6.5.1.1 Roof Slab Design (48" thick), N-S Reinforcement - INTSCUT7a slab width: $B := 40 \cdot ft$

$f'_c := 5000 \cdot psi$ $f_y := 60 \cdot ksi$

$\phi_b := 0.9$ Sect. 9.3.2.1, Ref. 2.2.14

$\phi_t := 0.9$ Sect. 9.3.2.2(a), Ref. 2.2.14

$\phi_c := 0.7$ Sect. 9.3.2.2(b), Ref. 2.2.14

$\phi_v := 0.6$ Sect. 9.3.4, Ref. 2.2.14

Load Combination I - DL+LL+SRSS

Axial Compression: **F1 = 96 kip (C)**

In-Plane Moment: **M3 = 1610 kip-ft**

In-Plane Shear: **F2 = 261 kip**

Out-of-Plane Moment: **M11 + M12 = 180 kip-ft/ft**

Out-of-Plane Shear: **V13 = 58 kip/ft**

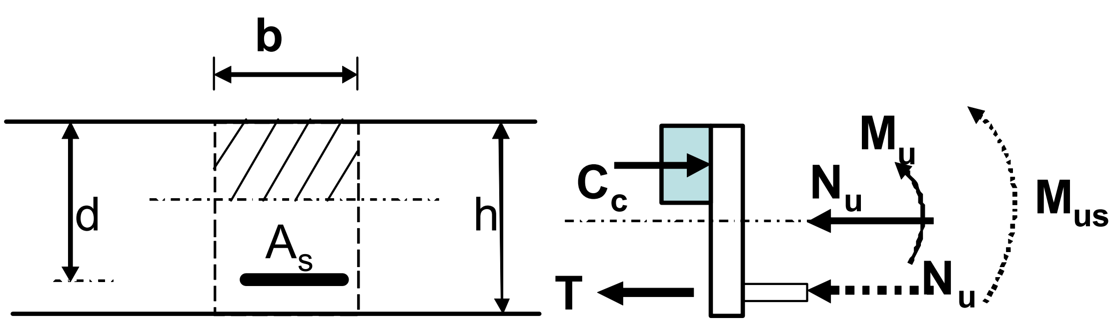
I-A Out-of-Plane Moment with Axial Compression

M11 + M12 = 180 kip-ft/ft **F1 = 96 kip (C)**

$M_{uo} := 180 \cdot \frac{kip \cdot ft}{ft}$ $N_u := \frac{96 \cdot kip}{B}$ $N_u = 2.4 \frac{kip}{ft}$

$b := 12 \cdot in$ $h := 48 \cdot in$ conc. cover for reinf.: $t_c := 2 \cdot in$

$d_{b11} := 1.41 \cdot in$ $d := h - t_c - d_{b11} \cdot 1.5$ (2nd layer) $d = 43.885\,in$



$M_{us} := M_{uo} + N_u \cdot \left(d - \frac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $M_{us} = 183.977\,kip \cdot \frac{ft}{ft}$

$m := \frac{f_y}{0.85 \cdot f'_c}$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$M_n := \frac{M_{us} \cdot b}{\phi_c}$ $M_n = 262.824\,kip \cdot ft$ $R_n := \frac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 136.469\,psi$

$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{f_y}}\right)$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 2.312 \times 10^{-3}$

$A_s := \rho \cdot b \cdot d - \frac{N_u \cdot b}{\phi_c \cdot f_y}$ (Commentary, Ref. 2.2.18) $A_s = 1.161\,in^2$

$A_{s\_min} := 0.0018 \cdot b \cdot h$ (Sect. 7.12.5, Ref. 2.2.14) $A_{s\_min} = 1.037\ in^2$

To count for D/C ratio (Demand/Capacity) = 0.6

$A_{s\_req} := \dfrac{A_s}{0.6}$ $A_{s\_req} = 1.934\ in^2$ try #11@9.5", As =1.97 in^2 O K

I-B Check Out-of-Plane Shear Capacity

Full Shear Capacity (neglect compression--- conservative)

$b = 12\ in$ $d = 43.885\ in$

$V_c := 2 \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d$ (Eq. 11-3, Ref. 2.2.14,) $V_c = 74.475\ kip$

$\phi_v \cdot V_c = 44.685\ kip$

Out-of-Plane Shear: **V13 = 58 kip/ft**

$V_{uo} := 58 \cdot \dfrac{kip}{ft}$ $\dfrac{V_{uo} \cdot b}{\phi_v \cdot V_c} = 1.298$ > D/C = 0.6 N G

Since V13 = 58 kip occured at corner point only, conservatively use average out-of plane shear + one-third of V13 for design.

Total Out-of Plane Shear at INTSCUT7a: F3 = 374 kip $B = 40\ ft$

$V := 374 \cdot kip$ $V_{ave} := \dfrac{V}{B}$ $V_{ave} = 9.35\ \dfrac{kip}{ft}$

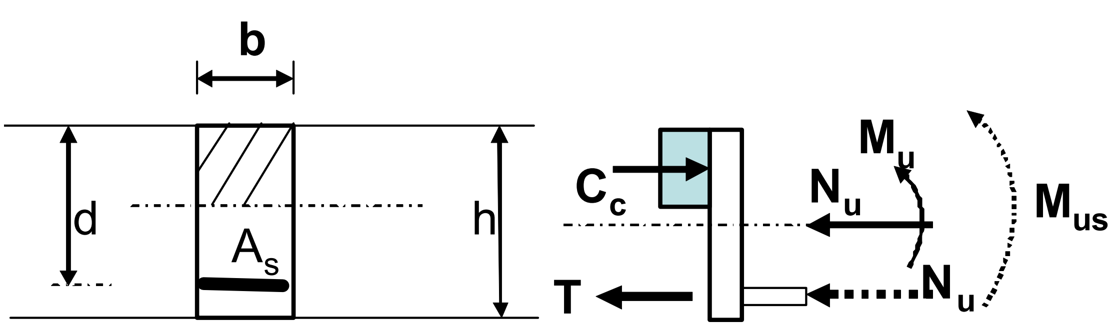
$V_u := \dfrac{V_{uo}}{3} + V_{ave}$ $V_u = 28.683\ \dfrac{kip}{ft}$

$\dfrac{V_u \cdot b}{\phi_v \cdot V_c} = 0.642$ Close to D/C = 0.6 O K

I-C Chord Reinf. at slab edge (east and west) due to In-Plane Moment

M3 = 1610 kip-ft **F1 = 96 kip (C)**

$M_{ui} := 1610 \cdot kip \cdot ft$ $N_u := 96 \cdot kip$ $b := 4 \cdot ft$ $h := 40 \cdot ft$ $d := h - 5 \cdot in$



$M_{us} := M_{ui} + N_u \cdot \left(d - \frac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $M_{us} = 3.49 \times 10^3\,\text{kip}\cdot\text{ft}$

$m := \frac{f_y}{0.85 \cdot f'_c}$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$M_n := \frac{M_{us}}{\phi_c}$ $M_n = 4.986 \times 10^3\,\text{kip}\cdot\text{ft}$ $R_n := \frac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 5.524\,\text{psi}$

$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{f_y}}\right)$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 9.213 \times 10^{-5}$

$A_s := \rho \cdot b \cdot d - \frac{N_u}{\phi_c \cdot f_y}$ (Commentary, Ref. 2.2.18) $A_s = -0.185\,\text{in}^2$ (tension reinf. not required----O K)

I-D Check In-Plane Shear Capacity **F2 = 261 kip**

$V_{ui} := 261 \cdot \text{kip}$ $b := 4 \cdot \text{ft}$ $h = 40\,\text{ft}$ $A_g := b \cdot h$ $d := h - 5 \cdot \text{in}$

$V_c := 2 \cdot \sqrt{f'_c \cdot \text{psi}} \cdot b \cdot d$ (Eq. 11-3, Ref. 2.2.14,) $V_c = 3.224 \times 10^3\,\text{kip}$

$\frac{V_{ui}}{\phi_v \cdot V_c} = 0.135$ < D/C = 0.6 O K

Load Combination II - DL+LL-SRSS

Axial Tension:	**F1 = -168 kip (T)**
In-Plane Moment:	**M3 = -1493 kip-ft**
In-Plane Shear:	**F2 = -350 kip**
Out-of-Plane Moment:	**M11 + M12 = -214 kip-ft/ft**
Out-of-Plane Shear:	**V13 = -57 kip/ft**

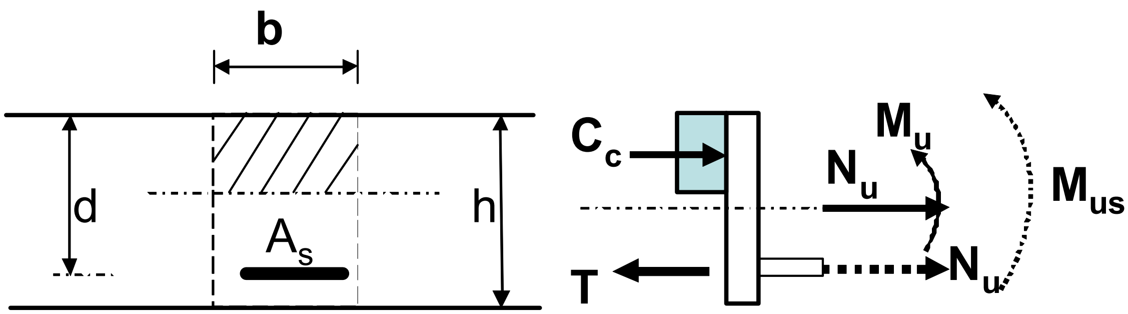
II-A Out-of-Plane Moment with Axial Tension slab width: $B := 40 \cdot \text{ft}$

M11 + M12 = -214 kip-ft/ft **F1 = -168 kip (T)**

$M_{uo} := 214 \cdot \frac{\text{kip}\cdot\text{ft}}{\text{ft}}$ $N_u := \frac{168 \cdot \text{kip}}{B}$ $N_u = 4.2\,\frac{\text{kip}}{\text{ft}}$

$b := 12 \cdot \text{in}$ $h := 48 \cdot \text{in}$ conc. cover for reinf.: $t_c := 2 \cdot \text{in}$

$d_{b11} := 1.41 \cdot \text{in}$ $d := h - t_c - d_{b11} \cdot 1.5$ (2nd layer) $d = 43.885\,\text{in}$



$M_{us} := M_{uo} - N_u \cdot \left(d - \frac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $M_{us} = 207.04\,\text{kip} \cdot \frac{\text{ft}}{\text{ft}}$

$m := \frac{f_y}{0.85 \cdot f_c}$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$M_n := \frac{M_{us} \cdot b}{\phi_t}$ $M_n = 230.045\,\text{kip} \cdot \text{ft}$ $R_n := \frac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 119.448\,\text{psi}$

$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{f_y}}\right)$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 2.02 \times 10^{-3}$

$A_s := \rho \cdot b \cdot d + \frac{N_u \cdot b}{\phi_t \cdot f_y}$ (Commentary, Ref. 2.2.18) $A_s = 1.141\,\text{in}^2$

$A_{s\_min} := 0.0018 \cdot b \cdot h$ (Sect. 7.12.5, Ref. 2.2.14) $A_{s\_min} = 1.037\,\text{in}^2$

To count for D/C ratio (Demand/Capacity) = 0.6

$A_{s\_req} := \frac{A_s}{0.6}$ $A_{s\_req} = 1.902\,\text{in}^2$ try #11@9.5", As =1.97 in^2 O K

II-B Check Out-of-Plane Shear Capacity **V13 = -57 kip/ft**

Shear Capacity with axial tension **F1 =-168 kip (T)**

$A_g := b \cdot h$ $h = 48\,\text{in}$ $A_g = 576\,\text{in}^2$ $N_u := \frac{-168 \cdot \text{kip}}{B}$ $N_u = -4.2 \frac{\text{kip}}{\text{ft}}$

$V_c := 2 \cdot \left(1 + \frac{N_u \cdot b}{500 \cdot A_g \cdot \text{psi}}\right) \cdot \sqrt{f_c \cdot \text{psi}} \cdot b \cdot d$ (Eq. 11-8, Ref. 2.2.14,) $V_c = 73.389\,\text{kip}$

$V_{uo} := 57 \cdot \frac{\text{kip}}{\text{ft}}$ $\frac{V_{uo} \cdot b}{\phi_v \cdot V_c} = 1.294$ > D/C = 0.6 N G

Since V13 = 57 kip occured at corner point only, conservatively use average out-of plane shear + one-third of V13 for design.

Total Out-of Plane Shear at INTSCUT7a: F3 = 118 kip $B = 40\,\text{ft}$

$V := 118 \cdot kip$ $\quad V_{ave} := \frac{V}{B}$ $\quad V_{ave} = 2.95 \frac{kip}{ft}$

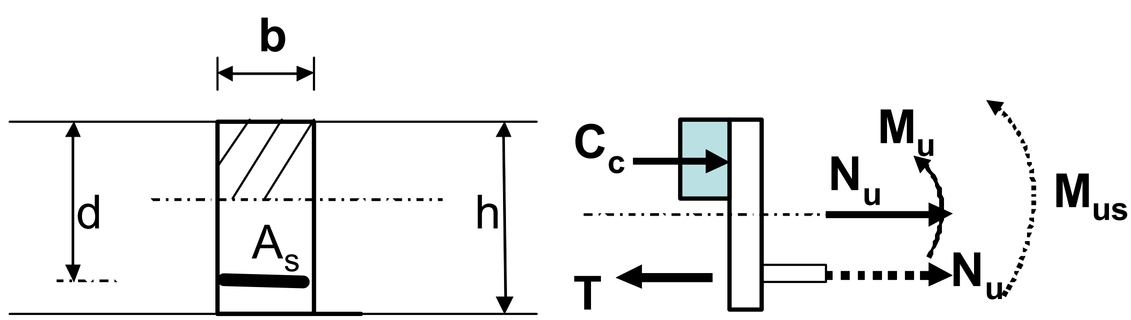
$V_u := \frac{V_{uo}}{3} + V_{ave}$ $\quad V_u = 21.95 \frac{kip}{ft}$

$\frac{V_u \cdot b}{\phi_v \cdot V_c} = 0.498$ < D/C = 0.6 O K

II-C Chord Reinf. at slab edge (east and west) due to In-Plane Moment

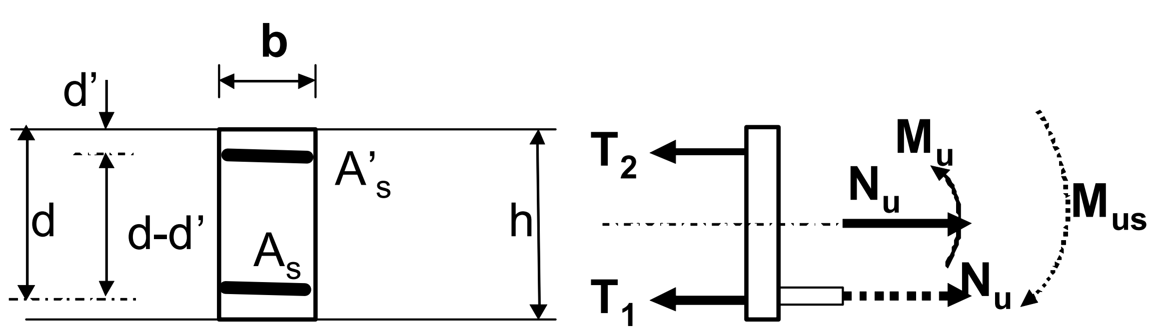
M3 = -1493 kip-ft **F1 = -168 kip (T)**

$M_{ui} := 1493 \cdot kip \cdot ft$ $\quad N_u := 168 \cdot kip$ $\quad b := 4 \cdot ft$ $\quad h := 40 \cdot ft$ $\quad d := h - 5 \cdot in$



$M_{us} := M_{ui} - N_u \cdot \left(d - \frac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $\quad M_{us} = -1.797 \times 10^3 \, kip \cdot ft$

Axial tension Nu is large compared to moment, redesign entire section in tension



$d' := 5 \cdot in$ $\quad A'_s := \frac{|M_{us}|}{\phi_b \cdot f_y \cdot (d - d')}$ (Commentary, Ref. 2.2.18) $\quad A'_s = 0.85 \, in^2$

$A_s := \frac{N_u}{\phi_t \cdot f_y} - A'_s$ (Commentary, Ref. 2.2.18) $\quad A_s = 2.261 \, in^2$

To count for D/C ratio (Demand/Capacity) = 0.6

$A_{s\_req\_ns} := \frac{A_s}{0.6}$ $\quad A_{s\_req\_ns} = 3.769\,in^2$

try 3#11 (N-S), As = 4.68 sq. in. O K

II-D Check In-Plane Shear Capacity **F2 = -350 kip** **F1 =-168 kip (T)**

$V_{ui} := 350 \cdot kip$ $\quad N_u := -168 \cdot kip$

$b := 4 \cdot ft$ $\quad h = 40\,ft$ $\quad A_g := b \cdot h$ $\quad d := h - 5 \cdot in$

$V_c := 2 \cdot \left(1 + \frac{N_u}{500 \cdot A_g \cdot psi}\right) \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d$ (Eq. 11-8, Ref. 2.2.14,) $\quad V_c = 3.177 \times 10^3\,kip$

$\frac{V_{ui}}{\phi_v \cdot V_c} = 0.184$ < D/C = 0.6 O K

6.5.1.2 Roof Slab Design (48" thick), E-W Reinforcement - INTSCUT7b

slab width: $B := 13.5 \cdot ft$

Load Combination I - DL+LL+SRSS

Axial Compression: **F2 = 326 kip (C)**

In-Plane Moment: **M3 = 1527 kip-ft**

In-Plane Shear: **F1 = 330 kip**

Out-of-Plane Moment: **M22 + M12 = 234 kip-ft/ft**

Out-of-Plane Shear: **V13 = 58 kip/ft**

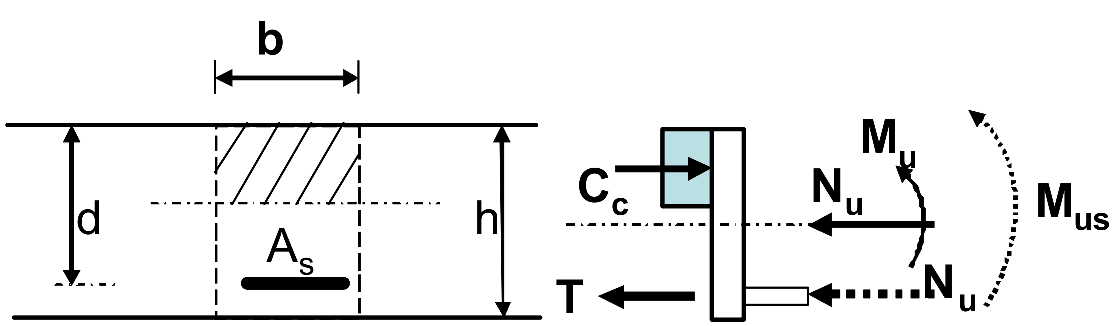
I-A Out-of-Plane Moment with Axial Compression

M22 + M12 = 234 kip-ft/ft **F2 = 326 kip (C)**

$M_{uo} := 234 \cdot \frac{kip \cdot ft}{ft}$ $N_u := \frac{326 \cdot kip}{B}$ $N_u = 24.148 \frac{kip}{ft}$

$b := 12 \cdot in$ $h := 48 \cdot in$ conc. cover for reinf.: $t_c := 2 \cdot in$

$d_{b11} := 1.41 \cdot in$ $d := h - t_c - d_{b11} \cdot 1.5$ (2nd layer) $d = 43.885\, in$



$M_{us} := M_{uo} + N_u \cdot \left(d - \frac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $M_{us} = 274.015\, kip \cdot \frac{ft}{ft}$

$m := \frac{f_y}{0.85 \cdot f'_c}$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$M_n := \frac{M_{us} \cdot b}{\phi_c}$ $M_n = 391.451\, kip \cdot ft$ $R_n := \frac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 203.257\, psi$

$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{f_y}}\right)$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 3.473 \times 10^{-3}$

$A_s := \rho \cdot b \cdot d - \dfrac{N_u \cdot b}{\phi_c \cdot f_y}$ (Commentary, Ref. 2.2.18) $A_s = 1.254\ in^2$

$A_{s\_min} := 0.0018 \cdot b \cdot h$ (Sect. 7.12.5, Ref. 2.2.14) $A_{s\_min} = 1.037\ in^2$

To count for D/C ratio (Demand/Capacity) = 0.6

$A_{s\_req} := \dfrac{A_s}{0.6}$ $A_{s\_req} = 2.09\ in^2$ try #11@9", As =2.08 in^2 O K

I-B Check Out-of-Plane Shear Capacity

Full Shear Capacity (neglect compression--- conservative)

$b = 12\ in$ $d = 43.885\ in$

$V_c := 2 \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d$ (Eq. 11-3, Ref. 2.2.14,) $V_c = 74.475\ kip$

$\phi_v \cdot V_c = 44.685\ kip$

Out-of-Plane Shear: **V13 = 58 kip/ft**

$V_{uo} := 58 \cdot \dfrac{kip}{ft}$ $\dfrac{V_{uo} \cdot b}{\phi_v \cdot V_c} = 1.298$ > D/C = 0.6 N G

Since V13 = 58 kip occured at corner point only, conservatively use average out-of plane shear + one-third of V13 for design.

Total Out-of Plane Shear at INTSCUT7b: F3 = 149 kip $B = 13.5\ ft$

$V := 149 \cdot kip$ $V_{ave} := \dfrac{V}{B}$ $V_{ave} = 11.037\ \dfrac{kip}{ft}$

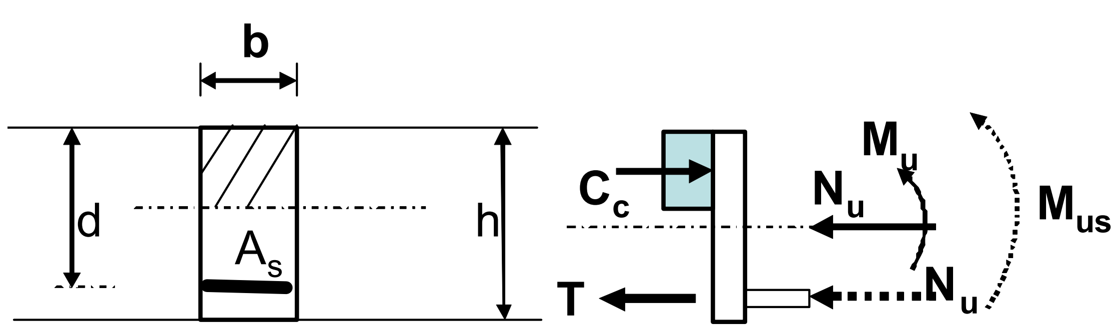
$V_u := \dfrac{V_{uo}}{3} + V_{ave}$ $V_u = 30.37\ \dfrac{kip}{ft}$

$\dfrac{V_u \cdot b}{\phi_v \cdot V_c} = 0.68$ Close to D/C = 0.6 O K

I-C Chord Reinf. at slab edge (east and west) due to In-Plane Moment

M3 = 1527 kip-ft **F2 = 326 kip (C)**

$M_{ui} := 1527 \cdot kip \cdot ft$ $N_u := 326 \cdot kip$ $b := 4 \cdot ft$ $h := 13.5 \cdot ft$ $d := h - 5 \cdot in$



$M_{us} := M_{ui} + N_u \cdot \left(d - \frac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $M_{us} = 3.592 \times 10^3\, kip \cdot ft$

$m := \frac{f_y}{0.85 \cdot f'_c}$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$M_n := \frac{M_{us}}{\phi_c}$ $M_n = 5.131 \times 10^3\, kip \cdot ft$ $R_n := \frac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 52.04\, psi$

$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{f_y}}\right)$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 8.727 \times 10^{-4}$

$A_s := \rho \cdot b \cdot d - \frac{N_u}{\phi_c \cdot f_y}$ (Commentary, Ref. 2.2.18) $A_s = -1.185\, in^2$ (tension reinf. not required----O K)

I-D Check In-Plane Shear Capacity **F1 = 330 kip**

$V_{ui} := 330 \cdot kip$ $b := 4 \cdot ft$ $h := 13.5 \cdot ft$ $A_g := b \cdot h$ $d := h - 5 \cdot in$

$V_c := 2 \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d$ (Eq. 11-3, Ref. 2.2.14,) $V_c = 1.066 \times 10^3\, kip$

$\frac{V_{ui}}{\phi_v \cdot V_c} = 0.516$ < D/C = 0.6 O K

Load Combination II - DL+LL-SRSS

Axial Tension: **F2 = -224 kip (T)**

In-Plane Moment: **M3 = -2062 kip-ft**

In-Plane Shear: **F1 = -252 kip**

Out-of-Plane Moment: **M22 + M12 = -249 kip-ft/ft**

Out-of-Plane Shear: **V13 = -57 kip/ft**

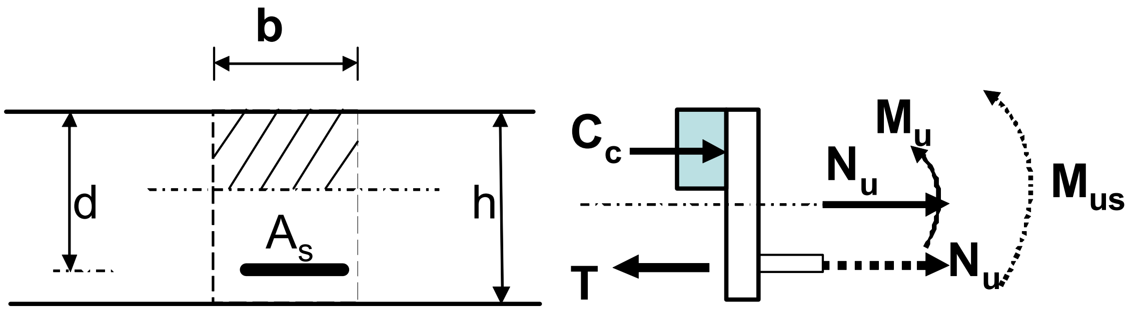
II-A Out-of-Plane Moment with Axial Tension slab width: $B := 13.5 \cdot ft$

M22 + M12 = -249 kip-ft/ft **F2 = -224 kip (T)**

$M_{uo} := 249 \cdot \frac{kip \cdot ft}{ft}$ $N_u := \frac{224 \cdot kip}{B}$ $N_u = 16.593 \frac{kip}{ft}$

$b := 12 \cdot in$ $h := 48 \cdot in$ conc. cover for reinf.: $t_c := 2 \cdot in$

$d_{b11} := 1.41 \cdot in$ $d := h - t_c - d_{b11} \cdot 1.5$ (2nd layer) $d = 43.885\, in$



$M_{us} := M_{uo} - N_u \cdot \left(d - \frac{h}{2} \right)$ (Commentary, Ref. 2.2.18) $M_{us} = 221.505 \, kip \cdot \frac{ft}{ft}$

$m := \frac{f_y}{0.85 \cdot f'_c}$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$M_n := \frac{M_{us} \cdot b}{\phi_t}$ $M_n = 246.116 \, kip \cdot ft$ $R_n := \frac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 127.793 \, psi$

$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{f_y}} \right)$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 2.163 \times 10^{-3}$

$A_s := \rho \cdot b \cdot d + \frac{N_u \cdot b}{\phi_t \cdot f_y}$ (Commentary, Ref. 2.2.18) $A_s = 1.446 \, in^2$

$A_{s\_min} := 0.0018 \cdot b \cdot h$ (Sect. 7.12.5, Ref. 2.2.14) $A_{s\_min} = 1.037 \, in^2$

To count for D/C ratio (Demand/Capacity) = 0.6

$A_{s\_req} := \frac{A_s}{0.6}$ $A_{s\_req} = 2.411 \, in^2$ try #11@8", As =2.34 in^2 O K

II-B Check Out-of-Plane Shear Capacity **V13 = -57 kip/ft**

Shear Capacity with axial tension **F2 =-224 kip (T)** $B = 13.5 \, ft$

$A_g := b \cdot h$ $h = 48 \, in$ $A_g = 576 \, in^2$ $N_u := \frac{-224 \cdot kip}{B}$ $N_u = -16.593 \, \frac{kip}{ft}$

$V_c := 2 \cdot \left(1 + \frac{N_u \cdot b}{500 \cdot A_g \cdot psi} \right) \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d$ (Eq. 11-8, Ref. 2.2.14,) $V_c = 70.185 \, kip$

$V_{uo} := 57 \cdot \frac{kip}{ft}$ $\frac{V_{uo} \cdot b}{\phi_v \cdot V_c} = 1.354$ > D/C = 0.6 N G

Since V13 = 57 kip occured at corner point only, conservatively use average out-of plane shear + one-third of V13 for design.

Total Out-of Plane Shear at INTSCUT7b: F3 = 2 kip $B = 13.5\,ft$

$V := 2 \cdot kip$ $V_{ave} := \frac{V}{B}$ $V_{ave} = 0.148\,\frac{kip}{ft}$

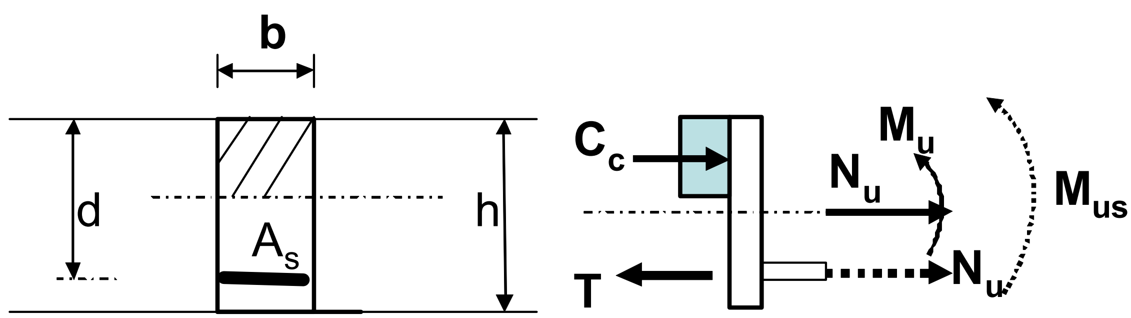
$V_u := \frac{V_{uo}}{3} + V_{ave}$ $V_u = 19.148\,\frac{kip}{ft}$

$\frac{V_u \cdot b}{\phi_v \cdot V_c} = 0.455$ < D/C = 0.6 O K

<u>II-C Chord Reinf. at slab edge (east and west) due to In-Plane Moment</u>

M3 = -2062 kip-ft **F2 = -224 kip (T)**

$M_{ui} := 2062 \cdot kip \cdot ft$ $N_u := 224 \cdot kip$ $b := 4 \cdot ft$ $h := 13.5 \cdot ft$ $d := h - 5 \cdot in$



$M_{us} := M_{ui} - N_u \cdot \left(d - \frac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $M_{us} = 643.333\,kip \cdot ft$

$m := \frac{f_y}{0.85 \cdot f'_c}$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$M_n := \frac{M_{us}}{\phi_t}$ $M_n = 714.815\,kip \cdot ft$ $R_n := \frac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 7.25\,psi$

$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{f_y}}\right)$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 1.209 \times 10^{-4}$

$A_s := \rho \cdot b \cdot d + \frac{N_u}{\phi_t \cdot f_y}$ (Commentary, Ref. 2.2.18) $A_s = 5.06\,in^2$

$A_{s\_min} := 0.0018 \cdot b \cdot h$ (Sect. 7.12.5, Ref. 2.2.14) $A_{s\_min} = 13.997\,in^2$

To count for D/C ratio (Demand/Capacity) = 0.6

$A_{s\_req} := \frac{A_s}{0.6}$ $A_{s\_req} = 8.433\,in^2$ try 6#11, As =9.36 in^2 O K

II-D Check In-Plane Shear Capacity **F1 = -252 kip** **F2 =-224 kip (T)**

$V_{ui} := 252 \cdot kip \qquad N_u := -224 \cdot kip$

$b := 4 \cdot ft \qquad h = 13.5\,ft \qquad A_g := b \cdot h \qquad d := h - 5 \cdot in$

$V_c := 2 \cdot \left(1 + \dfrac{N_u}{500 \cdot A_g \cdot psi}\right) \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d$ (Eq. 11-8, Ref. 2.2.14,) $V_c = 1.004 \times 10^3\,kip$

$\dfrac{V_{ui}}{\phi_v \cdot V_c} = 0.418$ < D/C = 0.6 O K

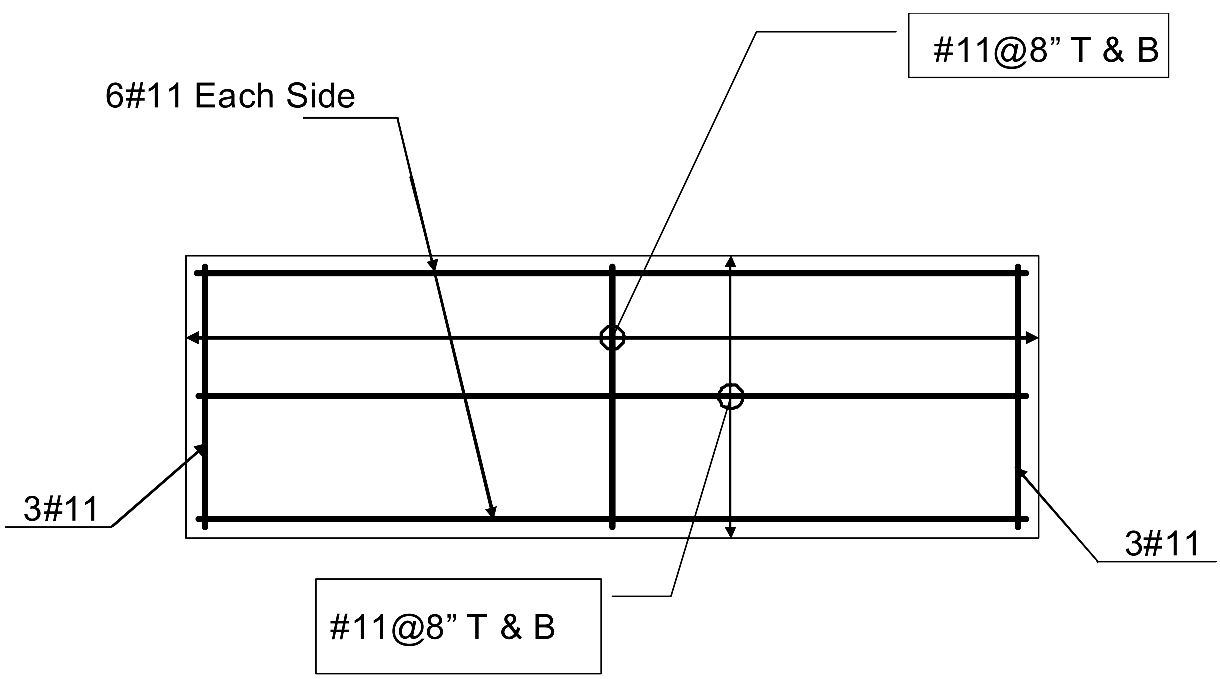
For simplicity of placing rebars, use Reinforcement as follow:

In North-South direction (short bar):

Provide #11@8" T & B, added Chord Bar 3#11 at east and west slab edges

In East-West direction (long bar):

Provide #11@8" T & B, added Chord Bar 6#11 at north and south slab edges



6.5.2

Roof Slab Design Loads, Column Lines 5-7

Roof Slab EL 37'-0"

Section cut design forces and moments, which follow a global axis system, for the roof slab at EL 37'-0" from column line 5 to 7 include in plane forces and moments: such as axial forces (tension/compression), in plane moment, and in plane shear. The section cut values are integrated along the section cut length, thus for INTSCUT8a & INTSCUT8b the length is equal to 50' and for INTSCUT8c & INTSCUT8d the length is equal to 37'. Out of plane values such as out of plane bending and out of plane shear are attained by shell element forces and moments from SAP2000 which follow a local axis system. The out of plane bending moments provided are M11 and M22, and a third moment M12, which is a twisting moment that is combined with M22 and M11. The ou of plane shear forces from the shell elements include V13 and V23.

INTSCUT8a & INTSCUT8b
F1 = Axial Force, Compression (+) / Tension (-)
F2 = In Plane Shear
M3 = In Plane Moment

INTSCUT8c & INTSCUT8d
F1 = In Plane Shear
F2 = Axial Force, Compression (+) / Tension (-)
M3 = In Plane Moment

Shell Element Forces and Moments
M11 = Out of Plane Moment
M22 = Out of Plane Moment
M12 = Twisting Moment
V13 = Out of Plane Shear
V23 = Out of Plane Shear

Accidental torsion factor= 15% (See assumption 3.1.4)

Section Cut	DL+LL						Seismic (SRSS)					
	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
INTSCUT8a	92	17	-	-	-	-232	239	1134	-	-	-	4540
INTSCUT8b	210	-26	-	-	-	-3061	117	928	-	-	-	4038
INTSCUT8c	14	-61	-	-	-	-152	908	533	-	-	-	8570
INTSCUT8d	-14	27	-	-	-	143	1130	689	-	-	-	5246

	Element Forces - Area Shells D+L						Element Forces - Area Shells SRSS				
MAX	M11	M22	M12	V13	V23	MAX	M11	M22	M12	V13	V23
MIN	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft		Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	72	46	16	10	8	MAX	84	62	22	10	9
MIN	-45	-68	-17	-11	-11						

Loads with accidental torsion factor

Section Cut	DL+LL						Seismic (SRSS)					
	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
INTSCUT8a	106	20	-	-	-	-267	274	1305	-	-	-	5221
INTSCUT8b	242	-30	-	-	-	-3520	135	1067	-	-	-	4643
INTSCUT8c	16	-71	-	-	-	-175	1044	612	-	-	-	9855
INTSCUT8d	-16	31	-	-	-	165	1299	792	-	-	-	6033

Loads with accidental torsion factor

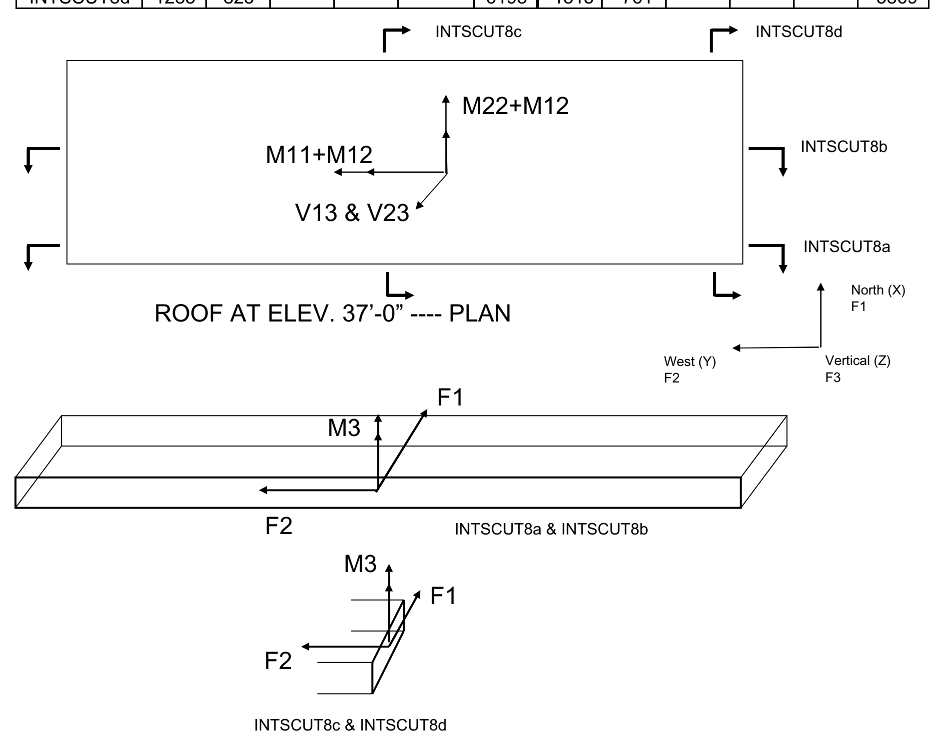
	Element Forces - Area Shells D+L						Element Forces - Area Shells SRSS				
MAX	M11	M22	M12	V13	V23	MAX	M11	M22	M12	V13	V23
MIN	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft		Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	83	52	18	12	9	MAX	96	71	26	12	10
MIN	-52	-78	-19	-12	-13						

Maximum Load Combination

Section Cut	DL+LL+Seismic (SRSS)						DL+LL-Seismic (SRSS)					
	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
INTSCUT8a	380	1324	-	-	-	4954	-169	-1285	-	-	-	-5488
INTSCUT8b	376	1037	-	-	-	1123	107	-1097	-	-	-	-8164
INTSCUT8c	1061	542	-	-	-	9680	-1028	-683	-	-	-	-10030
INTSCUT8d	1283	823	-	-	-	6198	-1315	-761	-	-	-	-5869

Maximum Load Combination

	DL+LL+Seismic(SRSS)						DL+LL-Seismic(SRSS)				
MAX	M11	M22	M12	V13	V23	MAX	M11	M22	M12	V13	V23
MIN	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft	MIN	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	179	123	44	24	19	MAX	-13	-18	-7	1	-1
MIN	44	-7	6	-1	-3	MIN	-148	-149	-45	-24	-23



6.5.2.1 Roof Slab Design (48" thick), N-S Reinforcement - INTSCUT8a slab width: $B := 50 \cdot ft$

$f'_c := 5000 \cdot psi$ $f_y := 60 \cdot ksi$

$\phi_b := 0.9$ Sect. 9.3.2.1, Ref. 2.2.14

$\phi_t := 0.9$ Sect. 9.3.2.2(a), Ref. 2.2.14

$\phi_c := 0.7$ Sect. 9.3.2.2(b), Ref. 2.2.14

$\phi_v := 0.6$ Sect. 9.3.4, Ref. 2.2.14

Load Combination I - DL+LL+SRSS

Axial Compression:	**F1 = 380 kip (C)**
In-Plane Moment:	**M3 = 4954 kip-ft**
In-Plane Shear:	**F2 = 1324 kip**
Out-of-Plane Moment:	**M11 + M12 = 223 kip-ft/ft**
Out-of-Plane Shear:	**V13 = 24 kip/ft**

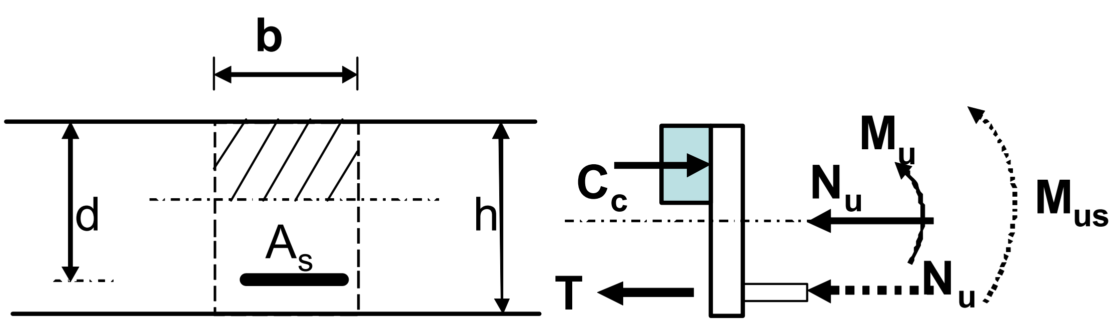
I-A Out-of-Plane Moment with Axial Compression

M11 + M12 = 223 kip-ft/ft **F1 = 380 kip (C)**

$M_{uo} := 223 \cdot \frac{kip \cdot ft}{ft}$ $N_u := \frac{380 \cdot kip}{B}$ $N_u = 7.6 \frac{kip}{ft}$

$b := 12 \cdot in$ $h := 48 \cdot in$ conc. cover for reinf.: $t_c := 2 \cdot in$

$d_{b11} := 1.41 \cdot in$ $d := h - t_c - d_{b11} \cdot 1.5$ (2nd layer) $d = 43.885\ in$



$M_{us} := M_{uo} + N_u \cdot \left(d - \frac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $M_{us} = 235.594\ kip \cdot \frac{ft}{ft}$

$m := \frac{f_y}{0.85 \cdot f'_c}$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$M_n := \frac{M_{us} \cdot b}{\phi_c}$ $M_n = 336.563\ kip \cdot ft$ $R_n := \frac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 174.757\ psi$

$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{f_y}}\right)$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 2.975 \times 10^{-3}$

$A_s := \rho \cdot b \cdot d - \frac{N_u \cdot b}{\phi_c \cdot f_y}$ (Commentary, Ref. 2.2.18) $A_s = 1.386\ in^2$

$A_{s\_min} := 0.0018 \cdot b \cdot h$ (Sect. 7.12.5, Ref. 2.2.14) $A_{s\_min} = 1.037\ in^2$

To count for D/C ratio (Demand/Capacity) = 0.6

$A_{s\_req} := \dfrac{A_s}{0.6}$ $A_{s\_req} = 2.31\ in^2$ try #11@8", As = 2.34 in^2 O K

I-B Check Out-of-Plane Shear Capacity

Full Shear Capacity (neglect compression--- conservative)

$b = 12\ in$ $d = 43.885\ in$

$V_c := 2 \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d$ (Eq. 11-3, Ref. 2.2.14,) $V_c = 74.475\ kip$

$\phi_v \cdot V_c = 44.685\ kip$

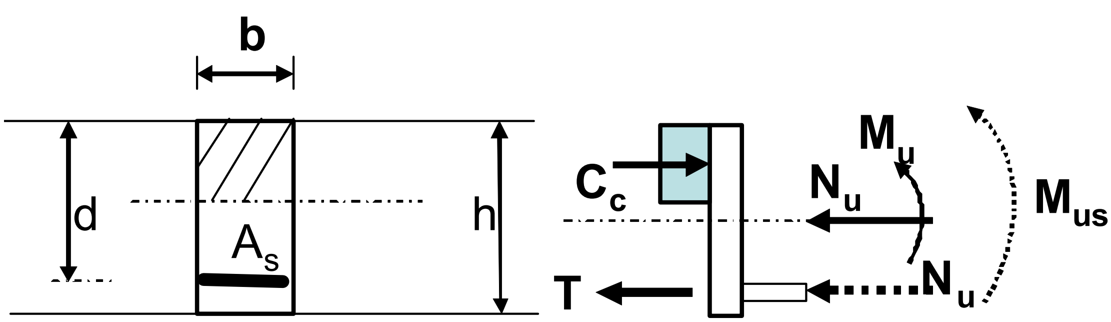
Out-of-Plane Shear: **V13 = 24 kip/ft**

$V_{uo} := 24 \cdot \dfrac{kip}{ft}$ $\dfrac{V_{uo} \cdot b}{\phi_v \cdot V_c} = 0.537$ < D/C = 0.6 O K

I-C Chord Reinf. at slab edge (east and west) due to In-Plane Moment

M3 = 4954 kip-ft **F1 = 380 kip (C)**

$M_{ui} := 4954 \cdot kip \cdot ft$ $N_u := 380 \cdot kip$ $b := 4 \cdot ft$ $h := 50 \cdot ft$ $d := h - 5 \cdot in$



$M_{us} := M_{ui} + N_u \cdot \left(d - \dfrac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $M_{us} = 1.43 \times 10^4\ kip \cdot ft$

$m := \dfrac{f_y}{0.85 \cdot f'_c}$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$M_n := \dfrac{M_{us}}{\phi_c}$ $M_n = 2.042 \times 10^4\ kip \cdot ft$ $R_n := \dfrac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 14.422\ psi$

$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{f_y}}\right)$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 2.408 \times 10^{-4}$

$A_s := \rho \cdot b \cdot d - \frac{N_u}{\phi_c \cdot f_y}$ (Commentary, Ref. 2.2.18) $A_s = -2.171\ \text{in}^2$ (tension reinf. not required----O K)

I-D Check In-Plane Shear Capacity **F2 = 1324 kip**

$V_{ui} := 1324 \cdot \text{kip}$ $b := 4 \cdot \text{ft}$ $h = 50\ \text{ft}$ $A_g := b \cdot h$ $d := h - 5 \cdot \text{in}$

$V_c := 2 \cdot \sqrt{f'_c \cdot \text{psi}} \cdot b \cdot d$ (Eq. 11-3, Ref. 2.2.14,) $V_c = 4.039 \times 10^3\ \text{kip}$

$\frac{V_{ui}}{\phi_v \cdot V_c} = 0.546$ < D/C = 0.6 O K

Load Combination II - DL+LL-SRSS

Axial Tension: **F1 = -169 kip (T)**

In-Plane Moment: **M3 = -5488 kip-ft**

In-Plane Shear: **F2 = -1285 kip**

Out-of-Plane Moment: **M11 + M12 = -193 kip-ft/ft**

Out-of-Plane Shear: **V13 = -24 kip/ft**

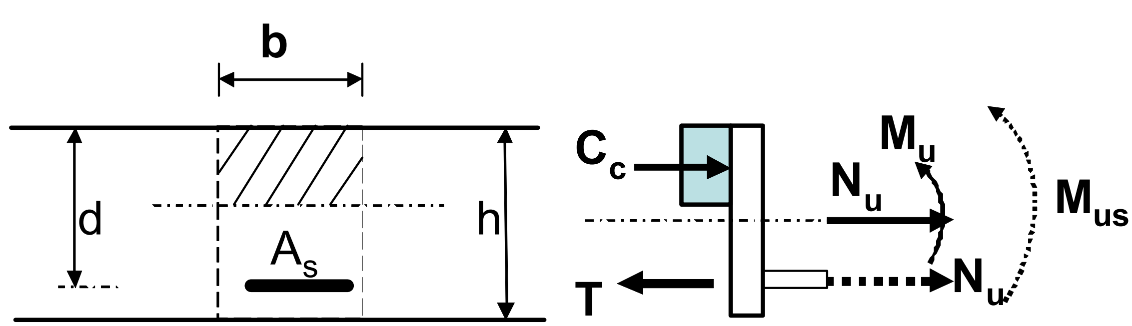
II-A Out-of-Plane Moment with Axial Tension slab width: $B := 50 \cdot \text{ft}$

M11 + M12 = -193 kip-ft/ft **F1 = -169 kip (T)**

$M_{uo} := 193 \cdot \frac{\text{kip} \cdot \text{ft}}{\text{ft}}$ $N_u := \frac{169 \cdot \text{kip}}{B}$ $N_u = 3.38\ \frac{\text{kip}}{\text{ft}}$

$b := 12 \cdot \text{in}$ $h := 48 \cdot \text{in}$ conc. cover for reinf.: $t_c := 2 \cdot \text{in}$

$d_{b11} := 1.41 \cdot \text{in}$ $d := h - t_c - d_{b11} \cdot 1.5$ (2nd layer) $d = 43.885\ \text{in}$



$M_{us} := M_{uo} - N_u \cdot \left(d - \frac{h}{2} \right)$ (Commentary, Ref. 2.2.18) $M_{us} = 187.399\ \text{kip} \cdot \frac{\text{ft}}{\text{ft}}$

$m := \frac{f_y}{0.85 \cdot f_c}$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$M_n := \frac{M_{us} \cdot b}{\phi_t}$ $M_n = 208.221\ \text{kip} \cdot \text{ft}$ $R_n := \frac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 108.117\ \text{psi}$

$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{f_y}} \right)$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 1.825 \times 10^{-3}$

$A_s := \rho \cdot b \cdot d + \frac{N_u \cdot b}{\phi_t \cdot f_y}$ (Commentary, Ref. 2.2.18) $A_s = 1.024\ \text{in}^2$

$A_{s\_min} := 0.0018 \cdot b \cdot h$ (Sect. 7.12.5, Ref. 2.2.14) $A_{s\_min} = 1.037\ \text{in}^2$

To count for D/C ratio (Demand/Capacity) = 0.6

$A_{s\_req} := \frac{A_s}{0.6}$ $A_{s\_req} = 1.707\ \text{in}^2$ try #11@10", As =1.87 in^2 O K

II-B Check Out-of-Plane Shear Capacity **V13 = -24 kip/ft**

Shear Capacity with axial tension **F1 =-169 kip (T)**

$A_g := b \cdot h$ $h = 48\ \text{in}$ $A_g = 576\ \text{in}^2$ $N_u := \frac{-169 \cdot \text{kip}}{B}$ $N_u = -3.38\ \frac{\text{kip}}{\text{ft}}$

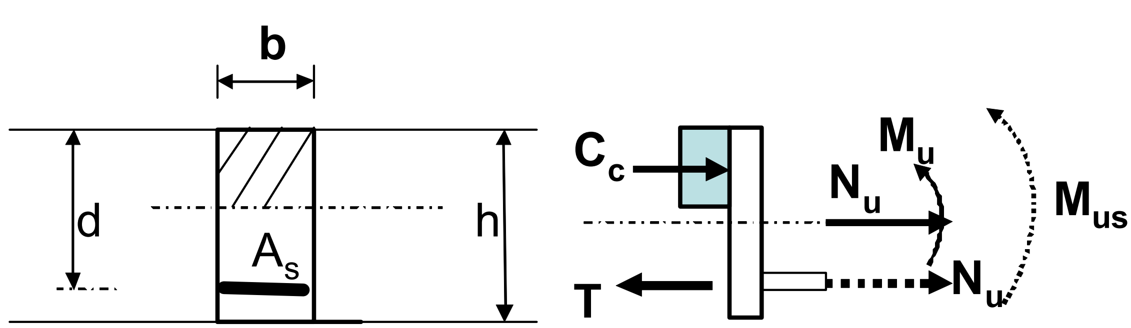
$V_c := 2 \cdot \left(1 + \frac{N_u \cdot b}{500 \cdot A_g \cdot \text{psi}} \right) \cdot \sqrt{f_c \cdot \text{psi}} \cdot b \cdot d$ (Eq. 11-8, Ref. 2.2.14,) $V_c = 73.601\ \text{kip}$

$V_{uo} := 24 \cdot \frac{\text{kip}}{\text{ft}}$ $\frac{V_{uo} \cdot b}{\phi_v \cdot V_c} = 0.543$ < D/C = 0.6 O K

II-C Chord Reinf. at slab edge (east and west) due to In-Plane Moment

M3 = -5488 kip-ft **F1 = -169 kip (T)**

$M_{ui} := 5488 \cdot \text{kip} \cdot \text{ft}$ $N_u := 169 \cdot \text{kip}$ $b := 4 \cdot \text{ft}$ $h := 50 \cdot \text{ft}$ $d := h - 5 \cdot \text{in}$



$M_{us} := M_{ui} - N_u \cdot \left(d - \frac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $M_{us} = 1.333 \times 10^3 \, kip \cdot ft$

$m := \frac{f_y}{0.85 \cdot f'_c}$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$M_n := \frac{M_{us}}{\phi_t}$ $M_n = 1.482 \times 10^3 \, kip \cdot ft$ $R_n := \frac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 1.046 \, psi$

$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{f_y}}\right)$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 1.744 \times 10^{-5}$

$A_s := \rho \cdot b \cdot d + \frac{N_u}{\phi_t \cdot f_y}$ (Commentary, Ref. 2.2.18) $A_s = 3.628 \, in^2$

To count for D/C ratio (Demand/Capacity) = 0.6

$A_{s\_req\_ns} := \frac{A_s}{0.6}$ $A_{s\_req\_ns} = 6.046 \, in^2$

try 4#11 (N-S), As = 6.24 sq. in. O K

II-D Check In-Plane Shear Capacity **F2 = -1285 kip** **F1 =-169 kip (T)**

$V_{ui} := 1285 \cdot kip$ $N_u := -169 \cdot kip$

$b := 4 \cdot ft$ $h = 50 \, ft$ $A_g := b \cdot h$ $d := h - 5 \cdot in$

$V_c := 2 \cdot \left(1 + \frac{N_u}{500 \cdot A_g \cdot psi}\right) \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d$ (Eq. 11-8, Ref. 2.2.14,) $V_c = 3.992 \times 10^3 \, kip$

$\frac{V_{ui}}{\phi_v \cdot V_c} = 0.537$ < D/C = 0.6 O K

6.5.2.2 Roof Slab Design (48" thick), N-S Reinforcement - INTSCUT8b slab width: $B := 50 \cdot ft$

Load Combination I - DL+LL+SRSS

Axial Compression: **F1 = 376 kip (C)**

In-Plane Moment: **M3 = 1123 kip-ft**

In-Plane Shear: **F2 = 1037 kip**

Out-of-Plane Moment: **M11 + M12 = 223 kip-ft/ft**

Out-of-Plane Shear: **V13 = 24 kip/ft**

I-A Out-of-Plane Moment with Axial Compression

M11 + M12 = 223 kip-ft/ft **F1 = 376 kip (C)**

Compared with section cut at INTSCUT8a, loads are smaller, use design at INTSCUT8a

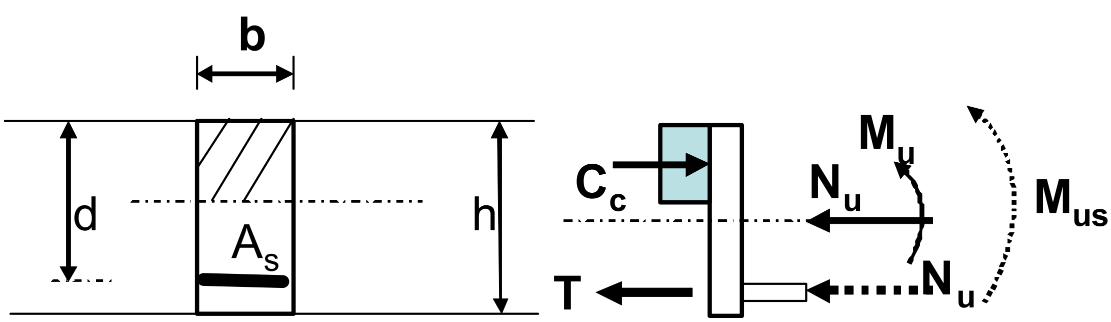
I-B Check Out-of-Plane Shear Capacity : **V13 = 24 kip/ft**

same as INTSCUT8a O K

I-C Chord Reinf. at slab edge (east and west) due to In-Plane Moment

M3 = 1123 kip-ft **F1 = 376 kip (C)**

$M_{ui} := 1123 \cdot kip \cdot ft$ $N_u := 376 \cdot kip$ $b := 4 \cdot ft$ $h := 50 \cdot ft$ $d := h - 5 \cdot in$



$M_{us} := M_{ui} + N_u \cdot \left(d - \frac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $M_{us} = 1.037 \times 10^4 \, kip \cdot ft$

$m := \frac{f_y}{0.85 \cdot f'_c}$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$M_n := \frac{M_{us}}{\phi_c}$ $M_n = 1.481 \times 10^4 \, kip \cdot ft$ $R_n := \frac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 10.458 \, psi$

$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{f_y}}\right)$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 1.745 \times 10^{-4}$

$A_s := \rho \cdot b \cdot d - \frac{N_u}{\phi_c \cdot f_y}$ (Commentary, Ref. 2.2.18) $A_s = -3.968\ in^2$ (tension reinf. not required----O K)

I-D Check In-Plane Shear Capacity **F2 = 1037 kip**

$V_{ui} := 1037 \cdot kip$ $b := 4 \cdot ft$ $h = 50\ ft$ $A_g := b \cdot h$ $d := h - 5 \cdot in$

$V_c := 2 \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d$ (Eq. 11-3, Ref. 2.2.14,) $V_c = 4.039 \times 10^3\ kip$

$\frac{V_{ui}}{\phi_v \cdot V_c} = 0.428$ < D/C = 0.6 O K

Load Combination II - DL+LL-SRSS

Axial Tension: **F1 = -107 kip (T)**

In-Plane Moment: **M3 = -8164kip-ft**

In-Plane Shear: **F2 = -1097 kip**

Out-of-Plane Moment: **M11 + M12 = -193 kip-ft/ft**

Out-of-Plane Shear: **V13 = -24 kip/ft**

II-A Out-of-Plane Moment with Axial Tension slab width: $B := 50 \cdot ft$

M11 + M12 = -193 kip-ft/ft **F1 = -107 kip (T)**

Compared with section cut at INTSCUT8a, loads are smaller, use design at INTSCUT8a

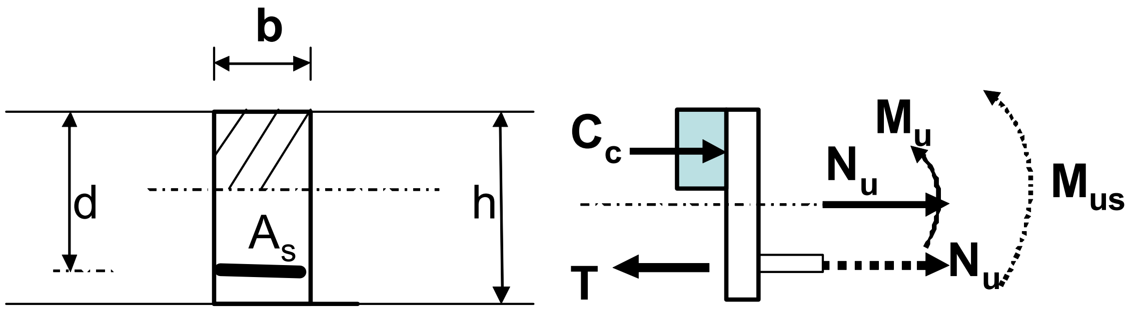
II-B Check Out-of-Plane Shear Capacity **V13 = -24 kip/ft** **F1 =-107 kip (T)**

Compared with section cut at INTSCUT8a, loads are smaller, use design at INTSCUT8a

II-C Chord Reinf. at slab edge (east and west) due to In-Plane Moment

M3 = -8164 kip-ft **F1 = -107 kip (T)**

$M_{ui} := 8164 \cdot kip \cdot ft$ $N_u := 107 \cdot kip$ $b := 4 \cdot ft$ $h := 50 \cdot ft$ $d := h - 5 \cdot in$



$$M_{us} := M_{ui} - N_u \cdot \left(d - \frac{h}{2}\right)$$ (Commentary, Ref. 2.2.18) $M_{us} = 5.534 \times 10^3 \, kip \cdot ft$

$$m := \frac{f_y}{0.85 \cdot f'_c}$$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$$M_n := \frac{M_{us}}{\phi_t}$$ $M_n = 6.148 \times 10^3 \, kip \cdot ft$ $R_n := \frac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 4.342 \, psi$

$$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{f_y}}\right)$$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 7.24 \times 10^{-5}$

$$A_s := \rho \cdot b \cdot d + \frac{N_u}{\phi_t \cdot f_y}$$ (Commentary, Ref. 2.2.18) $A_s = 4.049 \, in^2$

To count for D/C ratio (Demand/Capacity) = 0.6

$$A_{s\_req\_ns} := \frac{A_s}{0.6}$$ $A_{s\_req\_ns} = 6.749 \, in^2$

try 5#11 (N-S), As = 7.80 sq. in. O K

II-D Check In-Plane Shear Capacity **F2 = -1097 kip** **F1 =-107 kip (T)**

Compared with section cut at INTSCUT8a, loads are smaller, use design at INTSCUT8a

6.5.2.3 Roof Slab Design (48" thick), E-W Reinforcement - INTSCUT8c

slab width: $B := 37 \cdot ft$

Load Combination I - DL+LL+SRSS

Axial Compression:	**F2 = 542 kip (C)**
In-Plane Moment:	**M3 = 9680 kip-ft**
In-Plane Shear:	**F1 = 1061 kip**
Out-of-Plane Moment:	**M22 + M12 = 167 kip-ft/ft**
Out-of-Plane Shear:	**V13 = 24 kip/ft**

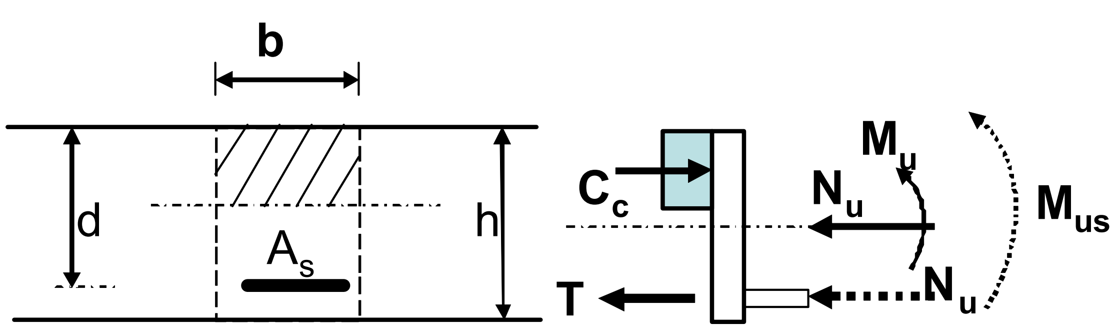
I-A Out-of-Plane Moment with Axial Compression

M22 + M12 = 167 kip-ft/ft **F2 = 542 kip (C)**

$M_{uo} := 167 \cdot \frac{kip \cdot ft}{ft}$ $N_u := \frac{542 \cdot kip}{B}$ $N_u = 14.649 \frac{kip}{ft}$

$b := 12 \cdot in$ $h := 48 \cdot in$ conc. cover for reinf.: $t_c := 2 \cdot in$

$d_{b11} := 1.41 \cdot in$ $d := h - t_c - d_{b11} \cdot 1.5$ (2nd layer) $d = 43.885\,in$



$M_{us} := M_{uo} + N_u \cdot \left(d - \frac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $M_{us} = 191.274\,kip \cdot \frac{ft}{ft}$

$m := \frac{f_y}{0.85 \cdot f'_c}$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$M_n := \frac{M_{us} \cdot b}{\phi_c}$ $M_n = 273.249\,kip \cdot ft$ $R_n := \frac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 141.881\,psi$

$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{f_y}}\right)$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 2.406 \times 10^{-3}$

$A_s := \rho \cdot b \cdot d - \dfrac{N_u \cdot b}{\phi_c \cdot f_y}$ (Commentary, Ref. 2.2.18) $A_s = 0.918\ in^2$

$A_{s\_min} := 0.0018 \cdot b \cdot h$ (Sect. 7.12.5, Ref. 2.2.14) $A_{s\_min} = 1.037\ in^2$

To count for D/C ratio (Demand/Capacity) = 0.6

$A_{s\_req} := \dfrac{A_s}{0.6}$ $A_{s\_req} = 1.53\ in^2$ try #11@12", As =1.56 in^2 O K

I-B Check Out-of-Plane Shear Capacity

Full Shear Capacity (neglect compression--- conservative)

$b = 12\ in$ $d = 43.885\ in$

$V_c := 2 \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d$ (Eq. 11-3, Ref. 2.2.14,) $V_c = 74.475\ kip$

$\phi_v \cdot V_c = 44.685\ kip$

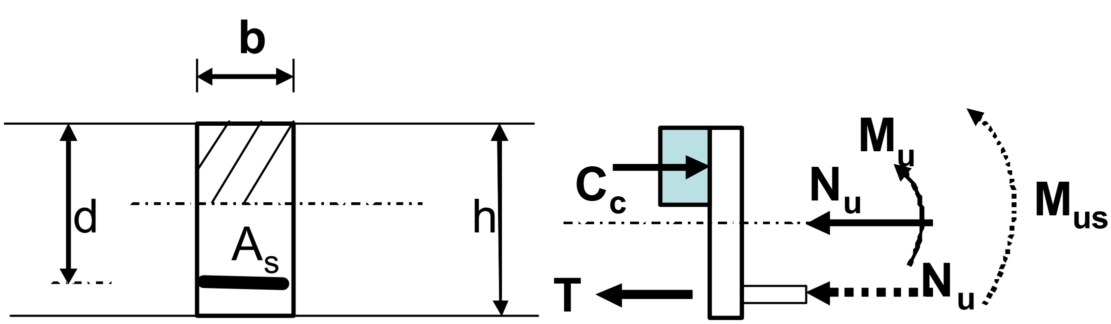
Out-of-Plane Shear: **V13 = 24 kip/ft**

$V_{uo} := 24 \cdot \dfrac{kip}{ft}$ $\dfrac{V_{uo} \cdot b}{\phi_v \cdot V_c} = 0.537$ < D/C = 0.6 O K

I-C Chord Reinf. at slab edge (east and west) due to In-Plane Moment

M3 = 9680 kip-ft **F2 = 542 kip (C)**

$M_{ui} := 9680 \cdot kip \cdot ft$ $N_u := 542 \cdot kip$ $b := 4 \cdot ft$ $h := 37 \cdot ft$ $d := h - 5 \cdot in$



$M_{us} := M_{ui} + N_u \cdot \left(d - \dfrac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $M_{us} = 1.948 \times 10^4\ kip \cdot ft$

$m := \dfrac{f_y}{0.85 \cdot f'_c}$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$M_n := \dfrac{M_{us}}{\phi_c}$ $M_n = 2.783 \times 10^4\,\text{kip}\cdot\text{ft}$ $R_n := \dfrac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 36.102\,\text{psi}$

$\rho := \dfrac{1}{m} \cdot \left(1 - \sqrt{1 - \dfrac{2 \cdot m \cdot R_n}{f_y}}\right)$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 6.043 \times 10^{-4}$

$A_s := \rho \cdot b \cdot d - \dfrac{N_u}{\phi_c \cdot f_y}$ (Commentary, Ref. 2.2.18) $A_s = -0.172\,\text{in}^2$ (tension reinf. not required----O K)

I-D Check In-Plane Shear Capacity **F1 = 1061 kip**

$V_{ui} := 1061 \cdot \text{kip}$ $b := 4 \cdot \text{ft}$ $h := 37 \cdot \text{ft}$ $A_g := b \cdot h$ $d := h - 5 \cdot \text{in}$

$V_c := 2 \cdot \sqrt{f'_c \cdot \text{psi}} \cdot b \cdot d$ (Eq. 11-3, Ref. 2.2.14,) $V_c = 2.98 \times 10^3\,\text{kip}$

$\dfrac{V_{ui}}{\phi_v \cdot V_c} = 0.593$ < D/C = 0.6 O K

Load Combination II - DL+LL-SRSS

Axial Compression:	**F2 = -683 kip (T)**
In-Plane Moment:	**M3 = -10030 kip-ft**
In-Plane Shear:	**F1 = -1028 kip**
Out-of-Plane Moment:	**M22 + M12 = -194 kip-ft/ft**
Out-of-Plane Shear:	**V13 = -24 kip/ft**

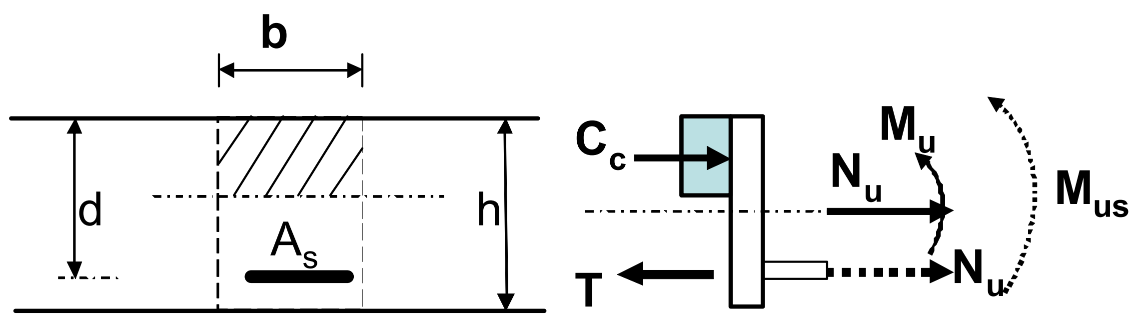
II-A Out-of-Plane Moment with Axial Tension slab width: $B := 37 \cdot \text{ft}$

M22 + M12 = -194 kip-ft/ft **F2 = -683 kip (T)**

$M_{uo} := 194 \cdot \dfrac{\text{kip} \cdot \text{ft}}{\text{ft}}$ $N_u := \dfrac{683 \cdot \text{kip}}{B}$ $N_u = 18.459\,\dfrac{\text{kip}}{\text{ft}}$

$b := 12 \cdot \text{in}$ $h := 48 \cdot \text{in}$ conc. cover for reinf.: $t_c := 2 \cdot \text{in}$

$d_{b11} := 1.41 \cdot \text{in}$ $d := h - t_c - d_{b11} \cdot 1.5$ (2nd layer) $d = 43.885\,\text{in}$



$M_{us} := M_{uo} - N_u \cdot \left(d - \frac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $M_{us} = 163.411\ \text{kip} \cdot \frac{\text{ft}}{\text{ft}}$

$m := \frac{f_y}{0.85 \cdot f'_c}$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$M_n := \frac{M_{us} \cdot b}{\phi_t}$ $M_n = 181.568\ \text{kip} \cdot \text{ft}$ $R_n := \frac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 94.277\ \text{psi}$

$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{f_y}}\right)$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 1.589 \times 10^{-3}$

$A_s := \rho \cdot b \cdot d + \frac{N_u \cdot b}{\phi_t \cdot f_y}$ (Commentary, Ref. 2.2.18) $A_s = 1.179\ \text{in}^2$

$A_{s\_min} := 0.0018 \cdot b \cdot h$ (Sect. 7.12.5, Ref. 2.2.14) $A_{s\_min} = 1.037\ \text{in}^2$

To count for D/C ratio (Demand/Capacity) = 0.6

$A_{s\_req} := \frac{A_s}{0.6}$ $A_{s\_req} = 1.965\ \text{in}^2$ try #11@9.5", As = 1.97 in^2 O K

<u>II-B Check Out-of-Plane Shear Capacity</u> **V13 = -24 kip/ft**

Shear Capacity with axial tension **F2 =-683 kip (T)** $B = 37\ \text{ft}$

$A_g := b \cdot h$ $h = 48\ \text{in}$ $A_g = 576\ \text{in}^2$ $N_u := \frac{-683 \cdot \text{kip}}{B}$ $N_u = -18.459\ \frac{\text{kip}}{\text{ft}}$

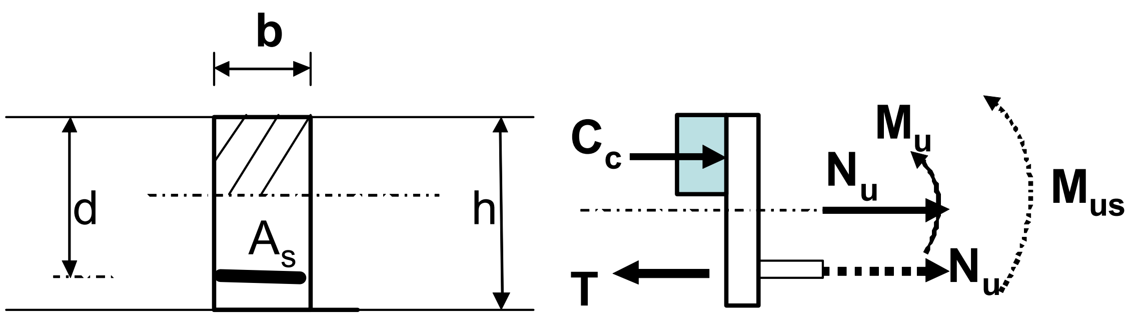
$V_c := 2 \cdot \left(1 + \frac{N_u \cdot b}{500 \cdot A_g \cdot \text{psi}}\right) \cdot \sqrt{f'_c \cdot \text{psi}} \cdot b \cdot d$ (Eq. 11-8, Ref. 2.2.14,) $V_c = 69.702\ \text{kip}$

$V_{uo} := 24 \cdot \frac{\text{kip}}{\text{ft}}$ $\frac{V_{uo} \cdot b}{\phi_v \cdot V_c} = 0.574$ < D/C = 0.6 O K

II-C Chord Reinf. at slab edge (east and west) due to In-Plane Moment

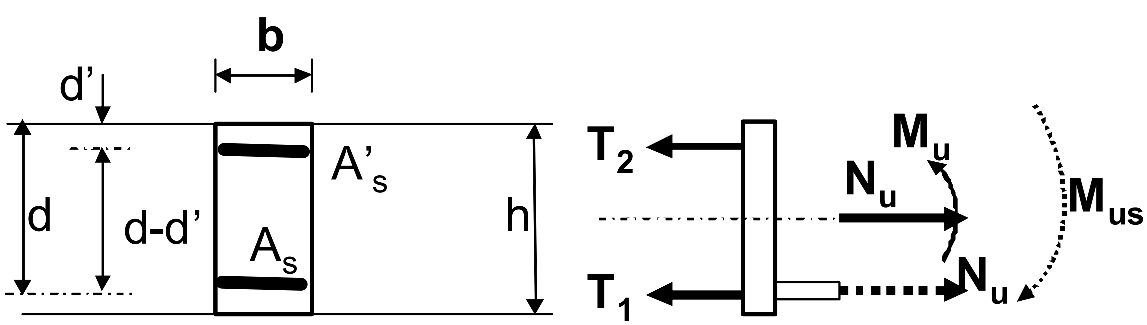
M3 = -10030 kip-ft **F2 = -683 kip (T)**

$M_{ui} := 10030 \cdot kip \cdot ft$ $N_u := 683 \cdot kip$ $b := 4 \cdot ft$ $h := 37 \cdot ft$ $d := h - 5 \cdot in$



$M_{us} := M_{ui} - N_u \cdot \left(d - \frac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $M_{us} = -2.321 \times 10^3 \, kip \cdot ft$

Axial tension Nu is large compared to moment, redesign entire section in tension



$d' := 5 \cdot in$ $A'_s := \frac{|M_{us}|}{\phi_b \cdot f_y \cdot (d - d')}$ (Commentary, Ref. 2.2.18) $A'_s = 1.188 \, in^2$

$A_s := \frac{N_u}{\phi_t \cdot f_y} - A'_s$ (Commentary, Ref. 2.2.18) $A_s = 11.46 \, in^2$

To count for D/C ratio (Demand/Capacity) = 0.6

$A_{s\_req\_ns} := \frac{A_s}{0.6}$ $A_{s\_req\_ns} = 19.1 \, in^2$

try 12#11 (N-S), As = 18.72 sq. in. O K

II-D Check In-Plane Shear Capacity **F1 = -1028 kip** **F2 =-683 kip (T)**

$V_{ui} := 1028 \cdot kip \qquad N_u := -683 \cdot kip$

$b := 4 \cdot ft \qquad h = 37\,ft \qquad A_g := b \cdot h \qquad d := h - 5 \cdot in$

$V_c := 2 \cdot \left(1 + \dfrac{N_u}{500 \cdot A_g \cdot psi}\right) \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d$ (Eq. 11-8, Ref. 2.2.14,) $V_c = 2.789 \times 10^3\,kip$

$\dfrac{V_{ui}}{\phi_v \cdot V_c} = 0.614$ close to D/C = 0.6 O K

6.5.2.4 Roof Slab Design (48" thick), E-W Reinforcement - INTSCUT8d slab width: $B := 37 \cdot ft$

Load Combination I - DL+LL+SRSS

Axial Compression:	**F2 = 823 kip (C)**
In-Plane Moment:	**M3 = 6198 kip-ft**
In-Plane Shear:	**F1 = 1283 kip**
Out-of-Plane Moment:	**M22 + M12 = 167 kip-ft/ft**
Out-of-Plane Shear:	**V13 = 24 kip/ft**

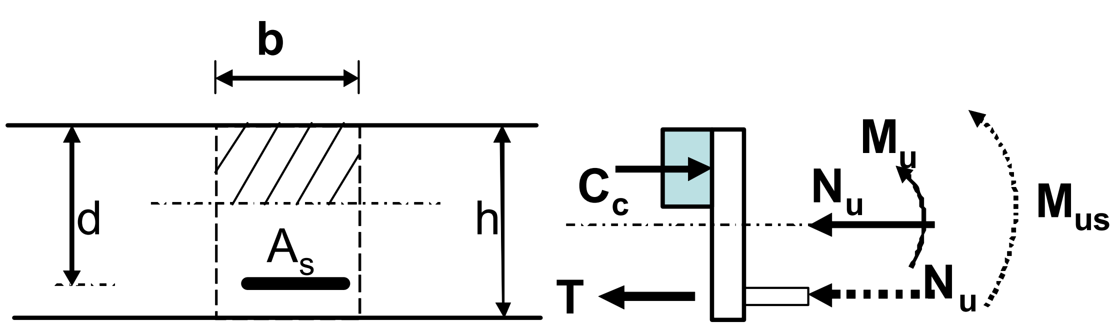
I-A Out-of-Plane Moment with Axial Compression

M22 + M12 = 167 kip-ft/ft **F2 = 823 kip (C)**

$M_{uo} := 167 \cdot \frac{kip \cdot ft}{ft}$ $N_u := \frac{823 \cdot kip}{B}$ $N_u = 22.243 \frac{kip}{ft}$

$b := 12 \cdot in$ $h := 48 \cdot in$ conc. cover for reinf.: $t_c := 2 \cdot in$

$d_{b11} := 1.41 \cdot in$ $d := h - t_c - d_{b11} \cdot 1.5$ (2nd layer) $d = 43.885\,in$



$M_{us} := M_{uo} + N_u \cdot \left(d - \frac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $M_{us} = 203.859\,kip \cdot \frac{ft}{ft}$

$m := \frac{f_y}{0.85 \cdot f'_c}$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$M_n := \frac{M_{us} \cdot b}{\phi_c}$ $M_n = 291.227\,kip \cdot ft$ $R_n := \frac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 151.217\,psi$

$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{f_y}}\right)$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 2.567 \times 10^{-3}$

$$A_s := \rho \cdot b \cdot d - \frac{N_u \cdot b}{\phi_c \cdot f_y}$$ (Commentary, Ref. 2.2.18) $A_s = 0.822\ \text{in}^2$

$$A_{s\_min} := 0.0018 \cdot b \cdot h$$ (Sect. 7.12.5, Ref. 2.2.14) $A_{s\_min} = 1.037\ \text{in}^2$

To count for D/C ratio (Demand/Capacity) = 0.6

$$A_{s\_req} := \frac{A_s}{0.6}$$ $A_{s\_req} = 1.37\ \text{in}^2$ try #11@12", As = 1.56 in^2 O K

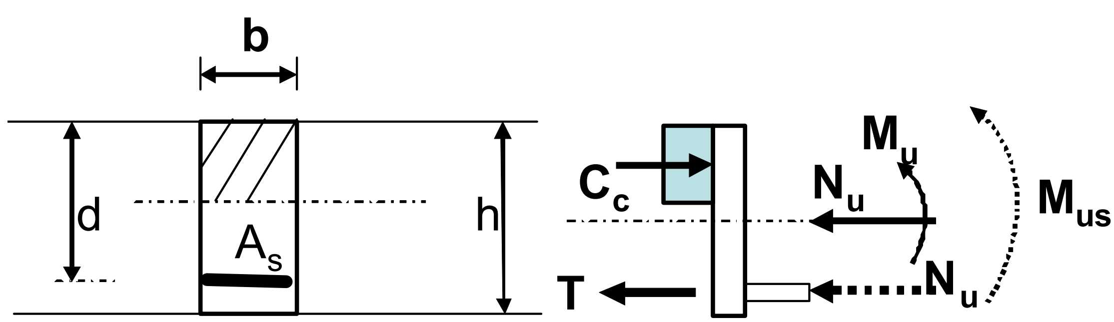
I-B Check Out-of-Plane Shear Capacity **V13 = 24 kip/ft**

same as INTSCUT8c O K

I-C Chord Reinf. at slab edge (east and west) due to In-Plane Moment

M3 = 6198 kip-ft **F2 = 823 kip (C)**

$M_{ui} := 6198 \cdot \text{kip} \cdot \text{ft}$ $N_u := 823 \cdot \text{kip}$ $b := 4 \cdot \text{ft}$ $h := 37 \cdot \text{ft}$ $d := h - 5 \cdot \text{in}$



$$M_{us} := M_{ui} + N_u \cdot \left(d - \frac{h}{2}\right)$$ (Commentary, Ref. 2.2.18) $M_{us} = 2.108 \times 10^4\ \text{kip} \cdot \text{ft}$

$$m := \frac{f_y}{0.85 \cdot f'_c}$$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$$M_n := \frac{M_{us}}{\phi_c}$$ $M_n = 3.012 \times 10^4\ \text{kip} \cdot \text{ft}$ $R_n := \frac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 39.066\ \text{psi}$

$$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{f_y}}\right)$$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 6.541 \times 10^{-4}$

$$A_s := \rho \cdot b \cdot d - \frac{N_u}{\phi_c \cdot f_y}$$ (Commentary, Ref. 2.2.18) $A_s = -5.812\ \text{in}^2$ (tension reinf. not required----O K)

I-D Check In-Plane Shear Capacity **F1 = 1283 kip**

$V_{ui} := 1283 \cdot kip$ $b := 4 \cdot ft$ $h := 37 \cdot ft$ $A_g := b \cdot h$ $d := h - 5 \cdot in$

$V_c := 2 \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d$ (Eq. 11-3, Ref. 2.2.14,) $V_c = 2.98 \times 10^3 \, kip$

$\frac{V_{ui}}{\phi_v \cdot V_c} = 0.718$ > D/C = 0.6 N G design shear reinf.for slab similar to wall panel

provide #11@8" Top and Bottom: $A_{s11} := 1.56 \cdot in^2$ $s_2 := 8 \cdot in$

Reinf. required for Out-of-Plane Moment: $A_{s\_opm} := 0.822 \cdot \frac{in^2}{ft}$

$A_v := 2 \cdot A_{s11} - A_{s\_opm} \cdot s_2$ $A_v = 2.572 \, in^2$

$l_w := 37 \cdot ft$ $d := 0.8 \cdot l_w$ (Section 11.10.4, Ref. 2.2.14)

$V_s := \frac{A_v \cdot f_y \cdot d}{s_2}$ (Eq. 11-33, Ref. 2.2.14) $V_s = 6.852 \times 10^3 \, kip$

$\frac{V_{ui}}{\phi_v \cdot (V_c + V_s)} = 0.217$ < D/C = 0.6 O K

Load Combination II - DL+LL-SRSS

Axial Tension: **F2 = -761 kip (T)**

In-Plane Moment: **M3 = -5869kip-ft**

In-Plane Shear: **F1 = -1315 kip**

Out-of-Plane Moment: **M22 + M12 = -194 kip-ft/ft**

Out-of-Plane Shear: **V13 = -24 kip/ft**

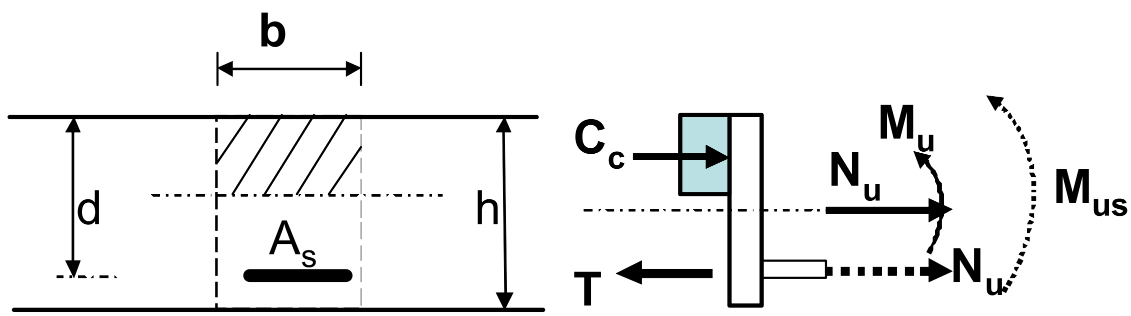
II-A Out-of-Plane Moment with Axial Tension slab width: $B := 37 \cdot ft$

M22 + M12 = -194 kip-ft/ft **F2 = -761 kip (T)**

$M_{uo} := 194 \cdot \frac{kip \cdot ft}{ft}$ $N_u := \frac{761 \cdot kip}{B}$ $N_u = 20.568 \, \frac{kip}{ft}$

$b := 12 \cdot in$ $h := 48 \cdot in$ conc. cover for reinf.: $t_c := 2 \cdot in$

$d_{b11} := 1.41 \cdot in$ $d := h - t_c - d_{b11} \cdot 1.5$ (2nd layer) $d = 43.885\,in$



$M_{us} := M_{uo} - N_u \cdot \left(d - \frac{h}{2} \right)$ (Commentary, Ref. 2.2.18) $M_{us} = 159.918\,kip \cdot \frac{ft}{ft}$

$m := \frac{f_y}{0.85 \cdot f'_c}$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$M_n := \frac{M_{us} \cdot b}{\phi_t}$ $M_n = 177.686\,kip \cdot ft$ $R_n := \frac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 92.262\,psi$

$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{f_y}} \right)$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 1.555 \times 10^{-3}$

$A_s := \rho \cdot b \cdot d + \frac{N_u \cdot b}{\phi_t \cdot f_y}$ (Commentary, Ref. 2.2.18) $A_s = 1.2\,in^2$

$A_{s\_min} := 0.0018 \cdot b \cdot h$ (Sect. 7.12.5, Ref. 2.2.14) $A_{s\_min} = 1.037\,in^2$

To count for D/C ratio (Demand/Capacity) = 0.6

$A_{s\_req} := \frac{A_s}{0.6}$ $A_{s\_req} = 1.999\,in^2$ try #11@8", As =2.34 in^2 O K

II-B Check Out-of-Plane Shear Capacity **V13 = -24 kip/ft**

Shear Capacity with axial tension **F2 =-761 kip (T)** $B = 37\,ft$

$A_g := b \cdot h$ $h = 48\,in$ $A_g = 576\,in^2$ $N_u := \frac{-761 \cdot kip}{B}$ $N_u = -20.568\,\frac{kip}{ft}$

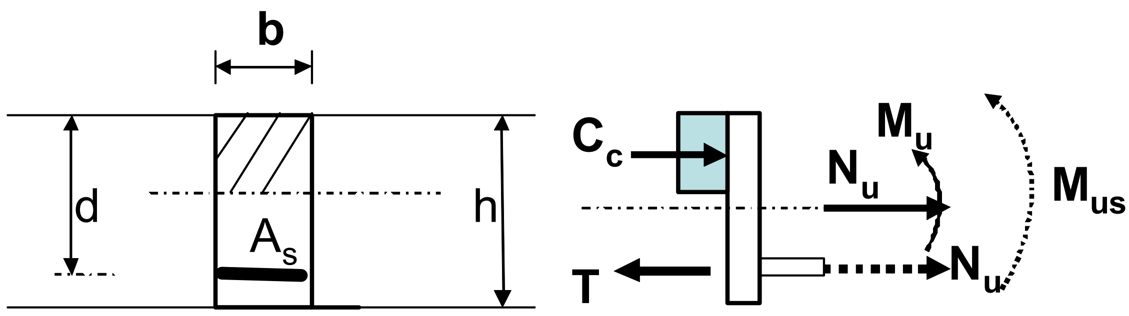
$V_c := 2 \cdot \left(1 + \frac{N_u \cdot b}{500 \cdot A_g \cdot psi} \right) \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d$ (Eq. 11-8, Ref. 2.2.14,) $V_c = 69.157\,kip$

$V_{uo} := 24 \cdot \frac{kip}{ft}$ $\frac{V_{uo} \cdot b}{\phi_v \cdot V_c} = 0.578$ < D/C = 0.6 O K

II-C Chord Reinf. at slab edge (east and west) due to In-Plane Moment

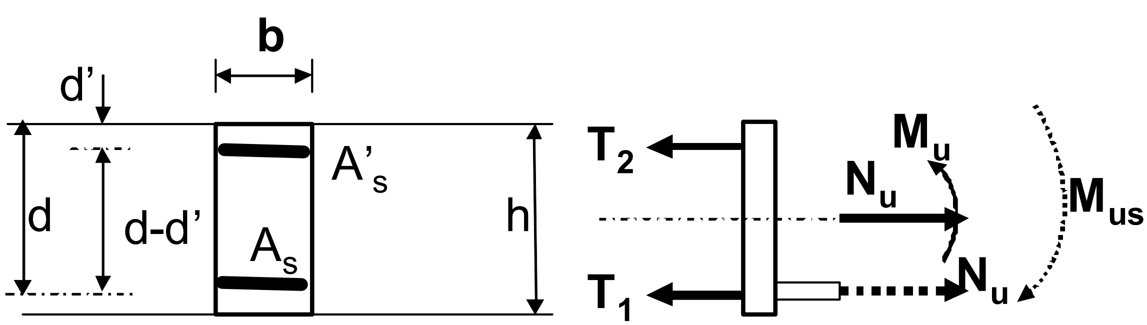
M3 = -5869 kip-ft **F2 = -761 kip (T)**

$M_{ui} := 5869 \cdot kip \cdot ft$ $N_u := 761 \cdot kip$ $b := 4 \cdot ft$ $h := 37 \cdot ft$ $d := h - 5 \cdot in$



$M_{us} := M_{ui} - N_u \cdot \left(d - \frac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $M_{us} = -7.892 \times 10^3 \, kip \cdot ft$

Axial tension Nu is large compared to moment, redesign entire section in tension



$d' := 5 \cdot in$ $A'_s := \frac{|M_{us}|}{\phi_b \cdot f_y \cdot (d - d')}$ (Commentary, Ref. 2.2.18) $A'_s = 4.041 \, in^2$

$A_s := \frac{N_u}{\phi_t \cdot f_y} - A'_s$ (Commentary, Ref. 2.2.18) $A_s = 10.051 \, in^2$

To count for D/C ratio (Demand/Capacity) = 0.6

$A_{s\_req\_ns} := \frac{A_s}{0.6}$ $A_{s\_req\_ns} = 16.752 \, in^2$ try 12#11 (N-S), As = 18.72 sq. in. O K

II-D Check In-Plane Shear Capacity **F1 = -1315 kip** **F2 =-761 kip (T)**

$V_{ui} := 1315 \cdot kip \qquad N_u := -761 \cdot kip$

$b := 4 \cdot ft \qquad h = 37\,ft \qquad A_g := b \cdot h \qquad d := h - 5 \cdot in$

$V_c := 2 \cdot \left(1 + \frac{N_u}{500 \cdot A_g \cdot psi}\right) \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d$ (Eq. 11-8, Ref. 2.2.14,) $V_c = 2.767 \times 10^3\,kip$

$\frac{V_{ui}}{\phi_v \cdot V_c} = 0.792$ > D/C = 0.6 N G design shear reinf.for slab similar to wall panel

provide #11@8" Top and Bottom: $A_{s11} := 1.56 \cdot in^2 \qquad s_2 := 8 \cdot in$

Reinf. required for Out-of-Plane Moment: $A_{s\_opm} := 1.2 \cdot \frac{in^2}{ft}$

$A_v := 2 \cdot A_{s11} - A_{s\_opm} \cdot s_2 \qquad A_v = 2.32\,in^2$

$l_w := 37 \cdot ft \qquad d := 0.8 \cdot l_w$ (Section 11.10.4, Ref. 2.2.14)

$V_s := \frac{A_v \cdot f_y \cdot d}{s_2}$ (Eq. 11-33, Ref. 2.2.14) $V_s = 6.18 \times 10^3\,kip$

$\frac{V_{ui}}{\phi_v \cdot (V_c + V_s)} = 0.245$ < D/C = 0.6 O K

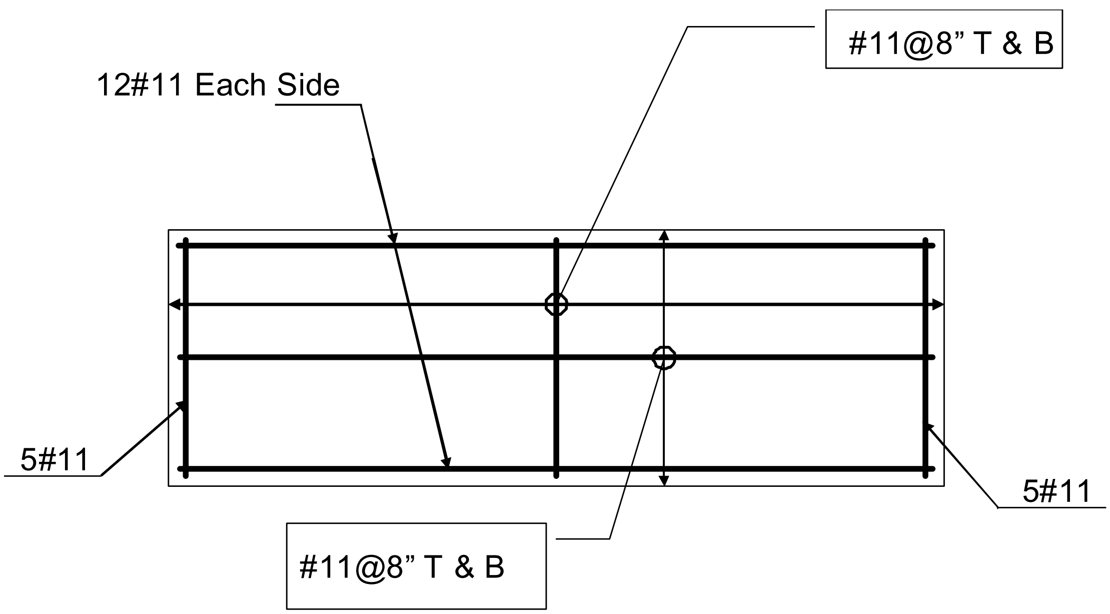
For simplicity of placing rebars, use Reinforcement as follow:

In North-South direction (short bar):

Provide #11@8" T & B, added Chord Bar 5#11 at east and west slab edges

In East-West direction (long bar):

Provide #11@8" T & B, added Chord Bar 12#11 at north and south slab edges



6.5.3

Roof Slab Design Loads, Column Lines 7-8

Roof Slab EL 37'-0"

Section cut design forces and moments, which follow a global axis system, for the roof slab at EL 37'-0" from column line 7 to 8 include in plane forces and moments: such as axial forces (tension/compression), in plane moment, and in plane shear. The section cut values are integrated along the section cut length, thus for INTSCUT9a the length is equal to 25' and for INTSCUT9 b the length is equal to 37'. Out of plane values such as out of plane bending and out of plane shear are attained by shell element forces and moments from SAP2000, which follow a local axis system. The out of plane bending moments provided are M11 and M22, and a third moment M12, which is a twisting moment that is combined with M22 and M11. The out of plane shear forces from the shell elements include V13 and V23.
Follow notation below to convert from SAP2000 labeling (i.e. F2, M22…) to appropriate design forces and moments.

INTSCUT9a
F1 = Axial Force, Compression (+) / Tension (-)
F2 = In Plane Shear
M3 = In Plane Moment

INTSCUT9b
F1 = In Plane Shear
F2 = Axial Force, Compression (+) / Tension (-)
M3 = In Plane Moment

Shell Element Forces and Moments
M11 = Out of Plane Moment
M22 = Out of Plane Moment
M12 = Twisting Moment
V13 = Out of Plane Shear
V23 = Out of Plane Shear

Accidental torsion factor= 15% (See assumption 3.1.4)

Section Cu	DL+LL						Seismic (SRSS)					
	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
NTSCUT9a	-30	-8	-	-	-	30	55	233	-	-	-	402
NTSCUT9b	2	-32	-	-	-	-86	210	423	-	-	-	3305

Element Forces - Area Shells D+L						Element Forces - Area Shells SRSS					
MAX	M11	M22	M12	V13	V23	MAX	M11	M22	M12	V13	V23
MIN	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft		Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	16	37	7	5	9	MAX	24	51	14	5	10
MIN	-16	-48	-6	-4	-6						

Loads with accidental torsion factor

Section Cu	DL+LL						Seismic (SRSS)					
	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
NTSCUT9a	-35	-9	-	-	-	34	63	268	-	-	-	462
NTSCUT9b	2	-37	-	-	-	-99	242	487	-	-	-	3801

Loads with accidental torsion factor

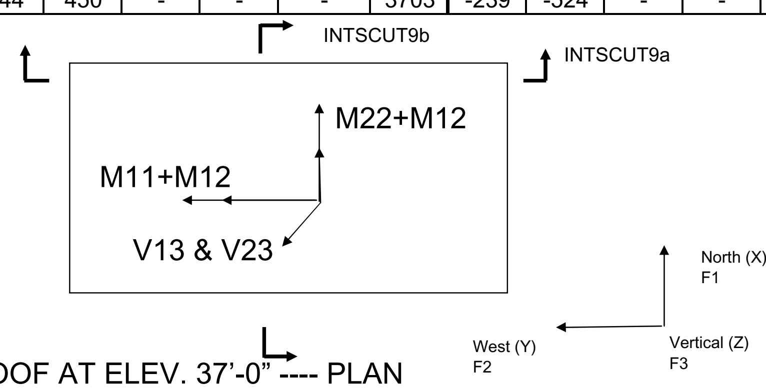
Element Forces - Area Shells D+L						Element Forces - Area Shells SRSS					
MAX	M11	M22	M12	V13	V23	MAX	M11	M22	M12	V13	V23
MIN	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft		Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	19	43	9	6	11	MAX	27	58	16	5	11
MIN	-18	-55	-6	-5	-7						

Maximum Load Combination

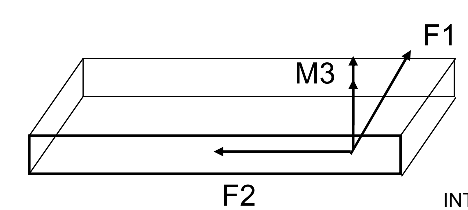
Section Cu	DL+LL+Seismic (SRSS)						DL+LL-Seismic (SRSS)					
	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
NTSCUT9a	28	259	-	-	-	496	-98	-277	-	-	-	-428
NTSCUT9b	244	450	-	-	-	3703	-239	-524	-	-	-	-3900

Maximum Load Combination

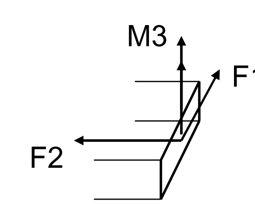
DL+LL+Seismic(SRSS)						DL+LL-Seismic(SRSS)					
MAX	M11	M22	M12	V13	V23	MAX	M11	M22	M12	V13	V23
MIN	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft	MIN	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	46	102	24	11	22	MAX	-9	-15	-7	1	-1
MIN	9	3	9	0	4	MIN	-46	-114	-22	-10	-19



ROOF AT ELEV. 37'-0" ---- PLAN



INTSCUT9a



INTSCUT9b

6.5.3.1 Roof Slab Design (48" thick), N-S Reinforcement - INTSCUT9a

slab width: $B := 25 \cdot ft$

$f'_c := 5000 \cdot psi$ $f_y := 60 \cdot ksi$

$\phi_b := 0.9$ Sect. 9.3.2.1, Ref. 2.2.14

$\phi_t := 0.9$ Sect. 9.3.2.2(a), Ref. 2.2.14

$\phi_c := 0.7$ Sect. 9.3.2.2(b), Ref. 2.2.14

$\phi_v := 0.6$ Sect. 9.3.4, Ref. 2.2.14

Load Combination I - DL+LL+SRSS

Axial Compression:	**F1 = 28 kip (C)**
In-Plane Moment:	**M3 = 496 kip-ft**
In-Plane Shear:	**F2 = 259 kip**
Out-of-Plane Moment:	**M11 + M12 = 70 kip-ft/ft**
Out-of-Plane Shear:	**V23 = 22 kip/ft**

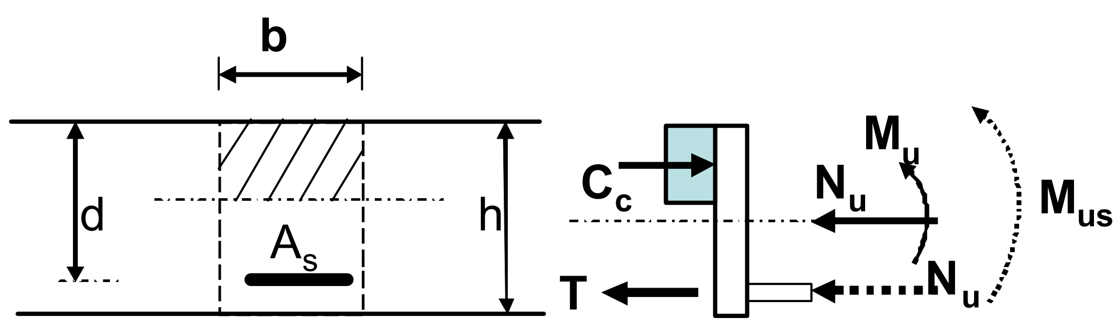
I-A Out-of-Plane Moment with Axial Compression

M11 + M12 = 70 kip-ft/ft **F1 = 28 kip (C)**

$M_{uo} := 70 \cdot \frac{kip \cdot ft}{ft}$ $N_u := \frac{28 \cdot kip}{B}$ $N_u = 1.12 \frac{kip}{ft}$

$b := 12 \cdot in$ $h := 48 \cdot in$ conc. cover for reinf.: $t_c := 2 \cdot in$

$d_{b11} := 1.41 \cdot in$ $d := h - t_c - d_{b11} \cdot 1.5$ (2nd layer) $d = 43.885\, in$



$M_{us} := M_{uo} + N_u \cdot \left(d - \frac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $M_{us} = 71.856\, kip \cdot \frac{ft}{ft}$

$m := \frac{f_y}{0.85 \cdot f'_c}$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$M_n := \frac{M_{us} \cdot b}{\phi_c}$ $M_n = 102.651\, kip \cdot ft$ $R_n := \frac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 53.301\, psi$

$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{f_y}}\right)$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 8.94 \times 10^{-4}$

$A_s := \rho \cdot b \cdot d - \frac{N_u \cdot b}{\phi_c \cdot f_y}$ (Commentary, Ref. 2.2.18) $A_s = 0.444\, in^2$

$A_{s\_min} := 0.0018 \cdot b \cdot h$ (Sect. 7.12.5, Ref. 2.2.14) $A_{s\_min} = 1.037\,in^2$

To count for D/C ratio (Demand/Capacity) = 0.6

$A_{s\_req} := \frac{A_s}{0.6}$ $A_{s\_req} = 0.74\,in^2$ try #9@12", As = 1.00 in^2 O K

I-B Check Out-of-Plane Shear Capacity

Full Shear Capacity (neglect compression--- conservative)

$b = 12\,in$ $d = 43.885\,in$

$V_c := 2 \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d$ (Eq. 11-3, Ref. 2.2.14,) $V_c = 74.475\,kip$

$\phi_v \cdot V_c = 44.685\,kip$

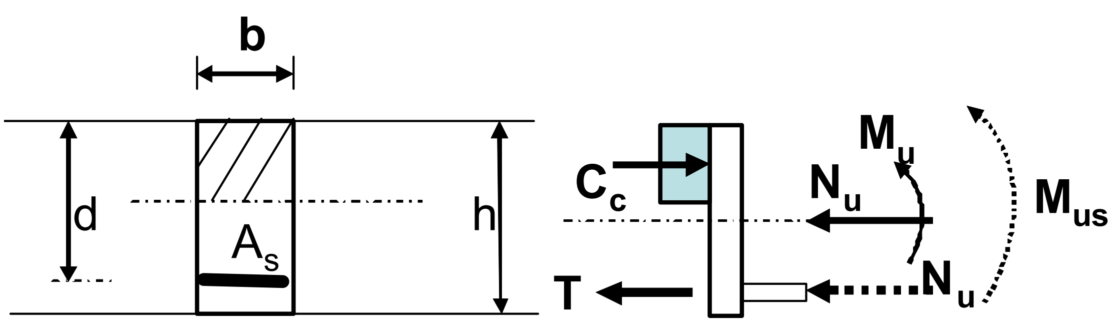
Out-of-Plane Shear: **V23 = 22 kip/ft**

$V_{uo} := 22 \cdot \frac{kip}{ft}$ $\frac{V_{uo} \cdot b}{\phi_v \cdot V_c} = 0.492$ < D/C = 0.6 O K

I-C Chord Reinf. at slab edge (east and west) due to In-Plane Moment

M3 = 496 kip-ft **F1 = 28 kip (C)**

$M_{ui} := 496 \cdot kip \cdot ft$ $N_u := 28 \cdot kip$ $b := 4 \cdot ft$ $h := 25 \cdot ft$ $d := h - 5 \cdot in$



$M_{us} := M_{ui} + N_u \cdot \left(d - \frac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $M_{us} = 834.333\,kip \cdot ft$

$m := \frac{f_y}{0.85 \cdot f'_c}$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$M_n := \frac{M_{us}}{\phi_c}$ $M_n = 1.192 \times 10^3\,kip \cdot ft$ $R_n := \frac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 3.424\,psi$

$$\rho := \frac{1}{m}\cdot\left(1-\sqrt{1-\frac{2\cdot m\cdot R_n}{f_y}}\right)$$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 5.709\times10^{-5}$

$$A_s := \rho\cdot b\cdot d - \frac{N_u}{\phi_c\cdot f_y}$$ (Commentary, Ref. 2.2.18) $A_s = 0.142\ in^2$ (negligible----O K)

I-D Check In-Plane Shear Capacity **F2 = 259 kip**

$V_{ui} := 259\cdot kip$ $b := 4\cdot ft$ $h = 25\ ft$ $A_g := b\cdot h$ $d := h - 5\cdot in$

$V_c := 2\cdot\sqrt{f'_c\cdot psi}\cdot b\cdot d$ (Eq. 11-3, Ref. 2.2.14,) $V_c = 2.003\times10^3\ kip$

$\frac{V_{ui}}{\phi_v\cdot V_c} = 0.216$ < D/C = 0.6 O K

Load Combination II - DL+LL-SRSS

Axial Tension: **F1 = -98 kip (T)**

In-Plane Moment: **M3 = -428 kip-ft**

In-Plane Shear: **F2 = -277 kip**

Out-of-Plane Moment: **M11 + M12 = -68 kip-ft/ft**

Out-of-Plane Shear: **V23 = -19 kip/ft**

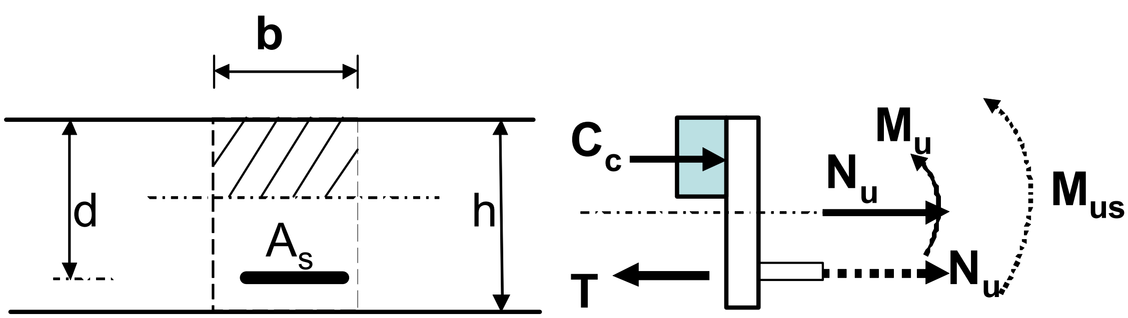
II-A Out-of-Plane Moment with Axial Tension slab width: $B := 25\cdot ft$

M11 + M12 = -68 kip-ft/ft **F1 = -98 kip (T)**

$M_{uo} := 68\cdot\frac{kip\cdot ft}{ft}$ $N_u := \frac{98\cdot kip}{B}$ $N_u = 3.92\frac{kip}{ft}$

$b := 12\cdot in$ $h := 48\cdot in$ conc. cover for reinf.: $t_c := 2\cdot in$

$d_{b11} := 1.41\cdot in$ $d := h - t_c - d_{b11}\cdot1.5$ (2nd layer) $d = 43.885\ in$



$M_{us} := M_{uo} - N_u \cdot \left(d - \frac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $M_{us} = 61.504\,kip \cdot \frac{ft}{ft}$

$m := \frac{f_y}{0.85 \cdot f_c}$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$M_n := \frac{M_{us} \cdot b}{\phi_t}$ $M_n = 68.338\,kip \cdot ft$ $R_n := \frac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 35.484\,psi$

$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{f_y}}\right)$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 5.939 \times 10^{-4}$

$A_s := \rho \cdot b \cdot d + \frac{N_u \cdot b}{\phi_t \cdot f_y}$ (Commentary, Ref. 2.2.18) $A_s = 0.385\,in^2$

$A_{s\_min} := 0.0018 \cdot b \cdot h$ (Sect. 7.12.5, Ref. 2.2.14) $A_{s\_min} = 1.037\,in^2$

To count for D/C ratio (Demand/Capacity) = 0.6

$A_{s\_req} := \frac{A_s}{0.6}$ $A_{s\_req} = 0.642\,in^2$ try #9@12", As = 100 in^2 O K

II-B Check Out-of-Plane Shear Capacity **V23 = -19 kip/ft**

Shear Capacity with axial tension **F1 =-98 kip (T)**

$A_g := b \cdot h$ $h = 48\,in$ $A_g = 576\,in^2$ $N_u := \frac{-98 \cdot kip}{B}$ $N_u = -3.92\,\frac{kip}{ft}$

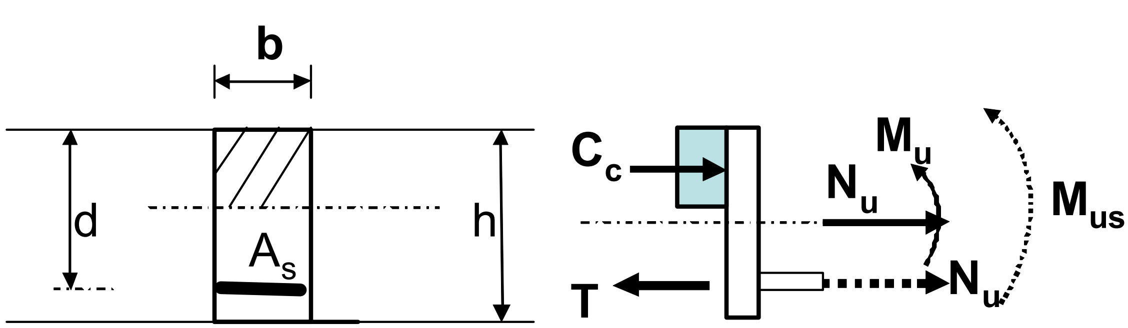
$V_c := 2 \cdot \left(1 + \frac{N_u \cdot b}{500 \cdot A_g \cdot psi}\right) \cdot \sqrt{f_c \cdot psi} \cdot b \cdot d$ (Eq. 11-8, Ref. 2.2.14,) $V_c = 73.462\,kip$

$V_{uo} := 19 \cdot \frac{kip}{ft}$ $\frac{V_{uo} \cdot b}{\phi_v \cdot V_c} = 0.431$ < D/C = 0.6 O K

II-C Chord Reinf. at slab edge (east and west) due to In-Plane Moment

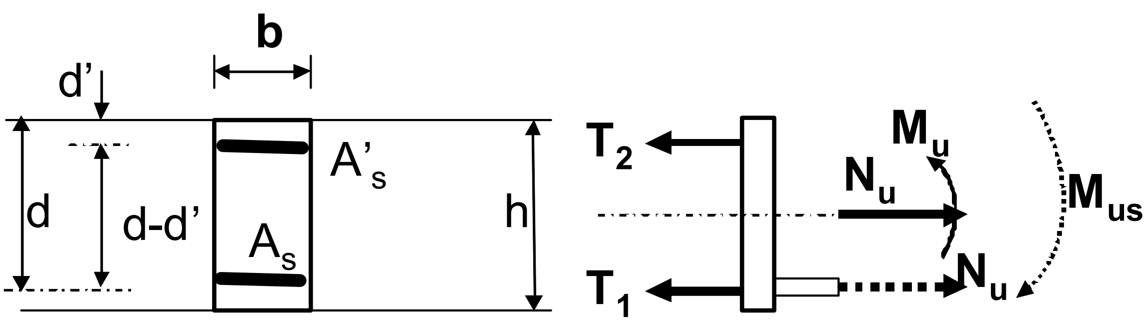
M3 = -428 kip-ft **F1 = -98 kip (T)**

$M_{ui} := 428 \cdot kip \cdot ft$ $N_u := 98 \cdot kip$ $b := 4 \cdot ft$ $h := 25 \cdot ft$ $d := h - 5 \cdot in$



$M_{us} := M_{ui} - N_u \cdot \left(d - \frac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $M_{us} = -756.167\ \text{kip} \cdot \text{ft}$

Axial tension Nu is large compared to moment, redesign entire section in tension



$d' := 5 \cdot \text{in}$ $A'_s := \frac{|M_{us}|}{\phi_b \cdot f_y \cdot (d - d')}$ (Commentary, Ref. 2.2.18) $A'_s = 0.579\ \text{in}^2$

$A_s := \frac{N_u}{\phi_t \cdot f_y} - A'_s$ (Commentary, Ref. 2.2.18) $A_s = 1.235\ \text{in}^2$

To count for D/C ratio (Demand/Capacity) = 0.6

$A_{s\_req\_ns} := \frac{A_s}{0.6}$ $A_{s\_req\_ns} = 2.059\ \text{in}^2$

try 2#11 (N-S), As = 3.12 sq. in. O K

II-D Check In-Plane Shear Capacity **F2 = -277 kip** **F1 =-98 kip (T)**

$V_{ui} := 277 \cdot \text{kip}$ $N_u := -98 \cdot \text{kip}$

$b := 4 \cdot \text{ft}$ $h = 25\ \text{ft}$ $A_g := b \cdot h$ $d := h - 5 \cdot \text{in}$

$V_c := 2 \cdot \left(1 + \frac{N_u}{500 \cdot A_g \cdot \text{psi}}\right) \cdot \sqrt{f'_c \cdot \text{psi}} \cdot b \cdot d$ (Eq. 11-8, Ref. 2.2.14,) $V_c = 1.975 \times 10^3\ \text{kip}$

$\frac{V_{ui}}{\phi_v \cdot V_c} = 0.234$ < D/C = 0.6 O K

6.5.3.2 Roof Slab Design (48" thick), E-W Reinforcement - INTSCUT9b

slab width: $B := 37 \cdot ft$

Load Combination I - DL+LL+SRSS

Axial Compression:	**F2 = 450 kip (C)**
In-Plane Moment:	**M3 = 3703 kip-ft**
In-Plane Shear:	**F1 = 244 kip**
Out-of-Plane Moment:	**M22 + M12 = 126 kip-ft/ft**
Out-of-Plane Shear:	**V13 = 22 kip/ft**

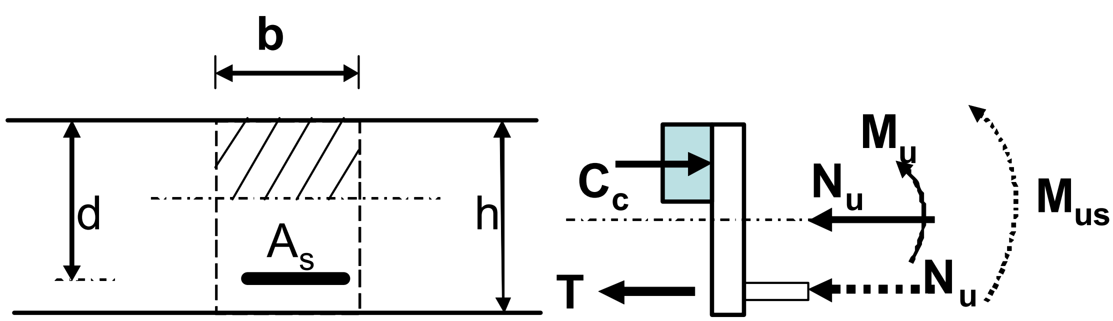
I-A Out-of-Plane Moment with Axial Compression

M22 + M12 = 126 kip-ft/ft **F2 = 450 kip (C)**

$M_{uo} := 126 \cdot \frac{kip \cdot ft}{ft}$ $N_u := \frac{450 \cdot kip}{B}$ $N_u = 12.162 \frac{kip}{ft}$

$b := 12 \cdot in$ $h := 48 \cdot in$ conc. cover for reinf.: $t_c := 2 \cdot in$

$d_{b11} := 1.41 \cdot in$ $d := h - t_c - d_{b11} \cdot 1.5$ (2nd layer) $d = 43.885\,in$



$M_{us} := M_{uo} + N_u \cdot \left(d - \frac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $M_{us} = 146.154\,kip \cdot \frac{ft}{ft}$

$m := \frac{f_y}{0.85 \cdot f'_c}$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$M_n := \frac{M_{us} \cdot b}{\phi_c}$ $M_n = 208.791\,kip \cdot ft$ $R_n := \frac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 108.413\,psi$

$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{f_y}}\right)$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 1.831 \times 10^{-3}$

$A_s := \rho \cdot b \cdot d - \dfrac{N_u \cdot b}{\phi_c \cdot f_y}$ (Commentary, Ref. 2.2.18) $A_s = 0.674\ in^2$

$A_{s\_min} := 0.0018 \cdot b \cdot h$ (Sect. 7.12.5, Ref. 2.2.14) $A_{s\_min} = 1.037\ in^2$

To count for D/C ratio (Demand/Capacity) = 0.6

$A_{s\_req} := \dfrac{A_s}{0.6}$ $A_{s\_req} = 1.124\ in^2$ try #11@12", As =1.56 in^2 O K

I-B Check Out-of-Plane Shear Capacity

Full Shear Capacity (neglect compression--- conservative)

$b = 12\,in$ $d = 43.885\,in$

$V_c := 2 \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d$ (Eq. 11-3, Ref. 2.2.14,) $V_c = 74.475\,kip$

$\phi_v \cdot V_c = 44.685\,kip$

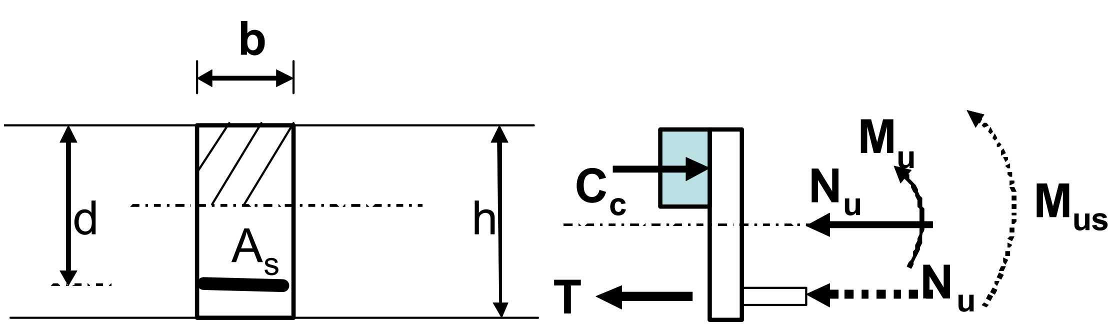
Out-of-Plane Shear: **V23 = 22 kip/ft**

$V_{uo} := 22 \cdot \dfrac{kip}{ft}$ $\dfrac{V_{uo} \cdot b}{\phi_v \cdot V_c} = 0.492$ < D/C = 0.6 O K

I-C Chord Reinf. at slab edge (east and west) due to In-Plane Moment

M3 = 3707 kip-ft **F2 = 450 kip (C)**

$M_{ui} := 3707 \cdot kip \cdot ft$ $N_u := 450 \cdot kip$ $b := 4 \cdot ft$ $h := 37 \cdot ft$ $d := h - 5 \cdot in$



$M_{us} := M_{ui} + N_u \cdot \left(d - \dfrac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $M_{us} = 1.184 \times 10^4\,kip \cdot ft$

$m := \dfrac{f_y}{0.85 \cdot f'_c}$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$M_n := \dfrac{M_{us}}{\phi_c}$ $M_n = 1.692 \times 10^4\,kip \cdot ft$ $R_n := \dfrac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 21.95\,psi$

$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{f_y}}\right)$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 3.668 \times 10^{-4}$

$A_s := \rho \cdot b \cdot d - \frac{N_u}{\phi_c \cdot f_y}$ (Commentary, Ref. 2.2.18) $A_s = -2.986\,in^2$ (tension reinf. not required----O K)

I-D Check In-Plane Shear Capacity **F1 = 244 kip**

$V_{ui} := 244 \cdot kip$ $b := 4 \cdot ft$ $h := 37 \cdot ft$ $A_g := b \cdot h$ $d := h - 5 \cdot in$

$V_c := 2 \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d$ (Eq. 11-3, Ref. 2.2.14,) $V_c = 2.98 \times 10^3\,kip$

$\frac{V_{ui}}{\phi_v \cdot V_c} = 0.136$ < D/C = 0.6 O K

Load Combination II - DL+LL-SRSS

Axial Compression: **F2 = -524 kip (T)**

In-Plane Moment: **M3 = -3900 kip-ft**

In-Plane Shear: **F1 = -239 kip**

Out-of-Plane Moment: **M22 + M12 = -136 kip-ft/ft**

Out-of-Plane Shear: **V23 = -19 kip/ft**

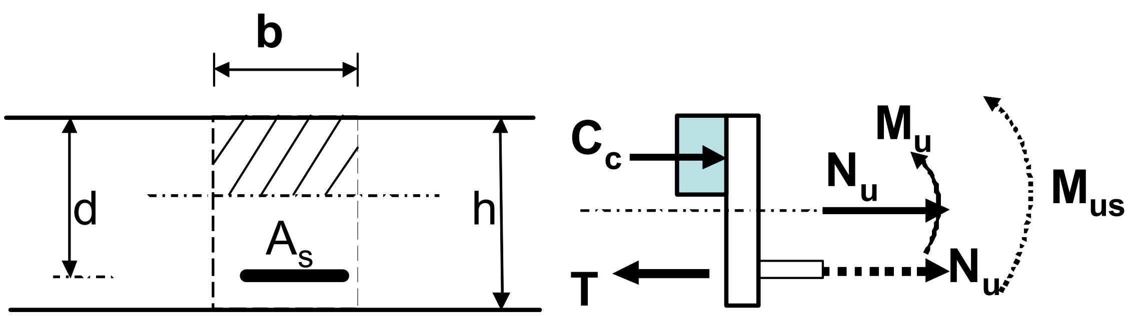
II-A Out-of-Plane Moment with Axial Tension slab width: $B := 37 \cdot ft$

M22 + M12 = -136 kip-ft/ft **F2 = -524 kip (T)**

$M_{uo} := 136 \cdot \frac{kip \cdot ft}{ft}$ $N_u := \frac{524 \cdot kip}{B}$ $N_u = 14.162\,\frac{kip}{ft}$

$b := 12 \cdot in$ $h := 48 \cdot in$ conc. cover for reinf.: $t_c := 2 \cdot in$

$d_{b11} := 1.41 \cdot in$ $d := h - t_c - d_{b11} \cdot 1.5$ (2nd layer) $d = 43.885\,in$



$$M_{us} := M_{uo} - N_u \cdot \left(d - \frac{h}{2}\right)$$ (Commentary, Ref. 2.2.18) $M_{us} = 112.532\,kip \cdot \frac{ft}{ft}$

$$m := \frac{f_y}{0.85 \cdot f'_c}$$ (Eq. 3.8.4a, Ref. 2.2.20,) $m = 14.118$

$$M_n := \frac{M_{us} \cdot b}{\phi_t}$$ $M_n = 125.036\,kip \cdot ft$ $R_n := \frac{M_n}{b \cdot d^2}$ (Eq. 3.8.4b, Ref. 2.2.20,) $R_n = 64.923\,psi$

$$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot m \cdot R_n}{f_y}}\right)$$ (Eq. 3.8.5, Ref. 2.2.20,) $\rho = 1.09 \times 10^{-3}$

$$A_s := \rho \cdot b \cdot d + \frac{N_u \cdot b}{\phi_t \cdot f_y}$$ (Commentary, Ref. 2.2.18) $A_s = 0.837\,in^2$

$$A_{s\_min} := 0.0018 \cdot b \cdot h$$ (Sect. 7.12.5, Ref. 2.2.14) $A_{s\_min} = 1.037\,in^2$

To count for D/C ratio (Demand/Capacity) = 0.6

$$A_{s\_req} := \frac{A_s}{0.6}$$ $A_{s\_req} = 1.394\,in^2$ try #11@12", As = 1.56 in^2 O K

II-B Check Out-of-Plane Shear Capacity **V23 = -19 kip/ft**

Shear Capacity with axial tension **F2 =-524 kip (T)** $B = 37\,ft$

$A_g := b \cdot h$ $h = 48\,in$ $A_g = 576\,in^2$ $N_u := \frac{-524 \cdot kip}{B}$ $N_u = -14.162\,\frac{kip}{ft}$

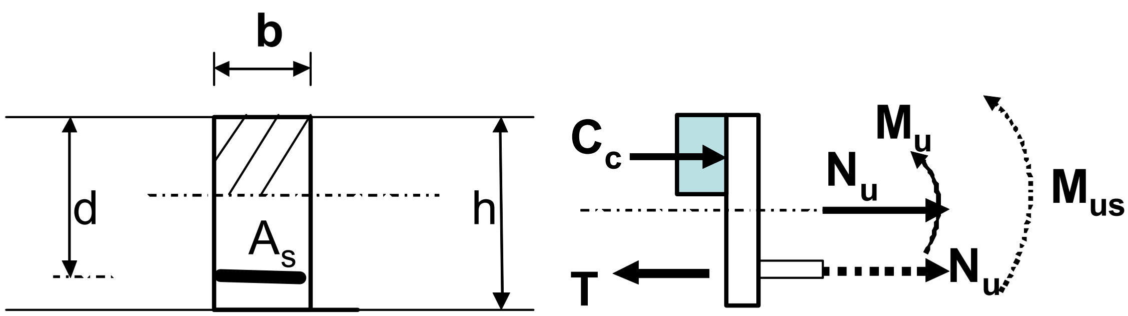
$$V_c := 2 \cdot \left(1 + \frac{N_u \cdot b}{500 \cdot A_g \cdot psi}\right) \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d$$ (Eq. 11-8, Ref. 2.2.14,) $V_c = 70.813\,kip$

$V_{uo} := 19 \cdot \frac{kip}{ft}$ $\frac{V_{uo} \cdot b}{\phi_v \cdot V_c} = 0.447$ < D/C = 0.6 O K

II-C Chord Reinf. at slab edge (east and west) due to In-Plane Moment

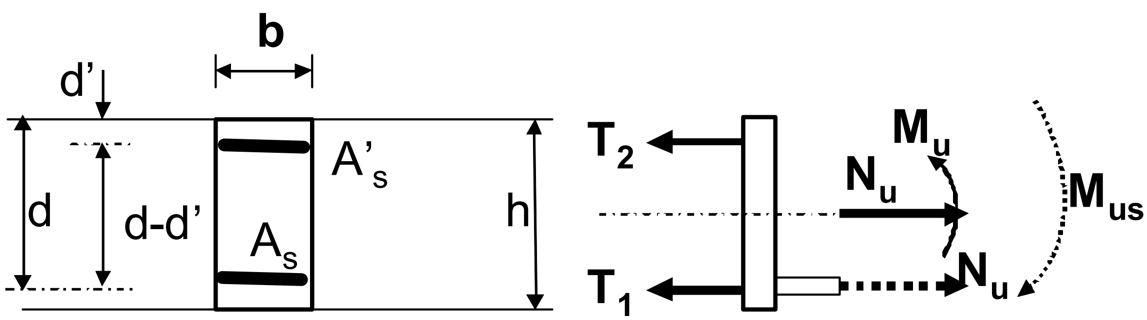
M3 = -3900 kip-ft **F2 = -524 kip (T)**

$M_{ui} := 3900 \cdot kip \cdot ft$ $N_u := 524 \cdot kip$ $b := 4 \cdot ft$ $h := 37 \cdot ft$ $d := h - 5 \cdot in$



$M_{us} := M_{ui} - N_u \cdot \left(d - \frac{h}{2}\right)$ (Commentary, Ref. 2.2.18) $M_{us} = -5.576 \times 10^3 \, kip \cdot ft$

Axial tension Nu is large compared to moment, redesign entire section in tension



$d' := 5 \cdot in$ $A'_s := \frac{|M_{us}|}{\phi_b \cdot f_y \cdot (d - d')}$ (Commentary, Ref. 2.2.18) $A'_s = 2.855 \, in^2$

$A_s := \frac{N_u}{\phi_t \cdot f_y} - A'_s$ (Commentary, Ref. 2.2.18) $A_s = 6.849 \, in^2$

To count for D/C ratio (Demand/Capacity) = 0.6

$A_{s\_req\_ns} := \frac{A_s}{0.6}$ $A_{s\_req\_ns} = 11.415 \, in^2$

try 8#11 (N-S), As = 12.48 sq. in. O K

II-D Check In-Plane Shear Capacity **F1 = -239 kip** **F2 =-524 kip (T)**

$V_{ui} := 239 \cdot kip$ $N_u := -524 \cdot kip$

$b := 4 \cdot ft$ $h = 37\,ft$ $A_g := b \cdot h$ $d := h - 5 \cdot in$

$V_c := 2 \cdot \left(1 + \frac{N_u}{500 \cdot A_g \cdot psi}\right) \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d$ (Eq. 11-8, Ref. 2.2.14,) $V_c = 2.833 \times 10^3\,kip$

$\frac{V_{ui}}{\phi_v \cdot V_c} = 0.141$ close to D/C = 0.6 O K

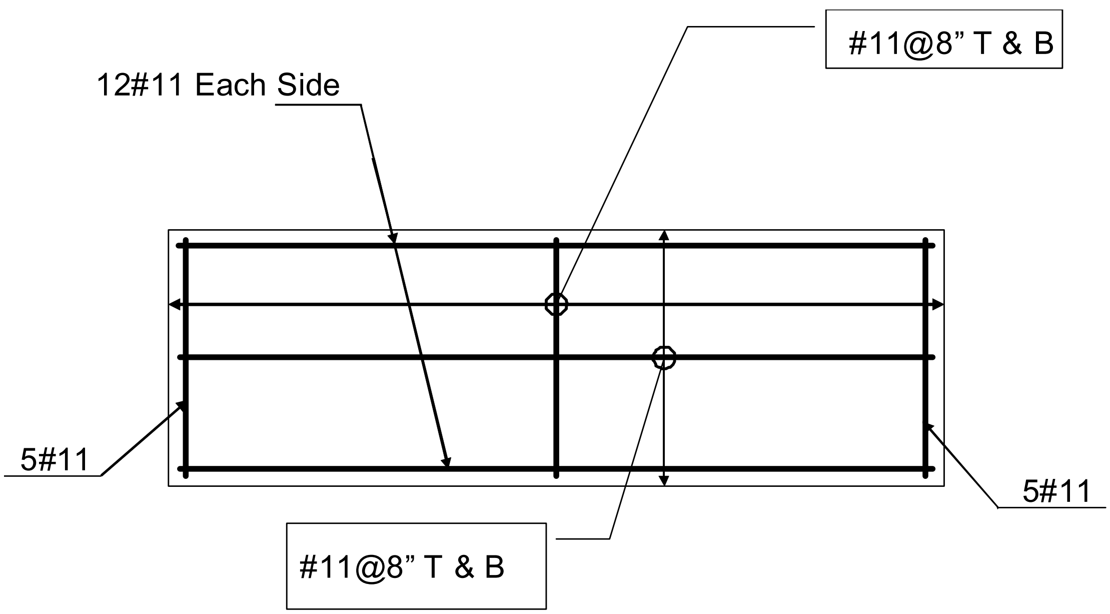
For simplicity of placing rebars, use Reinforcement as follow:

In North-South direction (short bar):

Provide #11@8" T & B, added Chord Bar 5#11 at east and west slab edges

In East-West direction (long bar):

Provide #11@8" T & B, added Chord Bar 12#11 at north and south slab edges



6.5.4

Longitudinal Walls Design Loads, Column Lines E

Accidental torsion factor = 15% (See assumption 3.1.4)

Section cut design forces and moments, which follow a global axis system, for the longitudinal wall on column E between column line 3.6 and 8 include in plane forces and moments: such as axial forces (tension/compression), in plane moment, and in plane shear. The section cut values are integrated along the section cut length, thus for INTSCUT2 the length is equal to 115'. Out of plane values such as out of plane bending and out of plane shear are attained by shell element forces and moments from SAP2000, which follow a local axis system. The out of plane bending moments provided are M22 and M12, which is a twisting moment that is combined with M22. The out of plane shear forces from the shell elements include V13 and V23.

Follow notation below to convert from SAP2000 labeling (i.e. F2, M22…) to appropriate design forces and moments.

INTSCUT2
F2 = In Plane Shear
F3 = Axial Force, Compression (+) / Tension (-)
M1 = In Plane Moment

Shell Element Forces and Moments
M22 = Out of Plane Moment
M12 = Twisting Moment
V13 = Out of Plane Shear
V23 = Out of Plane Shear

Section Cu	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
INTSCUT2	-	19	3088	-9193	-	-	-	1971	3406	49641	-	-

MAX MIN	M11	M22	M12	V13	V23	MAX	M11	M22	M12	V13	V23
	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft		Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	10	12	11	2	3	MAX	33	125	34	9	12
MIN	-8	-38	-12	-2	0						

Loads with accidental torsion factor

Section Cu	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
INTSCUT2	-	22	3551	-10572	-	-	-	2267	3917	57088	-	-

Loads with accidental torsion factor

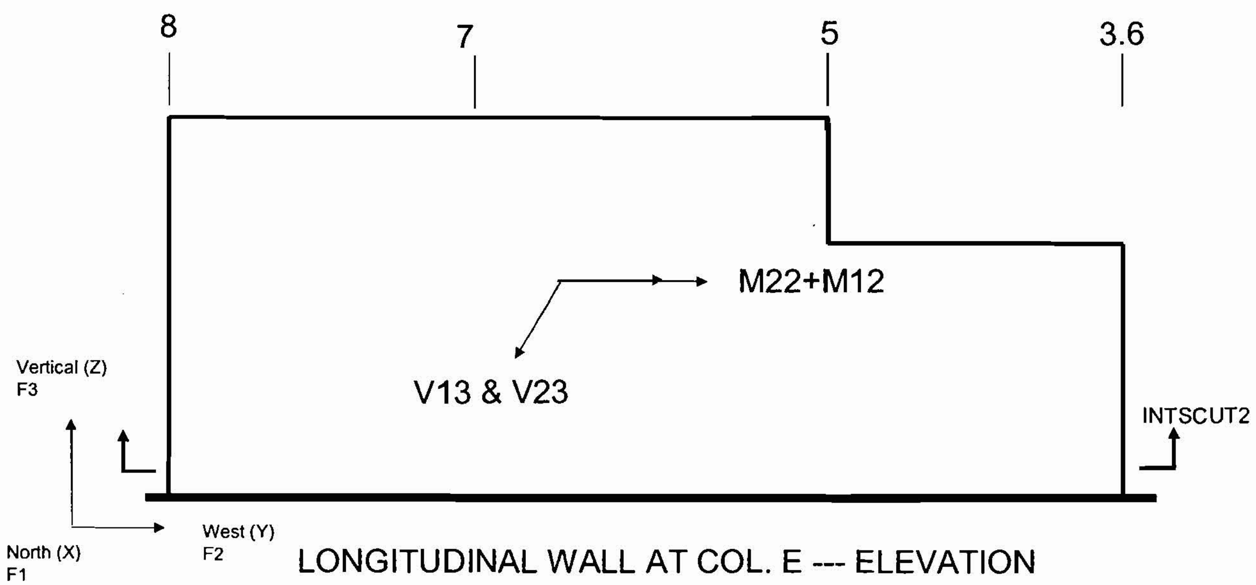
MAX MIN	M11	M22	M12	V13	V23	MAX	M11	M22	M12	V13	V23
	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft		Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	11	14	12	2	3	MAX	37	144	39	10	14
MIN	-10	-44	-14	-2	0						

Maximum Load Combination

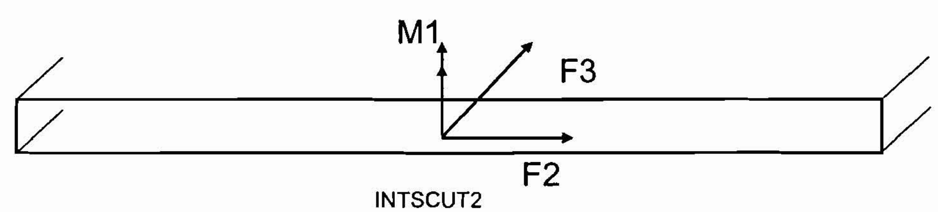
Section Cu	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
INTSCUT2	-	2289	7468	46515	-	-	-	-2245	-366	-67660	-	-

Maximum Load Combination

MAX MIN	M11	M22	M12	V13	V23	MAX MIN	M11	M22	M12	V13	V23
	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft		Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	49	158	51	12	17	MAX	-26	-130	-27	-8	-11
MIN	28	100	25	8	14	MIN	-47	-188	-53	-12	-15



LONGITUDINAL WALL AT COL. E --- ELEVATION



6.5.4.1 LONGITUDINAL NORTH WALL E, REINFORCEMENT COLUMN LINES 3.6 to 8

Longitudinal Wall COL E-INTSCUT2---D+L-SRSS

Design Loads

Axial Force (-Tension)	**Ft =**	-366	kips
Axial Force (+Comp)	**Fc =**	0	kips
In plane shear	**Vu =**	2245	kips
In plane Moment	**Mz =**	67660	ft-kips
Out of plane shear	**Vz =**	15	kips/ft
Out of plane Moment	**My =**	241	ft-kips/ft (M22+M12)

Shear Wall Section Properties

Height of Wall (segment)	h_w =	35	ft
Ht of Wall Between Floors	H =	35	ft
Length of Wall (Segment)	l_w =	115	ft
Thickness of Wall	t_w =	4	ft
Shear Area of Wall (Segment)	$Acv = l_w{*}t_w$ =	460	ft^2

Concrete & Rebar Properties

Concrete Design Strength	f'c =	5000	psi
Concrete Strain	ε_c =	0.002	
Rebar Yield Strength	fy =	60	ksi
Rebar Yield Strain	ε_y =	0.002	
Min Steel Required	ρ_{min} =	0.0025	
Concrete Cover		5	in

(Use 5" = 2" clear cover + diameter of the outer layer rebar + 1/2 diameter of the inner layer rebar)

Note: For ACI 349, see Ref. 2.2.14, for sections 1.0 to 6.0 of this design subset.

1.0 Check Shear on gross section - ACI 349: 21.6.5.6

Nominal Shear Capacity = 8*Acv*(f'c) $^{1/2}$ Vn (kips) = 37471

Factored Shear Load = Vu / ϕ Vu/ϕ (kips)= 3741.7
(ϕ = .6 per ACI 349 - 9.3.4)

Demand Capacity Ratio
Check Code 21.6.5.6 Demand/Capacity Ratio D/C = (Vu/ϕ)/Vn D/C = 0.10 **SHEAR WALL THICKNESS OK**

2.0 Horizontal Reinforcing Requirements

2.0a) ACI 349 - 21.6.5.3 Requirements

Determine α_c : h_w / l_w = 0.30 α_c = 3 α_c=3 for h_w/l_w<1.5, α_c varies linearly from 3 for h_w/l_w=1.5 to 2 for h_w/l_w=2.

Determine Concrete Shear Capacity Vc=Acv*α_c*(f'c) $^{1/2}$ Vc = 14051.6 kips

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc Vs = 0.0 kips

Determine Required Shear Reinforcing ρ=Vs/(fy*Acv) ρ = 0.0000

21.6.5.3 Shear Reinforcing Requirements **ρ_n = 0.0025** MINIMUM STEEL GOVERNS

2.0b) ACI 349 - 11.10.6 Equation 11-31 Requirements

Determine Concrete Shear Capacity Vc = 12292.2 kips $Vc=3.3{*}(f'_c)^{0.5}{*}(0.8{*}t_w{*}l_w)+F_t{*}0.8{*}l_w/(4{*}l_w)$

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc Vs = 0.0 kips

Determine Required Shear Reinforcing ρ = 0.0000 $\rho=Vs/(0.8{*}l_w{*}t_w{*}144{*}fy)$

11.10.6 - Equation 11-31 Shear Reinforcing Requirements **ρ_n = 0.0025** MINIMUM STEEL GOVERNS

2.0c) ACI 349 - 11.10.6 Equation 11-32 Requirements

Check Bounding Case Mu/Vu - l_w/2 = -27.36 **Mu/Vu-lw/2 < 0 equation 11-32 NOT APPLICABLE**

Determine Concrete Shear Capacity Vc= N/A $Vc=[0.6{*}f'^{0.5}_c+l_w{*}(1.25{*}f'^{0.5}_c+0.2{*}Ft{*}1000/(l_w{*}t_w{*}144))/(Mu/Vu-l_w/2)]{*}t_w{*}0.8{*}l_w{*}144/1000$

Determine Shear Carried by Steel Vs = N/A Vs=Vu/ϕ -Vc

Determine Required Shear Reinforcing Requirements ρ = N/A $\rho=Vs/(0.8{*}l_w{*}t_w{*}144{*}fy)$

11.10.6 - Equation 11-32 Shear Reinforcing Requirements **ρ_n = N/A** **EQUATION 11-32 NOT APPLICABLE**

2.0d) Horizontal Shear Reinforcing Requirements (max of 2a, 2b, 2c) **ρ_n = 0.0025**

2.0e) Select Horizontal Shear Reinforcing **Asn required per ft on each face =** **0.72** **in^2/ft each face** $(\rho_n{*}12{*}t_w{*}12/2)$

Use **1-#11@12"c/c EF** **Asn provided =** **1.56** **in^2/ft each face** **ρ_n (prov)=0.00542**

2.0f) Check Demand / Capacity Ratio for In-Plane Shear:

D/C = 0.11 $D/C=(Vu/\phi)/[Vc+(Asn{*}2{*}fy{*}l_w{*}t_w{*}144/(12{*}t_w{*}12))]$

3.0 Vertical Reinforcing Requirements

3.0a) ACI 349 21.6.5.5 and 11.10.9.4 - Minimum vertical reinforcing ratio : hw/lw = 0.30

ρ_v (min) =0.0025

If h_w/l_w>2.0, use: $\rho_{v\,min}$ =0.0025+0.5(2.5-h_w/l_w)(ρ_n-0.0025)<=ρ_n
If h_w/l_w<=2.0, use: ρ_v>=ρ_n

3.0b) Check Shear Friction Requirements

In plane shear per foot of wall: 19.52 kips/ft (Vu/l_w)

Transverse shear per ft of wall: 15.00 kips/ft (Vz)

Resultant Shear 24.62 kips/ft $[(\text{in-plane shear})^2+(\text{transverse shear})^2]^{0.5}$

Calculate limiting shear friction strength at joint per ACI 349-11.7.5:

Vn<.2f'cAv for f'c = 5000 psi Vn=1000Av
Vn<800Av The limiting value of 800Av controls
Vn (MAX) = 460.8 kips/ft **Vn(MAX>Resultant shear ....OK**

Calculate shear friction reinforcing requirements per ACI 349 - 11.7.4.1:

$Vn = Avf*fy*\mu$ μ = 1.0 for concrete placed against hardened concrete intentionally roughened to a full amplitude of 1/4 inch (ACI 349-11.7.9)

$Avf = Vu/2*\phi*\mu*fy$ (steel required per face)
Avf = 0.24 in^2/ft (steel required on each face for shear friction))

Calculate steel required for net Tension force

$At = T/2*\phi*fy*lw$
At = 0.03 in^2/ft (steel required on each face for direct tension)

Steel Requirements for Shear Friction Avf + At

Av = 0.27 in^2/ft (steel required on each face for shear friction + direct tension)

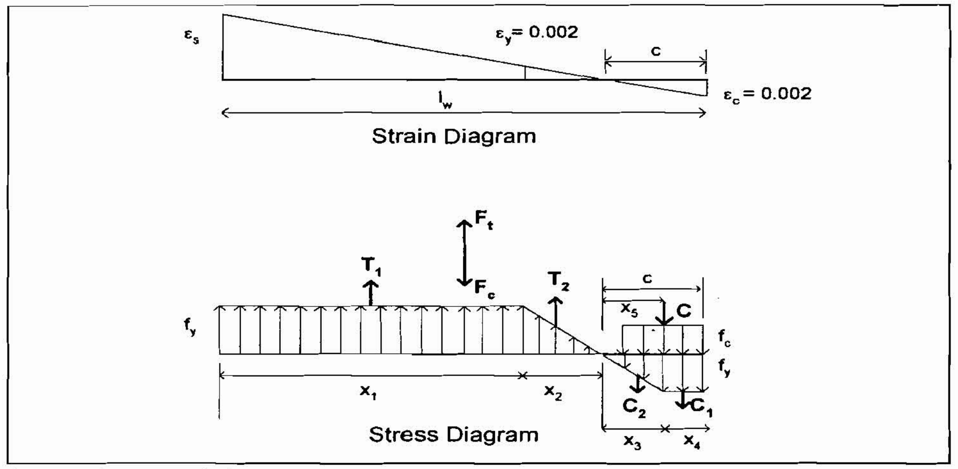
ρv(req'd) = 0.0009 $(\rho_{v\,req'd}=(2*Av)/(12*t_w*12)$

3.0c) Vertical Reinforcing Requirements (max of 3a,3b) **ρ_v = 0.0025**

3.0d) Select Vertical Reinforcing **Asv required per ft on each face =** **0.72** **in^2/ft each face** $(\rho_{v\,min}*12*t_w*12/2)$

Use 1-#11@12"c/c EF **Asv provided =** **1.56** **in^2/ft each face** **ρ_v (prov)= 0.00542**

3.0e) Perform Strain-Compatible Section Analysis - For Axial Force (Tension) **Ft =** -366 kips



ϕ : Strength reduction factor (= 0.9 for tension, ACI 349 9.3.2.2), ϕ = 0.9
c : Distance from compression face to neutral axis (ft)
l_w : Length of wall (ft)
$\varepsilon_{s\_max} = [(l_w - c) / c] * \varepsilon_c$
$X_1 = l_w - c - X_2$ (ft) (length of wall with tension steel reinforcing steel strain > ε_y)
$X_2 = X_3 = (\varepsilon_y / \varepsilon_c) * c$ (ft) (length of wall with tension / compression steel reinforcing steel strain < ε_y)
$X_4 = c - X_3$ (ft) (length of wall with compression steel reinforcing steel strain > ε_y)
$T_1 = X_1 * As * fy$ (kips)
$T_2 = C_2 = 0.5 * X_2 * As * fy$ (kips)
$C_1 = X_4 * As * fy$ (kips)
$C = 0.85 * 0.8 * c * t_w * f'_c * 144$ (kips)
Balanced condition : Tension = Compression, $T_1+T_2-C_2-C_1-C+Ft/\phi = 0$.
M_u : Total moment capacity = $\phi\{T_1(X_2+X_1/2) + T_2(X_2*2/3) + C(c-0.8*c/2) + F_t(l_w/2-c)/\phi + C_2(X_3*2/3) + C_1(X_3+X_4/2)\}$, (kip-ft)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
9.05	105.95	0.0234	96.89	9.05	0.00	18138	847	0	17731	0.000	1016819

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
10	105	0.0210	95.00	10.00	0.00	17784	936	0	19584	-2207	1019923
11	104.0	0.0189	93.00	11.00	0.00	17409.6	1029.6	0	21542	-4539	1025480
9	106	0.0236	97.00	9.00	0.00	18158.4	842.4	0	17626	126	1016705

Perform Strain-Compatible Section Analysis - For Axial Force (Comp) **Fc =** 0 kips

ϕ : Strength reduction factor (= 0.7 to 0.9 for comp, ACI 349 9.3.2.2), ϕ = 0.9000
If Fc<=0, ϕ=0.9; if Fc>0, ϕ=0.9-0.2*Fc/(0.1*Acv*144*f'c/1000)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
9.23	105.77	0.0229	96.54	9.23	0.00	18073	864	0	18073	0.000	1034901

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
10	105	0.0210	95.00	10.00	0.00	17784	936	0	19584	-1800	1037308
11	104.0	0.0189	93.00	11.00	0.00	17409.6	1029.6	0	21542	-4133	1042499
9	106	0.0236	97.00	9.00	0.00	18158.4	842.4	0	17626	533	1034456

3.0f) Check Demand / Capacity Ratio for In-Plane Moment:

Vertical Reinforcing Ratio Provided by Design : ρ_v (prov)= 0.0054 1-#11@12"c/c EF ρ_v = (2*Asv)/(12*t_w*12)
D/C = (in-plane moment Mz) / (Min of Mu-ten, Mu-comp) D/C = 0.07 **Section Adequate**
Vertical Reinforcing Ratio Required for In-Plane Moment: $\rho_{vl\ req}$ = 0.0004 $\rho_{v\ req} = \rho_v$*(D/C)

3.0g) Consider Out-of-Plane Moment:

Vertical Reinf Ratio Required for Out-of-Plane Bending: $\rho_{vt\ req}$ = 0.00246 (per face) $\rho f_y bd$^2*(1-.59$\rho f_y/f'_c$)=Mu/ϕ solve for ρ Ref. 2.2.19, Section 4-3

3.0h) Total Reinforcing required for axial force, in-plane and out of plane bending ρ_v = 0.0049 = $\rho_{vl}+\rho_{vt} > 2\rho_{vt}$

D/C = ρ_v(reqd) / ρ_v (prov) D/C = 0.91 **Section Adequate** = ρ_v(req'd) / ρ_v(prov)

4.0 Boundary Elements:

h_w / l_w = 0.30 <2, No Need To Check for Boundary Elements

5.0 Out-of-Plane Shear:

Nominal Shear Strength Provided by The Concrete Vc=2*(f'c)$^{1/2}$*b*d Vc = 73.0 kips/ft width of wall

Check Demand / Capacity Ratio: D/C = 0.24 No Shear Reinforcement Required
D/C = (out-of-plane shear Vz) / (0.85*Vc)

6.0 Tabulate Reinforcement Requirements And D/C Ratios:

Use 4 ft thick wall with 1-#11@12"c/c EF Horizontal Reinforcement
1-#11@12"c/c EF Vertical Reinforcement.

For Shear on Gross Section:	**D/C =**	**0.10**
For In-Plane Shear:	**D/C =**	**0.11**
For Out-of-Plane Shear:	**D/C =**	**0.24**
Bending + axial Loads	**D/C =**	**0.91**

Boundary Elements D/C= BOUNDARY ELEMENTS NOT REQUIRED

Longitudinal Wall COL E-INTSCUT2---D+L+SRSS

Design Loads			**Shear Wall Section Properties**			**Concrete & Rebar Properties**		
Axial Force (-Tension)	Ft =	0 kips	**Height of Wall (segment)**	h_w =	35 ft	**Concrete Design Strength**	f'c =	5000 psi
Axial Force (+Comp)	Fc =	7468 kips	**Ht of Wall Between Floors**	H =	35 ft	**Concrete Strain**	ε_c =	0.002
In plane shear	Vu =	2289 kips	**Length of Wall (Segment)**	l_w =	115 ft	**Rebar Yield Strength**	fy =	60 ksi
In plane Moment	Mz =	46515 ft-kips	**Thickness of Wall**	t_w =	4 ft	**Rebar Yield Strain**	ε_y =	0.002
			Shear Area of Wall (Segment)	$Acv = l_w*t_w$ =	460 ft^2	**Min Steel Required**	ρ_{min} =	0.0025
						Concrete Cover		5 in
Out of plane shear	Vz =	17 kips/ft						
Out of plane Moment	My =	209 ft-kips/ft (M22+M12)						

(Use 5" = 2" clear cover + diameter of the outer layer rebar + 1/2 diameter of the inner layer rebar)

Note: For ACI 349, see Ref. 2.2.14, for sections 1.0 to 6.0 of this design subset.

1.0 Check Shear on gross section - ACI 349: 21.6.5.6

Nominal Shear Capacity = $8*Acv*(f'c)^{1/2}$ — Vn (kips) = 37471

Factored Shear Load = Vu/ϕ — Vu/ϕ (kips)= 3815.0
(ϕ = .6 per ACI 349 - 9.3.4)

Demand Capacity Ratio
Check Code 21.6.5.6 Demand/Capacity Ratio — D/C = $(Vu/\phi)/Vn$ — D/C = 0.10 **SHEAR WALL THICKNESS OK**

2.0 Horizontal Reinforcing Requirements

2.0a) ACI 349 - 21.6.5.3 Requirements

Determine α_c : h_w/l_w = 0.30 — α_c = 3 — α_c=3 for h_w/l_w<1.5, α_c varies linearly from 3 for h_w/l_w=1.5 to 2 for h_w/l_w=2.

Determine Concrete Shear Capacity $Vc=Acv*\alpha_c*(f'c)^{1/2}$ — Vc = 14051.6 kips

Determine Shear Carried by Steel $Vs=Vu/\phi-Vc$ — Vs = 0.0 kips

Determine Required Shear Reinforcing $\rho=Vs/(fy*Acv)$ — ρ = 0.0000

21.6.5.3 Shear Reinforcing Requirements — ρ_n **= 0.0025** MINIMUM STEEL GOVERNS

2.0b) ACI 349 - 11.10.6 Equation 11-31 Requirements

Determine Concrete Shear Capacity — Vc = 12365.4 kips — $Vc=3.3*(f'_c)^{0.5}*(0.8*t_w*l_w)+F_t*0.8*l_w/(4*l_w)$

Determine Shear Carried by Steel $Vs=Vu/\phi-Vc$ — Vs = 0.0 kips

Determine Required Shear Reinforcing — ρ = 0.0000 — $\rho=Vs/(0.8*l_w*t_w*144*fy)$

11.10.6 - Equation 11-31 Shear Reinforcing Requirements — ρ_n **= 0.0025** MINIMUM STEEL GOVERNS

2.0c) ACI 349 - 11.10.6 Equation 11-32 Requirements

Check Bounding Case $Mu/Vu - l_w/2$ = -37.18 — **Mu/Vu-lw/2 < 0 equation 11-32 NOT APPLICABLE**

Determine Concrete Shear Capacity — Vc= N/A — $Vc=[0.6*f'_c{}^{0.5}+l_w*(1.25*f'_c{}^{0.5}+0.2*Ft*1000/(l_w*t_w*144))/(Mu/Vu-l_w/2)]*t_w*0.8*l_w*144/1000$

Determine Shear Carried by Steel — Vs = N/A — $Vs=Vu/\phi-Vc$

Determine Required Shear Reinforcing Requirements — ρ = N/A — $\rho=Vs/(0.8*l_w*t_w*144*fy)$

11.10.6 - Equation 11-32 Shear Reinforcing Requirements — ρ_n **= N/A** **EQUATION 11-32 NOT APPLICABLE**

2.0d) Horizontal Shear Reinforcing Requirements (max of 2a, 2b, 2c) — ρ_n **= 0.0025**

2.0e) Select Horizontal Shear Reinforcing — **Asn required per ft on each face = 0.72 in²/ft each face** $(\rho_n*12*t_w*12/2)$

Use 1-#11@12"c/c EF — **Asn provided = 1.56 in²/ft each face** — ρ_n **(prov)=0.00542**

2.0f) Check Demand / Capacity Ratio for In-Plane Shear:

D/C = 0.11 — $D/C=(Vu/\phi)/[Vc+(Asn*2*fy*l_w*t_w*144/(12*t_w*12))]$

3.0 Vertical Reinforcing Requirements

3.0a) ACI 349 21.6.5.5 and 11.10.9.4 - Minimum vertical reinforcing ratio : hw/lw = 0.30

If h_w/l_w>2.0, use: $\rho_{v\,min}=0.0025+0.5(2.5-h_w/l_w)(\rho_n-0.0025)<=\rho_n$
If h_w/l_w<=2.0, use: $\rho_v>=\rho_n$

ρ_v **(min) =0.0025**

3.0b) Check Shear Friction Requirements

In plane shear per foot of wall: 19.90 kips/ft (Vu/l_w)

Transverse shear per ft of wall: 17.00 kips/ft (Vz)

Resultant Shear 26.18 kips/ft $[(\text{in-plane shear})^2+(\text{transverse shear})^2]^{0.5}$

Calculate limiting shear friction strength at joint per ACI 349-11.7.5:

Vn<.2f'cAv for f'c = 5000 psi Vn=1000Av
Vn<800Av The limiting value of 800Av controls
Vn (MAX) = 460.8 kips/ft **Vn(MAX>Resultant shear ………OK**

Calculate shear friction reinforcing requirements per ACI 349 - 11.7.4.1:

Vn = Avf*fy*μ μ = 1.0 for concrete placed against hardened concrete intentionally roughened to a full amplitude of 1/4 inch (ACI 349-11.7.9)

Avf = Vu/2*ϕ*μ*fy (steel required per face)
Avf = 0.26 in^2/ft (steel required on each face for shear friction))

Calculate steel required for net Tension force

At = T/2*ϕ*fy*lw
At = 0.00 in^2/ft (steel required on each face for direct tension)

Steel Requirements for Shear Friction Avf + At

Av = 0.26 in^2/ft (steel required on each face for shear friction + direct tension)

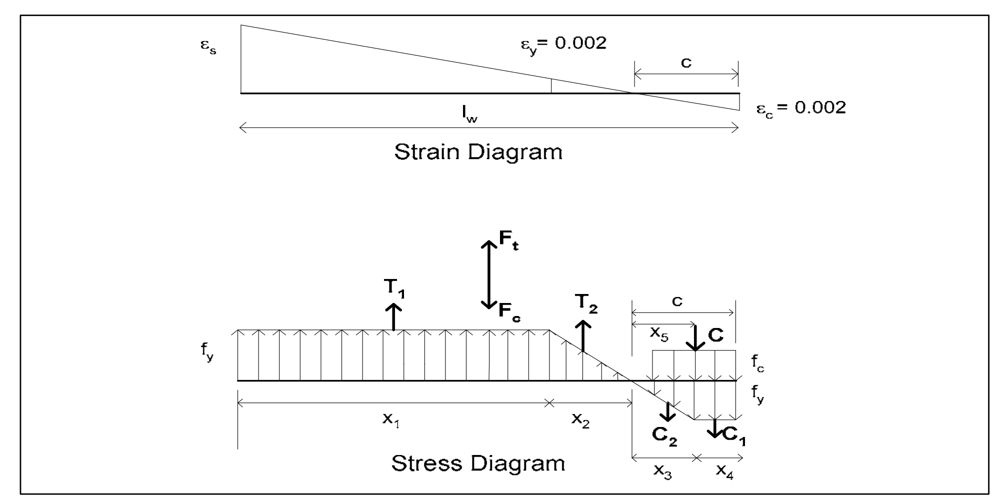
ρv(req'd) = 0.0009 $(\rho_{v\ req'd}=(2*Av)/(12*t_w*12)$

3.0c) Vertical Reinforcing Requirements (max of 3a,3b) ρ_v **= 0.0025**

3.0d) Select Vertical Reinforcing Asv required per ft on each face = 0.72 in^2/ft each face $(\rho_{v\ min}*12*t_w*12/2)$

Use 1-#11@12"c/c EF Asv provided = 1.56 in^2/ft each face ρ_v **(prov)= 0.0054**

3.0e) Perform Strain-Compatible Section Analysis - For Axial Force (Tension) Ft = 0 kips



ϕ : Strength reduction factor (= 0.9 for tension, ACI 349 9.3.2.2), ϕ = 0.9
c : Distance from compression face to neutral axis (ft)
l_w : Length of wall (ft)
$\varepsilon_{s\_max} = [(l_w - c) / c] * \varepsilon_c$
$X_1 = l_w - c - X_2$ (ft) (length of wall with tension steel reinforcing steel strain > ε_y)
$X_2 = X_3 = (\varepsilon_y / \varepsilon_c) * c$ (ft) (length of wall with tension / compression steel reinforcing steel strain < ε_y)
$X_4 = c - X_3$ (ft) (length of wall with compression steel reinforcing steel strain > ε_y)
$T_1 = X_1 * As * fy$ (kips)
$T_2 = C_2 = 0.5 * X_2 * As * fy$ (kips)
$C_1 = X_4 * As * fy$ (kips)
$C = 0.85 * 0.8 * c * t_w * f'_c * 144$ (kips)
Balanced condition : Tension = Compression, $T_1+T_2-C_2-C_1-C+Ft/\phi = 0$.
M_u : Total moment capacity = $\phi\{T_1(X_2+X_1/2) + T_2(X_2*2/3) + C(c-0.8*c/2) + F_t(l_w/2-c)/\phi + C_2(X_3*2/3) + C_1(X_3+X_4/2)\}$, (kip-ft)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
9.23	105.77	0.0229	96.54	9.23	0.00	18073	864	0	18073	0.000	1034901

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
10	105	0.0210	95.00	10.00	0.00	17784	936	0	19584	-1800	1037308
11	104.0	0.0189	93.00	11.00	0.00	17409.6	1029.6	0	21542	-4133	1042499

9	106	0.0236	97.00	9.00	0.00	18158.4	842.4	0	17626	533	1034456

Perform Strain-Compatible Section Analysis - For Axial Force (Comp) **Fc =** 7468 kips

ϕ : Strength reduction factor (= 0.7 to 0.9 for comp, ACI 349 9.3.2.2), ϕ = 0.8549
If Fc<=0, ϕ=0.9; if Fc>0, ϕ=0.9-0.2*Fc/(0.1*Acv*144*f'_c/1000)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
12.97	102.03	0.0157	89.05	12.97	0.00	16671	1214	0	25406	0.000	1339039

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
12	103	0.0172	91.00	12.00	0.00	17035.2	1123.2	0	23501	2270	1337211
11	104.0	0.0189	93.00	11.00	0.00	17409.6	1029.6	0	21542	4603	1337524
13	102	0.0157	89.00	13.00	0.00	16660.8	1216.8	0	25459	-63	1339119

3.0f) Check Demand / Capacity Ratio for In-Plane Moment:

Vertical Reinforcing Ratio Provided by Design : ρ_v (prov)= 0.0054 1-#11@12"c/c EF $\rho_v = (2*Asv)/(12*t_w*12)$
D/C = (in-plane moment Mz) / (Min of Mu-ten, Mu-comp) D/C = 0.04 **Section Adequate**
Vertical Reinforcing Ratio Required for In-Plane Moment: $\rho_{vl\ req}$ = 0.0002 $\rho_{v\ req} = \rho_v*(D/C)$

3.0g) Consider Out-of-Plane Moment:

Vertical Reinf Ratio Required for Out-of-Plane Bending: $\rho_{vt\ req}$ = 0.00213 (per face) $\rho f_y bd\text{^}2*(1-.59\rho f_y/f'_c)=Mu/\phi$ solve for ρ Ref. 2.2.19, Section 4-3

3.0h) Total Reinforcing required for axial force, in-plane and out of plane bending ρ_v = 0.0043 $= \rho_{vl}+\rho_{vt} > 2\rho_{vt}$

D/C = ρ_v(reqd) / ρ_v (prov) D/C = 0.78 **Section Adequate** = ρ_v(req'd) / ρ_v(prov)

4.0 Boundary Elements:

h_w / l_w = 0.30 <2, No Need To Check for Boundary Elements

5.0 Out-of-Plane Shear:

Nominal Shear Strength Provided by The Concrete Vc=2*(f'c)$^{1/2}$*b*d Vc = 73.0 kips/ft width of wall

Check Demand / Capacity Ratio: D/C = 0.27 No Shear Reinforcement Required
D/C = (out-of-plane shear Vz) / (0.85*Vc)

6.0 Tabulate Reinforcement Requirements And D/C Ratios:

Use 4 ft thick wall with 1-#11@12"c/c EF Horizontal Reinforcement
1-#11@12"c/c EF Vertical Reinforcement.

For Shear on Gross Section:	**D/C =**	**0.10**
For In-Plane Shear:	**D/C =**	**0.11**
For Out-of-Plane Shear:	**D/C =**	**0.27**
Bending + axial Loads	**D/C =**	**0.78**
Boundary Elements	**D/C=**	**BOUNDARY ELEMENTS NOT REQUIRED**

6.5.5

Longitudinal Walls Design Loads, Column Line F

Accidental torsion factor = 15% (See assumption 3.1.4)

Section cut design forces and moments, which follow a global axis system, for the longitudinal wall on column F between column line 3.6 and 7 include in plane forces and moments: such as axial forces (tension/compression), in plane moment, and in plane shear. The section cut values are integrated along the section cut length, thus for INTSCUT1 the length is equal to 90'. Out of plane values such as out of plane bending and out of plane shear are attained by shell element forces and moments from SAP2000, which follow a local axis system. The out of plane bending moments provided are M22 and M12, which is a twisting moment that is combined with M22. The out of plane shear forces from the shell elements include V13 and V23.

Follow notation below to convert from SAP2000 labeling (i.e. F2, M22...) to appropriate design forces and moments.

INTSCUT1
F2 = In Plane Shear
F3 = Axial Force, Compression (+) / Tension (-)
M1 = In Plane Moment

Shell Element Forces and Moments
M22 = Out of Plane Moment
M12 = Twisting Moment
V13 = Out of Plane Shear
V23 = Out of Plane Shear

Section Cut	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
INTSCUT1	-	-40	2510	-10952	-	-	-	1862	2734	31848	-	-

MAX MIN	M11	M22	M12	V13	V23	MAX	M11	M22	M12	V13	V23
	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft		Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	9	39	13	2	0	MAX	29	123	33	8	13
MIN	-8	-14	-11	-2	-4						

Loads with accidental torsion factor

Section Cut	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
INTSCUT1	-	-46	2886	-12595	-	-	-	2141	3144	36626	-	-

Loads with accidental torsion factor

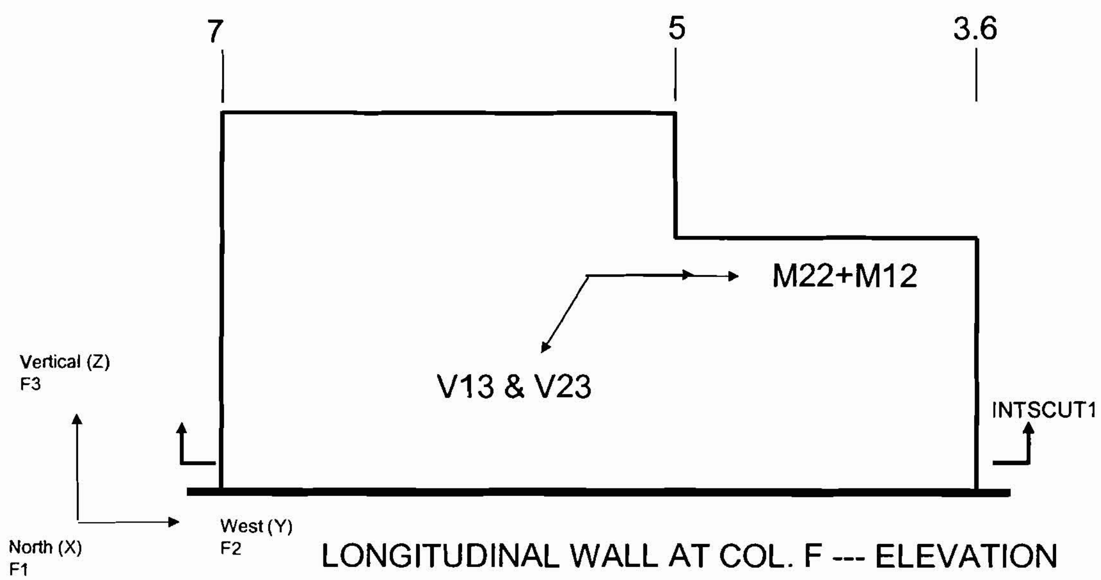
MAX MIN	M11	M22	M12	V13	V23	MAX	M11	M22	M12	V13	V23
	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft		Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	10	45	15	2	0	MAX	34	141	38	9	14
MIN	-9	-16	-13	-2	-4						

Maximum Load Combination

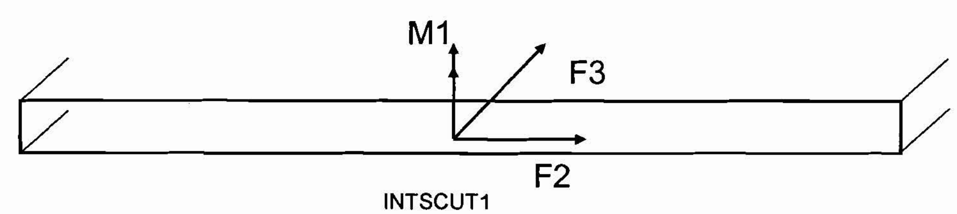
Section Cut	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
INTSCUT1	-	2095	6030	24031	-	-	-	-2187	-257	-49220	-	-

Maximum Load Combination

MAX MIN	M11	M22	M12	V13	V23	MAX MIN	M11	M22	M12	V13	V23
	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft		Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	44	186	53	11	15	MAX	-24	-96	-24	-7	-14
MIN	25	125	26	7	10	MIN	-42	-157	-51	-11	-19



LONGITUDINAL WALL AT COL. F --- ELEVATION



6.5.5.1 LONGITUDINAL SOUTH WALL F, REINFORCEMENT COLUMN LINES 3.6 to 7

Longitudinal Wall COL F-INTSCUT1---D+L-SRSS

Design Loads			**Shear Wall Section Properties**			**Concrete & Rebar Properties**		
Axial Force (-Tension)	**Ft =**	-257 kips	**Height of Wall (segment)**	h_w =	35 ft	**Concrete Design Strength**	**f'c =**	5000 psi
Axial Force (+Comp)	**Fc =**	0 kips	**Ht of Wall Between Floors**	**H =**	35 ft	**Concrete Strain**	ε_c =	0.002
In plane shear	**Vu =**	2187 kips	**Length of Wall (Segment)**	l_w =	90 ft	**Rebar Yield Strength**	**fy =**	60 ksi
In plane Moment	**Mz =**	49220 ft-kips	**Thickness of Wall**	t_w =	4 ft	**Rebar Yield Strain**	ε_y =	0.002
			Shear Area of Wall (Segment)	$Acv = l_w{*}t_w$ =	360 ft^2	**Min Steel Required**	ρ_{min} =	0.0025
						Concrete Cover		5 in

Out of plane shear **Vz =** 19 kips/ft

(Use 5" = 2" clear cover + diameter of the outer layer rebar + 1/2 diameter of the inner layer rebar)

Out of plane Moment **My =** 208 ft-kips/ft (M22+M12)

Note: For ACI 349, see Ref. 2.2.14, for sections 1.0 to 6.0 of this design subset.

1.0 Check Shear on gross section - ACI 349: 21.6.5.6

Nominal Shear Capacity = 8*Acv*(f'c) $^{1/2}$ Vn (kips) = 29325

Factored Shear Load = Vu / ϕ Vu/ϕ (kips)= 3645.0

(ϕ = .6 per ACI 349 - 9.3.4)

Demand Capacity Ratio

Check Code 21.6.5.6 Demand/Capacity Ratio D/C = (Vu/ϕ)/Vn D/C = 0.12 **SHEAR WALL THICKNESS OK**

2.0 Horizontal Reinforcing Requirements

2.0a) ACI 349 - 21.6.5.3 Requirements

Determine α_c : h_w / l_w = 0.39 α_c = 3 α_c=3 for h_w/l_w<1.5, α_c varies linearly from 3 for h_w/l_w=1.5 to 2 for h_w/l_w=2.

Determine Concrete Shear Capacity Vc=Acv*α_c*(f'c) $^{1/2}$ Vc = 10996.9 kips

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc Vs = 0.0 kips

Determine Required Shear Reinforcing ρ=Vs/(fy*Acv) ρ = 0.0000

21.6.5.3 Shear Reinforcing Requirements ρ_n **= 0.0025** MINIMUM STEEL GOVERNS

2.0b) ACI 349 - 11.10.6 Equation 11-31 Requirements

Determine Concrete Shear Capacity Vc = 9625.9 kips $Vc=3.3{*}(f'_c)^{0.5}{*}(0.8{*}t_w{*}l_w)+F_t{*}0.8{*}l_w/(4{*}l_w)$

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc Vs = 0.0 kips

Determine Required Shear Reinforcing ρ = 0.0000 $\rho=Vs/(0.8{*}l_w{*}t_w{*}144{*}fy)$

11.10.6 - Equation 11-31 Shear Reinforcing Requirements ρ_n **= 0.0025** MINIMUM STEEL GOVERNS

2.0c) ACI 349 - 11.10.6 Equation 11-32 Requirements

Check Bounding Case Mu/Vu - l_w/2 = -22.49 **Mu/Vu-lw/2 < 0 equation 11-32 NOT APPLICABLE**

Determine Concrete Shear Capacity Vc= N/A $Vc=[0.6{*}f'_c{}^{0.5}+l_w{*}(1.25{*}f'_c{}^{0.5}+0.2{*}Ft{*}1000/(l_w{*}t_w{*}144))/(Mu/Vu-l_w/2)]{*}t_w{*}0.8{*}l_w{*}144/1000$

Determine Shear Carried by Steel Vs = N/A Vs=Vu/ϕ -Vc

Determine Required Shear Reinforcing Requirements ρ = N/A $\rho=Vs/(0.8{*}l_w{*}t_w{*}144{*}fy)$

11.10.6 - Equation 11-32 Shear Reinforcing Requirements ρ_n **= N/A** **EQUATION 11-32 NOT APPLICABLE**

2.0d) Horizontal Shear Reinforcing Requirements (max of 2a, 2b, 2c) ρ_n **= 0.0025**

2.0e) Select Horizontal Shear Reinforcing **Asn required per ft on each face =** **0.72** **in^2/ft each face** ($\rho_n{*}12{*}t_w{*}12/2$)

Use **1-#11@12"c/c EF** **Asn provided =** **1.56** **in^2/ft each face** ρ_n **(prov)=0.00542**

2.0f) Check Demand / Capacity Ratio for In-Plane Shear:

D/C = 0.14 $D/C=(Vu/\phi)/[Vc+(Asn{*}2{*}fy{*}l_w{*}t_w{*}144/(12{*}t_w{*}12))]$

3.0 Vertical Reinforcing Requirements

3.0a) ACI 349 21.6.5.5 and 11.10.9.4 - Minimum vertical reinforcing ratio : hw/lw = 0.39

If h_w/l_w>2.0, use: $\rho_{v\ min}=0.0025+0.5(2.5-h_w/l_w)(\rho_n-0.0025)<=\rho_n$

ρ_v **(min) =0.0025** If h_w/l_w<=2.0, use: $\rho_v>=\rho_n$

3.0b) Check Shear Friction Requirements

In plane shear per foot of wall: 24.30 kips/ft (Vu/l_w)

Transverse shear per ft of wall: 19.00 kips/ft (Vz)

Resultant Shear 30.85 kips/ft $[(\text{in-plane shear})^2+(\text{transverse shear})^2]^{0.5}$

Calculate limiting shear friction strength at joint per ACI 349-11.7.5:

Vn<.2f'cAv for f'c = 5000 psi Vn=1000Av
Vn<800Av The limiting value of 800Av controls
Vn (MAX) = 460.8 kips/ft **Vn(MAX>Resultant shear ....OK**

Calculate shear friction reinforcing requirements per ACI 349 - 11.7.4.1:

Vn = Avf*fy*μ μ = 1.0 for concrete placed against hardened concrete intentionally roughened to a full amplitude of 1/4 inch (ACI 349-11.7.9)

Avf = Vu/2*ϕ*μ*fy (steel required per face)
Avf = 0.30 in^2/ft (steel required on each face for shear friction))

Calculate steel required for net Tension force

At = T/2*ϕ*fy*lw
At = 0.03 in^2/ft (steel required on each face for direct tension)

Steel Requirements for Shear Friction Avf + At

Av = 0.33 in^2/ft (steel required on each face for shear friction + direct tension)

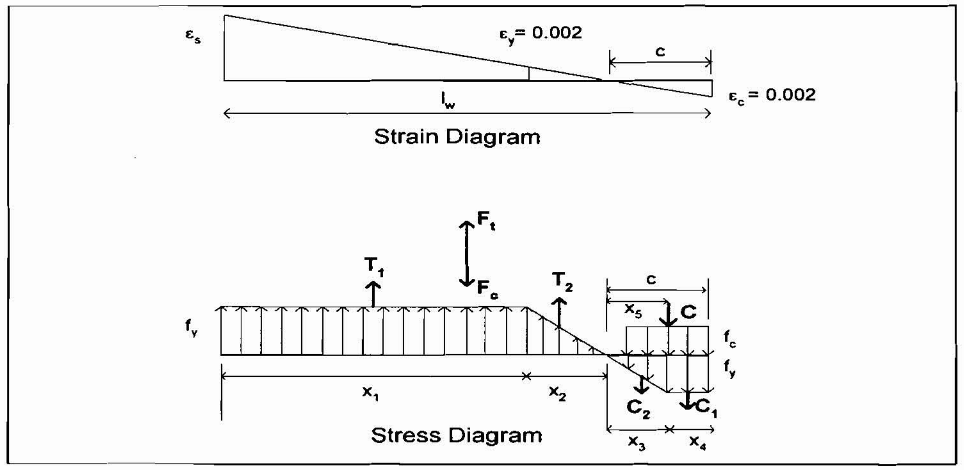
ρv(req'd) = 0.0011 $(\rho_{v\,req'd}=(2*Av)/(12*t_w*12)$

3.0c) Vertical Reinforcing Requirements (max of 3a,3b) **ρ_v = 0.0025**

3.0d) Select Vertical Reinforcing **Asv required per ft on each face = 0.72 in²/ft each face** $(\rho_{v\,min}*12*t_w*12/2)$

Use 1-#11@12"c/c EF **Asv provided = 1.56 in²/ft each face** **ρ_v (prov)= 0.00542**

3.0e) Perform Strain-Compatible Section Analysis - For Axial Force (Tension) **Ft =** -257 kips



ϕ : Strength reduction factor (= 0.9 for tension, ACI 349 9.3.2.2), ϕ = 0.9
c : Distance from compression face to neutral axis (ft)
l_w : Length of wall (ft)
$\varepsilon_{s\_max} = [(l_w - c) / c] * \varepsilon_c$
$X_1 = l_w - c - X_2$ (ft) (length of wall with tension steel reinforcing steel strain > ε_y)
$X_2 = X_3 = (\varepsilon_y / \varepsilon_c) * c$ (ft) (length of wall with tension / compression steel reinforcing steel strain < ε_y)
$X_4 = c - X_3$ (ft) (length of wall with compression steel reinforcing steel strain > ε_y)
$T_1 = X_1 * As * fy$ (kips)
$T_2 = C_2 = 0.5 * X_2 * As * fy$ (kips)
$C_1 = X_4 * As * fy$ (kips)
$C = 0.85 * 0.8 * c * t_w * f'_c * 144$ (kips)
Balanced condition : Tension = Compression, $T_1+T_2-C_2-C_1-C+Ft/\phi = 0$.
M_u : Total moment capacity = $\phi\{T_1(X_2+X_1/2) + T_2(X_2*2/3) + C(c-0.8*c/2) + F_t(l_w/2-c)/\phi + C_2(X_3*2/3) + C_1(X_3+X_4/2)\}$, (kip-ft)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
7.10	82.90	0.0234	75.80	7.10	0.00	14190	665	0	13904	0.000	623917

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
7	83	0.0237	76.00	7.00	0.00	14227.2	655.2	0	13709	233	623759
9	81.0	0.0180	72.00	9.00	0.00	13478.4	842.4	0	17626	-4433	631382
8	82	0.0205	74.00	8.00	0.00	13852.8	748.8	0	15667	-2100	626400

Perform Strain-Compatible Section Analysis - For Axial Force (Comp) **Fc =** 0 kips

ϕ : Strength reduction factor (= 0.7 to 0.9 for comp, ACI 349 9.3.2.2), ϕ = 0.9000
If Fc<=0, ϕ=0.9; if Fc>0, ϕ=0.9-0.2*Fc/(0.1*Acv*144*f'$_c$/1000)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
7.22	82.78	0.0229	75.56	7.22	0.00	14144	676	0	14144	0.000	633852

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
7	83	0.0237	76.00	7.00	0.00	14227.2	655.2	0	13709	518	633525
9	81.0	0.0180	72.00	9.00	0.00	13478.4	842.4	0	17626	-4147	640634
8	82	0.0205	74.00	8.00	0.00	13852.8	748.8	0	15667	-1814	635909

3.0f) Check Demand / Capacity Ratio for In-Plane Moment:

Vertical Reinforcing Ratio Provided by Design : ρ_v (prov)= 0.0054 1-#11@12"c/c EF ρ_v = (2*Asv)/(12*t_w*12)
D/C = (in-plane moment Mz) / (Min of Mu-ten, Mu-comp) D/C = 0.08 **Section Adequate**
Vertical Reinforcing Ratio Required for In-Plane Moment: $\rho_{vl\,req}$ = 0.0004 $\rho_{v\,req}$ = ρ_v*(D/C)

3.0g) Consider Out-of-Plane Moment:

Vertical Reinf Ratio Required for Out-of-Plane Bending: $\rho_{vt\,req}$ = 0.00211 (per face) $\rho f_y bd$^2*(1-.59$\rho f_y/f'_c$)=Mu/ϕ solve for ρ Ref. 2.2.19, Section 4-3

3.0h) Total Reinforcing required for axial force, in-plane and out of plane bending ρ_v = 0.0042 = $\rho_{vl}+\rho_{vt}$ > 2ρ_{vt}

D/C = ρ_v(reqd) / ρ_v (prov) D/C = 0.78 **Section Adequate** = ρ_v(req'd) / ρ_v(prov)

4.0 Boundary Elements:

h_w / l_w = 0.39 <2, No Need To Check for Boundary Elements

5.0 Out-of-Plane Shear:

Nominal Shear Strength Provided by The Concrete Vc=2*(f'c)$^{1/2}$*b*d Vc = 73.0 kips/ft width of wall

Check Demand / Capacity Ratio: D/C = 0.31 No Shear Reinforcement Required
D/C = (out-of-plane shear Vz) / (0.85*Vc)

6.0 Tabulate Reinforcement Requirements And D/C Ratios:

Use 4 ft thick wall with 1-#11@12"c/c EF Horizontal Reinforcement
1-#11@12"c/c EF Vertical Reinforcement.

For Shear on Gross Section:	**D/C =**	**0.12**
For In-Plane Shear:	**D/C =**	**0.14**
For Out-of-Plane Shear:	**D/C =**	**0.31**
Bending + axial Loads	**D/C =**	**0.78**

Boundary Elements D/C= BOUNDARY ELEMENTS NOT REQUIRED

Longitudinal Wall COL F-INTSCUT1---D+L+SRSS

Design Loads

Axial Force (-Tension)	**Ft =**	0 kips
Axial Force (+Comp)	**Fc =**	6030 kips
In plane shear	**Vu =**	2095 kips
In plane Moment	**Mz =**	24031 ft-kips
Out of plane shear	**Vz =**	15 kips/ft
Out of plane Moment	**My =**	239 ft-kips/ft (M22+M12)

Shear Wall Section Properties

Height of Wall (segment)	h_w =	35 ft
Ht of Wall Between Floors	**H =**	35 ft
Length of Wall (Segment)	l_w =	90 ft
Thickness of Wall	t_w =	4 ft
Shear Area of Wall (Segment)	$Acv = l_w{*}t_w$ =	360 ft^2

Concrete & Rebar Properties

Concrete Design Strength	**f'c =**	5000 psi
Concrete Strain	ε_c =	0.002
Rebar Yield Strength	**fy =**	60 ksi
Rebar Yield Strain	ε_y =	0.002
Min Steel Required	ρ_{min} =	0.0025
Concrete Cover		5 in

(Use 5" = 2" clear cover + diameter of the outer layer rebar + 1/2 diameter of the inner layer rebar)

Note: For ACI 349, see Ref. 2.2.14, for sections 1.0 to 6.0 of this design subset.

1.0 Check Shear on gross section - ACI 349: 21.6.5.6

Nominal Shear Capacity = 8*Acv*(f'c) $^{1/2}$ Vn (kips) = 29325

Factored Shear Load = Vu / ϕ Vu/ϕ (kips)= 3491.7
(ϕ = .6 per ACI 349 - 9.3.4)

Demand Capacity Ratio
Check Code 21.6.5.6 Demand/Capacity Ratio D/C = (Vu/ϕ)/Vn D/C = 0.12 **SHEAR WALL THICKNESS OK**

2.0 Horizontal Reinforcing Requirements

2.0a) ACI 349 - 21.6.5.3 Requirements

Determine α_c : h_w / l_w = 0.39 α_c = 3 α_c=3 for h_w/l_w<1.5, α_c varies linearly from 3 for h_w/l_w=1.5 to 2 for h_w/l_w=2.

Determine Concrete Shear Capacity Vc=Acv*α_c*(f'c) $^{1/2}$ Vc = 10996.9 kips

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc Vs = 0.0 kips

Determine Required Shear Reinforcing ρ=Vs/(fy*Acv) ρ = 0.0000

21.6.5.3 Shear Reinforcing Requirements **ρ_n = 0.0025** MINIMUM STEEL GOVERNS

2.0b) ACI 349 - 11.10.6 Equation 11-31 Requirements

Determine Concrete Shear Capacity Vc = 9677.3 kips $Vc=3.3*(f'_c)^{0.5}*(0.8*t_w*l_w)+F_t*0.8*l_w/(4*l_w)$

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc Vs = 0.0 kips

Determine Required Shear Reinforcing ρ = 0.0000 $\rho=Vs/(0.8*l_w*t_w*144*fy)$

11.10.6 - Equation 11-31 Shear Reinforcing Requirements **ρ_n = 0.0025** MINIMUM STEEL GOVERNS

2.0c) ACI 349 - 11.10.6 Equation 11-32 Requirements

Check Bounding Case Mu/Vu - l_w/2 = -33.53 **Mu/Vu-lw/2 < 0 equation 11-32 NOT APPLICABLE**

Determine Concrete Shear Capacity Vc= N/A $Vc=[0.6*f'_c{}^{0.5}+l_w*(1.25*f'_c{}^{0.5}+0.2*Ft*1000/(l_w*t_w*144))/(Mu/Vu-l_w/2)]*t_w*0.8*l_w*144/1000$

Determine Shear Carried by Steel Vs = N/A Vs=Vu/ϕ -Vc

Determine Required Shear Reinforcing Requirements ρ = N/A $\rho=Vs/(0.8*l_w*t_w*144*fy)$

11.10.6 - Equation 11-32 Shear Reinforcing Requirements **ρ_n = N/A** **EQUATION 11-32 NOT APPLICABLE**

2.0d) Horizontal Shear Reinforcing Requirements (max of 2a, 2b, 2c) **ρ_n = 0.0025**

2.0e) Select Horizontal Shear Reinforcing **Asn required per ft on each face =** **0.72** **in^2/ft each face** $(\rho_n*12*t_w*12/2)$

Use **1-#11@12"c/c EF** **Asn provided =** **1.56** **in^2/ft each face** **ρ_n (prov)= 0.0054167**

2.0f) Check Demand / Capacity Ratio for In-Plane Shear:

D/C = 0.13 $D/C=(Vu/\phi)/[Vc+(Asn*2*fy*l_w*t_w*144/(12*t_w*12))]$

3.0 Vertical Reinforcing Requirements

3.0a) ACI 349 21.6.5.5 and 11.10.9.4 - Minimum vertical reinforcing ratio : hw/lw = 0.39

If h_w/l_w>2.0, use: $\rho_{v\,min}=0.0025+0.5(2.5-h_w/l_w)(\rho_n-0.0025)<=\rho_n$
If h_w/l_w<=2.0, use: $\rho_v>=\rho_n$

ρ_v (min) =0.0025

3.0b) Check Shear Friction Requirements

In plane shear per foot of wall: 23.28 kips/ft (Vu/l_w)

Transverse shear per ft of wall: 15.00 kips/ft (Vz)

Resultant Shear 27.69 kips/ft $[(\text{in-plane shear})^2+(\text{transverse shear})^2]^{0.5}$

Calculate limiting shear friction strength at joint per ACI 349-11.7.5:

Vn<.2f'cAv for f'c = 5000 psi Vn=1000Av
Vn<800Av The limiting value of 800Av controls
Vn (MAX) = 460.8 kips/ft **Vn(MAX>Resultant shear ....OK**

Calculate shear friction reinforcing requirements per ACI 349 - 11.7.4.1:

Vn = Avf*fy*μ μ = 1.0 for concrete placed against hardened concrete intentionally roughened to a full amplitude of 1/4 inch (ACI 349-11.7.9)

Avf = Vu/2*ϕ*μ*fy (steel required per face)
Avf = 0.27 in^2/ft (steel required on each face for shear friction))

Calculate steel required for net Tension force

At = T/2*ϕ*fy*lw
At = 0.00 in^2/ft (steel required on each face for direct tension)

Steel Requirements for Shear Friction Avf + At

Av = 0.27 in^2/ft (steel required on each face for shear friction + direct tension)

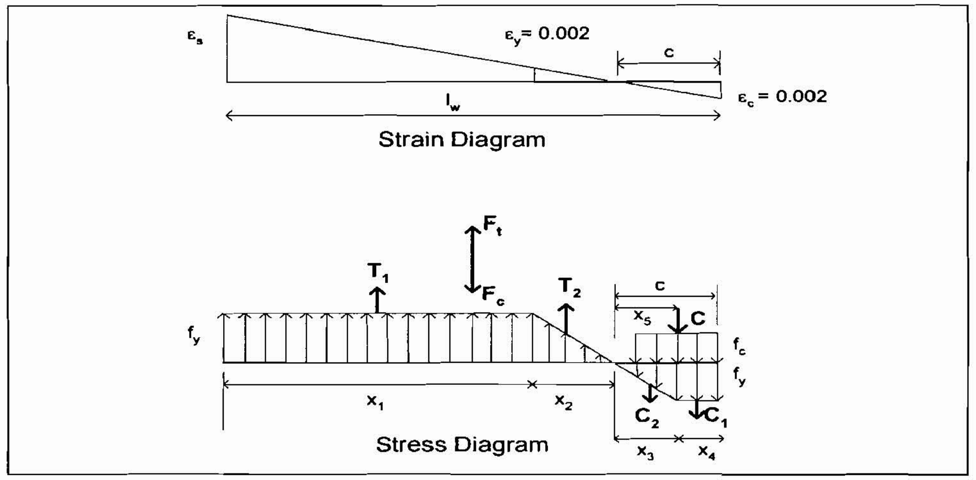
ρv(req'd) = 0.0009 $(\rho_{v\ req'd}=(2^*Av)/(12^*t_w{}^*12)$

3.0c) Vertical Reinforcing Requirements (max of 3a,3b) **ρ_v = 0.0025**

3.0d) Select Vertical Reinforcing **Asv required per ft on each face = 0.72 in²/ft each face** $(\rho_{v\,min}{}^*12^*t_w{}^*12/2)$

Use 1-#11@12"c/c EF Asv provided = 1.56 in²/ft each face ρ_v (prov)= 0.0054167

3.0e) Perform Strain-Compatible Section Analysis - For Axial Force (Tension) **Ft = 0 kips**



ϕ : Strength reduction factor (= 0.9 for tension, ACI 349 9.3.2.2), ϕ = 0.9
c : Distance from compression face to neutral axis (ft)
l_w : Length of wall (ft)
$\varepsilon_{s\_max} = [(l_w - c) / c] * \varepsilon_c$
$X_1 = l_w - c - X_2$ (ft) (length of wall with tension steel reinforcing steel strain > ε_y)
$X_2 = X_3 = (\varepsilon_y / \varepsilon_c) * c$ (ft) (length of wall with tension / compression steel reinforcing steel strain < ε_y)
$X_4 = c - X_3$ (ft) (length of wall with compression steel reinforcing steel strain > ε_y)
$T_1 = X_1 * As * fy$ (kips)
$T_2 = C_2 = 0.5 * X_2 * As * fy$ (kips)
$C_1 = X_4 * As * fy$ (kips)
$C = 0.85 * 0.8 * c * t_w * f'_c * 144$ (kips)
Balanced condition : Tension = Compression, $T_1+T_2-C_2-C_1-C+Ft/\phi = 0$.
M_u : Total moment capacity = $\phi\{T_1(X_2+X_1/2) + T_2(X_2*2/3) + C(c-0.8*c/2) + F_t(l_w/2-c)/\phi + C_2(X_3*2/3) + C_1(X_3+X_4/2)\}$, (kip-ft)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
7.22	82.78	0.0229	75.56	7.22	0.00	14144	676	0	14144	0.000	633852

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
7	83	0.0237	76.00	7.00	0.00	14227.2	655.2	0	13709	518	633525
9	81.0	0.0180	72.00	9.00	0.00	13478.4	842.4	0	17626	-4147	640634
8	82	0.0205	74.00	8.00	0.00	13852.8	748.8	0	15667	-1814	635909

Perform Strain-Compatible Section Analysis - For Axial Force (Comp) **Fc =** 6030 kips

ϕ : Strength reduction factor (= 0.7 to 0.9 for comp, ACI 349 9.3.2.2), ϕ = 0.8535
If Fc<=0, ϕ=0.9; if Fc>0, ϕ=0.9-0.2*Fc/(0.1*Acv*144*f'_c/1000)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
10.25	79.75	0.0156	69.50	10.25	0.00	13010	959	0	20075	0.000	825779

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
10	80	0.0160	70.00	10.00	0.00	13104	936	0	19584	585	825263
12	78.0	0.0130	66.00	12.00	0.00	12355.2	1123.2	0	23501	-4080	833257
11	79	0.0144	68.00	11.00	0.00	12729.6	1029.6	0	21542	-1748	828151

3.0f) Check Demand / Capacity Ratio for In-Plane Moment:

Vertical Reinforcing Ratio Provided by Design : ρ_v(prov)= 0.0054 1-#11@12"c/c EF ρ_v = (2*Asv)/(12*t_w*12)
D/C = (in-plane moment Mz) / (Min of Mu-ten, Mu-comp) D/C = 0.04 **Section Adequate**
Vertical Reinforcing Ratio Required for In-Plane Moment: $\rho_{vl\,req}$ = 0.0002 $\rho_{v\,req} = \rho_v$*(D/C)

3.0g) Consider Out-of-Plane Moment:

Vertical Reinf Ratio Required for Out-of-Plane Bending: $\rho_{vt\,req}$ = 0.00244 (per face) $\rho f_y bd$^2*(1-.59$\rho f_y/f'_c$)=Mu/ϕ solve for ρ Ref. 2.2.19, Section 4-3

3.0h) Total Reinforcing required for axial force, in-plane and out of plane bending ρ_v = 0.0049 = $\rho_{vl}+\rho_{vt} > 2\rho_{vt}$

D/C = ρ_v(reqd) / ρ_v (prov) D/C = 0.90 **Section Adequate** = ρ_v(req'd) / ρ_v(prov)

4.0 Boundary Elements:

h_w / l_w = 0.39 <2, No Need To Check for Boundary Elements

5.0 Out-of-Plane Shear:

Nominal Shear Strength Provided by The Concrete Vc=2*(f'c)$^{1/2}$*b*d Vc = 73.0 kips/ft width of wall

Check Demand / Capacity Ratio: D/C = 0.24 No Shear Reinforcement Required
D/C = (out-of-plane shear Vz) / (0.85*Vc)

6.0 Tabulate Reinforcement Requirements And D/C Ratios:

Use 4 ft thick wall with 1-#11@12"c/c EF Horizontal Reinforcement
1-#11@12"c/c EF Vertical Reinforcement.

For Shear on Gross Section:	**D/C =**	**0.12**
For In-Plane Shear:	**D/C =**	**0.13**
For Out-of-Plane Shear:	**D/C =**	**0.24**
Bending + axial Loads	**D/C =**	**0.90**
Boundary Elements	**D/C=**	**BOUNDARY ELEMENTS NOT REQUIRED**

6.5.6

Transverse Walls Design Loads, column line 3.6

Section cut design forces and moments, which follow a global axis system, for the transverse wall on column line 3.6 include inplane forces and moments: such as axial forces (tension/compression), in plane moment, and in plane shear. The section cut values are integrated along the section cut length, thus for INTSCUT3a and INTSCUT3b the length is equal to 8.5'. Out of plane values such as out of plane bending and out of plane shear are attained by shell element forces and moments from SAP2000, which follow a local axis system. The out of plane bending moments provided are M22 and M12, which is a twisting moment that is combined with M22. The out of plane shear forces from the shell elements include V13 and V23. From comparison of loads at INTSCUT3a and INTSCUT3b, loads at INSCUT3b are smaller than INTSCUT3a (except M2, but the difference is negligible). Thus, use loads from INTSCUT3a for designing both wall piers.
Follow notation below to convert from SAP2000 labeling (i.e F1, M22...) to appropriate design forces and moments.

INTSCUT3a & INTSCUT3b
F1 = In Plane Shear
F3 = Axial Force, Compression (+) / Tension (-)
M2 = In Plane Moment

Accidental torsion factor = 15% (See assumption 3.1.4)

Shell Element Forces and Moments
M22 = Out of Plane Moment
M12 = Twisting Moment
V13 = Out of Plane Shear
V23 = Out of Plane Shear

Section Cu	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
NTSCUT3a	-1	-	264	-	182	-	320	-	1007	-	3925	-
NTSCUT3b	2	-	267	-	-176	-	303	-	992	-	3641	-

MAX MIN	M11	M22	M12	V13	V23	MAX	M11	M22	M12	V13	V23
	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft		Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	5	27	12	1	0	MAX	16	36	13	7	6
MIN	-5	-4	-11	-1	-3						

Loads with accidental torsion factor

Section Cu	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
NTSCUT3a	-1	-	303	-	210	-	368	-	1158	-	4514	-
NTSCUT3b	2	-	307	-	-202	-	348	-	1141	-	4187	-

Loads with accidental torsion factor

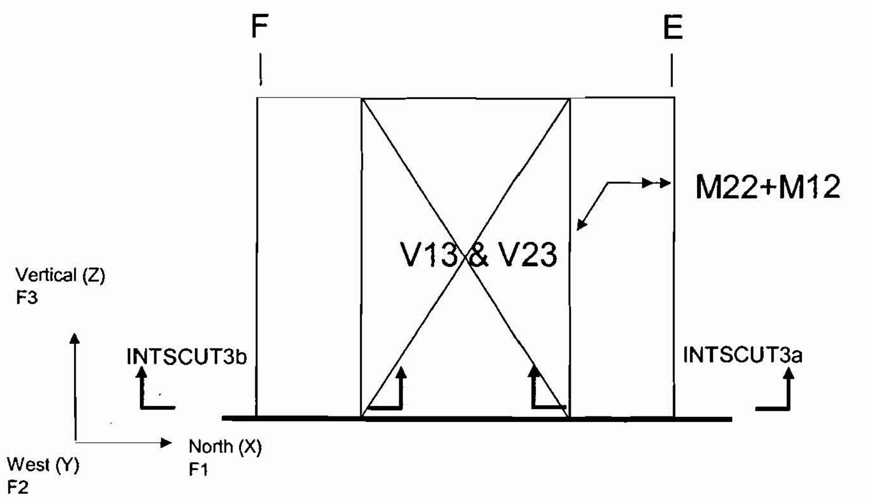
MAX MIN	M11	M22	M12	V13	V23	MAX	M11	M22	M12	V13	V23
	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft		Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	6	31	14	1	0	MAX	18	42	15	8	7
MIN	-6	-5	-13	-1	-3						

Maximum Load Combination

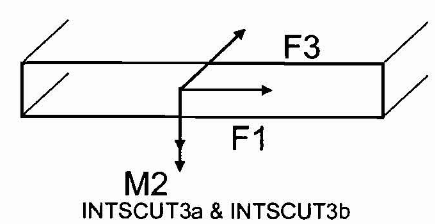
Section Cu	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
NTSCUT3a	368	-	1461	-	4723	-	-369	-	-854	-	-4304	-
NTSCUT3b	351	-	1447	-	3985	-	-346	-	-834	-	-4390	-

Maximum Load Combination

MAX MIN	M11	M22	M12	V13	V23	MAX MIN	M11	M22	M12	V13	V23
	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft		Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	24	73	29	9	7	MAX	-12	-11	-2	-7	-7
MIN	12	37	2	7	4	MIN	-23	-47	-28	-9	-10



TRANSVERSE WALL AT COL. 3.6 ---- ELEVATION



6.5.6.1 TRANSVERSE WALL REINFORCEMENT, COLUMN LINE 3.6

Transverse Wall COL 3.6-INTSCUT3a---D+L-SRSS

Design Loads			Shear Wall Section Properties			Concrete & Rebar Properties		
Axial Force (-Tension)	**Ft =**	-854 kips	**Height of Wall (segment)**	h_w =	26.08 ft	**Concrete Design Strength**	f'c =	5000 psi
Axial Force (+Comp)	**Fc =**	0 kips	**Ht of Wall Between Floors**	H =	26.08 ft	**Concrete Strain**	ε_c =	0.002
In plane shear	**Vu =**	369 kips	**Length of Wall (Segment)**	l_w =	8.5 ft	**Rebar Yield Strength**	fy =	60 ksi
In plane Moment	**Mz =**	4304 ft-kips	**Thickness of Wall**	t_w =	4 ft	**Rebar Yield Strain**	ε_y =	0.002
			Shear Area of Wall (Segment)	$Acv = l_w{}^*t_w$ =	34 ft^2	**Min Steel Required**	ρ_{min} =	0.0025
						Concrete Cover		5 in
Out of plane shear	**Vz =**	10 kips/ft				(Use 5" = 2" clear cover + diameter of the outer layer rebar + 1/2 diameter of the inner layer rebar)		
Out of plane Moment	**My =**	75 ft-kips/ft (M22+M12)						

Note: For ACI 349, see Ref. 2.2.14, for sections 1.0 to 6.0 of this design subset.

1.0 Check Shear on gross section - ACI 349: 21.6.5.6

Nominal Shear Capacity = 8*Acv*(f'c) $^{1/2}$ — Vn (kips) = 2770

Factored Shear Load = Vu / ϕ — Vu/ϕ (kips)= 615.0
(ϕ = .6 per ACI 349 - 9.3.4)

Demand Capacity Ratio
Check Code 21.6.5.6 Demand/Capacity Ratio — D/C = (Vu/ϕ)/Vn — D/C = 0.22 **SHEAR WALL THICKNESS OK**

2.0 Horizontal Reinforcing Requirements

2.0a) ACI 349 - 21.6.5.3 Requirements

Determine α_c : h_w / l_w = 3.07 — α_c = 2 — α_c=3 for h_w/l_w<1.5, α_c varies linearly from 3 for h_w/l_w=1.5 to 2 for h_w/l_w=2.

Determine Concrete Shear Capacity Vc=Acv*α_c*(f'c) $^{1/2}$ — Vc = 692.4 kips

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc — Vs = 0.0 kips

Determine Required Shear Reinforcing ρ=Vs/(fy*Acv) — ρ = 0.0000

21.6.5.3 Shear Reinforcing Requirements — **ρ_n = 0.0025** MINIMUM STEEL GOVERNS

2.0b) ACI 349 - 11.10.6 Equation 11-31 Requirements

Determine Concrete Shear Capacity — Vc = 743.2 kips — $Vc=3.3^*(f'_c)^{0.5*}(0.8^*t_w{}^*l_w)+F_t{}^*0.8^*l_w/(4^*l_w)$

Determine Shear Carried by Steel — Vs=Vu/ϕ -Vc — Vs = 0.0 kips

Determine Required Shear Reinforcing — ρ = 0.0000 — $\rho=Vs/(0.8^*l_w{}^*t_w{}^*144^*fy)$

11.10.6 - Equation 11-31 Shear Reinforcing Requirements — **ρ_n = 0.0025** MINIMUM STEEL GOVERNS

2.0c) ACI 349 - 11.10.6 Equation 11-32 Requirements

Check Bounding Case Mu/Vu - l_w/2 = 7.41 **Equation 11-32 APPLICABLE**

Determine Concrete Shear Capacity — Vc= 406.432831 — $Vc=[0.6^*f'_c{}^{0.5}+l_w{}^*(1.25^*f'_c{}^{0.5}+0.2^*Ft^*1000/(l_w{}^*t_w{}^*144))/(Mu/Vu-l_w/2)]^*t_w{}^*0.8^*l_w{}^*144/1000$

Determine Shear Carried by Steel — Vs = 208.567169 — Vs=Vu/ϕ -Vc

Determine Required Shear Reinforcing Requirements — ρ = 0.0009 — $\rho=Vs/(0.8^*l_w{}^*t_w{}^*144^*fy)$

11.10.6 - Equation 11-32 Shear Reinforcing Requirements — **ρ_n = 0.0025 MINIMUM STEEL GOVERNS**

2.0d) Horizontal Shear Reinforcing Requirements (max of 2a, 2b, 2c) — **ρ_n = 0.0025**

2.0e) Select Horizontal Shear Reinforcing — **Asn required per ft on each face =** **0.72** **in^2/ft each face** $(\rho_n{}^*12^*t_w{}^*12/2)$

Use **1-#11@12"c/c EF** **Asn provided =** **1.56** **in^2/ft each face** **ρ_n (prov)=0.00542**

2.0f) Check Demand / Capacity Ratio for In-Plane Shear:

D/C = 0.31 — $D/C=(Vu/\phi)/[Vc+(Asn^*2^*fy^*l_w{}^*t_w{}^*144/(12^*t_w{}^*12))]$

3.0 Vertical Reinforcing Requirements

3.0a) ACI 349 21.6.5.5 and 11.10.9.4 - Minimum vertical reinforcing ratio : — hw/lw = 3.07

If h_w/l_w>2.0, use: $\rho_{v\,min}=0.0025+0.5(2.5-h_w/l_w)(\rho_n-0.0025)<=\rho_n$
If h_w/l_w<=2.0, use: $\rho_v>=\rho_n$

ρ_v (min) =0.0025

3.0b) Check Shear Friction Requirements

In plane shear per foot of wall: 43.41 kips/ft (Vu/l_w)

Transverse shear per ft of wall: 10.00 kips/ft (Vz)

Resultant Shear 44.55 kips/ft $[(\text{in-plane shear})^2+(\text{transverse shear})^2]^{0.5}$

Calculate limiting shear friction strength at joint per ACI 349-11.7.5:

Vn<.2f'cAv for f'c = 5000 psi Vn=1000Av
Vn<800Av The limiting value of 800Av controls
Vn (MAX) = 460.8 kips/ft **Vn(MAX>Resultant shear ....OK**

Calculate shear friction reinforcing requirements per ACI 349 - 11.7.4.1:

Vn = Avf*fy*μ μ = 1.0 for concrete placed against hardened concrete intentionally roughened to a full amplitude of 1/4 inch (ACI 349-11.7.9)

Avf = Vu/2*ϕ*μ*fy (steel required per face)
Avf = 0.44 in^2/ft (steel required on each face for shear friction))

Calculate steel required for net Tension force

At = T/2*ϕ*fy*lw
At = 0.99 in^2/ft (steel required on each face for direct tension)

Steel Requirements for Shear Friction Avf + At

Av = 1.42 in^2/ft (steel required on each face for shear friction + direct tension)

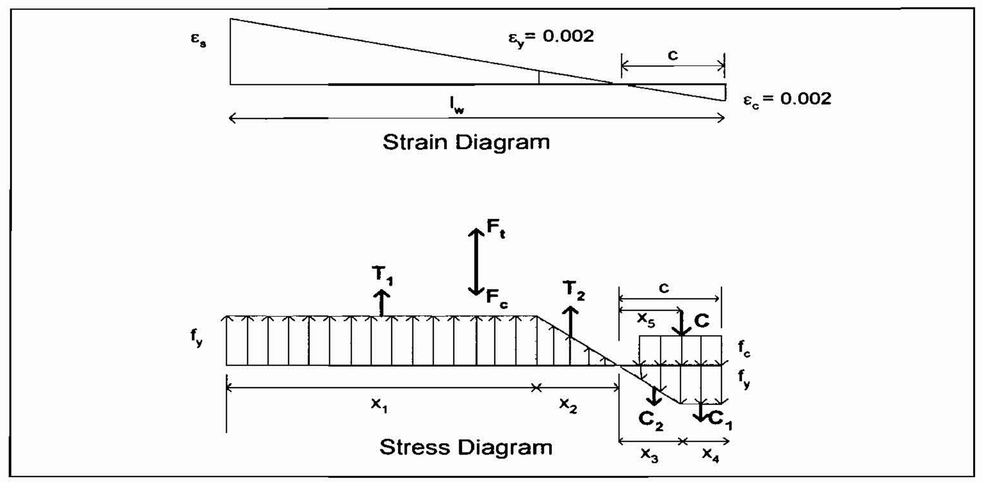
ρv(req'd) = 0.0049 ($\rho_{v\,req'd}$=(2*Av)/(12*t_w*12)

3.0c) Vertical Reinforcing Requirements (max of 3a,3b) **ρ_v = 0.0049**

3.0d) Select Vertical Reinforcing **Asv required per ft on each face =** **1.42** **in^2/ft each face** ($\rho_{v\,min}$*12*t_w*12/2)

Use 1-#11@6"c/c EF **Asv provided = 3.12** **in^2/ft each face** **ρ_v (prov)= 0.01083**

3.0e) Perform Strain-Compatible Section Analysis - For Axial Force (Tension) **Ft =** -854 kips



ϕ : Strength reduction factor (= 0.9 for tension, ACI 349 9.3.2.2), ϕ = 0.9
c : Distance from compression face to neutral axis (ft)
l_w : Length of wall (ft)
$\varepsilon_{s\_max} = [(l_w - c) / c] * \varepsilon_c$
$X_1 = l_w - c - X_2$ (ft) (length of wall with tension steel reinforcing steel strain > ε_y)
$X_2 = X_3 = (\varepsilon_y / \varepsilon_c) * c$ (ft) (length of wall with tension / compression steel reinforcing steel strain < ε_y)
$X_4 = c - X_3$ (ft) (length of wall with compression steel reinforcing steel strain > ε_y)
$T_1 = X_1 * As * fy$ (kips)
$T_2 = C_2 = 0.5 * X_2 * As * fy$ (kips)
$C_1 = X_4 * As * fy$ (kips)
$C = 0.85 * 0.8 * c * t_w * f'_c * 144$ (kips)
Balanced condition : Tension = Compression, $T_1+T_2-C_2-C_1-C+Ft/\phi = 0$.
M_u : Total moment capacity = $\phi\{T_1(X_2+X_1/2) + T_2(X_2*2/3) + C(c-0.8*c/2) + F_t(l_w/2-c)/\phi + C_2(X_3*2/3) + C_1(X_3+X_4/2)\}$, (kip-ft)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
0.83	7.67	0.0186	6.85	0.83	0.00	2565	154	0	1616	0.000	7757

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
2	6.5	0.0065	4.50	2.00	0.00	1684.8	374.4	0	3917	-3181	9652
1	7.5	0.0150	6.50	1.00	0.00	2433.6	187.2	0	1958	-474	7815
0.5	8	0.0320	7.50	0.50	0.00	2808	93.6	0	979	880	7859

Perform Strain-Compatible Section Analysis - For Axial Force (Comp) **Fc =** 0 kips

ϕ : Strength reduction factor (= 0.7 to 0.9 for comp, ACI 349 9.3.2.2), ϕ = 0.9000
If Fc<=0, ϕ=0.9; if Fc>0, ϕ=0.9-0.2*Fc/(0.1*Acv*144*f'c/1000)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	T_1+T_2-C_2-C_1-C+F_c/ϕ	M_u (k-ft)
1.18	7.32	0.0125	6.15	1.18	0.00	2302	220	0	2302	0.000	10578

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	T_1+T_2-C_2-C_1-C+F_c/ϕ	M_u (k-ft)
1	7.5	0.0150	6.50	1.00	0.00	2433.6	187.2	0	1958	475	10591
3	5.5	0.0037	2.50	3.00	0.00	936	561.6	0	5875	-4939	15120
2	6.5	0.0065	4.50	2.00	0.00	1684.8	374.4	0	3917	-2232	11573

3.0f) Check Demand / Capacity Ratio for In-Plane Moment:

Vertical Reinforcing Ratio Provided by Design : ρ_v (prov)= 0.0108 1-#11@6"c/c EF ρ_v = (2*Asv)/(12*t_w*12)
D/C = (in-plane moment Mz) / (Min of Mu-ten, Mu-comp) D/C = 0.55 **Section Adequate**
Vertical Reinforcing Ratio Required for In-Plane Moment: $\rho_{vl\ req}$ = 0.0060 $\rho_{v\ req}$ = ρ_v*(D/C)

3.0g) Consider Out-of-Plane Moment:

Vertical Reinf Ratio Required for Out-of-Plane Bending: $\rho_{vt\ req}$ = 0.00076 (per face) $\rho f_y bd$^2*(1-.59$\rho f_y/f'_c$)=Mu/ϕ solve for ρ Ref. 2.2.19, Section 4-3

3.0h) Total Reinforcing required for axial force, in-plane and out of plane bending ρ_v = 0.0068 = ρ_{vl}+ρ_{vt} > 2ρ_{vt}

D/C = ρ_v(reqd) / ρ_v(prov) D/C = 0.62 **Section Adequate** = ρ_v(req'd) / ρ_v(prov)

4.0 Boundary Elements:

h_w / l_w = 3.07 Concrete Strain limited to .002 No Need To Check for Boundary Elements

5.0 Out-of-Plane Shear:

Nominal Shear Strength Provided by The Concrete Vc=2*(f'c)$^{1/2}$*b*d Vc = 73.0 kips/ft width of wall

Check Demand / Capacity Ratio: D/C = 0.16 No Shear Reinforcement Required
D/C = (out-of-plane shear Vz) / (0.85*Vc)

6.0 Tabulate Reinforcement Requirements And D/C Ratios:

Use **4** **ft thick wall with** **1-#11@12"c/c EF** **Horizontal Reinforcement**
1-#11@6"c/c EF **Vertical Reinforcement.**

For Shear on Gross Section:	**D/C =**	**0.22**
For In-Plane Shear:	**D/C =**	**0.31**
For Out-of-Plane Shear:	**D/C =**	**0.16**
Bending + axial Loads	**D/C =**	**0.62**

Boundary Elements **D/C=** **BOUNDARY ELEMENTS NOT REQUIRED**

Transverse Wall COL 3.6-INTSCUT3a---D+L+SRSS

Design Loads			**Shear Wall Section Properties**			**Concrete & Rebar Properties**		
Axial Force (-Tension)	**Ft =**	0 kips	**Height of Wall (segment)**	**h_w =**	26.08 ft	**Concrete Design Strength**	**f'c =**	5000 psi
Axial Force (+Comp)	**Fc =**	1461 kips	**Ht of Wall Between Floors**	**H =**	26.08 ft	**Concrete Strain**	**ε_c =**	0.002
In plane shear	**Vu =**	368 kips	**Length of Wall (Segment)**	**l_w =**	8.5 ft	**Rebar Yield Strength**	**fy =**	60 ksi
In plane Moment	**Mz =**	4723 ft-kips	**Thickness of Wall**	**t_w =**	4 ft	**Rebar Yield Strain**	**ε_y =**	0.002
			Shear Area of Wall (Segment)	**Acv = l_w*t_w =**	34 ft^2	**Min Steel Required**	**ρ_{min} =**	0.0025
						Concrete Cover		5 in
Out of plane shear	**Vz =**	9 kips/ft						
Out of plane Moment	**My =**	102 ft-kips/ft (M22+M12)						

(Use 5" = 2" clear cover + diameter of the outer layer rebar + 1/2 diameter of the inner layer rebar)

Note: For ACI 349, see Ref. 2.2.14, for sections 1.0 to 6.0 of this design subset.

1.0 Check Shear on gross section - ACI 349: 21.6.5.6

Nominal Shear Capacity = 8*Acv*(f'c) $^{1/2}$ — Vn (kips) = 2770

Factored Shear Load = Vu / ϕ — Vu/ϕ (kips)= 613.3
(ϕ = .6 per ACI 349 - 9.3.4)

Demand Capacity Ratio
Check Code 21.6.5.6 Demand/Capacity Ratio — D/C = (Vu/ϕ)/Vn — D/C = 0.22 **SHEAR WALL THICKNESS OK**

2.0 Horizontal Reinforcing Requirements

2.0a) ACI 349 - 21.6.5.3 Requirements

Determine α_c : h_w / l_w = 3.07 — α_c = 2 — α_c=3 for h_w/l_w<1.5, α_c varies linearly from 3 for h_w/l_w=1.5 to 2 for h_w/l_w=2.

Determine Concrete Shear Capacity Vc=Acv*α_c*(f'c) $^{1/2}$ — Vc = 692.4 kips

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc — Vs = 0.0 kips

Determine Required Shear Reinforcing ρ=Vs/(fy*Acv) — ρ = 0.0000

21.6.5.3 Shear Reinforcing Requirements — **ρ_n = 0.0025** MINIMUM STEEL GOVERNS

2.0b) ACI 349 - 11.10.6 Equation 11-31 Requirements

Determine Concrete Shear Capacity — Vc = 914.0 kips — Vc=3.3*(f'$_c$)$^{0.5}$*(0.8*t_w*l_w)+F_t*0.8*l_w/(4*l_w)

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc — Vs = 0.0 kips

Determine Required Shear Reinforcing — ρ = 0.0000 — ρ=Vs/(0.8*l_w*t_w*144*fy)

11.10.6 - Equation 11-31 Shear Reinforcing Requirements — **ρ_n = 0.0025** MINIMUM STEEL GOVERNS

2.0c) ACI 349 - 11.10.6 Equation 11-32 Requirements

Check Bounding Case Mu/Vu - l_w/2 = 8.58 **Equation 11-32 APPLICABLE**

Determine Concrete Shear Capacity — Vc= 508.977895 — Vc=[0.6*f'$_c$$^{0.5}$+$l_w$*(1.25*f'$_c$$^{0.5}$+0.2*Ft*1000/($l_w$*$t_w$*144))/(Mu/Vu-$l_w$/2)]*$t_w$*0.8*$l_w$*144/1000

Determine Shear Carried by Steel — Vs = 104.355439 — Vs=Vu/ϕ -Vc

Determine Required Shear Reinforcing Requirements — ρ = 0.0004 — ρ=Vs/(0.8*l_w*t_w*144*fy)

11.10.6 - Equation 11-32 Shear Reinforcing Requirements — **ρ_n = 0.0025 MINIMUM STEEL GOVERNS**

2.0d) Horizontal Shear Reinforcing Requirements (max of 2a, 2b, 2c) — **ρ_n = 0.0025**

2.0e) Select Horizontal Shear Reinforcing — **Asn required per ft on each face = 0.72 in²/ft each face** (ρ_n*12*t_w*12/2)

Use 1-#11@12"c/c EF Asn provided = 1.56 in²/ft each face **ρ_n (prov)=0.00542**

2.0f) Check Demand / Capacity Ratio for In-Plane Shear:

D/C = 0.29 — D/C=(Vu/ϕ)/[Vc+(Asn*2*fy*l_w*t_w*144/(12*t_w*12))]

3.0 Vertical Reinforcing Requirements

3.0a) ACI 349 21.6.5.5 and 11.10.9.4 - Minimum vertical reinforcing ratio : — hw/lw = 3.07

If h_w/l_w>2.0, use: $\rho_{v\ min}$=0.0025+0.5(2.5-h_w/l_w)(ρ_n-0.0025)<=ρ_n

ρ_v (min) =0.0025 — If h_w/l_w<=2.0, use: ρ_v>=ρ_n

3.0b) Check Shear Friction Requirements

In plane shear per foot of wall: 43.29 kips/ft (Vu/l_w)

Transverse shear per ft of wall: 9.00 kips/ft (Vz)

Resultant Shear 44.22 kips/ft $[(\text{in-plane shear})^2+(\text{transverse shear})^2]^{0.5}$

Calculate limiting shear friction strength at joint per ACI 349-11.7.5:

Vn<.2f'cAv for f'c = 5000 psi Vn=1000Av
Vn<800Av The limiting value of 800Av controls
Vn (MAX) = 460.8 kips/ft **Vn(MAX>Resultant shear ....OK**

Calculate shear friction reinforcing requirements per ACI 349 - 11.7.4.1:

Vn = Avf*fy*μ μ = 1.0 for concrete placed against hardened concrete intentionally roughened to a full amplitude of 1/4 inch (ACI 349-11.7.9)

Avf = Vu/2*ϕ*μ*fy (steel required per face)
Avf = 0.43 in^2/ft (steel required on each face for shear friction))

Calculate steel required for net Tension force

At = T/2*ϕ*fy*lw
At = 0.00 in^2/ft (steel required on each face for direct tension)

Steel Requirements for Shear Friction Avf + At

Av = 0.43 in^2/ft (steel required on each face for shear friction + direct tension)

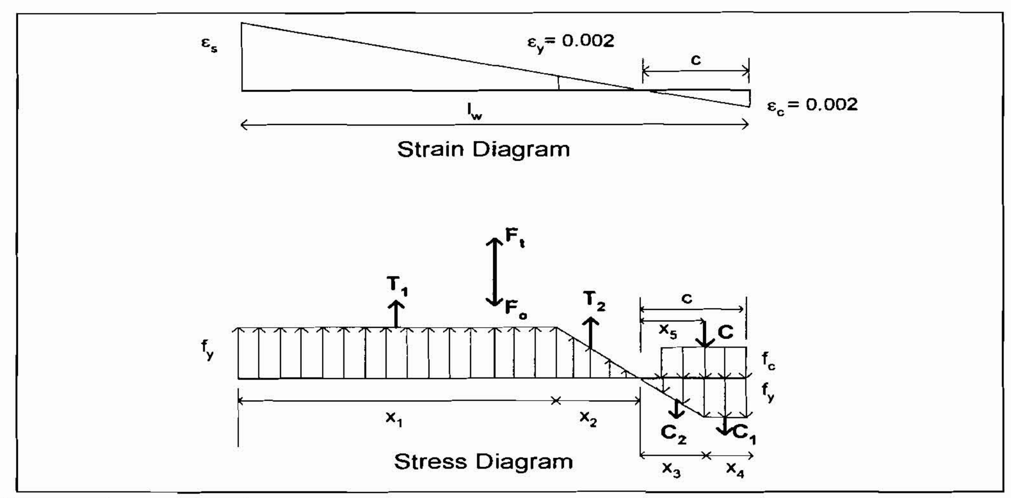
ρv(req'd) = 0.0015 ($\rho_{v\,req'd}=(2*Av)/(12*t_w*12)$)

3.0c) Vertical Reinforcing Requirements (max of 3a,3b) **ρ_v = 0.0025**

3.0d) Select Vertical Reinforcing **Asv required per ft on each face = 0.72 in^2/ft each face** ($\rho_{v\,min}*12*t_w*12/2$)

Use 1-#11@6"c/cEF **Asv provided = 3.12 in^2/ft each face** **ρ_v (prov)= 0.01083**

3.0e) Perform Strain-Compatible Section Analysis - For Axial Force (Tension) **Ft =** 0 kips



ϕ : Strength reduction factor (= 0.9 for tension, ACI 349 9.3.2.2), ϕ = 0.9
c : Distance from compression face to neutral axis (ft)
l_w : Length of wall (ft)
$\varepsilon_{s\_max} = [(l_w - c) / c] * \varepsilon_c$
$X_1 = l_w - c - X_2$ (ft) (length of wall with tension steel reinforcing steel strain > ε_y)
$X_2 = X_3 = (\varepsilon_y / \varepsilon_c) * c$ (ft) (length of wall with tension / compression steel reinforcing steel strain < ε_y)
$X_4 = c - X_3$ (ft) (length of wall with compression steel reinforcing steel strain > ε_y)
$T_1 = X_1 * As * fy$ (kips)
$T_2 = C_2 = 0.5 * X_2 * As * fy$ (kips)
$C_1 = X_4 * As * fy$ (kips)
$C = 0.85 * 0.8 * c * t_w * f'_c * 144$ (kips)
Balanced condition : Tension = Compression, $T_1+T_2-C_2-C_1-C+Ft/\phi = 0$.
M_u : Total moment capacity = $\phi\{T_1(X_2+X_1/2) + T_2(X_2*2/3) + C(c-0.8*c/2) + F_t(l_w/2-c)/\phi + C_2(X_3*2/3) + C_1(X_3+X_4/2)\}$, (kip-ft)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
1.18	7.32	0.0125	6.15	1.18	0.00	2302	220	0	2302	0.000	10578

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
1	7.5	0.0150	6.50	1.00	0.00	2433.6	187.2	0	1958	475	10591
3	5.5	0.0037	2.50	3.00	0.00	936	561.6	0	5875	-4939	15120

2	6.5	0.0065	4.50	2.00	0.00	1684.8	374.4	0	3917	-2232	11573

Perform Strain-Compatible Section Analysis - For Axial Force (Comp) **Fc =** 1461 kips

ϕ : Strength reduction factor (= 0.7 to 0.9 for comp, ACI 349 9.3.2.2), ϕ = 0.7806
If Fc<=0, ϕ=0.9; if Fc>0, ϕ=0.9-0.2*Fc/(0.1*Acv*144*f'$_c$/1000)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
1.87	6.63	0.0071	4.77	1.87	0.00	1784	349	0	3656	0.000	13278

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
1	7.5	0.0150	6.50	1.00	0.00	2433.6	187.2	0	1958	2347	13934
3	5.5	0.0037	2.50	3.00	0.00	936	561.6	0	5875	-3068	14941
2	6.5	0.0065	4.50	2.00	0.00	1684.8	374.4	0	3917	-360	13325

3.0f) Check Demand / Capacity Ratio for In-Plane Moment:

Vertical Reinforcing Ratio Provided by Design : ρ_v (prov)= 0.0108 1-#11@6"c/cEF $\rho_v = (2*Asv)/(12*t_w*12)$
D/C = (in-plane moment Mz) / (Min of Mu-ten, Mu-comp) D/C = 0.45 **Section Adequate**
Vertical Reinforcing Ratio Required for In-Plane Moment: $\rho_{vl\ req}$ = 0.0048 $\rho_{v\ req} = \rho_v*(D/C)$

3.0g) Consider Out-of-Plane Moment:

Vertical Reinf Ratio Required for Out-of-Plane Bending: $\rho_{vt\ req}$ = 0.00103 (per face) $\rho f_y bd^2(1-.59\rho f_y/f'_c)=Mu/\phi$ solve for ρ Ref. 2.2.19, Section 4-3

3.0h) Total Reinforcing required for axial force, in-plane and out of plane bending ρ_v = 0.0059 $= \rho_{vl}+\rho_{vt} > 2\rho_{vt}$

D/C = ρ_v(reqd) / ρ_v (prov) D/C = 0.54 **Section Adequate** = ρ_{vt}req'd) / ρ_v(prov)

4.0 Boundary Elements:

h_w / l_w = 3.07 Concrete Strain limited to .002 No Need To Check for Boundary Elements

5.0 Out-of-Plane Shear:

Nominal Shear Strength Provided by The Concrete Vc=2*(f'c)$^{1/2}$*b*d Vc = 73.0 kips/ft width of wall

Check Demand / Capacity Ratio: D/C = 0.15 No Shear Reinforcement Required
D/C = (out-of-plane shear Vz) / (0.85*Vc)

6.0 Tabulate Reinforcement Requirements And D/C Ratios:

Use **4** **ft thick wall with** **1-#11@12"c/c EF** **Horizontal Reinforcement**
1-#11@6"c/cEF **Vertical Reinforcement.**

For Shear on Gross Section:	**D/C =**	**0.22**
For In-Plane Shear:	**D/C =**	**0.29**
For Out-of-Plane Shear:	**D/C =**	**0.15**
Bending + axial Loads	**D/C =**	**0.54**

Boundary Elements **D/C=** **BOUNDARY ELEMENTS NOT REQUIRED**

6.5.7

Transverse Walls Design Loads, Column Line 5

Section cut design forces and moments, which follow a global axis system, for the transverse wall on column line 5 include inplane forces and moments: such as axial forces (tension/compression), in plane moment, and in plane shear. The section cut values are integrated along the section cut length, thus for INTSCUT4a and INTSCUT4b the length is equal to 8.5'. Out of plane values such as out of plane bending and out of plane shear are attained by shell element forces and moments from SAP2000, which follow a local axis system. The out of plane bending moments provided are M22 and M12, which is a twisting moment that is combined with M22. The out of plane shear forces from the shell elements include V13 and V23.

Follow notation below to convert from SAP2000 labeling (i.e F1, M22…) to appropriate design forces and moments.

INTSCUT4a & INTSCUT4b
F1 = In Plane Shear
F3 = Axial Force, Compression (+) / Tension (-)
M2 = In Plane Moment

Shell Element Forces and Moments
M22 = Out of Plane Moment
M12 = Twisting Moment
V13 = Out of Plane Shear
V23 = Out of Plane Shear

Accidental torsion factor = 15% (See assumption 3.1.4)

Section Cu	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
NTSCUT4a	-49	-	227	-	-246	-	432	-	933	-	3551	-
NTSCUT4b	55	-	217	-	294	-	438	-	984	-	3578	-

MAX MIN	M11	M22	M12	V13	V23	MAX	M11	M22	M12	V13	V23
	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft		Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	7	41	10	2	3	MAX	111	82	76	34	26
MIN	-8	-28	-10	-4	-5						

Loads with accidental torsion factor

Section Cu	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
NTSCUT4a	-56	-	261	-	-282	-	496	-	1073	-	4084	-
NTSCUT4b	63	-	249	-	338	-	504	-	1131	-	4115	-

Loads with accidental torsion factor

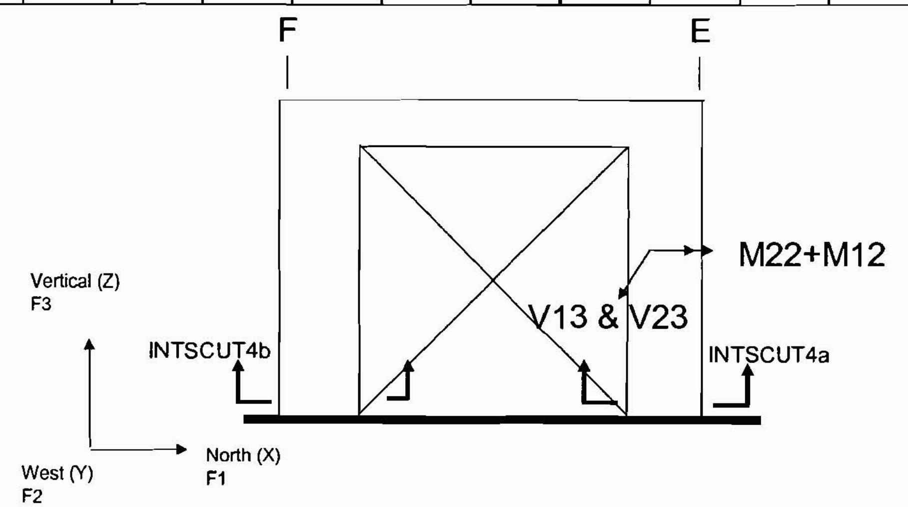
MAX MIN	M11	M22	M12	V13	V23	MAX	M11	M22	M12	V13	V23
	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft		Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	9	47	11	3	3	MAX	128	94	88	39	29
MIN	-10	-32	-12	-4	-6						

Maximum Load Combination

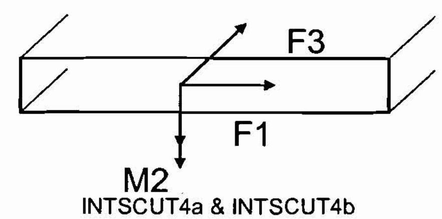
Section Cu	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
NTSCUT4a	440	-	1334	-	3802	-	-553	-	-812	-	-4366	-
NTSCUT4b	566	-	1380	-	4452	-	-441	-	-882	-	-3777	-

Maximum Load Combination

MAX	M11	M22	M12	V13	V23	MAX	M11	M22	M12	V13	V23
MIN	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft	MIN	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	136	141	99	42	32	MAX	-119	-48	-77	-36	-26
MIN	118	63	76	34	24	MIN	-137	-126	-100	-43	-35



TRANSVERSE WALL AT COL. 5 ---- ELEVATION



6.5.7.1 TRANSVERSE WALL REINFORCEMENT, COLUMN LINE 5

Transverse Wall COL 5-INTSCUT4a---D+L-SRSS

Design Loads

Axial Force (-Tension)	**Ft =**	-812 kips
Axial Force (+Comp)	**Fc =**	0 kips
In plane shear	**Vu =**	553 kips
In plane Moment	**Mz =**	4366 ft-kips
Out of plane shear	**Vz =**	43 kips/ft
Out of plane Moment	**My =**	226 ft-kips/ft (M22+M12)

Shear Wall Section Properties

Height of Wall (segment)	h_w =	35 ft
Ht of Wall Between Floors	H =	35 ft
Length of Wall (Segment)	l_w =	8.5 ft
Thickness of Wall	t_w =	4 ft
Shear Area of Wall (Segment)	$Acv = l_w*t_w$ =	34 ft^2

Concrete & Rebar Properties

Concrete Design Strength	f'c =	5000 psi
Concrete Strain	ε_c =	0.002
Rebar Yield Strength	fy =	60 ksi
Rebar Yield Strain	ε_y =	0.002
Min Steel Required	ρ_{min} =	0.0025
Concrete Cover		5 in

(Use 5" = 2" clear cover + diameter of the outer layer rebar + 1/2 diameter of the inner layer rebar)

Note: For ACI 349, see Ref. 2.2.14, for sections 1.0 to 6.0 of this design subset.

1.0 Check Shear on gross section - ACI 349: 21.6.5.6

Nominal Shear Capacity = 8*Acv*(f'c) $^{1/2}$ — Vn (kips) = 2770

Factored Shear Load = Vu / ϕ — Vu/ϕ (kips)= 921.7
(ϕ = .6 per ACI 349 - 9.3.4)

Demand Capacity Ratio
Check Code 21.6.5.6 Demand/Capacity Ratio — D/C = (Vu/ϕ)/Vn — D/C = 0.33 — **SHEAR WALL THICKNESS OK**

2.0 Horizontal Reinforcing Requirements

2.0a) ACI 349 - 21.6.5.3 Requirements

Determine α_c : h_w / l_w = 4.12 — α_c = 2 — α_c=3 for h_w/l_w<1.5, α_c varies linearly from 3 for h_w/l_w=1.5 to 2 for h_w/l_w=2.

Determine Concrete Shear Capacity Vc=Acv*α_c*(f'c) $^{1/2}$ — Vc = 692.4 kips

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc — Vs = 229.3 kips

Determine Required Shear Reinforcing ρ=Vs/(fy*Acv) — ρ = 0.0008

21.6.5.3 Shear Reinforcing Requirements — **ρ_n = 0.0025** — MINIMUM STEEL GOVERNS

2.0b) ACI 349 - 11.10.6 Equation 11-31 Requirements

Determine Concrete Shear Capacity — Vc = 751.6 kips — $Vc=3.3*(f'_c)^{0.5}*(0.8*t_w*l_w)+F_t*0.8*l_w/(4*l_w)$

Determine Shear Carried by Steel — Vs=Vu/ϕ -Vc — Vs = 170.1 kips

Determine Required Shear Reinforcing — ρ = 0.0007 — $\rho=Vs/(0.8*l_w*t_w*144*fy)$

11.10.6 - Equation 11-31 Shear Reinforcing Requirements — **ρ_n = 0.0025** — MINIMUM STEEL GOVERNS

2.0c) ACI 349 - 11.10.6 Equation 11-32 Requirements

Check Bounding Case Mu/Vu - l_w/2 = 3.65 — **Equation 11-32 APPLICABLE**

Determine Concrete Shear Capacity — Vc= 670.514927 — $Vc=[0.6*f'^{0.5}_c+l_w*(1.25*f'^{0.5}_c+0.2*Ft*1000/(l_w*t_w*144))/(Mu/Vu-l_w/2)]*t_w*0.8*l_w*144/1000$

Determine Shear Carried by Steel — Vs = 251.151739 — Vs=Vu/ϕ -Vc

Determine Required Shear Reinforcing Requirements — ρ = 0.0011 — $\rho=Vs/(0.8*l_w*t_w*144*fy)$

11.10.6 - Equation 11-32 Shear Reinforcing Requirements — **ρ_n = 0.0025** — **MINIMUM STEEL GOVERNS**

2.0d) Horizontal Shear Reinforcing Requirements (max of 2a, 2b, 2c) — **ρ_n = 0.0025**

2.0e) Select Horizontal Shear Reinforcing — **Asn required per ft on each face =** **0.72** **in^2/ft each face** — $(\rho_n*12*t_w*12/2)$

Use 1-#11@12"c/c EF — **Asn provided =** **1.56** **in^2/ft each face** — **ρ_n (prov)= 0.0054167**

2.0f) Check Demand / Capacity Ratio for In-Plane Shear:

D/C = 0.41 — $D/C=(Vu/\phi)/[Vc+(Asn*2*fy*l_w*t_w*144/(12*t_w*12))]$

3.0 Vertical Reinforcing Requirements

3.0a) ACI 349 21.6.5.5 and 11.10.9.4 - Minimum vertical reinforcing ratio : — hw/lw = 4.12

ρ_v (min) =0.0025

If h_w/l_w>2.0, use: $\rho_{v\,min}$ =0.0025+0.5(2.5-h_w/l_w)(ρ_n-0.0025)<=ρ_n
If h_w/l_w<=2.0, use: ρ_v>=ρ_n

3.0b) Check Shear Friction Requirements

In plane shear per foot of wall: 65.06 kips/ft (Vu/l_w)

Transverse shear per ft of wall: 43.00 kips/ft (Vz)

Resultant Shear 77.98 kips/ft $[(\text{in-plane shear})^2+(\text{transverse shear})^2]^{0.5}$

Calculate limiting shear friction strength at joint per ACI 349-11.7.5:

Vn<.2f'cAv for f'c = 5000 psi Vn=1000Av
Vn<800Av The limiting value of 800Av controls
Vn (MAX) = 460.8 kips/ft **Vn(MAX>Resultant shear ....OK**

Calculate shear friction reinforcing requirements per ACI 349 - 11.7.4.1:

Vn = Avf*fy*μ μ = 1.0 for concrete placed against hardened concrete intentionally roughened to a full amplitude of 1/4 inch (ACI 349-11.7.9)

Avf = Vu/2*ϕ*μ*fy (steel required per face)
Avf = 0.76 in^2/ft (steel required on each face for shear friction))

Calculate steel required for net Tension force

At = T/2*ϕ*fy*lw
At = 0.94 in^2/ft (steel required on each face for direct tension)

Steel Requirements for Shear Friction Avf + At

Av = 1.70 in^2/ft (steel required on each face for shear friction + direct tension)

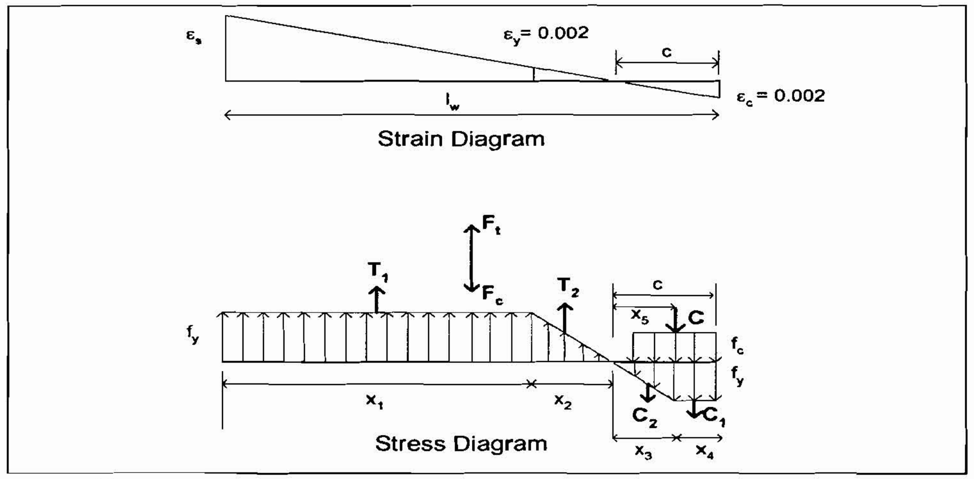
ρv(req'd) = 0.0059 $(\rho_{v\ req'd}=(2*Av)/(12*t_w*12)$

3.0c) Vertical Reinforcing Requirements (max of 3a,3b) **ρ_v = 0.0059**

3.0d) Select Vertical Reinforcing **Asv required per ft on each face = 1.70 in^2/ft each face** $(\rho_{v\ min}*12*t_w*12/2)$

Use 1-#11@6"c/c EF **Asv provided = 3.12 in^2/ft each face** **ρ_v (prov)= 0.0108333**

3.0e) Perform Strain-Compatible Section Analysis - For Axial Force (Tension) Ft = -812 kips



ϕ : Strength reduction factor (= 0.9 for tension, ACI 349 9.3.2.2), ϕ = 0.9
c : Distance from compression face to neutral axis (ft)
l_w : Length of wall (ft)
$\varepsilon_{s\_max} = [(l_w - c) / c] * \varepsilon_c$
$X_1 = l_w - c - X_2$ (ft) (length of wall with tension steel reinforcing steel strain > ε_y)
$X_2 = X_3 = (\varepsilon_y / \varepsilon_c) * c$ (ft) (length of wall with tension / compression steel reinforcing steel strain < ε_y)
$X_4 = c - X_3$ (ft) (length of wall with compression steel reinforcing steel strain > ε_y)
$T_1 = X_1 * As * fy$ (kips)
$T_2 = C_2 = 0.5 * X_2 * As * fy$ (kips)
$C_1 = X_4 * As * fy$ (kips)
$C = 0.85 * 0.8 * c * t_w * f'_c * 144$ (kips)
Balanced condition : Tension = Compression, $T_1+T_2-C_2-C_1-C+Ft/\phi = 0$.
M_u : Total moment capacity = $\phi\{T_1(X_2+X_1/2) + T_2(X_2*2/3) + C(c-0.8*c/2) + F_t(l_w/2-c)/\phi + C_2(X_3*2/3) + C_1(X_3+X_4/2)\}$, (kip-ft)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
0.84	7.66	0.0182	6.82	0.84	0.00	2552	158	0	1649	0.000	7903

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
2	6.5	0.0065	4.50	2.00	0.00	1684.8	374.4	0	3917	-3134	9746
1	7.5	0.0150	6.50	1.00	0.00	2433.6	187.2	0	1958	-427	7952
0.5	8	0.0320	7.50	0.50	0.00	2808	93.6	0	979	927	8016

Perform Strain-Compatible Section Analysis - For Axial Force (Comp) Fc = 0 kips

ϕ : Strength reduction factor (= 0.7 to 0.9 for comp, ACI 349 9.3.2.2), ϕ = 0.9000
If Fc<=0, ϕ=0.9; if Fc>0, ϕ=0.9-0.2*Fc/(0.1*Acv*144*f'$_c$/1000)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
1.18	7.32	0.0125	6.15	1.18	0.00	2302	220	0	2302	0.000	10578

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
1	7.5	0.0150	6.50	1.00	0.00	2433.6	187.2	0	1958	475	10591
3	5.5	0.0037	2.50	3.00	0.00	936	561.6	0	5875	-4939	15120
2	6.5	0.0065	4.50	2.00	0.00	1684.8	374.4	0	3917	-2232	11573

3.0f) Check Demand / Capacity Ratio for In-Plane Moment:

Vertical Reinforcing Ratio Provided by Design : ρ_v (prov)= 0.0108 1-#11@6"c/c EF ρ_v = (2*Asv)/(12*t_w*12)
D/C = (in-plane moment Mz) / (Min of Mu-ten, Mu-comp) D/C = 0.55 **Section Adequate**
Vertical Reinforcing Ratio Required for In-Plane Moment: $\rho_{vl\ req}$ = 0.0060 $\rho_{v\ req}$ = ρ_v*(D/C)

3.0g) Consider Out-of-Plane Moment:

Vertical Reinf Ratio Required for Out-of-Plane Bending: $\rho_{vt\ req}$ = 0.00230 (per face) $\rho f_y bd$^2*(1-.59$\rho f_y/f'_c$)=Mu/ϕ solve for ρ Ref. 2.2.19, Section 4-3

3.0h) Total Reinforcing required for axial force, in-plane and out of plane bending ρ_v = 0.0083 = $\rho_{vl}+\rho_{vt} > 2\rho_{vt}$

D/C = ρ_v(reqd) / ρ_v (prov) D/C = 0.76 **Section Adequate** = $\rho_v$req'd) / ρ_v(prov)

4.0 Boundary Elements:

h_w / l_w = 4.12 Concrete Strain limited to .002 No Need To Check for Boundary Elements

5.0 Out-of-Plane Shear:

Nominal Shear Strength Provided by The Concrete Vc=2*(f'c)$^{1/2}$*b*d Vc = 73.0 kips/ft width of wall

Check Demand / Capacity Ratio: D/C = 0.69 No Shear Reinforcement Required
D/C = (out-of-plane shear Vz) / (0.85*Vc)

6.0 Tabulate Reinforcement Requirements And D/C Ratios:

Use 4 ft thick wall with 1-#11@12"c/c EF Horizontal Reinforcement
1-#11@6"c/c EF Vertical Reinforcement.

For Shear on Gross Section:	**D/C =**	**0.33**
For In-Plane Shear:	**D/C =**	**0.41**
For Out-of-Plane Shear:	**D/C =**	**0.69**
Bending + axial Loads	**D/C =**	**0.76**

Boundary Elements D/C= BOUNDARY ELEMENTS NOT REQUIRED

Transverse Wall COL 5-INTSCUT4a---D+L+SRSS

Design Loads			**Shear Wall Section Properties**			**Concrete & Rebar Properties**		
Axial Force (-Tension)	**Ft =**	0 kips	**Height of Wall (segment)**	**h_w =**	35 ft	**Concrete Design Strength**	**f'c =**	5000 psi
Axial Force (+Comp)	**Fc =**	1334 kips	**Ht of Wall Between Floors**	**H =**	35 ft	**Concrete Strain**	**ε_c =**	0.002
In plane shear	**Vu =**	440	**Length of Wall (Segment)**	**l_w =**	8.5 ft	**Rebar Yield Strength**	**fy =**	60 ksi
In plane Moment	**Mz =**	3802 ft-kips	**Thickness of Wall**	**t_w =**	4 ft	**Rebar Yield Strain**	**ε_y =**	0.002
			Shear Area of Wall (Segment)	**Acv = l_w*t_w =**	34 ft^2	**Min Steel Required**	**ρ_{min} =**	0.0025
						Concrete Cover		5 in
Out of plane shear	**Vz =**	42 kips/ft						
Out of plane Moment	**My =**	240 ft-kips/ft (M22+M12)						

(Use 5" = 2" clear cover + diameter of the outer layer rebar + 1/2 diameter of the inner layer rebar)

Note: For ACI 349, see Ref. 2.2.14, for sections 1.0 to 6.0 of this design subset.

1.0 Check Shear on gross section - ACI 349: 21.6.5.6

Nominal Shear Capacity = 8*Acv*(f'c) $^{1/2}$ — Vn (kips) = 2770

Factored Shear Load = Vu / ϕ — Vu/ϕ (kips)= 733.3
(ϕ = .6 per ACI 349 - 9.3.4)

Demand Capacity Ratio
Check Code 21.6.5.6 Demand/Capacity Ratio — D/C = (Vu/ϕ)/Vn — D/C = 0.26 **SHEAR WALL THICKNESS OK**

2.0 Horizontal Reinforcing Requirements

2.0a) ACI 349 - 21.6.5.3 Requirements

Determine α_c : h_w / l_w = 4.12 — α_c = 2 — α_c=3 for h_w/l_w<1.5, α_c varies linearly from 3 for h_w/l_w=1.5 to 2 for h_w/l_w=2.

Determine Concrete Shear Capacity Vc=Acv*α_c*(f'c) $^{1/2}$ — Vc = 692.4 kips

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc — Vs = 40.9 kips

Determine Required Shear Reinforcing ρ=Vs/(fy*Acv) — ρ = 0.0001

21.6.5.3 Shear Reinforcing Requirements — **ρ_n = 0.0025** MINIMUM STEEL GOVERNS

2.0b) ACI 349 - 11.10.6 Equation 11-31 Requirements

Determine Concrete Shear Capacity — Vc = 914.0 kips — Vc=3.3*(f'c)$^{0.5}$*(0.8*t_w*l_w)+F_t*0.8*l_w/(4*l_w)

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc — Vs = 0.0 kips

Determine Required Shear Reinforcing — ρ = 0.0000 — ρ=Vs/(0.8*l_w*t_w*144*fy)

11.10.6 - Equation 11-31 Shear Reinforcing Requirements — **ρ_n = 0.0025** MINIMUM STEEL GOVERNS

2.0c) ACI 349 - 11.10.6 Equation 11-32 Requirements

Check Bounding Case Mu/Vu - l_w/2 = 4.39 **Equation 11-32 APPLICABLE**

Determine Concrete Shear Capacity — Vc= 836.354868 — Vc=[0.6*f'$_c^{0.5}$+l_w*(1.25*f'$_c^{0.5}$+0.2*Ft*1000/(l_w*t_w*144))/(Mu/Vu-l_w/2)]*t_w*0.8*l_w*144/1000

Determine Shear Carried by Steel — Vs = 0 — Vs=Vu/ϕ -Vc

Determine Required Shear Reinforcing Requirements — ρ = 0.0000 — ρ=Vs/(0.8*l_w*t_w*144*fy)

11.10.6 - Equation 11-32 Shear Reinforcing Requirements — **ρ_n = 0.0025 MINIMUM STEEL GOVERNS**

2.0d) Horizontal Shear Reinforcing Requirements (max of 2a, 2b, 2c) — **ρ_n = 0.0025**

2.0e) Select Horizontal Shear Reinforcing — **Asn required per ft on each face = 0.72 in^2/ft each face** (ρ_n*12*t_w*12/2)

Use 1-#11@12"c/c EF Asn provided = 1.56 in^2/ft each face — **ρ_n (prov)=0.00542**

2.0f) Check Demand / Capacity Ratio for In-Plane Shear:

D/C = 0.32 — D/C=(Vu/ϕ)/[Vc+(Asn*2*fy*l_w*t_w*144/(12*t_w*12))]

3.0 Vertical Reinforcing Requirements

3.0a) ACI 349 21.6.5.5 and 11.10.9.4 - Minimum vertical reinforcing ratio : — hw/lw = 4.12

ρ_v (min) =0.0025 — If h_w/l_w>2.0, use: $\rho_{v\,min}$ =0.0025+0.5(2.5-h_w/l_w)(ρ_n-0.0025)<=ρ_n
If h_w/l_w<=2.0, use: ρ_v>=ρ_n

3.0b) Check Shear Friction Requirements

In plane shear per foot of wall: 51.76 kips/ft (Vu/l_w)

Transverse shear per ft of wall: 42.00 kips/ft (Vz)

Resultant Shear 66.66 kips/ft $[(\text{in-plane shear})^2+(\text{transverse shear})^2]^{0.5}$

Calculate limiting shear friction strength at joint per ACI 349-11.7.5:

Vn<.2f'cAv for f'c = 5000 psi Vn=1000Av
Vn<800Av The limiting value of 800Av controls
Vn (MAX) = 460.8 kips/ft **Vn(MAX>Resultant shear ....OK**

Calculate shear friction reinforcing requirements per ACI 349 - 11.7.4.1:

Vn = Avf*fy*μ   μ = 1.0 for concrete placed against hardened concrete intentionally roughened to a full amplitude of 1/4 inch (ACI 349-11.7.9)

Avf = Vu/2*ϕ*μ*fy (steel required per face)
Avf = 0.65 in^2/ft (steel required on each face for shear friction))

Calculate steel required for net Tension force

At = T/2*ϕ*fy*lw
At = 0.00 in^2/ft (steel required on each face for direct tension)

Steel Requirements for Shear Friction Avf + At

Av = 0.65 in^2/ft (steel required on each face for shear friction + direct tension)

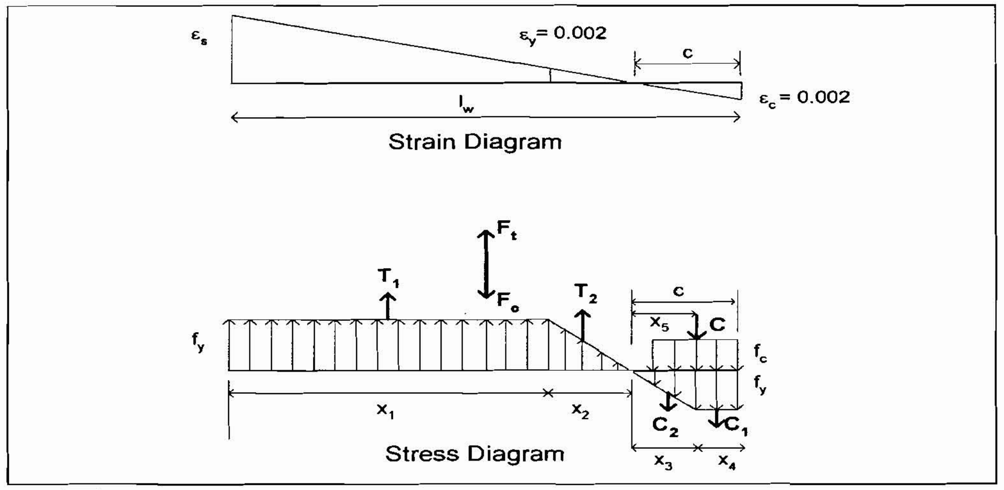
ρv(req'd) = 0.0023 $(\rho_{v\ req'd}=(2^*Av)/(12^*t_w{}^*12))$

3.0c) Vertical Reinforcing Requirements (max of 3a,3b) ρ_v **= 0.0025**

3.0d) Select Vertical Reinforcing **Asv required per ft on each face = 0.72 in²/ft each face** $(\rho_{v\ min}{}^*12^*t_w{}^*12/2)$

Use 1-#11@6"c/cEF **Asv provided = 3.12 in²/ft each face** ρ_v **(prov)= 0.01083**

3.0e) Perform Strain-Compatible Section Analysis - For Axial Force (Tension) **Ft =** 0 kips



ϕ : Strength reduction factor (= 0.9 for tension, ACI 349 9.3.2.2), ϕ = 0.9
c : Distance from compression face to neutral axis (ft)
l_w : Length of wall (ft)
$\varepsilon_{s\_max} = [(l_w - c) / c] * \varepsilon_c$
$X_1 = l_w - c - X_2$ (ft) (length of wall with tension steel reinforcing steel strain > ε_y)
$X_2 = X_3 = (\varepsilon_y / \varepsilon_c) * c$ (ft) (length of wall with tension / compression steel reinforcing steel strain < ε_y)
$X_4 = c - X_3$ (ft) (length of wall with compression steel reinforcing steel strain > ε_y)
$T_1 = X_1 * As * fy$ (kips)
$T_2 = C_2 = 0.5 * X_2 * As * fy$ (kips)
$C_1 = X_4 * As * fy$ (kips)
$C = 0.85 * 0.8 * c * t_w * f'_c * 144$ (kips)
Balanced condition : Tension = Compression, $T_1+T_2-C_2-C_1-C+Ft/\phi = 0$.
M_u : Total moment capacity = $\phi\{T_1(X_2+X_1/2) + T_2(X_2{}^*2/3) + C(c-0.8^*c/2) + F_t(l_w/2-c)/\phi + C_2(X_3{}^*2/3) + C_1(X_3+X_4/2)\}$, (kip-ft)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
1.18	7.32	0.0125	6.15	1.18	0.00	2302	220	0	2302	0.000	10578

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
1	7.5	0.0150	6.50	1.00	0.00	2433.6	187.2	0	1958	475	10591
3	5.5	0.0037	2.50	3.00	0.00	936	561.6	0	5875	-4939	15120

2	6.5	0.0065	4.50	2.00	0.00	1684.8	374.4	0	3917	-2232	11573

Perform Strain-Compatible Section Analysis - For Axial Force (Comp) **Fc =** 1334 kips

ϕ : Strength reduction factor (= 0.7 to 0.9 for comp, ACI 349 9.3.2.2), ϕ = 0.7910
If Fc<=0, ϕ=0.9; if Fc>0, ϕ=0.9-0.2*Fc/(0.1*Acv*144*f'$_c$/1000)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
1.80	6.70	0.0075	4.90	1.80	0.00	1836	337	0	3522	0.000	13087

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
1	7.5	0.0150	6.50	1.00	0.00	2433.6	187.2	0	1958	2162	13644
3	5.5	0.0037	2.50	3.00	0.00	936	561.6	0	5875	-3253	14956
2	6.5	0.0065	4.50	2.00	0.00	1684.8	374.4	0	3917	-546	13173

3.0f) Check Demand / Capacity Ratio for In-Plane Moment:

Vertical Reinforcing Ratio Provided by Design : ρ_v (prov)= 0.0108 1-#11@6"c/cEF ρ_v = (2*Asv)/(12*t_w*12)
D/C = (in-plane moment Mz) / (Min of Mu-ten, Mu-comp) D/C = 0.36 **Section Adequate**
Vertical Reinforcing Ratio Required for In-Plane Moment: $\rho_{vl\,req}$ = 0.0039 $\rho_{v\,req}$ = ρ_v*(D/C)

3.0g) Consider Out-of-Plane Moment:

Vertical Reinf Ratio Required for Out-of-Plane Bending: $\rho_{vt\,req}$ = 0.00245 (per face) $\rho f_y bd$^2*(1-.59$\rho f_y/f'_c$)=Mu/ϕ solve for ρ Ref. 2.2.19, Section 4-3

3.0h) Total Reinforcing required for axial force, in-plane and out of plane bending ρ_v = 0.0063 = $\rho_{vl}+\rho_{vt} > 2\rho_{vt}$

D/C = ρ_v(reqd) / ρ_v (prov) D/C = 0.59 **Section Adequate** = ρ_v(req'd) / ρ_v(prov)

4.0 Boundary Elements:

h_w / l_w = 4.12 Concrete Strain limited to .002 No Need To Check for Boundary Elements

5.0 Out-of-Plane Shear:

Nominal Shear Strength Provided by The Concrete Vc=2*(f'c)$^{1/2}$*b*d Vc = 73.0 kips/ft width of wall

Check Demand / Capacity Ratio: D/C = 0.68 No Shear Reinforcement Required
D/C = (out-of-plane shear Vz) / (0.85*Vc)

6.0 Tabulate Reinforcement Requirements And D/C Ratios:

Use 4 ft thick wall with 1-#11@12"c/c EF Horizontal Reinforcement
1-#11@6"c/cEF Vertical Reinforcement.

For Shear on Gross Section:	**D/C =**	**0.26**
For In-Plane Shear:	**D/C =**	**0.32**
For Out-of-Plane Shear:	**D/C =**	**0.68**
Bending + axial Loads	**D/C =**	**0.59**

Boundary Elements D/C= BOUNDARY ELEMENTS NOT REQUIRED

Transverse Wall COL 5-INTSCUT4b---D+L-SRSS

Design Loads

Axial Force (-Tension)	**Ft =**	882 kips
Axial Force (+Comp)	**Fc =**	0 kips
In plane shear	**Vu =**	441 kips
In plane Moment	**Mz =**	3777 ft-kips
Out of plane shear	**Vz =**	43 kips/ft
Out of plane Moment	**My =**	226 ft-kips/ft (M22+M12)

Shear Wall Section Properties

Height of Wall (segment)	h_w =	35 ft
Ht of Wall Between Floors	H =	35 ft
Length of Wall (Segment)	l_w =	8.5 ft
Thickness of Wall	t_w =	4 ft
Shear Area of Wall (Segment)	$Acv = l_w*t_w$ =	34 ft^2

Concrete & Rebar Properties

Concrete Design Strength	**f'c =**	5000 psi
Concrete Strain	ε_c =	0.002
Rebar Yield Strength	**fy =**	60 ksi
Rebar Yield Strain	ε_y =	0.002
Min Steel Required	ρ_{min} =	0.0025
Concrete Cover		5 in

(Use 5" = 2" clear cover + diameter of the outer layer rebar + 1/2 diameter of the inner layer rebar)

Note: For ACI 349, see Ref. 2.2.14, for sections 1.0 to 6.0 of this design subset.

1.0 Check Shear on gross section - ACI 349: 21.6.5.6

Nominal Shear Capacity = 8*Acv*(f'c) $^{1/2}$ Vn (kips) = 2770

Factored Shear Load = Vu / ϕ Vu/ϕ (kips)= 735.0
(ϕ = .6 per ACI 349 - 9.3.4)

Demand Capacity Ratio
Check Code 21.6.5.6 Demand/Capacity Ratio D/C = (Vu/ϕ)/Vn D/C = 0.27 **SHEAR WALL THICKNESS OK**

2.0 Horizontal Reinforcing Requirements

2.0a) ACI 349 - 21.6.5.3 Requirements

Determine α_c : h_w / l_w = 4.12 α_c = 2 α_c=3 for h_w/l_w<1.5, α_c varies linearly from 3 for h_w/l_w=1.5 to 2 for h_w/l_w=2.

Determine Concrete Shear Capacity Vc=Acv*α_c*(f'c) $^{1/2}$ Vc = 692.4 kips

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc Vs = 42.6 kips

Determine Required Shear Reinforcing ρ=Vs/(fy*Acv) ρ = 0.0001

21.6.5.3 Shear Reinforcing Requirements ρ_n **= 0.0025** MINIMUM STEEL GOVERNS

2.0b) ACI 349 - 11.10.6 Equation 11-31 Requirements

Determine Concrete Shear Capacity Vc = 1090.4 kips Vc=3.3*(f'$_c$)$^{0.5}$*(0.8*t_w*l_w)+F_t*0.8*l_w/(4*l_w)

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc Vs = 0.0 kips

Determine Required Shear Reinforcing ρ = 0.0000 ρ=Vs/(0.8*l_w*t_w*144*fy)

11.10.6 - Equation 11-31 Shear Reinforcing Requirements ρ_n **= 0.0025** MINIMUM STEEL GOVERNS

2.0c) ACI 349 - 11.10.6 Equation 11-32 Requirements

Check Bounding Case Mu/Vu - l_w/2 = 4.31 **Equation 11-32 APPLICABLE**

Determine Concrete Shear Capacity Vc= 1126.21626 Vc=[0.6*f'$_c$$^{0.5}$+$l_w$*(1.25*f'$_c$$^{0.5}$+0.2*Ft*1000/($l_w$*$t_w$*144))/(Mu/Vu-$l_w$/2)]*$t_w$*0.8*$l_w$*144/1000

Determine Shear Carried by Steel Vs = 0 Vs=Vu/ϕ -Vc

Determine Required Shear Reinforcing Requirements ρ = 0.0000 ρ=Vs/(0.8*l_w*t_w*144*fy)

11.10.6 - Equation 11-32 Shear Reinforcing Requirements ρ_n **= 0.0025 MINIMUM STEEL GOVERNS**

2.0d) Horizontal Shear Reinforcing Requirements (max of 2a, 2b, 2c) ρ_n **= 0.0025**

2.0e) Select Horizontal Shear Reinforcing Asn required per ft on each face = 0.72 in^2/ft each face (ρ_n*12*t_w*12/2)

Use 1-#11@12"c/c EF Asn provided = 1.56 in^2/ft each face ρ_n **(prov)=0.00542**

2.0f) Check Demand / Capacity Ratio for In-Plane Shear:

D/C = 0.32 D/C=(Vu/ϕ)/[Vc+(Asn*2*fy*l_w*t_w*144/(12*t_w*12))]

3.0 Vertical Reinforcing Requirements

3.0a) ACI 349 21.6.5.5 and 11.10.9.4 - Minimum vertical reinforcing ratio : hw/lw = 4.12

If h_w/l_w>2.0, use: $\rho_{v\,min}$ =0.0025+0.5(2.5-h_w/l_w)(ρ_n-0.0025)<=ρ_n
If h_w/l_w<=2.0, use: ρ_v>=ρ_n

ρ_v **(min) =0.0025**

3.0b) Check Shear Friction Requirements

In plane shear per foot of wall: 51.88 kips/ft (Vu/l_w)

Transverse shear per ft of wall: 43.00 kips/ft (Vz)

Resultant Shear 67.39 kips/ft $[(\text{in-plane shear})^2+(\text{transverse shear})^2]^{0.5}$

Calculate limiting shear friction strength at joint per ACI 349-11.7.5:

Vn<.2f'cAv for f'c = 5000 psi Vn=1000Av
Vn<800Av The limiting value of 800Av controls
Vn (MAX) = 460.8 kips/ft **Vn(MAX>Resultant shear ....OK**

Calculate shear friction reinforcing requirements per ACI 349 - 11.7.4.1:

Vn = Avf*fy*μ μ = 1.0 for concrete placed against hardened concrete intentionally roughened to a full amplitude of 1/4 inch (ACI 349-11.7.9)

Avf = Vu/2*ϕ*μ*fy (steel required per face)
Avf = 0.66 in^2/ft (steel required on each face for shear friction))

Calculate steel required for net Tension force

At = T/2*ϕ*fy*lw
At = -1.02 in^2/ft (steel required on each face for direct tension)

Steel Requirements for Shear Friction Avf + At

Av = -0.36 in^2/ft (steel required on each face for shear friction + direct tension)

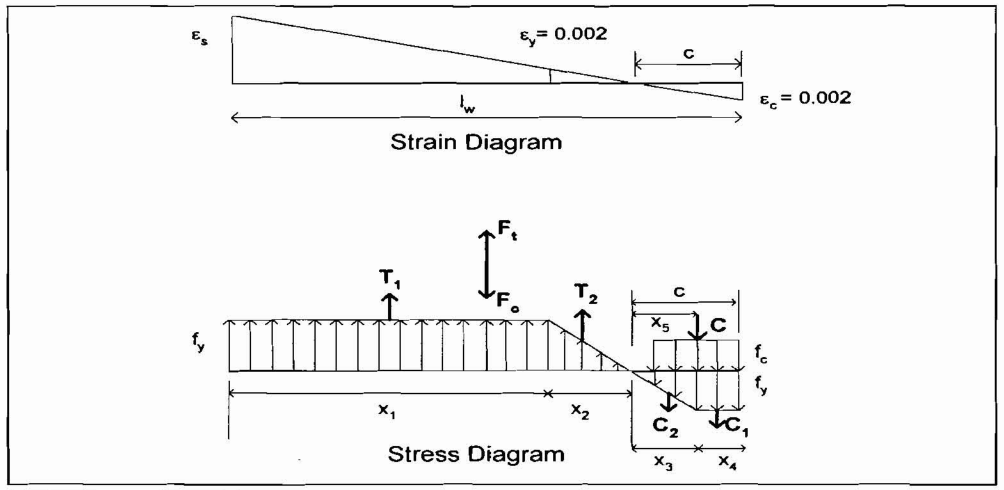
ρv(req'd) = -0.0012 ($\rho_{v\ req'd}$=(2*Av)/(12*t_w*12)

3.0c) Vertical Reinforcing Requirements (max of 3a,3b) **ρ_v = 0.0025**

3.0d) Select Vertical Reinforcing **Asv required per ft on each face = 0.72 in^2/ft each face** ($\rho_{v\ min}$*12*t_w*12/2)

Use 1-#11@6"c/c EF **Asv provided = 3.12 in^2/ft each face** **ρ_v (prov)= 0.01083**

3.0e) Perform Strain-Compatible Section Analysis - For Axial Force (Tension) **Ft =** 882 kips



ϕ : Strength reduction factor (= 0.9 for tension, ACI 349 9.3.2.2), ϕ = 0.9
c : Distance from compression face to neutral axis (ft)
l_w : Length of wall (ft)
$\varepsilon_{s\_max} = [(l_w - c) / c] * \varepsilon_c$
$X_1 = l_w - c - X_2$ (ft) (length of wall with tension steel reinforcing steel strain > ε_y)
$X_2 = X_3 = (\varepsilon_y / \varepsilon_c) * c$ (ft) (length of wall with tension / compression steel reinforcing steel strain < ε_y)
$X_4 = c - X_3$ (ft) (length of wall with compression steel reinforcing steel strain > ε_y)
$T_1 = X_1 * As * fy$ (kips)
$T_2 = C_2 = 0.5 * X_2 * As * fy$ (kips)
$C_1 = X_4 * As * fy$ (kips)
$C = 0.85 * 0.8 * c * t_w * f'_c * 144$ (kips)
Balanced condition : Tension = Compression, $T_1+T_2-C_2-C_1-C+Ft/\phi = 0$.
M_u : Total moment capacity = $\phi\{T_1(X_2+X_1/2) + T_2(X_2*2/3) + C(c-0.8*c/2) + F_t(l_w/2-c)/\phi + C_2(X_3*2/3) + C_1(X_3+X_4/2)\}$, (kip-ft)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
1.54	6.96	0.0091	5.42	1.54	0.00	2031	288	0	3011	0.000	13192

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
2	6.5	0.0065	4.50	2.00	0.00	1684.8	374.4	0	3917	-1252	13558
1	7.5	0.0150	6.50	1.00	0.00	2433.6	187.2	0	1958	1455	13457

1.5	7	0.0093	5.50	1.50	0.00	2059.2	280.8	0	2938	102	13187

Perform Strain-Compatible Section Analysis - For Axial Force (Comp) **Fc =** 0 kips

ϕ : Strength reduction factor (= 0.7 to 0.9 for comp, ACI 349 9.3.2.2), ϕ = 0.9000
If Fc<=0, ϕ=0.9; if Fc>0, ϕ=0.9-0.2*Fc/(0.1*Acv*144*f'$_c$/1000)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
1.18	7.32	0.0125	6.15	1.18	0.00	2302	220	0	2302	0.000	10578

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
1	7.5	0.0150	6.50	1.00	0.00	2433.6	187.2	0	1958	475	10591
3	5.5	0.0037	2.50	3.00	0.00	936	561.6	0	5875	-4939	15120
2	6.5	0.0065	4.50	2.00	0.00	1684.8	374.4	0	3917	-2232	11573

3.0f) Check Demand / Capacity Ratio for In-Plane Moment:

Vertical Reinforcing Ratio Provided by Design : ρ_v (prov)= 0.0108 1-#11@6"c/c EF ρ_v = (2*Asv)/(12*t_w*12)
D/C = (in-plane moment Mz) / (Min of Mu-ten, Mu-comp) D/C = 0.36 **Section Adequate**
Vertical Reinforcing Ratio Required for In-Plane Moment: $\rho_{vl\ req}$ = 0.0039 $\rho_{v\ req} = \rho_v$*(D/C)

3.0g) Consider Out-of-Plane Moment:

Vertical Reinf Ratio Required for Out-of-Plane Bending: $\rho_{vt\ req}$ = 0.00230 (per face) $\rho f_y bd$^2*(1-.59$\rho f_y/f'_c$)=Mu/ϕ solve for ρ Ref. 2.2.19, Section 4-3

3.0h) Total Reinforcing required for axial force, in-plane and out of plane bending ρ_v = 0.0062 = $\rho_{vl}+\rho_{vt} > 2\rho_{vt}$

D/C = ρ_v(reqd) / ρ_v (prov) D/C = 0.57 **Section Adequate** = ρ_v(req'd) / ρ_v(prov)

4.0 Boundary Elements:

h_w / l_w = 4.12 Concrete Strain limited to .002 No Need To Check for Boundary Elements

5.0 Out-of-Plane Shear:

Nominal Shear Strength Provided by The Concrete Vc=2*(f'c)$^{1/2}$*b*d Vc = 73.0 kips/ft width of wall

Check Demand / Capacity Ratio: D/C = 0.69 No Shear Reinforcement Required
D/C = (out-of-plane shear Vz) / (0.85*Vc)

6.0 Tabulate Reinforcement Requirements And D/C Ratios:

Use **4** **ft thick wall with** **1-#11@12"c/c EF** **Horizontal Reinforcement**
1-#11@6"c/c EF **Vertical Reinforcement.**

For Shear on Gross Section: **D/C =** **0.27**
For In-Plane Shear: **D/C =** **0.32**
For Out-of-Plane Shear: **D/C =** **0.69**
Bending + axial Loads **D/C =** **0.57**

Boundary Elements **D/C=** **BOUNDARY ELEMENTS NOT REQUIRED**

Transverse Wall COL 5-INTSCUT4b---D+L+SRSS

Design Loads			**Shear Wall Section Properties**			**Concrete & Rebar Properties**		
Axial Force (-Tension)	**Ft =**	0 kips	**Height of Wall (segment)**	**h_w =**	35 ft	**Concrete Design Strength**	**f'c =**	5000 psi
Axial Force (+Comp)	**Fc =**	1380 kips	**Ht of Wall Between Floors**	**H =**	35 ft	**Concrete Strain**	**ε_c =**	0.002
In plane shear	**Vu =**	566	**Length of Wall (Segment)**	**l_w =**	8.5 ft	**Rebar Yield Strength**	**fy =**	60 ksi
In plane Moment	**Mz =**	4452 ft-kips	**Thickness of Wall**	**t_w =**	4 ft	**Rebar Yield Strain**	**ε_y =**	0.002
			Shear Area of Wall (Segment)	**Acv = l_w*t_w =**	34 ft^2	**Min Steel Required**	**ρ_{min} =**	0.0025
						Concrete Cover		5 in

(Use 5" = 2" clear cover + diameter of the outer layer rebar + 1/2 diameter of the inner layer rebar)

Out of plane shear **Vz =** 42 kips/ft
Out of plane Moment **My =** 240 ft-kips/ft (M22+M12)

Note: For ACI 349, see Ref. 2.2.14, for sections 1.0 to 6.0 of this design subset.

1.0 Check Shear on gross section - ACI 349: 21.6.5.6

Nominal Shear Capacity = 8*Acv*(f'c) $^{1/2}$ — Vn (kips) = 2770

Factored Shear Load = Vu / ϕ — Vu/ϕ (kips)= 943.3
(ϕ = .6 per ACI 349 - 9.3.4)

Demand Capacity Ratio
Check Code 21.6.5.6 Demand/Capacity Ratio — D/C = (Vu/ϕ)/Vn — D/C = 0.34 **SHEAR WALL THICKNESS OK**

2.0 Horizontal Reinforcing Requirements

2.0a) ACI 349 - 21.6.5.3 Requirements

Determine α_c : h_w / l_w = 4.12 — α_c = 2 — α_c=3 for h_w/l_w<1.5, α_c varies linearly from 3 for h_w/l_w=1.5 to 2 for h_w/l_w=2.

Determine Concrete Shear Capacity Vc=Acv*α_c*(f'c) $^{1/2}$ — Vc = 692.4 kips

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc — Vs = 250.9 kips

Determine Required Shear Reinforcing ρ=Vs/(fy*Acv) — ρ = 0.0009

21.6.5.3 Shear Reinforcing Requirements — **ρ_n = 0.0025** MINIMUM STEEL GOVERNS

2.0b) ACI 349 - 11.10.6 Equation 11-31 Requirements

Determine Concrete Shear Capacity — Vc = 914.0 kips — Vc=3.3*$(f'_c)^{0.5}$*(0.8*t_w*l_w)+F_t*0.8*l_w/(4*l_w)

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc — Vs = 29.4 kips

Determine Required Shear Reinforcing — ρ = 0.0001 — ρ=Vs/(0.8*l_w*t_w*144*fy)

11.10.6 - Equation 11-31 Shear Reinforcing Requirements — **ρ_n = 0.0025** MINIMUM STEEL GOVERNS

2.0c) ACI 349 - 11.10.6 Equation 11-32 Requirements

Check Bounding Case Mu/Vu - l_w/2 = 3.62 **Equation 11-32 APPLICABLE**

Determine Concrete Shear Capacity — Vc= 980.036341 — Vc=[0.6*$f_c^{0.5}$+l_w*(1.25*$f_c^{0.5}$+0.2*Ft*1000/(l_w*t_w*144))/(Mu/Vu-l_w/2)]*t_w*0.8*l_w*144/1000

Determine Shear Carried by Steel — Vs = 0 — Vs=Vu/ϕ -Vc

Determine Required Shear Reinforcing Requirements — ρ = 0.0000 — ρ=Vs/(0.8*l_w*t_w*144*fy)

11.10.6 - Equation 11-32 Shear Reinforcing Requirements — **ρ_n = 0.0025 MINIMUM STEEL GOVERNS**

2.0d) Horizontal Shear Reinforcing Requirements (max of 2a, 2b, 2c) — **ρ_n = 0.0025**

2.0e) Select Horizontal Shear Reinforcing — **Asn required per ft on each face = 0.72 in^2/ft each face** (ρ_n*12*t_w*12/2)

Use 1-#11@12"c/c EF Asn provided = 1.56 in^2/ft each face ρ_n (prov)=0.00542

2.0f) Check Demand / Capacity Ratio for In-Plane Shear:

D/C = 0.41 — D/C=(Vu/ϕ)/[Vc+(Asn*2*fy*l_w*t_w*144/(12*t_w*12))]

3.0 Vertical Reinforcing Requirements

3.0a) ACI 349 21.6.5.5 and 11.10.9.4 - Minimum vertical reinforcing ratio : — hw/lw = 4.12

ρ_v (min) =0.0025 — If h_w/l_w>2.0, use: $\rho_{v\,min}$ =0.0025+0.5(2.5-h_w/l_w)(ρ_n-0.0025)<=ρ_n
If h_w/l_w<=2.0, use: ρ_v>=ρ_n

3.0b) Check Shear Friction Requirements

In plane shear per foot of wall:	66.59 kips/ft	(Vu/l_w)
Transverse shear per ft of wall:	42.00 kips/ft	(Vz)
Resultant Shear	78.73 kips/ft	$[(\text{in-plane shear})^2+(\text{transverse shear})^2]^{0.5}$

Calculate limiting shear friction strength at joint per ACI 349-11.7.5:

Vn<.2f'cAv for f'c = 5000 psi Vn=1000Av
Vn<800Av — The limiting value of 800Av controls
Vn (MAX) = 460.8 kips/ft **Vn(MAX>Resultant shear ....OK**

Calculate shear friction reinforcing requirements per ACI 349 - 11.7.4.1:

Vn = Avf*fy*μ — μ = 1.0 for concrete placed against hardened concrete intentionally roughened to a full amplitude of 1/4 inch (ACI 349-11.7.9)

Avf = Vu/2*ϕ*μ*fy (steel required per face)
Avf = 0.77 in^2/ft (steel required on each face for shear friction))

Calculate steel required for net Tension force

At = T/2*ϕ*fy*lw
At = 0.00 in^2/ft (steel required on each face for direct tension)

Steel Requirements for Shear Friction Avf + At

Av = 0.77 in^2/ft (steel required on each face for shear friction + direct tension)

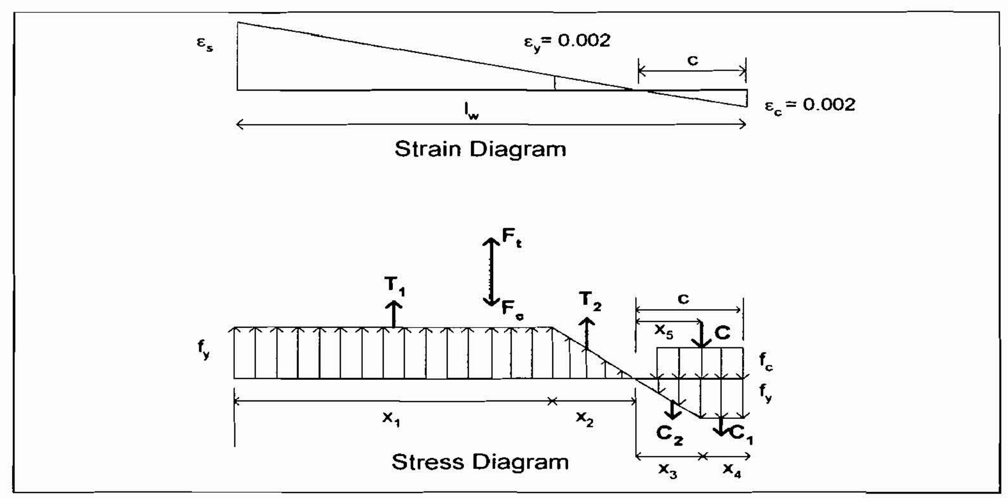
ρv(req'd) = 0.0027 ($\rho_{v\ req'd}$=(2*Av)/(12*t_w*12))

3.0c) Vertical Reinforcing Requirements (max of 3a,3b) **ρ_v = 0.0027**

3.0d) Select Vertical Reinforcing **Asv required per ft on each face = 0.77 in²/ft each face** ($\rho_{v\ min}$*12*t_w*12/2)

Use 1-#11@6"c/cEF **Asv provided = 3.12 in²/ft each face** **ρ_v (prov)= 0.01083**

3.0e) Perform Strain-Compatible Section Analysis - For Axial Force (Tension) **Ft =** 0 kips



ϕ : Strength reduction factor (= 0.9 for tension, ACI 349 9.3.2.2), ϕ = 0.9
c : Distance from compression face to neutral axis (ft)
l_w : Length of wall (ft)
$\varepsilon_{s\_max} = [(l_w - c) / c] * \varepsilon_c$
$X_1 = l_w - c - X_2$ (ft) (length of wall with tension steel reinforcing steel strain > ε_y)
$X_2 = X_3 = (\varepsilon_y / \varepsilon_c) * c$ (ft) (length of wall with tension / compression steel reinforcing steel strain < ε_y)
$X_4 = c - X_3$ (ft) (length of wall with compression steel reinforcing steel strain > ε_y)
$T_1 = X_1 * As * fy$ (kips)
$T_2 = C_2 = 0.5 * X_2 * As * fy$ (kips)
$C_1 = X_4 * As * fy$ (kips)
$C = 0.85 * 0.8 * c * t_w * f'_c * 144$ (kips)
Balanced condition : Tension = Compression, $T_1+T_2-C_2-C_1-C+Ft/\phi = 0$.
M_u : Total moment capacity = $\phi\{T_1(X_2+X_1/2) + T_2(X_2*2/3) + C(c-0.8*c/2) + F_t(l_w/2-c)/\phi + C_2(X_3*2/3) + C_1(X_3+X_4/2)\}$, (kip-ft)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
1.18	7.32	0.0125	6.15	1.18	0.00	2302	220	0	2302	0.000	10578

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
1	7.5	0.0150	6.50	1.00	0.00	2433.6	187.2	0	1958	475	10591
3	5.5	0.0037	2.50	3.00	0.00	936	561.6	0	5875	-4939	15120

2	6.5	0.0065	4.50	2.00	0.00	1684.8	374.4	0	3917	-2232	11573

Perform Strain-Compatible Section Analysis - For Axial Force (Comp) **Fc =** 1380 kips

ϕ : Strength reduction factor (= 0.7 to 0.9 for comp, ACI 349 9.3.2.2), ϕ = 0.7873
If Fc<=0, ϕ=0.9; if Fc>0, ϕ=0.9-0.2*Fc/(0.1*Acv*144*f'_c/1000)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
1.82	6.68	0.0073	4.85	1.82	0.00	1817	341	0	3570	0.000	13157

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
1	7.5	0.0150	6.50	1.00	0.00	2433.6	187.2	0	1958	2228	13749
3	5.5	0.0037	2.50	3.00	0.00	936	561.6	0	5875	-3186	14951
2	6.5	0.0065	4.50	2.00	0.00	1684.8	374.4	0	3917	-479	13228

3.0f) Check Demand / Capacity Ratio for In-Plane Moment:

Vertical Reinforcing Ratio Provided by Design : ρ_v (prov)= 0.0108 1-#11@6"c/cEF ρ_v = (2*Asv)/(12*t_w*12)
D/C = (in-plane moment Mz) / (Min of Mu-ten, Mu-comp) D/C = 0.42 **Section Adequate**
Vertical Reinforcing Ratio Required for In-Plane Moment: $\rho_{vl\ req}$ = 0.0046 $\rho_{v\ req}$ = ρ_v*(D/C)

3.0g) Consider Out-of-Plane Moment:

Vertical Reinf Ratio Required for Out-of-Plane Bending: $\rho_{vt\ req}$ = 0.00245 (per face) $\rho f_y bd$^2*(1-.59$\rho f_y/f'_c$)=Mu/ϕ solve for ρ Ref. 2.2.19, Section 4-3

3.0h) Total Reinforcing required for axial force, in-plane and out of plane bending ρ_v = 0.0070 = $\rho_{vl}+\rho_{vt}$ > $2\rho_{vt}$

D/C = ρ_v(reqd) / ρ_v (prov) D/C = 0.65 **Section Adequate** = ρ_v(req'd) / ρ_v(prov)

4.0 Boundary Elements:

h_w / l_w = 4.12 Concrete Strain limited to .002 No Need To Check for Boundary Elements

5.0 Out-of-Plane Shear:

Nominal Shear Strength Provided by The Concrete Vc=2*(f'c)$^{1/2}$*b*d Vc = 73.0 kips/ft width of wall

Check Demand / Capacity Ratio: D/C = 0.68 No Shear Reinforcement Required
D/C = (out-of-plane shear Vz) / (0.85*Vc)

6.0 Tabulate Reinforcement Requirements And D/C Ratios:

Use 4 ft thick wall with 1-#11@12"c/c EF Horizontal Reinforcement
1-#11@6"c/cEF Vertical Reinforcement.

For Shear on Gross Section:	**D/C =**	**0.34**
For In-Plane Shear:	**D/C =**	**0.41**
For Out-of-Plane Shear:	**D/C =**	**0.68**
Bending + axial Loads	**D/C =**	**0.65**

Boundary Elements D/C= BOUNDARY ELEMENTS NOT REQUIRED

6.5.8

Transverse Walls Design Loads, Column Line 7

Section cut design forces and moments, which follow a global axis system, for the transverse wall on column line 7 include in plane forces and moments: such as axial forces (tension/compression), in plane moment, and in plane shear. The section cut values are integrated along the section cut length, thus for INTSCUT5 the length is equal to 37'. Out of plane values such as out of plane bending and out of plane shear are attained by shell element forces and moments from SAP2000, which follow a local axis system. The out of plane bending moments provided are M22 and M12, which is a twisting moment that is combined with M22. The out of plane shear forces from the shell elements include V13 and V23.

Follow notation below to convert from SAP2000 labeling (i.e F1, M22...) to appropriate design forces and moments.

INTSCUT5
F1 = In Plane Shear
F3 = Axial Force, Compression (+) / Tension (-)
M2 = In Plane Moment

Accidental torsion factor = 15% (See assumption 3.1.4)

Section Cut	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
INTSCUT5	-3	-	1220	-	879	-	1442	-	911	-	20977	-

Shell Element Forces and Moments
M22 = Out of Plane Moment
M12 = Twisting Moment
V13 = Out of Plane Shear
V23 = Out of Plane Shear

MAX / MIN	M11	M22	M12	V13	V23	MAX	M11	M22	M12	V13	V23
	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft		Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	6	18	4	1	1	MAX	29	36	12	8	14
MIN	-3	-11	-6	-1	-3						

Loads with accidental torsion factor

Section Cut	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
INTSCUT5	-3	-	1403	-	1011	-	1658	-	1047	-	24123	-

Loads with accidental torsion factor

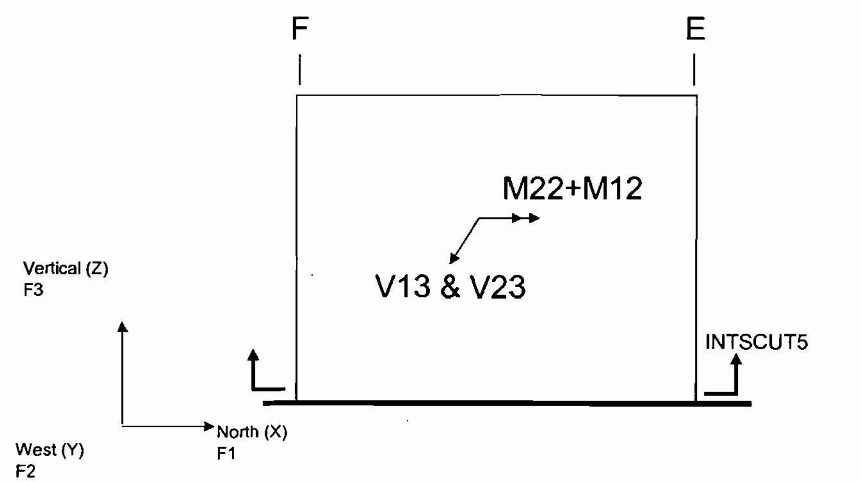
MAX / MIN	M11	M22	M12	V13	V23	MAX	M11	M22	M12	V13	V23
	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft		Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	7	20	5	2	1	MAX	33	41	13	9	17
MIN	-3	-12	-7	-1	-4						

Maximum Load Combination

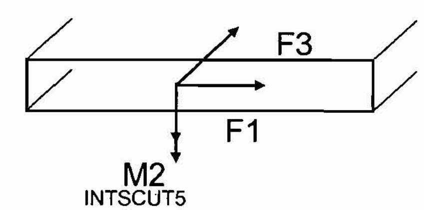
Section Cut	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
INTSCUT5	1655	-	2451	-	25135	-	-1661	-	356	-	-23112	-

Maximum Load Combination

MAX / MIN	M11	M22	M12	V13	V23	MAX / MIN	M11	M22	M12	V13	V23
	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft		Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	40	61	18	10	18	MAX	-27	-21	-8	-7	-15
MIN	30	29	7	8	13	MIN	-36	-53	-20	-10	-20



TRANSVERSE WALL AT COL. 7 ---- ELEVATION



6.5.8.1 TRANSVERSE WALL REINFORCEMENT, COLUMN LINE 7

Transverse Wall COL 7-INTSCUT5---D+L-SRSS

Design Loads

Axial Force (-Tension)	**Ft =**	0 kips
Axial Force (+Comp)	**Fc =**	356 kips
In plane shear	**Vu =**	1661 kips
In plane Moment	**Mz =**	23112 ft-kips
Out of plane shear	**Vz =**	20 kips/ft
Out of plane Moment	**My =**	73 ft-kips/ft (M22+M12)

Shear Wall Section Properties

Height of Wall (segment)	h_w =	35 ft
Ht of Wall Between Floors	H =	35 ft
Length of Wall (Segment)	l_w =	37 ft
Thickness of Wall	t_w =	4 ft
Shear Area of Wall (Segment)	$Acv = l_w*t_w$ =	148 ft^2

Concrete & Rebar Properties

Concrete Design Strength	f'c =	5000 psi
Concrete Strain	ε_c =	0.002
Rebar Yield Strength	fy =	60 ksi
Rebar Yield Strain	ε_y =	0.002
Min Steel Required	ρ_{min} =	0.0025
Concrete Cover		5 in

(Use 5" = 2" clear cover + diameter of the outer layer rebar + 1/2 diameter of the inner layer rebar)

Note: For ACI 349, see Ref. 2.2.14, for sections 1.0 to 6.0 of this design subset.

1.0 Check Shear on gross section - ACI 349: 21.6.5.6

Nominal Shear Capacity = $8*Acv*(f'c)^{1/2}$ — Vn (kips) = 12056

Factored Shear Load = Vu / ϕ — Vu/ϕ (kips)= 2768.3
(ϕ = .6 per ACI 349 - 9.3.4)

Demand Capacity Ratio
Check Code 21.6.5.6 Demand/Capacity Ratio — D/C = (Vu/ϕ)/Vn — D/C = 0.23 — **SHEAR WALL THICKNESS OK**

2.0 Horizontal Reinforcing Requirements

2.0a) ACI 349 - 21.6.5.3 Requirements

Determine α_c : h_w / l_w = 0.95 — α_c = 3 — α_c=3 for h_w/l_w<1.5, α_c varies linearly from 3 for h_w/l_w=1.5 to 2 for h_w/l_w=2.

Determine Concrete Shear Capacity $Vc=Acv*\alpha_c*(f'c)^{1/2}$ — Vc = 4521.0 kips

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc — Vs = 0.0 kips

Determine Required Shear Reinforcing ρ=Vs/(fy*Acv) — ρ = 0.0000

21.6.5.3 Shear Reinforcing Requirements — ρ_n **= 0.0025** — MINIMUM STEEL GOVERNS

2.0b) ACI 349 - 11.10.6 Equation 11-31 Requirements

Determine Concrete Shear Capacity — Vc = 3978.4 kips — $Vc=3.3*(f'_c)^{0.5}*(0.8*t_w*l_w)+F_t*0.8*l_w/(4*l_w)$

Determine Shear Carried by Steel — Vs=Vu/ϕ -Vc — Vs = 0.0 kips

Determine Required Shear Reinforcing — ρ = 0.0000 — $\rho=Vs/(0.8*l_w*t_w*144*fy)$

11.10.6 - Equation 11-31 Shear Reinforcing Requirements — ρ_n **= 0.0025** — MINIMUM STEEL GOVERNS

2.0c) ACI 349 - 11.10.6 Equation 11-32 Requirements

Check Bounding Case Mu/Vu - l_w/2 = -4.59 — **Mu/Vu-lw/2 < 0 equation 11-32 NOT APPLICABLE**

Determine Concrete Shear Capacity — Vc= N/A — $Vc=[0.6*f'_c{}^{0.5}+l_w*(1.25*f'_c{}^{0.5}+0.2*Ft*1000/(l_w*t_w*144))/(Mu/Vu-l_w/2)]*t_w*0.8*l_w*144/1000$

Determine Shear Carried by Steel — Vs = N/A — Vs=Vu/ϕ -Vc

Determine Required Shear Reinforcing Requirements — ρ = N/A — $\rho=Vs/(0.8*l_w*t_w*144*fy)$

11.10.6 - Equation 11-32 Shear Reinforcing Requirements — ρ_n **= N/A** — **EQUATION 11-32 NOT APPLICABLE**

2.0d) Horizontal Shear Reinforcing Requirements (max of 2a, 2b, 2c) — ρ_n **= 0.0025**

2.0e) Select Horizontal Shear Reinforcing — **Asn required per ft on each face =** **0.72** **in^2/ft each face** — $(\rho_n*12*t_w*12/2)$

Use **1-#11@12"c/c EF** **Asn provided =** **1.56** **in^2/ft each face** — ρ_n **(prov)= 0.0054167**

2.0f) Check Demand / Capacity Ratio for In-Plane Shear:

D/C = 0.25 — $D/C=(Vu/\phi)/[Vc+(Asn*2*fy*l_w*t_w*144/(12*t_w*12))]$

3.0 Vertical Reinforcing Requirements

3.0a) ACI 349 21.6.5.5 and 11.10.9.4 - Minimum vertical reinforcing ratio : — hw/lw = 0.95

ρ_v (min) = **0.0025**

If h_w/l_w>2.0, use: $\rho_{v\,min}=0.0025+0.5(2.5-h_w/l_w)(\rho_n-0.0025)<=\rho_n$
If h_w/l_w<=2.0, use: $\rho_v>=\rho_n$

3.0b) Check Shear Friction Requirements

In plane shear per foot of wall: 44.89 kips/ft (Vu/l_w)

Transverse shear per ft of wall: 20.00 kips/ft (Vz)

Resultant Shear 49.15 kips/ft $[(\text{in-plane shear})^2+(\text{transverse shear})^2]^{0.5}$

Calculate limiting shear friction strength at joint per ACI 349-11.7.5:

Vn<.2f'cAv for f'c = 5000 psi Vn=1000Av
Vn<800Av The limiting value of 800Av controls
Vn (MAX) = 460.8 kips/ft **Vn(MAX>Resultant shear ....OK**

Calculate shear friction reinforcing requirements per ACI 349 - 11.7.4.1:

Vn = Avf*fy*μ μ = 1.0 for concrete placed against hardened concrete intentionally roughened to a full amplitude of 1/4 inch (ACI 349-11.7.9)

Avf = Vu/2*ϕ*μ*fy (steel required per face)
Avf = 0.48 in^2/ft (steel required on each face for shear friction))

Calculate steel required for net Tension force

At = T/2*ϕ*fy*lw
At = 0.00 in^2/ft (steel required on each face for direct tension)

Steel Requirements for Shear Friction Avf + At

Av = 0.48 in^2/ft (steel required on each face for shear friction + direct tension)

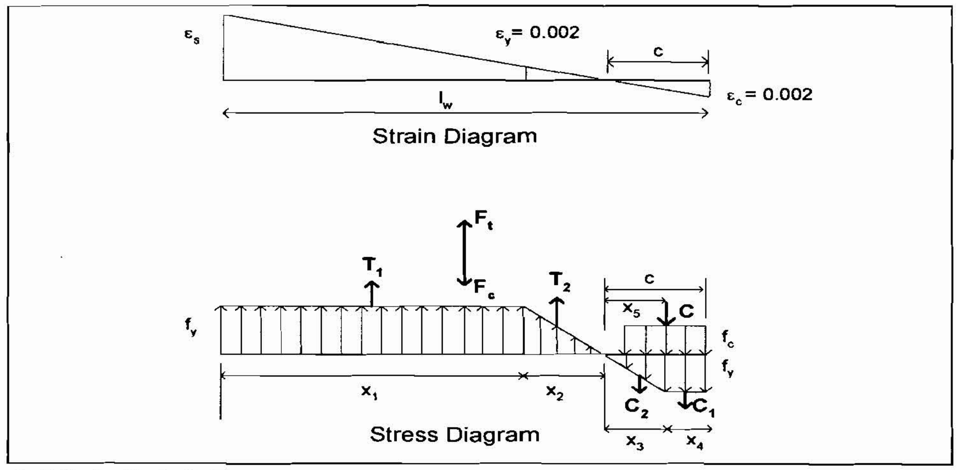
ρv(req'd) = 0.0017 $(\rho_{v\ req'd}=(2*Av)/(12*t_w*12))$

3.0c) Vertical Reinforcing Requirements (max of 3a,3b) **ρ_v = 0.0025**

3.0d) Select Vertical Reinforcing **Asv required per ft on each face = 0.72 in^2/ft each face** $(\rho_{v\ min}*12*t_w*12/2)$

Use 1-#11@12"c/c EF **Asv provided = 1.56 in^2/ft each face** **ρ_v (prov)= 0.0054167**

3.0e) Perform Strain-Compatible Section Analysis - For Axial Force (Tension) **Ft =** 0 kips



ϕ : Strength reduction factor (= 0.9 for tension, ACI 349 9.3.2.2), ϕ = 0.9
c : Distance from compression face to neutral axis (ft)
l_w : Length of wall (ft)
$\varepsilon_{s\_max} = [(l_w - c) / c] * \varepsilon_c$
$X_1 = l_w - c - X_2$ (ft) (length of wall with tension steel reinforcing steel strain > ε_y)
$X_2 = X_3 = (\varepsilon_y / \varepsilon_c) * c$ (ft) (length of wall with tension / compression steel reinforcing steel strain < ε_y)
$X_4 = c - X_3$ (ft) (length of wall with compression steel reinforcing steel strain > ε_y)
$T_1 = X_1 * As * fy$ (kips)
$T_2 = C_2 = 0.5 * X_2 * As * fy$ (kips)
$C_1 = X_4 * As * fy$ (kips)
$C = 0.85 * 0.8 * c * t_w * f'_c * 144$ (kips)
Balanced condition : Tension = Compression, $T_1+T_2-C_2-C_1-C+Ft/\phi = 0$.
M_u: Total moment capacity = $\phi\{T_1(X_2+X_1/2) + T_2(X_2*2/3) + C(c-0.8*c/2) + F_t(l_w/2-c)/\phi + C_2(X_3*2/3) + C_1(X_3+X_4/2)\}$, (kip-ft)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
2.97	34.03	0.0229	31.06	2.97	0.00	5815	278	0	5815	0.000	107129

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
2	35	0.0350	33.00	2.00	0.00	6177.6	187.2	0	3917	2261	107536
4	33.0	0.0165	29.00	4.00	0.00	5428.8	374.4	0	7834	-2405	109107
3	34	0.0227	31.00	3.00	0.00	5803.2	280.8	0	5875	-72	107152

Perform Strain-Compatible Section Analysis - For Axial Force (Comp) **Fc =** 356 kips

ϕ : Strength reduction factor (= 0.7 to 0.9 for comp, ACI 349 9.3.2.2), ϕ = 0.8933
If Fc<=0, ϕ=0.9; if Fc>0, ϕ=0.9-0.2*Fc/(0.1*Acv*144*f'$_c$/1000)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
3.14	33.86	0.0216	30.72	3.14	0.00	5751	294	0	6149	0.000	111957

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
2	35	0.0350	33.00	2.00	0.00	6177.6	187.2	0	3917	2659	112612
4	33.0	0.0165	29.00	4.00	0.00	5428.8	374.4	0	7834	-2006	113459
3	34	0.0227	31.00	3.00	0.00	5803.2	280.8	0	5875	327	111874

3.0f) Check Demand / Capacity Ratio for In-Plane Moment:

Vertical Reinforcing Ratio Provided by Design : ρ_v (prov)= 0.0054 1-#11@12"c/c EF ρ_v = (2*Asv)/(12*t_w*12)
D/C = (in-plane moment Mz) / (Min of Mu-ten, Mu-comp) D/C = 0.22 **Section Adequate**
Vertical Reinforcing Ratio Required for In-Plane Moment: $\rho_{vl\ req}$ = 0.0012 $\rho_{v\ req} = \rho_v$*(D/C)

3.0g) Consider Out-of-Plane Moment:

Vertical Reinf Ratio Required for Out-of-Plane Bending: $\rho_{vt\ req}$ = 0.00073 (per face) $\rho f_y bd$^2*(1-.59$\rho f_y/f'_c$)=Mu/ϕ solve for ρ Ref. 2.2.19, Section 4-3

3.0h) Total Reinforcing required for axial force, in-plane and out of plane bending ρ_v = 0.0019 $= \rho_{vl}+\rho_{vt} > 2\rho_{vt}$

D/C = ρ_v(reqd) / ρ_v (prov) D/C = 0.35 **Section Adequate** = ρ_v(req'd) / ρ_v(prov)

4.0 Boundary Elements:

h_w / l_w = 0.95 <2, No Need To Check for Boundary Elements

5.0 Out-of-Plane Shear:

Nominal Shear Strength Provided by The Concrete Vc=2*(f'c)$^{1/2}$*b*d Vc = 73.0 kips/ft width of wall

Check Demand / Capacity Ratio: D/C = 0.32 No Shear Reinforcement Required
D/C = (out-of-plane shear Vz) / (0.85*Vc)

6.0 Tabulate Reinforcement Requirements And D/C Ratios:

Use 4 ft thick wall with 1-#11@12"c/c EF Horizontal Reinforcement
1-#11@12"c/c EF Vertical Reinforcement.

For Shear on Gross Section: D/C = 0.23
For In-Plane Shear: D/C = 0.25
For Out-of-Plane Shear: D/C = 0.32
Bending + axial Loads D/C = 0.35

Boundary Elements D/C= BOUNDARY ELEMENTS NOT REQUIRED

Transverse Wall COL 7-INTSCUT5---D+L+SRSS

Design Loads

Axial Force (-Tension)	**Ft =**	0 kips
Axial Force (+Comp)	**Fc =**	2451 kips
In plane shear	**Vu =**	1655
In plane Moment	**Mz =**	25135 ft-kips
Out of plane shear	**Vz =**	18 kips/ft
Out of plane Moment	**My =**	79 ft-kips/ft (M22+M12)

Shear Wall Section Properties

Height of Wall (segment)	h_w =	35 ft
Ht of Wall Between Floors	H =	35 ft
Length of Wall (Segment)	l_w =	37 ft
Thickness of Wall	t_w =	4 ft
Shear Area of Wall (Segment)	$Acv = l_w*t_w$ =	148 ft^2

Concrete & Rebar Properties

Concrete Design Strength	f'c =	5000	psi
Concrete Strain	ε_c =	0.002	
Rebar Yield Strength	fy =	60	ksi
Rebar Yield Strain	ε_y =	0.002	
Min Steel Required	ρ_{min} =	0.0025	
Concrete Cover		5	in

(Use 5" = 2" clear cover + diameter of the outer layer rebar + 1/2 diameter of the inner layer rebar)

Note: For ACI 349, see Ref. 2.2.14, for sections 1.0 to 6.0 of this design subset.

1.0 Check Shear on gross section - ACI 349: 21.6.5.6

Nominal Shear Capacity = 8*Acv*(f'c) $^{1/2}$ Vn (kips) = 12056

Factored Shear Load = Vu / ϕ Vu/ϕ (kips)= 2758.3
(ϕ = .6 per ACI 349 - 9.3.4)

Demand Capacity Ratio
Check Code 21.6.5.6 Demand/Capacity Ratio D/C = (Vu/ϕ)/Vn D/C = 0.23 **SHEAR WALL THICKNESS OK**

2.0 Horizontal Reinforcing Requirements

2.0a) ACI 349 - 21.6.5.3 Requirements

Determine α_c : h_w / l_w = 0.95 α_c = 3 α_c=3 for h_w/l_w<1.5, α_c varies linearly from 3 for h_w/l_w=1.5 to 2 for h_w/l_w=2.

Determine Concrete Shear Capacity Vc=Acv*α_c*(f'c) $^{1/2}$ Vc = 4521.0 kips

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc Vs = 0.0 kips

Determine Required Shear Reinforcing ρ=Vs/(fy*Acv) ρ = 0.0000

21.6.5.3 Shear Reinforcing Requirements **ρ_n = 0.0025** MINIMUM STEEL GOVERNS

2.0b) ACI 349 - 11.10.6 Equation 11-31 Requirements

Determine Concrete Shear Capacity Vc = 3978.4 kips Vc=3.3*(f'$_c$)$^{0.5}$*(0.8*t_w*l_w)+F_t*0.8*l_w/(4*l_w)

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc Vs = 0.0 kips

Determine Required Shear Reinforcing ρ = 0.0000 ρ=Vs/(0.8*l_w*t_w*144*fy)

11.10.6 - Equation 11-31 Shear Reinforcing Requirements **ρ_n = 0.0025** MINIMUM STEEL GOVERNS

2.0c) ACI 349 - 11.10.6 Equation 11-32 Requirements

Check Bounding Case Mu/Vu - l_w/2 = -3.31 **Mu/Vu-lw/2 < 0 equation 11-32 NOT APPLICABLE**

Determine Concrete Shear Capacity Vc= N/A Vc=[0.6*f'$_c$$^{0.5}$+$l_w$*(1.25*f'$_c$$^{0.5}$+0.2*Ft*1000/($l_w$*$t_w$*144))/(Mu/Vu-$l_w$/2)]*$t_w$*0.8*$l_w$*144/1000

Determine Shear Carried by Steel Vs = N/A Vs=Vu/ϕ -Vc

Determine Required Shear Reinforcing Requirements ρ = N/A ρ=Vs/(0.8*l_w*t_w*144*fy)

11.10.6 - Equation 11-32 Shear Reinforcing Requirements **ρ_n = N/A** **EQUATION 11-32 NOT APPLICABLE**

2.0d) Horizontal Shear Reinforcing Requirements (max of 2a, 2b, 2c) **ρ_n = 0.0025**

2.0e) Select Horizontal Shear Reinforcing **Asn required per ft on each face =** **0.72** **in^2/ft each face** (ρ_n*12*t_w*12/2)

Use **1-#11@12"c/c EF** **Asn provided =** **1.56** **in^2/ft each face** **ρ_n (prov)=0.00542**

2.0f) Check Demand / Capacity Ratio for In-Plane Shear:

D/C = 0.25 D/C=(Vu/ϕ)/[Vc+(Asn*2*fy*l_w*t_w*144/(12*t_w*12))]

3.0 Vertical Reinforcing Requirements

3.0a) ACI 349 21.6.5.5 and 11.10.9.4 - Minimum vertical reinforcing ratio : hw/lw = 0.95

If h_w/l_w>2.0, use: $\rho_{v\,min}$=0.0025+0.5(2.5-h_w/l_w)(ρ_n-0.0025)<=ρ_n
ρ_v (min) = 0.0025 If h_w/l_w<=2.0, use: ρ_v>=ρ_n

3.0b) Check Shear Friction Requirements

In plane shear per foot of wall:	44.73 kips/ft	(V_u/l_w)
Transverse shear per ft of wall:	18.00 kips/ft	(V_z)
Resultant Shear	48.22 kips/ft	[(in-plane shear)^2+(transverse shear)^2]$^{0.5}$

Calculate limiting shear friction strength at joint per ACI 349-11.7.5:

Vn<.2f'cAv for f'c = 5000 psi Vn=1000Av
Vn<800Av — The limiting value of 800Av controls
Vn (MAX) = 460.8 kips/ft **Vn(MAX>Resultant shear ....OK**

Calculate shear friction reinforcing requirements per ACI 349 - 11.7.4.1:

Vn = Avf*fy*μ — μ = 1.0 for concrete placed against hardened concrete intentionally roughened to a full amplitude of 1/4 inch (ACI 349-11.7.9)

Avf = Vu/2*ϕ*μ*fy (steel required per face)
Avf = 0.47 in^2/ft (steel required on each face for shear friction))

Calculate steel required for net Tension force

At = T/2*ϕ*fy*lw
At = 0.00 in^2/ft (steel required on each face for direct tension)

Steel Requirements for Shear Friction Avf + At

Av = 0.47 in^2/ft (steel required on each face for shear friction + direct tension)

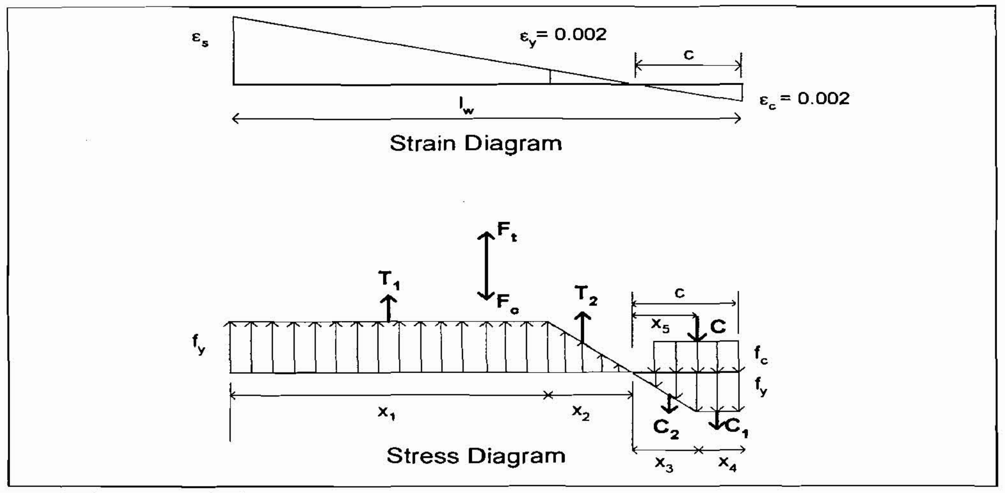
ρv(req'd) = 0.0016 ($\rho_{v\ req'd}$=(2*Av)/(12*t_w*12)

3.0c) Vertical Reinforcing Requirements (max of 3a,3b) **ρ_v = 0.0025**

3.0d) Select Vertical Reinforcing **Asv required per ft on each face = 0.72 in^2/ft each face** ($\rho_{v\ min}$*12*t_w*12/2)

Use 1-#11@12"c/cEF **Asv provided = 1.56 in^2/ft each face** **ρ_v (prov)= 0.00542**

3.0e) Perform Strain-Compatible Section Analysis - For Axial Force (Tension) **Ft =** 0 kips



ϕ : Strength reduction factor (= 0.9 for tension, ACI 349 9.3.2.2), ϕ = 0.9
c : Distance from compression face to neutral axis (ft)
l_w : Length of wall (ft)
$\varepsilon_{s\_max} = [(l_w - c) / c] * \varepsilon_c$
$X_1 = l_w - c - X_2$ (ft) (length of wall with tension steel reinforcing steel strain > ε_y)
$X_2 = X_3 = (\varepsilon_y / \varepsilon_c) * c$ (ft) (length of wall with tension / compression steel reinforcing steel strain < ε_y)
$X_4 = c - X_3$ (ft) (length of wall with compression steel reinforcing steel strain > ε_y)
$T_1 = X_1 * As * fy$ (kips)
$T_2 = C_2 = 0.5 * X_2 * As * fy$ (kips)
$C_1 = X_4 * As * fy$ (kips)
$C = 0.85 * 0.8 * c * t_w * f'_c * 144$ (kips)
Balanced condition : Tension = Compression, $T_1+T_2-C_2-C_1-C+Ft/\phi = 0$.
M_u : Total moment capacity = $\phi\{T_1(X_2+X_1/2) + T_2(X_2*2/3) + C(c-0.8*c/2) + F_t(l_w/2-c)/\phi + C_2(X_3*2/3) + C_1(X_3+X_4/2)\}$, (kip-ft)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
2.97	34.03	0.0229	31.06	2.97	0.00	5815	278	0	5815	0.000	107129

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
3	34	0.0227	31.00	3.00	0.00	5803.2	280.8	0	5875	-72	107152
4	33.0	0.0165	29.00	4.00	0.00	5428.8	374.4	0	7834	-2405	109107

2	35	0.0350	33.00	2.00	0.00	6177.6	187.2	0	3917	2261	107536

Perform Strain-Compatible Section Analysis - For Axial Force (Comp) **Fc =** 2451 kips

ϕ : Strength reduction factor (= 0.7 to 0.9 for comp, ACI 349 9.3.2.2), ϕ = 0.8540
If Fc<=0, ϕ=0.9; if Fc>0, ϕ=0.9-0.2*Fc/(0.1*Acv*144*f'_c/1000)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
4.20	32.80	0.0156	28.60	4.20	0.00	5354	393	0	8224	0.000	139217

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_c/\phi$	M_u (k-ft)
5	32	0.0128	27.00	5.00	0.00	5054.4	468	0	9792	-1868	140694
3	34.0	0.0227	31.00	3.00	0.00	5803.2	280.8	0	5875	2798	139666
4	33	0.0165	29.00	4.00	0.00	5428.8	374.4	0	7834	465	139070

3.0f) Check Demand / Capacity Ratio for In-Plane Moment:

Vertical Reinforcing Ratio Provided by Design : ρ_v (prov)= 0.0054 1-#11@12"c/cEF ρ_v = (2*Asv)/(12*t_w*12)
D/C = (in-plane moment Mz) / (Min of Mu-ten, Mu-comp) D/C = 0.23 **Section Adequate**
Vertical Reinforcing Ratio Required for In-Plane Moment: $\rho_{vl\,req}$ = 0.0013 $\rho_{v\,req}$ = ρ_v*(D/C)

3.0g) Consider Out-of-Plane Moment:

Vertical Reinf Ratio Required for Out-of-Plane Bending: $\rho_{vt\,req}$ = 0.00080 (per face) $\rho f_y bd$^2*(1-.59$\rho f_y/f'_c$)=Mu/ϕ solve for ρ Ref. 2.2.19, Section 4-3

3.0h) Total Reinforcing required for axial force, in-plane and out of plane bending ρ_v = 0.0021 = $\rho_{vl}+\rho_{vt} > 2\rho_{vt}$

D/C = ρ_v(reqd) / ρ_v (prov) D/C = 0.38 **Section Adequate** = ρ_v(req'd) / ρ_v(prov)

4.0 Boundary Elements:

h_w / l_w = 0.95 <2, No Need To Check for Boundary Elements

5.0 Out-of-Plane Shear:

Nominal Shear Strength Provided by The Concrete Vc=2*(f'c)$^{1/2}$*b*d Vc = 73.0 kips/ft width of wall

Check Demand / Capacity Ratio: D/C = 0.29 No Shear Reinforcement Required
D/C = (out-of-plane shear Vz) / (0.85*Vc)

6.0 Tabulate Reinforcement Requirements And D/C Ratios:

Use **4** **ft thick wall with** **1-#11@12"c/c EF** **Horizontal Reinforcement**
1-#11@12"c/cEF **Vertical Reinforcement.**

For Shear on Gross Section:	**D/C =**	**0.23**
For In-Plane Shear:	**D/C =**	**0.25**
For Out-of-Plane Shear:	**D/C =**	**0.29**
Bending + axial Loads	**D/C =**	**0.38**

Boundary Elements **D/C=** **BOUNDARY ELEMENTS NOT REQUIRED**

6.5.9

Transverse Walls Design Loads, Column Line 8

Section cut design forces and moments, which follow a global axis system, for the transverse wall on column line 8 include in plane forces and moments: such as axial forces (tension/compression), in plane moment, and in plane shear. The section cut values are integrated along the section cut length, thus for INTSCUT6 the length is equal to 37'. Out of plane values such as out of plane bending and out of plane shear are attained by shell element forces and moments from SAP2000, which follow a local axis system. The out of plane bending moments provided are M22 and M12, which is a twisting moment that is combined with M22. The out of plane shear forces from the shell elements include V13 and V23.

Follow notation below to convert from SAP2000 labeling (i.e F1, M22...) to appropriate design forces and moments.

INTSCUT6
F1 = In Plane Shear
F3 = Axial Force, Compression (+) / Tension (-)
M2 = In Plane Moment

Accidental torsion factor = 15% (See assumption 3.1.4)

Shell Element Forces and Moments
M22 = Out of Plane Moment
M12 = Twisting Moment
V13 = Out of Plane Shear
V23 = Out of Plane Shear

Section Cu	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
INTSCUT6	-3	-	1034	-	387	-	555	-	399	-	15210	-

MAX / MIN	M11	M22	M12	V13	V23	MAX	M11	M22	M12	V13	V23
	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft		Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	5	8	6	1	2	MAX	31	130	17	7	18
MIN	-4	-18	-2	-1	0						

Loads with accidental torsion factor

Section Cu	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
INTSCUT6	-4	-	1189	-	445	-	638	-	459	-	17492	-

Loads with accidental torsion factor

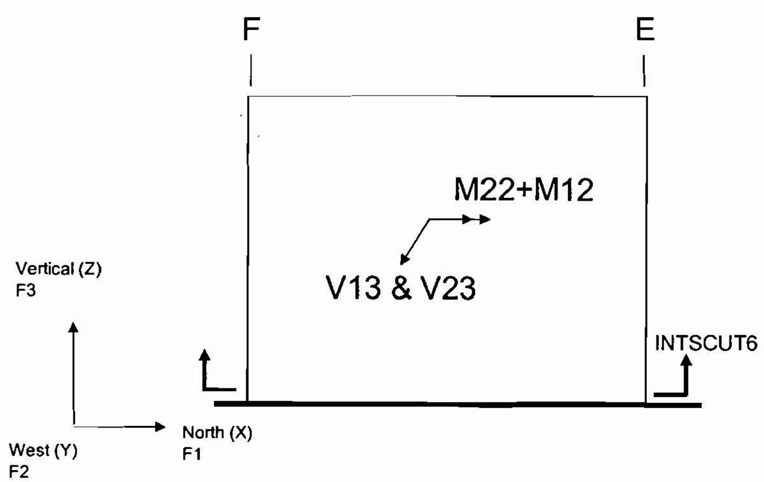
MAX / MIN	M11	M22	M12	V13	V23	MAX	M11	M22	M12	V13	V23
	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft		Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	6	10	7	1	2	MAX	36	150	19	8	20
MIN	-4	-21	-2	-1	0						

Maximum Load Combination

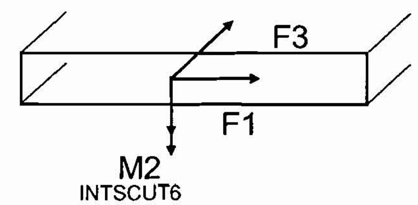
Section Cu	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
INTSCUT6	634	-	1647	-	17937	-	-642	-	730	-	-17046	-

Maximum Load Combination

MAX / MIN	M11	M22	M12	V13	V23	MAX / MIN	M11	M22	M12	V13	V23
	Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft		Kip-ft/ft	Kip-ft/ft	Kip-ft/ft	Kip/ft	Kip/ft
MAX	42	159	26	9	23	MAX	-30	-140	-13	-7	-18
MIN	32	129	17	7	20	MIN	-40	-171	-21	-9	-21



TRANSVERSE WALL AT COL. 8 ---- ELEVATION



Transverse Wall COL 8-INTSCUT6---D+L+SRSS

Transverse Wall COL 8-INTSCUT6---D+L-SRSS

Design Loads

Axial Force (-Tension)	**Ft =**	0 kips
Axial Force (+Comp)	**Fc =**	730 kips
In plane shear	**Vu =**	642 kips
In plane Moment	**Mz =**	17046 ft-kips
Out of plane shear	**Vz =**	20 kips/ft
Out of plane Moment	**My =**	192 ft-kips/ft (M22+M12)

Shear Wall Section Properties

Height of Wall (segment)	h_w =	35 ft
Ht of Wall Between Floors	H =	35 ft
Length of Wall (Segment)	l_w =	37 ft
Thickness of Wall	t_w =	4 ft
Shear Area of Wall (Segment)	$Acv = l_w{}^*t_w$ =	148 ft^2

Concrete & Rebar Properties

Concrete Design Strength	f'c =	5000 psi
Concrete Strain	ε_c =	0.002
Rebar Yield Strength	fy =	60 ksi
Rebar Yield Strain	ε_y =	0.002
Min Steel Required	ρ_{min} =	0.0025
Concrete Cover		5 in

(Use 5" = 2" clear cover + diameter of the outer layer rebar + 1/2 diameter of the inner layer rebar)

Note: For ACI 349, see Ref. 2.2.14, for sections 1.0 to 6.0 of this design subset.

1.0 Check Shear on gross section - ACI 349: 21.6.5.6

Nominal Shear Capacity = 8*Acv*(f'c) $^{1/2}$ Vn (kips) = 12056

Factored Shear Load = Vu / ϕ Vu/ϕ (kips)= 1070.0
(ϕ = .6 per ACI 349 - 9.3.4)

Demand Capacity Ratio
Check Code 21.6.5.6 Demand/Capacity Ratio D/C = (Vu/ϕ)/Vn D/C = 0.09 **SHEAR WALL THICKNESS OK**

2.0 Horizontal Reinforcing Requirements

2.0a) ACI 349 - 21.6.5.3 Requirements

Determine α_c : h_w / l_w = 0.95 α_c = 3 α_c=3 for h_w/l_w<1.5, α_c varies linearly from 3 for h_w/l_w=1.5 to 2 for h_w/l_w=2.

Determine Concrete Shear Capacity Vc=Acv*α_c*(f'c) $^{1/2}$ Vc = 4521.0 kips

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc Vs = 0.0 kips

Determine Required Shear Reinforcing ρ=Vs/(fy*Acv) ρ = 0.0000

21.6.5.3 Shear Reinforcing Requirements **ρ_n = 0.0025** MINIMUM STEEL GOVERNS

2.0b) ACI 349 - 11.10.6 Equation 11-31 Requirements

Determine Concrete Shear Capacity Vc = 3978.4 kips Vc=3.3*(f'c)$^{0.5}$*(0.8*t_w*l_w)+F_t*0.8*l_w/(4*l_w)

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc Vs = 0.0 kips

Determine Required Shear Reinforcing ρ = 0.0000 ρ=Vs/(0.8*l_w*t_w*144*fy)

11.10.6 - Equation 11-31 Shear Reinforcing Requirements **ρ_n = 0.0025** MINIMUM STEEL GOVERNS

2.0c) ACI 349 - 11.10.6 Equation 11-32 Requirements

Check Bounding Case Mu/Vu - l_w/2 = 8.05 **Equation 11-32 APPLICABLE**

Determine Concrete Shear Capacity Vc= 7648.66663 Vc=[0.6*f'c$^{0.5}$+l_w*(1.25*f'c$^{0.5}$+0.2*Ft*1000/(l_w*t_w*144))/(Mu/Vu-l_w/2)]*t_w*0.8*l_w*144/1000

Determine Shear Carried by Steel Vs = 0 Vs=Vu/ϕ -Vc

Determine Required Shear Reinforcing Requirements ρ = 0.0000 ρ=Vs/(0.8*l_w*t_w*144*fy)

11.10.6 - Equation 11-32 Shear Reinforcing Requirements **ρ_n = 0.0025** **MINIMUM STEEL GOVERNS**

2.0d) Horizontal Shear Reinforcing Requirements (max of 2a, 2b, 2c) **ρ_n = 0.0025**

2.0e) Select Horizontal Shear Reinforcing **Asn required per ft on each face =** **0.72** **in^2/ft each face** (ρ_n*12*t_w*12/2)

Use **1-#11@12"c/c EF** **Asn provided =** **1.56** **in^2/ft each face** **ρ_n (prov)= 0.0054167**

2.0f) Check Demand / Capacity Ratio for In-Plane Shear:

D/C = 0.10 D/C=(Vu/ϕ)/[Vc+(Asn*2*fy*l_w*t_w*144/(12*t_w*12))]

3.0 Vertical Reinforcing Requirements

3.0a) ACI 349 21.6.5.5 and 11.10.9.4 - Minimum vertical reinforcing ratio : hw/lw = 0.95

If h_w/l_w>2.0, use: $\rho_{v\ min}$ =0.0025+0.5(2.5-h_w/l_w)(ρ_n-0.0025)<=ρ_n
If h_w/l_w<=2.0, use: ρ_v>=ρ_n

ρ_v (min) =0.0025

3.0b) Check Shear Friction Requirements

In plane shear per foot of wall: 17.35 kips/ft (Vu/l_w)

Transverse shear per ft of wall: 20.00 kips/ft (Vz)

Resultant Shear 26.48 kips/ft $[(\text{in-plane shear})^2+(\text{transverse shear})^2]^{0.5}$

Calculate limiting shear friction strength at joint per ACI 349-11.7.5:

Vn<.2f'cAv for f'c = 5000 psi Vn=1000Av
Vn<800Av The limiting value of 800Av controls
Vn (MAX) = 460.8 kips/ft **Vn(MAX>Resultant shear ....OK**

Calculate shear friction reinforcing requirements per ACI 349 - 11.7.4.1:

Vn = Avf*fy*μ μ = 1.0 for concrete placed against hardened concrete intentionally roughened to a full amplitude of 1/4 inch (ACI 349-11.7.9)

Avf = Vu/2*ϕ*μ*fy (steel required per face)
Avf = 0.26 in^2/ft (steel required on each face for shear friction))

Calculate steel required for net Tension force

At = T/2*ϕ*fy*lw
At = 0.00 in^2/ft (steel required on each face for direct tension)

Steel Requirements for Shear Friction Avf + At

Av = 0.26 in^2/ft (steel required on each face for shear friction + direct tension)

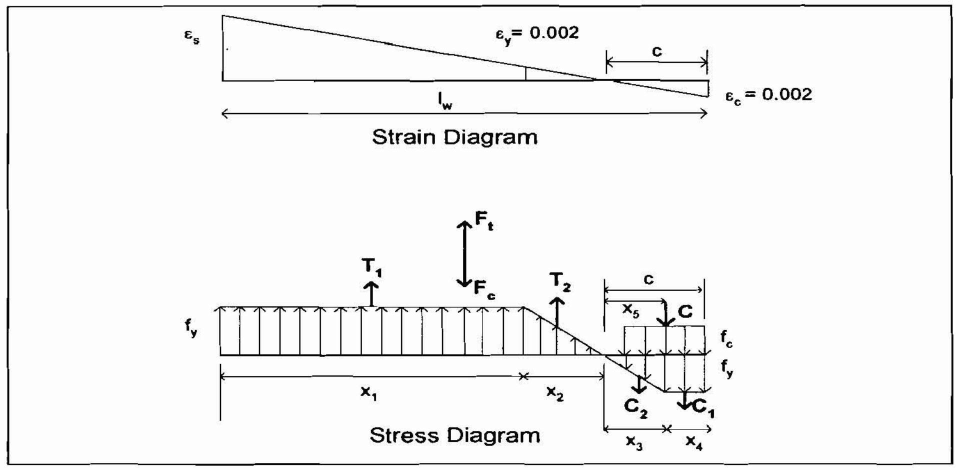
ρv(req'd) = 0.0009 $(\rho_{v\,req'd}=(2^*Av)/(12^*t_w^*12)$

3.0c) Vertical Reinforcing Requirements (max of 3a,3b) **ρ_v = 0.0025**

3.0d) Select Vertical Reinforcing **Asv required per ft on each face = 0.72 in^2/ft each face** $(\rho_{v\,min}{}^*12^*t_w{}^*12/2)$

Use 1-#11@12"c/c EF **Asv provided = 1.56 in^2/ft each face** **ρ_v (prov)= 0.0054167**

3.0e) Perform Strain-Compatible Section Analysis - For Axial Force (Tension) **Ft =** 0 kips



ϕ : Strength reduction factor (= 0.9 for tension, ACI 349 9.3.2.2), ϕ = 0.9
c : Distance from compression face to neutral axis (ft)
l_w : Length of wall (ft)
$\varepsilon_{s\_max} = [(l_w - c) / c] * \varepsilon_c$
$X_1 = l_w - c - X_2$ (ft) (length of wall with tension steel reinforcing steel strain > ε_y)
$X_2 = X_3 = (\varepsilon_y / \varepsilon_c) * c$ (ft) (length of wall with tension / compression steel reinforcing steel strain < ε_y)
$X_4 = c - X_3$ (ft) (length of wall with compression steel reinforcing steel strain > ε_y)
$T_1 = X_1 * As * fy$ (kips)
$T_2 = C_2 = 0.5 * X_2 * As * fy$ (kips)
$C_1 = X_4 * As * fy$ (kips)
$C = 0.85 * 0.8 * c * t_w * f'_c * 144$ (kips)
Balanced condition : Tension = Compression, $T_1+T_2-C_2-C_1-C+Ft/\phi = 0$.
M_u: Total moment capacity = $\phi\{T_1(X_2+X_1/2) + T_2(X_2*2/3) + C(c-0.8*c/2) + F_t(l_w/2-c)/\phi + C_2(X_3*2/3) + C_1(X_3+X_4/2)\}$, (kip-ft)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
2.97	34.03	0.0229	31.06	2.97	0.00	5815	278	0	5815	0.000	107129

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
3	34	0.0227	31.00	3.00	0.00	5803.2	280.8	0	5875	-72	107152
4	33.0	0.0165	29.00	4.00	0.00	5428.8	374.4	0	7834	-2405	109107
2	35	0.0350	33.00	2.00	0.00	6177.6	187.2	0	3917	2261	107536

Perform Strain-Compatible Section Analysis - For Axial Force (Comp) **Fc =** 730 kips

ϕ : Strength reduction factor (= 0.7 to 0.9 for comp, ACI 349 9.3.2.2), ϕ = 0.8863
If Fc<=0, ϕ=0.9; if Fc>0, ϕ=0.9-0.2*Fc/(0.1*Acv*144*f'$_c$/1000)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	T_1+T_2-C_2-C_1-C+F_c/ϕ	M_u (k-ft)
3.32	33.68	0.0203	30.36	3.32	0.00	5683	311	0	6506	0.000	116969

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	T_1+T_2-C_2-C_1-C+F_c/ϕ	M_u (k-ft)
5	32	0.0128	27.00	5.00	0.00	5054.4	468	0	9792	-3914	121531
4	33.0	0.0165	29.00	4.00	0.00	5428.8	374.4	0	7834	-1581	118031
3	34	0.0227	31.00	3.00	0.00	5803.2	280.8	0	5875	752	116836

3.0f) Check Demand / Capacity Ratio for In-Plane Moment:

Vertical Reinforcing Ratio Provided by Design : ρ_v (prov)= 0.0054 1-#11@12"c/c EF ρ_v = (2*Asv)/(12*t_w*12)
D/C = (in-plane moment Mz) / (Min of Mu-ten, Mu-comp) D/C = 0.16 **Section Adequate**
Vertical Reinforcing Ratio Required for In-Plane Moment: $\rho_{vl\ req}$ = 0.0009 $\rho_{v\ req}$ = ρ_v*(D/C)

3.0g) Consider Out-of-Plane Moment:

Vertical Reinf Ratio Required for Out-of-Plane Bending: $\rho_{vt\ req}$ = 0.00195 (per face) $\rho f_y bd$^2*(1-.59ρf_y/f'$_c$)=Mu/ϕ solve for ρ Ref. 2.2.19, Section 4-3

3.0h) Total Reinforcing required for axial force, in-plane and out of plane bending ρ_v = 0.0039 = ρ_{vl}+ρ_{vt} > 2ρ_{vt}

D/C = ρ_v(reqd) / ρ_v (prov) D/C = 0.72 **Section Adequate** = ρ_v(req'd) / ρ_v(prov)

4.0 Boundary Elements:

h_w / l_w = 0.95 <2, No Need To Check for Boundary Elements

5.0 Out-of-Plane Shear:

Nominal Shear Strength Provided by The Concrete Vc=2*(f'c)$^{1/2}$*b*d Vc = 73.0 kips/ft width of wall

Check Demand / Capacity Ratio: D/C = 0.32 No Shear Reinforcement Required
D/C = (out-of-plane shear Vz) / (0.85*Vc)

6.0 Tabulate Reinforcement Requirements And D/C Ratios:

Use 4 **ft thick wall with** **1-#11@12"c/c EF** **Horizontal Reinforcement**
1-#11@12"c/c EF **Vertical Reinforcement.**

For Shear on Gross Section:	**D/C =**	**0.09**
For In-Plane Shear:	**D/C =**	**0.10**
For Out-of-Plane Shear:	**D/C =**	**0.32**
Bending + axial Loads	**D/C =**	**0.72**

Boundary Elements **D/C=** **BOUNDARY ELEMENTS NOT REQUIRED**

Transverse Wall COL 8-INTSCUT6---D+L+SRSS

Design Loads			**Shear Wall Section Properties**			**Concrete & Rebar Properties**		
Axial Force (-Tension)	**Ft =**	0 kips	**Height of Wall (segment)**	**h_w =**	35 ft	**Concrete Design Strength**	**f'c =**	5000 psi
Axial Force (+Comp)	**Fc =**	1647 kips	**Ht of Wall Between Floors**	**H =**	35 ft	**Concrete Strain**	**ε_c =**	0.002
In plane shear	**Vu =**	634	**Length of Wall (Segment)**	**l_w =**	37 ft	**Rebar Yield Strength**	**fy =**	60 ksi
In plane Moment	**Mz =**	17937 ft-kips	**Thickness of Wall**	**t_w =**	4 ft	**Rebar Yield Strain**	**ε_y =**	0.002
			Shear Area of Wall (Segment)	**Acv = l_w*t_w =**	148 ft^2	**Min Steel Required**	**ρ_{min} =**	0.0025
						Concrete Cover		5 in

Out of plane shear	**Vz =**	23 kips/ft
Out of plane Moment	**My =**	185 ft-kips/ft (M22+M12)

(Use 5" = 2" clear cover + diameter of the outer layer rebar + 1/2 diameter of the inner layer rebar)

Note: For ACI 349, see Ref. 2.2.14, for sections 1.0 to 6.0 of this design subset.

1.0 Check Shear on gross section - ACI 349: 21.6.5.6

Nominal Shear Capacity = 8*Acv*(f'c) $^{1/2}$ Vn (kips) = 12056

Factored Shear Load = Vu / ϕ Vu/ϕ (kips)= 1056.7
(ϕ = .6 per ACI 349 - 9.3.4)

Demand Capacity Ratio
Check Code 21.6.5.6 Demand/Capacity Ratio D/C = (Vu/ϕ)/Vn D/C = 0.09 **SHEAR WALL THICKNESS OK**

2.0 Horizontal Reinforcing Requirements

2.0a) ACI 349 - 21.6.5.3 Requirements

Determine α_c : h_w / l_w = 0.95 α_c = 3 α_c=3 for h_w/l_w<1.5, α_c varies linearly from 3 for h_w/l_w=1.5 to 2 for h_w/l_w=2.

Determine Concrete Shear Capacity Vc=Acv*α_c*(f'c) $^{1/2}$ Vc = 4521.0 kips

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc Vs = 0.0 kips

Determine Required Shear Reinforcing ρ=Vs/(fy*Acv) ρ = 0.0000

21.6.5.3 Shear Reinforcing Requirements **ρ_n = 0.0025** MINIMUM STEEL GOVERNS

2.0b) ACI 349 - 11.10.6 Equation 11-31 Requirements

Determine Concrete Shear Capacity Vc = 3978.4 kips Vc=3.3*(f'$_c$)$^{0.5}$*(0.8*t_w*l_w)+F_t*0.8*l_w/(4*l_w)

Determine Shear Carried by Steel Vs=Vu/ϕ -Vc Vs = 0.0 kips

Determine Required Shear Reinforcing ρ = 0.0000 ρ=Vs/(0.8*l_w*t_w*144*fy)

11.10.6 - Equation 11-31 Shear Reinforcing Requirements **ρ_n = 0.0025** MINIMUM STEEL GOVERNS

2.0c) ACI 349 - 11.10.6 Equation 11-32 Requirements

Check Bounding Case Mu/Vu - l_w/2 = 9.79 **Equation 11-32 APPLICABLE**

Determine Concrete Shear Capacity Vc= 6417.75999 Vc=[0.6*f'$_c$$^{0.5}$+$l_w$*(1.25*f'$_c$$^{0.5}$+0.2*Ft*1000/($l_w$*$t_w$*144))/(Mu/Vu-$l_w$/2)]*$t_w$*0.8*$l_w$*144/1000

Determine Shear Carried by Steel Vs = 0 Vs=Vu/ϕ -Vc

Determine Required Shear Reinforcing Requirements ρ = 0.0000 ρ=Vs/(0.8*l_w*t_w*144*fy)

11.10.6 - Equation 11-32 Shear Reinforcing Requirements **ρ_n = 0.0025** **MINIMUM STEEL GOVERNS**

2.0d) Horizontal Shear Reinforcing Requirements (max of 2a, 2b, 2c) **ρ_n = 0.0025**

2.0e) Select Horizontal Shear Reinforcing **Asn required per ft on each face =** **0.72** **in^2/ft each face** (ρ_n*12*t_w*12/2)

Use **1-#11@12"c/c EF** **Asn provided =** **1.56** **in^2/ft each face** **ρ_n (prov)=0.00542**

2.0f) Check Demand / Capacity Ratio for In-Plane Shear:

D/C = 0.10 D/C=(Vu/ϕ)/[Vc+(Asn*2*fy*l_w*t_w*144/(12*t_w*12))]

3.0 Vertical Reinforcing Requirements

3.0a) ACI 349 21.6.5.5 and 11.10.9.4 - Minimum vertical reinforcing ratio : hw/lw = 0.95

If h_w/l_w>2.0, use: $\rho_{v\ min}$=0.0025+0.5(2.5-h_w/l_w)(ρ_n-0.0025)<=ρ_n

ρ_v (min) = 0.0025 If h_w/l_w<=2.0, use: ρ_v>=ρ_n

3.0b) Check Shear Friction Requirements

In plane shear per foot of wall: 17.14 kips/ft (Vu/l_w)

Transverse shear per ft of wall: 23.00 kips/ft (Vz)

Resultant Shear 28.68 kips/ft $[(\text{in-plane shear})^2+(\text{transverse shear})^2]^{0.5}$

Calculate limiting shear friction strength at joint per ACI 349-11.7.5:

Vn<.2f'cAv for f'c = 5000 psi Vn=1000Av
Vn<800Av The limiting value of 800Av controls
Vn (MAX) = 460.8 kips/ft **Vn(MAX>Resultant shear ....OK**

Calculate shear friction reinforcing requirements per ACI 349 - 11.7.4.1:

Vn = Avf*fy*μ   μ = 1.0 for concrete placed against hardened concrete intentionally roughened to a full amplitude of 1/4 inch (ACI 349-11.7.9)

Avf = Vu/2*ϕ*μ*fy (steel required per face)
Avf = 0.28 in^2/ft (steel required on each face for shear friction))

Calculate steel required for net Tension force

At = T/2*ϕ*fy*lw
At = 0.00 in^2/ft (steel required on each face for direct tension)

Steel Requirements for Shear Friction Avf + At

Av = 0.28 in^2/ft (steel required on each face for shear friction + direct tension)

ρv(req'd) = 0.0010 ($\rho_{v\,req'd}$=(2*Av)/(12*t_w*12)

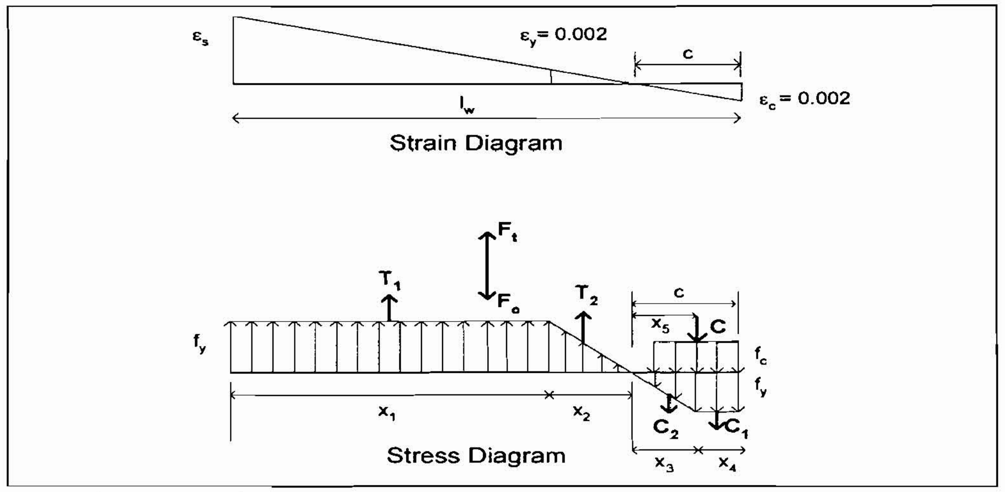
3.0c) Vertical Reinforcing Requirements (max of 3a,3b) **ρ_v = 0.0025**

3.0d) Select Vertical Reinforcing **Asv required per ft on each face = 0.72 in^2/ft each face** ($\rho_{v\,min}$*12*t_w*12/2)

Use 1-#11@12"c/cEF **Asv provided = 1.56 in^2/ft each face** **ρ_v (prov)= 0.00542**

3.0e) Perform Strain-Compatible Section Analysis - For Axial Force (Tension) **Ft =** 0 kips

ϕ : Strength reduction factor (= 0.9 for tension, ACI 349 9.3.2.2), ϕ = 0.9
c : Distance from compression face to neutral axis (ft)
l_w : Length of wall (ft)
$\varepsilon_{s\_max} = [(l_w - c) / c] * \varepsilon_c$
$X_1 = l_w - c - X_2$ (ft) (length of wall with tension steel reinforcing steel strain > ε_y)
$X_2 = X_3 = (\varepsilon_y / \varepsilon_c) * c$ (ft) (length of wall with tension / compression steel reinforcing steel strain < ε_y)
$X_4 = c - X_3$ (ft) (length of wall with compression steel reinforcing steel strain > ε_y)
$T_1 = X_1 * As * fy$ (kips)
$T_2 = C_2 = 0.5 * X_2 * As * fy$ (kips)
$C_1 = X_4 * As * fy$ (kips)
$C = 0.85 * 0.8 * c * t_w * f'_c * 144$ (kips)
Balanced condition : Tension = Compression, $T_1+T_2-C_2-C_1-C+Ft/\phi = 0$.
M_u : Total moment capacity = $\phi\{T_1(X_2+X_1/2) + T_2(X_2*2/3) + C(c-0.8*c/2) + F_t(l_w/2-c)/\phi + C_2(X_3*2/3) + C_1(X_3+X_4/2)\}$, (kip-ft)



Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
2.97	34.03	0.0229	31.06	2.97	0.00	5815	278	0	5815	0.000	107129

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	$X_2=X_3$ (ft)	X_4 (ft)	T_1 (k)	$T_2=C_2$ (k)	C_1 (k)	C (k)	$T_1+T_2-C_2-C_1-C+F_t/\phi$	M_u (k-ft)
4	33	0.0165	29.00	4.00	0.00	5428.8	374.4	0	7834	-2405	109107
3	34.0	0.0227	31.00	3.00	0.00	5803.2	280.8	0	5875	-72	107152

2	35	0.0350	33.00	2.00	0.00	6177.6	187.2	0	3917	2261	107536

Perform Strain-Compatible Section Analysis - For Axial Force (Comp) **Fc =** 1647 kips

ϕ : Strength reduction factor (= 0.7 to 0.9 for comp, ACI 349 9.3.2.2), ϕ = 0.8691
If Fc<=0, ϕ=0.9; if Fc>0, ϕ=0.9-0.2*Fc/(0.1*Acv*144*f'_c/1000)

Using Goal Seek to find "c" and "Mu"

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	T_1+T_2-C_2-C_1-C+F_c/ϕ	M_u (k-ft)
3.78	33.22	0.0176	29.44	3.78	0.00	5511	354	0	7406	0.000	128996

Verified that equations were correct

c (ft)	l_w - c (ft)	ε_{s_max}	X_1 (ft)	X_2=X_3 (ft)	X_4 (ft)	T_1 (k)	T_2=C_2 (k)	C_1 (k)	C (k)	T_1+T_2-C_2-C_1-C+F_c/ϕ	M_u (k-ft)
5	32	0.0128	27.00	5.00	0.00	5054.4	468	0	9792	-2843	131742
4	33.0	0.0165	29.00	4.00	0.00	5428.8	374.4	0	7834	-510	129241
3	34	0.0227	31.00	3.00	0.00	5803.2	280.8	0	5875	1823	129000

3.0f) Check Demand / Capacity Ratio for In-Plane Moment:

Vertical Reinforcing Ratio Provided by Design : ρ_v (prov)= 0.0054 1-#11@12"c/cEF ρ_v = (2*Asv)/(12*t_w*12)
D/C = (in-plane moment Mz) / (Min of Mu-ten, Mu-comp) D/C = 0.17 **Section Adequate**
Vertical Reinforcing Ratio Required for In-Plane Moment: $\rho_{vl\ req}$ = 0.0009 $\rho_{v\ req}$ = ρ_v*(D/C)

3.0g) Consider Out-of-Plane Moment:

Vertical Reinf Ratio Required for Out-of-Plane Bending: $\rho_{vt\ req}$ = 0.00188 (per face) $\rho f_y bd$^2*(1-.59$\rho f_y/f'_c$)=Mu/ϕ solve for ρ Ref. 2.2.19, Section 4-3

3.0h) Total Reinforcing required for axial force, in-plane and out of plane bending ρ_v = 0.0038 = ρ_{vl}+ρ_{vt} > 2ρ_{vt}

D/C = ρ_v(reqd) / ρ_v (prov) D/C = 0.69 **Section Adequate** = ρ_v(req'd) / ρ_v(prov)

4.0 Boundary Elements:

h_w / l_w = 0.95 <2, No Need To Check for Boundary Elements

5.0 Out-of-Plane Shear:

Nominal Shear Strength Provided by The Concrete Vc=2*(f'c)$^{1/2}$*b*d Vc = 73.0 kips/ft width of wall

Check Demand / Capacity Ratio: D/C = 0.37 No Shear Reinforcement Required
D/C = (out-of-plane shear Vz) / (0.85*Vc)

6.0 Tabulate Reinforcement Requirements And D/C Ratios:

Use 4 **ft thick wall with** **1-#11@12"c/c EF** **Horizontal Reinforcement**
1-#11@12"c/cEF **Vertical Reinforcement.**

For Shear on Gross Section:	**D/C =**	**0.09**
For In-Plane Shear:	**D/C =**	**0.10**
For Out-of-Plane Shear:	**D/C =**	**0.37**
Bending + axial Loads	**D/C =**	**0.69**

Boundary Elements **D/C=** **BOUNDARY ELEMENTS NOT REQUIRED**

6.6 Design Summary

WALLS DESIGN SUMMARY

Design, From page.	Wall Type	Wall Grid	Wall Segment	Vertical Reinforcement each face.	Horizontal Reinforcement each face.	Out-of-plane shear with tension, Max D/C	In-plane shear with tension, Max D/C
42-54	Transverse	1		# 11 @ 6"	#11@12"	0.32	0.50
68-80	Transverse	2, 2.3, 2.7		# 11 @ 6"	#11@12"	0.27	0.40
55-67	Transverse	3		# 11 @ 6"	#11@12"	0.32	0.53
147-153	Transverse	3.6		# 11 @ 6"	#11@12"	0.16	0.31
154-166	Transverse	5		# 11 @ 6"	#11@12"	0.69	0.41
167-173	Transverse	7		# 11 @ 12"	#11@12"	0.32	0.25
174-180	Transverse	8		# 11 @ 12"	#11@12"	0.37	0.10
35-41	Longitudinal	E,F	1 to 3	# 11 @ 12"	#11@12"	0.63	0.25
133-139	Longitudinal	E	3.6 to 8	# 11 @ 12"	#11@12"	0.27	0.11
140-146	Longitudinal	F	3.6 to 7	# 11 @ 12"	#11@12"	0.31	0.14

SLAB DESIGN SUMMARY

Design, From page.	Slab Type	Top of Slab Elevation	Slab between Grids	North-South Reinforcement each face.	East-West Reinforcement each face.	Out-of-plane shear with tension, Max D/C	In-plane shear with tension, Max D/C
16-34	Roof	60'-0"	1 to 3	# 11 @ 6"	# 11 @ 12"	0.579	0.594
81-95	2nd Floor	28'-1"	3.6 to 5	# 11 @ 8"	#11@8"	0.680	0.516
96-118	2nd Floor	37'-0"	5 to 7	# 11 @ 8"	#11@8"	0.578	0.400
119-132	2nd Floor	37'-0"	7 to 8	# 11 @ 8"	# 11 @ 8"	0.492	0.234

6.7 CONCRETE FLOOR SLAB AT EL 28’-1”

The floor slab is a one way slab, 2’-4” thick spanning N-S direction, separated from main concrete structure that is supported by steel framing with a metal deck; refer to Initial Handling Facility General Arrangement drawing, Ref. 2.2.5. Use #7 @ 12” T & B, E.W., in concrete slab. See Assumption 3.1.6.

6.8 CONCRETE FLOOR SLAB AT EL 26’-9”

The floor slab is a one way slab, 1’ thick spanning N-S direction, separated from main concrete structure that is supported by steel framing with a metal deck; refer to Initial Handling Facility General Arrangement drawing, Ref. 2.2.5. Use #7 @ 12” T & B, E.W., in concrete slab. See Assumption 3.1.6.

7. RESULTS AND CONCLUSIONS

7.1 Results

The concrete structure for the IHF has been designed to meet ACI 349-01(Ref. 2.2.14) requirements. The design summary shown in Section 6.6 indicates that the reinforcement in the 48” thick structural elements satisfy the requirements defined in ACI 349-01. Furthermore, it can be observed that the IHF concrete structural elements have a significant reserve margin available. This reserve capacity will be important in BDBGM seismic fragility evaluations made for the IHF concrete structure.

7.2 Conclusions

Based on the results from Section 7.1, it is concluded that the IHF concrete structure satisfies the ACI Code requirements for DBGM-2 seismic ground motions and 48” thick walls, roof, and floor slabs are satisfactory. Additionally, reserve capacity exists, which is significant when performing a seismic fragility evaluation for BDBGM in the limited probabilistic risk assessment.

ATTACHMENT A

Shield Plug & Port Slide Gate Loads

Shield Plug & Port Slide Gate Loads

Superimposed Dead Loads

π = 3.14159

$D_{FLOOR\ 2}$:

Conc Roof Slab thickness =	4.000 ft
Conc Unit Loads = γ_C =	0.150 kcf
Steel Unit Loads = γ_S =	0.490 kcf

Shield Plug:

...The ***circular*** Shield Plugs are embeded in the top part of the concrete slab.

Number of shield plugs = n_P =	1		
Shield plug dia = D_{sp} =	75.500 in	= 6.292 ft	Ref. 2.2.10 & 2.2.11
Shield plug thickness = t_{sp} =	15.000 in	= 1.250 ft	Ref. 2.2.10 & 2.2.11
Shield plug thickness material =	Steel		

Weight of shield plug less wt of concrete replaced =		$\gamma_S - \gamma_C$	n_P	$\pi / 4$	D_{SP}^2	t_{SP}
	=	0.340 kcf *	1 plugs	* 0.7854 *	39.59 sq-ft	* 1.250 ft
	=	**13.213 kips**				

Slide Gates:

...The ***rectangular*** Slide Gates are embeded in the top part of the concrete slab.

Number of Slide Gates = n_g =	2		
Length of Slide Gate assembly = L_g =	9.667 ft		Ref. 2.2.23
Width of Slide Gate assembly = W_g =	9.667 ft		Ref. 2.2.23
Thickness of Slide Gate assembly = t_g =	13.000 in	= 1.083 ft	Ref. 2.2.23

Weight of slide gate less wt of concrete replaced =		$\gamma_S - \gamma_C$	n_g	L_g	W_g	t_g
	=	0.340 kcf *	2 gates	* 9.667 ft	* 9.667 ft	* 1.083 ft
	=	**68.842 kips**				

Total Second Floor Superimposed Dead Load [Dfloor 2] @ Elev 37'-0" :

Length of roof = L_R =	75.000 ft	Ref 2.2.4 to 2.2.9			
Width of roof = W_R =	41.000 ft	Ref 2.2.4 to 2.2.9			
Total roof area = A_R =	3,075.00 sq-ft				
Roof area occupied by Shield Plug = A_{SP} =		n_P	$\pi / 4$	D_{SP}^2	
	=	1 plugs	* 0.7854 *	39.59 sq-ft	**= 31.09 sq-ft**

Roof area occupied by Slide Gate = A_g =		n_g	$L_{gO} - L_{gI}$	W_g	
	=	2 gates	* 9.667 ft	* 9.667 ft	**= 186.90 sq-ft**

Local area occupied by steel = A_{SP} + A_g = A_L =	**217.99 sq-ft**

SDL from Shield Plug =	13.213 kips
SDL from Slide Gate =	68.842 kips

Summary, $D_{FLOOR\ 2}$:

Conservatively assume that,

Plus one Shield Plug conc load =	**14.000 kips**	This is applied at TOC EL 37'-0".
And each of two Slide Gates conc loads =	**35.000 kips**	This is applied at TOC EL 37'-0".

ATTACHMENT B

SAP2000 Model Sketches

Page

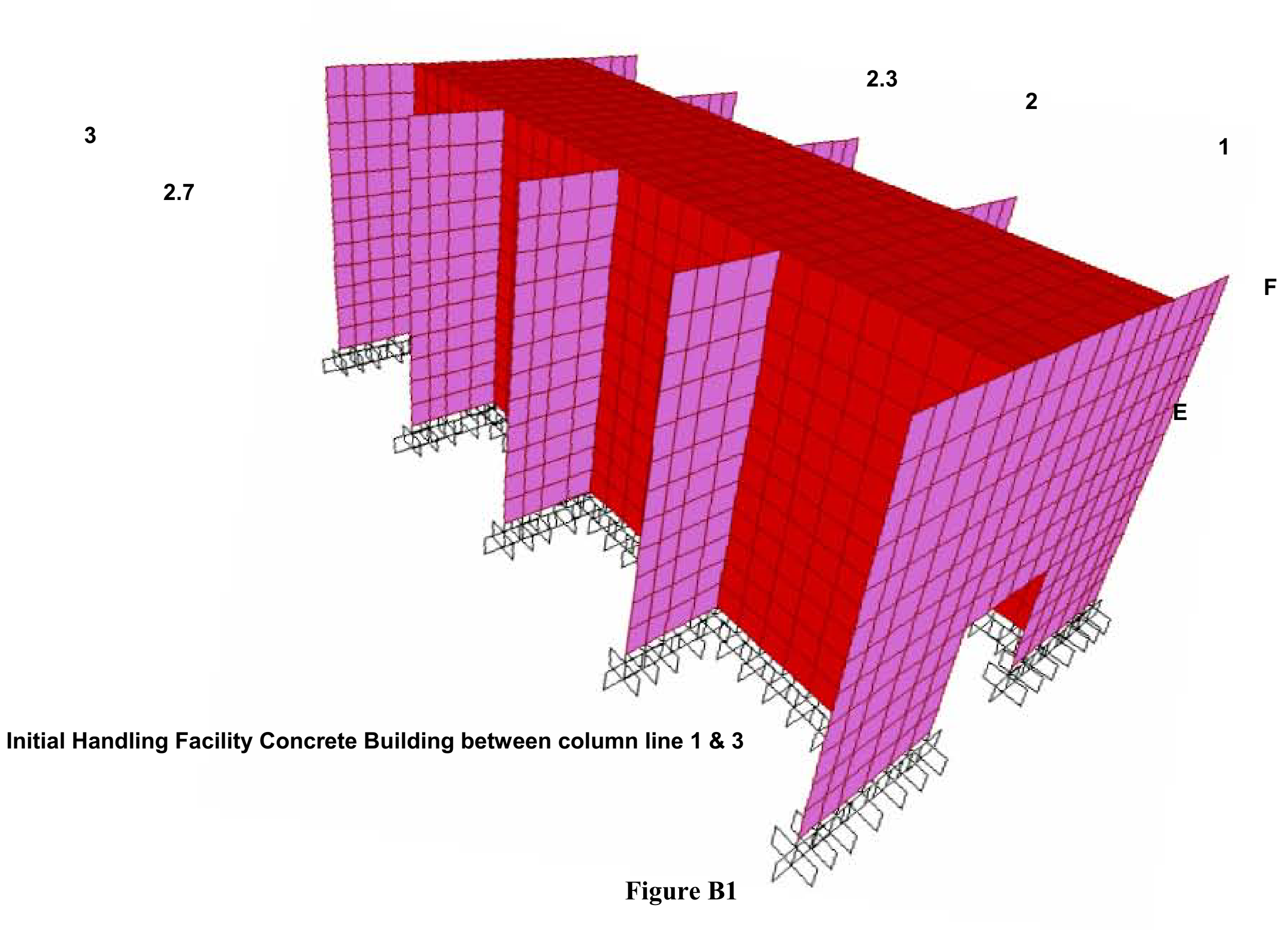
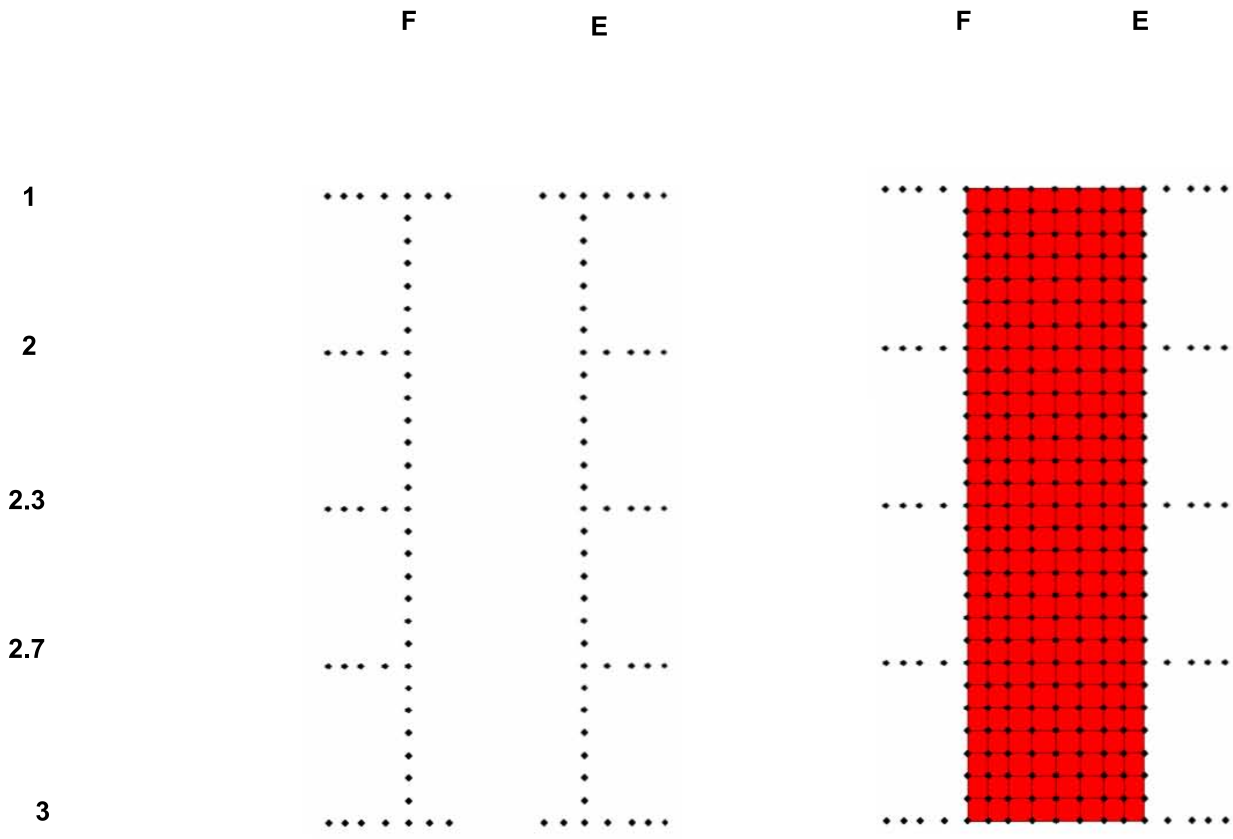

2.3
2
3
1
2.7
F
E
Initial Handling Facility Concrete Building between column line 1 & 3

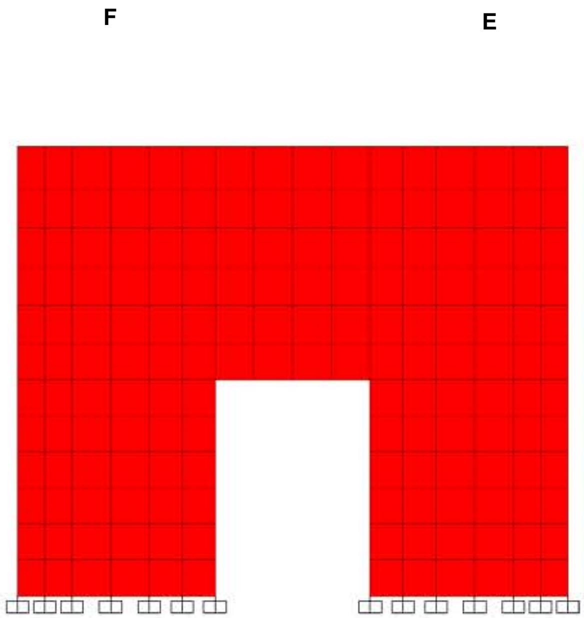
Figure B1



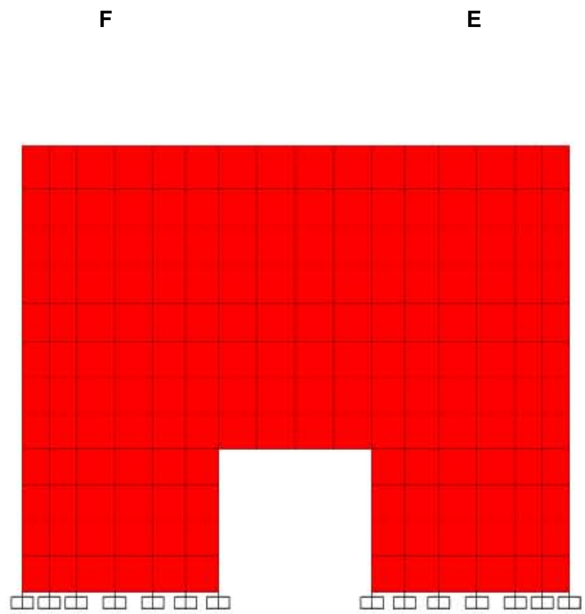
Plan at EL 0'-0"

Plan at EL 60'-0"

Figure B2

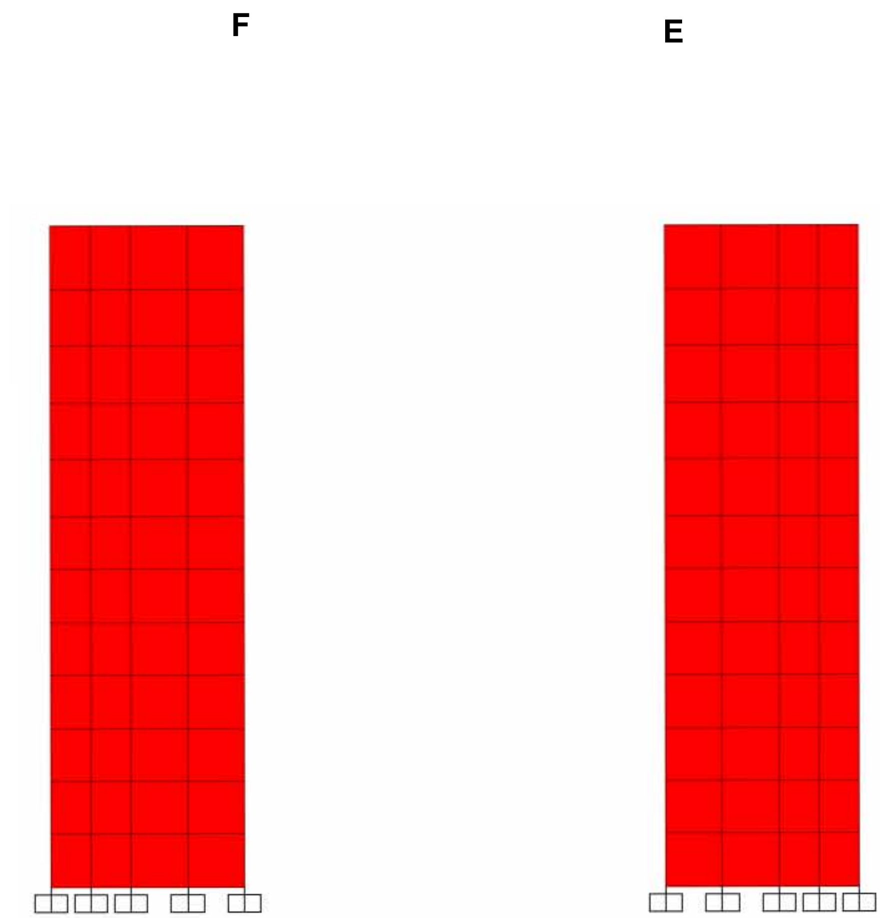


Elevation at column line 3



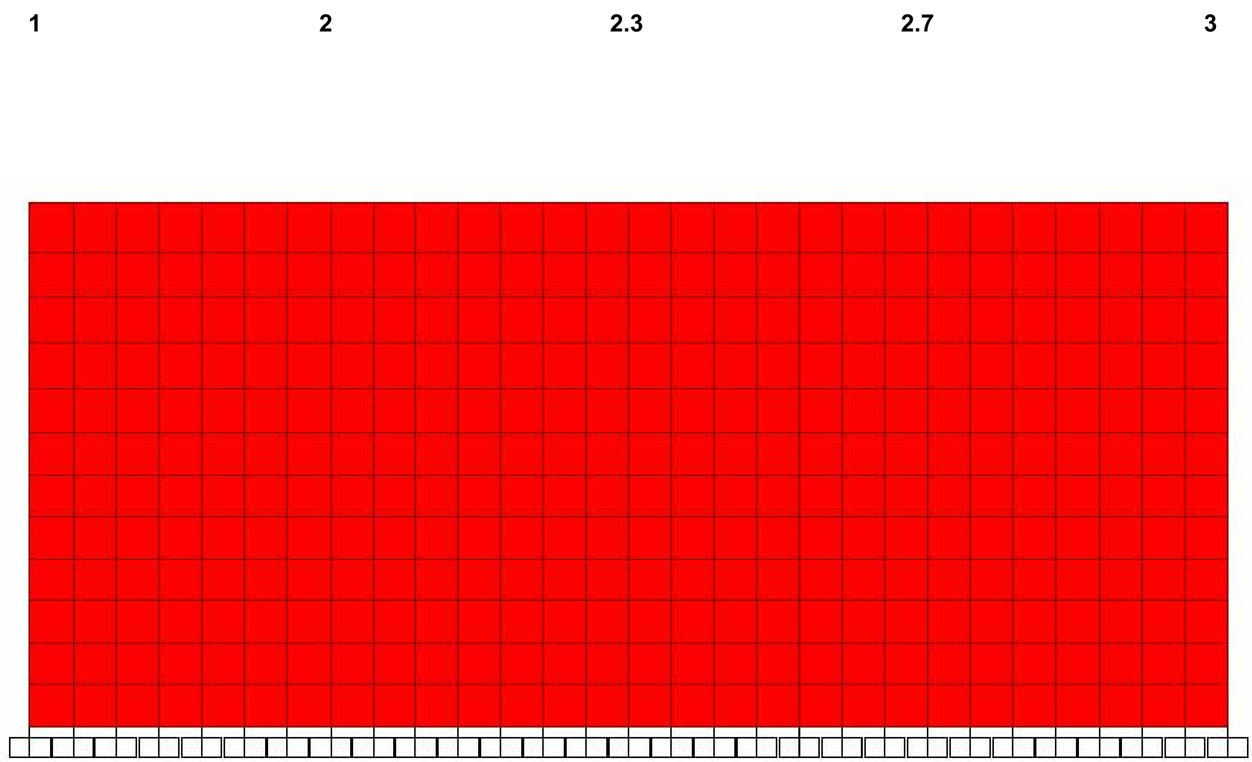
Elevation at column line 1

Figure B3



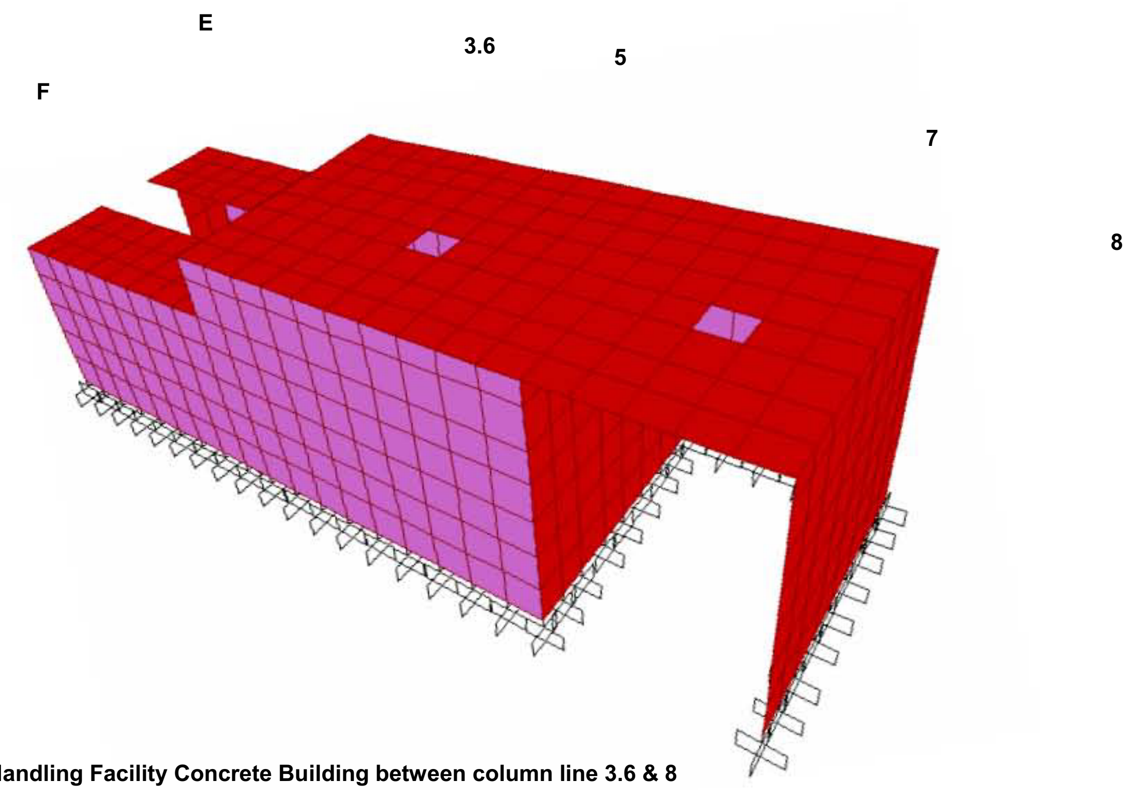
Elevation at column line 2, 2.3, & 2.7

Figure B4



Elevation at along column line E & F

Figure B5



Initial Handling Facility Concrete Building between column line 3.6 & 8

Figure B6

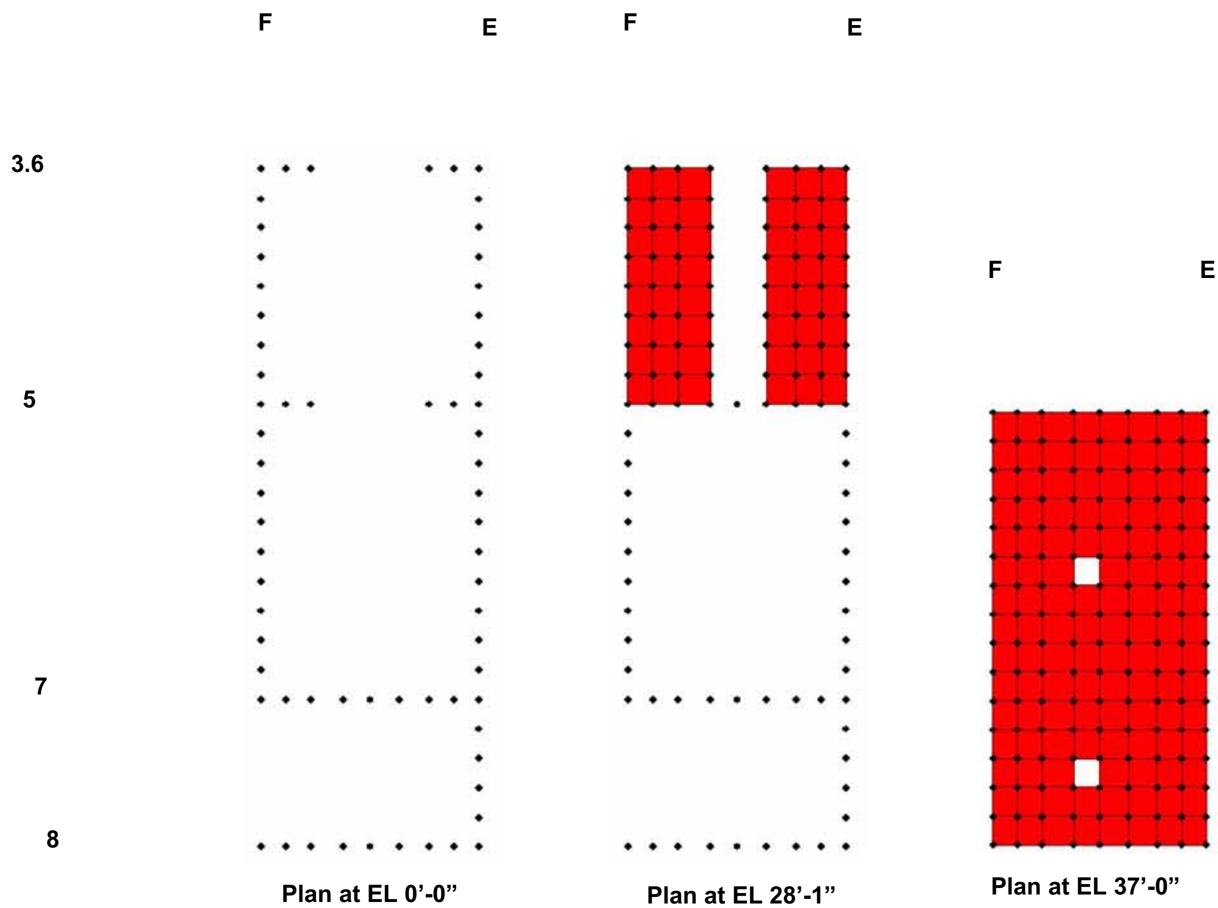
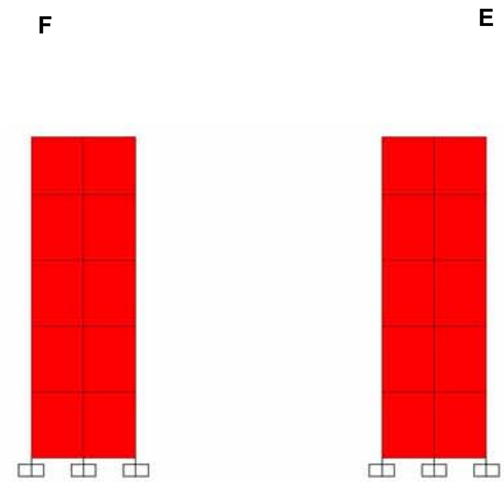
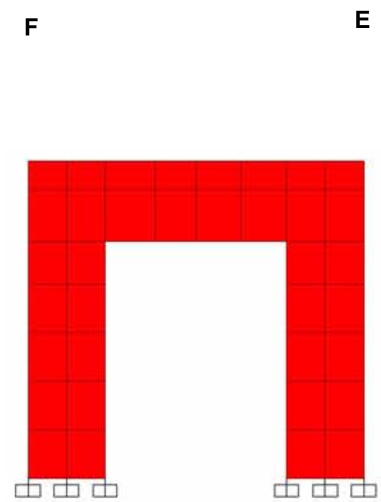


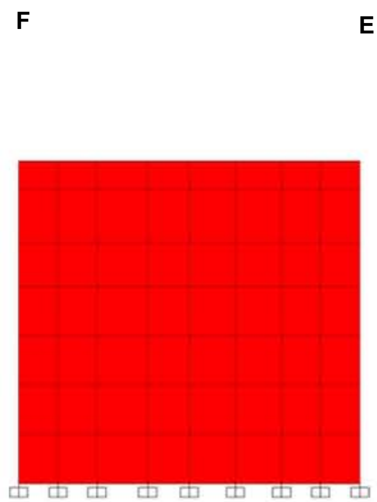
Figure B7



Elevation at column line 3.6

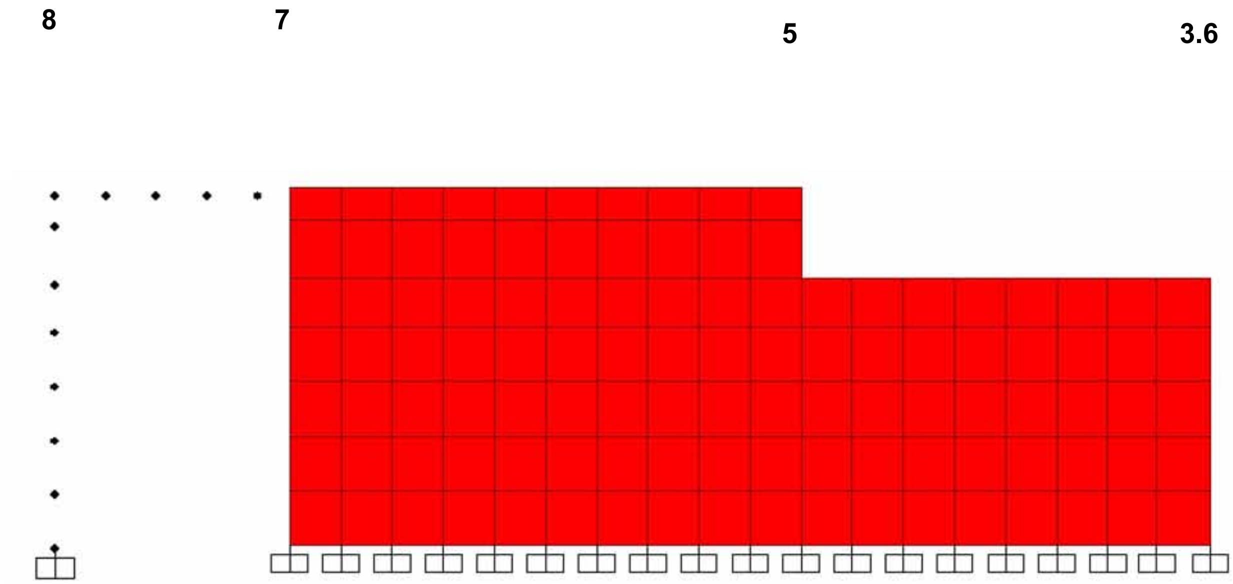


Elevation at column line 5

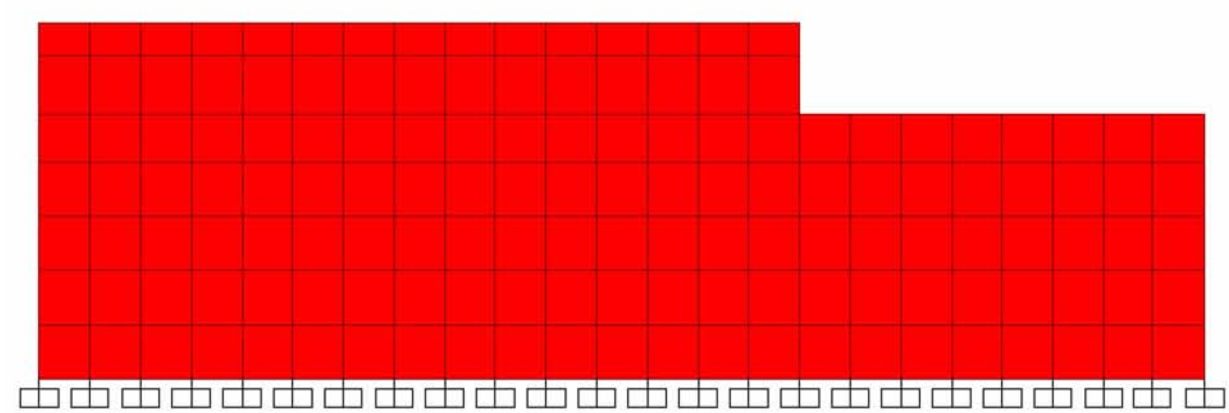


Elevation at column line 7 & 8

Figure B8



Elevation at along column line F



Elevation at along column line E

Figure B9

ATTACHMENT C

SAP2000 Analysis Model of IHF Concrete Structure

CD

A Attachment C contains the SAP2000 models for the IHF concrete structure between COL lines 1 through 3 and COL lines 3.6 to 8. The exterior structure is within COL lines 1 and 3; the interior structure is within COL lines 3.6 and 8.

- Two directories exist in Attachment C: Exterior Structure Model COL 1-3 and Interior Structure Model COL 3.6-8.
    - There is a sub-folder inside each directory above; Exterior Structure Model COL 1-3 contains two sub-folders: Crane Position 1 and Crane Position 2. Interior Structure Model COL 3.6-8 contains the corresponding SAP2000 model and input file.
        - The sub-folders Crane Position 1 and Crane Position 2 contain their corresponding SAP2000 models and input files.

ATTACHMENT D

SAP2000 Output Forces & Moments and Results

CD

Attachment D contains the SAP2000 design output forces and moments for the IHF concrete structure between column lines 1 through 3 and column lines 3.6 to 8, as well as the results from the SAP2000 model.

- Three directories exist in Attachment D: Output Loads for column 1-3, Output Loads for column 3.6-8, and SAP2000 Results.
    - Inside the Output Loads directory exists the appropriate design load output excel files for longitudinal walls, transverse walls, buttress wall, and slabs in the IHF concrete structure.
    - The SAP2000 Results directory contains three excel files which contain the results of the three models used in this calculation: Model 1-3 Crane Position 1 Results, Model 1-3 Crane Position 2 Results, and Model 3.6-8 Results.

ATTACHMENT E

Foundation Basemat Input & Output Forces

CD

Attachment E contains the foundation basemat input and output forces from the IHF concrete structure between column lines 1 through 3 and column lines 3.6 to 8.

- Two directories exist in Attachment E: Large Basemat for Interior Structure and Small Basemat for Exterior Structure.
    - Inside each directory exist the corresponding SAP2000 file, which was used to output the basemat forces for the IHF foundation, and an excel output of the joint and basemat reactions.