# Table of Contents

# SITE SPECIFIC DESIGN PARAMETERS

# Design Maps Summary Report

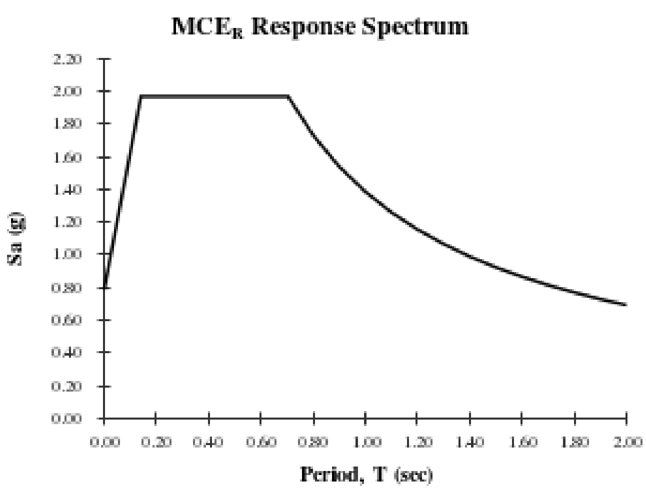
## User–Specified Input

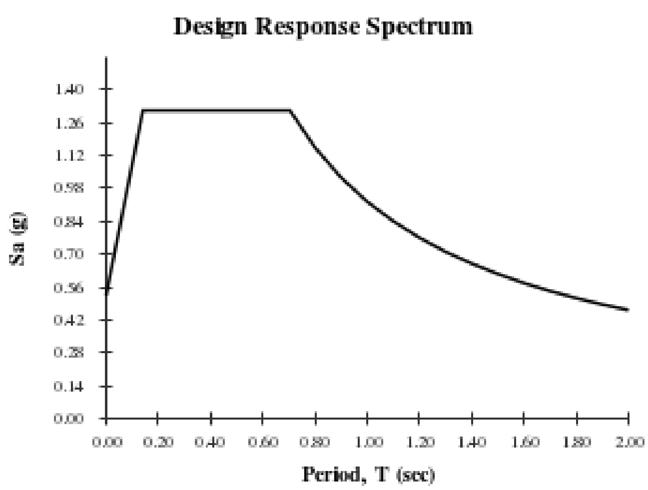
| | |
|---|---|
| **Report Title** | 816 Taraval Street, San Francisco, California<br>Wed September 12, 2018 15:32:44 UTC |
| **Building Code Reference Document** | 2012/2015 International Building Code<br>(which utilizes USGS hazard data available in 2008) |
| **Site Coordinates** | 37.74316°N, 122.47482°W |
| **Site Soil Classification** | Site Class D – "Stiff Soil" |
| **Risk Category** | I/II/III |


## USGS–Provided Output

| | | |
|---|---|---|
| $S_S =$ 1.967 g | $S_{MS} =$ 1.967 g | $S_{DS} =$ 1.312 g |
| $S_1 =$ 0.923 g | $S_{M1} =$ 1.385 g | $S_{D1} =$ 0.923 g |

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.



**SEISMIC DESIGN CATEGORY - MAPPED ACCELERATION VALUES**
**ASCE 7-10 SECTION 11.4 - SEISMIC GROUND MOTION VALUES**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**1. Spectral Response Accelerations (USGS Website)**

Longitude: -122.47482° W
Latitude: 37.74316° N

| | Period (secs) | Sa (g's) |
|---|---|---|
| $S_s$ | 0.20 | 1.967 |
| $S_1$ | 1.00 | 0.923 |

**2. Site Class and Occupancy Category (ASCE 11.4)**

i) Site Class — Site Class = D

Note: Where soil properties are not known in sufficient detail to determine Site Class, Site Class D shall be assumed.

ii) Occupancy Category

Occupancy Category = II (ASCE 7-10 Table 1.5-1)

I = 1.00 Importance Factor (Table 11.5-2)

| Table 11.5-2 | |
|---|---|
| Occupancy Category | I |
| I or II | 1.00 |
| III | 1.25 |
| IV | 1.50 |

**3. Approximate Fundamental Period (Section 12.8.2)**

$h_n$ = **45.0** feet (Building Height)

| | N-S Direction | W-E Direction |
|---|---|---|
| LFRS | TMBR SW | TMBR SW |
| $C_t$ | 0.020 | 0.020 |
| x | 0.75 | 0.75 |

| ASCE 7-10 Table 12.8-2 | | |
|---|---|---|
| **System** | **$C_t$** | **x** |
| Steel MRF | 0.028 | 0.80 |
| Concrete MRF | 0.016 | 0.90 |
| EBF | 0.030 | 0.75 |
| All other systems | 0.020 | 0.75 |

$T_a = C_t \; h^x_n$ (12.8-7)

Thus, $T_a$ = 0.35 seconds (N-S Direction)
= 0.35 seconds (W-E Direction)

**4. Design Spectral Acceleration Parameters (ASCE 11.4)**

**a) At Short Periods**

$S_{DS} = 0.67\, S_{ms} = 0.67\, F_a\, S_s$

$S_s$ = 1.967 g's (from USGS website)

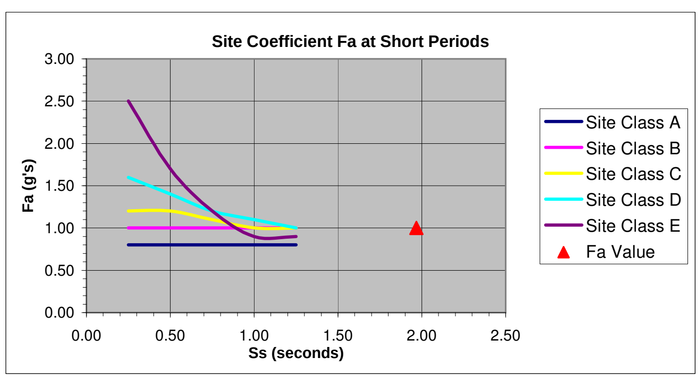
$Fa = y_1+((y_2-y_1)/(x_2-x_1))*(S_1-x_1)$

$x_1$ = 1.00 secs   $y_1$ = 1.00
$x_2$ = 1.25 secs   $y_2$ = 1.00

**$F_a$ = 1.000 (Interpolated Site Coefficient Value - Table 11.4-1)**

| | Table 11.4-1 SITE COEFFICIENT, Fa | | | | |
|---|---|---|---|---|---|
| | Mapped Maximum Considered Earthquake Spectral Response Acceleration parameter at Short Period ($S_s$) | | | | |
| Site Class | **0.25** | **0.50** | **0.75** | **1.00** | **1.25** |
| A | 0.80 | 0.80 | 0.80 | 0.80 | 0.80 |
| B | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| C | 1.20 | 1.20 | 1.10 | 1.00 | 1.00 |
| D | 1.60 | 1.40 | 1.20 | 1.10 | 1.00 |
| E | 2.50 | 1.70 | 1.20 | 0.90 | 0.90 |
| F | See Section 11.4.7 | | | | |
| **$F_a$ Values** | 1.60 | 1.40 | 1.20 | 1.10 | 1.00 |

**$S_{DS}$ = 1.311 g's (Site Design Coefficient - Short Period)**



**SEISMIC DESIGN CATEGORY - MAPPED ACCELERATION VALUES**
**ASCE 7-10 SECTION 11.4 - SEISMIC GROUND MOTION VALUES**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**5. Design Spectral Acceleration Parameters - Continued**

**b) At T = 1.0 Seconds**

$S_{D1}$ = 0.67 $S_{m1}$ = 0.67 $F_v S_1$

$S_1$ = 0.923 g's (from USGS website)

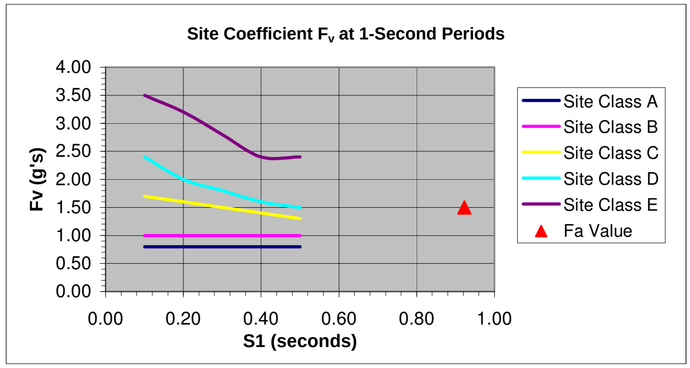
Fv = $y_1+((y_2-y_1)/(x_2-x_1))*(S_1-x_1)$

$x_1$ = 0.40 secs $y_1$ = 1.50
$x_2$ = 0.50 secs $y_2$ = 1.50

**$F_v$ = 1.500 (Interpolated Site Coefficient Value - Table 11.4-2)**

**$S_{D1}$ = 0.923 g's (Site Design Coefficient - at 1-Second Period)**

| | **Table 11.4-2 SITE COEFFICIENT, $F_v$** | | | | |
|---|---|---|---|---|---|
| | Mapped Maximum Considered Earthquake Spectral Response Acceleration parameter at 1-sec Period ($S_1$) | | | | |
| Site Class | **0.10** | **0.20** | **0.30** | **0.40** | **0.50** |
| A | 0.80 | 0.80 | 0.80 | 0.80 | 0.80 |
| B | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| C | 1.70 | 1.60 | 1.50 | 1.40 | 1.30 |
| D | 2.40 | 2.00 | 1.80 | 1.60 | 1.50 |
| E | 3.50 | 3.20 | 2.80 | 2.40 | 2.40 |
| F | See Section 11.4.7 | | | | |
| **$F_v$ Values** | 2.40 | 2.00 | 1.80 | 1.60 | 1.50 |




**6. Seismic Design Category (ASCE 11.6)**

Occupancy Category = II

$S_1$ = 0.923 g's (from USGS website)

| Occupancy Category | $S_1$ < 0.75 g's | $S_1$ >= 0.75 g's |
|---|---|---|
| I | Per Tables 11-6-1 and 11-6-2 | E |
| II | | E |
| III | | E |
| IV | | F |

**a) At Short Periods**

$S_{DS}$ = 1.311 g's (Site Design Coefficient - Short Period)

**SDC = D (Seismic Design Category, per Table 11.6-1)**

| **Table 11.6-1** Seismic Design Category Based on Short Period Response Acceleration Parameter | | | |
|---|---|---|---|
| **Value of $S_{DS}$** | **Occupancy Category** | | |
| | **I or II** | **III** | **IV** |
| $S_{DS}$ < 0.167 | A | A | A |
| 0.167 <= $S_{DS}$ <=0.33 | B | B | C |
| 0.33 <= $S_{DS}$ <=0.50 | C | C | D |
| 0.50 <= $S_{DS}$ | D | D | D |

$T_s = S_{D1}/S_{DS}$ Where $S_{D1}$ = 0.923 g's
$S_{DS}$ = 1.311 g's

$T_s$ = 0.70

Conditions to use Table 11.6-1 **Only** (Section 11.6):

| | Yes | No |
|---|---|---|
| 1. $S_1$ < 0.75 seconds | | x |
| 2. In both orthogonal directions, 0.80 $T_s$ = 0.80 $S_{D1}/S_{DS}$ ≥ $T_a$ | x | |
| 3. In both orthogonal directions, T ≤ $T_s$ | x | |
| 4. Equation $C_s = S_{DS}$ I/R (12.8-2) is used. | x | |
| 5. Diaphragms are rigid per 12.3.1, or if flexible span ≤ 40 ft. | x | |

(Note: 0.80 Ts = 0.80*Sd1/Sds = 0.56 secs ≥ Ta = 0.35 (N-S), ≥ Ta = 0.35 (W-E ))

(Note: T = Ta per 12.8.2)

**Need to use Table 11.6-2 too!**

**b) At T = 1.0 Seconds**

$S_{D1}$ = 0.923 g's (Site Design Coefficient - at 1-Second Period)

**SDC = D (Seismic Design Category, per Table 11.6-2)**

| **Table 11.6-2** Seismic Design Category Based on 1-Second Period Response Acceleration Parameter | | | |
|---|---|---|---|
| **Value of $S_{D1}$** | **Occupancy Category** | | |
| | **I or II** | **III** | **IV** |
| $S_{D1}$ < 0.067 | A | A | A |
| 0.067 <= $S_{D1}$ <=0.133 | B | B | C |
| 0.133 <= $S_{D1}$ <=0.20 | C | C | D |
| 0.20 <= $S_{D1}$ | D | D | D |

**c) Seismic Design Category - Governing**

**SDC = E (Seismic Design Category)**

**BASE SHEAR AND VERTICAL FORCE DISTRIBUTION**
**ASCE 7-10 CHAPTER 12 - SEISMIC REQUIREMENTS FOR BUILDING STRUCTURES**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**1. Seismic Parameter Data**

Occupancy Category = II (ASCE 7-10 Table 1.5-1)

**SDC = E (Seismic Design Category)**

$S_{DS}$ = 1.311 g's (Site Design Coefficient - Short Period)
$S_{D1}$ = 0.923 g's (Site Design Coefficient - at 1-Second Period)
$S_1$ = 0.923 g's (from USGS website)

| | Loading Direction | | |
|---|---|---|---|
| | N-S | E-W | |
| LFRS | TMBR SW | TMBR SW | |
| $T_a$ | 0.35 | 0.35 | seconds (Approximate Fundamental Period) |
| R | 6.5 | 6.5 | Response Modification Factor (Table 12.2-1) |

$T_L$ = 8.00 Seconds (Long Period Transition Period from Figure 22-15)

I = 1.00 Importance Factor (Table 11.5-2)

**2. Determination of Seismic Response Coefficient, Cs (Section 12.8.1.1)**

$C_s = S_{DS}\ I / R$ (12.8-2)

Where $S_{DS}$ = 1.311 g's (Site Design Coefficient - Short Period)
I = 1.00 Importance Factor
R = 6.5 N-S Response Modification Factor
= 6.5 W-E

= $C_s$ = 0.202 (N-S Direction)
= 0.202 (W-E Direction)

Max $C_s$ values:

$$C_S = \frac{S_{D1}\ I}{T\ R} \quad \text{for } T \le T_L \qquad (12.8-3)$$

$$C_S = \frac{S_{D1}\ T_L\ I}{T^2\ R} \quad \text{for } T > T_L \qquad (12.8-4)$$

Where $S_{D1}$ = 0.923 g's (Site Design Coefficient - Short Period)
I = 1.00 Importance Factor
R = 6.5 Response Modification Factor
= 6.5 W-E
$T = T_a$ = 0.35 seconds (Approximate Fundamental Period - N-S direction)
= 0.35 seconds ( " - W-E Direction)
$T_L$ = 8.00 Seconds (Long Period Transition Period from Figure 22-15)

= $C_{s\,max}$ = 0.409 (N-S Max Cs Value)
= 0.409 (W-E Max Cs Value)

Min Cs values:

$C_s$ = 0.044 $S_{DS}$ I > 0.01 (12.8-5)
= 0.058

Where $S_{DS}$ = 1.311 g's (Site Design Coefficient - Short Period)
I = 1.00 Importance Factor

$$C_S = \frac{0.5 S_1\ I}{R} \quad \text{for } S_1 \ge 0.6 \text{ g's} \qquad (12.8-6)$$

= 0.071

Where $S_1$ = 0.923 g's (from USGS website)
I = 1.00 Importance Factor
R = 6.5 Response Modification Factor
= 6.5 W-E

$C_s$ = 0.071 (N-S Min Cs Value)
= 0.071 (W-E Min Cs Value)

Seismic Coefficient Cs - Governing Value:

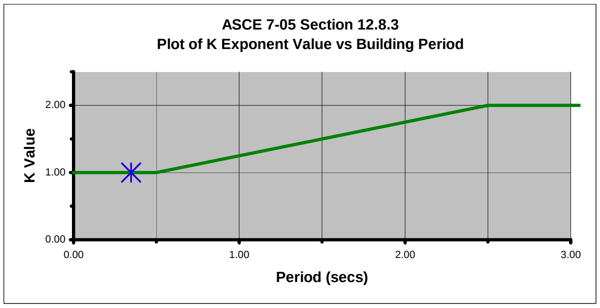
**$C_s$ = 0.202 g's (N-S Direction)**
**= 0.202 g's (W-E Direction)**

**BASE SHEAR AND VERTICAL FORCE DISTRIBUTION**
**ASCE 7-10 CHAPTER 12 - SEISMIC REQUIREMENTS FOR BUILDING STRUCTURES**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**3. Determination of Base Shear, V (Section 12.8)**

$V = C_s W$ (12.8-1) Where $C_s$ = 0.202 g's (N-S Direction)
= 0.202 g's (W-E Direction)
W = 628 kips (Building Weight)

| | | |
|---|---|---|
| **V =** | **127** | **kips (N-S Direction)** |
| **=** | **127** | **kips (W-E Direction)** |




**4. Vertical Distribution of Seismic Forces (Section 12.8.3)**

$$F_x = C_{vx} V \qquad (12.8-11)$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad (12.8-12)$$

Where K = 1.00 N-S for T = $T_a$ = 0.35 seconds
= 1.00 W-E for T = Ta = 0.35 seconds
$W_x$ = See Below
$h_x$ = See Below

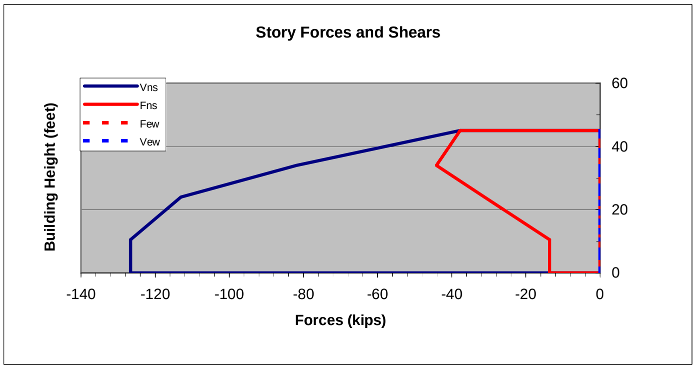
| Level | Story Weight, $W_x$ (kips) | Height, $h_x$ (feet) | N-S and E-W Directions: $W_x H^k_x$ | Lateral Force, $F_x$ (kips) | Story Shear, $V_x$ (kips) | $V_x * h_x$ (kip-ft) | Overturning Moment, $M_{ot}$ (kip-ft) | $W_x H^k_x$ | Lateral Force, $F_x$ (kips) | Story Shear, $V_x$ (kips) | $V_x * h_x$ (kip-ft) | Overturning Moment, $M_{ot}$ (kip-ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Roof | 111 | 45.0 | 5,004 | 38 | 38 | 1,698 | 1,698 | | | | | |
| 3 | 172 | 34.0 | 5,855 | 44 | 82 | 2,783 | 4,481 | | | | | |
| 2 | 172 | 24.0 | 4,133 | 31 | 113 | 2,713 | 7,194 | | | | | |
| 1 | 172 | 10.5 | 1,808 | 14 | 127 | 1,330 | 8,524 | | | | | |

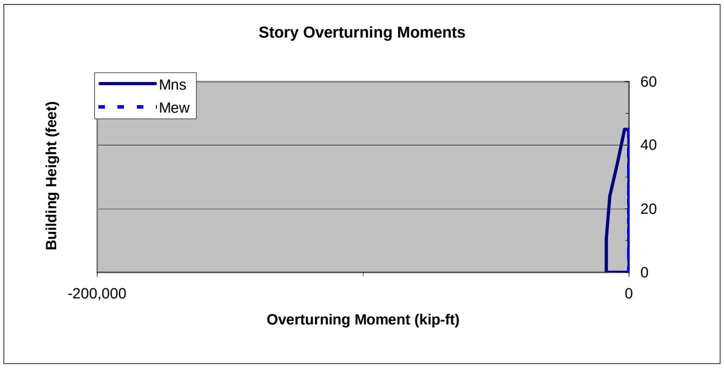
$\sum_{i=1}^{n} w_i h_i^k$ = 16,800 $\qquad \sum_{i=1}^{n} w_i h_i^k$ = 0

Σ Weight = 628 kips

V = 127 kips
$M_{ot}$ = 8,524 kip-ft

V = kips
$M_{ot}$ =



**BASE SHEAR AND VERTICAL FORCE DISTRIBUTION**
**ASCE 7-10 CHAPTER 12 - SEISMIC REQUIREMENTS FOR BUILDING STRUCTURES**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**5. Determination of Diaphragm Forces, $F_{px}$ (Section 12.10.1.1)**

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} W_i} W_{px} \qquad (12.10-1)$$

Where $W_i$ = Weight tributary to level i
$W_{px}$ = Weight tributary to the diaphragm at Level x

Minimum Value: $F_{px} = 0.20\, S_{DS}\, I\, W_{px}$ $F_{px}$ = 0.2623 $W_{px}$

Where $S_{DS}$ = 1.311 g's (Site Design Coefficient - Short Period)
I = 1.00 Importance Factor

Maximum Value: $F_{px} = 0.40\, S_{DS}\, I\, W_{px}$ $F_{px}$ = 0.5245 $W_{px}$

| | | | N-S and E-W Directions | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Level** | **$W_{px} = W_x$ (kips)** | **Height, $h_x$ (feet)** | **$\Sigma W_x$ (kips)** | **$F_x$ (kips)** | **$\Sigma F_x$ (kips)** | **$\Sigma F_x / \Sigma W_x$** | **$F_{px}$ (kips)** | **$\Sigma W_x$ (kips)** | **$F_x$ (kips)** | **$\Sigma F_x$ (kips)** | **$\Sigma F_x / \Sigma W_x$** | **$F_{px}$ (kips)** |
| Roof | 111.2 | 45.00 | 111 | 38 | 38 | 0.3393 | 38 | | | | | |
| 3 | 172.2 | 34.00 | 283 | 44 | 82 | 0.2889 | 50 | | | | | |
| 2 | 172.2 | 24.00 | 456 | 31 | 113 | 0.2623 | 45 | | | | | |
| 1 | 172.2 | 10.50 | 628 | 14 | 127 | 0.2623 | 45 | | | | | |

**DETERMINATION OF CODE LEVEL FORCES AT UPPER FLOOR LEVELS**
**ASCE 7-10 CHAPTER 12 - SEISMIC REQUIREMENTS FOR BUILDING STRUCTURES**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

Note : Strength Level forces copied from ASCE 7-10 Determination of Code Level Forces Spreadsheet.

for $C_s$ = 0.202 g's

and determined as follows:

$A_F$ = Floor Acceleration at Floor Level x resulting from Seismic Coefficient $C_s$ determined

= $\rho\ W_x / F_x$ for $\rho$ = 1.30 Redundancy Factor (ASCE 7-10 Section 12.3.4)

$W_x$ = Story Weight from table below;

$F_x$ = Lateral Force from table below;

| | | | N-S and E-W Directions | | | | | Floor Acceleration, $A_F$ (g's) | | |
|---|---|---|---|---|---|---|---|---|---|---|
| **Level** | **Story Weight, $W_x$ (kips)** | **Height, $h_x$ (feet)** | **$W_x H^k_x$** | **Lateral Force, $F_x$ (kips)** | **Story Shear, $V_x$ (kips)** | **$V_x * h_x$ (kip-ft)** | **Overturning Moment, $M_{ot}$ (kip-ft)** | **Strength Level** | **Adjusted for Redundancy Factor, ρ** | **A4 Service Level** |
| Roof | 111 | 45.00 | 5,004 | 38 | 38 | 1,698 | 1,698 | 0.34 | 0.44 | 0.24 |
| 3 | 172 | 34.00 | 5,855 | 44 | 82 | 2,783 | 4,481 | 0.26 | 0.33 | 0.18 |
| 2 | 172 | 24.00 | 4,133 | 31 | 113 | 2,713 | 7,194 | 0.18 | 0.24 | 0.13 |
| 1 | 172 | 10.50 | 1,808 | 14 | 127 | 1,330 | 8,524 | 0.08 | 0.10 | 0.06 |

$\sum_{i=1}^{n} w_i\, h_i^k =$ 0

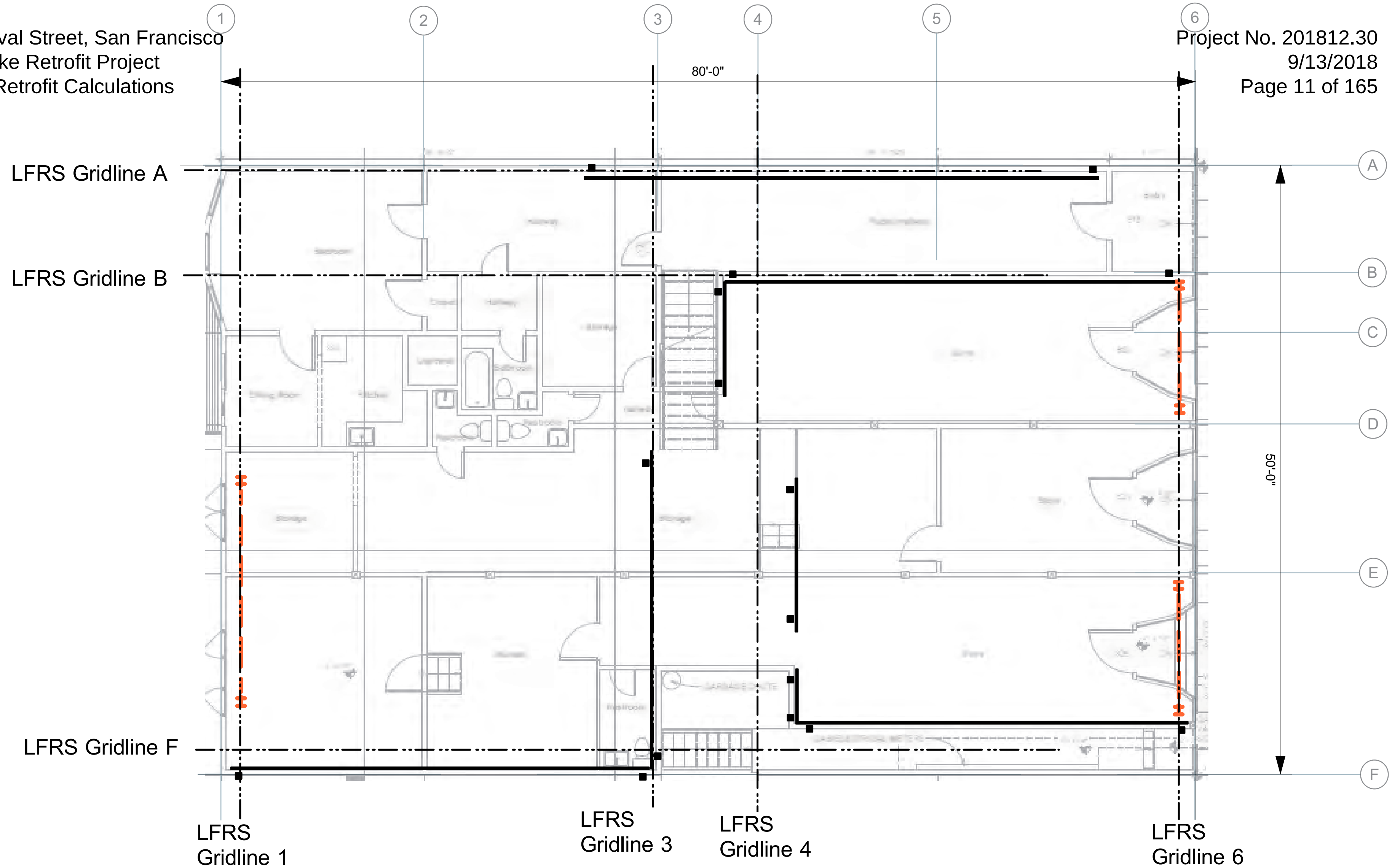
$\sum_{i=1}^{n} w_i\, h_i^k$ =

Weight = 628 kips

V = 127 kips

$M_{ot}$ = 8,524 kip-ft

# FLEXIBLE DIAPHRAGM ANALYSIS

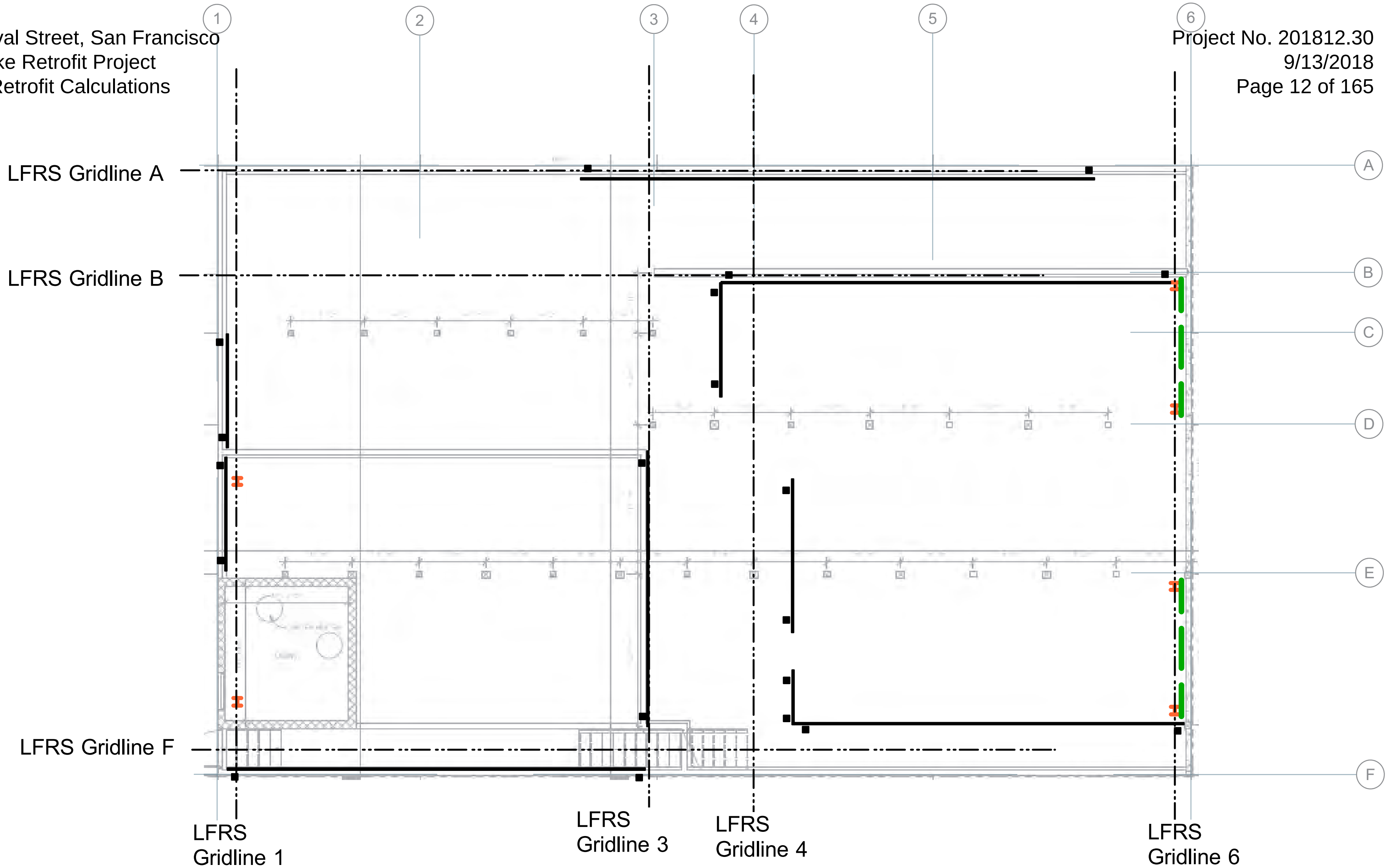



# (E) LEVEL 1 PLAN - SHEET A2
NTS

Lateral Force Resisting System (LFRS) Elements :

(N) Shear Wall

Simpson Strong Frame




(E) BASEMENT PLAN - SHEET A1
NTS

Lateral Force Resisting System (LFRS) Elements :

(N) Shear Wall

Simpson Strong Frame

Diaphragm Supported Edges
at Retaining Wall

**DETERMINATION OF DIAPHRAGM SELF WEIGHT**
**SEISMIC LOADS**
**816 TAVARAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**1. Roof Dead Loads**

| | | | |
|---|---|---|---|
| Roofing (Built-up) : | | 6.0 | psf |
| 2x Straight Sheathing (3 psf x 1 1/2) : | | 4.5 | psf |
| Roof Framing (2 x 10 @ 16"oc) : | | 2.6 | psf |
| Insulation (Loose): | | 1.0 | psf |
| 1/2" Plaster over Wood Lath (10.0 psf x 1/2) | | 5.0 | psf |
| Miscellaneous (mechanical, etc) | | 1.0 | psf |
| | RDL Totals : | 20.1 | psf |
| | **RDL =** | 21.0 | **psf** |

**2. Floor Dead Loads**

| | | | |
|---|---|---|---|
| Hardwood Floors : | | 4.5 | psf |
| 2x Straight Sheathing (3 psf x 1 1/2) : | | 4.5 | psf |
| Framing (2 x 10 @ 16"oc) : | | 2.6 | psf |
| Insulation (Loose): | | 0.5 | psf |
| 1/2" Plaster over Wood Lath (10.0 psf x 1/2) | | 5.0 | psf |
| Miscellaneous (mechanical, plumbing, etc) | | 3.0 | psf |
| | **Sub-Total :** | **20.1** | **psf** |
| Partition Loads : | | 10.0 | **psf** |
| | **FDL Totals :** | **30.1** | **psf** |
| | **FDL =** | **31.0** | **psf** |

**3. Metal Grating Firescape Exit Balconies**

| | | | |
|---|---|---|---|
| 2 1/2 x 5/16 19-W-4 Heavy Duty Welded Metal Grating | | 25.0 | psf |
| Misc Steel Framing + Railings | | 5.0 | psf |
| | **FSDL =** | **30.0** | **psf** |

**DETERMINATION OF WALL WEIGHTS**
**SEISMIC LOADS**
**816 TAVARAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 1. Interior Wall Loads

Note: Interior wall loads are assumed as 10 psf partition load over floor areas.

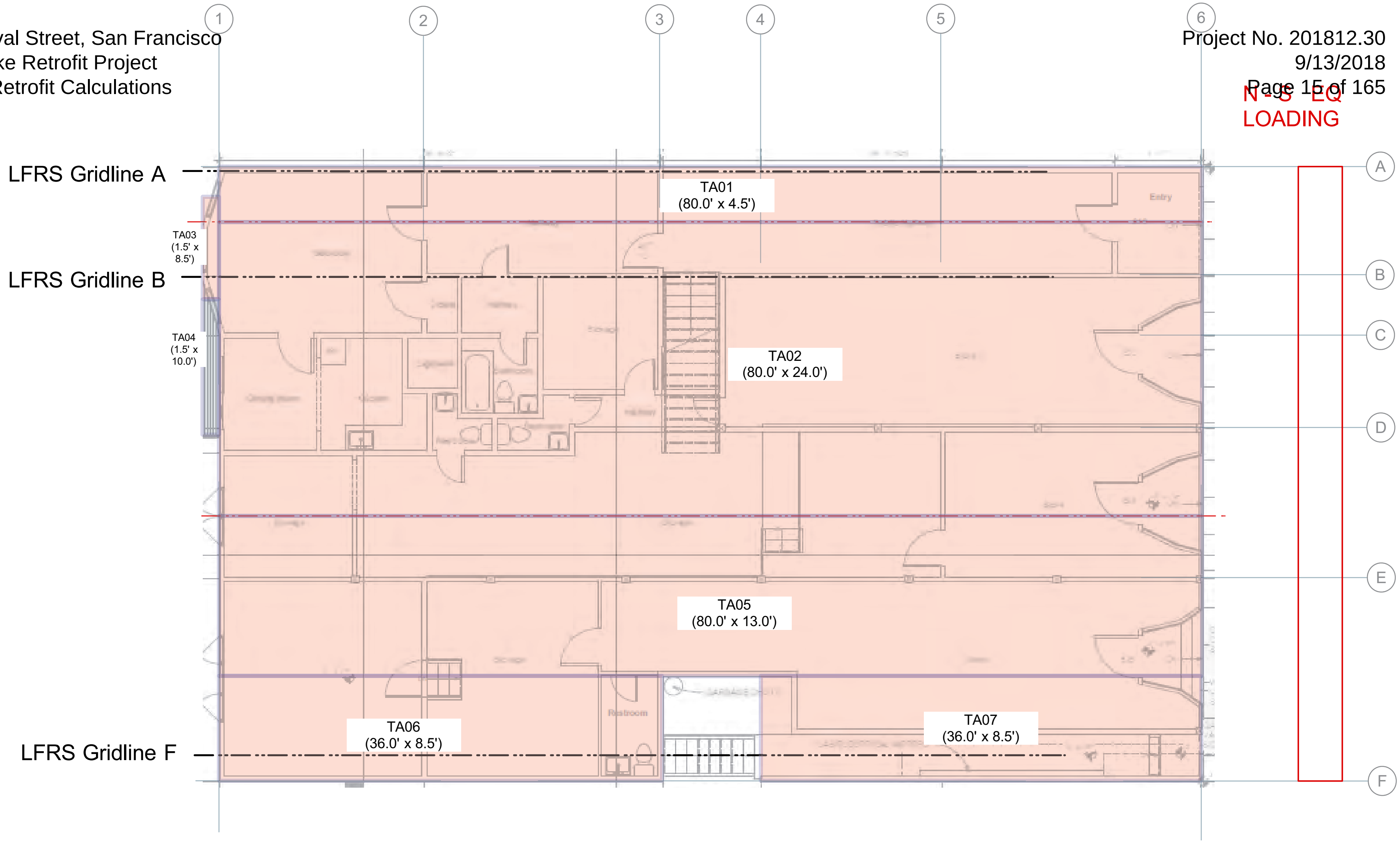
## 2. Exterior Wall Loads

a) Exterior Wall facing Street (North)

| | | |
|---|---|---|
| 1" Stucco (142 pcf, from PCA website) | 11.8 | psf |
| 1-1x Straight Sheathing Layers (3 psf x 1.0") : | 3.0 | psf |
| 2 x 6 studs @ 16"oc (Plus plates, blocking, etc) | 2.5 | psf |
| 1/2" Plaster over Wood Lath (10.0 psf x 1/2) | 5.0 | psf |
| Insulation, Loose | 1.0 | psf |
| **Total wall loads:** | **23.4** | **psf** |
| **$WDL_a$ =** | **24.0** | **psf** |

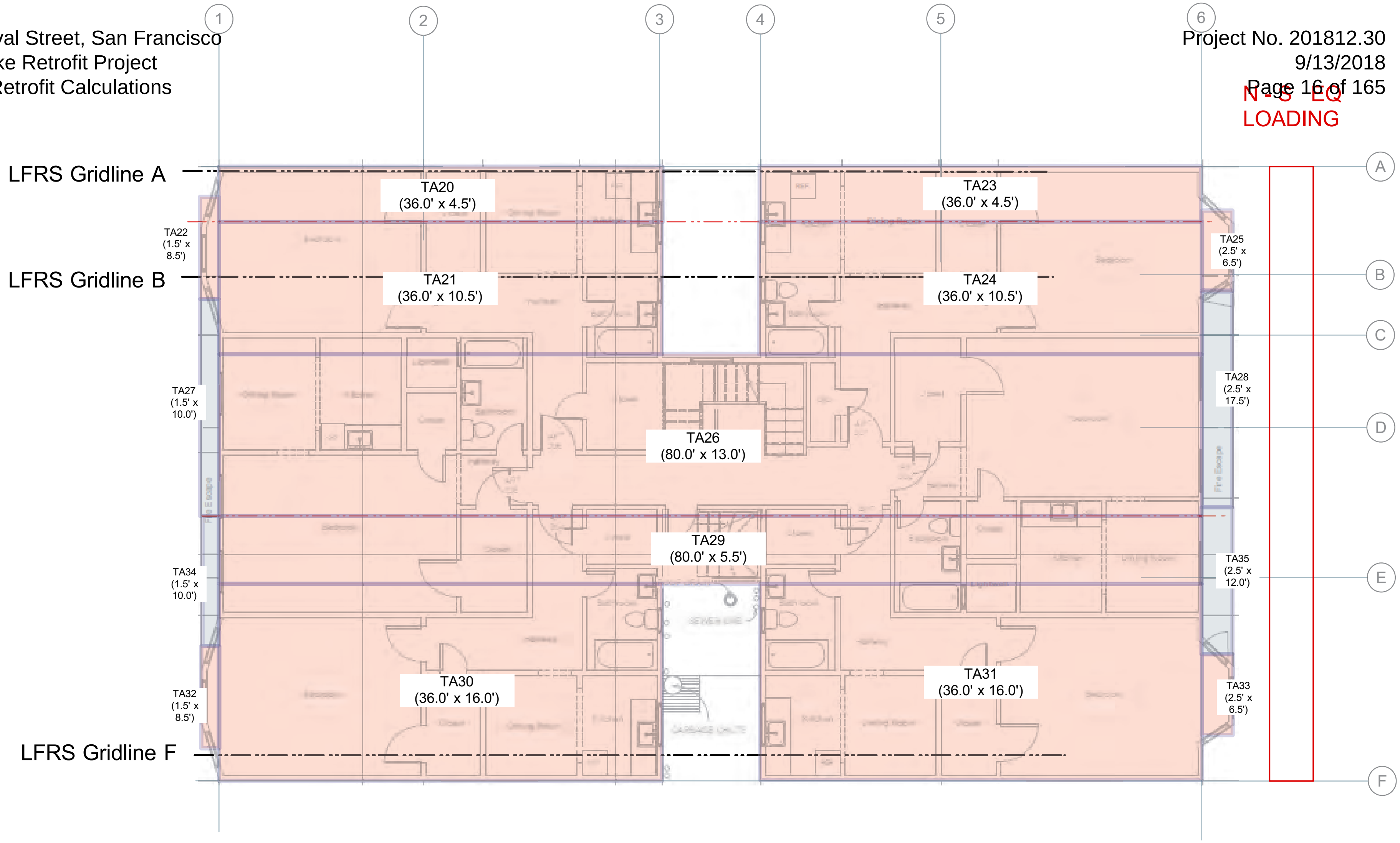
b) Other Sides of Building (South, East and West)

| | | |
|---|---|---|
| 2-1x Straight Sheathing Layers (3 psf x 1.0") : | 6.0 | psf |
| 2 x 6 studs @ 16"oc (Plus plates, blocking, etc) | 2.0 | psf |
| Insulation, Loose | 0.5 | psf |
| 1/2" Plaster over Wood Lath (10.0 psf x 1/2) | 5.0 | psf |
| **Total wall loads:** | **13.5** | **psf** |
| **$WDL_c$ =** | **14.0** | **psf** |



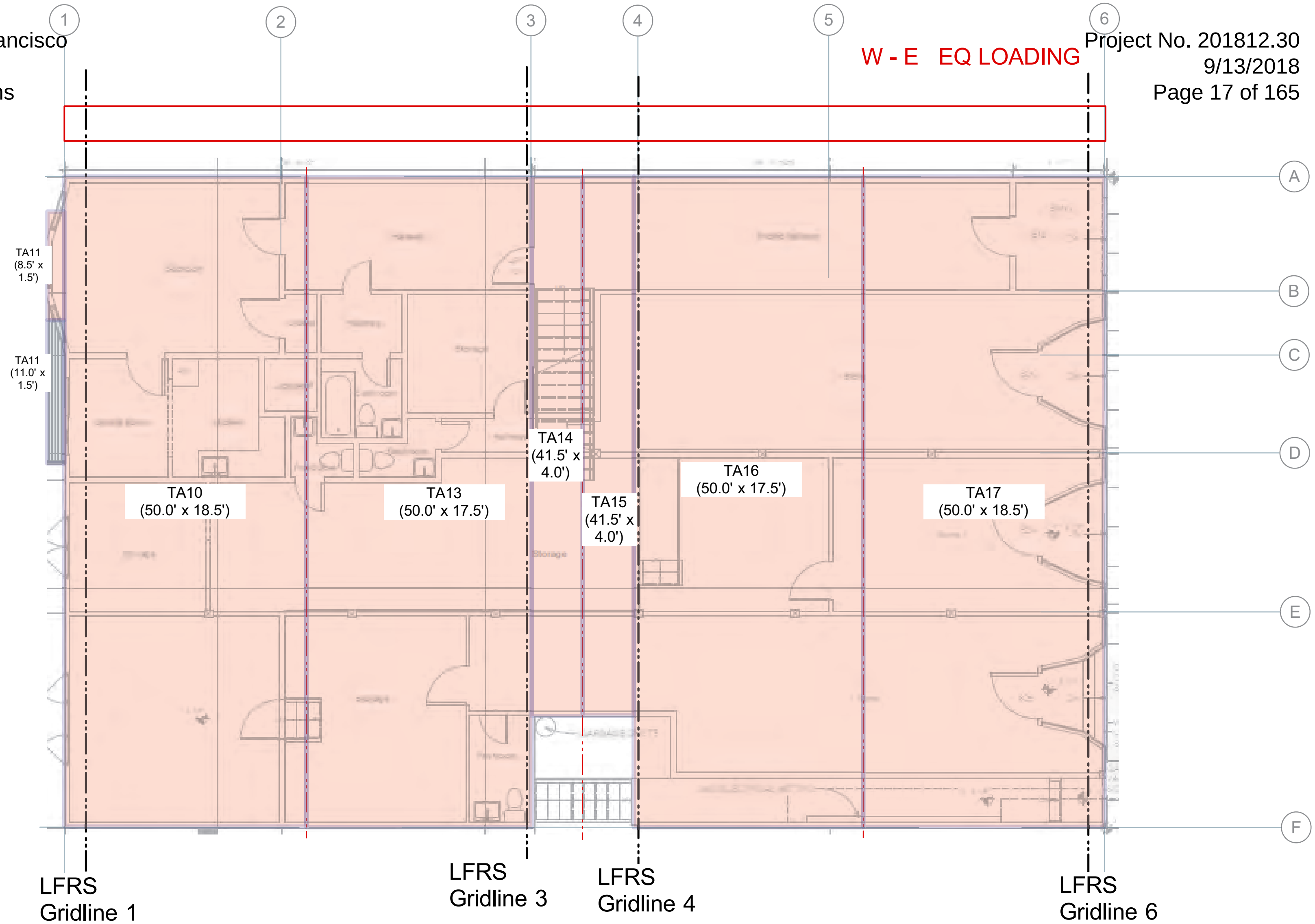

LEVELS 1-2 FLOOR - N-S TRIBUTARY AREAS

NTS



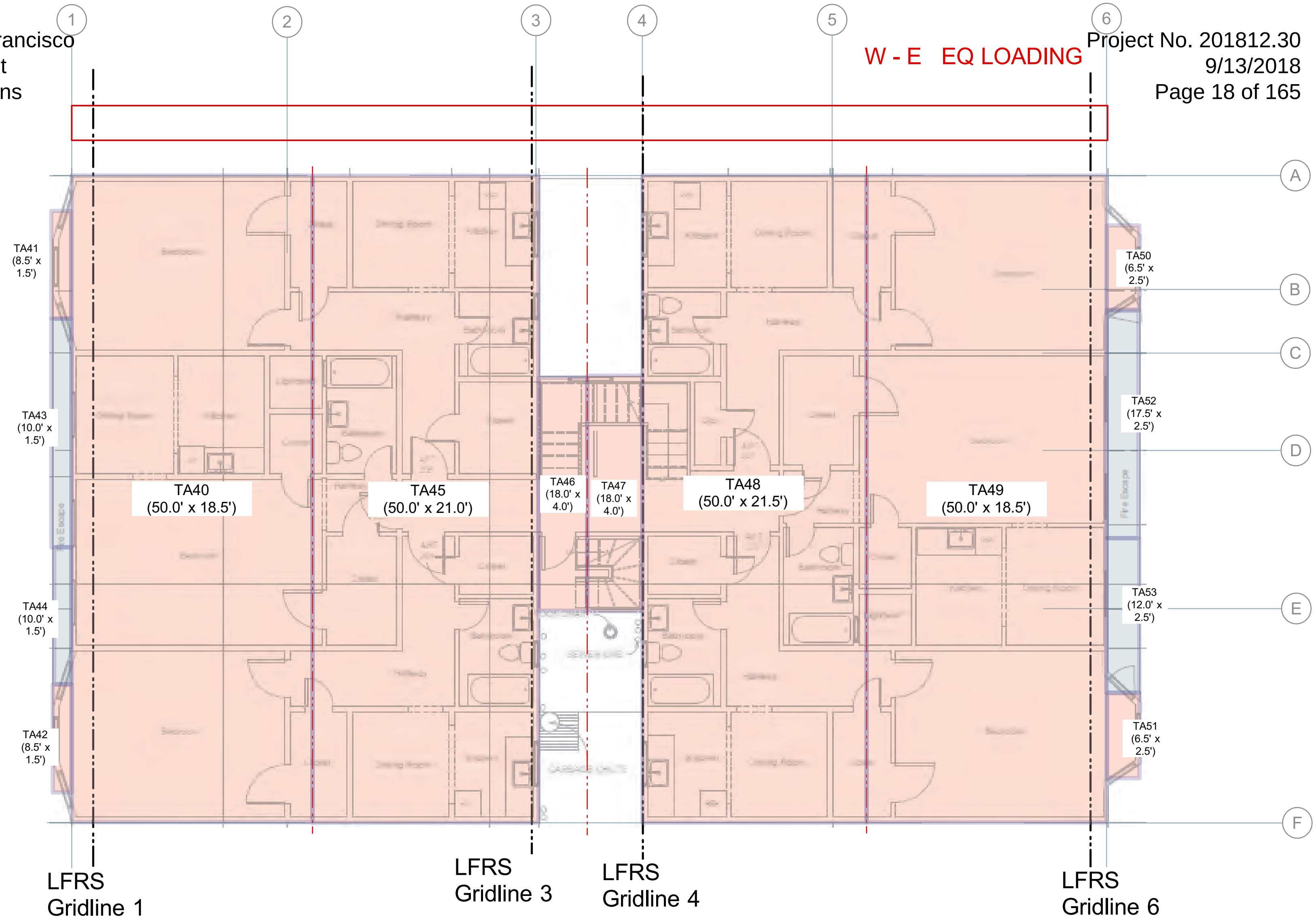

## LEVEL 3 FLOOR & ROOF - N-S TRIBUTARY AREAS

NTS




LEVELS 1-2 - W-E TRIBUTARY AREAS

NTS




LEVEL 3 FLOOR AND ROOF - W-E TRIBUTARY AREAS

NTS

**DETERMINATION OF SEISMIC LOADS TO LFRS AT GRIDLINES - FLEXIBLE DIAPHRAGM ASSUMPTIONS**
**ASCE 7-10 CHAPTER 12 - SEISMIC REQUIREMENTS FOR BUILDING STRUCTURES**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**1. Input Data**

$\rho$ = 1.30 Redundancy Factor (ASCE 7-10 Section 12.3.4)

| | Floors | | | | | |
|---|---|---|---|---|---|---|
| | **Roof** | **Floor A** | **Floor B** | **Decks** | **RC Stairs** | **Fire Escape** |
| **DL (psf)** | 22 | 32 | | | | 30 |
| **LL (psf)** | - | - | | - | - | - |

| Exterior Walls | | | |
|---|---|---|---|
| **Type a** | **Type b** | **Type c** | **Type d** |
| 25 | 15 | | |
| - | - | - | - |

| Code Level Floor Loads | | |
|---|---|---|
| **Level** | **$W_x$** | **$A_F$** |
| Roof | 111 | 0.24 |
| 3 | 172 | 0.18 |
| 2 | 172 | 0.13 |
| 1 | 172 | 0.06 |

Sum = 628 Kips

Where $W_x$ = Story Weight at Level x

$A_F$ = Floor Acceleration at Level x

**V = 0.202 *W (Seismic Base Shear - Strength Level)**
**V = 0.152 *W ( " - 75% V for (E) Bldg per IEBC12 Section A4)**
**V = 0.197 *W ( " - adjusted for Redundancy Factor, $\rho$)**
**V = 0.141 *W (Seismic Base Shear - ASD Level)**

Weight Check:
N-S = 634 kips
W-E = 635 kips
(0.28 %)

**2. Determination of Tributary Loads**

| Loading Direction | Wall Gridline Location | Floor Level | Tributary Area Segment | Floor Type | Diaphragm Loads: Dead Load (psf) | Partition Loads (psf) | Length (feet) | Width (feet) | Area (ft^2) | DL | Wall Loads: Wall Segment | Wall Dead Load (psf) | Length (feet) | Height (feet) | Wall Area (ft^2) | DL (lbs) | Seismic Weight: At Floor Level (lbs) | Sum (lbs) | Seismic Loads Code Level: Forces (lbs) | Shears (lbs) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **N-S** | **A** | R | TA20 | R | 22.0 | | 36.00 | 4.50 | 162 | 3,564 | a | 25.00 | 4.50 | 8.00 | 36 | 900 | 12,483 | 12,483 | 2,949 | 2,949 |
| | | | TA21 | R | 22.0 | | 36.00 | 4.50 | 162 | 3,564 | b | 15.00 | 4.50 | 6.00 | 27 | 405 | | | | |
| | | | | | | | | | | | b | 15.00 | 4.50 | 6.00 | 27 | 405 | | | | |
| | | | | | | | | | | | b | 15.00 | 4.50 | 6.00 | 27 | 405 | | | | |
| | | | | | | | | | | | b | 15.00 | 36.00 | 6.00 | 216 | 3,240 | | | | |
| | | | | | | | | | | | b | 15.00 | 36.00 | 6.00 | | | | | | |
| | | **3** | TA20 | F | 32.0 | | 36.00 | 4.50 | 162 | 5,184 | a | 25.00 | 4.50 | 10.00 | 45 | 1,125 | 18,918 | 31,401 | 3,377 | 6,327 |
| | | | TA21 | F | 32.0 | | 36.00 | 4.50 | 162 | 5,184 | b | 15.00 | 4.50 | 10.00 | 45 | 675 | | | | |
| | | | | | | | | | | | b | 15.00 | 4.50 | 10.00 | 45 | 675 | | | | |
| | | | | | | | | | | | b | 15.00 | 4.50 | 10.00 | 45 | 675 | | | | |
| | | | | | | | | | | | b | 15.00 | 36.00 | 10.00 | 360 | 5,400 | | | | |
| | | | | | | | | | | | b | 15.00 | 36.00 | 10.00 | | | | | | |
| | | **2** | TA01 | F | 32.0 | | 80.00 | 4.50 | 360 | 11,520 | a | 25.00 | 4.50 | 12.00 | 54 | 1,350 | 28,080 | 59,481 | 3,538 | 9,865 |
| | | | | | | | | | | | b | 15.00 | 4.50 | 12.00 | 54 | 810 | | | | |
| | | | | | | | | | | | b | 15.00 | 80.00 | 12.00 | 960 | 14,400 | | | | |
| | | **1** | TA01 | F | 32.0 | | 80.00 | 4.50 | 360 | 11,520 | a | 25.00 | 4.50 | 8.00 | 36 | 900 | 21,225 | 80,706 | 1,170 | 11,035 |
| | | | | | | | | | | | b | 15.00 | 4.50 | 6.00 | 27 | 405 | | | | |
| | | | | | | | | | | | b | 15.00 | 80.00 | 7.00 | 560 | 8,400 | | | | |
| | **B** | R | TA21 | R | 22.0 | | 36.00 | 10.50 | 378 | 8,316 | a | 25.00 | 10.50 | 8.00 | 84 | 2,100 | 48,839 | 48,839 | 11,539 | 11,539 |
| | | | TA24 | R | 22.0 | | 36.00 | 10.50 | 378 | 8,316 | b | 15.00 | 10.50 | 6.00 | 63 | 945 | | | | |
| | | | TA26 | R | 22.0 | | 80.00 | 13.00 | 1,040 | 22,880 | b | 15.00 | 10.50 | 6.00 | 63 | 945 | | | | |
| | | | TA22 | R | 22.0 | | 1.50 | 8.00 | 12 | 264 | b | 15.00 | 10.50 | 6.00 | 63 | 945 | | | | |
| | | | TA25 | R | 22.0 | | 2.50 | 6.50 | 16 | 358 | a | 25.00 | 13.00 | 8.00 | 104 | 2,600 | | | | |
| | | | | | | | | | | | b | 15.00 | 13.00 | 6.00 | 78 | 1,170 | | | | |
| | | **3** | TA21 | F | 32.0 | | 36.00 | 10.50 | 378 | 12,096 | a | 25.00 | 10.50 | 10.00 | 105 | 2,625 | 73,004 | 121,842 | 13,032 | 24,571 |
| | | | TA24 | F | 32.0 | | 36.00 | 10.50 | 378 | 12,096 | b | 15.00 | 10.50 | 10.00 | 105 | 1,575 | | | | |
| | | | TA26 | F | 32.0 | | 80.00 | 13.00 | 1,040 | 33,280 | b | 15.00 | 10.50 | 10.00 | 105 | 1,575 | | | | |
| | | | TA22 | F | 32.0 | | 1.50 | 8.00 | 12 | 384 | b | 15.00 | 10.50 | 10.00 | 105 | 1,575 | | | | |
| | | | TA25 | F | 32.0 | | 2.50 | 6.50 | 16 | 520 | a | 25.00 | 13.00 | 10.00 | 130 | 3,250 | | | | |
| | | | TA27 | FS | 30.0 | | 1.50 | 17.00 | 26 | 765 | b | 15.00 | 13.00 | 10.00 | 130 | 1,950 | | | | |
| | | | TA28 | FS | 30.0 | | 2.50 | 17.50 | 44 | 1,313 | | | | | | | | | | |
| | | **2** | TA02 | F | 32.0 | | 80.00 | 24.00 | 1,920 | 61,440 | a | 25.00 | 24.00 | 12.00 | 288 | 7,200 | 75,966 | 197,808 | 9,572 | 34,144 |
| | | | TA03 | F | 32.0 | | 1.50 | 8.50 | 13 | 408 | b | 15.00 | 24.00 | 12.00 | 288 | 4,320 | | | | |
| | | | TA25 | F | 32.0 | | 2.50 | 6.50 | 16 | 520 | | | | | | | | | | |
| | | | TA27 | FS | 30.0 | | 1.50 | 17.00 | 26 | 765 | | | | | | | | | | |
| | | | TA28 | FS | 30.0 | | 2.50 | 17.50 | 44 | 1,313 | | | | | | | | | | |
| | | **1** | TA02 | F | 32.0 | | 80.00 | 24.00 | 1,920 | 61,440 | a | 25.00 | 24.00 | 8.00 | 192 | 4,800 | 68,808 | 266,616 | 3,793 | 37,937 |
| | | | TA03 | F | 32.0 | | 1.50 | 8.50 | 13 | 408 | b | 15.00 | 24.00 | 6.00 | 144 | 2,160 | | | | |

**DETERMINATION OF SEISMIC LOADS TO LFRS AT GRIDLINES - FLEXIBLE DIAPHRAGM ASSUMPTIONS**
**ASCE 7-10 CHAPTER 12 - SEISMIC REQUIREMENTS FOR BUILDING STRUCTURES**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**1. Input Data**

$\rho$ = 1.30 Redundancy Factor (ASCE 7-10 Section 12.3.4)

| | Floors | | | | | | Exterior Walls | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Roof | Floor A | Floor B | Decks | RC Stairs | Fire Escape | Type a | Type b | Type c | Type d |
| DL (psf) | 22 | 32 | | | | 30 | 25 | 15 | | |
| LL (psf) | - | - | | - | - | - | - | - | - | - |

| Code Level Floor Loads | | |
|---|---|---|
| Level | $W_x$ | $A_F$ |
| Roof | 111 | 0.24 |
| 3 | 172 | 0.18 |
| 2 | 172 | 0.13 |
| 1 | 172 | 0.06 |
| | | |

Sum = 628 Kips

Where $W_x$ = Story Weight at Level x

$A_F$ = Floor Acceleration at Level x

**V = 0.202 *W (Seismic Base Shear - Strength Level)**
**V = 0.152 *W ( " - 75% V for (E) Bldg per IEBC12 Section A4)**
**V = 0.197 *W ( " - adjusted for Redundancy Factor, ρ)**
**V = 0.141 *W (Seismic Base Shear - ASD Level)**

Weight Check:
N-S = 634 kips
W-E = 635 kips
(0.28 %)

**2. Determination of Tributary Loads**

| Loading Direction | Wall Gridline Location | Floor Level | Tributary Area Segment | Floor Type | Diaphragm Loads: Dead Load (psf) | Partition Loads (psf) | Length (feet) | Width (feet) | Area (ft^2) | DL | Wall Loads: Wall Segment | Wall Dead Load (psf) | Length (feet) | Height (feet) | Wall Area (ft^2) | DL (lbs) | Seismic Weight: At Floor Level (lbs) | Sum (lbs) | Seismic Loads Code Level: Forces (lbs) | Shears (lbs) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **N-S (Cont)** | **F** | R | TA29 | R | 22.0 | | 80.00 | 5.50 | 440 | 9,680 | a | 25.00 | 21.50 | 8.00 | 172 | 4,300 | 55,217 | 55,217 | 13,046 | 13,046 |
| | | | TA30 | R | 22.0 | | 36.00 | 16.00 | 576 | 12,672 | b | 15.00 | 16.00 | 6.00 | 96 | 1,440 | | | | |
| | | | TA31 | R | 22.0 | | 36.00 | 16.00 | 576 | 12,672 | b | 15.00 | 16.00 | 6.00 | 96 | 1,440 | | | | |
| | | | TA32 | R | 22.0 | | 1.50 | 8.50 | 13 | 281 | b | 15.00 | 21.50 | 6.00 | 129 | 1,935 | | | | |
| | | | TA33 | R | 22.0 | | 2.50 | 6.50 | 16 | 358 | a | 25.00 | 36.00 | 8.00 | 288 | 7,200 | | | | |
| | | | | | | | | | | | b | 15.00 | 36.00 | 6.00 | 216 | 3,240 | | | | |
| | | **3** | TA29 | F | 32.0 | | 80.00 | 5.50 | 440 | 14,080 | a | 25.00 | 21.50 | 10.00 | 215 | 5,375 | 81,112 | 136,329 | 14,480 | 27,526 |
| | | | TA30 | F | 32.0 | | 36.00 | 16.00 | 576 | 18,432 | b | 15.00 | 16.00 | 10.00 | 160 | 2,400 | | | | |
| | | | TA31 | F | 32.0 | | 36.00 | 16.00 | 576 | 18,432 | b | 15.00 | 16.00 | 10.00 | 160 | 2,400 | | | | |
| | | | TA32 | F | 32.0 | | 1.50 | 8.50 | 13 | 408 | b | 15.00 | 21.50 | 10.00 | 215 | 3,225 | | | | |
| | | | TA33 | F | 32.0 | | 2.50 | 6.50 | 16 | 520 | a | 25.00 | 36.00 | 10.00 | 360 | 9,000 | | | | |
| | | | TA34 | FS | 30.0 | | 1.50 | 12.00 | 18 | 540 | b | 15.00 | 36.00 | 10.00 | 360 | 5,400 | | | | |
| | | | TA35 | FS | 30.0 | | 2.50 | 12.00 | 30 | 900 | | | | | | | | | | |
| | | **2** | TA05 | F | 32.0 | | 80.00 | 13.00 | 1,040 | 33,280 | a | 25.00 | 21.50 | 12.00 | 258 | 6,450 | 81,572 | 217,901 | 10,279 | 37,805 |
| | | | TA06 | F | 32.0 | | 36.00 | 8.50 | 306 | 9,792 | b | 15.00 | 21.50 | 12.00 | 258 | 3,870 | | | | |
| | | | TA07 | F | 32.0 | | 36.00 | 8.50 | 306 | 9,792 | b | 15.00 | 36.00 | 12.00 | 432 | 6,480 | | | | |
| | | | TA32 | F | 32.0 | | 1.50 | 8.50 | 13 | 408 | b | 15.00 | 36.00 | 12.00 | 432 | 6,480 | | | | |
| | | | TA33 | F | 32.0 | | 2.50 | 6.50 | 16 | 520 | b | 15.00 | 8.50 | 12.00 | 102 | 1,530 | | | | |
| | | | TA34 | FS | 30.0 | | 1.50 | 12.00 | 18 | 540 | b | 15.00 | 8.50 | 12.00 | 102 | 1,530 | | | | |
| | | | TA35 | FS | 30.0 | | 2.50 | 12.00 | 30 | 900 | | | | | | | | | | |
| | | **1** | TA05 | F | 32.0 | | 80.00 | 13.00 | 1,040 | 33,280 | a | 25.00 | 21.50 | 8.00 | 172 | 4,300 | 68,444 | 286,345 | 3,773 | 41,578 |
| | | | TA06 | F | 32.0 | | 36.00 | 8.50 | 306 | 9,792 | b | 15.00 | 21.50 | 6.00 | 129 | 1,935 | | | | |
| | | | TA07 | F | 32.0 | | 36.00 | 8.50 | 306 | 9,792 | b | 15.00 | 36.00 | 7.00 | 252 | 3,780 | | | | |
| | | | | | | | | | | | b | 15.00 | 36.00 | 7.00 | 252 | 3,780 | | | | |
| | | | | | | | | | | | b | 15.00 | 8.50 | 7.00 | 60 | 893 | | | | |
| | | | | | | | | | | | b | 15.00 | 8.50 | 7.00 | 60 | 893 | | | | |

**DETERMINATION OF SEISMIC LOADS TO LFRS AT GRIDLINES - FLEXIBLE DIAPHRAGM ASSUMPTIONS**
**ASCE 7-10 CHAPTER 12 - SEISMIC REQUIREMENTS FOR BUILDING STRUCTURES**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**1. Input Data**

$\rho$ = 1.30 Redundancy Factor (ASCE 7-10 Section 12.3.4)

| | Floors | | | | | | Exterior Walls | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Roof | Floor A | Floor B | Decks | RC Stairs | Fire Escape | Type a | Type b | Type c | Type d |
| **DL (psf)** | 22 | 32 | | | | 30 | 25 | 15 | | |
| **LL (psf)** | - | - | | - | - | - | - | - | - | - |

| Code Level Floor Loads | | |
|---|---|---|
| Level | $W_x$ | $A_F$ |
| Roof | 111 | 0.24 |
| 3 | 172 | 0.18 |
| 2 | 172 | 0.13 |
| 1 | 172 | 0.06 |
| | | |
| | | |

Sum = 628 Kips

Where $W_x$ = Story Weight at Level x

$A_F$ = Floor Acceleration at Level x

**V = 0.202 *W (Seismic Base Shear - Strength Level)**
**V = 0.152 *W ( " - 75% V for (E) Bldg per IEBC12 Section A4)**
**V = 0.197 *W ( " - adjusted for Redundancy Factor, ρ)**
**V = 0.141 *W (Seismic Base Shear - ASD Level)**

Weight Check:
N-S = 634 kips
W-E = 635 kips
(0.28 %)

**2. Determination of Tributary Loads**

| | | | | | Diaphragm Loads | | | | | | Wall Loads | | | | | | Seismic Weight | | Seismic Loads Code Level | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Loading Direction | Wall Gridline Location | Floor Level | Tributary Area Segment | Floor Type | Dead Load (psf) | Partition Loads (psf) | Length (feet) | Width (feet) | Area (ft^2) | DL | Wall Segment | Wall Dead Load (psf) | Length (feet) | Height (feet) | Wall Area (ft^2) | DL (lbs) | At Floor Level (lbs) | Sum (lbs) | Forces (lbs) | Shears (lbs) |
| **W-E** | **1** | R | TA40 | R | 22.0 | | 50.00 | 18.00 | 900 | 19,800 | b | 15.00 | 18.00 | 6.00 | 108 | 1,620 | 33,601 | 33,601 | 7,939 | 7,939 |
| | | | TA41 | R | 22.0 | | 8.50 | 1.50 | 13 | 281 | b | 15.00 | 18.00 | 6.00 | 108 | 1,620 | | | | |
| | | | TA42 | R | 22.0 | | 8.50 | 1.50 | 13 | 281 | a | 25.00 | 50.00 | 8.00 | 400 | 10,000 | | | | |
| | | **3** | TA40 | F | 32.0 | | 50.00 | 18.00 | 900 | 28,800 | b | 15.00 | 18.00 | 10.00 | 180 | 2,700 | 48,844 | 82,445 | 8,719 | 16,658 |
| | | | TA41 | F | 32.0 | | 8.50 | 1.50 | 13 | 408 | b | 15.00 | 18.00 | 10.00 | 180 | 2,700 | | | | |
| | | | TA42 | F | 32.0 | | 8.50 | 1.50 | 13 | 408 | a | 25.00 | 50.00 | 10.00 | 500 | 12,500 | | | | |
| | | | TA43 | FS | 30.0 | | 17.50 | 1.50 | 26 | 788 | | | | | | | | | | |
| | | | TA44 | FS | 30.0 | | 12.00 | 1.50 | 18 | 540 | | | | | | | | | | |
| | | **2** | TA10 | F | 32.0 | | 50.00 | 18.00 | 900 | 28,800 | b | 15.00 | 18.00 | 12.00 | 216 | 3,240 | 52,016 | 134,460 | 6,555 | 23,213 |
| | | | TA11 | F | 32.0 | | 8.50 | 1.50 | 13 | 408 | b | 15.00 | 18.00 | 12.00 | 216 | 3,240 | | | | |
| | | | TA43 | FS | 30.0 | | 17.50 | 1.50 | 26 | 788 | a | 25.00 | 50.00 | 12.00 | 600 | 15,000 | | | | |
| | | | TA44 | FS | 30.0 | | 12.00 | 1.50 | 18 | 540 | | | | | | | | | | |
| | | **1** | TA10 | F | 32.0 | | 50.00 | 18.00 | 900 | 28,800 | b | 15.00 | 18.00 | 8.00 | 144 | 2,160 | 43,528 | 177,988 | 2,400 | 25,612 |
| | | | TA11 | F | 32.0 | | 8.50 | 1.50 | 13 | 408 | b | 15.00 | 18.00 | 8.00 | 144 | 2,160 | | | | |
| | | | | | | | | | | | a | 25.00 | 50.00 | 8.00 | 400 | 10,000 | | | | |
| | **3** | R | TA45 | R | 22.0 | | 50.00 | 17.50 | 875 | 19,250 | b | 15.00 | 21.50 | 6.00 | 129 | 1,935 | 27,854 | 27,854 | 6,581 | 6,581 |
| | | | TA46 | R | 22.0 | | 18.00 | 4.00 | 72 | 1,584 | b | 15.00 | 21.50 | 6.00 | 129 | 1,935 | | | | |
| | | | | | | | | | | | b | 15.00 | 17.50 | 6.00 | 105 | 1,575 | | | | |
| | | | | | | | | | | | b | 15.00 | 17.50 | 6.00 | 105 | 1,575 | | | | |
| | | **3** | TA45 | F | 32.0 | | 50.00 | 17.50 | 875 | 28,000 | b | 15.00 | 21.50 | 10.00 | 215 | 3,225 | 42,004 | 69,858 | 7,498 | 14,079 |
| | | | TA46 | F | 32.0 | | 18.00 | 4.00 | 72 | 2,304 | b | 15.00 | 21.50 | 10.00 | 215 | 3,225 | | | | |
| | | | | | | | | | | | b | 15.00 | 17.50 | 10.00 | 175 | 2,625 | | | | |
| | | | | | | | | | | | b | 15.00 | 17.50 | 10.00 | 175 | 2,625 | | | | |
| | | **2** | TA13 | F | 32.0 | | 50.00 | 17.50 | 875 | 28,000 | b | 15.00 | 21.50 | 12.00 | 258 | 3,870 | 42,852 | 112,710 | 5,400 | 19,479 |
| | | | TA14 | F | 32.0 | | 41.50 | 4.00 | 166 | 5,312 | b | 15.00 | 21.50 | 12.00 | 258 | 3,870 | | | | |
| | | | | | | | | | | | b | 15.00 | 10.00 | 12.00 | 120 | 1,800 | | | | |
| | | **1** | TA13 | F | 32.0 | | 50.00 | 17.50 | 875 | 28,000 | b | 15.00 | 21.50 | 7.00 | 151 | 2,258 | 38,877 | 151,587 | 2,143 | 21,623 |
| | | | TA14 | F | 32.0 | | 41.50 | 4.00 | 166 | 5,312 | b | 15.00 | 21.50 | 7.00 | 151 | 2,258 | | | | |
| | | | | | | | | | | | b | 15.00 | 10.00 | 7.00 | 70 | 1,050 | | | | |

**DETERMINATION OF SEISMIC LOADS TO LFRS AT GRIDLINES - FLEXIBLE DIAPHRAGM ASSUMPTIONS**
**ASCE 7-10 CHAPTER 12 - SEISMIC REQUIREMENTS FOR BUILDING STRUCTURES**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**1. Input Data**

$\rho$ = 1.30 Redundancy Factor (ASCE 7-10 Section 12.3.4)

| | Floors | | | | | |
|---|---|---|---|---|---|---|
| | **Roof** | **Floor A** | **Floor B** | **Decks** | **RC Stairs** | **Fire Escape** |
| **DL (psf)** | 22 | 32 | | | | 30 |
| **LL (psf)** | - | - | | - | - | - |

| Exterior Walls | | | |
|---|---|---|---|
| **Type a** | **Type b** | **Type c** | **Type d** |
| 25 | 15 | | |
| - | - | - | - |

| Code Level Floor Loads | | |
|---|---|---|
| **Level** | $W_x$ | $A_F$ |
| Roof | 111 | 0.24 |
| 3 | 172 | 0.18 |
| 2 | 172 | 0.13 |
| 1 | 172 | 0.06 |
| | | |

Sum = 628 Kips

Where $W_x$ = Story Weight at Level x

$A_F$ = Floor Acceleration at Level x

**V = 0.202 *W (Seismic Base Shear - Strength Level)**
**V = 0.152 *W ( " - 75% V for (E) Bldg per IEBC12 Section A4)**
**V = 0.197 *W ( " - adjusted for Redundancy Factor, $\rho$)**
**V = 0.141 *W (Seismic Base Shear - ASD Level)**

Weight Check:
N-S = 634 kips
W-E = 635 kips
(0.28 %)

**2. Determination of Tributary Loads**

| | | | | | Diaphragm Loads | | | | | | Wall Loads | | | | | | Seismic Weight | | Seismic Loads Code Level | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Loading Direction** | **Wall Gridline Location** | **Floor Level** | Tributary Area Segment | Floor Type | Dead Load (psf) | Partition Loads (psf) | Length (feet) | Width (feet) | Area (ft^2) | DL | Wall Segment | Wall Dead Load (psf) | Length (feet) | Height (feet) | Wall Area (ft^2) | DL (lbs) | At Floor Level (lbs) | Sum (lbs) | Forces (lbs) | Shears (lbs) |
| **W-E** | **4** | R | TA48 | R | 22.0 | | 50.00 | 17.50 | 875 | 19,250 | b | 15.00 | 21.50 | 6.00 | 129 | 1,935 | 27,854 | 27,854 | 6,581 | 6,581 |
| **(Cont)** | | | TA47 | R | 22.0 | | 18.00 | 4.00 | 72 | 1,584 | b | 15.00 | 21.50 | 6.00 | 129 | 1,935 | | | | |
| | | | | | | | | | | | b | 15.00 | 17.50 | 6.00 | 105 | 1,575 | | | | |
| | | | | | | | | | | | b | 15.00 | 17.50 | 6.00 | 105 | 1,575 | | | | |
| | | **3** | TA48 | F | 32.0 | | 50.00 | 17.50 | 875 | 28,000 | b | 15.00 | 21.50 | 10.00 | 215 | 3,225 | 42,004 | 69,858 | 7,498 | 14,079 |
| | | | TA47 | F | 32.0 | | 18.00 | 4.00 | 72 | 2,304 | b | 15.00 | 21.50 | 10.00 | 215 | 3,225 | | | | |
| | | | | | | | | | | | b | 15.00 | 17.50 | 10.00 | 175 | 2,625 | | | | |
| | | | | | | | | | | | b | 15.00 | 17.50 | 10.00 | 175 | 2,625 | | | | |
| | | **2** | TA16 | F | 32.0 | | 50.00 | 17.50 | 875 | 28,000 | b | 15.00 | 21.50 | 12.00 | 258 | 3,870 | 37,182 | 107,040 | 4,685 | 18,765 |
| | | | TA15 | F | 32.0 | | 41.50 | 4.00 | 166 | 5,312 | b | 15.00 | 21.50 | 0.00 | 0 | | | | | |
| | | | | | | | | | | | b | 15.00 | 10.00 | 0.00 | 0 | | | | | |
| | | **1** | TA16 | F | 32.0 | | 50.00 | 17.50 | 875 | 28,000 | b | 15.00 | 21.50 | 6.00 | 129 | 1,935 | 38,082 | 145,122 | 2,099 | 20,864 |
| | | | TA15 | F | 32.0 | | 41.50 | 4.00 | 166 | 5,312 | b | 15.00 | 21.50 | 6.00 | 129 | 1,935 | | | | |
| | | | | | | | | | | | b | 15.00 | 10.00 | 6.00 | 60 | 900 | | | | |
| | **6** | R | TA49 | R | 22.0 | | 50.00 | 18.50 | 925 | 20,350 | b | 15.00 | 18.50 | 6.00 | 111 | 1,665 | 28,895 | 28,895 | 6,827 | 6,827 |
| | | | TA50 | R | 22.0 | | 6.50 | 2.50 | 16 | 358 | b | 15.00 | 18.50 | 6.00 | 111 | 1,665 | | | | |
| | | | TA51 | R | 22.0 | | 6.50 | 2.50 | 16 | 358 | b | 15.00 | 50.00 | 6.00 | 300 | 4,500 | | | | |
| | | **3** | TA49 | F | 32.0 | | 50.00 | 18.50 | 925 | 29,600 | b | 15.00 | 18.50 | 10.00 | 185 | 2,775 | 45,903 | 74,798 | 8,194 | 15,021 |
| | | | TA50 | F | 32.0 | | 6.50 | 2.50 | 16 | 520 | b | 15.00 | 18.50 | 10.00 | 185 | 2,775 | | | | |
| | | | TA51 | F | 32.0 | | 6.50 | 2.50 | 16 | 520 | b | 15.00 | 50.00 | 10.00 | 500 | 7,500 | | | | |
| | | | TA52 | FS | 30.0 | | 17.50 | 2.50 | 44 | 1,313 | | | | | | | | | | |
| | | | TA53 | FS | 30.0 | | 12.00 | 2.50 | 30 | 900 | | | | | | | | | | |
| | | **2** | TA17 | F | 32.0 | | 50.00 | 18.50 | 925 | 29,600 | b | 15.00 | 18.50 | 12.00 | 222 | 3,330 | 48,513 | 123,310 | 6,113 | 21,134 |
| | | | TA50 | F | 32.0 | | 6.50 | 2.50 | 16 | 520 | b | 15.00 | 18.50 | 12.00 | 222 | 3,330 | | | | |
| | | | TA51 | F | 32.0 | | 6.50 | 2.50 | 16 | 520 | b | 15.00 | 50.00 | 12.00 | 600 | 9,000 | | | | |
| | | | TA52 | FS | 30.0 | | 17.50 | 2.50 | 44 | 1,313 | | | | | | | | | | |
| | | | TA53 | FS | 30.0 | | 12.00 | 2.50 | 30 | 900 | | | | | | | | | | |
| | | **1** | TA17 | F | 32.0 | | 50.00 | 18.50 | 925 | 29,600 | b | 15.00 | 18.50 | 6.00 | 111 | 1,665 | 37,430 | 160,740 | 2,064 | 23,198 |
| | | | | | | | | | | | b | 15.00 | 18.50 | 6.00 | 111 | 1,665 | | | | |
| | | | | | | | | | | | b | 15.00 | 50.00 | 6.00 | 300 | 4,500 | | | | |

# FLEXIBLE DIAPHRAGM ANALYSIS
# - N-S LOADING AT GRIDLINES

**SHEARWALL LOAD DISTRIBUTION - FLEXIBLE DIAPHRAGM ANALYSIS**
**NDS - SDPWS 2015 SHEAR WALL CRITERIA**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Wall Location:** Gridline A

Loading: EQ
Loading Direction: N-S

## 1. Diaphragm and Shear Wall Dimensions along Plane of Assembled Walls

| | Story Forces | | Foundation | | | Diaphragm | | |
|---|---|---|---|---|---|---|---|---|
| Level | Strength Load (lbs) | **Service Load (lbs)** | Offset (feet) | Length (feet) | Edge (feet) | Offset (feet) | Length (feet) | Edge (feet) |
| R | | **2,949** | | | | 0 | 80.00 | 80.00 |
| 3 | | **3,377** | 0 | 0 | 0.00 | 0 | 80.00 | 80.00 |
| 2 | | **3,538** | 0 | 0 | 0.00 | 0 | 80.00 | 80.00 |
| 1 | | **1,170** | 0 | 0.00 | 0.00 | 0.00 | 80.00 | 80.00 |
| 0 | - | | | 80.00 | 80.00 | | - | |

| | | Wall Segments | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Wall 1 | | Wall 2 | | Wall 3 | | Wall 4 | | Wall 5 | |
| Wall Levels | Wall Height (feet) | Offset* (feet) | Length (feet) | Offset (feet) | Length (feet) | Offset (feet) | Length (feet) | Offset (feet) | Length (feet) | Offset (feet) | Length (feet) |
| 3 Level Tied * | 11.00 | 0.00 | 36.00 | 8.00 | 36.00 | | | | | | |
| 2 Level Tied * | 10.00 | 0.00 | 36.00 | 8.00 | 36.00 | | | | | | |
| 1 Level Tied * | 13.50 | 29.50 | 42.00 | | | | | | | | |
| 0 Level Tied * | 10.00 | 29.50 | 42.00 x | | | | | | | | |

| Summation of Segments | | |
|---|---|---|
| Wall Length (feet) | Floor Length (feet) | Tied to Foundation (feet)* |
| 72.00 | 80.00 | 0.00 |
| 72.00 | 80.00 | 0.00 |
| 42.00 | 80.00 | 0.00 |
| 42.00 | 80.00 | 42.00 |

* Notes : 1. Wall segment offset defined from edge of diaphragm (Diaphragm offset).
2. Marked automatically with an X if Wall segment is tied to foundation.
**3. After all data is complete , run macro w/ Crtl - w to update spreadsheet.**

## 2. Vertical Wall Distribution and Shear Wall Loads

| | | | Story Shear | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Level | Story Force (lbs) | Total Shear (lbs) | To Foundation (lbs) | To Walls (lbs) | Total Shear (lbs) | Wall Length (feet) | Diaphragm Length (feet) | Wall Shear (lbs/ft) | Diaphragm Shear (lbs/ft) |
| R | 2,949 | | | | | | 80.00 | | 37 |
| | | 2,949 | | 2,949 | | 72.00 | | 41 | |
| 3 | 3,377 | | 0 | | 2,949 | | 80.00 | | 42 |
| | | 6,326 | | 6,326 | | 72.00 | | 88 | |
| 2 | 3,538 | | 0 | | 6,326 | | 80.00 | | 44 |
| | | 9,864 | | 9,864 | | 42.00 | | 235 | |
| 1 | 1,170 | | 0 | | 9,864 | | 80.00 | | 15 |
| | | 11,034 | | 11,034 | | 42.00 | | 263 | |
| 0 | | | 11,034 | | 11,034 | | | | |

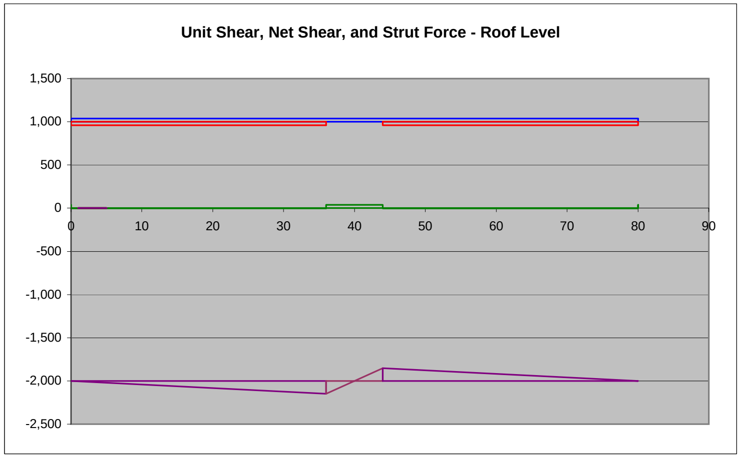

**Notes:** 1. Diaphragm connected to foundation transfers all load to shear walls; diaphragm connection to wall calculated separately (conservative).
2. Load transferred to floor below is proportional to wall length over diaphragm/total wall length;

**SHEARWALL LOAD DISTRIBUTION - FLEXIBLE DIAPHRAGM ANALYSIS**
**NDS - SDPWS 2015 SHEAR WALL CRITERIA**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Wall Location:** Gridline A

Loading: EQ
Loading Direction: N-S

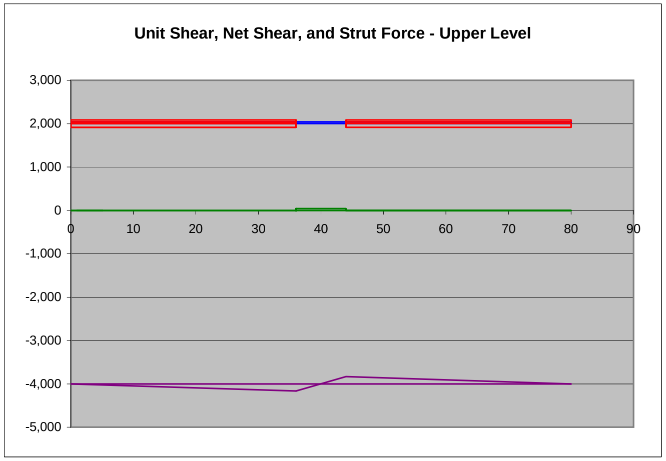
## 3. Plots of Unit and Net Shears and Strut Force at Wall Levels




**R Level Demands:**

Vsw = 41 lb/ft

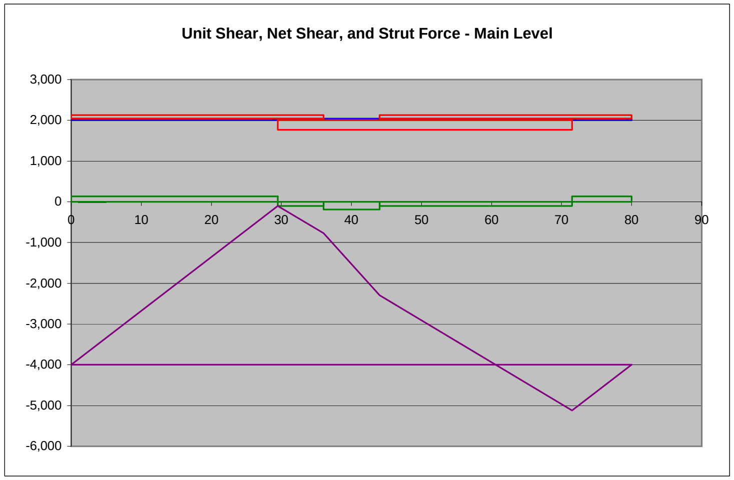
$F_{strut}$ = 147 lbs




**3 Level Demands:**

Vsw = 88 lb/ft

$F_{strut}$ = 169 lbs




**2 Level Demands:**

Vsw = 235 lb/ft

Fstrut = 3,897 lbs


**1 Level Demands:**

Vsw = 263 lb/ft

Fstrut = 431 lbs

**SHEARWALL LOAD DISTRIBUTION - FLEXIBLE DIAPHRAGM ANALYSIS**
**NDS - SDPWS 2015 SHEAR WALL CRITERIA**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Wall Location:** Gridline B

Loading: EQ
Loading Direction: N-S

## 1. Diaphragm and Shear Wall Dimensions along Plane of Assembled Walls

| | Story Forces | | Foundation | | | Diaphragm | | |
|---|---|---|---|---|---|---|---|---|
| Level | Strength Load (lbs) | **Service Load (lbs)** | Offset (feet) | Length (feet) | Edge (feet) | Offset (feet) | Length (feet) | Edge (feet) |
| R | | **11,539** | | | | 0 | 80.00 | 80.00 |
| 3 | | **13,032** | 0 | 0 | 0.00 | 0 | 80.00 | 80.00 |
| 2 | | **9,572** | 0 | 0 | 0.00 | 0 | 80.00 | 80.00 |
| 1 | | **3,793** | 0 | 0.00 | 0.00 | 0.00 | 80.00 | 80.00 |
| 0 | - | | | 80.00 | 80.00 | | - | |

| | | Wall Segments | | | | | | | | | | Summation of Segments | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Wall 1 | | Wall 2 | | Wall 3 | | Wall 4 | | Wall 5 | | | | |
| Wall Levels | Wall Height (feet) | Offset* (feet) | Length (feet) | Offset (feet) | Length (feet) | Offset (feet) | Length (feet) | Offset (feet) | Length (feet) | Offset (feet) | Length (feet) | Wall Length (feet) | Floor Length (feet) | Tied to Foundation (feet)* |
| 3 Level | 11.00 | 0.00 | 25.00 | 4.00 | 9.00 | 3.50 | 9.00 | 4.00 | 25.00 | | | 68.00 | 80.00 | 0.00 |
| Tied * | | | | | | | | | | | | | | |
| 2 Level | 10.00 | 0.00 | 25.00 | 4.00 | 9.00 | 3.50 | 9.00 | 4.00 | 25.00 | | | 68.00 | 80.00 | 0.00 |
| Tied * | | | | | | | | | | | | | | |
| 1 Level | 13.50 | 41.00 | 36.00 | | | | | | | | | 36.00 | 80.00 | 0.00 |
| Tied * | | | | | | | | | | | | | | |
| 0 Level | 10.00 | 41.00 | 36.00 | | | | | | | | | 36.00 | 80.00 | 36.00 |
| Tied * | | | x | | | | | | | | | | | |

<u>* Notes :</u> 1. Wall segment offset defined from edge of diaphragm (Diaphragm offset).
2. Marked automatically with an X if Wall segment is tied to foundation.
**3. After all data is complete , run macro w/ Crtl - w to update spreadsheet.**


## 2. Vertical Wall Distribution and Shear Wall Loads

| Level | Story Force (lbs) | Total Shear (lbs) | Story Shear: To Foundation (lbs) | Story Shear: To Walls (lbs) | Story Shear: Total Shear (lbs) | Wall Length (feet) | Diaphragm Length (feet) | Wall Shear (lbs/ft) | Diaphragm Shear (lbs/ft) |
|---|---|---|---|---|---|---|---|---|---|
| R | 11,539 | | | | | | 80.00 | | 144 |
| | | 11,539 | | 11,539 | | 68.00 | | 170 | |
| 3 | 13,032 | | 0 | | 11,539 | | 80.00 | | 163 |
| | | 24,571 | | 24,571 | | 68.00 | | 361 | |
| 2 | 9,572 | | 0 | | 24,571 | | 80.00 | | 120 |
| | | 34,143 | | 34,143 | | 36.00 | | 948 | |
| 1 | 3,793 | | 0 | | 34,143 | | 80.00 | | 47 |
| | | 37,936 | | 37,936 | | 36.00 | | 1054 | |
| 0 | | | 37,936 | | 37,936 | | | | |

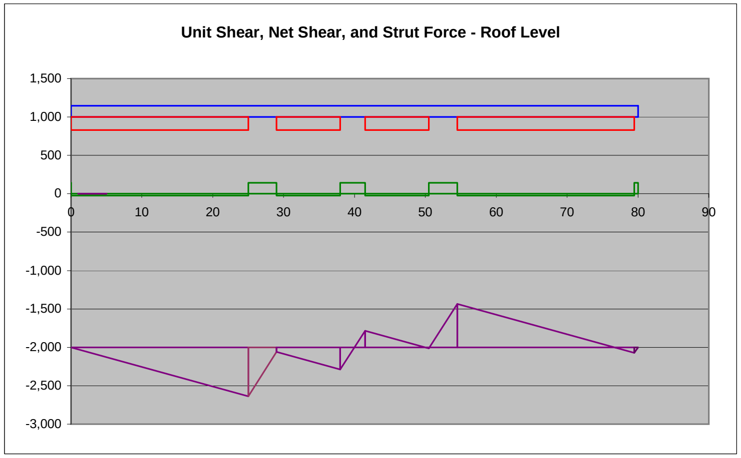
**Notes:** 1. Diaphragm connected to foundation transfers all load to shear walls; diaphragm connection to wall calculated separately (conservative).
2. Load transferred to floor below is proportional to wall length over diaphragm/total wall length;

**SHEARWALL LOAD DISTRIBUTION - FLEXIBLE DIAPHRAGM ANALYSIS**
**NDS - SDPWS 2015 SHEAR WALL CRITERIA**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Wall Location:** Gridline B

Loading: EQ
Loading Direction: N-S

## 3. Plots of Unit and Net Shears and Strut Force at Wall Levels




**R Level Demands:**

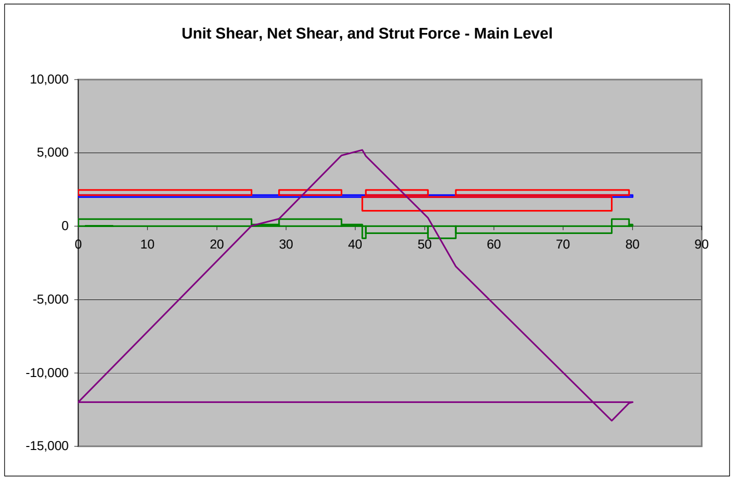
Vsw = 170 lb/ft

$F_{strut}$ = 636 lbs


**3 Level Demands:**

Vsw = 361 lb/ft

$F_{strut}$ = 719 lbs




**2 Level Demands:**

Vsw = 948 lb/ft

Fstrut = 17,191 lbs


**1 Level Demands:**

Vsw = 1,054 lb/ft

Fstrut = 1,944 lbs

**SHEARWALL LOAD DISTRIBUTION - FLEXIBLE DIAPHRAGM ANALYSIS**
**NDS - SDPWS 2015 SHEAR WALL CRITERIA**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Wall Location:** Gridline F

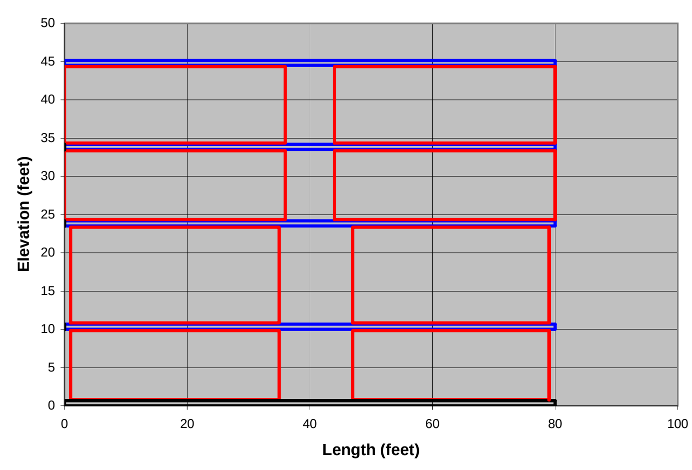
Loading: EQ
Loading Direction: N-S

## 1. Diaphragm and Shear Wall Dimensions along Plane of Assembled Walls

| | Story Forces | | Foundation | | | Diaphragm | | |
|---|---|---|---|---|---|---|---|---|
| Level | Strength Load (lbs) | **Service Load (lbs)** | Offset (feet) | Length (feet) | Edge (feet) | Offset (feet) | Length (feet) | Edge (feet) |
| R | | **13,046** | | | | 0 | 80.00 | 80.00 |
| 3 | | **14,480** | 0 | 0 | 0.00 | 0 | 80.00 | 80.00 |
| 2 | | **10,279** | 0 | 0 | 0.00 | 0 | 80.00 | 80.00 |
| 1 | | **3,773** | 0 | 0.00 | 0.00 | 0.00 | 80.00 | 80.00 |
| 0 | - | | | 80.00 | 80.00 | | - | |

| | | **Wall Segments** | | | | | | | | | | Summation of Segments | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Wall 1 | | Wall 2 | | Wall 3 | | Wall 4 | | Wall 5 | | | | |
| Wall Levels | Wall Height (feet) | Offset* (feet) | Length (feet) | Offset (feet) | Length (feet) | Offset (feet) | Length (feet) | Offset (feet) | Length (feet) | Offset (feet) | Length (feet) | Wall Length (feet) | Floor Length (feet) | Tied to Foundation (feet)* |
| 3 Level | 11.00 | 0.00 | 36.00 | 8.00 | 36.00 | | | | | | | 72.00 | 80.00 | 0.00 |
| Tied * | | | | | | | | | | | | | | |
| 2 Level | 10.00 | 0.00 | 36.00 | 8.00 | 36.00 | | | | | | | 72.00 | 80.00 | 0.00 |
| Tied * | | | | | | | | | | | | | | |
| 1 Level | 13.50 | 1.00 | 34.00 | 12.00 | 32.00 | | | | | | | 66.00 | 80.00 | 0.00 |
| Tied * | | | | | | | | | | | | | | |
| 0 Level | 10.00 | 1.00 | 34.00 | 12.00 | 32.00 | | | | | | | 66.00 | 80.00 | 66.00 |
| Tied * | | | x | | x | | | | | | | | | |

<u>* Notes :</u> 1. Wall segment offset defined from edge of diaphragm (Diaphragm offset).
2. Marked automatically with an X if Wall segment is tied to foundation.
**3. After all data is complete , run macro w/ Crtl - w to update spreadsheet.**




## 2. Vertical Wall Distribution and Shear Wall Loads

| | | | Story Shear | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Level | Story Force (lbs) | Total Shear (lbs) | To Foundation (lbs) | To Walls (lbs) | Total Shear (lbs) | Wall Length (feet) | Diaphragm Length (feet) | Wall Shear (lbs/ft) | Diaphragm Shear (lbs/ft) |
| R | 13,046 | | | | | | 80.00 | | 163 |
| | | 13,046 | | 13,046 | | 72.00 | | 181 | |
| 3 | 14,480 | | 0 | | 13,046 | | 80.00 | | 181 |
| | | 27,526 | | 27,526 | | 72.00 | | 382 | |
| 2 | 10,279 | | 0 | | 27,526 | | 80.00 | | 128 |
| | | 37,805 | | 37,805 | | 66.00 | | 573 | |
| 1 | 3,773 | | 0 | | 37,805 | | 80.00 | | 47 |
| | | 41,578 | | 41,578 | | 66.00 | | 630 | |
| 0 | | | 41,578 | | 41,578 | | | | |

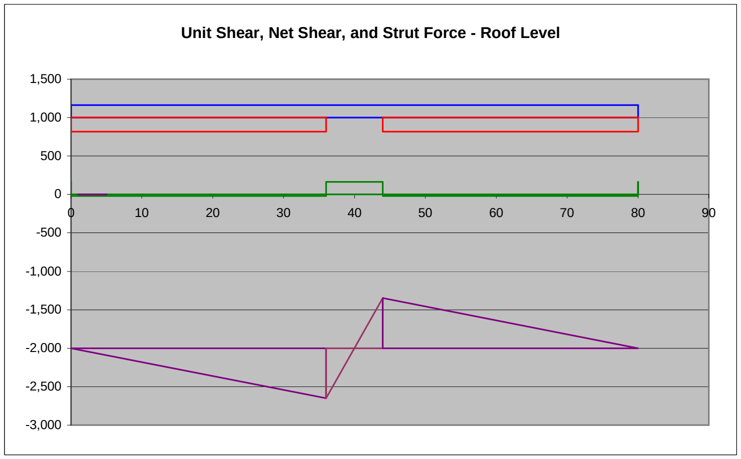
**Notes:** 1. Diaphragm connected to foundation transfers all load to shear walls; diaphragm connection to wall calculated separately (conservative).
2. Load transferred to floor below is proportional to wall length over diaphragm/total wall length;

**SHEARWALL LOAD DISTRIBUTION - FLEXIBLE DIAPHRAGM ANALYSIS**
**NDS - SDPWS 2015 SHEAR WALL CRITERIA**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Wall Location:** Gridline F

Loading: EQ
Loading Direction: N-S

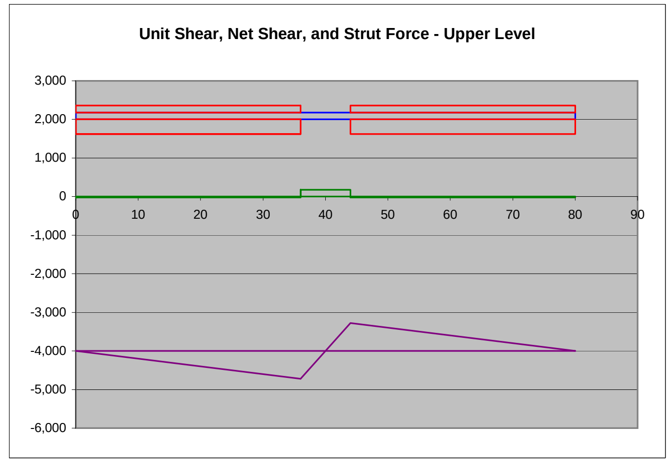
## 3. Plots of Unit and Net Shears and Strut Force at Wall Levels




**R Level Demands:**

Vsw = 181 lb/ft

$F_{strut}$ = 652 lbs




**3 Level Demands:**

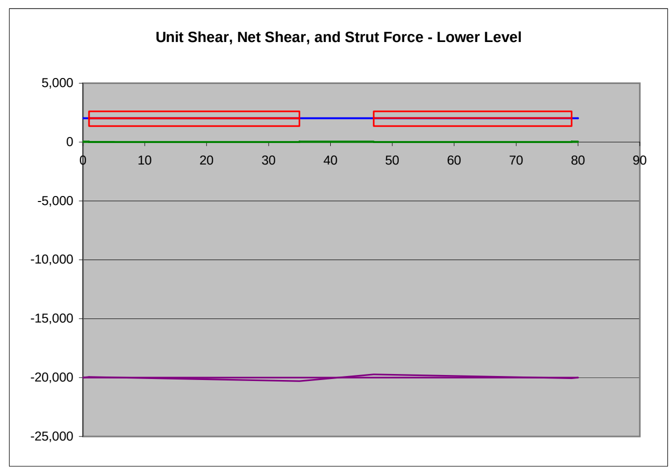
Vsw = 382 lb/ft

$F_{strut}$ = 724 lbs


**2 Level Demands:**

Vsw = 573 lb/ft

Fstrut = 1,598 lbs




**1 Level Demands:**

Vsw = 630 lb/ft

Fstrut = 293 lbs

# FLEXIBLE DIAPHRAGM ANALYSIS
# - W-E LOADING AT GRIDLINES

**SHEARWALL LOAD DISTRIBUTION - FLEXIBLE DIAPHRAGM ANALYSIS**
**NDS - SDPWS 2015 SHEAR WALL CRITERIA**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Wall Location:** Gridline 1

Loading: EQ
Loading Direction: N-S

## 1. Diaphragm and Shear Wall Dimensions along Plane of Assembled Walls

| | Story Forces | | Foundation | | | Diaphragm | | |
|---|---|---|---|---|---|---|---|---|
| Level | Strength Load (lbs) | **Service Load (lbs)** | Offset (feet) | Length (feet) | Edge (feet) | Offset (feet) | Length (feet) | Edge (feet) |
| R | | **7,939** | | | | 0 | 50.00 | 50.00 |
| 3 | | **8,719** | 0 | 0 | 0.00 | 0 | 50.00 | 50.00 |
| 2 | | **6,555** | 0 | 0 | 0.00 | 0 | 50.00 | 50.00 |
| 1 | | **2,400** | 0 | 0.00 | 0.00 | 0.00 | 50.00 | 50.00 |
| 0 | - | | | 50.00 | 50.00 | | - | |

| | | **Wall Segments** | | | | | | | | | | Summation of Segments | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Wall 1 | | Wall 2 | | Wall 3 | | Wall 4 | | Wall 5 | | | | |
| Wall Levels | Wall Height (feet) | Offset* (feet) | Length (feet) | Offset (feet) | Length (feet) | Offset (feet) | Length (feet) | Offset (feet) | Length (feet) | Offset (feet) | Length (feet) | Wall Length (feet) | Floor Length (feet) | Tied to Foundation (feet)* |
| 3 Level Tied * | 11.00 | 12.00 | 3.00 | 3.50 | 2.50 | 3.50 | 5.00 | 3.50 | 4.00 | | | 14.50 | 50.00 | 0.00 |
| 2 Level Tied * | 10.00 | 12.00 | 3.00 | 3.50 | 2.50 | 3.50 | 5.00 | 3.50 | 4.00 | | | 14.50 | 50.00 | 0.00 |
| 1 Level Tied * | 13.50 | 25.50 | 18.50 | | | | | | | | | 18.50 | 50.00 | 0.00 |
| 0 Level Tied * | 10.00 | 25.50 | 18.50 x | | | | | | | | | 18.50 | 50.00 | 18.50 |

* Notes : 1. Wall segment offset defined from edge of diaphragm (Diaphragm offset).
2. Marked automatically with an X if Wall segment is tied to foundation.
**3. After all data is complete , run macro w/ Crtl - w to update spreadsheet.**

## 2. Vertical Wall Distribution and Shear Wall Loads

| | | | Story Shear | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Level | Story Force (lbs) | Total Shear (lbs) | To Foundation (lbs) | To Walls (lbs) | Total Shear (lbs) | Wall Length (feet) | Diaphragm Length (feet) | Wall Shear (lbs/ft) | Diaphragm Shear (lbs/ft) |
| R | 7,939 | | | | | | 50.00 | | 159 |
| | | 7,939 | | 7,939 | | 14.50 | | 548 | |
| 3 | 8,719 | | 0 | | 7,939 | | 50.00 | | 174 |
| | | 16,658 | | 16,658 | | 14.50 | | 1149 | |
| 2 | 6,555 | | 0 | | 16,658 | | 50.00 | | 131 |
| | | 23,213 | | 23,213 | | 18.50 | | 1255 | |
| 1 | 2,400 | | 0 | | 23,213 | | 50.00 | | 48 |
| | | 25,613 | | 25,613 | | 18.50 | | 1384 | |
| 0 | | | 25,613 | | 25,613 | | | | |

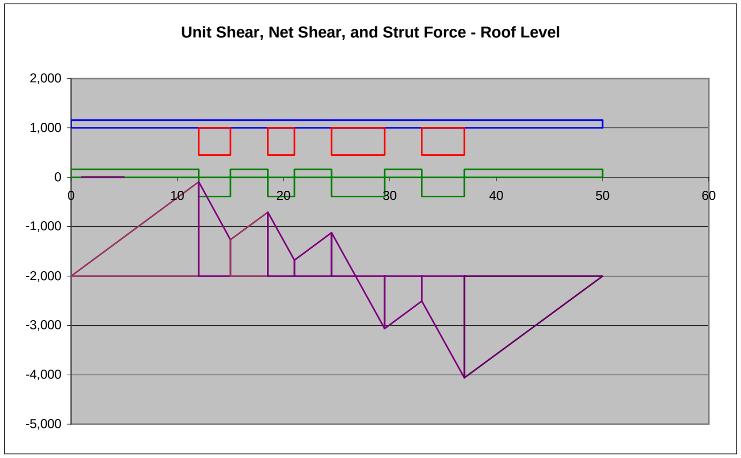

**Notes:** 1. Diaphragm connected to foundation transfers all load to shear walls; diaphragm connection to wall calculated separately (conservative).
2. Load transferred to floor below is proportional to wall length over diaphragm/total wall length;

**SHEARWALL LOAD DISTRIBUTION - FLEXIBLE DIAPHRAGM ANALYSIS**
**NDS - SDPWS 2015 SHEAR WALL CRITERIA**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Wall Location:** Gridline 1

Loading: EQ
Loading Direction: N-S

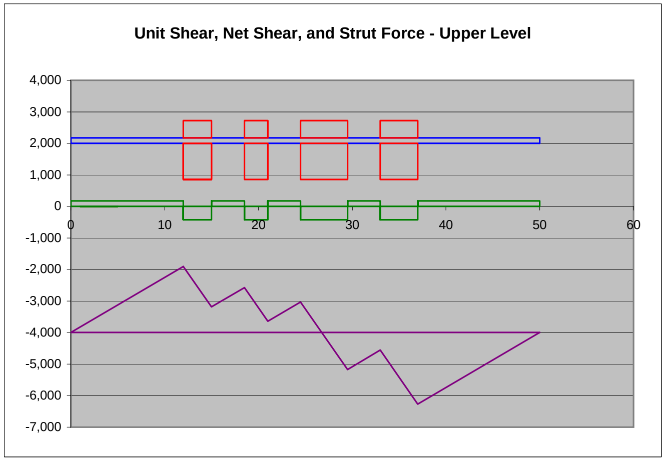
## 3. Plots of Unit and Net Shears and Strut Force at Wall Levels




**R Level Demands:**

Vsw = 548 lb/ft

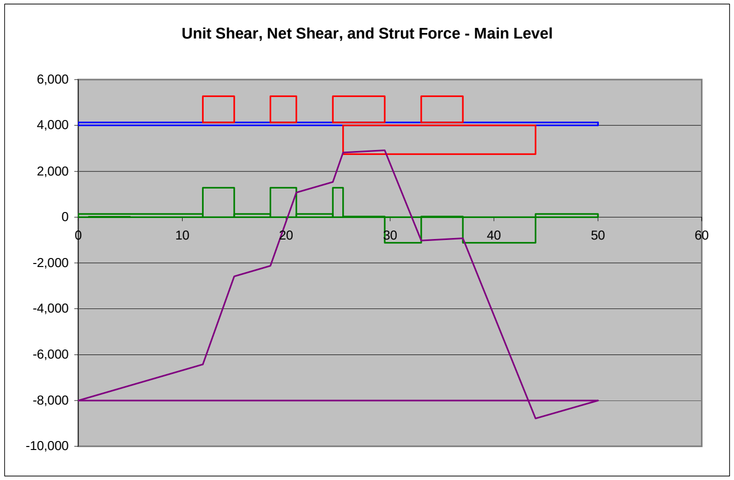
$F_{strut}$ = 2,064 lbs




**3 Level Demands:**

Vsw = 1,149 lb/ft

$F_{strut}$ = 2,267 lbs




**2 Level Demands:**

Vsw = 1,255 lb/ft

Fstrut = 10,911 lbs


**1 Level Demands:**

Vsw = 1,384 lb/ft

Fstrut = 1,224 lbs

**SHEARWALL LOAD DISTRIBUTION - FLEXIBLE DIAPHRAGM ANALYSIS**
**NDS - SDPWS 2015 SHEAR WALL CRITERIA**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Wall Location:** Gridline 3

Loading: EQ
Loading Direction: W-E

## 1. Diaphragm and Shear Wall Dimensions along Plane of Assembled Walls

| | Story Forces | | Foundation | | | Diaphragm | | |
|---|---|---|---|---|---|---|---|---|
| Level | Strength Load (lbs) | **Service Load (lbs)** | Offset (feet) | Length (feet) | Edge (feet) | Offset (feet) | Length (feet) | Edge (feet) |
| R | | **6,581** | | | | 0 | 50.00 | 50.00 |
| 3 | | **7,498** | 0 | 0 | 0.00 | 0 | 50.00 | 50.00 |
| 2 | | **5,400** | 0 | 0 | 0.00 | 0 | 50.00 | 50.00 |
| 1 | | **2,143** | 0 | 0.00 | 0.00 | 0.00 | 50.00 | 50.00 |
| 0 | - | | | 50.00 | 50.00 | | - | |

| | | **Wall Segments** | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Wall 1 | | Wall 2 | | Wall 3 | | Wall 4 | | Wall 5 | |
| Wall Levels | Wall Height (feet) | Offset* (feet) | Length (feet) | Offset (feet) | Length (feet) | Offset (feet) | Length (feet) | Offset (feet) | Length (feet) | Offset (feet) | Length (feet) |
| 3 Level | 11.00 | 0.00 | 3.00 | 3.50 | 3.00 | 2.00 | 12.50 | 4.00 | 14.00 | 4.50 | 3.00 |
| Tied * | | | | | | | | | | | |
| 2 Level | 10.00 | 0.00 | 3.00 | 3.50 | 3.00 | 2.00 | 12.50 | 4.00 | 14.00 | 4.50 | 3.00 |
| Tied * | | | | | | | | | | | |
| 1 Level | 13.50 | 23.50 | 26.00 | | | | | | | | |
| Tied * | | | | | | | | | | | |
| 0 Level | 10.00 | 23.50 | 26.00 | | | | | | | | |
| Tied * | | | x | | | | | | | | |

| Summation of Segments | | |
|---|---|---|
| Wall Length (feet) | Floor Length (feet) | Tied to Foundation (feet)* |
| 35.50 | 50.00 | 0.00 |
| 35.50 | 50.00 | 0.00 |
| 26.00 | 50.00 | 0.00 |
| 26.00 | 50.00 | 26.00 |

* Notes : 1. Wall segment offset defined from edge of diaphragm (Diaphragm offset).
2. Marked automatically with an X if Wall segment is tied to foundation.
**3. After all data is complete , run macro w/ Crtl - w to update spreadsheet.**


## 2. Vertical Wall Distribution and Shear Wall Loads

| | | | Story Shear | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Level | Story Force (lbs) | Total Shear (lbs) | To Foundation (lbs) | To Walls (lbs) | Total Shear (lbs) | Wall Length (feet) | Diaphragm Length (feet) | Wall Shear (lbs/ft) | Diaphragm Shear (lbs/ft) |
| R | 6,581 | | | | | | 50.00 | | 132 |
| | | 6,581 | | 6,581 | | 35.50 | | 185 | |
| 3 | 7,498 | | 0 | | 6,581 | | 50.00 | | 150 |
| | | 14,079 | | 14,079 | | 35.50 | | 397 | |
| 2 | 5,400 | | 0 | | 14,079 | | 50.00 | | 108 |
| | | 19,479 | | 19,479 | | 26.00 | | 749 | |
| 1 | 2,143 | | 0 | | 19,479 | | 50.00 | | 43 |
| | | 21,622 | | 21,622 | | 26.00 | | 832 | |
| 0 | | | 21,622 | | 21,622 | | | | |

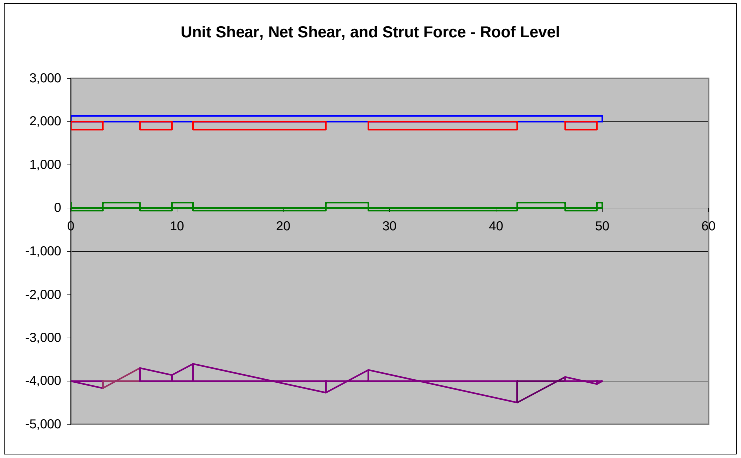
**Notes:** 1. Diaphragm connected to foundation transfers all load to shear walls; diaphragm connection to wall calculated separately (conservative).
2. Load transferred to floor below is proportional to wall length over diaphragm/total wall length;

**SHEARWALL LOAD DISTRIBUTION - FLEXIBLE DIAPHRAGM ANALYSIS**
**NDS - SDPWS 2015 SHEAR WALL CRITERIA**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Wall Location:** Gridline 3

Loading: EQ
Loading Direction: W-E

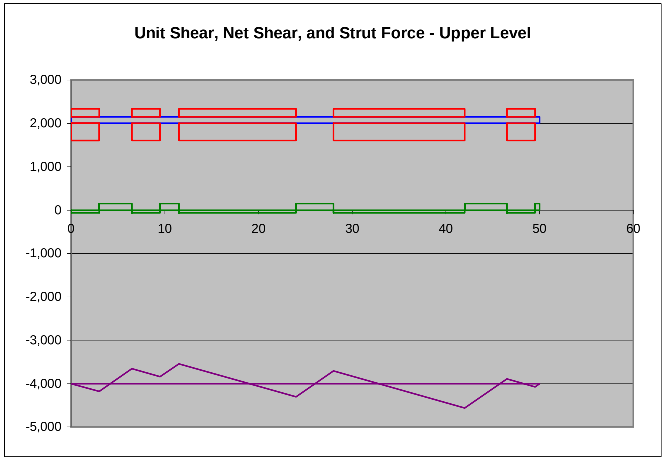
## 3. Plots of Unit and Net Shears and Strut Force at Wall Levels




**R Level Demands:**

Vsw = 185 lb/ft

$F_{strut}$ = 497 lbs




**3 Level Demands:**

Vsw = 397 lb/ft

$F_{strut}$ = 566 lbs


**2 Level Demands:**

Vsw = 749 lb/ft

Fstrut = 9,677 lbs


**1 Level Demands:**

Vsw = 832 lb/ft

Fstrut = 1,007 lbs

**SHEARWALL LOAD DISTRIBUTION - FLEXIBLE DIAPHRAGM ANALYSIS**
**NDS - SDPWS 2015 SHEAR WALL CRITERIA**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Wall Location:** Gridline 4

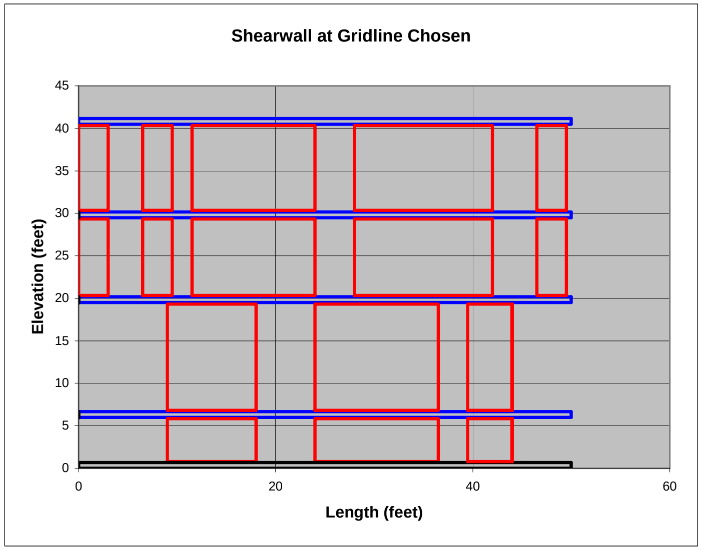
Loading: EQ
Loading Direction: W-E

## 1. Diaphragm and Shear Wall Dimensions along Plane of Assembled Walls

| | Story Forces | | Foundation | | | Diaphragm | | |
|---|---|---|---|---|---|---|---|---|
| Level | Strength Load (lbs) | **Service Load (lbs)** | Offset (feet) | Length (feet) | Edge (feet) | Offset (feet) | Length (feet) | Edge (feet) |
| R | | **6,581** | | | | 0 | 50.00 | 50.00 |
| 3 | | **7,498** | 0 | 0 | 0.00 | 0 | 50.00 | 50.00 |
| 2 | | **4,685** | 0 | 0 | 0.00 | 0 | 50.00 | 50.00 |
| 1 | | **2,099** | 0 | 0.00 | 0.00 | 0.00 | 50.00 | 50.00 |
| 0 | - | | | 50.00 | 50.00 | | - | |

| | | **Wall Segments** | | | | | | | | | | Summation of Segments | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Wall 1 | | Wall 2 | | Wall 3 | | Wall 4 | | Wall 5 | | | | |
| Wall Levels | Wall Height (feet) | Offset* (feet) | Length (feet) | Offset (feet) | Length (feet) | Offset (feet) | Length (feet) | Offset (feet) | Length (feet) | Offset (feet) | Length (feet) | Wall Length (feet) | Floor Length (feet) | Tied to Foundation (feet)* |
| 3 Level | 11.00 | 0.00 | 3.00 | 3.50 | 3.00 | 2.00 | 12.50 | 4.00 | 14.00 | 4.50 | 3.00 | 35.50 | 50.00 | 0.00 |
| Tied * | | | | | | | | | | | | | | |
| 2 Level | 10.00 | 0.00 | 3.00 | 3.50 | 3.00 | 2.00 | 12.50 | 4.00 | 14.00 | 4.50 | 3.00 | 35.50 | 50.00 | 0.00 |
| Tied * | | | | | | | | | | | | | | |
| 1 Level | 13.50 | 9.00 | 9.00 | 6.00 | 12.50 | 3.00 | 4.50 | | | | | 26.00 | 50.00 | 0.00 |
| Tied * | | | | | | | | | | | | | | |
| 0 Level | 6.00 | 9.00 | 9.00 | 6.00 | 12.50 | 3.00 | 4.50 | | | | | 26.00 | 50.00 | 26.00 |
| Tied * | | | x | | x | | x | | | | | | | |

* Notes : 1. Wall segment offset defined from edge of diaphragm (Diaphragm offset).
2. Marked automatically with an X if Wall segment is tied to foundation.
**3. After all data is complete , run macro w/ Crtl - w to update spreadsheet.**




## 2. Vertical Wall Distribution and Shear Wall Loads

| | | | Story Shear | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Level | Story Force (lbs) | Total Shear (lbs) | To Foundation (lbs) | To Walls (lbs) | Total Shear (lbs) | Wall Length (feet) | Diaphragm Length (feet) | Wall Shear (lbs/ft) | Diaphragm Shear (lbs/ft) |
| R | 6,581 | | | | | | 50.00 | | 132 |
| | | 6,581 | | 6,581 | | 35.50 | | 185 | |
| 3 | 7,498 | | 0 | | 6,581 | | 50.00 | | 150 |
| | | 14,079 | | 14,079 | | 35.50 | | 397 | |
| 2 | 4,685 | | 0 | | 14,079 | | 50.00 | | 94 |
| | | 18,764 | | 18,764 | | 26.00 | | 722 | |
| 1 | 2,099 | | 0 | | 18,764 | | 50.00 | | 42 |
| | | 20,863 | | 20,863 | | 26.00 | | 802 | |
| 0 | | | 20,863 | | 20,863 | | | | |

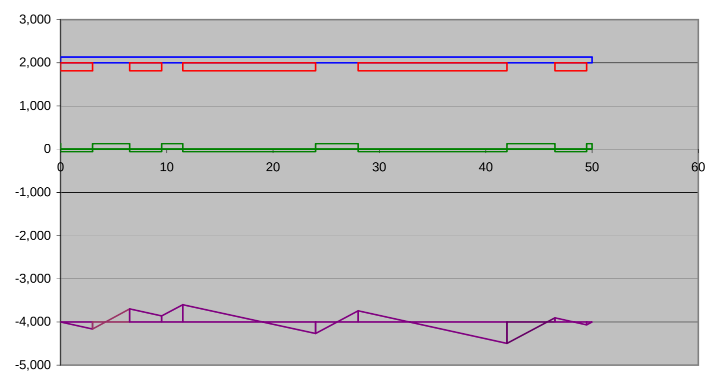
**Notes:** 1. Diaphragm connected to foundation transfers all load to shear walls; diaphragm connection to wall calculated separately (conservative).
2. Load transferred to floor below is proportional to wall length over diaphragm/total wall length;

**SHEARWALL LOAD DISTRIBUTION - FLEXIBLE DIAPHRAGM ANALYSIS**
**NDS - SDPWS 2015 SHEAR WALL CRITERIA**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Wall Location:** Gridline 4

Loading: EQ
Loading Direction: W-E

**3. Plots of Unit and Net Shears and Strut Force at Wall Levels**

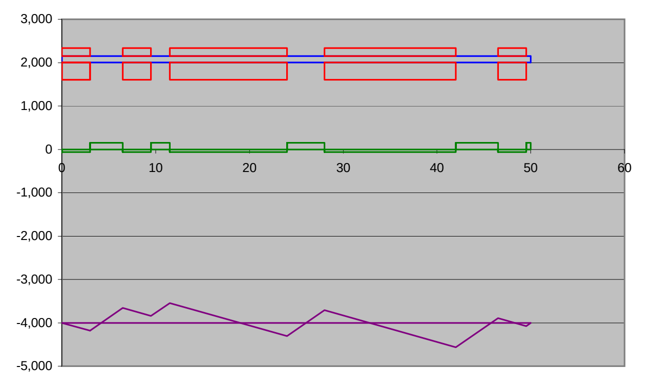


**Unit Shear, Net Shear, and Strut Force - Roof Level**

**R Level Demands:**

Vsw = 185 lb/ft

$F_{strut}$ = 497 lbs

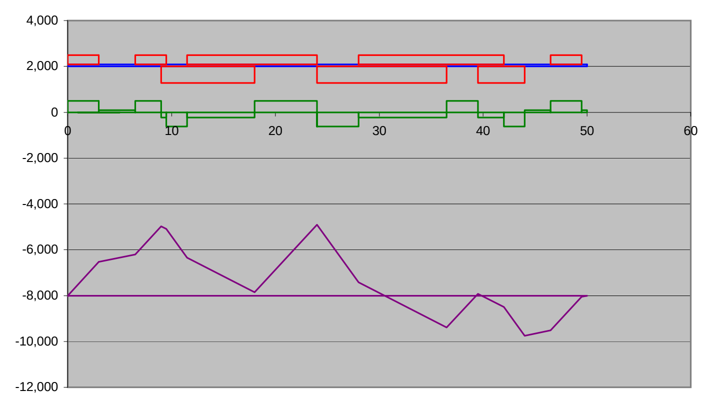


**Unit Shear, Net Shear, and Strut Force - Upper Level**

**3 Level Demands:**

Vsw = 397 lb/ft

$F_{strut}$ = 566 lbs

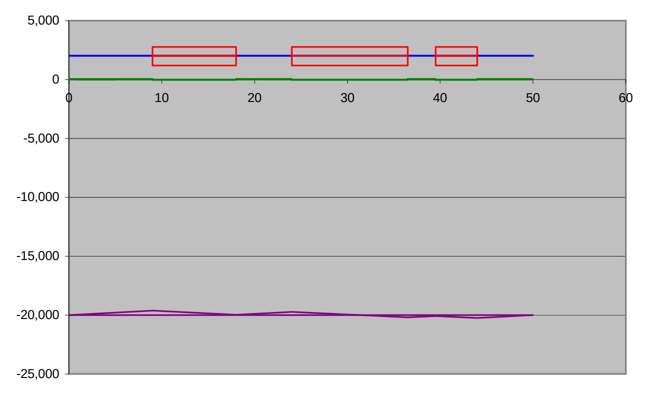


**Unit Shear, Net Shear, and Strut Force - Main Level**

**2 Level Demands:**

Vsw = 722 lb/ft

Fstrut = 3,091 lbs



**Unit Shear, Net Shear, and Strut Force - Lower Level**

**1 Level Demands:**

Vsw = 802 lb/ft

Fstrut = 378 lbs

**SHEARWALL LOAD DISTRIBUTION - FLEXIBLE DIAPHRAGM ANALYSIS**
**NDS - SDPWS 2015 SHEAR WALL CRITERIA**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Wall Location:** Gridline 6

Loading: EQ
Loading Direction: W-E

## 1. Diaphragm and Shear Wall Dimensions along Plane of Assembled Walls

| | Story Forces | | Foundation | | | Diaphragm | | |
|---|---|---|---|---|---|---|---|---|
| Level | Strength Load (lbs) | **Service Load (lbs)** | Offset (feet) | Length (feet) | Edge (feet) | Offset (feet) | Length (feet) | Edge (feet) |
| R | | **6,827** | | | | 0 | 50.00 | 50.00 |
| 3 | | **8,194** | 0 | 0 | 0.00 | 0 | 50.00 | 50.00 |
| 2 | | **6,113** | 0 | 0 | 0.00 | 0 | 50.00 | 50.00 |
| 1 | | **2,064** | 0 | 0.00 | 0.00 | 0.00 | 50.00 | 50.00 |
| 0 | - | | | 50.00 | 50.00 | | - | |

| | | **Wall Segments** | | | | | | | | | | Summation of Segments | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Wall 1 | | Wall 2 | | Wall 3 | | Wall 4 | | Wall 5 | | | | |
| Wall Levels | Wall Height (feet) | Offset* (feet) | Length (feet) | Offset (feet) | Length (feet) | Offset (feet) | Length (feet) | Offset (feet) | Length (feet) | Offset (feet) | Length (feet) | Wall Length (feet) | Floor Length (feet) | Tied to Foundation (feet)* |
| 3 Level | 11.00 | 0.00 | 2.00 | 8.00 | 5.00 | 8.00 | 8.00 | 3.50 | 4.00 | 8.00 | 3.00 | 22.00 | 50.00 | 0.00 |
| Tied * | | | | | | | | | | | | | | |
| 2 Level | 10.00 | 0.00 | 2.00 | 8.00 | 5.00 | 8.00 | 8.00 | 3.50 | 4.00 | 8.00 | 3.00 | 22.00 | 50.00 | 0.00 |
| Tied * | | | | | | | | | | | | | | |
| 1 Level | 13.50 | 9.50 | 10.50 | 13.50 | 10.50 | | | | | | | 21.00 | 50.00 | 0.00 |
| Tied * | | | | | | | | | | | | | | |
| 0 Level | 4.00 | 9.50 | 10.50 | 13.50 | 10.50 | | | | | | | 21.00 | 50.00 | 21.00 |
| Tied * | | | x | | x | | | | | | | | | |

<u>* Notes :</u> 1. Wall segment offset defined from edge of diaphragm (Diaphragm offset).
2. Marked automatically with an X if Wall segment is tied to foundation.
**3. After all data is complete , run macro w/ Crtl - w to update spreadsheet.**


## 2. Vertical Wall Distribution and Shear Wall Loads

| Level | Story Force (lbs) | Total Shear (lbs) | Story Shear: To Foundation (lbs) | Story Shear: To Walls (lbs) | Story Shear: Total Shear (lbs) | Wall Length (feet) | Diaphragm Length (feet) | Wall Shear (lbs/ft) | Diaphragm Shear (lbs/ft) |
|---|---|---|---|---|---|---|---|---|---|
| R | 6,827 | | | | | | 50.00 | | 137 |
| | | 6,827 | | 6,827 | | 22.00 | | 310 | |
| 3 | 8,194 | | 0 | | 6,827 | | 50.00 | | 164 |
| | | 15,021 | | 15,021 | | 22.00 | | 683 | |
| 2 | 6,113 | | 0 | | 15,021 | | 50.00 | | 122 |
| | | 21,134 | | 21,134 | | 21.00 | | 1006 | |
| 1 | 2,064 | | 0 | | 21,134 | | 50.00 | | 41 |
| | | 23,198 | | 23,198 | | 21.00 | | 1105 | |
| 0 | | | 23,198 | | 23,198 | | | | |

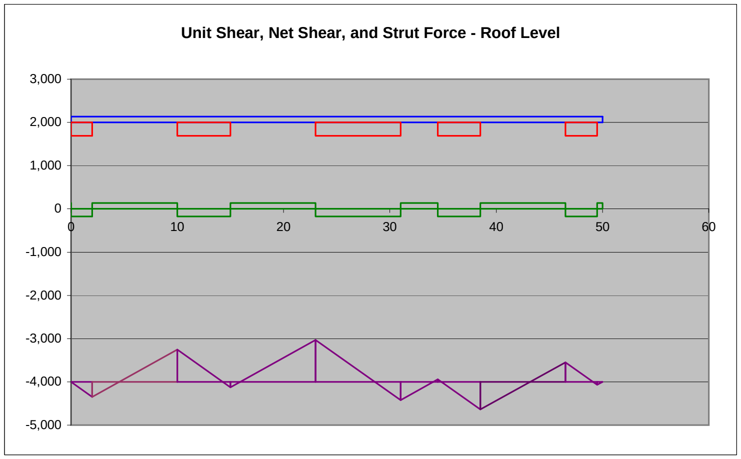
**Notes:** 1. Diaphragm connected to foundation transfers all load to shear walls; diaphragm connection to wall calculated separately (conservative).
2. Load transferred to floor below is proportional to wall length over diaphragm/total wall length;

**SHEARWALL LOAD DISTRIBUTION - FLEXIBLE DIAPHRAGM ANALYSIS**
**NDS - SDPWS 2015 SHEAR WALL CRITERIA**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Wall Location:** Gridline 6

Loading: EQ
Loading Direction: W-E

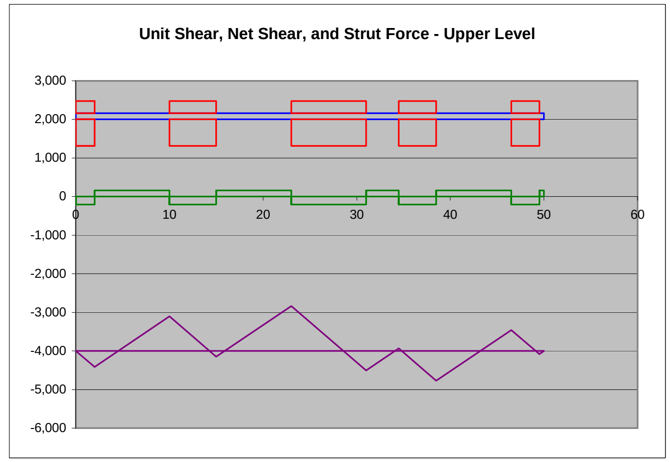
**3. Plots of Unit and Net Shears and Strut Force at Wall Levels**




**R Level Demands:**

Vsw = 310 lb/ft

$F_{strut}$ = 968 lbs




**3 Level Demands:**

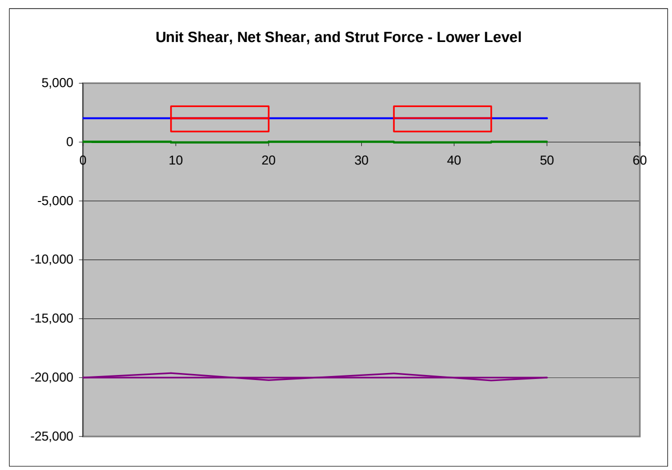
Vsw = 683 lb/ft

$F_{strut}$ = 1,162 lbs


**2 Level Demands:**

Vsw = 1,006 lb/ft

Fstrut = 3,770 lbs




**1 Level Demands:**

Vsw = 1,105 lb/ft

Fstrut = 392 lbs

# SHEAR WALL DESIGN SUMMARY

201812.30
9/13/2018

**SHEARWALL DESIGN SUMMARY - FLEXIBLE DIAPHRAGM ASSUMPTIONS**
**NDS - SDPWS 2015 SHEAR WALL CRITERIA**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

Note: Note: Collector Loads in areas of discontinuities will be amplified by 1.25 as per ASCE 7-10 12.3.3.4 (in Blue), if applicable.

**Table 4.3.4 Maximum Shear Wall Aspect Ratios**

| Shear Wall Sheathing Type | Maximum $h/b_s$ Ratio |
|---|---|
| Wood structural panels, all edges nailed | 3-½:1[1] |
| Particleboard, all edges nailed | 2:1 |
| Diagonal sheathing, conventional | 2:1 |
| Gypsum wallboard | 2:1[2] |
| Portland cement plaster | 2:1[2] |
| Fiberboard | 1-½:1 |

1. For design to resist seismic forces, the shear wall aspect ratio shall not exceed 2:1 unless the nominal unit shear capacity is multiplied by $2b_s/h$.
2. Walls having aspect ratios exceeding 1-½:1 shall be blocked.

** Note: Value reduced by 2w/h for EQ loads for walls with 2.0<=h/b<=3.5 per NDS SDPWS-2015 Table 4.3.4.

Connector Capacities:

Z = 174 lbs (nail shear capacity)
A34 = 412 lbs (Framing angle capacity - Reduced by 1.25 per ASCE 7-10 12.3.3.4)
SDS Screw = 340 lbs (SDS 1/4 x 3 1/2 Screw)
Fanchor = 1,516 lbs (Foundation Anchor capacity)

Assumed for (N) design, modeified for (E) conditions later.

**Sources :** 2012 California Building Code, Table 2306.4.1, Page 324.
Simpson Catalog C-2014.

| | | | | | Wall Dimensions | | | | | Panel Data | | Nail Data | | | Allowable Wall Shear | | | Required Hardware: Shear Walls | | | | | Shearwall to Floor | | Mudsill Anchors | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Loading Direction** | Gridline Wall Location | Floor Level | Normal Gridline | **$F_{MAX}$ (kips)** | Height (feet) | Width (feet) | Service Load (lb/ft) | Collector Force (lbs) | Shearwall Chord Force (lbs) | No. Panels | Thickness (inches) | Size | Edge (inches) | Field (inches) | Tabular Value (lb/ft) | Modified** (lb/ft) | Check | Shearwall Chords | Holdown | Anchor Diameter | Required Coiled Strap Perpendicular to Framing | No. Framing Angles/ Wall | Framing Angle Spacing (inches) | SDS Screw Spacing (inches) | No. Anchors | Anchor Spacing (inches) |
| **N-S** | A | 1L | 2.5 - 5.5 | 9.87 | 12.67 | 42.00 | 235 | 3,987 | 2,977 | 1 | 0.47 | 10d | 6 | 12 | 340 | 340 | **ok** | 2 - 2x6 | HDU4 w/ 2-2x | 5/8" w/ 8.50" Embed | Use CMST14 Strap w/ 16d | 32 - A34" Angle | 16.00 | 16.00 | 16 - 5/8" Bolts | 32 |
| | | B | 2.5 - 5.5 | 11.05 | 4.00 | 42.00 | 263 | 431 | 1,052 | 1 | 0.47 | 10d | 6 | 12 | 340 | 340 | **ok** | 2 - 2x6 | HDU2 w/ 2-2x | 5/8" w/ 8.50" Embed | Use CS18 Strap w/ 10d | 32 - A34" Angle | 16.00 | 14.82 | 16 - 5/8" Bolts | 32 |
| | B | 1L | 3.5 - 6 | 34.13 | 12.67 | 36.00 | 948 | 17,191 | 12,008 | 2 | 0.47 | 10d | 3 | 12 | 1330 | 1,330 | **ok** | 4 - 2x6 | HDU14 w/ 6x | 7/8" w/ 14.00" Embe | Use 2 CMSTC12 Straps w/ 16d | 83 - A34" Angle | 5.14 | 4.24 | 23 - 5/8" Bolts | 19 |
| | | B | 3.5 - 6 | 37.94 | 10.00 | 36.00 | 1,054 | 1,944 | 10,540 | 2 | 0.47 | 10d | 3 | 12 | 1330 | 1,330 | **ok** | 3 - 2x6 | HDU14 w/ 6x | 7/8" w/ 14.00" Embe | Use CMSTC16 Strap w/ 16d | 93 - A34" Angle | 4.60 | 3.82 | 26 - 5/8" Bolts | 17 |
| | F | 1L | 1 - 3 | 19.48 | 12.67 | 34.00 | 573 | 1,598 | 7,258 | 1 | 0.47 | 10d | 3 | 12 | 665 | 665 | **ok** | 3 - 2x6 | HDU14 w/ 3-2x | 3/4" w/ 12.50" Embed | Use CS14 Strap w/ 10d | 48 - A34" Angle | 8.33 | 6.92 | 13 - 5/8" Bolts | 31 |
| | | B | 1 - 3 | 21.42 | 7.00 | 34.00 | 630 | 293 | 4,410 | 1 | 0.47 | 10d | 2 | 12 | 870 | 870 | **ok** | 3 - 2x6 | HDU8 w/ 2-2x | 5/8" w/ 10.00" Embed | Use CS18 Strap w/ 10d | 52 - A34" Angle | 7.70 | 6.38 | 15 - 5/8" Bolts | 27 |
| | | 1L | 4 - 6 | 18.34 | 12.67 | 32.00 | 573 | 1,598 | 7,258 | 1 | 0.47 | 10d | 3 | 12 | 665 | 665 | **ok** | 3 - 2x6 | HDU14 w/ 3-2x | 3/4" w/ 12.50" Embed | Use CS14 Strap w/ 10d | 45 - A34" Angle | 8.35 | 6.98 | 13 - 5/8" Bolts | 30 |
| | | B | 4 - 6 | 20.16 | 7.00 | 32.00 | 630 | 293 | 4,410 | 1 | 0.47 | 10d | 2 | 12 | 870 | 870 | **ok** | 3 - 2x6 | HDU8 w/ 2-2x | 5/8" w/ 10.00" | Use CS18 Strap w/ 10d | 49 - A34" Angle | 7.68 | 6.30 | 14 - 5/8" Bolts | 27 |

201812.30
9/13/2018

**SHEARWALL DESIGN SUMMARY - FLEXIBLE DIAPHRAGM ASSUMPTIONS**
**NDS - SDPWS 2015 SHEAR WALL CRITERIA**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

Note: Note: Collector Loads in areas of discontinuities will be amplified by 1.25 as per ASCE 7-10 12.3.3.4 (in Blue), if applicable.

**Sources :** 2012 California Building Code, Table 2306.4.1, Page 324.
Simpson Catalog C-2014.

**Table 4.3.4 Maximum Shear Wall Aspect Ratios**

| Shear Wall Sheathing Type | Maximum h/$b_s$ Ratio |
|---|---|
| Wood structural panels, all edges nailed | 3-½:1[1] |
| Particleboard, all edges nailed | 2:1 |
| Diagonal sheathing, conventional | 2:1 |
| Gypsum wallboard | 2:1[2] |
| Portland cement plaster | 2:1[2] |
| Fiberboard | 1-½:1 |

1. For design to resist seismic forces, the shear wall aspect ratio shall not exceed 2:1 unless the nominal unit shear capacity is multiplied by $2b_s/h$.
2. Walls having aspect ratios exceeding 1-½:1 shall be blocked.

** Note: Value reduced by 2w/h for EQ loads for walls with 2.0<=h/b<=3.5 per NDS SDPWS-2015 Table 4.3.4.

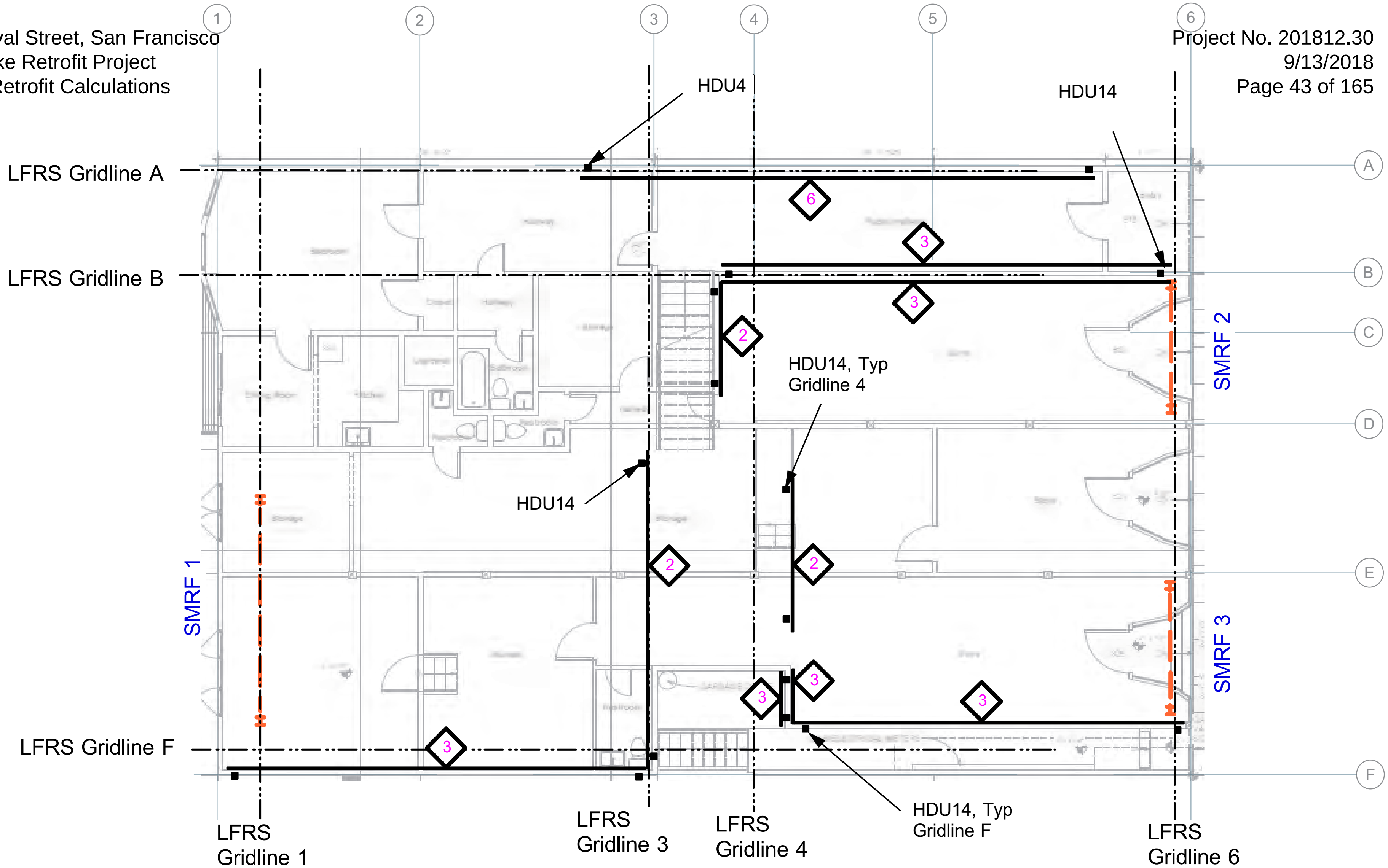
Connector Capacities:

| | | |
|---|---|---|
| Z = | 174 | lbs (nail shear capacity) |
| A34 = | 412 | lbs (Framing angle capacity - Reduced by 1.25 per ASCE 7-10 12.3.3.4) |
| SDS Screw = | 340 | lbs (SDS 1/4 x 3 1/2 Screw) |
| Fanchor = | 1,516 | lbs (Foundation Anchor capacity) |

Assumed for (N) design, modeified for (E) conditions later.

| | | | | | Wall Dimensions | | | | | Panel Data | | Nail Data | | | Allowable Wall Shear | | | Required Hardware | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | | | | | | | | Shear Walls | | | | Shearwall to Floor | | | Mudsill Anchors | |
| Loading Direction | Gridline Wall Location | Floor Level | Normal Gridline | $F_{MAX}$ (kips) | Height (feet) | Width (feet) | Service Load (lb/ft) | Collector Force (lbs) | Shearwall Chord Force (lbs) | No. Panels | Thickness (inches) | Size | Edge (inches) | Field (inches) | Tabular Value (lb/ft) | Modified** (lb/ft) | Check | Shearwall Chords | Holdown | Anchor Diameter | Required Coiled Strap Perpendicular to Framing | No. Framing Angles/ Wall | Framing Angle Spacing (inches) | SDS Screw Spacing (inches) | No. Anchors | Anchor Spacing (inches) |
| **W-E** | 1 | 1L | D - F | 23.22 | 12.67 | 18.50 | 1,255 | 8,320 | 15,897 | Simpson FB SMRF 1 | | | | | | - | - | - | - | - | Use CMST12 Strap w/ 16d | - | - | - | - | - |
| | | B | C - D | 5.53 | 4.00 | 9.00 | 614 | 1,224 | 2,456 | 1 | 0.47 | 10d | 2 | 12 | 870 | 870 | **ok** | 2 - 2x6 | HDU4 w/ 2-2x | 5/8" w/ 8.50" Embed | Use CS16 Strap w/ 10d | 14 - A34" Angle | 7.20 | 6.00 | 4 - 5/8" Bolts | 27 |
| | | B | D - E | 5.53 | 4.00 | 9.00 | 614 | 1,224 | 2,456 | 1 | 0.47 | 10d | 2 | 12 | 870 | 870 | **ok** | 2 - 2x6 | HDU4 w/ 2-2x | 5/8" w/ 8.50" Embed | Use CS16 Strap w/ 10d | 14 - A34" Angle | 7.20 | 6.00 | 4 - 5/8" Bolts | 27 |
| | 3 | 1L | C - D | 19.47 | 12.67 | 26.00 | 749 | 9,677 | 9,487 | 1 | 0.47 | 10d | 2 | 12 | 870 | 870 | **ok** | 3 - 2x6 | HDU14 w/ 3-2x | 7/8" w/ 14.00" Embe | Use 2 CMSTC12 Straps w/ 16d | 48 - A34" Angle | 6.37 | 5.29 | 13 - 5/8" Bolts | 24 |
| | | B | C - D | 21.63 | 2.00 | 26.00 | 832 | 1,007 | 1,664 | 2 | 0.47 | 10d | 4 | 12 | 1060 | 1,060 | **ok** | 2 - 2x6 | HDU2 w/ 2-2x | 5/8" w/ 8.50" Embed | Use CS18 Strap w/ 10d | 53 - A34" Angle | 5.78 | 4.80 | 15 - 5/8" Bolts | 21 |
| | 4 | 1L | B - C.5 | 6.50 | 12.67 | 9.00 | 722 | 3,091 | 9,145 | 1 | 0.47 | 10d | 2 | 12 | 870 | 870 | **ok** | 3 - 2x6 | HDU14 w/ 3-2x | 7/8" w/ 14.00" Embe | Use CMSTC16 Strap w/ 16d | 16 - A34" Angle | 6.35 | 5.14 | 5 - 5/8" Bolts | 22 |
| | | B | B - C.5 | 7.22 | 7.00 | 9.00 | 802 | 378 | 5,614 | 2 | 0.47 | 10d | 4 | 12 | 1060 | 1,060 | **ok** | 3 - 2x6 | HDU8 w/ 3-2x | 5/8" w/ 10.00" Embed | Use CS18 Strap w/ 10d | 18 - A34" Angle | 5.68 | 4.70 | 5 - 5/8" Bolts | 22 |
| | | 1L | D.5 - E.5 | 9.03 | 12.67 | 12.50 | 722 | 3,091 | 9,145 | 1 | 0.47 | 10d | 2 | 12 | 870 | 870 | **ok** | 3 - 2x6 | HDU14 w/ 3-2x | 7/8" w/ 14.00" Embe | Use CMSTC16 Strap w/ 16d | 22 - A34" Angle | 6.52 | 5.36 | 6 - 5/8" Bolts | 25 |
| | | B | D.5 - E.5 | 10.03 | 7.00 | 12.50 | 802 | 378 | 5,614 | 2 | 0.47 | 10d | 4 | 12 | 1060 | 1,060 | **ok** | 3 - 2x6 | HDU8 w/ 3-2x | 5/8" w/ 10.00" Embed | Use CS18 Strap w/ 10d | 25 - A34" Angle | 5.77 | 4.84 | 7 - 5/8" Bolts | 21 |
| | | 1L | E.5 - E.8 | 3.25 | 12.67 | 4.50 | 722 | 3,091 | 9,145 | 2 | 0.47 | 10d | 3 | 12 | 1330 | 945 | **ok** | 3 - 2x6 | HDU14 w/ 3-2x | 7/8" w/ 14.00" Embe | Use CMSTC16 Strap w/ 16d | 8 - A34" Angle | 6.00 | 4.91 | 3 - 5/8" Bolts | 18 |
| | | B | E.5 - E.8 | 3.61 | 7.00 | 4.50 | 802 | 378 | 5,614 | 2 | 0.47 | 10d | 3 | 12 | 1330 | 1,330 | **ok** | 3 - 2x6 | HDU8 w/ 3-2x | 5/8" w/ 10.00" Embed | Use CS18 Strap w/ 10d | 9 - A34" Angle | 5.40 | 4.50 | 3 - 5/8" Bolts | 18 |
| | 6 | 1L | B - D | 10.58 | 12.67 | 10.50 | 1,008 | 2,875 | 12,768 | Simpson FB SMRF 2 | | | | | | - | - | - | - | - | Use CMSTC16 Strap w/ 16d | - | - | - | - | - |
| | | B | B - D | 5.53 | - | 11.00 | 502 | - | - | Diaphragm Supported Edge at Retaining Wall | | | | | | - | - | - | - | - | - | 14 - A34" Angle | 8.80 | - | 5 - 5/8" Bolts | 32 |
| | | 1L | E - F | 10.58 | 12.67 | 10.50 | 1,008 | 2,875 | 12,768 | Simpson FB SMRF 3 | | | | | | - | - | - | - | - | Use CMSTC16 Strap w/ 16d | - | - | - | - | - |
| | | B | E - F | 5.53 | - | 11.00 | 502 | - | - | Diaphragm Supported Edge at Retaining Wall | | | | | | - | - | - | - | - | - | 14 - A34" Angle | 8.80 | - | 5 - 5/8" Bolts | 32 |

# RETROFIT PLANS




## FIRST FLOOR LEVEL - RETROFIT PLAN

NTS

Lateral Force Resisting System (LFRS) Elements :

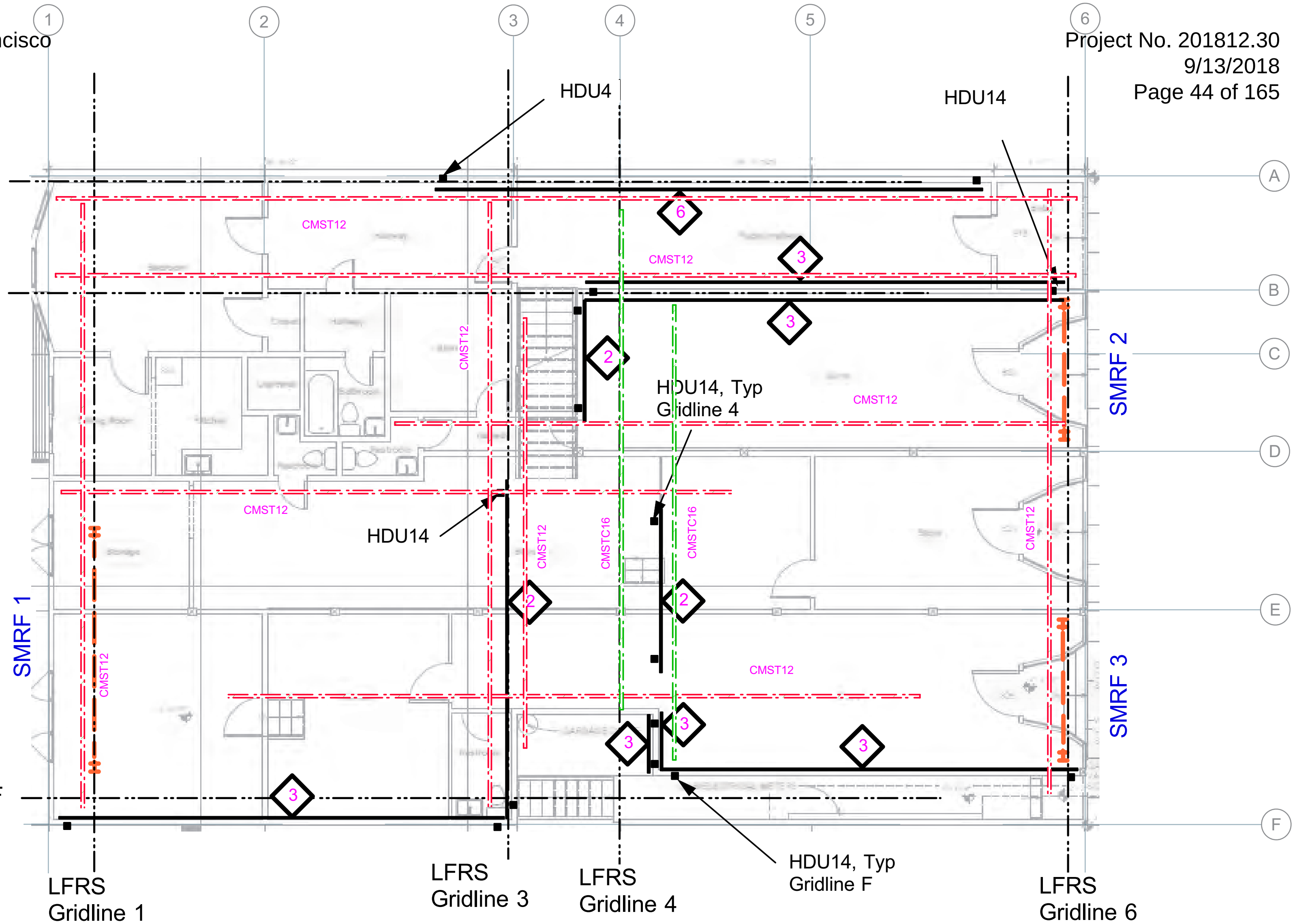
(N) Shear Wall

Simpson Strong Frame

**RETROFIT LEGEND:**

3 — 1/2" Struct I Sheathing w/ 10d Edge Nailing

Holdown each side, as indicated




## FIRST FLOOR LEVEL - RETROFIT PLAN

NTS

**Lateral Force Resisting System (LFRS) Elements :**

(N) Shear Wall

Simpson Strong Frame

**RETROFIT LEGEND:**

3 — 1/2" Struct I Sheathing w/ 10d Edge Nailing

■ Holdown each side, as indicated

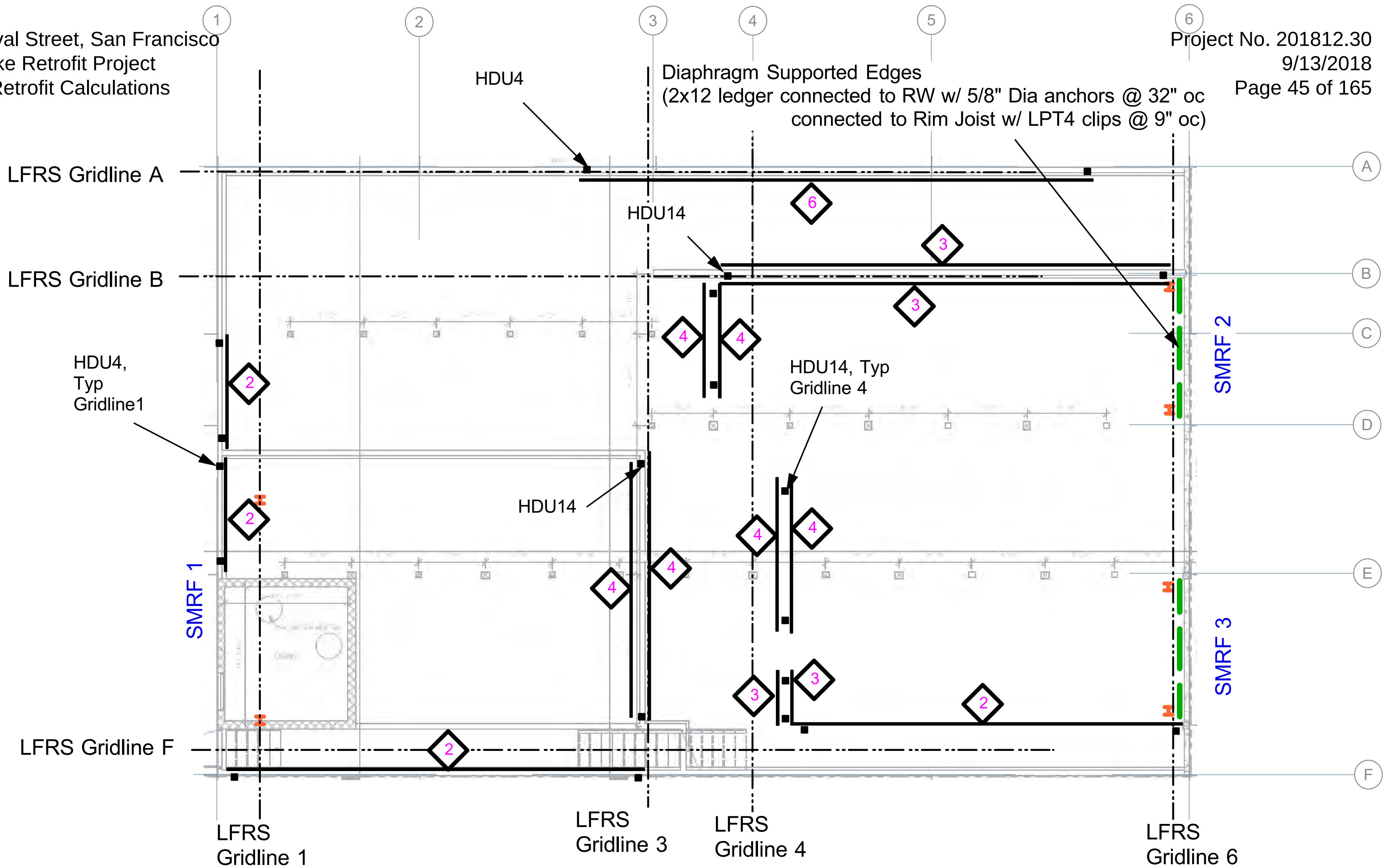
**STRAP LEGEND:**

Note: Straps in N-S direction can be replaced with 6 - 3/4" carriage bolts at Floor joist splices at lines of support.

CS14 — CS14 Strap w/ 10d (2,490 #)

CMSTC16 — CMSTC16 Strap w/ 16d (4,585 #)

CMST12 — CMST12 Strap w/ 16d (9,215 #)




## BASEMENT LEVEL - RETROFIT PLAN

NTS

Lateral Force Resisting System (LFRS) Elements :

(N) Shear Wall

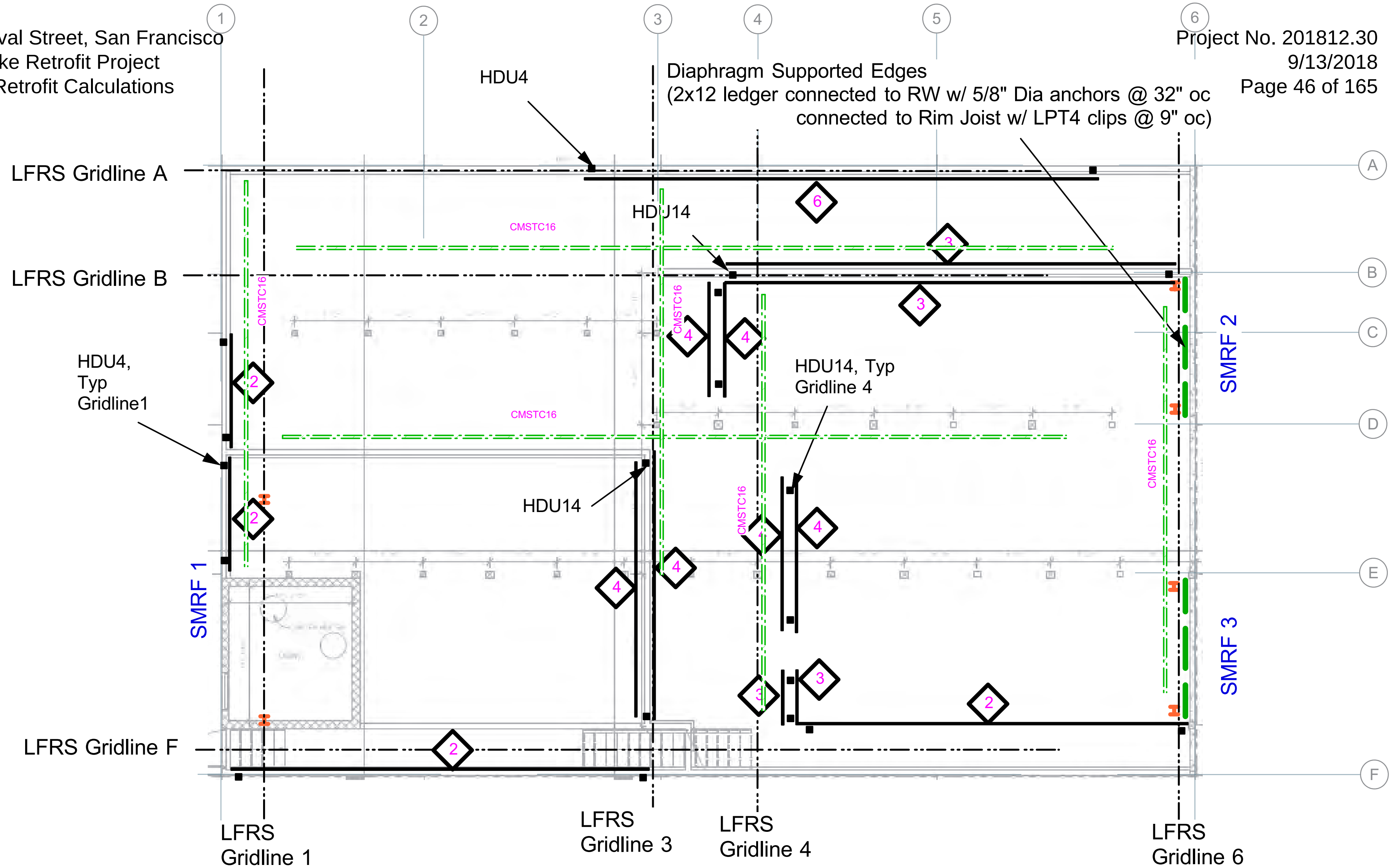
Simpson Strong Frame

Diaphragm Supported Edges
at Retaining Wall

**RETROFIT LEGEND:**

3 1/2" Struct I Sheathing w/ 10d Edge Nailing

Holdown each side, as indicated




## BASEMENT LEVEL - RETROFIT PLAN

NTS

Lateral Force Resisting System (LFRS) Elements :

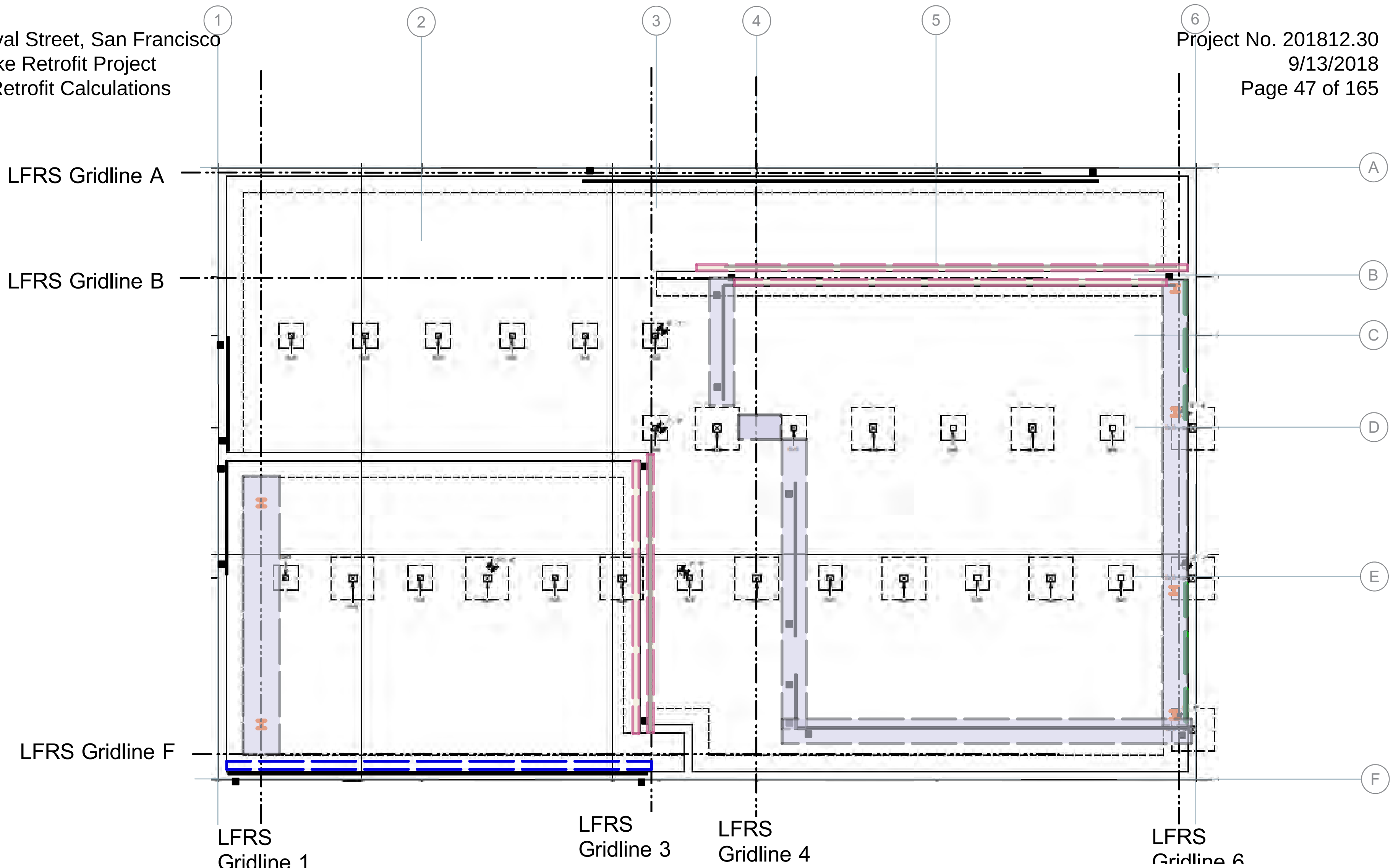
- (N) Shear Wall
- Simpson Strong Frame
- Diaphragm Supported Edges at Retaining Wall

**RETROFIT LEGEND:**

- 3 — 1/2" Struct I Sheathing w/ 10d Edge Nailing
- ■ — Holdown each side, as indicated

**STRAP LEGEND:** Note: Straps in N-S direction can be replaced with 6 - 3/4" carriage bolts at Floor joist splices at lines of support.

- CS14 — CS14 Strap w/ 10d (2,490 #)
- CMSTC16 — CMSTC16 Strap w/ 16d (4,585 #)
- CMST12 — CMST12 Strap w/ 16d (9,215 #)




FOUNDATION RETROFIT PLAN - LEVEL 0

(E) FOUNDATION PLAN - SHEET S1

NTS

Lateral Force Resisting System (LFRS) Elements :

- (N) Shear Wall
- Simpson Strong Frame
- Diaphragm Supported Edges at Retaining Wall

FOUNDATION RETROFIT ELEMENTS:

- (N) Footings
- (N) 2-Sided Wall Retrofit
- (N) 1-Sided Wall Retrofit

# COMPARISON OF FLOOR STRENGTHS
- LEVEL 1 AND 2

**COMPARISON OF EXPECTED STRENGTHS - FLOOR LEVELS 2 AND 1**
**2013 SAN FRANCISCO BUILDING CODE**
**816 TAVARAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**1. Summary of Expected Strength Floor Strengths - N-S Direction**

| | | Expected Composite Strength (Kips) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | **Floor Level** | Drift (%) | | | | | | | | |
| | | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 |
| N-S | 2 | 365 | 447 | 388 | 409 | 21 | 23 | 25 | 0 | 0 |
| | 1 | 270 | 335 | 392 | 438 | 322 | 321 | 321 | 242 | 0 |
| | Story Force Ratio = | 0.74 | 0.75 | 1.01 | 1.07 | 15.44 | 14.20 | 13.02 | 0.00 | 0.00 |
| | | NG | NG | OK | OK | NG | NG | NG | | |

Average = 5.14 (for all Drift Values)

**Upper Floor Level looses strength**

**2. Summary of Expected Strength Floor Strengths - W-E Direction**

| | | Expected Composite Strength (Kips) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | **Floor Level** | Drift (%) | | | | | | | | |
| | | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 |
| W-E | 2 | 232 | 282 | 222 | 208 | 2 | 2 | 2 | 0 | 0 |
| | 1 | 121 | 152 | 182 | 208 | 200 | 201 | 204 | 178 | 74 |
| | Story Force Ratio = | 0.52 | 0.54 | 0.82 | 1.00 | 95.17 | 88.35 | 82.11 | 0.00 | 0.00 |
| | | NG | NG | OK | OK | NG | NG | NG | | |

Average = 29.84 (for all Drift Values)

**COMPARISON OF EXPECTED STRENGTHS - FLOOR LEVELS 2 AND 1**
**2013 SAN FRANCISCO BUILDING CODE**
**816 TAVARAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Table B5.1.1 Expected Strength for Load-Drift Curves (plf)**

| Sheathing ID | Sheathing Material | Drift (%) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 |
| 1 | Stucco | 333 | 320 | 262 | 0 | - | - | - | - | - |
| 2 | Horizontal Wood Sheathing or Wood Siding | 85 | 96 | 110 | 132 | 145 | 157 | 171 | 0 | - |
| 3 | Diagonal Wood Sheathing | 429 | 540 | 686 | 913 | 0 | - | - | - | - |
| 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - |
| 5 | Plywood panel siding (T1-11), 6d @ 6"oc | 354 | 420 | 496 | 549 | 565 | 505 | 449 | 0 | - |
| 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 |
| 7 | Plaster on Gypsum Lath | 402 | 347 | 304 | 0 | - | - | - | - | - |
| 8 | WSP, 8d @ 6" oc | 521 | 621 | 732 | 812 | 836 | 745 | 686 | 0 | - |
| 9 | WSP, 8d @ 4" oc | 513 | 684 | 826 | 943 | 1,018 | 1,080 | 1,112 | 798 | 0 |
| 10 | WSP, 8d @ 3" oc | 1,072 | 1,195 | 1,318 | 1,482 | 1,612 | 1,664 | 1,686 | 1,638 | 0 |
| 11 | WSP, 8d @ 2" oc | 1,393 | 1,553 | 1,713 | 1,926 | 2,096 | 2,163 | 2,192 | 2,130 | 0 |
| 12 | WSP, 10d @ 6" oc | 548 | 767 | 946 | 1,023 | 1,038 | 1,055 | 1,065 | 843 | 0 |
| 13 | WSP, 10d @ 4" oc | 707 | 990 | 1,275 | 1,420 | 1,466 | 1,496 | 1,496 | 1,185 | 0 |
| 14 | WSP, 10d @ 3" oc | 940 | 1,316 | 1,696 | 1,889 | 1,949 | 1,990 | 1,990 | 1,576 | 0 |
| 15 | WSP, 10d @ 2" oc | 1,120 | 1,568 | 1,999 | 2,248 | 2,405 | 2,512 | 2,512 | 2,231 | 0 |
| 16 | Moment Frame | 1,200 | 1,600 | 2,100 | 2,250 | 2,500 | 2,650 | 2,800 | 3,000 | 3,200 |
| 17 | Moment Frame | 625 | 875 | 1,250 | 1,875 | 2,110 | 2,248 | 2,386 | 2,662 | 2,800 |
| 18 | Moment Frame | 250 | 350 | 500 | 750 | 844 | 899 | 954 | 1,065 | 1,120 |
| 19 | Steel Strong Wall | 350 | 489 | 699 | 1,049 | 1,180 | 1,257 | 1,335 | 1,489 | 0 |

**B5.1.1.1. Wall assemblies without wood structural panel sheathing.** The assembly load drift curve is the sum of the load drift curves for each of the sheathing layers.

**B5.1.1.2. Wall assemblies with wood structural panel sheathing.** The assembly load drift curve is whichever of the following two load-drift curves has the larger peak strength:

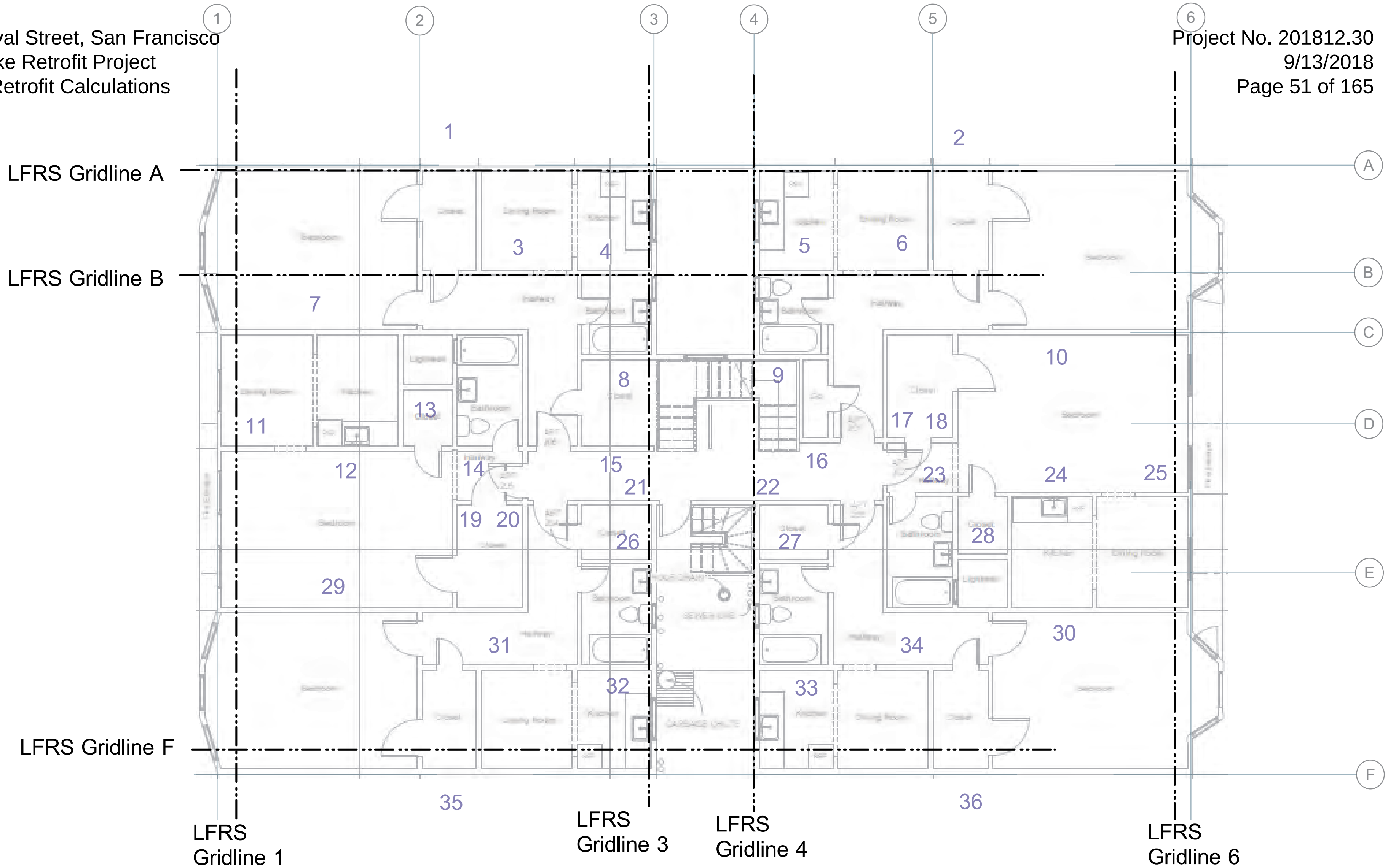
1. The assembly load-drift curve using 50 percent of the strength of the wood structural panel layers and 100 percent of the strength of the other sheathing materials.

2. The assembly load-drift curve using 100 percent of the strength of the wood structural panel layers and 50 percent of the strength of the other sheathing materials.

**B5.1.2. Wall line assignment.** Each segment of sheathed wall framing within a story shall be assigned to a wall line. Wall lines shall satisfy the following rules:

1. Full-height wall segments separated by window or door openings but connected by sheathed segments and continuous framing above or below the opening shall be assigned to the same wall line, unless other rules require them to be treated separately.

2. Wall segments assigned to the same wall line shall not be offset out-of-plane from adjacent segments by more than four feet.




(E) LEVEL 2 FLOOR PLAN - SHEET A3
NTS

**LEVEL 2 - N-S WALL ID'S**

**EXPECTED STRENGTH OF LEVEL 2 WALLS - N-S DIRECTION**
**2013 SAN FRANCISCO BUILDING CODE**
**816 TAVARAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

Floor Level = 2

Loading Direction = N-S

**1. Expected Strength of Floor Level 2 Walls - N-S Direction**

| Wall ID | Wall Length (feet) | No. Layers | Sheathing ID | Sheathing Material | Expected Strength Per Layer (plf) | | | | | | | | | Expected Composite Strength (lbs) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Drift (%) | | | | | | | | | Drift (%) | | | | | | | | |
| | | | | | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 |
| | | 1 | 3 | iagonal Wood Sheathing | 429 | 540 | 686 | 913 | 0 | - | - | - | - | | | | | | | | | |
| 1 | 36.00 | 1 | 2 | Wood Sheathing or Woo | 85 | 96 | 110 | 132 | 145 | 157 | 171 | 0 | - | 34,344 | 42,264 | 43,560 | 51,696 | 5,220 | 5,652 | 6,156 | 0 | 0 |
| | | 1 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | | | | | | | | | |
| | | 1 | 3 | iagonal Wood Sheathing | 429 | 540 | 686 | 913 | 0 | - | - | - | - | | | | | | | | | |
| 2 | 36.00 | 1 | 2 | Wood Sheathing or Woo | 85 | 96 | 110 | 132 | 145 | 157 | 171 | 0 | - | 34,344 | 42,264 | 43,560 | 51,696 | 5,220 | 5,652 | 6,156 | 0 | 0 |
| | | 1 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | | | | | | | | | |
| 3 | 6.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 5,280 | 6,456 | 4,968 | 4,692 | 0 | 0 | 0 | 0 | 0 |
| 4 | 7.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 6,160 | 7,532 | 5,796 | 5,474 | 0 | 0 | 0 | 0 | 0 |
| 5 | 7.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 6,160 | 7,532 | 5,796 | 5,474 | 0 | 0 | 0 | 0 | 0 |
| 6 | 6.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 5,280 | 6,456 | 4,968 | 4,692 | 0 | 0 | 0 | 0 | 0 |
| 7 | 25.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 22,000 | 26,900 | 20,700 | 19,550 | 0 | 0 | 0 | 0 | 0 |
| 8 | 9.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 7,920 | 9,684 | 7,452 | 7,038 | 0 | 0 | 0 | 0 | 0 |
| 9 | 8.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 7,040 | 8,608 | 6,624 | 6,256 | 0 | 0 | 0 | 0 | 0 |
| 10 | 25.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 22,000 | 26,900 | 20,700 | 19,550 | 0 | 0 | 0 | 0 | 0 |
| 11 | 4.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 3,520 | 4,304 | 3,312 | 3,128 | 0 | 0 | 0 | 0 | 0 |
| 12 | 9.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 7,920 | 9,684 | 7,452 | 7,038 | 0 | 0 | 0 | 0 | 0 |

**EXPECTED STRENGTH OF LEVEL 2 WALLS - N-S DIRECTION**
**2013 SAN FRANCISCO BUILDING CODE**
**816 TAVARAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

Floor Level = 2

Loading Direction = N-S

**1. Expected Strength of Floor Level 2 Walls - N-S Direction**

| Wall ID | Wall Length (feet) | No. Layers | Sheathing ID | Sheathing Material | Expected Strength Per Layer (plf) | | | | | | | | | Expected Composite Strength (lbs) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Drift (%) | | | | | | | | | Drift (%) | | | | | | | | |
| | | | | | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 |
| 13 | 4.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 3,520 | 4,304 | 3,312 | 3,128 | 0 | 0 | 0 | 0 | 0 |
| 14 | 4.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 3,520 | 4,304 | 3,312 | 3,128 | 0 | 0 | 0 | 0 | 0 |
| 15 | 6.50 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 5,720 | 6,994 | 5,382 | 5,083 | 0 | 0 | 0 | 0 | 0 |
| 16 | 3.50 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 3,080 | 3,766 | 2,898 | 2,737 | 0 | 0 | 0 | 0 | 0 |
| 17 | 2.50 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 2,200 | 2,690 | 2,070 | 1,955 | 0 | 0 | 0 | 0 | 0 |
| 18 | 2.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 1,760 | 2,152 | 1,656 | 1,564 | 0 | 0 | 0 | 0 | 0 |
| 19 | | | | | | | | | | | | | | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 20 | 2.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 1,760 | 2,152 | 1,656 | 1,564 | 0 | 0 | 0 | 0 | 0 |
| 21 | 7.50 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 6,600 | 8,070 | 6,210 | 5,865 | 0 | 0 | 0 | 0 | 0 |
| 22 | 12.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 10,560 | 12,912 | 9,936 | 9,384 | 0 | 0 | 0 | 0 | 0 |
| 23 | 4.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 3,520 | 4,304 | 3,312 | 3,128 | 0 | 0 | 0 | 0 | 0 |
| 24 | 8.50 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 7,480 | 9,146 | 7,038 | 6,647 | 0 | 0 | 0 | 0 | 0 |

**EXPECTED STRENGTH OF LEVEL 2 WALLS - N-S DIRECTION**
**2013 SAN FRANCISCO BUILDING CODE**
**816 TAVARAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

Floor Level = 2

Loading Direction = N-S

**1. Expected Strength of Floor Level 2 Walls - N-S Direction**

| | | | | | Expected Strength Per Layer (plf) | | | | | | | | | Expected Composite Strength (lbs) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Drift (%) | | | | | | | | | Drift (%) | | | | | | | | |
| **Wall ID** | **Wall Length (feet)** | No. Layers | Sheathing ID | Sheathing Material | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 |
| 25 | 4.50 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 3,960 | 4,842 | 3,726 | 3,519 | 0 | 0 | 0 | 0 | 0 |
| 26 | 6.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 5,280 | 6,456 | 4,968 | 4,692 | 0 | 0 | 0 | 0 | 0 |
| 27 | 5.50 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 4,840 | 5,918 | 4,554 | 4,301 | 0 | 0 | 0 | 0 | 0 |
| 28 | 4.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 3,520 | 4,304 | 3,312 | 3,128 | 0 | 0 | 0 | 0 | 0 |
| 29 | 25.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 22,000 | 26,900 | 20,700 | 19,550 | 0 | 0 | 0 | 0 | 0 |
| 30 | 25.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 22,000 | 26,900 | 20,700 | 19,550 | 0 | 0 | 0 | 0 | 0 |
| 31 | 6.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 5,280 | 6,456 | 4,968 | 4,692 | 0 | 0 | 0 | 0 | 0 |
| 32 | 7.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 6,160 | 7,532 | 5,796 | 5,474 | 0 | 0 | 0 | 0 | 0 |
| 33 | 7.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 6,160 | 7,532 | 5,796 | 5,474 | 0 | 0 | 0 | 0 | 0 |
| 34 | 6.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 5,280 | 6,456 | 4,968 | 4,692 | 0 | 0 | 0 | 0 | 0 |
| | | 1 | 3 | iagonal Wood Sheathing | 429 | 540 | 686 | 913 | 0 | - | - | - | - | | | | | | | | | |
| 35 | 36.00 | 1 | 2 | Wood Sheathing or Woo | 85 | 96 | 110 | 132 | 145 | 157 | 171 | 0 | - | 34,344 | 42,264 | 43,560 | 51,696 | 5,220 | 5,652 | 6,156 | 0 | 0 |
| | | 1 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | | | | | | | | | |

**EXPECTED STRENGTH OF LEVEL 2 WALLS - N-S DIRECTION**
**2013 SAN FRANCISCO BUILDING CODE**
**816 TAVARAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

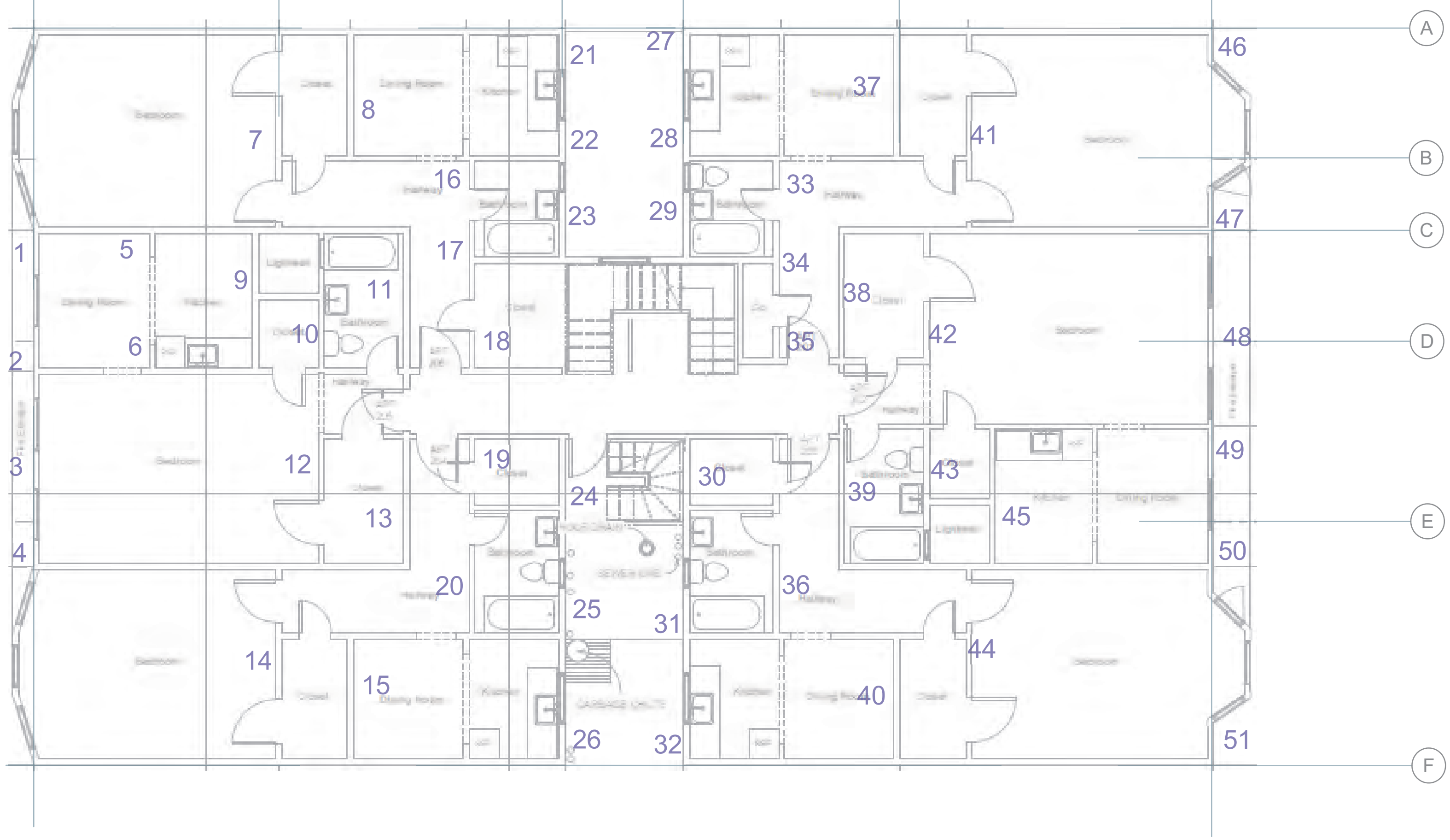
Floor Level = 2

Loading Direction = N-S

**1. Expected Strength of Floor Level 2 Walls - N-S Direction**

| Wall ID | Wall Length (feet) | No. Layers | Sheathing ID | Sheathing Material | Expected Strength Per Layer (plf) Drift (%) 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 | Expected Composite Strength (lbs) Drift (%) 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 36 | 36.00 | 1 | 3 | iagonal Wood Sheathing | 429 | 540 | 686 | 913 | 0 | - | - | - | - | 34,344 | 42,264 | 43,560 | 51,696 | 5,220 | 5,652 | 6,156 | 0 | 0 |
| | | 1 | 2 | Wood Sheathing or Woc | 85 | 96 | 110 | 132 | 145 | 157 | 171 | 0 | - | | | | | | | | | |
| | | 1 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | | | | | | | | | |

| Sum of Expected Composite Wall Strengths (Kips) Drift (%) 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 |
|---|---|---|---|---|---|---|---|---|
| 365 | 447 | 388 | 409 | 21 | 23 | 25 | 0 | 0 |




(E) LEVEL 2 FLOOR PLAN - SHEET A3
NTS

**LEVEL 2 - W-E WALL ID'S**

**EXPECTED STRENGTH OF LEVEL 2 WALLS - W-E DIRECTION**
**2013 SAN FRANCISCO BUILDING CODE**
**816 TAVARAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

Floor Level = 2

Loading Direction = W-E

**2. Expected Strength of Floor Level 2 Walls - W-E Direction**

| Wall ID | Wall Length (feet) | No. Layers | Sheathing ID | Sheathing Material | Expected Strength Per Layer (plf) Drift (%) | | | | | | | | | Expected Composite Strength (lbs) Drift (%) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 |
| | | 1 | 3 | iagonal Wood Sheathing | 429 | 540 | 686 | 913 | 0 | - | - | - | - | | | | | | | | | |
| 1 | 4.00 | 1 | 2 | Wood Sheathing or Wood | 85 | 96 | 110 | 132 | 145 | 157 | 171 | 0 | - | 3,816 | 4,696 | 4,840 | 5,744 | 580 | 628 | 684 | 0 | 0 |
| | | 1 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | | | | | | | | | |
| | | 1 | 3 | iagonal Wood Sheathing | 429 | 540 | 686 | 913 | 0 | - | - | - | - | | | | | | | | | |
| 2 | 5.00 | 1 | 2 | Wood Sheathing or Wood | 85 | 96 | 110 | 132 | 145 | 157 | 171 | 0 | - | 4,770 | 5,870 | 6,050 | 7,180 | 725 | 785 | 855 | 0 | 0 |
| | | 1 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | | | | | | | | | |
| | | 1 | 3 | iagonal Wood Sheathing | 429 | 540 | 686 | 913 | 0 | - | - | - | - | | | | | | | | | |
| 3 | 2.50 | 1 | 2 | Wood Sheathing or Wood | 85 | 96 | 110 | 132 | 145 | 157 | 171 | 0 | - | 2,385 | 2,935 | 3,025 | 3,590 | 363 | 393 | 428 | 0 | 0 |
| | | 1 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | | | | | | | | | |
| | | 1 | 3 | iagonal Wood Sheathing | 429 | 540 | 686 | 913 | 0 | - | - | - | - | | | | | | | | | |
| 4 | 3.00 | 1 | 2 | Wood Sheathing or Wood | 85 | 96 | 110 | 132 | 145 | 157 | 171 | 0 | - | 2,862 | 3,522 | 3,630 | 4,308 | 435 | 471 | 513 | 0 | 0 |
| | | 1 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | | | | | | | | | |
| 5 | 2.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 1,760 | 2,152 | 1,656 | 1,564 | 0 | 0 | 0 | 0 | 0 |
| 6 | 2.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 1,760 | 2,152 | 1,656 | 1,564 | 0 | 0 | 0 | 0 | 0 |
| 7 | 6.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 5,280 | 6,456 | 4,968 | 4,692 | 0 | 0 | 0 | 0 | 0 |
| 8 | 8.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 7,040 | 8,608 | 6,624 | 6,256 | 0 | 0 | 0 | 0 | 0 |
| 9 | 9.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 7,920 | 9,684 | 7,452 | 7,038 | 0 | 0 | 0 | 0 | 0 |
| 10 | 7.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 6,160 | 7,532 | 5,796 | 5,474 | 0 | 0 | 0 | 0 | 0 |
| 11 | 11.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 9,680 | 11,836 | 9,108 | 8,602 | 0 | 0 | 0 | 0 | 0 |
| 12 | 4.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 3,520 | 4,304 | 3,312 | 3,128 | 0 | 0 | 0 | 0 | 0 |

**EXPECTED STRENGTH OF LEVEL 2 WALLS - W-E DIRECTION**
**2013 SAN FRANCISCO BUILDING CODE**
**816 TAVARAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

Floor Level = 2
Loading Direction = W-E

**2. Expected Strength of Floor Level 2 Walls - W-E Direction**

| Wall ID | Wall Length (feet) | No. Layers | Sheathing ID | Sheathing Material | Expected Strength Per Layer (plf) Drift (%) | | | | | | | | | Expected Composite Strength (lbs) Drift (%) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 |
| 13 | 9.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 7,920 | 9,684 | 7,452 | 7,038 | 0 | 0 | 0 | 0 | 0 |
| 14 | 5.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 4,400 | 5,380 | 4,140 | 3,910 | 0 | 0 | 0 | 0 | 0 |
| 15 | 8.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 7,040 | 8,608 | 6,624 | 6,256 | 0 | 0 | 0 | 0 | 0 |
| 16 | 3.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 2,640 | 3,228 | 2,484 | 2,346 | 0 | 0 | 0 | 0 | 0 |
| 17 | 5.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 4,400 | 5,380 | 4,140 | 3,910 | 0 | 0 | 0 | 0 | 0 |
| 18 | 3.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 2,640 | 3,228 | 2,484 | 2,346 | 0 | 0 | 0 | 0 | 0 |
| 19 | 2.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 1,760 | 2,152 | 1,656 | 1,564 | 0 | 0 | 0 | 0 | 0 |
| 20 | 7.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 6,160 | 7,532 | 5,796 | 5,474 | 0 | 0 | 0 | 0 | 0 |
| 21 | 2.50 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 2,200 | 2,690 | 2,070 | 1,955 | 0 | 0 | 0 | 0 | 0 |
| 22 | 3.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 2,640 | 3,228 | 2,484 | 2,346 | 0 | 0 | 0 | 0 | 0 |
| 23 | 5.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 4,400 | 5,380 | 4,140 | 3,910 | 0 | 0 | 0 | 0 | 0 |
| 24 | 8.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 7,040 | 8,608 | 6,624 | 6,256 | 0 | 0 | 0 | 0 | 0 |

**EXPECTED STRENGTH OF LEVEL 2 WALLS - W-E DIRECTION**
**2013 SAN FRANCISCO BUILDING CODE**
**816 TAVARAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

Floor Level = 2
Loading Direction = W-E

**2. Expected Strength of Floor Level 2 Walls - W-E Direction**

| Wall ID | Wall Length (feet) | No. Layers | Sheathing ID | Sheathing Material | Expected Strength Per Layer (plf) Drift (%) | | | | | | | | | Expected Composite Strength (lbs) Drift (%) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 |
| 25 | 6.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 5,280 | 6,456 | 4,968 | 4,692 | 0 | 0 | 0 | 0 | 0 |
| 26 | 2.50 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 2,200 | 2,690 | 2,070 | 1,955 | 0 | 0 | 0 | 0 | 0 |
| 27 | 2.50 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 2,200 | 2,690 | 2,070 | 1,955 | 0 | 0 | 0 | 0 | 0 |
| 28 | 3.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 2,640 | 3,228 | 2,484 | 2,346 | 0 | 0 | 0 | 0 | 0 |
| 29 | 4.50 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 3,960 | 4,842 | 3,726 | 3,519 | 0 | 0 | 0 | 0 | 0 |
| 30 | 8.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 7,040 | 8,608 | 6,624 | 6,256 | 0 | 0 | 0 | 0 | 0 |
| 31 | 6.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 5,280 | 6,456 | 4,968 | 4,692 | 0 | 0 | 0 | 0 | 0 |
| 32 | 2.50 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 2,200 | 2,690 | 2,070 | 1,955 | 0 | 0 | 0 | 0 | 0 |
| 33 | 3.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 2,640 | 3,228 | 2,484 | 2,346 | 0 | 0 | 0 | 0 | 0 |
| 34 | 5.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 4,400 | 5,380 | 4,140 | 3,910 | 0 | 0 | 0 | 0 | 0 |
| 35 | 2.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 1,760 | 2,152 | 1,656 | 1,564 | 0 | 0 | 0 | 0 | 0 |
| 36 | 7.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 6,160 | 7,532 | 5,796 | 5,474 | 0 | 0 | 0 | 0 | 0 |

**EXPECTED STRENGTH OF LEVEL 2 WALLS - W-E DIRECTION**
**2013 SAN FRANCISCO BUILDING CODE**
**816 TAVARAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

Floor Level = 2

Loading Direction = W-E

**2. Expected Strength of Floor Level 2 Walls - W-E Direction**

| | | | | | Expected Strength Per Layer (plf) | | | | | | | | | Expected Composite Strength (lbs) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Drift (%) | | | | | | | | | Drift (%) | | | | | | | | |
| **Wall ID** | **Wall Length (feet)** | No. Layers | Sheathing ID | Sheathing Material | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 |
| 37 | 8.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 7,040 | 8,608 | 6,624 | 6,256 | 0 | 0 | 0 | 0 | 0 |
| 38 | 10.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 8,800 | 10,760 | 8,280 | 7,820 | 0 | 0 | 0 | 0 | 0 |
| 39 | 10.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 8,800 | 10,760 | 8,280 | 7,820 | 0 | 0 | 0 | 0 | 0 |
| 40 | 8.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 7,040 | 8,608 | 6,624 | 6,256 | 0 | 0 | 0 | 0 | 0 |
| 41 | 6.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 5,280 | 6,456 | 4,968 | 4,692 | 0 | 0 | 0 | 0 | 0 |
| 42 | 4.50 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 3,960 | 4,842 | 3,726 | 3,519 | 0 | 0 | 0 | 0 | 0 |
| 43 | 7.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 6,160 | 7,532 | 5,796 | 5,474 | 0 | 0 | 0 | 0 | 0 |
| 44 | 5.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 4,400 | 5,380 | 4,140 | 3,910 | 0 | 0 | 0 | 0 | 0 |
| 45 | 9.00 | 2 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 7,920 | 9,684 | 7,452 | 7,038 | 0 | 0 | 0 | 0 | 0 |
| | | 1 | 1 | Stucco | 333 | 320 | 262 | 0 | - | - | - | - | - | | | | | | | | | |
| 46 | 2.00 | 1 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 1,546 | 1,716 | 1,352 | 782 | 0 | 0 | 0 | 0 | 0 |
| | | 1 | 1 | Stucco | 333 | 320 | 262 | 0 | - | - | - | - | - | | | | | | | | | |
| 47 | 4.50 | 1 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 3,479 | 3,861 | 3,042 | 1,760 | 0 | 0 | 0 | 0 | 0 |
| | | 1 | 1 | Stucco | 333 | 320 | 262 | 0 | - | - | - | - | - | | | | | | | | | |
| 48 | 4.00 | 1 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | 3,092 | 3,432 | 2,704 | 1,564 | 0 | 0 | 0 | 0 | 0 |

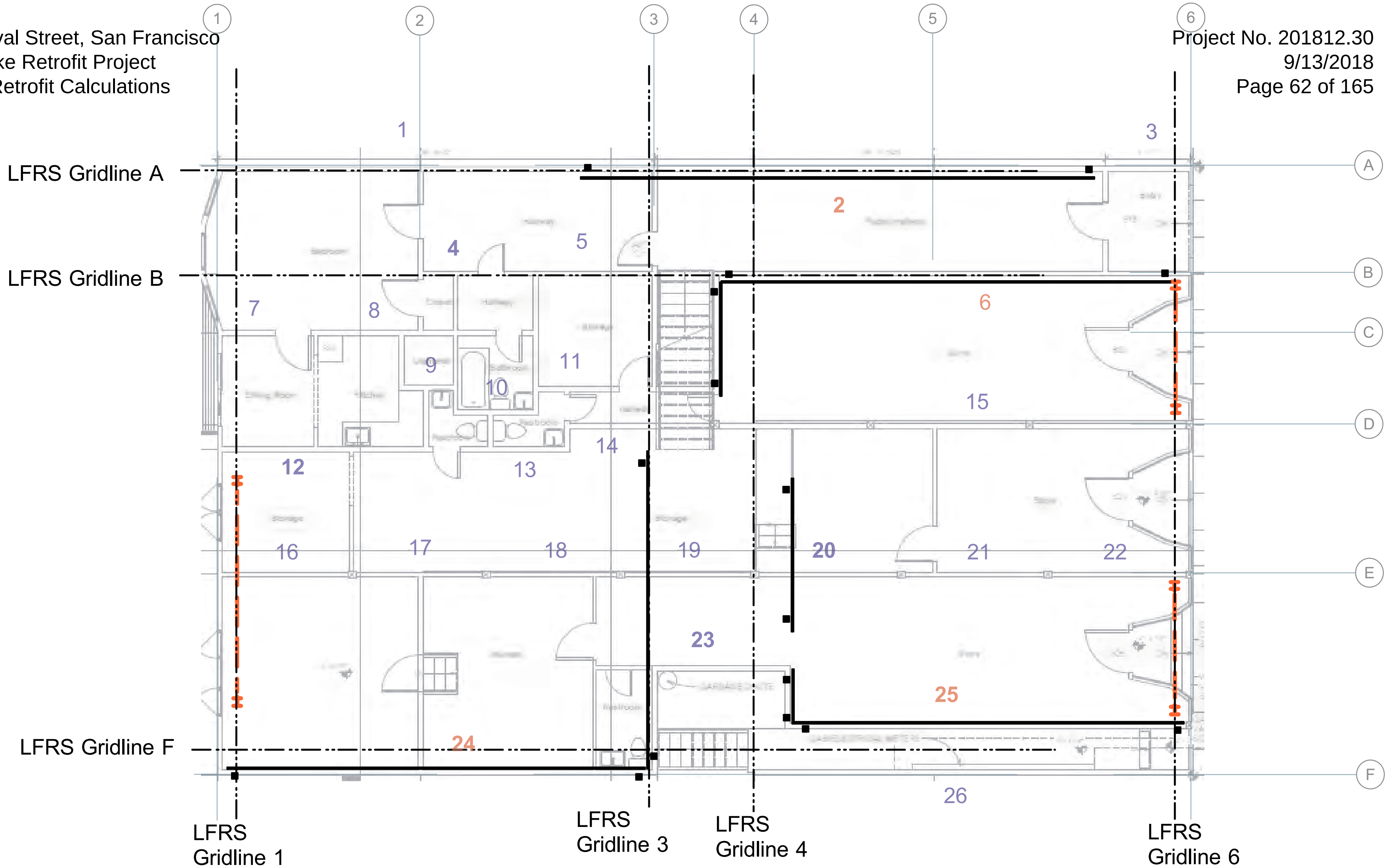
**EXPECTED STRENGTH OF LEVEL 2 WALLS - W-E DIRECTION**
**2013 SAN FRANCISCO BUILDING CODE**
**816 TAVARAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

Floor Level = 2
Loading Direction = W-E

**2. Expected Strength of Floor Level 2 Walls - W-E Direction**

| Wall ID | Wall Length (feet) | No. Layers | Sheathing ID | Sheathing Material | Expected Strength Per Layer (plf) Drift (%) 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 | Expected Composite Strength (lbs) Drift (%) 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 49 | 4.00 | 1 | 1 | Stucco | 333 | 320 | 262 | 0 | - | - | - | - | - | 3,092 | 3,432 | 2,704 | 1,564 | 0 | 0 | 0 | 0 | 0 |
| | | 1 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| 50 | 4.00 | 1 | 1 | Stucco | 333 | 320 | 262 | 0 | - | - | - | - | - | 3,092 | 3,432 | 2,704 | 1,564 | 0 | 0 | 0 | 0 | 0 |
| | | 1 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |
| 51 | 3.00 | 1 | 1 | Stucco | 333 | 320 | 262 | 0 | - | - | - | - | - | 2,319 | 2,574 | 2,028 | 1,173 | 0 | 0 | 0 | 0 | 0 |
| | | 1 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | |

| Sum of Expected Composite Wall Strengths (Kips) Drift (%) 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 |
|---|---|---|---|---|---|---|---|---|
| 232 | 282 | 222 | 208 | 2 | 2 | 2 | 0 | 0 |




(E) LEVEL 1 PLAN - SHEET A2

NTS

**LEVEL 1 - N-S WALL ID'S**

Lateral Force Resisting System (LFRS) Elements :

(N) Shear Wall

Simpson Strong Frame

**EXPECTED STRENGTH OF LEVEL 1 WALLS - N-S DIRECTION**
**2013 SAN FRANCISCO BUILDING CODE**
**816 TAVARAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

Floor Level = 1

Loading Direction = N-S

**3. Expected Strength of Floor Level 1 Walls - N-S Direction**

| | | | | | Expected Strength Per Layer (plf) | | | | | | | | | Expected Composite Strength (lbs) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Drift (%) | | | | | | | | | Drift (%) | | | | | | | | |
| **Wall ID** | **Wall Length (feet)** | No. Layers | Sheathing ID | Sheathing Material | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 |
| 1 | 30.00 | 1 | 3 | iagonal Wood Sheathing | 429 | 540 | 686 | 913 | 0 | - | - | - | - | | | | | | | | | |
| | | 1 | 2 | Wood Sheathing or Wood | 85 | 96 | 110 | 132 | 145 | 157 | 171 | 0 | - | 21,480 | 25,470 | 30,000 | 36,900 | 9,510 | 9,240 | 9,480 | 3,210 | 0 |
| | | 1 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | | | | | | | | | |
| 2 | 42.00 | 1 | 3 | iagonal Wood Sheathing | 429 | 540 | 686 | 913 | 0 | - | - | - | - | | | | | | | | | |
| | | 1 | 2 | Wood Sheathing or Wood | 85 | 96 | 110 | 132 | 145 | 157 | 171 | 0 | - | 33,810 | 45,570 | 56,448 | 65,373 | 46,641 | 47,607 | 48,321 | 35,406 | 0 |
| | | 1 | 12 | WSP, 10d @ 6" oc | 548 | 767 | 946 | 1,023 | 1,038 | 1,055 | 1,065 | 843 | 0 | | | | | | | | | |
| 3 | 8.00 | 1 | 3 | iagonal Wood Sheathing | 429 | 540 | 686 | 913 | 0 | - | - | - | - | | | | | | | | | |
| | | 1 | 2 | Wood Sheathing or Wood | 85 | 96 | 110 | 132 | 145 | 157 | 171 | 0 | - | 7,632 | 9,392 | 9,680 | 11,488 | 1,160 | 1,256 | 1,368 | 0 | 0 |
| | | 1 | 4 | Plaster on Wood Lath | 440 | 538 | 414 | 391 | 0 | - | - | - | - | | | | | | | | | |
| 4 | 4.50 | | | | | | | | | | | | | | | | | | | | | |
| | | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 1,818 | 1,917 | 1,836 | 1,665 | 1,548 | 1,359 | 1,305 | 963 | 0 |
| | | | | | | | | | | | | | | | | | | | | | | |
| 5 | 12.00 | | | | | | | | | | | | | | | | | | | | | |
| | | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 4,848 | 5,112 | 4,896 | 4,440 | 4,128 | 3,624 | 3,480 | 2,568 | 0 |
| | | | | | | | | | | | | | | | | | | | | | | |
| 6 | 36.00 | 1 | 15 | WSP, 10d @ 2" oc | 1,120 | 1,568 | 1,999 | 2,248 | 2,405 | 2,512 | 2,512 | 2,231 | 0 | | | | | | | | | |
| | | 1 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 43,956 | 60,282 | 75,636 | 84,258 | 89,676 | 93,150 | 93,042 | 82,242 | 0 |
| | | | | | | | | | | | | | | | | | | | | | | |
| 7 | 4.50 | | | | | | | | | | | | | | | | | | | | | |
| | | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 1,818 | 1,917 | 1,836 | 1,665 | 1,548 | 1,359 | 1,305 | 963 | 0 |
| | | | | | | | | | | | | | | | | | | | | | | |
| 8 | 15.00 | | | | | | | | | | | | | | | | | | | | | |
| | | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 6,060 | 6,390 | 6,120 | 5,550 | 5,160 | 4,530 | 4,350 | 3,210 | 0 |
| | | | | | | | | | | | | | | | | | | | | | | |
| 9 | 4.00 | | | | | | | | | | | | | | | | | | | | | |
| | | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 1,616 | 1,704 | 1,632 | 1,480 | 1,376 | 1,208 | 1,160 | 856 | 0 |
| | | | | | | | | | | | | | | | | | | | | | | |
| 10 | 7.00 | | | | | | | | | | | | | | | | | | | | | |
| | | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 2,828 | 2,982 | 2,856 | 2,590 | 2,408 | 2,114 | 2,030 | 1,498 | 0 |
| | | | | | | | | | | | | | | | | | | | | | | |
| 11 | 6.50 | | | | | | | | | | | | | | | | | | | | | |
| | | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 2,626 | 2,769 | 2,652 | 2,405 | 2,236 | 1,963 | 1,885 | 1,391 | 0 |
| | | | | | | | | | | | | | | | | | | | | | | |
| 12 | 17.00 | | | | | | | | | | | | | | | | | | | | | |
| | | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 6,868 | 7,242 | 6,936 | 6,290 | 5,848 | 5,134 | 4,930 | 3,638 | 0 |
| | | | | | | | | | | | | | | | | | | | | | | |

**EXPECTED STRENGTH OF LEVEL 1 WALLS - N-S DIRECTION**
**2013 SAN FRANCISCO BUILDING CODE**
**816 TAVARAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

Floor Level = 1

Loading Direction = N-S

**3. Expected Strength of Floor Level 1 Walls - N-S Direction**

| Wall ID | Wall Length (feet) | No. Layers | Sheathing ID | Sheathing Material | Expected Strength Per Layer (plf) | | | | | | | | | Expected Composite Strength (lbs) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Drift (%) | | | | | | | | | Drift (%) | | | | | | | | |
| | | | | | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 |
| 13 | 9.00 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 3,636 | 3,834 | 3,672 | 3,330 | 3,096 | 2,718 | 2,610 | 1,926 | 0 |
| 14 | 12.00 | | | | | | | | | | | | | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 15 | 38.00 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 15,352 | 16,188 | 15,504 | 14,060 | 13,072 | 11,476 | 11,020 | 8,132 | 0 |
| 16 | 10.00 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 4,040 | 4,260 | 4,080 | 3,700 | 3,440 | 3,020 | 2,900 | 2,140 | 0 |
| 17 | 10.00 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 4,040 | 4,260 | 4,080 | 3,700 | 3,440 | 3,020 | 2,900 | 2,140 | 0 |
| 18 | 11.00 | | | | | | | | | | | | | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 19 | 11.00 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 4,444 | 4,686 | 4,488 | 4,070 | 3,784 | 3,322 | 3,190 | 2,354 | 0 |
| 20 | | | | | | | | | | | | | | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 21 | 11.00 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 4,444 | 4,686 | 4,488 | 4,070 | 3,784 | 3,322 | 3,190 | 2,354 | 0 |
| 22 | 11.00 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 4,444 | 4,686 | 4,488 | 4,070 | 3,784 | 3,322 | 3,190 | 2,354 | 0 |
| 23 | 11.00 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 4,444 | 4,686 | 4,488 | 4,070 | 3,784 | 3,322 | 3,190 | 2,354 | 0 |
| | | 1 | 3 | iagonal Wood Sheathing | 429 | 540 | 686 | 913 | 0 | - | - | - | - | | | | | | | | | |
| 24 | 34.00 | 1 | 2 | Wood Sheathing or Wood | 85 | 96 | 110 | 132 | 145 | 157 | 171 | 0 | - | 32,776 | 44,472 | 56,882 | 66,045 | 52,309 | 53,533 | 53,771 | 40,290 | 0 |
| | | 1 | 13 | WSP, 10d @ 4" oc | 707 | 990 | 1,275 | 1,420 | 1,466 | 1,496 | 1,496 | 1,185 | 0 | | | | | | | | | |

**EXPECTED STRENGTH OF LEVEL 1 WALLS - N-S DIRECTION**
**2013 SAN FRANCISCO BUILDING CODE**
**816 TAVARAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

Floor Level = 1

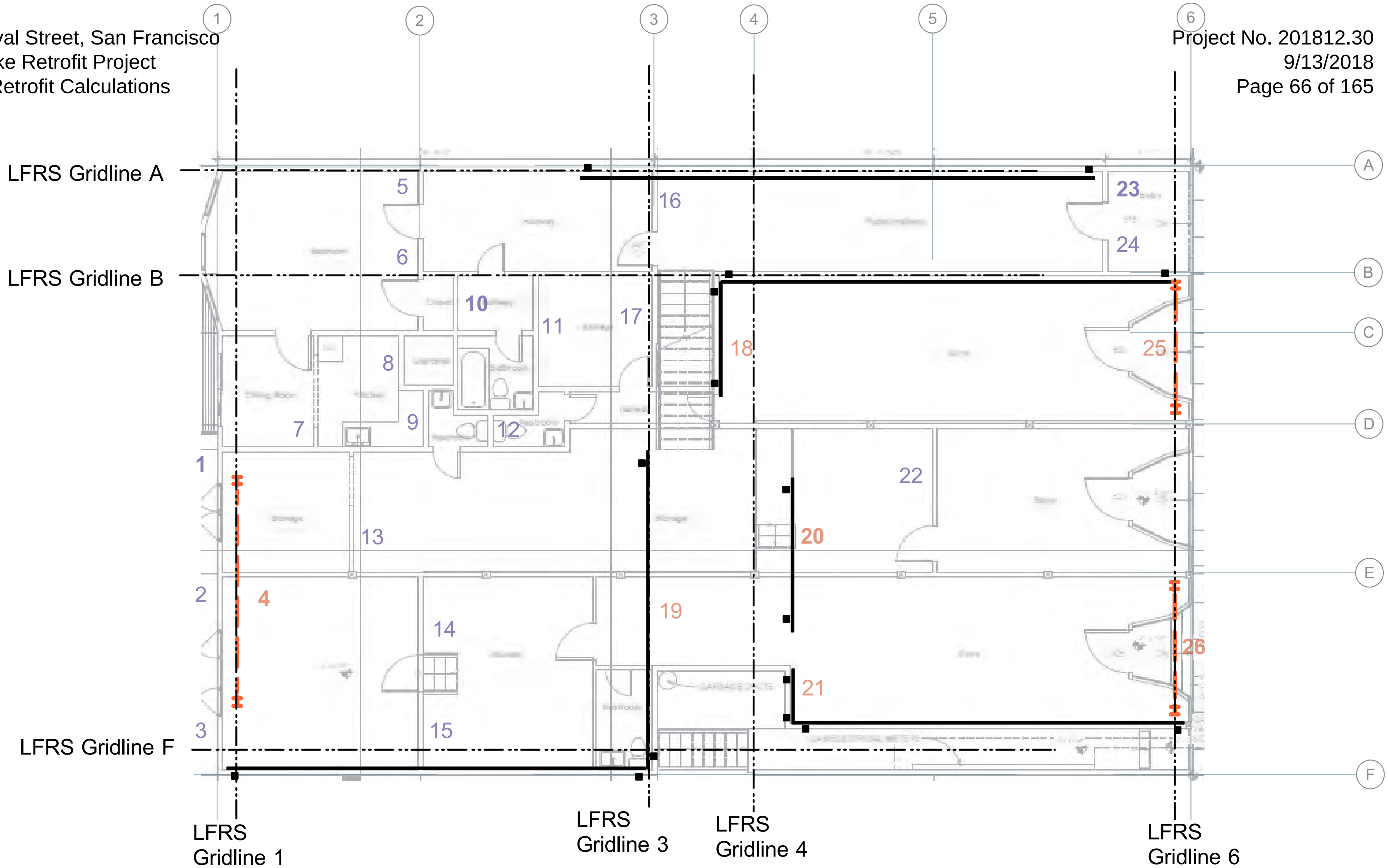
Loading Direction = N-S

**3. Expected Strength of Floor Level 1 Walls - N-S Direction**

| Wall ID | Wall Length (feet) | No. Layers | Sheathing ID | Sheathing Material | Expected Strength Per Layer (plf) Drift (%) 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 25 | 32.00 | 1 | 3 | iagonal Wood Sheathing | 429 | 540 | 686 | 913 | 0 | - | - | - | - |
| | | 1 | 2 | Wood Sheathing or Wood | 85 | 96 | 110 | 132 | 145 | 157 | 171 | 0 | - |
| | | 1 | 13 | WSP, 10d @ 4" oc | 707 | 990 | 1,275 | 1,420 | 1,466 | 1,496 | 1,496 | 1,185 | 0 |
| 26 | 36.00 | 1 | 3 | iagonal Wood Sheathing | 429 | 540 | 686 | 913 | 0 | - | - | - | - |
| | | 1 | 2 | Wood Sheathing or Wood | 85 | 96 | 110 | 132 | 145 | 157 | 171 | 0 | - |
| | | 1 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 |

| Wall ID | Expected Composite Strength (lbs) Drift (%) 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 |
|---|---|---|---|---|---|---|---|---|---|
| 25 | 30,848 | 41,856 | 53,536 | 62,160 | 49,232 | 50,384 | 50,608 | 37,920 | 0 |
| 26 | 25,776 | 30,564 | 36,000 | 44,280 | 11,412 | 11,088 | 11,376 | 3,852 | 0 |

| **Sum of Expected Composite Wall Strengths (Kips)** Drift (%) 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 |
|---|---|---|---|---|---|---|---|---|
| 270 | 335 | 392 | 438 | 322 | 321 | 321 | 242 | 0 |




(E) LEVEL 1 PLAN - SHEET A2

NTS

Lateral Force Resisting System (LFRS) Elements :

LEVEL 0 - W-E WALL ID'S

(N) Shear Wall

Simpson Strong Frame

**EXPECTED STRENGTH OF LEVEL 1 WALLS - W-E DIRECTION**
**2013 SAN FRANCISCO BUILDING CODE**
**816 TAVARAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

Floor Level = 1

Loading Direction = W-E

**4. Expected Strength of Floor Level 1 Walls - W-E Direction**

| Wall ID | Wall Length (feet) | No. Layers | Sheathing ID | Sheathing Material | Expected Strength Per Layer (plf) Drift (%) 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 | Expected Composite Strength (lbs) Drift (%) 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 4.50 | 1 | 3 | iagonal Wood Sheathing | 429 | 540 | 686 | 913 | 0 | - | - | - | - | 3,222 | 3,821 | 4,500 | 5,535 | 1,427 | 1,386 | 1,422 | 482 | 0 |
| | | 1 | 2 | Wood Sheathing or Wood | 85 | 96 | 110 | 132 | 145 | 157 | 171 | 0 | - | | | | | | | | | |
| | | 1 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | | | | | | | | | |
| 2 | 7.00 | 1 | 3 | iagonal Wood Sheathing | 429 | 540 | 686 | 913 | 0 | - | - | - | - | 5,012 | 5,943 | 7,000 | 8,610 | 2,219 | 2,156 | 2,212 | 749 | 0 |
| | | 1 | 2 | Wood Sheathing or Wood | 85 | 96 | 110 | 132 | 145 | 157 | 171 | 0 | - | | | | | | | | | |
| | | 1 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | | | | | | | | | |
| 3 | 4.50 | 1 | 3 | iagonal Wood Sheathing | 429 | 540 | 686 | 913 | 0 | - | - | - | - | 3,222 | 3,821 | 4,500 | 5,535 | 1,427 | 1,386 | 1,422 | 482 | 0 |
| | | 1 | 2 | Wood Sheathing or Wood | 85 | 96 | 110 | 132 | 145 | 157 | 171 | 0 | - | | | | | | | | | |
| | | 1 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | | | | | | | | | |
| 4 | 18.50 | 1 | 17 | Moment Frame | 625 | 875 | 1,250 | 1,875 | 2,110 | 2,248 | 2,386 | 2,662 | 2,800 | 11,563 | 16,188 | 23,125 | 34,688 | 39,041 | 41,593 | 44,145 | 49,248 | 51,800 |
| 5 | 2.50 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 1,010 | 1,065 | 1,020 | 925 | 860 | 755 | 725 | 535 | 0 |
| 6 | 2.50 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 1,010 | 1,065 | 1,020 | 925 | 860 | 755 | 725 | 535 | 0 |
| 7 | 2.00 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 808 | 852 | 816 | 740 | 688 | 604 | 580 | 428 | 0 |
| 8 | 4.50 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 1,818 | 1,917 | 1,836 | 1,665 | 1,548 | 1,359 | 1,305 | 963 | 0 |
| 9 | 4.50 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 1,818 | 1,917 | 1,836 | 1,665 | 1,548 | 1,359 | 1,305 | 963 | 0 |
| 10 | 11.00 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 4,444 | 4,686 | 4,488 | 4,070 | 3,784 | 3,322 | 3,190 | 2,354 | 0 |
| 11 | 11.00 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 4,444 | 4,686 | 4,488 | 4,070 | 3,784 | 3,322 | 3,190 | 2,354 | 0 |
| 12 | 2.50 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 1,010 | 1,065 | 1,020 | 925 | 860 | 755 | 725 | 535 | 0 |

**EXPECTED STRENGTH OF LEVEL 1 WALLS - W-E DIRECTION**
**2013 SAN FRANCISCO BUILDING CODE**
**816 TAVARAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

Floor Level = 1

Loading Direction = W-E

**4. Expected Strength of Floor Level 1 Walls - W-E Direction**

| | | | | | Expected Strength Per Layer (plf) | | | | | | | | | Expected Composite Strength (lbs) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Drift (%) | | | | | | | | | Drift (%) | | | | | | | | |
| **Wall ID** | **Wall Length (feet)** | No. Layers | Sheathing ID | Sheathing Material | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 |
| 13 | 5.00 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 2,020 | 2,130 | 2,040 | 1,850 | 1,720 | 1,510 | 1,450 | 1,070 | 0 |
| 14 | 6.00 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 2,424 | 2,556 | 2,448 | 2,220 | 2,064 | 1,812 | 1,740 | 1,284 | 0 |
| 15 | 6.50 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 2,626 | 2,769 | 2,652 | 2,405 | 2,236 | 1,963 | 1,885 | 1,391 | 0 |
| 16 | 5.00 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 2,020 | 2,130 | 2,040 | 1,850 | 1,720 | 1,510 | 1,450 | 1,070 | 0 |
| 17 | 10.00 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 4,040 | 4,260 | 4,080 | 3,700 | 3,440 | 3,020 | 2,900 | 2,140 | 0 |
| | | 1 | 14 | WSP, 10d @ 3" oc | 940 | 1,316 | 1,696 | 1,889 | 1,949 | 1,990 | 1,990 | 1,576 | 0 | | | | | | | | | |
| 18 | 9.00 | 1 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 9,369 | 12,803 | 16,182 | 17,834 | 18,315 | 18,590 | 18,563 | 14,666 | 0 |
| | | 1 | 14 | WSP, 10d @ 3" oc | 940 | 1,316 | 1,696 | 1,889 | 1,949 | 1,990 | 1,990 | 1,576 | 0 | | | | | | | | | |
| 19 | 26.00 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 29,692 | 39,754 | 49,400 | 53,924 | 55,146 | 55,666 | 55,510 | 43,758 | 0 |
| | | 1 | 14 | WSP, 10d @ 3" oc | 940 | 1,316 | 1,696 | 1,889 | 1,949 | 1,990 | 1,990 | 1,576 | 0 | | | | | | | | | |
| 20 | 12.50 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 14,275 | 19,113 | 23,750 | 25,925 | 26,513 | 26,763 | 26,688 | 21,038 | 0 |
| | | 1 | 14 | WSP, 10d @ 3" oc | 940 | 1,316 | 1,696 | 1,889 | 1,949 | 1,990 | 1,990 | 1,576 | 0 | | | | | | | | | |
| 21 | 4.50 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 5,139 | 6,881 | 8,550 | 9,333 | 9,545 | 9,635 | 9,608 | 7,574 | 0 |
| 22 | 8.00 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 3,232 | 3,408 | 3,264 | 2,960 | 2,752 | 2,416 | 2,320 | 1,712 | 0 |
| 23 | 2.50 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 1,010 | 1,065 | 1,020 | 925 | 860 | 755 | 725 | 535 | 0 |
| 24 | 2.50 | 2 | 6 | Gypsum Wallboard | 202 | 213 | 204 | 185 | 172 | 151 | 145 | 107 | 0 | 1,010 | 1,065 | 1,020 | 925 | 860 | 755 | 725 | 535 | 0 |

**EXPECTED STRENGTH OF LEVEL 1 WALLS - W-E DIRECTION**
**2013 SAN FRANCISCO BUILDING CODE**
**816 TAVARAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

Floor Level = 1

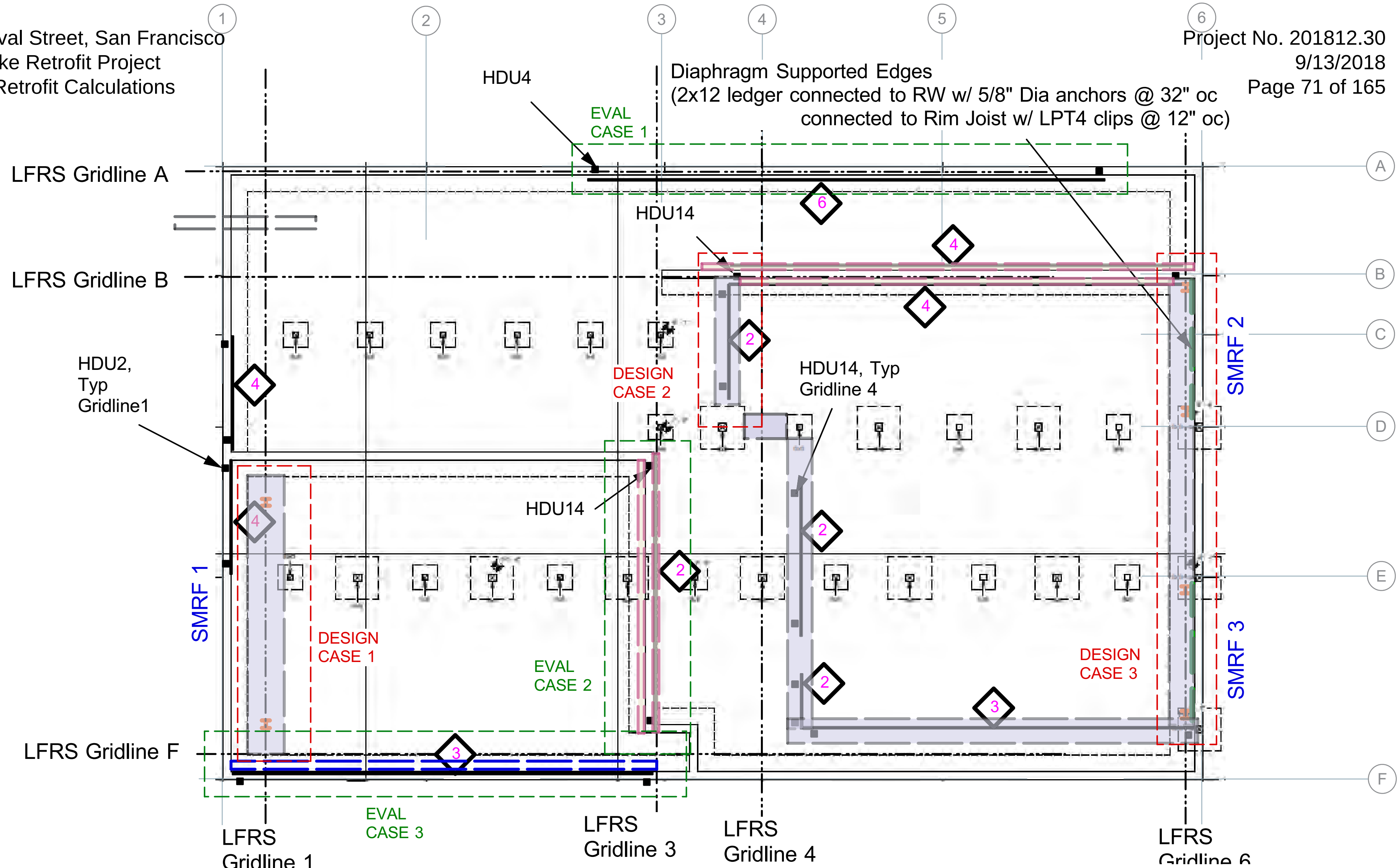
Loading Direction = W-E

**4. Expected Strength of Floor Level 1 Walls - W-E Direction**

| | | | | | Expected Strength Per Layer (plf) | | | | | | | | | Expected Composite Strength (lbs) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Drift (%) | | | | | | | | | Drift (%) | | | | | | | | |
| **Wall ID** | **Wall Length (feet)** | No. Layers | Sheathing ID | Sheathing Material | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 | 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 |
| 25 | 10.00 | 1 | 18 | Moment Frame | 250 | 350 | 500 | 750 | 844 | 899 | 954 | 1,065 | 1,120 | 2,500 | 3,500 | 5,000 | 7,500 | 8,441 | 8,993 | 9,545 | 10,648 | 11,200 |
| 26 | 10.00 | 1 | 18 | Moment Frame | 250 | 350 | 500 | 750 | 844 | 899 | 954 | 1,065 | 1,120 | 2,500 | 3,500 | 5,000 | 7,500 | 8,441 | 8,993 | 9,545 | 10,648 | 11,200 |

| **Sum of Expected Composite Wall Strengths (Kips)** | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Drift (%) | | | | | | | | |
| 0.50 | 0.70 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 |
| 121 | 152 | 182 | 208 | 200 | 201 | 204 | 178 | 74 |

# EVALUATION OF EXISTING FOUNDATIONS:




# FOUNDATION RETROFIT PLAN - LEVEL 0

## FOUNDATION RETROFIT PLAN - FOOTING EVALUATION/DESIGN CASES

NTS

**Lateral Force Resisting System (LFRS) Elements :**

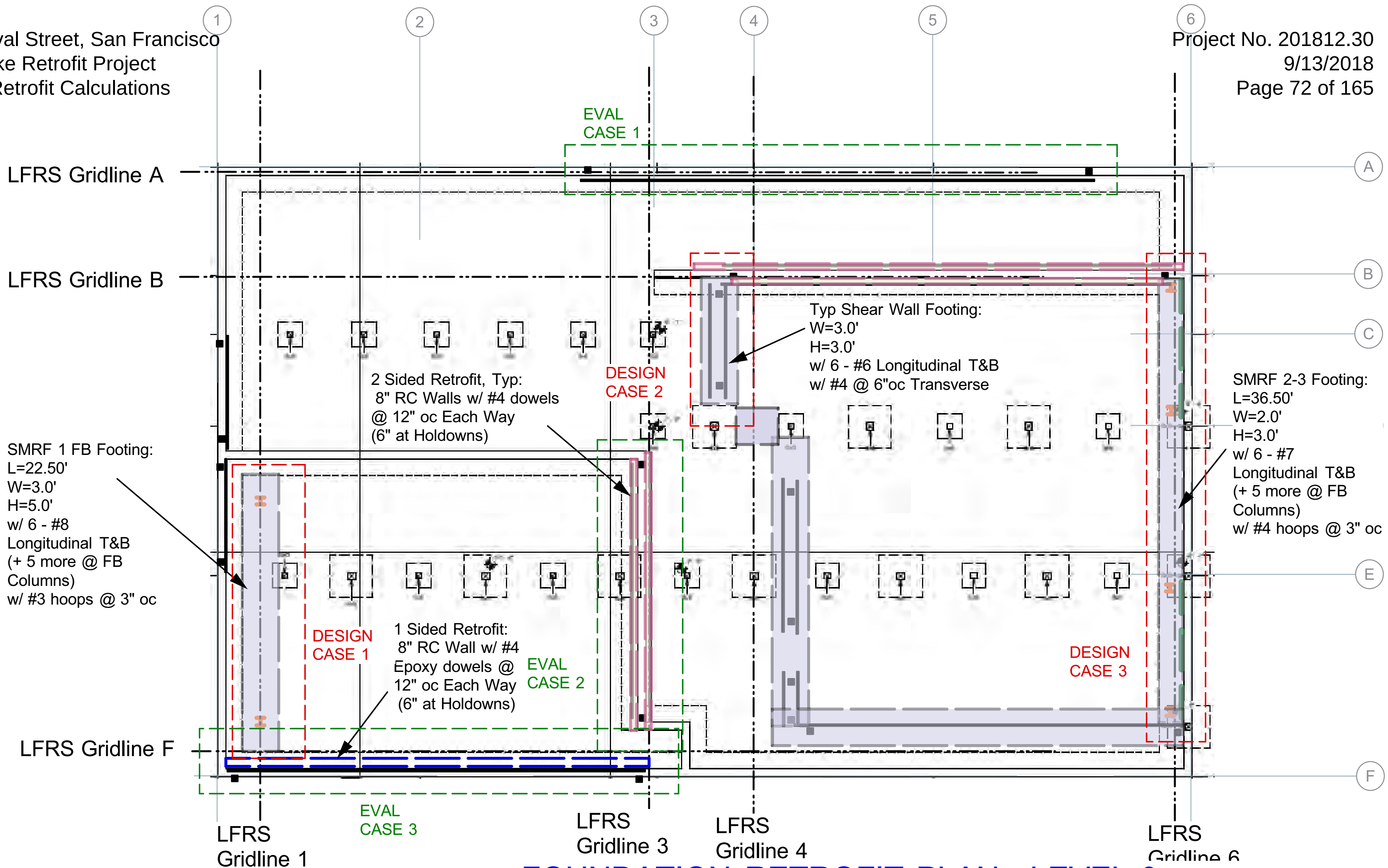
- (N) Shear Wall
- Simpson Strong Frame
- Diaphragm Supported Edges at Retaining Wall

**FOUNDATION RETROFIT ELEMENTS:**

- (N) Footings
- (N) 2-Sided Wall Retrofit
- (N) 1-Sided Wall Retrofit

**RETROFIT LEGEND:**

- 3 — 1/2" Struct I Sheathing w/ 10d Edge Nailing
- Holdown each side, as indicated




## FOUNDATION RETROFIT PLAN - LEVEL 0

## FOUNDATION RETROFIT PLAN - FOOTING EVALUATION/DESIGN CASES

NTS

Lateral Force Resisting System (LFRS) Elements :

(N) Shear Wall

Simpson Strong Frame

Diaphragm Supported Edges at Retaining Wall

FOUNDATION RETROFIT ELEMENTS:

(N) Footings

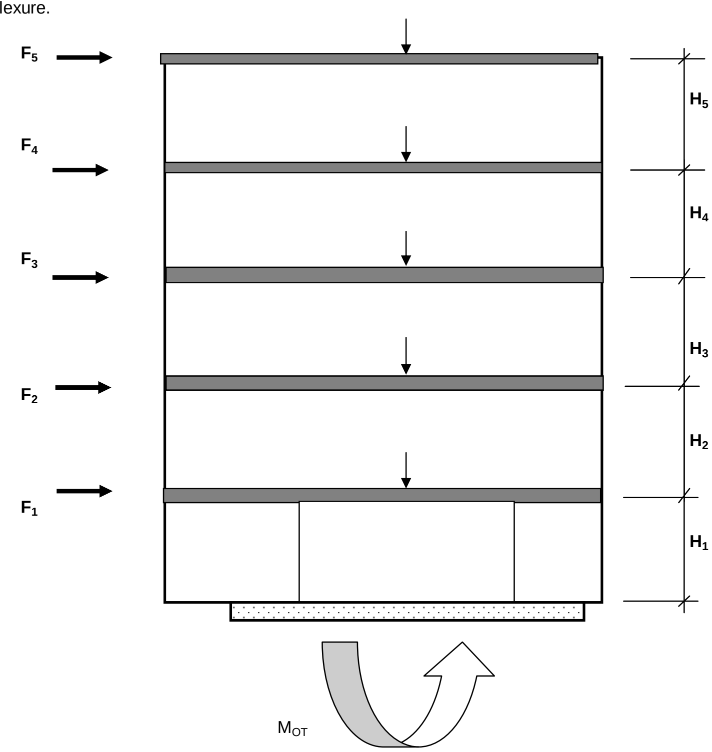
(N) 2-Sided Wall Retrofit

(N) 1-Sided Wall Retrofit

**EXISTING FOUNDATION EVALUATION- SHEAR WALL AT GRIDLINE A (EVAL CASE E-1)**
**DETERMINATION OF VERTICAL AND LATERAL LOADS TO FOUNDATION**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Assumptions**

1. Existing walls are treated as pinned Columns. Footing is assumed rigid.
2. Column loads are located in transverse center of footing; limit of 2 columns w/o flexure.
3. Footing has no shear reinforcement.
4. Concrete is Normal Weight Concrete with uncoated bars.




**1. Lateral Loads and Load Effects**

V = 10.00 kips (Base Shear - A4 **ASD**)

| Floor Level | Height (feet) | Loading ID | $V_x$ / V | Shear (Kips) | Force (Kips) | Moment (Kip-ft) |
|---|---|---|---|---|---|---|
| | | 5 | | | | |
| R | 11.00 | 4 | 0.30 | 3.00 | 3.00 | 135.0 |
| 3 | 10.00 | 3 | 0.65 | 6.50 | 3.50 | 254.0 |
| 2 | 13.50 | 2 | 0.89 | 8.90 | 2.40 | 311.6 |
| 1 | 10.50 | 1 | 1.00 | 10.00 | 1.10 | 323.2 |

**$M_{OT}$ = 323.15 Kip-ft**

**2. Vertical Loads and Load Effects**

| | | Floor Tributary Loads | | | | | Wall Tributary Loads | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Wall** | Floor Level | DL (psf) | Length (feet) | Width (feet) | Area (ft$^2$) | Weight (kips) | WL (psf) | Length (feet) | Height (feet) | Area (ft$^2$) | Weight (kips) |
| | | | | | | | | | | | |
| **1** | R | 20 | 42.00 | 2.00 | 84 | 1.68 | 14 | 42.00 | 11.00 | 462 | 6.47 |
| | 3 | 30 | 42.00 | 2.00 | 84 | 2.52 | 14 | 42.00 | 10.00 | 420 | 5.88 |
| | 2 | 30 | 42.00 | 2.00 | 84 | 2.52 | 14 | 42.00 | 13.50 | 567 | 7.94 |
| | 1 | 30 | 42.00 | 2.00 | 84 | 2.52 | 14 | 42.00 | 10.50 | 441 | 6.17 |

Sum of Floor Weight = 9.24 Kips

Sum of Wall Weight = 26.46 Kips

**$P_{D1}$ = 35.70 Kips**

**EXISTING FOUNDATION EVALUATION- SHEAR WALL AT GRIDLINE A (EVAL CASE E-1)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Assumptic** 1. Footing has no shear reinforcement.
2. Concrete is Normal Weight Concrete with uncoated bars.

## Footing Parameters :

Footing Size :
$L_x$ = 48.0 feet $d_s$ = 0.0 feet (depth of soil)
$L_y$ = 1.4 feet
$h_f$ = 3.0 feet

Wall Location :
$x_c$ = 24.0 feet (Wall centerline distance from Left Edge)
$y_c$ = 1.1 feet (Wall centerline distance from Bottom Edge)

Wall Size :
$C_x$ = 42.0 feet (Wall length)
$C_y$ = 0.5 feet (Wall width)

Interconnected Slab at Sides:

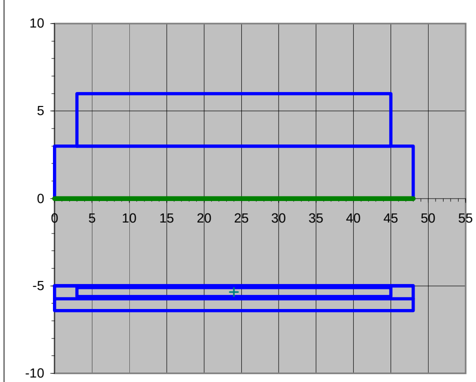
Note : Slabs at sides are used only to reduce soil bearing pressure; footing is designed to take all loads.

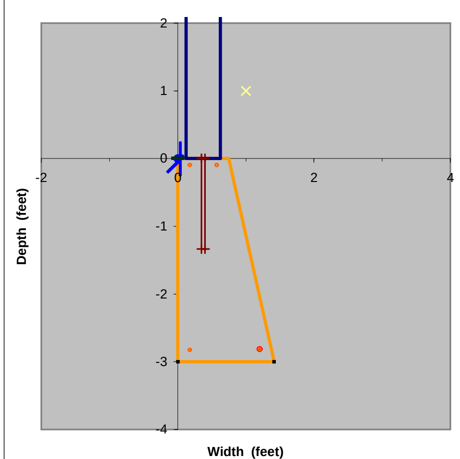
Side : Left Right
t Inches (Slab Thickness)
X Feet (distance to other Slab Edge Support
$f'_c$ Ksi
Conn Type (D= Dowel, C= Continuous)

Concrete :
$f'_c$ = 3.25 Ksi
$f_y$ = 40.00 Ksi
$\rho_c$ = 0.150 kip/ft^3

Reinforcement:
$d_c$ = 1.00 inches (bar clearance - top)
$d_c$ = 2.00 inches (bar clearance - bottom)
= 2.00 inches (bar clearance - sides)

| | Orientation | Bar Size | N Bars | Bottom Layer | d (inches) | Bar Diameter (inches) | Bar Area Per Bar (in^2) | Bar Area Total (in^2) |
|---|---|---|---|---|---|---|---|---|
| Top Mat | x | 7 | 1 | x | 32.13 | 0.88 | 0.60 | 0.60 |
| Bottom Mat | x | 7 | 1 | | 32.75 | 0.88 | 0.60 | 0.60 |




$L_B$ = 1.42 feet (Bearing Length)

Note: Reinf Layout = 0 (Evaluation)

## 1. Design of Slab-to-Footing Connections

**No Slabs connected to Footing.**

## 2. Lateral Resistance of Foundation

**Foundation OK for Sliding**

## 3. Soil Pressure due to Applied Loads

**Footing Bearing stress OK**

## 5. Adequacy of Footing - Anchor Pull-out in Existing Footing

**Use 2 - HDU4 with 0.63 Dia anchors @ 16" oc Min EA side**

## 5. Adequacy of Footing - Shear

**Existing Footing OK for Shear, DC Ratio =0.32**

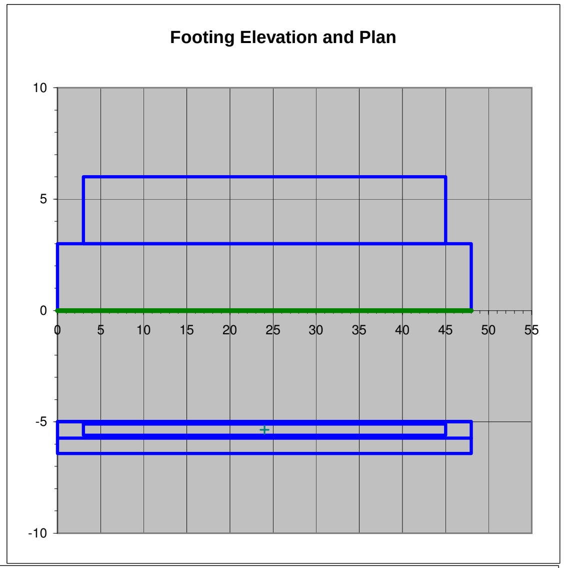
## 6. Adequacy of Footing - Flexure

**Existing Reinforcement OK, DC Ratio = 0.47**

**EXISTING FOUNDATION EVALUATION- SHEAR WALL AT GRIDLINE A (EVAL CASE E-1)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Assumptions :**
1. Existing Footing has no shear reinforcement.
2. Concrete is Normal Weight Concrete with uncoated bars.
3. Wall is centered on Footing; Longitudinal Heel and Toe are equal in length.

## Footing Parameters :




Wall Size :
$C_x$ = 42.0 feet (Wall length)
$C_y$ = 0.5 feet (Wall width)

Wall Location :
$x_c$ = 24.0 feet (Wall centerline distance from Left Edge)
$y_c$ = 1.1 feet (Wall centerline distance from Bottom Edge)

Holdown Anchor :
Holdown = **HDU4**
$t_{HD}$ = 0.63 inches (Diameter of Holdown Anchor)
Note: $t'_{HD}$ = 0.63 inches (Diameter of Holdown Anchor Required for Holdown)

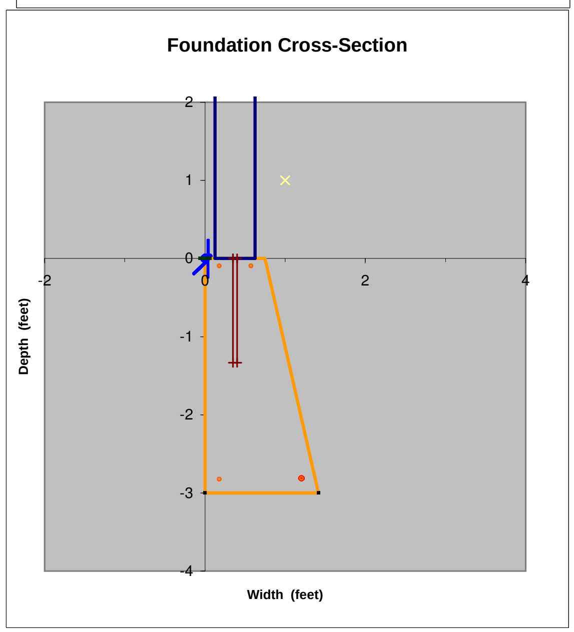
$F_{CA}$ = 4.67 Kips (Allowable Capacity of Holdown)
$F_{CU}$ = 6.53 Kips (Ultimate Capacity of Holdown)

$H_{HD}$ = 16.00 inches (Embedment depth of Holdown Anchor)
OK

Footing Dimensions :
$L_x$ = 48.00 feet (Footing Length)
$h_f$ = 3.00 feet (Footing Height)

Left Right
$L_H$ = feet (Length of Heel, if any)
$h_H$ = feet (Thickness of Heel, if any)

(E) Footing Width:
$W_T$ = 9.00 inches (Footing Width - at Top of Wall)
= 0.00 inches (Increase in Width - at Left)
= 8.00 inches (Increase in Width - at Right)
$W_B$ = 1.42 feet (Footing Width - at Bottom of Wall)




(N) Retrofit Walls: Note: Reinf Layout = 0 (Evaluation)

Concrete Weight : NWC LWC - Lean Weight Concrete, NWC otherwise)

Left Right
$t_T$ = inches (Retrofit Wall Thickness - Top of Wall)
$t_B$ = inches (Retrofit Wall Thickness - Bottom of Wall)

$W'_T$ = 0.75 feet (Footing Width - at Top of Wall)
$W'_B$ = 1.42 feet (Footing Width - at Bottom of Wall)

$t_H$ = inches (Retrofit Wall Thickness of Heel, if any)

Dowel Data : No. 4 Dowels
$S_D$ = 12.00 inches (Dowel Spacing - Horiz and Vertical)

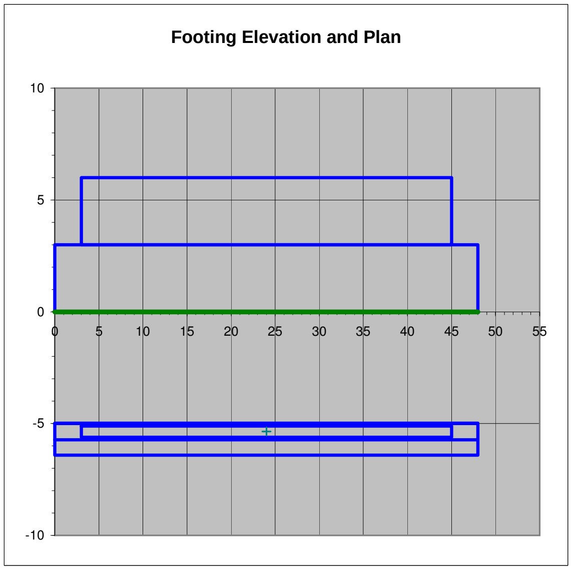
$L_B$ = 1.42 feet (Bearing Length)

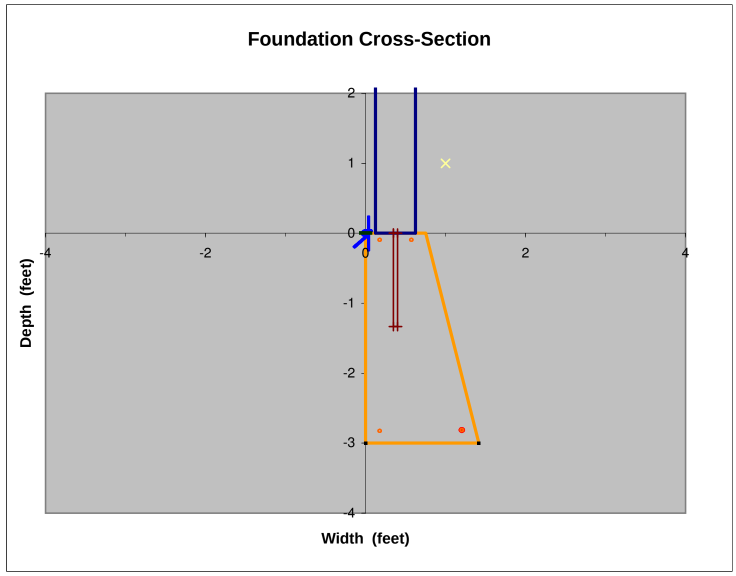
Footing Loads : 9.9

| | Service | Strength | |
|---|---|---|---|
| P = | 35.7 | 50.0 | kips |
| $M_y$ = | 323.2 | 452.4 | kip-ft |
| $V_x$ = | 10.00 | 14.0 | kips |
| $M_x$ = | 0 | 0.1 | kip-ft |
| $V_y$ = | 0.0 | 0.0 | kips |

Plastic Hinge Centroidal Heights:
@ $h_{px}$ = 0.00 feet
@ $h_{py}$ = 0.00 feet

**EXISTING FOUNDATION EVALUATION- SHEAR WALL AT GRIDLINE A (EVAL CASE E-1)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**




Capacity Factors :

Concrete :

$\phi_v$ = 0.75 (Shear)
$\phi_b$ = 0.65 (Bearing)

Steel Anchor in Concrete:

$\phi_{EQ}$ = 0.75 (Steel Anchor - Seismic Region - ACI Section D.3.3.3)
$\phi_{sa,t}$ = 0.75 (Steel Anchor - Tension, Ductile Steel Element - ACI D.4.4)
$\phi_{sa,cb}$ = 0.65 (Steel Anchor - Concrete Breakout Category 2 - ACI D.4.4)
Note: Cat 1 : Low Sensitivity to installation and High Reliability

Material Properties :

Concrete :

$f'_{ce}$ = 3.25 Ksi (Existing Concrete)
$f'_{cn}$ = 2.50 Ksi (New Concrete)
$f_y$ = 40 Ksi (Existing Reinforcement)
$\rho_c$ = 0.150 kip/ft^3

Steel Anchor in Concrete:

$f_{ya}$ = 60.00 Ksi (PCA Notes Table 34-1 - ASTM A307)

Note : Year Built : 1921

Interconnected Slab at Sides:

Note : Slabs at sides are used only to reduce soil bearing pressure; footing is designed to take all loads.

| Side : | Left | Right | |
|---|---|---|---|
| t | | | Inches (Slab Thickness) |
| h | | | Inches (Distance to Top of Wall) |
| X | | | Feet (distance to other Slab Edge Support |
| $f'_c$ | | | Ksi |
| Conn Type | | | (D= Dowel, C= Continuous) |

Reinforcement:

$d_c$ = 1.00 inches (bar clearance - top)
$d_c$ = 2.00 inches (bar clearance - bottom)
= 2.00 inches (bar clearance - sides)

Existing Reinforcement:

| | | |
|---|---|---|
| Top | 2 | No. 4 |
| Bottom | 2 | No. 4 |

| New Flexural Reinforcement EA Side | | | | | | | Bar Area | |
|---|---|---|---|---|---|---|---|---|
| Location | Bar Size | N Bars (Max 3) | Bottom Layer | d (inches) | b (inches) | Bar Diameter (inches) | Per Bar (in^2) | Total (in^2) |
| Top | 7 | 1 | x | 32.13 | 9.00 | 0.88 | 0.60 | 0.60 |
| Bottom | 7 | 1 | | 32.75 | 17.00 | 0.88 | 0.60 | 0.60 |

Soil Parameters :

$\sigma_{allow}$ = 2.00 ksf (Allowable Bearing Pressure)
$\sigma_p$ = 0.30 ksf/ft (Passive Soil Pressure)
$\mu$ = 0.25 ksf (Coefficient of Friction)

**EXISTING FOUNDATION EVALUATION- SHEAR WALL AT GRIDLINE A (EVAL CASE E-1)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 2. Lateral Resistance of Foundation

2A. **Longitudinal Loading**

$F_{Rx} = 0.5\, L'_y\, h'^2_f\, \sigma_p + 0.6\,(W_f + P)\,\mu$

Where $L'_y = W'_B$    $W'_B =$ 1.42 feet (Footing Width - at Bottom of Wall)

$L'_y =$ 1.42 feet (Bearing Width at Ends of Footing)

$h'_f = h_f + h_{sk}$    and $h_f =$ 3.00 feet

$h_{sk} =$ 0.00 feet (Additional height of Shear Key at Footing E

$h'_f =$ 3.0 feet (Bearing Height at Ends of Footing)

= (6.4) (0.30) + 0.6 (66.3) (0.25)    $\sigma_p =$ 0.30 ksf/ft (Passive Soil Pressure)

= (1.91) + (10.10)    $W_f = \rho_c\, L_x\, L_y\, h_f$    and $\rho_c =$ 0.150 kip/ft^3

$L_x =$ 48.0 feet    0

$L_y =$ 1.4 feet

$h_f =$ 3.0 feet

$W_f =$ 30.60 Kips (Footing Weight)

$P =$ 35.7 Kips (Service Load)

$\mu =$ 0.25 ksf (Coefficient of Friction)

$F_{Rx} =$ 12.02 kips    Note : $V_x =$ 10.00 kips

**OK**

**Foundation OK for Sliding**

## 3. Soil Pressure due to Applied Loads

3A. **Longitudinal Loading**

a) Loading Eccentricity    Note: Effective transverse width of footing for bearing is assumed as Bottom of Footing Width, $L_y = L'_B$

$e_x = \Sigma M_y / P'$    Where $\Sigma M_y = M_y + V_x H_{px} - P\,(0.5\, L_x - x_c)$    and $M_y =$ 323 kip-ft

$V_x =$ 10 kips

@ $h_{px} =$ 0.00 feet

$P =$ 36 Kips

$L_x =$ 48.0 feet

= 323 kip-ft + 0 kip-ft - 0 kip-ft    $x_c =$ 24.0 feet (Column centerline distance from Left Edge)

$\Sigma M_y =$ 323 Kip-in

$P' = P + P_F$    and $P =$ 36 Kips

$P_F = \rho_c\, L_x\, L_y\, h_f$    for $\rho_c =$ 0.150 kip/ft^3

$L_x =$ 48.0 feet

$L_y = 0.5\,(L'_T + L'_B) =$ 1.1 feet

$h_f =$ 3.0 feet

$P_F =$ 23.40 kips (footing weight)

$P' =$ 59 Kips

$e_x =$ 5.47 feet

Note: $L_x/6 =$ 8.00 feet (Footing Middle Third)

**EXISTING FOUNDATION EVALUATION- SHEAR WALL AT GRIDLINE A (EVAL CASE E-1)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

b) Bearing Stresses

i) for $e_x \le L_x/6$ (within Middle $L_x/3$) **<= Governs!**

$\sigma_{max}$ = P (1 + 6 $e_x$ / $L_x$) / ($L_x$ $L_y$) Where P = 59 Kips

$\sigma_{min}$ = P (1 - 6 $e_x$ / $L_x$) / ($L_x$ $L_y$) $e_x$ = 5.47 feet

$L_x$ = 48.0 feet

$L_y$ = $L_B$ = 1.4 feet

| | | |
|---|---|---|
| $\sigma_{max}$ = | 1.46 | Ksf |
| $\sigma_{min}$ = | 0.28 | Ksf |

ii) for $e_x > L_x/6$ (outside Middle $L_x/3$)

$\sigma_{max}$ = 2 P / ($L_{bx}$ $L_y$) Where P = 59 Kips

$L_{bx}$ = 3 (0.5 $L_x$ - $e_x$) and $L_x$ = 48.0 feet

$e_x$ = 5.47 feet

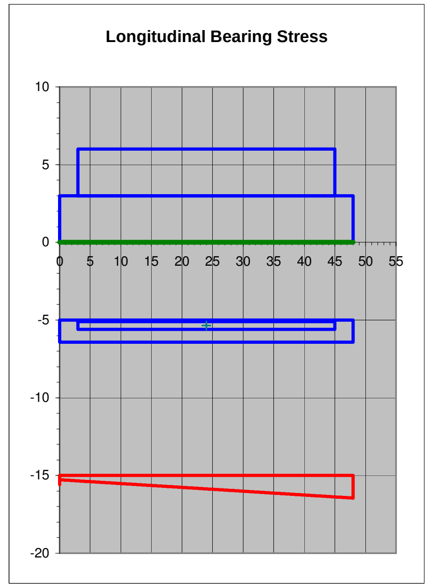
| | | |
|---|---|---|
| $L_{bx}$ = | 48.00 | feet |

$L_y$ = $L_B$ = 1.4 feet

| | | |
|---|---|---|
| $\sigma_{max}$ = | 1.74 | Ksf |
| $\sigma_{min}$ = | 0.00 | Ksf |

iii) Governing Condition

| | | |
|---|---|---|
| $\sigma_{max}$ = | 1.46 | Ksf |
| $\sigma_{min}$ = | 0.28 | Ksf |




3B. **Transverse Loading**

a) Loading Eccentricity

Note: Effective transverse width of footing for bearing is assumed as Bottom of Footing Width, $L_y$ = $L'_B$

$e_y$ = $\Sigma M_x / P'$

Where $\Sigma M_x$ = $M_x$ + $V_y$ $H_{py}$ - P (0.5 $L_y$ - $y_c$) and $M_x$ = 0 kip-ft

$V_y$ = 0 kips

@ $h_{py}$ = 0.00 feet

P = 36 Kips

$L_y$ = $L_B$ = 1.4 feet

= 0 kip-ft + 0 kip-ft - -13 kip-ft $y_c$ = 1.1 feet (Column centerline distance from Left Edge)

| | | |
|---|---|---|
| $\Sigma M_y$ = | 13 | Kip-in |

P' = 59 Kips

| | | |
|---|---|---|
| $e_y$ = | 0.22 | feet |

Note: $L_y/6$ = 0.24 feet (Footing Middle Third)

b) Bearing Stresses

i) for $e_y \le L_y/6$ (within Middle $L_y/3$) **<= Governs!**

$\sigma_{max}$ = P' (1 + 6 $e_y$ / $L_y$) / ($L_x$ $L_y$) Where P' = 59 Kips

$\sigma_{min}$ = P' (1 - 6 $e_y$ / $L_y$) / ($L_x$ $L_y$) $e_y$ = 0.22 feet

$L_y$ = $L_B$ = 1.4 feet

$L_x$ = 48.0 feet

| | | |
|---|---|---|
| $\sigma_{max}$ = | 1.66 | Ksf |
| $\sigma_{min}$ = | 0.08 | Ksf |

ii) for $e_x > L_x/6$ (outside Middle $L_x/3$)

$\sigma_{max}$ = 2 P' / ($L_{by}$ $L_x$) Where P' = 59 Kips

$L_{by}$ = 3 (0.5 $L_y$ - $e_y$) ≤ $L_y$ and $L_y$ = $L_B$ = 1.4 feet

$e_y$ = 0.22 feet

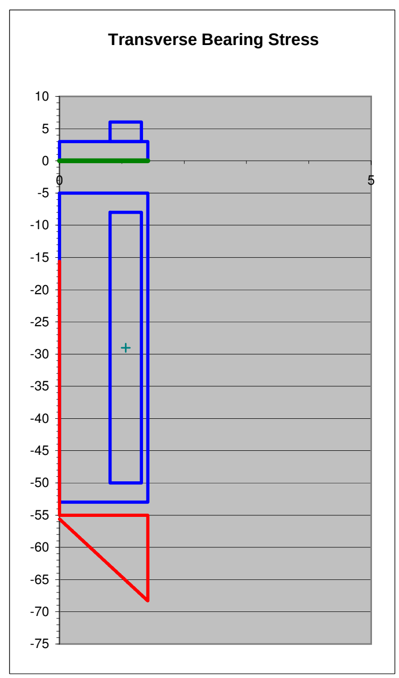
| | | |
|---|---|---|
| $L_{by}$ = | 1.42 | feet |

$L_x$ = 48.0 feet

| | | |
|---|---|---|
| $\sigma_{max}$ = | 1.74 | Ksf |
| $\sigma_{min}$ = | 0.00 | Ksf |

iii) Governing Condition

| | | |
|---|---|---|
| $\sigma_{max}$ = | 1.66 | Ksf |
| $\sigma_{min}$ = | 0.08 | Ksf |



**EXISTING FOUNDATION EVALUATION- SHEAR WALL AT GRIDLINE A (EVAL CASE E-1)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**3C. Check of Bearing Stresses on Soil**

a) Assuming all loads resisted by Footing only

$\sigma_{max}$ = Max($\sigma_{max,x}$,$\sigma_{max,y}$)    Where $\sigma_{max}$ = 1.46 Ksf (Longitudinal Direction)
= 1.66 Ksf (Transverse Direction)

b) Assuming all loads resisted by Footing + Adjoining Slabs    Note: Effective transverse width of footing for bearing is assumed as Bottom of Footing Width, $L_y = L'_B$

$\sigma_{max}$ = Max($\sigma_{max,x}$,$\sigma_{max,y}$) * $L_y$ / b    Where $\sigma_{max}$ = 1.46 Ksf (Longitudinal Direction)    and $L_y = L_B$ = 1.4 feet
= 1.66 Ksf (Transverse Direction)    b = 17.00 inches
= 1.42 feet

= ( 1.66 * 1.00 )

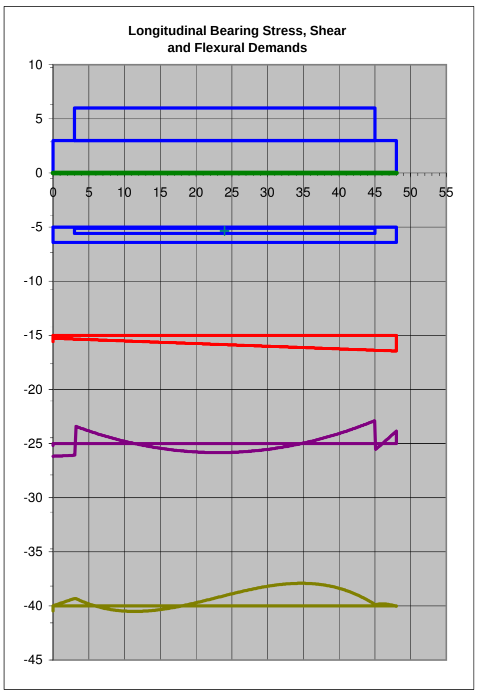
| $\sigma_{max}$ = 1.66 Ksf |
|---|

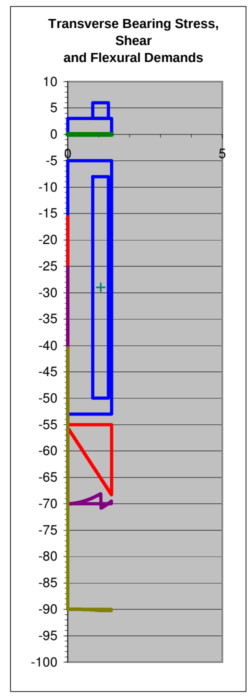
OK

Note: $\sigma_{allow}$ = 2.00 ksf (allowable bearing pressure)

| Footing Bearing stress OK |
|---|

## 4. Applied Loading and Demands on Footing



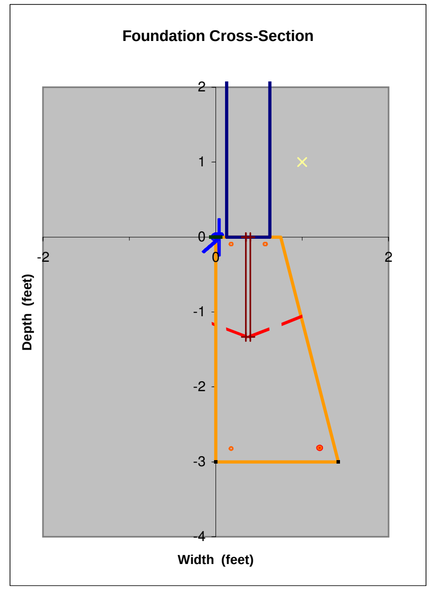




| | Longitudinal Direction | | | | |
|---|---|---|---|---|---|
| | Left End | Left Face of Wall | Wall Centerline | Right Face of Wall | Right End |
| Location (feet) | 0 | 3.00 | 24.00 | 45.00 | 48.00 |
| $\sigma = q_u/L_y$ (ksf) | 0.39 | 0.48 | 1.21 | 1.94 | 2.05 |
| P (kips) | - | - | 50 | - | - |
| $M_y + V\,h_{px}$ (kip-ft) | - | - | 452 | - | - |
| V+ (kips) | - | 6 | - | 8 | 5 |
| V- (kips) | -5 | -4 | -3 | -2 | - |
| M+ (kip-ft) | 0 | 14 | 18 | 3 | - |
| M- (kip-ft) | - | - | - | - | -1 |

| | Transverse Direction | | | | |
|---|---|---|---|---|---|
| | Left End | Left Face of Column | Column Centerline | Right Face of Column | Right End |
| Location (feet) | 0.00 | 0.81 | 1.06 | 1.31 | 1.42 |
| $\sigma = q_u/L_y$ (ksf) | 0.08 | 0.99 | 1.27 | 1.54 | 1.66 |
| P (kips) | - | - | 50 | - | - |
| $M_y + V\,h_{px}$ (kip-ft) | - | - | 0 | - | - |
| V+ (kips) | 0 | 21 | 34 | 1 | 9 |
| V- (kips) | - | - | - | - | - |
| M+ (kip-ft) | 0 | - | - | - | - |
| M- (kip-ft) | - | -6 | -13 | -11 | -12 |

**EXISTING FOUNDATION EVALUATION- SHEAR WALL AT GRIDLINE A (EVAL CASE E-1)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 5. Adequacy of Footing - Anchor Pull-out in Existing Footing




Holdown = HDU4

$F_{CU}$ = 6.53 Kips (Ultimate Capacity of Holdown)

$H_{HD}$ = 16.00 inches (Embedment depth of Holdown Anchor)

a) Bolt Design Strength - Tension (ACI 318-08 D3.3.3, D4.4, D5.1)

$\phi_{EQ}\ \phi_{sa,t}\ N_{sa} = \phi_{EQ}\ \phi_{sa,t}\ n\ A_{se,N}\ f_{uta}$ (D-3)

Where $\phi_{EQ}$ = 0.75 (Steel Anchor - Seismic Region - ACI Section D.3.3.3)

$\phi_{sa,t}$ = 0.75 (Steel Anchor - Tension, Ductile Steel Element - ACI Section D.4.4)

n = 1 (number of anchors)

$A_{se,N} = \pi\ t_{HD}^2$ for $t_{HD}$ = 0.63 inches (Diameter of Holdown Anchor)

$\boxed{A_{se,N} = 0.307\ in^2}$

$f_{uta}$ = Min (1.6 $f_{ya}$, 125) for $f_{ya}$ = 60.00 Ksi (PCA Notes Table 34-1 - ASTM A307)

$\boxed{f_{uta} = 96.00\ Ksi}$

$\boxed{\phi_{EQ}\ \phi_{sa,t}\ N_{sa} = 16.57\ Kips}$ (Bolt Design Strength - Tension)

b) Concrete Breakout Strength - Tension (ACI 318-08 Section D5.2)

Note: Condition B is assumed per Section D4.4, where Supplementary reinforcement in not present in failure prism.

$\phi_{EQ}\ \phi_{sa,cb}\ N_{cb} = \phi_{EQ}\ \phi_{sa,cb}\ N_b\ A_{NC} / A_{NCO}\ \Psi_1\ \Psi_2\ \Psi_3$ (D-4)

Where $\phi_{EQ}$ = 0.75 (Steel Anchor - Seismic Region - ACI Section D.3.3.3)

$\phi_{sa,cb}$ = 0.65 (Steel Anchor - Concrete Breakout Category 2 - ACI D.4.4)
Note: Cat 1 : Low Sensitivity to installation and High Reliability

$N_b$ = Basic concrete Break-out Strength of a single anchor in Tension in Cracked Concrete (ACI D.5.2.2)

$= K_C\ f'^{0.5}_c\ H_{ef}^{1.5}\ \lambda$ and $K_C$ = 17 Post-installed Anchors (D5.2.2)

$f'_c$ = 3.25 Ksi
= 3,250 Psi

$H_{ef} = H_{HD}$ = 16.00 inches (Embedment depth of Holdown Anchor)

$\lambda$ = 1.00 (1.0 for NWC, 0.75 for LWC)
Note: NWC - Normal Weight Concrete Assumed

$\boxed{N_b = 62.03\ Kips}$

$A_{NC}$ = Projected Concrete Failure Area - Actual (ACI D5.2.1)

$= 2\ (1.5\ H_{ef})\ (C_{a1} + C_{a2})$ and $H_{ef} = H_{HD}$ = 16.00 inches (Embedment depth of Holdown Anchor)

$C_{a1}$ = 4.50 inches (Distance from Wall CL to Left Edge of Existing Footing Wall)

$C_{a2}$ = 7.17 inches (Distance from Wall CL to Right Edge of Existing Footing Wall)

$\boxed{A_{NC} = 560\ in^2}$

$A_{NCO}$ = Projected Concrete Failure Area - Ideal (ACI D5.2.1)

$= 9\ h_{ef}^2$ (D-6) and $h_{ef} = H_{HD}$ = 16.00 inches (Embedment depth of Holdown Anchor)

$\boxed{A_{NCO} = 2{,}304\ in^2}$

$\Psi_1 = \Psi_{ed,N}$ = Modification for Edge Effects (ACI D.5.2.5)

= 1.0 if $C_{a,min} >= 1.5\ H_{ef}$ Where $C_{a1}$ = 4.50 inches (Distance from Wall CL to Left Edge of Existing Footing Wall)

$= 0.7 + 0.3\ C_{a,min} / (1.5\ H_{ef})$ if $C_{a,min} < 1.5\ H_{ef}$ $C_{a2}$ = 7.17 inches (Distance from Wall CL to Right Edge of Existing Footing Wall)

$\boxed{C_{a,min} = 4.50\ inches}$

$H_{ef} = H_{HD}$ = 16.00 inches (Embedment depth of Holdown Anchor)

$\boxed{\Psi_1 = \Psi_{ed,N} = 0.76}$ Modification for Edge Effects (ACI D.5.2.5)

**EXISTING FOUNDATION EVALUATION- SHEAR WALL AT GRIDLINE A (EVAL CASE E-1)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

$\Psi_2 = \Psi_{c,N}$ = Modification for Uncracked Concrete (ACI D.5.2.6)

= 1.0 Note: Existing Concrete is assume to crack at anchor bolt limit state or capacity of holdown.

| $\Psi_2 = \Psi_{c,N}$ = 1.0 Modification for Uncracked Concrete (ACI D.5.2.6) |
|---|

$\Psi_3 = \Psi_{cp,N}$ = Modification for Post-Installed Anchors (ACI D.5.2.7)

= 1.0 if $C_{a,min}$ >= $C_{ac}$ for $C_{a,min}$ = 4.50 inches

$C_{ac}$ = 2.5 $H_{ef}$ and $H_{ef} = H_{HD}$ = 16.00 inches (Embedment depth of Holdown Anchor)

| $C_{ac}$ = 40.0 inches |
|---|

= Max ( $C_{a,min}$ / $C_{ac}$ , 1.5 $H_{ef}$ / $C_{ac}$ ) if $C_{a,min}$ < $C_{ac}$

= Max (0.11, 0.60 )

| $\Psi_3 = \Psi_{cp,N}$ = 0.60 Modification for Post-Installed Anchors (ACI D.5.2.7) |
|---|

| $\phi_{EQ}$ $\phi_{sa,cb}$ $N_{cb}$ = 3.33 Kips |
|---|

(Concrete Break-out Strength - Tension)

c) <u>Pullout Strength of Anchor - Tension</u> (ACI 318-08 Section D5.3)

Note: Condition B is assumed per Section D4.4, where Supplementary reinforcement in not present in failure prism.

$\phi_{EQ}$ $\phi_{sa,po}$ $N_p$ = $\phi_{EQ}$ $\phi_{sa,po}$ $N_p$ $\Psi_4$ (D-14)

Where $\phi_{EQ}$ = 0.75 (Steel Anchor - Seismic Region - ACI Section D.3.3.3)

$\phi_{sa,po} = \phi_{sa,cb}$ = 0.65 (Steel Anchor - Concrete Breakout/Pullout Category 2 - ACI D.4.4)
Cat 1 : Low Sensitivity to installation and High Reliability

$N_p$ = Pull-out Strength of a single anchor in Tension in Cracked Concrete (ACI D.5.2.2)

= 8 $A_{brg}$ $f'_c$ (D-15) and $A_{brg} = A_{se,N}$ = Bearing Area of Anchor Bolt

= 0.307 $in^2$ for $t_{HD}$ = 0.63 inches (Diameter of Holdown Anchor)

$f'_c$ = 3.25 Ksi
= 3,250 Psi

| $N_p$ = 7.98 kips |
|---|

$\Psi 4 = \Psi_{c,P}$ = Modification for uncracked Concrete (ACI D.5.2.7)

= 1.0 Note: Existing Concrete is assumed to crack at anchor bolt limit state or capacity of holdown.

| $\Psi 4 = \Psi_{c,P}$ = 1.0 Modification for Uncracked Concrete (ACI D.5.2.6) |
|---|

| $\phi_{EQ}$ $\phi_{sa,po}$ $N_p$ = 3.89 Kips |
|---|

(Concrete Pull-out Strength - Tension)

d) <u>Concrete Side-face Blowout Strength of Anchor - Tension</u> (ACI 318-08 Section D5.4)

$\phi_{EQ}$ $\phi_{sa,sf}$ $N_p$ = $\phi_{EQ}$ $\phi_{sa,po}$ $N_p$

Where $\phi_{EQ}$ = 0.75 (Steel Anchor - Seismic Region - ACI Section D.3.3.3)

$\phi_{sa,sf} = \phi_{sa,cb}$ = 0.65 (Steel Anchor - Concrete Breakout/Pullout/Side Blowout Category 2 - ACI D.4.4)
Cat 1 : Low Sensitivity to installation and High Reliability

$N_p = 160\ C_{a1}\ A_{brg}^{0.5}\ \lambda\ f'^{0.5}_c$ Where $C_{a1}$ = 4.50 inches (Distance from Wall CL to Left Edge of Existing Footing Wall)

$C_{a2}$ = 7.17 inches (Distance from Wall CL to Right Edge of Existing Footing Wall)

| $C_{a,min}$ = 4.50 inches |
|---|

and $A_{brg} = A_{se,N}$ = 0.307 $in^2$ for $t_{HD}$ = 0.63 inches (Diameter of Holdown Anchor)

$\lambda$ = 1.00 (1.0 for NWC, 0.75 for LWC)
<u>Note:</u> NWC - Normal Weight Concrete Assumed

$f'_c$ = 3.25 Ksi
= 3,250 Psi

| $N_p$ = 22.74 Kips |
|---|

| $\phi_{EQ}$ $\phi_{sa,sf}$ $N_p$ = 11.1 Kips |
|---|

(Concrete Side-face Blowout Strength - Tension)

**EXISTING FOUNDATION EVALUATION- SHEAR WALL AT GRIDLINE A (EVAL CASE E-1)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

e) Limiting Governing Strength of Anchor - Tension Only

Note: Holdown = HDU4

$F_{CU}$ = 6.53 Kips (Ultimate Capacity of Holdown)

$H_{HD}$ = 16.00 inches (Embedment depth of Holdown Anchor)

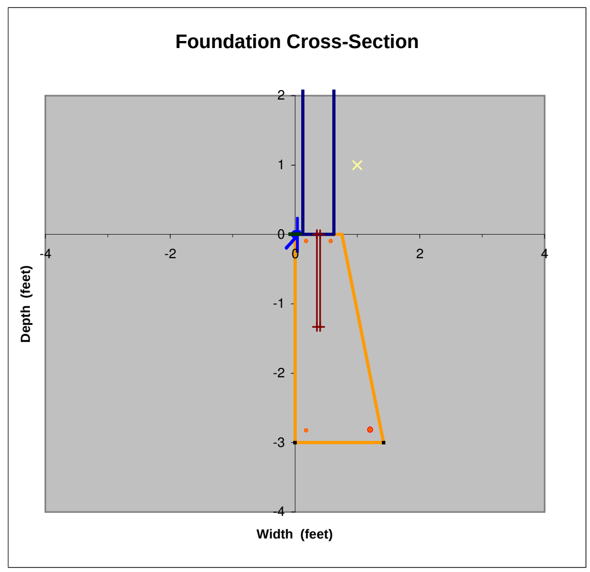
$t_{HD}$ = 0.63 inches

Limiting Strength of Anchor in Tension:

$T_{UC} = \text{Min}\,(\phi_{EQ}\ \phi_{sa,t}\ N_{sa},\ \phi_{EQ}\ \phi_{sa,cb}\ N_{cb},\ \phi_{EQ}\ \phi_{sa,po}\ N_p,\ \phi_{EQ}\ \phi_{sa,sf}\ N_p)$

| | | | |
|---|---|---|---|
| Where $\phi_{EQ}\ \phi_{sa,t}\ N_{sa}$ = | 16.57 | Kips | (Bolt Design Strength) |
| $\phi_{EQ}\ \phi_{sa,cb}\ N_{cb}$ = | 3.33 | Kips | (Concrete Break-out Strength) |
| $\phi_{EQ}\ \phi_{sa,po}\ N_p$ = | 3.89 | Kips | (Concrete Pull-out Strength) |
| $\phi_{EQ}\ \phi_{sa,sf}\ N_p$ = | 11.1 | Kips | (Concrete Side-face Blowout Strength) |

$\boxed{T_{UC} = 3.33 \text{ Kips}}$




Required Number of Holdown Anchors:

$N_A = F_{CU} / T_{UC}$ Where $F_{CU}$ = 6.53 Kips (Ultimate Capacity of Holdown)

= 1.96 $T_{UC}$ = 3.33 Kips (Strength per Anchor)

$N_A$ = 2

**Use 2 - HDU4 with 0.63 Dia anchors @ 16" oc Min EA side**

**EXISTING FOUNDATION EVALUATION- SHEAR WALL AT GRIDLINE A (EVAL CASE E-1)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

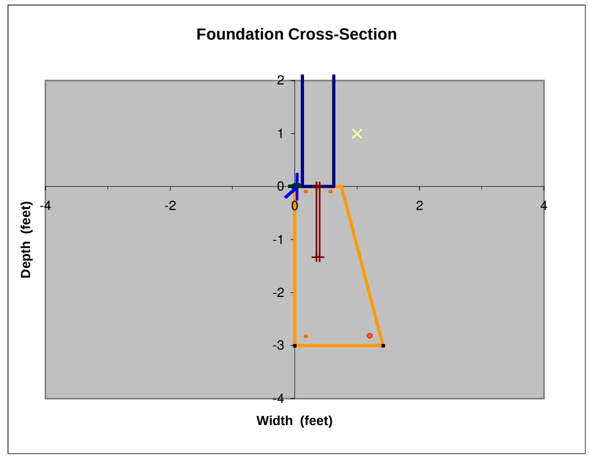
## 6. Adequacy of Footing - Shear

### A. Flexural/One-Way Shear on Full Cross-Section

Note: Effective transverse width of footing for Shear is assumed as Top of Footing Width, $L_y = L'_T$

Shear demands: $V_{ux}$ = 6.3 Kips @ $x_L$ = 0.27 feet (locations at distance d from face of Wall - Left side)
= 8.1 Kips @ $x_R$ = 47.73 feet ( - Right side)

= > $V_{ux}$ = 8.1 Kips




ii) Shear capacity of concrete without shear reinforcement

$$\varphi V_c = \varphi\left(1.9\sqrt{f'_c} + 2500\,\rho_w \frac{V_u d}{M_u}\right) b_w d \;\le\; 3.5\sqrt{f'_c}\, b_w d \qquad (11\text{-}5)$$

Note: $V_u d/M_u$ value must be ≤ 1.0

Where $\phi$ = 0.75
$f'_c$ = 3,250 psi

$\rho_w = A_{sx}/(L_y d_x)$ and $A_{sx}$ = 0.60 in^2
$L_y$ = 0.75 feet
= 9.0 inches
d = 32.75 inches

$\rho_w$ = 0.002036

$V_u$ = 8 kips
d = 32.75 inches

$M_u$ = 3 Kip-ft @ $V_{ux}$ = 8 Kips (location of shear value)

Check of $V_u$ d/$M_u$ value limit:
$V_u d/M_u$ = 7.78 where Vu = 8 kips
**NG, value taken as unity.**
d = 32.75 inches
$M_u$ = 3 kip-ft
= 34 kip -in

$b_w = L'_T$ = 0.8 feet
= 9.0 inches
d = 32.75 inches

= > $\phi V_c$ = 25.1 kips

Comparison w/ Equation 11-3:
$\phi V_c = \phi\, 2\, f'^{0.5}_c\, b_w d$
Where $\phi$ = 0.75
$f'_c$ = 3,250 psi
$b_w = L'_T$ = 9.0 inches
d = 32.75 inches

$\phi V_c$ = 25.2 kips

Check of upper value limit:
$\phi V_{c,max} = 3.5\, f'^{0.5}_c\, b_w d$
Where $f'_c$ = 3,250 psi
$b_w = L'_T$ = 9.0 inches
d = 32.75 inches

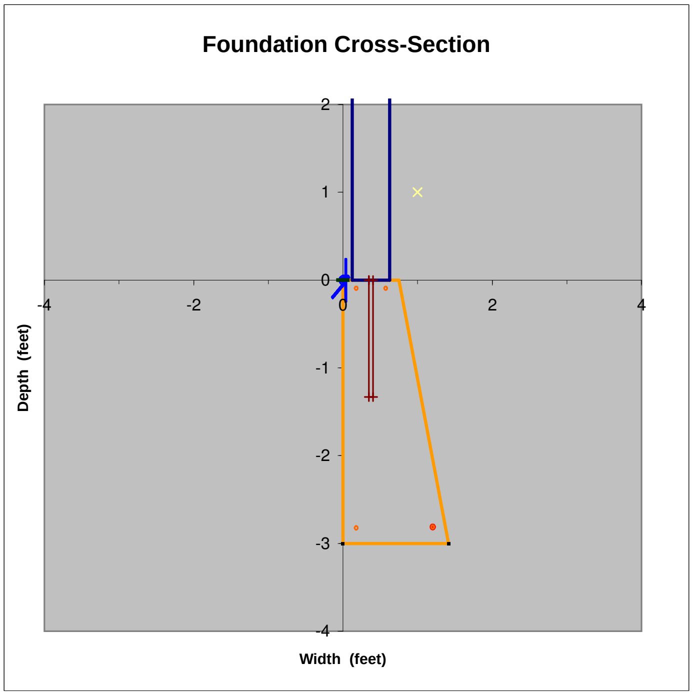
$\phi V_{c,max}$ = 58.8 kips

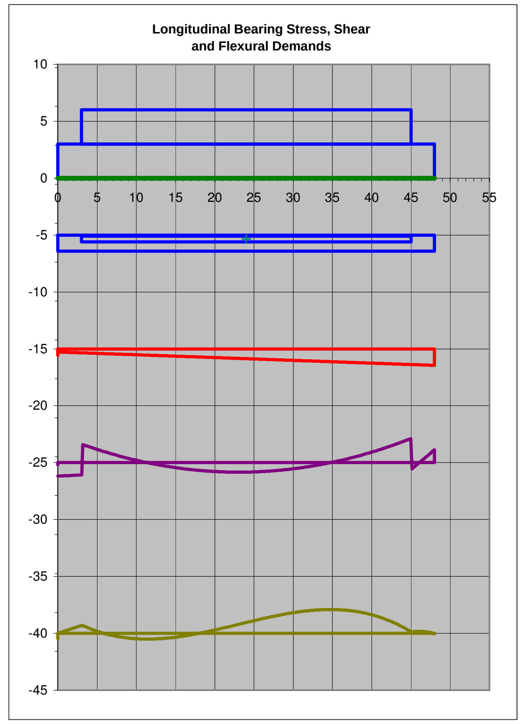
$\phi V_c$ = 25.1 kips
**OK, > Vu**

Note: D/C Ratio = 0.32 (Demand to Capacity Ratio - Shear)

**EXISTING FOUNDATION EVALUATION- SHEAR WALL AT GRIDLINE A (EVAL CASE E-1)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 7. Adequacy of Footing - Flexure




a) Flexural demands $M_{ux}$ = 14 Kip-ft @ $x_L$ = 3.00 feet (locations at face of column - Left side)
= 163 Kip-in Note: $X_f$ = 3.00 feet (Cantilever Length)

= 18 Kip-ft @ Wall and Footing Centerline
= 221 Kip-in

= 3 Kip-ft @ $x_R$ = 45.00 feet ( - Right side)
= 34 Kip-in Note: $X_f$ = 3.00 feet (Cantilever Length)

= > $\boxed{M_{ux} = 221 \text{ Kip-in}}$

b) Required Reinforcement Ratio (ACI 10.2)

$$\rho_r = \frac{0.85\, f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2\, M_u}{0.765\, b\, d^2\, f'_c}}\right]$$

Where $f'_c$ = 3.25 Ksi
$f_y$ = 40.00 Ksi
$M_u$ = 221 kip-in
$L_y = L_B$ = 1.4 feet
= 17 inches
$d_x$ = 32.75 inches

$\boxed{\rho_r = 0.000338}$

c) Reinforcement Ratio Provided

$\rho_w = A_{sx}/(L_y\, d_x)$ Where $A_{sx} = A_N + A_E$

$A_N$ = 0.00 $in^2$ (New Flexural Reinforcement)
$A_E$ = 0.40 $in^2$ (Existing Flexural Reinforcement)

$\boxed{A_{sx} = 0.40 \ in^2}$

$L_y = L_B$ = 1.4 feet
= 17.0 inches
d = 32.75 inches

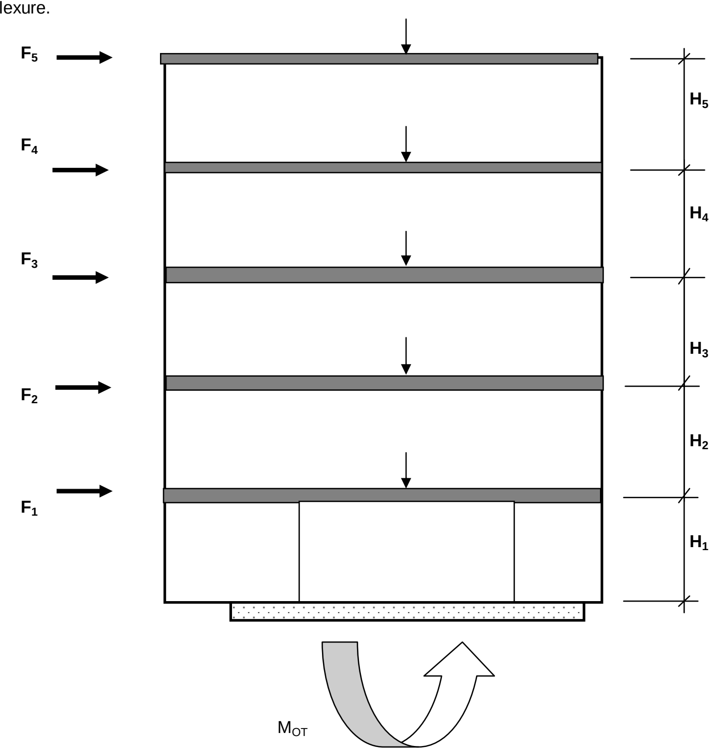
$\boxed{\rho_w = 0.000718}$ (reinforcement ratio provided) Note: D/C Ratio = 0.47 (Demand to Capacity Ratio - Flexure)

OK

**Existing Reinforcement OK, DC Ratio = 0.47**

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE 3 (EVAL CASE E-2R)**
**DETERMINATION OF VERTICAL AND LATERAL LOADS TO FOUNDATION**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Assumptions**
1. Existing walls are treated as pinned Columns. Footing is assumed rigid.
2. Column loads are located in transverse center of footing; limit of 2 columns w/o flexure.
3. Footing has no shear reinforcement.
4. Concrete is Normal Weight Concrete with uncoated bars.




**1. Lateral Loads and Load Effects**

V = 21.60 kips (Base Shear - A4 **ASD**)

| Floor Level | Height (feet) | Loading ID | $V_x$ / V | Shear (Kips) | Force (Kips) | Moment (Kip-ft) |
|---|---|---|---|---|---|---|
| | | 5 | | | | |
| R | 11.00 | 4 | 0.30 | 6.48 | 6.48 | 291.6 |
| 3 | 10.00 | 3 | 0.65 | 14.04 | 7.56 | 548.6 |
| 2 | 13.50 | 2 | 0.89 | 19.22 | 5.18 | 673.1 |
| 1 | 10.50 | 1 | 1.00 | 21.60 | 2.38 | 698.0 |

**$M_{OT}$ = 698.00 Kip-ft**

**2. Vertical Loads and Load Effects**

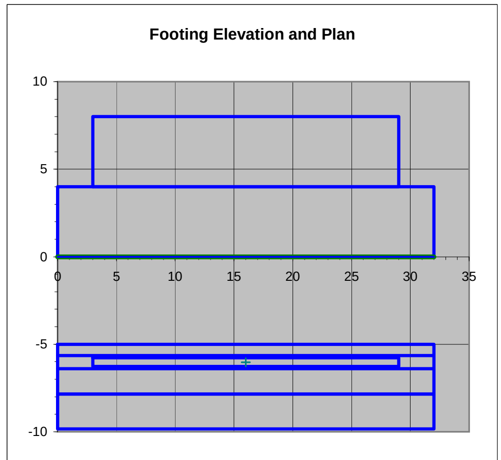
| | | Floor Tributary Loads | | | | | Wall Tributary Loads | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Wall** | Floor Level | DL (psf) | Length (feet) | Width (feet) | Area ($ft^2$) | Weight (kips) | WL (psf) | Length (feet) | Height (feet) | Area ($ft^2$) | Weight (kips) |
| | | | | | | | | | | | |
| **1** | R | 20 | 26.00 | 2.00 | 52 | 1.04 | 14 | 26.00 | 11.00 | 286 | 4.00 |
| | 3 | 30 | 26.00 | 2.00 | 52 | 1.56 | 14 | 26.00 | 10.00 | 260 | 3.64 |
| | 2 | 30 | 26.00 | 2.00 | 52 | 1.56 | 14 | 26.00 | 13.50 | 351 | 4.91 |
| | 1 | 30 | 26.00 | 2.00 | 52 | 1.56 | 14 | 26.00 | 10.50 | 273 | 3.82 |

Sum of Floor Weight = 5.72 Kips

Sum of Wall Weight = 16.38 Kips

**$P_{D1}$ = 22.10 Kips**

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE 3 (EVAL CASE E-2R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**



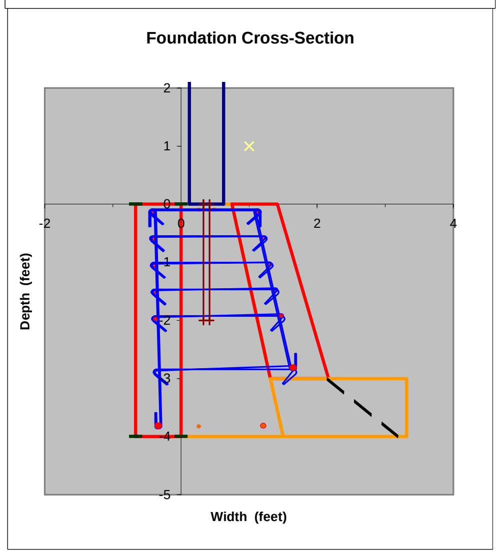

**Assumptic** 1. Footing has no shear reinforcement.
2. Concrete is Normal Weight Concrete with uncoated bars.

## Footing Parameters :

Footing Size :
$L_x$ = 32.0 feet    $d_s$ = 0.0 feet (depth of soil)
$L_y$ = 2.8 feet
$h_f$ = 4.0 feet

Wall Location :
$x_c$ = 16.0 feet (Wall centerline distance from Left Edge)
$y_c$ = 1.8 feet (Wall centerline distance from Bottom Edge)

Wall Size :
$C_x$ = 26.0 feet (Wall length)
$C_y$ = 0.5 feet (Wall width)




Interconnected Slab at Sides:

Note : Slabs at sides are used only to reduce soil bearing pressure; footing is designed to take all loads.

Side : Left Right
t    Inches (Slab Thickness)
X    Feet (distance to other Slab Edge Support
f'c    Ksi
Conn Type    (D= Dowel, C= Continuous)

Concrete :
$f'_c$ = 3.25 Ksi
$f_y$ = 40.00 Ksi
$\rho_c$ = 0.150 kip/ft^3

Reinforcement:
$d_c$ = 1.00 inches (bar clearance - top)
$d_c$ = 2.00 inches (bar clearance - bottom)
= 3.00 inches (bar clearance - sides)

| | Orientation | Bar Size | N Bars | Bottom Layer | d (inches) | Bar Diameter (inches) | Bar Area Per Bar (in^2) | Bar Area Total (in^2) |
|---|---|---|---|---|---|---|---|---|
| Top Mat | x | 6 | 1 | x | 44.25 | 0.75 | 0.44 | 0.44 |
| Bottom Mat | x | 6 | 1 | | 44.88 | 0.75 | 0.44 | 0.44 |

$L_B$ = 3.83 feet (Bearing Length)

Note: Reinf Layout = 2
(Two-Sided Retrofit W:

## 1. Design of Slab-to-Footing Connections

**No Slabs connected to Footing.**

## 2. Lateral Resistance of Foundation

**Foundation OK for Sliding**

## 3. Soil Pressure due to Applied Loads

**Footing Bearing stress OK**

## 5. Adequacy of Footing - Anchor Pull-out in Existing Footing

**Use 2 - HDU14 with 1.13 Dia anchors @ 24" oc Min EA side**

## 5. Adequacy of Footing - Shear

**Footing OK for Shear**

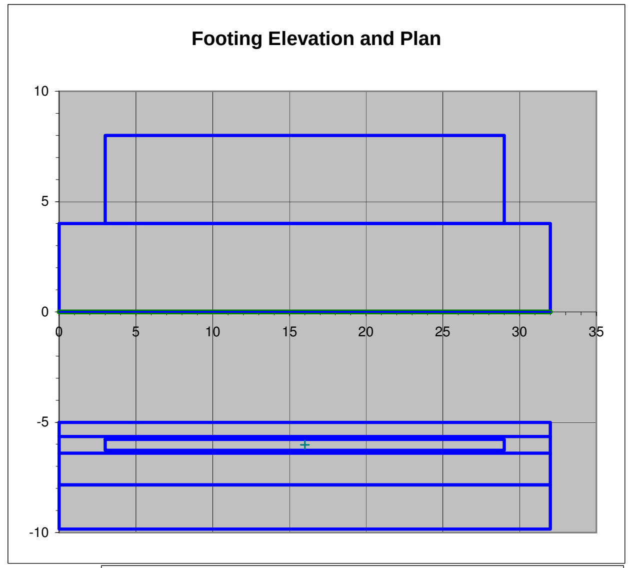
## 6. Adequacy of Footing - Flexure

**1 - # 6 Bars OK for Longitudinal Flexure with DC Ratio = 0.41**

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE 3 (EVAL CASE E-2R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Assumptions :**
1. Existing Footing has no shear reinforcement.
2. Concrete is Normal Weight Concrete with uncoated bars.
3. Wall is centered on Footing; Longitudinal Heel and Toe are equal in length.

## Footing Parameters :




Wall Size :
$C_x$ = 26.0 feet (Wall length)
$C_y$ = 0.5 feet (Wall width)

Wall Location :
$x_c$ = 16.0 feet (Wall centerline distance from Left Edge)
$y_c$ = 1.8 feet (Wall centerline distance from Bottom Edge)

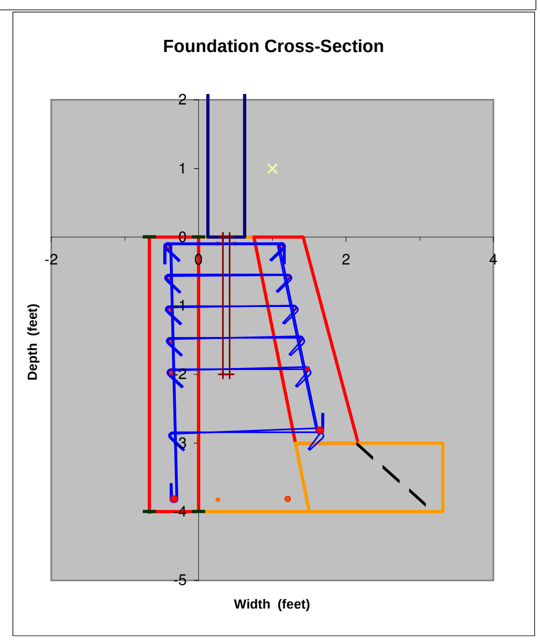
Holdown Anchor :
Holdown = **HDU14**
$t_{HD}$ = 1.13 inches (Diameter of Holdown Anchor)
Note: $t'_{HD}$ = 1.00 inches (Diameter of Holdown Anchor Required for Holdown)

$F_{CA}$ = 14.38 Kips (Allowable Capacity of Holdown)
$F_{CU}$ = 20.13 Kips (Ultimate Capacity of Holdown)

$H_{HD}$ = 24.00 inches (Embedment depth of Holdown Anchor)
OK

Footing Dimensions :
$L_x$ = 32.00 feet (Footing Length)
$h_f$ = 4.00 feet (Footing Height)

| | Left | Right | |
|---|---|---|---|
| $L_H$ = | | 2.00 | feet (Length of Heel, if any) |
| $h_H$ = | | 1.00 | feet (Thickness of Heel, if any) |




(E) Footing Width:
$W_T$ = 9.00 inches (Footing Width - at Top of Wall)
= 0.00 inches (Increase in Width - at Left)
= 9.00 inches (Increase in Width - at Right)
$W_B$ = 1.50 feet (Footing Width - at Bottom of Wall)

(N) Retrofit Walls: Note: Reinf Layout = 2 (Two-Sided Retrofit Walls)

Concrete Weight : NWC LWC - Lean Weight Concrete, NWC otherwise)

| | Left | Right | |
|---|---|---|---|
| $t_T$ = | 8.00 | 8.00 | inches (Retrofit Wall Thickness - Top of Wall) |
| $t_B$ = | 8.00 | 8.00 | inches (Retrofit Wall Thickness - Bottom of Wall) |

$W'_T$ = 2.08 feet (Footing Width - at Top of Wall)
$W'_B$ = 2.83 feet (Footing Width - at Bottom of Wall)

$t_H$ = inches (Retrofit Wall Thickness of Heel, if any)

Dowel Data : No. 4 Dowels
$S_D$ = 12.00 inches (Dowel Spacing - Horiz and Vertical)

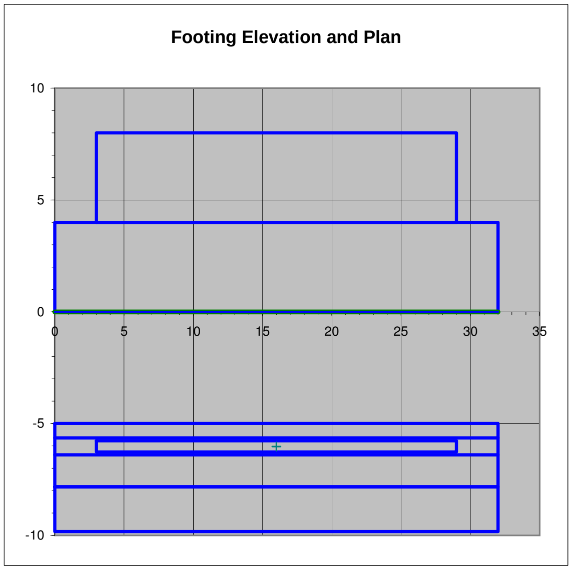
$L_B$ = 3.83 feet (Bearing Length)

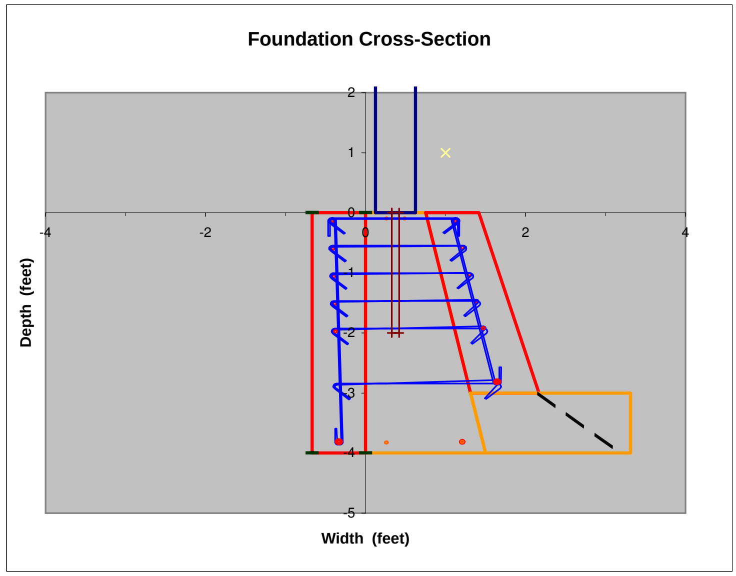
Footing Loads : 21.6

| | Service | Strength | |
|---|---|---|---|
| P = | 22.1 | 30.9 | kips |
| $M_y$ = | 698.0 | 977.2 | kip-ft |
| $V_x$ = | 21.60 | 30.2 | kips |
| $M_x$ = | 0 | 0.3 | kip-ft |
| $V_y$ = | 0.0 | 0.0 | kips |

Plastic Hinge Centroidal Heights:
@ $h_{px}$ = 0.00 feet
@ $h_{py}$ = 0.00 feet

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE 3 (EVAL CASE E-2R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**




Capacity Factors :

Concrete :

$\phi_v$ = 0.75 (Shear)
$\phi_b$ = 0.65 (Bearing)

Steel Anchor in Concrete:

$\phi_{EQ}$ = 0.75 (Steel Anchor - Seismic Region - ACI Section D.3.3.3)
$\phi_{sa,t}$ = 0.75 (Steel Anchor - Tension, Ductile Steel Element - ACI D.4.4)
$\phi_{sa,cb}$ = 0.65 (Steel Anchor - Concrete Breakout Category 2 - ACI D.4.4)
Note: Cat 1 : Low Sensitivity to installation and High Reliability

Material Properties :

Concrete :

$f'_{ce}$ = 3.25 Ksi (Existing Concrete)
$f'_{cn}$ = 2.50 Ksi (New Concrete)
$f_y$ = 40 Ksi (Existing Reinforcement)
$\rho_c$ = 0.150 kip/ft^3

Note : Year Built : 1921

Steel Anchor in Concrete:

$f_{ya}$ = 60.00 Ksi (PCA Notes Table 34-1 - ASTM A307)

Interconnected Slab at Sides:

Note : Slabs at sides are used only to reduce soil bearing pressure; footing is designed to take all loads.

| Side : | Left | Right | |
|---|---|---|---|
| t | | | Inches (Slab Thickness) |
| h | | | Inches (Distance to Top of Wall) |
| X | | | Feet (distance to other Slab Edge Support |
| $f'_c$ | | | Ksi |
| Conn Type | | | (D= Dowel, C= Continuous) |

Reinforcement:

$d_c$ = 1.00 inches (bar clearance - top)
$d_c$ = 2.00 inches (bar clearance - bottom)
= 3.00 inches (bar clearance - sides)

Existing Reinforcement:

| | | | |
|---|---|---|---|
| Top | 2 | No. | 4 |
| Bottom | 2 | No. | 4 |

| New Flexural Reinforcement EA Side | | | | | | | Bar Area | |
|---|---|---|---|---|---|---|---|---|
| Location | Bar Size | N Bars (Max 3) | Bottom Layer | d (inches) | b (inches) | Bar Diameter (inches) | Per Bar (in^2) | Total (in^2) |
| Top | 6 | 1 | x | 44.25 | 25.00 | 0.75 | 0.44 | 0.44 |
| Bottom | 6 | 1 | | 44.88 | 58.00 | 0.75 | 0.44 | 0.44 |

Soil Parameters :

$\sigma_{allow}$ = 2.04 ksf (Allowable Bearing Pressure)
$\sigma_p$ = 0.30 ksf/ft (Passive Soil Pressure)
$\mu$ = 0.27 ksf (Coefficient of Friction)

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE 3 (EVAL CASE E-2R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 2. Lateral Resistance of Foundation

2A. **Longitudinal Loading**

$F_{Rx} = 0.5\, L'_y\, h'^2_f\, \sigma_p + 0.6\,(W_f + P)\,\mu$

Where $L'_y = W'_B$ — $W'_B =$ 3.83 feet (Footing Width - at Bottom of Wall)

$L'_y =$ 3.83 feet (Bearing Width at Ends of Footing)

$h'_f = h_f + h_{sk}$ and $h_f =$ 4.00 feet

$h_{sk} =$ 0.00 feet (Additional height of Shear Key at Footing E

$h'_f =$ 4.0 feet (Bearing Height at Ends of Footing)

= (30.7) (0.30) + 0.6 (76.5) (0.27) — $\sigma_p =$ 0.30 ksf/ft (Passive Soil Pressure)

= (9.20) + (12.58) — $W_f = \rho_c\, L_x\, L_y\, h_f$ and $\rho_c =$ 0.150 kip/ft^3

$L_x =$ 32.0 feet 0

$L_y =$ 2.8 feet

$h_f =$ 4.0 feet

$W_f =$ 54.40 Kips (Footing Weight)

$P =$ 22.1 Kips (Service Load)

$\mu =$ 0.27 ksf (Coefficient of Friction)

$F_{Rx} =$ 21.78 kips — Note : $V_x =$ 21.60 kips

**OK**

**Foundation OK for Sliding**

## 3. Soil Pressure due to Applied Loads

3A. **Longitudinal Loading**

a) Loading Eccentricity

Note: Effective transverse width of footing for bearing is assumed as Bottom of Footing Width, $L_y = L'_B$

$e_x = \Sigma M_y / P'$

Where $\Sigma M_y = M_y + V_x H_{px} - P\,(0.5\, L_x - x_c)$ and $M_y =$ 698 kip-ft

$V_x =$ 22 kips

@ $h_{px} =$ 0.00 feet

$P =$ 22 Kips

$L_x =$ 32.0 feet

= 698 kip-ft + 0 kip-ft - 0 kip-ft — $x_c =$ 16.0 feet (Column centerline distance from Left Edge)

$\Sigma M_y =$ 698 Kip-in

$P' = P + P_F$ and $P =$ 22 Kips

$P_F = \rho_c\, L_x\, L_y\, h_f$ for $\rho_c =$ 0.150 kip/ft^3

$L_x =$ 32.0 feet

$L_y = 0.5\,(L'_T + L'_B) =$ 2.5 feet

$h_f =$ 4.0 feet

$P_F =$ 47.20 kips (footing weight)

$P' =$ 69 Kips

$e_x =$ 10.07 feet

Note: $L_x/6 =$ 5.33 feet (Footing Middle Third)

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE 3 (EVAL CASE E-2R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

b) Bearing Stresses

i) for $e_x \le L_x/6$ (within Middle $L_x/3$)

$\sigma_{max}$ = P (1 + 6 $e_x$ / $L_x$) / ($L_x$ $L_y$) — Where P = 69 Kips

$\sigma_{min}$ = P (1 - 6 $e_x$ / $L_x$) / ($L_x$ $L_y$) — $e_x$ = 10.07 feet

$L_x$ = 32.0 feet

$L_y = L_B$ = 3.8 feet

| | | |
|---|---|---|
| $\sigma_{max}$ = | 1.63 | Ksf |
| $\sigma_{min}$ = | -0.50 | Ksf |

ii) for $e_x > L_x/6$ (outside Middle $L_x/3$) **<= Governs!**

$\sigma_{max}$ = 2 P / ($L_{bx}$ $L_y$) — Where P = 69 Kips

$L_{bx}$ = 3 (0.5 $L_x$ - $e_x$) — and $L_x$ = 32.0 feet; $e_x$ = 10.07 feet

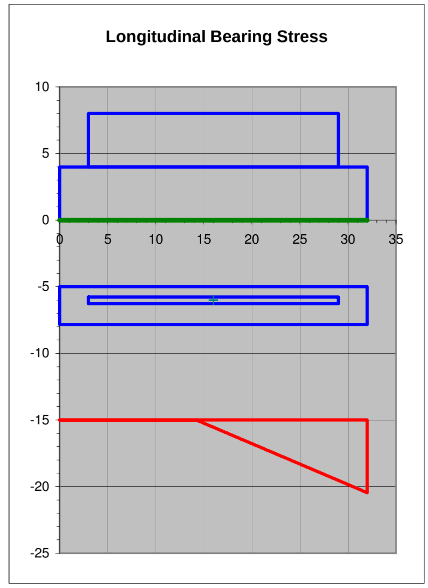
| | | |
|---|---|---|
| $L_{bx}$ = | 17.78 | feet |

$L_y = L_B$ = 3.8 feet

| | | |
|---|---|---|
| $\sigma_{max}$ = | 2.03 | Ksf |
| $\sigma_{min}$ = | 0.00 | Ksf |

iii) Governing Condition

| | | |
|---|---|---|
| $\sigma_{max}$ = | 2.03 | Ksf |
| $\sigma_{min}$ = | 0.00 | Ksf |




3B. **Transverse Loading**

a) Loading Eccentricity

Note: Effective transverse width of footing for bearing is assumed as Bottom of Footing Width, $L_y = L'_B$

$e_y = \Sigma M_x / P'$

Where $\Sigma M_x = M_x + V_y H_{py}$ - P (0.5 $L_y$ - $y_c$) — and $M_x$ = 0 kip-ft

$V_y$ = 0 kips

@ $h_{py}$ = 0.00 feet

P = 22 Kips

$L_y = L_B$ = 3.8 feet

= 0 kip-ft + 0 kip-ft - 2 kip-ft — $y_c$ = 1.8 feet (Column centerline distance from Left Edge)

| | | |
|---|---|---|
| $\Sigma M_y$ = | -2 | Kip-in |

P' = 69 Kips

| | | |
|---|---|---|
| $e_y$ = | -0.03 | feet |

Note: $L_y/6$ = 0.64 feet (Footing Middle Third)

b) Bearing Stresses

i) for $e_y \le L_y/6$ (within Middle $L_y/3$) **<= Governs!**

$\sigma_{max}$ = P' (1 + 6 $e_y$ / $L_y$) / ($L_x$ $L_y$) — Where P' = 69 Kips

$\sigma_{min}$ = P' (1 - 6 $e_y$ / $L_y$) / ($L_x$ $L_y$) — $e_y$ = -0.03 feet

$L_y = L_B$ = 3.8 feet

$L_x$ = 32.0 feet

| | | |
|---|---|---|
| $\sigma_{max}$ = | 0.59 | Ksf |
| $\sigma_{min}$ = | 0.54 | Ksf |

ii) for $e_x > L_x/6$ (outside Middle $L_x/3$)

$\sigma_{max}$ = 2 P' / ($L_{by}$ $L_x$) — Where P' = 69 Kips

$L_{by}$ = 3 (0.5 $L_y$ - $e_y$) $\le$ $L_y$ — and $L_y = L_B$ = 3.8 feet; $e_y$ = -0.03 feet

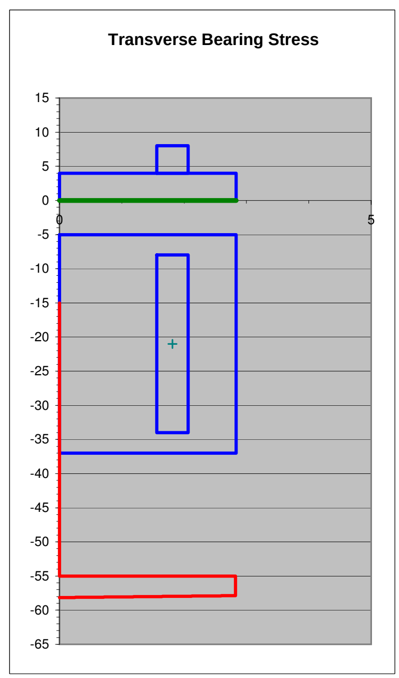
| | | |
|---|---|---|
| $L_{by}$ = | 3.83 | feet |

$L_x$ = 32.0 feet

| | | |
|---|---|---|
| $\sigma_{max}$ = | 1.13 | Ksf |
| $\sigma_{min}$ = | 0.00 | Ksf |

iii) Governing Condition

| | | |
|---|---|---|
| $\sigma_{max}$ = | 0.59 | Ksf |
| $\sigma_{min}$ = | 0.54 | Ksf |



**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE 3 (EVAL CASE E-2R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**3C. Check of Bearing Stresses on Soil**

a) Assuming all loads resisted by Footing only

$\sigma_{max} = \text{Max}(\sigma_{max,x}, \sigma_{max,y})$ Where $\sigma_{max}$ = 2.03 Ksf (Longitudinal Direction)

= 0.59 Ksf (Transverse Direction)

b) Assuming all loads resisted by Footing + Adjoining Slabs

Note: Effective transverse width of footing for bearing is assumed as Bottom of Footing Width, $L_y = L'_B$

$\sigma_{max} = \text{Max}(\sigma_{max,x}, \sigma_{max,y}) * L_y / b$ Where $\sigma_{max}$ = 2.03 Ksf (Longitudinal Direction) and $L_y = L_B$ = 3.8 feet

= 0.59 Ksf (Transverse Direction) b = 46.00 inches

= ( 2.03 * 1.00 ) = 3.83 feet

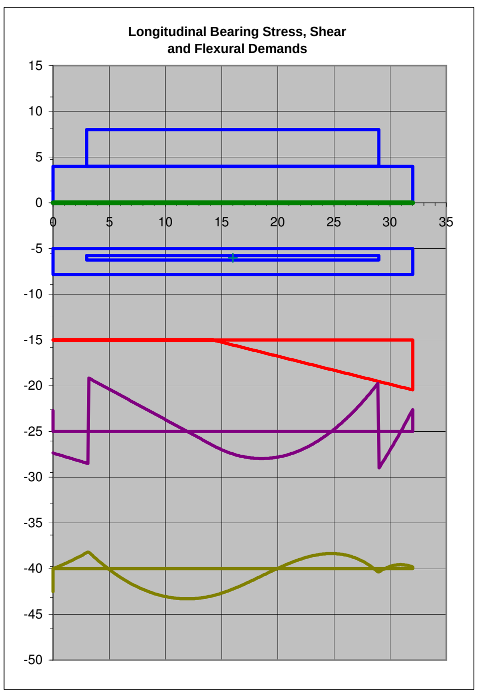
$\boxed{\sigma_{max} = 2.03 \text{ Ksf}}$

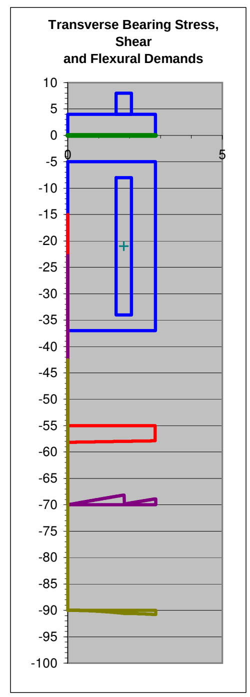
OK

Note: $\sigma_{allow}$ = 2.04 ksf (allowable bearing pressure)

**Footing Bearing stress OK**

## 4. Applied Loading and Demands on Footing



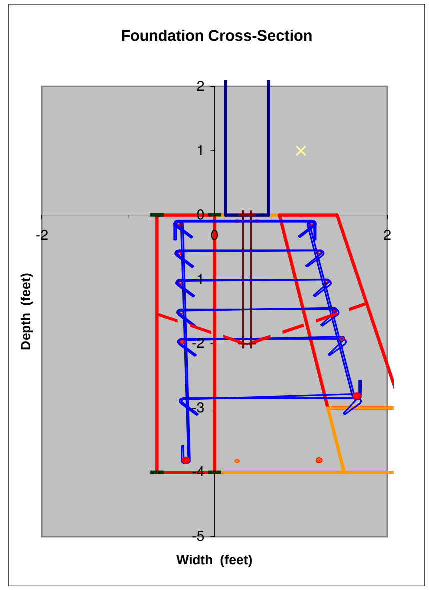




| | Longitudinal Direction | | | | |
|---|---|---|---|---|---|
| | Left End | Left Face of Wall | Wall Centerline | Right Face of Wall | Right End |
| Location (feet) | 0 | 3.00 | 16.00 | 29.00 | 32.00 |
| $\sigma = q_u/L_y$ (ksf) | 0.00 | 0.00 | 0.27 | 2.35 | 2.85 |
| P (kips) | - | - | 31 | - | - |
| $M_y + V\,h_{px}$ (kip-ft) | - | - | 977 | - | - |
| V+ (kips) | - | 23 | - | 20 | 10 |
| V- (kips) | -9 | -14 | -10 | -15 | - |
| M+ (kip-ft) | 0 | 36 | - | - | 4 |
| M- (kip-ft) | - | - | -46 | -5 | - |

| | Transverse Direction | | | | |
|---|---|---|---|---|---|
| | Left End | Left Face of Column | Column Centerline | Right Face of Column | Right End |
| Location (feet) | 0.00 | 1.56 | 1.81 | 2.06 | 2.83 |
| $\sigma = q_u/L_y$ (ksf) | 0.59 | 0.56 | 0.56 | 0.55 | 0.54 |
| P (kips) | - | - | 31 | - | - |
| $M_y + V\,h_{px}$ (kip-ft) | - | - | 0 | - | - |
| V+ (kips) | 0 | 29 | 33 | 7 | 20 |
| V- (kips) | - | - | - | - | - |
| M+ (kip-ft) | 0 | - | - | - | - |
| M- (kip-ft) | - | -23 | -30 | -32 | -43 |

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE 3 (EVAL CASE E-2R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 5. Adequacy of Footing - Anchor Pull-out in Existing Footing




Holdown = HDU14

$F_{CU}$ = 20.13 Kips (Ultimate Capacity of Holdown)

$H_{HD}$ = 24.00 inches (Embedment depth of Holdown Anchor)

a) Bolt Design Strength - Tension (ACI 318-08 D3.3.3, D4.4, D5.1)

$\phi_{EQ}\ \phi_{sa,t}\ N_{sa} = \phi_{EQ}\ \phi_{sa,t}\ n\ A_{se,N}\ f_{uta}$ (D-3)

Where $\phi_{EQ}$ = 0.75 (Steel Anchor - Seismic Region - ACI Section D.3.3.3)

$\phi_{sa,t}$ = 0.75 (Steel Anchor - Tension, Ductile Steel Element - ACI Section D.4.4)

n = 1 (number of anchors)

$A_{se,N} = \pi\ t_{HD}^2$ for $t_{HD}$ = 1.13 inches (Diameter of Holdown Anchor)

$\boxed{A_{se,N} = 0.994 \text{ in}^2}$

$f_{uta}$ = Min (1.6 $f_{ya}$, 125) for $f_{ya}$ = 60.00 Ksi (PCA Notes Table 34-1 - ASTM A307)

$\boxed{f_{uta} = 96.00 \text{ Ksi}}$

$\boxed{\phi_{EQ}\ \phi_{sa,t}\ N_{sa} = 53.68 \text{ Kips}}$ (Bolt Design Strength - Tension)

b) Concrete Breakout Strength - Tension (ACI 318-08 Section D5.2)

Note: Condition B is assumed per Section D4.4, where Supplementary reinforcement in not present in failure prism.

$\phi_{EQ}\ \phi_{sa,cb}\ N_{cb} = \phi_{EQ}\ \phi_{sa,cb}\ N_b\ A_{NC} / A_{NCO}\ \Psi_1\ \Psi_2\ \Psi_3$ (D-4)

Where $\phi_{EQ}$ = 0.75 (Steel Anchor - Seismic Region - ACI Section D.3.3.3)

$\phi_{sa,cb}$ = 0.65 (Steel Anchor - Concrete Breakout Category 2 - ACI D.4.4)

Note: Cat 1 : Low Sensitivity to installation and High Reliability

$N_b$ = Basic concrete Break-out Strength of a single anchor in Tension in Cracked Concrete (ACI D.5.2.2)

= $K_C\ f'^{0.5}_c\ H_{ef}^{1.5}\ \lambda$ and $K_C$ = 17 Post-installed Anchors (D5.2.2)

$f'_c$ = 3.25 Ksi
= 3,250 Psi

$H_{ef} = H_{HD}$ = 24.00 inches (Embedment depth of Holdown Anchor)

$\lambda$ = 1.00 (1.0 for NWC, 0.75 for LWC)

Note: NWC - Normal Weight Concrete Assumed

$\boxed{N_b = 113.95 \text{ Kips}}$

$A_{NC}$ = Projected Concrete Failure Area - Actual (ACI D5.2.1)

= 2 (1.5 $H_{ef}$ ) ($C_{a1}$ + $C_{a2}$) and $H_{ef} = H_{HD}$ = 24.00 inches (Embedment depth of Holdown Anchor)

$C_{a1}$ = 12.50 inches (Distance from Wall CL to Left Edge of Existing Footing Wall)

$C_{a2}$ = 15.88 inches (Distance from Wall CL to Right Edge of Existing Footing Wall)

$\boxed{A_{NC} = 2{,}043 \text{ in}^2}$

$A_{NCO}$ = Projected Concrete Failure Area - Ideal (ACI D5.2.1)

= 9 $h_{ef}^2$ (D-6) and $h_{ef} = H_{HD}$ = 24.00 inches (Embedment depth of Holdown Anchor)

$\boxed{A_{NCO} = 5{,}184 \text{ in}^2}$

$\Psi_1 = \Psi_{ed,N}$ = Modification for Edge Effects (ACI D.5.2.5)

= 1.0 if $C_{a,min} >= 1.5\ H_{ef}$ Where $C_{a1}$ = 12.50 inches (Distance from Wall CL to Left Edge of Existing Footing Wall)

= 0.7 + 0.3 $C_{a,min}$ / (1.5 $H_{ef}$) if $C_{a,min}$ < 1.5 $H_{ef}$ $C_{a2}$ = 15.88 inches (Distance from Wall CL to Right Edge of Existing Footing Wall)

$\boxed{C_{a,min} = 12.50 \text{ inches}}$

$H_{ef} = H_{HD}$ = 24.00 inches (Embedment depth of Holdown Anchor)

$\boxed{\Psi_1 = \Psi_{ed,N} = 0.80}$ Modification for Edge Effects (ACI D.5.2.5)

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE 3 (EVAL CASE E-2R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

$\Psi_2 = \Psi_{c,N} =$ Modification for Uncracked Concrete (ACI D.5.2.6)

= 1.0 Note: Existing Concrete is assume to crack at anchor bolt limit state or capacity of holdown.

$\Psi_2 = \Psi_{c,N} =$ 1.0 Modification for Uncracked Concrete (ACI D.5.2.6)

$\Psi_3 = \Psi_{cp,N} =$ Modification for Post-Installed Anchors (ACI D.5.2.7)

= 1.0 if $C_{a,min} >= C_{ac}$ for $C_{a,min} =$ 12.50 inches

$C_{ac} =$ 2.5 $H_{ef}$ and $H_{ef} = H_{HD} =$ 24.00 inches (Embedment depth of Holdown Anchor)

$C_{ac} =$ 60.0 inches

= Max ( $C_{a,min} / C_{ac}$ , 1.5 $H_{ef} / C_{ac}$ ) if $C_{a,min} < C_{ac}$

= Max (0.21, 0.60 )

$\Psi_3 = \Psi_{cp,N} =$ 0.60 Modification for Post-Installed Anchors (ACI D.5.2.7)

$\phi_{EQ}\ \phi_{sa,cb}\ N_{cb} =$ 10.56 Kips (Concrete Break-out Strength - Tension)

c) Pullout Strength of Anchor - Tension (ACI 318-08 Section D5.3)

Note: Condition B is assumed per Section D4.4, where Supplementary reinforcement in not present in failure prism.

$\phi_{EQ}\ \phi_{sa,po}\ N_p = \phi_{EQ}\ \phi_{sa,po}\ N_p\ \Psi_4$ (D-14)

Where $\phi_{EQ} =$ 0.75 (Steel Anchor - Seismic Region - ACI Section D.3.3.3)

$\phi_{sa,po} = \phi_{sa,cb} =$ 0.65 (Steel Anchor - Concrete Breakout/Pullout Category 2 - ACI D.4.4)
Cat 1 : Low Sensitivity to installation and High Reliability

$N_p =$ Pull-out Strength of a single anchor in Tension in Cracked Concrete (ACI D.5.2.2)

= 8 $A_{brg}$ $f'_c$ (D-15) and $A_{brg} = A_{se,N} =$ Bearing Area of Anchor Bolt

= 0.994 $in^2$ for $t_{HD} =$ 1.13 inches (Diameter of Holdown Anchor)

$f'_c =$ 3.25 Ksi
= 3,250 Psi

$N_p =$ 25.84 kips

$\Psi 4 = \Psi_{c,P} =$ Modification for uncracked Concrete (ACI D.5.2.7)

= 1.0 Note: Existing Concrete is assumed to crack at anchor bolt limit state or capacity of holdown.

$\Psi 4 = \Psi_{c,P} =$ 1.0 Modification for Uncracked Concrete (ACI D.5.2.6)

$\phi_{EQ}\ \phi_{sa,po}\ N_p =$ 12.60 Kips (Concrete Pull-out Strength - Tension)

d) Concrete Side-face Blowout Strength of Anchor - Tension (ACI 318-08 Section D5.4)

$\phi_{EQ}\ \phi_{sa,sf}\ N_p = \phi_{EQ}\ \phi_{sa,po}\ N_p$

Where $\phi_{EQ} =$ 0.75 (Steel Anchor - Seismic Region - ACI Section D.3.3.3)

$\phi_{sa,sf} = \phi_{sa,cb} =$ 0.65 (Steel Anchor - Concrete Breakout/Pullout/Side Blowout Category 2 - ACI D.4.4)
Cat 1 : Low Sensitivity to installation and High Reliability

$N_p = 160\ C_{a1}\ A_{brg}^{0.5}\ \lambda\ f'^{0.5}_c$ Where $C_{a1} =$ 12.50 inches (Distance from Wall CL to Left Edge of Existing Footing Wall)

$C_{a2} =$ 15.88 inches (Distance from Wall CL to Right Edge of Existing Footing Wall)

$C_{a,min} =$ 12.50 inches

and $A_{brg} = A_{se,N} =$ 0.994 $in^2$ for $t_{HD} =$ 1.00 inches (Diameter of Holdown Anchor)

$\lambda =$ 1.00 (1.0 for NWC, 0.75 for LWC)
Note: NWC - Normal Weight Concrete Assumed

$f'_c =$ 3.25 Ksi
= 3,250 Psi

$N_p =$ 113.68 Kips

$\phi_{EQ}\ \phi_{sa,sf}\ N_p =$ 55.4 Kips (Concrete Side-face Blowout Strength - Tension)

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE 3 (EVAL CASE E-2R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

e) Limiting Governing Strength of Anchor - Tension Only

Note: Holdown = HDU14

$F_{CU}$ = 20.13 Kips (Ultimate Capacity of Holdown)

$H_{HD}$ = 24.00 inches (Embedment depth of Holdown Anchor)

$t_{HD}$ = 1.13 inches

Limiting Strength of Anchor in Tension:

$T_{UC}$ = Min ($\phi_{EQ}\ \phi_{sa,t}\ N_{sa}$ , $\phi_{EQ}\ \phi_{sa,cb}\ N_{cb}$ , $\phi_{EQ}\ \phi_{sa,po}\ N_p$ , $\phi_{EQ}\ \phi_{sa,sf}\ N_p$ )

Where $\phi_{EQ}\ \phi_{sa,t}\ N_{sa}$ = 53.68 Kips (Bolt Design Strength)

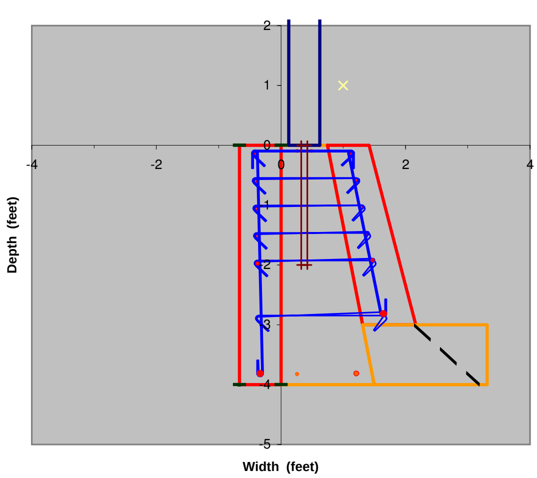
$\phi_{EQ}\ \phi_{sa,cb}\ N_{cb}$ = 10.56 Kips (Concrete Break-out Strength)

$\phi_{EQ}\ \phi_{sa,po}\ N_p$ = 12.60 Kips (Concrete Pull-out Strength)

$\phi_{EQ}\ \phi_{sa,sf}\ N_p$ = 55.4 Kips (Concrete Side-face Blowout Strength)

$T_{UC}$ = 10.56 Kips

**Foundation Cross-Section**




Required Number of Holdown Anchors:

$N_A$ = $F_{CU}$ / $T_{UC}$ Where $F_{CU}$ = 20.13 Kips (Ultimate Capacity of Holdown)

= 1.91 $T_{UC}$ = 10.56 Kips (Strength per Anchor)

$N_A$ = 2

**Use 2 - HDU14 with 1.13 Dia anchors @ 24" oc Min EA side**

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE 3 (EVAL CASE E-2R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

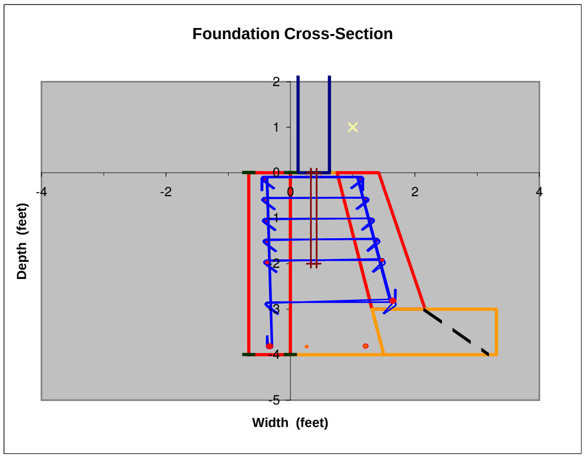
## 6. Adequacy of Footing - Shear

**A. Flexural/One-Way Shear on Full Cross-Section**

Note: Effective transverse width of footing for Shear is assumed as Top of Footing Width, $L_y = L'_T$

Shear demands: $V_{ux}$ = 23.3 Kips @ $x_L$ = 3.00 feet (locations at distance d from face of Wall - Left side)
= 19.7 Kips @ $x_R$ = 29.00 feet ( - Right side)

=> $V_{ux}$ = 23.3 Kips




ii) Shear capacity of concrete without shear reinforcement

$$\varphi V_c = \varphi\left(1.9\sqrt{f'_c} + 2500\,\rho_w \frac{V_u\,d}{M_u}\right) b_w\,d \;\le\; 3.5\sqrt{f'_c}\,b_w\,d \qquad (11\text{-}5)$$

Note: $V_u d/M_u$ value must be ≤ 1.0

Where $\phi$ = 0.75
$f'_c$ = 3,250 psi

$\rho_w = A_{sx}/(L_y\, d_x)$ and $A_{sx}$ = 0.44 in^2
$L_y$ = 2.08 feet
= 25.0 inches
d = 44.88 inches

$\rho_w$ = 0.000392

$V_u$ = 23 kips
d = 44.88 inches

$M_u$ = 36 Kip-ft @ $V_{ux}$ = 23 Kips (location of shear value)

Check of $V_u$ d/$M_u$ value limit:
$V_u$ d/$M_u$ = 2.44 where Vu = 23 kips
**NG, value taken as unity.**
d = 44.88 inches
$M_u$ = 36 kip-ft
= 429 kip -in

$b_w = L'_T$ = 2.1 feet
= 25.0 inches
d = 44.88 inches

=> $\phi$ $V_c$ = 92.0 kips

Comparison w/ Equation 11-3:
$\phi V_c = \phi\, 2\, f'^{0.5}_c\, b_w\, d$

Where $\phi$ = 0.75
$f'_c$ = 3,250 psi
$b_w = L'_T$ = 25.0 inches
d = 44.88 inches

$\phi$ $V_c$ = 95.9 kips

Check of upper value limit:
$\phi V_{c,max} = 3.5\, f'^{0.5}_c\, b_w\, d$

Where $f'_c$ = 3,250 psi
$b_w = L'_T$ = 25.0 inches
d = 44.88 inches

$\phi$ $V_{c,max}$ = 223.8 kips

$\phi$ $V_c$ = 92.0 kips
**OK, > Vu**

Note: D/C Ratio = 0.25 (Demand to Capacity Ratio - Shear)

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE 3 (EVAL CASE E-2R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**B. Shear Across Vertical Plane between Existing and New Concrete Retrofit Wals**

Note: Reinf Layout = 2 (Two-Sided Retrofit Walls)

a) Capacity of Single Dowel - Shear Friction (ACI 11.6.4)

$\phi V_n = \phi A_{vf} f_y \mu \lambda$ (11-25) Where $\phi$ = 0.75

$A_{vf}$ = 0.50 $in^2$ (Dowel Area) Note: Dowels are No. 4 bars

$f_y$ = 60.00 Ksi

$\mu$ = 0.60 ( RC - RC Surface not roughened)

$\lambda$ = 1.00 (1.0 for NWC, 0.75 for LWC) Note: NWC - Normal Weight Concrete Assumed

| $\phi V_n$ = 13.50 Kips |
|---|

Upper Limit Restrictions (ACI 11.6.5) :

$\phi V_{MAX} = \phi$ MIN ( 0.2 $f'_c$, 0.48 + 0.08 $f'_c$, 0.80 ) $A_c$ Where $\phi$ = 0.75

$f'_c$ = 3,250 Psi

= $\phi$ MIN (0.65, 0.74, 0.80 ) Ac = 3.25 Ksi

= 0.75 (0.65 ) Ac $A_c = \pi R_D^2$ for $R_D$ = Min ($L_e$, $S_D$)

$L_e$ = 6.00 inches (Depth of Embedment)

$S_D$ = 12.00 inches (Dowel Spacing)

| $R_D$ = 6.00 inches |
|---|

= 0.49 ( 113 ) $A_c$ = 113 $in^2$ ( Concrete Area)

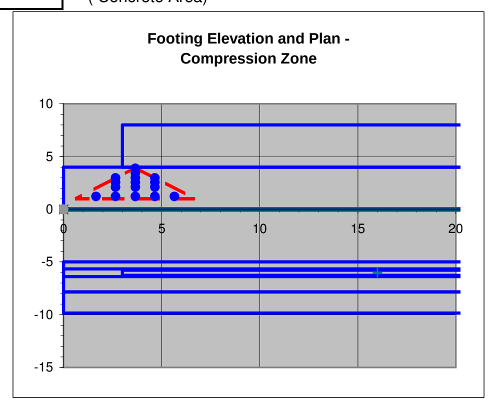
| $\phi V_{MAX}$ = 55 kips |
|---|

(Upper Limit on Dowel Strength)

OK

| $\phi V_n$ = 13.50 Kips |
|---|

(Shear Capacity of Single Dowel)




b) Demands on Dowels - **Compression** at Holdown Anchor

$V_{uc} = P_c = P_{EQ} + 1.2 P_{DL} /2$ Where $P_{EQ} = M_{OT} / L_W$ and $M_{OT}$ = 698.00 Kip-ft

$L_W$ = 26.00 feet

| $P_{EQ}$ = 37.58 Kips |
|---|

$P_{DL}$ = 30.94 Kips

| **$V_{uc}$ = 56.15 Kips** |
|---|

- Assume triangular Compression pressure distribution as shown.

Determination of Effective Dowels at Holdowns in Compression Zone :

$N_D$ = **16** (Number of Dowels)

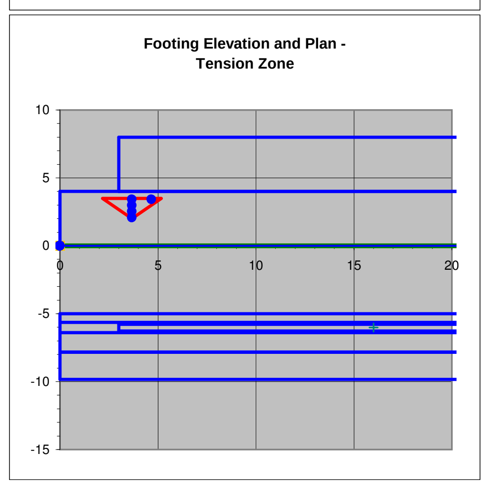
$N_{VP}$ = **2** (Number of Shear Planes - Double Shear)

$\phi V_n$ = 13.50 Kips (Shear Capacity of Single Dowel)

| $\phi V_n N_D N_V$ = 432 Kips |
|---|

(Shear Capacity of Compression Zone)

OK




c) Demands on Dowels - **Tension** at Holdown Anchor

$V_{ut} = P_t$ = MAX( $P_{EQ}$ - 0.6 $P_{DL}$ /2 , $F_{UC}$ ) Where $P_{EQ}$ = 37.58 Kips

= Max ( 28.30 , 14.38 ) $P_{DL}$ = 30.94 Kips

$F_{CU}$ = 14.38 Kips (Ultimate Capacity of Holdown)

| **$V_{ut}$ = 28.30 Kips** |
|---|

- Assume Tension triangular pressure distribution as shown.

Determination of Effective Dowels at Holdowns in Tension Zone :

$N_D$ = **5** (Number of Dowels)

$N_{VP}$ = **2** (Number of Shear Planes - Double Shear)

$\phi V_n$ = 13.50 Kips (Shear Capacity of Single Dowel)

| $\phi V_n N_D N_V$ = 135.0 Kips |
|---|

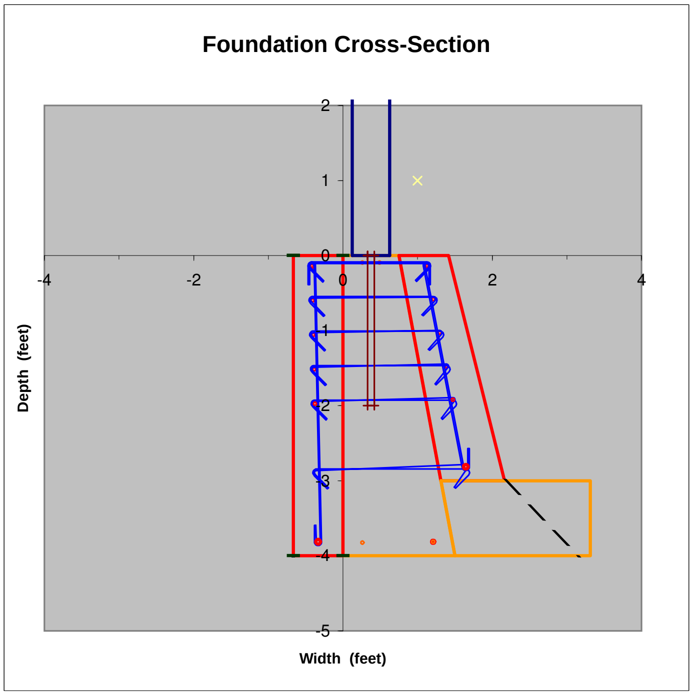
(Shear Capacity of Tension Zone) Note: D/C Ratio = 0.21 (Demand to Capacity Ratio - S

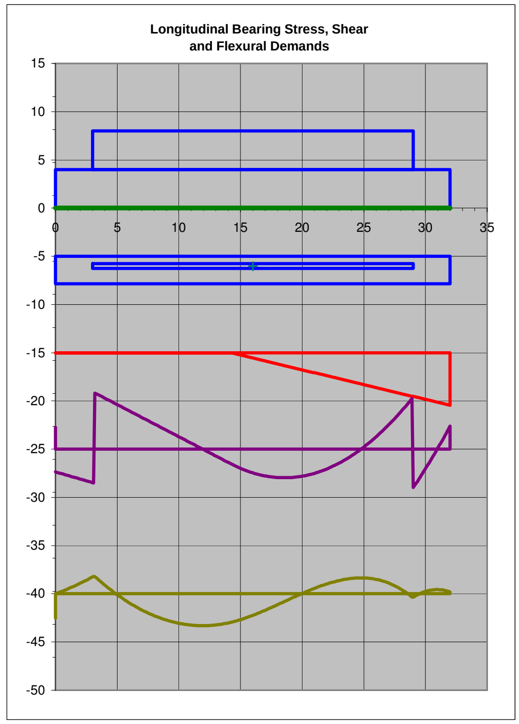
OK

| **Footing OK for Shear** |
|---|

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE 3 (EVAL CASE E-2R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 7. Adequacy of Footing - Flexure




a) Flexural demands $M_{ux}$ = 36 Kip-ft @ $x_L$ = 3.00 feet (locations at face of column - Left side)
= 429 Kip-in — Note: $X_f$ = 3.00 feet (Cantilever Length)

= 46 Kip-ft @ Wall and Footing Centerline
= 549 Kip-in

= 5 Kip-ft @ $x_R$ = 29.00 feet ( - Right side)
= 65 Kip-in — Note: $X_f$ = 3.00 feet (Cantilever Length)

= > $M_{ux}$ = 549 Kip-in

b) Required Reinforcement Ratio (ACI 10.2)

$$\rho_r = \frac{0.85\ f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2\ M_u}{0.765\ b\ d^2\ f'_c}}\right]$$

Where $f'_c$ = 3.25 Ksi
$f_y$ = 40.00 Ksi
$M_u$ = 549 kip-in
$L_y = L_B$ = 3.8 feet
= 46 inches
$d_x$ = 44.88 inches

$\rho_r$ = 0.000165

c) Reinforcement Ratio Provided

$\rho_w = A_{sx}/(L_y\ d_x)$ — Where $A_{sx} = A_N + A_E$

$A_N$ = 0.44 in$^2$ (New Flexural Reinforcement)
$A_E$ = 0.40 in$^2$ (Existing Flexural Reinforcement)

$A_{sx}$ = 0.84 in$^2$

$L_y = L_B$ = 3.8 feet
= 46.0 inches
d = 44.88 inches

$\rho_w$ = 0.000407 (reinforcement ratio provided) — Note: D/C Ratio = 0.41 (Demand to Capacity Ratio - Flexure)

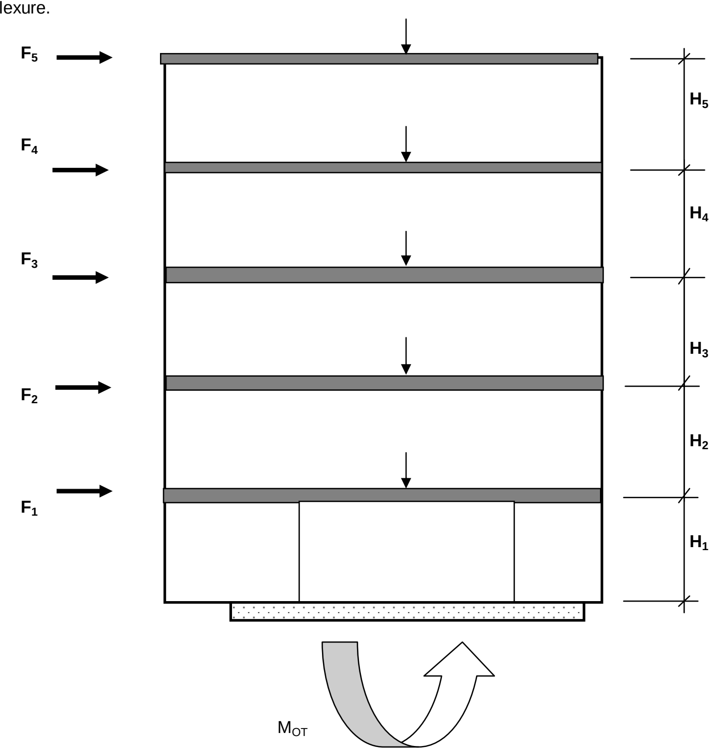
**OK**

**1 - # 6 Bars OK for Longitudinal Flexure with DC Ratio = 0.41**

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE F (EVAL CASE E-3R)**
**DETERMINATION OF VERTICAL AND LATERAL LOADS TO FOUNDATION**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Assumptions**

1. Existing walls are treated as pinned Columns. Footing is assumed rigid.
2. Column loads are located in transverse center of footing; limit of 2 columns w/o flexure.
3. Footing has no shear reinforcement.
4. Concrete is Normal Weight Concrete with uncoated bars.




**1. Lateral Loads and Load Effects**

V = 21.42 kips (Base Shear - A4 **ASD**)

| Floor Level | Height (feet) | Loading ID | $V_x$ / V | Shear (Kips) | Force (Kips) | Moment (Kip-ft) |
|---|---|---|---|---|---|---|
| | | 5 | | | | |
| R | 11.00 | 4 | 0.30 | 6.43 | 6.43 | 289.2 |
| 3 | 10.00 | 3 | 0.65 | 13.92 | 7.50 | 544.1 |
| 2 | 13.50 | 2 | 0.89 | 19.06 | 5.14 | 667.4 |
| 1 | 10.50 | 1 | 1.00 | 21.42 | 2.36 | 692.2 |

**$M_{OT}$ = 692.19 Kip-ft**

**2. Vertical Loads and Load Effects**

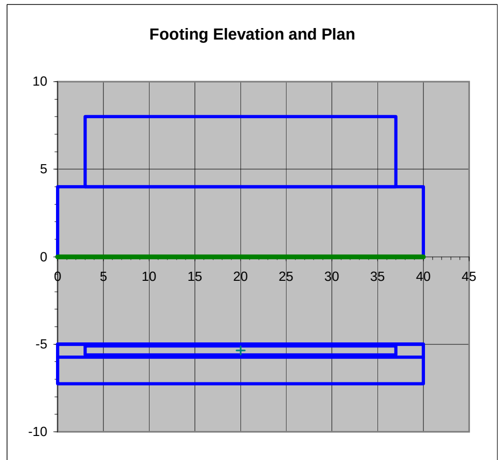
| | | Floor Tributary Loads | | | | | Wall Tributary Loads | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Wall** | Floor Level | DL (psf) | Length (feet) | Width (feet) | Area ($ft^2$) | Weight (kips) | WL (psf) | Length (feet) | Height (feet) | Area ($ft^2$) | Weight (kips) |
| | | | | | | | | | | | |
| **1** | R | 20 | 34.00 | 8.00 | 272 | 5.44 | 14 | 34.00 | 11.00 | 374 | 5.24 |
| | 3 | 30 | 34.00 | 8.00 | 272 | 8.16 | 14 | 34.00 | 10.00 | 340 | 4.76 |
| | 2 | 30 | 34.00 | 8.00 | 272 | 8.16 | 14 | 34.00 | 13.50 | 459 | 6.43 |
| | 1 | 30 | 34.00 | 8.00 | 272 | 8.16 | 14 | 34.00 | 10.50 | 357 | 5.00 |

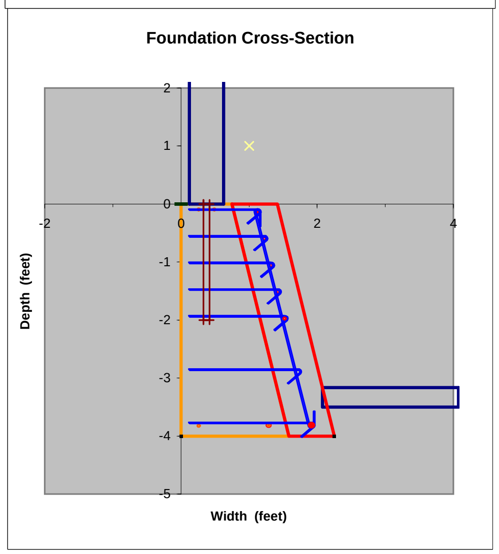
Sum of Floor Weight = 29.92 Kips

Sum of Wall Weight = 21.42 Kips

**$P_{D1}$ = 51.34 Kips**

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE F (EVAL CASE E-3R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**




**Assumptic** 1. Footing has no shear reinforcement.
2. Concrete is Normal Weight Concrete with uncoated bars.

## Footing Parameters :

Footing Size :
$L_x$ = 40.0 feet   $d_s$ = 0.0 feet (depth of soil)
$L_y$ = 2.3 feet
$h_f$ = 4.0 feet

Wall Location :
$x_c$ = 20.0 feet (Wall centerline distance from Left Edge)
$y_c$ = 1.9 feet (Wall centerline distance from Bottom Edge)

Wall Size :
$C_x$ = 34.0 feet (Wall length)
$C_y$ = 0.5 feet (Wall width)

Interconnected Slab at Sides:

Note : Slabs at sides are used only to reduce soil bearing pressure; footing is designed to take all loads.

| Side : | Left | Right | |
|---|---|---|---|
| t | | 4.00 | Inches (Slab Thickness) |
| X | | 20.00 | Feet (distance to other Slab Edge Support |
| $f'_c$ | | 1.00 | Ksi |
| Conn Type | | D | (D= Dowel, C= Continuous) |

Concrete :
$f'_c$ = 3.25 Ksi
$f_y$ = 40.00 Ksi
$\rho_c$ = 0.150 kip/ft^3

Reinforcement:
$d_c$ = 1.00 inches (bar clearance - top)
$d_c$ = 2.00 inches (bar clearance - bottom)
= 3.00 inches (bar clearance - sides)

| | Orientation | Bar Size | N Bars | Bottom Layer | d (inches) | Bar Diameter (inches) | Bar Area Per Bar (in^2) | Bar Area Total (in^2) |
|---|---|---|---|---|---|---|---|---|
| Top Mat | x | 6 | 1 | x | 44.25 | 0.75 | 0.44 | 0.44 |
| Bottom Mat | x | 6 | 1 | | 44.88 | 0.75 | 0.44 | 0.44 |

$L_B$ = 2.25 feet (Bearing Length)

Note: Reinf Layout = 1
(One-Sided Retrofit W

## 1. Design of Slab-to-Footing Connections

**Use No. 4 bars @ 8.00 inches on-center for Slab-to-Footing Connections**

## 2. Lateral Resistance of Foundation

**Foundation OK for Sliding**

## 3. Soil Pressure due to Applied Loads

**Footing Bearing stress OK**

## 5. Adequacy of Footing - Anchor Pull-out in Existing Footing

**Use 3 - HDU14 with 1.13 Dia anchors @ 24" oc Min EA side**

## 5. Adequacy of Footing - Shear

**Footing OK for Shear**

## 6. Adequacy of Footing - Flexure

**1 - # 6 Bars OK for Longitudinal Flexure with DC Ratio = 0.30**

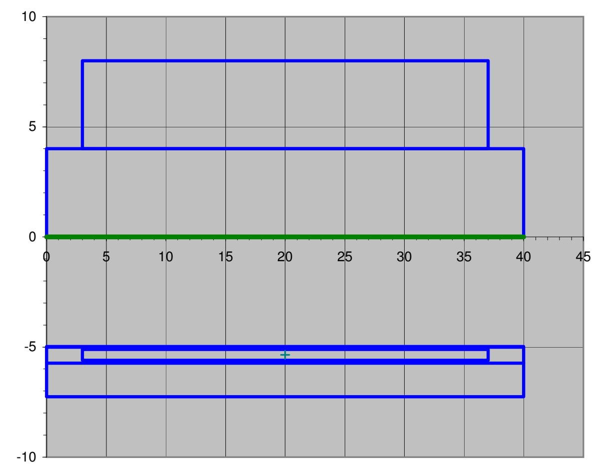
**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE F (EVAL CASE E-3R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Assumptions :**

1. Existing Footing has no shear reinforcement.
2. Concrete is Normal Weight Concrete with uncoated bars.
3. Wall is centered on Footing; Longitudinal Heel and Toe are equal in length.

## Footing Parameters :

**Footing Elevation and Plan**




Wall Size :

$C_x$ = 34.0 feet (Wall length)
$C_y$ = 0.5 feet (Wall width)

Wall Location :

$x_c$ = 20.0 feet (Wall centerline distance from Left Edge)
$y_c$ = 1.9 feet (Wall centerline distance from Bottom Edge)

Holdown Anchor :

Holdown = **HDU14**
$t_{HD}$ = 1.13 inches (Diameter of Holdown Anchor)

Note: $t'_{HD}$ = 1.00 inches (Diameter of Holdown Anchor Required for Holdown)

$F_{CA}$ = 14.38 Kips (Allowable Capacity of Holdown)
$F_{CU}$ = 20.13 Kips (Ultimate Capacity of Holdown)

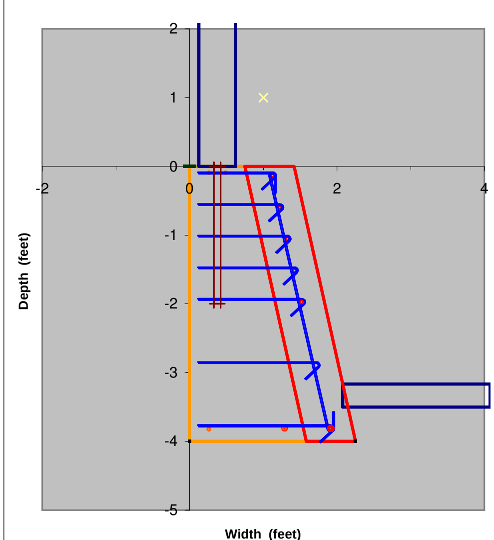
$H_{HD}$ = 24.00 inches (Embedment depth of Holdown Anchor)
OK

Footing Dimensions :

$L_x$ = 40.00 feet (Footing Length)
$h_f$ = 4.00 feet (Footing Height)

| | Left | Right | |
|---|---|---|---|
| $L_H$ = | | | feet (Length of Heel, if any) |
| $h_H$ = | | | feet (Thickness of Heel, if any) |

**Foundation Cross-Section**




(E) Footing Width:

$W_T$ = 9.00 inches (Footing Width - at Top of Wall)
= 0.00 inches (Increase in Width - at Left)
= 10.00 inches (Increase in Width - at Right)
$W_B$ = 1.58 feet (Footing Width - at Bottom of Wall)

(N) Retrofit Walls: Note: Reinf Layout = 1 (One-Sided Retrofit Walls)

Concrete Weight : NWC LWC - Lean Weight Concrete, NWC otherwise)

| | Left | Right | |
|---|---|---|---|
| $t_T$ = | | 8.00 | inches (Retrofit Wall Thickness - Top of Wall) |
| $t_B$ = | | 8.00 | inches (Retrofit Wall Thickness - Bottom of Wall) |

$W'_T$ = 1.42 feet (Footing Width - at Top of Wall)
$W'_B$ = 2.25 feet (Footing Width - at Bottom of Wall)

$t_H$ = inches (Retrofit Wall Thickness of Heel, if any)

Dowel Data : No. 4 Dowels
$S_D$ = 12.00 inches (Dowel Spacing - Horiz and Vertical)

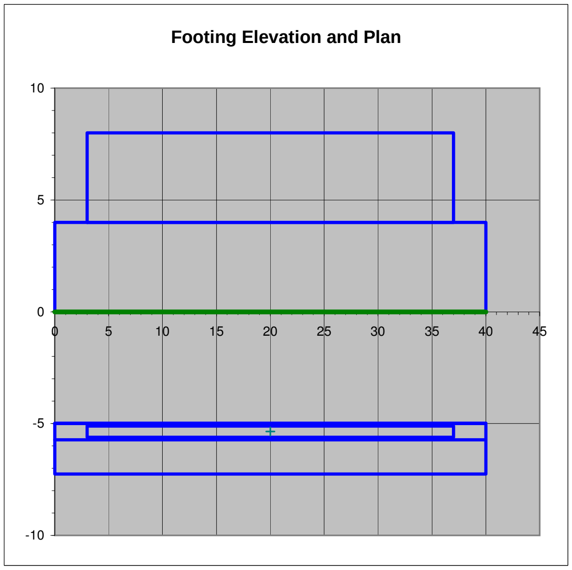
$L_B$ = 2.25 feet (Bearing Length)

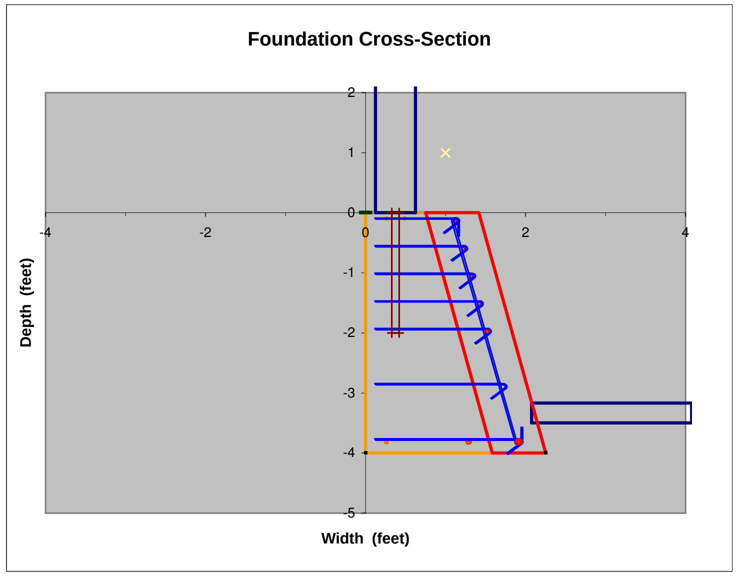
Footing Loads : 16.5 21.42 1.30

| | Service | Strength | |
|---|---|---|---|
| P = | 51.3 | 71.9 | kips |
| $M_y$ = | 692.2 | 969.1 | kip-ft |
| $V_x$ = | 21.42 | 30.0 | kips |
| $M_x$ = | 0 | 0.3 | kip-ft |
| $V_y$ = | 0.0 | 0.0 | kips |

Plastic Hinge Centroidal Heights:
@ $h_{px}$ = 0.00 feet
@ $h_{py}$ = 0.00 feet

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE F (EVAL CASE E-3R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**




Capacity Factors :

Concrete :

$\phi_v$ = 0.75 (Shear)
$\phi_b$ = 0.65 (Bearing)

Steel Anchor in Concrete:

$\phi_{EQ}$ = 0.75 (Steel Anchor - Seismic Region - ACI Section D.3.3.3)
$\phi_{sa,t}$ = 0.75 (Steel Anchor - Tension, Ductile Steel Element - ACI D.4.4)
$\phi_{sa,cb}$ = 0.65 (Steel Anchor - Concrete Breakout Category 2 - ACI D.4.4)
Note: Cat 1 : Low Sensitivity to installation and High Reliability

Material Properties :

Concrete :

$f'_{ce}$ = 3.25 Ksi (Existing Concrete)
$f'_{cn}$ = 2.50 Ksi (New Concrete)
$f_y$ = 40 Ksi (Existing Reinforcement)
$\rho_c$ = 0.150 kip/ft^3

Note : Year Built : 1921

Steel Anchor in Concrete:

$f_{ya}$ = 60.00 Ksi (PCA Notes Table 34-1 - ASTM A307)

Interconnected Slab at Sides:

Note : Slabs at sides are used only to reduce soil bearing pressure; footing is designed to take all loads.

| Side : | Left | Right | |
|---|---|---|---|
| t | | 4.00 | Inches (Slab Thickness) |
| h | | 38.00 | Inches (Distance to Top of Wall) |
| X | | 20.00 | Feet (distance to other Slab Edge Support |
| $f'_c$ | | 1.00 | Ksi |
| Conn Type | | D | (D= Dowel, C= Continuous) |

Reinforcement:

$d_c$ = 1.00 inches (bar clearance - top)
$d_c$ = 2.00 inches (bar clearance - bottom)
= 3.00 inches (bar clearance - sides)

Existing Reinforcement:

| | | | |
|---|---|---|---|
| Top | 2 | No. | 4 |
| Bottom | 2 | No. | 4 |

| New Flexural Reinforcement EA Side | | | | | | | Bar Area | |
|---|---|---|---|---|---|---|---|---|
| Location | Bar Size | N Bars (Max 3) | Bottom Layer | d (inches) | b (inches) | Bar Diameter (inches) | Per Bar (in^2) | Total (in^2) |
| Top | 6 | 1 | x | 44.25 | 17.00 | 0.75 | 0.44 | 0.44 |
| Bottom | 6 | 1 | | 44.88 | 27.00 | 0.75 | 0.44 | 0.44 |

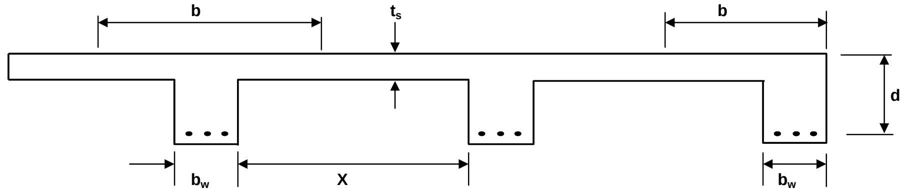
Soil Parameters :

$\sigma_{allow}$ = 2.00 ksf (Allowable Bearing Pressure)
$\sigma_p$ = 0.30 ksf/ft (Passive Soil Pressure)
$\mu$ = 0.25 ksf (Coefficient of Friction)

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE F (EVAL CASE E-3R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 1. Design of Slab-to-Footing Connections

a) Effective Slab Width (ACI ACI 21.6.2.2)




i) Symmetrical T-Beams (ACI 8.12.2)

$b \leq$ Min ($L/4 + b_w$, $16\ t_s + b_w$, $X + b_w$)

Where L = feet (Footing span length)
= inches

$t_s$ = inches (Slab thickness - Average value)

$b_w = L'_B$ = feet (Footing Width)
= inches

X = feet (Slab Supported Length - Average Value)
= inches

=

| b = | inches |
|---|---|

ii) Slabs on One Side (ACI 8.12.3)

$b \leq$ Min ($b_w + L/12$, $b_w + 6\ t_s$, $X/2 + b_w$)

Where $b_w = L'_B$ = 2.25 feet (Footing Width)
= 27.00 inches

L = 40.00 feet (Footing span length)
= 480.00 inches

$t_s$ = 4.00 inches (Slab thickness)

X = 20.00 feet (Slab Supported Length)
= 240 inches

= Min (67.00, 51.00, 147.00)

| b = | 51.00 | inches |
|---|---|---|

| **b =** | **51.00** | **inches** |
|---|---|---|
| **=** | **4.25** | **feet** |

b) Required Slab Reinforcement Area

$A_{sr} = 0.0018\ b\ t_s$ (ACI 7.12.2.1)

Where b = 51.00 inches (Effective Slab Width)

$t_s$ = 4.0 inches (Slab thickness)

| $A_{sr}$ = | 0.37 | in$^2$ |
|---|---|---|

c) Required Slab Reinforcement Spacing

$S_{sr} = b / n_b \leq 2\ t_s$ (ACI 13.3.2)

Where b = 51.00 inches (Effective Slab Width)

$n_b$ = Ceiling ($A_{sr}/A_b$) for $A_{sr}$ = 0.37 in$^2$ (Slab Reinforcement Area - Required)

$A_b$ = 0.20 in$^2$ for No. 4 bars

= CEILING(MIN(25.50,8.00,1)

| $n_b$ = | 2 | bars |
|---|---|---|

$t_s$ = 4.0 inches
$2\ t_s$ = 8.0 inches

| $S_{sr}$ = | 8.00 | inches |
|---|---|---|

**Use No. 4 bars @ 8.00 inches on-center for Slab-to-Footing Connections**

201812.30

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE F (EVAL CASE E-3R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 2. Lateral Resistance of Foundation

### 2A. Longitudinal Loading

$F_{Rx} = 0.5\ L'_y\ h'^2_f\ \sigma_p + 0.6\ (W_f + P)\ \mu$

Where $L'_y = W'_B$ — $W'_B =$ 2.25 feet (Footing Width - at Bottom of Wall)

$\boxed{L'_y = 2.25 \text{ feet}}$ (Bearing Width at Ends of Footing)

$h'_f = h_f + h_{sk}$ and $h_f =$ 4.00 feet

$h_{sk} =$ 0.00 feet (Additional height of Shear Key at Footing E

$\boxed{h'_f = 4.0 \text{ feet}}$ (Bearing Height at Ends of Footing)

= (18.0) (0.30) + 0.6 (105.3) (0.25) — $\sigma_p =$ 0.30 ksf/ft (Passive Soil Pressure)

= (5.40) + (16.05) — $W_f = \rho_c\ L_x\ L_y\ h_f$ and $\rho_c =$ 0.150 kip/ft^3

$L_x =$ 40.0 feet 0

$L_y =$ 2.3 feet

$h_f =$ 4.0 feet

$\boxed{W_f = 54.00 \text{ Kips}}$ (Footing Weight)

P = 51.3 Kips (Service Load)

$\mu =$ 0.25 ksf (Coefficient of Friction)

$\boxed{F_{Rx} = 21.45 \text{ kips}}$ — Note : $V_x =$ 21.42 kips

**OK**

**Foundation OK for Sliding**

## 3. Soil Pressure due to Applied Loads

### 3A. Longitudinal Loading

a) Loading Eccentricity — Note: Effective transverse width of footing for bearing is assumed as Bottom of Footing Width, $L_y = L'_B$

$e_x = \Sigma M_y / P'$ — Where $\Sigma M_y = M_y + V_x H_{px} - P\ (0.5\ L_x - x_c)$ and $M_y =$ 692 kip-ft

$V_x =$ 21 kips

@ $h_{px} =$ 0.00 feet

P = 51 Kips

$L_x =$ 40.0 feet

= 692 kip-ft + 0 kip-ft - 0 kip-ft — $x_c =$ 20.0 feet (Column centerline distance from Left Edge)

$\boxed{\Sigma M_y = 692 \text{ Kip-in}}$

$P' = P + P_F$ and P = 51 Kips

$P_F = \rho_c\ L_x\ L_y\ h_f$ for $\rho_c =$ 0.150 kip/ft^3

$L_x =$ 40.0 feet

$L_y = 0.5\ (L'_T + L'_B) =$ 1.8 feet

$h_f =$ 4.0 feet

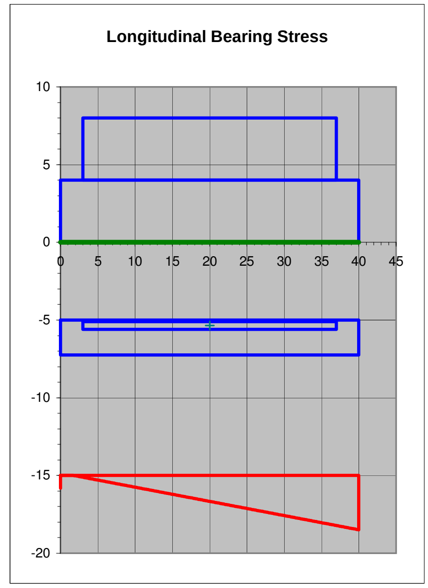
$P_F =$ 44.00 kips (footing weight)

P' = 95 Kips

$\boxed{e_x = 7.26 \text{ feet}}$

Note: $L_x/6 =$ 6.67 feet (Footing Middle Third)

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE F (EVAL CASE E-3R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**




b) Bearing Stresses

i) for $e_x \le L_x/6$ (within Middle $L_x/3$)

$\sigma_{max}$ = P (1 + 6 $e_x$ / $L_x$) / ($L_x$ $L_y$) Where P = 95 Kips

$\sigma_{min}$ = P (1 - 6 $e_x$ / $L_x$) / ($L_x$ $L_y$) $e_x$ = 7.26 feet

$L_x$ = 40.0 feet

$L_y = L_B$ = 2.3 feet

| | | |
|---|---|---|
| $\sigma_{max}$ = | 2.21 | Ksf |
| $\sigma_{min}$ = | -0.09 | Ksf |

ii) for $e_x > L_x/6$ (outside Middle $L_x/3$) **<= Governs!**

$\sigma_{max}$ = 2 P / ($L_{bx}$ $L_y$) Where P = 95 Kips

$L_{bx}$ = 3 (0.5 $L_x$ - $e_x$) and $L_x$ = 40.0 feet

$e_x$ = 7.26 feet

| | | |
|---|---|---|
| $L_{bx}$ = | 38.22 | feet |

$L_y = L_B$ = 2.3 feet

| | | |
|---|---|---|
| $\sigma_{max}$ = | 2.22 | Ksf |
| $\sigma_{min}$ = | 0.00 | Ksf |

iii) Governing Condition

| | | |
|---|---|---|
| $\sigma_{max}$ = | 2.22 | Ksf |
| $\sigma_{min}$ = | 0.00 | Ksf |

3B. **Transverse Loading**

a) Loading Eccentricity

Note: Effective transverse width of footing for bearing is assumed as Bottom of Footing Width, $L_y = L'_B$

$e_y = \Sigma M_x / P'$ Where $\Sigma M_x = M_x + V_y H_{py} - P (0.5 L_y - y_c)$ and $M_x$ = 0 kip-ft

$V_y$ = 0 kips

@ $h_{py}$ = 0.00 feet

P = 51 Kips

$L_y = L_B$ = 2.3 feet

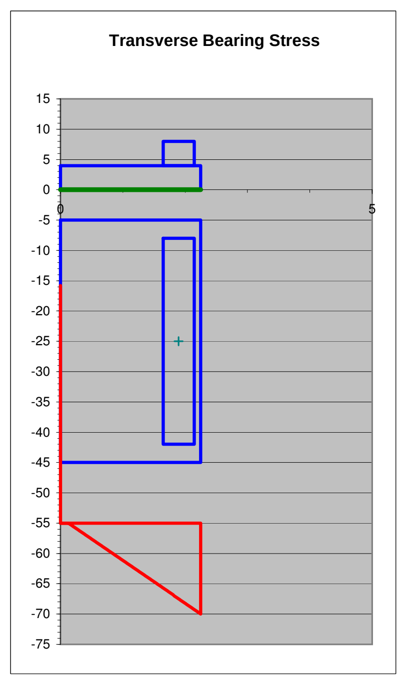
= 0 kip-ft + 0 kip-ft - -40 kip-ft $y_c$ = 1.9 feet (Column centerline distance from Left Edge)

| | | |
|---|---|---|
| $\Sigma M_y$ = | 40 | Kip-in |

P' = 95 Kips

| | | |
|---|---|---|
| $e_y$ = | 0.42 | feet |

Note: $L_y/6$ = 0.38 feet (Footing Middle Third)




b) Bearing Stresses

i) for $e_y \le L_y/6$ (within Middle $L_y/3$)

$\sigma_{max}$ = P' (1 + 6 $e_y$ / $L_y$) / ($L_x$ $L_y$) Where P' = 95 Kips

$\sigma_{min}$ = P' (1 - 6 $e_y$ / $L_y$) / ($L_x$ $L_y$) $e_y$ = 0.42 feet

$L_y = L_B$ = 2.3 feet

$L_x$ = 40.0 feet

| | | |
|---|---|---|
| $\sigma_{max}$ = | 2.24 | Ksf |
| $\sigma_{min}$ = | -0.12 | Ksf |

ii) for $e_x > L_x/6$ (outside Middle $L_x/3$) **<= Governs!**

$\sigma_{max}$ = 2 P' / ($L_{by}$ $L_x$) Where P' = 95 Kips

$L_{by}$ = 3 (0.5 $L_y$ - $e_y$) $\le$ $L_y$ and $L_y = L_B$ = 2.3 feet

$e_y$ = 0.42 feet

| | | |
|---|---|---|
| $L_{by}$ = | 2.12 | feet |

$L_x$ = 40.0 feet

| | | |
|---|---|---|
| $\sigma_{max}$ = | 2.25 | Ksf |
| $\sigma_{min}$ = | 0.00 | Ksf |

iii) Governing Condition

| | | |
|---|---|---|
| $\sigma_{max}$ = | 2.25 | Ksf |
| $\sigma_{min}$ = | 0.00 | Ksf |

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE F (EVAL CASE E-3R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**3C. Check of Bearing Stresses on Soil**

a) Assuming all loads resisted by Footing only

$\sigma_{max}$ = Max($\sigma_{max,x}$,$\sigma_{max,y}$)      Where $\sigma_{max}$ = 2.22 Ksf (Longitudinal Direction)
= 2.25 Ksf (Transverse Direction)

b) Assuming all loads resisted by Footing + Adjoining Slabs      Note: Effective transverse width of footing for bearing is assumed as Bottom of Footing Width, $L_y = L'_B$

$\sigma_{max}$ = Max($\sigma_{max,x}$,$\sigma_{max,y}$) * $L_y$ / b      Where $\sigma_{max}$ = 2.22 Ksf (Longitudinal Direction)      and $L_y = L_B$ = 2.3 feet
= 2.25 Ksf (Transverse Direction)      b = 51.00 inches
= ( 2.25 * 0.53 )      = 4.25 feet

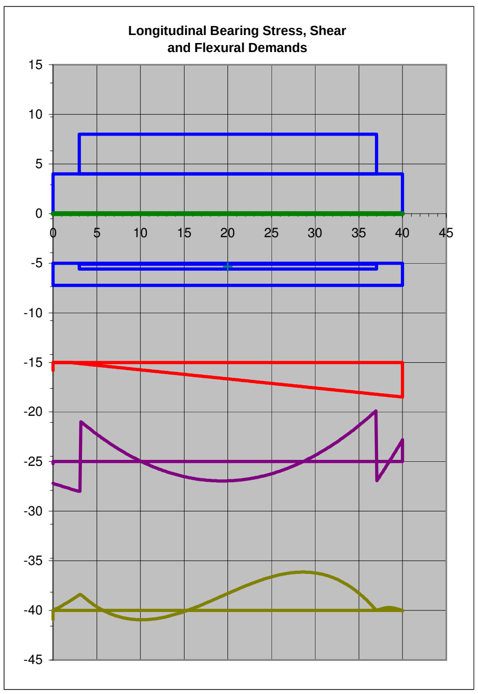
| $\sigma_{max}$ = 1.19 Ksf |
|---|

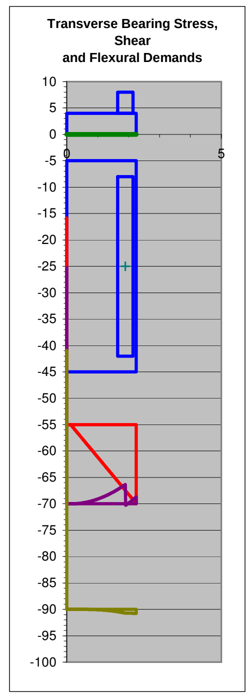
OK

Note: $\sigma_{allow}$ = 2.00 ksf (allowable bearing pressure)

| Footing Bearing stress OK |
|---|

## 4. Applied Loading and Demands on Footing



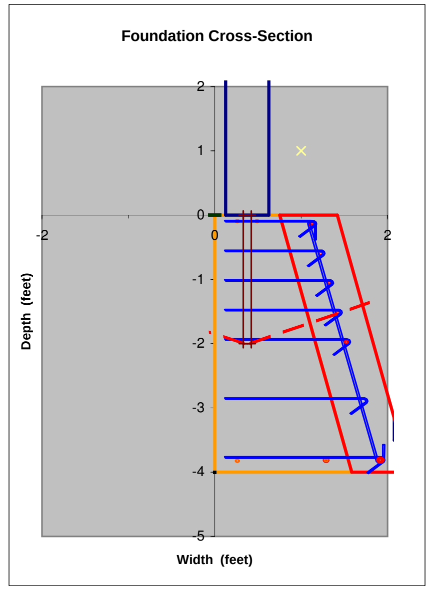




| | Longitudinal Direction | | | | |
|---|---|---|---|---|---|
| | Left End | Left Face of Wall | Wall Centerline | Right Face of Wall | Right End |
| Location (feet) | 0 | 3.00 | 20.00 | 37.00 | 40.00 |
| $\sigma = q_u/L_y$ (ksf) | 0.00 | 0.09 | 1.48 | 2.86 | 3.10 |
| P (kips) | - | - | 72 | - | - |
| $M_y + V h_{px}$ (kip-ft) | - | - | 969 | - | - |
| V+ (kips) | - | 16 | - | 20 | 9 |
| V- (kips) | -9 | -12 | -8 | -7 | - |
| M+ (kip-ft) | 0 | 32 | 34 | 1 | - |
| M- (kip-ft) | - | - | - | - | -1 |

| | Transverse Direction | | | | |
|---|---|---|---|---|---|
| | Left End | Left Face of Column | Column Centerline | Right Face of Column | Right End |
| Location (feet) | 0.00 | 1.65 | 1.90 | 2.15 | 2.25 |
| $\sigma = q_u/L_y$ (ksf) | 0.00 | 1.60 | 1.86 | 2.13 | 2.25 |
| P (kips) | - | - | 72 | - | - |
| $M_y + V h_{px}$ (kip-ft) | - | - | 0 | - | - |
| V+ (kips) | 0 | 49 | 66 | 14 | 23 |
| V- (kips) | - | - | - | - | - |
| M+ (kip-ft) | 0 | - | - | - | - |
| M- (kip-ft) | - | -25 | -39 | -41 | -43 |

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE F (EVAL CASE E-3R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 5. Adequacy of Footing - Anchor Pull-out in Existing Footing




Holdown = HDU14

$F_{CU}$ = 20.13 Kips (Ultimate Capacity of Holdown)

$H_{HD}$ = 24.00 inches (Embedment depth of Holdown Anchor)

a) Bolt Design Strength - Tension (ACI 318-08 D3.3.3, D4.4, D5.1)

$\phi_{EQ}\ \phi_{sa,t}\ N_{sa} = \phi_{EQ}\ \phi_{sa,t}\ n\ A_{se,N}\ f_{uta}$ (D-3)

Where $\phi_{EQ}$ = 0.75 (Steel Anchor - Seismic Region - ACI Section D.3.3.3)

$\phi_{sa,t}$ = 0.75 (Steel Anchor - Tension, Ductile Steel Element - ACI Section D.4.4)

n = 1 (number of anchors)

$A_{se,N} = \pi\ t_{HD}^2$ for $t_{HD}$ = 1.13 inches (Diameter of Holdown Anchor)

$A_{se,N}$ = 0.994 in$^2$

$f_{uta}$ = Min (1.6 $f_{ya}$, 125) for $f_{ya}$ = 60.00 Ksi (PCA Notes Table 34-1 - ASTM A307)

$f_{uta}$ = 96.00 Ksi

$\phi_{EQ}\ \phi_{sa,t}\ N_{sa}$ = 53.68 Kips (Bolt Design Strength - Tension)

b) Concrete Breakout Strength - Tension (ACI 318-08 Section D5.2)

Note: Condition B is assumed per Section D4.4, where Supplementary reinforcement in not present in failure prism.

$\phi_{EQ}\ \phi_{sa,cb}\ N_{cb} = \phi_{EQ}\ \phi_{sa,cb}\ N_b\ A_{NC} / A_{NCO}\ \ \Psi_1\ \Psi_2\ \Psi_3$ (D-4)

Where $\phi_{EQ}$ = 0.75 (Steel Anchor - Seismic Region - ACI Section D.3.3.3)

$\phi_{sa,cb}$ = 0.65 (Steel Anchor - Concrete Breakout Category 2 - ACI D.4.4)

Note: Cat 1 : Low Sensitivity to installation and High Reliability

$N_b$ = Basic concrete Break-out Strength of a single anchor in Tension in Cracked Concrete (ACI D.5.2.2)

$= K_C\ f'^{0.5}_c\ H_{ef}^{1.5}\ \lambda$ and $K_C$ = 17 Post-installed Anchors (D5.2.2)

$f'_c$ = 3.25 Ksi
= 3,250 Psi

$H_{ef} = H_{HD}$ = 24.00 inches (Embedment depth of Holdown Anchor)

$\lambda$ = 1.00 (1.0 for NWC, 0.75 for LWC)

Note: NWC - Normal Weight Concrete Assumed

$N_b$ = 113.95 Kips

$A_{NC}$ = Projected Concrete Failure Area - Actual (ACI D5.2.1)

= 2 (1.5 $H_{ef}$ ) ($C_{a1}$ + $C_{a2}$) and $H_{ef} = H_{HD}$ = 24.00 inches (Embedment depth of Holdown Anchor)

$C_{a1}$ = 4.50 inches (Distance from Wall CL to Left Edge of Existing Footing Wall)

$C_{a2}$ = 16.25 inches (Distance from Wall CL to Right Edge of Existing Footing Wall)

$A_{NC}$ = 1,494 in$^2$

$A_{NCO}$ = Projected Concrete Failure Area - Ideal (ACI D5.2.1)

= 9 $h_{ef}^2$ (D-6) and $h_{ef} = H_{HD}$ = 24.00 inches (Embedment depth of Holdown Anchor)

$A_{NCO}$ = 5,184 in$^2$

$\Psi_1 = \Psi_{ed,N}$ = Modification for Edge Effects (ACI D.5.2.5)

= 1.0 if $C_{a,min}$ >= 1.5 $H_{ef}$ Where $C_{a1}$ = 4.50 inches (Distance from Wall CL to Left Edge of Existing Footing Wall)

= 0.7 + 0.3 $C_{a,min}$ / (1.5 $H_{ef}$) if $C_{a,min}$ < 1.5 $H_{ef}$ $C_{a2}$ = 16.25 inches (Distance from Wall CL to Right Edge of Existing Footing Wall)

$C_{a,min}$ = 4.50 inches

$H_{ef} = H_{HD}$ = 24.00 inches (Embedment depth of Holdown Anchor)

$\Psi_1 = \Psi_{ed,N}$ = 0.74 Modification for Edge Effects (ACI D.5.2.5)

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE F (EVAL CASE E-3R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

$\Psi_2 = \Psi_{c,N} =$ Modification for Uncracked Concrete (ACI D.5.2.6)

= 1.0 Note: Existing Concrete is assume to crack at anchor bolt limit state or capacity of holdown.

$\Psi_2 = \Psi_{c,N} =$ 1.0 Modification for Uncracked Concrete (ACI D.5.2.6)

$\Psi_3 = \Psi_{cp,N} =$ Modification for Post-Installed Anchors (ACI D.5.2.7)

= 1.0 if $C_{a,min} >= C_{ac}$ for $C_{a,min} =$ 4.50 inches

$C_{ac} =$ 2.5 $H_{ef}$ and $H_{ef} = H_{HD} =$ 24.00 inches (Embedment depth of Holdown Anchor)

$C_{ac} =$ 60.0 inches

= Max ( $C_{a,min}$ / $C_{ac}$ , 1.5 $H_{ef}$ / $C_{ac}$ ) if $C_{a,min} < C_{ac}$

= Max (0.08, 0.60 )

$\Psi_3 = \Psi_{cp,N} =$ 0.60 Modification for Post-Installed Anchors (ACI D.5.2.7)

$\phi_{EQ}\ \phi_{sa,cb}\ N_{cb} =$ 7.08 Kips (Concrete Break-out Strength - Tension)

c) Pullout Strength of Anchor - Tension (ACI 318-08 Section D5.3)

Note: Condition B is assumed per Section D4.4, where Supplementary reinforcement in not present in failure prism.

$\phi_{EQ}\ \phi_{sa,po}\ N_p = \phi_{EQ}\ \phi_{sa,po}\ N_p\ \Psi_4$ (D-14)

Where $\phi_{EQ} =$ 0.75 (Steel Anchor - Seismic Region - ACI Section D.3.3.3)

$\phi_{sa,po} = \phi_{sa,cb} =$ 0.65 (Steel Anchor - Concrete Breakout/Pullout Category 2 - ACI D.4.4)
Cat 1 : Low Sensitivity to installation and High Reliability

$N_p =$ Pull-out Strength of a single anchor in Tension in Cracked Concrete (ACI D.5.2.2)

= 8 $A_{brg}$ $f'_c$ (D-15) and $A_{brg} = A_{se,N} =$ Bearing Area of Anchor Bolt

= 0.994 $in^2$ for $t_{HD} =$ 1.13 inches (Diameter of Holdown Anchor)

$f'_c =$ 3.25 Ksi
= 3,250 Psi

$N_p =$ 25.84 kips

$\Psi 4 = \Psi_{c,P} =$ Modification for uncracked Concrete (ACI D.5.2.7)

= 1.0 Note: Existing Concrete is assumed to crack at anchor bolt limit state or capacity of holdown.

$\Psi 4 = \Psi_{c,P} =$ 1.0 Modification for Uncracked Concrete (ACI D.5.2.6)

$\phi_{EQ}\ \phi_{sa,po}\ N_p =$ 12.60 Kips (Concrete Pull-out Strength - Tension)

d) Concrete Side-face Blowout Strength of Anchor - Tension (ACI 318-08 Section D5.4)

$\phi_{EQ}\ \phi_{sa,sf}\ N_p = \phi_{EQ}\ \phi_{sa,po}\ N_p$

Where $\phi_{EQ} =$ 0.75 (Steel Anchor - Seismic Region - ACI Section D.3.3.3)

$\phi_{sa,sf} = \phi_{sa,cb} =$ 0.65 (Steel Anchor - Concrete Breakout/Pullout/Side Blowout Category 2 - ACI D.4.4)
Cat 1 : Low Sensitivity to installation and High Reliability

$N_p = 160\ C_{a1}\ A_{brg}^{0.5}\ \lambda\ f'^{0.5}_c$ Where $C_{a1} =$ 4.50 inches (Distance from Wall CL to Left Edge of Existing Footing Wall)

$C_{a2} =$ 16.25 inches (Distance from Wall CL to Right Edge of Existing Footing Wall)

$C_{a,min} =$ 4.50 inches

and $A_{brg} = A_{se,N} =$ 0.994 $in^2$ for $t_{HD} =$ 1.00 inches (Diameter of Holdown Anchor)

$\lambda =$ 1.00 (1.0 for NWC, 0.75 for LWC)
Note: NWC - Normal Weight Concrete Assumed

$f'_c =$ 3.25 Ksi
= 3,250 Psi

$N_p =$ 40.92 Kips

$\phi_{EQ}\ \phi_{sa,sf}\ N_p =$ 20.0 Kips (Concrete Side-face Blowout Strength - Tension)

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE F (EVAL CASE E-3R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

e) Limiting Governing Strength of Anchor - Tension Only

Note: Holdown = HDU14

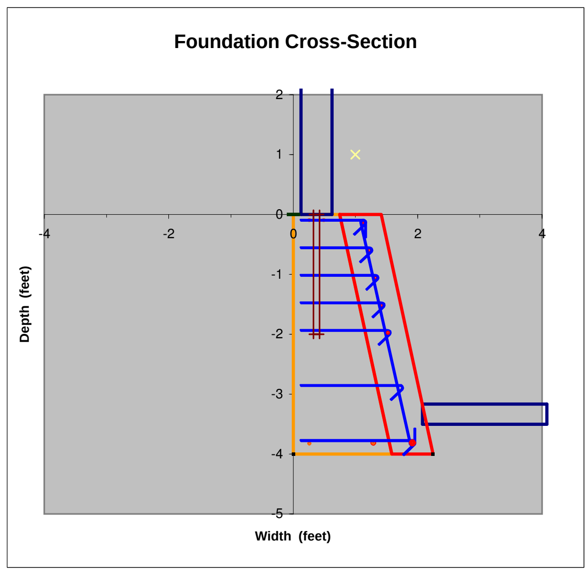
| | | | |
|---|---|---|---|
| $F_{CU}$ = | 20.13 | Kips (Ultimate Capacity of Holdown) |
| $H_{HD}$ = | 24.00 | inches (Embedment depth of Holdown Anchor) |
| $t_{HD}$ = | 1.13 | inches |

Limiting Strength of Anchor in Tension:

$T_{UC}$ = Min ($\phi_{EQ}$ $\phi_{sa,t}$ $N_{sa}$ , $\phi_{EQ}$ $\phi_{sa,cb}$ $N_{cb}$ , $\phi_{EQ}$ $\phi_{sa,po}$ $N_p$ , $\phi_{EQ}$ $\phi_{sa,sf}$ $N_p$ )

| | | | |
|---|---|---|---|
| Where $\phi_{EQ}$ $\phi_{sa,t}$ $N_{sa}$ = | 53.68 | Kips | (Bolt Design Strength) |
| $\phi_{EQ}$ $\phi_{sa,cb}$ $N_{cb}$ = | 7.08 | Kips | (Concrete Break-out Strength) |
| $\phi_{EQ}$ $\phi_{sa,po}$ $N_p$ = | 12.60 | Kips | (Concrete Pull-out Strength) |
| $\phi_{EQ}$ $\phi_{sa,sf}$ $N_p$ = | 20.0 | Kips | (Concrete Side-face Blowout Strength) |

$T_{UC}$ = 7.08 Kips




Required Number of Holdown Anchors:

$N_A$ = $F_{CU}$ / $T_{UC}$ Where $F_{CU}$ = 20.13 Kips (Ultimate Capacity of Holdown)

= 2.84 $T_{UC}$ = 7.08 Kips (Strength per Anchor)

$N_A$ = 3

**Use 3 - HDU14 with 1.13 Dia anchors @ 24" oc Min EA side**

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE F (EVAL CASE E-3R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

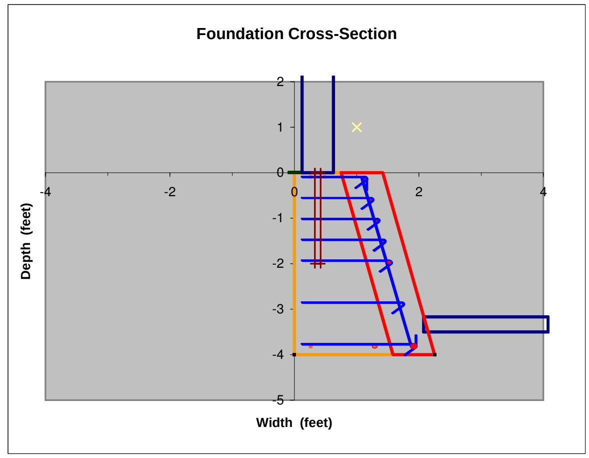
## 6. Adequacy of Footing - Shear

**A. Flexural/One-Way Shear on Full Cross-Section**

Note: Effective transverse width of footing for Shear is assumed as Top of Footing Width, $L_y = L'_T$

Shear demands: $V_{ux}$ = 16.1 Kips @ $x_L$ = 3.00 feet (locations at distance d from face of Wall - Left side)
= 19.5 Kips @ $x_R$ = 37.00 feet ( - Right side)

=> $\boxed{V_{ux} = 19.5 \text{ Kips}}$




ii) Shear capacity of concrete without shear reinforcement

$$\varphi\, V_c = \varphi \left( 1.9 \sqrt{f'_c} + 2500\, \rho_w \frac{V_u\, d}{M_u} \right) b_w\, d \;\; \le \;\; 3.5 \sqrt{f'_c}\, b_w\, d \qquad (11\text{-}5)$$

Note: $V_u\,d/M_u$ value must be ≤ 1.0

Where $\phi$ = 0.75
$f'_c$ = 3,250 psi

$\rho_w = A_{sx}/(L_y\, d_x)$ and $A_{sx}$ = 0.44 in^2
$L_y$ = 1.42 feet
= 17.0 inches
d = 44.88 inches

$\boxed{\rho_w = 0.000577}$

$V_u$ = 20 kips
d = 44.88 inches
$M_u$ = 1 Kip-ft @ $V_{ux}$ = 20 Kips (location of shear value)

Check of $V_u\, d/M_u$ value limit:
$V_u\, d/M_u$ = 59.61 where Vu = 20 kips
**NG, value taken as unity.**
d = 44.88 inches
$M_u$ = 1 kip-ft
= 15 kip -in

$b_w = L'_T$ = 1.4 feet
= 17.0 inches
d = 44.88 inches

=> $\boxed{\phi\, V_c = 62.8 \text{ kips}}$

Comparison w/ Equation 11-3:
$\phi\, V_c = \phi\, 2\, f'^{\,0.5}_c\, b_w\, d$ Where $\phi$ = 0.75
$f'_c$ = 3,250 psi
$b_w = L'_T$ = 17.0 inches
d = 44.88 inches

$\boxed{\phi\, V_c = 65.2 \text{ kips}}$

Check of upper value limit:
$\phi\, V_{c,max} = 3.5\, f'^{\,0.5}_c\, b_w\, d$ Where $f'_c$ = 3,250 psi
$b_w = L'_T$ = 17.0 inches
d = 44.88 inches

$\boxed{\phi\, V_{c,max} = 152.2 \text{ kips}}$

$\boxed{\phi\, V_c = 62.8 \text{ kips}}$
**OK, > Vu**

Note: D/C Ratio = 0.31 (Demand to Capacity Ratio - Shear)

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE F (EVAL CASE E-3R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**B. Shear Across Vertical Plane between Existing and New Concrete Retrofit Wals**

Note: Reinf Layout = 1 (One-Sided Retrofit Walls)

a) Capacity of Single Dowel - Shear Friction (ACI 11.6.4)

$\phi V_n = \phi A_{vf} f_y \mu \lambda$ (11-25) Where $\phi$ = 0.75

$A_{vf}$ = 0.50 $in^2$ (Dowel Area) Note: Dowels are No. 4 bars

$f_y$ = 60.00 Ksi

$\mu$ = 0.60 ( RC - RC Surface not roughened)

$\lambda$ = 1.00 (1.0 for NWC, 0.75 for LWC) Note: NWC - Normal Weight Concrete Assumed

$\phi V_n$ = 13.50 Kips

Upper Limit Restrictions (ACI 11.6.5) :

$\phi V_{MAX} = \phi MIN(0.2 f'_c, 0.48 + 0.08 f'_c, 0.80) A_c$ Where $\phi$ = 0.75

$f'_c$ = 3,250 Psi

= $\phi$ MIN (0.65, 0.74, 0.80 ) Ac = 3.25 Ksi

= 0.75 (0.65 ) Ac $A_c = \pi R_D^2$ for $R_D$ = Min ($L_e$, $S_D$)

$L_e$ = 6.00 inches (Depth of Embedment)

$S_D$ = 12.00 inches (Dowel Spacing)

$R_D$ = 6.00 inches

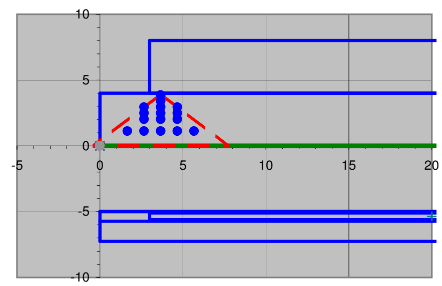
= 0.49 ( 113 ) $A_c$ = 113 $in^2$ ( Concrete Area)

$\phi V_{MAX}$ = 55 kips (Upper Limit on Dowel Strength)

OK

$\phi V_n$ = 13.50 Kips (Shear Capacity of Single Dowel)

Footing Elevation and Plan - Compression Zone



b) Demands on Dowels - **Compression** at Holdown Anchor

$V_{uc} = P_c = P_{EQ} + 1.2 P_{DL}/2$ Where $P_{EQ} = M_{OT} / L_W$ and $M_{OT}$ = 692.19 Kip-ft

$L_W$ = 34.00 feet

$P_{EQ}$ = 28.50 Kips

$P_{DL}$ = 71.88 Kips

**$V_{uc}$ = 71.63 Kips**

- Assume triangular Compression pressure distribution as shown.

Determination of Effective Dowels at Holdowns in Compression Zone :

$N_D$ = **16** (Number of Dowels)

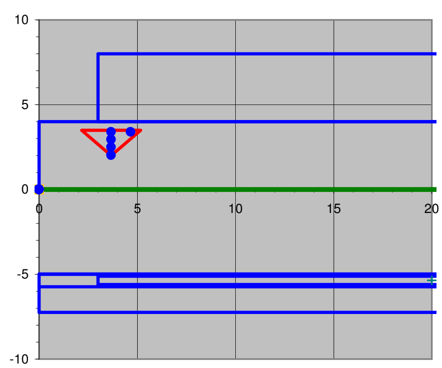
$N_{VP}$ = **1** (Number of Shear Planes - Single Shear)

$\phi V_n$ = 13.50 Kips (Shear Capacity of Single Dowel)

$\phi V_n N_D N_V$ = 216 Kips (Shear Capacity of Compression Zone)

OK

Footing Elevation and Plan - Tension Zone



c) Demands on Dowels - **Tension** at Holdown Anchor

$V_{ut} = P_t = MAX(P_{EQ} - 0.6 P_{DL}/2, F_{UC})$ Where $P_{EQ}$ = 28.50 Kips

= Max ( 6.94 , 14.38 ) $P_{DL}$ = 71.88 Kips

$F_{CU}$ = 14.38 Kips (Ultimate Capacity of Holdown)

**$V_{ut}$ = 14.38 Kips**

- Assume Tension triangular pressure distribution as shown.

Determination of Effective Dowels at Holdowns in Tension Zone :

$N_D$ = **5** (Number of Dowels)

$N_{VP}$ = **1** (Number of Shear Planes - Single Shear)

$\phi V_n$ = 13.50 Kips (Shear Capacity of Single Dowel)

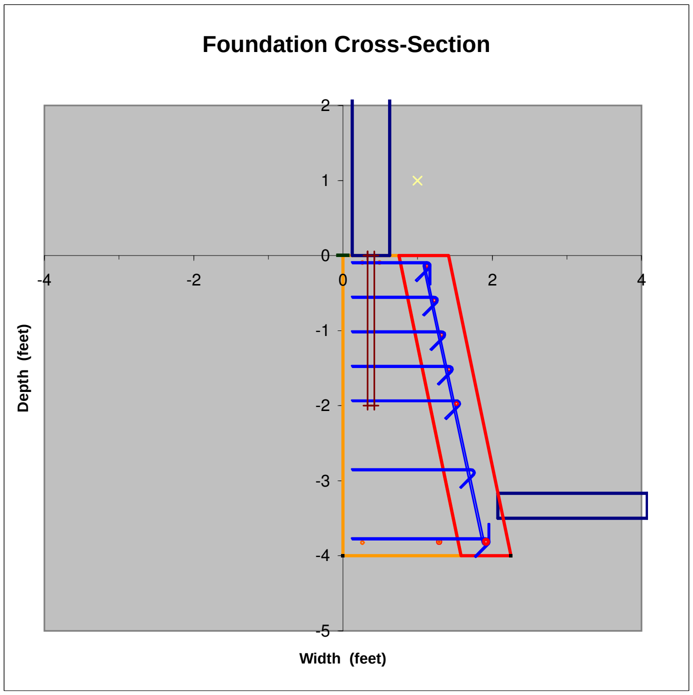
$\phi V_n N_D N_V$ = 67.5 Kips (Shear Capacity of Tension Zone) Note: D/C Ratio = 0.33 (Demand to Capacity Ratio - S

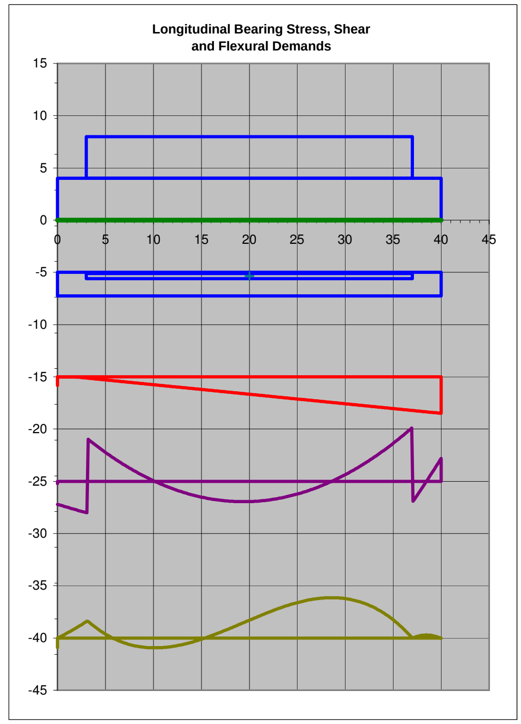
OK

**Footing OK for Shear**

**EXISTING FOUNDATION RETROFIT- SHEAR WALL AT GRIDLINE F (EVAL CASE E-3R)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 7. Adequacy of Footing - Flexure




a) Flexural demands

| | | | | | | |
|---|---|---|---|---|---|---|
| $M_{ux}$ = | 32 | Kip-ft | @ $x_L$ = | 3.00 | feet (locations at face of column - Left side) | |
| = | 382 | Kip-in | | | Note: $X_f$ = 3.00 | feet (Cantilever Length) |
| = | 34 | Kip-ft | @ Wall and Footing Centerline | | | |
| = | 410 | Kip-in | | | | |
| = | 1 | Kip-ft | @ $x_R$ = | 37.00 | feet ( - Right side) | |
| = | 15 | Kip-in | | | Note: $X_f$ = 3.00 | feet (Cantilever Length) |

=> $\boxed{M_{ux} = 410 \text{ Kip-in}}$

b) Required Reinforcement Ratio (ACI 10.2)

$$\rho_r = \frac{0.85\, f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2\, M_u}{0.765\, b\, d^2\, f'_c}}\right]$$

Where $f'_c$ = 3.25 Ksi
$f_y$ = 40.00 Ksi
$M_u$ = 410 kip-in
$L_y = L_B$ = 2.3 feet
= 27 inches
$d_x$ = 44.88 inches

$\boxed{\rho_r = 0.000210}$

c) Reinforcement Ratio Provided

$\rho_w = A_{sx}/(L_y\, d_x)$

Where $A_{sx} = A_N + A_E$

$A_N$ = 0.44 in$^2$ (New Flexural Reinforcement)

$A_E$ = 0.40 in$^2$ (Existing Flexural Reinforcement)

$\boxed{A_{sx} = 0.84 \text{ in}^2}$

$L_y = L_B$ = 2.3 feet
= 27.0 inches
d = 44.88 inches

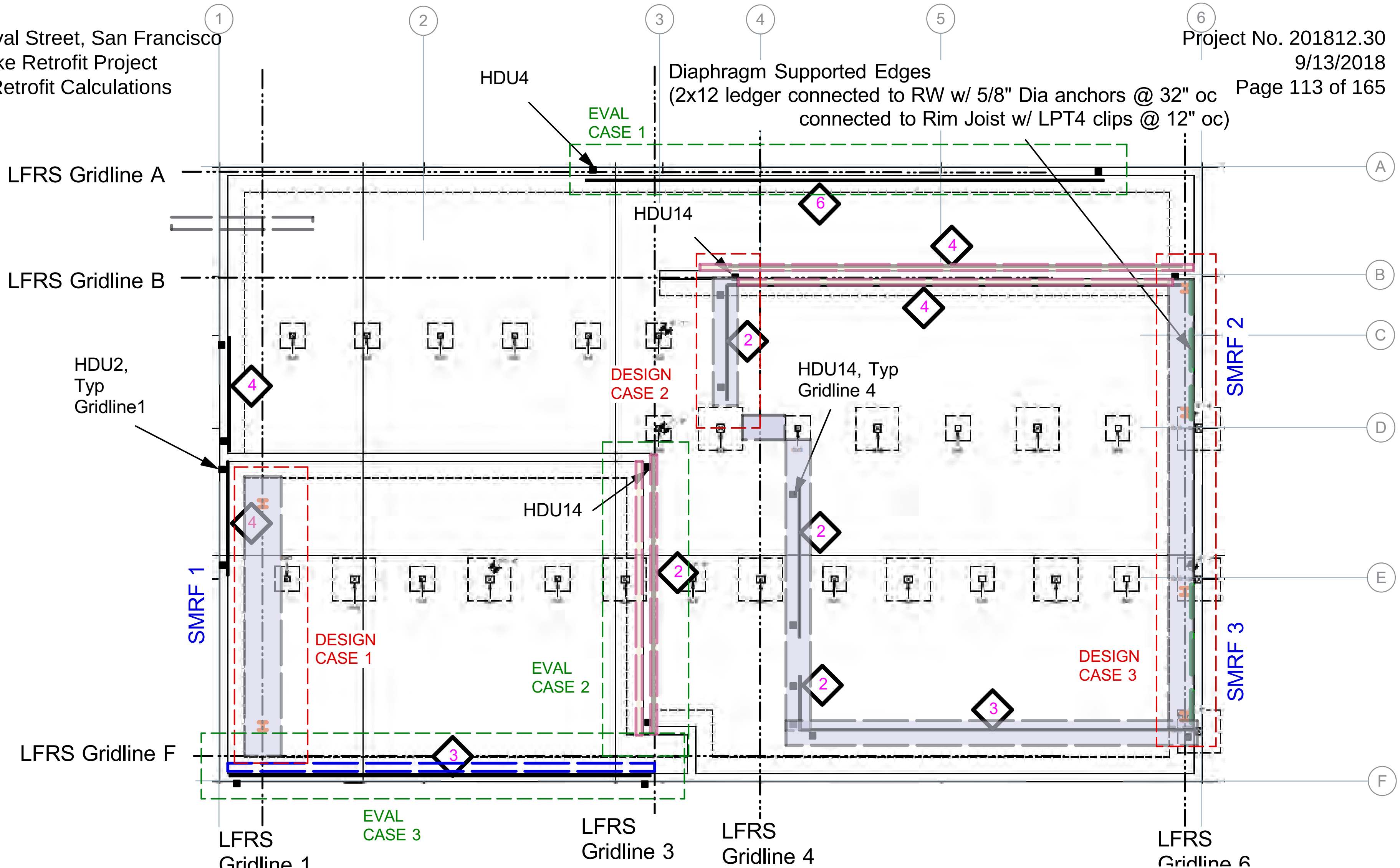
$\boxed{\rho_w = 0.000693}$ (reinforcement ratio provided)

Note: D/C Ratio = 0.30 (Demand to Capacity Ratio - Flexure)

**OK**

**1 - # 6 Bars OK for Longitudinal Flexure with DC Ratio = 0.30**

# DESIGN OF NEW FOUNDATIONS:




# FOUNDATION RETROFIT PLAN - LEVEL 0

## FOUNDATION RETROFIT PLAN - FOOTING EVALUATION/DESIGN CASES

NTS

**RETROFIT LEGEND:**

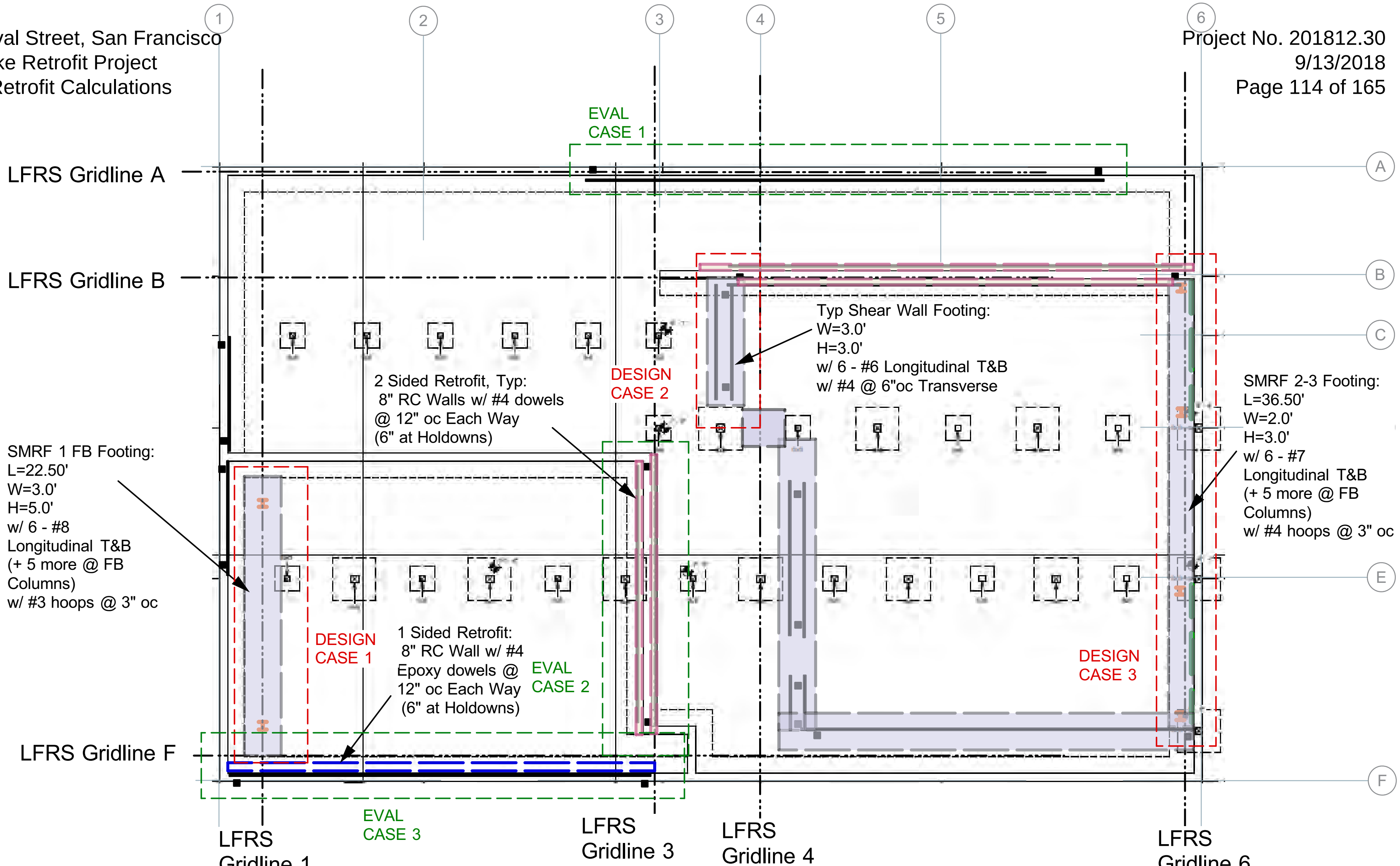
- 3 — 1/2" Struct I Sheathing w/ 10d Edge Nailing
- ■ — Holdown each side, as indicated

### Lateral Force Resisting System (LFRS) Elements :

- (N) Shear Wall
- Simpson Strong Frame
- Diaphragm Supported Edges at Retaining Wall

### FOUNDATION RETROFIT ELEMENTS:

- (N) Footings
- (N) 2-Sided Wall Retrofit
- (N) 1-Sided Wall Retrofit




## FOUNDATION RETROFIT PLAN - LEVEL 0

## FOUNDATION RETROFIT PLAN - FOOTING EVALUATION/DESIGN CASES

NTS

Lateral Force Resisting System (LFRS) Elements :

- (N) Shear Wall
- Simpson Strong Frame
- Diaphragm Supported Edges at Retaining Wall

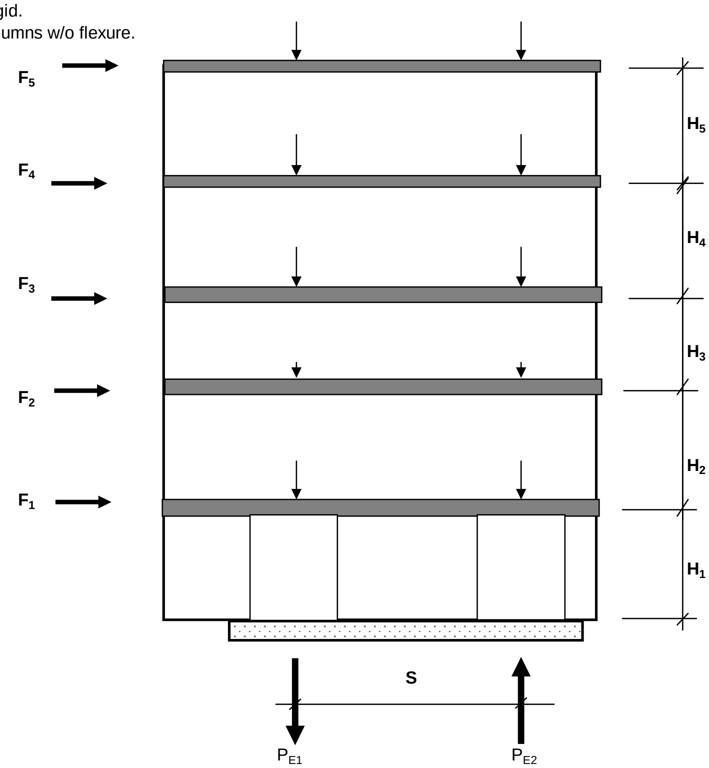
FOUNDATION RETROFIT ELEMENTS:

- (N) Footings
- (N) 2-Sided Wall Retrofit
- (N) 1-Sided Wall Retrofit

**SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE 1 (W-E EQ LOADS) - CASE N-1**
**DETERMINATION OF VERTICAL AND LATERAL LOADS TO FOUNDATION**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Assumptions**

1. Existing walls are treated as pinned Columns. Footing is assumed rigid.
2. Column loads are located in transverse center of footing; limit of 2 columns w/o flexure.
3. Footing has no shear reinforcement.
4. Concrete is Normal Weight Concrete with uncoated bars.




**1. Lateral Loads and Load Effects**

V = 23.25 kips (Base Shear - A4 ASD)

S = 18.50 feet (Separation between Wall centerlines)

| Floor Level | Height (feet) | Loading ID | $V_x$ / V | Shear (Kips) | Force (Kips) |
|---|---|---|---|---|---|
| | | 5 | | | |
| R | 11.00 | 4 | 0.65 | 15.11 | 15.11 |
| 3 | 10.00 | 3 | 0.89 | 20.69 | 5.58 |
| 2 | 13.50 | 2 | 1.00 | 23.25 | 2.56 |
| 1 | 10.50 | 1 | 0.00 | 0.00 | -23.25 |

From summation of moments :

**$P_{E1}$ = -37.14 Kips**

**$P_{E2}$ = 37.14 Kips**

**2. Vertical Loads and Load Effects**

| | | Floor Tributary Loads | | | | | Wall Tributary Loads | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Wall** | Floor Level | DL (psf) | Length (feet) | Width (feet) | Area (ft$^2$) | Weight (kips) | WL (psf) | Length (feet) | Height (feet) | Area (ft$^2$) | Weight (kips) |
| | | | | | | | | | | | |
| | R | 20 | 13.88 | 2.00 | 28 | 0.56 | 14 | 13.88 | 11.00 | 153 | 2.14 |
| **1** | 3 | 30 | 13.88 | 2.00 | 28 | 0.83 | 14 | 13.88 | 13.50 | 187 | 2.62 |
| | 2 | 30 | 13.88 | 2.00 | 28 | 0.83 | 14 | 13.88 | 13.50 | 187 | 2.62 |
| | 1 | 0 | 13.88 | 2.00 | 28 | 0.00 | 0 | 13.88 | 10.50 | 146 | 0.00 |

Sum of Floor Weight = 2.22 Kips    Sum of Wall Weight = 7.38 Kips

**$P_{D1}$ = 9.60 Kips**

| | | Floor Tributary Loads | | | | | Wall Tributary Loads | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Wall** | Floor Level | DL (psf) | Length (feet) | Width (feet) | Area (ft$^2$) | Weight (kips) | WL (psf) | Length (feet) | Height (feet) | Area (ft$^2$) | Weight (kips) |
| | | | | | | | | | | | |
| | R | 20 | 13.88 | 2.00 | 28 | 0.56 | 14 | 13.88 | 11.00 | 153 | 2.14 |
| **2** | 3 | 30 | 13.88 | 2.00 | 28 | 0.83 | 14 | 13.88 | 13.50 | 187 | 2.62 |
| | 2 | 30 | 13.88 | 2.00 | 28 | 0.83 | 14 | 13.88 | 13.50 | 187 | 2.62 |
| | 1 | 0 | 13.88 | 2.00 | 28 | 0.00 | 0 | 13.88 | 10.50 | 146 | 0.00 |

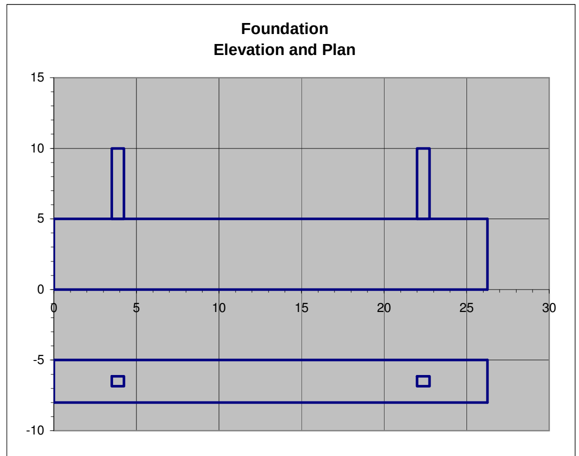
Sum of Floor Weight = 2.22 Kips    Sum of Wall Weight = 7.38 Kips

**$P_{D2}$ = 9.60 Kips**

**SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE 1 (W-E EQ LOADS) - CASE N-1**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Assumptions**

1. Column loads are located in transverse center of footing; limit of 2 columns w/o flexure.
2. Footing has no shear reinforcement.
3. Concrete is Normal Weight Concrete with uncoated bars.




**Footing Parameters :**

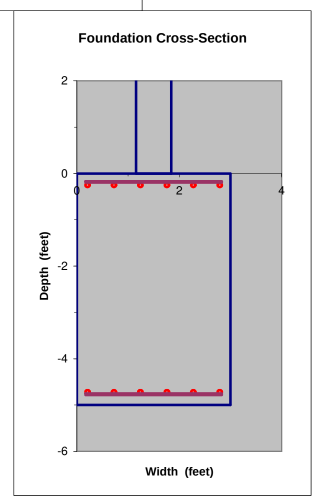
Note: Includes 2.0' portion of (E) Footing each end for Bearing purposes.

Footing Size :
$L_x$ = 26.25 feet
$L_y$ = 3.00 feet
$h_f$ = 5.00 feet

Column Sizes :
$C_{1x}$ = 0.8 feet (column length)
$C_{1y}$ = 0.7 feet (column width)
$x_1$ = 3.88 feet (distance from edge of footing to $C_1$ Centerline)

$C_{2x}$ = 0.8 feet (column length)
$C_{2y}$ = 0.7 feet (column width)
$x_2$ = 22.38 feet (distance from edge of footing to $C_2$ Centerline)

Note: S = 18.50 feet (Separation between column centerlines)




Interconnected Slab at Sides:

Note : Slabs at sides are used only to reduce soil bearing pressure; footing is designed to take all loads.

Side : Left Right
t Inches (Slab Thickness)
X Feet (distance to other Slab Edge Support
$f'_c$ Ksi
Conn Type (D= Dowel, C= Continuous)

17.7 23.22 1.31

Footing Loads :
$V_x$ = 23.25 kips (Base Shear - A4 ASD)
$V_y$ = 2.33 kips

Load Factors : Strength = S (S for Strength, OS for Over-Strength)
$\Omega$ = 3.00 (Overstrength Factor - SMRF)

| | Unfactored Loads | | | Service Loads | | | | Strength Loads | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Load | D | L | EQ | 1.0 D + EQ | 0.6 D + EQ | L | $P_S$ | 1.2 D + 1.4 EQ | 0.9 D + 1.4 EQ | 1.6 L | $P_U$ |
| $P_1$ | 10 | | -37 | -28 | -31 | 0 | -31 | -40 | -43 | 0 | -43 |
| $P_2$ | 10 | | 37 | 47 | 43 | 0 | 47 | 64 | 61 | 0 | 64 |

Capacity Factors : $\phi_v$ = 0.75 (Shear)
$\alpha$ = 40 (40 for interior columns, 30 for edge columns, 20 for corner columns)

Concrete : $f'_c$ = 3.25 Ksi
$f_y$ = 60.00 Ksi
$\rho_c$ = 0.150 kip/ft$^3$

Reinforcement: $d_c$ = 2.00 inches (bar clearance - top)
= 3.00 inches (bar clearance - bottom)
= 2.00 inches (bar clearance - sides)

| | Orientation | Bar Size | N Bars | Bottom Layer | d (inches) | Bar Spacing (inches) | Bar Diameter (inches) | Bar Area Per Bar (in^2) | Bar Area Total (in^2) |
|---|---|---|---|---|---|---|---|---|---|
| Top Mat | x | 8 | 6 | x | 57.00 | 6.20 | 1.00 | 0.79 | 4.74 |
| | y | 3 | 100 | | 57.25 | 3.14 | 0.38 | 0.11 | 11.00 |
| Bottom Mat | x | 8 | 6 | | 55.63 | 6.20 | 1.00 | 0.79 | 4.74 |
| | y | 3 | 100 | x | 57.63 | 3.14 | 0.38 | 0.11 | 11.00 |

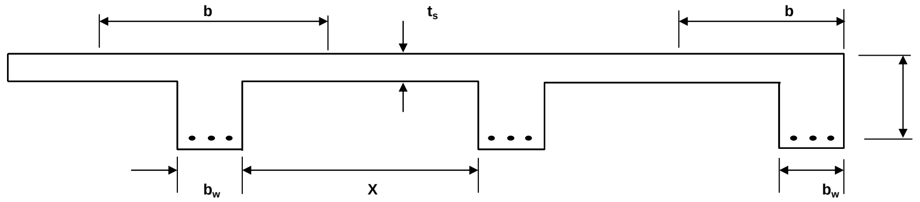
Note: Used for placing top bars only.

Soil Parameters : Soil density = 120 pcf
$\sigma_{allow}$ = 2.00 ksf (allowable bearing pressure)
$\sigma_p$ = 0.30 ksf/ft (Passive Soil Pressure)
$\mu$ = 0.25 ksf (Coefficient of Friction)

**SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE 1 (W-E EQ LOADS) - CASE N-1**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 1. Design of Slab-to-Footing Connections

a) Effective Slab Width (ACI ACI 21.6.2.2)




i) Symmetrical T-Beams (ACI 8.12.2)

$b \le$ Min ($L/4 + b_w$, $16\ t_s + b_w$, $X + b_w$)

Where L = feet (Footing span length)
= inches
$t_s$ = inches (Slab thickness - Average value)
$b_w = L_y$ = feet (Footing Width)
= inches
X = feet (Slab Supported Length - Average Value)
= inches
=

| b = | inches |
|---|---|

ii) Slabs on One Side (ACI 8.12.3)

$b \le$ Min ($b_w + L/12$, $b_w + 6\ t_s$, $X/2 + b_w$)

Where $b_w = L_y$ = feet (Footing Width)
= inches
L = feet (Footing span length)
= inches
$t_s$ = inches (Slab thickness)
X = feet (Slab Supported Length)
= inches
=

| b = | inches |
|---|---|

| **b =** | **inches** |
|---|---|
| **=** | **feet** |

b) Required Slab Reinforcement Area

$A_{sr} = 0.0018\ b\ t_s$ (ACI 7.12.2.1)

Where b = inches (Effective Slab Width)
$t_s$ = inches (Slab thickness)

| $A_{sr}$ = | in$^2$ |
|---|---|

c) Required Slab Reinforcement Spacing

$S_{sr} = b / n_b \le 2\ t_s$ (ACI 13.3.2)

Where b = inches (Effective Slab Width)

$n_b$ = Ceiling ($A_{sr}/A_b$)

for $A_{sr}$ = in$^2$ (Slab Reinforcement Area - Required)
$A_b$ = 0.20 in$^2$ for No. 4 bars

=

| $n_b$ = | bars |
|---|---|

$t_s$ = inches
$2\ t_s$ = inches

| $S_{sr}$ = | inches |
|---|---|

**RC Slab not Needed**

**SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE 1 (W-E EQ LOADS) - CASE N-1**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 2. Lateral Resistance of Foundation

### 2A. Longitudinal Loading

$F_{Rx} = 0.5\,L'_y\,h'^2_f\,\sigma_p + 0.6\,(W_f + P)\,\mu$

Where $L'_y = L_y + 2\,t_{nw}$ and $L_y =$ 3.0 feet

$t_{ew} =$ 1.00 feet (Thickness of (E) connected walls at ends)

$L'_y =$ 5.00 feet (Bearing Width at Ends of Footing)

$h'_f = h_f + h_{sk}$ and $h_f =$ 5.0 feet

$h_{sk} =$ 0.00 feet (Additional height of Shear Key at Footing End)

$h'_f =$ 5.0 feet (Bearing Height at Ends of Footing)

= (62.5) (0.30) + 0.6 (78.3) (0.25)

$\sigma_p =$ 0.30 ksf/ft (Passive Soil Pressure)

= (18.75) + (11.74)

$W_f = \rho_c\ L_x\ L_y\ h_f$ and $\rho_c =$ 0.150 kip/ft^3

$L_x =$ 26.3 feet

$L_y =$ 3.0 feet

$h_f =$ 5.0 feet

$W_f =$ 59.06 Kips (Footing Weight)

$P = P_1 + P_2$ and $P_1 =$ 9.6 Kips

$P_2 =$ 9.6 Kips

$P =$ 19.2 Kips (Service Load)

$\mu =$ 0.25 ksf (Coefficient of Friction)

$F_{Rx} =$ 30.49 kips

**OK**

Note : $V_x =$ 23.25 kips

**Foundation OK for Sliding**

## 3. Soil Pressure due to Applied Loads - Service and Strength Loading

a) Applied soil stress - uniform

$q = (P_1 + P_2)/L_x$

| | Service | Strength | |
|---|---|---|---|
| Where $P_1 =$ | -31 | -43 | kips |
| $P_2 =$ | 47 | 64 | kips |
| $L_x =$ | 26.3 | feet | |

$q =$ 0.59 kip/ft **Service**
$=$ 0.77 kip/ft **Strength**

b) Centroid of Factored Loads - from Left Edge of Footing

$X_R = (x_1\,P_1 + x_2\,P_2 + W_f\,L_x/2) / (P_1 + P_2 + W_f)$

| | Service | Strength | |
|---|---|---|---|
| Where $X_1 =$ | 3.88 | 3.88 | feet (distance from edge of footing to $C_1$ Centerline) |
| $P_1 =$ | -31 | -43 | kips |
| $X_2 =$ | 22.38 | 22.38 | feet (distance from edge of footing to $C_2$ Centerline) |
| $P_2 =$ | 47 | 64 | kips |
| $W_f =$ | 59.06 | Kips (Footing Weight) | |
| $L_x =$ | 26.3 | feet | |

$X_R =$ 22.83 feet **Service**
$=$ 25.60 feet **Strength**

c) Applied soil stress - **Trapezoidal**

Note: This conditions applies when $\Delta q < q$, and $L_b = L_x$ **<= Does not apply**

$\Sigma M_o = -X_1\,P_1 - X_2\,P_2 + (q - \Delta q)\,L_x^2/2 + 1/2\,(2\,\Delta q)\,L_x\,L_{\Delta q}$

=> $\Delta q = (\,q\,L_x^2/2 - X_1\,P_1 - X_2\,P_2)/(L_x^2/2 - L_x\,L_{\Delta q})$

| | Service | Strength | |
|---|---|---|---|
| Where $X_1 =$ | 3.88 | 3.88 | feet (distance from edge of footing to $C_1$ Centerline) |
| $P_1 =$ | -31 | -43 | kips |
| $X_2 =$ | 22.38 | 22.38 | feet (distance from edge of footing to $C_2$ Centerline) |
| $P_2 =$ | 47 | 64 | kips |
| $q =$ | 0.59 | 0.77 | kips/ft |
| $L_x =$ | 26.3 | 26.3 | feet |

$L_{\Delta q} = L_x/3$ if $X_R < 0.5\,L_x$
$= 2\,L_x/3$ if $X_R > 0.5\,L_x$

$L_{\Delta q} =$ 17.50 feet **Service**
$=$ 17.50 feet **Strength**

Incremental Soil Bearing Stresses :

$\Delta q =$ 6.29 **Service**
**Soil Stress is NOT Trapezoidal**

$\Delta q =$ 8.61 kips/ft **Strength**
**Soil Stress is NOT Trapezoidal**

**SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE 1 (W-E EQ LOADS) - CASE N-1**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

d) Applied soil stress - **Triangular** **<= Governs**

Notes: This condition applies when $\Delta q \geq q$, and $L_b \leq L_x$

$X_R$ = 22.83 feet => **Large Rotation to Right** **Service**
= 25.60 feet **Strength**

$0.5\ L_x$ = 13.1 feet

i) Rotation to Left - Footing Bearing Length

$L_b = 3\ (0.5\ L_x - e)$ Where $0.5\ L_x$ = 13.1 feet

$e = X_R - 0.5\ L_x$ and $X_R$ = 22.83 feet **Service**
$X_R$ = 25.60 feet **Strength**

| | | | |
|---|---|---|---|
| e = | 9.71 | feet | **Service** |
| = | 12.48 | feet | **Strength** |

| | | | |
|---|---|---|---|
| **Service** | $L_b$ = | NA | feet |
| **Strength** | = | NA | feet |

ii) Rotation to Right - Footing Bearing Length

$L_b = 3\ (0.5\ L_x - e)$ Where $0.5\ L_x$ = 13.1 feet

$e = X_R - 0.5\ L_x$ and $X_R$ = 22.83 feet **Service**
$X_R$ = 25.60 feet **Strength**

| | | | |
|---|---|---|---|
| e = | 9.71 | feet | **Service** |
| = | 12.48 | feet | **Strength** |

| | | | |
|---|---|---|---|
| $L_b$ = | 10.25 | feet | **Service** |
| | 1.94 | feet | **Strength** |

iii) Resulting Soil Bearing Length and Triangular Pressure

$\Delta q = 2(P_1 + P_2)/L_b$

| | Service | Strength | |
|---|---|---|---|
| Where $P_1$ = | -31 | -43 | kips |
| $P_2$ = | 47 | 64 | kips |
| $L_b$ = | 10.25 | 1.94 | feet |

| | | | |
|---|---|---|---|
| $\Delta q$ = | 3 | kips/ft | **Service** |
| = | 21 | kips/ft | **Strength** |

e) Applied Soil Stresses - Governing

**Service Loads :**

Note: Large Rotation to Right

q = 0.00 kips/ft $X_R$ = 22.83 feet
$\Delta q$ = 3.00 kips/ft $0.5\ L_x$ = 13.13 feet

$L_b$ = 10.25 feet
Note: $L_o$ = 16.00 feet (location of soil zero value)

**Strength Loads :**

Note: Large Rotation to Right

q = 0.00 kips/ft $X_R$ = 25.60 feet
$\Delta q$ = 20.74 kips/ft $0.5\ L_x$ = 13.13 feet

$L_b$ = 1.94 feet
Note: $L_o$ = 24.31 feet (location of soil zero value)

f) Check of Soil Bearing Stress

**Service Loads :**

$\sigma_b = q_{max} / L_y \leq \sigma_b$ Where $q_{max}$ = 3.00 kips/ft
$L_y = b$ = 3.00 feet

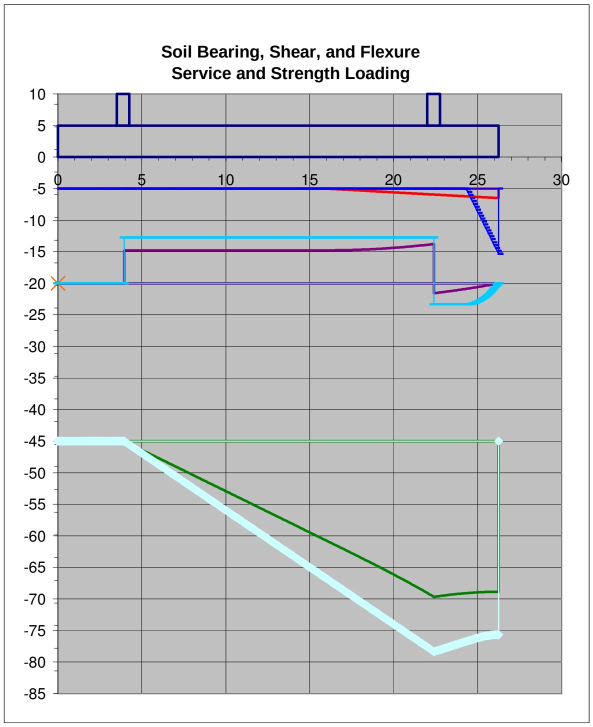
| | | | |
|---|---|---|---|
| $\sigma_b$ = | 1.00 | Ksf | **Service** |

**OK**

**Strength Loads :**

$\sigma_{bu} = q_u / L_y \leq \sigma_b$ Where $q_u$ = 20.74 kips/ft
$L_y = b$ = 3.00 feet

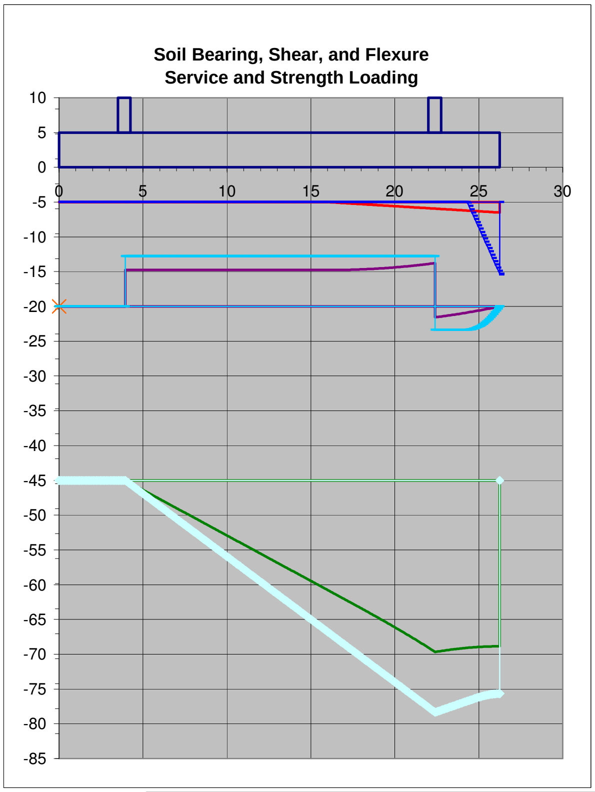
| | | | |
|---|---|---|---|
| $\sigma_b$ = | 6.91 | Ksf | **Strength** |



**SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE 1 (W-E EQ LOADS) - CASE N-1**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 4. Applied Loading and Demands on Footing - Strength Loads

| | Left End | Left Column Centerline | Inflection Point | Right Column Centerline | Right End |
|---|---|---|---|---|---|
| Location (feet) | 0 | 3.88 | | 22.38 | 26.25 |
| Load (kips) | - | -31 | - | 47 | - |
| $V_L$ (kips) | 0 | 0 | | 43 | |
| $V_R$ (kips) | - | 43 | - | -20 | - |
| $M_L$ (kip-ft) | 0 | 0 | - | -793 | - |
| $M_R$ (kip-ft) | - | 0 | | -800 | |




## 5. Adequacy of Footing - Shear

### 5A. Check of Flexural/One-Way Shear (ACI 15.5.2 and 11.1.3.1)

Shear demands: $V_{max}$ = 43 Kips @ x = 3.88 feet

$V_u = V_{max} - q\,(d + C/2)$

Where $V_{max}$ = 43 Kips

$q_u$ = 0.00 Kips/ft @ x = 3.88 feet

$d = h_f - d_c - d_b$ and $h_f$ = 5.0 feet
= 60.00 inches

$d_c$ = 3.00 inches
$d_b$ = 0.375 inches

| d = | 56.63 | inches |
|---|---|---|
| = | 4.72 | feet |

C = 0.00 feet

**$V_u$ = 43 Kips**

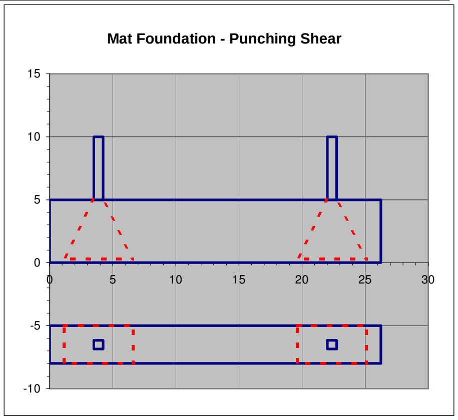
b) Shear Strength provided by Concrete (ACI 11.3.1.1)

$\phi V_c = \phi\, 2\, f'_c{}^{0.5}\, b_w\, d$

Where $\phi$ = 0.75
$f'_c$ = 3,250 psi
$b = L_y$ = 3.0 feet
= 36.0 inches
d = 56.63 inches

**$\phi V_c$ = 174 kips**
**OK, > Vu**

Note: if footing shear reinforcement is needed, use EQ (11-5) in ACI 11.3.2.1.




### 5B. Punching/Two Way Shear (ACI 11.5.5.2, 11.12.1.2)

**A. Left Column**

Shear demands: $V_u$ = 43 Kips

a) Failure Perimeter

$b_0 = 2\,(b_1 + b_2)$

Where $b_1 = X_1 + 0.5(C_{2x} + d)$ <= $C_{2x} + d$

and $X_1$ = 3.88 feet
= 46.50 inches
$C_{2x}$ = 9.00 inches
d = 56.63 inches

**$b_1$ = 65.63 inches**

$b_2 = C_{2y} + d$ <= $L_y$

and $C_{2y}$ = 8.28 inches
d = 56.63 inches
$L_y$ = 36.0 inches

**$b_2$ = 36.00 inches**

**$b_0$ = 203.3 inches**

**SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE 1 (W-E EQ LOADS) - CASE N-1**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

b) Factored Shear Capacity (ACI 11.12.2.1)

$$\varphi V_c = \varphi \, \mathrm{Min}\left[2 + \frac{4}{\beta}, \frac{\alpha d}{b_o} + 2, 4\right]\sqrt{f'_c}\, b_o\, d$$

Where $\phi$ = 0.75

$\beta = C_{max}/C_{min}$ and $C_{max}$ = 9.00 inches
$C_{min}$ = 8.28 inches

$\beta$ = 1.09

$\alpha$ = 40 (40 for interior columns, 30 for edge columns, 20 for corner columns)

$d = \mathrm{Min}(d_1, d_2)$ $d_1$ = 55.63 inches
$d_2$ = 57.63 inches

d = 55.63 inches

$b_0$ = 203.3 inches

$f'_c$ = 3,250 psi

$\phi V_c$ = 1,934 kips
**OK, > Vu**

**B. Right Column**

Shear demands: $V_u$ = 43 Kips

a) Failure Perimeter

$b_0 = 2\,(b_1 + b_2)$ Where $b_1 = 0.5(C_{2x} + d) + (L_x - X_2)$ <= $C_{2x} + d$

and $L_x$ = 26.3 feet
$X_2$ = 22.38 feet

$L_x - X_2$ = 3.88 feet
= 46.50 inches

$C_{2x}$ = 9.00 inches
d = 56.63 inches

$b_1$ = 65.63 inches

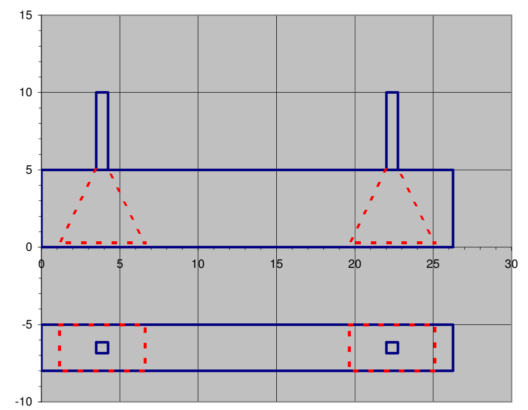
$b_2 = C_{2y} + d$ <= $L_y$

and $C_{2y}$ = 8.28 inches
d = 56.63 inches

$L_y$ = 36.0 inches

$b_2$ = 36.00 inches

$b_0$ = 203.3 inches




b) Factored Shear Capacity (ACI 11.12.2.1)

$$\varphi V_c = \varphi \, \mathrm{Min}\left[2 + \frac{4}{\beta}, \frac{\alpha d}{b_o} + 2, 4\right]\sqrt{f'_c}\, b_o\, d$$

Where $\phi$ = 0.75

$\beta = C_{max}/C_{min}$ and $C_{max}$ = 9.00 inches
$C_{min}$ = 8.28 inches

$\beta$ = 1.09

$\alpha$ = 40 (40 for interior columns, 30 for edge columns, 20 for corner columns)

$d = \mathrm{Min}(d_1, d_2)$ $d_1$ = 55.63 inches
$d_2$ = 57.63 inches

d = 55.63 inches

$b_0$ = 203.3 inches

$f'_c$ = 3,250 psi

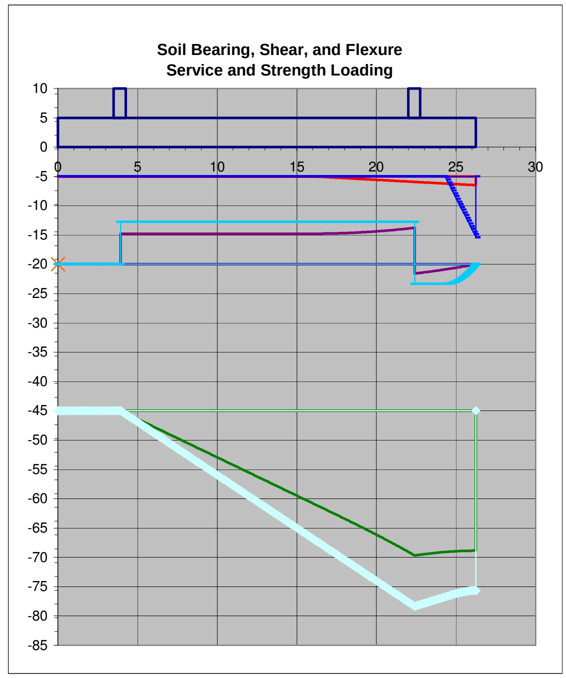
$\phi V_c$ = 1,934 kips
**OK, > Vu**

**Footing OK for Shear**

**SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE 1 (W-E EQ LOADS) - CASE N-1**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 6. Adequacy of Footing - Flexure

### 6A. Longitudinal Top Reinforcement Check




Flexural demands: $M_u$ = 0 Kip-ft @ x = feet (Inflection Point)

a) Required Reinforcement Ratio (ACI 10.2)

$$\rho_r = \frac{0.85\ f'_c}{f_y}\left[1-\sqrt{1-\frac{2\ M_u}{0.765\ b\ d^2\ f'_c}}\right]$$

Where $f'_c$ = 3.25 Ksi
$f_y$ = 60.00 Ksi
$M_u$ = 0
= 0 kip-in
$b = L_y$ = 3.0 feet
= 36 inches
$d_{top}$ = 57.00 inches

$\rho_r$ = 0.0000

b) Maximum Reinforcement Ratio (ACI 10.3.4)

$$\rho_t = 0.85\,\beta_1 \frac{f'_c}{f_y}\left[\frac{\varepsilon_c}{\varepsilon_c+\varepsilon_s}\right]$$

Where $f'_c$ = 3.25 Ksi
$f_y$ = 60.00 Ksi
$\beta_1$ = 0.85 for $f'_c$ ≤ 4.0 Ksi
= 0.85 - 0.05 ($f'_c$ - 4.0), ≥ 0.65 for $f'_c$ > 4.0 Ksi

$\beta_1$ = 0.85

$\varepsilon_c$ = 0.003 (ACI Section 10.3.4)
$\varepsilon_s$ = 0.005

$\rho_t$ = 0.0147

c) Minimum Reinforcement of Flexural Members (ACI 10.5, 7.12.2.1)

$\rho_{min}$ = Max [3 $f'_c{}^{0.5}/f_y$, 200/$f_y$] ≤ Max [1.33 $\rho_r$, 0.0018] Where $f'_c$ = 3,250 psi
= Max [0.0029,0.0033] ≤ Max[0.0000,0.0018] $f_y$ = 60,000 psi
= Max [0.0033] ≤ Max[0.0018] $\rho_r$ = 0.0000 (Required Reinforcement Ratio)

$\rho_{min}$ = 0.0018 ≤

d) Required Reinforcement Area

$A_{req} = \rho\ L_y\ d_2$

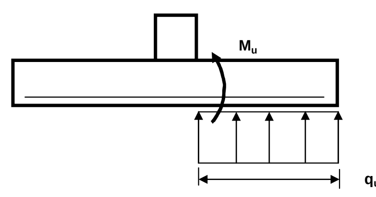
Where $\rho = \rho_r$ if $\rho_r \le \rho_{max}$ and $\rho_r \ge \rho_{min}$ and $\rho_r$ = 0.00000
= $\rho_{max}$ if $\rho_r \ge \rho_{max}$ $\rho_{max}$ = 0.01468
= $\rho_{min}$ if $\rho_r \le \rho_{min}$ $\rho_{min}$ = 0.00180

Where $\rho_r$ = 0.0018
$L_y$ = 3.0 feet
= 36.0 inches
$d_{top}$ = 57.00 inches

$A_{req}$ = 3.69 in^2 Note: $A_{s1}$ = 4.74 in^2 (reinforcement provided)
OK

**6 - No. 8 Longitudinal Top Bars OK**

### 6B. Longitudinal Bottom Reinforcement Check




a) Flexural Demands (ACI 15.4.2)

$M_u$ = 0 Kip-ft @ x = 3.88 feet (at face of Left Column)
= -800 Kip-ft @ x = 22.38 feet (at face of Right Column)

$M_u$ = 800 kip-ft
= 9,604 kip-in

b) Required Reinforcement Ratio (ACI 10.2)

$$\rho_r = \frac{0.85\ f'_c}{f_y}\left[1-\sqrt{1-\frac{2\ M_u}{0.765\ b\ d^2\ f'_c}}\right]$$

Where $f'_c$ = 3.25 Ksi
$f_y$ = 60.00 Ksi
$M_u$ = 9,604 kip-in
$b = L_y$ = 3.0 feet
= 36 inches
$d_{bott}$ = 55.63 inches

$\rho_r$ = 0.0016

**SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE 1 (W-E EQ LOADS) - CASE N-1**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

c) Required Reinforcement of Flexural Members

Note: $A_{sb}$ = 4.74 $in^2$ (Bottom flexural steel provided)

$A_{req} = \rho\, b\, d$

Where $\rho = \rho_r$ for $\rho_r \le \rho_t$ and $\rho_r \ge \rho_{min}$
$= \rho_{min}$ for $\rho_r \le \rho_{min}^{*}$
$= 0$ Otherwise

Where $\rho_r$ = 0.0016
$\rho_t$ = 0.0147
$\rho_{min}$ = 0.0018

$\rho$ = 0.0018

Note: * for $\rho_r \le \rho_{min}$ condition, check Minimum Reinforcement exception (ACI 10.5.3)

$A_{min} = 1.33\, \rho_r\, b\, d$

Where $\rho_r$ = 0.0016 (required reinforcement ratio)
$b = L_y$ = 36.0 inches
$d = d_{bott}$ = 55.63 inches

$A_{min}$ = 4.33 $in^2$

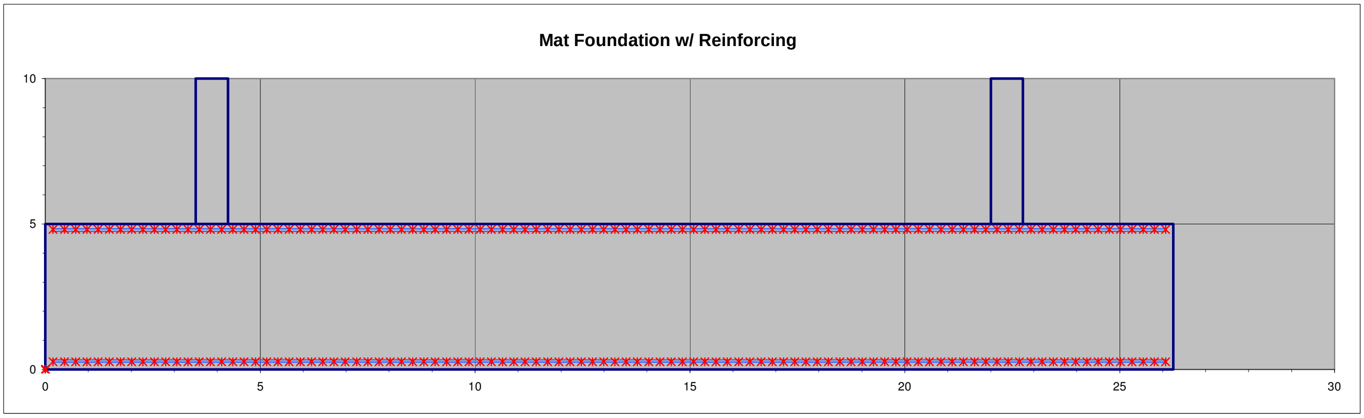
$b = L_y$ = 36.0 inches
$d = d_{bott}$ = 55.63 inches

$A_{req}$ = $in^2$

**6 - No. 8 Longitudinal Bottom Bars OK**

**SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE 1 (W-E EQ LOADS) - CASE N-1**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 7. Footing Reinforcement Summary




**Footing Parameters :**

Footing Size :
$L_x$ = 26.3 feet
$L_y$ = 3.0 feet
$h_f$ = 5.0 feet

Reinforcement Summary: $d_c$ = 2.00 inches (bar clearance - top)
= 3.00 inches (bar clearance - bottom)
= 2.00 inches (bar clearance - sides)

| | Orientation | Bar Size | N Bars | Bottom Layer | d (inches) | Bar Spacing (inches) | Bar Diameter (inches) | Bar Area Per Bar (in^2) | Bar Area Total (in^2) |
|---|---|---|---|---|---|---|---|---|---|
| Top Mat | x | 8 | 6 | x | 57.00 | 6.20 | 1.00 | 0.79 | 4.74 |
| | y | 3 | 100 | | 57.25 | 3.14 | 0.38 | 0.11 | 11.00 |
| Bottom Mat | x | 8 | 6 | | 55.63 | 6.20 | 1.00 | 0.79 | 4.74 |
| | y | 3 | 100 | x | 57.63 | 3.14 | 0.38 | 0.11 | 11.00 |

Note: Used for placing top bars only.

### 1. Design of Slab-to-Footing Connections

Interconnected Slab at Sides:

Note : Slabs at sides are used only to reduce soil bearing pressure; footing is designed to take all loads.

Side : Left Right

t Inches (Slab Thickness)

X Feet (distance to other Slab Edge Support

$f'_c$ Ksi

Conn Type (D= Dowel, C= Continuous)

**RC Slab not Needed**

### 2. Lateral Resistance of Foundation

**Foundation OK for Sliding**

### 3. Soil Pressure due to Applied Loads

$\sigma_b$ = 1.00 Ksf
**OK**

Note: $\sigma_{allow}$ = 2.00 ksf (allowable bearing pressure)

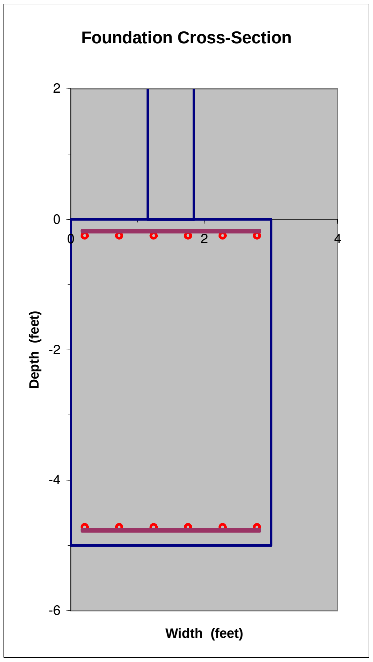
### 5. Adequacy of Footing - Shear

**Footing OK for Shear**

### 6. Adequacy of Footing - Flexure

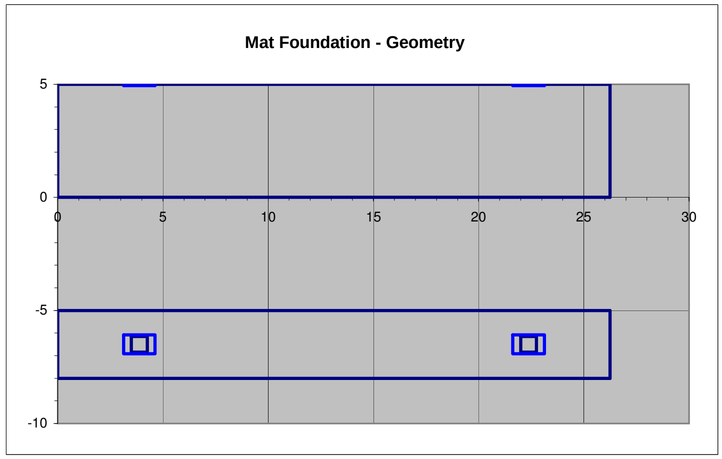
**6 - No. 8 Longitudinal Top Bars OK**

**6 - No. 8 Longitudinal Bottom Bars OK**



**SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE 1 (W-E EQ LOADS) - CASE N-1**
**DETERMINATION OF ADDITIONAL REINFORCEMENT REQUIRED - FIXED BASE COLUMN CONDITION**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 1. Parameters




Footing Size :

$L_x$ = 26.25 feet
$L_y$ = 3.00 feet
$h_f$ = 5.00 feet

Base Plate Dimensions:

**Note:** Base Plate design done elsewhere.

N = 18.00 inches (Base Plate - Length)
B = 10.00 inches (Base Plate - Width)
$t_{PL}$ = 0.75 inches (Base Plate - Thickness)

Column: W8x67

d = 9.00 inches (Wide Flange - Depth)
$b_f$ = 8.28 inches (Wide Flange - Width)
$t_f$ = 0.94 inches (Wide Flange - Thickness)
$Z_x$ = 70.1 $in^3$ (Wide Flange - Plastic Section)
A = 19.70 $in^2$ (Wide Flange - Area)
$F_y$ = 50 Ksi

Concrete :

$f'_c$ = 3.25 Ksi
$f_y$ = 60.00 Ksi
$\rho_c$ = 0.15 $kip/ft^3$

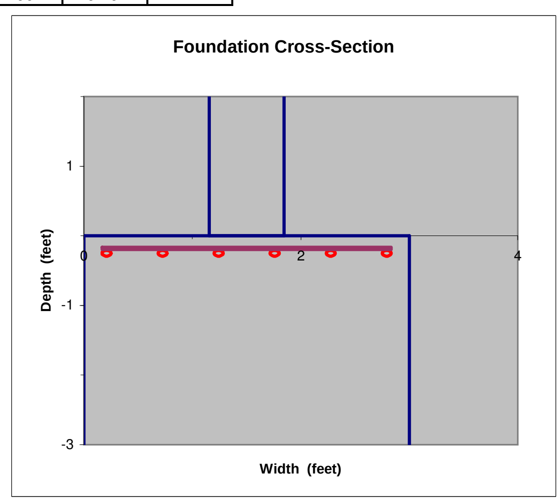
Reinforcement:

$d_c$ = 2.00 inches (bar clearance - top)
= 3.00 inches (bar clearance - bottom)
= 2.00 inches (bar clearance - sides)

| | Orientation | Bar Size | N Bars | Bottom Layer | d (inches) | Bar Spacing (inches) | Bar Diameter (inches) | Bar Area Per Bar (in^2) | Bar Area Total (in^2) |
|---|---|---|---|---|---|---|---|---|---|
| Top Mat | x | 8 | 6 | x | 57.00 | 6.20 | 1.00 | 0.79 | 4.74 |
| Bottom Mat | x | 8 | 6 | 0 | 55.63 | 6.20 | 1.00 | 0.79 | 4.74 |

Soil Parameters :

Soil density = 120 pcf
$\sigma_{allow}$ = 2.00 ksf (allowable bearing pressure)
$\sigma_p$ = 0.30 ksf/ft (Passive Soil Pressure)
$\mu$ = 0.25 ksf (Coefficient of Friction)




Design Parameters :

$\phi_v$ = 0.75 (Shear; ACI 318-11 9.3.2.3)
$\Omega$ = 3.00 (Overstrength Factor - SMRF)

**SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE 1 (W-E EQ LOADS) - CASE N-1**
**DETERMINATION OF ADDITIONAL REINFORCEMENT REQUIRED - FIXED BASE COLUMN CONDITION**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 2. Additional Required Reinforcement at Columns

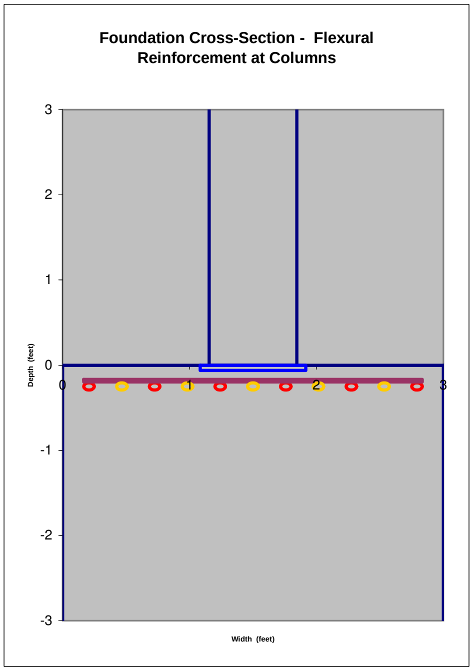
a) Column Probable Expected Flexural Capacity

$M_{FB}$ = 100% of Column Flexural Capacity

= $1.0\ Z_x\ F_y$  Where $Z_x$ = 70.1 $in^3$ (Wide Flange - Plastic Section)

$F_y$ = 50 Ksi

| | | |
|---|---|---|
| $M_{FB}$ = | 3,505 | Kip-in |
| = | 292.1 | Kip-ft |




b) Required Reinforcement Ratio (ACI 10.2)

$$\rho_r = \frac{0.85\ f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2\ M_u}{0.765\ b\ d^2\ f'_c}}\right]$$

Where $f'_c$ = 3.25 Ksi
$f_y$ = 60.00 Ksi

$M_u = M_F + M_{FB}$  for $M_F$ = 9,604 kip-in (Footing Flexural Demands)
$M_{FB}$ = 3,505 kip-in (Column Flexural Capacity)

$M_u$ = 13,109 kip-in

$b = L_y$ = 3.0 feet
= 36 inches

$d_x$ = 55.63 inches

| | |
|---|---|
| $\rho_r$ = | 0.00223 |

c) Reinforcement Ratio Provided

$\rho_w = A_{sx}/(L_y\ d_x)$  Where $A_{sx} = A_F + A_{FB}$  Where $A_F$ = 4.74 $in^2$ (Reinforcement Provided - Footing Flexure)

$A_{FB}$ = Reinforcement Required for Resisting Fixed Base Column Flexural Capacity

= $(N-1)\ A_b$  for N = 6 bars provided

$A_b$ = 0.79 $in^2$ for 8 bars

Note: db = 1.00 $in^2$ (Bar Diameter)

$A_{FB}$ = 3.95 $in^2$

| | | |
|---|---|---|
| $A_{sx}$ = | 8.69 | $in^2$ |

$L_y = L_B$ = 3.0 feet
= 36.0 inches
d = 55.63 inches

| | |
|---|---|
| $\rho_w$ = | 0.00434 |

(reinforcement ratio provided)  Note: D/C Ratio = 0.51 (Demand to Capacity Ratio - Flexure)

**OK**

**Use Additional 5 - # 8 Bars for Column Flexure with DC Ratio = 0.51**

**SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE 1 (W-E EQ LOADS) - CASE N-1**
**DETERMINATION OF ADDITIONAL REINFORCEMENT REQUIRED - FIXED BASE COLUMN CONDITION**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

d) Flexural reinforcement development length (ACI 12.2.2 and 12.2.5)

i) Development Length (ACI 12.2.2 - 12.2.4)

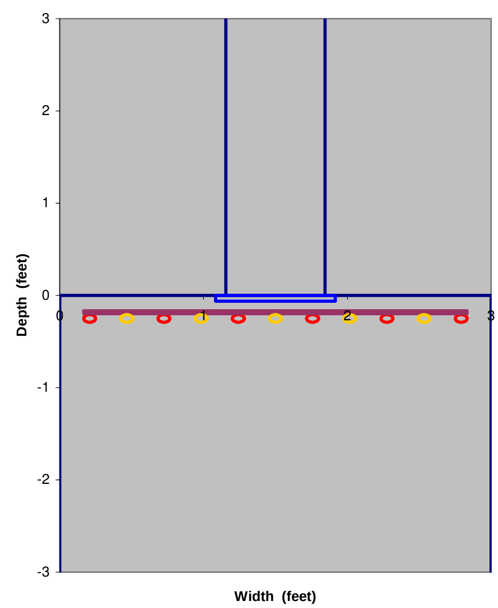
Bar Size = 8

$d_s$ = S - $d_b$ Where S = 3.13 inches (Bar spacing provided)
$d_b$ = 1.00 inches

$d_s$ = 2.13 inches inches (Clear spacing provided)

$d_c$ = 2.00 inches (Clear Cover provided)

**Foundation Cross-Section - Flexural Reinforcement at Columns**




| | Provided (inches) | Lower Limit | Upper Limit |
|---|---|---|---|
| **Clear Cover** | 2.00 | $d_b$ = 1.00 inches **OK** | 2 $d_b$ = 2.00 inches **NG** |
| **Clear Spacing** | 2.13 | 2 $d_b$ = 2.00 inches **OK** | 4 $d_b$ = 4.00 inches **NG** |
| | Equations | $l_d = \left(\frac{f_y \Psi_s \Psi_t \Psi_e \lambda}{25\sqrt{f'_c}}\right) d_b$ | $l_d = \frac{3}{40}\left(\frac{f_y \Psi_s \Psi_t \Psi_e \lambda}{2.5\sqrt{f'_c}}\right) d_b$ |
| | Values | $l_d$ = 52.62 $d_b$<br>$l_d$ = 52.6 inches | $l_d$ = 31.57 $d_b$<br>$l_d$ = 31.6 inches |

Note: Normal Weight Concrete with uncoated bars is assumed.

Where $f_y$ = 60.00 Ksi
$\Psi_s$ = 1.00 (ACI 12.2.4)

$\Psi_t = \Psi_e = \lambda$ = 1.00

$f'_c$ = 3.25 Ksi

$d_b$ = 1.00 inches

ii) Excess Reinforcement (ACI 12.2.5)

$l'_d = l_d\, \rho_r/\rho_w$ Where $l_d$ = 52.6 inches
$\rho_r$ = 0.0022 (required reinforcement ratio)
$\rho_w$ = 0.0043 (reinforcement ratio provided)

$l'_d$ = 27.1 inches (Required development length)

iii) Available Anchorage length

$L_{da} = x_f - d_{cs} > l'_d$ Where $x_f$ = 3.88 feet (Cantilever Length at Column Centerline)
= 46.50 inches

$d_{cs}$ = 2.00 inches (bar clearance - sides)

$L_{da}$ = 44.50 inches
**OK**

**Use Additional 5 - # 8 Bars for Column Flexure; Use 0 in Development Length beyond Ends of Base Plates**

**SPREAD FOOTING DESIGN - SMRF 1 AT GRIDLINE 1 (W-E EQ LOADS) - CASE N-1**
**DETERMINATION OF ADDITIONAL REINFORCEMENT REQUIRED - FIXED BASE COLUMN CONDITION**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 3. Foundation Demands at Fixed Base Columns

a) Column Fixed Base Plastic Shear Demands

$V_P = M_P / d$ Where $M_P = F_y Z_x$ for $F_y =$ 50 Ksi

$Z_x =$ 70.1 $in^3$ (Wide Flange - Plastic Section)

$\boxed{M_P = 3{,}505 \text{ Kip-in}}$

d = 55.63 inches (Efffective depth of footing)

$\boxed{V_P = 63.0 \text{ Kips}}$

b) Amplified Column Axial Demands - Overstrength Shear Demands on Foundation

$P_U =$ Max ( Abs ($P_1$), Abs ($P_2$) ) Where $P_1 =$ -43 Kips

$P_2 =$ 64 Kips

$\boxed{P_U = 63.5 \text{ Kips}}$

$V_O = \Omega\ P_U$ Where $\Omega =$ 3.00 (Overstrength Factor - SMRF)

$P_U =$ 63.5 Kips

$\boxed{V_O = 190.5 \text{ Kips}}$

c) Controlling Shear Demands on Foundation

$V_U =$ Max ( $V_P$, $V_O$ ) Where $V_P =$ 63.0 Kips (Column Plastic Shear)

$V_O =$ 190.5 Kips (Column Overstrength Demands)

$\boxed{V_U = 190.5 \text{ Kips}}$

## 4. Foundation Capacity at Fixed Base Columns

a) Shear Strength provided by Concrete (ACI 11.3.1.1)

$V_C = 2\ f'_c{}^{0.5}\ b_w\ d$ Where $f'_c =$ 3.25 Ksi

= 3,250 psi

$b = L_y =$ 3.0 feet

= 36.0 inches

d = 55.63 inches (Efffective depth of footing)

$\boxed{V_C = 171 \text{ kips}}$

b) Shear Strength provided by Shear Reinforcement (ACI 11.3.1.1)

Note: Assume transverse flexural reinforcement provided for footing is part of a reinrforcement cage.

$V_S = A_S\ F_y\ d\ /\ S$ (11-15) $A_S =$ 0.22 for No. 3 bars => hoops (from Footing Design)

$\leq 4\ V_C$ $F_y =$ 50 Ksi

d = 55.63 inches (Efffective depth of footing)

S = 3.14 inches (bar spacing - from Footing Design)

$\boxed{V_S = 195.0 \text{ kips}}$

b) Factored Shear Capacity of Footing (ACI 11.1)

$\phi\ V_n = \phi\ (V_C + V_S)$ Where $\phi =$ 0.75 (Shear; ACI 318-11 9.3.2.3)

$V_C =$ 171 kips

$V_S =$ 195.0 kips

$\boxed{\phi\ V_n = 274.7 \text{ Kips}}$ Note: $V_U =$ 190.5 Kips

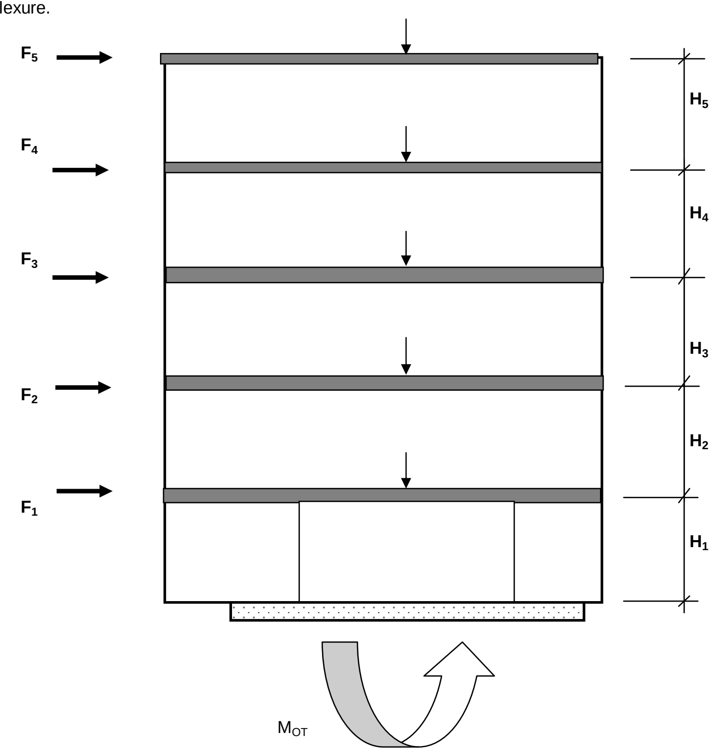
**OK**

**Footing OK for Shear**

**FOOTING DESIGN CASE N-2 - SHEAR WALL AT GRIDLINE 4 (W-E LOADS)**
**DETERMINATION OF VERTICAL AND LATERAL LOADS TO FOUNDATION**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Assumptions**

1. Existing walls are treated as pinned Columns. Footing is assumed rigid.
2. Column loads are located in transverse center of footing; limit of 2 columns w/o flexure.
3. Footing has no shear reinforcement.
4. Concrete is Normal Weight Concrete with uncoated bars.




**1. Lateral Loads and Load Effects**

V = 10.00 kips (Base Shear - A4 **ASD**)

| Floor Level | Height (feet) | Loading ID | $V_x$ / V | Shear (Kips) | Force (Kips) | Moment (Kip-ft) |
|---|---|---|---|---|---|---|
| | | 5 | | | | |
| R | 11.00 | 4 | 0.30 | 3.00 | 3.00 | 33.00 |
| 3 | 10.00 | 3 | 0.65 | 6.50 | 3.50 | 68.00 |
| 2 | 13.50 | 2 | 0.89 | 8.90 | 2.40 | 100.40 |
| 1 | 6.00 | 1 | 1.00 | 10.00 | 1.10 | 107.00 |

**$M_{OT}$ = 107.00 Kip-ft**

**2. Vertical Loads and Load Effects**

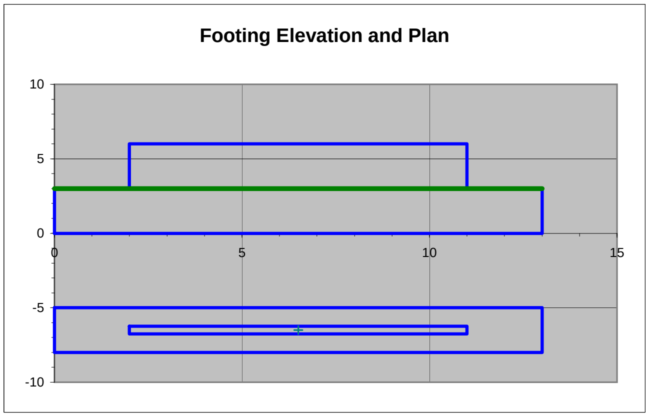
| | | Floor Tributary Loads | | | | | Wall Tributary Loads | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Wall** | Floor Level | DL (psf) | Length (feet) | Width (feet) | Area (ft$^2$) | Weight (kips) | WL (psf) | Length (feet) | Height (feet) | Area (ft$^2$) | Weight (kips) |
| | | | | | | | | | | | |
| **1** | R | 20 | 9.00 | 2.00 | 18 | 0.36 | 10 | 9.00 | 11.00 | 99 | 0.99 |
| | 3 | 30 | 9.00 | 2.00 | 18 | 0.54 | 10 | 9.00 | 10.00 | 90 | 0.90 |
| | 2 | 30 | 9.00 | 2.00 | 18 | 0.54 | 10 | 9.00 | 13.50 | 122 | 1.22 |
| | 1 | 30 | 9.00 | 2.00 | 18 | 0.54 | 10 | 9.00 | 6.00 | 54 | 0.54 |

Sum of Floor Weight = 1.98 Kips

Sum of Wall Weight = 3.65 Kips

**$P_{D1}$ = 5.63 Kips**

**FOOTING DESIGN CASE N-2 - SHEAR WALL AT GRIDLINE 4 (W-E LOADS)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**



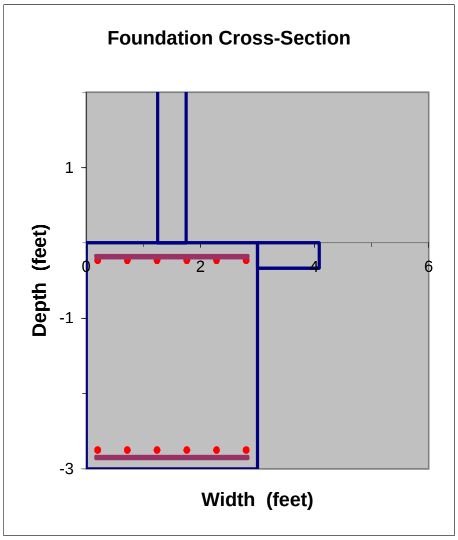

**Assumptic** 1. Footing has no shear reinforcement.
2. Concrete is Normal Weight Concrete with uncoated bars.

## Footing Parameters :

Footing Size :
$L_x$ = 13.0 feet — $d_s$ = 0.0 feet (depth of soil)
$L_y$ = 3.0 feet
$h_f$ = 3.0 feet

Column Location :
$x_c$ = 6.5 feet (Column centerline distance from Left Edge)
$y_c$ = 1.5 feet (Column centerline distance from Bottom Edge)

Column Size :
$C_x$ = 9.0 feet (column length)
$C_y$ = 0.5 feet (column width)




Interconnected Slab at Sides:

Note : Slabs at sides are used only to reduce soil bearing pressure; footing is designed to take all loads.

| Side : | Left | Right | |
|---|---|---|---|
| t | | 4.00 | Inches (Slab Thickness) |
| X | | 20.00 | Feet (distance to other Slab Edge Support |
| f'c | | 2.50 | Ksi |
| Conn Type | | D | (D= Dowel, C= Continuous) |

Concrete :
$f'_c$ = 3.25 Ksi
$f_y$ = 60.00 Ksi
$\rho_c$ = 0.150 kip/ft^3

Reinforcement:
$d_c$ = 2.00 inches (bar clearance - top)
$d_c$ = 3.00 inches (bar clearance - bottom)
= 2.00 inches (bar clearance - sides)

| | Orientation | Bar Size | N Bars | Bottom Layer | d (inches) | Bar Spacing (inches) | | Bar Diameter (inches) | Bar Area Per Bar (in^2) | Bar Area Total (in^2) |
|---|---|---|---|---|---|---|---|---|---|---|
| Top Mat | x | 6 | 6 | x | 32.25 | 6.25 | | 0.75 | 0.44 | 2.64 |
| | y | 3 | 30 | | 32.25 | 5.23 | | 0.38 | 0.11 | 3.30 |
| Bottom Mat | x | 6 | 6 | | 31.88 | 6.25 | | 0.75 | 0.44 | 2.64 |
| | y | 3 | 30 | x | 32.63 | 5.23 | | 0.38 | 0.11 | 3.30 |

## 1. Design of Slab-to-Footing Connections

**Use No. 4 bars @ 8.00 inches on-center for Slab-to-Footing Connections**

## 2. Lateral Resistance of Foundation

**Foundation OK for Sliding**

## 3. Soil Pressure due to Applied Loads

**Footing Bearing stress OK**

## 5. Adequacy of Footing - Shear

**Footing OK for Shear**

## 6. Adequacy of Footing - Flexure

**6 - # 6 Bars OK for Longitudinal Flexure**

**30 - # 3 Bars OK for Transverse Flexure**

**FOOTING DESIGN CASE N-2 - SHEAR WALL AT GRIDLINE 4 (W-E LOADS)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

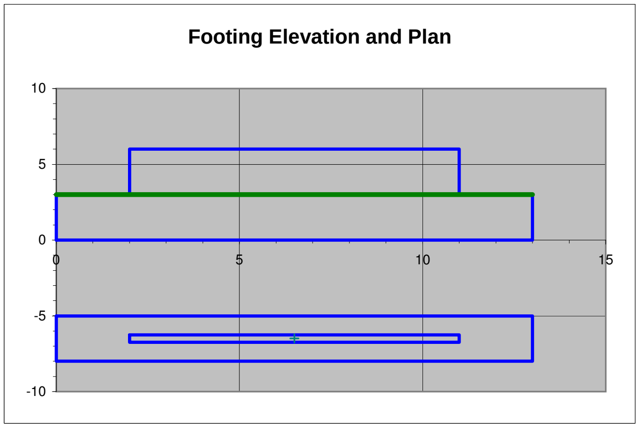
**Assumptions :**
1. Footing has no shear reinforcement.
2. Concrete is Normal Weight Concrete with uncoated bars.

## Footing Parameters :

Footing Size :
$L_x$ = 13.0 feet
$L_y$ = 3.0 feet
$h_f$ = 3.0 feet
$d_s$ = 0.0 feet (depth of soil)

Wall Location :
$x_c$ = 6.5 feet (Wall centerline distance from Left Edge)
$y_c$ = 1.5 feet (Wall centerline distance from Bottom Edge)

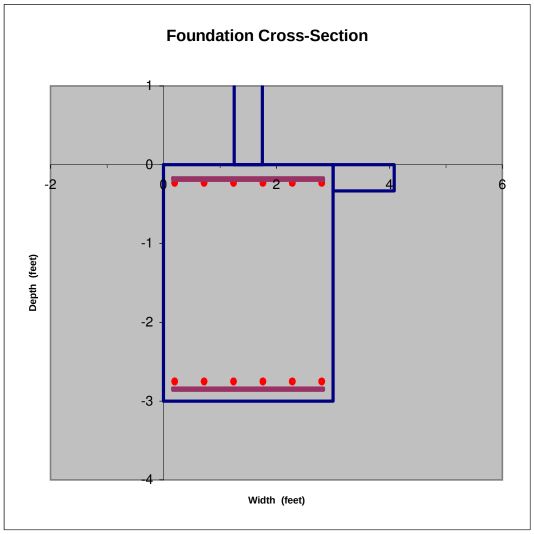
Wall Size :
$C_x$ = 9.0 feet (Wall length)
$C_y$ = 0.5 feet (Wall width)




Interconnected Slab at Sides:

Note : Slabs at sides are used only to reduce soil bearing pressure; footing is designed to take all loads.

| Side : | Left | Right | |
|---|---|---|---|
| t | | 4.00 | Inches (Slab Thickness) |
| X | | 20.00 | Feet (distance to other Slab Edge Support " |
| $f'_c$ | | 2.50 | Ksi |
| Conn Type | | D | (D= Dowel, C= Continuous) |




| Footing Loads : | Service | Strength | |
|---|---|---|---|
| P = | 5.6 | 7.9 | kips |
| $M_y$ = | 107.0 | 149.8 | kip-ft |
| $V_x$ = | 10.00 | 14.0 | kips |
| $M_x$ = | 1 | 1.4 | kip-ft |
| $V_y$ = | 0.1 | 0.1 | kips |

Plastic Hinge Centroidal Heights:
@ $h_{px}$ = 0.00 feet
@ $h_{py}$ = 0.00 feet

Capacity Factors :
$\phi_v$ = 0.75 (Shear)
$\phi_b$ = 0.65 (Bearing)
$\alpha$ = 40 (40 for interior columns, 30 for edge columns, 20 for corner columns)

Concrete :
$f'_c$ = 3.25 Ksi
$f_y$ = 60.00 Ksi
$\rho_c$ = 0.150 kip/ft^3

Reinforcement:
$d_c$ = 2.00 inches (bar clearance - top)
$d_c$ = 3.00 inches (bar clearance - bottom)
= 2.00 inches (bar clearance - sides)

| | Orientation | Bar Size | N Bars | Bottom Layer | d (inches) | Bar Spacing (inches) | Bar Diameter (inches) | Bar Area Per Bar (in^2) | Bar Area Total (in^2) |
|---|---|---|---|---|---|---|---|---|---|
| Top Mat | x | 6 | 6 | x | 32.25 | 6.25 | 0.75 | 0.44 | 2.64 |
| | y | 3 | 30 | | 32.25 | 5.23 | 0.38 | 0.11 | 3.30 |
| Bottom Mat | x | 6 | 6 | | 31.88 | 6.25 | 0.75 | 0.44 | 2.64 |
| | y | 3 | 30 | x | 32.63 | 5.23 | 0.38 | 0.11 | 3.30 |

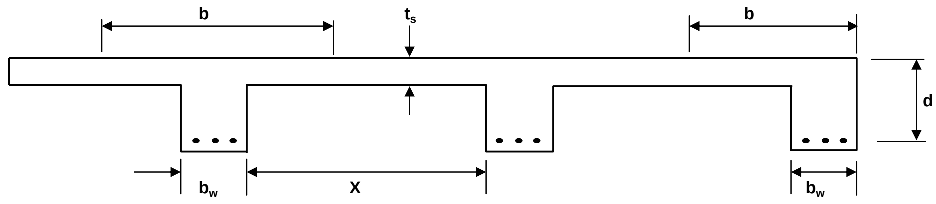
Note: Used for placing top bars only.

Soil Parameters :
$\rho_s$ = 120 pcf
$\sigma_{allow}$ = 2.02 ksf (allowable bearing pressure)
$\sigma_p$ = 0.30 ksf/ft (Passive Soil Pressure)
$\mu$ = 0.25 ksf (Coefficient of Friction)

**FOOTING DESIGN CASE N-2 - SHEAR WALL AT GRIDLINE 4 (W-E LOADS)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 1. Design of Slab-to-Footing Connections

a) Effective Slab Width (ACI 21.6.2.2)




i) Symmetrical T-Beams (ACI 8.12.2)

$b \leq$ Min ($L/4 + b_w$, $16\ t_s + b_w$, $X + b_w$)

Where L = feet (Footing span length)
= inches

$t_s$ = inches (Slab thickness - Average value)

$b_w = L_y$ = feet (Footing Width)
= inches

X = feet (Slab Supported Length - Average Value)
= inches

=

| b = | inches |
|---|---|

ii) Slabs on One Side (ACI 8.12.3)

$b \leq$ Min ($b_w + L/12$, $b_w + 6\ t_s$, $X/2 + b_w$)

Where $b_w = L_y$ = 3.00 feet (Footing Width)
= 36.00 inches

L = 13.00 feet (Footing span length)
= 156.00 inches

$t_s$ = 4.00 inches (Slab thickness)

X = 20.00 feet (Slab Supported Length)
= 240 inches

= Min (49.00, 60.00, 156.00)

| b = | 49.00 | inches |
|---|---|---|

| **b =** | **49.00** | **inches** |
|---|---|---|
| **=** | **4.08** | **feet** |

b) Required Slab Reinforcement Area

$A_{sr} = 0.0018\ b\ t_s$ (ACI 7.12.2.1)

Where b = 49.00 inches (Effective Slab Width)

$t_s$ = 4.0 inches (Slab thickness)

| $A_{sr}$ = | 0.35 | $in^2$ |
|---|---|---|

c) Required Slab Reinforcement Spacing

$S_{sr} = b / n_b \leq 2\ t_s$ (ACI 13.3.2)

Where b = 49.00 inches (Effective Slab Width)

$n_b$ = Ceiling ($A_{sr}/A_b$) for $A_{sr}$ = 0.35 $in^2$ (Slab Reinforcement Area - Required)

$A_b$ = 0.20 $in^2$ for No. 4 bars

= CEILING(MIN(24.50,8.00,1)

| $n_b$ = | 2 | bars |
|---|---|---|

$t_s$ = 4.0 inches
$2\ t_s$ = 8.0 inches

| $S_{sr}$ = | 8.00 | inches |
|---|---|---|

**Use No. 4 bars @ 8.00 inches on-center for Slab-to-Footing Connections**

**FOOTING DESIGN CASE N-2 - SHEAR WALL AT GRIDLINE 4 (W-E LOADS)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 2. Lateral Resistance of Foundation

2A. **Longitudinal Loading**

$F_{Rx} = 0.5\, L'_y\, h'^2_f\, \sigma_p + 0.6\,(W_f + P)\,\mu$

Where $L'_y = L_y + 2\, t_{nw}$ and $L_y$ = 3.0 feet

$t_{ew}$ = 3.00 feet (Thickness of (E) connected walls at ends)

$L'_y$ = 9.00 feet (Bearing Width at Ends of Footing)

$h'_f = h_f + h_{sk}$ and $h_f$ = 3.0 feet

$h_{sk}$ = 0.00 feet (Additional height of Shear Key at Footing E

$h'_f$ = 3.0 feet (Bearing Height at Ends of Footing)

= (40.5) (0.30) + 0.6 (23.2) (0.25)

$\sigma_p$ = 0.30 ksf/ft (Passive Soil Pressure)

= (12.15) + (3.48)

$W_f = \rho_c\, L_x\, L_y\, h_f$ and $\rho_c$ = 0.150 kip/ft^3

$L_x$ = 13.0 feet

$L_y$ = 3.0 feet

$h_f$ = 3.0 feet

$W_f$ = 17.55 Kips (Footing Weight)

P = 5.6 Kips (Service Load)

$\mu$ = 0.25 ksf (Coefficient of Friction)

$F_{Rx}$ = 15.63 kips

OK

Note : $V_x$ = 10.00 kips

**Foundation OK for Sliding**

## 3. Soil Pressure due to Applied Loads

3A. **Longitudinal Loading**

a) Loading Eccentricity

$e_x = \Sigma M_y / P'$

Where $\Sigma M_y = M_y + V_x H_{px} - P\,(0.5\, L_x - x_c)$ and $M_y$ = 107 kip-ft

$V_x$ = 10 kips

@ $h_{px}$ = 0.00 feet

P = 6 Kips

$L_x$ = 13.0 feet

= 107 kip-ft + 0 kip-ft - 0 kip-ft

$x_c$ = 6.5 feet (Column centerline distance from Left Edge)

$\Sigma M_y$ = 107 Kip-in

$P' = P + P_F$ and P = 6 Kips

$P_F = \rho_c\, L_x\, L_y\, h_f$ for $\rho_c$ = 0.150 kip/ft^3

$L_x$ = 13.0 feet

$L_y$ = 3.0 feet

$h_f$ = 3.0 feet

$P_F$ = 17.55 kips (footing weight)

P' = 23 Kips

$e_x$ = 4.62 feet

Note: $L_x/6$ = 2.17 feet (Footing Middle Third)

**FOOTING DESIGN CASE N-2 - SHEAR WALL AT GRIDLINE 4 (W-E LOADS)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

b) Bearing Stresses

i) for $e_x \le L_x/6$ (within Middle $L_x/3$)

$\sigma_{max} = P (1 + 6 e_x / L_x) / (L_x L_y)$ — Where P = 23 Kips

$\sigma_{min} = P (1 - 6 e_x / L_x) / (L_x L_y)$

$e_x$ = 4.62 feet
$L_x$ = 13.0 feet
$L_y$ = 3.0 feet

| | | |
|---|---|---|
| $\sigma_{max}$ = | 1.86 | Ksf |
| $\sigma_{min}$ = | -0.67 | Ksf |

ii) for $e_x > L_x/6$ (outside Middle $L_x/3$) **<= Governs!**

$\sigma_{max} = 2 P / (L_{bx} L_y)$ — Where P = 23 Kips

$L_{bx} = 3 (0.5 L_x - e_x)$ — and $L_x$ = 13.0 feet, $e_x$ = 4.62 feet

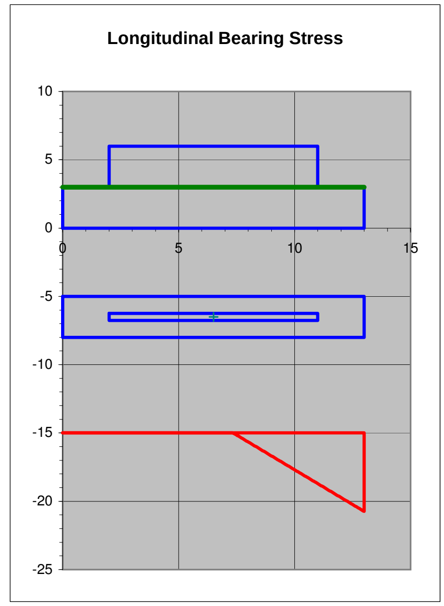
| | | |
|---|---|---|
| $L_{bx}$ = | 5.65 | feet |

$L_y$ = 3.0 feet

| | | |
|---|---|---|
| $\sigma_{max}$ = | 2.74 | Ksf |
| $\sigma_{min}$ = | 0.00 | Ksf |

iii) Governing Condition

| | | |
|---|---|---|
| $\sigma_{max}$ = | 2.74 | Ksf |
| $\sigma_{min}$ = | 0.00 | Ksf |




**3B. Transverse Loading**

a) Loading Eccentricity

$e_y = \Sigma M_x / P'$ — Where $\Sigma M_x = M_x + V_y H_{py} - P (0.5 L_y - y_c)$

and $M_x$ = 1 kip-ft
$V_y$ = 0 kips
@ $h_{py}$ = 0.00 feet
P = 6 Kips
$L_y$ = 3.0 feet
$y_c$ = 1.5 feet (Column centerline distance from Left Edge)

= 1 kip-ft + 0 kip-ft - 0 kip-ft

| | | |
|---|---|---|
| $\Sigma M_y$ = | 1 | Kip-in |

P' = 23 Kips

| | | |
|---|---|---|
| $e_y$ = | 0.04 | feet |

Note: $L_y/6$ = 0.50 feet (Footing Middle Third)

b) Bearing Stresses

i) for $e_y \le L_y/6$ (within Middle $L_y/3$) **<= Governs!**

$\sigma_{max} = P' (1 + 6 e_y / L_y) / (L_x L_y)$ — Where P' = 23 Kips

$\sigma_{min} = P' (1 - 6 e_y / L_y) / (L_x L_y)$

$e_y$ = 0.04 feet
$L_y$ = 3.0 feet
$L_x$ = 13.0 feet

| | | |
|---|---|---|
| $\sigma_{max}$ = | 0.65 | Ksf |
| $\sigma_{min}$ = | 0.54 | Ksf |

ii) for $e_x > L_x/6$ (outside Middle $L_x/3$)

$\sigma_{max} = 2 P' / (L_{by} L_x)$ — Where P' = 23 Kips

$L_{by} = 3 (0.5 L_y - e_y) \le L_y$ — and $L_y$ = 3.0 feet, $e_y$ = 0.04 feet

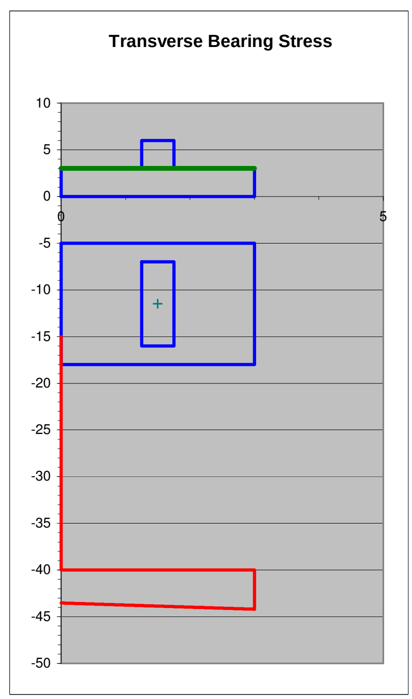
| | | |
|---|---|---|
| $L_{by}$ = | 3.00 | feet |

$L_x$ = 13.0 feet

| | | |
|---|---|---|
| $\sigma_{max}$ = | 1.19 | Ksf |
| $\sigma_{min}$ = | 0.00 | Ksf |

iii) Governing Condition

| | | |
|---|---|---|
| $\sigma_{max}$ = | 0.65 | Ksf |
| $\sigma_{min}$ = | 0.54 | Ksf |



**FOOTING DESIGN CASE N-2 - SHEAR WALL AT GRIDLINE 4 (W-E LOADS)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**3C. Check of Bearing Stresses on Soil**

a) Assuming all loads resisted by Footing only

$\sigma_{max}$ = Max($\sigma_{max,x}$,$\sigma_{max,y}$) Where $\sigma_{max}$ = 2.74 Ksf (Longitudinal Direction)

= 0.65 Ksf (Transverse Direction)

b) Assuming all loads resisted by Footing + Adjoining Slabs

$\sigma_{max}$ = Max($\sigma_{max,x}$,$\sigma_{max,y}$) * $L_y$ / b Where $\sigma_{max}$ = 2.74 Ksf (Longitudinal Direction) and $L_y$ = 3.0 feet

= 0.65 Ksf (Transverse Direction) b = 49.00 inches

= ( 2.74 * 0.73 ) = 4.08 feet

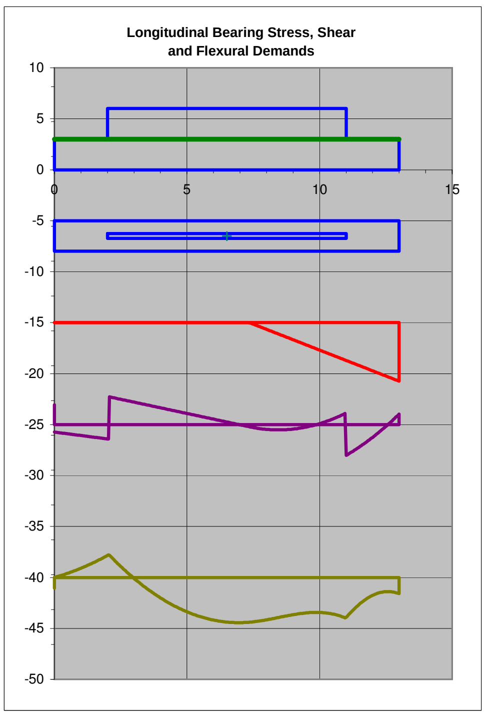
$\sigma_{max}$ = 2.01 Ksf

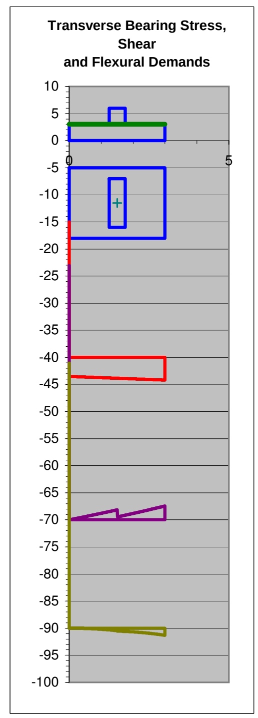
OK

Note: $\sigma_{allow}$ = 2.02 ksf (allowable bearing pressure)

Footing Bearing stress OK

## 4. Applied Loading and Demands on Footing




| | Longitudinal Direction | | | | |
|---|---|---|---|---|---|
| | Left End | Left Face of Wall | Wall Centerline | Right Face of Wall | Right End |
| Location (feet) | 0 | 2.00 | 6.50 | 11.00 | 13.00 |
| $\sigma$ = $q_u$/$L_y$ (ksf) | 0.00 | 0.00 | 0.00 | 2.45 | 3.83 |
| P (kips) | - | - | 8 | - | - |
| $M_y$ + V $h_{px}$ (kip-ft) | - | - | 150 | - | - |
| V+ (kips) | - | 11 | 1 | 4 | 4 |
| V- (kips) | -3 | -6 | - | -12 | - |
| M+ (kip-ft) | 0 | 9 | - | - | - |
| M- (kip-ft) | - | - | -18 | -15 | -6 |

| | Transverse Direction | | | | |
|---|---|---|---|---|---|
| | Left End | Left Face of Column | Column Centerline | Right Face of Column | Right End |
| Location (feet) | 0.00 | 1.25 | 1.50 | 1.75 | 3.00 |
| $\sigma$ = $q_u$/$L_y$ (ksf) | 0.54 | 0.59 | 0.59 | 0.60 | 0.65 |
| P (kips) | - | - | 8 | - | - |
| $M_y$ + V $h_{px}$ (kip-ft) | - | - | 1 | - | - |
| V+ (kips) | 0 | 9 | 11 | 5 | 15 |
| V- (kips) | - | - | - | - | - |
| M+ (kip-ft) | 0 | - | - | - | - |
| M- (kip-ft) | - | -6 | -8 | -11 | -23 |

**FOOTING DESIGN CASE N-2 - SHEAR WALL AT GRIDLINE 4 (W-E LOADS)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 5. Adequacy of Footing - Shear

### A. Flexural/One-Way Shear

a) Longitudinal Direction

Shear demands: $V_{ux}$ = 10.8 Kips @ $x_L$ = 2.00 feet (locations at distance d from face of Wall - Left side)
= 11.8 Kips @ $x_R$ = 11.00 feet ( - Right side)

=> $V_{ux}$ = 11.8 Kips

ii) Shear capacity of concrete without shear reinforcement

$$\varphi V_c = \varphi\left(1.9\sqrt{f'_c} + 2500\,\rho_w \frac{V_u\,d}{M_u}\right) b_w\,d \;\le\; 3.5\sqrt{f'_c}\,b_w\,d \qquad (11\text{-}5)$$

Note: $V_u\,d/M_u$ value must be ≤ 1.0

Where $\phi$ = 0.75
$f'_c$ = 3,250 psi

$\rho_w = A_{sx}/(L_y\,d_x)$ and $A_{sx}$ = 2.64 in^2
$L_y$ = 3.00 feet
= 36.0 inches
d = 31.88 inches

$\rho_w$ = 0.002301

$V_u$ = 12 kips
d = 31.88 inches
$M_u$ = 15 Kip-ft @ $V_{ux}$ = 12 Kips (location of shear value)

Check of $V_u\,d/M_u$ value limit:
$V_u\,d/M_u$ = 2.07 where Vu = 12 kips
**NG, value taken as unity.**
d = 31.88 inches
$M_u$ = 15 kip-ft
= 182 kip -in

$b_w = L_2$ = 3.0 feet
= 36.0 inches
d = 31.88 inches

=> $\phi\,V_c$ = 98.2 kips

Comparison w/ Equation 11-3:
$\phi\,V_c = \phi\,2\,f'^{0.5}_c\,b_w\,d$
Where $\phi$ = 0.75
$f'_c$ = 3,250 psi
$b_w = L_2$ = 36.0 inches
d = 31.88 inches

$\phi\,V_c$ = 98.1 kips

Check of upper value limit:
$\phi\,V_{c,max} = 3.5\,f'^{0.5}_c\,b_w\,d$
Where $f'_c$ = 3,250 psi
$b_w = L_2$ = 36.0 inches
d = 31.88 inches

$\phi\,V_{c,max}$ = 229.0 kips

$\phi\,V_c$ = 98.2 kips
**OK, > Vu**

**FOOTING DESIGN CASE N-2 - SHEAR WALL AT GRIDLINE 4 (W-E LOADS)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

b) Transverse Direction

Note: approach is for concrete column, not column or pedestal with steel base plates.

Shear demands: $V_{uy}$ = 9 Kips @ $x_L$ = 1.25 feet (locations at distance d from face of column - Left side)
= 5 Kips @ $x_R$ = 1.75 feet ( - Right side)

=> $V_{uy}$ = 9 Kips

ii) Shear capacity of concrete without shear reinforcement

$$\varphi V_c = \varphi\left(1.9\sqrt{f'_c} + 2500\,\rho_w \frac{V_u d}{M_u}\right) b_w d \;\leq\; 3.5\sqrt{f'_c}\, b_w d \qquad (11\text{-}5)$$

Note: $V_u d/M_u$ value must be ≤ 1.0

Where $\phi$ = 0.75
$f'_c$ = 3,250 psi

$\rho_w = A_{sy}/(L_x d_y)$ and $A_{sy}$ = 3.30 in^2

$L_x$ = 13.00 feet
= 156.0 inches

$d_y$ = 32.63 inches

$\rho_w$ = 0.000648

$V_u$ = 9 kips
d = 32.63 inches

$M_u$ = 11 Kip-ft @ $V_{ux}$ = 9 Kips (location of shear value)

Check of $V_u$ d/$M_u$ value limit:

$V_u$ d/$M_u$ = 2.33 where Vu = 9 kips
**Value taken as unity.** $d_y$ = 32.63 inches
$M_u$ = 11 kip-ft
= 127 kip -in

$b_w = L_x$ = 13.0 feet
= 156.0 inches

$d_y$ = 32.63 inches

=> $\phi V_c$ = 420 kips

Comparison w/ Equation 11-3:

$\phi V_c = 2 f'^{0.5}_c b_w d$

Where $\phi$ = 0.75
$f'_c$ = 3,250 psi
$b_w = L_x$ = 156.0 inches
$d_y$ = 32.63 inches

$\phi V_c$ = 435 kips

Check of upper value limit:

$\phi V_{c,max} = 3.5 f'^{0.5}_c b_w d$

Where $f'_c$ = 3,250 psi
$b_w = L_2$ = 156.0 inches
$d_y$ = 32.63 inches

$\phi V_{c,max}$ = 1,016 kips

$\phi V_c$ = 420 kips
**OK, > Vu**

**FOOTING DESIGN CASE N-2 - SHEAR WALL AT GRIDLINE 4 (W-E LOADS)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**B. Punching/Two Way Shear**

Sources: ACI 11.5.5.2, 11.12.1.2

a) Failure Perimeter

$b_0 = 2\,(b_1 + b_2)$ Where $b_1 = C_x + d_x$ and $C_x =$ 108.00 inches

$d_x =$ 31.88 inches

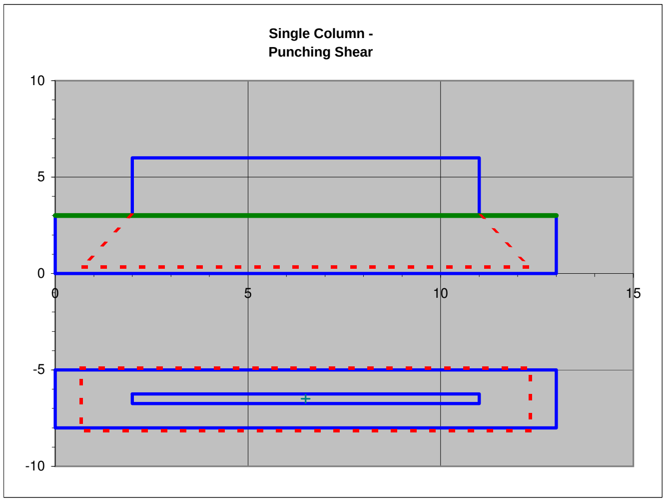
$b_1 =$ 139.88 inches

$b_2 = C_y + d_y$ and $C_y =$ 6.00 inches

$d_y =$ 32.63 inches

$b_2 =$ 38.63 inches

$b_0 =$ 357.0 inches




b) Shear Demands

$V_u = 1.4\,\sigma_{CL}\,(L_x L_y - b_1 b_2)$ Where $\sigma_{CL} =$ 0.59 Ksf

$L_x =$ 13.00 feet

$L_y =$ 3.00 feet

$b_1 =$ 139.88 inches

= 11.66 feet

$b_2 =$ 38.63 inches

= 3.22 feet

$V_u =$ 1.2 kips

c) Factored Shear Capacity (ACI 11.12.2.1)

$$\varphi\,V_c = \varphi\,\mathrm{Min}\left[2 + \frac{4}{\beta},\ \frac{\alpha\,d}{b_o} + 2,\ 4\right]\sqrt{f'_c}\ b_o\ d$$

Where $\phi =$ 0.75

$\beta = C_{max}/C_{min}$ and $C_{max} =$ 108.00 inches

$C_{min} =$ 6.00 inches

$\beta =$ 18.00

$\alpha =$ 40 (40 for interior columns, 30 for edge columns, 20 for corner columns)

$d = \mathrm{Min}(d_x, d_y, L_y)$ $d_x =$ 31.88 inches

$d_y =$ 32.63 inches

$L_y =$ 36.0 inches

$d =$ 31.88 inches

$b_0 =$ 357.0 inches

$f'_c =$ 3,250 psi

$\phi\,V_c =$ 1,081 kips

**OK, > Vu**

**Footing OK for Shear**

**FOOTING DESIGN CASE N-2 - SHEAR WALL AT GRIDLINE 4 (W-E LOADS)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 6. Adequacy of Footing - Flexure

### A. Longitudinal Flexure

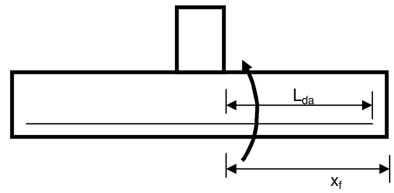
a) Flexural demands

$M_{ux}$ = 9 Kip-ft @ $x_L$ = 2.00 feet (locations at face of column - Left side)
= 104 Kip-in — Note: $X_f$ = 2.00 feet (Cantilever Length)

= 15 Kip-ft @ $x_R$ = 11.00 feet ( - Right side)
= 182 Kip-in — Note: $X_f$ = 2.00 feet (Cantilever Length)

=> $\boxed{M_{ux} = 182 \text{ Kip-in}}$

b) Required Reinforcement Ratio (ACI 10.2)




$$\rho_r = \frac{0.85\, f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2\, M_u}{0.765\, b\, d^2\, f'_c}}\right]$$

Where $f'_c$ = 3.25 Ksi
$f_y$ = 60.00 Ksi
$M_u$ = 182 kip-in
$b = L_x$ = 3.0 feet
= 36 inches
$d_x$ = 31.88 inches

$\boxed{\rho_r = 0.0001}$ — Note: $\rho_w$ = 0.0023 (reinforcement ratio provided)
**OK**

c) Maximum Reinforcement Ratio (ACI 10.3.4)

$$\rho_t = 0.85\, \beta_1 \frac{f'_c}{f_y}\left[\frac{\varepsilon_c}{\varepsilon_c + \varepsilon_s}\right]$$

Where $f'_c$ = 3.25 Ksi
$f_y$ = 60.00 Ksi

$\beta_1$ = 0.85 — for $f'_c \le 4.0$ Ksi
= 0.85 - 0.05 ($f'_c$ - 4.0), ≥ 0.65 — for $f'_c > 4.0$ Ksi

$\boxed{\beta_1 = 0.85}$

$\varepsilon_c$ = 0.003 (ACI Section 10.3.4)
$\varepsilon_s$ = 0.005

$\boxed{\rho_t = 0.0147}$ — Note: $\rho_w$ = 0.0023 (reinforcement ratio provided)
**OK, Tension controlled section**

d) Minimum Reinforcement Area (ACI 7.12.2.1)

$A_{min} = \rho_{min}\, A_g$

Where $\rho_{min}$ = 0.002 — for $f_y \le 50.0$ Ksi
0.0018 — for $f_y = 60.0$ Ksi
0.0018 60/$f_y$ — for $f_y \ge 60.0$ Ksi

$f_y$ = 60.00 Ksi

$\boxed{\rho_{min} = 0.0018}$

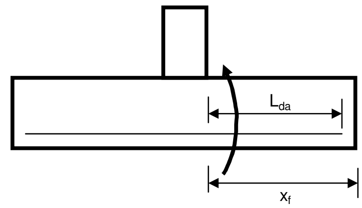
$A_g = L_y\, h_f$ — $L_y$ = 3.0 feet
= 36 inches
$h_f$ = 3.00 feet
= 36.00 inches

$\boxed{A_g = 1{,}296 \text{ in}^2}$

$\boxed{A_{min} = 2.33 \text{ in}^2}$ — Note: $A_{s1}$ = 2.64 in^2 (reinforcement provided)
**OK**

**FOOTING DESIGN CASE N-2 - SHEAR WALL AT GRIDLINE 4 (W-E LOADS)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

e) Flexural reinforcement development length (ACI 12.2.2 and 12.2.5)




i) Development Length (ACI 12.2.2 - 12.2.4)

Bar Size = 6

$d_s$ = S - $d_b$ Where S = 6.25 inches (Bar spacing provided)
$d_b$ = 0.75 inches

$d_s$ = 5.50 inches — inches (Clear spacing provided)

$d_c$ = 3.00 inches (Clear Cover provided)

| | Provided (inches) | Lower Limit | Upper Limit |
|---|---|---|---|
| **Clear Cover** | 3.00 | $d_b$ = 0.75 inches OK | 2 $d_b$ = 1.50 inches OK |
| **Clear Spacing** | 5.50 | 2 $d_b$ = 1.5 inches OK | 4 $d_b$ = 3 inches OK |
| | Equations | $l_d = \left( \frac{f_y \Psi_s \Psi_t \Psi_e \lambda}{25 \sqrt{f'_c}} \right) d_b$ | $l_d = \frac{3}{40} \left( \frac{f_y \Psi_s \Psi_t \Psi_e \lambda}{2.5 \sqrt{f'_c}} \right) d_b$ |
| | Values | $l_d$ = 42.10 $d_b$<br>$l_d$ = 31.6 inches | $l_d$ = 25.26 $d_b$<br>$l_d$ = 18.9 inches |

Note: Normal Weight Concrete with uncoated bars is assumed.

Where $f_y$ = 60.00 Ksi
$\Psi_s$ = 0.80 (ACI 12.2.4)
$\Psi_t = \Psi_e = \lambda$ = 1.00
$f'_c$ = 3.25 Ksi
$d_b$ = 0.75 inches

ii) Excess Reinforcement (ACI 12.2.5)

$l'_d = l_d \, \rho_r / \rho_w$ Where $l_d$ = 18.9 inches
$\rho_r$ = 0.0001 (required reinforcement ratio)
$\rho_w$ = 0.0023 (reinforcement ratio provided)

$l'_d$ = 0.8 inches (Required development length)

iii) Available Anchorage length

$L_{da} = x_f - d_{cs}$ > $l'_d$ Where $x_f$ = 2.00 feet (Cantilever Length at $M_{ux}$)
= 24.00 inches
$d_{cs}$ = 2.00 inches (bar clearance - sides)

$L_{da}$ = 22.00 inches
OK

6 - # 6 Bars OK for Longitudinal Flexure

**B. Transverse Flexure**

a) Flexural demands $M_{ux}$ = 6 Kip-ft @ $x_L$ = 1.25 feet (locations at face of column - Left side)
= 67 Kip-in Note: $X_f$ = 1.25 feet (Cantilever Length)
= 11 Kip-ft @ $x_R$ = 1.75 feet ( - Right side)
= 127 Kip-in Note: $X_f$ = 1.25 feet (Cantilever Length)

=> $M_{ux}$ = 127 Kip-in

b) Required Reinforcement Ratio (ACI 10.2)

$$\rho_r = \frac{0.85 f'_c}{f_y} \left[ 1 - \sqrt{1 - \frac{2 M_u}{0.765 \, b \, d^2 \, f'_c}} \right]$$

Where $f'_c$ = 3.25 Ksi
$f_y$ = 60.00 Ksi
$M_u$ = 127 kip-in
$b = L_x$ = 13.0 feet
= 156 inches
$d_y$ = 32.63 inches

$\rho_r$ = 0.00001

c) Required Reinforcement Area

$A_{req} = \rho_r L_1 d_2$ Where $\rho_r$ = 0.00001
$L_x$ = 13.0 feet
= 156.0 inches
$d_y$ = 32.63 inches

$A_{req}$ = 0.07 in^2 Note: $A_{s1}$ = 3.30 in^2 (reinforcement provided)
OK

**FOOTING DESIGN CASE N-2 - SHEAR WALL AT GRIDLINE 4 (W-E LOADS)**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

d) Flexural reinforcement development length (ACI 12.2.2 and 12.2.5)

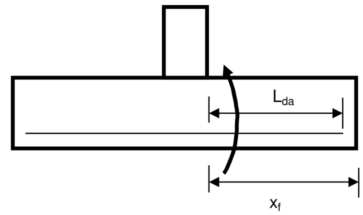
i) Development Length (ACI 12.2.2 - 12.2.4)

Bar Size = 3

$d_s$ = S - $d_b$ Where S = 5.23 inches (Bar spacing provided)
$d_b$ = 0.38 inches

$d_s$ = 4.85 inches | inches (Clear spacing provided)

$d_c$ = 3.00 inches (Clear Cover provided)




| | Provided (inches) | Lower Limit | Upper Limit |
|---|---|---|---|
| **Clear Cover** | 3.00 | $d_b$ = 0.38 inches **OK** | 2 $d_b$ = 0.75 inches **OK** |
| **Clear Spacing** | 4.85 | 2 $d_b$ = 0.75 inches **OK** | 4 $d_b$ = 1.5 inches **OK** |
| | Equations | $l_d = \left( \frac{f_y\ \Psi_s\ \ \Psi_t\ \Psi_e\ \lambda}{25\sqrt{f'_c}} \right) d_b$ | $l_d = \frac{3}{40} \left( \frac{f_y\ \Psi_s\ \Psi_t\ \Psi_e\ \lambda}{2.5\sqrt{f'_c}} \right) d_b$ |
| | Values | $l_d$ = 42.10 $d_b$<br>$l_d$ = 31.6 inches | $l_d$ = 25.26 $d_b$<br>$l_d$ = 18.9 inches |

Note: Normal Weight Concrete with uncoated bars is assumed.

Where $f_y$ = 60.00 Ksi
$\Psi_s$ = 0.80 (ACI 12.2.4)
$\Psi_t = \Psi_e = \lambda$ = 1.00
$f'_c$ = 3.25 Ksi
$d_b$ = 0.75 inches

ii) Excess Reinforcement (ACI 12.2.5)

$l'_d = l_d\ \rho_r/\rho_w$ Where $l_d$ = 18.9 inches
$\rho_r$ = 0.0000 (required reinforcement ratio)
$\rho_w$ = 0.0006 (reinforcement ratio provided)

$l'_d$ = 0.42 inches | (Required development length)

iii) Available Anchorage length

$L_{da} = x_f - d_{cs}$ $> l'_d$ Where $x_f$ = 1.25 feet
= 15.00 inches

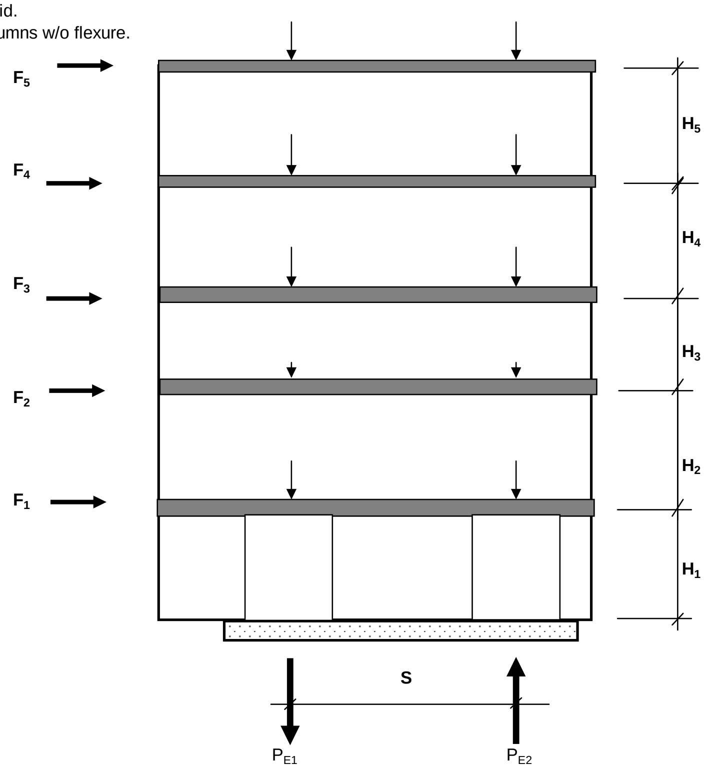
$d_{cs}$ = 2.00 inches (bar clearance - sides)

$L_{da}$ = 13.00 inches
**OK**

**30 - # 3 Bars OK for Transverse Flexure**

**SPREAD FOOTING DESIGN - SMRF's 2-3 AT GRIDLINE 6 (W-E EQ LOADS) - CASE N-3**
**DETERMINATION OF VERTICAL AND LATERAL LOADS TO FOUNDATION**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Assumptions**
1. Existing walls are treated as pinned Columns. Footing is assumed rigid.
2. Column loads are located in transverse center of footing; limit of 2 columns w/o flexure.
3. Footing has no shear reinforcement.
4. Concrete is Normal Weight Concrete with uncoated bars.




**1. Lateral Loads and Load Effects**

V = 19.60 kips (Base Shear - A4 **ASD**)

S = 25.00 feet (Separation between Wall centerlines)

| Floor Level | Height (feet) | Loading ID | $V_x$ / V | Shear (Kips) | Force (Kips) |
|---|---|---|---|---|---|
| | | 5 | | | |
| R | 11.00 | 4 | 0.65 | 12.74 | 12.74 |
| 3 | 10.00 | 3 | 0.89 | 17.44 | 4.70 |
| 2 | 13.50 | 2 | 1.00 | 19.60 | 2.16 |
| 1 | 2.00 | 1 | 0.00 | 0.00 | 0.00 |

From summation of moments :

**$P_{E1}$ = -24.74 Kips**
**$P_{E2}$ = 24.74 Kips**

**2. Vertical Loads and Load Effects**

| | | Floor Tributary Loads | | | | | Wall Tributary Loads | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Wall** | Floor Level | DL (psf) | Length (feet) | Width (feet) | Area ($ft^2$) | Weight (kips) | WL (psf) | Length (feet) | Height (feet) | Area ($ft^2$) | Weight (kips) |
| | | | | | | | | | | | |
| **1** | R | 20 | 25.00 | 2.00 | 50 | 1.00 | 20 | 25.00 | 11.00 | 275 | 5.50 |
| | 3 | 30 | 25.00 | 2.00 | 50 | 1.50 | 20 | 25.00 | 13.50 | 338 | 6.75 |
| | 2 | 30 | 25.00 | 2.00 | 50 | 1.50 | 20 | 25.00 | 13.50 | 338 | 6.75 |
| | 1 | 0 | 25.00 | 2.00 | 50 | 0.00 | 0 | 25.00 | 2.00 | 50 | 0.00 |

Sum of Floor Weight = 4.00 Kips    Sum of Wall Weight = 19.00 Kips

**$P_{D1}$ = 23.00 Kips**

| | | Floor Tributary Loads | | | | | Wall Tributary Loads | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Wall** | Floor Level | DL (psf) | Length (feet) | Width (feet) | Area ($ft^2$) | Weight (kips) | WL (psf) | Length (feet) | Height (feet) | Area ($ft^2$) | Weight (kips) |
| | | | | | | | | | | | |
| **2** | R | 20 | 25.00 | 2.00 | 50 | 1.00 | 20 | 25.00 | 11.00 | 275 | 5.50 |
| | 3 | 30 | 25.00 | 2.00 | 50 | 1.50 | 20 | 25.00 | 13.50 | 338 | 6.75 |
| | 2 | 30 | 25.00 | 2.00 | 50 | 1.50 | 20 | 25.00 | 13.50 | 338 | 6.75 |
| | 1 | 0 | 25.00 | 2.00 | 50 | 0.00 | 0 | 25.00 | 2.00 | 50 | 0.00 |

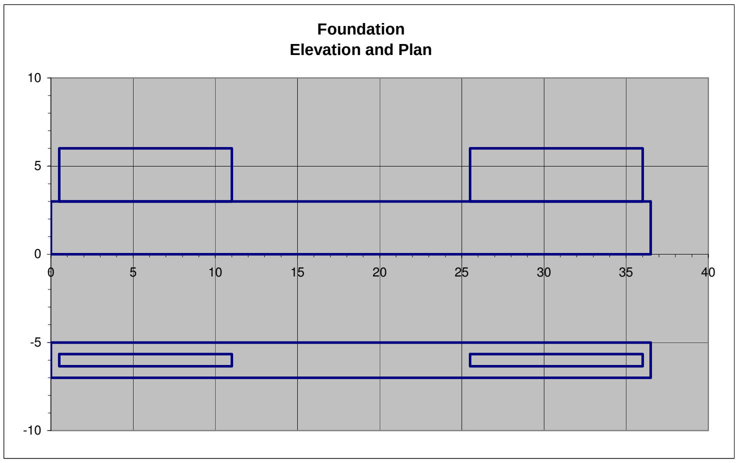
Sum of Floor Weight = 4.00 Kips    Sum of Wall Weight = 19.00 Kips

**$P_{D2}$ = 23.00 Kips**

**SPREAD FOOTING DESIGN - SMRF's 2-3 AT GRIDLINE 6 (W-E EQ LOADS) - CASE N-3**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

**Assumptions**

1. Column loads are located in transverse center of footing; limit of 2 columns w/o flexure.
2. Footing has no shear reinforcement.
3. Concrete is Normal Weight Concrete with uncoated bars.



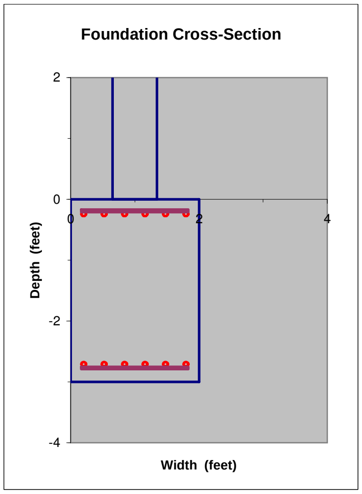

## Footing Parameters :

Footing Size :
$L_x$ = 36.50 feet
$L_y$ = 2.00 feet
$h_f$ = 3.00 feet

Column Sizes :
$C_{1x}$ = 10.5 feet (column length)
$C_{1y}$ = 0.7 feet (column width)
$x_1$ = 5.75 feet (distance from edge of footing to $C_1$ Centerline)

$C_{2x}$ = 10.5 feet (column length)
$C_{2y}$ = 0.7 feet (column width)
$x_2$ = 30.75 feet (distance from edge of footing to $C_2$ Centerline)

Note: S = 25.00 feet (Separation between column centerlines)




Interconnected Slab at Sides:
Note : Slabs at sides are used only to reduce soil bearing pressure; footing is designed to take all loads.
Side : Left Right
t Inches (Slab Thickness)
X Feet (distance to other Slab Edge Support
$f'_c$ Ksi
Conn Type (D= Dowel, C= Continuous)

16.0 19.6 1.23

Footing Loads :
$V_x$ = 19.60 kips (Base Shear - A4 ASD)
$V_y$ = 1.96 kips

Load Factors : Strength = S (S for Strength, OS for Over-Strength)
$\Omega$ = 3.00 (Overstrength Factor - SMRF)

| | Unfactored Loads | | | Service Loads | | | | Strength Loads | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Load | D | L | EQ | 1.0 D + EQ | 0.6 D + EQ | L | $P_S$ | 1.2 D + 1.4 EQ | 0.9 D + 1.4 EQ | 1.6 L | $P_U$ |
| $P_1$ | 23 | | -25 | -2 | -11 | 0 | -11 | -7 | -14 | 0 | -14 |
| $P_2$ | 23 | | 25 | 48 | 39 | 0 | 48 | 62 | 55 | 0 | 62 |

Capacity Factors : $\phi_v$ = 0.75 (Shear)
$\alpha$ = 40 (40 for interior columns, 30 for edge columns, 20 for corner columns)

Concrete : $f'_c$ = 3.25 Ksi
$f_y$ = 60.00 Ksi
$\rho_c$ = 0.150 kip/ft$^3$

Reinforcement: $d_c$ = 2.00 inches (bar clearance - top)
= 3.00 inches (bar clearance - bottom)
= 2.00 inches (bar clearance - sides)

| | Orientation | Bar Size | N Bars | Bottom Layer | d (inches) | Bar Spacing (inches) | Bar Diameter (inches) | Bar Area Per Bar (in^2) | Bar Area Total (in^2) |
|---|---|---|---|---|---|---|---|---|---|
| Top Mat | x | 7 | 6 | x | 33.13 | 3.83 | 0.88 | 0.60 | 3.60 |
| | y | 4 | 130 | | 33.00 | 3.36 | 0.50 | 0.20 | 26.00 |
| Bottom Mat | x | 7 | 6 | | 31.63 | 3.83 | 0.88 | 0.60 | 3.60 |
| | y | 4 | 130 | x | 33.50 | 3.36 | 0.50 | 0.20 | 26.00 |

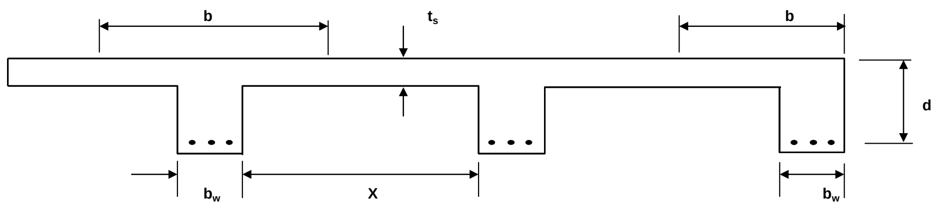
Note: Used for placing top bars only.

Soil Parameters : Soil density = 120 pcf
$\sigma_{allow}$ = 2.00 ksf (allowable bearing pressure)
$\sigma_p$ = 0.30 ksf/ft (Passive Soil Pressure)
$\mu$ = 0.25 ksf (Coefficient of Friction)

**SPREAD FOOTING DESIGN - SMRF's 2-3 AT GRIDLINE 6 (W-E EQ LOADS) - CASE N-3**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 1. Design of Slab-to-Footing Connections

a) Effective Slab Width (ACI ACI 21.6.2.2)




i) Symmetrical T-Beams (ACI 8.12.2)

$b \le$ Min ($L/4 + b_w$, $16\ t_s + b_w$, $X + b_w$)

Where L = feet (Footing span length)
= inches
$t_s$ = inches (Slab thickness - Average value)
$b_w = L_y$ = feet (Footing Width)
= inches
X = feet (Slab Supported Length - Average Value)
= inches

=

| b = | inches |
|---|---|

ii) Slabs on One Side (ACI 8.12.3)

$b \le$ Min ($b_w + L/12$, $b_w + 6\ t_s$, $X/2 + b_w$)

Where $b_w = L_y$ = feet (Footing Width)
= inches
L = feet (Footing span length)
= inches
$t_s$ = inches (Slab thickness)
X = feet (Slab Supported Length)
= inches

=

| b = | inches |
|---|---|

| **b =** | **inches** |
|---|---|
| **=** | **feet** |

b) Required Slab Reinforcement Area

$A_{sr} = 0.0018\ b\ t_s$ (ACI 7.12.2.1)

Where b = inches (Effective Slab Width)
$t_s$ = inches (Slab thickness)

| $A_{sr}$ = | in$^2$ |
|---|---|

c) Required Slab Reinforcement Spacing

$S_{sr} = b / n_b \le 2\ t_s$ (ACI 13.3.2)

Where b = inches (Effective Slab Width)

$n_b$ = Ceiling ($A_{sr}/A_b$) for $A_{sr}$ = in$^2$ (Slab Reinforcement Area - Required)
$A_b$ = 0.20 in$^2$ for No. 4 bars

=

| $n_b$ = | bars |
|---|---|

$t_s$ = inches
$2\ t_s$ = inches

| $S_{sr}$ = | inches |
|---|---|

**RC Slab not Needed**

**SPREAD FOOTING DESIGN - SMRF's 2-3 AT GRIDLINE 6 (W-E EQ LOADS) - CASE N-3**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 2. Lateral Resistance of Foundation

### 2A. Longitudinal Loading

$F_{Rx} = 0.5\, L'_y\, h'^2_f\, \sigma_p + 0.6\,(W_f + P)\,\mu$

Where $L'_y = L_y + 2\, t_{nw}$ and $L_y =$ 2.0 feet

$t_{ew} =$ 2.00 feet (Thickness of (E) connected walls at ends)

$L'_y =$ 6.00 feet (Bearing Width at Ends of Footing)

$h'_f = h_f + h_{sk}$ and $h_f =$ 3.0 feet

$h_{sk} =$ 0.00 feet (Additional height of Shear Key at Footing End)

$h'_f =$ 3.0 feet (Bearing Height at Ends of Footing)

= (27.0) (0.30) + 0.6 (78.9) (0.25)

$\sigma_p =$ 0.30 ksf/ft (Passive Soil Pressure)

= (8.10) + (11.83)

$W_f = \rho_c\, L_x\, L_y\, h_f$ and $\rho_c =$ 0.150 kip/ft^3

$L_x =$ 36.5 feet

$L_y =$ 2.0 feet

$h_f =$ 3.0 feet

$W_f =$ 32.85 Kips (Footing Weight)

$P = P_1 + P_2$ and $P_1 =$ 23.0 Kips

$P_2 =$ 23.0 Kips

$P =$ 46.0 Kips (Service Load)

$\mu =$ 0.25 ksf (Coefficient of Friction)

$F_{Rx} =$ 19.93 kips

**OK**

Note : $V_x =$ 19.60 kips

**Foundation OK for Sliding**

## 3. Soil Pressure due to Applied Loads - Service and Strength Loading

a) Applied soil stress - uniform

$q = (P_1 + P_2)/L_x$

| | Service | Strength | |
|---|---|---|---|
| Where $P_1 =$ | -11 | -14 | kips |
| $P_2 =$ | 48 | 62 | kips |
| $L_x =$ | 36.5 | feet | |

$q =$ 1.01 kip/ft **Service**
= 1.32 kip/ft **Strength**

b) Centroid of Factored Loads - from Left Edge of Footing

$X_R = (x_1\, P_1 + x_2\, P_2 + W_f L_x/2) / (P_1 + P_2 + W_f)$

| | Service | Strength | |
|---|---|---|---|
| Where $X_1 =$ | 5.75 | 5.75 | feet (distance from edge of footing to $C_1$ Centerline) |
| $P_1 =$ | -11 | -14 | kips |
| $X_2 =$ | 30.75 | 30.75 | feet (distance from edge of footing to $C_2$ Centerline) |
| $P_2 =$ | 48 | 62 | kips |
| $W_f =$ | 32.85 | Kips | (Footing Weight) |
| $L_x =$ | 36.5 | feet | |

$X_R =$ 28.78 feet **Service**
= 29.98 feet **Strength**

c) Applied soil stress - **Trapezoidal**

Note: This conditions applies when **Δq < q**, and **$L_b = L_x$** **<= Does not apply**

$\Sigma M_o = -X_1 P_1 - X_2 P_2 + (q - \Delta q)\, L_x^2/2 + 1/2\,(2\,\Delta q)\, L_x\, L_{\Delta q}$

=> $\Delta q = (\, q\, L_x^2/2 - X_1 P_1 - X_2 P_2)/(L_x^2/2 - L_x L_{\Delta q})$

| | Service | Strength | |
|---|---|---|---|
| Where $X_1 =$ | 5.75 | 5.75 | feet (distance from edge of footing to $C_1$ Centerline) |
| $P_1 =$ | -11 | -14 | kips |
| $X_2 =$ | 30.75 | 30.75 | feet (distance from edge of footing to $C_2$ Centerline) |
| $P_2 =$ | 48 | 62 | kips |
| $q =$ | 1.01 | 1.32 | kips/ft |
| $L_x =$ | 36.5 | 36.5 | feet |

$L_{\Delta q} = L_x/3$ if $X_R < 0.5\, L_x$
$= 2\, L_x/3$ if $X_R > 0.5\, L_x$

$L_{\Delta q} =$ 24.33 feet **Service**
= 24.33 feet **Strength**

Incremental Soil Bearing Stresses :

$\Delta q =$ 3.30 **Service**
**Soil Stress is NOT Trapezoidal**

$\Delta q =$ 4.29 kips/ft **Strength**
**Soil Stress is NOT Trapezoidal**

**SPREAD FOOTING DESIGN - SMRF's 2-3 AT GRIDLINE 6 (W-E EQ LOADS) - CASE N-3**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

d) Applied soil stress - **Triangular** **<= Governs**

Notes: This condition applies when $\Delta q \geq q$, and $L_b \leq L_x$

$X_R$ = 28.78 feet => **Large Rotation to Right** **Service**
= 29.98 feet **Strength**

$0.5\ L_x$ = 18.3 feet

i) Rotation to Left - Footing Bearing Length

$L_b = 3\ (0.5\ L_x - e)$ Where $0.5\ L_x$ = 18.3 feet

$e = X_R - 0.5\ L_x$ and $X_R$ = 28.78 feet **Service**
$X_R$ = 29.98 feet **Strength**

| | | | |
|---|---|---|---|
| e = | 10.53 | feet | **Service** |
| = | 11.73 | feet | **Strength** |

| | | | |
|---|---|---|---|
| **Service** | $L_b$ = | NA | feet |
| **Strength** | = | NA | feet |

ii) Rotation to Right - Footing Bearing Length

$L_b = 3\ (0.5\ L_x - e)$ Where $0.5\ L_x$ = 18.3 feet

$e = X_R - 0.5\ L_x$ and $X_R$ = 28.78 feet **Service**
$X_R$ = 29.98 feet **Strength**

| | | | |
|---|---|---|---|
| e = | 10.53 | feet | **Service** |
| = | 11.73 | feet | **Strength** |

| | | | |
|---|---|---|---|
| $L_b$ = | 23.16 | feet | **Service** |
| | 19.56 | feet | **Strength** |

iii) Resulting Soil Bearing Length and Triangular Pressure

$\Delta q = 2(P_1 + P_2)/L_b$

| | Service | Strength | |
|---|---|---|---|
| Where $P_1$ = | -11 | -14 | kips |
| $P_2$ = | 48 | 62 | kips |
| $L_b$ = | 23.16 | 19.56 | feet |

| | | | |
|---|---|---|---|
| $\Delta q$ = | 3 | kips/ft | **Service** |
| = | 5 | kips/ft | **Strength** |

e) Applied Soil Stresses - Governing

**Service Loads :**

Note: Large Rotation to Right

q = 0.00 kips/ft $X_R$ = 28.78 feet
$\Delta q$ = 3.18 kips/ft $0.5\ L_x$ = 18.25 feet

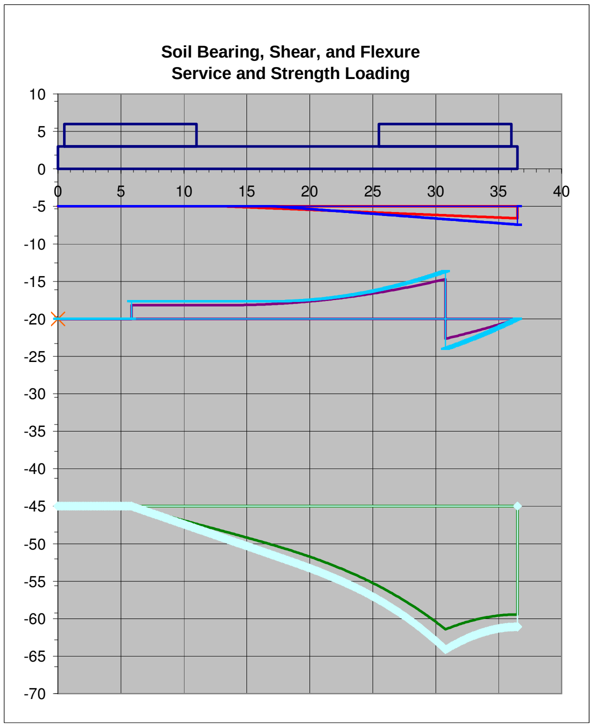
$L_b$ = 23.16 feet
Note: $L_o$ = 13.34 feet (location of soil zero value)

**Strength Loads :**

Note: Large Rotation to Right

q = 0.00 kips/ft $X_R$ = 29.98 feet
$\Delta q$ = 4.94 kips/ft $0.5\ L_x$ = 18.25 feet

$L_b$ = 19.56 feet
Note: $L_o$ = 16.94 feet (location of soil zero value)




f) Check of Soil Bearing Stress

**Service Loads :**

$\sigma_b = q_{max} / L_y \leq \sigma_b$ Where $q_{max}$ = 3.18 kips/ft
$L_y = b$ = 2.00 feet

| | | | |
|---|---|---|---|
| $\sigma_b$ = | 1.59 | Ksf | **Service** |

**OK**

**Strength Loads :**

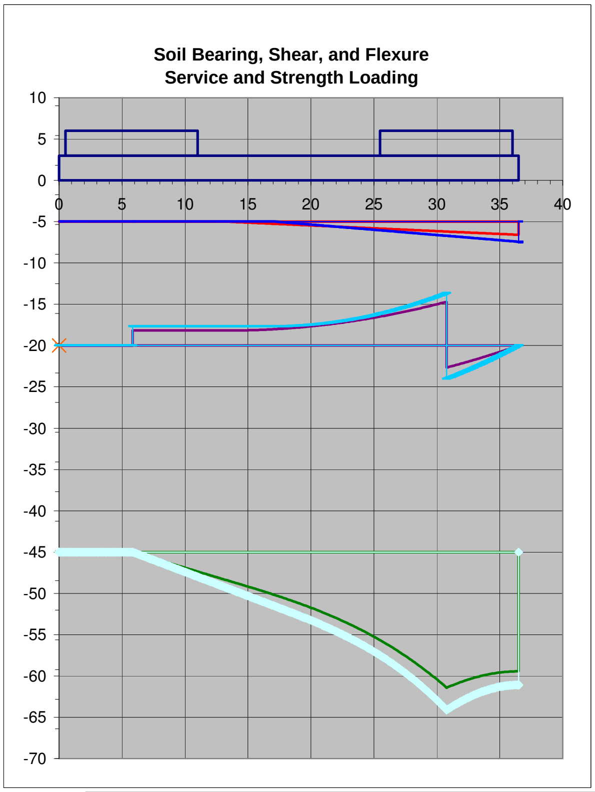
$\sigma_{bu} = q_u / L_y \leq \sigma_b$ Where $q_u$ = 4.94 kips/ft
$L_y = b$ = 2.00 feet

| | | | |
|---|---|---|---|
| $\sigma_b$ = | 2.47 | Ksf | **Strength** |

**SPREAD FOOTING DESIGN - SMRF's 2-3 AT GRIDLINE 6 (W-E EQ LOADS) - CASE N-3**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 4. Applied Loading and Demands on Footing - Strength Loads

| | Left End | Left Column Centerline | Inflection Point | Right Column Centerline | Right End |
|---|---|---|---|---|---|
| Location (feet) | 0 | 5.75 | | 30.75 | 36.50 |
| Load (kips) | - | -11 | - | 48 | - |
| $V_L$ (kips) | 0 | 0 | | 37 | |
| $V_R$ (kips) | - | 14 | - | -24 | - |
| $M_L$ (kip-ft) | 0 | 0 | - | -450 | - |
| $M_R$ (kip-ft) | - | 0 | | -459 | |




## 5. Adequacy of Footing - Shear

### 5A. Check of Flexural/One-Way Shear (ACI 15.5.2 and 11.1.3.1)

Shear demands: $V_{max}$ = 37 Kips @ x = 30.75 feet

$V_u = V_{max} - q\ (d + C/2)$

Where $V_{max}$ = 37 Kips

$q_u$ = 2.38 Kips/ft @ x = 30.75 feet

$d = h_f - d_c - d_b$ and $h_f$ = 3.0 feet
= 36.00 inches

$d_c$ = 3.00 inches
$d_b$ = 0.5 inches

d = 32.50 inches
= 2.71 feet

C = 0.00 feet

$V_u$ = 31 Kips

b) Shear Strength provided by Concrete (ACI 11.3.1.1)

$\phi V_c = \phi\, 2\, f'_c{}^{0.5}\, b_w\, d$ Where $\phi$ = 0.75

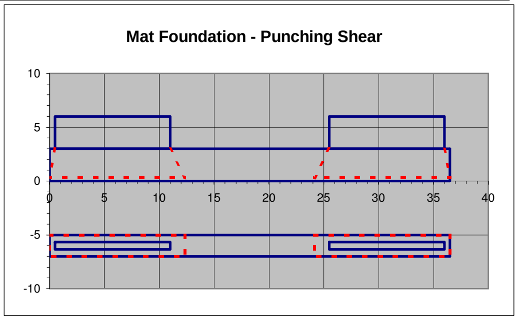
$f'_c$ = 3,250 psi

$b = L_y$ = 2.0 feet
= 24.0 inches

d = 32.50 inches

$\phi V_c$ = 67 kips
**OK, > Vu**

Note: if footing shear reinforcement is needed, use EQ (11-5) in ACI 11.3.2.1.




### 5B. Punching/Two Way Shear (ACI 11.5.5.2, 11.12.1.2)

**A. Left Column**

Shear demands: $V_u$ = 14 Kips

a) Failure Perimeter

$b_0 = 2\ (b_1 + b_2)$ Where $b_1 = X_1 + 0.5(C_{2x} + d) <= C_{2x} + d$ and $X_1$ = 5.75 feet
= 69.00 inches

$C_{2x}$ = 126.00 inches
d = 32.50 inches

$b_1$ = 148.25 inches

$b_2 = C_{2y} + d <= L_y$ and $C_{2y}$ = 8.28 inches
d = 32.50 inches

$L_y$ = 24.0 inches

$b_2$ = 24.00 inches

$b_0$ = 320.5 inches

**SPREAD FOOTING DESIGN - SMRF's 2-3 AT GRIDLINE 6 (W-E EQ LOADS) - CASE N-3**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

b) Factored Shear Capacity (ACI 11.12.2.1)

$$\varphi V_c = \varphi \, \text{Min}\left[2 + \frac{4}{\beta}, \frac{\alpha d}{b_o} + 2, 4\right]\sqrt{f'_c}\, b_o\, d$$

Where $\phi$ = 0.75

$\beta = C_{max}/C_{min}$ and $C_{max}$ = 126.00 inches

$C_{min}$ = 8.28 inches

$\beta$ = 15.22

$\alpha$ = 30 (40 for interior columns, 30 for edge columns, 20 for corner columns)

d = Min($d_1$,$d_2$) $d_1$ = 31.63 inches

$d_2$ = 33.50 inches

d = 31.63 inches

$b_0$ = 320.5 inches

$f'_c$ = 3,250 psi

$\phi V_c$ = 981 kips

**OK, > Vu**

**B. Right Column**

Shear demands: $V_u$ = 37 Kips

a) Failure Perimeter

$b_0 = 2\,(b_1 + b_2)$ Where $b_1 = 0.5(C_{2x} + d) + (L_x - X_2)$ <= $C_{2x} + d$

and $L_x$ = 36.5 feet

$X_2$ = 30.75 feet

$L_x - X_2$ = 5.75 feet

= 69.00 inches

$C_{2x}$ = 126.00 inches

d = 32.50 inches

$b_1$ = 148.25 inches

$b_2 = C_{2y} + d$ <= $L_y$

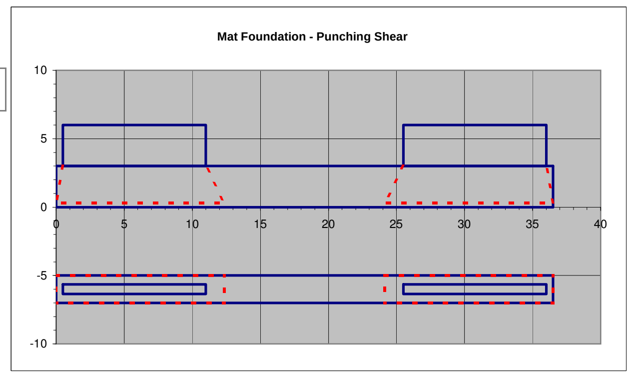
and $C_{2y}$ = 8.28 inches

d = 32.50 inches

$L_y$ = 24.0 inches

$b_2$ = 24.00 inches

$b_0$ = 320.5 inches




b) Factored Shear Capacity (ACI 11.12.2.1)

$$\varphi V_c = \varphi \, \text{Min}\left[2 + \frac{4}{\beta}, \frac{\alpha d}{b_o} + 2, 4\right]\sqrt{f'_c}\, b_o\, d$$

Where $\phi$ = 0.75

$\beta = C_{max}/C_{min}$ and $C_{max}$ = 126.00 inches

$C_{min}$ = 8.28 inches

$\beta$ = 15.22

$\alpha$ = 30 (40 for interior columns, 30 for edge columns, 20 for corner columns)

d = Min($d_1$,$d_2$) $d_1$ = 31.63 inches

$d_2$ = 33.50 inches

d = 31.63 inches

$b_0$ = 320.5 inches

$f'_c$ = 3,250 psi

$\phi V_c$ = 981 kips

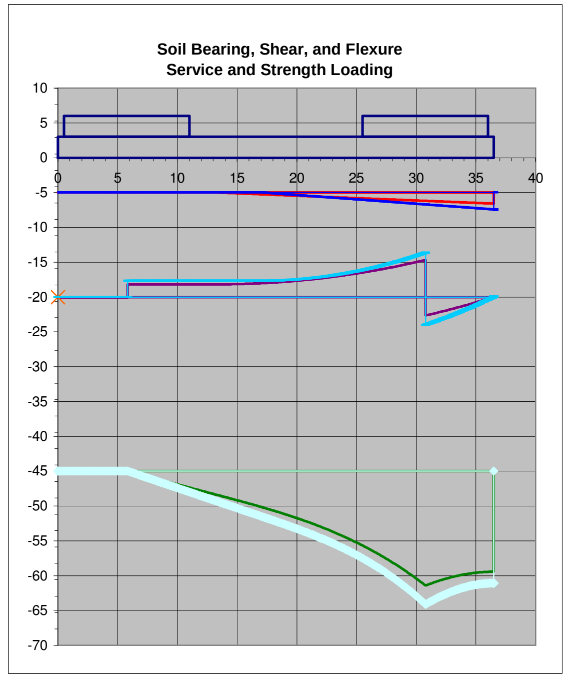
**OK, > Vu**

**Footing OK for Shear**

**SPREAD FOOTING DESIGN - SMRF's 2-3 AT GRIDLINE 6 (W-E EQ LOADS) - CASE N-3**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 6. Adequacy of Footing - Flexure

### 6A. Longitudinal Top Reinforcement Check




Flexural demands: $M_u$ = 0 Kip-ft @ x = feet (Inflection Point)

a) Required Reinforcement Ratio (ACI 10.2)

$$\rho_r = \frac{0.85\ f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2\ M_u}{0.765\ b\ d^2\ f'_c}}\right]$$

Where $f'_c$ = 3.25 Ksi
$f_y$ = 60.00 Ksi
$M_u$ = 0
= 0 kip-in
$b = L_y$ = 2.0 feet
= 24 inches
$d_{top}$ = 33.13 inches

$\rho_r$ = 0.0000

b) Maximum Reinforcement Ratio (ACI 10.3.4)

$$\rho_t = 0.85\ \beta_1 \frac{f'_c}{f_y}\left[\frac{\varepsilon_c}{\varepsilon_c + \varepsilon_s}\right]$$

Where $f'_c$ = 3.25 Ksi
$f_y$ = 60.00 Ksi

$\beta_1$ = 0.85 for $f'_c \le$ 4.0 Ksi
= 0.85 - 0.05 ($f'_c$ - 4.0), ≥ 0.65 for $f'_c$ > 4.0 Ksi

$\beta_1$ = 0.85

$\varepsilon_c$ = 0.003 (ACI Section 10.3.4)
$\varepsilon_s$ = 0.005

$\rho_t$ = 0.0147

c) Minimum Reinforcement of Flexural Members (ACI 10.5, 7.12.2.1)

$\rho_{min}$ = Max [3 $f'_c{}^{0.5}/f_y$, 200/$f_y$] ≤ Max [1.33 $\rho_r$, 0.0018]
= Max [0.0029,0.0033] ≤ Max[0.0000,0.0018]
= Max [0.0033] ≤ Max[0.0018]

Where $f'_c$ = 3,250 psi
$f_y$ = 60,000 psi
$\rho_r$ = 0.0000 (Required Reinforcement Ratio)

$\rho_{min}$ = 0.0018

≤

d) Required Reinforcement Area

$A_{req} = \rho\ L_y\ d_2$

Where $\rho = \rho_r$ if $\rho_r \le \rho_{max}$ and $\rho_r \ge \rho_{min}$
$= \rho_{max}$ if $\rho_r \ge \rho_{max}$
$= \rho_{min}$ if $\rho_r \le \rho_{min}$

and $\rho_r$ = 0.00000
$\rho_{max}$ = 0.01468
$\rho_{min}$ = 0.00180

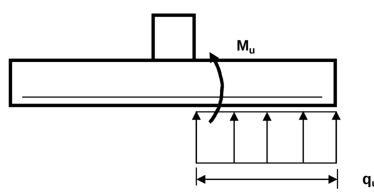
Where $\rho_r$ = 0.0018
$L_y$ = 2.0 feet
= 24.0 inches
$d_{top}$ = 33.13 inches

$A_{req}$ = 1.43 in^2

Note: $A_{s1}$ = 3.60 in^2 (reinforcement provided)
OK

**6 - No. 7 Longitudinal Top Bars OK**

### 6B. Longitudinal Bottom Reinforcement Check




a) Flexural Demands (ACI 15.4.2)

$M_u$ = 0 Kip-ft @ x = 5.75 feet (at face of Left Column)
= -459 Kip-ft @ x = 30.75 feet (at face of Right Column)

$M_u$ = 459 kip-ft
= 5,508 kip-in

b) Required Reinforcement Ratio (ACI 10.2)

$$\rho_r = \frac{0.85\ f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2\ M_u}{0.765\ b\ d^2\ f'_c}}\right]$$

Where $f'_c$ = 3.25 Ksi
$f_y$ = 60.00 Ksi
$M_u$ = 5,508 kip-in
$b = L_y$ = 2.0 feet
= 24 inches
$d_{bott}$ = 31.63 inches

$\rho_r$ = 0.0045

**SPREAD FOOTING DESIGN - SMRF's 2-3 AT GRIDLINE 6 (W-E EQ LOADS) - CASE N-3**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

c) Required Reinforcement of Flexural Members

Note: $A_{sb}$ = 3.60 in$^2$ (Bottom flexural steel provided)

$A_{req} = \rho\, b\, d$

Where $\rho = \rho_r$ for $\rho_r \le \rho_t$ and $\rho_r \ge \rho_{min}$
$= \rho_{min}$ for $\rho_r \le \rho_{min}$*
$= 0$ Otherwise

Where $\rho_r$ = 0.0045
$\rho_t$ = 0.0147
$\rho_{min}$ = 0.0018

$\rho$ = 0.0045

Note: * for $\rho_r \le \rho_{min}$ condition, check Minimum Reinforcement exception (ACI 10.5.3)

$A_{min} = 1.33\, \rho_r\, b\, d$

Where $\rho_r$ = (required reinforcement ratio)
$b = L_y$ = inches
$d = d_{bott}$ = inches

$A_{min}$ = NA in$^2$

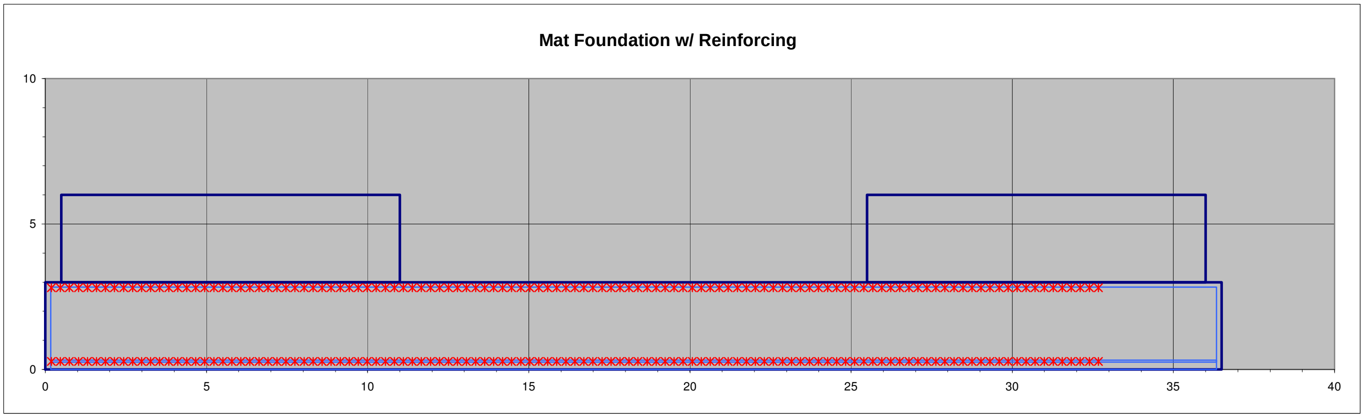
$b = L_y$ = 24.0 inches
$d = d_{bott}$ = 31.63 inches

$A_{req}$ = 3.39 in$^2$
**OK**

**6 - No. 7 Longitudinal Bottom Bars OK**

**SPREAD FOOTING DESIGN - SMRF's 2-3 AT GRIDLINE 6 (W-E EQ LOADS) - CASE N-3**
**ACI 318-11 LOADS AND DESIGN**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 7. Footing Reinforcement Summary




**Footing Parameters :**

Footing Size :

$L_x$ = 36.5 feet
$L_y$ = 2.0 feet
$h_f$ = 3.0 feet

Reinforcement Summary: $d_c$ = 2.00 inches (bar clearance - top)
= 3.00 inches (bar clearance - bottom)
= 2.00 inches (bar clearance - sides)

| | Orientation | Bar Size | N Bars | Bottom Layer | d (inches) | Bar Spacing (inches) | Bar Diameter (inches) | Bar Area Per Bar (in^2) | Bar Area Total (in^2) |
|---|---|---|---|---|---|---|---|---|---|
| Top Mat | x | 7 | 6 | x | 33.13 | 3.83 | 0.88 | 0.60 | 3.60 |
| | y | 4 | 130 | | 33.00 | 3.36 | 0.50 | 0.20 | 26.00 |
| Bottom Mat | x | 7 | 6 | | 31.63 | 3.83 | 0.88 | 0.60 | 3.60 |
| | y | 4 | 130 | x | 33.50 | 3.36 | 0.50 | 0.20 | 26.00 |

Note: Used for placing top bars only.

### 1. Design of Slab-to-Footing Connections

Interconnected Slab at Sides:

Note : Slabs at sides are used only to reduce soil bearing pressure; footing is designed to take all loads.

Side : Left Right

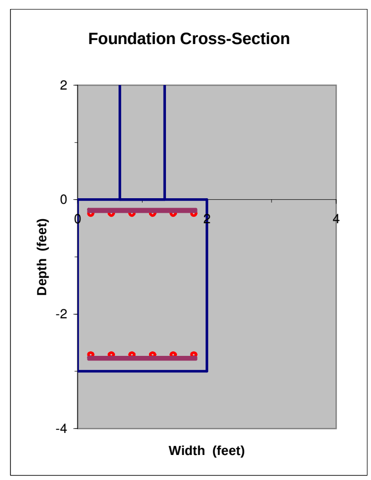
t Inches (Slab Thickness)

X Feet (distance to other Slab Edge Support

$f'_c$ Ksi

Conn Type (D= Dowel, C= Continuous)

**RC Slab not Needed**




### 2. Lateral Resistance of Foundation

**Foundation OK for Sliding**

### 3. Soil Pressure due to Applied Loads

$\sigma_b$ = 1.59 Ksf
**OK**

Note: $\sigma_{allow}$ = 2.00 ksf (allowable bearing pressure)

### 5. Adequacy of Footing - Shear

**Footing OK for Shear**

### 6. Adequacy of Footing - Flexure

**6 - No. 7 Longitudinal Top Bars OK**

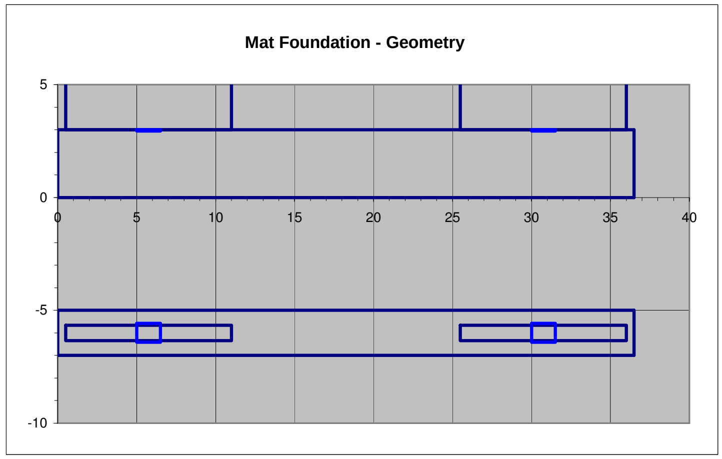
**6 - No. 7 Longitudinal Bottom Bars OK**

**SPREAD FOOTING DESIGN - SMRF's 2-3 AT GRIDLINE 6 (W-E EQ LOADS) - CASE N-3**
**DETERMINATION OF ADDITIONAL REINFORCEMENT REQUIRED - FIXED BASE COLUMN CONDITION**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 1. Parameters

Footing Size :

$L_x$ = 36.50 feet
$L_y$ = 2.00 feet
$h_f$ = 3.00 feet




Base Plate Dimensions:

**Note:** Base Plate design done elsewhere.

N = 18.00 inches (Base Plate - Length)
B = 10.00 inches (Base Plate - Width)
$t_{PL}$ = 0.75 inches (Base Plate - Thickness)

Column: W8x67

d = 9.00 inches (Wide Flange - Depth)
$b_f$ = 8.28 inches (Wide Flange - Width)
$t_f$ = 0.94 inches (Wide Flange - Thickness)
$Z_x$ = 70.1 $in^3$ (Wide Flange - Plastic Section)
A = 19.70 $in^2$ (Wide Flange - Area)
$F_y$ = 50 Ksi

Concrete :

$f'_c$ = 3.25 Ksi
$f_y$ = 60.00 Ksi
$\rho_c$ = 0.15 kip/ft$^3$

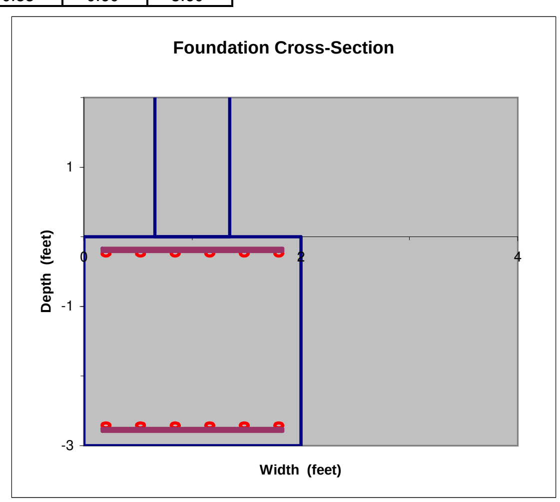
Reinforcement:

$d_c$ = 2.00 inches (bar clearance - top)
= 3.00 inches (bar clearance - bottom)
= 2.00 inches (bar clearance - sides)

| | Orientation | Bar Size | N Bars | Bottom Layer | d (inches) | Bar Spacing (inches) | Bar Diameter (inches) | Bar Area Per Bar (in^2) | Bar Area Total (in^2) |
|---|---|---|---|---|---|---|---|---|---|
| Top Mat | x | 7 | 6 | x | 33.13 | 3.83 | 0.88 | 0.60 | 3.60 |
| Bottom Mat | x | 7 | 6 | 0 | 31.63 | 3.83 | 0.88 | 0.60 | 3.60 |

Soil Parameters :

Soil density = 120 pcf
$\sigma_{allow}$ = 2.00 ksf (allowable bearing pressure)
$\sigma_p$ = 0.30 ksf/ft (Passive Soil Pressure)
$\mu$ = 0.25 ksf (Coefficient of Friction)




Design Parameters :

$\phi_v$ = 0.75 (Shear; ACI 318-11 9.3.2.3)
$\Omega$ = 3.00 (Overstrength Factor - SMRF)

**SPREAD FOOTING DESIGN - SMRF's 2-3 AT GRIDLINE 6 (W-E EQ LOADS) - CASE N-3**
**DETERMINATION OF ADDITIONAL REINFORCEMENT REQUIRED - FIXED BASE COLUMN CONDITION**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 2. Additional Required Reinforcement at Columns

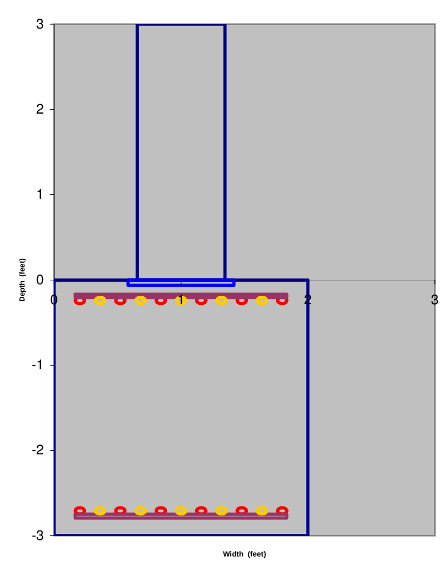
a) Column Probable Expected Flexural Capacity

$M_{FB}$ = 100% of Column Flexural Capacity

= $1.0\, Z_x\, F_y$ Where $Z_x$ = 70.1 $in^3$ (Wide Flange - Plastic Section)

$F_y$ = 50 Ksi

| | | |
|---|---|---|
| $M_{FB}$ = | 3,505 | Kip-in |
| = | 292.1 | Kip-ft |




b) Required Reinforcement Ratio (ACI 10.2)

$$\rho_r = \frac{0.85\ f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2\ M_u}{0.765\ b\ d^2\ f'_c}}\right]$$

Where $f'_c$ = 3.25 Ksi
$f_y$ = 60.00 Ksi

$M_u = M_F + M_{FB}$ for $M_F$ = 5,508 kip-in (Footing Flexural Demands)
$M_{FB}$ = 3,505 kip-in (Column Flexural Capacity)

$M_u$ = 9,013 kip-in

$b = L_y$ = 2.0 feet
= 24 inches

$d_x$ = 31.63 inches

| | |
|---|---|
| $\rho_r$ = | 0.00758 |

c) Reinforcement Ratio Provided

$\rho_w = A_{sx}/(L_y\, d_x)$ Where $A_{sx} = A_F + A_{FB}$ Where $A_F$ = 3.60 $in^2$ (Reinforcement Provided - Footing Flexure)

$A_{FB}$ = Reinforcement Required for Resisting Fixed Base Column Flexural Capacity

= $(N-1)\, A_b$ for N = 6 bars provided

$A_b$ = 0.60 $in^2$ for 7 bars

Note: db = 0.88 $in^2$ (Bar Diameter)

$A_{FB}$ = 3.00 $in^2$

| | | |
|---|---|---|
| $A_{sx}$ = | 6.60 | $in^2$ |

$L_y = L_B$ = 2.0 feet
= 24.0 inches
d = 31.63 inches

| | |
|---|---|
| $\rho_w$ = | 0.00870 |

(reinforcement ratio provided) Note: D/C Ratio = 0.87 (Demand to Capacity Ratio - Flexure)

OK

**Use Additional 5 - # 7 Bars for Column Flexure with DC Ratio = 0.87**

**SPREAD FOOTING DESIGN - SMRF's 2-3 AT GRIDLINE 6 (W-E EQ LOADS) - CASE N-3**
**DETERMINATION OF ADDITIONAL REINFORCEMENT REQUIRED - FIXED BASE COLUMN CONDITION**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

d) Flexural reinforcement development length (ACI 12.2.2 and 12.2.5)

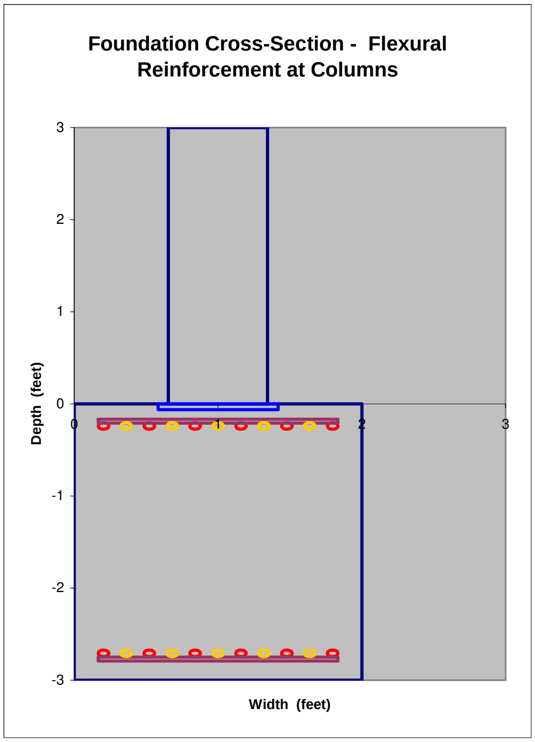
i) Development Length (ACI 12.2.2 - 12.2.4)

Bar Size = 7

$d_s$ = S - $d_b$ Where S = 1.93 inches (Bar spacing provided)
$d_b$ = 0.88 inches

$d_s$ = 1.06 inches (Clear spacing provided)

$d_c$ = 2.00 inches (Clear Cover provided)




| | Provided (inches) | Lower Limit | Upper Limit |
|---|---|---|---|
| **Clear Cover** | 2.00 | $d_b$ = 0.88 inches<br>OK | 2 $d_b$ = 1.75 inches<br>OK |
| **Clear Spacing** | 1.06 | 2 $d_b$ = 1.75 inches<br>NG | 4 $d_b$ = 3.50 inches<br>NG |
| | Equations | $l_d = \left( \frac{f_y \Psi_s \Psi_t \Psi_e \lambda}{25\sqrt{f'_c}} \right) d_b$ | $l_d = \frac{3}{40} \left( \frac{f_y \Psi_s \Psi_t \Psi_e \lambda}{2.5\sqrt{f'_c}} \right) d_b$ |
| | Values | $l_d$ = 52.62 $d_b$<br>$l_d$ = 46.0 inches | $l_d$ = 31.57 $d_b$<br>$l_d$ = 27.6 inches |

Note: Normal Weight Concrete with uncoated bars is assumed.

Where $f_y$ = 60.00 Ksi
$\Psi_s$ = 1.00 (ACI 12.2.4)

$\Psi_t = \Psi_e = \lambda$ = 1.00

$f'_c$ = 3.25 Ksi

$d_b$ = 0.88 inches

ii) Excess Reinforcement (ACI 12.2.5)

$l'_d = l_d\, \rho_r/\rho_w$ Where $l_d$ = 0.0 inches
$\rho_r$ = 0.0076 (required reinforcement ratio)
$\rho_w$ = 0.0087 (reinforcement ratio provided)

$l'_d$ = 0.0 inches (Required development length)

iii) Available Anchorage length

$L_{da} = x_f - d_{cs}$ > $l'_d$ Where $x_f$ = 5.75 feet (Cantilever Length at Column Centerline)
= 69.00 inches

$d_{cs}$ = 2.00 inches (bar clearance - sides)

$L_{da}$ = 67.00 inches
OK

**Use Additional 5 - # 7 Bars for Column Flexure; Use 0 in Development Length beyond Ends of Base Plates**

**SPREAD FOOTING DESIGN - SMRF's 2-3 AT GRIDLINE 6 (W-E EQ LOADS) - CASE N-3**
**DETERMINATION OF ADDITIONAL REINFORCEMENT REQUIRED - FIXED BASE COLUMN CONDITION**
**816 TARAVAL STREET, SAN FRANCISCO - SEISMIC RETROFIT**

## 3. Foundation Demands at Fixed Base Columns

a) Column Fixed Base Plastic Shear Demands

$V_P = M_P / d$ Where $M_P = F_y Z_x$ for $F_y =$ 50 Ksi

$Z_x =$ 70.1 $in^3$ (Wide Flange - Plastic Section)

$M_P =$ 3,505 Kip-in

$d =$ 31.63 inches (Efffective depth of footing)

$V_P =$ 110.8 Kips

b) Amplified Column Axial Demands - Overstrength Shear Demands on Foundation

$P_U =$ Max ( Abs ($P_1$), Abs ($P_2$) ) Where $P_1 =$ -14 Kips

$P_2 =$ 62 Kips

$P_U =$ 62.2 Kips

$V_O = \Omega P_U$ Where $\Omega =$ 3.00 (Overstrength Factor - SMRF)

$P_U =$ 62.2 Kips

$V_O =$ 186.7 Kips

c) Controlling Shear Demands on Foundation

$V_U =$ Max ( $V_P$, $V_O$ ) Where $V_P =$ 110.8 Kips (Column Plastic Shear)

$V_O =$ 186.7 Kips (Column Overstrength Demands)

$V_U =$ 186.7 Kips

## 4. Foundation Capacity at Fixed Base Columns

a) Shear Strength provided by Concrete (ACI 11.3.1.1)

$V_C = 2\ f'_c{}^{0.5}\ b_w\ d$ Where $f'_c =$ 3.25 Ksi

= 3,250 psi

$b = L_y =$ 2.0 feet

= 24.0 inches

$d =$ 31.63 inches (Efffective depth of footing)

$V_C =$ 65 kips

b) Shear Strength provided by Shear Reinforcement (ACI 11.3.1.1)

Note: Assume transverse flexural reinforcement provided for footing is part of a reinrforcement cage.

$V_S = A_S F_y\ d\ /\ S$ (11-15) $A_S =$ 0.40 for No. 4 bars => hoops (from Footing Design)

$\leq 4 V_C$ $F_y =$ 50 Ksi

$d =$ 31.63 inches (Efffective depth of footing)

$S =$ 3.36 inches (bar spacing - from Footing Design)

$V_S =$ 188.2 kips

b) Factored Shear Capacity of Footing (ACI 11.1)

$\phi V_n = \phi\ (V_C + V_S)$ Where $\phi =$ 0.75 (Shear; ACI 318-11 9.3.2.3)

$V_C =$ 65 kips

$V_S =$ 188.2 kips

$\phi V_n =$ 189.8 Kips Note: $V_U =$ 186.7 Kips

**OK**

**Footing OK for Shear**

# MISCELLANEOUS WOOD FRAME CONNECTIONS AND DESIGN VALUES

**TABLE 2306.4.1**
**ALLOWABLE SHEAR (POUNDS PER FOOT) FOR WOOD STRUCTURAL PANEL SHEAR WALLS WITH FRAMING OF DOUGLAS FIR-LARCH OR SOUTHERN PINE[a] FOR WIND OR SEISMIC LOADING[b, h, i, j, l, m]**

| PANEL GRADE | MINIMUM NOMINAL PANEL THICKNESS (inch) | MINIMUM FASTENER PENETRATION IN FRAMING (inches) | PANELS APPLIED DIRECT TO FRAMING | | | | | PANELS APPLIED OVER $^1/_2$" OR $^5/_8$" GYPSUM SHEATHING | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | NAIL (common or galvanized box) or staple size[k] | Fastener spacing at panel edges (inches) | | | | NAIL (common or galvanized box) or staple size[k] | Fastener spacing at panel edges (inches) | | | |
| | | | | 6 | 4 | 3 | 2[e] | | 6 | 4 | 3 | 2[e] |
| Structural I Sheathing | $^5/_{16}$ | $1^1/_4$ | 6d (2 × 0.113" common, 2" × 0.099" galvanized box) | 200 | 300 | 390 | 510 | 8d ($2^1/_2$" × 0.131" common, $2^1/_2$" × 0.113" galvanized box) | 200 | 300 | 390 | 510 |
| | | 1 | $1^1/_2$ 16 Gage | 165 | 245 | 325 | 415 | 2 16 Gage | 125 | 185 | 245 | 315 |
| | $^3/_8$ | $1^3/_8$ | 8d ($2^1/_2$" × 0.131" common, $2^1/_2$" × 0.113" galvanized box) | 230[d] | 360[d] | 460[d] | 610[d] | 10d (3" × 0.148" common, 3" × 0.128" galvanized box) | 280 | 430 | 550[f] | 730 |
| | | 1 | $1^1/_2$ 16 Gage | 155 | 235 | 315 | 400 | 2 16 Gage | 155 | 235 | 310 | 400 |
| | $^7/_{16}$ | $1^3/_8$ | 8d ($2^1/_2$" × 0.131" common, $2^1/_2$" × 0.113" galvanized box) | 255[d] | 395[d] | 505[d] | 670[d] | 10d (3" × 0.148" common, 3" × 0.128" galvanized box) | 280 | 430 | 550[f] | 730 |
| | | 1 | $1^1/_2$ 16 Gage | 170 | 260 | 345 | 440 | 2 16 Gage | 155 | 235 | 310 | 400 |
| | $^{15}/_{32}$ | $1^3/_8$ | 8d ($2^1/_2$" × 0.131" common, $2^1/_2$" × 0.113" galvanized box) | 280 | 430 | 550 | 730 | 10d (3" × 0.148" common, 3" × 0.1218" galvanized box) | 280 | 430 | 550[f] | 730 |
| | | 1 | $1^1/_2$ 16 Gage | 185 | 280 | 375 | 475 | 2 16 Gage | 155 | 235 | 300 | 400 |
| | | $1^1/_2$ | 10d (3" × 0.148" common, 3" × 0.128" galvanized box) | 340 | 510 | 665[f] | 870 | 10d (3" × 0.148" common, 3" × 0.128" galvanized box) | — | — | — | — |
| Sheathing, plywood siding[g] except Group 5 Species | $^5/_{16}$ or $^1/_4$[c] | $1^1/_4$ | 6d (2" × 0.113" common, 2" × 0.099" galvanized box) | 180 | 270 | 350 | 450 | 8d ($2^1/_2$" × 0.131" common, $2^1/_2$" × 0.113" galvanized box) | 180 | 270 | 350 | 450 |
| | | 1 | $1^1/_2$ 16 Gage | 145 | 220 | 295 | 375 | 2 16 Gage | 110 | 165 | 220 | 285 |
| | $^3/_8$ | $1^1/_4$ | 6d (2" × 0.113" common, 2" × 0.099" galvanized box) | 200 | 300 | 390 | 510 | 8d ($2^1/_2$" × 0.131" common, $2^1/_2$" × 0.113" galvanized box) | 200 | 300 | 390 | 510 |
| | | $1^3/_8$ | 8d ($2^1/_2$" × 0.131" common, $2^1/_2$" × 0.113" galvanized box) | 220[d] | 320[d] | 410[d] | 530[d] | 10d (3" × 0.148" common, 3" × 0.128" galvanized box) | 260 | 380 | 490[f] | 640 |
| | | 1 | $1^1/_2$ 16 Gage | 140 | 210 | 280 | 360 | 2 16 Gage | 140 | 210 | 280 | 360 |
| | $^7/_{16}$ | $1^3/_8$ | 8d ($2^1/_2$" × 0.131" common, $2^1/_2$" × 0.113" galvanized box) | 240[d] | 350[d] | 450[d] | 585[d] | 10d (3" × 0.148" common, 3" × 0.128" galvanized box) | 260 | 380 | 490[f] | 640 |
| | | 1 | $1^1/_2$ 16 Gage | 155 | 230 | 310 | 395 | 2 16 Gage | 140 | 210 | 280 | 360 |
| | $^{15}/_{32}$ | $1^3/_8$ | 8d ($2^1/_2$" × 0.131" common, $2^1/_2$" × 0.113" galvanized box) | 260 | 380 | 490 | 640 | 10d (3" × 0.148" common, 3" × 0.128" galvanized box) | 260 | 380 | 490[f] | 640 |
| | | $1^1/_2$ | 10d (3" × 0.148" common, 3" × 0.128" galvanized box) | 310 | 460 | 600[f] | 770 | — | — | — | — | — |
| | | 1 | $1^1/_2$ 16 Gage | 170 | 255 | 335 | 430 | 2 16 Gage | 140 | 210 | 280 | 360 |
| | $^{19}/_{32}$ | $1^1/_2$ | 10d (3" × 0.148" common, 3" × 0.128" galvanized box) | 340 | 510 | 665[f] | 870 | — | — | — | — | — |
| | | 1 | $1^3/_4$ 16 Gage | 185 | 280 | 375 | 475 | — | — | — | — | — |
| | | | Nail Size (galvanized casing) | | | | | Nail Size (galvanized casing) | | | | |
| | $^5/_{16}$[c] | $1^1/_4$ | 6d (2" × 0.099") | 140 | 210 | 275 | 360 | 8d ($2^1/_2$" × 0.113") | 140 | 210 | 275 | 360 |
| | $^3/_8$ | $1^3/_8$ | 8d ($2^1/_2$" × 0.113") | 160 | 240 | 310 | 410 | 10d (3" × 0.128") | 160 | 240 | 310[f] | 410 |

(continued)

SIMPSON Strong-Tie

# HDU/DTT2Z Holdowns

VALUE ENGINEERED

***This product is preferable to similar connectors because of a) easier installation, b) higher loads, c) lower installed cost, or a combination of these features.***

HDU holdowns are pre-deflected during the manufacturing process, virtually eliminating deflection under load due to material stretch. They use Simpson Strong-Tie® Strong-Drive® SDS screws which install easily, reduce fastener slip and provide a greater net section when compared to bolts.

The HDU series of holdowns are designed to replace previous versions of the product such as PHD's as well as bolted holdowns. The HDU2, 4 and 5 are direct replacements for the PHD2, 5 and 6, respectively.

The DTT2Z tension tie is suitable for lighter-duty holdown applications on single or double 2x posts, and installs easily with Strong-Drive SDS screws *(included)*. The DTT2Z has been tested in accordance with the ICC-ES acceptance criteria for Holdowns Attached to Wood Members (AC155) and meets the minimum requirements for many alternate braced wall panels per section R602.10.3.2 of the 2009 IRC *(see table R602.10.6, item 1)*.

For more information on holdown options, contact Simpson Strong-Tie.

**HDU SPECIAL FEATURES**:

- Pre-deflected body virtually eliminates deflection due to material stretch.
- Uses SDS screws which install easily, reduce fastener slip, and provide a greater net section area of the post compared to bolts.
- SDS screws are supplied with the holdowns to ensure proper fasteners are used.
- No stud bolts to countersink at openings.

**MATERIAL**: See table

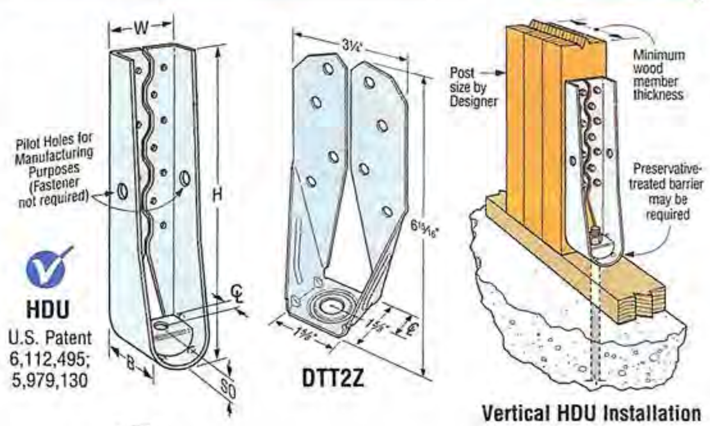
**FINISH**: HDU – Galvanized; DTT2Z – ZMAX® coating; DTT2SS – stainless steel

**INSTALLATION**: • Use all specified fasteners. See General Notes.

- For use in vertical and horizontal applications.
- The HDU requires no additional washer, the DTT requires a standard cut washer *(included)* be installed between the nut and the seat.
- To tie multiple 2x members together, the Designer must determine the fasteners required to join the members without splitting the wood. See page 26 for SDS values.
- See SB and SSTB Anchor Bolts on pages 33-37 for anchorage options.
- SDS screws install best with a low speed high torque drill with a ⅜" hex head driver.

**CODES**: See page 13 for Code Reference Key Chart.

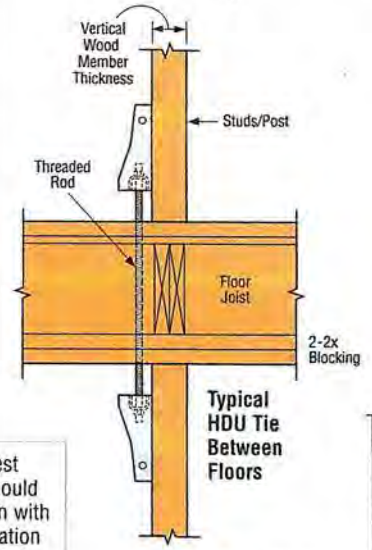
For holdowns, per ASTM test standards, anchor bolt nut should be finger-tight plus ⅓ to ½ turn with a hand wrench, with consideration given to possible future wood shrinkage. Care should be taken to not over-torque the nut. Impact wrenches should not be used.



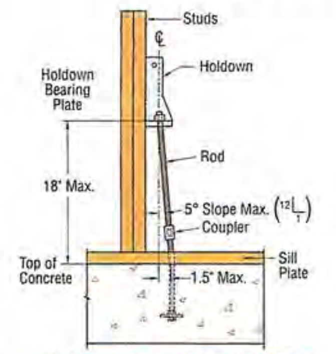

HDU
U.S. Patent 6,112,495; 5,979,130

DTT2Z

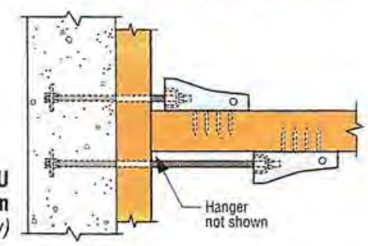
Vertical HDU Installation




Typical HDU Tie Between Floors




Holdown Raised Off Sill Plate




Horizontal HDU Offset Installation *(Plan View)*

See footnote 7

*These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.*

| Model No. | Ga | Dimensions (in.) | | | | | Fasteners | | Minimum Wood Member Thickness[4] (in.) | Allowable Tension Loads (lbs.) (160)[1] | | | Code Ref. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | W | H | B | ℄ | SO | Anchor Bolt Dia. (in.) | SDS Screws | | DF/SP | SPF/HF | Deflection at Allowable Load (in.) | |
| DTT2Z | 14 | 3¼ | 6¹⁵⁄₁₆ | 1⅝ | ¹³⁄₁₆ | ³⁄₁₆ | ½ | 8-SDS ¼"x1½" | 1½ | 1825 | 1800 | 0.105 | I6, L8, F5 |
| | | | | | | | | 8-SDS ¼"x1½" | 3 | 2145 | 1835 | 0.128 | |
| DTT2Z-SDS2.5 | | | | | | | | 8-SDS ¼"x2½" | 3 | 2145 | 2105 | 0.128 | 170 |
| HDU2-SDS2.5 | 14 | 3 | 8¹¹⁄₁₆ | 3¼ | 1⁵⁄₁₆ | 1⅜ | ⅝ | 6-SDS ¼"x2½" | 3 | 3075 | 2215 | 0.088 | I6, L8, F5 |
| HDU4-SDS2.5 | 14 | 3 | 10¹⁵⁄₁₆ | 3¼ | 1⁵⁄₁₆ | 1⅜ | ⅝ | 10-SDS ¼"x2½" | 3 | 4565 | 3285 | 0.114 | |
| HDU5-SDS2.5 | 14 | 3 | 13³⁄₁₆ | 3¼ | 1⁵⁄₁₆ | 1⅜ | ⅝ | 14-SDS ¼"x2½" | 3 | 5645 | 4065 | 0.115 | |
| HDU8-SDS2.5 | 10 | 3 | 16⅝ | 3½ | 1⅜ | 1½ | ⅞ | 20-SDS ¼"x2½" | 3 | 5980 | 4305 | 0.084 | |
| | | | | | | | | | 3½ | 6970 | 5020 | 0.116 | |
| | | | | | | | | | 4½ | 7870 | 5665 | 0.113 | |
| HDU11-SDS2.5 | 10 | 3 | 22¼ | 3½ | 1⅜ | 1½ | 1 | 30-SDS ¼"x2½" | 5½ | 9535 | 6865 | 0.137 | |
| | | | | | | | | | 7¼ | 11175 | 8045 | 0.137 | |
| HDU14-SDS2.5 | 7 | 3 | 25¹¹⁄₁₆ | 3½ | 1⁹⁄₁₆ | 1⁹⁄₁₆ | 1 | 36-SDS ¼"x2½" | 4x6[10] | 10770 | 7755 | 0.122 | 170 |
| | | | | | | | | | 7¼ | 14375[9] | 10435[9] | 0.177 | I6, L8, F5 |
| | | | | | | | | | 5½[8] | 14445[8,9] | 10350[9] | 0.177 | |

1. Allowable loads have been increased for earthquake or wind load durations with no further increase allowed; reduce where other load durations govern.
2. The Designer must specify anchor bolt type, length and embedment. See SB and SSTB Anchor Bolts *(pages 33-37)*.
3. Structural composite lumber columns have sides that show either the wide face or the edges of the lumber strands/veneers. Values in the tables reflect installation into the wide face. See technical bulletin T-SCLCOLUMN for values on the narrow face (edge) *(see page 232 for details)*.
4. Post design by Specifier. Tabulated loads are based on a minimum 3½" wide post (in a 3½" wall). Post may consist of multiple members provided they are connected independently of the holdown fasteners. See pages 226-227 for common post allowable loads.
5. Tension values are valid for holdowns flush or raised off of sill plate.
6. Deflection at Allowable Tension Load includes fastener slip, holdown deformation and anchor rod elongation for holdowns installed up to 6" above top of concrete. Holdowns may be installed raised up to 18" above top of concrete with no load reduction provided that additional elongation of the anchor rod is accounted for.
7. Tabulated loads may be doubled when the holdowns are installed on opposite sides of the wood member provided either the post is large enough to prevent opposing holdown screw interference or the holdowns are offset to eliminate screw interferences.
8. Noted HDU14 allowable loads are based on a 5½" wide post *(6x6 min.)*.
9. Requires heavy hex anchor nut to achieve tabulated loads *(supplied with holdown)*.
10. Loads are applicable to installation on either narrow or wide face of post.

HOLDOWNS CONSIDERED

# CS/CMST Coiled Straps

SIMPSON Strong-Tie®

CMSTC provides nail slots for easy installation; it can be cut to length. CS are continuous utility straps which can be cut to length on the job site. Packaged in lightweight *(about 40 pounds)* cartons.

**FINISH**: Galvanized. Some products available in ZMAX® coating; see Corrosion Information, page 14-15.

**INSTALLATION**: • Use all specified fasteners. See General Notes.

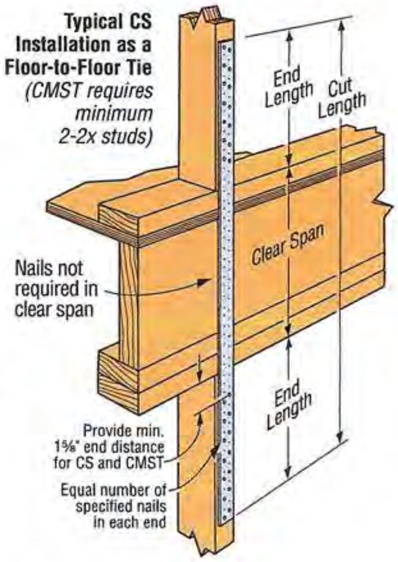
- Wood shrinkage after strap installation across horizontal wood members may cause strap to buckle outward.
- Refer to the applicable code for minimum nail penetration and minimum wood edge and end distances.
- The table shows the maximum allowable loads and the nails required to obtain them. Fewer nails may be used; reduce the allowable load as shown in footnote #3.
- The cut length of the strap shall be equal to twice the "End Length" noted in the table plus the clear span dimension.
- CMST only–Use every other round hole if the wood tends to split. Use round and triangle holes for comparable MST loads, providing wood does not tend to split.
- For lap slice and alternate nailing information, refer to technical bulletin T-CMST *(see page 230 for details)*.
- CS straps are available in 25' lengths, order CS14-R, CS16-R, CS18-R, CS20-R or CS22-R.

**CODES:** See page 13 for Code Reference Key Chart.

*These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.*

*These products are approved for installation with the Strong-Drive SD Structural-Connector screw. See page 27 for more information.*

**Typical CS Installation as a Floor-to-Floor Tie** *(CMST requires minimum 2-2x studs)*

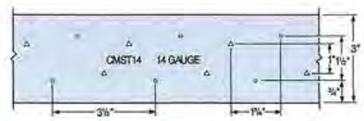



| Model No. | Total L | Ga | DF/SP Fasteners (160) | DF/SP End Length (160) | SPF/HF Fasteners (160) | SPF/HF End Length (160) | Allowable Tension Loads (160) | Code Ref. |
|---|---|---|---|---|---|---|---|---|
| CMST12 | 40' | 12 | 74 - 16d | 33" | 84 - 16d | 38" | 9215 | I4, L3, F2 |
| | | | 86 - 10d | 39" | 98 - 10d | 44" | 9215 | |
| CMST14 | 52½' | 14 | 56 - 16d | 26" | 66 - 16d | 30" | 6490 | |
| | | | 66 - 10d | 30" | 76 - 10d | 34" | 6490 | |
| CMSTC16 | 54' | 16 | 50 - 16d sinker | 20" | 58 - 16d sinker | 25" | 4585 | |
| CS14 | 100' | 14 | 26 - 10d | 15" | 30 - 10d | 16" | 2490 | |
| | | | 30 - 8d | 16" | 36 - 8d | 19" | 2490 | |
| CS16 | 150' | 16 | 20-10d | 11" | 22 - 10d | 12" | 1705 | |
| | | | 22 - 8d | 13" | 26 - 8d | 14" | 1705 | |
| CS18 | 200' | 18 | 16 - 10d | 9" | 18 - 10d | 10" | 1370 | |
| | | | 18 - 8d | 11" | 22 - 8d | 12" | 1370 | |
| CS20 | 250' | 20 | 12 - 10d | 6" | 14 - 10d | 8" | 1030 | |
| | | | 14 - 8d | 9" | 16 - 8d | 9" | 1030 | |
| CS22 | 300' | 22 | 10 - 10d | 7" | 12 - 10d | 7" | 845 | |
| | | | 12 - 8d | 6" | 14 - 8d | 8" | 845 | |

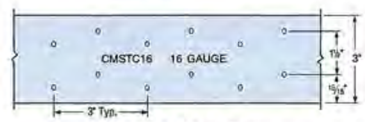
1. Loads include a 60% load duration increase on the fasteners for wind or seismic loading.
2. Use half of the required nails in each member being connected to achieve the listed loads.
3. Calculate the connector value for a reduced number of nails as follows: $\text{Allowable Load} = \frac{\text{No. of Nails Used}}{\text{No. of Nails in Table}} \times \text{Table Load}$

   *Example:* CMSTC16 in DF/SP with 40 nails total. *(Half of the nails in each member being connected)* $\text{Allowable Load} = \frac{\text{40 Nails (Used)}}{\text{50 Nails (Table)}} \times 4585 \text{ lbs} = 3668 \text{ lbs}$
4. Tension loads apply for uplift when installed vertically.
5. **NAILS:** 16d = 0.162" dia. x 3½" long, 16d Sinker = 0.148" dia. x 3¼" long, 10d = 0.148" dia. x 3" long. See page 22-23 for other nail sizes and information.


**CS16 Hole Pattern**
*(all other CS straps similar)*




**CMST14 Hole Pattern**
*(CMST12 similar)*



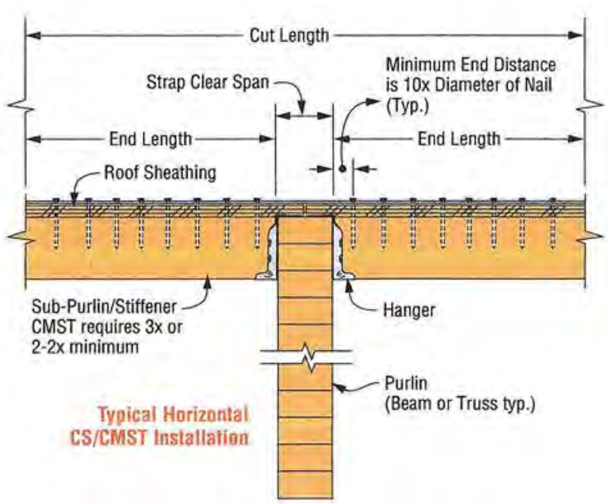

**CMSTC16 Hole Pattern**

*Gauge stamped on part for easy identification*

**NOT SURE HOW MUCH COIL STRAP YOU NEED?**

Simpson Strong-Tie has a web-based app, the Coil Strap Length Calculator, which can help you quickly determine the cut length of each strap and the total amount of coil strap needed for each application on a project.

For more information or to access, go to ***www.strongtie.com/software.***




**Typical Horizontal CS/CMST Installation**

Straps & Ties

# SDS & SD *Wood Screws*

The Simpson Strong-Tie® Strong-Drive® screw (SDS) is a ¼" diameter structural wood screw ideal for various connector installations as well as wood-to-wood applications. It installs with no predrilling and has been extensively tested in various applications. The new SDS is improved with a patented easy driving 4CUT™ tip and a corrosion resistant double-barrier coating.

The SD8 #8x1¼" wafer head screw is ideal for miscellaneous fastening applications. The needle point ensures fast starts and deep #2 Phillips drive reduces cam-out and stripping.

**SDS FEATURES:**

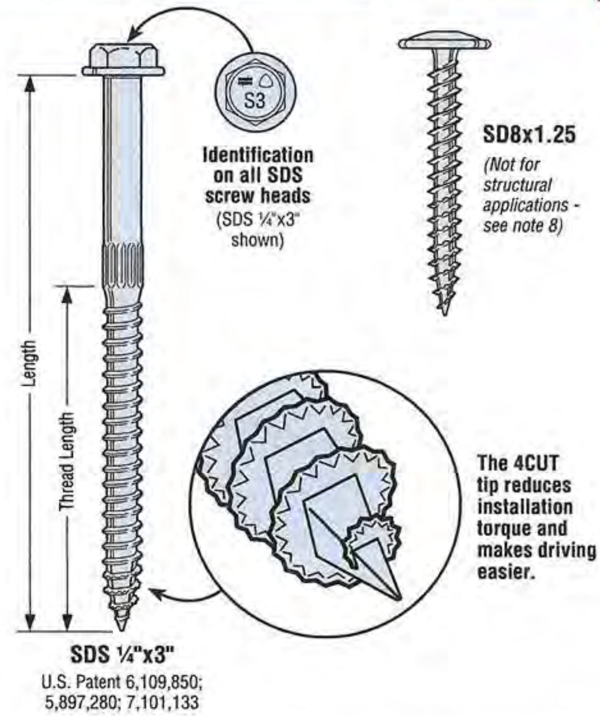
- The patented 4CUT tip has a square core and serrated threads to reduce installation torque and make driving easier with no predrilling and minimal wood splitting.
- A double-barrier coating finish provides corrosion resistance equivalent to hot-dip galvanization. Now one screw can handle interior, exterior and certain pressure-treated wood applications *(see Corrosion Information on page 14-15 for more information).*
- ⅜" hex washer head is stamped with the No-Equal sign and fastener length for easy identification after installation.

**MATERIAL:** Heat-treated carbon steel, Type-316 stainless steel

**FINISH:** SDS—Double-barrier coating. SDS screws may also be available yellow zinc dichromate or HDG *(Not all sizes are available in all coatings – Contact Simpson Strong-Tie for product availability and ordering information)*; SD8x1.25—Electro Galvanized.

**CODES:** See page 13 for Code Reference Key Chart.

**WARNING:** Industry studies show that hardened fasteners can experience performance problems in wet or corrosive environments. Accordingly, the SD8x1.25 should be used in dry, interior, and noncorrosive environments only.




**SDS ¼"x3"**
U.S. Patent 6,109,850; 5,897,280; 7,101,133

*These products feature additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.*

## SDS and SD Wood Screws

| Size (in.) | Model No. | Thread Length (in.) | Fasteners per Carton[6] | DF/SP Allowable Loads[4] | | | | | | SPF/HF Allowable Loads[4] | | | | | | Code Ref. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Shear (100)[1] | | | | | Withdrawal[5] (100) | Shear (100) | | | | | Withdrawal[5] (100) | |
| | | | | Wood Side Plate[3] | | Steel Side Plate | | | | Wood Side Plate[3] | | Steel Side Plate | | | | |
| | | | | 1½" | 1¾" SCL | 16 ga | 14 ga & 12 ga | 10 ga or Greater | Wood or Steel Side Plate | 1½" | 1¾" SPF LVL | 16 ga | 14 ga & 12 ga | 10 ga or Greater | Wood or Steel Side Plate | |
| ⁵⁄₃₂ x 1¼ | SD8x1.25[8] | — | — | — | — | 50 | 50 | 50 | — | — | — | 45 | 45 | 45 | — | 170 |
| ¼ x 1½ | SDS25112 | 1 | 1500 | — | — | 250 | 250 | 250 | 170 | — | — | 180 | 180 | 180 | 120 | I5, L1, F20 |
| ¼ x 2 | SDS25200 | 1¼ | 1300 | — | — | 250 | 290 | 290 | 215 | — | — | 180 | 210 | 210 | 150 | |
| ¼ x 2½ | SDS25212 | 1½ | 1100 | 190 | — | 250 | 390 | 420 | 255 | 135 | — | 180 | 280 | 300 | 180 | |
| ¼ x 3 | SDS25300 | 2 | 950 | 280 | — | 250 | 420 | 420 | 345 | 200 | — | 180 | 300 | 300 | 240 | |
| ¼ x 3½ | SDS25312 | 2¼ | 900 | 340 | 340 | 250 | 420 | 420 | 385 | 245 | 245 | 180 | 300 | 300 | 270 | |
| ¼ x 4½ | SDS25412 | 2¾ | 800 | 350 | 340 | 250 | 420 | 420 | 475 | 250 | 245 | 180 | 300 | 300 | 330 | |
| ¼ x 5 | SDS25500 | 2¾ | 500 | 350 | 340 | 250 | 420 | 420 | 475 | 250 | 245 | 180 | 300 | 300 | 330 | |
| ¼ x 6 | SDS25600 | 3¼ | 600 | 350 | 340 | 250 | 420 | 420 | 560 | 250 | 245 | 180 | 300 | 300 | 395 | |
| ¼ x 8 | SDS25800 | 3¼ | 400 | 350 | 340 | 250 | 420 | 420 | 560 | 250 | 245 | 180 | 300 | 300 | 395 | |

## Stainless-Steel SDS Wood Screws

| Size (in.) | Model No. | Thread Length (in.) | Fasteners per Carton[6] | DF/SP Allowable Loads[4] | | | | | | SPF/HF Allowable Loads[4] | | | | | | Code Ref. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Shear (100) | | | | | Withdrawal[5] (100) | Shear (100) | | | | | Withdrawal[5] (100) | |
| | | | | Wood Side Plate | | Steel Side Plate | | | | Wood Side Plate | | Steel Side Plate | | | | |
| | | | | 1½" | 1¾" SCL | 16 ga | 14 ga & 12 ga | 10 ga or Greater | Wood or Steel Side Plates | 1½" | 1¾" SCL | 16 ga | 14 ga & 12 ga | 10 ga or Greater | Wood or Steel Side Plates | |
| ¼ x 1½ | SDS25112SS | 1 | 1500 | — | — | 250 | 250 | 250 | 170 | — | — | 180 | 180 | 180 | 120 | I5, L1, F20 |
| ¼ x 2 | SDS25200SS | 1¼ | 1300 | — | — | 250 | 290 | 290 | 215 | — | — | 180 | 210 | 210 | 150 | |
| ¼ x 2½ | SDS25212SS | 1½ | 1100 | 190 | — | 250 | 390 | 420 | 255 | 135 | — | 180 | 280 | 300 | 180 | |
| ¼ x 3 | SDS25300SS | 2 | 950 | 280 | — | 250 | 420 | 420 | 345 | 200 | — | 180 | 300 | 300 | 240 | |
| ¼ x 3½ | SDS25312SS | 2¼ | 900 | 340 | 340 | 250 | 420 | 420 | 385 | 245 | 245 | 180 | 300 | 300 | 270 | |

1. Screws may be provided with the 4CUT or Type 17 tip.
2. SDS screws install best with a low speed ½" drill with a ⅜" hex head driver.
3. All applications are based on full thread penetration into the main member. For other wood side plate values, see *Fastening Systems* catalog (C-FS).
4. Allowable loads are shown at the wood load duration factor of $C_D$=1.00. Loads may be increased for load duration per the building code up to a $C_D$=1.60.
5. Withdrawal loads shown are in pounds (lbs.) and are based on the entire threaded section installed into the main member. If thread penetration into the main member is less than the Thread Length as shown in the table, reduce allowable load by 172 lbs. x inches of thread not in main member. Use 121 lbs./inch for SPF.
6. Fasteners per Carton represent the quantity of screws which are available in bulk packaging. Screws are also available in mini bulk and retail packs. Refer to Simpson Strong-Tie® List Price book. Contact Simpson Strong-Tie for more information.
7. LSL wood-to-wood applications that require 4½", 5", 6" or 8" SDS screws are limited to interior-dry use only.
8. SD8x1.25 requires ¾" minimum penetration. DO NOT USE SD8x1.25 wood screws with structural connectors unless specified and stated in this catalog.
9. Where predrilling is required for SDS, predrill diameter is ⁵⁄₃₂".
10. Minimum spacing edge and end distance requirements are listed in ICC-ES ESR-2236.

# LTP4/LTP5/A34/A35 *Framing Angles & Plates*

The larger LTP5 spans subfloor at the top of the blocking or rim joist. The embossments enhance performance.

The LTP4 Lateral Tie Plate transfers shear forces for top plate-to-rim joist or blocking connections. Nail holes are spaced to prevent wood splitting for single and double top plate applications. May be installed over plywood sheathing.

The A35 anchor's exclusive bending slot allows instant, accurate field bends for all two- and three-way ties. Balanced, completely reversible design permits the A35 to secure a great variety of connections.

**MATERIAL**: LTP4/LTP5–20 gauge; all others–18 gauge

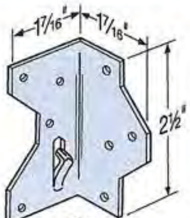
**FINISH**: Galvanized. Some products available in stainless steel or ZMAX® coating; see Corrosion Information, page 14-15.

**INSTALLATION**: • Use all specified fasteners. See General Notes.
• A35–Bend one time only.

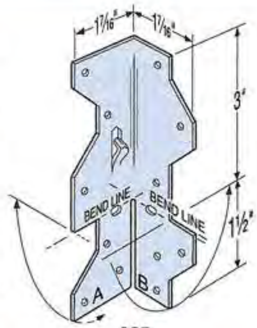
**CODES**: See page 13 for Code Reference Key Chart.

*These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.*

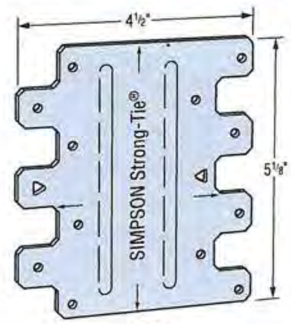
*These products are approved for installation with the Strong-Drive SD Structural-Connector screw. See page 27 for more information.*



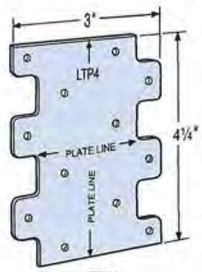
A34



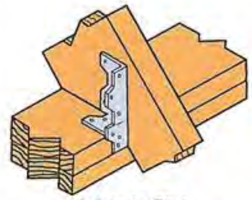
A35

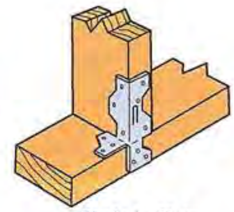


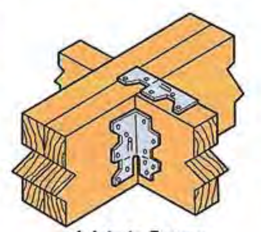
LTP5

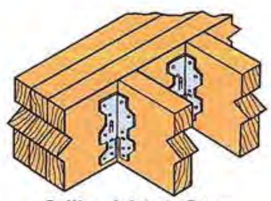


LTP4


Joists to Plate with A Leg Inside

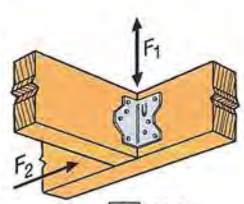

Studs to Plate with B Leg Outside


Joists to Beams

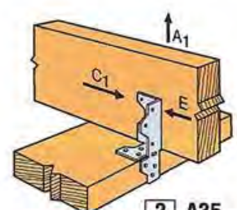

Ceiling Joists to Beam

| Model No. | Type of Connection | Fasteners | Direction of Load | DF/SP Allowable Loads | | | SPF/HF Allowable Loads | | | Code Ref. |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Floor (100) | Roof (125) | (160) | Floor (100) | Roof (125) | (160) | |
| A34 | 1 | 8-8dx1½ | $F_1$ | 395 | 485 | 515 | 340 | 415 | 445 | IP1, L5, L18, F13 |
| | | | $F_2$[6] | 395 | 455 | 455 | 340 | 390 | 390 | |
| A35 | 2 | 9-8dx1½ | $A_1$, E | 295 | 365 | 395 | 255 | 315 | 340 | |
| | | | $C_1$ | 210 | 210 | 210 | 180 | 180 | 180 | |
| | 3 | 12-8dx1½ | $A_2$ | 295 | 365 | 380 | 255 | 315 | 325 | |
| | | | $C_2$ | 295 | 365 | 370 | 255 | 315 | 320 | |
| | | | D | 230 | 230 | 230 | 200 | 200 | 200 | |
| | 4 | 12-8dx1½ | $F_1$ | 595 | 695 | 695 | 510 | 600 | 600 | |
| | | | $F_2$[6] | 595 | 670 | 670 | 510 | 575 | 575 | |
| LTP4 | 5 | 12-8dx1½ | G | 515 | 645 | 670 | 445 | 555 | 575 | I14, L5, F13 |
| | | | H | 515 | 645 | 670 | 445 | 555 | 575 | |
| LTP5 | 6 | 12-8dx1½ | G | 585 | 620 | 620 | 505 | 535 | 535 | IP1, L18, F25 |
| | | | H | 545 | 545 | 545 | 470 | 470 | 470 | |

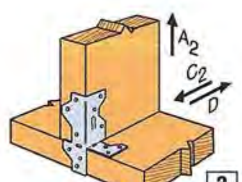
1. Allowable loads are for one anchor. When anchors are installed on each side of the joist, the minimum joist thickness is 3".
2. Some illustrations show connections that could cause cross-grain tension or bending of the wood during loading if not reinforced sufficiently. In this case, mechanical reinforcement should be considered.
3. LTP4 can be installed over ⅜" wood structural panel sheathing with 8dx1½ nails and achieve 0.72 of the listed load, or over ½" and achieve 0.64 of the listed load. 8d commons will achieve 100% load.
4. The LTP5 may be installed over wood structural panel sheathing up to ½" thick using 8dx1½ nails with no reduction in load.
5. Connectors are required on both sides to achieve $F_2$ loads in both directions.
6. **NAILS:** 8dx1½ = 0.131" dia. x 1½" long. See page 22-23 for other nail sizes and information.



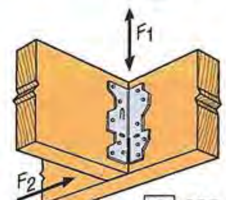
1 A34



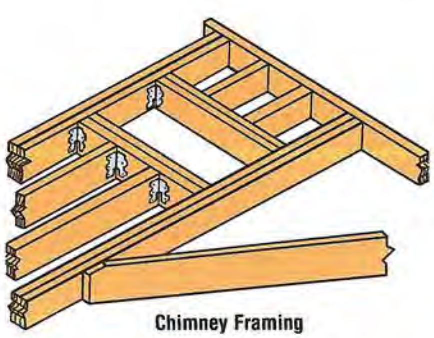
2 A35

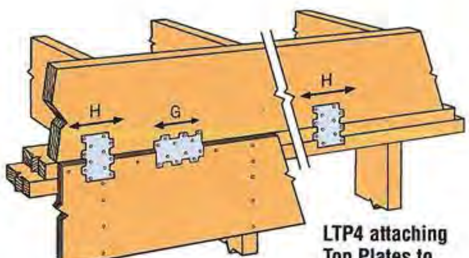


3 A35



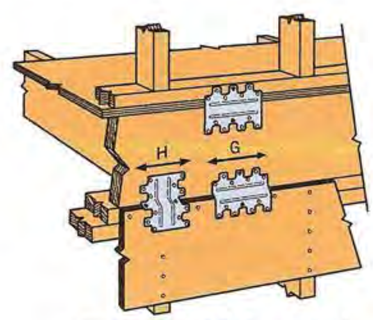
4 A35


Chimney Framing



5 LTP4 Installed over Wood Structural Panel Sheathing

LTP4 attaching Top Plates to Rim Joist



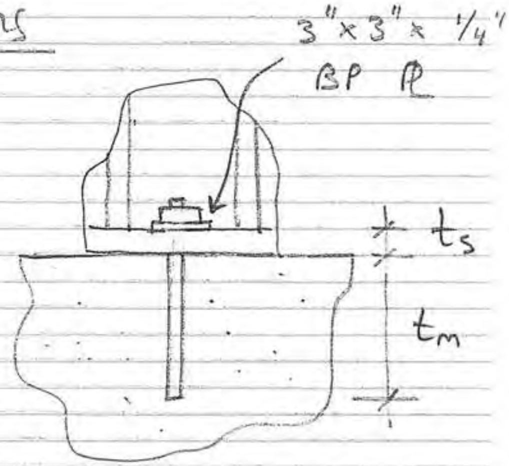
6 LTP5 Installed over Wood Structural Panel Sheathing

Straps & Ties

Project
Job No
By
Date
Sheet ____ of ____

North Bay Seismic Design
Structural Analysis and Design Services
PO Box 55, Inverness, CA 94937
Tel and Fax (415) 669-9678
www.NorthBaySeismicDesign.com

SHEAR WALL TO FOUNDATION ANCHORS




SOURCE: NDS Table 11 E

USING DF-L,

$t_m = 6.0"$ (embedment into concrete w/ $f'_c = 2000$ psi)

$t_s = 2.5"$ (3" sill PL per CBC 2305.3.11) w/ BP 5/8 - 3

$D = 5/8"$

$\boxed{\therefore Z_{\parallel} = 1140 \text{ lbs}}$

Factored Capacity (NDS Table 7.3.1)

$Z' = Z_{\parallel} \cdot C_D \cdot C_m \cdot C_t \cdot C_g \cdot C_\Delta$

$= (1140 \text{ lbs})(1.33)(1.0) \longrightarrow (1.0) = \boxed{Z' = 1516 \text{ lbs}}$

Required No. Anchors

$N = \dfrac{v \cdot b}{Z'}$ where $v = 319$ lb/ft (N-S Loading, Gridline (2), Basement wall 1)

$= \dfrac{(319 \text{ lb/ft})\, 9.42'}{1516 \text{ lbs}} = 1.98$ $b = 9.42'$

$Z' = 1516$ lbs

$\boxed{\therefore \text{USE } 2 - 5/8" \text{ ANCHORS}}$

Note: Remainder of foundation anchor checked in adjoining spreadsheet

## Table 11E BOLTS: Reference Lateral Design Values (Z) for Single Shear (two member) Connections[1,2,3,4]

for sawn lumber or SCL to concrete

| Embedment Depth in Concrete $t_m$ in. | Side Member $t_s$ in. | Bolt Diameter D in. | G=0.67 Red Oak $Z_{\parallel}$ lbs. | G=0.67 Red Oak $Z_{\perp}$ lbs. | G=0.55 Mixed Maple Southern Pine $Z_{\parallel}$ lbs. | G=0.55 Mixed Maple Southern Pine $Z_{\perp}$ lbs. | G=0.50 Douglas Fir-Larch $Z_{\parallel}$ lbs. | G=0.50 Douglas Fir-Larch $Z_{\perp}$ lbs. | G=0.49 Douglas Fir-Larch(N) $Z_{\parallel}$ lbs. | G=0.49 Douglas Fir-Larch(N) $Z_{\perp}$ lbs. | G=0.46 Douglas Fir(S) Hem-Fir(N) $Z_{\parallel}$ lbs. | G=0.46 Douglas Fir(S) Hem-Fir(N) $Z_{\perp}$ lbs. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 6.0 and greater | 1-1/2 | 1/2 | 770 | 480 | 680 | 410 | 650 | 380 | 640 | 380 | 620 | 360 |
| | | 5/8 | 1070 | 660 | 970 | 580 | 930 | 530 | 920 | 520 | 890 | 470 |
| | | 3/4 | 1450 | 890 | 1330 | 660 | 1270 | 590 | 1260 | 560 | 1230 | 520 |
| | | 7/8 | 1890 | 960 | 1750 | 720 | 1690 | 630 | 1680 | 600 | 1640 | 550 |
| | | 1 | 2410 | 1020 | 2250 | 770 | 2100 | 680 | 2060 | 650 | 1930 | 600 |
| | 1-3/4 | 1/2 | 830 | 510 | 740 | 430 | 700 | 400 | 690 | 390 | 670 | 370 |
| | | 5/8 | 1160 | 680 | 1030 | 600 | 980 | 550 | 970 | 550 | 940 | 530 |
| | | 3/4 | 1530 | 900 | 1390 | 770 | 1330 | 680 | 1310 | 660 | 1270 | 600 |
| | | 7/8 | 1970 | 1120 | 1800 | 840 | 1730 | 740 | 1720 | 700 | 1680 | 640 |
| | | 1 | 2480 | 1190 | 2290 | 890 | 2210 | 790 | 2200 | 750 | 2150 | 700 |
| | 2-1/2 | 1/2 | 830 | 590 | 790 | 520 | 770 | 470 | 760 | 460 | 750 | 440 |
| | | 5/8 | 1290 | 800 | 1230 | 670 | 1180 | 610 | 1170 | 610 | 1120 | 570 |
| | | 3/4 | 1840 | 1000 | 1630 | 850 | 1540 | 800 | 1520 | 780 | 1460 | 750 |
| | | 7/8 | 2290 | 1240 | 2050 | 1080 | 1940 | 1020 | 1920 | 1000 | 1860 | 920 |
| | | 1 | 2800 | 1520 | 2530 | 1280 | 2410 | 1130 | 2390 | 1080 | 2310 | 1000 |
| | 3-1/2 | 1/2 | 830 | 590 | 790 | 540 | 770 | 510 | 760 | 500 | 750 | 490 |
| | | 5/8 | 1290 | 880 | 1230 | 810 | 1200 | 730 | 1190 | 720 | 1170 | 670 |
| | | 3/4 | 1860 | 1190 | 1770 | 980 | 1720 | 900 | 1720 | 880 | 1680 | 830 |
| | | 7/8 | 2540 | 1410 | 2410 | 1190 | 2320 | 1100 | 2290 | 1070 | 2200 | 1020 |
| | | 1 | 3310 | 1670 | 2970 | 1420 | 2800 | 1330 | 2770 | 1300 | 2660 | 1260 |

| Embedment Depth in Concrete $t_m$ in. | Side Member $t_s$ in. | Bolt Diameter D in. | G=0.43 Hem-Fir $Z_{\parallel}$ lbs. | G=0.43 Hem-Fir $Z_{\perp}$ lbs. | G=0.42 Spruce-Pine-Fir $Z_{\parallel}$ lbs. | G=0.42 Spruce-Pine-Fir $Z_{\perp}$ lbs. | G=0.37 Redwood (open grain) $Z_{\parallel}$ lbs. | G=0.37 Redwood (open grain) $Z_{\perp}$ lbs. | G=0.36 Eastern Softwoods Spruce-Pine-Fir(S) Western Cedars Western Woods $Z_{\parallel}$ lbs. | G=0.36 Eastern Softwoods Spruce-Pine-Fir(S) Western Cedars Western Woods $Z_{\perp}$ lbs. | G=0.35 Northern Species $Z_{\parallel}$ lbs. | G=0.35 Northern Species $Z_{\perp}$ lbs. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 6.0 and greater | 1-1/2 | 1/2 | 590 | 340 | 590 | 340 | 550 | 310 | 540 | 290 | 530 | 290 |
| | | 5/8 | 860 | 420 | 850 | 410 | 810 | 350 | 800 | 330 | 780 | 320 |
| | | 3/4 | 1200 | 460 | 1190 | 450 | 1130 | 370 | 1120 | 360 | 1100 | 350 |
| | | 7/8 | 1580 | 500 | 1540 | 490 | 1360 | 410 | 1330 | 390 | 1280 | 370 |
| | | 1 | 1800 | 540 | 1760 | 530 | 1560 | 440 | 1520 | 420 | 1460 | 410 |
| | 1-3/4 | 1/2 | 640 | 360 | 630 | 350 | 580 | 320 | 580 | 310 | 560 | 310 |
| | | 5/8 | 910 | 490 | 900 | 480 | 840 | 400 | 830 | 380 | 810 | 370 |
| | | 3/4 | 1230 | 540 | 1220 | 530 | 1160 | 430 | 1140 | 420 | 1120 | 410 |
| | | 7/8 | 1630 | 580 | 1610 | 570 | 1540 | 470 | 1520 | 460 | 1490 | 430 |
| | | 1 | 2090 | 630 | 2060 | 610 | 1820 | 510 | 1770 | 490 | 1710 | 470 |
| | 2-1/2 | 1/2 | 730 | 410 | 730 | 400 | 700 | 360 | 690 | 340 | 680 | 340 |
| | | 5/8 | 1070 | 540 | 1060 | 530 | 980 | 480 | 960 | 470 | 940 | 460 |
| | | 3/4 | 1400 | 710 | 1380 | 700 | 1290 | 620 | 1270 | 600 | 1240 | 580 |
| | | 7/8 | 1790 | 830 | 1770 | 810 | 1660 | 680 | 1640 | 660 | 1600 | 610 |
| | | 1 | 2230 | 900 | 2210 | 880 | 2080 | 730 | 2060 | 700 | 2030 | 680 |
| | 3-1/2 | 1/2 | 730 | 470 | 730 | 470 | 700 | 430 | 690 | 410 | 690 | 400 |
| | | 5/8 | 1140 | 620 | 1140 | 610 | 1090 | 550 | 1080 | 530 | 1070 | 520 |
| | | 3/4 | 1650 | 780 | 1640 | 770 | 1540 | 680 | 1510 | 670 | 1470 | 660 |
| | | 7/8 | 2100 | 960 | 2070 | 950 | 1910 | 870 | 1880 | 850 | 1840 | 820 |
| | | 1 | 2550 | 1190 | 2520 | 1180 | 2340 | 1020 | 2310 | 980 | 2260 | 950 |

1. Tabulated lateral design values (Z) for bolted connections shall be multiplied by all applicable adjustment factors (see Table 10.3.1).
2. Tabulated lateral design values (Z) are for "full diameter" bolts (see Appendix L) with bending yield strength ($F_{yb}$) of 45,000 psi.
3. Tabulated lateral design values (Z) are based on dowel bearing strength ($F_e$) of 7,500 psi for concrete with minimum $f_c'$ = 2,500 psi.
4. Six inch anchor embedment assumed.

SIMPSON 1-BAY SINGLE STORY
FIXED BASE
SPECIAL MOMENT RESISTING FRAME 1
AT GRIDLINE 1

SIMPSON 1-BAY SINGLE STORY
FIXED BASE
SPECIAL MOMENT RESISTING FRAME 2
AT GRIDLINE 6