FEMA P-807-1

Calculation Package 1:

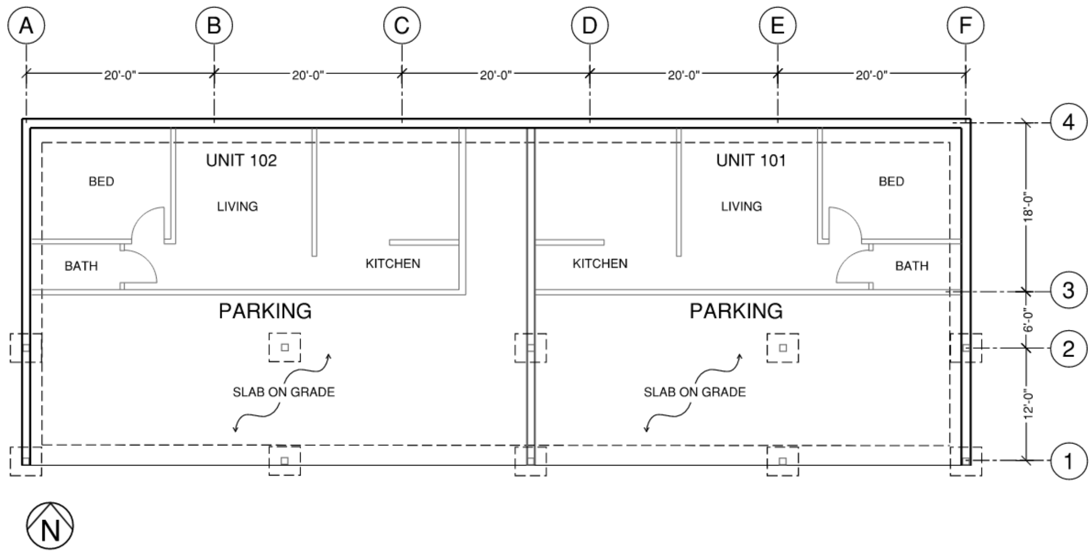
Optimized Line Retrofit Example

Section 5.3 Design Example Calculations Using Line Retrofit Method

1. Introduction

1.1 Building Description

The example building is located in Los Angeles, CA. Tuck-under parking is located along Grid Line 1. The building consists of wood, stucco, and steel gravity framing.



1.2 Detailed Weight Takeoff

$W_{Roof} := 12.2\ psf$ — Roof weight.

$W_{Floor3} := 11.2\ psf$ — 3rd floor weight.

$W_{Floor2} := 20.7\ psf$ — 2nd floor weight. Accounts for additional stucco weight above parking.

$W_{Deck} := 11.6\ psf$ — Entry deck weight.

$W_{IntWall} := 7\ psf$ — Interior wall weight.

$W_{Window} := 8\ psf$ — Window weight.

$W_{ExtWall} := 14.5\ psf \cdot 0.85 + W_{Window} \cdot 0.15 = 13.53\ psf$ — Exterior wall weight. Assumes 15% window openings.

1.3 Seismic Design Parameters

$R := 2.5$ — Response modification coefficient. LA ordinance.

$\Omega_0 := 1.25$ — Seismic overstrength factor. ASCE 7-16, Table 12.2-1.

$I_e := 1.0$ — Importance factor. ASC 7-16, Section 11.5.1

$\rho := 1.0$ — Redundancy factor. See Chapter 3 for comment.

$S_s := 1.975\ g$ — Short period response acceleration. USGS Hazard Map.

$S_1 := 0.705\ g$ — 1-second spectral response acceleration. USGS Hazard Map.

2. Base Shear and Vertical Force Distribution

The following calculations are based on the 2013 Los Angeles Soft-Story Equivalent Lateral Force Procedure using a site class assignment of "D" and an occupation category of "II". An address of 220 North Spring Street, Los Angeles, CA is assumed.

2.1 Spectral Accelerations

$S_s = 1.98 \quad g$

$S_1 = 0.71 \quad g$

$F_a := 1.00$ — Site coefficient. ASCE 7-16, Table 11.4-1.

$F_v := 1.50$ — Site coefficient. ASCE 7-16, Table 11.4-2.

$S_{MS} := F_a \cdot S_s = 1.98\, g$ — Modified show period acceleration. ASCE 7-16, EQ 11.4-1.

$S_{M1} := F_v \cdot S_1 = 1.06\, g$ — Modified 1-second period acceleration. ASCE 7-16, EQ 11.4-2.

$S_{DS} := \frac{2}{3} S_{MS} = 1.32 g$ — Design short period acceleration. ASCE 7-16, EQ 11.4-3.

$S_{D1} := \frac{2}{3} S_{M1} = 0.71\, g$ — Design 1-second period acceleration. ASCE 7-16, EQ 11.4-4.

2.2 Building Period

$H := 27\ ft$ — Total building height, base to roof.

$C_t := 0.02$ — Building system coefficient. ASCE 7-16, Table 12.8-2.

$x := 0.75$ — Period determination exponent. ASCE 7-16, Table 12.8-2.

$n_{story} := 3$ — Number of stories

$T_L := 8 \quad sec$ — Long-period transition period. ASCE 7-16, Tables 22-15 to 22-20.

$T_S := \frac{S_{D1}}{S_{DS}} = 0.54 \quad sec$ — ASCE 7-16, Section 11.4.5.

$T_0 := 0.2 \cdot T_S = 0.11\ sec$ — ASCE 7-16, Section 11.4.5.

$T_a := C_t \cdot \left(\frac{H}{ft}\right)^x = 0.24 \quad sec$ — Approximate fundamental period of vibration. ASCE 7-16, EQ 21.8-7.

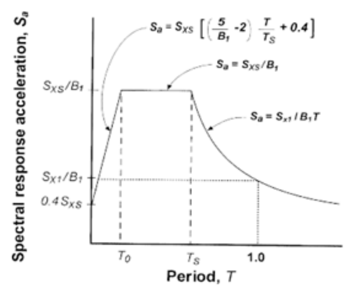
$C_u := 1.4$ — ASCE 7-16, Table 12.8-1.

$T_{lim} := C_u \cdot T_a = 0.33 \quad sec$ — Limiting structural period.

$T := min\left(T_a, T_{lim}\right) = 0.24 \quad sec$ — Design structural period. ASCE 7-16, Section 12.8.2.

2.3 Response Spectrum

Figure 1-1 General Horizontal Response Spectrum



$T_0 = 0.11 \quad sec$

$T_S = 0.54 \quad sec$

$T = 0.24 \quad sec$

$S_a := S_{DS} = 1.32 \; sec$

2.4 Base Shear

$W_{Roof} = 12.2 \; \boldsymbol{psf}$

$W_{Floor3} = 11.2 \; \boldsymbol{psf}$

$W_{Floor2} = 20.7 \; \boldsymbol{psf}$

$W_{Deck} = 11.6 \; \boldsymbol{psf}$

$W_{IntWall} = 7 \; \boldsymbol{psf}$

$W_{ExtWall} = 13.53 \; \boldsymbol{psf}$

$L := 100 \; \boldsymbol{ft}$ Building length.

$W := 36 \; \boldsymbol{ft}$ Building width.

$h_{wall} := 8 \; \boldsymbol{ft}$ Wall height.

$deck := 5 \; \boldsymbol{ft}$ Entry deck width.

$$Roof := L \cdot W \cdot \left(W_{Roof} + \frac{W_{IntWall}}{2} \right) + 2 \left(L + W \right) \left(\frac{h_{wall}}{2} \cdot W_{ExtWall} \right) = 71.24 \; \boldsymbol{kip}$$

$$Story_2 := L \cdot W \cdot \left(W_{Floor3} + W_{IntWall} \right) + \left(2 \left(L + W \right) \left(\frac{h_{wall}}{2} \right) + 2 \left(L + W \right) \left(\frac{h_{wall}}{2} \right) \right) \left(W_{ExtWall} \right) + W_{Deck} \cdot L \cdot deck = 100.75 \; \boldsymbol{kip}$$

$$Story_1 := L \cdot \frac{W}{2} \left(W_{Floor2} + W_{IntWall} \right) + L \cdot \frac{W}{2} \left(W_{Floor3} + W_{IntWall} \right) + \left(2 \left(L + W \right) \cdot \frac{h_{wall}}{2} + 2 \left(L + W \right) \frac{h_{wall}}{2} \right) \cdot W_{ExtWall} + W_{Deck} \cdot L \cdot deck = 117.85 \; \boldsymbol{kip}$$

$W_{Total} := Roof + Story_2 + Story_1 = 289.84 \; \boldsymbol{kip}$ Building seismic weight.

$$C_s := \frac{S_{DS}}{\left(\frac{R}{I_e} \right)} = 0.53$$

Controlling seismic response coefficient. ASCE 7-16, Section 12.8.1.1.

$V' := C_s \cdot W_{Total} = 152.65 \; \boldsymbol{kip}$ Base shear. ASCE 7-16, EQ 12.8-1.

$V := 0.75 \cdot V' = 114.49 \; \boldsymbol{kip}$ LA Ordinance design base shear.

2.5 Line Forces

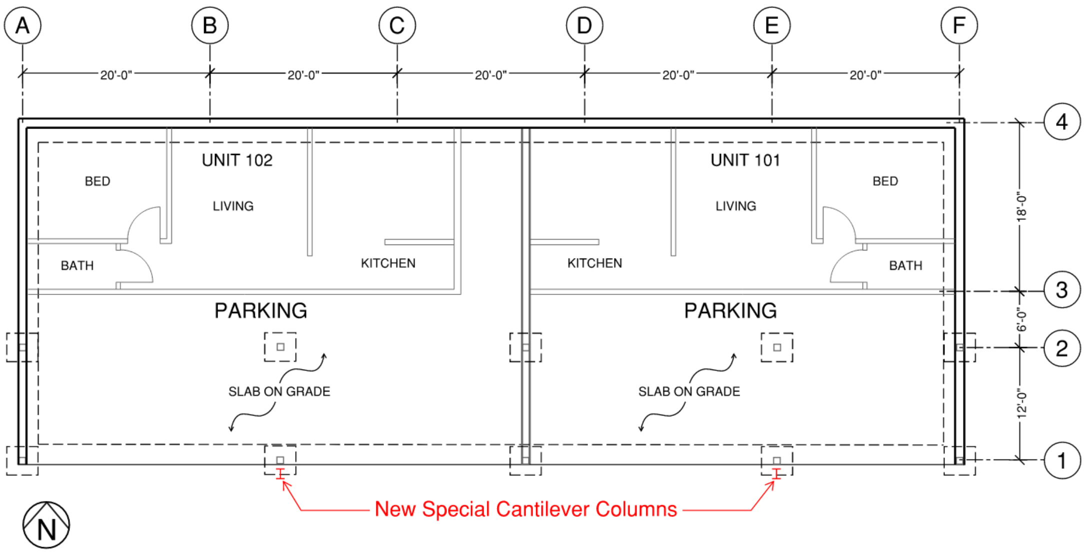
$$w_{parking} := \frac{6 \; \boldsymbol{ft} + 12 \; \boldsymbol{ft}}{2} = 9 \; \boldsymbol{ft}$$

Parking garage tributary width.

$V := \frac{w_{parking} \cdot L}{L \cdot W}$ $V = 28.62\ kip$ Tributary force on retrofit line.

3. Special Steel Cantilever Column Design

Design retrofit using a pair of special cantilever columns.



3.1 Column Information

$R_y := 1.1$ Expected yield strength factor. AISC, Table A3.1

$F_y := 50\ ksi$ Yield strength

$E := 29000\ ksi$ Modulus of elasticity

$h_{wall} = 8\ ft$

$L_{GB} := 50\ ft$ Length of concrete grade beam.

3.2 Seismic Demands

$V = 28.62\ kip$

$V_{ue} := \rho \cdot V = 28.62\ kip$ Factored story shear. ASCE 7-16, Chapter 12.

Resistance Factors

$\phi_c := 0.9$ Compression resistance factor. AISC Specification, Chapter E.

$\phi_b := 0.9$ Moment resistance factor. AISC Specification, Chapter F.

$\phi_t := 0.9$ Tension resistance factor. AISC Specification, Chapter D.

$\phi_v := 1.0$ Shear resistance factor. AISC Specification, Chapter G.

3.3 Column Sizing

Use a W8x40 column and design as special cantilever column.

$A_g := 11.7\ in^2$	AISC, Table 1-1.
$d := 8.25\ in$	AISC, Table 1-1.
$t_w := 0.36\ in$	AISC, Table 1-1.
$Z_x := 39.8\ in^3$	AISC, Table 1-1.
$L_{unbr} := h_{wall} = 8\ ft$	Unbraced length.
$K := 1.0$	Effective length factor.
$r_y := 2.04\ in$	AISC, Table 1-1.

$$\frac{K \cdot L_{unbr}}{r_y} = 47.06 \qquad 4.71\sqrt{\frac{E}{F_y}} = 113.43$$

$$F_e := \frac{\pi^2\ E}{\left(\frac{K \cdot L_{unbr}}{r_y}\right)^2} = 129.25\ ksi$$

Elastic critical buckling stress. AISC, EQ E3-4.

$$F_{cr} := 0.658^{\frac{F_y}{F_e}}\ F_y = 42.53\ ksi$$

Flexural buckling stress. AISC EQ E3-2.

$$M_n := F_y \cdot Z_x = 165.83\ kip \cdot ft$$

$$\phi_b M_n := \phi_b \cdot M_n = 149.25\ kip \cdot ft$$

Available flexural strength.

$C_v := 1.0$ — AISC Specification, Chapter G.

$A_w := d \cdot t_w = 2.97\ in^2$

$$\phi_v V_n := \phi_v \cdot 0.6 \cdot F_y \cdot A_w \cdot C_v = 89.1\ kip$$

Available shear strength. AISC Specification, Chapter G.

3.3.1 Seismic b/t & h/t Requirements

$btf_{ratio} := 7.21$ — AISC, Table 1-1.

$$\lambda_{hd\_flange} := 0.32 \cdot \sqrt{\frac{E}{R_y \cdot F_y}} = 7.35$$

AISC 314-16, Table D1.1.

Compactness < Required; OK.

$htw_{ratio} := 17.6$ — AISC, Table 1-1.

$$\lambda_{hd\_web} := 2.57 \cdot \sqrt{\frac{E}{R_y \cdot F_y}} = 59.01$$

AISC 314-16, Table D1.1.

Compactness < Required; OK. Pu assumed to be zero, therefore Ca is zero.

3.4 Column Demand Loads

(N) Special Cantilever Columns are not part of gravity system. These retrofit elements are designed to only support lateral seismic loads.

$V_{ue} = 28.62\ \boldsymbol{kip}$ — Factored story shear

$V_u := \dfrac{V_{ue}}{2} = 14.31\ \boldsymbol{kip}$ — Shear demand load per column.

$h_{wall} = 8\ \boldsymbol{ft}$

$M_u := V_u \cdot h_{wall} = 114.49\ \boldsymbol{kip} \cdot \boldsymbol{ft}$ — Moment demand load per column.

3.5 Column Demand-to-Capacity Checks

$$DCR_M := \frac{M_u}{\phi_b M_n} = 0.77$$

$$DCR_V := \frac{V_u}{\phi_v V_n} = 0.16$$

3.6 Column Stability Bracing

Member requirements for Special Cantilever Column Systems are specified in AISC 341-16, Section E6. Stability bracing then points to Section D1.2a for moderately ductile members.

$r_y = 2.04\ \boldsymbol{in}$

$L_b := \dfrac{0.19\ r_y \cdot E}{R_y \cdot F_y} = 17.03\ \boldsymbol{ft}$ — Maximum bracing spacing. AISC 341-16, EQ D1-2.

$h_{wall} = 8\ \boldsymbol{ft}$

$\dfrac{L_b}{h_{wall}} = 2.13$ — Required bracing spacing is greater than twice the height of the special cantilever column. Stability bracing not required.

3.7 Column Torsion

Due to the eccentricity of the (N) Special Cantilever Columns from the exterior line of the building, there will be torsion induced in these members at their top connection. To brace the columns, angles are used in a configuration forming a truss. This spreads out the lateral force, reducing the point loads entering the diaphragm.

$V_u = 14.31\ \boldsymbol{kip}$

$e_{col} := 1.33\ \boldsymbol{ft}$

$Torsion_{col} := V_u \cdot e_{col} = 19.03\ \boldsymbol{kip} \cdot \boldsymbol{ft}$

$d = 8.25\ \boldsymbol{in}$

Use 3x3x1/2 L angles to form truss tying the column to the collector.

$d_{angle} := 3\ in$ AISC Table 1-7

$b_{angle} := 3\ in$ AISC Table 1-7

$t_{angle} := 0.5\ in$ AISC Table 1-7

$k_{angle} := 0.875\ in$ AISC Table 1-7

$A_{g\_angle} := 2.76\ in^2$ AISC Table 1-7

$S_y := 1.06\ in^3$ AISC Table 1-7

$r_z := 1.58\ in$ AISC Table 1-7

$$bt_{ratio\_angle} := \frac{b_{angle}}{t_{angle}} = 6$$

$F_{y\_angle} := 36\ ksi$ AISC Table 2-4

$F_{u\_angle} := 58\ ksi$ AISC Table 2-4

$$TC_{col} := \frac{Torsion_{col}}{16\ in \cdot 3} = 4.76\ kip$$

Assume (E) floor joists are 16in O.C.

$b_f := 8.07\ in$ Column flange width; AISC Table 1-1.

Tension:

$\phi_{ty} := 0.9$

$\phi_{tr} := 0.75$

$l_1 := b_{angle} - k_{angle} = 2.13\ in$

$l_2 := l_1 = 2.13\ in$

$$l_{weld} := \frac{l_1 + l_2}{2} = 2.13\ in$$

$w := b_{angle} = 3\ in$

$x_{bar} := 0.929\ in$ AISC Table 1-7

$$U := \frac{3\ {l_{weld}}^2}{3\ {l_{weld}}^2 + w^2} \left(1 - \frac{x_{bar}}{l_{weld}}\right) = 0.34$$

AISC Table D3.1

$A_{e\_angle} := A_{g\_angle} \cdot U = 0.93\ in^2$ AISC EQ D3-1

$$P_n := min\left(\phi_{ty} \cdot F_{y\_angle} \cdot A_{g\_angle}, \phi_{tr} \cdot F_{u\_angle} \cdot A_{e\_angle}\right) = 40.6\ kip$$

AISC EQ's D2-1 & D2-2

$$DCR_{angle\_T} := \frac{TC_{col}}{P_n} = 0.12$$

Compression:

$bt_{ratio\_angle} = 6$

$$0.45 \sqrt{\frac{E}{F_{y\_angle}}} = 12.77$$

Angle is compact for compression.

$K_{angle} := 1.0$ Assume pin-pin connection for truss

$r_{angle} := 0.895\ in$ AISC Table 1-7

$L_{b\_angle} := e_{col} - \frac{b_f}{2} - 4\ in = 7.93\ in$ Assume existing collecter is W8x

$\frac{K_{angle} \cdot L_{b\_angle}}{r_{angle}} = 8.85$ $4.71 \sqrt{\frac{E}{F_y}} = 113.43$

$F_{e\_angle} := \frac{\pi^2 E}{\left(\frac{K_{angle} \cdot L_{b\_angle}}{r_{angle}}\right)^2} = 3650.44\ ksi$ AISC EQ E3-4

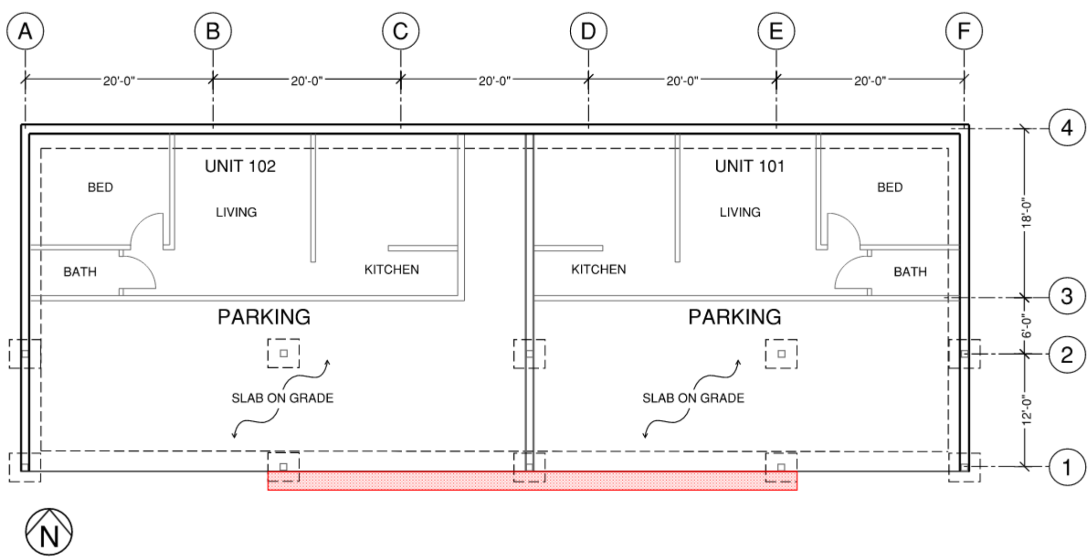
$F_{cr\_angle} := \left(0.658^{\frac{F_y}{F_{e\_angle}}}\right) F_y = 49.71\ ksi$ AISC EQ E3-2

$P_{n\_angle} := F_{cr\_angle} \cdot A_{g\_angle} = 137.21\ kip$ AISC EQ E3-1

$DCR_{angle\_FB} := \frac{TC_{col}}{\phi_c \cdot P_{n\_angle}} = 0.04$

4. Concrete Grade Beam Design

Design grade beam to span across the middle two bays along Grid Line 1. Total length will be the out-to-out distance of the existing column footings. As best practice, grade beam should be doweled to adjacent (E) foundation. Any vertical bearing capacity lost when (E) footings are demolished should be restored.



4.1 Grade Beam Dimensions

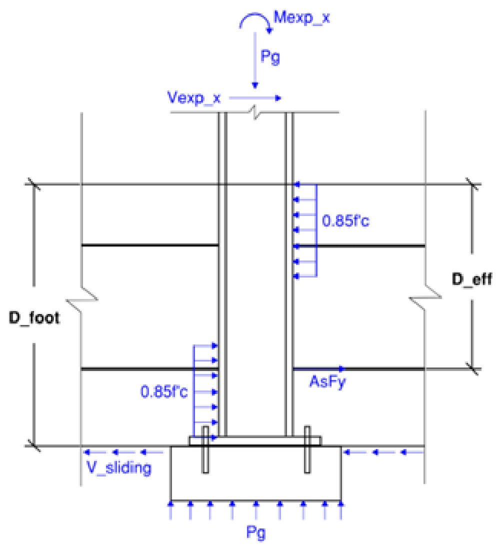
$L_{foot} := 56\ ft$ — Total length of footing.

$B_{foot} := 24\ in$ — Width of footing.

$D_{foot} := 24\ in$ — Depth of footing.

$A_{foot} := L_{foot} \cdot B_{foot} = 112\ ft^2$

4.2 Grade Beam Reinforcement



$d_{\#6} := 0.75\ in$ — Diameter of #6 bar.

$A_{\#6} := \dfrac{\pi \cdot \left(d_{\#6}\right)^2}{4} = 0.44\ in^2$ — Area of #6 bar.

$A_s := 5 \cdot A_{\#6} = 2.21\ in^2$ — Use (5) #6 bars.

$f'_c := 4000\ psi$ — Concrete compressive strength.

$f_y := 60\ ksi$ — Steel reinforcement yield strength.

$D_{eff} := D_{foot} - 3\ in = 21\ in$ — Effective footing depth.

$A_{s\_min} := \max\left(\dfrac{3 \cdot \sqrt{f'_c \cdot psi}}{f_y} \cdot B_{foot} \cdot D_{eff}, \dfrac{200\ psi}{f_y} \cdot B_{foot} \cdot D_{eff}\right) = 1.68\ in^2$ — ACI 318-19, Section 9.6.1.2; As > As_min; OK.

4.3 Grade Beam Demands

$R_y = 1.1$

$F_y = 50\ ksi$

$Z_x = 39.8\ in^3$

$M_{exp\_x} := \phi_b \cdot R_y \cdot F_y \cdot Z_x = 164.18\ kip \cdot ft$ — Expected moment capacity of column for in-plane direction. ASIC, EQ D1-1.

$h_{wall} = 8\ ft$

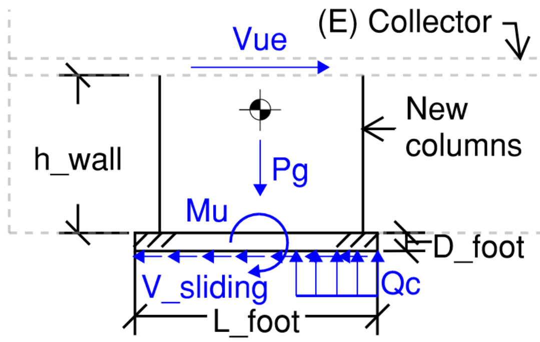
$V_{exp\_x} := \dfrac{M_{exp\_x}}{h_{wall}} = 20.52\ kip$ — Expected shear capacity from expected moment capacity

4.4 Grade Beam Demand-to-Capacity Check

$M_{cap\_x} := A_s \cdot f_y \cdot D_{eff} = 231.94\ kip \cdot ft$ — Moment capacity of footing.

$DCR_{Mx} := \dfrac{M_{exp\_x}}{M_{cap\_x}} = 0.71$

4.5 Global Soil Bearing



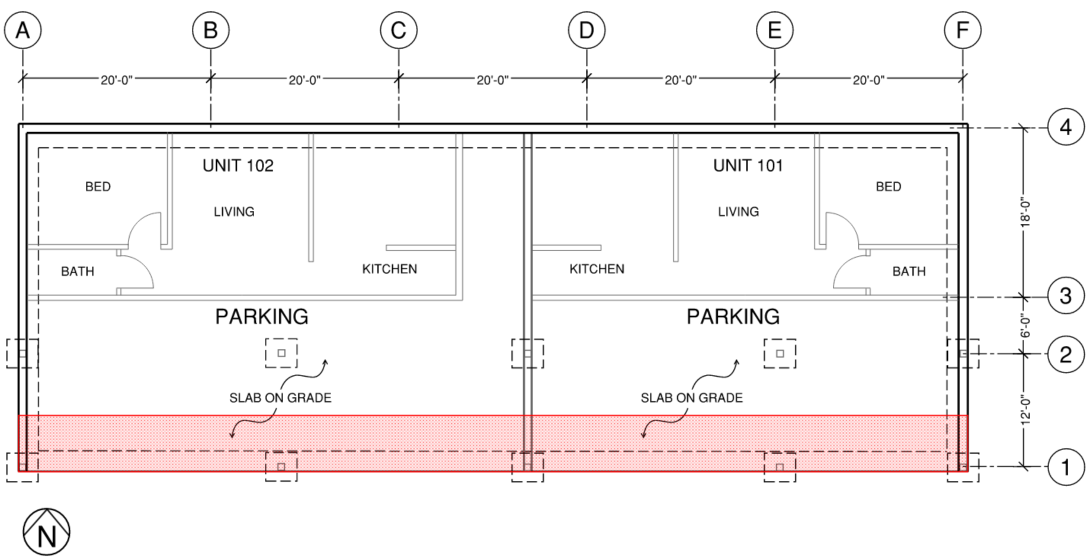
$h_{story} := 9\ ft$ — Story height

$W_{col} := 40\ plf$ — Weight of column per foot

$M_{u\_bear} := 0.7 \cdot V \cdot (h_{story} + D_{foot}) = 220.38\ kip \cdot ft$ — Conversion factor applied to compare with ASD presumptive soil bearing capacities in CBC

$P_{col} := W_{col} \cdot h_{wall} \cdot 2 = 0.64\ kip$ — Weight of (2) columns.

$P_{foot} := 150\ pcf \cdot D_{foot} \cdot B_{foot} \cdot L_{foot} = 33.6\ kip$ — Weight of footing. Conservatively assume full weight of footing for foundation.



$A_{trib} := L \cdot \frac{12\ ft}{2} = 600\ ft^2$ — Tributary area

$W_{Floor2} = 20.7\ psf$

$W_{Roof} = 12.2\ psf$

$W_{ExtWall} = 13.53\ psf$

$W_{IntWall} = 7\ psf$

$LL := 40\ psf$ — Interior live load. LA Information bulletin.

$LL_{roof} := 20\ psf$ — Roof live load. LA Information bulletin.

$W_{roof} := \left(1.2\ W_{Roof} + LL_{roof}\right) \cdot A_{trib} = 20.78\ \boldsymbol{kip}$

Tributary weight of roof

$$W_{3rd} := \left(1.2\ W_{Floor3} + LL\right) \cdot A_{trib} + 1.2\ W_{ExtWall} \cdot \left(L + 2 \cdot \frac{12\ ft}{2}\right) \cdot h_{wall} + 1.2\ W_{IntWall} \cdot A_{trib} = 51.65\ \boldsymbol{kip}$$

Tributary weight of 3rd floor

$$W_{2nd} := \left(1.2\ W_{Floor2} + LL\right) \cdot A_{trib} + 1.2\ W_{ExtWall} \cdot \left(L + 2 \cdot \frac{12\ ft}{2}\right) \cdot h_{wall} + 1.2\ W_{IntWall} \cdot A_{trib} = 58.49\ \boldsymbol{kip}$$

Tributary weight of 2nd floor

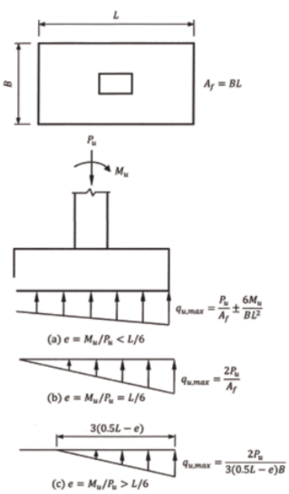
$P_{trib} := W_{2nd} + W_{3rd} + W_{roof} = 130.92\ \boldsymbol{kip}$

$P_g := P_{col} + P_{foot} + P_{trib} = 165.16\ \boldsymbol{kip}$

$e := \dfrac{M_{u\_bear}}{P_g} = 1.33\ \boldsymbol{ft}$ $\qquad x := \dfrac{L_{foot}}{6} = 9.33\ \boldsymbol{ft}$

Values compared on following page.

$$q_{max} := \begin{cases} \dfrac{P_g}{A_{foot}} + \left(\dfrac{6\ M_u}{B_{foot} \cdot L_{foot}^{\ 2}}\right) & \text{if } e < x \\ \dfrac{2\ P_g}{3\ \left(0.5\ L_{foot} - e\right)\ B_{foot}} & \text{else if } e > x \\ \dfrac{2\ P_g}{A_{foot}} & \text{else} \end{cases} = 1584.13\ \boldsymbol{psf}$$



$$q_{min} := \frac{P_g}{L_{foot} \cdot B_{foot}} - \left(\frac{6\ M_u}{B_{foot} \cdot L_{foot}^{\ 2}}\right) = 1365.09\ \boldsymbol{psf}$$

$q_a := 1500\ \boldsymbol{psf} \cdot 1.3 = 1950\ \boldsymbol{psf}$

Assumed soil bearing value with seismic factor.

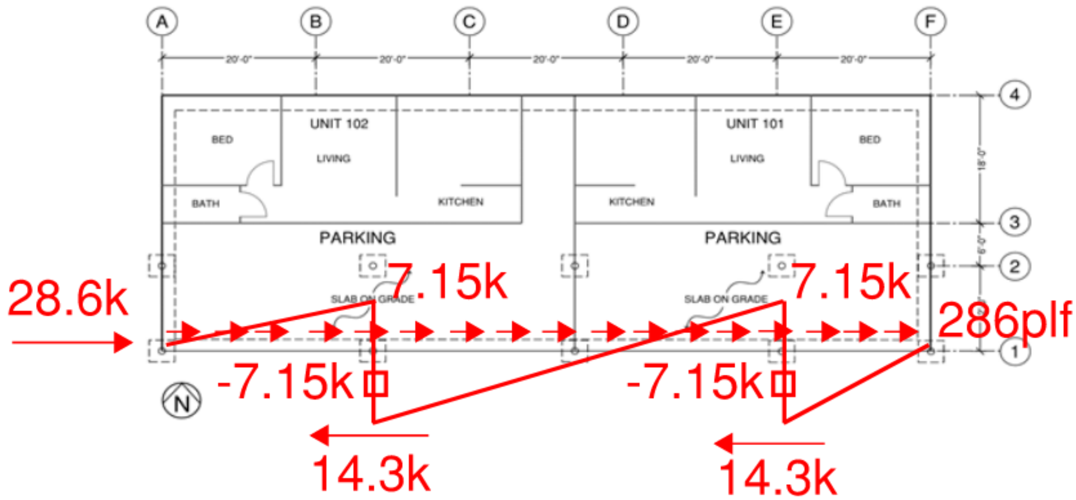
$DCR_{soil} := \dfrac{q_{max}}{q_a} = 0.81$

4.6 Global Sliding

As discussed in Chapter 4, meeting sliding resistance is not required. Instead, best practice is to dowel the new foundation into the existing foundation for sliding resistance. When detailing in this manner, deformation compatability between the new and existing foundations must be considered.

5. Steel Collector Design

Design collector to run full length of open front.



$V_{ue'} := \dfrac{V_{ue}}{L} = 286.21\ plf$

$P_{uL} := V_{ue'} \cdot 25\ ft = 7.16\ kip$ Load acting on collector on building left.

$V_{exp\_x} = 20.52\ kip$

$V_u = 14.31\ kip$

$\Omega_0 := \dfrac{V_{exp\_x} \cdot R_y}{V_u} = 1.58$

$P_{uL} := \Omega_0 \cdot P_{uL} = 11.29\ kip$

5.1 Collector Sizing

Determine smallest acceptable beam required. Compare this size with existing collector beam to determine if adequate for retrofit. Try a W8x31 beam.

$A_{g\_coll} := 9.13\ in^2$ AISC, Table 1-1.

$L_{b\_coll} := 30\ ft$ Maximum unbraced length.

$K = 1$

$r_{y\_coll} := 2.02\ in$ AISC, Table 1-1.

$\dfrac{K \cdot L_{b\_coll}}{r_{y\_coll}} = 178.22$ $\quad 4.71\sqrt{\dfrac{E}{F_y}} = 113.43$

$$F_{e\_coll} := \frac{\pi^2\ E}{\left(\frac{K \cdot L_{b\_coll}}{r_{y\_coll}}\right)^2} = 9.01\ ksi$$

Elastic critical buckling stress. AISC, EQ E3-4.

$$F_{cr\_coll} := 0.877\ F_{e\_coll} = 7.9\ ksi$$

Flexural buckling stress. AISC EQ E3-3.

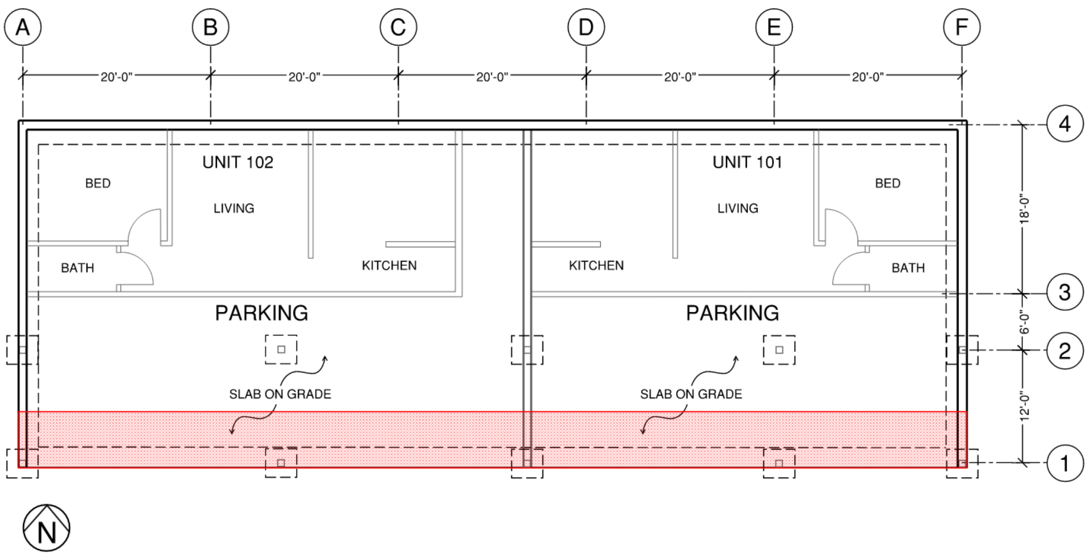
$$\phi_c P_{n\_coll} := \phi_c \cdot F_{cr\_coll} \cdot A_{g\_coll} = 64.94\ kip$$

Nominal compressive strength. AISC EQ E3-1.

$$\phi_t P_{n\_coll} := \phi_t \cdot F_y \cdot A_{g\_coll} = 410.85\ kip$$

Available tensile strength.

5.2 Gravity Loading on Collector



$$A_{trib} = 600\ ft^2$$

$$W_{Floor2} = 20.7\ psf$$
$$W_{Roof} = 12.2\ psf$$
$$W_{ExtWall} = 13.53\ psf$$
$$W_{IntWall} = 7\ psf$$

$$LL = 40\ psf$$
$$LL_{roof} = 20\ psf$$

$$W_{2nd} = 58.49\ kip$$

$$W_g := \frac{W_{2nd}}{L} = 0.58\ \frac{kip}{ft}$$

Distributed weight across full collector length.

$$M_{u\_coll} := \frac{W_g \cdot (L_{b\_coll})^2}{8} = 66\ kip \cdot ft$$

Maximum moment on longest unbraced length of collector.

$M_{n\_coll} := F_y \cdot 27.5\ in^3 = 114.58\ kip \cdot ft$

Available strong-axis moment capacity. AISC, Table 1-1

5.3 Collector Demand-to-Capacity Checks

$$DCR_{P\_coll} := \frac{P_{uL}}{\phi_c P_{n\_coll}} = 0.17$$

$$DCR_{T\_coll} := \frac{P_{uL}}{\phi_t P_{n\_coll}} = 0.03$$

$$DCR_{M\_coll} := \frac{M_{u\_coll}}{M_{n\_coll}} = 0.57$$

$$DCR_{collector} := \begin{array}{l} \text{if } DCR_{P\_coll} \geq 0.2 \\ \quad \left\| DCR_{P\_coll} + \frac{8}{9} DCR_{M\_coll} \right. \\ \text{else} \\ \quad \left\| \frac{1}{2} DCR_{P\_coll} + DCR_{M\_coll} \right. \end{array} = 0.66$$

Existing collector must be minimum of W8x31 to be sufficient for retrofit.

5.4 Collector Connection to Diaphragm

It is assumed the existing diaphragm-to-collector load path is adequate in this example. However, the design professional should confirm the field conditions provide this to be true based on the demands to the strengthening system.

$V_{ue} = 28.62\ kip$ — Tributary force on retrofit line

$L = 100\ ft$ — Collector length

$V_{ue'} = 286.21\ plf$ — Collector load entering diaphragm

5.5 Connections to Diaphragm for Column Torsion

The tension tie and clips presented here are to resolve the torsion couple created by eccentricity of in-plane forces.

LTTP2 Tension Tie:

$T_{LTTP2\_ASD} := 2320\ lbf$ — Simpson Strong Tie Wood Construction Connectors 2021-2023

$K_F := 3.32$ — NDS 2015 Table N1

$\phi_z := 0.65$ — NDS 2015 Table N2

$\lambda := 1.0$ — NDS 2015 Table N3

$T_{LTTP2\_LRFD} := T_{LTTP2\_ASD} \cdot K_F \cdot \phi_z \cdot \lambda = 5.01\ kip$

$$DCR_{LTTP2} := \frac{TC_{col}}{T_{LTTP2\_LRFD}} = 0.95$$

LTP4 Clips:

$LTP4_{cap\_ASD} := 580\ lbf$

$LTP4_{cap\_LRFD} := LTP4_{cap\_ASD} \cdot K_F \cdot \phi_z \cdot \lambda = 1251.64\ lbf$

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$$\frac{TC_{col}}{LTP4_{cap\_LRFD}} = 3.8$$

Use 4 clips to attach each (N) sister joist to (E) floor joist

A35 Clips:

$A35_{cap\_ASD} := 420\ lbf$

$A35_{cap\_LRFD} := A35_{cap\_ASD} \cdot K_F \cdot \phi_z \cdot \lambda = 906.36\ lbf$

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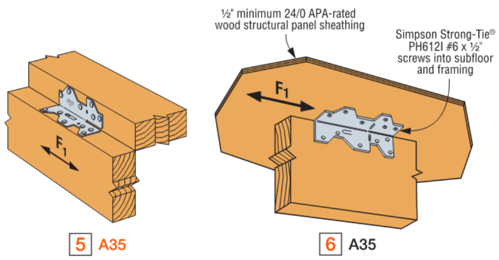
$$Diaph_{cap} := 505\ \frac{lbf}{ft}$$

Assumed diaphragm capacity from Section 2.2.4.



$$\frac{A35_{cap\_LRFD}}{Diaph_{cap}} = 1.79\ ft$$

Space clips 2ft o.c. so as not to exceed the capacity of the diaphragm.

$$n_{A35\_clips} := \frac{TC_{col}}{A35_{cap\_LRFD}} = 5.25$$

Use 6 clips to attach (E) floor joist to diaphragm over a span of 12ft (6*2=12).

6. Out-of-Plane Loading on Cantilever Column:

6.1 Expected Column Strength in Weak-Axis Direction

AISC 341-16, Section D1.2a

$R_y = 1.1$

$F_y = 50\ ksi$

$Z_y := 18.5\ in^3$

Section modulus about Y-Y axis. AISC, Table 1-1.

$M_{expWeak} := R_y \cdot F_y \cdot Z_y = 84.79\ kip \cdot ft$

Equation D1-1

$h_{wall} = 8\ ft$

$$V_{Weak} := \frac{M_{expWeak}}{h_{wall}} = 10.6\ kip$$

Expected shear capacity of column in weak-axis bending

$$\frac{V_{Weak}}{4} = 2.65\ kip$$

Assume the out-of-plane shear is resolved over the (4) new angles.

$$\frac{\frac{V_{Weak}}{4}}{TC_{col}} = 0.56$$

This out-of-plane shear value is less than the TC couple created by torsion found in Section 3.7, therefore OK by inspection.