# Project 3 - Report

SE 140A - Group 18

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# I. Introduction

This project details the seismic force resisting system for an eight-story structure with steel Special Moment Frames (SMF) in the north-south direction. All four bays along Column Lines A and G are to be mobilized. Reduced Beam Section (RBS) connections are used.

The building is located in San Jose with a zip code of 95113. The relevant seismic parameter values for this location were determined using the ASCE Hazard Tool. The structure is made of concrete slabs on a metal deck and supported with steel framing, composed of 35-foot bays. Each story is 15 feet high. A992 steel was used for the W-shapes, A572 Gr. 50 was used for connection plates, and E70 was used for the weld electrodes.

# II. Equivalent Lateral Force (ELF) Procedure

The design loads were as follows, using the three tables below. Note that the additional "line" dead load, displayed below in Table 1.2, was calculated by multiplying the floor area loads by the tributary height of that floor. These line loads were applied later on just to the perimeter of the floor.

Table 2.1 - Area Dead Load Information

| **Area DL** | Floor Area Loads (psf) | Roof Area Loads (psf) |
|---|---:|---:|
| Concrete Slab | 43.5 | 0 |
| 3-in Metal Deck | 2.2 | 2.2 |
| Steel Framing | 15 | 12 |
| Superimposed DL | 15 | 12 |
| TOTAL DL | **75.7** | **26.2** |

Table 2.2 - Additional Line Dead Load Information

| **DL Line Loads** | Floor Line Loads (plf) = (Hs) (13 psf) | Roof Line Load (plf) = (H□/2 + 6')(13) |
|---|---:|---:|
| Curtain Wall Weight (13 psf) | 195 | 175.5 |

Table 2.3 - Area Live Load Information

| **Area LL** | Floor Area Loads (psf) | Roof Area Loads (psf) |
|---|---:|---:|
| Partition Weight (ASCE 7 §12.7.2) | 10 | 5 |

The length of the building can be found by adding the length of six bays and the two feet of overhang, resulting in a length of $212\ ft$. The width of the building is composed of four bays and two feet of overhang, resulting in a width of $142\ ft$. The perimeter was found to be $708\ ft$. The area was found to be $30,104\ ft^2$.

To design the lateral seismic force resisting systems, the equivalent lateral force (ELF) procedure was utilized. The seismic weight was found by adding up all the individual dead load components which include the area load of the flooring, partition weight, and curtain wall weight. The equation used to compute this final seismic weight is shown below in Equation 1.

$$w_i = \frac{(D_{RF} + W_{P-j})A_{FL} + W_{Cw-j}L_{PERIM}}{1000} \text{ (kips)} \qquad \text{[Equation 1]}$$

Table 2.4 – Effective Seismic Weight & Weight Per Floor

| **Effective Seismic Weight Determination** | | | | | |
|---|---|---|---|---|---|
| Floor | DOF | Area Loads | Partition Weight | Curtain Wall Weight | Seismic Weight |
| j | i | D□ (psf) | w□-j (plf) | w_cw - j (plf) | $w_i$ (kips) |
| RF | 8 | 26.2 | 5 | 175.5 | **1063.50** |
| 8 | 7 | 75.7 | 10 | 195 | **2717.97** |
| 7 | 6 | 75.7 | 10 | 195 | **2717.97** |
| 6 | 5 | 75.7 | 10 | 195 | **2717.97** |
| 5 | 4 | 75.7 | 10 | 195 | **2717.97** |
| 4 | 3 | 75.7 | 10 | 195 | **2717.97** |
| 3 | 2 | 75.7 | 10 | 195 | **2717.97** |
| 2 | 1 | 75.7 | 10 | 195 | **2717.97** |
| | | | | Eff. Seismic Weight (kips) Σwi = | **20089.31** |

# III. Seismic Parameter Values

The seismic forces acting on the structure were calculated using the following variables.

### *1. Approximate Period, Ta.*

The Approximate Period, Ta, of the structure was determined from ASCE ASCE 7-22 §12.8, Table 12.8-2. For the 120-foot tall building, steel moment frames are to be used as the primary seismic force resisting systems for the N-S direction. Thus, the following parameters were used.

Table 3.1 – Structure Approximate Period

| **Period Ta Determination: [ASCE 7-22 §12.8]** | |
|---|---|
| Factors | Steel SMF |
| C□ (Table 12.8-2) | 0.028 |
| x (Table 12.8-2) | 0.75 |
| h□ (from base to Floor 9) (ft) | 120 |
| $T_a$ = C□h□$^x$ | 1.015 |
| $C_u$ (Table 12.8-1) | 1.4 |
| $T_a$□$_a$ = $C_u T_a$ | **1.42** |

### *2. Building Design Information*

The building specification given by the project and needed to determine the seismic forces are summarized below. The Risk Category was determined based on ASCE 7-22 §1.5 in Table 1.5-1. The soil site class was determined based on ASCE 7-22 §11.4.2.1.

Table 3.2 – Given Building Design Information

| **Design Info** | |
|---|---|
| Building Type: | Office Building |
| Risk Category: | II |
| Site Conditions: | C, CD, D |
| Location: | San Jose, CA |
| ZIP Code | 95113 |

### *3. Spectral Response Spectrum Acceleration ($S_a$)*

The Multi-Period Design Response Spectrum for San Jose, CA was found on the USGS Seismic Design Geodatabase at https://asce7hazardtool.online/. The Spectrum Acceleration ($S_a$) was found and is summarized in the table below for the three different Site Conditions. The Spectrum Acceleration at the structure's approximate period, Ta = 1.42, is highlighted below for all three site conditions.

Table 3.3 – Spectrum Acceleration in San Jose

| **Spectral Response Spectrum Acceleration ($S_a$) Determination** | | | |
|---|---|---|---|
| T (period) | $S_a$ | | |
| | Site Condition: C | Site Condition: CD | Site Condition: D |
| **0** | 0.49 | 0.49 | 0.46 |
| **0.01** | 0.49 | 0.5 | 0.47 |
| **0.02** | 0.49 | 0.5 | 0.47 |
| **0.03** | 0.53 | 0.52 | 0.47 |
| **0.05** | 0.64 | 0.59 | 0.51 |
| **0.075** | 0.8 | 0.72 | 0.64 |
| **0.1** | 0.91 | 0.84 | 0.75 |
| **0.15** | 1.07 | 1 | 0.88 |
| **0.2** | 1.18 | 1.11 | 0.96 |
| **0.25** | **1.19** | 1.22 | 1.05 |
| **0.3** | 1.11 | **1.26** | 1.14 |
| **0.4** | 1.02 | 1.21 | **1.2** |
| **0.5** | 0.92 | 1.15 | **1.2** |
| **0.75** | 0.71 | 0.94 | 1.05 |
| **1** | 0.57 | 0.78 | 0.93 |
| **1.42** | 0.427 | 0.595 | **0.761** |
| **1.5** | 0.4 | 0.56 | 0.73 |
| **2** | 0.3 | 0.43 | 0.59 |
| **3** | 0.23 | 0.3 | 0.42 |
| **4** | 0.2 | 0.23 | 0.31 |
| **5** | 0.18 | 0.2 | 0.24 |
| **7.5** | 0.11 | 0.12 | 0.15 |
| **10** | 0.07 | 0.078 | 0.095 |

### 4. *Seismic Design Category*

The Seismic Design Category was found according to ASCE 7-22 §11.6 for all three site conditions and for Risk Category II. The highest value for $S_{ds}$ and $S_{d1}$ determined the category and the governing site condition was found. The information is summarized in the table below.

Table 3.4 – Seismic Design Category

| Seismic Design Category Determination for **Risk Category II** Building | | | | | |
|---|---|---|---|---|---|
| | Seismic Design Category | | | | |
| Factors | Site Condition: C | Site Condition: CD | Site Condition: D | Category Determination | Governing |
| **S_1** | 0.62 | 0.62 | 0.62 | N/A | |
| **S_ds** | 1.07 | **1.14** | 1.08 | CD | |
| **S_d1** | 0.57 | 0.89 | **1.13** | D | **D** |

### 5. *Seismic Design Coefficients & Factors*

The Response Modification Coefficient (R), Overstrength Factor (**$\Omega_o$**), Deflection Amplification Factor (Cd), and Structural Height Limit (h□ Limit) were found in ASCE 7-22 §12.2 using Table 12.2-1, Design Coefficients and Factors for Seismic Force-Resisting Systems. These values were dependent on the building frame system that we were designing for, the steel moment frames. Seismic Importance Factor ($I_e$) was found in ASCE 7-22 §1.5 using Table 1.5-2, the Importance Factors by Risk Category of Buildings and Other Structures for Earthquake Loads table. The value was determined by the Risk Category of the structure, Risk Category II.

Table 3.5 – Seismic Design Coefficients & Factors

| **Seismic Design Coefficients and Factors (*R*, $\Omega_o$, *Cd*, $I_e$)** | |
|---|---|
| Factors | Seismic Design Category D |
| ***R*** | 8 |
| **$\Omega_o$** | 3 |
| ***Cd*** | 5.5 |
| **h□ Limit** | 160 |
| **$I_e$** | 1 |

### 6. *Seismic Response Coefficient & Base Shear*

The variables needed to calculate the Seismic Response Coefficient were summarized for each site condition. The Spectrum Acceleration, $S_a$, is from Table 2.3. S_1, S_ds, and S_d1 are from Table 2.4. $I_e$ and R are from Table 2.5. The value for Cs, the seismic response coefficient, is found by following procedures outlined in ASCE 7-22 §12.8.1.1 with equation 12.8-2. There are two checks for the lower bounds of Cs given in equations 12.8-6 and 12.8-7 when $S_1$ is greater than 0.6. From the three site conditions, Site Condition D governs with the greatest value of Cs being 0.0952. The governing Cs value, 0.1162, was then multiplied by the Effective Seismic Weight (Σwi = 20089.31) which is shown in Table 2.4, the Effective Lateral Force Per Floor table. This gives a base shear value of 1912.25 kips.

Table 3.6 – Seismic Response Coefficient & Base Shear

| | Seismic Base Shear Determination | | | |
|---|---|---|---|---|
| | | Seismic Design Category | | |
| | **ASCE 7-22 §12.8.1.1** | Site Condition: C | Site Condition: CD | Site Condition: D |
| | $S_a$ | 0.427 | 0.595 | 0.761 |
| | S_1 | 0.62 | 0.62 | 0.62 |
| | S_ds | 1.07 | 1.14 | 1.08 |
| | S_d1 | 0.57 | 0.89 | 1.13 |
| | $I_e$ | 1 | 1 | 1 |
| Factors | R | 8 | 8 | 8 |
| [Eq. 12.8-2] | Cs = $S_a$ ($I_e$/R) | **0.0533** | **0.0743** | **0.0952** |
| [Eq. 12.8-6] | Cs□in = 0.044(S_ds)($I_e$) >= 0.01 | 0.04708 | 0.05016 | 0.04752 |
| [Eq. 12.8-7] $S_1$ > 0.6 | Cs□in = 0.5$S_1$ / (R/$I_e$) | 0.03875 | 0.03875 | 0.03875 |
| | Governing Cs | **0.0533** | **0.0743** | **0.0952** |
| | | | Site Condition D Governs | |
| | Base Shear: V = CsW (kips) | **1912.25** | | |

## 7. *Seismic Force Calculation*

The seismic response coefficient was found using equation 2.

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad \text{[Equation 2]}$$

The exponent K was found using linear interpolation between the values 1 and 2 based on the period of the structure. This is shown in Table 2.7.1.

Table 3.7 – K Calculation

| **K Calculation (using linear interpolation)** | |
|---|---|
| T (s) | k |

| | |
|---|---|
| 0.5 | 1 |
| 1.42 | **1.614** |
| 2 | 2 |

The seismic force for each level was found by multiplying the seismic response coefficient for the corresponding floor and the seismic base shear. The numerator of the seismic response factor was found by multiplying the level's seismic weight by its structural height to the 1.614 power, the K value. The denominator is the sum of all multiplied seismic weight and story heights. $C_{vx}$ is shown as the vertical distance factor. The shear on each individual floor is shown as the Total Lateral Force, $F_x$. The demand for each special moment frame, SMF, is the sum of the seismic force divided by the number of SMF, 3, and the force caused by the additional torsional effect. The total force for a single SMF comes out to 701.16 kips.

Table 3.8 – Effective Seismic Force & Force Per Floor

| **Effective Seismic Force Calculation** | | | | | | | |
|---|---|---|---|---|---|---|---|
| DOF | Seismic Weight | Structural Height | | Vertical Distance Factor | Total Lateral Force | Force per SMF | Final Force per SMF w/ Accidental Torsion |
| (i = x) | $w_i$ (kips) | $h_i$ (ft) | $w_i h_i^k$(k-ft) | $C_{vx} = w_i h_i^k / (\Sigma w_i h_i^k)$ | $F_x$ (kips) | $f_x = F_x/3$(kips) | $f_{i,\square o\square a\square} = 1.1f_x$ (kips) |
| 8 | 1063.50 | 120 | 2414843 | 0.132 | 252.12 | 84.04 | 92.44 |
| 7 | 2717.97 | 105 | 4974944 | 0.272 | 519.41 | 173.14 | 190.45 |
| 6 | 2717.97 | 90 | 3879044 | 0.212 | 404.99 | 135.00 | 148.50 |
| 5 | 2717.97 | 75 | 2890099 | 0.158 | 301.74 | 100.58 | 110.64 |
| 4 | 2717.97 | 60 | 2015967 | 0.110 | 210.48 | 70.16 | 77.18 |
| 3 | 2717.97 | 45 | 1267101 | 0.069 | 132.29 | 44.10 | 48.51 |
| 2 | 2717.97 | 30 | 658521 | 0.036 | 68.75 | 22.92 | 25.21 |
| 1 | 2717.97 | 15 | 215108 | 0.012 | 22.46 | 7.49 | 8.23 |
| **Σ** | 20089.31 | | **18315628** | 1.000 | **1912.25** | | **701.16** |

# IV. Beam & Column Preliminary Sizing

The columns were spliced twice: once at the third floor and once at the seventh floor. The load of the top column, above the first splice on the seventh floor, is the cumulative lateral force at the eighth, seventh, and sixth degree of freedom. The load on the middle column is the cumulation of the degrees of freedom above it. The same is for the bottom most column. These values are summarized in the table below.

Table 4.1 – Lateral Forces on Columns

| **SMF Forces** | | | |
|---|---|---|---|
| Structural Height | DOF | Final Force per SMF (three) | Total Splice Shear |
| $h_i$ (ft) | (i = x) | $f_{i,\square o\square a\square}$ = 1.1$f_x$(kips) | Forces Above Splice (kips) |
| | 8 | 92.44 | |
| | 7 | 190.45 | |
| | 6 | 148.50 | 431.39 |
| | 5 | 110.64 | |
| | 4 | 77.18 | |
| | 3 | 48.51 | 667.71 |
| | 2 | 25.21 | |
| | 1 | 8.23 | 701.16 |
| | **Σ** | 701.16 | |

When considering the moment frame with the portal method, there is no moment at the midspan of beams and columns. A trial shear at the midspan of the internal and external columns can be set with the portal method assumption that the external column shear, $V_e$, is half of the internal column shear, $V_i$. The external shear is found using Equation 3. The 2 in the denominator is due to there being 2 exterior columns. The numerator is the sum of the lateral loads at each of the floors connected to the column. With the relationship of $V_e = 0.5V_i$, the internal trial shear is twice the external shear.

$$V_e = \frac{\sum_{i=j}^{N} F_{i,\,total}}{3 + 2\frac{V_e}{V_i}} \qquad \text{[Equation 3]}$$

Table 4.2 – Trial Column Shear Forces

| Interior and Exterior Column Shear from (ELF) | | | |
|---|---|---|---|
| Story Cut @ | $V_e c$ / $V_i c$ | $V_i c$ (kips) | $V_e c$ (kips) |
| 6th | 0.5 | 107.85 | 53.92 |
| 3rd | 0.5 | 166.93 | 83.46 |
| 1st | 0.5 | 175.29 | 87.64 |

The story drift of the structure must be less than the allowable inter-story displacement, $\Delta_a$. The left side of the inequality in Equation 4 is the inter-story displacement under force level 3. The right side of the equation is the allowable displacement with 0.02 being the allowable inter-story drift angle and 0.85 being a modification so that 15% of the drift goes to panel zone deformation. Equation 4 can be simplified to Equation 5 when the height of each story remains consistent for each floor. The capacity of the beams and columns must be large enough to prevent the allowable displacement.

$$\Delta = C_d \Delta_s \leq \left[ \frac{\Delta_a}{\rho} = \frac{0.02\left(\frac{h_j}{2} + \frac{h_{j+1}}{2}\right)[0.85]}{1.0} \right]$$

[Equation 4]

$$C_d \Delta_i \leq \frac{0.02\, h\, [0.85]}{1.0} \qquad \text{[Equation 5]}$$

The beam and column members are chosen from the minimum moment of inertia required for the story drift to be less than the maximum allowable drift. $I_b$ and $I_c$ will be assumed to be equivalent since the moment of inertia for both the beam and column are unknowns. Equation 6 can then be solved to find $I_{req}$.

$$1.1 \left[ \frac{V_i h^2}{12E} \left( \frac{L}{I_b} + \frac{h}{I_c} \right) \right] \leq \frac{0.02\, h\, [0.85]}{1.0\, C_d} \qquad \text{[Equation 6]}$$

Table 4.3 – Moment of Inertia Required for Story Drift Limit

| Required Moment of Inertia | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Story Cut @ | $V_i c$ | Δa = 0.02(h)(0.85) | p | Δa / p | RBS | Cd | $\Delta s_{re} q$ = Δa/(p * Cd) | I_req ($in^4$) |
| 6th | 107.85 | 3.06 | 1 | 3.06 | 1 | 5.5 | 0.556 | 11911.41 |
| 3rd | 166.93 | 3.06 | 1 | 3.06 | 1 | 5.5 | 0.556 | 18436.59 |
| 1st | 175.29 | 3.06 | 1 | 3.06 | 1 | 5.5 | 0.556 | 19360.04 |

The required moments of inertia for each story cut was used to choose the preliminary beam and columns. The moment of inertia found in Table 4.3 was set as the minimum requirement for both the beam and column of at each slice because the assumption taken before was that the moment of inertia of the beam, $I_b$ , was the same as that of the column, $I_c$. The W-shape sizes that work are shown in Table 4.4. The beams were then checked to see if they would have to be braced using the $L_{n,max}$ value in order to comply with the requirements listed in AISC 341 §E3.4b.

Table 4.4 – Preliminary Beam and Column Sizes

| **Preliminary Selection of Beams and Columns (AISCM Table 3-3)** | | | | |
|---|---|---|---|---|
| Story Cut @ | Type | Size | I (in⁴) | L_b, max (ft) |
| 6th | Beam | **W30x261** | 13100 | 13.3 |
| | Column | **W30X292** | 14900 | 13.5 |
| 3rd | Beam | **W40x249** | 19600 | 13.4 |
| | Column | **W30x357** | 18700 | 13.8 |
| 1st | Beam | **W40x249** | 19600 | 13.4 |
| | Column | **W30x391** | 20700 | 13.9 |

The initial assumptions made for the shear at the columns shown in Table 4.2 was checked to see if they are still valid for the W-shapes chosen as the preliminary sizes. The assumed shear for the interior and exterior columns were plugged into Equation 7 and 8 respectively with the moment of inertia of the beam, $I_b$ , and the moment of inertia of the column, $I_c$ , being the values from the W-shapes chosen.

$$\Delta_i = \Delta_b + \Delta_c = \frac{V_i h^2}{12E}\left(\frac{L}{I_b} + \frac{h}{I_c}\right) \qquad \text{[Equation 7]}$$

$$\Delta_e = \Delta_b + \Delta_c = \frac{V_e h^2}{6E}\left(\frac{L}{I_b} + \frac{h}{2I_c}\right) \qquad \text{[Equation 8]}$$

Goal Seek was used to set the story drift of internal and external members, $\Delta_i$ and $\Delta_e$, equal to each other with the difference between the two, $\Delta_{diff}$, being set to zero. The shear of the internal and external columns, $V_i$ and $V_e$ , are then automatically adjusted to make the story drift consistent. The ratio between the internal and external shear, $\left(\frac{V_e}{V_i}\right)$, is no longer 0.5 but rather closer to 0.6.

Table 4.5 – Initial Shear Assumption Check

| Check Initial Assumptions | | | | | | | |
|---|---|---|---|---|---|---|---|
| Story Cut @ | $V_ec$ / $V_ic$ | $V_ic$ (kips) | $V_ec$ (kips) | $\Delta_i$ | $\Delta_e$ | Δ_diff = Δi - Δe | (Goal Seek) |
| 6th | 0.579 | 103.73 | 60.10 | 0.426 | 0.426 | 0.000 | -0.01% |
| 3rd | 0.592 | 159.60 | 94.45 | 0.461 | 0.462 | 0.000 | -0.01% |
| 1st | 0.584 | 168.21 | 98.27 | 0.472 | 0.472 | 0.000 | 0.01% |

The real story drift on Table 4.5 is then checked to see if it is under the allowed limit. The maximum allowable story drift shown on the right side in Equation 5 is checked against the actual story drift at the sixth, third, and first story shown on the left side in Equation 5. The demand capacity ratio of the drifts are shown in the two right most columns in Table 4.6 and all of the selected preliminary beam and column sizes deflected less than the maximum length.

Table 4.6 – Final Drift Check

| Final Drift Check | | | | | | | |
|---|---|---|---|---|---|---|---|
| Story Cut @ | $\Delta_i$ | $\Delta_e$ | Deflection Check 0.02(h)(0.85) / (1.0Cd) | $1.1\Delta_i$ | $1.1\Delta_e$ | $1.1\Delta_i$ < Check | $1.1\Delta_e$ < Check |
| 6th | 0.426 | 0.426 | 0.556 | 0.469 | 0.469 | 0.843 | 0.843 |
| 3rd | 0.461 | 0.462 | 0.556 | 0.508 | 0.508 | 0.912 | 0.912 |
| 1st | 0.472 | 0.472 | 0.556 | 0.519 | 0.519 | 0.933 | 0.933 |

# V. SMF Design Drawings

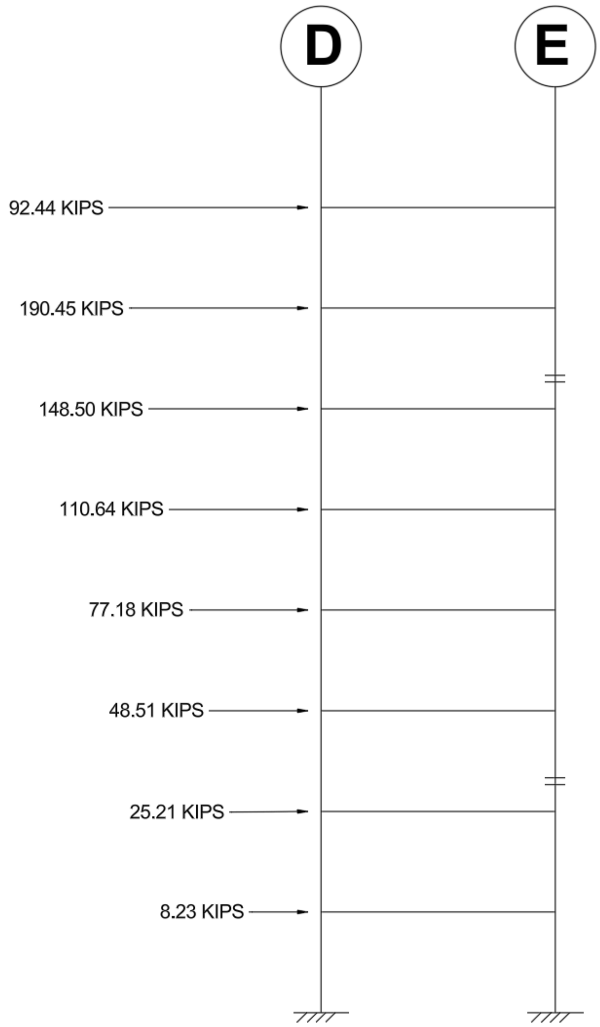



Figure 4.1 - ELF Procedure Results

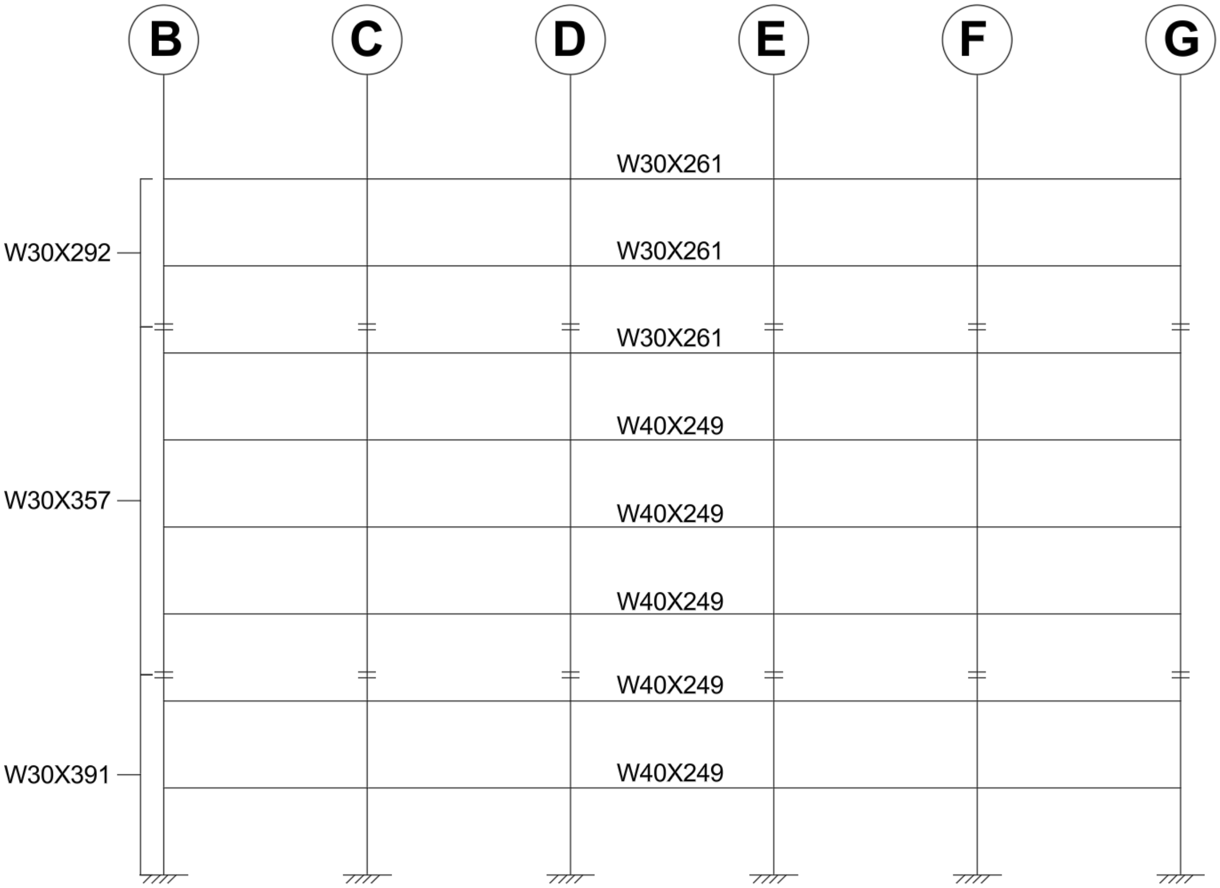



Figure 4.2 - SMF Preliminary Sizing

# VI. Interior Beam-to-Column Connection

## *1. Reduced Beam Section & Weld Access Hole Design*

For the beam-to-column connection, the flange width of the column must be greater than that of the beam so the sizes were changed from the preliminary design.

Table 6.1.1 – Modified Beam & Column Sizes

| Story Cut @ | Type | Size |
|---|---|---|
| 6th | Beam | **W30x261** |
| | Column | **W40x362** |
| 3rd | Beam | **W40x249** |
| | Column | **W40x362** |
| 1st | Beam | **W40x249** |
| | Column | **W40x362** |

The reduced beam section, RBS, is where the beam is cut and reduced to protect the beam-to-column welding. Trial dimensions for the reduction were chosen based on the dimension limitations. a is the distance of the cut from the column and is dependent on the width of the beam flange: $0.5b_{bf} \leq a \leq 0.75b_{bf}$. For our case, $0.55b_{bf}$ was used. b is the length of the cut and is dependent on the beam depth: $0.65d \leq b \leq 0.85d$. For our case, $0.7d$ was used. c is how deep the cut is made and is dependent on the width of the beam flange and is dependent on the width of the beam flange: $0.1b_{bf} \leq c \leq 0.25b_{bf}$. For our case, $0.2b_{bf}$ was used. The beams also pass the width-to-thickness ratio: $\frac{b'_f}{2t_f} < \lambda_{hd}$ .

The trial values selected are summarized in Table 6.2.

Table 6.1.2 – Trial Values for RBS Dimensions

| Trial RBS Dimension | | | | | | | |
|---|---|---|---|---|---|---|---|
| | Shape | b_bf | d | a | b | c | λ_hd Check |
| | (-) | (in) | (in) | (in) | (in) | (in) | AISC 341 Table 1-3B |
| SMF 1 | W40X249 | 15.8 | 39.4 | 8.75 | 27.5 | 3.25 | Y (Highly Ductile) |
| SMF 2 | W40X249 | 15.8 | 39.4 | 8.75 | 27.5 | 3.25 | Y (Highly Ductile) |
| SMF 3 | W30X261 | 15.2 | 31.6 | 8.25 | 22 | 3 | Y (Highly Ductile) |

Each beam's web access hole was checked based on SISC SDM Table 1-3b. The weld access hole design values were then found in Table 1-1 based on the web access hole letter.

Table 6.1.3 – Beam Weld Access Hole Dimensions

| Beam Weld Access Hole Design | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Shape | Table 1-3b | 1 | 2 | 3 | 4 | 5 | Root Opening |
| | (-) | (-) | (deg) | (in) | (in) | (in) | (in) | (in) |
| SMF 1 | W40X249 | F | 30 | 1.5 | 1.75 | 0.5 | 5.5 | 0.375 |
| SMF 2 | W40X249 | F | 30 | 1.5 | 1.75 | 0.5 | 5.5 | 0.375 |
| SMF 3 | W30X261 | G | 30 | 1.75 | 2 | 0.5 | 6.5 | 0.375 |

## 2. *Capacity Design*

The capacity of the beam in bending was found with the beam dimensions like $Z_x$, d, $t_{bf}$, and $t_w$. The plastic section modulus is reduced due to the cut in the beam. The reduced value is found using Equation 9. Using this value, the moment at the midpoint of the cut, $M_{pr}$, can be found using Equation 10. $M_{pe}$, the moment capacity of the unreduced beam, is found using Equation 11. $S_h$ is the distance from the column to the midpoint of the cut.

$$Z_{RBS} = Z_x - 2c\,t_{bf}\,(d - t_{bf}) \quad \text{[Equation 9]}$$

$$M_{pr} = C_{pr}\,(R_y F_y)\,Z_{RBS} \quad \text{[Equation 10]}$$

$$M_{pe} = (R_y F_y)\,Z_x \quad \text{[Equation 11]}$$

Table 6.2.1 – Moment at RBS & Unreduced Beam

| Capacity Design | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Z_x | d | t_bf | t_w | Z_RBS | Cpr | Ry | Fy | M_pr | M_pe | S_h |
| | (in^3) | (in) | (in) | (in) | (in^3) | (-) | (-) | (ksi) | (kip-ft) | (kip-ft) | (in) |
| SMF 1 | 1120 | 39.4 | 1.42 | 0.75 | 769.44 | 1.15 | 1.1 | 50 | 4055.61 | 5133.33 | 31.875 |
| SMF 2 | 1120 | 39.4 | 1.42 | 0.75 | 769.44 | 1.15 | 1.1 | 50 | 4055.61 | 5133.33 | 31.875 |
| SMF 3 | 943 | 31.6 | 1.65 | 0.93 | 646.50 | 1.15 | 1.1 | 50 | 3407.57 | 4322.08 | 26.125 |

The distributed loading on the beams were found from the dead and live load. $w_u$ is the total factored distributed load and the shear caused by the distributed load is $V_G$. These loads are the same on all floors.

$$w_u = (1.2 + 0.2S_{DS})w_D + 0.5w_L \quad \text{[Equation 12]}$$

$$V_G = \frac{w_u L_h}{2} \quad \text{[Equation 13]}$$

Table 6.2.2 – Beam Loading on SMF

| Gravity Force Demands (Same for all SMFs) | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| A_T | Trib Width | d_c | L_h | S_DS | DL | LL | Reduced LL | w_D | w_L | w_u | Vg |
| (ft^2) | (ft) | (in) | (ft) | (-) | (psf) | (psf) | (psf) | (kip/ft) | (kip/ft) | (kip/ft) | (kips) |
| 647.5 | 18.5 | 33.2 | 26.92 | 1.14 | 75.7 | 80 | 67.16 | 1.40 | 1.24 | 2.62 | 35.28 |

The moment and shear for each beam was checked to compare the demand and strength. This is done to ensure that the beam flange CJP groove weld will be safe in the case of seismic activity. The moment demand is found using Equation 14 and the strength is $M_{pe}$ found on Table 6.4. The shear demand and strength are found using Equation 16 and 17 respectively.

$$M_f = M_{pr} + V_{RBS} S_h \quad \text{[Equation 14]}$$

$$M^*_{be} = M_{pr} + V_{RBS}\left(S_h + \frac{d_c}{2}\right) \quad \text{[Equation 15]}$$

$$V_u = V_{RBS} + w_u S_h \quad \text{[Equation 16]}$$

$$\phi_v V_n = 1.0\,(0.6F_y)\,(d\,t_w) \quad \text{[Equation 17]}$$

Table 6.2.3 – Moment & Shear Checks

| Seismic Force Demands and Checks | | | | | | | |
|---|---|---|---|---|---|---|---|
| | V_RBS | M_f | M_be* | DCR: Moment Check | Vu | ϕVn | DCR: Shear Check |
| | (kips) | (kip-ft) | (kip-ft) | M_f < ϕM_pe | (kips) | (kips) | Vu < ϕVn |
| SMF 1 | 336.58 | 4949.65 | 5415.26 | 0.964 | 343.54 | 886.5 | 0.388 |
| SMF 2 | 336.58 | 4949.65 | 5415.26 | 0.964 | 343.54 | 886.5 | 0.388 |
| SMF 3 | 288.44 | 4035.51 | 4434.52 | 0.934 | 294.14 | 881.64 | 0.334 |

### 3. *Strong Column-Weak Beam Condition*

The strong column-weak beam (SCWB) condition requires that the columns are stronger than the fully yielded and strain-hardened beams, the capacity. The following condition must be satisfied: $\Sigma M^*_{pc} \geq \Sigma M^*_{be}$ . $\Sigma M^*_{pc}$ is the capacity of the columns and $\Sigma M^*_{be}$ is the demand that comes from the capacity of the beams. $M^*_{be}$ has been found in Table 6.6. To find $M^*_{pc}$ , the cantilever method was used. The ELF forces on the structure are computed to find $P_u$ which is found using Equation 18.

$$P_u = (1.2 + 0.2S_{DS})P_D + 0.5P_L + P_{uc} \qquad \text{[Equation 18]}$$

$$M^*_{pc} = Z_c\left(F_{yc} - \frac{P_u}{A_g}\right) \qquad \text{[Equation 19]}$$

Table 6.7 summarizes the ELF forces at each degree of freedom and the distance that force is from each story cut.

Table 6.3.1 – ELF Forces on Structure & Distances from Story Cuts

| **Strong Column-Weak Beam Condition** | | | | | |
|---|---|---|---|---|---|
| DOF | Final Force per SMF | Amplified Seismic Force | Story Moment Arms (ft) | | |
| (i = x) | $f_{i,\square o\square a\square}$ (kips) | $f_{\square ei\square} = f_i * 3$ (kips) | 1st Story Cut | 3rd Story Cut | 6th Story Cut |
| 8 | 92.44 | 277.32 | 112.5 | 82.5 | 37.5 |
| 7 | 190.45 | 571.35 | 97.5 | 67.5 | 22.5 |
| 6 | 148.50 | 445.5 | 82.5 | 52.5 | 7.5 |
| 5 | 110.64 | 331.92 | 67.5 | 37.5 | |
| 4 | 77.18 | 231.54 | 52.5 | 22.5 | |
| 3 | 48.51 | 145.53 | 37.5 | 7.5 | |
| 2 | 25.21 | 75.63 | 22.5 | | |
| 1 | 8.23 | 24.69 | 7.5 | | |
| **Σ** | 701.16 | 2103.48 | | | |

The columns were changed from the preliminary column sizes to account for the required minimum column flange width. $A_g$, $Z_c$, and $F_{yc}$ are all measurements of the column selected and are known. $P_u$ is found from the ELF forces acting on the structure using Equation 18. The interior stress is $\frac{P_u}{A_g}$ . From calculating $M^*_{pc}$ using Equation 19 it is clear that the capacity of hte columns are adequate for the demand from the beam capacities.

Table 6.3.2 – Capacity of Columns

| Story Cut | | Columns* | Z_c | A_g | F_yc | Pu | Interior Stress | M_pc* = Z_c *(F_yc - Stress) | M_be* | DCR Check: |
|---|---|---|---|---|---|---|---|---|---|---|
| (-) | (-) | (-) | (in³) | (in²) | (ksi) | (kips) | (ksi) | (kip-ft) | (kip-ft) | M□c* > M_be* |
| 1 | SMF 1 | **W40x362** | **1640** | 95.3 | **50** | 473.04 | 4.96 | -57815.29 | 5415.26 | -0.094 |
| 3 | SMF 2 | **W40x362** | **1640** | 95.3 | **50** | 383.90 | 4.03 | -45633.02 | 5415.26 | -0.119 |
| 6 | SMF 3 | **W40x362** | **1640** | 95.3 | **50** | 224.74 | 2.36 | -23880.91 | 4434.52 | -0.186 |

### 4. *Continuity Plate Design*

The column flange strength is checked to see whether or not a continuity plate is required for this design. There is a pulling force at the top and bottom flange of the beams, $P_f$ , being applied to the column flanges on both sides; this is the demand. The column flange must be adequate to resist this pulling force. If it is not adequate, a continuity plate is required. Two checks were done and both passed with the demand being larger than the capacity so the continuity plate is required.

The first check is comparing the pulling force, $P_f$ , with the column flange local bending and web local yielding. The second check is comparing the beam flange width and thickness of the column flange.

$$\textit{Flange Local Bending:}\quad \phi R_n = 0.9(6.25F_{yt}t_{cf}^2) \qquad \text{[Equation 20]}$$

$$\textit{Web Local Yielding:}\quad \phi R_n = 1.0\left[F_{yw}t_w\left(5k + t_{bf}\right)\right] \qquad \text{[Equation 21]}$$

6.4.1 – Continuity Plate Requirement Check

| **Column Flange Strength Check** | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Story Cut | Vrbs = 2M□r / L□ + V_G (k) | Mf = M□r + Vrbs*(a + b/2) | d* = d_b - t_bf | t_w | t_cf | b_bf / 6 | Pf = 0.85 * Mf /d* | Column Flange Local Bending | Web Local Yielding | Pf > ΦRn? | t_cf < b_bf / 6 |
| (-) | (kip) | (k-ft) | (in) | (in) | (in) | (in) | (k) | ΦRn = 0.9(6.25)(F_yf)(t_cf)² | ΦRn = 1.0(F_yw)(t_w)(5k + t_bf) | | |
| 1 | 336.58 | 5283.43 | 37.98 | 1 | 1.81 | 2.63 | 1418.93 | 921.40 | 321 | Y | Y |
| 3 | 336.58 | 5283.43 | 37.98 | 1 | 1.81 | 2.63 | 1418.93 | 921.40 | 321 | Y | Y |
| 6 | 288.44 | 4266.26 | 38.39 | 1 | 1.81 | 2.53 | 1133.52 | 921.40 | 332.5 | Y | Y |
| | | | | | | | | | | Continuity | |

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | Plate Needed |

The continuity plate will be a two sided connection so the thickness must be greater than 0.75* $t_{bf}$ where $t_{bf}$ is the thickness of the beam flange. This is then rounded up to the nearest 1/8". The width-to-thickness ratio check is done to check for continuity plate buckling.

$$\frac{b}{t} \leq 0.56\sqrt{\frac{E}{R_y F_y}} \qquad \text{[Equation 22]}$$

The dimensions of the continuity plate were then designed. The width, $b_{cp}$, and cut off points, cc and cc', were determined from the column size dimensions and are summarized in Table 6.4.2.

Table 6.4.2 – Continuity Plate Design

| Continuity Plate Check | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Story Cut | 2-Sided Connection: t >= 0.75t_bf | Rounded t | Width-To-Thickness Ratio Check | | Design | | | | |
| (-) | (in) | (in) | 0.56*sqrt(E / RyFy) | b□ax (in) = t * [0.56*sqrt(E / RyFy)] | **b_CP (in)** | k1 (in) | **cc (in)** | k_det - t_f + 1.5 (in) | **cc' (in)** |
| 1 | 1.065 | 1.125 | 12.86 | 14.47 | **7.5** | 1.75 | **2** | 2.74 | **3** |
| 3 | 1.065 | 1.125 | 12.86 | 14.47 | **7.5** | 1.75 | **2** | 2.74 | **3** |
| 6 | 1.2375 | 1.25 | 12.86 | 16.07 | **7.25** | 1.75 | **2** | 2.74 | **3** |

### 5. *Doubler Plate Design*

Shear at the panel zone creates the need for a doubler plate when the demand of $V_{pz}$ is greater than the supply, $V_n$. The demand, $V_{pz}$, is determined from Equation 23 and is dependent on the pulling force due to the beams and shear on the columns. $V_n$ is determined by Equation 24. The demand is greater so doubler plates are required.

$$V_{pz} = \left(\Sigma P_f\right) - V_c = \left(\Sigma \frac{M_f}{0.95 d_b}\right) - V_c \qquad \text{[Equation 23]}$$

$$V_n = 0.60 F_y d_c t_w \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_w}\right)\left(1.9 - \frac{1.2 P_u}{P_y}\right) \qquad \text{[Equation 24]}$$

Table 6.5.1 – Doubler Plate Requirement Check

| Panel Zone Strength Check | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Story Cut @ | Vb = 2*M□r / L□ | V_c = (V_b)(L)/h | Pf = Mf / (0.95 * d_b) | V□z = ∑Pf - V_c (kips) | Pu (kips) < | 0.75FyAg (kips) | Vn = 0.6Fy(d_c)(t_w)(1 + 3b_cf(t_cf)²/(d_b * d_c * t_w)) | DCR: V□z > Vn |
| 1 | 301.30 | 703.03 | 1693.859 | 2684.69 | 473.039 | 3573.75 | 1115.74 | Y |
| 3 | 301.30 | 703.03 | 1693.859 | 2684.69 | 383.900 | 3573.75 | 1115.74 | Y |
| 6 | 253.15 | 590.69 | 1705.368 | 2820.04 | 224.738 | 3573.75 | 1145.29 | Y |
| | | | | | Y, Use Equation 1 | | Doubler Plate Needed | |

The required thickness of the doubler plate was found through the demand and supply equations. It was then rounded up to the nearest 1/8”.

Table 6.5.2 – Doubler Plate Thickness

| Supply Recheck | | | |
|---|---|---|---|
| Story Cut @ | $t_r$eq (in) | t_dp = $t_{re}$q - t_cw (in) | FINAL t_dp (in) |
| 1 | 2.58 | 1.58 | **1.625** |
| 3 | 2.58 | 1.58 | **1.625** |
| 6 | 2.68 | 1.68 | **1.75** |

There must not be local buckling due to shear. To check this, the thinner of the column web or doubler plate was compared against the location of the plug welds. The column web is thinner as it is 1”. The location of the plug welds are determined by $w_g$, the horizontal distance between plug welds, and $d_g$, the vertical distance between plug welds. The design must satisfy the following inequality: $t \geq \frac{d_z + w_z}{90}$ .

Table 6.5.3 – Column Local Buckling Check

| Local Buckling Check | | | | |
|---|---|---|---|---|
| t_cw (in) | d_z = (d_b) - 2(t_fb) (in) | w_z = d_c - 2t_fc (in) | (d_z + w_z) / 90 | DCR: t > (d_z + w_z) / 90 |
| 1 | 36.56 | 29.58 | 0.735 | Y |
| 1 | 36.56 | 29.58 | 0.735 | Y |
| 1 | 28.3 | 29.58 | 0.643 | Y |

The layout for the doubler plate with the continuity plate was determined and is summarized below in Table 6.5.4.

Table 6.5.4 – Doubler Plate & Continuity Plate Layout

| Doubler Plate and Continuity Plate Layout | | | | | | |
|---|---|---|---|---|---|---|
| Story Cut @ | k1 (in) | k1 - t_cp | k1 + t_cp + 0.5 | **cc (in)** | **cc' (in)** | **b_CP (in)** |
| 1 | 1.75 | 0.625 | 3.375 | **2** | **3** | **7.5** |
| 3 | 1.75 | 0.625 | 3.375 | **2** | **3** | **7.5** |
| 6 | 1.75 | 0.5 | 3.5 | **2** | **3** | **7.25** |
| | | | | There will be a doubler plate on only one side. Two continuity plates will be installed on each side of the column web. See diagram. | | |

### 6. *Beam Lateral Bracing Design*

The beam must be laterally braced at every third of the length. The length of the beam is 35' so the bracing must be placed at 11.67' and 23.33'. The maximum braced length is found using Equation 25 and the braced length is less than the maximum length of 13.3' and 13.4'.

$$L_{b\,(max)} = \frac{0.095\,\gamma_y E}{R_y F_y} \qquad \text{[Equation 25]}$$

Table 6.6.1 – Beam Lateral Bracing Locations

| Beam Lateral Bracing | | | | |
|---|---|---|---|---|
| Story Cut @ | Type | Size | L_b, max (ft) | Bracing Every Lb / 3 (ft) |
| 6th | Beam | **W30x261** | 13.3 | 11.67 |
| 3rd | Beam | **W40x249** | 13.4 | 11.67 |
| 1st | Beam | **W40x249** | 13.4 | 11.67 |

## 7. *Column Compressive Strength Check*

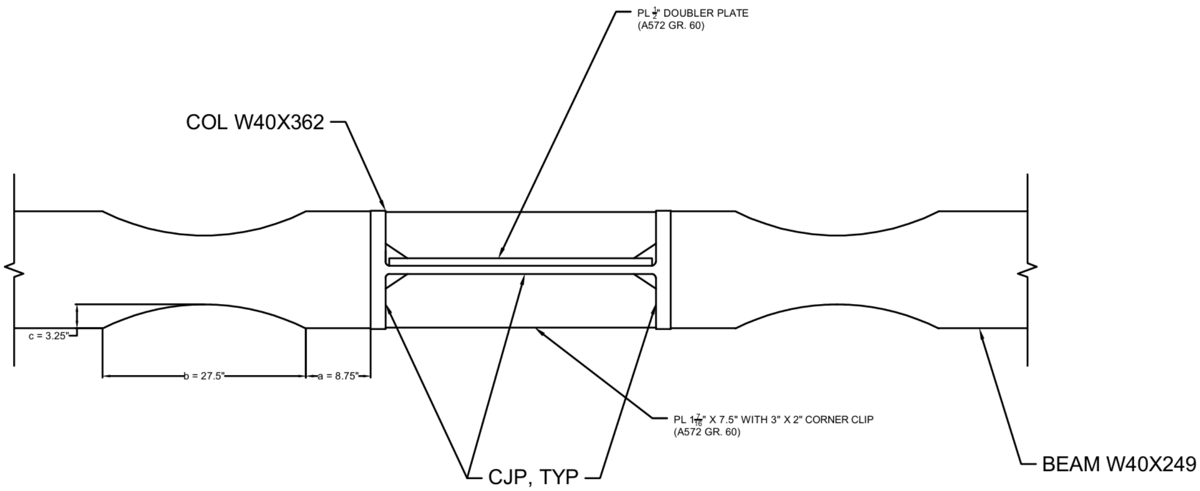
The column compressive strength was rechecked to ensure that the compressive demands due to seismic and gravity loads are not greater than the column's capacity. These checks were all satisfied and the design is complete.

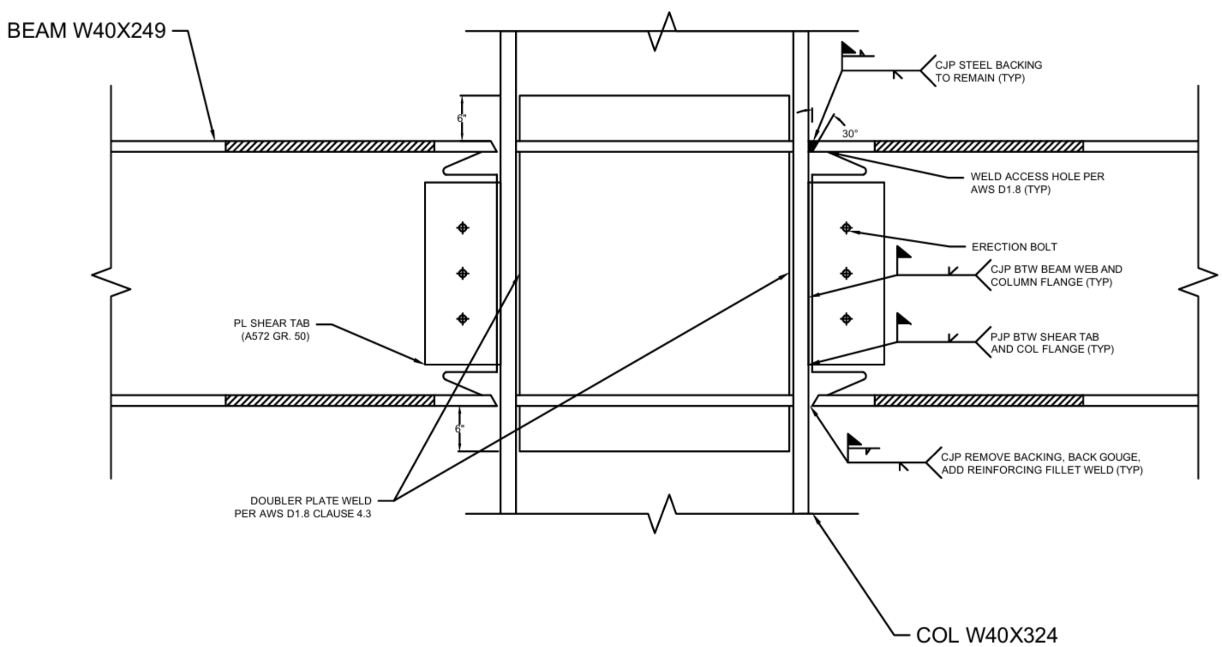
Table 6.7.1 – Column Compressive Strength Check

| Recheck Column Compressive Strength: W40x362 | | | | |
|---|---|---|---|---|
| Story Cut | Pu | Pr,max (AISC SDM Table 1-3b) | ΦPn (AISCM Super Table) | Pu < min(Pr,max ; ΦPn) |
| (-) | (kips) | (kips) | (kips) | DCR |
| 1 | 473.04 | 1400 | 3970 | 0.338 |
| 3 | 383.90 | 1400 | 3970 | 0.274 |
| 6 | 224.74 | 1400 | 3970 | 0.161 |

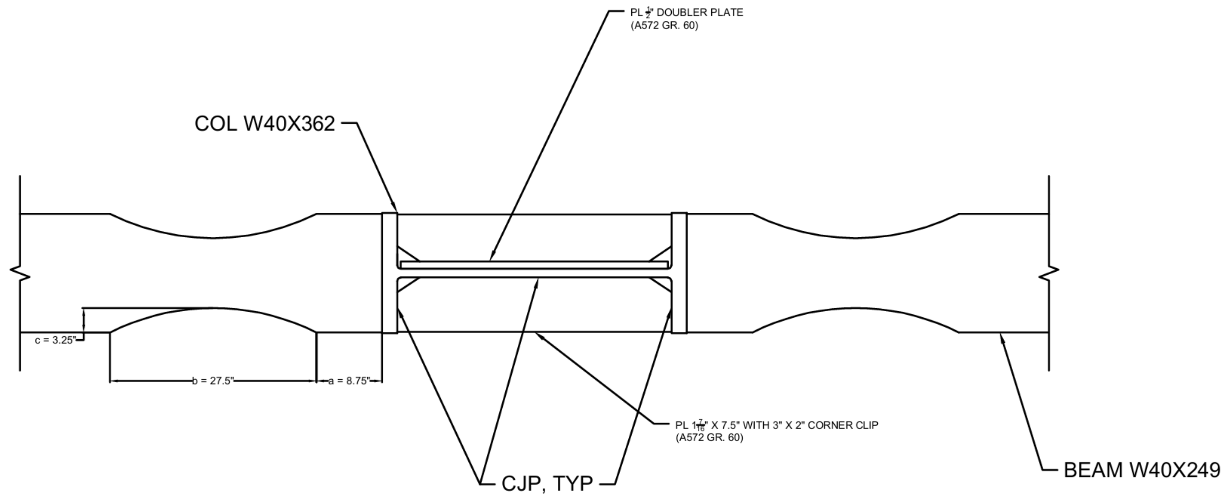
## *8. Interior Beam-to-Column Connection Drawing*

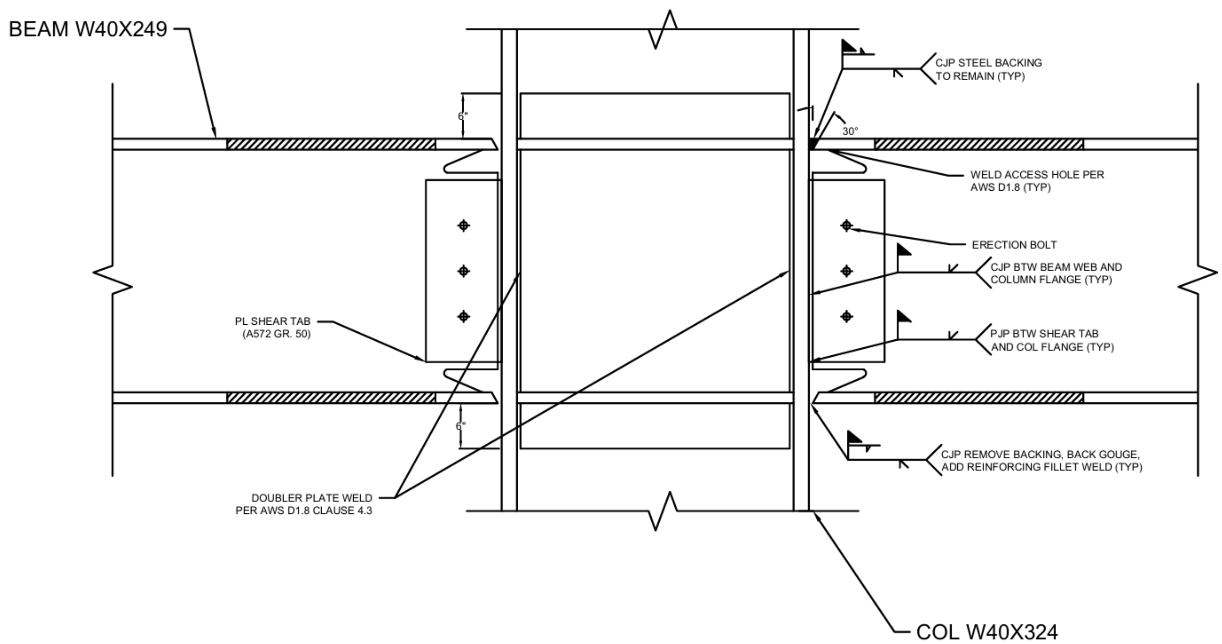
### First Floor SMF



## Third Floor SMF



# Sixth Floor SMF