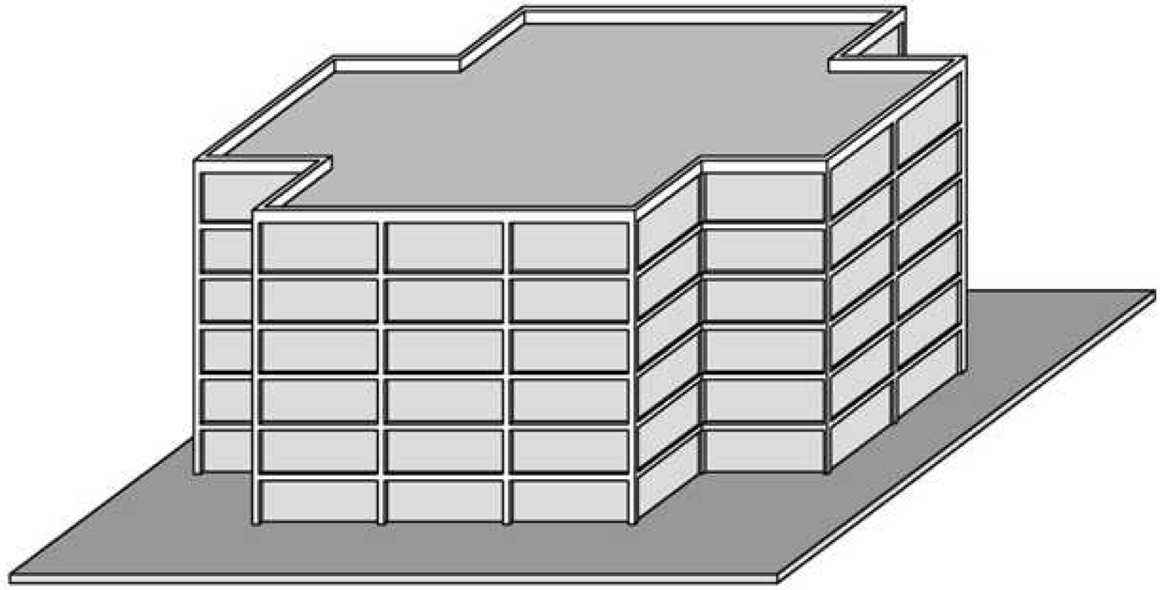
Design Example 1
Special Moment Frame



OVERVIEW

Structural steel special moment frames (SMF) are typically comprised of wide-flange beams, columns, and beam-column connections. Connections are proportioned and detailed to resist internal forces (flexural, axial, and shear) that result from imposed displacement as a result of wind or earthquake ground shaking. Inelasticity and energy dissipation are achieved through localized yielding of the beam element outside of the beam-column connection. Special proportioning and detailing of this connection is essential to achieving the desired inelastic behavior.

The anticipated seismic behavior of the SMF system is long-period, high-displacement motion, with well-distributed inelastic demand shared by all participating beam-column connections. System-yielding mechanisms are generally limited to frame beams with the intent to invoke yielding at the base of frame columns. In many cases, engineers may model an SMF system with pin-based columns as significant stiffness is required to yield the base of large wide-flange members. If yielding at the base of the frame is desired to occur within the column section, the column might be extended below grade and tied into a basement wall or a ground-level beam, which is added to create a beam-column connection. Economies of construction usually limit the size of beam and column elements based on imposed displacement/drift limits.

Design regulations for steel SMF are promulgated in a series of standards: ASCE/SEI 7, ANSI/AISC 341, ANSI/AISC 358, and ANSI/AISC 360. AISC 358 provides specific regulations related to prequalification of certain SMF connection types that preclude project-specific testing required by AISC 341. This design example follows the provisions of AISC 358 for the RBS connection type for the steel SMF seismic-force-resisting system.

The six-story steel office structure depicted in the figure above has a lateral-force-resisting system comprising structural steel special moment frames. The typical floor framing plan is shown in Figure 1-1. A typical frame elevation is depicted in Figure 1-2. This design example utilizes simplifying assumptions for ease of calculation or computational efficiency. Because bay sizes vary, the example frames can be designed with different participating bays in each direction, which will result in different sizes of beams and columns for each frame, depending on location. This example explores the design of a single frame and a single connection of that frame. Assumptions related to base-of-column rotational restraint (assumed fixed), applied forces (taken from the base example assumptions), and applied wind force (not considered) are all incorporated into the example in "silent" consideration. Beam and column element sizes were determined using a linear elastic computer model of the building. These element sizes were determined through iteration such that code-required drift limits, element characteristics, and strength requirements were met.

While this example is accurate and appropriate for the design of steel SMF structures, different methodologies for analysis, connection design, and inelastic behavior can be utilized, including the use of proprietary SMF connection design. This example does not explore every possible option, nor is it intended to be integrated with other examples in this document (i.e., base plate design).

OUTLINE

1. Building Geometry and Loads
2. Calculation of the Design Base Shear and Load Combinations
3. Vertical and Horizontal Distribution of Load
4. SMF Frame
5. Element and RBS Connection Design
6. Detailing of RBS Connection

1. Building Geometry and Loads

1.1 GIVEN INFORMATION

- Per Appendix A
    - Office occupancy on all floors
    - Located in San Francisco, CA, at the latitude and longitude given
    - Site Class D
    - 120 feet × 150 feet in plan with typical floor framing shown in Figure 1-1
    - Frame beam and column sizes for lines 1 and 5 (Figure 1-2)
        - Beam and column sizes will vary from those on lines A and F
    - Six stories, as shown in Figure 1-2

- Structural materials
    - Wide-flange shapes — ASTM A992 (F_y = 50 ksi)
    - Plates — ASTM A572, Grade 50
    - Weld electrodes — E70XX

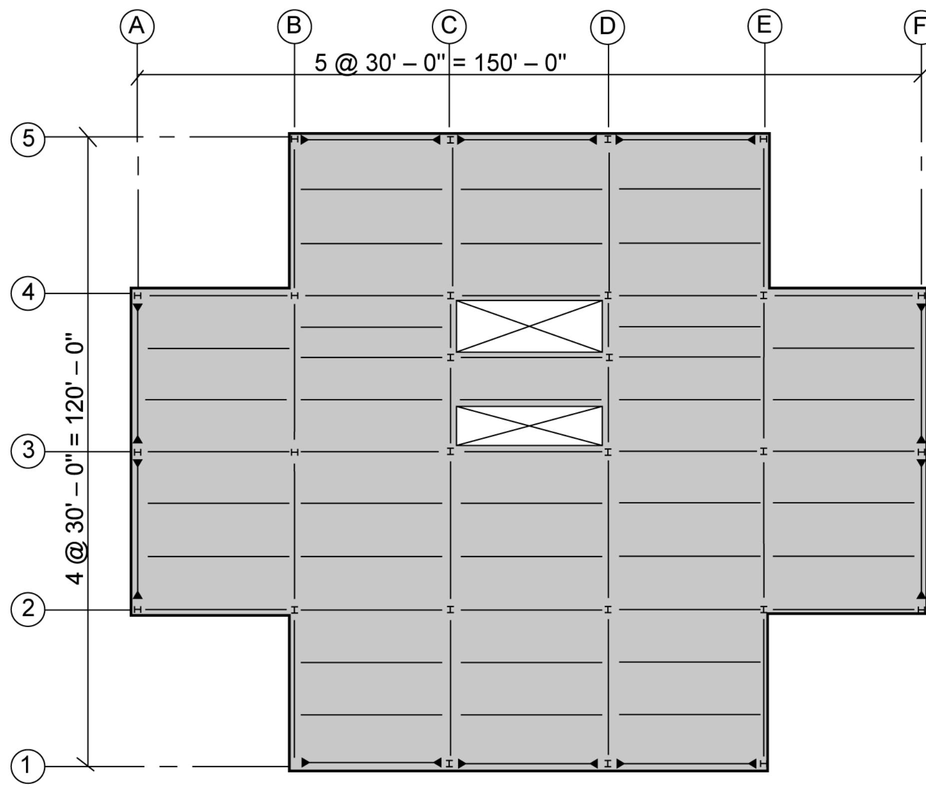


Figure 1-1. Typical floor framing plan

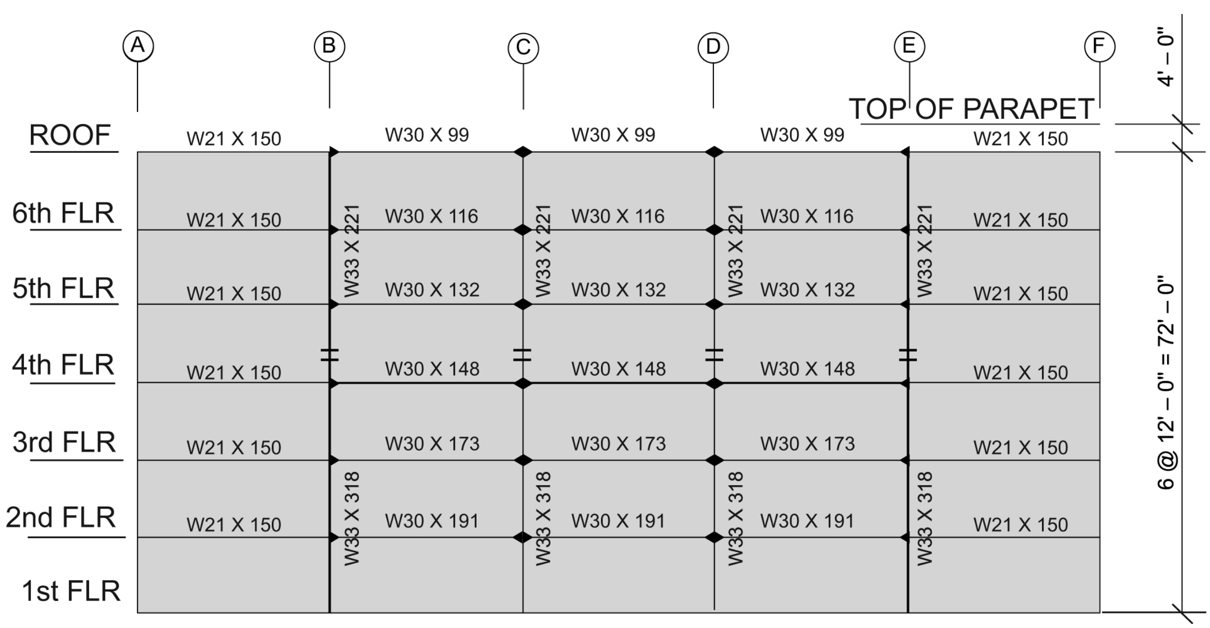


Figure 1-2. Frame elevation—line 1 (line 2 in background)

1.2 FLOOR WEIGHTS

For development of seismic forces per Appendix A:

Table 1-1. Development of seismic forces per Appendix A

Level	Assembly	Unit Weight (psf)	Area (ft^2)	Weight (kips)	Floor Weight (kips)
Typical floor	Floor	78	15,220	1187	1315
	Ext wall	19	6990	133	
Roof	Roof	36	15,220	548	656
	Ext wall/parapet	19	5700	108	

W = 5(1320 kips) + 656 kips = 7256 kips

2. Calculation of the Design Base Shear and Load Combinations ASCE 7

2.1 CLASSIFY THE STRUCTURAL SYSTEM AND DETERMINE SPECTRAL ACCELERATIONS

Per ASCE 7 Table 12.2-1 for special steel moment frame:

$$\boxed{R = 8.0 \quad \Omega_0 = 3 \quad C_d = 5.5}$$

2.2 DESIGN SPECTRAL ACCELERATIONS

The spectral accelerations to be used in design are derived in Appendix A:

$$\boxed{S_{DS} = 1.00g \quad S_{D1} = 0.60g}$$

2.3 DESIGN RESPONSE SPECTRUM

Determine the approximate fundamental building period, T_a, using Section 12.8.2.1:

$C_t = 0.028$ and $x = 0.8$ T 12.8-2

$T_a = C_t h_n^x = 0.028 \times 72^{0.8} = 0.86$ sec (see the following discussion) Eq 12.8-7

$$\boxed{T_a = 0.86 \text{ sec}}$$

$T_o = 0.2\dfrac{S_{D1}}{S_{DS}} = 0.2\dfrac{0.60}{1.00} = 0.12$ sec §11.4.5

$$S_a = S_{DS}\left(0.4 + 0.6\frac{T}{T_o}\right) = 0.4 + 5.0T \quad \text{For } T < T_o \qquad \text{Eq 11.4-5}$$

$$T_S = \frac{S_{D1}}{S_{DS}} = \frac{0.60}{1.00} = 0.60 \text{ sec} \qquad \text{§11.4.5}$$

$$S_a = \frac{S_{D1}}{T} = \frac{0.60}{T} \quad \text{For } T > T_S \qquad \text{Eq 11.4-6}$$

The long-period equation for S_a does not apply here because the long-period transition occurs at 12 seconds (from ASCE 7 Figure 22-12).

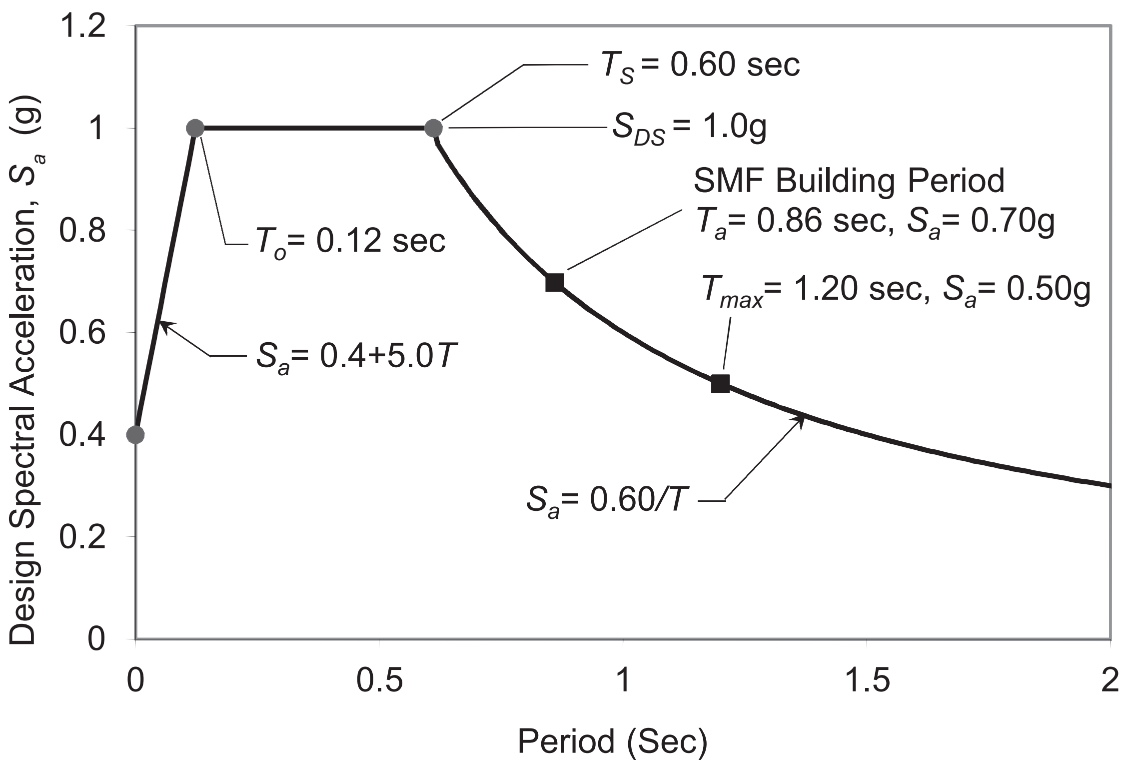


Figure 1-3. Design response spectrum for the example building

Figure 1-3 depicts the design spectral acceleration determined from T, which is greater than T_S, so the design spectral acceleration S_a is 0.70g.

ASCE 7 Section 12.8.2 indicates that the fundamental period of the structure "can be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis," which might allow a linear elastic modal analysis to suffice. Section 12.8.2, however, limits the period that can be used to calculate spectral acceleration to a value of $T_{max} = C_u \times T_a$, where C_u is a factor found in Table 12.8-1. In this case $T_{max} = 1.4 \times 0.86 = 1.20$ seconds. For preliminary design, the approximate period, T_a, will be used to design the SMF. As SMF designs are heavily dependent on meeting drift requirements, the initial value (usually found to be much lower than the period found through mathematical modeling) will suffice for the first design iteration.

2.4 HORIZONTAL IRREGULARITIES **T12.3-1**

1a. and 1b. Torsional Irregularity—A torsional irregularity exists when the maximum story drift computed, including accidental torsion, is more than 1.2 times the average story drift. Extreme torsional irregularity exists when the maximum story drift computed, including accidental torsion, is more than 1.4 times the average story drift. A static linear elastic three-dimensional computer analysis is used to obtain the displacement at the corners of the building. This building has no torsional response, so the difference between the maximum drift and average drift is 1.0. Table 1-2 provides an example of how one would evaluate the presence of a torsional irregularity for the earthquake load case in the longitudinal direction with positive accidental eccentricity and differences between maximum and average drift.

Table 1-2. Story displacements, line 1 and line 5, torsional irregularity check

Story	δ_x at Line 5 (in)	δ_x at Line 1 (in)	δ_{avg} (in)	$\delta_{max}/\delta_{avg}$
Roof	15.8	11.00	13.4	1.18
5	14.3	10.5	12.4	1.16
4	12.0	9.44	10.7	1.12
3	8.86	7.22	8.04	1.10
2	5.26	4.51	4.89	1.08
1	1.85	1.80	1.83	1.01

NO TORSIONAL IRREGULARITY: $\delta_{max}/\delta_{avg} < 1.2$

2. Reentrant corner irregularity exists where both plan projections of the structure beyond a reentrant corner are greater than 15 percent of the plan dimension of the structure in the given direction. The plan projections in longitudinal and transverse directions are 30 feet. The plan dimensions are 150 feet and 120 feet in the longitudinal and transverse direction respectively:

30 feet/150 feet = 20 percent in the longitudinal direction →
REENTRANT CORNER IRREGULARITY

30 feet/120 feet = 25 percent in the transverse direction →
REENTRANT CORNER IRREGULARITY

3. to 5. By inspection, the building does not qualify for any of these horizontal structural irregularities.

REENTRANT CORNER IRREGULARITY EXISTS

Per Section 12.3.3.4, forces for the connections of diaphragms to vertical elements and collectors and the design of collectors and their connections must be increased by 25 percent. If forces for the design of collectors and their connections are calculated using seismic load effects including the overstrength factor, the 25 percent increase is not required.