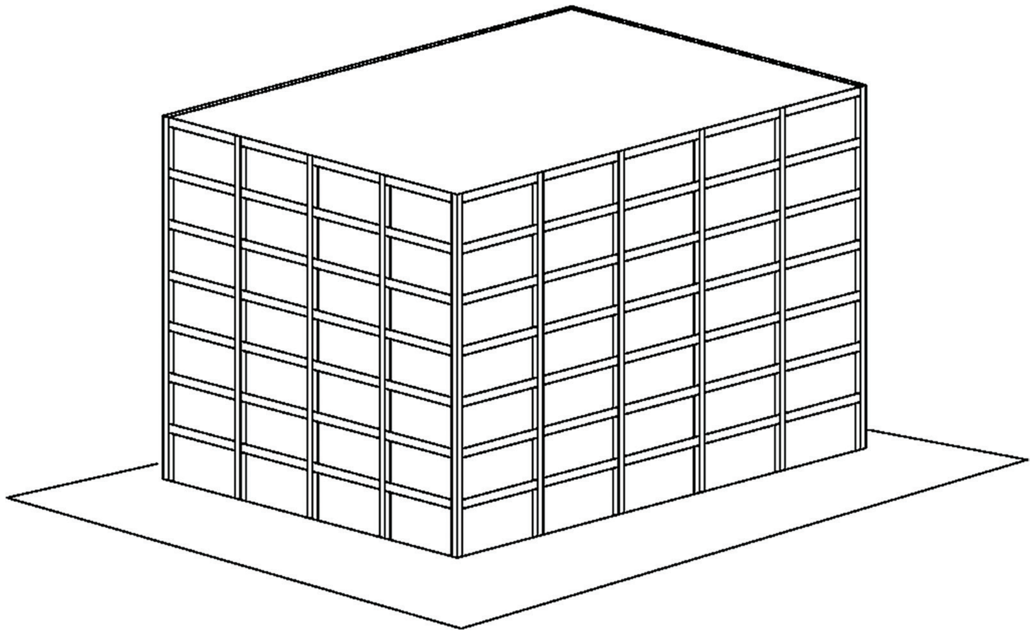
Design Example 3
Reinforced Concrete Special Moment Frame



OVERVIEW

Concrete frame buildings, especially ones with older, non-ductile frames, have frequently experienced significant structural damage in earthquakes. Several non-ductile frame buildings collapsed in the 1971 San Fernando earthquake. Because the collapses were attributed to detailing issues, special requirements for ductile concrete frames were introduced in the code.

Today ductile reinforced concrete frames are designated as SMRF (special moment-resisting frames). All reinforced concrete moment-frame structures built in Seismic Design Category D, E, or F locations must be SMRF as required by ASCE 7 Table 12.2-1. Ordinary moment-resisting frames (OMRF) are prohibited in Seismic Design Categories C, D, E and F, and intermediate moment-resisting frames (IMRF) are prohibited in Seismic Design Categories D, E, and F.

This example illustrates the seismic design of a seven-story concrete SMRF. A conceptual elevation of the building is shown above, with the typical floor plan shown in Figure 3-1. The building has seven stories with a SMRF on each perimeter wall. A typical building elevation is shown in Figure 3-2.

This design example follows the general code requirements of the 2024 *International Building Code* (2024 IBC), which adopt ASCE 7 and ACI 318 by reference. The example also includes occasional discussions referencing the 2009 edition of the *SEAOC Blue Book Seismic Design Recommendations (SEAOC Blue Book Seismic Design Recommendations,* 2009, Article 09.01.010 “Reinforced Concrete Structures”).

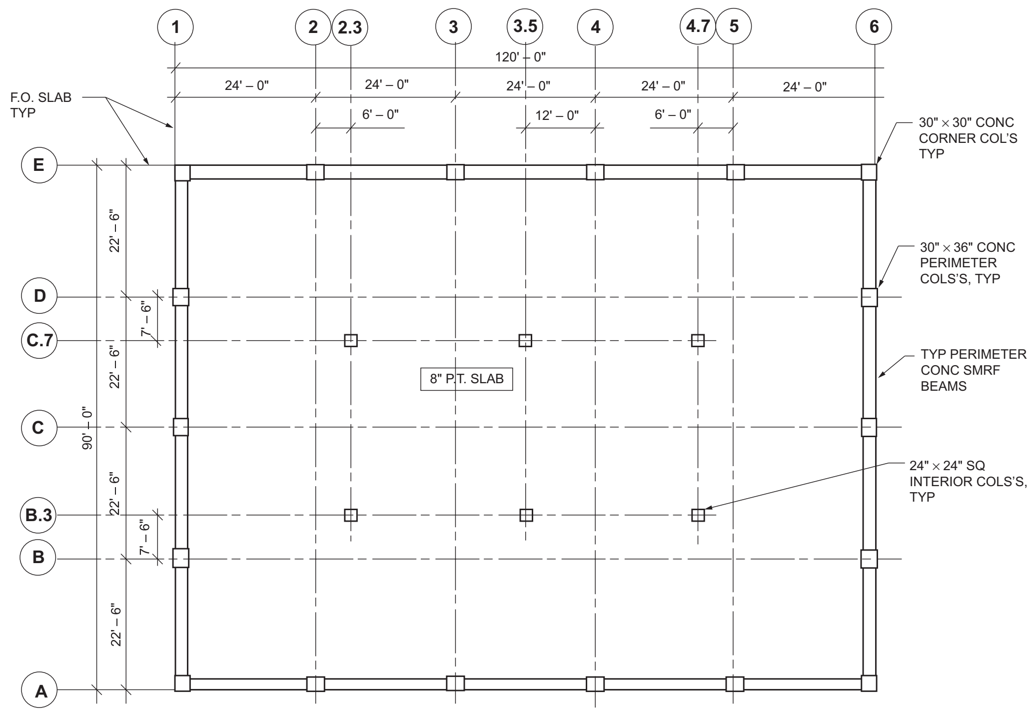


Figure 3-1. Typical floor plan

Note: Elevator, stair, and shaft openings are not shown on the plan.

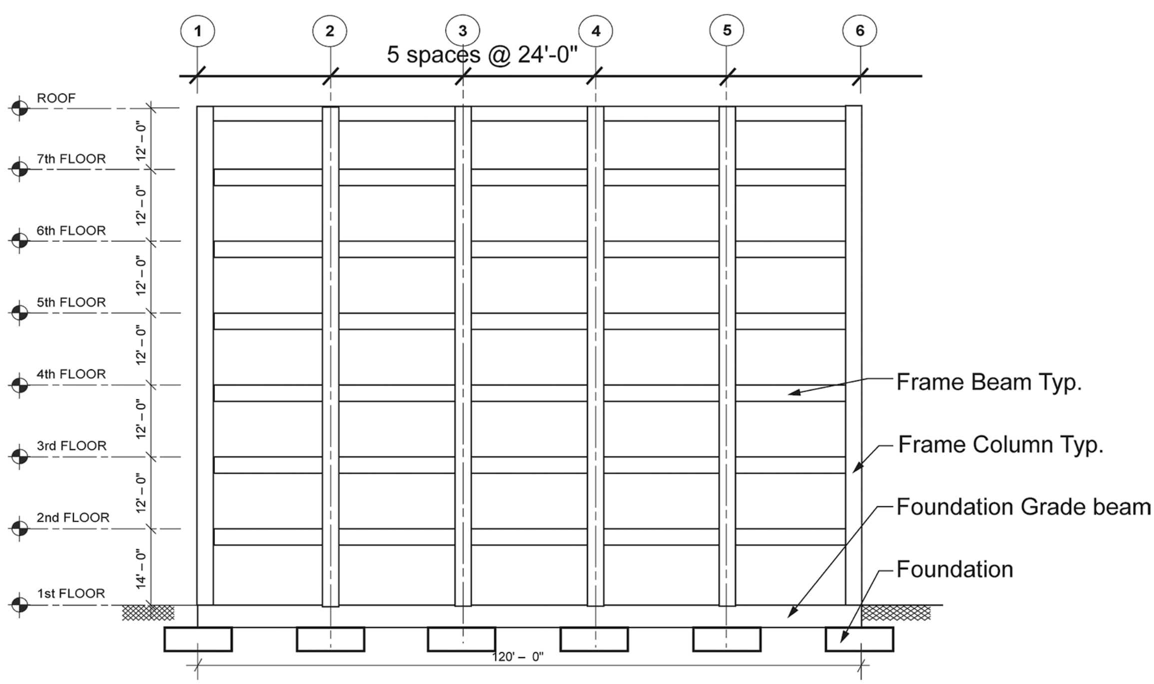


Figure 3-2. Typical frame elevation, line A

OUTLINE

Determine the controlling seismic parameters and the seismic forces to design structural elements per the following outline:

1. Determine Site Ground Motion
2. Establish Design Base Shear Coefficient
3. Determine Redundancy Factor
4. Determine Combined Effect of Horizontal and Vertical Earthquake-Induced Forces
5. Calculate Vertical Distribution of Seismic Forces
6. Establish Frame Nodal and Member Forces
7. Perform Analysis of the Frame and Evaluate Frame Drifts
8. Design SMRF Beam
9. Design SMRF Column
10. Perform Joint Shear Analysis
11. Detail Beams and Columns
12. Foundation Design Considerations and Detailing

GIVEN INFORMATION

The building has a floor system consisting of post-tensioned flat plate slabs. Vertical loads are carried by the slab, interior columns, and the perimeter frame system. The underside of the slab will be exposed architecturally. Use of perimeter SMRF provides for flexibility and voluminous interior spaces.

Seismic and site data:

Mapped spectral response accelerations
(from Chapter 22 of ASCE 7 or USGS Seismic Design Geodatabase)

$S_{MS} = 1.25$ §11.4.3

$S_1 = 0.64$ §11.4.3

$I_e = 1.0$ (Occupancy Category II) §11.5.1 and T 1.5.2

Site Class C

Seismic Loading Criteria[1]

Average story weights for seismic design loads are assumed to be distributed over the floor plate area:

Roof loads:

Material	**Roof (psf)**
Roofing	5.0
Concrete slab	100.0
Girders	15.0
Columns	9.0
Partition walls (seismic)[3]	5.0
Curtain wall	3.0
MEP	2.0[2]
Miscl.	1.0[2]
Total dead for seismic	140.0
L_r (roof live load)	20[5]

Typical floor loads:

Material	**2nd–5th Floors (psf)**	**6th–7th Floors (psf)**
Floor covering	1.0	1.0
Concrete slab[4]	100.0	100.0
Girders	27.0	15.0
Columns	18.5	18.5
Partition walls (seismic)[3]	10.0	10.0
Curtain wall	5.0	5.0
MEP	2.0[2]	2.0[2]
Miscl.	1.5[2]	1.5[2]
Total dead for seismic	165.0[2]	153.0[2]
L (floor live load)	50[5]	50[5]

Structural material specifications are given as follows:

Concrete $f'_c = 4000$ psi (normal weight)

Reinforcing A706, ($f_y = 60$ ksi) for longitudinal bars in special moment frames. Please note that A706 Grade 80 ksi is also permitted by ACI 318 Section 20.2.2.4.

Reinforcing A615, Grade 60 ($f_y = 60$ ksi) for stirrups and bars other than located in special moment frames. It should be noted Grade 80 ksi is also permitted for stirrups by ACI 318 Section 20.2.2.4.

Notes:

1. Customarily, different loading criteria are developed for gravity and seismic design of buildings. The loading criteria presented for this example are intended for seismic design; however, the same loading criteria are used for gravity loading calculations in this example for simplicity. This approach can lead to underestimating loading for gravity design of elements in some instances. It is best to develop separate loading criteria for gravity and seismic loading. In general, loading conditions for the design of buildings should be carefully evaluated and developed as they serve as the basis of design for the building.
2. A distributed MEP and misc. loading on the order of 5 psf is often considered for gravity design in office buildings. This distributed loading is intended to capture the loading impact of various MEP loading conditions, including impact of localized concentrated MEP loads on elements such as slabs and beams. Often the same MEP distributed loading is used for seismic mass calculations. However, this loading may be reduced in some instances for seismic mass calculation, as the average calculated MEP mass when distributed over the entire floor area will likely be less than the uniform loading considered for gravity design. This reduced MEP loading, if used, should ideally be justified through calculations. In this example, an MEP and misc. distributed loading of 3.0–3.5 psf is used, which has presumably been justified through calculations for the building. It should be noted that in addition to distributed MEP loading, the weight of mechanical units on the rooftop or on the floors, where they occur, needs to be considered in seismic mass calculations per ASCE 7 Section 12.7.2.
3. For partition loading, ASCE 7 Section 12.7.2 prescribes a minimum live loading of 15 psf for gravity design and 10 psf for seismic mass calculations.
4. Customarily, a ceiling loading on the order of 5 psf is considered for office buildings, but in the case of this example, it is assumed that the underside of the post-tensioned slab will be left exposed and a ceiling will not be added at any time in the future.
5. The code allows for roof and floor live load reduction, depending on the tributary area of the element considered. In this example, live load reduction has not been implemented for simplicity. It should be noted that designers at times elect to use higher live loads or forgo live load reduction for the design of some elements in the office buildings such as slabs and beams. This provides for more versatility for those instances where higher loading capacity may be required in the future.

1. Site Ground Motion — ASCE 7-16 §11.4

The maximum considered earthquake spectral response acceleration for short periods, S_{MS}, and at 1.0-second period, S_{M1}, adjusted for site class effects, shall be determined by ASCE 7 Equations 11.4-1 and 11.4-2.

$$S_{MS} = 1.5$$
$$S_{M1} = 0.90$$

Design spectral response acceleration at short periods, S_{DS}, and at 1.0-second period, S_{D1}, adjusted for site class effects, shall be determined by Equations 11.4-1 and 11.4-2.

$$S_{DS} = 2/3S_{MS} = (2/3)(1.5) = 1.0 \qquad \text{Eq 11.4-1}$$
$$S_{D1} = 2/3S_{M1} = (2/3)(0.90) = 0.60 \qquad \text{Eq 11.4-2}$$

2. Design Base Shear Coefficient — ASCE 7-16 §12.8

The coefficients for a reinforced concrete special moment frame building per ASCE 7 Table 12.2-1:

Response modification factor, $R = 8$ — T 12.2-1

System overstrength factor, $\Omega_0 = 3$

Deflection amplification factor, $C_d = 5.5$

The approximate fundamental building period, per ASCE 7 Section 12.8.2.1, is

$$T_a = C_t h_n^x = (0.016)(86\text{ ft})^{0.9} = 0.88\text{ sec} < T_L \qquad \text{Eq 12.8-8}$$

Alternately,

$$T_a = 0.1N = 0.1(7) = 0.7\text{ sec} \qquad \text{Eq 12.8-9}$$

Therefore, use $T_a = 0.88$ sec

Per ASCE 7 Section 12.8.2, it is permitted to calculate an upper limit period of $C_u \times T_a$ provided that it is confirmed through the analysis of the building that the building period equals or exceeds this upper limit. Once confirmed, this upper limit period can then be used for calculating seismic base shear for the strength design of the lateral system. In the case of this example, the calculated T_a is conservatively used without an increase, allowed by ASCE 7 Section 12.8.2.

The seismic base shear, V, in a given direction shall be determined in accordance with the following equation:

$$V = C_s W \qquad \text{Eq 12.8-1}$$

The calculation of seismic response coefficient, C_s, shall be

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{1.0}{\left(\frac{8}{1.0}\right)} = 0.125 \qquad \text{Eq 12.8-3}$$

From ASCE 7 Section 11.4.5.2, $T_S = S_{D1}/S_{DS} = (0.60/1.0) = 0.60\text{ sec} < T_a = 0.88$

Therefore, C_s is in the velocity range controlled by Equation 12.8-3 (see following calculations).

Calculation of $T_L = 8$ per ASCE 7 Figures 22-14 through 22-17.

C_s need not exceed the following:

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.6}{0.88\left(\frac{8}{1.0}\right)} = 0.085 \text{ for } T \le T_L \qquad \text{Eq 12.8-4}$$