Chapter 5

Linear Static Seismic Lateral Force Procedures

Roger M. Di Julio Jr., Ph.D., P.E.
Professor of Engineering, California State University, Northridge

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Abstract: The purpose of this chapter is to review and compare the sections of current seismic design provisions, which deal with the specification of seismic design forces. Emphasis will be on the equivalent static force procedures as contained in the 2000 edition of the International Building Code and the 1997 Edition of the Uniform Building Code. There are two commonly used procedures for specifying seismic design forces: The "Equivalent Static Force Procedure" and "Dynamic Analysis". In the equivalent static force procedure, the inertial forces are specified as static forces using empirical formulas. The empirical formulas do not explicitly account for the "dynamic characteristics" of the particular structure being designed or analyzed. The formulas were, however, developed to adequately represent the dynamic behavior of what are called "regular" structures, which have a reasonably uniform distribution of mass and stiffness. For such structures, the equivalent static force procedure is most often adequate. Structures that do not fit into this category are termed "irregular". Common irregularities include large floor-to-floor variation in mass or center of mass and soft stories. Such structures violate the assumptions on which the empirical formulas, used in the equivalent static force procedure, are based. Therefore, its use may lead to erroneous results. In these cases, a dynamic analysis should be used to specify and distribute the seismic design forces. Principles and procedures for dynamic analysis of structures were presented in Chapter 4.
5.1 INTRODUCTION

In order to design a structure to withstand an earthquake the forces on the structure must be specified. The exact forces that will occur during the life of the structure cannot be known. A realistic estimate is important, however, since the cost of construction, and therefore the economic viability of the project depends on a safe and cost efficient final product.

The seismic forces in a structure depend on a number of factors including the size and other characteristics of the earthquake, distance from the fault, site geology, and the type of lateral load resisting system. The use and the consequences of failure of the structure may also be of concern in the design. These factors should be included in the specification of the seismic design forces.

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Structures that do not fit into this category are termed "irregular". Common irregularities include large floor-to-floor variation in mass or center of mass and soft stories. Such structures violate the assumptions on which the empirical formulas, used in the equivalent static force procedure, are based. Therefore, its use may lead to erroneous results. In these cases, a dynamic analysis should be used to specify and distribute the seismic design forces.

A dynamic analysis can take a number of forms, but should account for the irregularities of the structure by modeling its "dynamic characteristics" including natural frequencies, mode shapes and damping.

The purpose of this chapter is to review and compare the sections of current seismic design provisions, which deal with the specification of seismic design forces. Emphasis will be on, as in the documents discussed, the equivalent static force procedure.

The following seismic design provisions are included in the discussion, which follows:


5.2 CODE PHILOSOPHY

The philosophy of a particular document indicates the general level of protection that it can be expected to provide. Most code documents clearly state that their standards are minimum requirements that are meant to provide for life safety but not to insure against damage.

The code-specified forces are generally lower than the actual forces that would occur in a large or moderate size earthquake. This is because the structure is designed to carry the specified loads within allowable stresses and deflections, which are considerably less than the ultimate or yield capacity (when using working stress design) of the materials and system. It is assumed that the larger loads that actually occur will be accounted for by the factors of safety and by the redundancy and
ductility of the system. Life safety is thereby insured but structural damage may be sustained.

5.3 UBC-97 PROVISIONS

UBC-97, basically provides for the use of the equivalent static force procedure or a dynamic analysis for regular structures under 240 feet tall and irregular structures 65 feet or less in height. A dynamic analysis is required for regular structures over 240 feet tall, irregular structures over 65 feet tall, and buildings that are located on poor soils (type Sₚ) and have a period greater than 0.7 seconds.

Although UBC-97 allows for both working stress design and alternately strength or load and resistance factor design, the earthquake loads are specified for use with the latter. This is a departure from previous editions where the earthquake loads were specified at the working stress level.

5.3.1 Design Base Shear V

The design base shear is specified by the formula:

\[ V = \frac{C_v I W}{RT} \] (5-1)

Where, \( T \) is the fundamental period of the structure in the direction under consideration, \( I \) is the seismic importance factor, \( C_v \) is a numerical coefficient dependent on the soil conditions at the site and the seismicity of the region, \( W \) is the seismic dead load, and \( R \) is a factor which accounts for the ductility and overstrength of the structural system. Additionally the base shear is dependent on the seismic zone factor, \( Z \). The base shear as specified by Equation 5-1 is subject to three limits:

The design base shear need not exceed:

\[ V = 2.5C_a I W \] (5-2)

And cannot be less than:

\[ V = 0.11C_a I W \] (5-3)

Where \( C_a \) is another seismic co-efficient dependent on the soil conditions at the site and regional seismicity.

Additionally in the zone of highest seismicity (zone 4) the design base shear must be greater than:

\[ V = \frac{0.8ZN_v I W}{R} \] (5-4)

Where \( N_v \) is a near-source factor that depends on the proximity to and activity of known faults near the structure. Faults are identified by seismic source type, which reflect the slip rate and potential magnitude of earthquake generated by the fault.

The near source factor \( N_v \) is also used in determining the seismic co-efficient \( C_v \) for buildings located in seismic zone 4.

5.3.2 Seismic Zone Factor Z

Five seismic zones, numbered 1 2A, 2B, 3 and 4 are defined. The zone for a particular site is determined from a seismic zone map (See Figure 5-1). The numerical values of \( Z \) are:

<table>
<thead>
<tr>
<th>Zone</th>
<th>1</th>
<th>2A</th>
<th>2B</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Z</td>
<td>0.075</td>
<td>0.15</td>
<td>0.2</td>
<td>0.3</td>
<td>0.4</td>
</tr>
</tbody>
</table>

The value of the coefficient thus normalized can be viewed as the peak ground acceleration, in percent of gravity, in each zone.

5.3.3 Seismic Importance Factor I

The importance factor \( I \) is used to increase the margin of safety for essential and hazardous facilities. For such structures \( I = 1.25 \). Essential structures are those that must remain operative immediately following an earthquake such as emergency treatment areas and fire stations. Hazardous facilities include those housing toxic or explosive substances (See Table 5-1).
5.3.4 Building Period T

The building period may be determined by analysis or using empirical formulas. A single empirical formula may be used for all framing systems:

\[ T = C \left( \frac{h}{h_n} \right)^{3/2} \] \hspace{1cm} (5-5)

where

\[ C = \begin{cases} 
0.035 & \text{for steel moment frames} \\
0.030 & \text{for concrete moment frames} \\
0.030 & \text{for eccentric braced frames} \\
0.020 & \text{for all other buildings}
\end{cases} \]

\( h_n \) = the height of the building in feet.

If the period is determined using Rayleigh's formula or another method of analysis, the value of \( T \) is limited. In Seismic Zone 4, the period cannot be over 30% greater than that determined by Equation 5-5 and in Zones 1, 2 and 3 it cannot be more than 40% greater. This provision is included to eliminate the possibility of using an excessively long period to justify an unreasonably low base shear. This limitation does not apply when checking drifts.

5.3.5 Structural System Coefficient R

The structural system coefficient, \( R \) is a measure of the ductility and overstrength of the structural system, based primarily on performance of similar systems in past earthquakes.

The values of \( R \) for various structural systems are found in Table 5-2. A higher number has the effect of reducing the design base shear. For example, for a steel special moment resisting frame the factor has value of 8.5, while and ordinary moment resisting frame the value is 4.5. This reflects the fact that a special moment resisting frame is expected to perform better during an earthquake.

5.3.6 Seismic Dead Load W

The dead load \( W \), used to calculate the base shear, includes not only the total dead load of the structures but also partitions, 25% of the floor live load in storage and warehouse occupancies and the weight of snow when the design snow load is greater than 30 pounds per square foot. The snow load may be reduced by up to 75% if its duration is short.

The rationale for including a portion of the snow load in heavy snow areas is the fact that in these areas a significant amount of ice can build up and remain on roofs.

5.3.7 Seismic Coefficients \( C_v \) and \( C_a \)

The seismic coefficients \( C_v \) & \( C_a \) are measures of the expected ground acceleration at the site. They may be found in Tables 5-3 and 5-4.

The co-efficient, and hence the expected ground accelerations are dependent on the seismic zone and soil profile type. They therefore reflect regional seismicity and soil conditions at the site.

Additionally in seismic zone 4 they also depend on the seismic source type and near source factors \( N_s \) and \( N_v \). These factors reflect local seismicity in the region of highest seismic activity.

5.3.8 Soil Profile Type \( S \)

The soil profile type reflects the effect of soil conditions at the site on ground motion. They are found in Table 5-5 and are labeled \( S_A \) through \( S_F \).
Table 5-1: Seismic Importance Factor

<table>
<thead>
<tr>
<th>Occupancy Category</th>
<th>Occupancy or Functions of Structure</th>
<th>Seismic Importance Factor, I</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Essential facilities</td>
<td>Group I, Division 1 Occupancies having surgery and emergency treatment areas. Fire and police stations. Garages and shelters for emergency vehicles and emergency aircraft. Structures and shelters in emergency-preparedness centers. Aviation control towers. Structures and equipment in government communication centers and other facilities required for emergency response. Standby power-generating equipment for Category 1 facilities. Tanks or other structures containing housing or supporting water or other fire-suppression material or equipment required for the protection of Category 1, 2 or 3 structures.</td>
<td>1.25</td>
</tr>
<tr>
<td>2. Hazardous facilities</td>
<td>Group H, Divisions 1, 2, 6 and 7 Occupancies and structures therein housing or supporting toxic or explosive chemicals or substances. Nonbuilding structures housing, supporting or containing quantities of toxic or explosive substances that, if contained within a building, would cause that building to be classified as a Group H, Division 1, 2 or 7 Occupancy.</td>
<td>1.25</td>
</tr>
<tr>
<td>3. Special occupancy structures</td>
<td>Group A, Divisions 1, 2 and 2.1 Occupancies. Buildings housing Group E, Divisions 1 and 3 Occupancies with a capacity greater than 300 students. Buildings housing Group B Occupancies used for college or adult education with a capacity greater than 500 students. Group I, Divisions 1 and 2 Occupancies with 50 or more resident incapacitated patients, but not included in Category 1. Group I, Division 3 Occupancies. All structures with an occupancy greater than 5,000 persons. Structures and equipment in power-generating stations, and other public utility facilities not included in Category 1 or Category 2 above, and required for continued operation.</td>
<td>1.00</td>
</tr>
<tr>
<td>4. Standard occupancy</td>
<td>All structures housing occupancies or having functions not listed in Category 1, 2 or 3 and Group U Occupancy towers.</td>
<td>1.00</td>
</tr>
<tr>
<td>5. Miscellaneous</td>
<td>Group U Occupancies except for towers.</td>
<td>1.00</td>
</tr>
</tbody>
</table>
Table 5-2. Structural Systems

<table>
<thead>
<tr>
<th>Basic Structural System</th>
<th>Lateral-Force-Resisting System Description</th>
<th>R</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Bearing wall system</td>
<td>1. Light-framed walls with shear panels</td>
<td></td>
</tr>
<tr>
<td></td>
<td>a. Wood Structural panel walls for structures three stories or less</td>
<td>5.5</td>
</tr>
<tr>
<td></td>
<td>b. All other light-framed walls</td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td>2. Shear walls</td>
<td></td>
</tr>
<tr>
<td></td>
<td>a. Concrete</td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td>b. Masonry</td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td>3. Light steel-framed bearing walls with tension only bracing</td>
<td>2.8</td>
</tr>
<tr>
<td></td>
<td>4. Braced frames where bracing carries gravity load</td>
<td></td>
</tr>
<tr>
<td></td>
<td>a. Steel</td>
<td>4.4</td>
</tr>
<tr>
<td></td>
<td>b. Concrete</td>
<td>2.8</td>
</tr>
<tr>
<td></td>
<td>c. Heavy timber</td>
<td>2.8</td>
</tr>
<tr>
<td>2. Building frame system</td>
<td>1. Steel eccentrically braced frame (EBF)</td>
<td>7.0</td>
</tr>
<tr>
<td></td>
<td>2. Light-framed walls with shear panels</td>
<td></td>
</tr>
<tr>
<td></td>
<td>a. Wood structural panel walls for structures three stories or less</td>
<td>6.5</td>
</tr>
<tr>
<td></td>
<td>b. All other light-framed walls</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>3. Shear walls</td>
<td></td>
</tr>
<tr>
<td></td>
<td>a. Concrete</td>
<td>5.5</td>
</tr>
<tr>
<td></td>
<td>b. Masonry</td>
<td>5.5</td>
</tr>
<tr>
<td></td>
<td>4. Ordinary braced frames</td>
<td></td>
</tr>
<tr>
<td></td>
<td>a. Steel</td>
<td>5.6</td>
</tr>
<tr>
<td></td>
<td>b. Concrete</td>
<td>5.6</td>
</tr>
<tr>
<td></td>
<td>c. Heavy timber</td>
<td>5.6</td>
</tr>
<tr>
<td></td>
<td>5. Special concentrically braced frames</td>
<td></td>
</tr>
<tr>
<td></td>
<td>a. Steel</td>
<td>6.4</td>
</tr>
<tr>
<td>3. Moment-resisting frame system</td>
<td>1. Special moment-resisting frame (SMRF)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>a. Steel</td>
<td>8.5</td>
</tr>
<tr>
<td></td>
<td>b. Concrete</td>
<td>8.5</td>
</tr>
<tr>
<td></td>
<td>2. Masonry moment-resisting wall frame (MMRWF)</td>
<td>6.5</td>
</tr>
<tr>
<td></td>
<td>3. Concrete intermediate moment-resisting frame (IMRF)</td>
<td>5.5</td>
</tr>
<tr>
<td></td>
<td>4. Ordinary moment-resisting frame (OMRF)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>a. Steel</td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td>b. Concrete</td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td>5. Special truss moment frames of steel (STMF)</td>
<td>6.5</td>
</tr>
<tr>
<td>4. Dual systems</td>
<td>1. Shear walls</td>
<td></td>
</tr>
<tr>
<td></td>
<td>a. Concrete with SMRF</td>
<td>8.5</td>
</tr>
<tr>
<td></td>
<td>b. Concrete with steel OMRF</td>
<td>4.2</td>
</tr>
<tr>
<td></td>
<td>c. Concrete with concrete IMRF</td>
<td>6.5</td>
</tr>
<tr>
<td></td>
<td>d. Masonry with SMRF</td>
<td>5.5</td>
</tr>
<tr>
<td></td>
<td>e. Masonry with steel OMRF</td>
<td>4.2</td>
</tr>
<tr>
<td></td>
<td>f. Masonry with concrete IMRF</td>
<td>4.2</td>
</tr>
<tr>
<td></td>
<td>g. Masonry with masonry MMRWF</td>
<td>6.0</td>
</tr>
<tr>
<td></td>
<td>2. Steel EBF</td>
<td></td>
</tr>
<tr>
<td></td>
<td>a. With steel SMRF</td>
<td>8.5</td>
</tr>
<tr>
<td></td>
<td>b. With steel OMRF</td>
<td>4.2</td>
</tr>
<tr>
<td></td>
<td>3. Ordinary braced frames</td>
<td></td>
</tr>
<tr>
<td></td>
<td>a. Steel with steel SMRF</td>
<td>6.5</td>
</tr>
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<td></td>
<td>c. Concrete with concrete SMRF</td>
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</tr>
<tr>
<td></td>
<td>d. Concrete with concrete IMRF</td>
<td>4.2</td>
</tr>
<tr>
<td></td>
<td>4. Special concentrically braced frames</td>
<td></td>
</tr>
<tr>
<td></td>
<td>a. Steel with steel SMRF</td>
<td>7.5</td>
</tr>
<tr>
<td></td>
<td>b. Steel with steel OMRF</td>
<td>4.2</td>
</tr>
<tr>
<td>5. Cantilevered column building systems</td>
<td>1. Cantilevered column elements</td>
<td>2.2</td>
</tr>
<tr>
<td>6. Shear wall-frame interaction systems</td>
<td>1. Concrete</td>
<td>5.5</td>
</tr>
</tbody>
</table>
The soil profile types are broadly defined in generic terms, for example “Hard Rock” for type $S_A$. They are also defined by the physical properties of the soil determined by standard tests including; shear wave velocity, standard penetration test, and undrained shear strength.

5.3.9 Seismic Source Type A, B and C

The seismic source type is used to specify the capability and activity of faults in the immediate vicinity of the structure. It is used only in seismic zone 4.

The seismic source types, labeled A, B or C, are found in Table 5-6. They are defined in terms of the slip rate of the fault and the maximum magnitude earthquake it is capable of generating. For example, the highest seismic risk is posed by seismic source type A, which is defined by a maximum moment magnitude of 7.0 or greater and a slip rate of 5mm/year or greater.

5.3.10 Near Source Factors $N_a$ and $N_v$

The near source factors $N_a$ and $N_v$ are found in Tables 5-7 and 5-8. In seismic zone 4, they are used in conjunction with the soil profile type to determine the seismic coefficients $C_v$ and $C_a$ (See Tables 5-3 and 5-4). For example, for seismic source type A at a distance to the fault of less than 2km, $N_a = 1.5$ (See Table 5-7). This is then used with Table 5-4 to determine the seismic co-efficient, $C_a$.

5.3.11 Distribution of Lateral Force $F_x$

The base shear $V$, as determined from Equations 5-1 through 5-4 are distributed over the height of the structure as a force at each level $F_i$, plus an extra force $F_t$ at the top:

$$V = F_i + \sum_{i=1}^{n} F_i$$  \hspace{1cm} (5-6)

The extra force at the top is:

$$F_t = 0.077V \leq 0.25V \quad \text{if} \quad T > 0.7 \text{sec.} \hspace{1cm} (5-7a)$$

$$F_t = 0.0 \quad \text{if} \quad T \leq 0.7 \text{sec.} \hspace{1cm} (5-7b)$$

$F_t$ accounts for the greater participation of higher modes in the response of longer period structures.

<table>
<thead>
<tr>
<th>Soil Profile Type</th>
<th>$Z = 0.075$</th>
<th>$Z = 0.15$</th>
<th>$Z = 0.2$</th>
<th>$Z = 0.3$</th>
<th>$Z = 0.4$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_A$</td>
<td>0.06</td>
<td>0.12</td>
<td>0.16</td>
<td>0.24</td>
<td>0.32 $N_v$</td>
</tr>
<tr>
<td>$S_B$</td>
<td>0.08</td>
<td>0.15</td>
<td>0.20</td>
<td>0.30</td>
<td>0.40 $N_v$</td>
</tr>
<tr>
<td>$S_C$</td>
<td>0.13</td>
<td>0.25</td>
<td>0.32</td>
<td>0.45</td>
<td>0.56 $N_v$</td>
</tr>
<tr>
<td>$S_D$</td>
<td>0.18</td>
<td>0.32</td>
<td>0.40</td>
<td>0.54</td>
<td>0.64 $N_v$</td>
</tr>
<tr>
<td>$S_E$</td>
<td>0.26</td>
<td>0.50</td>
<td>0.64</td>
<td>0.84</td>
<td>0.96 $N_v$</td>
</tr>
<tr>
<td>$S_F$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

$S_F$ Site-specific geotechnical investigation and dynamic site response analysis shall be performed.

<table>
<thead>
<tr>
<th>Soil Profile Type</th>
<th>$Z = 0.075$</th>
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<th>$Z = 0.4$</th>
</tr>
</thead>
<tbody>
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<td>0.12</td>
<td>0.16</td>
<td>0.24</td>
<td>0.32 $N_v$</td>
</tr>
<tr>
<td>$S_B$</td>
<td>0.08</td>
<td>0.15</td>
<td>0.20</td>
<td>0.30</td>
<td>0.40 $N_v$</td>
</tr>
<tr>
<td>$S_C$</td>
<td>0.09</td>
<td>0.18</td>
<td>0.24</td>
<td>0.33</td>
<td>0.40 $N_v$</td>
</tr>
<tr>
<td>$S_D$</td>
<td>0.12</td>
<td>0.22</td>
<td>0.28</td>
<td>0.36</td>
<td>0.44 $N_v$</td>
</tr>
<tr>
<td>$S_E$</td>
<td>0.19</td>
<td>0.30</td>
<td>0.34</td>
<td>0.36</td>
<td>0.36 $N_v$</td>
</tr>
<tr>
<td>$S_F$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

$S_F$ Site-specific geotechnical investigation and dynamic site response analysis shall be performed.
The remaining portion of the total base shear \((V - F_t)\) is distributed over the height, including the top, by the formula:

\[
F_x = \left( V - F_t \right) \left( w_i h_i \right) / \sum_{i=1}^{n} w_i h_i
\]

\[(5-8)\]

Where, \(w\) is the weight at a particular level and \(h\) is the height of a particular level above the shear base. At each floor, the force is located at the center of mass.

For equal story heights and weights, Equation 5-8 distributes the force linearly, increasing towards the top. Any significant variation from this triangular distribution indicates an irregular structure.

**Table 5-5. Soil Profile Types**

<table>
<thead>
<tr>
<th>Soil Profile Type</th>
<th>Soil Profile Name/Generic Description</th>
<th>Average Soil Properties for Top 100 Feet (30 480 mm) of Soil Profile</th>
</tr>
</thead>
<tbody>
<tr>
<td>(S_A)</td>
<td>Hard Rock</td>
<td>Shear Wave Velocity, feet/second (m/s)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&gt; 5,000</td>
</tr>
<tr>
<td>(S_B)</td>
<td>Rock</td>
<td>2,500 to 5,000</td>
</tr>
<tr>
<td>(S_C)</td>
<td>Very Dense Soil and Soft Rock</td>
<td>1,200 to 2,500</td>
</tr>
<tr>
<td>(S_D)</td>
<td>Stiff Soil Profile</td>
<td>600 to 1,200</td>
</tr>
<tr>
<td>(S_E)</td>
<td>Soft Soil Profile</td>
<td>&lt; 600</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(180)</td>
</tr>
<tr>
<td>(S_F)</td>
<td>Soil Requiring Site-specific Evaluation.</td>
<td></td>
</tr>
</tbody>
</table>

**Table 5-6. Seismic Source Type**

<table>
<thead>
<tr>
<th>Seismic Source Type</th>
<th>Seismic Source Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Faults that are capable of producing large magnitude events and that have a high rate of seismic activity.</td>
</tr>
<tr>
<td></td>
<td>(M \geq 7.0) SR (\geq 5)</td>
</tr>
<tr>
<td>B</td>
<td>All faults other than Types A and C.</td>
</tr>
<tr>
<td></td>
<td>(M \geq 7.0) SR &lt; 5</td>
</tr>
<tr>
<td></td>
<td>(M &lt; 7.0) SR &gt; 2</td>
</tr>
<tr>
<td></td>
<td>(M \geq 6.5) SR &lt; 2</td>
</tr>
<tr>
<td>C</td>
<td>Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity.</td>
</tr>
<tr>
<td></td>
<td>(M &lt; 6.5) SR (\leq 2)</td>
</tr>
</tbody>
</table>

**Table 5-7. Near-Source Factor \(N_a\)**

<table>
<thead>
<tr>
<th>Seismic Source Type</th>
<th>Closest Distance to Known Seismic Source</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(\leq 2) km</td>
</tr>
<tr>
<td>A</td>
<td>1.5</td>
</tr>
<tr>
<td>B</td>
<td>1.3</td>
</tr>
<tr>
<td>C</td>
<td>1.0</td>
</tr>
</tbody>
</table>

**Table 5-8. Near-Source Factor \(N_v\)**

<table>
<thead>
<tr>
<th>Seismic Source Type</th>
<th>Closest Distance to Known Seismic Source</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(\leq 2) km</td>
</tr>
<tr>
<td>A</td>
<td>2.0</td>
</tr>
<tr>
<td>B</td>
<td>1.6</td>
</tr>
<tr>
<td>C</td>
<td>1.0</td>
</tr>
</tbody>
</table>
5.3.12 Story Shear and Overturning Moment $V_X$ and $M_X$

The story shear at level $x$ is the sum of all the story forces at and above that level:

$$ V_x = F_i + \sum_{i=x}^n F_i $$

(5-9)

The overturning moment at a particular level $M_X$ is the sum of the moments of the story forces above, about that level. Hence:

$$ M_x = F_i(h_i - h_x) + \sum_{i=x}^n F_i(h_i - h_x) $$

(5-10)

Design must be based on the overturning moment as well as the shear at each level.

5.3.13 Torsion and P-Delta Effect

Accidental torsion, due to uncertainties in the mass and stiffness distribution, must be added to the calculated eccentricity. This is done by adding a torsional moment at each floor equal to the story shear multiplied by 5% of the floor dimension, perpendicular to the direction of the force. This procedure is equivalent to moving the center of mass by 5% of the plan dimension, in a direction perpendicular to the force.

If the deflection at either end of the building is more than 20% greater than the average deflection, it is classified as torsionally irregular and the accidental eccentricity must be amplified using the formula:

$$ A_x = \left[ \frac{\delta_{MAX}}{1.2\delta_{AVG}} \right]^2 \leq 3.0 $$

(5-11)

where

$\delta_{avg}$ = the average displacement at level $x$
$\delta_{max}$ = the maximum displacement at level $x$

P-Delta effects must be included in determining member forces and story displacements where significant.

5.3.14 Reliability / Redundancy Factor $\rho$

The seismic design forces and hence the base shear as determined from Equations 5-1 through 5-4, must be multiplied be a reliability/redundancy factor for the lateral load resisting system:

$$ 1 \leq \rho = 2 - \frac{20}{r_{max} \sqrt{A_B}} \leq 1.5 $$

(5-12)

Where, $A_B$ is the ground floor area of the structure in square feet and $r_{max}$ is the maximum element-story shear ratio.

The element story shear ratio ($r_i$) at a particular level is the ratio of the shear in the most heavily loaded member to the total story shear. The maximum ratio, $r_{max}$ is defined as the largest value of $r_i$ in the lower two-thirds of the building.

Special provisions for calculating $r$, for different lateral load resisting systems, are demonstrated in the examples that follow.

For special moment-resisting frames, if $\rho$ exceeds 1.25, additional bays must be added. For the purposes of determining drift (displacement), and in seismic zones 0, 1 and 2, $\rho = 1.0$.

5.3.15 Drift Limitations

The deflections due to the design seismic forces are called the design level response displacements, $\Delta_s$. The seismic forces used to determine $\Delta_s$ may be calculated using a reliability/redundancy factor equal to one, ignoring the limitation represented by Equation 5-3, and using an analytically determined period greater than the limits outlined in section 5.3.4.

The maximum inelastic response is defined as:
5. Linear Static Seismic Lateral Force Procedures

\[ \Delta_{st} = 0.7 R \Delta_s \]  
\hspace{2cm} (5-13)

Where, \( R \) is the structural system coefficient defined in Table 5-2.

Deflection control is specified in terms of the story drift, which is defined as the lateral displacement of one level relative to the level below. The story drift is determined from the maximum inelastic response as defined by Equation 5-13.

The displacement must include both translation and torsion. Hence, the drift must be checked in the plane of the lateral load resisting elements, generally at the ends of the building. P-Delta displacements must be included where significant.

For structures with a period less than 0.7 seconds, the maximum story drift is limited to:

\[ \Delta_s \leq 0.025 h \]  
\hspace{2cm} (5-14)

Where, \( h \) is the story height.

For structures with a period greater than 0.7 seconds:

\[ \Delta_s \leq 0.020 h \]  
\hspace{2cm} (5-15)

5.3.16 Irregular Structures

UBC-97 quantifies the notion of irregularity, which it breaks into two broad categories:

<table>
<thead>
<tr>
<th>Irregularity Type and Definition</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stiffness irregularity - soft story</td>
<td>A soft story is one in which the lateral stiffness is less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above.</td>
</tr>
<tr>
<td>Weight (mass) irregularity</td>
<td>Mass irregularity shall be considered to exist where the effective mass of any story is more than 150 percent of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.</td>
</tr>
<tr>
<td>Vertical geometric irregularity</td>
<td>Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral-force-resisting system in any story is more than 130 percent of that in an adjacent story. One-story penthouses need not be considered.</td>
</tr>
<tr>
<td>In-plane discontinuity in vertical lateral-force-resisting element</td>
<td>An in-plane offset of the lateral-load-resisting elements greater than the length of those elements.</td>
</tr>
<tr>
<td>Discontinuity in capacity - weak story</td>
<td>A weak story is one in which the story strength is less than 80 percent of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Irregularity Type and Definition</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Torsional irregularity - to be considered when diaphragms are not flexible</td>
<td>Torsional irregularity shall be considered to exist when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts of the two ends of the structure.</td>
</tr>
<tr>
<td>Re-entrant corners</td>
<td>Plan configurations of a structure and its lateral-force-resisting system contain re-entrant corners, where both projections of the structure beyond a re-entrant corner are greater than 15 percent of the plan dimension of the structure in the given direction.</td>
</tr>
<tr>
<td>Diaphragm discontinuity</td>
<td>Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50 percent of the gross enclosed area of the diaphragm, or changes in effective diaphragm stiffness of more than 50 percent from one story to the next.</td>
</tr>
<tr>
<td>Out-of-plane offsets</td>
<td>Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements.</td>
</tr>
<tr>
<td>Nonparallel systems</td>
<td>The vertical lateral-load-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral-force-resisting system.</td>
</tr>
</tbody>
</table>
vertical structural and plan structural irregularity. Vertical irregularities include soft or weak stories, large changes in mass from floor to floor and large discontinuities in the dimensions or in-plane locations of lateral load resisting elements. Plan irregular buildings include those which undergo substantial torsion when subjected to seismic loads, have re-entrant corners, discontinuities in floor diaphragms, discontinuity in the lateral force path, or lateral load resisting elements which are not parallel to each other or to the axes of the building.

The precise definitions of these irregularities are found in Tables 5-9 and 5-10. For a more detailed discussion of irregularity, see Chapter Six.

5.3.17 Dynamic Lateral Force Procedure

UBC-97 requires that, if the base shear determined by a dynamic analysis using a site-specific spectra is less than that specified by the static lateral force procedure, it must be scaled to equal that determined by the equivalent static force procedure. Similarly, if the base shear obtained from a dynamic analysis is greater than that specified by the static lateral force procedure, it may be scaled down. In this manner, the dynamic characteristics of the structure are modeled, and thus the forces are distributed properly, while the code level forces are maintained. If a site-specific spectrum is not available, the spectra provided in UBC-97 (see Figure 5-2) can be used.

![Design Response Spectra](image)

*Figure 5-2. Design Response Spectra*
5.3.18 Examples

Example 5-1:

Determine the UBC-97 design seismic forces for a three-story concrete shear wall office building. It is located in Southeastern California on rock with a shear wave velocity of 3000 ft/sec. The story heights are 13 feet for the first floor and 11 feet for the second and third floors. The story dead loads are 2200, 2000 and 1700 kips from the bottom up. The plan dimensions are 180 feet by 120 feet. The walls in the direction under consideration are 120 feet long and are without openings. The shear walls do not carry vertical loads. Sample calculations are presented and a complete tabulation is found in Table 5-11.

- Base Shear:

\[ V = \frac{C_s IW}{RT} \]  

\[ I = 1.0 \quad \text{Table 5-1} \]
\[ R = 5.5 \quad \text{(Shear Walls)} \]
\[ Z = 0.3 \quad \text{Section 5.3.2} \]
\[ C_v = 0.3 \quad \text{Table 5-3} \]
\[ T = 0.02(35)^{0.25} = .29 \text{ Seconds} \quad \text{Equation 5-5} \]
\[ W = 1700 + 2000 + 2200 = 5900 \text{ k} \]
\[ V = \frac{.3(1.0)}{5.5(.29)}(5900) = 1109.2 \text{ k} \]

\[ V \leq 2.5 \frac{C_s IW}{R} \]  

\[ C_a = 0.3 \quad \text{Table 5-4} \]
\[ V \leq 2.5 \frac{(3)(1)}{5.5} (5900) = 804.5 \text{ k} < 1109.2 \]

- Vertical Distribution:

\[ V \geq .11 \text{ Ca IW} \]  

\[ V \geq .11 (0.3) (1) (5900) = 194.7 < 804.5 \]

\[ V = 804.5 \text{ k} \]

- Story Shear:

\[ V = F_i + \sum_{i=1}^{n} F_i \quad \text{Equation 5-9} \]
\[ V_3 = 351.7 \text{ k} \]
\[ V_2 = 351.7 + 283.7 = 635.4 \text{ k} \]
\[ V_3 = 351.7 + 283.7 = 169.1 + 804.5 \text{ k} \]

- Overturning Moment:

\[ M_i = F_i (h_i - h_1) + \sum_{i=1}^{n} F_i (h_i - h_1) \quad \text{Eq. 5-10} \]
\[ M_3 = 351.7 (11) = 3869 \text{ ft-k} \]
\[ M_2 = 351.7 (22) + 283.7 (11) = 10,858 \text{ ft-k} \]
\[ M_1 = 351.7 (35) + 283.7 (24) + 169.1 (13) = 21,317 \]

<table>
<thead>
<tr>
<th>Table 5-11: Example 5-1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level</td>
</tr>
<tr>
<td>-------</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>Σ</td>
</tr>
</tbody>
</table>

- Allowable Inelastic Story Displacement:

\[ T \leq .7 \text{ seconds} \]
\[ \Delta_s \leq 0.025 \text{h} \]  
\[ \Delta_s \leq 0.025 (11 \times 12) = 3.3 \text{ inches} \]

\[ \Delta_s \leq 0.025 (11 \times 13) = 3.56 \text{ inches} \]

- Equivalent Elastic Story Displacement:

\[ \Delta \leq \frac{0.025h}{.7R} = \frac{0.025}{.7(5.5)} h = 0.0065h \]  
Eq. 5-13

\[ \Delta \leq 0.0065 (11 \times 12) = 0.858 \text{ inches} \]

\[ \Delta \leq 0.0065 (13 \times 12) = 1.01 \text{ inches} \]

- Reliability / Redundancy Factor:

For shear walls, \( r_i \) is the maximum value of the product of the wall shear and \( 10/l_w \), divided by the total shear, where \( l_w \) is the length of wall in feet (120 ft).

An approximation of \( r_{\text{max}} \) can be obtained by assuming that half the story shear is carried by each wall.

\[ r_{\text{max}} = \frac{(V_S/2)(10/120)}{V_S} = 0.04 \]

\[ A_B = 120 \times 180 = 21,600 \text{ ft}^2 \]

\[ \rho = 2 - \frac{20}{r_{\text{max}} \sqrt{A_B}} \]  
Equation 5-12

\[ \rho = 2 - \frac{20}{0.04 \sqrt{21,600}} = -1.4 < \rho_{\text{min}} = 1.0 \]

\[ \rho = 1.0 \]

**Example 5-2:**

Determine the UBC-97 design seismic forces for a nine story ductile moment resisting steel frame office building located in Los Angeles, California on very dense soil and soft rock. The building is located 5km from a fault capable of large magnitude earthquakes and that has a moderate slip rate (M>7, SR>2mm/yr). The story heights are all thirteen feet. The plan area is 100 feet by 170 feet. The total dead load is 100 pounds per square foot at all levels. The moment frames consist of two four bay frames in the transverse direction and two seven bay frames in the longitudinal direction. Sample calculations are presented and a complete tabulation is found in Table 5-12.

- Base Shear:

\[ V = \frac{C_s IW}{RT} \]  
Equation 5-1

\[ I = 1.0 \]  
Table 5-1
\[ R = 8.5 \text{ (SMRF)} \]  
Table 5-2
\[ \text{Seismic Zone 4} \]  
Figure 5-1
\[ Z = 0.4 \]  
Section 5.3.2
\[ \text{Soil Profile Type S}_c \]  
Table 5-5
\[ \text{Seismic Source Type B} \]  
Table 5-6
\[ N_c = 1.2 \]  
Table 5-7
\[ C_s = 0.56 \]  
Table 5-4

\[ N_s = 1.0 \]  
Table 5-7
\[ C_a = 0.4 \]  
Table 5-4

\[ V = \frac{67(1.0)}{85(1.25)} \times W = 0.063W = 0.063(15,300) = 964.8K \]

\[ T = 0.035 \times (117)^{3/4} = 1.25 \text{ seconds} \]  
Equation 5-5

\[ W = 0.1 \times (170) = 1700 \text{ k / floor} \]
\[ W = 0.9 \times (1700) = 15,300 \text{ k} \]

\[ V \leq 2.5 \frac{C_s I}{R} W \]  
Equation 5-2

\[ V \geq 0.11 C_a IW \]  
Equation 5-3
Since the building is in zone 4:

\[ V \geq \frac{0.8 N I}{R W} \]

\[ V \geq \frac{0.8(1.2)(1.0)}{8.5}(15,300) = 691.2 \text{k} < 964.8 \text{k} \]

\[ V = 964.8 \text{k} \]

- Vertical Distribution:

\[ T > 0.7 \text{ sec} \]

\[ F_t = 0.07 TV = 0.07(1.25)(964.8) = 84.4 \text{k} \]

\[ .25V = .25(964.8) = 241.2 > 84.4 \]

\[ F_i = 84.4 \text{k} \]

\[ (V-F_i) = 964.8 - 84.4 = 880.4 \]

\[ F_x = \frac{(V - F_i)(w_i h_i)}{\sum w_i h_i} \]

\[ 9 \quad 117 \quad 198.9 \]

\[ 8 \quad 104 \quad 176.8 \]

\[ 7 \quad 91 \quad 155.7 \]

\[ 6 \quad 78 \quad 132.6 \]

\[ 5 \quad 65 \quad 110.5 \]

\[ 4 \quad 52 \quad 88.4 \]

\[ 3 \quad 39 \quad 66.3 \]

\[ 2 \quad 26 \quad 45.2 \]

\[ 1 \quad 13 \quad 22.1 \]

\[ \Sigma \quad 15300. \quad 996.5 \quad 964.8 \]

\[ \Delta_x < 0.02 h = 0.02(13 \times 12) = 3.12 \text{ inches} \]

- Equivalent Elastic Story Displacement:

\[ \Delta_e \leq \frac{0.02 h}{R} = \frac{0.02 h}{7(8.5)} = 0.00336h \]

\[ \Delta \leq 0.00336 (13 \times 12) = 0.52 \text{ inches} \]

- Reliability / Redundancy Factor

For moment frames, \( r_i \) is normally 70% of the shear in two adjacent interior columns. An approximation for \( r_i \) can be obtained by assuming all interior columns carry equal shear and external columns carry half as much.

\[ \rho = 2 - \frac{20}{r_{max} \sqrt{A_B}} \]

\[ A_B = 100 \times 170 = 17,000 \text{ ft}^2 \]

Transverse Direction:
Two 4 Bay Frames

\[ r_{max} = 0.7(V/8 + V/8)/V = 0.175 \]

\[ \rho = 2 - \frac{20}{0.175 \sqrt{17,000}} = 1.12 \leq 1.25 \]

\[ \rho_{max} = 1.25 \quad \text{for special moment frame ok.} \]

\[ \rho = 1.12 \]

Longitudinal Direction:
Two 7 Bay Frames
Chapter 5

5.4 IBC2000 PROVISIONS

IBC2000 is broadly similar to UBC-97, but does contain significant differences. These include ground accelerations specified on a local basis by a set seismic risk maps.

The concept of a seismic use group, which is related to the importance factor in UBC-97, is introduced. In addition to defining the importance factor it is used to designate the seismic design category and to establish the allowable story drift.

The seismic design category determines the analysis procedures to be used and height and system limitations.

5.4.1 Seismic Use Group I, II, III

Each structure is assigned to a seismic use group based on the occupancy of the building and the consequences of severe earthquake damage. Three seismic hazard groups are defined:

GROUP III...."having essential facilities that are required for post-earthquake recovery and those containing substantial quantities of hazardous substances ". These facilities include fire and police stations, hospitals, medical facilities having emergency treatment facilities, emergency preparedness centers, operation centers, communication centers, utilities required for emergency backup, and structures containing significant toxic or explosive substances.

GROUP II...."have a substantial public hazard due to occupancy or use...". These include high occupancy buildings and utilities not required for emergency backup.

GROUP I -- All other buildings.

5.4.2 Occupancy Importance Factor I

An occupancy importance factor is assigned based on the seismic use group. This factor is used to increase the design base shear for structures in seismic use groups II and III.

The values of the importance occupancy factor are:

<table>
<thead>
<tr>
<th>Seismic Use Group</th>
<th>I</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>1.0</td>
</tr>
<tr>
<td>II</td>
<td>1.25</td>
</tr>
<tr>
<td>III</td>
<td>1.50</td>
</tr>
</tbody>
</table>

5.4.3 Maximum Considered Earthquake Ground Motion

Regional seismicity is specified by a series of maps. The maps provide the spectral response accelerations at short periods, $S_s$ and at a period of one second, $S_1$ (see Figures 5-3 and 5-4).

In areas of low seismic activity ($S_s \leq 0.15g$, $S_1 \leq 0.04g$) the acceleration need not be determined.

5.4.4 Site Class

The soil conditions at the site determine the structures “site class”. These are virtually identical to the soil profile types in UBC-97 (see Table 5-5).

5.4.5 Site Coefficients $F_a$ and $F_v$

The regional seismicity, as expressed by the maximum considered earthquake ground motion, $S_s$ and $S_1$, must be modified for the soil conditions at the site. These are defined by the site class. The maximum considered earthquake spectral response accelerations adjusted for site class effects, are:

$$S_{MS} = F_a S_s$$  \hspace{1cm} (5-16 a)

$$S_{M1} = F_v S_1$$  \hspace{1cm} (5-16 b)
Figure 5-4 IBC 2000 Spectral Map for Intermediate Period Range (T=1.0 Sec).
Where, $F_a$ and $F_v$ are the site coefficients defined in Tables 5-13 and 5-14.

For site class F, and site class E in regions of high seismicity ($S_s > 1.25g$ or $S_1 > 5g$), a site-specific geotechnical investigation must be performed.

5.4.6 Design Spectral Response Accelerations $S_{DS}$ and $S_{DI}$

The spectral accelerations for the design earthquake are:

$$S_{DS} = \frac{2}{3}S_{MS} \quad (5-17a)$$

$$S_{DI} = \frac{2}{3}S_{M1} \quad (5-17b)$$

These are the accelerations used to determine the design base shear.

5.4.7 Seismic Design Category

The structure must be assigned a seismic design category, which determines the permissible structural systems, limitations on height and irregularity, those components of the structure that must be designed for seismic loads, and the types of analysis required.

The seismic design categories, designated A through F, are presented in Tables 5-15 and 5-16. They depend on the seismic use group and the design spectral acceleration coefficients, $S_{DS}$ and $S_{DI}$. The structure is assigned the more severe of the two values taken for these tables.

5.4.8 Design Base Shear $V$

IBC2000 specifies the design base shear by the formula:

$$V = C_s W \quad (5-18)$$

The base shear is a percentage, $C_s$ of the total dead load $W$.

5.4.9 Total Dead Load $W$

The seismic dead load consists of the total weight of the structure, plus partitions and permanent equipment. It also includes 25% of floor live load in areas used for storage, and the
snow load if it is greater than 30 lb/ft². The snow load may be reduced by up to 80% if its duration is short.

5.4.10 Seismic Response Coefficient \( C_s \)

The seismic response coefficient is determined from the formula:

\[
C_s = \frac{S_{DS}}{R/I}
\]  \hspace{1cm} (5-19)

where

\( S_{DS} = \) the design spectral acceleration in the short period range
\( R = \) the response modification factor from Table 5-17 and defined below
\( I = \) the occupancy importance factor

The coefficient \( C_s \), as specified by Equation 5-19, is subject to three limits.

It need not exceed:

\[
C_s = \frac{5S_{I}}{R/I}
\]  \hspace{1cm} (5-20)

It must be greater than:

\[
C_s = \frac{S_{D1}}{T \cdot R/I}
\]  \hspace{1cm} (5-21)

Additionally for structures in seismic design categories E and F, and for structures with a 1 second spectral response greater than or equal to .6g, it cannot be less than:

\[
C_s = \frac{5S_{I}}{R/I}
\]  \hspace{1cm} (5-22)

where

\( S_{D1} = \) the design spectral response at a 1.0 second period
\( T = \) the fundamental period of the structure
\( S_1 = \) the maximum considered earthquake spectral response acceleration at a 1 second period

5.4.11 Building Period \( T \)

The building period can be estimated using the empirical formula:

\[
T_s = C_s \cdot h_o^{3/4}
\]  \hspace{1cm} (5-23)

where

\( h_o = \) the height of the structure

\[ Table 5-15. \text{Seismic Design Category Based on Short Period Response Accelerations} \]

<table>
<thead>
<tr>
<th>Value of ( S_{DS} )</th>
<th>Seismic Use Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>( S_{DS} &lt; 0.167g )</td>
<td>I</td>
</tr>
<tr>
<td>( 0.167g \leq S_{DS} &lt; 0.33g )</td>
<td>A</td>
</tr>
<tr>
<td>( 0.33g \leq S_{DS} &lt; 0.50g )</td>
<td>B</td>
</tr>
<tr>
<td>( 0.50g \leq S_{DS} )</td>
<td>D*</td>
</tr>
</tbody>
</table>

*Seismic Use Group I and II structures located on sites with mapped maximum considered earthquake spectral response acceleration at 1 second period, \( S_1 \), equal to or greater than 0.75g shall be assigned to Seismic Design Category E and Seismic Use Group III structure located on such sites shall be assigned to Seismic Design Category F.

\[ Table 4-16. \text{Seismic Design Category Based on 1Second Period Response Accelerations} \]

<table>
<thead>
<tr>
<th>Value of ( S_{D1} )</th>
<th>Seismic Use Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>( S_{D1} &lt; 0.067g )</td>
<td>I</td>
</tr>
<tr>
<td>( 0.067g \leq S_{D1} &lt; 0.133g )</td>
<td>B</td>
</tr>
<tr>
<td>( 0.133g \leq S_{D1} &lt; 0.20g )</td>
<td>C</td>
</tr>
<tr>
<td>( 0.20g \leq S_{D1} )</td>
<td>D*</td>
</tr>
</tbody>
</table>
5. Linear Static Seismic Lateral Force Procedures

\[
C_r = \begin{cases} 
0.035 & \text{for steel moment frames} \\
0.030 & \text{for concrete moment frames} \\
0.030 & \text{for eccentric braced frames} \\
0.020 & \text{for all other buildings}
\end{cases}
\]

\[h_n = \text{the height of the building in feet.}\]

An alternate formula is provided for steel and concrete moment frame buildings twelve stories or less in height and with story heights ten feet or greater:

\[T_a = 0.1 N \quad (5-24)\]

where, \(N\) is the number of stories.

The period may also be determined by an analysis. The period used to determine the base shear is subject to an upper limit, which is based on the design spectral response acceleration at a period of one second, \(S_{D1}\). The relationship between \(S_{D1}\) and the maximum allowable period used to specify the base shear is:

\[
\begin{array}{c|c}
S_{D1} & T_{max}/T_a \\\n\hline
\geq 0.4 & 1.2 \\\n0.3 & 1.3 \\\n0.2 & 1.4 \\\n0.15 & 1.5 \\\n\leq 0.1 & 1.7 \\
\end{array}
\]

This provision insures that an excessively long analytically determined period is not used to justify an unrealistically low design base shear. When determining drifts these limits do not apply.

5.4.12 Response Modification Factor R

The response modification factor, \(R\) serves the same function as the structural system coefficient in UBC–97. It reduces the design loads to account for the damping and ductility of the structural system. An abbreviated set for values for \(R\) is found in Table 5-17.

<table>
<thead>
<tr>
<th>Basic Seismic-Force-Resisting System</th>
<th>Response Modifications Coefficient, (R)</th>
<th>Deflection Amplification Factor, (C_d)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing Wall Systems</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Special reinforced concrete shear walls</td>
<td>5.5</td>
<td>5</td>
</tr>
<tr>
<td>Ordinary reinforced concrete shear walls</td>
<td>4.5</td>
<td>4</td>
</tr>
<tr>
<td>Building Frame Systems</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Special steel concretically braced frames</td>
<td>6</td>
<td>5</td>
</tr>
<tr>
<td>Special reinforced concrete shear walls</td>
<td>6</td>
<td>5</td>
</tr>
<tr>
<td>Moment Resisting Frame Systems</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Special steel moment frames</td>
<td>8</td>
<td>5.5</td>
</tr>
<tr>
<td>Ordinary steel moment frames</td>
<td>4</td>
<td>3.5</td>
</tr>
<tr>
<td>Dual Systems with Intermediate Moment Frames Capable of Resisting at Least 25% of Prescribed Seismic Forces</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Special reinforced concrete shear walls</td>
<td>6</td>
<td>5</td>
</tr>
<tr>
<td>Ordinary reinforced concrete shear walls</td>
<td>5.5</td>
<td>4.5</td>
</tr>
<tr>
<td>Inverted Pendulum Systems and Cantilevered Column Systems</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Special steel moment frames</td>
<td>2.5</td>
<td>2.5</td>
</tr>
<tr>
<td>Ordinary steel moment frames</td>
<td>1.25</td>
<td>2.5</td>
</tr>
</tbody>
</table>
5.4.13 Vertical Distribution of Force $F_x$

The seismic force at any level is a portion of the total base shear:

$$F_x = C_{vx} V$$  \hspace{1cm} (5-25)$$

where

$$C_{vx} = \frac{w_i h_i^k}{\sum_{i=1}^n w_i h_i^k}$$  \hspace{1cm} (5-26)$$

where

- $w_i, w_x =$ the portion of the dead load at or assigned to level $i$ or $x$
- $h_i, h_x =$ height above the base to level $i$ or $x$
- $k =$ an exponent related to the building period as follows:

For buildings with a period of 0.5 seconds or less, $k=1.0$. If the period is 2.5 seconds or more, $k=2.0$. For buildings with a period between 0.5 and 2.5 seconds, it may be taken as 2.0 or determined by linear interpolation between 1.0 and 2.0.

For $k=1.0$ the distribution is a straight line. This is reasonable for short buildings with a regular distribution of mass and stiffness. Hence, $k=1.0$ for buildings with a period of 0.5 seconds or less.

For $k=2.0$ the distribution is a parabola with the vertex at the base. This is reasonable for tall regular buildings where the participation of higher modes is significant. Hence, $k=2.0$ for buildings with a period of 2.5 seconds or more. This effect is accounted for by the force $F_t$, placed at the roof in UBC-97.

5.4.14 Overturning Moment $M_x$

IBC2000 allows for a reduction in the design overturning moment:

$$M_x = \tau \sum_{i=x}^n F_i (h_i - h_x)$$  \hspace{1cm} (5-27)$$

where

- $\tau =$ 1.0 for the top 10 stories
- $\tau =$ 0.8 for the 20th story from the top and below and is interpolated between 0.8 and 1.0 for stories in between.

Part of the reasoning behind this reduction is that the design story forces are an envelope of the maximaums at each floor, and it is unlikely that they will all reach a maximum simultaneously.

5.4.15 Drift Limitations

For buildings, other than masonry, over four stories the allowable drifts are:

- $\Delta \leq \Delta a = 0.010 h_{sx}$ Use Group III \hspace{1cm} (5-28)
- $\Delta \leq \Delta a = 0.015 h_{sx}$ Use Group II \hspace{1cm} (5-29)
- $\Delta \leq \Delta a = 0.020 h_{sx}$ Use Group I \hspace{1cm} (5-30)

For buildings four stories or less and height, other than masonry, the allowable drifts are:

- $\Delta \leq \Delta a = 0.015 h_{sx}$ Use Group III \hspace{1cm} (5-28a)
- $\Delta \leq \Delta a = 0.020 h_{sx}$ Use Group II \hspace{1cm} (5-29a)
- $\Delta \leq \Delta a = 0.025 h_{sx}$ Use Group I \hspace{1cm} (5-30a)

where

- $\Delta =$ the design interstory displacement
- $\Delta a =$ the allowable story displacement
- $h_{sx} =$ the height of the story below level $x$

The design interstory displacement $\Delta$, is the difference in the deflections $\delta_x$, at the top and bottom of the story under consideration. It is based on the calculated deflections and is evaluated by the formula:

$$\delta_x = \frac{C_d \delta_{se}}{I}$$  \hspace{1cm} (5-31)$$

where

- $C_d =$ the deflection amplification factor
- $\delta_{se} =$ the deflections determined by an elastic analysis.
- $I =$ the occupancy importance factor
The deflection amplification factor $C_d$ is assigned values from 1.25 to 5.5 and accounts for the ductility of the system and the properties of the materials from which it is constructed (see Table 5-17).

In determining these deflections the period determined by an analysis may be used to calculate the base shear without considering the limitation on the period discussed in Section 5.4.11. This has the implication that lower story forces may be used to determine deflections than are used to determine member forces. A similar provision is contained in UBC-97.

Where significant, P-Delta and torsional deflections must be considered in satisfying the drift limitation. This is discussed further in the next section.

5.4.16 Torsion and P-Delta Effect

Torsion is accounted for in same manner as in UBC-97. The torsional moment resulting from the location of the center of mass plus that resulting from an assumed movement of five percent of the plan dimension must be accounted for.

For buildings with torsional irregularity, in seismic design categories C through F, the five percent accidental torsion must be amplified using Equation 5-11. For this purpose a building is irregular if the diaphragm is rigid and the maximum interstory displacement is more than 1.2 times the average.

The P-Delta effect must be included in the computation of story shears, story drifts and member forces when the value of the "stability coefficient" has a value, for any story, such that:

$$\theta = \frac{P_x \Delta}{V_x h_x C_d} > 0.10 \quad (5-32)$$

where

$\Delta$ = the design story drift
$V_x$ = the seismic force acting between level x and x-1
$h_x$ = the story height below level x
$P_x$ = total gravity load at and above level x
$C_d$ = the deflection amplification factor

The stability coefficient can be visualized as the ratio of the P-Delta moment ($P_x \Delta$) to the lateral force story moment ($V_x h_x$). Hence if the

<table>
<thead>
<tr>
<th>Irregularity Type and Description</th>
<th>Plan Structural Irregularities</th>
</tr>
</thead>
<tbody>
<tr>
<td>1a Torsional Irregularity—to be considered when diaphragms are not flexible</td>
<td>Torsional irregularity shall be considered to exist when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure.</td>
</tr>
<tr>
<td>1b Extreme Torsional Irregularity – to be considered when diaphragms are not flexible</td>
<td>Extreme torsional irregularity shall be considered to exist when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure.</td>
</tr>
<tr>
<td>2 Re-entrant Corners</td>
<td>Plan configurations of a structure and its lateral force-resisting system contain re-entrant corners, where both projections of the structure beyond a re-entrant corner are greater than 15 percent of the plan dimension of the structure in the given direction.</td>
</tr>
<tr>
<td>3 Diaphragm Discontinuity</td>
<td>Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50 percent of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50 percent from one story to the next.</td>
</tr>
<tr>
<td>4 Out-of-Plane Offsets</td>
<td>Discontinuities in a lateral force resistance path, such as out-of-plane offsets of the vertical elements.</td>
</tr>
<tr>
<td>5 Nonparallel Systems</td>
<td>The vertical lateral force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral force-resisting system.</td>
</tr>
</tbody>
</table>
Chapter 5

P-Delta moment is equal to 10 percent of the story moment at any floor the P-Delta effect should be considered. The code also specifies an upper limit on the stability coefficient.

5.4.17 Irregularity

IBC2000 defines irregularity in a manner similar to UBC-97, but goes further by assigning a building to a seismic design category based on its irregularity. It distinguishes between the two broad categories of plan and vertical irregularity.

Plan irregularities include: a non-symmetrical geometric configuration, re-entrant corners, significant torsion due to eccentricity between mass and stiffness, nonparallel lateral force resisting elements, out of plane offsets and discontinuous diaphragms.

Vertical irregularities include: soft and weak stories, large changes in mass-stiffness ratios between adjacent floors, large changes in plan dimension from floor to floor and significant horizontal offsets in the lateral load system.

The definitions of plan and vertical structural irregularities and their assigned seismic design categories are found in Tables 5-18 and 5-19.

5.4.18 Reliability Factor $\rho$

The reliability factor $\rho$ is identical to and serves the same function as in UBC-97 (See section 5.3.14, Equation 5-12). It is assigned a value of 1.0 for seismic design categories A, B and C. For special moment resisting frames in Seismic Design Category D, $\rho$ cannot exceed 1.25. For special moment resisting frames in Seismic Design Categories E and F, $\rho$ cannot exceed 1.1.

5.4.19 Analysis Procedures

The minimum level of structural analysis is dependent on the seismic design category. For buildings in category A, the design lateral force at all floors is 1 % of gravity. Buildings in categories B and C, whether regular or irregular, may be analyzed using the equivalent lateral force procedure.

The analysis procedure for buildings in categories D, E & F is specified as follows. Regular buildings up to 240 feet in height may be analyzed using the equivalent lateral force procedure. Buildings that are either over 240 feet tall, irregular, located on poor soils, or

### Table 5-19. Vertical Structural Irregularities

<table>
<thead>
<tr>
<th>Irregularity Type and Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1a</td>
</tr>
<tr>
<td>1b</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>4</td>
</tr>
<tr>
<td>5</td>
</tr>
</tbody>
</table>

- A soft story is one in which the lateral stiffness is less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above.
- An extreme soft story is one in which the lateral stiffness is less than 60 percent of that in the story above or less than 70 percent of the average stiffness of the three stories above.
- Mass Irregularity shall be considered to exist where the effective mass of any story is more than 150 percent of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.
- Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral force-resisting system in any story is more than 130 percent of that in an adjacent story.
- An in-plane offset of the lateral force-resisting elements greater than the length of those elements or a reduction in stiffness of the resisting element in the story below.
- A weak story is one in which the story lateral strength is less than 80 percent of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.
close to known faults in areas of high seismicity require various types of dynamic analysis.

5.4.20 Dynamic Analysis

Provisions are included for a simplified two dimensional version of modal analysis which is applicable to regular structures with independent orthogonal seismic force resisting systems. For such structures the motion is predominantly planar and a two dimensional model may be appropriate.

For irregular structures or with interacting seismic force resisting systems a three dimensional model is required.

The required base shear is equal to that determined by Equation 5-18, where the period used may be 20 percent longer than the maximum period allowed in the equivalent lateral force procedure (see Section 5.4.11). The justification for this is that a modal analysis is more accurate than a static analysis. Although the total force on the building does not change appreciably its distribution over the height is more accurately modeled.

5.4.21 Examples

Example 5-3:

Rework Example 5-1 using IBC2000 and special reinforced concrete shear walls.

- Base Shear:
  \[ V = C_s W \]  
  \[ \text{Equation 5-18} \]

  Seismic use group I
  \[ I = 1.0 \]  
  \[ \text{Section 5.4.1} \]
  \[ S_s = .5g \]  
  \[ \text{Figure 5-3} \]
  \[ S_1 = .2g \]  
  \[ \text{Figure 5-4} \]
  Site class B
  \[ F_a = 1.0 \]  
  \[ \text{Table 5-5} \]
  \[ F_r = 1.0 \]  
  \[ \text{Table 5-13} \]
  \[ S_{MS} = F_a S_s = .5g \]  
  \[ \text{Eq. 5-16a} \]
  \[ S_{M1} = F_r S_1 = .2g \]  
  \[ \text{Eq. 5-16b} \]
  \[ S_{DS} = 2/3 S_{MS} = 2/3 (.5) = .333g \]  
  \[ \text{Eq. 5-17a} \]

- Allowable Inelastic Story Displacements:
seismic use group I
less than four stories
\[ \Delta_a = 0.025h_{sx} \]  
Equation 5-30a

1st Floor:
\[ \Delta = 0.025(13)(12) = 3.9 \text{ inches} \]
2nd and 3rd Floors:
\[ \Delta = 0.025(11)(12) = 3.3 \text{ inches} \]

- Equivalent Elastic Story Displacement:
\[ \delta = \frac{C_d \Delta}{I} \]  
Equation 5-31

\[ C_d = 5 \]  
Table 5-17

\[ \delta = 5 \delta_{xe} \]

1st Floor:
\[ \Delta = \Delta_a / 5 = 3.9 / 5 = 0.78 \text{ inches} \]
2nd and 3rd Floors:
\[ \Delta = \Delta_a / 5 = 3.3 / 5 = 0.66 \text{ inches} \]

- Reliability Factor:
\[ \rho = 1.0 \]  
Section 5.4.18

seismic design category C
\[ \rho = 1.0 \]  
Section 5.4.18

**Example 5-4:**

Rework Example 5-2 using IBC2000.

- Base Shear:

\[ V = C_s W \]  
Equation 5-18

seismic use group I

\[ S_s = 2.05g \]  
Section 5.4.11

\[ S_i = 0.81g \]  
Section 5.4.12

site class C

\[ F_s = 1.0 \]  
Table 5-5

\[ F_v = 1.3 \]  
Table 5-13

\[ S_{MS} = F_s S_s = 1.0(2.05) = 2.05 \]  
Eq. 5-16a

\[ S_{MI} = F_v S_i = 1.3(.81) = 1.05 \]  
Eq. 5-16b

\[ S_{DS} = 2/3 S_{MS} = 2/3(2.05) = 1.37g \]  
Eq. 5-17a

\[ S_{DI} = 2/3 S_{MI} = 2/3 (1.05) = 0.7g \]  
Eq. 5-17b

\[ S_1 \geq 0.75g \]  
Tables 5-15, 16; footnote a.

seismic design category E

\[ R = 8 \] (special moment frame)

\[ C_s = \frac{S_{DS}}{R/I} = \frac{1.37}{8/1} = .171 \]  
Equation 5-19

\[ T = 0.035(117)^{1/3} = 1.25 \text{ sec} \]  
Equation 5-23

\[ C_s \leq \frac{S_{DI}}{TR/I} = \frac{.7}{1.25(8/1)} = .07 \]  
Equation 5-20

\[ C_s \geq .04S_{DI} I = .044(1.37)(1) = .0603 \text{ Eq. 5-21} \]

\[ C_s \geq .5S_1 I = \frac{.5(81)}{8/1} = .051 \]  
Equation 5-22

\[ C_s = .07g \]

\[ V = C_s W = .07 (15,300) = 1071 \text{ k} \]

- Vertical Distribution:

\[ C_{vx} = \frac{w_i h_i^k}{\sum_{i=1}^{n} w_i h_i^k} \]  
Equation 5-26

Interpolate to find k:

\[ k = 1.0 + (1.25-.5)/(2.5-.5) = 1.375 \]

\[ h_{1.375}^{1.375} = 117 \text{ k} \]

\[ C_{v9} = 1700(697.8)/1700(3000.9) = .233 \]

\[ F_9 = .233(1071) = 250 \text{ k} \]

See Table 5-21.

The story shear is determined by the same procedure as UBC-97.

- Overturning Moment:

\[ M_x = \tau \sum_{i=x}^{n} F_i(h_i - h_x) \]  
Equation 5-27

\[ \tau = 1.0 \] for top ten stories

Since \( \tau = 1.0 \) the procedure is the same as for UBC-97. See Table 5-21.
Table 5-21: Example 5-4

<table>
<thead>
<tr>
<th>Level</th>
<th>hx (ft)</th>
<th>wx (k)</th>
<th>hx \times C_{vx}^{1.375} (k)</th>
<th>F_x (k)</th>
<th>V_x (k)</th>
<th>M_x (ft-k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>117</td>
<td>1700</td>
<td>697.8</td>
<td>233</td>
<td>250</td>
<td>3250</td>
</tr>
<tr>
<td>8</td>
<td>104</td>
<td>1700</td>
<td>593.5</td>
<td>198</td>
<td>212</td>
<td>462</td>
</tr>
<tr>
<td>7</td>
<td>91</td>
<td>1700</td>
<td>493.9</td>
<td>165</td>
<td>177</td>
<td>639</td>
</tr>
<tr>
<td>6</td>
<td>78</td>
<td>1700</td>
<td>399.6</td>
<td>133</td>
<td>142</td>
<td>781</td>
</tr>
<tr>
<td>5</td>
<td>65</td>
<td>1700</td>
<td>311.0</td>
<td>104</td>
<td>111</td>
<td>892</td>
</tr>
<tr>
<td>4</td>
<td>52</td>
<td>1700</td>
<td>228.8</td>
<td>76</td>
<td>81</td>
<td>973</td>
</tr>
<tr>
<td>3</td>
<td>39</td>
<td>1700</td>
<td>155.1</td>
<td>51</td>
<td>55</td>
<td>1028</td>
</tr>
<tr>
<td>2</td>
<td>26</td>
<td>1700</td>
<td>88.2</td>
<td>29</td>
<td>31</td>
<td>1059</td>
</tr>
<tr>
<td>1</td>
<td>13</td>
<td>1700</td>
<td>35.0</td>
<td>11</td>
<td>12</td>
<td>1071</td>
</tr>
<tr>
<td>Σ</td>
<td>15300</td>
<td>3000.9</td>
<td>1.0</td>
<td>1071</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- Allowable Inelastic Story Displacements:

Seismic use group I
\[ \Delta_a = 0.02h_x = 0.02(13)(12) = 3.12 \text{ inches} \quad \text{Eq. 5-30} \]

- Equivalent Elastic Story Displacements:

\[ C_d = 5.5 \quad \text{Table 5-17} \]
\[ \delta = C_d \delta_{el}/ I = 5.5 \delta_{el} \quad \text{Equation 5-31} \]
\[ \Delta \leq 2.34/5.5 = 0.567 \text{ inches} \]

- Reliability Factor:

The calculations are the same as for UBC-97 (See example 5-2):
\[ \rho = 1.0 \quad \text{Longitudinal} \]
\[ \rho = 1.12 \quad \text{Transverse} \]

But in seismic design category E:
\[ \rho_{\text{max}} = 1.1 \quad \text{Section 5.4.18} \]

Therefore, we need more transverse bays. Note that \( \rho \) will be even higher using actual shears.

### 5.5 CONCLUSION

Basic linear static lateral force procedures of the 1997 UBC, the 1997 NEHRP, and the 2000 IBC codes were discussed. Numerical examples were provided to highlight practical applications of these procedures.