EARTHQUAKE RESISTANT STEEL STRUCTURES

BASIC CONCEPTS
The building is expected to behave mainly in elastic range and should not undergo considerable plastic deformations (damage) under service loads.

Whereas, it is aimed to sustain large plastic deformations while preserving global stability under extreme earthquake loads.

Doesn’t it seem quite contradictory: we don’t want the buildings to deform under service loads while we want them to deform as much as they can under rare seismic loads ??? Let’s think about this together!
How do we assure such expectations?

- We choose

  **Appropriate load factors** (especially \( R \) factor for design earthquake load),

  **Combinations of loading cases** consistent with our basic goals.
How do we assure such expectations?

- We guarantee the development of large inelastic deformations without the stability loss via the use of:
  - Capacity based design principles
  - Proper seismic detailing
  - Limiting lateral displacements and considering P-Delta effects under design earthquake loads.
- In other words, by means of all these considerations we assure global ductility.
Isn’t there any other way of the design?

- It is possible to design buildings in a way that they are not expected to undergo inelastic deformations under extreme events and remain generally elastic. Such a building would have extremely high elastic stiffness compared to the current design practice in this case. In addition, there wouldn’t be any need for capacity based design, special detailing requirements, limiting displacements; briefly achieving adequate global ductility wouldn’t be a necessity…

- It seems easier to design in such a way. So why don’t we choose this philosophy??? Could you provide your comments, please?
What is ductility?

- Material ductility
- Member ductility
- Global ductility

Devrim Ozhendekci
Material ductility is an important index of the ability of a material to withstand deformation without fracture. Material ductility, $\mu_M$, is usually represented by the equation:

$$\mu_M = \frac{\varepsilon_u}{\varepsilon_y}$$

where $\varepsilon_u$ and $\varepsilon_y$, are the material strains at fracture and onset of yielding, respectively. At room temperature typical structural steels will have static ductility on order of 20. However, under cyclic loads, the material ductility decreases because of fatigue. Typical structural steels may be regarded as having available ductility for seismic purposes on the order of 10.
As a special case, for axially loaded tensile members, material ductility and the member ductility are the same. Whereas, plastic hinge/s develop and affect the member curvature and end rotations in flexural elements. Member curvature would be:

\[ \phi = \frac{\varepsilon_T + \varepsilon_C}{d} \]

Extreme tensile strain

Extreme compression strain

Cross section height

CURVATURE
What is ductility? – Individual Member Ductility.
What is ductility? -Individual Member

Ductility-

- Inelastic moment-curvature behavior of beam.
- Idealized moment-curvature distribution for inelastic beam.

Deformation of beam under applied external moment.
Global system ductility can be defined by the expression:

$$\mu = \frac{\Delta u}{\Delta y}$$

where $\Delta u$ and $\Delta y$ are, respectively, the lateral deflection of the structure at failure and onset of yielding. Determination of the point of yielding in a structure is difficult, as is selection of a point of ultimate strength.

Following figure presents a pushover curve for an arbitrary steel structure. Accordingly,

- Yielding of individual beams $\sim 5$ in.
- Failure of individual connections $\sim 30$ in.
- Eventually, total loss of stability and collapse $\sim 40$ in.
What is ductility? –Global Ductility–

Global ductility ~ 3 for this arbitrary structure. (It is aimed to be on the order of 3 to 4 in seismic design codes.)
What is capacity based design?

- Simply speaking, manipulating the strength capacities of structural elements during design phase for creating weaker links on purpose. In a way, we decide in which location the plasticity (damage) would lump under a rare seismic event. This is relieving since we have the opportunity to project the probable failure modes and we design to result in the most harmless plastic mechanisms in terms of collapse prevention.
For special moment resisting frames (SMFs), it is expected that most of the inelastic deformation will take place as rotation in beam plastic hinges, with limited deformation in the panel zone of the column. We generally avoid formation of plastic hinges on columns except for column bases, since this will result in the formation of soft story mechanism (collapse). This is achieved via the manipulation of design strength capacities of the structural elements.
In Eccentrically Braced Frames (EBFs) most of the inelastic action is restricted to the links, and the links are intended to serve as structural fuses yielding under rare seismic events while the other frame components remain essentially elastic by means of capacity based design.
Non-ductile versus Ductile Buildings
(Low-Dissipative versus Dissipative Buildings)

Devrim Ozhendekci
How do we determine the design earthquake load?

- We have to start with describing the earthquake response spectrum.
- A response spectrum is constructed via making a set of maximum dynamic responses of single degree of freedom systems (SDOFs). Every SDOF system is represented with its elastic period during calculations for a given earthquake record and damping ratio.
How do we determine the design earthquake load? - PSA

$m\ddot{x}(t) + c\dot{x}(t) + kx(t) = -m\ddot{x}_g(t)$

$\ddot{x}(t) + 2\xi\omega\dot{x}(t) + \omega^2x(t) = -\ddot{x}_g(t)$

$A(t) = \ddot{x}(t) + \ddot{x}_g(t) = -2\dot{x}(t) - \omega^2x(t)$

$SA = |A(t)|_{max} = |2\dot{x}(t) + \omega^2x(t)|_{max}$

$D(t) = x(t)$

$SD = |D(t)|_{max} = |X(t)|_{max}$

If we assume that the term including damping coefficient in the equation giving SA equals zero (but it is not) for the time step giving maximum displacement, pseudo spectral acceleration will be (actually it equals zero for undamped systems):

$PSA = |\omega^2x(t)| = \omega^2|x(t)|_{max} = \omega^2SD$

$PSA = \omega^2SD$
How do we determine the design earthquake load? - PSA

- Based on the above discussion, it is obvious that we consider the maximum displacement response instead of maximum acceleration response. We derive the acceleration in a consequent step from maximum displacement value via neglecting damping. That’s why we call the resultant value pseudo spectral acceleration (PSA). It should be underlined that we do not neglect damping during the response displacement calculations. Neglecting damping is appropriate since the characteristic values of material strength are determined based on static loading conditions, whereas loading rate would most probably increase these values.
How do we determine the design earthquake load?—Response Spectrum

Loma Prieta — 1989
Station: Bran
Magnitude: 6.93

As fore-mentioned, PSA is used to construct this elastic response spectrum.
How do we determine the design earthquake load? – Design Response Spectrum

- Is one ground motion’s response spectrum, capable of representing the seismic hazard level of the relevant location? **Of course not...** One record can be an example of a rare extreme event or may be a frequent event. Thus, we use **design response spectrum** which is a smoothed and normalized approximation for **many different ground motions**.
How do we determine the design earthquake load? – Design Response Spectrum

\[ S_a = \frac{S_{DL}}{T} \]

\[ S_a = \frac{S_{DA} \cdot T_i}{T^2} \]

Figure 11.4-1 Design response spectrum.

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11.4.5 Design Response Spectrum. Where a design response spectrum is required by this standard and site-specific ground motion procedures are not used, the design response spectrum curve shall be developed as indicated in Fig. 11.4-1 and as follows:

1. For periods less than $T_0$, the design spectral response acceleration, $S_a$, shall be taken as given by Eq. 11.4-5:

$$S_a = S_{DS} \left(0.4 + 0.6 \frac{T}{T_0}\right) \quad (11.4-5)$$

2. For periods greater than or equal to $T_0$ and less than or equal to $T_S$, the design spectral response acceleration, $S_a$, shall be taken equal to $S_{DS}$.

3. For periods greater than $T_S$, and less than or equal to $T_L$, the design spectral response acceleration, $S_a$, shall be taken as given by Eq. 11.4-6:

$$S_a = \frac{S_{D1}}{T} \quad (11.4-6)$$

4. For periods greater than $T_L$, $S_a$ shall be taken as given by Eq. 11.4-7:

$$S_a = \frac{S_{D1}T_L}{T^2} \quad (11.4-7)$$

where

- $S_{DS}$ = the design spectral response acceleration parameter at short periods
- $S_{D1}$ = the design spectral response acceleration parameter at 1-s period
- $T$ = the fundamental period of the structure, s
- $T_0 = 0.2 \frac{S_{D1}}{S_{DS}}$
- $T_S = \frac{S_{D1}}{S_{DS}}$ and
- $T_L = $ long-period transition period (s)

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11.4.1 Mapped Acceleration Parameters. The parameters \( S_S \) and \( S_I \) shall be determined from the 0.2 and 1.0 s spectral response accelerations shown on Figs. 22-1 through 22-14, respectively. Where \( S_I \) is less than or equal to 0.04 and \( S_S \) is less than or equal to 0.15, the structure is permitted to be assigned to Seismic Design Category A and is only required to comply with Section 11.7.

11.4.2 Site Class. Based on the site soil properties, the site shall be classified as Site Class A, B, C, D, E, or F in accordance with Chapter 20. Where the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the authority having jurisdiction or geotechnical data determines Site Class E or F soils are present at the site.

11.4.3 Site Coefficients and Adjusted Maximum Considered Earthquake (MCE) Spectral Response Acceleration Parameters. The MCE spectral response acceleration for short periods (\( S_{MS} \)) and at 1 s (\( S_{M1} \)), adjusted for Site Class effects, shall be determined by Eqs. 11.4-1 and 11.4-2, respectively.

\[
S_{MS} = F_a S_c \quad (11.4-1)
\]

\[
S_{M1} = F_v S_I \quad (11.4-2)
\]

where \( S_S \) = the mapped MCE spectral response acceleration at short periods as determined in accordance with Section 11.4.1, and

\( S_I \) = the mapped MCE spectral response acceleration at a period of 1 s as determined in accordance with Section 11.4.1

where site coefficients \( F_a \) and \( F_v \) are defined in Tables 11.4-1 and 11.4-2, respectively. Where the simplified design procedure of Section 12.14 is used, the value of \( F_a \) shall be determined in accordance with Section 12.14.8.1, and the values for \( F_v, S_{MS}, \) and \( S_{M1} \) need not be determined.

11.4.4 Design Spectral Acceleration Parameters. Design earthquake spectral response acceleration parameter at short period, \( S_{DS} \), and at 1 s period, \( S_{D1} \), shall be determined from Eqs. 11.4-3 and 11.4-4, respectively. Where the alternate simplified design procedure of Section 12.14 is used, the value of \( S_{DS} \) shall be determined in accordance with Section 12.14.8.1, and the value for \( S_{D1} \) need not be determined.

\[
S_{DS} = \frac{2}{3} S_{MS} \quad (11.4-3)
\]

\[
S_{D1} = \frac{2}{3} S_{M1} \quad (11.4-4)
\]

These chapters are from ASCE 7-05, in the current code (ASCE 7-10) all of the above equations are the same but the main difference is \( MCE_r \) will substitute MCE. Since Risk Targeted Maximum Considered Earthquake is taken into consideration in ASCE 7-10.

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Local Soil Condition Effects

- It has long been recognized that the effects of local soil conditions on ground motion characteristics should be considered in building design.

- The 1989 Loma Prieta earthquake provided abundant strong motion data that was used extensively together with other information in introducing the site coefficients $F_a$ and $F_v$ into the 1994 provisions.

- Following Figure presents average response spectra of ground motions recorded on soft clay and rock sites in San Francisco and Oakland during the Loma Prieta earthquake. The peak acceleration was about 0.08 to 0.1 g at the rock sites and was amplified 2 to 3 times to 0.2 g or 0.3 g at the soft soil sites. For longer periods (0.5 to 1.5 s), the amplifications were greater about 3 to 6 times.
Figure C3.3.2-1. Average spectra recorded during the 1989 Loma Prieta earthquake in San Francisco Bay area at rock sites and soft soil sites (modified after Housner, 1990).

Devrim Ozhendekci
# Local Soil Condition Effects

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at Short Period</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$S_S &lt; 0.25$</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.2</td>
</tr>
<tr>
<td>D</td>
<td>1.6</td>
</tr>
<tr>
<td>E</td>
<td>2.5</td>
</tr>
<tr>
<td>F</td>
<td></td>
</tr>
</tbody>
</table>

NOTE: Use straight-line interpolation for intermediate values of $S_S$.  

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at 1-s Period</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$S_I &lt; 0.1$</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.7</td>
</tr>
<tr>
<td>D</td>
<td>2.4</td>
</tr>
<tr>
<td>E</td>
<td>3.5</td>
</tr>
<tr>
<td>F</td>
<td></td>
</tr>
</tbody>
</table>

NOTE: Use straight-line interpolation for intermediate values of $S_I$.  

Devrim Ozhendekci
Actually, design response spectrum is based on three characteristic regions that proposed by Newmark and Hall.
12.8.1 Seismic BaseShear. The seismic base shear, $V$, in a given direction shall be determined in accordance with the following equation:

$$ V = C_s W $$  \hspace{1cm} (12.8-1) 

where

$C_s =$ the seismic response coefficient determined in accordance with Section 12.8.1.1

$W =$ the effective seismic weight per Section 12.7.2.

12.8.1.1 Calculation of Seismic Response Coefficient. The seismic response coefficient, $C_s$, shall be determined in accordance with Eq. 12.8-2.

$$ C_s = \frac{S_{D_S}}{\left(\frac{R}{I}\right)} $$ \hspace{1cm} (12.8-2) 

where

$S_{D_S} =$ the design spectral response acceleration parameter in the short period range as determined from Section 11.4.4

$R =$ the response modification factor in Table 12.2-1

$I =$ the occupancy importance factor determined in accordance with Section 11.5.1

The value of $C_s$ computed in accordance with Eq. 12.8-2 need not exceed the following:

$$ C_s = \frac{S_{D_1}}{T \left(\frac{R}{I}\right)} $$  \hspace{1cm} (12.8-3) 

$$ C_s = \frac{S_{D_1}T_L}{T^2 \left(\frac{R}{I}\right)} $$  \hspace{1cm} (12.8-4) 

$C_s$ shall not be less than

$$ C_s = 0.01 $$  \hspace{1cm} (12.8-5) 

In addition, for structures located where $S_1$ is equal to or greater than 0.6g, $C_s$ shall not be less than

$$ C_s = \frac{0.5S_1}{\left(\frac{R}{I}\right)} $$  \hspace{1cm} (12.8-6) 

Devrim Ozhendekci
12.8.1 Seismic Base Shear

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

$$ V = C_s W $$  \hspace{1cm} (12.8-1)

where

$C_s =$ the seismic response coefficient determined in accordance with Section 12.8.1.1

$W =$ the effective seismic weight per Section 12.7.2

12.8.1.1 Calculation of Seismic Response Coefficient

The seismic response coefficient, $C_s$, shall be determined in accordance with Eq. 12.8-2.

$$ C_s = \frac{S_{DS}}{R} \left( \frac{R}{I_e} \right) $$  \hspace{1cm} (12.8-2)

where

$S_{DS} =$ the design spectral response acceleration parameter in the short period range as determined from Section 11.4.4 or 11.4.7

$R =$ the response modification factor in Table 12.2-1

$I_e =$ the importance factor determined in accordance with Section 11.5.1

The value of $C_s$ computed in accordance with Eq. 12.8-2 need not exceed the following:

$$ C_s = \frac{S_{Di}}{T \left( \frac{R}{I_e} \right)} $$  \hspace{1cm} for $T \leq T_L$  \hspace{1cm} (12.8-3)

$$ C_s = \frac{S_{Di}T_L}{T^2 \left( \frac{R}{I_e} \right)} $$  \hspace{1cm} for $T > T_L$  \hspace{1cm} (12.8-4)

$C_s$ shall not be less than

$$ C_s = 0.044 S_{DS} I_e \geq 0.01 $$  \hspace{1cm} (12.8-5)

In addition, for structures located where $S_1$ is equal to or greater than 0.6g, $C_s$ shall not be less than

$$ C_s = 0.5 S_1 / (R/I_e) $$  \hspace{1cm} (12.8-6)

where $I_e$ and $R$ are as defined in Section 12.8.1.1 and $S_{Di} =$ the design spectral response acceleration parameter at a period of 1.0 s, as determined from Section 11.4.4 or 11.4.7

$T =$ the fundamental period of the structure(s) determined in Section 12.8.2

$T_L =$ long-period transition period(s) determined in Section 11.4.5

$S_1 =$ the mapped maximum considered earthquake spectral response acceleration parameter determined in accordance with Section 11.4.1 or 11.4.7
Actually, the US seismic design codes intend to provide a uniform margin against collapse at the design ground motion. In order to achieve this objective, ground motion hazards are defined in terms of Risk-targeted maximum considered earthquake (MCE_R) ground motions, which are based on a set of rules that depend on the seismic hazard of a region. MCE_R is represented by S_s and S_1 values, which are provided in seismic maps of USA.
How do we determine the design earthquake load? — Hazard Level

- The fore-mentioned set of rules are sophisticated and beyond the scope of this course. But briefly, there are two main approaches to determine $MCE_R$ ground motions:
  - Deterministic Approach
  - Probabilistic Approach

Both of these approaches are adopted in Provisions.
Seismic hazards around the USA were defined at a uniform 10 percent probability of exceedance in 50 years. (Basic philosophy is the same with the current Turkish code.) While this approach provided for a uniform likelihood throughout the country that the design ground motion would not be exceeded, it did not provide for a uniform probability of failure for structures designed for that ground motion. The reason for this is that the rate of change of earthquake ground motion versus likelihood is not constant in different regions of the US.
The approach adopted in ASCE 7-05 is intended to provide for a uniform margin against collapse at the design ground motion. In order to accomplish this, ground motion hazards were defined in terms of maximum considered earthquake (MCE) ground motions. The maximum considered earthquake ground motions are based on a set of rules that depend on the seismicity of an individual region. The design ground motions are based on a lower bound estimate of the margin against collapse inherent in structures designed to the Provisions. This lower bound was judged, based on experience, the design earthquake ground motion was selected at a ground shaking level that is $1/1.5 \times 2/3$ of the MCE.
For most regions of USA, the MCE ground motion is defined with a **uniform probability of exceedance of 2% in 50 years** (return period of about 2500 years).

However, in regions of high seismicity, such as California, ground shaking calculated at a 2% probability of exceedance in 50 years would be much larger than that which would be expected based on the characteristic magnitudes of earthquakes on these known active faults. That is why the median estimate of ground motion resulting for the characteristics event is multiplied by 1.5 for such regions instead of a probabilistic hazard analysis for the calculation of the maximum earthquake ground motion.
Hazard definition for design is changed from motion with a 2% probability of exceedance in 50 years to the **1% chance of collapse in 50 years**

11.4.3 Site Coefficients, Risk Coefficients, and Risk-targeted Maximum Considered Earthquake (MCE<sub>R</sub>) Spectral Response Acceleration Parameters. The spectral response acceleration for short periods (\(S_d\)), adjusted for the target risk of collapse, shall be determined as the lesser value of Equations 11.4-1 and 11.4-2:

\[
S_s = C_{RS} S_{SUH} \quad (11.4-1)
\]

\[
S_s = S_{SD} \quad (11.4-2)
\]

and the spectral response acceleration at a period of 1 second (\(S_1\)), adjusted for the target risk of collapse, shall be determined as the lesser value of Equations 11.4-3 and 11.4-4:

\[
S_1 = C_{R1} S_{UH} \quad (11.4-3)
\]

\[
S_1 = S_{1D} \quad (11.4-4)
\]

where

- \(S_{SD}\) = mapped deterministic, 5 percent damped, spectral response acceleration parameter at short periods as defined in Section 11.4.1
- \(S_{SUH}\) = mapped uniform-hazard, 5 percent damped, spectral response acceleration parameter at short periods as defined in Section 11.4.1
- \(C_{RS}\) = mapped value of the risk coefficient at short periods as defined in Section 11.4.1
- \(S_{1D}\) = mapped deterministic, 5 percent damped, spectral response acceleration parameter at a period of 1 second as defined in Section 11.4.1
$S_{UH} = \text{mapped uniform-hazard, 5 percent damped, spectral response acceleration parameter at a period of 1 second as defined in Section 11.4.1}$

$C_{R1} = \text{mapped value of the risk coefficient at a period of 1 second as defined in Section 11.4.1}$

The MCE_R spectral response acceleration for short periods ($S_{MS}$) and at 1 second ($S_{M1}$), adjusted for Site Class effects and the target risk of collapse, shall be determined by Equations 11.4-5 and 11.4-6, respectively.

$$S_{MS} = F_a S_s \quad (11.4-5)$$

$$S_{M1} = F_v S_1 \quad (11.4-6)$$
1- Uniform hazard ground motions are adjusted for Site Class B
The uniform hazard (2% in 50 years probability of exceedance) mapped spectral response acceleration values for short periods and for 1 second period are multiplied with corresponding mapped risk coefficients. The risk coefficients adjust these uniform-hazard spectral accelerations to achieve building designs with 1% probability of collapse in 50 years.

2- The minimum of probabilistic and deterministic ground motions is taken for Site Class B
Above steps are already applied and the spectral acceleration maps are modified in the current code.

The designer should adjust for the relevant site class using Fa and Fv coefficients
Seismic Hazard – ASCE 7-10

Step 1 – Adjust uniform-hazard ground motions (Site Class B) for target risk of collapse

\[ \frac{S_{U}^{\text{100Y}}}{C_{R}} = \text{Map of probabilistic ground motions} \]

Step 2 – Take minimum of probabilistic and deterministic ground motions (Site Class B)

\[ \frac{C_{R} \times S_{U}^{\text{100Y}}}{C_{D} \times S_{D}} = \text{Map of Site Class B ground motions} \]

Step 3 – Adjust Site Class B ground motions for site conditions (e.g., Site Class D)

\[ S_{D} \times F_{C} = \text{Map of Site Class D ground motions} \]

Figure C11.4-1: Illustration of process for developing 1-second MCEg Site Class D ground motions using formulas of Section 11.4.3 and associated mapped values of ground motions and risk coefficients of Chapter 22.

Figure C11.4-2: Map illustrating values of the MCEg 1-second spectral response acceleration parameter, \( S_{D} \) (\( S_{g} \)), and associated regions of Seismic Design Category, assuming Site Class D conditions.
Seismic Hazard – ASCE 7-10

FIGURE 22-1 5% Risk-Adjusted Maximum Considered Earthquake (MCEa) Ground Motion Parameter for the Continuous United States for 0.2 s Spectral Response Acceleration (5% of Critical Damping), Site Class B.

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Seismic Response Modification Factor

Figure C12.1-1 Inelastic force-deformation curve.
Seismic Response Modification Factor

- **R** factor represents the ratio of the forces that would develop under the specified ground motion if the structure had entirely linear-elastic response (Ve) to the prescribed design forces (Vs).

- **R=Ve/Vs**, is always greater than 1, which indicates that the buildings are designed to behave inelastically under the effect of design ground motion.

- The structure must be designed so that level of significant yield exceeds the prescribed design force.

- Furthermore, it should be designed to be capable of developing large inelastic displacements without global stability loss which is crucial in terms of collapse prevention. This can be achieved by the means of a number of fore-mentioned considerations.
Seismic Response Modification Factor

- The components of R factor: $R = \Omega \times R_d$

- $\Omega$: Overstrength factor

- Some of the sources of overstrength:
  - Actual material strength is generally higher than the material design strength,
  - The conservatism of the design procedures
  - The effects of loading rate (strength, damping)
  - The contribution of nonstructural elements
R_d : Ductility reduction factor

This reduction is possible for a number of reasons. As the structure begins to yield and deform inelastically, the effective period of response of the structure tends to lengthen, which for many structures, results in a reduction in strength demand. Furthermore, the inelastic action results in a significant amount of energy dissipation, also known as hysteretic damping, in addition to the viscous damping.

This factor is generally derived with an analogy of idealized pushover load-displacement curves of MDOF systems to the load-displacement curves of SDOF systems. There are a number of SDOF system models (elasto-plastic, strength degrading, stiffness degrading, etc.)
Seismic Response Modification Factor

- Though redundancy factor was assumed to be one of the multipliers of R factor in previous research. Redundancy is taken into consideration via the use of $\rho$ factor in the Provisions.

- Structural engineering textbooks generally define redundancy as the number of equations that are required for solution, in addition to the equilibrium equations. But this definition is quite inadequate in view of the complicated nonlinear structural behaviors under random earthquakes excitations and the effects of uncertainty in demand and capacity. It has positive effects on the global performance.
Let's think about *how we use R factor*, from both design and performance assessment aspects.
12.8.2 Period Determination

The fundamental period of the structure, $T$, in the direction under consideration shall be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The fundamental period, $T$, shall not exceed the product of the coefficient for upper limit on calculated period ($C_u$) from Table 12.8-1 and the approximate fundamental period, $T_a$, determined in accordance with Section 12.8.2.1. As an alternative to performing an analysis to determine the fundamental period, $T$, it is permitted to use the approximate building period, $T_a$, calculated in accordance with Section 12.8.2.1, directly.

12.8.2.1 Approximate Fundamental Period

The approximate fundamental period ($T_a$), in s, shall be determined from the following equation:

$$T_a = C_i h_n^x$$  \hspace{0.5cm} (12.8-7)

where $h_n$ is the structural height as defined in Section 11.2 and the coefficients $C_i$ and $x$ are determined from Table 12.8-2.

Alternatively, it is permitted to determine the approximate fundamental period ($T_a$), in s, from the following equation for structures not exceeding 12 stories above the base as defined in Section 11.2 where the seismic force-resisting system consists entirely of concrete or steel moment resisting frames and the average story height is at least 10 ft (3 m):

$$T_a = 0.1N$$  \hspace{0.5cm} (12.8-8)

where $N$ = number of stories above the base.
Period Determination

Table 12.8-2 Values of Approximate Period Parameters $C_i$ and $x$

<table>
<thead>
<tr>
<th>Structure Type</th>
<th>$C_i$</th>
<th>$x$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment-resisting frame systems in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel moment-resisting frames</td>
<td>0.028 (0.0724)</td>
<td>0.8</td>
</tr>
<tr>
<td>Concrete moment-resisting frames</td>
<td>0.016 (0.0466)</td>
<td>0.9</td>
</tr>
<tr>
<td>Steel eccentrically braced frames in accordance with Table 12.2-1 lines B1 or D1</td>
<td>0.03 (0.0731)</td>
<td>0.75</td>
</tr>
<tr>
<td>Steel buckling-restrained braced frames</td>
<td>0.03 (0.0731)</td>
<td>0.75</td>
</tr>
<tr>
<td>All other structural systems</td>
<td>0.02 (0.0488)</td>
<td>0.75</td>
</tr>
</tbody>
</table>

*Metric equivalents are shown in parentheses.*

Approximate period values and upper limits are derived from the regression analyses of compiled real data. Upper limit is essential since generally the effects of nonstructural components are not taken into consideration during the calculations of fundamental period values, next to the level of proximity of calculated values to the real values.

Devrim Ozhendekci
Period Determination

- Why do we need approximate period formulae?
- Could you provide your comments please?
12.8.3 Vertical Distribution of Seismic Forces

The lateral seismic force \( F_x \) (kip or kN) induced at any level shall be determined from the following equations:

\[
F_x = C_{vx} V \tag{12.8-11}
\]

and

\[
C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{12.8-12}
\]

where

\[ C_{vx} \text{ = vertical distribution factor} \]

\[ V \text{ = total design lateral force or shear at the base of the structure (kip or kN)} \]

\[ w_i \text{ and } w_x \text{ = the portion of the total effective seismic weight of the structure } (W) \text{ located or assigned to Level } i \text{ or } x \]

\[ h_i \text{ and } h_x \text{ = the height (ft or m) from the base to Level } i \text{ or } x \]

\[ k = \text{ an exponent related to the structure period as follows:} \]

\[ \text{for structures having a period of } 0.5 \text{ s or less, } k = 1 \]

\[ \text{for structures having a period of } 2.5 \text{ s or more, } k = 2 \]

\[ \text{for structures having a period between } 0.5 \text{ and } 2.5 \text{ s, } k \text{ shall be } 2 \text{ or shall be determined by linear interpolation between 1 and 2} \]

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12.4.2 Seismic Load Effect

The seismic load effect, $E$, shall be determined in accordance with the following:

1. For use in load combination 5 in Section 2.3.2 or load combinations 5 and 6 in Section 2.4.1, $E$ shall be determined in accordance with Eq. 12.4-1 as follows:

$$E = E_h + E_v \quad (12.4-1)$$

2. For use in load combination 7 in Section 2.3.2 or load combination 8 in Section 2.4.1, $E$ shall be determined in accordance with Eq. 12.4-2 as follows:

$$E = E_h - E_v \quad (12.4-2)$$

where

$E$ = seismic load effect

$E_h$ = effect of horizontal seismic forces as defined in Section 12.4.2.1

$E_v$ = effect of vertical seismic forces as defined in Section 12.4.2.2

12.4.2.1 Horizontal Seismic Load Effect

The horizontal seismic load effect, $E_h$, shall be determined in accordance with Eq. 12.4-3 as follows:

$$E_h = \rho Q_E \quad (12.4-3)$$

$Q_E$ = effects of horizontal seismic forces from $V$ or $F_p$.

Where required by Section 12.5.3 or 12.5.4, such effects shall result from application of horizontal forces simultaneously in two directions at right angles to each other.

$\rho$ = redundancy factor, as defined in Section 12.3.4

12.4.2.2 Vertical Seismic Load Effect

The vertical seismic load effect, $E_v$, shall be determined in accordance with Eq. 12.4-4 as follows:

$$E_v = 0.2S_{DS}D \quad (12.4-4)$$

where

$S_{DS}$ = design spectral response acceleration parameter at short periods obtained from Section 11.4.4

$D$ = effect of dead load

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12.4.2.3 Seismic Load Combinations

Where the prescribed seismic load effect, \( E \), defined in Section 12.4.2 is combined with the effects of other loads as set forth in Chapter 2, the following seismic load combinations for structures not subject to flood or atmospheric ice loads shall be used in lieu of the seismic load combinations in either Section 2.3.2 or 2.4.1:

Basic Combinations for Strength Design (see Sections 2.3.2 and 2.2 for notation).

5. \( (1.2 + 0.2S_{DS})D + \rho Q_E + L + 0.2S \)
6. \( (0.9 - 0.2S_{DS})D + \rho Q_E + 1.6H \)

12.8.4.2 Accidental Torsion

Where diaphragms are not flexible, the design shall include the inherent torsional moment \( (M_t) \) resulting from the location of the structure masses plus the accidental torsional moments \( (M_{at}) \) caused by assumed displacement of the center of mass each way from its actual location by a distance equal to 5 percent of the dimension of the structure perpendicular to the direction of the applied forces.

Where earthquake forces are applied concurrently in two orthogonal directions, the required 5 percent displacement of the center of mass need not be applied in both of the orthogonal directions at the same time, but shall be applied in the direction that produces the greater effect.
12.8.6 Story Drift Determination

The design story drift (Δ) shall be computed as the difference of the deflections at the centers of mass at the top and bottom of the story under consideration. See Fig. 12.8-2. Where centers of mass do not align vertically, it is permitted to compute the deflection at the bottom of the story based on the vertical projection of the center of mass at the top of the story. Where allowable stress design is used, Δ shall be computed using the strength level seismic forces specified in Section 12.8 without reduction for allowable stress design.

For structures assigned to Seismic Design Category C, D, E, or F having horizontal irregularity Type 1a or 1b of Table 12.3-1, the design story drift, Δ, shall be computed as the largest difference of the deflections of vertically aligned points at the top and bottom of the story under consideration along any of the edges of the structure.

The deflection at Level x (δx) (in. or mm) used to compute the design story drift, Δ, shall be determined in accordance with the following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I_e} \quad (12.8-15)$$

where

- $C_d$ = the deflection amplification factor in Table 12.2-1
- $\delta_{xe}$ = the deflection at the location required by this section determined by an elastic analysis
- $I_e$ = the importance factor determined in accordance with Section 11.5.1

12.8.6.1 Minimum Base Shear for Computing Drift

The elastic analysis of the seismic force-resisting system for computing drift shall be made using the prescribed seismic design forces of Section 12.8.

**EXCEPTION:** Eq. 12.8-5 need not be considered for computing drift.

12.8.6.2 Period for Computing Drift

For determining compliance with the story drift limits of Section 12.12.1, it is permitted to determine the elastic drifts, (δxe), using seismic design forces based on the computed fundamental period of the structure without the upper limit ($C_d T_e$) specified in Section 12.8.2.

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12.8.7 P-Delta Effects

P-delta effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects are not required to be considered where the stability coefficient (θ) as determined by the following equation is equal to or less than 0.10:

\[ \theta = \frac{P_x \Delta I_e}{V_x h_{xx} C_d} \quad (12.8-16) \]

where

- \( P_x \) = the total vertical design load at and above Level \( x \) (kip or kN); where computing \( P_x \), no individual load factor need exceed 1.0
- \( \Delta \) = the design story drift as defined in Section 12.8.6 occurring simultaneously with \( V_x \) (in. or mm)
- \( I_e \) = the importance factor determined in accordance with Section 11.5.1

\( V_x \) = the seismic shear force acting between Levels \( x \) and \( x - 1 \) (kip or kN)
\( h_{xx} \) = the story height below Level \( x \) (in. or mm)
\( C_d \) = the deflection amplification factor in Table 12.2-1

The stability coefficient (θ) shall not exceed \( \theta_{\max} \) determined as follows:

\[ \theta_{\max} = \frac{0.5}{\beta C_d} \leq 0.25 \quad (12.8-17) \]

where \( \beta \) is the ratio of shear demand to shear capacity for the story between Levels \( x \) and \( x - 1 \). This ratio is permitted to be conservatively taken as 1.0.

Where the stability coefficient (θ) is greater than 0.10 but less than or equal to \( \theta_{\max} \), the incremental factor related to P-delta effects on displacements and member forces shall be determined by rational analysis. Alternatively, it is permitted to multiply displacements and member forces by 1.0/(1 - θ).

Where ϑ is greater than \( \theta_{\max} \), the structure is potentially unstable and shall be redesigned.

Where the P-delta effect is included in an automated analysis, Eq. 12.8-17 shall still be satisfied, however, the value of θ computed from Eq. 12.8-16 using the results of the P-delta analysis is permitted to be divided by (1 + θ) before checking Eq. 12.8-17.