# 3.5 Tier 1 Analysis

## 3.5.1 Overview

Analyses performed as part of Tier 1 of the Evaluation Process are limited to Quick Checks. Quick Checks shall be used to calculate the stiffness and strength of certain building components to determine whether the building complies with certain evaluation criterion. Quick Checks shall be performed in accordance with Section 3.5.3 when triggered by evaluation statements from the Checklists of Section 3.7. Seismic shear forces for use in the Quick Checks shall be computed in accordance with Section 3.5.2.

## 3.5.2 Seismic Shear Forces

### 3.5.2.1 Pseudo Lateral Force

The pseudo lateral force, in a given horizontal direction of a building, shall be calculated in accordance with Equations (3-1) and (3-2).

$$V=CS_aW \tag{3-1}$$

where:

- V = Pseudo lateral force;
- C = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response; C shall be taken from Table 3-4;
- $S_a$ = Response spectral acceleration at the fundamental period of the building in the direction under consideration. The value of $S_a$ shall be calculated in accordance with the procedures in Section 3.5.2.3.
- W = Total dead load and anticipated live load as follows:
  - In storage and warehouse occupancies, a minimum of 25% of the floor live load; The actual partition weight or minimum weight of 10 psf of floor area, whichever is greater;
  - The applicable snow load;
  - The total weight of permanent equipment and furnishings.

Alternatively, for buildings with shallow foundations and without basements being evaluated for the Life Safety Performance Level, Equation (3-2) may be used to compute the pseudo lateral force:

$$V = 0.75W \tag{3-2}$$

If Equation (3-2) is used, an *m*-factor of 1.0 shall be used to compute the component forces and stresses for the Quick Checks of Section 3.5.3 and acceptance criteria of Section 4.2.4.

Table 3-4. Modification Factor, C

| Building Type[1] | Number of Stories | | | |
|---|---|---|---|---|
| | 1 | 2 | 3 | ≥ 4 |
| Wood (W1, W1A, W2)<br>Moment Frame (S1, S3, C1, PC2A) | 1.3 | 1.1 | 1.0 | 1.0 |
| Shear Wall (S4, S5, C2, C3, PC1A, PC2, RM2, URMA)<br>Braced Frame (S2) | 1.4 | 1.2 | 1.1 | 1.0 |
| Unreinforced Masonry (URM)<br>Flexible Diaphragms (S1A, S2A, S5A, C2A, C3A, PC1, RM1) | 1.0 | 1.0 | 1.0 | 1.0 |

[1]Defined in Table 2-2.

**Commentary:**

The seismic evaluation procedure of this Handbook, as well as the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings* and the *Uniform Building Code*, is based on a widely-accepted philosophy that permits nonlinear response of a building when subjected to a ground motion that is representative of the design earthquake. The *NEHRP Recommended Provisions for Seismic Regulations for New Buildings*, the *Uniform Building Code* and FEMA 178 account for nonlinear seismic response in a linear static analysis procedure by including a response modification factor, R, in calculating a reduced equivalent base shear to produce a rough approximation of the internal forces during a design earthquake. In other words, the base shear is equivalent to what the bulding is expected to resist strength-wise, but the building displacement using this base shear are significantly less than the displacements the building will actually experience during a design earthquake. Thus, this approach increases the base shear by another factor ($C_d$, .7R, etc.) when checking drift and ductility requirements. In summary, this procedure is based on equivalent lateral forces and pseudo displacements.

The linear static analysis procedure in this Handbook, as well as in FEMA 273, takes a different approach to account for the nonlinear seismic response. Pseudo static lateral forces are applied to the structure to obtain "actual" displacements during a design earthquake. The pseudo lateral force of Equation (3-1) represents the force required, in a linear static analysis, to impose the expected actual deformation of the structure in its yielded state when subjected to the design earthquake motions.

It does not represent an actual lateral force that the building must resist in traditional design codes or FEMA 178. In summary, this procedure is based on equivalent displacements and pseudo lateral forces. For additional commentary regarding this linear static analysis approach, please refer to the commentary for Section 4.2.2.1 and FEMA 273 and 274.

Instead of applying a ductility related response reduction factor, *R*, to the applied loads, this Handbook uses ductility related *m*-factors in the acceptability checks of each component. Thus, instead of using a single *R*-value for the entire structure, different *m*-factors are used depending on the ductility of the component being evaluated. The *m*-factors specified for each Tier of analysis shall not be used for other Tiers of analysis (i.e., Tier 3 values of *m* may not be used when a Tier 1 or Tier 2 analysis is performed).

For short and stiff buildings with low ductility located in regions of high seismicity, the required building strength in accordance with Equation (3-1) may exceed the force required to cause sliding at the foundation level. The strength of the structure, however, does not need to exceed the strength of the ground. Thus, when Equation (3-2) is applied to these buildings, the required strength of structural components need not exceed 0.75W.

#### 3.5.2.2 Story Shear Forces

For multi-story buildings, the pseudo lateral force computed in accordance with Section 3.5.2.1 shall be distributed vertically in accordance with Equation (3-3).

$$V_j = \left(\frac{n+j}{n+1}\right)\left(\frac{W_j}{W}\right)V \qquad (3\text{-}3)$$

where:

- $V_j$ = Story shear at story level j,
- n = Total number of stories above ground level,
- j = Number of story level under consideration,
- $W_j$ = Total seismic weight of all stories above level j,
- W = Total seismic weight per Section 3.5.2.1,
- V = Pseudo lateral force from Equation (3-1) or (3-2).

For buildings with flexible diaphragms (Types S1A, S2A, S5A, C2A, C3A, PC1, RM1, URM), story shear shall be calculated separately for each line of lateral resistance. This value shall be calculated using Equation (3-3) with $W_j$ defined as the seismic weight of all stories above level j tributary to the line of resistance under consideration.

#### 3.5.2.3 Spectral Acceleration

Spectral acceleration for use in computing the pseudo lateral force shall be computed in accordance with this section. Spectral acceleration shall be based on mapped spectral accelerations, defined in Section 3.5.2.3.1, for the site of the building being evaluated. Alternatively, a site specific response spectrum may be developed according to Section 3.5.2.3.2.

##### 3.5.2.3.1 Mapped Spectral Acceleration

The mapped spectral acceleration, $S_a$, shall be computed in accordance with Equation (3-4).

$$S_a = \frac{S_{D1}}{T}, \text{ but} \qquad (3\text{-}4)$$

$S_a$ shall not exceed $S_{DS}$ ;

where:

$$S_{D1} = \frac{2}{3}F_v S_1 \qquad (3\text{-}5)$$

$$S_{DS} = \frac{2}{3}F_a S_s \qquad (3\text{-}6)$$

T = Fundamental period of vibration of the building calculated in accordance with Section 3.5.2.4.

$S_s$ and $S_1$ are short period response acceleration and spectral response acceleration at a one second period parameters, respectively, for the Maximum Considered Earthquake (MCE). $S_s$ and $S_1$ shall be obtained from the *Seismic Map Package*. $F_v$ and $F_a$ are site coefficients and shall be determined from Tables 3-5 and 3-6, respectively, based on the site class and the values of the response acceleration parameters $S_s$ and $S_1$. The site class of the building shall be defined as one of the following:

- **Class A**: Hard rock with measured shear wave velocity, $\bar{v}_s$ > 5,000 ft/sec;
- **Class B**: Rock with 2,500 ft/sec ≤ $\bar{v}_s$ ≤ 5,000 ft/sec.
- **Class C**: Very dense soil and soft rock with 1,200 ft/sec < $\bar{v}_s$ ≤ 2,500 ft/sec or with either standard blow count $\bar{N}$ > 50 or undrained shear strength $\bar{s}_u$ > 2,000 psf.
- **Class D**: Stiff soil with 600 ft/sec < $\bar{v}_s$ ≤ 1,200 ft/sec or with 15 < $\bar{N}$ ≤ 50 or 1,000 psf ≤ $\bar{s}_u$ < 2000 psf.
- **Class E**: Any profile with more than 10 feet of soft clay defined as soil with plasticity index PI >20, or water content w > 40 percent, and $\bar{s}_u$ < 500 psf or a soil profile with $\bar{v}_s$ < 600 ft/sec.
- **Class F**: Soils requiring a site-specific geotechnical investigation and dynamic site response analyses:
  - Soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly-sensitive clays, collapsible weakly-cemented soils;
  - Peats and/or highly organic clays (H>10 feet of peat and/or highly organic clay; where H = thickness of soil);
  - Very high plasticity clays (H > 25 feet with PI > 75 percent);
  - Very thick soft/medium stiff clays (H > 120 feet).

For a soil profile classified as Class F, a Class E soil profile may be assumed for a Tier 1 Evaluation. If sufficient data is not available to classify a soil profile, a Class E profile shall be assumed. For one- and two-story buildings with a roof height equal to or less than 25 feet, a Class D soil profile may be assumed if site conditions are not known.

Table 3-5. Values of $F_v$ as a Function of Site Class and Mapped Spectral Acceleration at a One Second Period, $S_1$

| Site Class | Mapped Spectral Acceleration at One Second Period[1] | | | | |
|---|---|---|---|---|---|
| | $S_1 \le 0.1$ | $S_1 = 0.2$ | $S_1 = 0.3$ | $S_1 = 0.4$ | $S_1 \ge 0.5$ |
| A | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 |
| B | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| C | 1.7 | 1.6 | 1.5 | 1.4 | 1.3 |
| D | 2.4 | 2.0 | 1.8 | 1.6 | 1.5 |
| E | 3.5 | 3.4 | 2.8 | 2.4 | 2.2 |
| F | * | * | * | * | * |

[1]Note: Use straight-line interpolation for intermediate values of $S_1$.

* See Class F soil profile.

Table 3-6. Values of $F_a$ as a Function of Site Class and Mapped Short-Period Spectral Acceleration, $S_s$

| Site Class | Mapped Spectral Acceleration at Short Periods[1] | | | | |
|---|---|---|---|---|---|
| | $S_s \le 0.25$ | $S_s = 0.50$ | $S_s = 0.75$ | $S_s = 1.00$ | $S_s \ge 1.25$ |
| A | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 |
| B | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| C | 1.2 | 1.2 | 1.1 | 1.0 | 1.0 |
| D | 1.6 | 1.4 | 1.2 | 1.1 | 1.0 |
| E | 2.5 | 1.7 | 1.2 | 0.9 | 0.9 |
| F | * | * | * | * | * |

[1]NOTE: Use straight-line interpolation for intermediate values of $S_s$.

*See Class F soil profile.

**Commentary:**

The short period response acceleration and spectral response acceleration at a one second period parameters, $S_s$ and $S_1$, are provided in the *Seismic Map Package*. The values of $S_s$ and $S_1$ represent an earthquake with a 2% probability of exceedance in 50 years with deterministic-based maximum values near known fault sources. For information on obtaining a copy of the *Seismic Map Package*, please contact the FEMA Distribution Facility at 1-800-480-2520.

#### 3.5.2.3.2 Site-Specific Spectral Acceleration

Development of site-specific response spectra shall be based on the geologic, seismological, and soil characteristics associated with the specific site of the building being evaluated. Site-specific response spectra shall be based on input ground motions with a 2% probability of exceedance in 50 years (2500 year return interval) and developed for an equivalent viscous damping ratio of 5%. The site specific response spectra need not exceed the mean deterministic spectra for faults with known slip rates. When the 5% damped site specific spectrum has spectral amplitudes in the period range of greatest significance to the structural response that are less than 70% of the mapped spectral amplitudes, an independent third-party review of the spectrum shall be made by an individual with expertise in the evaluation of ground motion.

#### 3.5.2.4 Period

The fundamental period of a building, in the direction under consideration, shall be calculated in accordance with Equation (3-7).

$$T = C_t h_n^{3/4} \tag{3-7}$$

where:

T = Fundamental period (in seconds) in the direction under consideration;

$C_t$ = 0.060 for wood buildings (Building Types W1, W1A, and W2);
= 0.035 for moment-resisting frame systems of steel (Building Types S1 and S1A);
= 0.030 for moment-resisting frames of reinforced concrete (Building Type C1);
= 0.030 for eccentrically-braced steel frames (Building Types S2 and S2A);
= 0.020 for all other framing systems;

$h_n$ = height (in feet) above the base to the roof level.

Alternatively, for steel or reinforced-concrete moment frames of 12 stories or less the fundamental period of the building may be calculated as follows:

$$T = 0.10N \tag{3-8}$$

where:

N = number of stories above the base.

### 3.5.3 Quick Checks for Strength and Stiffness

Quick Checks shall be used to compute the stiffness and strength of building components. Quick Checks are triggered by evaluation statements in the Checklists of Section 3.7 and are required to determine the compliance of certain building components. The seismic shear forces used in the Quick Checks shall be calculated in accordance with Section 3.5.2.

**Commentary:**

The quick check equations used here are essentially the same as those used in FEMA 178, modified for use with the pseudo lateral forces and the appropriate material *m*-factors.

#### 3.5.3.1 Story Drift for Moment Frames

Equation (3-9) shall be used to calculate the drift ratios of regular, multistory, multibay moment frames with columns continuous above and below the story under consideration. The drift ratio is based on the deflection due to flexural displacement of a representative column, including the effect of end rotation due to bending of the representative girder.

$$DR = \left(\frac{k_b + k_c}{k_b \cdot k_c}\right)\left(\frac{h}{12E}\right) V_c \tag{3-9}$$

where:

DR = Drift Ratio = Interstory displacement divided by story height,

$k_b$ = I/L for the representative beam,

$k_c$ = I/h for the representative column,

h = Story height (in.),

I = Moment of inertia ($in^4$),

$L$ = Center to center length of columns (in.),
$E$ = Modulus of elasticity (ksi),
$V_c$ = Shear in the column (kips).

The column shear forces shall be taken as a portion of the story shear forces, computed in accordance with Section 3.5.2.2. For reinforced concrete frames, an equivalent cracked section moment of inertia equal to one half of gross value shall be used.

Equation (3-9) may also be used for the first floor of the frame if columns are fixed against rotation at the bottom. However, if columns are pinned at the bottom, an equivalent story height equal to twice the actual story height shall be used in calculating the value of $k_c$.

For other configurations of frames, the quick check need not be performed as a Full-Building Tier 2 Evaluation including calculation of the drift ratio shall be completed based on principles of structural mechanics.

### 3.5.3.2 Shear Stress in Concrete Frame Columns

The average shear stress, $v_{avg}$, in the columns of concrete frames shall be computed in accordance with Equation (3-10).

$$v_{avg} = \frac{1}{m}\left(\frac{n_c}{n_c - n_f}\right)\left(\frac{V_j}{A_c}\right) \qquad (3\text{-}10)$$

where:
$n_c$ = Total number of columns;
$n_f$ = Total number of frames in the direction of loading;
$A_c$ = Summation of the cross sectional area of all columns in the story under consideration; and
$V_j$ = Story shear computed in accordance with Section 3.5.2.2.
$m$ = component modification factor; m shall be taken equal to 2.0 for buildings being evaluated to the Life Safety Performance Level and 1.3 for buildings being evaluated to the Immediate Occupancy Performance Level.

**Commentary:**

Equation (3-10) assumes that all of the columns in the frame have similar stiffness.

### 3.5.3.3 Shear Stress in Shear Walls

The average shear stress in shear walls, $v_{avg}$, shall be calculated in accordance with Equation (3-11).

$$v_{avg} = \frac{1}{m}\left(\frac{V_j}{A_w}\right) \qquad (3\text{-}11)$$

where:
$V_j$ = Story shear at level j computed in accordance with Section 3.5.2.2;
$A_w$ = Summation of the horizontal cross sectional area of all shear walls in the direction of loading. Openings shall be taken into consideration when computing $A_w$. For masonry walls, the net area shall be used. For wood framed walls, the length shall be used rather than the area.
$m$ = component modification factor; m shall be taken from Table 3-7.

Table 3-7. m-factors for Shear Walls

| Wall Type | Level of Performance[1] | |
|---|---|---|
| | LS | IO |
| Reinforced Concrete, Precast Concrete, and Wood | 4.0 | 2.0 |
| Reinforced Masonry | 3.0 | 1.5 |
| Unreinforced Masonry | 1.5 | N/A |

[1]Defined in Section 2.4.

### 3.5.3.4 Diagonal Bracing

The average axial stress in diagonal bracing elements, $f_{br}$, shall be calculated in accordance with Equation (3-12).

$$f_{br} = \frac{1}{m}\left(\frac{V_j}{sN_{br}}\right)\left(\frac{L_{br}}{A_{br}}\right) \qquad (3\text{-}12)$$

where:
$L_{br}$ = Average length of the braces (ft);
$N_{br}$ = Number of braces in tension and compression if the braces are designed for compression; if not, use the number of braces in tension, if the braces are not designed for compression;
$s$ = Average span length of braced spans (ft);
$A_{br}$ = Average area of a diagonal brace (in$^2$);

$V_j$ = Maximum story shear at each level (kips);
m = component modification factor; m shall be taken from Table 3-8.

Table 3-8. m-factors for Diagonal Braces

| Brace Type | (d/t) * | Level of Performance[1] LS | IO |
|---|---|---|---|
| Tube | $\leq 90/(F_{ye})^{1/2}$ | 6.0 | 2.5 |
| | $> 190/(F_{ye})^{1/2}$ | 3.0 | 1.5 |
| Pipe | $\leq 1500/F_{ye}$ | 6.0 | 2.5 |
| | $> 6000/F_{ye}$ | 3.0 | 1.5 |
| Tension-only | | 3.0 | 1.5 |
| All others | | 6.0 | 2.5 |

[1]Defined in Section 2.4.
*Interpolation permitted.
$F_{ye} = 1.25F_y$; expected yield stress as defined by Section 4.2.4.4.

### 3.5.3.5 Precast Connections

The precast connection in precast concrete moment frames shall be able to develop the moment in the girder, $M_g$, calculated in accordance with Equation (3-13).

$$M_g = \frac{V_j}{m}\left(\frac{n_c}{n_c - n_f}\right)\left(\frac{h}{2}\right) \qquad (3\text{-}13)$$

where:

$n_c$ = Total number of columns;
$n_f$ = Total number of frames in the direction of loading;
$V_j$ = Story shear at the level directly below the connection under consideration;
h = Typical column story height;
m = Component modification factor taken equal to 2.0 for buildings being evaluated to the Life Safety Performance Level and 1.3 for buildings being evaluated to the Immediate Occupancy Performance Level.

### 3.5.3.6 Axial Stress Due to Overturning

The axial stress of columns subjected to overturning forces, $p_{ot}$, shall be calculated in accordance with Equation (3-14).

$$p_{ot} = \frac{1}{m}\left(\frac{2}{3}\right)\left(\frac{Vh_n}{Ln_f}\right) \qquad (3\text{-}14)$$

where:

$n_f$ = Total number of frames in the direction of loading;
V = Pseudo lateral force;
$h_n$ = height (in feet) above the base to the roof level.
L = Total length of the frame (in feet);
m = Component modification factor taken equal to 2.0 for buildings being evaluated to the Life Safety Performance Level and 1.3 for buildings being evaluated to the Immediate Occupancy Performance Level.

## 3.6 Region Of Low Seismicity Checklist

This Region of Low Seismicity Checklist shall be completed when required by Table 3-2.

Each of the evaluation statements on this checklist shall be marked compliant (C), non-compliant (NC), or not applicable (N/A) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this Handbook, while non-compliant statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 evaluation procedure; the section numbers in parentheses following each evaluation statement correspond to Tier 2 evaluation procedures.

### Structural Components

**C NC N/A** LOAD PATH: The structure shall contain one complete load path for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation. (Tier 2: Sec. 4.3.1.1)

**C NC N/A** WALL ANCHORAGE: Exterior concrete or masonry walls shall be anchored for out-of-plane forces at each diaphragm level with steel anchors or straps that are developed into the diaphragm. (Tier 2: Sec. 4.6.1.1)

### Geologic Site and Foundation Components

**C NC N/A** FOUNDATION PERFORMANCE: There shall be no evidence of excessive foundation movement such as settlement or heave that would affect the integrity or strength of the structure. (Tier 2: Sec. 4.7.2.1)

### Nonstructural Components

**C NC N/A** EMERGENCY LIGHTING: Emergency lighting equipment shall be anchored to prevent falling or swaying during an earthquake. (Tier 2: Sec. 4.8.3.2)

**C NC N/A** CLADDING ANCHORS: Cladding components weighing more than 10 psf shall be anchored to the exterior wall framing at a spacing equal to or less than 6 ft. (Tier 2: Sec. 4.8.4.1)

**C NC N/A** GLAZING: Glazing in curtain walls and individual panes over 16 square feet in area, located up to a height of 10 feet above an exterior walking surface, shall be laminated annealed or heat strengthened safety glass that will remain in the frame when cracked. (Tier 2: Sec. 4.8.4.9)

**C NC N/A** PARAPETS: There shall be no laterally unsupported unreinforced masonry parapets or cornices above the highest anchorage level with height-to-thickness ratios greater than 2.5. (Tier 2: Sec. 4.8.8.1)

**C NC N/A** CANOPIES: Canopies located at building exits shall be anchored at a spacing of 10 ft. (Tier 2: Sec. 4.8.8.2)

**C NC N/A** STAIRS: Walls around stair enclosures shall not consist of unbraced hollow clay tile or unreinforced masonry. (Tier 2: Sec. 4.8.10.1)

**C NC N/A** EMERGENCY POWER: Equipment used as part of an emergency power system shall be anchored. (Tier 2: Sec. 4.8.12.1)