10.5 ARCHITECTURAL FEATURES AND COMPONENTS

10.5.1 UNREINFORCED MASONRY (URM) WALLS

This section provides guidance in evaluating unreinforced non-bearing masonry (URM) walls for seismic adequacy. It should be noted that the approaches presented herein address only the out-of-plane behavior of non-bearing unreinforced masonry walls with respect to seismic loads. It is important to have a list of masonry walls selected before the Seismic Review Team (SRT) begins its seismic evaluation. The Seismic Capability Engineers (SCEs) that make up the SRT are not necessarily the ones expected to assemble the list of selected masonry walls for evaluation. That is a separate task to be performed by others (see Chapter 4).

The selected masonry wall is first examined by non-destructive evaluation (NDE) methods to determine if it is hollow or grouted solid. If the wall is found to be hollow in every cell (or only hollow in the cells that contain rebar), then it is considered to be unreinforced. If the wall is grouted solid in a specified minimum number of vertical cells, then it is further investigated by NDE methods to determine if it is either reinforced or unreinforced. If the wall is found to contain enough rebar to be categorized as reinforced, it is considered to be "out-of-scope" of the evaluation guidance provided in this module. If a URM wall is determined to be a load-bearing wall, it is also considered "out-of-scope" for this module. The URM walls included in the guidance herein are assumed to be either: (1) walls that in-fill a concrete or steel frame, or (2) partitions inside a concrete or steel-framed building.

One screening approach and three methods of URM wall evaluation for out-of-plane bending are presented in this module and are the following: (1) Screening based on height/thickness ratio, (2) The Elastic Method (also called the ACI working stress approach), (3) The Reserve Energy Method, and (4) The Arching Action Method. The Elastic Method is generally the most conservative and yields a relatively low capacity for the wall in question. The Arching Action Method provides the highest capacity for the wall. Both the Reserve Energy Method and the Arching Action Method are considered to be post-elastic approaches and account for additional wall strength after wall cracking. The methods are shown in Figure 10.5.1-1.

10.5.1.1 List of Selected Masonry Walls

This task should be performed by others before the Seismic Capability Engineers (SCEs) begin their URM wall evaluation. A list of selected masonry walls must be generated so that the SCEs can begin their evaluation of walls. The Seismic Equipment List (SEL) is discussed in Chapter 4. If masonry walls are included on the SEL, use that list.

Questions that should be addressed during the selection of masonry walls might include:

- Is seismic interaction credible?
- Is critical equipment in the vicinity of or attached to the masonry wall?
- Is the masonry wall in question used for:
    - confinement of hazardous material?
    - shielding?
    - fire protection?
    - security concerns?

A more detailed list of questions to be addressed can be found in Reference 117, Pages 18-21.

10.5.1.2 Type of Unreinforced Masonry Wall

The three main types of masonry walls considered are:

- Concrete Masonry Unit (CMU)
- Hollow-Clay Tile (HCT)
- Brick

It will also make a difference whether each cell of the wall is grouted solid or left hollow. The hollow cell of masonry block will attract a smaller seismic loading, since it has less mass than the cell of masonry block which is fully grouted. If construction documents or installation records are not available, one must perform a non-destructive evaluation to determine the condition of the selected masonry wall. For determination of hollow cell vs. grouted cell, drilling a small hole through the face of the cell is one simple method. To ascertain whether only a few cells are grouted, check several consecutive blocks along a course of the selected wall. In some parts of the United States, insulation is placed in ungrouted cells of masonry walls. The weight of this insulation should be included when conducting the evaluations presented in this section.

It is also important to find out if the masonry wall is reinforced. The scope of the guidance in this section only includes unreinforced masonry walls. For detection of rebar, a hand-held ferromagnetic detector with a display meter or an audio signal can be easily used in many cases. An alternate method involves using imaging impulse radar. With either method, it is important to locate the positions of the following:

- vertical reinforcing steel and its approximate spacing
- horizontal reinforcing steel and its approximate spacing

An unreinforced masonry wall is a masonry wall in which the area of reinforcing steel is less than 25 percent of the minimum steel ratios required by the 1994 Uniform Building Code (UBC) for reinforced masonry (Ref. 69). Lightly or poorly reinforced walls are considered to be URM walls and can be evaluated by the methods presented in this Section.

10.5.1.3 Determine Physical Condition of Wall

As part of the seismic evaluation of the selected URM wall, it is important to examine the condition of mortar joints, openings, and existing cracks. If the mortar joints are not sound or if there are substantial cracks in the mortar or faces of the masonry units, the Elastic Method (ACI Working Stress Approach) in Section 10.5.1.5 may not be applicable.

The top connection is often not fully grouted and thus may be a free joint. Simple supports at the top and/or side should result from structural-steel angle "keepers" or dovetail slots in columns or overhead beams. There needs to be some positive means of carrying the out-of-plane load from the wall panel and into the support if it is to be considered a simple support boundary condition. If not, the wall may have to be evaluated as a cantilever.

10.5.1.4 Screening Based on Height-to-Thickness Ratio

A conservative screening approach based on the Elastic Method may be used to screen out walls from further evaluation. The top of the wall must be laterally supported to use this approach, there should be a tight fit between the supporting member, or suitable restraining members should be provided to prevent lateral motion of the top of the wall.

The wall may be screened out if:

$$\left(\frac{H}{t}\right)_{actual} \leq \left(\frac{H}{t}\right)_{max}$$

where:

$$\left(\frac{H}{t}\right)_{max} = \left(\frac{H}{t}\right)_{N} \frac{\alpha_D}{\sqrt{\frac{S_{A_{max}}}{g}}}$$

$\left(\frac{H}{t}\right)_{N}$ can be found in Table 10.5.1-1 as a function of actual wall thickness t

H = wall height

t = actual wall thickness

α_D = $\sqrt{150/\rho}$ or from Table 10.5.1-6

ρ = weight density of masonry in #/ft^3

$S_{A_{max}}$ = maximum spectral acceleration from 5% damped input spectra for appropriate Performance Category and location above grade in facility (see Section 5.2). Values in Table 10.5.1-2 may only be used for Performance Category 1 masonry walls at grade.

g = acceleration of gravity

Development of this screening approach is discussed in Section 10.5.1.8.

For walls that are not screened out by this process, continue with the analysis methods presented in Sections 10.5.1.5, 10.5.1.6, and 10.5.1.7.

10.5.1.5 Elastic Method

Estimate Maximum Flexural Tensile Stress in URM Wall

For the elastic method, this module makes extensive use of Reference 117. The following topics are considered in arriving at an estimate of the maximum flexural tensile stress in the URM wall:

- natural frequency prediction for a single-wythe, uncracked masonry wall,
- determine horizontal seismic acceleration,
- estimate maximum out-of-plane bending stress for a single-wythe, uncracked, masonry wall of height H and width L

Multiple-wythe masonry walls with sufficient header courses to insure composite action can also be evaluated by this procedure. Header courses are used to tie single-wythe masonry walls together.

Determine boundary conditions of the selected URM wall

To properly use the seismic guidance in this document, it is important to determine boundary conditions of the selected URM walls. Table 10.5.1-3 lists many combinations of boundary conditions, some of which include: 1) simply supported on all four edges; 2) simply supported on top and bottom, free on sides; and 3) simply supported on bottom and sides, free on top.

Cross walls will provide support to the wall sides. Using doorways as free edges may be appropriate. However, using a window as a free edge may be overly-conservative if the window is less than half of the height of the URM wall in question.

Estimate the fundamental natural frequency of the wall

Once the boundary conditions are verified, the fundamental natural frequency can be estimated as follows:

$$f = (B_f)(F)(\alpha_E)(\alpha_D)(\alpha_T)$$

- f has units of cycles per second (Hz)
- boundary condition factor, B_f for fundamental frequency calculation from Table 10.5.1-3
- frequency factor, F from Table 10.5.1-4
- elastic modulus factor, α_E from Table 10.5.1-5
- weight density factor, α_D from Table 10.5.1-6
- orthotropic behavior adjustment factor, α_T from Table 10.5.1-7
- special considerations (for cases of partial grouting, partially filled joints, and multi-wythe walls), see Table 10.5.1-8.

Estimate the spectral acceleration of the wall

If the wall is at the ground level, the site-specific 5% damped ground response spectrum can be entered with the URM wall frequency to determine the spectral acceleration for the selected wall (see Section 5.2). If the wall is at a higher elevation in the building or if it has a basement, the appropriate floor spectrum should be used when determining the spectral acceleration of the selected wall.

Estimate the maximum flexural stress in the URM wall.

With the maximum flexural tensile stress tables, the estimated maximum flexural tensile stress for the selected wall can be scaled according to the wall spectral acceleration.

$$\sigma_b = (B_s)(S)(A_H)(1/\alpha_D)^2$$

- σ_b has units of pounds per square inch
- boundary condition factor, B_s from Table 10.5.1-9

- stress factor, S from Table 10.5.1-10
- horizontal seismic acceleration, A_H (in g's)
- weight density factor, α_D from Table 10.5.1-6.

Capacity by Elastic Method

Compare the allowable stress, due to out-of-plane seismic loads, at mortar/masonry unit interface with the estimated maximum flexural tensile stress above.

When evaluating URM walls using the Elastic Method, the following should be considered:

1. ACI 530 Table 6.3.1.1 (Ref. 118) has conservative values of allowable flexural tensile stress. Only URM walls that are located in geographic regions with low values of seismic acceleration will meet these ACI 530 code values of allowable stress.

2. The location of maximum stress depends on the specific masonry wall boundary conditions. For example, the maximum moment and stresses in many cases will occur at the fixed boundary in the form of a negative moment. In-filled walls with simple supports at the edges will most likely have the maximum out-of-plane bending stress located near the center of the wall (approximately mid-height and mid-span).

3. Values that may be used for allowable flexural stress for good quality masonry, as stated in Ref. 117, are the following:

   – 33 psi for hollow masonry
   – 52 psi for solid or fully grouted masonry

4. If site-specific test data exist, a safety factor of 2 to 3 against measured flexural tensile stress at fracture should be applied to the test results and the safety factor chosen should be consistent with the scatter of the site-specific data (Ref. 117).

Example problems illustrating application of this method are shown in Section 10.5.1.10.

10.5.1.6 Reserve Energy Method

The formulas for screening non bearing unreinforced masonry walls are developed from the arching action method with the initial confining force at the top of the wall taken as zero, (Reference 119 and 120).

For the two rigid block rocking (see Figure 10.5.1-2), the spectral acceleration capacity, S_{AP}, is

$$\frac{S_{AP}}{g} = 6\,\phi\,\frac{b}{H}\left(1 - \frac{\delta_H}{2b}\right)$$

For the cantilever wall (see Figure 10.5.1-3), the spectral acceleration capacity is

$$\frac{S_{AP}}{g} = 2\,\phi\,\frac{b}{H}\left(1 - \frac{\delta_H}{2b}\right)$$

where:

g	=	acceleration of gravity
ϕ	=	capacity reduction factor (may be taken as 0.67)
t	=	actual wall thickness
b	=	effective wall thickness = 0.9t
H	=	wall height
δ_H	=	any specified out-of-plane displacement (δ_H should be limited to no more than b for wall stability)

The Spectral Acceleration Demand, S_{AD}, can be determined by the average of the 5% damped, peak-broadened floor spectra for the floors above and below the wall at the effective frequency, f_e (see Section 5.2).

$$f_e = \frac{1}{2\pi}\sqrt{\frac{1.5\left(\frac{S_{AP}}{g}\right)g}{\delta_H}} \quad \text{(Hz)}$$

If $\frac{S_{AP}}{g} \geq \frac{S_{AD}}{g}$, then the wall is acceptable.

If the capacity is less than the demand for all values of δ_H from 0 to b, the wall becomes an outlier. Wall displacement is the lowest δ_H at which $S_{AP} = S_{AD}$.

The capacity trend using the Reserve Energy Method is shown in Figure 10.5.1-4. It can be seen that the ultimate capacity S_{AP} occurs at low lateral displacement. However, the demand S_{AD} is also likely to reduce at even a faster rate with increasing δ_H (see example problems) so that the largest ratio of (S_{AP} / S_{AD}) is most likely to occur when δ_H equals the stability limit b = 0.9t.

When evaluating URM walls using the Reserve Energy Method, the following should be considered:

1. Neglect cracking strength of the unreinforced masonry wall.
2. Assume an idealized rigid-body motion of the wall.
3. Assume that the URM wall is a non-load bearing wall. Load bearing walls can also be assessed by a more complex version of the Reserve Energy Method.
4. Failure of a URM wall is identified when the response exceeds the effective wall thickness b.

Example problems illustrating application of this method are shown in Section 10.5.1.10.

10.5.1.7 Arching Action Method

Check for applicability of Arching Action. When this method can be justified, it provides the highest out-of-plane seismic capacity.

It is critical that the boundary conditions of the URM walls do not include any significant gaps (> 1/16 inch) between the top of the selected URM wall and the beam or floor above for the Arching Action Method to apply. If gaps occur, then there may be limited, or reduced, ability for the wall to develop arching action. To take credit for arching action, it is also important to check the maximum allowable compressive stress in the masonry unit and compare it to the maximum stresses developed at the edges of critical masonry units (Ref. 119).

When the rotational restraints at the boundaries are considered, a higher capacity can be achieved for the URM wall. The rotational restraint due to the wall's horizontal displacement induces an arching mechanism (Ref. 119). This arching mechanism is illustrated in Figure 10.5.1-2.

Assuming rigid body rocking develops after the masonry wall has cracked at a location αH above the base, as shown in Figure 10.5.1-2, the Reserve Energy method can be used to calculate the ultimate out-of-plane spectral acceleration capacity of a nonload bearing wall including arching action as:

$$\frac{S_{AP}}{g} = \phi\left(\frac{b}{H}\right)\left[2\,f_p\left(\frac{P_{R\delta}}{wH}\right)\left(1 - \frac{\delta_H}{b}\right) + 6\left(1 - \frac{\delta_H}{2b}\right)\right]$$

where:

g	=	acceleration of gravity
ϕ	=	capacity reduction factor (may be taken as 0.67)
t	=	actual wall thickness
b	=	effective wall thickness = 0.9t
H	=	wall height
f_p	=	$1.03 + 3.0\left(\frac{e}{b} + 0.5\right)^{0.65}$
e	=	eccentricity of P_R (see Figure 10.5.1-2)
w	=	weight/unit area of masonry wall
δ_H	=	any specified out-of-plane displacement. To take credit for arching action, δ_H should not exceed δ_p

δ_p = out-of-plane displacement at which ultimate capacity is reached $= \dfrac{0.00045H^2}{f_D t}$

except $\dfrac{\delta_p}{b} \leq \dfrac{2\,F_e}{(3 - F_e)}$

f_D = 1.0 for concrete block and single wythe hollow clay tile walls
1.5 for double wythe hollow clay tile walls

F_e = $\dfrac{e}{b} + 0.5$

$P_{R\delta}$ = confining force at displacement δ_H
(increases with displacement until the displacement δ_p is reached at which the ultimate capacity occurs)

$P_{R\delta}$ = $P_c\ f_R$

P_c = crushing capacity of block = $0.125\ t\ f'_m$

f'_m = ultimate compressive strength of masonry
[analogus to ultimate compressive strength of concrete, f'_c,
typically 1000 - 1500 psi for concrete block (1350 psi typical) ,
possibly as low as 275 psi for hollow clay tile]

f_R = relative boundary element flexibility factor (See Section 10.5.1.9 for approach used to compute f_R)

f_R should not exceed $\left(1 - \dfrac{wH}{P_c}\right)$.

The first term of the arching action capacity equation, shown above, defines the arching effect and generally dominates. For walls with large H/t and small boundary stiffness (low f_R) the second term can become very significant.

Instability will occur when δ_H reaches 0.9t. If δ_H substantially exceeds δ_p, the wall should be assumed to have lost its in-plane capacity.

The increase in capacity over the Reserve Energy Method is shown in Figure 10.5.1-5.

The effective frequency f_e is:

$$f_e = \frac{1}{2\pi}\sqrt{\frac{1.5\left(\frac{S_{AP}}{g}\right)g}{\delta_H}}$$

The spectral acceleration demand, S_{AD} can be determined from the average of the 5% damped, peak-broadened floor spectra (see Section 5.2) for the floors above and below the wall at the effective frequency f_e.

In order to determine δ_H for a given input response spectrum, start with a low δ_H and compute S_{AP}, f_e, and S_{AD}. Keep increasing δ_H until the spectral acceleration demand S_{AD} at f_e drops below the spectral acceleration capacity S_{AP} corresponding to δ_H. The lowest δ_H at which $S_{AP} \leq S_{AD}$ represents the appropriate δ_H for the given input response spectrum.

When δ_H reaches δ_p, the masonry is assumed to crush sufficiently that arching benefit is lost. For larger δ_H up to 0.9t, the capacity may be conservatively estimated by the Reserve Energy Approach discussed in the previous subsection.

The ground motion level at which the wall is acceptable can be generally established by the larger of:

1. Elastic Method Capacity

2. Reserve Energy Method Capacity with $\delta_H = b = 0.9t$

3. Arching Method Capacity with $\delta_H = \delta_p$

It is always conservative to use the larger of these three capacities. In some cases, a greater $\left(S_{AP} / S_{AD}\right)$ ratio might occur at lesser δ_H values than the values defined above. However, in most cases, this increase is not sufficiently significant to warrant considering these intermediate δ_H values unless it is desired to have an estimate of the wall displacement for a given input spectrum.

Example problems illustration application of this method are in Section 10.5.1.10.

10.5.1.8 <u>Development of Screening Approach Based on Elastic Method</u>

A conservative screening approach has been developed to rapidly screen out walls from further analysis if they meet the screening criteria. This approach is based on the Elastic Method for walls simply supported top and bottom and free on both sides. The equations and terms used are those defined in subsection 10.5.1.5.

$$\sigma_b = B_s \; S \; A_H \left(\frac{1}{\alpha_D} \right)^2$$

$A_H = S_{A_{max}}$ = Peak of the 5% damped response spectra for the site and Performance Category, (in g's).

Use the peak of the in-structure spectra if wall is not located at grade.

$B_s = 0.125$ for walls simply supported top and bottom and free on the side.

$\alpha_D = \sqrt{150/\rho}$ from Table 10.5.1-6

$\sigma_b =$ 33 psi for hollow masonry and 52 psi for solid masonry

Therefore,

For hollow masonry:

$$S = \frac{\sigma_b \, \alpha_D^{\,2}}{B_s \, A_H} = \frac{33 \, \alpha_D^{\,2}}{0.125 \, S_{A_{max}}} = \frac{264 \, \alpha_D^{\,2}}{S_{A_{max}}}$$

or for solid masonry:

$$S = \frac{52 \, \alpha_D^{\,2}}{0.125 \, S_{A_{max}}} = \frac{416 \, \alpha_D^{\,2}}{S_{A_{max}}}$$

and for solid masonry:

$$S = H^2 \, w \, \frac{c}{I'} \text{ from Table 10.5.1-10}$$

$$w = \rho t$$

$$c = \frac{t}{2}$$

$$I' = \frac{t^3}{12}$$

$$S = \frac{H^2 \, \rho \, t\left(\frac{t}{2}\right)}{\frac{t^3}{12}}$$

Therefore $S = 6 \, \rho \, t \left(\frac{H}{t}\right)^2$

For hollow masonry, actual values for w and I' must be used.

Set

$$\frac{264 \, \alpha_D^{\,2}}{S_{A_{max}}} = H^2 \, w_{hollow} \, \frac{c}{I'_{hollow}}$$

where w_{hollow} and I'_{hollow} are the actual values for hollow masonry used to develop stress factors, S, in Table 10.5.1-10 or for solid masonry

$$\frac{416\,\alpha_D{}^2}{S_{A_{max}}} = 6\,\rho\,t\left(\frac{H}{t}\right)^2$$

and determine $\left(\frac{H}{t}\right)$ from the smaller value. This becomes the developed values of $\left(\frac{H}{t}\right)_N$ presented in Table 10.5.1-1.

10.5.1.9 <u>Method of Calculating Boundary Member Flexibility Factor</u> f_R

The average value of P_R along the length of the top beam can be approximated as shown in Figure 10.5.1-7. The load on the beam reaches the local block crushing capacity P_c over length *a* at each end of the beam, and is zero over the central region of the beam.

The length *a* is from the end of the beam to point 1 of Figure 10.5.1-7 at which the upward displacement δ_1 reaches

$$\delta_1 = \delta_u - \delta_g$$

where δ_g = height of any pre-existing gap between the beam and the top of the wall.
(Recall Arching Action may provide limited additional capacity if $\delta_g > \frac{1}{16}$ in.)

Vertical displacement of a simply supported beam restrained against twisting due to arching of wall is:

$$\delta_1 = \underbrace{\frac{P_c\,L^4}{32\,E I_B}\,f_R^3\left[1-\left(\frac{7}{12}\right)f_R\right]}_{\text{Flexural Term}} + \underbrace{\frac{P_c\,e_b^2\,L^2\,f_R^2}{8\,G\,J_B}}_{\text{Torsion Term}} \qquad (*)$$

where:

P_c = crushing capacity of block

L = length of beam and wall

I_B = moment of inertia of beam

J_B = polar moment of inertia of beam

E = elastic modulus of beam

G = shear modulus of beam

f_R = beam flexibility factor

e_b = eccentricity to load from beam centerline

Vertical displacement of wall due to horizontal displacement is calculated next.

As the wall blocks rock, the point at which P_R is applied lifts and presses against the boundary beam. This wall uplift at the location of P_R is given by:

$$\delta_u = \delta_H \left(\frac{b}{H}\right) f_p$$

Uplift Factor

$$f_p = 1.03 + 3.0\left(\frac{e}{b} + 0.5\right)^{0.65}$$

where e is the load eccentricity measured from the center line of the wall (see Figure 10.5.1-8), and b = 0.9t to account for block crushing.

Set vertical displacement of wall equal to vertical displacement of beam.

$$\delta_l + \delta_g = \delta_u = \delta_H \left(\frac{b}{H}\right) f_p$$

or

$$\delta_H = \left(\frac{\delta_g + \delta_l}{f_p}\right)\frac{H}{b} \qquad (**)$$

Horizontal displacement of wall at ultimate capacity

$$\delta_p = \frac{.00045\, L^2}{t} \qquad \frac{\delta_p}{b} \le \frac{2\, F_e}{3 - F_e}$$

$$\delta_H \le \delta_p$$

The value of f_R can then be found by trial and error until the maximum permissible value of f_R is reached.

The following procedure can be used:

Pick f_R, start low $f_R = 0.1$, calculate δ_l from (*) on the previous page, calculate δ_H from (**) above and repeat until $\delta_H = \delta_p$

A tabular form is convenient

f_R	δ_l	δ_H
		stop when $\delta_H = \delta_p$

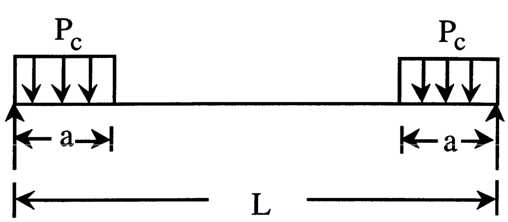
The following data will assist the calculation

$$C = f_R^3 \left[1 - \left(\frac{7}{12} \right) f_R \right]$$

then $$\delta_1 = \frac{P_c\, L^4}{32\, E I_B}\, C + \frac{P_c\, e_b^2\, L^2\, f_R^2}{8\, G\, J_B}$$

f_R	C
0.	0.
.1	0.000942
.2	0.00707
.3	0.0223
.4	0.0491
.5	0.0885
.6	0.140
.7	0.203
.8	0.273
.9	0.346
1.0	0.417

The boundary member capacity must also be checked. Moment capacity M_c can place an upper limit on f_R. Torsion capacity T_c can place an upper limit on e_b.



$$a = \frac{f_R L}{2}$$

$$M = P_c \frac{a^2}{2} = \frac{P_c\, L^2}{8}\, f_R^2 \le M_c$$

$$f_R \le \left(\frac{8\, M_c}{P_c\, L^2} \right)^{1/2}$$

$$T = P_c\, e_b\, a = \frac{P_c\, f_R\, L}{2}\, e_b \le T_c$$

$$e_b\, f_R \le \left(\frac{2\, T_c}{P_c\, L} \right)$$

10.5.1.10 Example Problems

The following example problems are presented to demonstrate application of the methods in this section to a typical URM wall.

A 6 inch hollow concrete block wall at the Portsmouth Gaseous Diffusion Plant is evaluated by the Elastic, Reserve Energy, and Arching Action Methods using ground motion described by a Portsmouth Site Specific Spectra and a Newmark and Hall Generic Spectra (Ref. 72) for a soil site.

6" Concrete Block Wall

$$f'_m = 1000 \text{ psi}$$

$$H = 12' = 144"$$

$$L = 18' = 216"$$

$$\rho = 135 \text{ lbs/ft}^3$$

Simply supported top and bottom, free on sides

Portsmouth Site with 0.15g spectrum (see Figure 10.5.1-6A)

Screening Approach (Section 10.5.1.4)

$$\left(\frac{H}{t}\right)_{actual} = \frac{144}{5.625} = 25.6$$

$$SA_{max} = 0.4g \text{ (Portsmouth)}$$

$$SA_{max} = 2.12 \text{ x } .15 = 0.32g \text{ (Newmark \& Hall)}$$

$$\alpha_D = \sqrt{\frac{150}{135}} = 1.054$$

$$\left(\frac{H}{t}\right)_{max} = \left(\frac{H}{t}\right)_N \frac{\alpha_D}{\sqrt{SA_{max}}}$$

$$\left(\frac{H}{t}\right)_N = 11.5 \text{ for a 6" wall from Table 10.5.1-1}$$

$$\left(\frac{H}{t}\right)_{max} = \frac{(11.5)\,(1.054)}{\sqrt{0.4}} = 19.17 \qquad \text{(Portsmouth ground motion)}$$

$$\left(\frac{H}{t}\right)_{max} = \frac{(11.5)\,(1.054)}{\sqrt{0.32}} = 21.43 \qquad \text{(Newmark and Hall ground motion)}$$

$$\left(\frac{H}{t}\right)_{actual} > \left(\frac{H}{t}\right)_{max}$$

Wall is not screened out.

<u>Elastic Method (Section 10.5.1.5)</u>

Estimate seismic capacity from:

$$\sigma_b = B_s \; S \; A_H \left(\frac{1}{\alpha_D}\right)^2$$

$$\sigma_b \;=\; \sigma_b \text{ allowable} = 33 \text{ psi}$$

$$\frac{H}{L} = \frac{12'}{18'} = 0.67, \;\; B_s \;=\; 0.125 \text{ from Table 10.5.1-9}$$

$$S \;=\; 1245 \text{ psi from Table 10.5.1-10}$$

$$\alpha_D = \sqrt{\frac{150}{135}} = 1.054$$

$$A_H = S_{AP} = \frac{\sigma_b \, \alpha_D^2}{B_S S} = \frac{(33)\,(1.054)^2}{(0.125)\,(1245)} = 0.24g$$

Estimate frequency from:

$$f = B_f \; F \; \alpha_E \; \alpha_D \; \alpha_T$$

$$\frac{H}{L} = 0.67, \;\; B_f = 1.571 \text{ from Table 10.5.1-3}$$

6" hollow concrete block, H = 12', F = 6.70 from Table 10.5.1-4

$$\alpha_E \;=\; 1 \text{ from Table 10.5.1-5}$$

$$\alpha_D \;=\; 1.054$$

$$\alpha_T \;=\; 0.97 \text{ from Table 10.5.1-7 for 6" wall}$$

$$f = (1.571)(6.70)(1)(1.054)(0.97) = 10.8 \text{ Hz}$$

$$T = \frac{1}{f} = 0.093 \text{sec}$$

$S_{AD} = 0.4g$ from 0.15g Portsmouth 5% damped spectra at 0.093 sec

$$\text{Capacity to Demand Ratio} = \frac{S_{AP}}{S_{AD}} = \frac{0.24}{0.40} = 0.6 < 1.0$$

Wall Fails Elastically

The maximum elastic peak ground acceleration that will not fail the wall elastically is

$$a_g = (0.60)(0.15g) = 0.09g$$

<u>Reserve Energy Method (Section 10.5.1.6)</u>

b = $0.9t = 0.9\,(6") = 5.4"$
(Note: 6" is the nominal wall thickness, the actual wall thickness should be used in the calculation).

$$\frac{S_{AP}}{g} = 6\phi\frac{b}{H}\left(1-\frac{\delta_H}{2b}\right)$$

$$\frac{S_{AP}}{g} = 6\,(0.67)\left(\frac{5.4}{144}\right)\left[1-\frac{\delta_H}{2(5.4)}\right]$$

$$\frac{S_{AP}}{g} = 0.151\left[1-\frac{\delta_H}{10.8}\right]$$

$$f_e = \frac{1}{2\pi}\sqrt{\frac{1.5\,S_{AP}\,g}{\delta_H}} = \frac{1}{2\pi}\sqrt{\frac{(1.5)\,S_{AP}\,(386.4)}{\delta_H}} = 3.83\left(\frac{S_{AP}}{\delta_H}\right)^{0.5}\ \text{Hz}$$

Find S_{AP}, f_e, and S_{AD} at various δ_H up to stability limit of 5.4".

Reserve Energy Results in tabular form:

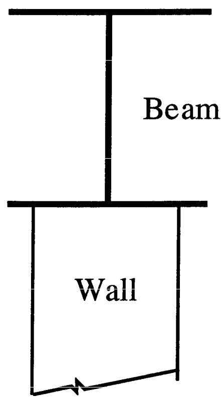
δ_H (inch)	Capacity S_{AP} (g)	Frequency f_e (Hz)	Period T (sec)	Demand S_{AD} (g)	Capacity/ Demand $\frac{S_{AP}}{S_{AD}}$	a_g (g)
0.2	.148	3.29	0.30	.215	0.69	0.10
0.4	.145	2.31	0.43	.145	1.00	0.15(1)
1.0	.137	1.42	0.70	.066	2.08	0.31
2.0	.123	.95	1.05	.036	3.42	0.51
5.4	.076	.45	2.22	.012	6.33	0.95(2)

1. Wall displaces only 0.4" for 0.15g Spectrum
2. Wall reaches stability limit at 0.95g Spectrum

Much greater capacity than for Elastic Method because spectrum drops quickly at lower frequencies.

Arching Action Method (Section 10.5.1.7) - Case 1

Case 1: Simply supported steel W8 x 28 beam centered on top of wall with no gap between beam and top of wall



Negligible torsional resistance web of beam lines up with centerline of wall, $e_o = 0$ (see Figure 10.5.1-8)

Use
$$e = 0$$
$$e_b = 0$$
$$E = 29 \times 10^6 \text{ psi}$$
$$I_B = 98 \text{ in.}^4$$
$$L = 216 \text{ in.}$$

Masonry:

$$f'_m = 1000 \text{ psi}$$

$$P_c = .125 t f'_m = .125\,(6")\,(1000 \text{ psi}) = 750\,{}^{\#}\!/_{\text{in.}}$$

$$w = \rho t = (135)\,(0.5)\,{}^{\#}\!/_{\text{ft}}\,2 = 0.469 \text{ psi}$$

$$\text{gap} = \delta_g = 0$$

Vertical displacement of beam:

$$\delta_1 = \frac{P_c L^4}{32 E I_B} f_R^3 \left\{1 - \left(\frac{7}{12}\right) f_R\right\} = \frac{750\,{}^{\#}\!/_{\text{in.}}\,(216 \text{ in.})^4}{32\left(29 \times 10^6 \text{ psi}\right)\left(98 \text{ in}^4\right)} f_R^3\,(1 - .583\, f_R)$$

$$\delta_1 = 17.95"\; f_R^3 (1 - .583\, f_R) = 17.95\, C$$

Displacement at ultimate capacity:

$$\delta_p = \frac{.00045(144")^2}{(6")} = 1.56"$$

Set $\delta_H \leq \delta_p = 1.56"$

Uplift factor:

$$f_p = 1.03 + 3.0\left(\frac{e}{b} + 0.5\right)^{0.65} = 1.03 + 3.0\,(.5)^{.65} = 2.94$$

$$\delta_u = \delta_H\left(\frac{b}{H}\right) f_p = 2.94\left(\frac{5.4}{144}\right)\delta_H = 0.110\,\delta_H$$

$$\delta_u - \delta_g = \delta_1$$

$$\delta_u = \delta_1 = 0.110\,\delta_H$$

$$\delta_H = \frac{\delta_1}{0.110}$$

Maximum permissible f_R:

$$f_R \leq \left(1 - \frac{wH}{P_c}\right) \leq 0.91$$

Check steel W8 x 28 beam A36 steel:

$$M_{CAP} = \phi\, F_y\, Z_x = (0.9)\,(36\,\text{ksi})\left(27.2\,\text{in.}^3\right) = 881 \text{ k-in} \qquad \text{(LRFD Method)}$$

$$f_R \leq \left[\frac{8\,(881)}{.750\,(216)^2}\right]^{\frac{1}{2}} = 0.45$$

thus $f_R \leq 0.45$

$T_{CAP} \approx$ 0 for wide flange held only on web at ends

$$e_b = 0$$

$$e = e_b - e_o = 0 - 0 = 0$$

Start by picking a $f_R = 0.10$, calculate δ_1 and δ_H until $\delta_H = \delta_p = 1.56$:

f_R	δ_1 (in)	δ_H (in)
0.10	.0169	.154
0.14	.0452	.411
0.20	.127	1.15
0.22	.167	1.52
0.225	.178	1.61

← max δ_H for arching (block begins to crush)
$f_R \approx 0.222$

$$P_{R\delta} = P_c\, f_R$$

$$\frac{S_{AP}}{g} = \phi\left(\frac{b}{H}\right)\left[2\, f_p\, \frac{P_{R\delta}\left(1-\frac{\delta_H}{b}\right)}{wH} + 6\left(1-\frac{\delta_H}{2b}\right)\right]$$

$$\frac{S_{AP}}{g} = 0.67\left(\frac{5.4}{144}\right)\left[2(2.94)\,\frac{750\frac{\#}{''}\, f_R\left(1-\frac{\delta_H}{5.4''}\right)}{.469\,\text{psi}\ (144'')} + 6\left(1-\frac{\delta_H}{10.8}\right)\right]$$

$$\frac{S_{AP}}{g} = \underbrace{1.64\, f_R\left(1-\frac{\delta_H}{5.4}\right)}_{\substack{\text{Arching} \\ \text{(only good up to 1.56")}}} + \underbrace{0.151\left(1-\frac{\delta_H}{10.8}\right)}_{\substack{\text{Reserve} \\ \text{Energy}}}$$

$$f_e = 3.83\left(\frac{S_{AP}}{\delta_H}\right)^{0.5}$$

Arching Action results:

δ_H (inch)	Capacity S_{AP} (g)	Frequency f_e (Hz)	Period T (sec)	Demand S_{AD} (g)	$\frac{S_{AP}}{S_{AD}}$	a_g (g)	
0.154	0.300	5.35	.187	.342	0.88	0.13	
0.200					1.00	0.15	
0.411	0.357	3.57	.280	.228	1.57	0.24	Arching Action
1.15	0.393	2.24	.446	.129	3.05	0.46	
1.56	0.388	1.91	.524	.101	3.84	0.58	
2.0	0.123	.95	1.05	.036	3.42	0.51	Reserve Energy
5.4	0.076	.45	2.22	.012	6.33	0.95	

Wall displaces only 0.2" for 0.15g Spectrum (by interpolation) (Only about 50% of Reserve Energy deflection)

Stability limit is still 0.95g Spectrum (Same as for Reserve Energy)

Not much benefit from arching because of flexibility of support beam and quick drop-off with lowering frequency for input spectrum.

Arching Action Method (Section 10.5.1.7) - Case 2

Case 2: Same wall, but supported by a large simply supported, torsionally restrained reinforced concrete beam with the following properties:

$$I_B = 6000\,\text{in}^4 \qquad E = 3\text{x}10^6\,\text{psi}$$

$$J_B = 7000\,\text{in}^4 \qquad G = 1.2\text{x}10^6\,\text{psi}$$

see Figure 10.5.1-8

$$e_o = 0 \qquad e/b = 0.5 \qquad e_b = \frac{b}{2} - e_o = 0.45t - 0 = 2.7"$$

$$f_p = 1.03 + 3.0\,(1.0)^{.65} = 4.03$$

$$\delta_u - \delta_g = \delta_1 = \delta_H \left(\frac{5.4"}{144"}\right)(4.03) = 0.151\,\delta_H$$

$$\delta_H = \frac{\delta_1}{0.151}$$

$$\delta_1 = \frac{P_c L^4}{32\, EI_B} f_R^3 \left\{1 - \frac{7}{12} f_R\right\} + \frac{P_c e_b^2 L^2 f_R^2}{8\, GJ_B}$$

$$\delta_1 = \underbrace{\frac{750\,\#/'' \,(216'')^4}{32\left(3\times10^6\text{ psi}\right)\left(6000''^4\right)} f_R^3 \left(1 - .583 f_R\right)}_{\text{Flexure}} + \underbrace{\frac{750\,\#/''\,(2.7'')^2\,(216'')^2}{8\left(1.2\times10^6\text{ psi}\right)\left(7000''^4\right)} f_R^2}_{\text{Torsion}}$$

$$\delta_1 = 2.83''\, f_R^3 \left(1 - .583 f_R\right) + (\approx 0)$$

Maximum permissible f_R:

$$f_R \le \left(1 - \frac{wH}{P_c}\right) = 1 - \frac{.469\text{ psi}\,(144'')}{750\,\#/''} = 0.91$$

Check concrete beam $\left(12'' \text{ x } 24'' \text{ Deep}, A_s \ge 2\,\text{in}^2\right)$ with some torsional steel:

$$M_{CAP} = 2000 \text{ k-in.}$$

$$f_R \le \left[\frac{8\,(2000)}{.750\,(216)^2}\right]^{\frac{1}{2}} = 0.68$$

thus $f_R \le 0.68$

$$T_{CAP} = 120 \text{ k-in.}$$

$$e_b\, f_R \le \left[\frac{2\,(120)}{.750\,(216)}\right] = 1.48$$

e_b must be reduced below 2.7" if
f_R exceeds $\frac{1.48}{2.7} = 0.55$

f_R	δ_1 (in.)	$\delta_H = \frac{\delta_1}{0.151}$ (in.)
0.20	.0200	0.132
0.25	.0378	0.250
0.30	.0630	0.417
0.40	.139	0.920
0.486		1.56
0.50	.251	1.66

$$\frac{S_{AP}}{g} = 2.25\, f_R \left(1 - \frac{\delta_H}{5.4}\right) + 0.151\left(1 - \frac{\delta_H}{10.8}\right)$$

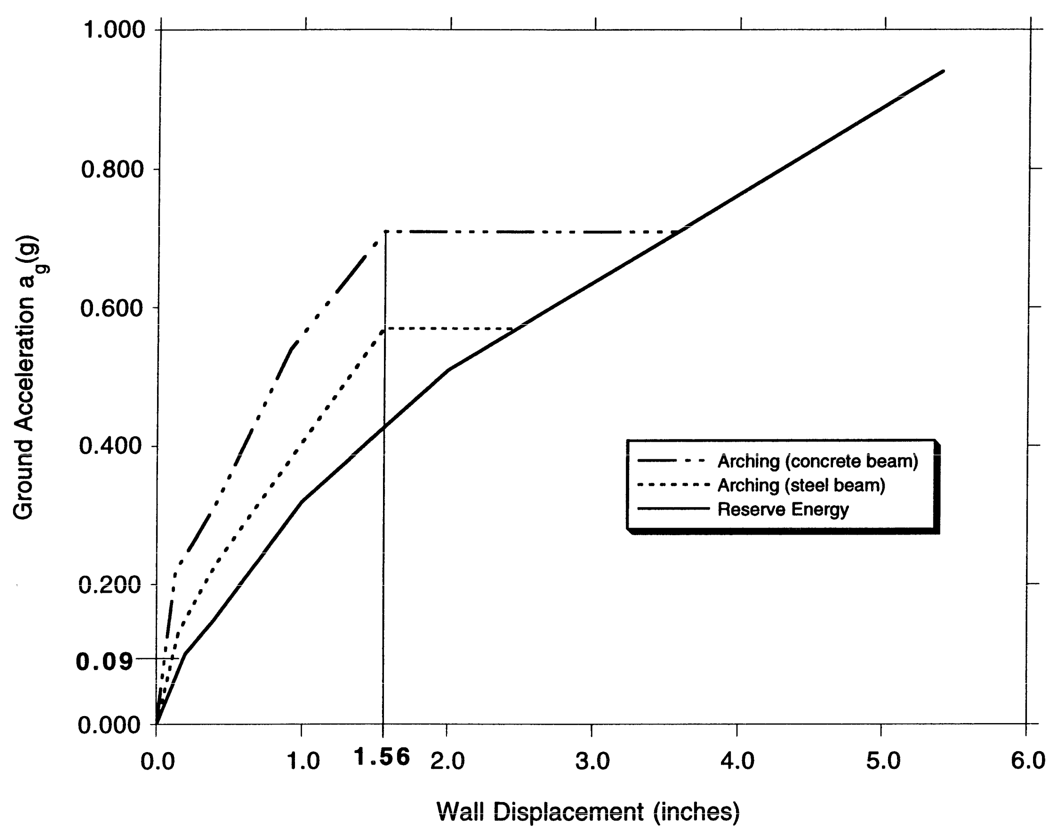
$$f_e = 3.83\sqrt{\frac{S_{AP}}{\delta_H}}$$

δ_H (inch)	S_{AP} (g)	f_e (Hz)	T (sec)	S_{AD} (g)	$\frac{S_{AP}}{S_{AD}}$	a_g (g)	
0.132	.588	8.08	0.124	0.40	1.47	0.22	Arching Action
0.250	.684	6.33	0.158	0.40	1.71	0.26	
0.417	.768	5.20	0.192	0.338	2.27	0.34	
0.920	.885	3.76	0.266	0.245	3.61	0.54	
1.56	.907	2.92	0.342	0.191	4.75	0.71	
2.0	.123	0.95	1.05	.036	3.42	0.51	Reserve Energy
5.4	.076	0.45	2.22	.012	6.33	0.95	

Wall displays only 0.13 inches for a 0.22g input
However, stability limit is still 0.95g

Arching Action did not increase stability limit because of shape of input spectrum.

<u>Comparison of results for Portsmouth input spectrum shape:</u>



Rework same example with NUREG/CR-0098 (Ref. 72) input median spectrum for a soil site to illustrate the importance of the input spectrum shape on relative results.

Spectrum properties for 5% damping are given below and shown in Figure 10.5.1-6B:

$$8\,\text{Hz} \le f \le 33\,\text{Hz}: \qquad S_{AD} = a_g \left(f/33\text{Hz}\right)^{-0.53}$$

$$1.64\ \text{Hz} \le f \le 8\,\text{Hz} \qquad S_{AD} = 2.12\ a_g$$

$$0.25\ \text{Hz} \le f \le 1.64\ \text{Hz} \qquad S_{AD} = 1.29\,\text{sec}\ f\ a_g$$

$$f \le 0.25\ \text{Hz} \qquad S_{AD} = 5.08\,\text{sec}\ f^2\ a_g$$

Elastic Method (Section 10.5.1.5)

$$S_{AP} = 0.24g$$

$$f = 10.8\,\text{Hz} \longrightarrow S_{AD} = 1.81\ a_g = 0.27g$$

$$\frac{S_{AP}}{S_{AD}} = \frac{0.24}{0.27} = 0.89 < 1.0$$

$$a_g = (0.89)\,(0.15g) = 0.13g$$

Reserve Energy Method (Section 10.5.1.6)

Using previous results:

δ_H (in.)	S_{AP} (g)	f_e (Hz)	S_{AD}/a_g	$a_g = \left(\frac{S_{AP}}{S_{AD}/a_g}\right)$ (g)	
0.20	.148	3.29	2.12	0.07	less than elastic
0.40	.145	2.31	2.12	0.07	"
1.0	.137	1.42	1.83	0.07	"
2.0	.123	.95	1.23	0.10	"
5.4	.076	.45	0.58	0.13	"

No value over Elastic Method

For NUREG/CR-0098 soil spectrum, wall becomes unstable when it exceeds 0.13g elastic capacity, no advantage to Reserve Energy Method. (Spectrum has lots of low frequency)

Arching Action Method (Section 10.5.1.7) - Case 1

Case 1 - Steel beam

Using previous results:

δ_H (in.)	S_{AP} (g)	f_e (Hz)	S_{AD} / a_g	$a_g = \left(\frac{S_{AP}}{\frac{S_{AD}}{a_g}} \right)$ (g)
.154	.300	5.35	2.12	0.14
.411	.357	3.57	2.12	0.17
1.15	.393	2.24	2.12	0.18
1.56	.388	1.91	2.12	0.18

Maximum $a_g \approx 1.4$ * Elastic capacity for NUREG/CR-0098 soil spectrum

Arching Action Method (Section 10.5.1.7) - Case 2

Case 2 - Concrete beam

Using previous results:

δ_H (in.)	S_{AP} (g)	f_e (Hz)	S_{AD} / a_g	$a_g = \frac{S_{AP}}{\frac{S_{AD}}{g}}$ (g)
.132	.588	8.08	2.11	0.28
.250	.684	6.33	2.12	0.32
.417	.768	5.20	2.12	0.36
.920	.885	3.76	2.12	0.42
1.56	.907	2.92	2.12	0.43

Maximum $a_g \approx 3.3$ *Elastic capacity for NUREG/CR-0098 soil spectrum

Summary of Section 10.5.1.10

	Factor Over Elastic a_g Capacity	
	Portsmouth Spectrum	NUREG/CR-0098 Soil Spectrum
Reserve Energy	10.6	1.0
Arching Case 1 (Steel Beam)	10.6	1.4
Arching Case 2 (Concrete Beam)	10.6	3.3

Whether Reserve Energy results in increased capacity over Elastic Method is highly sensitive to shape of input demand spectrum.

Increase in capacity from Arching Action is significantly influenced by stiffness of boundary element and shape of input demand spectrum.

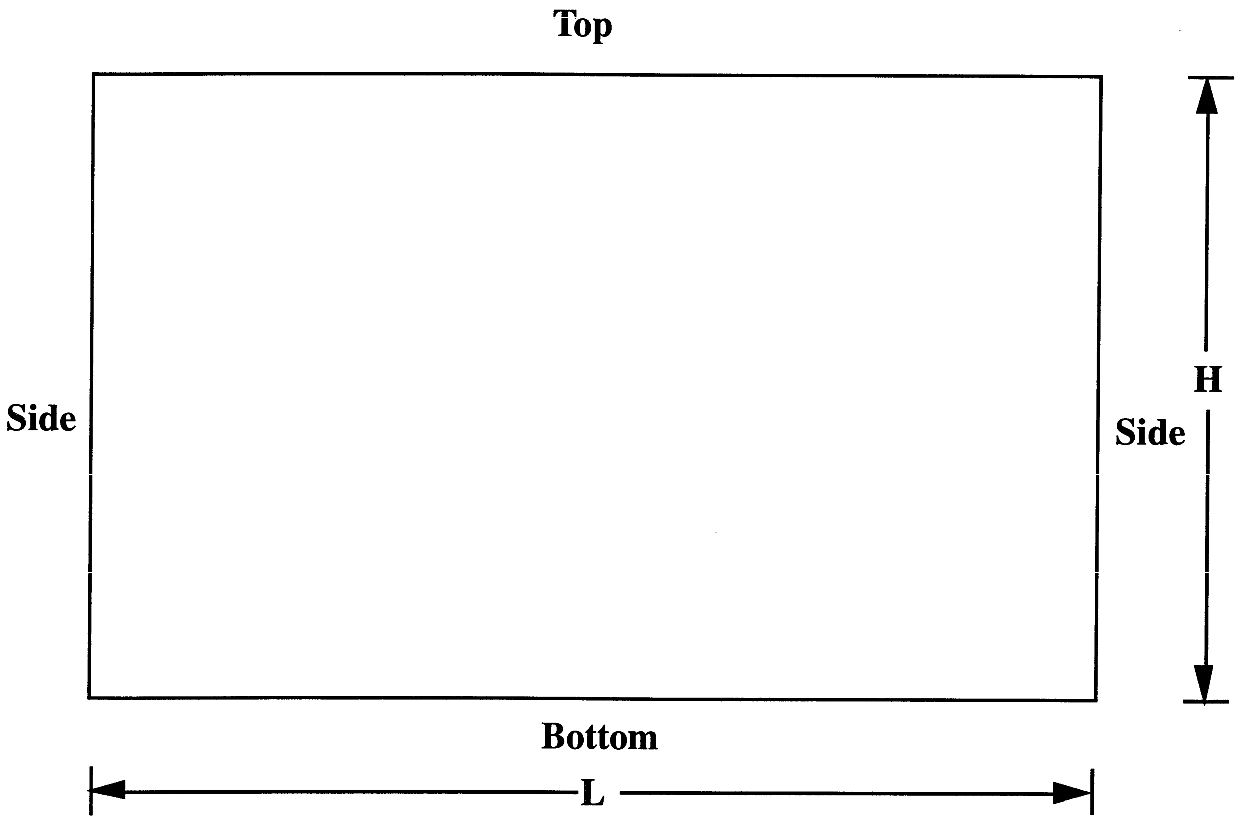
Table 10.5.1-1 $\left(\frac{H}{t}\right)_N$ versus Wall Thickness for use in URM Wall Screening (based on Sections 10.5.1.4 and 10.5.1.8)

Nominal URM Wall Thickness	Actual Concrete Block Wall Thickness	Minimum Concrete Block Flange Thickness	$\left(\frac{H}{t}\right)_N$
4"	3.625"	.75"	13.5
6"	5.625"	1.0"	11.5
8"	7.625"	1.25"	10.0
10"	9.625"	1.375"	9.0
12"	11.625"	1.5"	8.0

Table 10.5.1-2 DBE Ground Motion $S_{A_{max}}$ from UBC Seismic Zone (May be used for PC 1 Structures, Systems, and Components, Ref. 6)

DOE Site	Seismic Zone	$S_{A_{max}}$
Kansas City	2A	0.41
LANL	2B	0.55
Mound	1	0.21
Pantex Plant	1	0.21
Rocky Flats	1	0.21
Sandia, Albuquerque	2B	0.55
Sandia, Livermore	4	1.10
Pinellas Plant	0	0.10
Argonne-East	0	0.10
Argonne-West	2B	0.55
Brookhaven	2A	0.41
Princeton	2A	0.41
INEL	2B	0.55
Feed Materials Production Center	1	0.21
Oak Ridge	2A	0.41
Paducah	2A	0.41
Portsmouth	1	0.21
Nevada Test Site	3	0.83
Hanford	2B	0.55
LBL	4	1.10
LLNL	4	1.10
ETEC	4	1.10
SLAC	4	1.10
Savannah River	2A	0.41

Table 10.5.1-3 Boundary Condition Factors, B_F, for Fundamental Frequency Calculation (Table 1 of Reference 117)



Case 1: Simple Support Top/Simple Support Bottom with Specified Combination of Side Supports

H/L	Free-Free	SS-Free	Fixed-Free	SS-SS	SS-Fixed	Fixed-Fixed
≤ 0.20	1.571	1.571	1.571	1.571	1.571	1.571
0.4	1.571	1.612	1.622	1.822	1.870	1.931
0.667	1.571	1.698	1.748	2.270	2.480	2.765
1.0	1.571	1.859	2.020	3.142	3.764	4.608
1.5	1.571	2.182	2.677	5.106	6.769	8.968
2.5	1.571	2.992	4.875	11.39	16.54	23.16

Table 10.5.1-3 (Continued)

Case 2: Fixed Top/Fixed Bottom with Specified Combination of Side Supports

H/L	Free-Free	SS-Free	Fixed-Free	SS-SS	SS-Fixed	Fixed-Fixed
≤ 0.20	3.561	3.561	3.561	3.561	3.561	3.561
0.4	3.561	3.587	3.594	3.706	3.731	3.764
0.667	3.561	3.638	3.664	3.986	4.116	4.299
1.0	3.561	3.734	3.823	4.608	5.066	5.730
1.5	3.561	3.944	4.254	6.221	7.666	9.672
2.5	3.561	4.545	5.994	12.07	17.05	23.52

Case 3: Simple Support Top/Fixed Bottom (or Vice-Versa) with Specified Combination of Side Supports

H/L	Free-Free	SS-Free	Fixed-Free	SS-SS	SS-Fixed	Fixed-Fixed
≤ 0.20	2.454	2.454	2.454	2.454	2.454	2.454
0.4	2.454	2.491	2.499	2.646	2.682	2.727
0.667	2.454	2.558	2.593	3.008	3.175	3.407
1.0	2.454	2.685	2.804	3.764	4.307	5.066
1.5	2.454	2.951	3.349	5.579	7.144	9.260
2.5	2.454	3.672	5.344	11.69	16.76	23.32

Table 10.5.1-3 (Continued)

Case 4: Free Top/Fixed Bottom with Specified Combination of Side Supports

H/L	Free-Free	SS-Free	Fixed-Free	SS-SS	SS-Fixed	Fixed-Fixed
≤0.20	0.560	0.560	0.560	0.560	0.560	0.560
0.4	0.560	0.613	0.634	0.780	0.855	0.959
0.667	0.560	0.704	0.793	1.190	1.488	1.891
1.0	0.560	0.897	1.105	2.020	2.804	3.823
1.5	0.560	1.103	1.786	3.932	5.833	8.243
2.5	0.560	1.607	3.965	10.14	15.62	22.46

Case 5: Free Top/Simple Support Bottom with Specified Combination of Side Supports

H/L	Free-Free*	SS-Free	Fixed-Free	SS-SS	SS-Fixed	Fixed-Fixed
≤0.2	0	0.107	0.159	0.224	0.258	0.285
0.4	0	0.210	0.257	0.479	0.587	0.727
0.667	0	0.356	0.491	0.971	1.313	1.755
1.0	0	0.536	0.854	1.859	2.685	3.734
1.5	0	0.800	1.585	3.821	5.755	8.186
2.5	0	1.313	3.834	10.08	15.57	22.42

* Rigid Body Mode

Table 10.5.1-4 Frequency Factors, F (Table 2 of Reference 117)

WALL HEIGHT H	HOLLOW MASONRY THICKNESS					SOLID MASONRY THICKNESS				
	4"	6"	8"	10"	12"	4"	6"	8"	10"	12"
6'	17.4	26.8	36.5	45.8	55.1	13.5	20.9	28.3	35.8	43.2
8'	9.81	15.1	20.5	25.7	31.0	7.57	11.8	15.9	20.1	24.3
10'	6.28	9.65	13.1	16.5	19.8	4.85	7.52	10.2	12.9	15.5
12'	4.36	6.70	9.13	11.4	13.8	3.37	5.22	7.08	8.94	10.8
14'	3.20	4.92	6.71	8.41	10.1	2.47	3.84	5.20	6.57	7.94
16'	2.45	3.77	5.14	6.44	7.75	1.89	2.94	3.98	5.03	6.07
18'	1.94	2.98	4.06	5.09	6.13	1.50	2.32	3.15	3.97	4.79
20'	1.57	2.41	3.29	4.12	4.96	1.21	1.88	2.55	3.22	3.88
24'	1.09	1.68	2.28	2.86	3.45	.841	1.31	1.77	2.23	2.70
30'	.698	1.07	1.46	1.83	2.21	.538	.836	1.13	1.43	1.73

$$F = (1/H^2) * (EI'g/w)^{1/2}$$

where

H = Wall Height (in)

E = Elastic Modulus = 1×10^6 #/in^2

I' = Effective Plate Moment of Inertia (in^4/in)

g = Acceleration of Gravity = 386.4 in/sec^2

w = Distributed Load per Unit Surface Area (#/in^2) based on masonry weight density = 150 #/ft^3

Table 10.5.1-5 Elastic Modulus Factor (α_E)
(Table 3 of Reference 117)

The Frequency Factor, F, is based on E = 1 x 10^6 psi. To adjust f for other values of E, $\alpha_E = \sqrt{E/(1x10^6)}$. For masonry, E is typically taken as 1000 f'_m, where f'_m is the compressive strength of the masonry unit/mortar combination. The typical range of E is 0.7 x 10^6 psi to 2.5 x 10^6 psi. Site-specific testing can be utilized to determine E.

The following table shows α_E vs. E for the range of interest:

E (psi)	α_E
0.5 x 10^6	0.71
0.7 x 10^6	0.84
0.9 x 10^6	0.95
1.0 x 10^6	1.0
1.25 x 10^6	1.12
1.50 x 10^6	1.22
1.75 x 10^6	1.32
2.00 x 10^6	1.41
2.25 x 10^6	1.50
2.50 x 10^6	1.58
2.75 x 10^6	1.66
3.00 x 10^6	1.73

Table 10.5.1-6 Weight Density Factor (α_D)
(Table 4 of Reference 117)

The Frequency Factor, F, is based on a weight density, ρ, of 150#/ft^3 for the masonry material. Based on the density, the masonry block construction (solid vs. hollow), and the nominal block thickness (4", 6", 8", 10", 12"), the surface loading, w, is defined in #/in^2.

The density of masonry may vary over a wide range, depending on the application. By varying aggregate density and constituent ratios, ρ can range from 75 #/ft^3 to 200 #/ft^3. For most DOE facilities, the reference value of ρ = 150 #/ft^3 should be a suitable, slightly conservative value.

To account for cases where there is significant difference, based on site-specific design specifications or sample testing, the following table provides values of α_D vs. ρ for the expected range of variation:

ρ (#/ft.^{3})	α_D
200	0.87
175	0.93
150	1.0
125	1.10
100	1.22
75	1.41

To adjust f for other values of ρ, $\alpha_D = \sqrt{150 / \rho}$

Additional Weight of Attachments

To account for the additional weight of attachments to the wall, an effective weight density can be estimated as follows:

1. Estimate total weight of attachments, WT_A

2. Divide WT_A by gross wall volume (HxLxt) to get effective increase in density

   $\rho_A = WT_A / (HLt)$ [#/ft^3]

3. For solid masonry, effective total density is

   $\rho = \rho_{masonry} + \rho_A$

4. For hollow masonry, effective total density is

   $\rho = \rho_{masonry} + 2\ (\rho_A)$

   The factor of 2 on ρ_A for hollow masonry accounts for the fact that the net volume is approximately 50% of the gross volume.

5. Select factor α_D based on the effective total density.

Table 10.5.1-7 Orthotropic Behavior Adjustment Factor (α_T) (Table 5 of Reference 117)

A. Solid Masonry

For solid masonry (including hollow masonry with completely grouted cells), isotropic out-of-plane bending behavior is expected. Consequently,

$$\alpha_T = 1.0$$

B. Hollow Masonry

Based on the geometry of the hollow masonry, the section properties resisting out-of-plane bending are different for bending about axes perpendicular to and parallel to the cell axis direction. Assuming completely mortared web joints between masonry units, the webs contribute to the bending resistance about an axis perpendicular to the cell axis direction. For bending about an axis parallel to the cell axis direction, the webs are considered to be ineffective; this results in a modest reduction of bending resistance, which is a function of the masonry unit thickness. The significance of this reduction on the out-of-plane natural frequency depends on the plate aspect ratio and the cell axis direction. The worst case reduction factors are provided in the table below for the range of masonry unit thicknesses:

Hollow Masonry Unit Thickness (in.)	α_T (minimum value)
4"	0.98
6"	0.97
8"	0.96
10"	0.94
12"	0.91

A more accurate value for α_T can be determined by the following procedure:

1) Calculate the wall aspect ratio (AR), defined as the lineal dimension parallel to the cell axis divided by the lineal dimension perpendicular to the cell axis:
2) For AR $\leq$ 0.2, use $\alpha_T = 1.0$.
3) For AR $\geq$ 5.0, use α_T (min) = 0.91.
4) For AR = 1.0, use $\alpha_T = 0.5\ [1.0 + \alpha_T\ (\text{min})]$.
5) For 0.2 < AR < 1.0, use linear interpolation between 1.0 and $0.5\ [1.0 + \alpha_T\ (\text{min})]$.
6) For 1.0 < AR < 5.0, use linear interpolation between $0.5\ [1.0 + \alpha_T\ (\text{min})]$ and α_T (min).

Table 10.5.1-8 Special Considerations for Elastic Method (Table 6 of Reference 117)

A) Partial Grouting of Cells in Hollow Masonry

If selected cells are grouted from top to bottom of the wall, in a regular pattern, then both wall mass and stiffness are increased. This would tend to decrease the applicable frequency factor, F. Therefore, the solid masonry values in Table 10.5.1-4 can be used as a conservative lower bound for F. Alternately, interpolation between the solid and hollow masonry values can be used, based on the percentage of cells filled.

B) Partially Filled Mortar Joints

1) Solid Masonry

This is an undesirable condition, which raises questions about the original construction workmanship. A technical basis for such construction should be investigated. In addition, a significant amount of in-situ sampling is probably required to characterize the mortar joints

2) Hollow Masonry

The original construction may not have specified mortaring of the webs in the bed joints. If this condition has been verified by in-situ sampling then the Orthotropic Behavior Adjustment Factor, α_T, is set to the appropriate minimum value from Table 10.5.1-5 in the calculation of the wall frequency. This effectively eliminates any contribution to bending stiffness from the webs.

Any other deviation from fully mortared joints is an undesirable condition. Refer to discussion above for solid masonry.

C) Multi-Wythe and Composite Construction

The possible combinations are too numerous to quantify. However, certain guidance can be provided for the assessment of such walls.

1) If adequate connectivity between wythes cannot be demonstrated, then each wythe must be treated as a separate wall. In this case, the formulas and data provided here should be applicable to each wythe.

2) Adequate connectivity should be verified by definitive design and fabrication documentation, supported by in-situ sampling.

3) The Boundary Condition Factor, B_f from Table 10.5.1-3 is applicable to multi-wythe and composite construction. A case-specific Frequency Factor, F, would have to be developed for composite bending behavior.

Table 10.5.1-9 Boundary Condition Factors, B_S, for Maximum Bending Stress Calculation (Table 7 of Reference 117)

Case 1: SS Top/SS Bottom

H/L	Free-Free Sides	SS-SS Sides	Fixed-Fixed Sides
≤ 0.20	0.125	0.125	0.125
0.4	0.125	0.110	0.122
0.667	0.125	0.081	0.105
1.0	0.125	0.048	0.070
1.5	0.125	0.036	0.037
2.5	0.125	0.018	0.013

Case 2: Fixed Top/Fixed Bottom

H/L	Free-Free Sides	SS-SS Sides	Fixed-Fixed Sides
≤ 0.20	0.083	0.083	0. 083
0.4	0.083	0.083	0.083
0.667	0.083	0.082	0.076
1.0	0.083	0.070	0.051
1.5	0.083	0.047	0.034
2.5	0.083	0.020	0.013

Table 10.5.1-9 (Continued)

Case 3: SS Top/Fixed Bottom (or Vice-Versa)

H/L	Free-Free Sides	SS-SS Sides	Fixed-Fixed Sides
≤ 0.20	0.125	0.125	0.125
0.4	0.125	0.125	0.119
0.667	0.125	0.110	0.095
1.0	0.125	0.084	0.060
1.5	0.125	0.050	0.034
2.5	0.125	0.020	0.013

Case 4: Free Top/Fixed Bottom

H/L	Free-Free Sides	SS-SS Sides	Fixed-Fixed Sides
≤ 0.20	0.50	0.50	0.50
0.4	0.50	0.375	0.275
0.667	0.50	0.227	0.173
1.0	0.50	0.119	0.085
1.5	0.50	0.055	0.037
2.5	0.50	0.021	0.013

Table 10.5.1-9 (Continued)

Case 5: Free Top/Simple Support Bottom

H/L	Free-Free Sides	SS-SS Sides	Fixed-Fixed Sides
0.2	*	0.78	0.78
0.4	*	0.34	0.34
0.667	*	0.187	0.187
1.0	*	0.112	0.085
1.5	*	0.057	0.037
2.5	*	0.021	0.013

* Unstable Condition

Table 10.5.1-10 Stress Factors, S (psi)
(Table 8 of Reference 117)

WALL HEIGHT H	HOLLOW MASONRY THICKNESS					SOLID MASONRY THICKNESS				
	4"	6"	8"	10"	12"	4"	6"	8"	10"	12"
6'	460	310	230	195	170	745	480	355	280	230
8'	815	555	410	345	305	1,325	850	630	500	415
10'	1,275	865	640	545	475	2,075	1,330	985	780	645
12'	1,835	1,245	925	780	680	2,985	1,915	1,415	1,120	930
14'	2,500	1,695	1,255	1,065	930	4,065	2,610	1,930	1,525	1,265
16'	3,260	2,215	1,640	1,390	1,215	5,310	3,405	2,520	1,995	1,650
18'	4,130	2,805	2,075	1,760	1,535	6,720	4,310	3,185	2,525	2,090
20'	5,100	3,460	2,565	2,170	1,895	8,295	5,320	3,935	3,115	2,580
24'	7,340	4,985	3,690	3,125	2,730	11,945	7,665	5,665	4,485	3,715
30'	11,470	7,790	5,765	4,885	4,265	18,660	11,975	8,850	7,010	5,805

$S = H^2 * (wc/I')$

where
- H = Wall Height (in)
- I' = Effective Plate Moment of Inertia (in^4/in)
- c = Distance from Neutral Axis to Extreme Fiber (in)
- w = Distributed Load per Unit Surface Area ($\#/in^2$) based on masonry weight density = $150\#/ft^3$

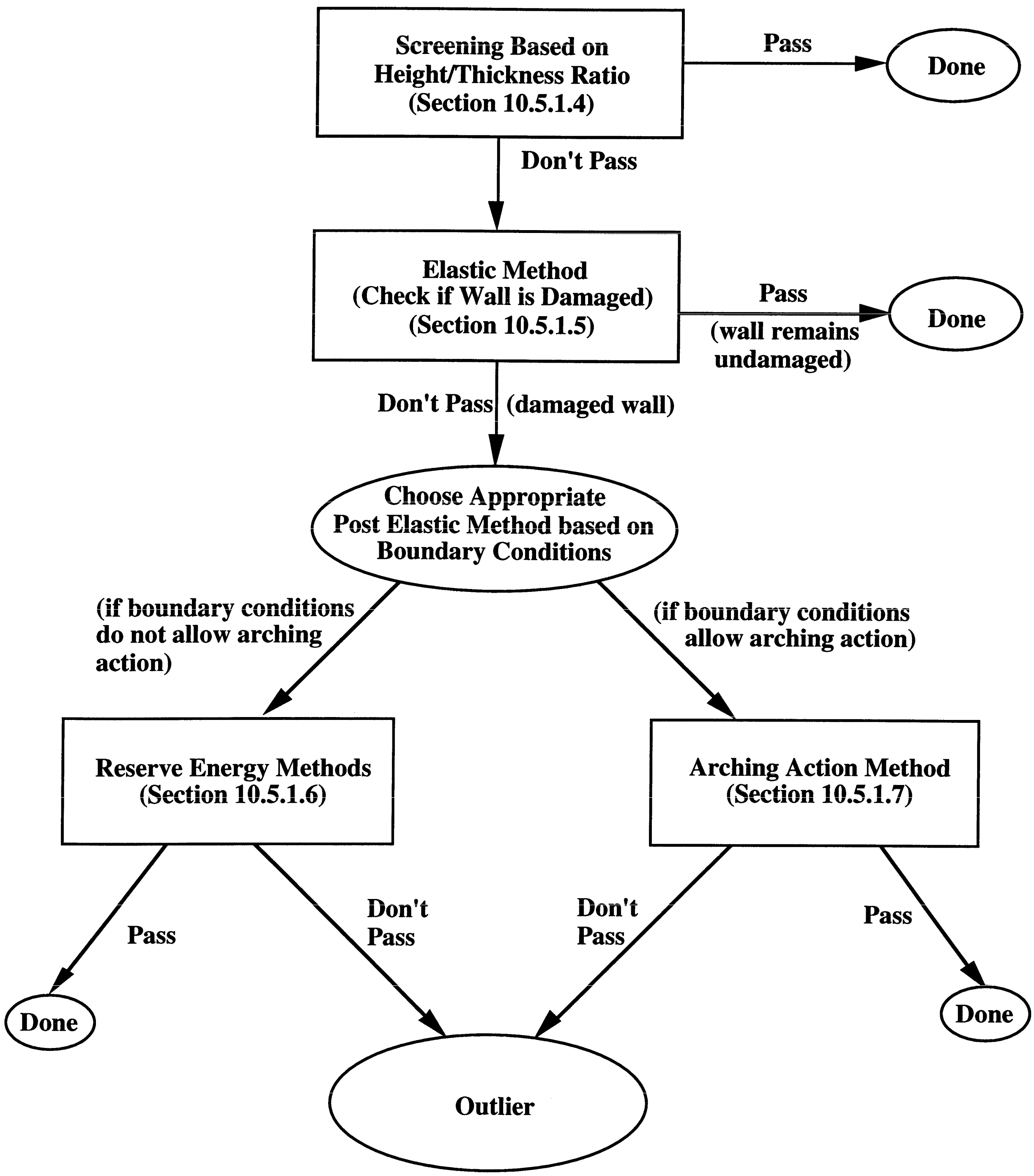
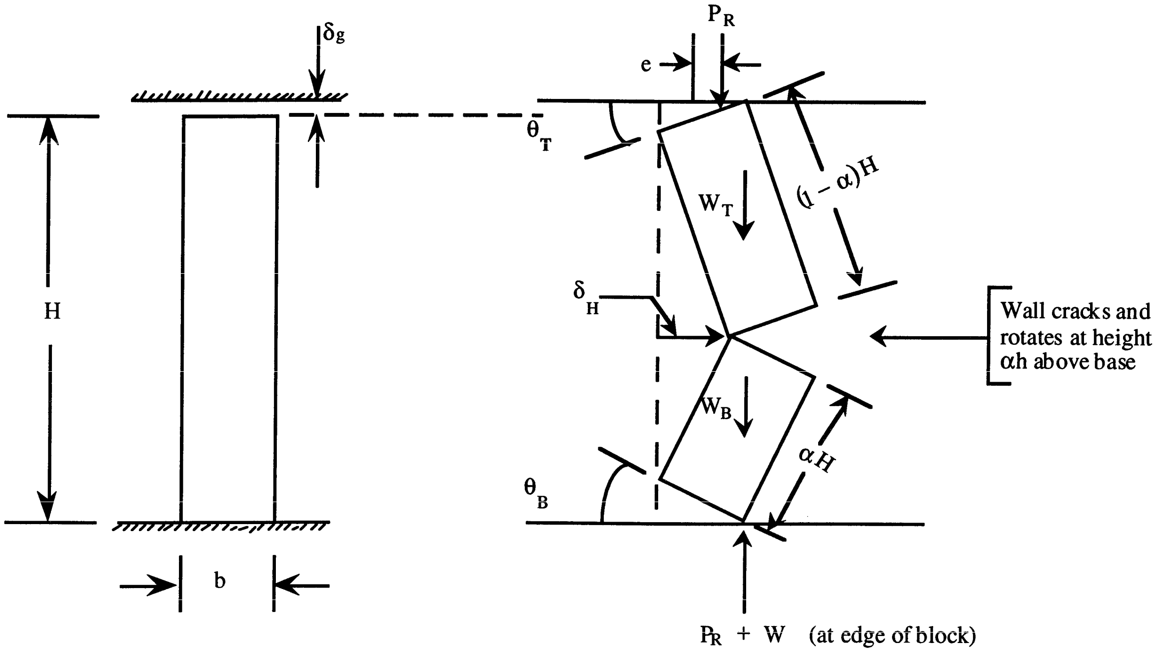


Figure 10.5.1-1 Methods for Evaluation of Out-of-Plane Bending of Non-Bearing Infill or Partition Unreinforced Masonry Walls in Section 10.5.1



P_R = in-plane compressive force

zero for Reserve Energy Method (non load bearing wall)
increases with displacement for Arching Action Method

W_B = $W\alpha$

W_T = $W(1-\alpha)$

W = block wall weight

α = parameter which locates crack location

e = load eccentricity from centerline of wall

H = wall height

b = effective wall thickness (= 0.9 actual wall thickness)

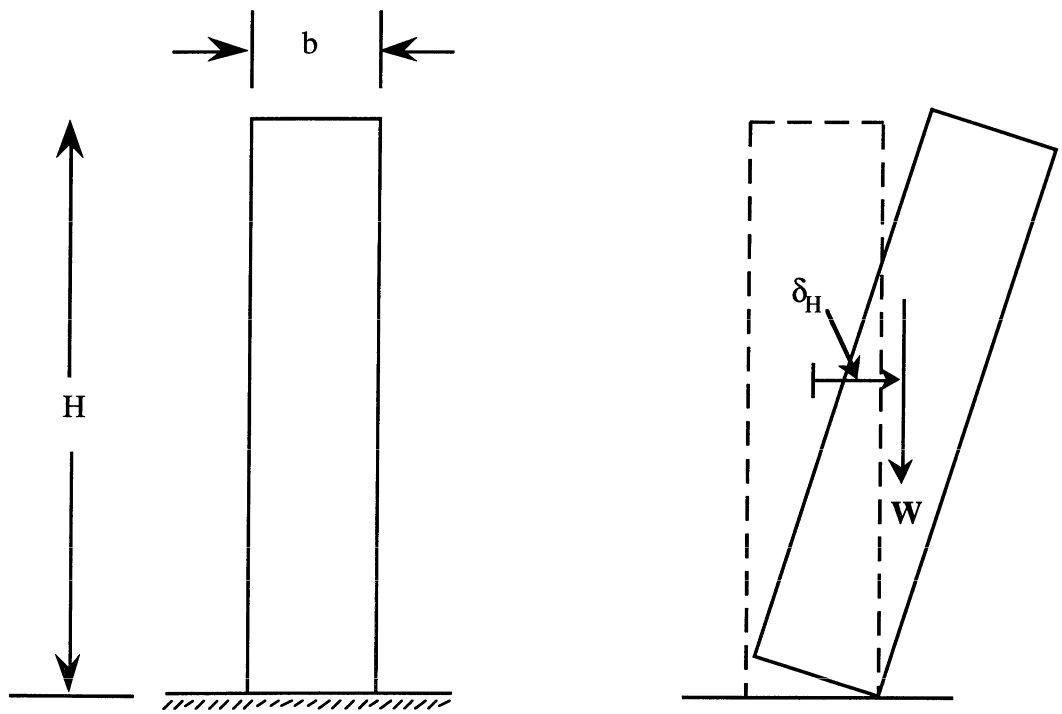
δ_H = lateral displacement

δ_g = gap between wall and upper support

θ_B = angle of rotation of bottom block

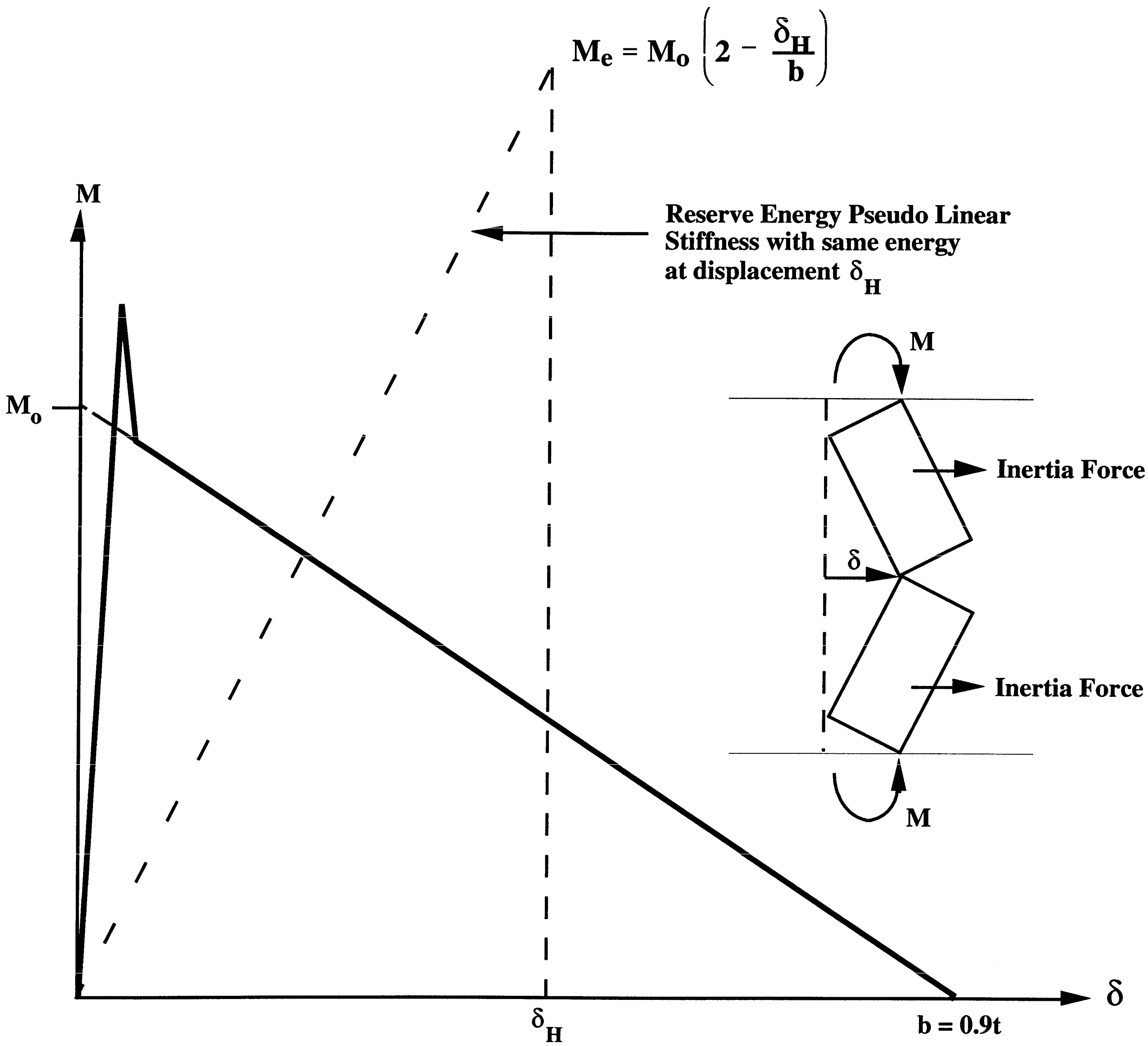
θ_T = angle of rotation of top block

Figure 10.5.1-2 Wall Properties for Reserve Energy and Arching Action Methods



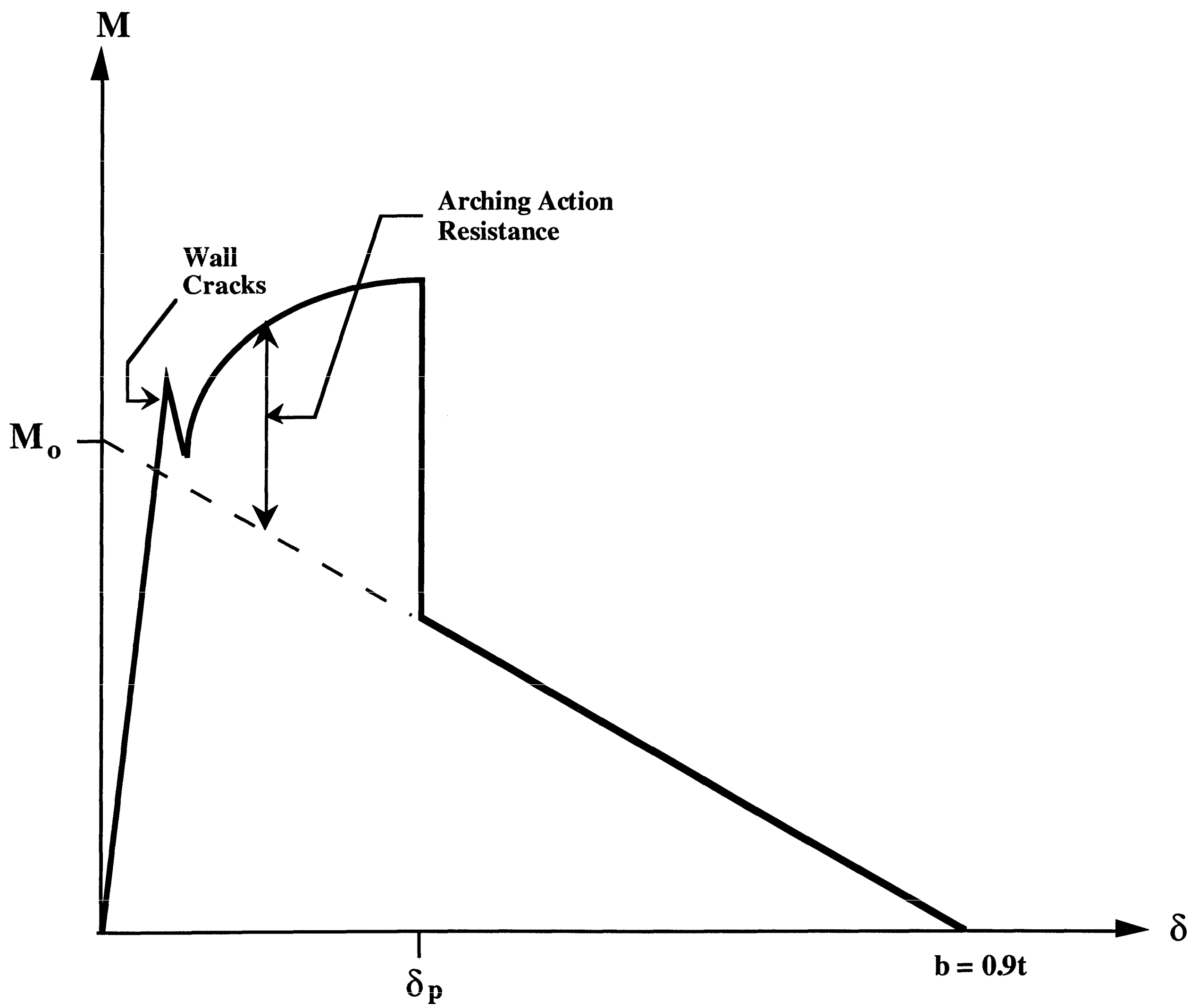
W	=	Block wall weight
H	=	wall height
b	=	effective wall thickness (= 0.9 actual wall thickness)
δ_H	=	Lateral displacement

Figure 10.5.1-3 Properties for a Cantilever Wall for Reserve Energy Method (Large gap at top of wall, non load bearing, and no lateral restraint at top of wall)



M	=	restoring moment
M_o	=	actual moment at zero displacement
M_e	=	effective moment
b	=	effective wall thickness
t	=	actual wall thickness
$\delta_1\ \delta_H$	=	out-of-plane displacements

Figure 10.5.1-4 Restoring Force for Reserve Energy Method



M	=	restoring moment
M_o	=	actual moment at zero displacement
b	=	effective wall thickness
t	=	actual wall thickness
δ_p	=	out-of-plane displacement at which ultimate capacity is reached
δ	=	out-of-plane displacement

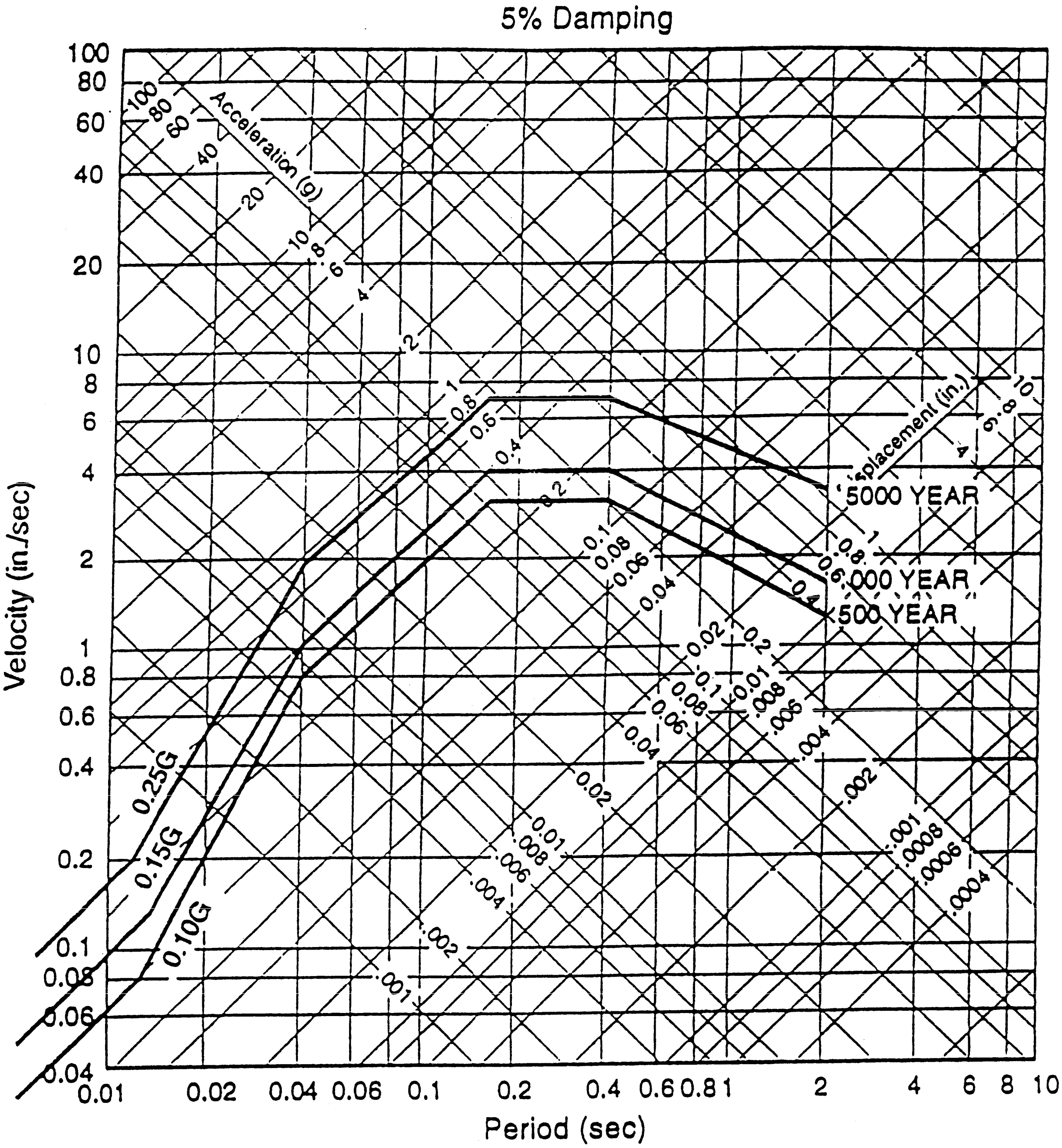


Figure 10.5.1-6A Portsmouth-Uniform Hazard Response Spectra for Horizontal Ground Motion

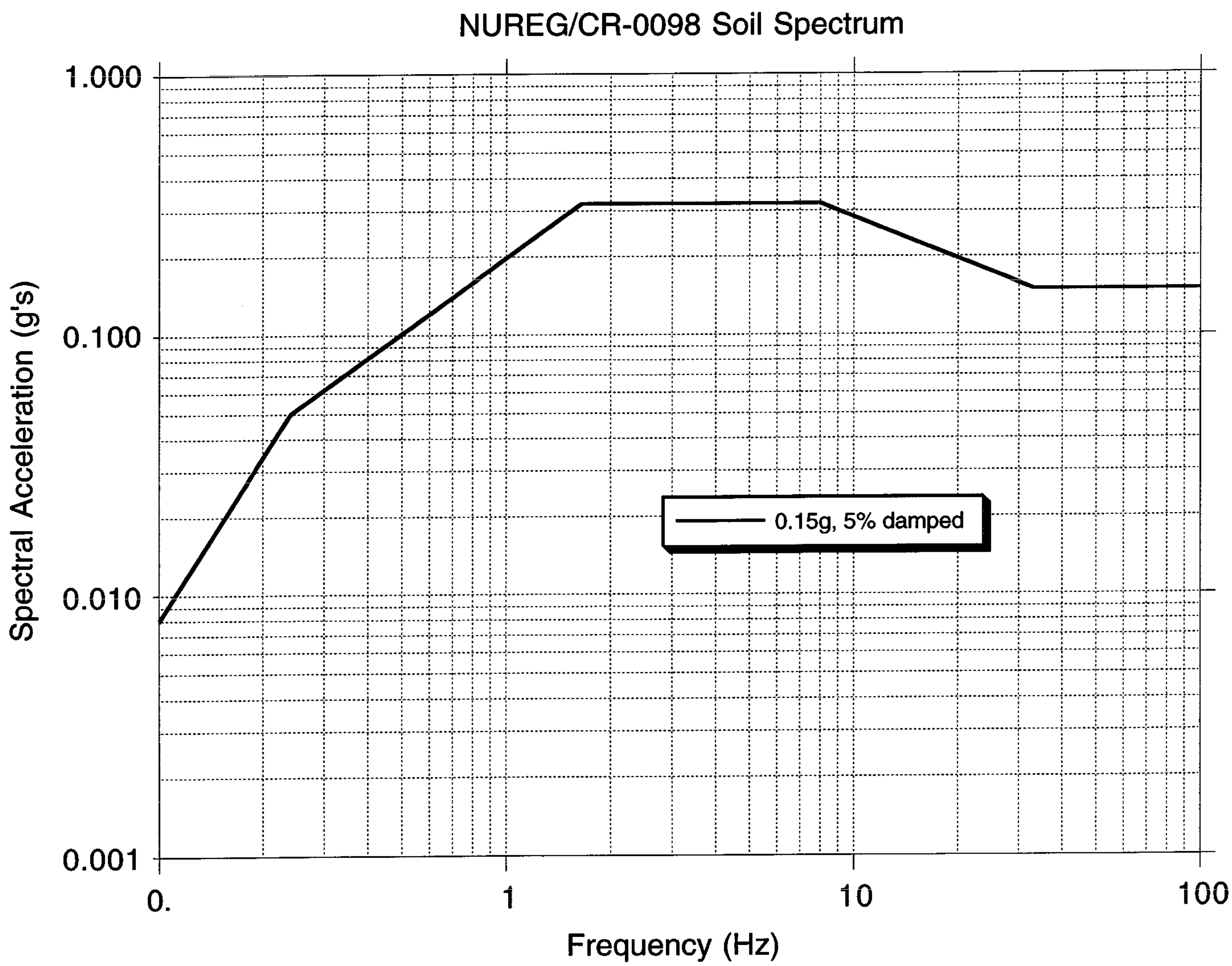


Figure 10.5.1-6B NUREG/CR-0098 Median Soil Spectrum (Ref. 72)

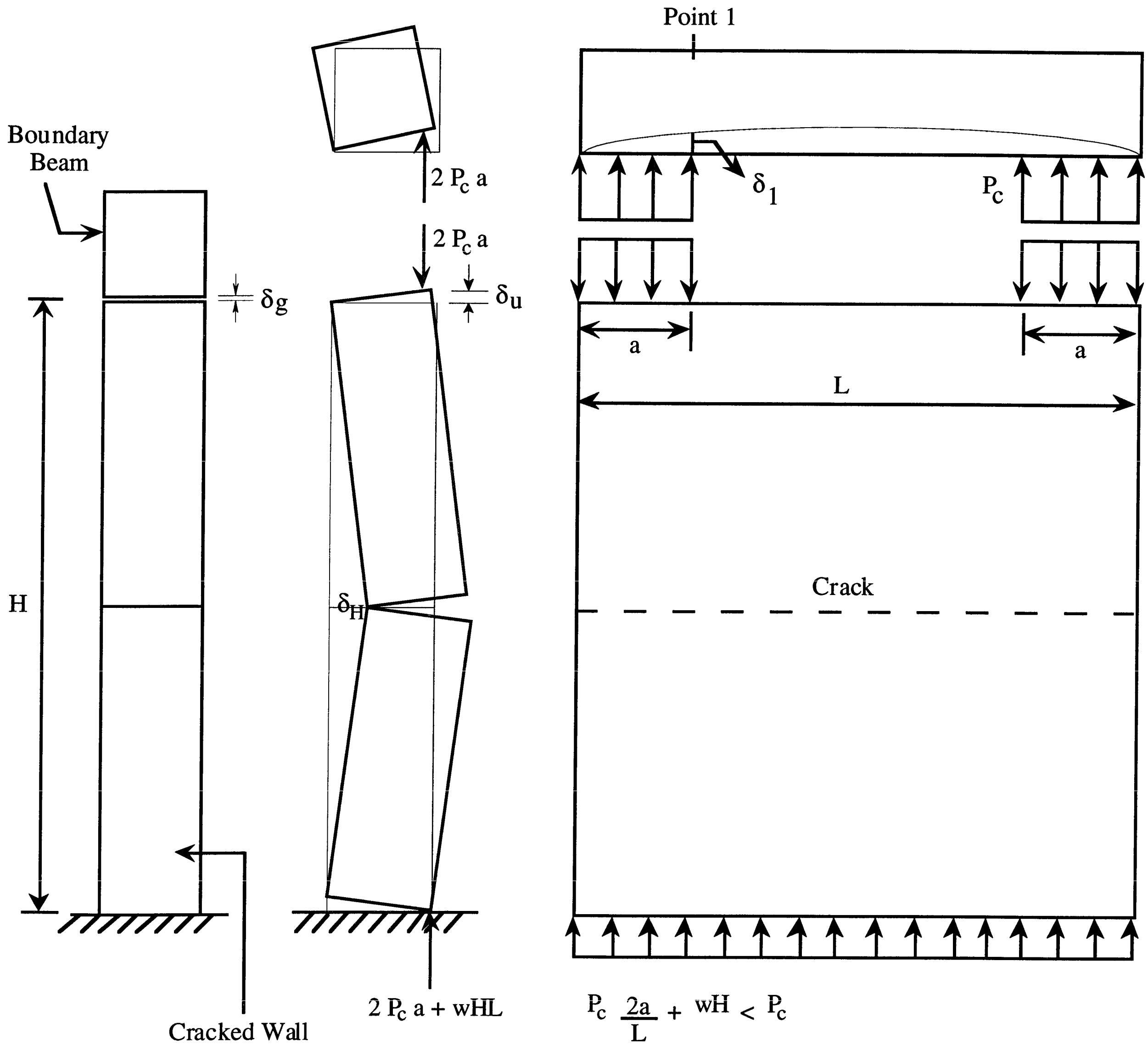
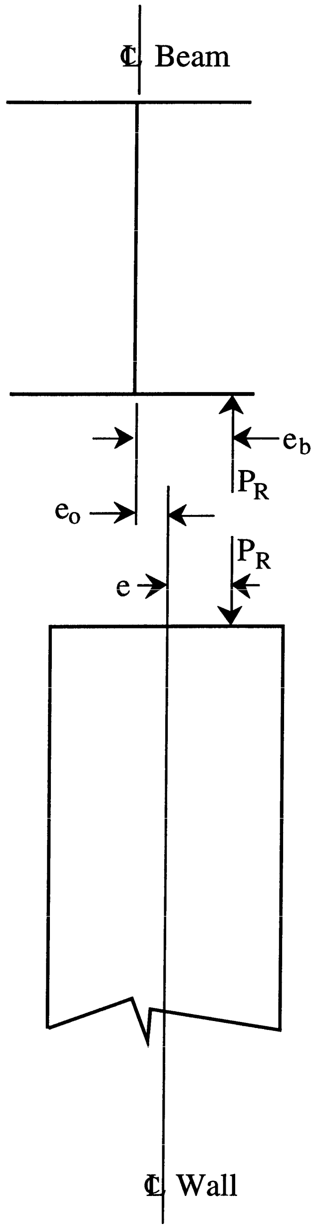


Figure 10.5.1-7 Arching Kinematics and Assumed Load Distribution along Length of Top Beam



e_o = offset between beam centerline and wall centerline

e = eccentricity to load P_R from wall centerline

e_b = eccentricity to load P_R from beam centerline

If beam twists more freely than top of wall rotates (typical for steel beam)

take $e_b = o$
$e = -e_o$

If twisting stiffness of beam is sufficiently large, then the beam twists less than the top of the wall rotates (typical for concrete beam)

take $e \approx 0.45\, b_f - e_o \leq 0.45t$
$e_b = e + e_o$
$b_f =$ flange width of beam

Figure 10.5.1-8 Geometry of Beam, Wall, and Confining Force

10.5.2 RAISED FLOORS

This section describes general guidelines that can be used for evaluating and upgrading the seismic adequacy of raised floors which are included in the Seismic Equipment List (SEL). The guidelines contained in this section are based on Section 4.4 of "Practical Equipment Seismic Upgrade and Strengthening Guidelines" (Ref. 60), Chapter 6 of "Data Processing Facilities: Guidelines for Earthquake Hazard Mitigation" (Ref. 121), and Chapter 9c of the "Seismic Safety Manual" (Ref. 32). In Chapter 6 of Reference 121, further detailed information on the seismic performance of raised floors and techniques for upgrading their seismic capacity is contained in the following sections: Descriptions of some of the more common floor systems and their strengths and weaknesses under earthquake loading; Specific guidelines for the seismic design, analysis, testing, and inspection of new raised floor systems; and Guidelines for analysis, retrofit design, and testing of existing raised access floors. Guidelines in this section of the DOE Seismic Evaluation Procedure cover those features of raised floors which experience has shown can be vulnerable to seismic loadings.

Because of extensive cabling requirements, components in computer facilities, data processing facilities, and control rooms are often supported on a raised floor with removable panels that may or may not be supported by stringers. A typical raise floor system is shown in Figure 10.5.2-1. A raised floor system forms the basic foundation or support for computer and data processing equipment, creates a space for a HVAC air plenum, and provides a protective shield for subfloor utilities vital to the operation of the equipment. The equipment supported on raised floors often costs hundreds of times more than the cost of the floor. Because of the cost of the equipment on a raised floor, earthquake-induced damage to the floor has a very high property loss potential. Furthermore, reconstruction of the collapsed floor and reinstallation of subfloor power, cooling, and signal cables could take a considerable amount of time. Potential damage evidenced in raised floor systems include buckling of support pedestals, buckling of floor panels, misalignment of floor penetrations, shifting of the entire floor system, and tipping of equipment supported by the floor.

For raised floor systems, the following seismic parameters should be evaluated:

- Seismic Demand Spectrum (SDS) at location of floor anchorage (see Section 5.2)
- dynamic stability or ability to withstand tipping and buckling capacity of pedestals
- type of anchorage system (leveling pads, skids, adhesives, clips, bolts, none)
- load path to load-bearing floor or foundation
- geometry and size (aspect ratio, height, width, length)
- relative strength and stiffness (stiff, flexible, strong, medium, weak)
- spacing of pedestals
- penetrations in the raised floor system
- operational considerations (weight being supported by floor, distribution of weight)

Large computer or control room raised floors may be susceptible to earthquake-induced damage due to tipping of the support pedestals. Figures 10.5.2-2 and 10.5.2-3 show examples of support pedestals that are typically slender, relatively long, and unanchored to the load-bearing floor or foundation. In addition, many raised floor systems lack lateral bracing between the pedestals (see Figure 10.5.2-4) which would provide horizontal stiffness.

To resist potential earthquake-induced damage, raised floor systems should be properly anchored by drilling holes in the base plates of supporting pedestals and installing anchor bolts. The anchor bolts can be evaluated using the procedures in Chapter 6. Many raised floor systems use an adhesive to attach the pedestals to the load-bearing floor or foundation. Test results have indicated that this adhesive is not adequate for withstanding significant lateral motion.

Earthquake and test experience has indicated that the unbraced pedestals and the weld to the pedestal base plate are often too weak to transfer the required lateral loads. Bracing schemes as shown in Figures 10.5.2-5 should be provided to create moment-frame action of the raised floor systems, to increase the lateral stiffness of the raised floor system, and to avoid concerns about the weld to the pedestal base plate. Potential flexibility of the threaded screw connections and weak welds, such as tack welds, to the pedestal should be evaluated.

In addition to strengthening the raised floor support system, the penetrations in the floor systems should be carefully evaluated. In many cases, the equipment on the raised floor is not anchored so there needs to be adequate accommodations for movement of the equipment during an earthquake. If there are extensive floor penetrations, the equipment on the raised floor may roll into, tip on, or catch on the penetrations. This action may cause a large concentrated lateral overload on the floor system as well as cause local floor breakup due to panel buckling. The floor penetrations should be modified to prevent equipment entry or covered with special air vents that permit the equipment to traverse the floor without penetration. Special precautions may be required to anchor the equipment through the raised floor or tether it to prevent it from catching in the penetrations. For light equipment on a braced floor, connecting to the bracing at the stringers may be adequate restraint. The use of tethers is discussed below.

Strengthening of the raised floor will not necessarily provide a system capable of resisting the lateral loads associated with heavy computer or control equipment. Separate anchorage for these items of equipment should be provided. The most desirable strategy for upgrading the seismic capacity of computer equipment typically involves either floor anchorage, vertical bracing schemes, or the use of tethers. The anchorage of the equipment on the raised floor may be used for the following conditions:

- the equipment is relatively heavy
- analysis of the equipment indicates that it will tip
- the equipment is closely spaced and will impact
- the internal components have low vulnerability to vibratory motion
- the cabinet frame has sufficient strength and stiffness to support the equipment without supplemental bracing.

Because unbraced raised floors cannot carry significant lateral loads, independent anchorage and support for equipment meeting one or more of the conditions listed above should be to a load-bearing floor or foundation. With the independent support, the raised floor should not be part of the load path for the anchorage of large computer and control equipment. The base of the equipment should be evaluated to determine if it has adequate capacity to support the anchorage loads.

An approach for independently securing equipment on top of a raised floor is to use under-floor cable tethers which allow for limited movement of the equipment. The cable tethers secure the equipment by providing a support path between a floor or load-bearing wall and the base of the equipment. As discussed in Reference 32, the following factors should be considered when using a tethering system:

- openings in the raised floor should have raised edges or curbs to prevent the base of the equipment from sliding into the opening
- the equipment should be stable against overturning when an appropriate coefficient of friction (judgment is required) is assumed between the raised floor and the base of the equipment
- there should be sufficient space between equipment to prevent seismic interactions
- elastomeric pads or bumpers may be used between closely spaced equipment
- the location of tether anchors and cable attachments to the equipment should consider the distribution of mass and stiffness within the equipment
- the design of the tether anchorage should consider the interaction with the raised floor if the cable becomes taut
- attached lines to the equipment should have sufficient slack to accommodate the constrained movement of the equipment

A second approach for independently anchoring computer equipment to a load-bearing floor or foundation is to use a separate support system, such as a diagonally - braced frame, for the equipment. This support system must be adequately anchored, have adequate lateral bracing, and have an appropriate load path from the equipment to the support system. If the equipment anchorage to the separate support system passes through an unbraced raised floor, interactions between the floor and the equipment anchorage should be considered.

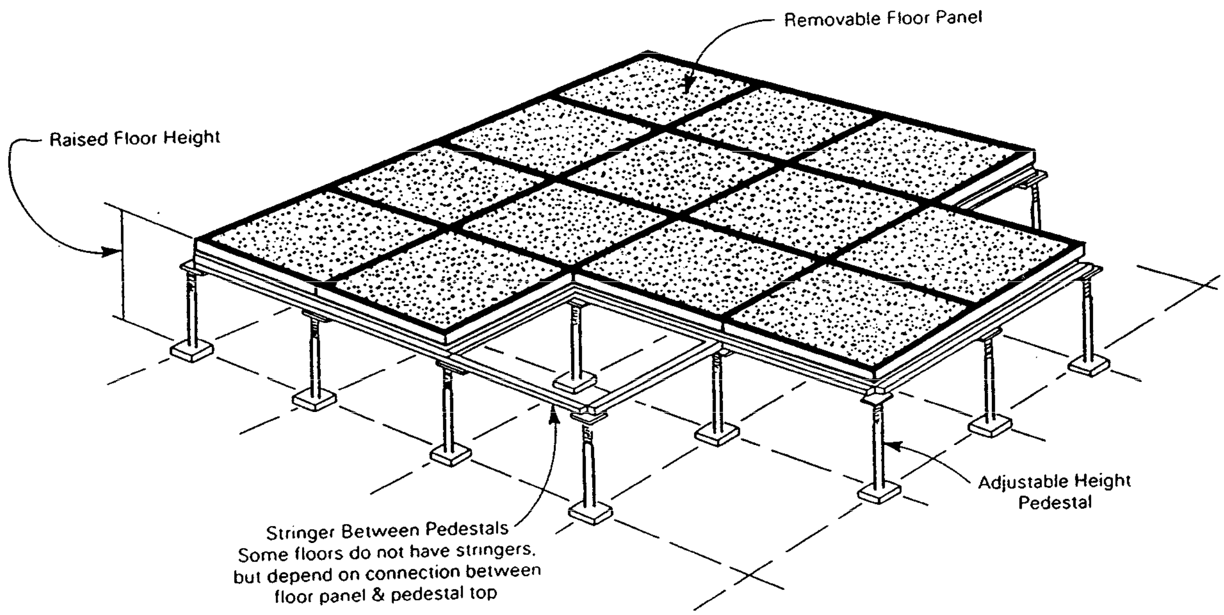


Figure 10.5.2-1 Raised Floor System (Figure 6.1 of Reference 121)

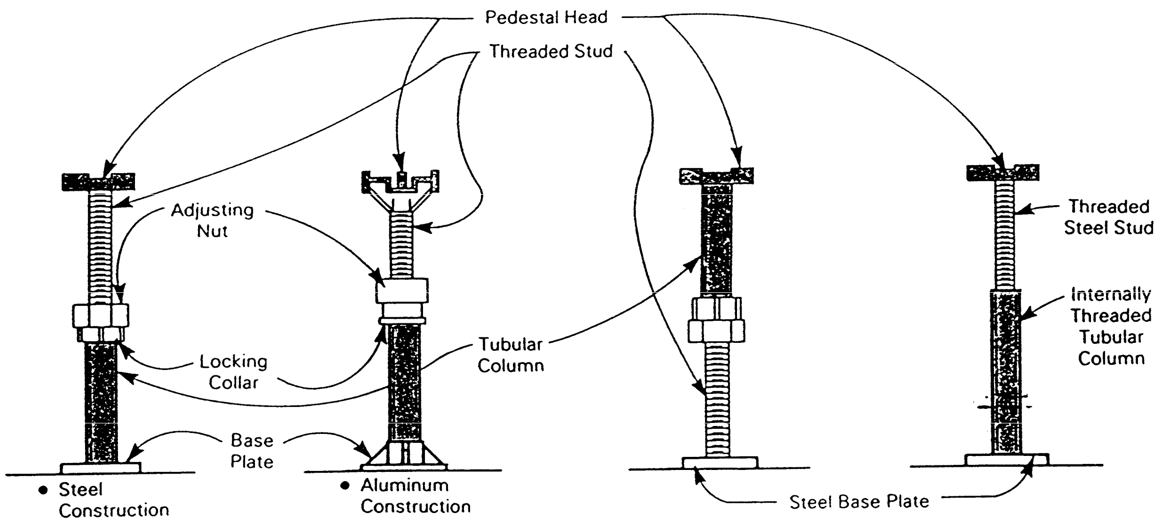


Figure 10.5.2-2 Pedestal Types (Figure 6.2 of Reference 121)

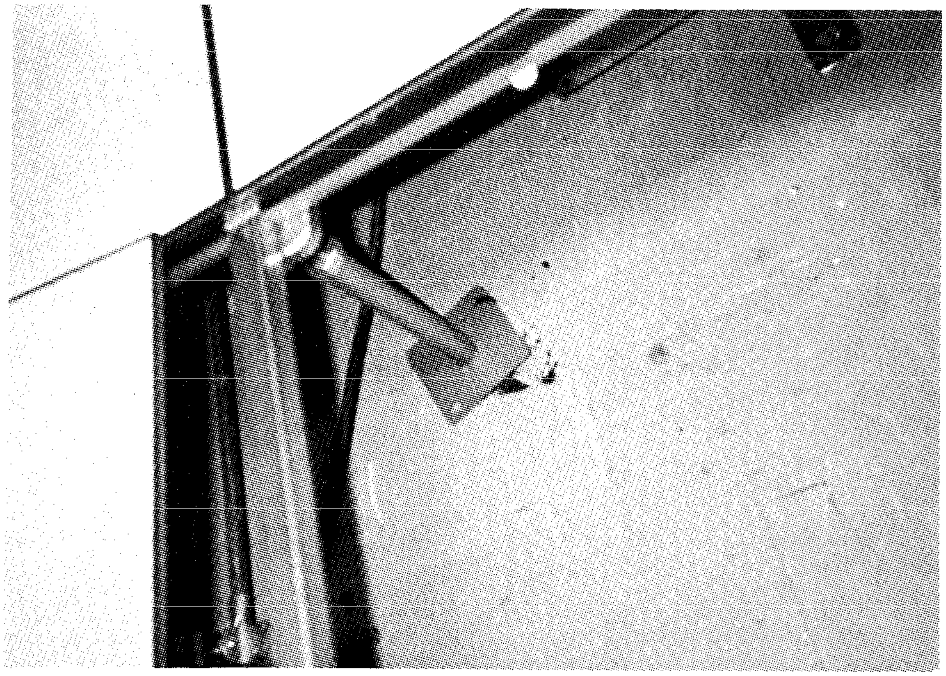


Figure 10.5.2-3 Raised Computer Floor Supported by Pedestal and Leveling Screw (Figure 4-30 of Reference 60)

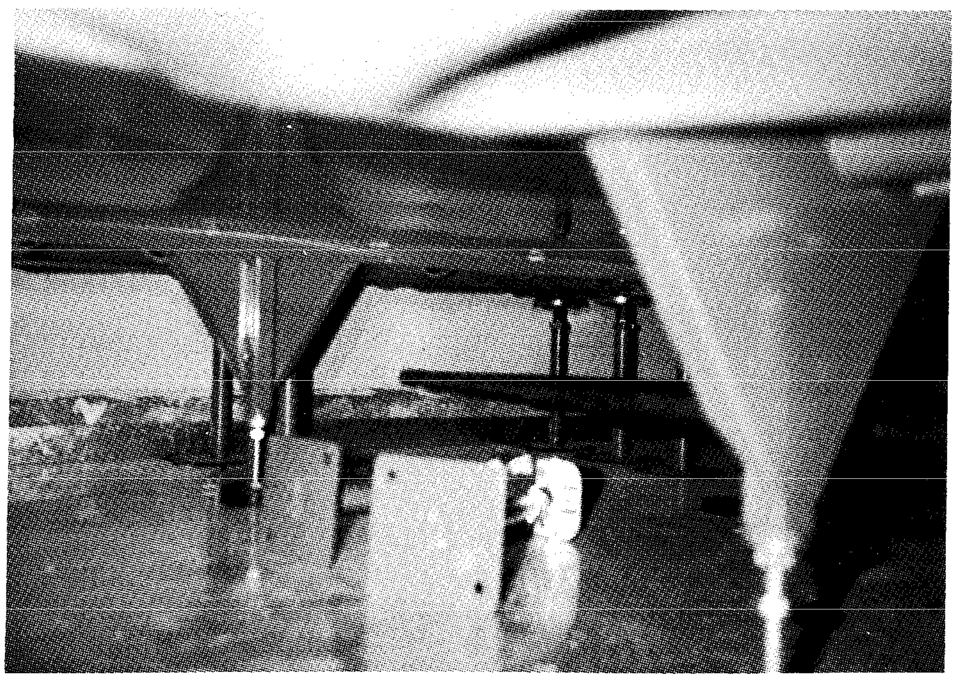


Figure 10.5.2-4 Raised Computer Floor Showing Lack of Lateral Bracing (Figure 4-31 of Reference 60)

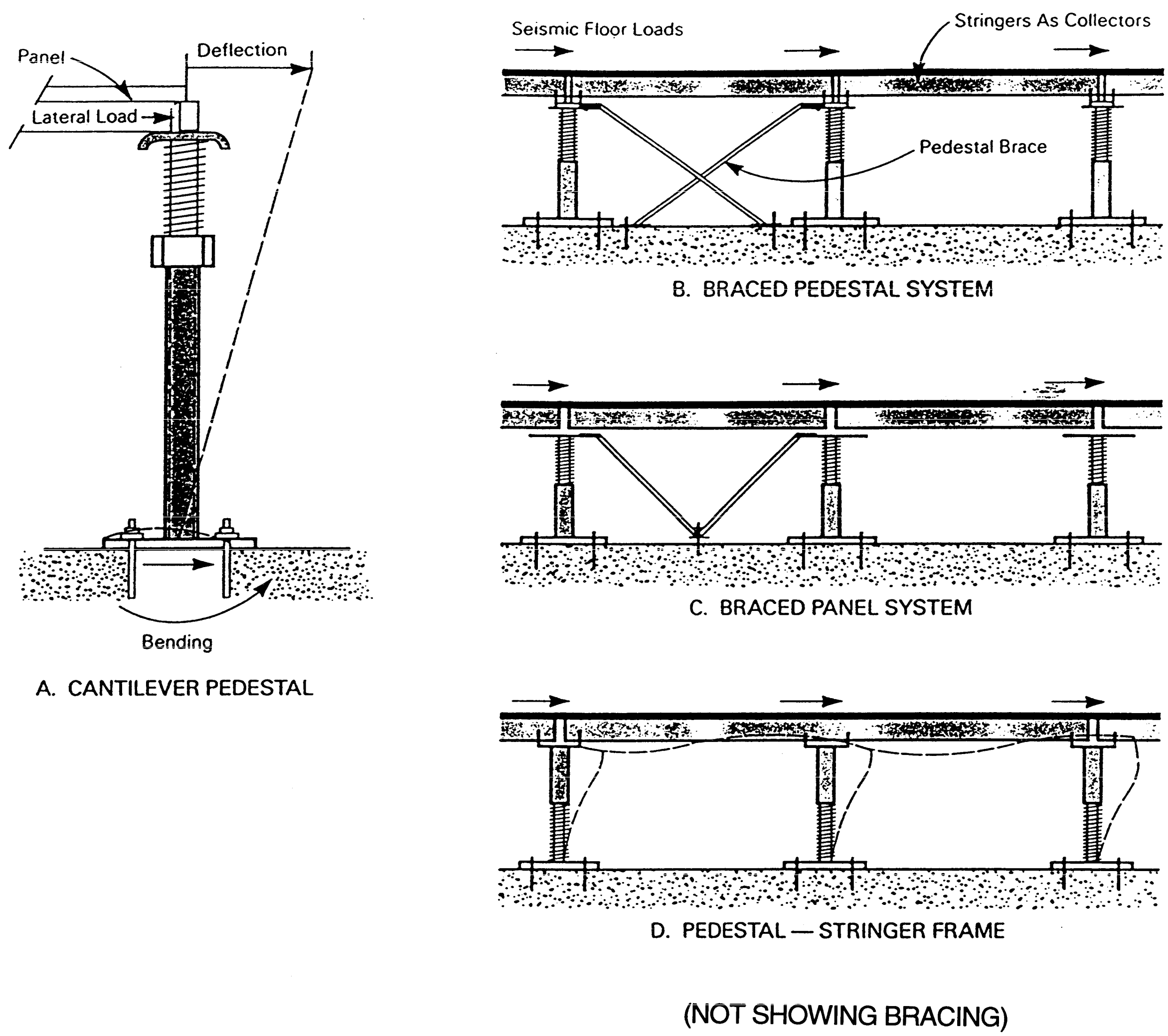


Figure 10.5.2-5 Lateral Force Resisting Systems (Figure 6.5 of Reference 121)

10.5.3 STORAGE RACKS

This section describes general guidelines that can be used for evaluating and upgrading the seismic adequacy of storage racks which are included in the Seismic Equipment List (SEL). The guidelines contained in this section are based on Sections 4.6.5 and 4.8 of "Practical Equipment Seismic Upgrade and Strengthening Guidelines" (Ref. 60). Guidelines in this section cover those features of storage racks which experience has shown can be vulnerable to seismic loadings.

Raw materials and finished products are typically stored on racks, in bins, or in stacks. Storage racks range from light metal shelving (see Figure 10.5.3-1) to heavy industrial grade shelving (see Figure 10.5.3-2). Inventory is extremely susceptible to earthquake-induced damage if racks or bins have no identifiable lateral load carrying system (see Figure 10.5.3-3). During an earthquake, items may slide off shelves or shelving may collapse which causes the contents to spill to the floor. If hazardous chemicals are involved, the resulting toxic chemical spill can be extremely dangerous and expensive to clean up.

The seismic evaluation of storage racks should emphasize the following considerations:

- anchorage
- structural capacity
- lateral bracing
- load path
- connection details
- restraints for contents

The structural capacity of a storage rack should be evaluated, especially its capacity for lateral loads. It may be difficult to determine the capacity of the rack without performing some calculations to determine member strengths and the modal, or stiffness, characteristics of the frame. Judgment may be required for determining the appropriate model for the connection details in a rack system. The connections in rack systems range from welded connections to slip joints. According to the provisions of Section 5.4, the capacity of the rack should be compared to the Seismic Demand Spectrum (SRS) at the anchorage location of the rack.

Storage racks should be evaluated to determine if they have adequate anchorage and if lateral bracing is present and of sufficient size to accommodate seismic loads. Tall racks should be anchored to walls with adequate capacity, the floor, and/or each other to prevent overturning. Most rack units have holes provided in their base plates and legs to accommodate anchor bolts. The screening evaluation for anchor bolts is provided in Chapter 6. The capacity of the floor to resist the anchorage loads should be evaluated. Many rack systems are leveled with shims and the excessive use of shims may reduce the capacity of the anchorage for those systems. If the rack is anchored to an unreinforced masonry (URM) wall, the capacity of the wall should be evaluated according to the provisions of Section 10.5.1 including the lateral loads of the racks.

Since racks are relatively flexible, extensive use of lateral bracing is useful in increasing the seismic capacity of the rack and in limiting earthquake-induced damage. Bracing should be provided at the ends and along the back side as shown in Figure 10.5.3-4. In addition to bracing, the load path in the structure should be evaluated. The bracing should attach to the structural members of the rack and these members should have sufficient capacity to withstand the earthquake-induced lateral demand. Many racks are designed only for vertical loads, so the effects of lateral loads should be evaluated. Additional information on the seismic design of storage racks is available from the Rack Manufacturer's Institute. Finally, possible reductions in the structural capacity of a storage rack may result from improper assembly of the rack or damage from operational use, such as forklift damage. Manufacturer's data should be used to determine if the rack was properly assembled and is being used as designed.

Horizontal shelves that are structurally attached to the supporting frame members are preferred as part of the connection details in a storage rack. If the rack has removable shelves, these shelves cannot be considered part of the lateral force resisting system. Loose pieces of wood spanning between frames may fall during an earthquake and should be restrained. Heavier stock should be moved to lower shelves to prevent injury to personnel and to minimize damage. Whenever possible, restraint should be provided for equipment or stock that can slide off during earthquake motions. Methods of achieving restrain include installation of a steel angle (lip) at the front edge of each shelf or an elastic band or tensioned wire across the opening. If feasible, removable restraints can also be provided across the front of the rack to preclude materials from sliding off shelves as shown in Figure 10.5.3-4.

During an earthquake, the support structure for drums supported on a rack may collapse if it does not have adequate lateral bracing and seismic anchorage. Poorly restrained canisters and drums may fall and/or roll causing them to possibly spill their contents, to damage other equipment, and injure personnel. Methods of restraining them include providing positive anchorage to the floor or a wall with adequate capacity, storing them in well-braced and anchored racks, or storing them horizontally on the floor.

Storage bins are temporary storage containers stacked on top of each other. Bins are often stacked very high with no lateral supports. In a strong earthquake, the upper bins can fall causing damage to contents and pose a possible life safety hazard. Materials stored in bins or stacks should be assessed to determine their stability under earthquake loads. Often, the seismic requirements of these components is in direct conflict with operational requirements. However, if materials are extremely hazardous or are expensive to replace, mitigation measures should be considered to provide positive restraint. These measures might include the installation of permanent racks, minimizing stack heights to 2 or 3 layers in height, or restraining existing stacks through tiedowns.

Figure 10.5.3-1 Unanchored Light Storage Racks Storing Hazardous Chemicals (Figure 4-62 of Reference 60)

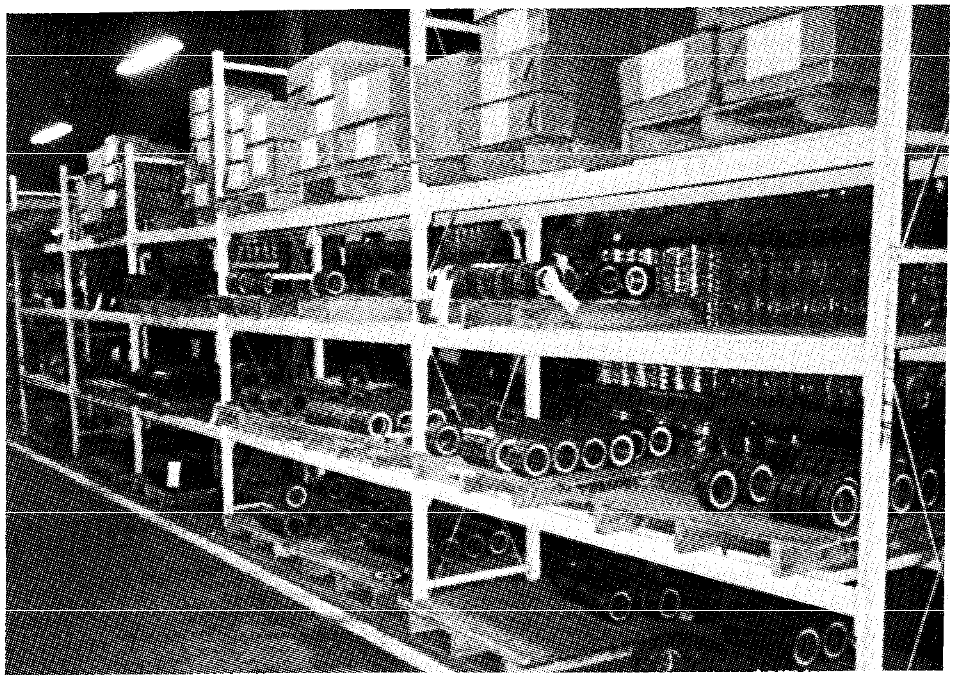


Figure 10.5.3-2 Unanchored Industrial Grade Shelving (Figure 4-63 of Reference 60)

Figure 10.5.3-3 Unanchored Storage Bins (Figure 4-64 of Reference 60)

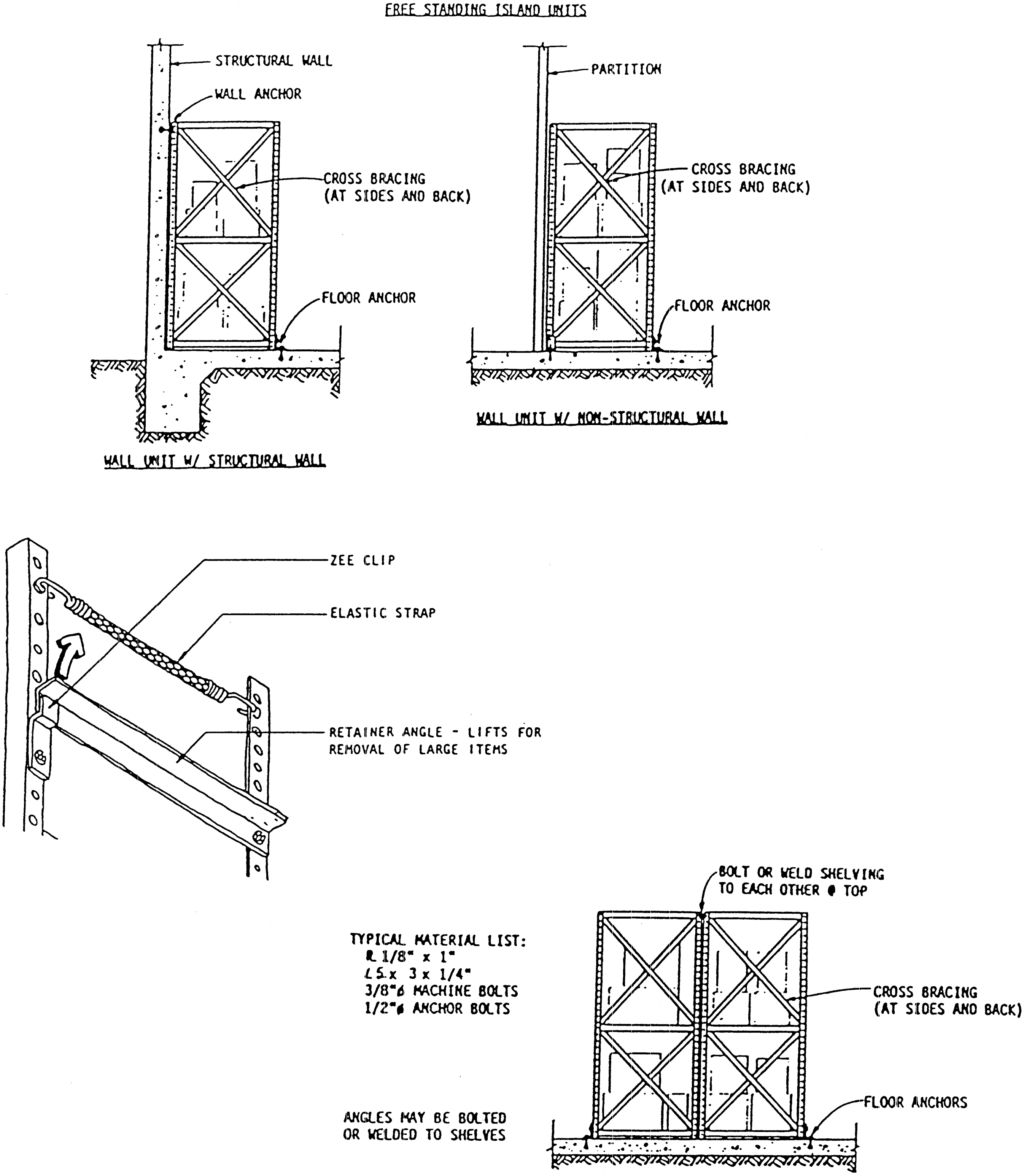


Figure 10.5.3-4 Approaches for Anchoring Storage Racks (Figure 4-67 of Reference 60)