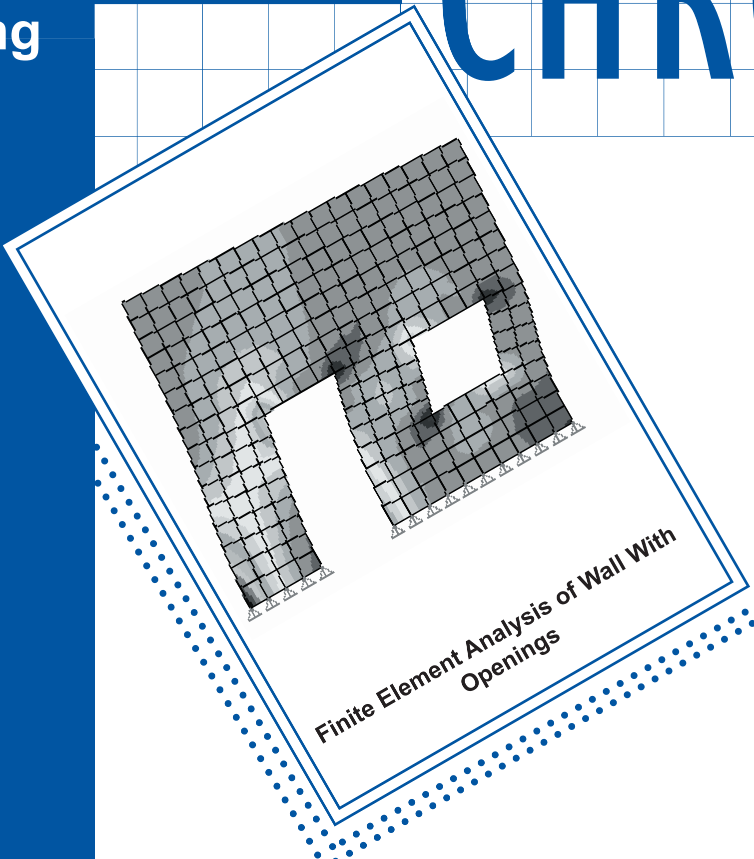
Engineering Notes For Design With Concrete Block Masonry

MASONRY CHRONICLES

Spring 2009




# OPENINGS IN CONCRETE MASONRY WALLS (Part II)

## Introduction

As discussed in the previous edition of "Masonry Chronicles," several aspects need to be considered when designing concrete masonry walls with openings such as doors, windows and vents. Those aspects include:

1. Design walls over openings to resist in-plane gravity load (dead, live, snow, etc.).
2. Design walls with openings to resist in-plane lateral loads (wind and seismic).
3. Design walls with openings to resist out-of-plane lateral loads (wind and seismic).

The previous edition of "Masonry Chronicles" covered the design of walls above openings to resist gravity loads. This edition will discuss the analysis of masonry walls with openings to resist in-plane lateral loads. The article will focus on how to determine the forces in the various segments of walls subjected to in-plane lateral loads. While some discussion on the design of walls will be presented, detailing requirements and procedures for determining the necessary wall reinforcement were previously presented in other editions of "Masonry Chronicles" (Winter 2006-2007, Fall 2007, Winter 2007-08) and elsewhere.

There are three methods that are commonly used to calculate the forces in segments of walls subjected to in-plane lateral loads:

1. The equivalent stiffness approach.
2. Elastic analysis.
3. Plastic analysis.

For comparison purposes, solutions will be determined for an example problem using the three methods.

## Equivalent Stiffness Method

The equivalent stiffness method is an approximate method that distributes the in-plane force to wall segments based on relative stiffness or rigidity. Openings increase a wall's flexibility and the approach is based on determining the deflection of the solid wall and increasing this deflection due to the effect of the openings. The wall rigidity is then calculated from the total wall deflection.

Since it as an approximate method, the equivalent stiffness approach should not be applied to walls with large openings or walls with configurations that require the wall assemblage be analyzed as a frame rather than as an individual wall.

Figure 1 illustrates the process of incorporating the effect of openings on the deflection of a wall. After the deflection of a solid cantilever wall is obtained, the deflection of a solid strip of wall equal to the height of the openings is subtracted and replaced by the deflection due to the piers around the openings. Thus the total wall deflection caused by a unit force on the wall is given by:

$$\Delta_{wall} = \Delta_{solid} - \Delta_{solid\ strip} + \Delta_{piers} \qquad (1)$$

and the relative rigidity of the wall is equal to:

$$R_{wall} = 1/\Delta_{wall} \qquad (2)$$

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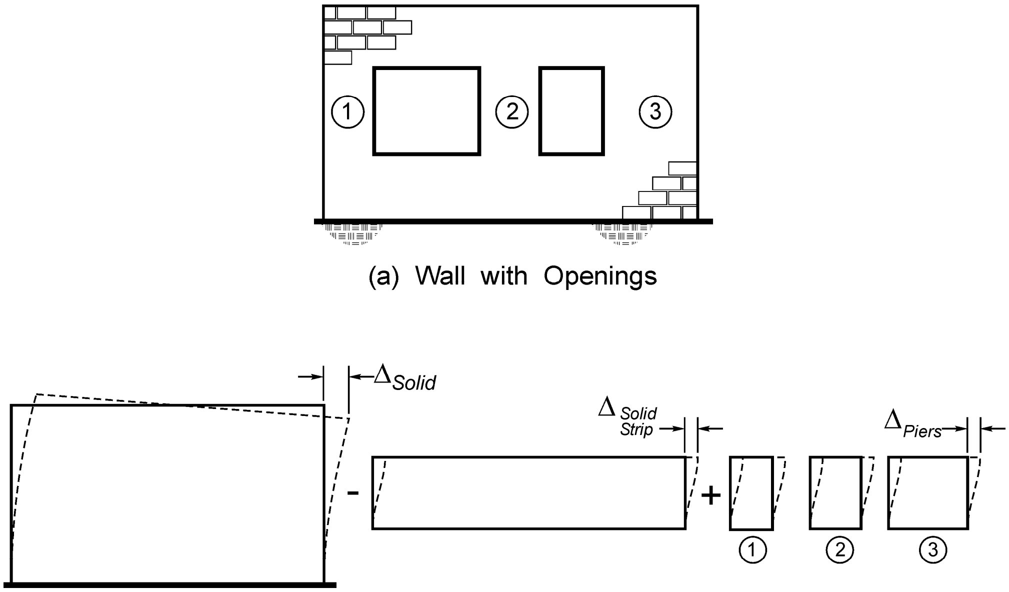
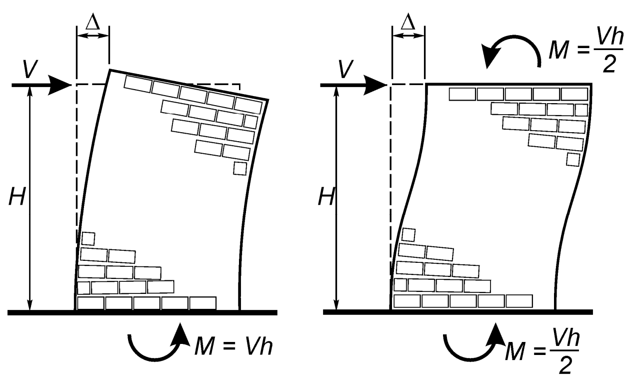


(a) Wall with Openings

(b) Deflection of Components

**Figure 1 – Deflection of Wall with Openings**

The deflection components are calculated using the basic strength of materials equations for fixed-fixed piers and fixed-free piers, depending on the boundary conditions, as shown in Figure 2. The deflections of the solid strip and pier are typically obtained assuming a fixed-fixed condition, since there is usually sufficient amount of wall above the openings to restrain rotation at the top of the piers.



(a) Cantilever Wall/Pier (b) Fixed-Fixed Pier

**Figure 2 – Deformation of Walls and Piers**

For fixed-free wall segments, the deflection is equal to:

$$\Delta = \frac{VH^3}{3E_m I} + \frac{1.2VH}{AE_v} \tag{3}$$

where $H$ is the height of the wall or pier and $L$ is the length. The first term represents bending or flexural deformation and the second term is the shear deformation. The bending term considers the wall as a simple vertical cantilever beam with a moment of inertia, $I$, which includes returns or pilasters at the ends of the wall. The cross-sectional area, $A$, is the area of the web and omits the flange areas. $E_m$ and $E_v$ are the Young's modulus and shear modulus, respectively, which are given in Section 1.8.2.2 of ACI 530-05/ASCE 5-05/TMS402-05 [1], also referred to as the 2005 Masonry Standards Joint Committee Building Code (MSJC), as:

$$E_m = 900 f'_m \tag{4}$$

$$E_v = 0.4E_m \tag{5}$$

where $f'_m$ is the masonry compressive strength. Assuming a unit lateral load that causes a deflection of $\bar{\Delta}$ is applied, Equations (3) and (5) can be used to obtain:

$$E_m \bar{\Delta} = \left(\frac{H^3}{3I}\right) + \left(\frac{3H}{A}\right) \tag{6}$$

For walls with rectangular cross-sections (no flanges) the cross-sectional area and moment of inertia are equal to:

$$A = tL \tag{7}$$

$$I = \frac{tL^3}{12} \tag{8}$$

Therefore, Equation (6) can be further simplified to:

$$tE_m\overline{\Delta} = 4\left(\frac{H}{L}\right)^3 + 3\left(\frac{H}{L}\right) \qquad (9)$$

It is important to note that when determining the distribution of earthquake loads, it is the relative rigidity of the walls that is required. Therefore, for walls with the same thickness the leading terms in Equation (9) can be ignored and the relative rigidity of each pier or wall segment is equal to:

$$R = \frac{1}{\Delta} = \frac{1}{\left[4\left(\frac{H}{L}\right)^3 + 3\left(\frac{H}{L}\right)\right]} \qquad (10)$$

The tops of masonry shear walls or piers are sometimes restrained by deep beams so that the deformation occurs with no rotation at the top of the wall, as shown in Figure 2(b). In order to obtain the deformed shape shown in Figure 2(b) the restraining beam must possess both the stiffness and strength to resist the moment that develops at the top of the wall. From beam theory, the deflection of a wall or pier that is prevented from rotating at the top is equal to:

$$\Delta = \frac{VH^3}{12E_mI} + \frac{1.2VH}{AE_v} \qquad (11)$$

Noting that $E_v$ = 0.4$E_m$ and substituting Equations (7) and (8) for the wall area and moment of inertia:

$$\Delta = \left[\left(\frac{H}{L}\right)^3 + 3\left(\frac{H}{L}\right)\right]\left(\frac{V}{tE_m}\right) \qquad (12)$$

Neglecting the common terms, the relative rigidity of "fixed-fixed" walls or piers is given by:

$$R = \frac{1}{\Delta} = \frac{1}{\left[\left(\frac{H}{L}\right)^3 + 3\left(\frac{H}{L}\right)\right]} \qquad (13)$$

Once the stiffness of each pier or wall segment is determined, the lateral force to the wall will be distributed to individual elements based on the relative rigidity:

$$V_i = \frac{R_i}{\sum_{i=1}^{n} R_i}V \qquad (14)$$

If the openings are at different elevations, this method becomes more complex as can be seen in the examples. It should be noted the procedure described above for obtaining the distribution of earthquake loads to shear walls is an approximate method at best. Calculations are based on elastic, uncracked masonry wall cross-sections. While it may be justifiable to use uncracked section properties for extremely low levels of loading, actual response of buildings during design level earthquakes will be extremely nonlinear and result in cracking of the masonry shear walls. Determining the stiffness of cracked reinforced masonry shear walls can be quite complex and the stiffness of a cracked masonry wall varies significantly depending on the degree of cracking. Thus, it is important to emphasize that the relative rigidity only provides an estimate of the distribution of earthquake loads and that the true distribution will not be completely predictable during a major earthquake. Nevertheless, a good design that utilizes walls of similar rigidity that are placed in a symmetrical pattern will have a more predictable seismic response behavior, even with the significant cracking expected during major earthquakes.

**Example 1**

Determine the distribution of the forces in the shear walls shown in Figure 3 using the equivalent stiffness approach. A 30 kip lateral force is applied at the top of the walls and a drag strut distributes the force to the two walls.

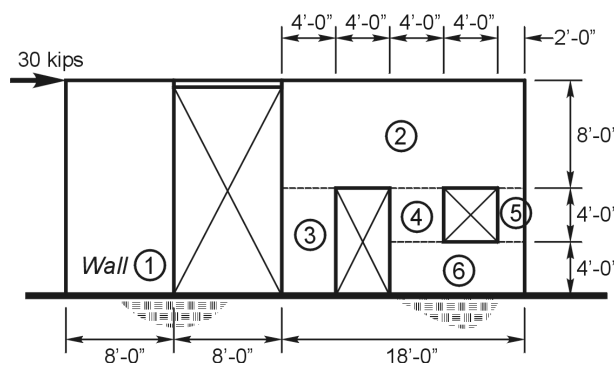



**Figure 3 – Example Walls**

**Solution:**

Determine the relative rigidity of Wall 1(fixed-free):

$$\Delta_1 = 4\left(\frac{H}{L}\right)^3 + 3\left(\frac{H}{L}\right)$$

$$= 4\left(\frac{16}{8}\right)^3 + 3\left(\frac{16}{8}\right) = 38.0$$

$$R_1 = \frac{1}{\Delta} = \frac{1}{38.0} = 0.026$$

Determine the relative rigidity of Wall 2 (fixed-free):

$$\Delta_{\substack{\text{Solid}\\\text{wall}}} = 4\left(\frac{16}{18}\right)^3 + 3\left(\frac{16}{18}\right) = 5.476$$

For the solid strip that contains piers 3, 4, 5 and 6 (fixed-fixed):

$$\Delta_{\substack{\text{Solid}\\\text{Strip}}} = \left(\frac{8}{18}\right)^3 + 3\left(\frac{8}{18}\right) = 1.421$$

For pier 3:

$$\Delta_3 = \left(\frac{8}{4}\right)^3 + 3\left(\frac{8}{4}\right) = 14.0;$$

$$R_3 = \frac{1}{14} = 0.071$$

For the solid strip that contains piers 4, 5 and 6:

$$\Delta_{\substack{solid\\456}} = \left(\frac{8}{10}\right)^3 + 3\left(\frac{8}{10}\right) = 2.912$$

and for piers 4 and 5:

$$\Delta_{\substack{solid\\strip}} = \left(\frac{4}{10}\right)^3 + 3\left(\frac{4}{10}\right) = 1.264$$

$$\Delta_4 = \left(\frac{4}{4}\right)^3 + 3\left(\frac{4}{4}\right) = 4.0;\ R_4 = \frac{1}{4.0} = 0.25$$

$$\Delta_5 = \left(\frac{4}{2}\right)^3 + 3\left(\frac{4}{2}\right) = 14.0;\ R_5 = \frac{1}{14.0} = 0.071$$

$$\Delta_{piers} = \frac{1}{\frac{1}{\Delta_4} + \frac{1}{\Delta_5}} = \frac{1}{\frac{1}{4.0} + \frac{1}{14.0}} = 3.115$$

Therefore the total deflection of piers 4, 5 and 6 is equal to:

$$\Delta_{456} = \Delta_{\substack{solid\\456}} - \Delta_{strip} + \Delta_{piers}$$

$$= 2.912 - 1.264 + 3.115 = 4.763$$

$$R_{456} = \frac{1}{\Delta} = 0.21$$

Similarly, for piers 3, 4, 5 and 6:

$$\Delta_{piers} = \frac{1}{\frac{1}{\Delta_3} + \frac{1}{\Delta_{456}}} = \frac{1}{\frac{1}{14.0} + \frac{1}{4.763}} = 3.559$$

and for the entire wall:

$$\Delta_2 = \Delta_{\substack{solid\\wall}} - \Delta_{\substack{solid\\strip}} + \Delta_{piers}$$

$$= 5.476 - 1.421 + 3.559 = 7.614$$

$$R_2 = \frac{1}{\Delta_2} = \frac{1}{7.614} = 0.131$$

The forces in each wall pier can then be calculated as follows:

$$F_1 = \frac{R_1}{R_1 + R_2} \times 30$$

$$= \frac{0.026}{0.026 + 0.131} \times 30 = 4.97 \text{ kips}$$

$$F_2 = \frac{R_2}{R_1 + R_2} \times 30$$

$$= \frac{0.131}{0.026 + 0.131} \times 30 = 25.03 \text{ kips}$$

$$F_3 = \frac{R_3}{R_3 + R_{456}} \times 25.03$$

$$= \frac{0.071}{0.21 + 0.071} \times 25.03 = 6.32 \text{ kips}$$

$$F_{4-5-6} = 25.03 - 6.32 = 18.71 \text{ kips}$$

$$F_4 = \frac{R_4}{R_4 + R_5} \times 18.71$$

$$= \frac{0.25}{0.25 + 0.071} \times 18.71 = 14.57 \text{ kips}$$

$$F_5 = 18.71 - 14.57 = 4.14 \text{ kips}$$

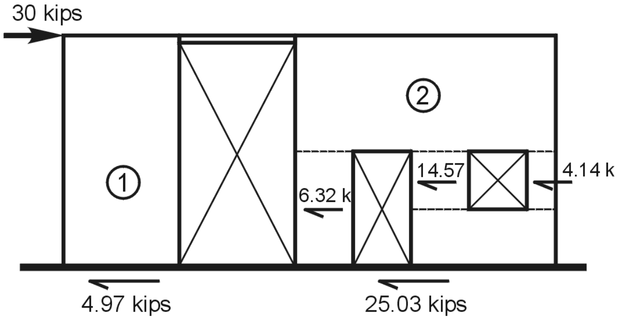



**Figure 4 – Solution to Example 1**

## Elastic Analysis

An elastic analysis of walls with openings can be performed to determine the forces in each of the wall segments. This typically involves the use of a finite element analysis program that can model a wall with membrane or shell elements in order to adequately capture the behavior of the wall and the effect of openings. The finite element grid must be fine enough to capture the flexural and shear deformation components of deflection.

## Example 2

Determine the distribution of the forces in the shear walls shown in Figure 3 using the elastic analysis.

### Solution:

The walls shown in Figure 3 were modeled using the computer program SAP 2000® [2]. The computer model is shown in Figure 5 and the deformed shape due to the lateral load is shown in Figure 6. Figure 7 shows the forces in the various wall segments that resulted from the computer analysis.

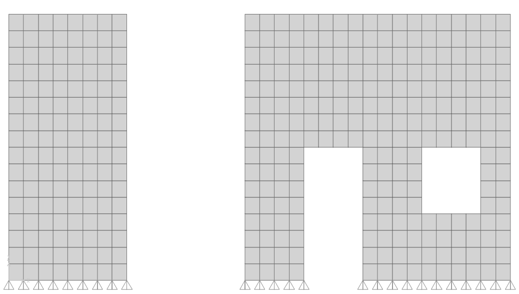


**Figure 5 – Finite Element Model for Example 2**

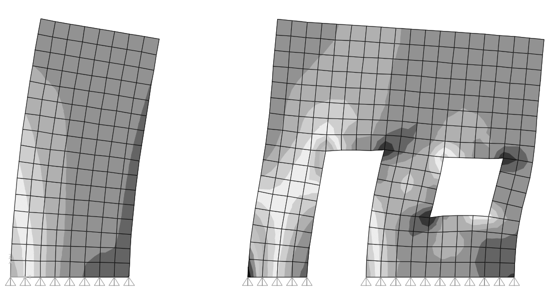


**Figure 6 – Deformed Shape of Walls**

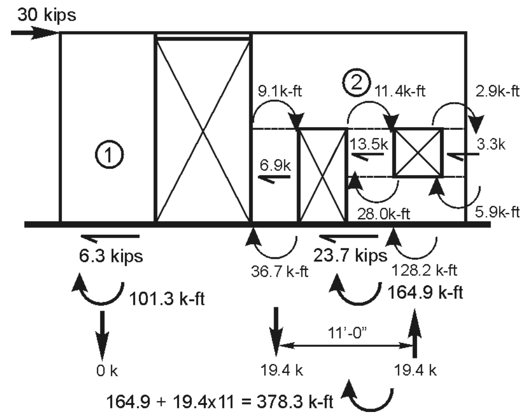



**Figure 7 – Solution to Example 2**

Comparisons of Figures 4 and 7 indicate that there are significant differences in the results obtained from the equivalent stiffness method and the computer analysis. The differences are highlighted by the shear force in Wall 1, which increases by about 27% when a finite element analysis is performed. In addition, the equivalent stiffness approach does not provide information on the axial loads that occur in the walls segments due to the overturning effect of the lateral load.

A primary reason for the difference in results is the assumption that wall segments are either perfectly "fixed-fixed" or "fixed-free" when using the equivalent stiffness approach. In reality, the boundary conditions at the ends of wall segments may be significantly different from these idealized assumptions, and the coupling effect of the horizontal wall segments results in axial loads that depend on the relative stiffness. This often results in an overestimation of the stiffness of walls with openings. Computer models allow the designer to incorporate the rotation at the ends of piers and the coupling effect of horizontal wall segments.

## Plastic Analysis

The elastic analysis computer analysis clearly provides better results than the equivalent stiffness method. However, since response during large earthquakes occurs in the nonlinear range of structural response, an elastic analysis is still an approximation and may not always represent the true nonlinear response of a concrete masonry wall with openings.

Plastic analysis of masonry walls has been suggested in various forms, for several years [3, 4]. Terms such as limit analysis, limit state design and displacement analysis are also often used to describe the procedure. A primary advantage of plastic or limit state analysis is the engineer's ability to dictate building performance during seismic events without extensive analysis. The engineer selects a plastic mechanism that defines the wall behavior and then verifies that the various wall segments have sufficient ductility to incur deformation demands that correspond to the selected mechanism. This typically involves design to avoid brittle failure modes such as shear failures, and a verification that the compressive strains are within acceptable limits at the maximum displacement.

This article only addresses some basic aspects of plastic design of masonry walls as a detailed discussion of the method and different approaches would require a more extensive discussion. The Masonry Standards Joint Committee is currently working on developing standardized procedures for performing plastic design on masonry walls.

**Example 3**

Determine the distribution of the forces in the shear walls shown in Figure 3 using the plastic analysis.

**Solution:**

The first step is the determination of a mechanism for the wall. While mechanisms with plastic hinges in the beams are preferred, this is not always possible when the configuration of openings results in deep beams and narrow piers. Figure 8 shows the selected mechanism for the example problem. As can be seen, pier mechanisms with plastic hinges in the piers can often create large deformation demands in the piers.

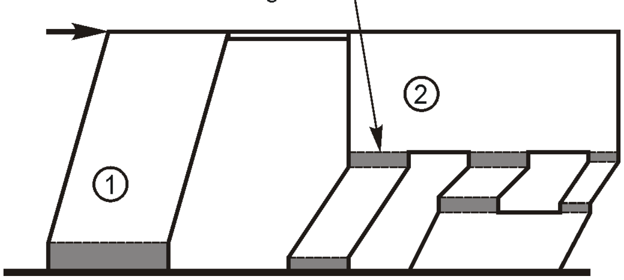



**Figure 8 – Plastic Mechanism for Example 3**

The next step is the distribution of wall loads to the wall segments. Theoretically, any distribution of forces that satisfies equilibrium and is compatible with the selected mechanism may be used. However, it is best to distribute the loads by taking into consideration the potential force and deformation capacity of the various wall segments.

For this example, the total shear force is distributed to the piers and walls segments in proportion to their lengths. The resulting forces are shown in Figure 9.

Note that since Wall 1 is not attached to the remainder of the assembly by a coupling beam, it does not experience any axial load. Wall segment 2 acts as a coupling beam between the piers and induces axial loads in addition to the shear and flexural forces on the piers.

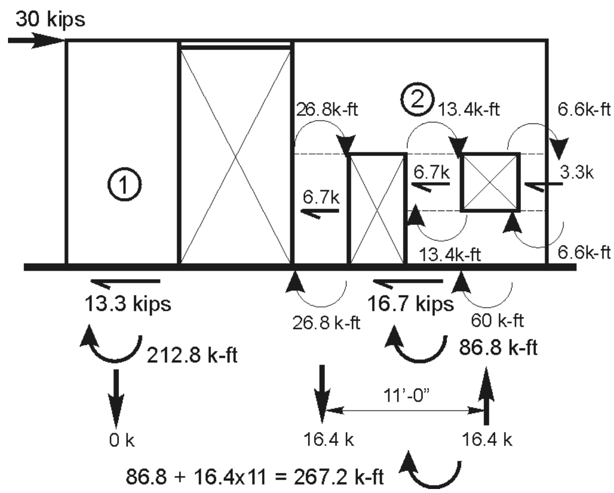



**Figure 9 – Solution to Example 3**

It should be noted that while the plastic mechanism defines the loads on each plastic hinge at the maximum wall displacement (ultimate limit state), each hinge does not occur at the same time. This means that the inelastic demands on some wall segments may be significantly different from others. To ensure that the wall has adequate ductility, the deformation capacity of each pier and walls segment must be verified.

A simplified approach is to conservatively ignore the elastic deformation of the wall and assume that all displacement is as a result of plastic rotation. Then the plastic rotation demand, $\theta_{pu}$, is given by:

$$\theta_{pu} \frac{\Delta_u}{(H' - 0.5H_p)} \qquad (15)$$

where $H'$ is height of the equivalent cantilever (half the height of piers subjected to double curvature) and $H_p$ is the plastic hinge length, which is usually taken as half the length of the wall segment. $\Delta_u$ is the maximum displacement of the wall calculated from the analysis of the structure. The plastic rotation capacity is based on the maximum usable compressive strain in the masonry:

$$\theta_{pn} = \frac{\varepsilon_{mu}}{c} H_p \qquad (16)$$

where the maximum usable compressive strain, $\varepsilon_{mu}$, is equal to 0.0025 for concrete masonry and $c$ is the depth of the neutral axis corresponding to the maximum usable compressive strain.

## Design and Detailing of Walls with Openings

Irrespective of the method used to determine the forces in the walls, piers and wall segments, the walls must be designed to satisfy the requirements of the International Building Code (IBC) [5] and the MSJC code [1]. Gravity loads must be combined with lateral loads using the appropriate load combinations.

Section 3.1.3.1 of the MSJC code states that at each story level and line of resistance of buildings assigned to seismic design category C and greater, at least 80 percent of the lateral stiffness must be provided by lateral-force-resisting walls. Piers and columns may be used to provide earthquake load resistance if a response modification factor, *R*, of no greater than 1.5 is used to calculate earthquake loads.

One interpretation of this stipulation is that if all piers satisfy the design and detailing requirements for shear walls required in a seismic design category, the values of *R* for shear walls may be used (i.e. 3.5 for intermediate reinforced masonry shear walls and 5 for special reinforced shear walls in bearing wall systems). This means that in addition to other detailing requirements, the piers must satisfy the requirements for minimum and maximum reinforcement, reinforcement spacing and the amplified shear demands on walls in seismic regions. This will ensure that the wall segments possess sufficient ductility to justify a use of the selected response modification factor. It is recommended that strength design procedures are used to design masonry shear walls with openings since this provides a more accurate determination of the capacity of wall segments subjected to earthquake loading.

If the wall design and detailing requirements cannot be satisfied, and this is often the case with walls that have short squat piers, a response modification factor of 1.5 should be used and the wall must satisfy the less stringent pier detailing requirements, some of which are as follows (MSJC Section 3.3.4.3):

a. One bar shall be provided in the end cells.
b. Minimum area of longitudinal reinforcement shall Be 0.0007*bd.*
c. Longitudinal reinforcement shall be uniformly Distributed throughout the depth of the element.

## Conclusions

Various methods have been presented for analyzing concrete masonry with openings that are subjected to in-plane lateral loads. The equivalent stiffness approach, while easily performed with hand calculations, is extremely approximate and can often lead to non-conservative designs. The equivalent stiffness approach also does not provide information on the axial loads resulting from the coupling effect of beams in walls. This is a significant shortcoming since the strength and deformation capacity of concrete masonry elements is often highly dependent on axial load.

Elastic analysis with computer programs overcome several of the limitations of the equivalent stiffness approach and assist in the design of safe, earthquake resistant structures. Plastic analysis can provide the engineer with the ability to control the earthquake response of walls using capacity design procedures. Standardized procedures for plastic design of masonry walls are currently being developed.

## References

[1] Masonry Standards Joint Committee (MSJC), Building Code Requirements for Masonry Structures, Masonry Standards Joint Committee, Boulder, Colorado, 2005.

[2] Computers and Structures, Inc, *SAP 2000 User's Manual*, Berkeley California, 2008.

[3] Leiva, G., and Klingner, R., Technical Coordinating Committee for Masonry Research (TCCMaR) Report No. 3.1(c)-2: In-plane Seismic Resistance of Two-story Concrete Masonry Shear Walls with Openings, 1991.

[4] Drysdale, R. G., Hamid, A. A., *Masonry Structures: Behavior and Design*, 3rd edition, The Masonry Society, Boulder, CO 2005.

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