# Code Approaches to Seismic Design of Masonry-Infilled Reinforced Concrete Frames: A State-of-the-Art Review

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Masonry infill (MI) walls are remarkable in increasing the initial stiffness of reinforced concrete (RC) frames, and being the stiffer component, attract most of the lateral seismic shear forces on buildings, thereby reducing the demand on the RC frame members. However, behavior of MI is difficult to predict because of significant variations in material properties and because of failure modes that are brittle in nature. As a result, MI walls have often been treated as nonstructural elements in buildings, and their effects are not included in the analysis and design procedure. However, experience shows that MI may have significant positive or negative effects on the global behavior of buildings and, therefore, should be addressed appropriately. Various national codes differ greatly in the manner effects of MI are to be considered in the design process from aseismic performance point of view. This paper reviews and compares analysis and design provisions related to MI-RC frames in seismic design codes of 16 countries and identifies important issues that should be addressed by a typical model code. [DOI: 10.1193/1.2360907]

## INTRODUCTION

Masonry infill (MI) walls confined by reinforced concrete (RC) frames on all four sides play a vital role in resisting the lateral seismic loads on buildings. It has been shown experimentally that MI walls have a very high initial lateral stiffness and low deformability (Moghaddam and Dowling 1987). Thus introduction of MI in RC frames changes the lateral-load transfer mechanism of the structure from predominant frame action to predominant truss action (Murty and Jain 2000), as shown in Figure 1, which is responsible for reduction in bending moments and increase in axial forces in the frame members. In addition, construction of MI is cheaper because it uses locally available material and labor skills. Moreover, it has good sound and heat insulation and waterproofing properties, resulting in greater occupant comforts and economy.

Buildings can become *irregular* in plan and elevation because of uncertain position of MI walls and openings in them. Often MI walls are rearranged to suit the changing

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*Earthquake Spectra,* Volume 22, No. 4, pages 961–983, November 2006; © 2006, Earthquake Engineering Research Institute

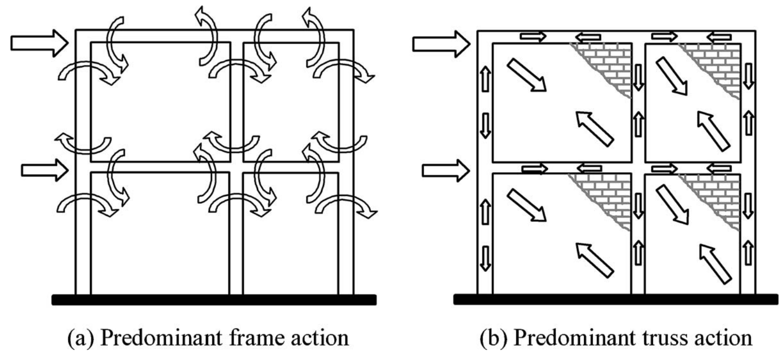


**Figure 1.** Change in lateral-load transfer mechanism due to masonry infills (Murty and Jain 2000).

functional needs of the occupants, the changes being carried out without considering their adverse effects on the overall structural behavior because MI walls are generally regarded as nonstructural elements of buildings. MI can be distributed in RC frames in several patterns, for example, as shown in Figure 2. Thus it is not only difficult to construct a *regular* MI-RC frame building, but also it cannot be taken for granted that it will remain *regular* after it is constructed.

As already discussed, MI is used extensively in RC buildings in several countries worldwide and have significant effect on their behavior during earthquakes. It is important to know how national codes of various countries control behavior and design of MI-RC frames. This paper aims at reviewing and comparing the seismic codes of 16 countries from different parts of the world having provisions for design of MI-RC frames. In addition to these national codes, *FEMA-306* (ATC 1999) is included in the study because of its comprehensive treatment of MI in RC frames. *FEMA-306* is not a

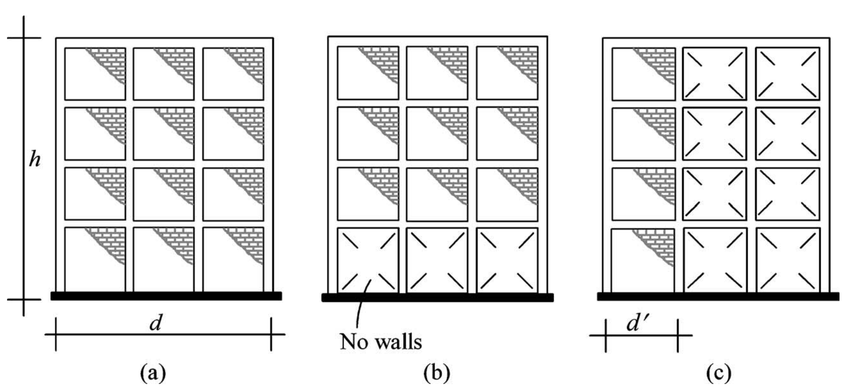


**Figure 2.** Different arrangements of masonry infill walls in RC frame.

code of standard practice of any country and it is intended for evaluation of earthquake-damaged buildings. Besides the national codes, Paz (1994) and IAEE (2004) were also studied. The present documentation would certainly help the design engineers, code developers, and researchers working on the behavior, analysis, and design of MI-RC frame buildings.

## COMPARISON OF NATIONAL CODES

Various national codes can be broadly grouped in two categories of those that consider or do not consider the role of MI walls while designing RC frames. A very few codes specifically recommend isolating the MI from the RC frames such that the stiffness of MI does not play any role in the overall stiffness of the frame (NZS-3101 1995, SNIP-II-7-81 1996). As a result, MI walls are not considered in the analysis and design procedure. The isolation helps to prevent the problems associated with the brittle behavior and asymmetric placement of MI.

Another group of national codes prefers to take advantage of certain characteristics of MI walls such as high initial lateral stiffness, cost-effectiveness, and ease in construction. These codes require that the beneficial effects of MI are appropriately included in the analysis and design procedure and that the detrimental effects are mitigated. In other words, these codes tend to maximize the role of MI as a first line of defense against seismic actions, and to minimize their potential detrimental effects through proper selection of their layout and quality control.

A list of all codes discussed in the paper, along with the key features used for the comparison, is shown in Table 1. It is observed that some provisions are quite similar in most of the national codes, which is expected as the code-writing committees are usually familiar with all the existing codes, a fact that has been noted in similar studies (Luft 1989). The seismic force–resisting-systems in the *International Building Code* (*IBC*) (ICC 2003) have several systems with RC frames and masonry for new buildings; however, unreinforced masonry as infill is not permitted. In the following sections, some of the important issues related to the combined effect of MI and RC frames discussed in these codes are compared and reviewed.

### METHOD OF ANALYSIS

Most national codes recognize that structures with simple and *regular* geometry perform well during earthquakes, and unsymmetrical placement of MI walls may introduce irregularities into them. These codes permit static analysis methods for *regular* short buildings located in regions of low seismicity. However, for other buildings (Table 1), dynamic analyses are recommended, in which it is generally expected but not specifically required that all components imparting mass and stiffness to the structure are adequately modeled. Most codes restrict the use of seismic design force obtained from dynamic analysis such that it does not differ greatly from a minimum value that is based on the code-prescribed empirical estimate of natural period. This restriction prevents the design of buildings for unreasonably low forces that may result from various uncertainties involved in a dynamic analysis.

**Table 1.** Summary of contents of national codes on masonry infilled RC frames

| Country/Code | D[1] | $T_a$[2] | Min. design force (%) Frame | Min. design force (%) Infill | Irregularity Plan | Irregularity Elev. | $K$[3] | Drift | Infill $\sigma_i$[4] | Infill $K_i$[4] | Infill O[4] | Out-of-Plane |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Albania (1989) | Y | Y | × | × | × | × | 1.2–1.5 | × | × | × | × | × |
| Algeria (1988) | Y | Y | 25 | × | × | × | 1.42 | × | × | × | × | × |
| Bulgaria (1987) | Y | × | × | × | × | Y | 1.5–3.0[5] | × | × | × | × | × |
| China (GBJ-11-89 1989) | Y | × | × | × | × | × | × | Y | × | × | × | × |
| Columbia (NSR-98 1998) | Y | Y | 25 | 100 | × | × | × | Y | × | × | × | × |
| Costa Rica (1986) | Y | Y | × | × | Y | Y | × | Y | × | × | × | × |
| Egypt (1988) | Y | Y | 25 | 100 | × | × | 2.0 | × | × | × | × | × |
| Ethiopia (ESCP-1 1983) | Y | Y | 25 | 100 | × | × | 1.25 | × | × | × | × | × |
| Eurocode 8 (2003) | Y | Y | 50–65 | × | Y | Y | 1.2 | Y | × | × | Y | Y |
| France (AFPS-90 1990) | Y | Y | × | × | × | × | × | × | × | × | × | × |
| USA (IBC 2003) | × | × | × | × | × | × | × | × | × | × | × | × |
| India (IS-1893 2002) | Y | Y | × | × | × | Y | × | × | × | × | × | × |
| Israel (SI-413 1995) | Y | Y | 25 | × | Y | Y | 1.15 | × | Y | × | × | × |
| Nepal (NBC-105, 201 1995) | Y | Y | 25 | × | Y | Y | 2.0 | Y | Y | × | Y | Y |
| Philippines (NSCP 1992) | Y | Y | × | × | × | × | 1.5 | × | × | × | × | × |
| Venezuela (1988) | Y | Y | 25 | × | × | × | × | × | × | × | × | × |
| *FEMA-306*[6] | Y | × | × | × | × | × | × | Y | Y | Y | Y | Y |

[1] Dynamic analysis is required for *irregular* buildings, tall buildings, important buildings, and buildings located in high seismic regions. The specific requirements vary among different codes.
[2] $T_a$ is the fundamental natural period of vibration for MI-RC frames.
[3] $K$ is the ratio of seismic design forces for MI-RC frames to that for the RC frames without MI due to the difference in response reduction factor.
[4] $\sigma_i$, and $K_i$ are the strength and stiffness of MI, respectively, and O is the openings in MI.
[5] Response coefficient for the soft-story buildings are required to be increased by two times the value for *regular* buildings with MI and three times the value for buildings without MI.
[6] *FEMA-306* (ATC 1999) is not a code of standard practice of any country.

Nepal code (NBC-201 1995) adopts a different analysis procedure in which axial forces in the frame members are estimated by assuming a pin-jointed frame and representing MI by compression diagonal struts. A method of distributing the lateral shear force on various MI walls in a story is specified in the code, which depends upon the seismic base shear on the frame and cross-sectional and material properties of MI and RC frame members.

## EMPIRICAL FORMULAE FOR NATURAL PERIOD

Natural periods of vibration of buildings depend upon their mass and lateral stiffness. Presence of non-isolated MI walls in buildings increases both the mass and stiffness of buildings; however, the contribution of latter is more significant. Consequently, the natural periods of an MI-RC frame are normally lower than that of the corresponding bare frame. Therefore, the seismic design forces for MI frames are generally higher than those for the bare frames. Although, all national codes explicitly specify empirical formulae for the fundamental natural period calculations of bare RC frames, only a few specify the formulae for MI-RC frames.

Several codes—IS-1893 (2002); NBC-105 (1995); NSR-98 (1998); Egyptian code (1988); Venezuelan code (1988); Algerian code (1988); ESCP-1 (1983)—suggest using an empirical formula given by Equation 1 to calculate the natural period of MI-RC frames, $T_a$.

$$T_a = \frac{0.09h}{\sqrt{d}} \quad \text{units}\begin{Bmatrix} T_a \text{ in s} \\ h, d \text{ in m} \end{Bmatrix} \tag{1}$$

where $h$ is the height of the building and $d$ the base dimension of building at the plinth level along the considered direction of the lateral force.

For $T_a$ estimation, French code (AFPS-90 1990) recommends using the most unfavorable of Equation 1 and the following equation that is specified for masonry buildings:

$$T = 0.06\frac{h}{\sqrt{d}}\sqrt{\frac{h}{2d+h}} \quad \text{units:}\{T \text{ in s}\} \tag{2}$$

In Equations 1 and 2, total base width of buildings is used to calculate $T_a$, which may not be appropriate. For example, $d$ will be equal to the total base dimension for all the frames in Figure 2 irrespective of the distribution of MI in the frame. However, for frame in Figure 2c, it is more appropriate to consider $d'$ as the effective base width, rather than total width $d$ of the building. Therefore, Equations 1 and 2 may not estimate correct $T_a$ values for different frames shown in Figure 2.

Empirical formula suggested by the Costa Rican code (1986) for MI-RC frame buildings is given by

$$T_a = 0.08N \tag{3}$$

where $N$ is number of stories in the building. A flat 20% reduction from that of bare frame ($T_a$=0.1$N$) is specified to account for the increased stiffness of frames due to presence of MI.

According to the Israeli seismic code (SI-413 1995), $T_a$ is determined as follows:

$$T_a = 0.049h^{0.75} \tag{4}$$

In addition, Israeli code recommends that natural period calculated by any structural dynamics method shall not be larger than the following:

$$T_a = 0.068h^{0.75} \tag{5}$$

This requirement ensures that the seismic design base shear is not less than 80% of the base shear determined using the period obtained by the empirical relation given by Equation 4.

According to the Algerian code (1988), $T_a$ is taken as the smaller value between the values given by Equation 1 and the following expression:

$$T_a = 0.05h^{0.75} \tag{6}$$

According to the empirical Equations 1–6 of various codes, $T_a$ is not a function of amount of infills in different stories and their distribution along the height. Therefore, Equations 1–6, when used independently, will estimate same $T_a$ values for different frames shown in Figure 2. A few other codes (*Eurocode 8* 2003, NSR-98 1998, and NSCP 1992) recommend estimating $T_a$ using a more realistic approach, which is discussed below.

*Eurocode 8* (2003) recommends the following equations for buildings up to 40 m high:

$$T_a = C_t h^{0.75} \tag{7}$$

where

$$C_t = \frac{0.075}{\sqrt{A_c}} \tag{8}$$

and

$$A_c = \sum A_i \left(0.2 + \frac{l_{wi}}{h}\right)^2; \frac{l_{wi}}{h} \leqslant 0.9 \quad \text{units:} \begin{Bmatrix} A_c, A_i & \text{in} & \text{m}^2 \\ l_{wi} & \text{in} & \text{m} \end{Bmatrix} \tag{9}$$

where $C_t$ is the correction factor for MI, which is more for a flexible building, $A_c$ is the combined effective area of MI in the first story, $A_i$ is the effective cross-sectional area of wall $i$ in the first story, and $l_{wi}$ is length of the wall $i$ in the first story in the considered direction.

An older version of the Columbian code (NSR-84 1984) recommended Equation 1 for estimating $T_a$ with the following expression for $d$ that includes distribution of MI in frames:

$$d = d_{s\,\max} \sum_{s=1}^{N_S} \left(\frac{d_s}{d_{s\,\max}}\right)^2 \quad \text{units:}\{d, d_s, d_{s\,\max} \text{ in m}\} \tag{10}$$

where $d_s$ is the length of a segment of wall, $d_{s\,max}$ is the length of the largest segment of wall, and $N_s$ is the number of segments of wall in the considered direction. Since Equation 10 considers the amount of MI present in the frame, $T_a$ can be estimated with better accuracy than that given by Equation 1. In the latest version of this code (NSR-98 1998), Equations 7 and 8 are recommended for $T_a$ calculations with $C_t$ restricted to a maximum value of 0.07 and a new formula for $A_c$ as given below:

$$A_c = \sum A_i\left(0.02 + \left(\frac{l_{wi}}{h}\right)^2\right); \quad \frac{l_{wi}}{h} \leqslant 0.9 \tag{11}$$

The Philippine code (NSCP 1992) has specified Equation 7 for $T_a$, where $C_t$ is taken as

$$C_t = \frac{0.03048}{\sqrt{A_c}} \tag{12}$$

and where $A_c$ is defined by the following expression:

$$A_c = \sum A_i\left(0.2 + \left(\frac{l_{wi}}{h}\right)^2\right); \quad \frac{l_{wi}}{h} \leqslant 0.9 \tag{13}$$

Ratio $l_{wi}/h$ can become very large for squat-type buildings in which the length of a building is large in comparison to its height. Therefore, an upper limit of 0.9 on $l_{wi}/h$ is specified in Equations 9, 11, and 13 to prevent computation of unrealistically larger values of $A_c$. There is no upper limit on $C_t$ in Equations 8 and 12 recommended by *Eurocode 8* and Philippine code, respectively. $C_t$ can become unrealistically high for open first-story buildings when $A_c$=0 (Figure 2b). This may imply that open first-story buildings are not permitted by *Eurocode 8* and Philippine code; however, it is not clearly mentioned in these codes.

Amount of MI in the first story greatly influences $T_a$, while MI in the upper stories simply adds to the total mass of frames, and its contribution to the overall stiffness is considerably smaller. Equation 7 used by *Eurocode 8*, Columbian code, and Philippine code require details of MI only in the first story. Consequently, this $T_a$ estimation may be more accurate when compared with other empirical equations; however, Equation 7 is not valid for an open first-story frame unless there is an upper limit on $C_t$, as specified in the Columbian code.

All the above empirical equations for $T_a$ have certain limitations; therefore, a few codes (*Eurocode 8* 2003; NSR-98 1998; Costa Rican code 1986; Venezuelan code 1988; NSCP 1992; Algerian code 1988) recommend the use of Rayleigh formula for $T_a$ calculations:

$$T_a = 2\pi\sqrt{\frac{\sum_{i=1}^{N} W_i\delta_{ei}^2}{g\sum_{i=1}^{N} F_i\delta_{ei}}} \quad \text{units:}\left\{\begin{array}{ll} W_i & \text{in kg} \\ F_i & \text{in N} \\ g & \text{in m/s}^2 \\ \delta_{ei} & \text{in m} \end{array}\right\} \tag{14}$$

where $W_i$, $\delta_{ei}$, and $F_i$ are the seismic weight, elastic displacement, and seismic force, respectively, at level $i$. $\delta_{ei}$ is calculated in the first cycle of analysis using $T_a$ from Equation 1 or by any other empirical formulae. A modified Rayleigh formula is also proposed by *Eurocode 8* and Algerian code to estimate $T_a$:

$$T_a = 2\sqrt{\Delta} \quad \text{units:}\{\Delta \text{ in m}\} \tag{15}$$

where $\Delta$ is the lateral displacement at the top of the building due to the gravity loads applied horizontally; as a result, $\Delta$ depends upon the distribution of MI in the building frame. Rayleigh formula is based on the method of structural dynamics, which includes the mass and stiffness of all structural members. The codes generally require $T_a$ given by Rayleigh formula not to exceed by more than 20–30% of values obtained from empirical formulae.

In reality, empirical $T_a$ may be more reliable than $T_a$ computed using methods of structural dynamics, because there are considerable uncertainties in modeling a building for dynamic analysis, such as stiffness contribution of nonstructural elements and MI, modulus of elasticity of concrete and masonry materials, and area and moment of inertia of participating structural members. These uncertainties may give rise to unduly large natural periods and result in lower design seismic forces. Therefore, most of the national codes have put an upper limit on $T_a$ values obtained by Rayleigh method to safeguard against unrealistically lower values of design seismic forces.

## LATERAL LOAD SHARING BETWEEN INFILL AND FRAME

The combined behavior of MI-RC frames is such that the total seismic design force is resisted in proportion to the lateral stiffnesses of the RC frame and MI walls at all story levels. MI walls, which are normally very stiff initially, attract most of the lateral forces, but may fail prematurely because of the brittle behavior. In such cases, RC frames must have sufficient backup strength to avoid the collapse of the structure.

*Eurocode 8* (2003) requires the RC frames to resist full vertical loads and at least 50–65% of the total lateral loads on buildings. The Columbian (NSR-98 1998), Egyptian (1988), and Ethiopian (ESCP-1 1983) codes also require that MI should resist full design lateral seismic loads without any assistance from the RC frame. In such cases, provisions must be made to structurally connect MI walls to the surrounding RC frame. The Algerian code (1988) requires MI walls to carry at the most 20% of the total vertical loads of the building.

According to most codes, MI is not expected to carry any gravity loads other than its self-weight. The contribution of MI in resisting the lateral loads can be substantial. However, to safeguard against an RC frame being designed for a very low seismic force, the frame alone is required to be designed to independently resist at least 25% of the design seismic forces in addition to the forces due to vertical loads (Table 1).

## PLAN IRREGULARITIES

Plan irregularities are introduced into buildings because of asymmetric placement of MI walls, thus increasing shear demand in RC frame members, especially columns. Although, national codes mention torsional irregularity, only a few address the problem in the context of MI, e.g., *Eurocode 8* (2003), NBC-201 (1995), Costa Rican code (1986), and SI-413 (1995).

According to *Eurocode 8* (2003), slight plan irregularities may be taken into account by doubling the accidental eccentricity. In case of severe plan irregularities due to excessive unsymmetrical placement of MI walls, three-dimensional analysis is required considering stiffness distribution related to the uncertain position of MI. In addition, a sensitivity analysis is required for the position and properties of MI by disregarding one out of three or four MI panels in a planar frame, especially on the more flexible sides. Accidental eccentricity is assumed to take care of approximations in computation of eccentricity, possible relocation of the center of mass due to changes in its usage during service life of buildings, and probable torsional component of ground motion.

In the Nepal code (NBC-201 1995), eccentricity between center of mass and center of rigidity along each principal direction is limited to 10% of the building dimension along that direction. The above requirement may be satisfied by adjusting thicknesses of walls. According to the Costa Rican code (1986), eccentricity in each direction must not exceed 5% of the total dimension in that direction. Maximum allowed eccentricity for *irregular* structures is limited to 30% of the plan dimension in any of the directions. According to the Israeli code (SI-413 1995), eccentricity in each direction is restricted to 10% of the building dimension along that direction. If this condition is not satisfied, center of rigidity shall be calculated including the stiffness of MI.

A few codes take cognizance of the added torsional forces that may develop in the frame members due to plan irregularity introduced by asymmetrical placement of MI walls. In such cases, the codes have put a restriction on the amount of plan eccentricity that a building can have, because the effect of eccentricity is greater under dynamic conditions than that is calculated for static conditions.

## VERTICAL IRREGULARITIES

Vertical irregularities are introduced into MI-RC frames due to reduction or absence of MI in a particular story compared to adjacent stories, e.g., buildings with parking space in the first story and MI on upper stories. In general, this gives rise to mass, stiffness, and strength irregularities along height of buildings. Vertical irregularities in the bottom stories make the beams and columns of those stories more susceptible to damage or failure. A few national codes penalize beams and/or columns of the *irregular* stories, as they are required to be designed for higher seismic forces to compensate for the reduction in the strength due to absence of MI in the *irregular* stories.

The Indian seismic code (IS-1893 2002) requires members of the soft story (story stiffness less than 70% of that in the story above or less than 80% of the average lateral stiffness of the three stories above) to be designed for 2.5 times the seismic story shears and moments, obtained without considering the effects of MI in any story. The factor of 2.5 is specified for all the buildings with soft stories irrespective of the extent of irregularities; and the method is quite empirical. The other option is to provide symmetric RC shear walls, designed for 1.5 times the design story shear force in both directions of the building as far away from the center of the building as feasible. In this case, the columns can be designed for the calculated story shears and moments without considering the effects of MI. Alternatively, the code requires nonlinear dynamic analysis of such buildings considering the mass and stiffness of MI and inelastic deformations in all structural members.

*Eurocode 8* (2003) recommends increasing the resistance of columns in the less-infilled story in proportion to the amount of deficit in strength of MI. In the older version of *Eurocode 8* (1996), increase in design forces was sought in beams and columns of the concerned story. However, further research (Fardis and Panagiotakos 1997) has shown that increasing the beam resistance would further increase the seismic demands on the columns, thus seismic design forces in only columns are increased by a factor $\eta$ given by

$$\eta = 1 + \frac{\Delta V_{RW}}{\sum V_{Ed}} \leqslant q \quad \text{units:} \begin{Bmatrix} \Delta V_{RW} & \text{in N} \\ V_{ED} & \text{in N} \end{Bmatrix} \tag{16}$$

where $\Delta V_{RW}$ is the total reduction in lateral resistance of MI in a story compared to the story above, and $\Sigma V_{Ed}$ is the sum of seismic shear forces acting on all structural vertical elements of the story concerned. The behavior factor, $q$, which accounts for the energy dissipation capacity of the structure, varies from a minimum value of 1.5 to 4.68 depending upon the building systems, ductility classes, and plan regularity in the building. The design forces are not required to be increased if the factor $\eta$ is less than 1.1.

Maximum vertical irregularities allowed by *Eurocode 8* (2003) in buildings are such that $\eta$ is never more than 4.68, which is larger than the factor 2.5 given in the Indian code (IS-1893 2002). Also, $\eta$ is applied only to columns of the soft story, whereas in the Indian code, both beams and columns of the soft story are required to be designed for increased forces. *Eurocode 8* (2003) does not clearly mention whether the buildings with open first story are permitted; it only restricts the value of $\eta$.

*Eurocode 8* (2003) requires adequate confinement in the form of shear reinforcement along the full height of the first-story columns because of the particular vulnerability of MI in the first story. Confinement along full column height is also required in case of frames containing MI with partial heights to reduce damage due to the short column effect. Similar confinement is also needed if MI walls are present on only one side of a column along a particular direction (e.g., corner columns).

According to the Bulgarian code (1987), members of the soft stories (story stiffness less than half the stiffness of the adjacent stories) are required to be designed for increased forces by introducing a coefficient while calculating the design forces. The value of coefficient for *regular* RC frames with MI is 0.3 as compared to a value of 0.2 for the bare frames, and the coefficient for the RC frames with a soft story is 0.6. Therefore, the soft-story members are required to be designed for three times the design seismic forces for corresponding *regular* bare frames (Table 1). For buildings with asymmetrically distributed masses, the code requires that analyses be done for the most unfavorable direction of the seismic excitation.

Costa Rican code (1986) requires that all structural-resisting systems must be continuous from the foundation to the top of buildings, and stiffness of a story must not be less than 50% of that of the story below. Also, the weight of two adjacent stories must not differ by more than 15%, except at the roof level and at those stories located in the first 20% of the height of tall buildings. These clauses are intended to help reduce the adverse effects of the vertical irregularities in buildings.

According to the Israeli code (SI-413 1995), a flexible (soft) story is that story whose lateral stiffness is less than 70% of that of the story above, or less than 80% of the average stiffness of the three stories above, and which contains less than half the length of walls (with thickness of 150 mm or more) as compared to the story above it, in at least one of its principal directions. Weak story is defined as a story with lateral shear capacity in a direction less than 80% of that of the story above in the same direction.

Israeli code allows a flexible (soft) or a weak story, including open first-story buildings, in buildings with low or medium ductility levels only, which correspond to the buildings of little or moderate importance only. The design forces for the flexible or weak story members, and for the members in the story above and below, are required to be increased by a factor $0.6R$. The response reduction factor, $R$, for the building system is discussed in the next section. For MI-RC frame buildings, $R$ is 3.5 for low ductility level, and 5.0 for medium ductility level. Therefore, beams and columns of the flexible or weak story, and also of the two adjacent stories, are required to be designed for at least 2.1–3.0 times the actual design forces for the *irregular* story, depending upon the ductility level of the building. Confinement in columns in the flexible or weak story, and in the story above and below, is required to be increased such that the maximum spacing of shear reinforcement (min. 8 mm diameter) shall not exceed 100 mm throughout the height of columns. In addition, the overlapping length of column longitudinal bars in the flexible or weak story, and in the two adjacent stories is required to be 30% more than that for the corresponding *regular* columns.

Israeli code allows construction of an extremely weak story, whose shear resistance is less than 65% of that of the story above, in buildings with height up to 2 stories or 9 m, whichever is less. The height restriction is waived if the combined shear resistance of the weak story and the stories above and below is at least equal to $0.75R$ times the seismic design base shear for the building. In any case, members of an extremely weak story and two adjacent stories are required to be designed for the increased forces ($0.6R$), as discussed in the previous paragraph.

According to the Nepal code (NBC-201 1995), at least two lateral load–resisting walls shall be used in each principal direction at any level in a building. At least 20% of the total length of the walls in the $x$ direction shall be placed in each *Area 1* and *Area 2*, and in the $y$ direction in each *Area 3* and *Area 4*, as shown in Figure 3. In each principal direction, the ratio of lumped mass of each story to the sum of thicknesses of walls including plaster finish in the story shall not be more than 125% of the same ratio for any higher story except at the roof level. This provision keeps a check on the plan and vertical irregularities arising in buildings due to unsymmetrical placement of MI.

Most national codes do recognize the vulnerability of frame members of the stories that are rendered soft/weak due to the absence or reduction of MI. These codes require increasing the seismic design forces of the concerned story members several times,

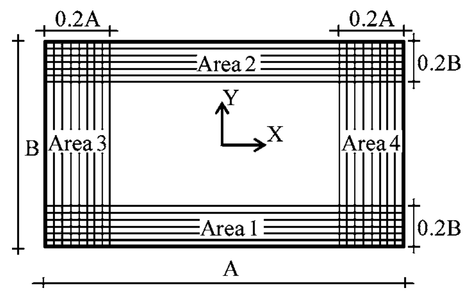


**Figure 3.** Plan of a typical building showing requirement of minimum 20% of the total length of lateral load–resisting walls along both *x* and *y* directions to be placed in each of the external frames in both directions (NBC-201 1995).

varying from 1.5 to 4.68 times, depending upon the extent of irregularities, building systems, ductility, and energy dissipation capacity. In some buildings, it is not feasible to increase capacity of the columns in a soft/weak story. Therefore, the Indian code (IS-1893, 2002) recommends providing symmetric RC shear walls, designed for 1.5 times the seismic design forces, in the weak/soft story, preferably on the periphery of buildings.

## RESPONSE REDUCTION FACTOR

Elastic force resultants in RC frame members are reduced by an appropriate value of *R* to account for overstrength, redundancy, and ductility in the structure. It is difficult to compare *R* values across different codes because of significant differences in the design philosophies, and safety and load factors used on the final design values. Therefore, *R* values are compared for different building systems within a particular code only. The ratio of seismic design forces for frames with MI to frames without MI, due to the difference in *R* as specified by various codes, is given in Table 1. *R* value for MI-RC frames is generally less than that for bare frames, thus most codes require MI-RC frames to be designed for higher force levels than the corresponding bare frames (about 1.15 to 3.0 times).

In *Eurocode 8* (2003), behavior factors are specified for different types of building systems, such that vertically *irregular* frames are required to be designed for 1.2 times the design forces for corresponding *regular* frames. The code does not differentiate between the behavior factors for RC frames with or without MI.

## LATERAL DISPLACEMENT AND INTERSTORY DRIFT

Lateral deformations at various levels in MI-RC frame buildings depend upon the distribution of MI walls in buildings. If more walls are present at the base, lateral deformations will be less and evenly distributed along the height of buildings. On the other hand, if more walls are present on the upper stories, then lateral deformations will be concentrated at the bottom, where stories are lesser infilled. Lateral deformations and interstory drift will also depend upon the ductility and damping of buildings.

Chinese code (GBJ-11-89 1989) has provisions to control the deformability of MI-RC frames. Seismic deformations must be checked for the limit state of deformability. Elastic story relative displacement, $\Delta U_e$, caused by the design value of frequent-earthquake action should not exceed the limit given by

$$\Delta U_e \leqslant (\theta_e)H \quad \text{units:}\left\{\begin{array}{lll}\Delta U_e, H & \text{in} & \text{m} \\ \theta_e & \text{in} & \text{rad}\end{array}\right\} \tag{17}$$

where $\theta_e$ is the elastic drift taken as $1/550$ for frames considering the stiffness of MI. Elasto-plastic story relative displacement ($\Delta U_p$) at weak locations caused by rare large earthquakes is limited by

$$\Delta U_p \leqslant (\theta_p)H_w \quad \text{units:}\left\{\begin{array}{lll}\Delta U_p, H_w & \text{in} & \text{m} \\ \theta_p & \text{in} & \text{rad}\end{array}\right\} \tag{18}$$

where $\theta_p$ is the elasto-plastic drift with a value of $1/50$ (increased by 20% for ductile columns), and $H_w$ is height of the weak story. Such buildings shall not have any abrupt change in story rigidity, and height shall not be more than 12 stories. By restricting elastic and elasto-plastic relative story displacement in MI-RC frames, an attempt has been made to reduce the brittle and out-of-plane failure of MI.

A few national codes, such as *Eurocode 8* (2003), NBC-105 (1995), NSR-98 (1998), and Costa Rican code (1986) have restricted the interstory drift ratio for MI-RC frames to about 1%. These drift ratios are calculated using displacements obtained from elastic forces, which are amplified. *FEMA-306* (ATC 1999) recommends the following interstory drift limit states for different solid panels: for brick masonry, 1.5%; for grouted concrete block masonry, 2.0%; and for ungrouted concrete block masonry, 2.5%. However, there is a concern that these values are too large and further experimental studies are needed to verify these limit states.

## STRENGTH OF MASONRY INFILL

Strength of MI does not have any direct implications on the ultimate strength of ductile RC frames; however, in some cases, failure modes of MI control the failure modes of nonductile RC frames. Failure mode of MI depends upon relative strength of MI in different actions, like compression, shear, etc. For example, if RC columns are not sufficiently confined with shear reinforcement, then shear-sliding failure mode of MI along a bed joint may trigger shear failure of columns.

As per *Eurocode 8* (2003), shear capacity of columns is required to be checked for shear forces generated by the diagonal strut action of MI by considering the vertical component of the width of strut as the contact area between RC frame and MI. Recommended strut width is an unspecified fraction of the panel diagonal length. Minimum wall thickness of 240 mm and maximum slenderness ratio (height/thickness) of 15 is specified for MI. Nepal code (NBC-201 1995) also requires MI to be modeled as diagonal struts, without specifying their cross-sectional properties. A minimum wall thickness of half brick is allowed to be used as infill.

According to the Israeli code (SI-413 1995), MI of thickness 150 mm or more are considered to resist the seismic story shear force, and total story resistance is given by

$$V_R = 10\left(\sum A_c\right)f_{vd} + 0.4\left(\sum A_m\right)f_{mk} \quad \text{units:}\left\{\begin{array}{lll} V_R & \text{in} & \text{N} \\ A_c, A_m & \text{in} & \text{mm}^2 \\ f_{vd}, f_{mk} & \text{in} & \text{MPa} \end{array}\right\} \tag{19}$$

where $\Sigma A_c$ and $\Sigma A_m$ are total cross-sectional areas of RC columns and MI, respectively, along the direction considered, $f_{vd}$ is the design strength of concrete in shear, and $f_{mk}$ is the characteristic shear strength of MI, which may be taken as 0.2 MPa for walls with mortar of at least 10 MPa compressive strength, and 0.1 MPa for walls with weaker mortar.

*FEMA-306* recommends the following equation to calculate the effective width of diagonal compression strut $a$, which can be used in the strength calculation of MI:

$$a = 0.175(\lambda_1 h_{col})^{-0.4} r_{\text{inf}} \quad \text{units:}\{a, h_{col}, r_{\text{inf}} \text{ in in.}\} \tag{20}$$

where $h_{col}$ is the column height, $r_{inf}$ is the diagonal length of the MI panel, and $\lambda_1$ is given by

$$\lambda_1 = \left[\frac{E_{me} t_{\text{inf}} \sin 2\theta}{4E_{fe} I_{col} h_{\text{inf}}}\right]^{0.25} \quad \text{units:}\left\{\begin{array}{lll} \lambda_1 & \text{in} & \text{in.}^{-1} \\ E_{me}, E_{fe} & \text{in} & \text{psi} \\ t_{\text{inf}}, h_{\text{inf}} & \text{in} & \text{in.} \\ \theta & \text{in} & \text{rad} \\ I_{col} & \text{in} & \text{in.}^4 \end{array}\right\} \tag{21}$$

where $E_{me}$ and $E_{fe}$ are expected modulus of elasticity of masonry (secant modulus of elasticity between 5% and 33% of masonry prism strength) and frame material, respectively. In the absence of tests, recommended value of $E_{me}$ is specified as 550 times the prism strength of masonry ($f'_{me}$). In Equation 21, $t_{inf}$ is the actual thickness of MI in contact with frame, $\theta$ is the inclination of diagonal strut with horizontal, $I_{col}$ is the moment of inertia of column, and $h_{inf}$ is the height of MI panel. Thickness of equivalent strut is taken to be equal to actual thickness of the wall.

*FEMA-306* identifies four possible failure modes for MI that also give an indication of potential crack and damage patterns in MI. The failure modes are sliding-shear failure, compression failure, diagonal tension cracking, and general shear failure. Equations given in *FEMA-306* to calculate strength corresponding to each failure mode are summarized in the Appendix. These equations must be used with caution as their further experimental verification may be necessary.

In addition to these failure modes of MI, RC frame may fail due to the flexural failure of beams and/or columns due to yielding of tension steel, shear failure of beams

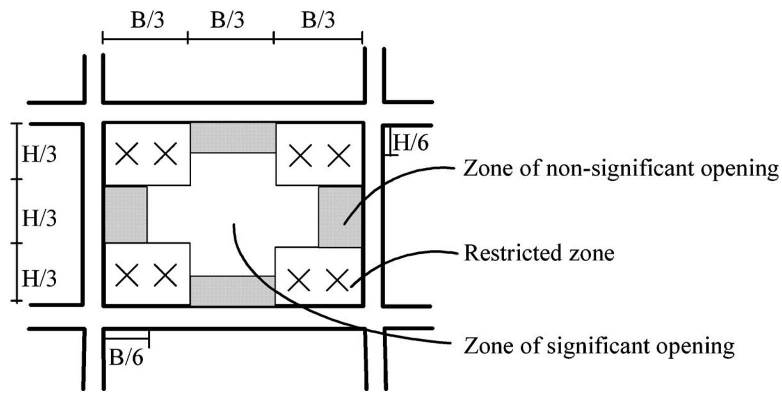


**Figure 4.** Possible location of openings in infill walls (NBC-201 1995).

and/or columns, and shear failure and bond failure of beam-column joints. Strength associated with these failure modes of RC frames members is generally calculated from different RC codes.

## Effect of Openings in Masonry Infill on Strength

Door and window openings in MI are provided because of functional and ventilation requirements of buildings. Presence of openings in MI changes the actual behavior of RC frames because of reduction in lateral strength and stiffness. Most national codes, in general, do not discuss the effects of openings on the strength and stiffness of MI-RC frames.

As per *Eurocode 8*, only the solid walls or walls with a single door or window opening are assumed to be imparting any significant strength to the structure. Large openings are required to be framed with RC elements across the full length and thickness of walls. Vertical RC elements of at least 150 mm dimension are required at both sides of any opening larger than a 1.5 m$^2$ area. Longitudinal steel in the element shall not be less than 300 mm$^2$ or 1% of the cross-sectional area of the element. Shear reinforcement in the form of stirrups of at least 5 mm diameter is required with a minimum spacing of 150 mm.

According to the Nepal code (NBC-201 1995), only those walls with an opening area less than 10% of the gross panel area are considered as resisting seismic loads. Openings shall be outside the restricted zone (Figure 4), and if these openings are located inside the middle two-thirds of a panel, then they need to be strengthened by providing RC elements around them (Figure 5a). RC tie beams at both the top and bottom of openings along the full length and width of the wall, and vertical elements on both sides of the opening shall be provided with longitudinal reinforcement of two bars of 8 mm diameter. Shear reinforcement in the form of minimum 6 mm diameter bars at every 150 mm is required in the elements. Such strengthening elements are not required for openings in a nonsignificant area (Figure 5b).

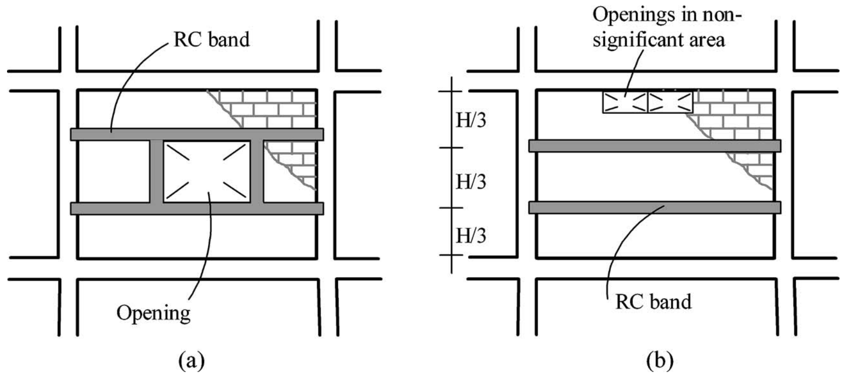


**Figure 5.** (a) Framed opening in a significant area, and (b) tie-bands and openings in a non-significant area, in infill walls (NBC-201 1995).

### Strength Associated with Out-of-Plane Collapse of Masonry Infills

During earthquakes, MI walls are subjected to high in-plane shear forces because of their high initial stiffness. Tension cracks are formed along the loaded diagonal in MI, which causes reduction in their lateral strength. In addition, connection between the RC frame and MI is generally weak, and MI may get separated from RC frames during the in-plane or out-of-plane ground motion, and thus become susceptible for collapse in the out-of-plane direction. However, such out-of-plane collapses are not common for walls of low slenderness value (ratio of unsupported length or height to thickness) and when MI walls are sufficiently confined in an RC frame. Under the action of out-of-plane forces, bending of MI takes place in the out-of-plane direction, which exerts tension (elongation) in the leeward face of MI. Because of elongation, an in-plane compressive reaction is induced in MI by the surrounding RC frames in which MI walls are confined. Therefore, an arching action is developed in MI due to which a considerable amount of out-of-plane forces are resisted by MI.

*Eurocode 8* (2003) suggests several preventive measures to avoid brittle failure, premature disintegration, and out-of-plane failure of MI walls during earthquakes, especially for slender walls (ratio of the smaller of length or height to thickness greater than 15). The measures includes providing light wire meshes adequately anchored on MI walls and on RC frames, wall ties fixed to columns and cast into bedding planes of masonry, and concrete posts and belts across the panels and through the full thickness of the MI. Nepal code (NBC-201 1995) recommends provision of RC bands throughout the length of walls at about one-third and two-thirds of the story height, as shown in Figure 5b. These bands are similar to the framing components, as discussed above in the section on openings.

Based on Angel et al. (1994), *FEMA-306* recommends calculating the capacity of MI to resist out-of-plane seismic forces. Capacity of MI is calculated in the form of a uniform pressure applied on MI walls in the out-of-plane direction, which depends upon strength of masonry, slenderness ratio, and damage sustained in MI walls and surrounding columns due to in-plane seismic forces.

### STIFFNESS OF MASONRY INFILL

MI walls are laterally much stiffer than RC frames, and therefore, the initial stiffness of the MI-RC frames largely depends upon the stiffness of MI. Stiffness of MI-RC frames significantly depends on the distribution of MI in the frame; generally, the MI-RC frames with regular distribution of MI in plan as well as along height are stiffer than the *irregular* MI-RC frames. Lateral stiffness of MI-RC frames reduces with the presence of openings in infills; however, this issue has not been addressed by the national codes.

*Eurocode 8* (2003), Nepal code (NBC-201 1995), and *FEMA-306* recommend modeling the MI as equivalent diagonal struts. However, *Eurocode 8* and Nepal code do not specify the width of strut. *FEMA-306* recommends Equation 20 to calculate the width of strut for use in stiffness calculations; however, experimental research by Al-Chaar (2002) has reported that Equation 20 can estimate only the lower-bound stiffness values of MI.

Nepal code specifies the modulus of elasticity of MI as 2,400 to 3,000 MPa for various grades of mortar. On the other hand, *FEMA-306* recommends using modulus of elasticity as 550 times the masonry prism strength in the absence of tests. As per *FEMA-306*, the only MI walls assumed to provide stiffness are those that are in full contact with RC frames, or those that are structurally connected to RC frames.

## SHORTCOMINGS IN NATIONAL CODES

The present paper has illustrated several shortcomings of various national codes on issues related to seismic design of MI-RC frame buildings. Major problem areas in various national codes needing further attention may be summarized as follows:

1. *Empirical estimation of natural period* addresses very simple and *regular* MI-RC frames and does not cover the frames rendered *irregular* because of unsymmetrical distribution of MI. Because of practical reasons, most RC buildings become *irregular* when MI walls are added in RC frames. Therefore, most of the empirical equations may not estimate the natural periods of such buildings with sufficient accuracy.
2. *Enhanced design of weak/soft-story frame members* is done in different national codes based on empirical or semi-empirical relations. Very limited literature is available in support of these relations. Hence there is an urgent need for more research in this area.
3. *Strength and stiffness of MI* are among the most important concerns related to MI-RC frame buildings, yet national codes are not very specific on these issues. The presence of openings in MI walls and vulnerability in the out-of-plane direction further complicates the matter. Only limited research is available to address issues related to reduction in strength and stiffness of MI because of the

presence of openings, and to assess the strength associated with the out-of-plane collapse of MI.

4. *Response reduction factor and allowable story drift* control the seismic design forces and deformability requirements for buildings, respectively. There is no consensus in various national codes on values of response reduction factor, which reflects that more research is needed on reliable estimation of strength and ductility of such buildings. Similarly, there is an ambiguity over the specifications in various national codes regarding allowable story drift in such buildings.

Clearly, a substantial amount of research is required to enhance our understanding of these widely used structures, and to overcome the shortcomings in the national codes associated with the seismic design of such structures.

## SUMMARY AND CONCLUSIONS

MI in RC frames acts as a diaphragm in vertical plane that imparts significant lateral strength and stiffness to RC frames under lateral loads. Infilled frames also tend to be substantially stronger, but less deformable, than otherwise identical bare frames. In symmetric buildings with vertically continuous infilled frames, the increased stiffness and strength may protect a building from damage associated with excessive lateral drift or inadequate strength. Because of its higher stiffness, infill panels may attract significantly greater forces that may lead to premature failure of infill, and possibly of the whole structure. Therefore, it is essential for designers to consider the effects of infills in the design of RC buildings.

In the paper, seismic design provisions in various national codes for MI-RC frames have been reviewed and compared. As shown in Table 1, there is no single code that contains all the relevant information required for the seismic design of such buildings. Most of the codes agree that MI-RC frame buildings require special treatment, and they specify clauses on several important issues related to such buildings. However, the codes differ greatly in specifications of the individual clauses.

Various codes recommend simplified static analysis methods for *regular* buildings, and detailed three-dimensional dynamic analysis methods for *irregular* buildings. Several empirical formulae are suggested to estimate the natural period of MI-RC frames, which have their own shortcomings. Thus some codes recommend using Rayleigh formula to calculate the period more accurately. Most of the codes require that the periods estimated by Rayleigh formula are not more than 20–30% of those given by empirical formulae.

Although MI attracts most of the lateral forces coming on buildings, RC frames must have sufficient strength to prevent the premature failure of buildings in case of failure of masonry walls because of their brittle behavior. Thus most codes recommend designing RC frames to independently resist at least 25% of the design seismic base shear.

Several codes address the problems associated with plan and vertical irregularities in MI-RC frames. The codes restrict the amount of eccentricity between center of mass and center of rigidity to safeguard the building components against the adverse effects of plan irregularities. In case of vertical irregularities, codes recommend increasing the capacity of *irregular* story members by 1.5–4.68 times, depending upon the extent of irregularities, building systems, ductility of the members, etc.

National codes specify lower values of response reduction factors for MI-RC frame buildings as compared to the buildings without MI, such that MI frames are required to be designed for 1.15–3 times the design forces for the corresponding bare frames. Lower value of response reduction factor is considered for MI-RC frames because of lower ductility and a higher degree of uncertainty and seismic vulnerability associated with MI. A few codes have specified limitations on the elastic and inelastic deformations and interstory drift ratio of MI-RC frames for damage limitation requirements.

A few codes recommend modeling MI using equivalent diagonal struts; however, the required sectional properties for the struts are not specified. Strength and stiffness of MI reduces with the presence of openings; however, the issue is not looked upon by any code. Various ways of reducing the damage in MI due to openings have been discussed in a few codes, e.g., framing the openings using RC elements. Full strength and stiffness of MI is not utilized when out-of-plane collapse of infills takes place. A few codes specify limits on slenderness ratio (ratio of length or height to thickness) to prevent out-of-plane failure of masonry infill. Some national codes recommend using light wire mesh and RC tie-bands along the length of walls at various locations to avoid out-of-plane collapse of MI.

Shortcomings of various national codes and recommendations for possible future research on MI-RC frames are discussed in the paper. A comprehensive design code for MI-RC frames is urgently needed as a significantly large number of buildings belongs to this category.

## ACKNOWLEDGMENTS

The authors acknowledge the financial assistance provided by the Ministry of Human Resource Development (MHRD), Government of India, in the research work.

## APPENDIX

*FEMA-306* recognizes four possible failure modes of MI, and the strength associated with each failure mode is discussed in the following:

### SLIDING-SHEAR FAILURE

In ductile RC frames, failure of MI with weak mortar joints and strong brick units may take place by sliding through the horizontal bed joint of wall. Initial sliding-shear capacity of MI is calculated by the Mohr-Coulomb failure criteria given by

$$V^1_{slide} = (\tau_0 + \sigma_y \tan\phi)L_{\inf}t_{\inf} = \mu N \quad \text{units:}\begin{Bmatrix} V^1_{slide}, N & \text{in} & \text{lbf} \\ \tau_0, \sigma_y & \text{in} & \text{psi} \\ \phi & \text{in} & \text{rad} \\ L_{\inf} & \text{in} & \text{in.} \end{Bmatrix} \tag{A1}$$

where $\sigma_y$ is the vertical stress on a MI panel, $\phi$ is the angle of sliding friction of masonry along a bed joint, and $L_{inf}$ is the length of the panel. $\tau_0$ is the cohesive capacity of mortar beds, which, in the absence of data, may be taken as 0.05 times $f'_{me90}$, where $f'_{me90}$ (psi) is the expected strength of masonry in the horizontal direction, which may be set at 50% of the expected prism strength of masonry, $f'_{me}$ (psi). $\mu$ is the coefficient of sliding friction along the bed joint and $N$ is the vertical load in the masonry panel.

After the cohesive bond in a mortar bed is destroyed ($\tau_0 = 0$) as a result of cyclic loading, MI still has some ability to resist sliding through shear friction in the bed joints. If lateral deformations are small, then $V^1_{slide} \approx 0$, because $\sigma_y$ may only result from the self-weight of panels. However, if the interstory drifts become large, then the bounding columns impose a vertical load due to shortening of the height of panels. Vertical shortening strain $\varepsilon$ in panels is

$$\varepsilon = \frac{\delta}{h_{col}} = \theta_d \frac{\Delta_d}{h_{col}} = \theta_d^2 \quad \text{units:}\begin{Bmatrix} \delta, \Delta_d, h_{col} & \text{in} & \text{in.} \\ \theta_d & \text{in} & \text{rad} \end{Bmatrix} \tag{A2}$$

where $\delta$ is the downward movement of the upper beam as a result of the interstory drift angle, $\theta_d$, and $\Delta_d$ is the interstory drift. Axial load on MI can be calculated by

$$N = \varepsilon L_{\inf} t_{\inf} E_{me} \tag{A3}$$

Sliding-shear capacity of MI is then calculated by

$$V^1_{slide} = \mu L_{\inf} t_{\inf} E_{me} \theta_d^2 \tag{A4}$$

## COMPRESSION FAILURE

Failure of MI may take place by compression failure of the equivalent diagonal strut if the mortar joints and brick units are strong and RC frames are sufficiently ductile. Horizontal shear force required for the failure of equivalent diagonal strut is calculated by

$$V_c = a t_{\inf} f'_{me90} \cos\theta \quad \text{units:}\{V_c \text{ in lbf}\} \tag{A5}$$

## DIAGONAL TENSION CRACKING

This is not a failure mode; however, it helps other failure modes to propagate. Under lateral in-plane loading, high compressive stresses develop in MI along the loaded diagonal, and transverse to these compressive diagonal stresses, tension cracks develop in MI. Using the recommendations of Saneinejad and Hobbs (1995), the cracking shear in MI is given by

$$V_{cr} = 2\sqrt{2}t_{\inf}h_{\inf}\sigma_{cr}\cos^2\theta \quad \text{units:}\begin{Bmatrix} V_{cr} & \text{in} & \text{lbf} \\ \sigma_{cr} & \text{in} & \text{psi} \end{Bmatrix} \tag{A6}$$

In the absence of tests results, the cracking capacity of masonry, $\sigma_{cr}$, which is approximately equal to the cohesive capacity of the mortar beds, may be taken as 0.05 times $f'_{me90}$. The cracking capacity, $\sigma_{cr}$ may also be calculated as

$$\sigma_{cr} = 20\sqrt{f'_{me}} \tag{A7}$$

**GENERAL SHEAR FAILURE OF PANEL**

Initial and final contributions of shear carried by MI panels are defined as

$$V_{mi} = A_{vh}2\sqrt{f'_{me}} \quad \text{units:}\begin{Bmatrix} V_{mi}, V_{mf} & \text{in lbf} \\ f'_{me} & \text{in psi} \\ A_{vh} & \text{in in.}^2 \end{Bmatrix} \tag{A8}$$

and

$$V_{mf} = 0.3V_{mi} \tag{A9}$$

where $V_{mi}$ is the available initial shear capacity consumed during the first half-cyclic (monotonic) loading, $V_{mf}$ is the final shear capacity as a result of cyclic loading effects, and $A_{vh}$ is the net horizontal shear area of MI panels. For panels without openings, $A_{vh}$ is calculated by the total length times the thickness of walls. Lower value among those given by Equations A1 and A8 will be the governing shear strength of infill.

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(Received 19 July 2005; accepted 22 December 2005)