MASONRY STRUCTURAL SYSTEMS FOR REGIONS OF HIGH SEISMICITY

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SUMMARY

This paper discusses different engineered masonry systems and assesses probable seismic performance in terms of established principles of earthquake engineering and the special properties of masonry. It is concluded that masonry buildings whose lateral load resistance is provided by simple linked cantilever shear walls are likely to exhibit better seismic performance than conventional pierced-wall designs. The importance of structural simplicity and the need for energy to be dissipated in carefully detailed plastic hinges are stressed. Recent research into base isolation and the influence of foundation compliance and structural rocking are discussed in relation to design of masonry shear walls.

INTRODUCTION

Masonry is one of the most common forms of construction in seismic zones, and collapse of inadequately designed masonry structures under seismic attack has probably been the single greatest contributor to death tolls in recent major earthquakes. Although all but a small fraction of recent masonry failures has been of undesigned unreinforced masonry built on traditional lines the extent of damage in recent earthquakes, in particular the 1971 San Fernando earthquake[1] to reinforced masonry buildings designed in accordance with modern building codes has been disquietening. In part the poor performance can be traced to a lack of research effort into the seismic performance of structural masonry. Although steel and reinforced concrete construction have received wide research attention in the past twenty years, structural masonry has until recently remained something of a 'poor relation' and has been designed on traditional lines or as a low grade concrete, without recognition of the special limitations imposed by material behaviour.

Research effort in structural concrete has sought to establish preferred structural forms, and to improve seismic performance by detailing for ductility and by use of special design philosophies, such as the Capacity Design approach[2]. This paper reviews common masonry structural systems in an attempt to establish structural forms and design methods that are most suited to masonry construction. The arguments are restricted to reinforced masonry, as unreinforced masonry, like unreinforced concrete, cannot be relied on for lateral load resistance except for light buildings in regions of low seismicity.

DUCTILE RESPONSE OF STRUCTURAL MASONRY SYSTEMS

Lateral Force Levels

Although many material codes still specify elastic design procedures for structural masonry under seismic loads, the levels of lateral loads

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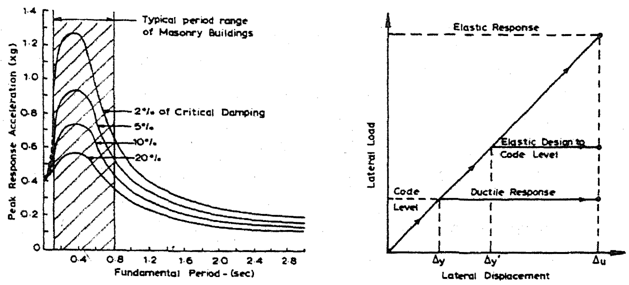
specified are inadequate to ensure that behaviour remains elastic under the design level earthquake. Fig. 1a shows smoothed composite acceleration response spectra developed by Skinner[3] from eight California accelerograms scaled to El Centro 1940 N-S intensity, an accelerogram which is now accepted as having only moderate damage potential. Masonry structures, being stiff, typically have fundamental periods in the range 0.1 - 0.8 sec., thus spanning the frequency range of maximum response. Assuming 5% equivalent viscous damping, peak elastic response of the order of 0.8g can be anticipated.



(a) Composite Response Spectra from Eight Accelerograms Scaled to El Centro 1940 N-S (after Skinner)

(b) Peak Lateral Load using "Equal-displacement" Principle

Fig. 1. Seismic Loads for Masonry Walls

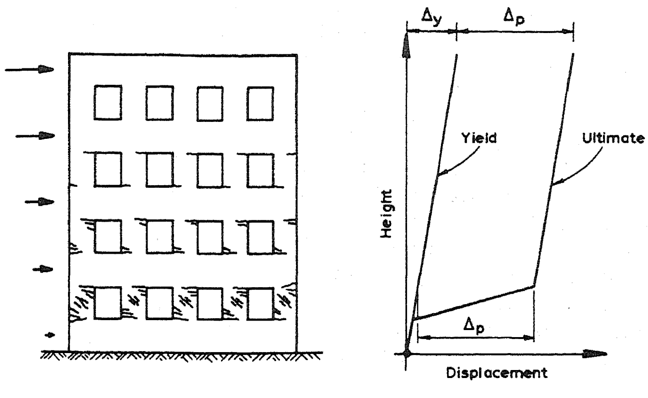
Seismic coefficients included in most loadings codes are based on ductile response, with structure ductility in the vicinity of four. Reference to Fig. 1b shows that elastic design to these load levels will still result in the ultimate capacity being attained, but with a reduction in the required structure ductility. Unless masonry structures are designed for the true elastic force levels likely to occur during the anticipated structural life, it is essential to recognise that ductile response will be required, and to design accordingly by ensuring that the materials and structural system adopted are capable of sustaining the required ductility without excessive strength or stiffness degradation.

Materials

Space limitations mitigate against a description of differences in behaviour resulting from use of different materials, such as concrete or clay-brick. These have been discussed elsewhere[4]. Behaviour is sufficiently similar to enable valid generalisations to be made, and unless specifically stated it will be assumed that the masonry construction consists of clay or concrete hollow-cell units, or double-skin wall, jointed with competent mortar, reinforced within the cavities and grouted with a concrete grout.

Pierced Shear Wall with Pier Ductility

Masonry structures almost always gain their lateral load capacity from shear walls rather than beam and column type construction. Traditionally the most common construction method has consisted of peripheral structural masonry shear walls pierced by window and door openings, as idealised in Figs. 2a, 3a. Under lateral loading the weakest link will typically be the flexural or shear strength of the pier units between the window openings. Unless the piers are designed to resist elastically the forces resulting from the design earthquake, the piers will be required to exhibit ductility. Plastic displacement (flexural or shear) will inevitably be concentrated in the piers of one storey, generally the lowest, with consequential extremely high ductility demand at that level. Consider the deflection profiles at yield and ultimate illustrated in Fig. 2b. Design is on the basis of a



(a) Crack Pattern (b) Displacement Profiles

Fig. 2. Pierced Shear Wall with Pier Ductility

specified ductility μ, requiring a roof level plastic displacement

$$\Delta_p = (\mu - 1)\ \Delta_y \tag{1}$$

Under a typical triangular distribution of lateral seismic loads (Fig. 2a) the yield load will be attained when the flexural or shear strength of the bottom level piers is reached. Other elements of the structure will not have yielded at this stage. Subsequent deformation of the yielding piers will occur at constant or decreasing lateral load, thus ensuring that the other elements are protected from inelastic action and concentrating the plastic deformation Δ_p in the yielding piers. If the structure has n stories and pier height is half storey height, then the elastic displacement over the height of the piers at yield will be

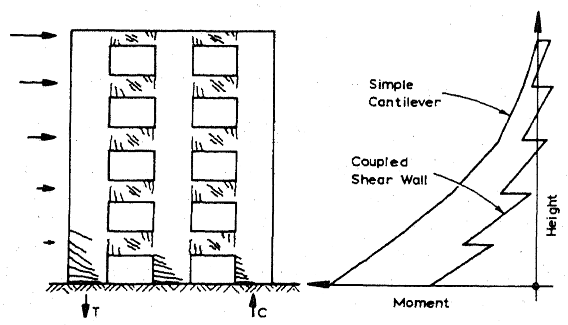
$$\Delta_{y1} = \Delta_y/2n \tag{2}$$

From Eqs. 1 and 2 the displacement ~~ductility~~ factor μ_1 required of the piers will thus be

$$\mu_1 = 2n(\mu - 1) + 1 \tag{3}$$

Thus for a 10 storey masonry shear wall designed for a structural displacement ductility factor $\mu = 4$, the ductility required of the piers would be $\mu_1 = 61$. Extensive recent experimental research on pier units at the University of California, Berkeley[5] has indicated extreme difficulty in obtaining reliable ductility levels an order of magnitude less than this value. It is this concluded that the structural system of Fig. 2a, with ductile piers, is only suitable if very low structural ductilities are required.

Pierced Shear Wall with Spandrel Ductility



(a) Crack Pattern (b) Wall Moments

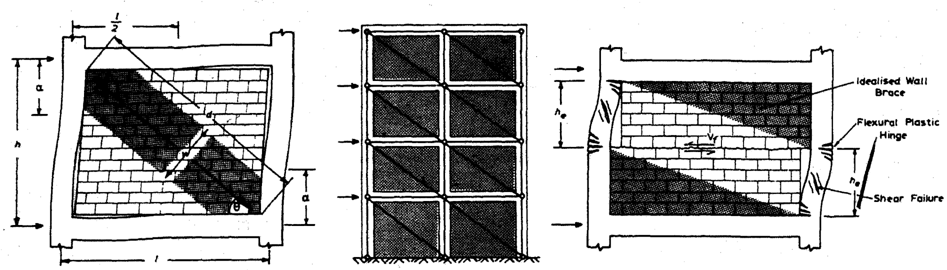
Fig. 3. Coupled Masonry Shear Walls

Occasionally openings in masonry walls will be of such proportions that spandrels will be relatively weaker than piers, and behaviour will approximate coupled shear walls, with crack patterns as illustrated in Fig. 3a. Well designed coupled shear walls in reinforced concrete constitute an efficient structural system for seismic resistance[2]. However, to satisfy the high ductility demand generated in the spandrel beams diagonal reinforcement is generally required. Such a system is unsuitable for structural masonry, and rapid strength and stiffness degradation is likely resulting in an increase in wall moments towards those appropriate to simple linked cantilevers (see Fig. 3b). If the wall moment capacities have been proportioned on the basis of ductile coupled shear wall action, then the moment increase implied by Fig. 3b will not be possible, and the consequence will be excessive ductility demand at the wall-base plastic hinges. To ensure satisfactory performance from a masonry wall of the type shown in Fig. 3a, it is advisable to ignore the strength of the spandrels under seismic loading, and proportion the wall moment capacities on the basis of the simple linked cantilever moments in Fig. 3b. 'Basketting' reinforcement is then adopted in spandrels to avoid total collapse of these elements under the comparatively large relative displacements that will result.

Masonry Infilled Frames

There are numerous examples of earthquake damage that can be traced to structural modification of the behaviour of well designed frames by so-called non-structural masonry infill. The infill results in a decrease in fundamental period, and an increase in seismic shears, frequently resulting in shear failure of the frame columns.

Two options are available to the designer. The infill and frame may be structurally separated on the sides and top to allow free deformation of the frame relative to the panel, maintaining the basic frame action. However, as separation at the bottom of the panel, and adequate resistance to seismic face-loads, are difficult to achieve, this can be hazardous. Alternatively, the structural action of the infill can be recognized by designing for composite frame/panel action. Although at low loads full contact is maintained between frame and panel, separation occurs at high load levels, as shown in Fig. 4a, resulting in behaviour characterised by the diagonal braced frame shown in Fig. 4b. Ultimate capacity may be reached in a number



(a) Deformation under Shear Load. (b) Equivalent Braced Frame for 2 Bay 4 Storey Infilled Wall. (c) Knee Braced Frame Concept for Sliding Shear Failure

Fig. 4. Behaviour of Masonry Infilled Frames Under Lateral Load

of modes, including tension yielding of the tension column, compression failure of the diagonal masonry strut, sliding shear failure of the masonry along horizontal mortar courses (generally at or close to panel midheight), or flexural or shear failure of the column. This final failure mode is generally preceded by the sliding shear mode, illustrated in Fig. 4c. As this involves a shear-type plastic deformation at the level where sliding shear failure occurs, ductility demand may be very high for structures of substantial numbers of floors by exactly the same reasoning used in discussion of pierced walls with pier ductility. Although Klinger and Bertero[6] have demonstrated that comparatively high ductilities car be obtained in this mode if the panel is extensively reinforced vertically and horizontally and fully connected to the frame round the entire perimeter, the more common construction technique using unreinforced or lightly reinforced infill masonry will result in rapid strength degradation. Consequently such structures should be designed to remain elastic under the design-level seismic attack. More complete descriptions of infill behaviour are given in reference 4.

Linked Cantilever Shear Wall

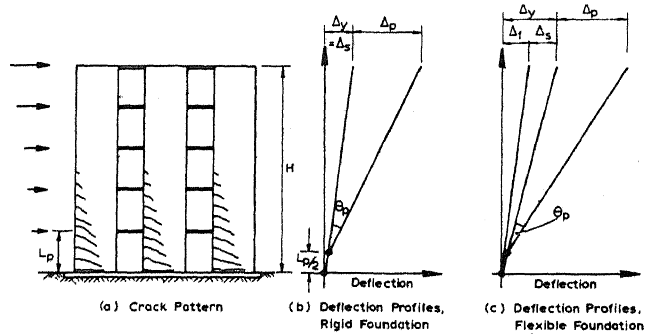


Fig. 5. Cantilever Shear Walls Linked by Flexible Floor Slabs

Fig. 5 illustrates the masonry structural system preferred in New Zealand for ductile seismic response. Seismic loads are carried by cantilever shear walls linked by flexible floor slabs which inhibit shear transfer between the walls. Energy dissipation occurs only in carefully detailed plastic hinges at the base of each wall. A ductile flexural failure mode is ensured by adopting a capacity design approach for shear strength of the walls, and carrying all shear within potential plastic hinge zones by horizontal shear steel.

The system illustrated in Fig. 5a has the advantage over those of Figs. 2a and 3a in that ductility is provided by plastic rotation of a hinge at the wall base, rather than by a shear-type plastic deformation. Hence the structural and member ductility demand in Fig. 5a are identical.

Because of the very brittle compression behaviour of masonry it is advisable to check that sufficient ductility is available at the ultimate compression strain, which may be taken as $\varepsilon_c = .003$, as for concrete. In Fig. 5b, representing walls built on rigid foundations, the average plastic curvature ϕ_p in the plastic hinge zone may be expressed as

$$\phi_p = \theta_p/L_p = (\mu - 1)\Delta_s/L_p(H - L_p/2) \qquad (4)$$

where L_p is the plastic hinge length and Δ_s is the structural yield displacement. With a knowledge of steel and masonry material properties and the axial load carried by the wall, the peak compression strain e_p corresponding to ϕ_p can be calculated. If e_p exceeds .003, a compression failure of the compression zone is likely. The wall must then be redesigned for higher force levels to reduce the required plastic displacements, and hence the ultimate compression strain, or alternatively, stainless steel confinement plates[7] may be inserted in mortar beds at the wall ends within the plastic hinge zone. It has been shown[5,7] that the inclusion of 3 mm thick confining plates increase the ultimate compression strain of masonry by inhibiting the vertical splitting which precedes compression failure. Tests on cantilever shear wall units designed by capacity design principles have indicated dependable flexural failures with good ductility[7,8].

When cantilever walls are constructed on a flexible foundation, foundation compliance will increase the yield displacement by an amount Δ_f proportional to the rotation of the base. Thus the yield displacement Δ_y can be expressed as

$$\Delta_y = \Delta_s + \Delta_f = C.\Delta_s \tag{5}$$

where C is a constant expressing the increase in elastic flexibility. However, assuming elasto-plastic behaviour of the flexural hinge, all plastic displacement will occur at constant load by rotation of the plastic hing. Thus for a specified structural displacement ductility factor μ, the required plastic curvature will be

$$\phi_p = (\mu - 1)\Delta_y/L_p(H - L_p/2) = C.(\mu - 1)\Delta_s/L_p(H - L_p/2) \tag{6}$$

Consequently foundation compliance will have the effect of increasing plastic curvature, and hence ultimate compression strain for a specified ductility. Conversely, for a specified ultimate compression strain the available structural ductility μ will be decreased by foundation flexibility. This influence is not widely recognized by designers.

Although the structural system represented in Fig. 5a is simple and structurally attractive, the implications of large storey-height glazed areas between walls may be architecturally undesirable. In these cases the system represented in Fig. 3a may be adopted provided the design is based on the concepts of this section. For preference, the spandrel units should be structurally separated from the cantilever walls by a seismic gap at each end filled with a flexible layer (e.g. polystyrene) to allow relative deformations to occur. The spandrels should still contain 'basketting' reinforcement to ensure integrity is maintained under seismic displacements.

INNOVATIVE DESIGN CONCEPTS

In recent years a number of structural systems applicable to masonry wall design have been developed that allow the engineer to design walls to remain elastic under the design earthquake while still designing at force levels appropriate to full ductile response. Two such techniques are briefly outlined below.

Base Isolation

Fig. 6 shows in schematic form a masonry building consisting of three linked cantilever shear walls separated from its foundations by use of mechanical energy dissipators. Different hysteretic energy dissipators have been developed, and characteristics are available in the literature[9]. Perhaps the most promising recent development is the use of an elastomeric bearing incorporating a central plug of lead[10]. The elastomeric bearing

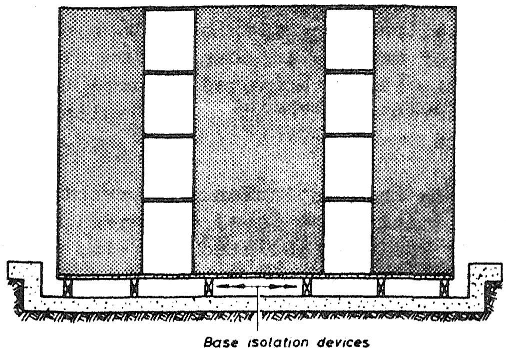


Fig.6. Base Isolation for Masonry Walls

provides the vertical load carrying capacity while the lead plug provides the requisite lateral load strength. Under seismic attack energy dissipation is provided by plastic deformation in shear of the lead plug, and the masonry structure above is protected from inelastic action by designing to a force level higher than the maximum base shear developed in the dissipators. Dynamic analyses of masonry buildings[11] showed considerable economies for base-isolated short period structures, resulting from reduced wall moments, and obviation of the need for a capacity design approach.

Structural Rocking

Overturning moments developed at flexural capacity of shear walls can be difficult to sustain. Fig. 7a shows a typical mixed structural system where the end masonry shear walls carry all the shear load but only a small proportion of the gravity loads. For stability under peak seismic response it may be necessary to incorporate a massive foundation beam (Fig. 8b). An alternative approach currently receiving considerable research attention would use a light foundation beam and let the wall rock, or tip, when the seismic overturning moment exceeds the gravity restoring moment. Again, elastic response of the masonry wall can be assured by designing the load capacity of the wall at an adequate margin above the load required to initiate tipping. Total instability collapse is extremely unlikely due to the large energy input required. Analyses for peak deflections can be carried out using an iterative response spectrum approach [12] based on Housner's rocking-block-model[13], but recent research[14] indicates that dynamic inelastic analyses are advisable.

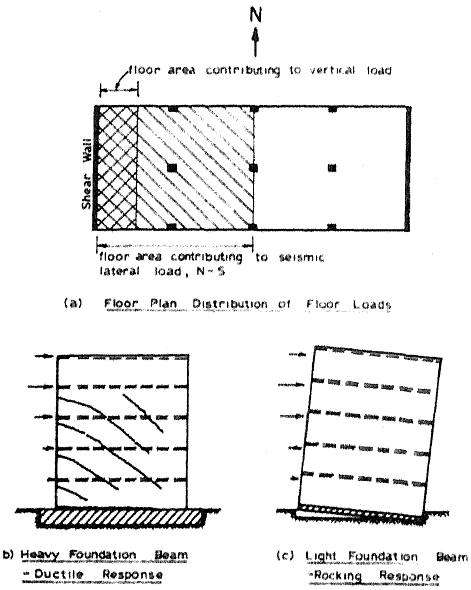


Fig. 7. Foundation Rocking

CONCLUSIONS

Traditional structural masonry systems consisting of reinforced pierced peripheral shear walls are unsuitable for seismic resistance unless a low level of ductility is specified. This will generally mean designing for close to a full elastic response, which will often be economically prohibitive. Better performance can be expected from a ductile design consisting of simple vertical cantilever shear walls linked by flexible floor slabs. Because of the extremely brittle nature of masonry in compression, it is necessary to ensure the required ductility can be achieved at acceptable peak compression strain levels, particularly when foundation compliance adds significantly to yield displacements.

Alternative design techniques such as base-isolation and structural rocking show promise as mechanisms for limiting the level of seismic response accelerations, enabling the masonry walls so isolated to remain elastic. More research is needed before these techniques can become suitable for routine design.

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