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Edited by D.P. Abrams[1]
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16. Abstract (Limit: 200 words)

This volume offers the proceedings of a two-day workshop focused on the seismic response and performance of masonry infills. Researchers and engineers made a total of sixteen presentations, which are reproduced here, and a number of separate groups convened to discuss the mathematical modelling of masonry infills, overall system behaviour, evaluation criteria, and the rehabilitation of existing buildings with masonry infill. Overall, the papers dealt with concrete or clay-unit masonry in concrete or steel frames subjected to static or dynamic, in-plane or out-of-plane seismic forces. Six papers treat laboratory experiments. One paper describes field experiments. Five concern analytical studies and the remaining four papers present case studies. A number of general themes emerged from the workshop: lateral strength of infill frames could be represented with equivalent struts; standard guidelines for the evaluation of rehabilitated or repaired infill systems are needed; standard relations for the assessment of infill strength are also necessary. Finally, one of the workshop's highlights was the joint participation and interaction of both researchers and practiced working engineers.

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EXECUTIVE SUMMARY

NCEER Workshop on Seismic Response of Masonry Infills
San Francisco, February 4th and 5th, 1994

PREFACE

Research on seismic performance of masonry infill panels is not new. Early studies date back over forty years. Despite the continued research interest, building codes still do not address how a structural engineer may design a new infill-frame system, or evaluate and rehabilitate an existing one. This is because of two reasons. One, masonry infills have long been recognized as a nonstructural partition, thus being exempt from building code specifications for building structures. Two, the interaction between a masonry infill and a structural material made of steel or reinforced concrete is not only complex from a structural mechanics view, but also a difficult problem to codify because steel and concrete codes do not address masonry materials.

The next few years will bring about the formulation of guidelines for seismic rehabilitation of existing buildings. This exercise will demand that standard methods for infill evaluation be established. In this context, codes of engineering practice for existing construction will precede those for new construction. The time will soon be here when the multitude of research results on masonry infills will have to be consolidated and formed into meaningful engineering standards. Because nearly all of the research projects on this topic have been disjointed studies, such an endeavor will be a great challenge which will probably yield a number of needed future studies.

As a precursor to this future research consolidation, a workshop on masonry infill research was proposed to the National Center for Earthquake Engineering Research a few years ago. As a very minimum, the workshop was felt to suffice as a forum for several currently funded research projects. Through no premeditation, five separate infill research projects were funded by the National Science Foundation at the same time. Two of these projects were supported through NCEER, two others through the NSF Repair and Rehabilitation Research Program, and one other through the NSF program on Large Structural Systems. At the same time, a major seismic evaluation program on hollow-clay tile infills was underway at a Department of Energy facility, and another research program was underway at the US Army Construction Engineering Research Laboratory. Because post-coordination is better than no coordination at all, a meeting of investigators from each of these research projects was proposed.

After the workshop was funded, and planing started to procede, it was decided to do more than simply bring a group of researchers together. Practicing engineers, well versed at seismic engineering, were invited to attend so that they could express their current practices for evaluation, design and redesign of masonry infill-frame systems, as well as their ideas for needed research. For this reason, the workshop was held in a major west coast city. Local structural engineers, comprising over 60% of the participants, provided an excellent sounding board for researchers presenting their results, and for expressing needs of the practice.

GENERAL WORKSHOP FINDINGS

The NCEER Workshop on Seismic Response of Masonry Infills was held on February 4th and 5th, 1994 at the Holiday Inn Golden Gateway in San Francisco. The two-day program consisted of sixteen presentations by researchers and engineers on the somewhat narrow topic of masonry infills. Discussion groups were held on mathematical modeling of infill component and system behavior as well as criteria for evaluation and rehabilitation of existing building systems.

Despite the relatively narrow focus of the workshop topic, each of the sixteen presentations presented a different research or engineering perspective. Papers dealt with concrete or clay-unit masonry in concrete or steel frames subjected to static or dynamic, in-plane or out-of-plane seismic forces. Six papers were on laboratory experiments. One paper was on field experiments. Five papers were on analytical studies. Four papers were on case studies.

The numerous perspectives on a narrow topic such as masonry infills suggested a definite lack of coordination. Also, the fact that nearly all research projects were in their initial phase, and few were in a continuing stage, gave evidence that the objectives of the research programs were independently sporadic. Needs were expressed by all researchers in attendance for better coordination of research objectives, standarization in experimental methods, and consistent development of computational models.

Several themes tended to emerge and reoccur over the day and a half. Nearly all of the workshop participants agreed that the lateral strength and response of an infill-frame system could be represented with equivalent struts, and that characterization of local behavior required a more complex formulation than a single strut. Standard guidelines need to be established for evaluating infill systems that have been rehabilitated or repaired using either traditional, or non traditional methods. Standard relations need to defined for assessing infill strength, and how various insitu measurements may be extrapolated to estimate component strength or performance. Research needs to be done on behavior and strength of infills with openings, and infill panels and/or cladding that is off center from the plane of a surrounding frame.

The workshop did provide an initial forum for the exchange of research and engineering information. The participants found the workshop to be of worth for initial communication on complementary problems.

One final resolution endorsed by all in attendance was that a second workshop be held in two years.

RESOLUTIONS

Each of the four discussion groups formulated a list of resolutions that were presented in a final plenary session for consensus approval of the whole. Resolutions are grouped together below by topic of each discussion group. Listings of the individuals in each discussion group are given at the end of these proceedings.

Discussion Group IA:

Modeling Global Response of Building Systems with Masonry Infills

1. Infill panels dramatically affect stiffness and strength of a building structural system, and should be considered in computations of global building response.
2. A two-dimensional compressive strut is a reasonable representation for the in-plane infill stiffness.
3. Properties of an equivalent strut may be developed with the use of physical or numerical models, or semi-empirical expressions.
4. Strength and stiffness degradation of infill panels should be accounted for in the structural analysis. A piece-wise nonlinear analysis is acceptable for such an analysis.
5. Global drift limits need to be established for infill-frame systems. Limits should assure that local panel performance criteria are met.
6. Vertical loads should be included in the development of an equivalent strut model.
7. Biaxial material properties are desirable in modeling infills, particularly near the corners of panels.
8. In-plane and out-of-plane loading effects may be considered separately, particularly at low to moderate force levels.
9. Future research investigations should examine:
    a) biaxial properties of panel materials
    b) effects of vertical loads on equivalent struts
    c) appropriate levels of global damping
    d) behavior of panels with openings
    e) criteria for formulation of equivalent struts in terms of system drift and local panel deformation and degradation.

Discussion Group IB:

Modeling of Infill Panel Behavior: Normal and Transverse Loadings

1. The behavior of infills with openings is not well understood.
2. Gaps between a frame and an infill panel will significantly influence the lateral force-deflection behavior of a frame-infill system. Field evaluation methods are needed to assess the presence and condition of these gaps.
3. Tests of out-of-plane performance for infill panels are presently being run using either static or dynamic methods. A set of standard recommendations summarizing the merits and limits of each type of test method should be formulated.
4. The influence of various parameters on infill behavior should be studied with nonlinear finite element models that have been calibrated with experimental data.
5. A unified method needs to be developed for assessing the strength of an infill panel.
6. Future research investigations should examine:
   a) the sensitivities of finite element models to various parameters
   b) the range in different frame-panel interface conditions in existence throughout the nation
   c) the feasibilities of simple methods for estimating seismic strength of infill panels such as (i) a nominal, average shear stress, (ii) plastic analysis methods or (iii) equivalent strut models
   d) the feasibilities of using a two-level analysis to estimate the global system response and the behavior of local panels and surrounding frames
   e) the feasibilities of developing performance-based design methods that rely on knowledge of stiffness and damage at various levels
   f) the precision of field test methods to measure shear and tensile strengths of mortar joints, and compressive strength of masonry units
   g) behavior in masonry infills subjected to bi-directional ground motions, particularly out-of-plane stability under large in-plane displacements.

Discussion Group IIA:

Criteria for Rehabilitation of Infills and Infill Systems

1. Acceptance criteria for infill performance need to be established.

2. Drift limits need to be set that prohibit strength degradation.

3. Appropriate techniques for rehabilitation of infill-frame systems include:

   a) addition of reinforced shear walls, or braced frames

   b) rehabilitate the infill panel using (i) non-traditional materials such as fiber glass coatings, (ii) gunite or shotcrete coatings, (iii) grouting for hollow-unit ungrouted masonry, (iv) strengthening window openings with steel confining frames, or (v) anchorage of infill panels to frames.

4. Future research investigations should examine:

   a) the effects of new materials used for rehabilitation

   b) the development of new computer programs for modeling response of rehabilitated infill-frame systems

   c) the feasibilities of basing acceptance criteria on lateral drift

   d) the response of undamaged infill systems using analytical models to determine why they worked, and if conventional modeling techniques would have predicted actual behavior

   e) the behavior of infills with openings and methods for their rehabilitation

   f) the effects of various kinds of frames on stiffness of infill-frame systems.

Discussion Group IIB:

Criteria for Evaluation of Infills and Infill Systems

1. Standard methods need to be developed for structural evaluation of infills with gaps, openings, eccentricities between frame and infill panels, and partial cracking.
2. Standard methods need to be developed for assessing the quality and mechanical properties of infill materials.
3. Analysis procedures need to be developed that include the in-plane stiffness of infill panels.
4. Drift limits at various limit states (initial cracking, strength degradation, life safety, etc.) need to be given standard definitions.
5. Standard guidelines need to be established on how to model infill-frame systems with veneers or cladding that are eccentric to plane of frame.
6. Standard definitions of failure modes for infill-frame systems need to be developed.
7. Methods need to be developed for evaluating strength and behavior of infill panels with openings.
8. Evaluation methods based on research done with single story, single bay frames need to be extrapolated for multi-bay, mulit-story frames.
9. Future research investigations should examine:
   a) development of standard evaluation methods
   b) development of analytical methods
   c) standardized procedures for estimating infill strength
   d) lateral-force behavior of infills with openings
   e) drift limit states for infill-frame systems
   f) behavior of infills with weak, non-ductile frames.

ACKNOWLEDGMENTS

The Workshop on Seismic Response of Masonry Infills was funded through a grant from the National Center for Earthquake Engineering Research to the University of Illinois at Urbana-Champaign (Project # 923107).

The workshop was co-sponsored by The Masonry Society and the Earthquake Engineering Research Institute.

Appreciation is extended to Susan Abrams for her assistance with the workshop.

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Section I

Synopses of Research Projects

SEISMIC RETROFIT OF FLAT-SLAB BUILDINGS WITH MASONRY INFILLS

A. J. Durrani[1] and Y. H. Luo[2]

ABSTRACT

Lateral drift of older flat-slab buildings subjected to seismic loading can be controlled by appropriately placing and mobilizing masonry infills. The modeling of infills as equivalent diagonal compression struts is examined. Finite element analysis is used to identify the parameters having most effect on infill-frame interaction under lateral loading. An equivalent effective width of the diagonal strut is proposed for masonry infill panels with and without openings for use in lateral load analysis of reinforced concrete flat-slab frames.

INTRODUCTION

Older flat-slab buildings typically have low lateral stiffness and lack the reinforcing detail necessary for protection against progressive collapse. As such, these buildings are vulnerable to severe damage during earthquakes of moderate intensity. Retrofit strategy for the older flat-slab buildings thus mainly consists of controlling the lateral drift and providing protection against progressive collapse. By limiting the lateral drift, the demand on slab-column connections is also reduced. Individual slab-column connections can be retrofitted to safeguard against progressive collapse. However, it is more practical to increase the lateral stiffness through a prudent global retrofit scheme. The addition of shear walls in existing buildings is an expensive preposition. Masonry infill walls, which have high in-plane stiffness and can be economically added to the existing building frames, are an attractive choice for control of lateral drift in older flat-slab buildings.

Masonry infills of different types are commonly present in buildings for functional and architectural reasons. Their contribution to lateral stiffness and strength of flat-slab frames is usually neglected during the design of new buildings. Retrofit of older buildings for seismic resistance requires an accurate evaluation of the building response including the contribution of the existing infills. As such, appropriate analytical tools for elastic and inelastic analysis of reinforced concrete frames with masonry infills need to be developed and verified through laboratory tests. At present, the test data on the interaction of masonry infills with the concrete frames under lateral loading is very limited and the analytical models for infills are not yet fully developed. Tests on reinforced concrete frames with masonry infills are currently in progress to investigate the potential of utilizing masonry infills to improve the seismic resistance of older flat-slab buildings. The lateral load behavior of the test frames with masonry infills was first studied with finite element analysis. This paper presents the analytical results and examines the commonly used analytical models for masonry infills. Tests (2,3,4,5) have shown that the increase in lateral stiffness and strength of frames depends upon the thickness of the infill wall, its aspect ratio, presence and size of openings, and stiffness of the bound-

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ing members. The infill walls are commonly modelled as diagonal struts (4,5) which can transfer only the compressive force between the diagonally opposite joints. A key element of this approach is the determination of the effective width of the equivalent diagonal compression strut.

MODELLING OF INFILLS

Based on a number of tests on 6 in. x 6 in. square mortar infills bounded with steel frame subjected to diagonal static loading, Smith (5) proposed simple equations for the effective width of diagonal strut at cracking and ultimate loads. The effective width factor γ for the compression strut has been commonly defined as a ratio of the width of the equivalent diagonal strut to the net diagonal width of the infill panel as given by

$$\gamma = w_e/w_d = w_e/(d\sin(2\theta)) \quad (1)$$

where w_e = effective width of the equivalent diagonal strut; d = diagonal length of the infill; and θ = slope of the diagonal. Previous investigations (4,5) have shown the effective width factor to be a function of the relative stiffness of the infill and the boundary frame. The length of contact between the square infill subjected to diagonal compression and the boundary members has been suggested (5) as

$$\alpha = \frac{\pi l}{2\lambda l} \quad (2)$$

where l = length of frame members bounding the square infill; and λl = a non-dimensional parameter for the relative stiffness of frame and infill similar to that used in beam on elastic foundation theory.

Mainstone (4) extended the above procedure to rectangular infill walls. He expressed the lateral stiffness of a single story frame as $K_f = (mE_cI_c)/H^3$ where the coefficient m depends on the ratio of beam to column stiffness, and varies between 6 for a very flexible beam to 24 for a very stiff beam. The lateral stiffness of the infill expressed as stiffness of the diagonal strut is $K_i = E_it\sin 2\theta$. The stiffness ratio of the infill panel and the frame is expressed as

$$R = \frac{H^4E_it\sin 2\theta}{mE_cI_cb} \quad (3)$$

where H = story height; E_i = modulus of the infill; t = thickness of the infill; θ = angle between beam and diagonal strut; E_c = modulus of column; I_c = moment of inertia of column; and b = height of the infill panel. The effective width of the diagonal compression strut, has been proposed (4) as a function of infill to frame stiffness ratio in the form of $\gamma = A(R)^B$ in which the coefficients A and B are calibrated from experimental results. For infill walls bounded by reinforced concrete members, the effective width factors suggested by Mainstone (4) are

$$\gamma_{ek} = A_{ek}\left(\frac{H^4E_it\sin 2\theta}{E_cI_cb}\right)^{-0.1} \quad (4)$$

$$\gamma_{eu} = A_{eu}\left(\frac{H^4E_it\sin 2\theta}{E_cI_cb}\right)^{-0.1} \quad (5)$$

$$\gamma_{ec} = A_{ec}\left(\frac{H^4E_it\sin 2\theta}{E_cI_cb}\right)^{-0.22} \quad (6)$$

where $\gamma_{ek}, \gamma_{eu}, \gamma_{ec}$ = equivalent width factor for effective secant stiffness, ultimate strength, and first-cracking strength of the infill, respectively; A_{ek}, A_{eu}, A_{ec} = 0.20, 0.192, 0.76, respectively, for brick infill and 0.133, 0.288, 1.14, respectively, for concrete.

FINITE ELEMENT ANALYSIS

The lateral load response of the test RC frames with masonry infill was studied analytically with finite element analysis. Eight node quadratic elements were used to model the infill panel and the bounding members. The interface between infill and the bounding members was modeled with gap elements. The stress contours in the panel under lateral loading (Fig. 1) clearly indicate the diagonal compression strut as a primary mechanism of shear transfer in the infill panel.

Solid Infill Panel

The effect of column stiffness, infill thickness, beam stiffness, and aspect ratio of the infill on the width of the diagonal strut was investigated. As shown in Fig. 2, the effective width of the diagonal strut decreased as the infill thickness increased. Furthermore, the effective widths at initial stiffness and at ultimate strength are quiet different. Variations in the column stiffness did not significantly influence the effective width of the infill panel (Fig. 3) compared with that predicted by Eq. 6. Mainstone's empirical approach neglected the stiffness of beams in determining the effective width factors. As shown in Fig. 4, the finite element analysis confirmed that the effective width factor increased only slightly with the increase in the beam stiffness. Figure 5 shows variation of the effective width factor with respect to the infill aspect ratio as represented by the angle of the diagonal with the beam. For different aspect ratios of the infill, the factor $H^4E_it\sin 2\theta/(E_cI_cb)$ was kept approximately constant by adjusting the column moment of inertia. The finite element analysis gave effective width factors which are quite different from those obtained by Mainstone's equations. The effective width factors as calculated by Klingner (3) are also plotted in Fig. 5. His approach, which is also based on Mainstone's formulation, gives correct effective width factors for square infill panels only.

Based on the finite element analysis results, the effective width for the initial stiffness of the infill is calibrated as

$$\gamma = 0.32\sqrt{\sin 2\theta}\left(\frac{H^4E_it}{mE_cI_cb}\right)^{-0.1} \tag{7}$$

where $m = 6(1 + 6\,\mathrm{atan}\,(E_bI_bH/(E_cI_cL))/\pi)$.

Infill Panel with Opening

The effect of openings in infills on strength and stiffness of reinforced concrete frames was also studied. When the opening is relatively small, as in Fig. 6, the transfer of shear is still possible with a diagonal strut. However, when the opening is relatively large, as in Fig. 7, the diagonal compression strut mechanism cannot develop. The effect of various sizes of concentric openings in infills on the effective width factors was investigated. Based on the finite element results, reduction factors are proposed for the effective width to account for the openings of various aspect ratios in the infill panel. The effective width reduction factor is defined as

$$\kappa = \frac{w_{eo}}{w_e} \tag{8}$$

where w_{eo} = effective width of infill panel with opening, and w_e = effective width of infill panel without opening. The reduction factors κ for an infill panel with a rectangular opening having centroid at the same location as that of the infill panel are shown in Fig. 8. These factors are plotted in terms of the square of the ratio of areas A_d enclosing the opening to the total area of the infill A_t as illustrated in the figure. The reduction factor for the effective width for an infill with an opening are determined by

$$\kappa = 1 - \left(\frac{A_d}{ab}\right)^2 \tag{9}$$

$$A_d = ab - \frac{(d\sin(2\theta) - d_o\sin(\theta + \theta_o))^2}{2\sin(2\theta)} \tag{10}$$

where $d = \sqrt{a^2 + b^2}$; $d_o = \sqrt{a_o^2 + b_o^2}$; a = width of infill; b = height of infill panel; a_o = width of opening; b_o = height of opening; θ = angle between diagonal of infill and beam; and θ_o = angle between diagonal of opening and horizontal. When the opening within the infill extends across the full width or height of the panel, the effective width should be conservatively taken as zero.

CONCLUSIONS

Based on the simulated response of concrete frames with masonry infills under lateral loading, the following conclusions are drawn:

1. Masonry infills in reinforced concrete frames subjected to lateral loading can be reasonably modelled with a diagonal compression strut. An equation is proposed to calculate the effective width for initial stiffness of the masonry infills.

2. Aspect ratio of the infill has the most effect on the effective width of the diagonal compression strut. A square panel has the largest effective width which decreases with increase or decrease in the infill aspect ratio.

3. Increasing the stiffness of columns and beams results in a larger infill effective width. The effective width is more sensitive to the stiffness of the columns than the stiffness of the beams.

4. The opening in the infill panel significantly reduces the effective width of the diagonal strut. A reduction factor for the effective width is proposed to account for the opening in infills.

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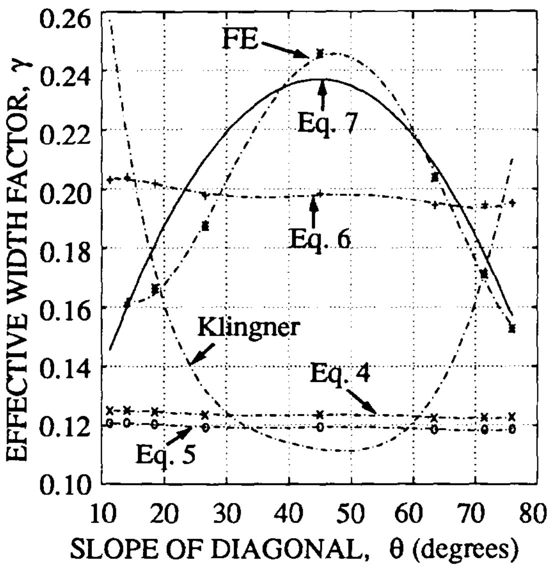

Fig. 5 Effective Width vs. Slope of Diagonal

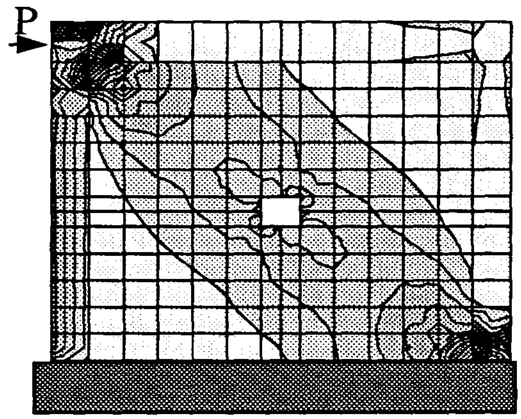

Fig. 6 Infill with Small Opening

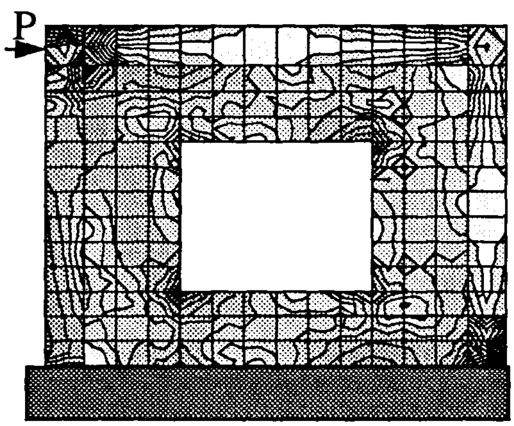

Fig. 7 Infill with Large Opening

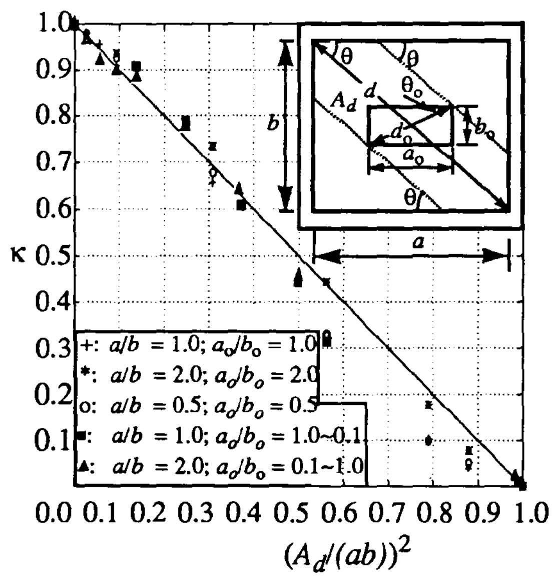

Fig. 8 Reduction Factor vs. Area Ratio

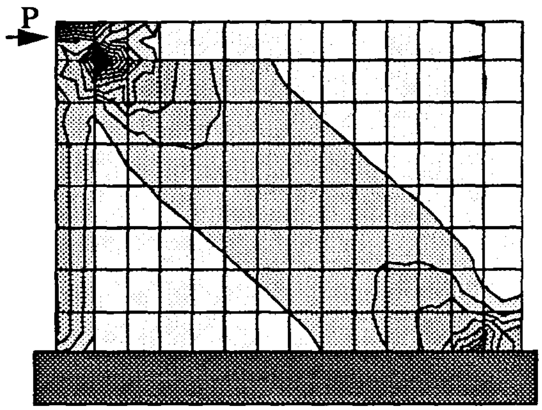

Fig. 1 Stress Contour Plot for Solid Infill

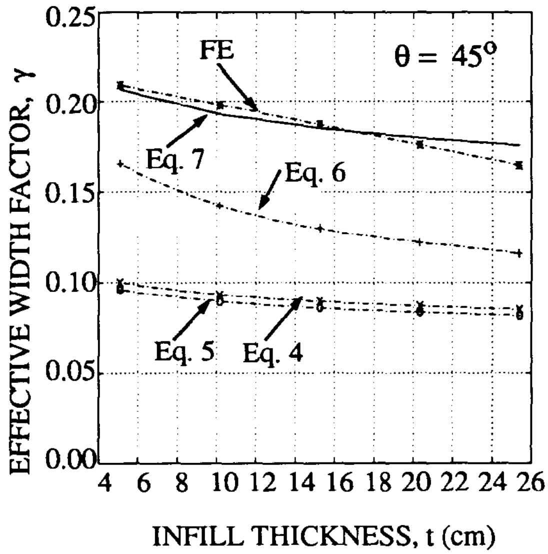

Fig. 2 Effective Width vs. Infill Thickness

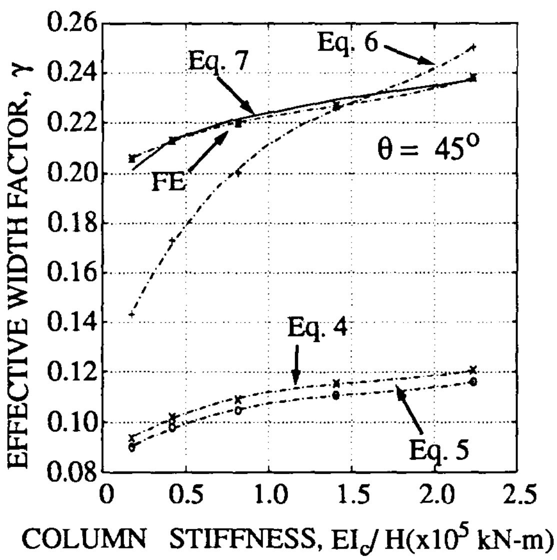

Fig. 3 Effective Width vs. Column Stiffness

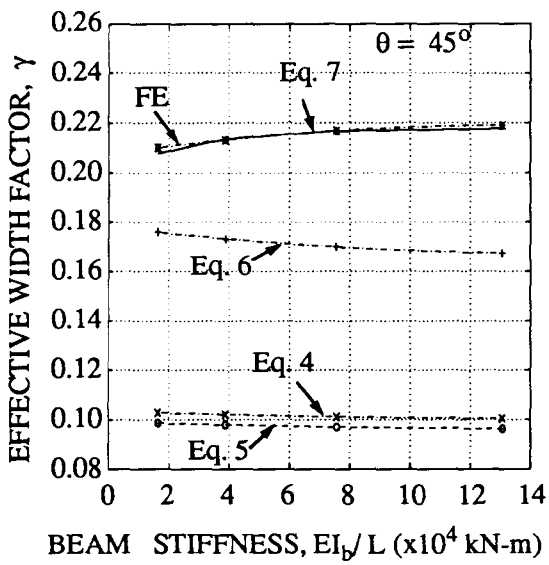

Fig. 4 Effective Width vs. Beam Stiffness

OUT-OF-PLANE STRENGTH EVALUATION OF URM INFILL PANELS

Richard Angel[1] and Daniel P. Abrams[2]

ABSTRACT

An out–of–plane strength evaluation procedure for unreinforced masonry infill panels in both undamaged and damaged states is presented. The evaluation method was based on an analytical model that considers the development of arching action in panels when subjected to out–of–plane loadings. Strength estimates are compared to a series of experimental results carried out on full–scale specimens. Test specimens consisted of clay or block infills in a reinforced concrete frame.

INTRODUCTION

Masonry infills are stiff and brittle elements that often attract large lateral story shears when loaded parallel to their plane. Following a severe earthquake, an *x* crack pattern extending to the corners may be found. This crack pattern is the result of large in–plane stiffness, but small in–plane diagonal tensile strength. The probability is high that a lighter earthquake may occur and shake a cracked infill panel loose from its surrounding frame with inertial forces applied normal to its plane. The *x* pattern of cracks resulting from in–plane forces is similar to the crack pattern for a square panel subjected to out–of–plane forces. This implies that the transverse strength can be substantially weakened by in–plane cracking. Because of this, evaluation of out–of–plane strength for a cracked infill is often surmised to be quite small, and repair measures may be prescribed unnecessarily.

Past research on out–of–plane strength of unreinforced masonry infills has shown that arching effects may be dominant for panels that are restrained at their edges by relatively stiff frames, or through continuity with an adjacent infill. The ultimate limit state of an infill panel has been found to be precipitated by the failure in compression of the different panel segments along the edges.

A method is presented for determining the transverse uniform pressure that cracked or uncracked masonry infill panels can resist. The method is based on arching action for a strip of infill that spans between two rigid supports. If panels are located in adjacent bays or stories, then by continuity, rotations at boundaries may be considered to be fully restrained.

A research project was undertaken at the University of Illinois to examine losses in transverse strength resulting from in–plane shear cracking for unreinforced masonry infills. Full–scale, single–story, single–bay reinforced concrete frames were constructed, and filled with clay brick or concrete block masonry. Test specimens were first subjected to in–plane lateral forces until masonry infills cracked in shear. Then, the same panels were subjected to normal pressures using an air bag. Estimates of transverse strength and behavior are determined using the analytical model. This paper summarizes the evaluation procedure, and presents correlations between measured and calculated behavior.

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EXPERIMENTAL PROGRAM

The experimental program consisted of testing unreinforced clay and concrete masonry infills that were placed within a concrete frame as shown in Fig. 1. The concrete frame was designed according

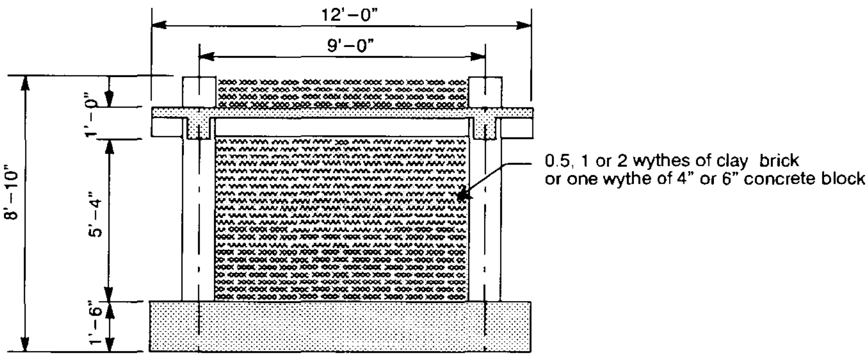


Fig. 1 Dimensions of Test Specimen

to the 1989 ACI–318 requirements so that it was both ductile and tough when subjected to load reversals. The lateral frame strength was higher than the in–plane shear strength of the strongest infill so that any frame–infill interaction was minimized.

Static, in–plane lateral forces were applied at the center of the beam span until cracking in the masonry infill. To assure that a fully cracked condition was reached, cycles of reversed shears were continued until lateral deflections were twice that observed at first cracking. The amount of in–plane shear required to crack an infill was representative of the shear force that would be developed at the base story of a multistory building.

Following the in–plane loading, panels were subjected to pressures applied across their plane using the air–bag arrangement. Pressures were increased monotonically until ultimate loads were reached. Unlike the in–plane test, the out–of–plane test simulated the condition at the top story of a building where lateral accelerations would be the largest, and no continuity would be present with the panel above it.

A total of eight infill specimens were tested. Parameters of the study were the type of unit, the h/t ratio for the infill and the mortar type. Both clay brick and concrete block infills were tested. The clay units were a low strength reclaimed brick (Chicago common) laid in a single wythe running bond. Concrete units were standard 4–inch or 6–inch blocks laid in a single wythe running bond. A typical Type N mortar (a 1:1:6 mix of Portland Cement, lime and sand) was used as the control mortar. Another mortar comprised only of lime and sand (1:3) represented mortars used at the earlier part of the century.

ANALYTICAL MODEL

An infill panel was idealized as a strip of unit width that spans between two supports fully restrained against translation and rotation. A uniformly distributed lateral load was applied normal to the plane of the panel. Because of a previous in–plane loading, the panel was considered cracked in an x pattern. This was modeled with the worst case situation using a unit one–way strip that was cracked at mid–span. Cracking separated the strip into two segments that rotate as rigid bodies about their supported ends as shown in Fig. 2. Although the tensile strength of the panels was neglected and formation of cracks was not important for estimation of the out–of–plane strength of the panels, the deterioration in the infill caused by the repetitive cyclic in–plane loading varied the out–of–plane behavior and strength of the panels. A factor to account for this effect is developed later in the paper.

The uniform lateral load, w, was estimated based on statics. The free body diagram for the lateral load resisting mechanism is presented in Fig. 2. As shown in Fig. 2, the direction of the thrust force with

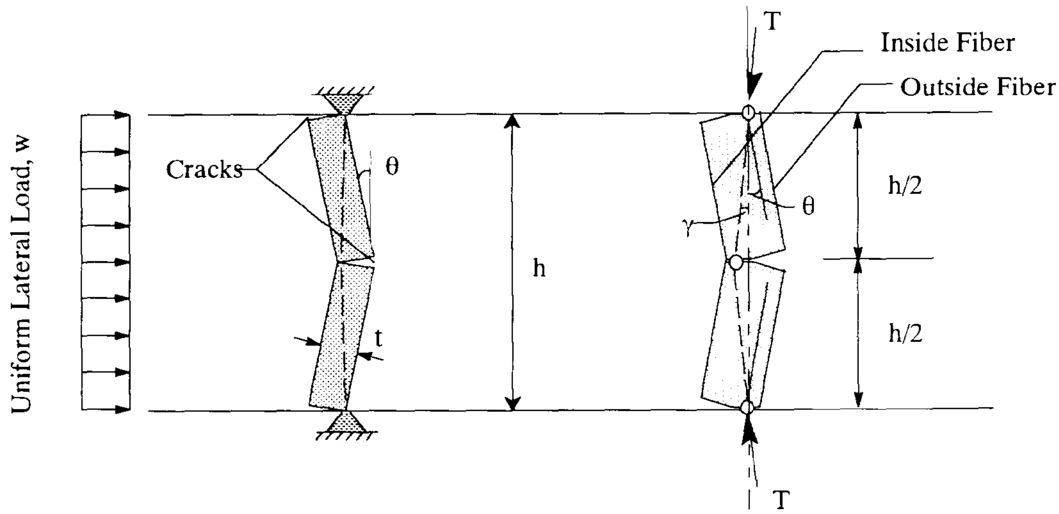


Fig. 2 Idealized Loading and Behavior of Unit Strip of Infill Panel

respect to an undisturbed vertical reference line, γ, is dependent on the rotation of the half span, θ, and on the location of the thrust resultant. The centroid of the force was dependent on the bearing width, b, and on the compressive stress distribution along this width. Therefore, the primary variables for panel strength were γ, b, and θ. These variables were functions of the compressive edge strain at the support, and the distribution of strain along the height.

The uniform transverse load can be related to the thrust force by summing horizontal forces that act at the mid–span hinge (Fig. 2). If the thrust force is equated to the internal compressive force, then expression Eq. [1] can be obtained relating the load, w, to the maximum compressive stress at the support. Eq. [1] is valid only for small angles. The expression considers only the component of the forces developed by thrust in the arch, excluding the minimal contribution by flexure as a beam. Any developed flexural stresses in the segments of the beam are at most an order of magnitude smaller than the developed axial stresses forming the thrust in the arch. This can be observed by summing moments at the boundary of the beam segment and considering the large difference in the lever arms of the component of the thrust force, and the applied lateral load. The term f_b is the maximum compressive

$$w = \frac{4\ k_1 \left(\frac{b}{t}\right) f_b \frac{\cos\gamma}{\cos\theta} \sin\gamma}{\left(\frac{h}{t}\right)} \qquad \text{Eq. [1]}$$

stress at the support, and may be determined from the corresponding strain if the stress–strain relation for the masonry in compression is known (k_1 represents the ratio of peak stress to average stress in the masonry). The strain, ε_{max}, can be expressed in terms of the total shortening along the outside face which is the variable that is used to determine the angles, γ and θ, and the compressed width, b.

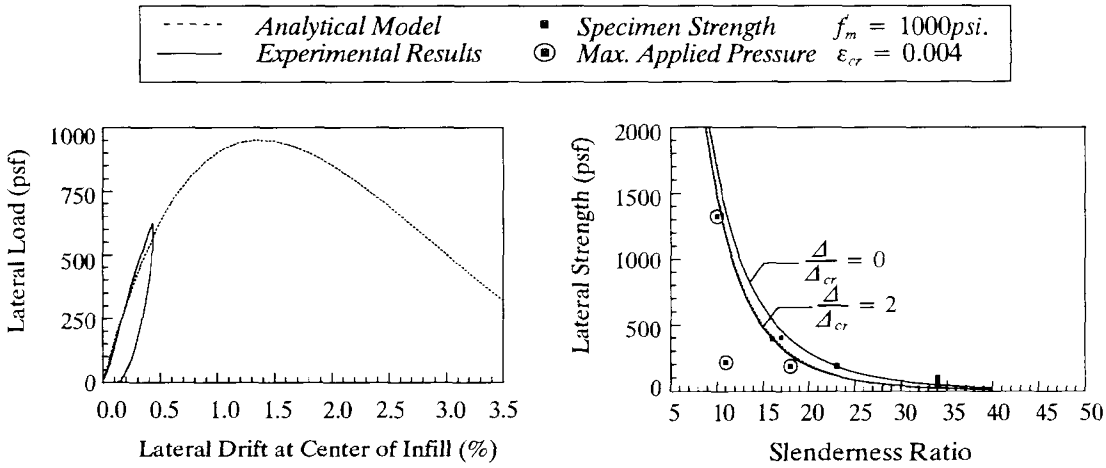
IN–PLANE CRACKING EFFECTS

Out–of–plane capacity decreased with in–plane cracking. Based on the experimental results, the theory was modified to account for the in–plane damage previously done to the panels.

The out–of–plane strength of the panels was reduced by the amount of in–plane damage. For the same amount of in–plane damage, the out–of–plane strength reduction varied with the slenderness ratio of the panels. The reduction factor was evaluated as the panel strengths calculated based on the modified model for in–plane cracked panels normalized to the strength of the panel in a virgin state. The strength reduction caused by the in–plane damage was not linearly related to the slenderness ratio. Slender infills were greatly affected by in–plane damage. The strength for these slender panels can be reduced by a factor of two. Experimental results support this observation. According to this model, the out–of–plane strength of infills with a lower slenderness ratio are not affected as much by in–plane damage.

CORRELATION WITH EXPERIMENTAL RESULTS

The out–of–plane strength of a series of panels exceeded the capacity of the loading rig. The response observed during the testing of the specimens is compared to their corresponding analytical predictions as evaluated from the analytical method, and results are presented in Fig. 3(a). Comparing the



(a) (b)

Fig. 3 Predicted Behavior

experimental results to the estimated analytical results shown in this figure indicates that the initial behavior and stiffness of the panels was well predicted. Based on initial panel behavior, and on the type of behavior observed throughout the loading sequence of the panel, it is reasonable to assume that the actual strength of the panel was well predicted by the method. A reduction factor for the out–of–plane strength was included in the model to account for in–plane damage.

Results obtained using the analytical model gave out–of–plane strength values similar to the strengths obtained experimentally. Comparison between out–of–plane strengths as calculated by the analytical model and the experimental results are illustrated in Fig. 3(b). Measured strengths have been normalized to a compressive strength of 1000 psi for comparison with strength curves. Notice that some panels were not tested to failure since their strength exceeded the capacity of the loading rig. For these panels, the maximum applied pressure was recorded rather than their strength. Predicted strengths for panels with no existing in–plane damage are illustrated with $\Delta/\Delta_{cr} = 0$, while strengths for panels damaged in the in–plane direction corresponding to a maximum in–plane drift of twice the required for cracking of the panel are illustrated with $\Delta/\Delta_{cr} = 2$. The out–of–plane strength reduction obtained from the applied in–plane damage along with all the experimental results are presented in Fig. 3(b). As shown in Fig. 3(b), the strength of the panels varies with the slenderness ratio of the panel and with the amount of in–plane damage that the panel has experienced.

SUGGESTED PROCEDURE FOR ESTIMATING OUT–OF–PLANE STRENGTH OF CRACKED PANELS

Simplifications

Parameters $b/t, f'_b$, and $\sin(\gamma)$, depend on the crushing strain and on the slenderness ratio of the panel. Thus, expression Eq. [1] for the out–of–plane strength of panels can be simplified by considering a constant masonry crushing strain. The crushing strain of the masonry was taken equal to 0.004. These parameters are combined to obtain a dimensionless parameter λ (Eq. [2]). This dimensionless

$$\lambda = \left[\left(\frac{f_b}{f_m}\right)\left(\frac{b}{t}\right)\sin\gamma\right] \qquad \text{Eq. [2]}$$

$$w = \frac{2\ f_m}{\left(\frac{h}{t}\right)}\lambda \qquad \text{Eq. [3]}$$

parameter was evaluated for a range of slenderness ratios, and results are presented in Table 1. Substituting λ into Eq. [1] produces expression Eq. [3].

Table 1 Parameter Approximation

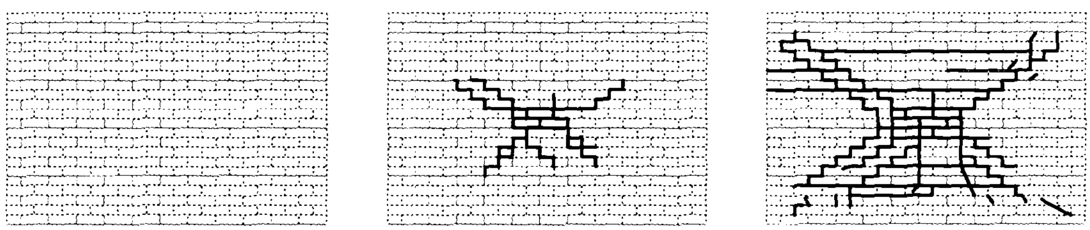
$\frac{h}{t}$	λ	R_I for ratio of Δ/Δ_{cr}	
		1	2
5	0.129	0.997	0.994
10	0.060	0.946	0.894
15	0.034	0.888	0.789
20	0.021	0.829	0.688
25	0.013	0.776	0.602
30	0.008	0.735	0.540
35	0.005	0.716	0.512
40	0.003	0.727	0.528

Analysis required for the evaluation of infill.

1.) In–plane damage assessment.

There are two methods for quantifying the amount of damage for cracked panels: 1) visual inspection which is described in detail in this paper, and 2) analysis of the maximum deflection experienced by the structure in terms of the displacement observed at cracking of the infill panel explained in detail by Angel [2].

A method used to evaluate the damage of a panel is visual inspection. Based on experimental results, visual inspection of the panel can classify the amount of existing panel damage into three different ranges as illustrated in Fig. 4. The three different cracking stages were obtained from experimental results and normalized in terms of the lateral deflection required for cracking of the infill.



$\frac{\Delta}{\Delta_{cr}} = 0$ No Damage

$\frac{\Delta}{\Delta_{cr}} = 1$ Moderate Damage

$\frac{\Delta}{\Delta_{cr}} = 2$ Severe Damage

Fig. 4 Physical Infill Cracking Damage

Out–of–plane strength reduction factors given a known amount of in–plane damage (R_I) for a range of panel slenderness ratios have been tabulated and results are presented in Table 1.

2.) Flexibility of confining frame.

Infill panels confined within frames with all sides continuous (neighboring panel in every direction) may assume to have fully restrained boundary conditions ($R_2 = 1$). For infill panels confined within frames with at least one side not continuous (neighboring panel missing on any panel direction) a reduction factor for the out–of–plane strength is applied (R_2). Evaluation of the stiffness of the smallest frame member on the non–continuous panel side should be performed, and results are to be used in conjunction with Eq. [4] and Eq. [5].

$$R_2 = 0.357 + 7.14X10^{-8}EI \qquad \text{for } 2.0E6\ k-in \le EI \le 9.0E6\ k-in \qquad \text{Eq. [4]}$$

$$R_2 = 1 \qquad \text{for } EI > 9.0E6\ k-in \qquad \text{Eq. [5]}$$

3.) Out–of–plane strength of the panel.

The out–of–plane strength of previously cracked, or uncracked infill panels within confining frames at any location of a structure may be evaluated by Eq. [6]. Values for λ for a range of slenderness ratio

$$w = \frac{2\ f_m}{\left(\frac{h}{t}\right)}\ R_1\ R_2\ \lambda \qquad \text{Eq. [6]}$$

are given in Table 1.

RETROFIT OR REHABILITATION TECHNIQUE

The rehabilitation or retrofit method recommended to increase the out–of–plane strength of the panel consists of parging a ferrocement coating to one or both faces of the infill panel. Application of the coating decreases the slenderness ratio of the panel, and also increases the compressive strength of the panel. The out–of–plane strength of the panel is then largely increased by the repair method since the strength depends: 1)linearly on the compressive strength of the material, and 2) on the square of the slenderness ratio of the panel.

The out–of–plane strength of repaired infill panels may be evaluated by Eq. [7] (R_1 is not considered because once the panel was repaired the existing in–plane damage did not affect the strength of the panel). The value for the slenderness ratio should consider the thickness of the panel once repairing

$$w = \frac{2\ f_{m-repaired}}{\left(\frac{h}{t}\right)}\ R_2\ \lambda \qquad \text{Eq. [7]}$$

has been completed. The compressive strength for the panel should the lesser of the masonry or of the repair coating. Values for λ for a range of slenderness ratios are given in Table 1.

SUMMARY

An evaluation procedure designed to estimate the out–of–plane strength of uncracked and cracked panels is presented. The strength of the panels vary with the compressive strength of the masonry, and with the corresponding slenderness ratio. Visual inspection is a preferred method to quantify the extend of the damage existing in a panel. Reduction factors are calculated to account for the amount of existing in–plane damage in the panel, and the flexibility of the frame. A rehabilitation or retrofit technique consisting of parging a ferrocement coating to one or both faces of the infill panel is recommended.

ACKNOWLEDGMENTS

The research presented in this paper was part of a study at the University of Illinois on seismic evaluation and repair of masonry infills. The project is one part of the national coordinated program on *Repair and Rehabilitation Research for Seismic Resistance of Structures* that is funded by the National Science Foundation (Grant #BCS 90–156509). The authors wish to acknowledge the laboratory assistance of Paul Blaszczyk, and SOH & Associates in San Francisco for participation in the research.

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OUT-OF PLANE STRENGTH OF MASONRY WALLS RETROFITTED WITH FIBER COMPOSITES

Mohammad R. Ehsani[†] and Hamid Saadatmanesh[†]

ABSTRACT

A new approach for seismic retrofitting of URM structures is presented where a fiber composite fabric is epoxy bonded to the wall. Results indicate that both flexural and shear strength of the wall as well as its ductility is significantly enhanced.

INTRODUCTION

Various methods for strengthening masonry walls have been studied in recent years. These usually require the addition of framing elements to reduce the loads on the walls, or surface treatments such as shotcrete to increase the strength and ductility of the walls. Such retrofits often add significant mass to the structure and are time-consuming and costly to perform.

Recent studies at the University of Arizona have demonstrated that the strength of concrete beams and columns can be significantly increased by epoxy bonding composite laminates to the critically stressed regions of these members (1,3). The method presented here is an extension of the above studies, where for ease of application, a thin flexible fabric of glass is epoxied to the masonry wall (2). The steps required in strengthening an in-fill frame, for example, include: a) cleaning the wall surface(s) and if required, filling the mortar joints flush with the surface of the wall; (b) applying a thin layer of epoxy to the wall surface(s) and the adjacent frame elements; (c) placing the composite fabric on the epoxied surfaces and pressing it firmly against the wall; and (d) applying an additional layer of epoxy to the outer surface of the fabric (Fig. 1).

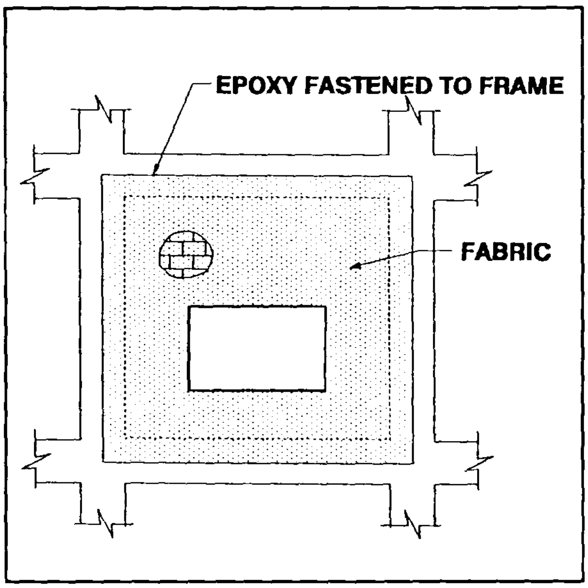

Fig. 1. Proposed retrofitting system

If desired, the edges of the fabric could be bolted to the frame using a steel angle. The surface of the wall could also be covered with plaster. This may be desirable for exterior applications to

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prevent ultraviolet damage to the epoxy. However, it is not necessary for interior walls. In fact, depending on the type of the resin used, it is possible to maintain the appearance of the walls virtually unchanged. In some of the specimens, after the fabric was attached to the wall, the only difference in the wall appearance was a slightly glossy finish to the wall surface; i.e. the clay bricks and joints remained distinctly visible.

EXPERIMENTAL STUDY

A study is currently under way to examine the feasibility of this retrofitting technique. The results for a few masonry beams and an in-plane shear test are reported here. The beams consist of 19 clay bricks, each with a dimension of 2½*4*8½ in., stacked in a single wythe (stack bond). This results in beams which are 8½-in. wide, 4-in. high and 57-in. long. The beams are loaded statically to failure with two concentrated loads over a clear span of 47 in., as shown in Fig. 2.

Each beam is identified with a combination of 4 characters. The first numeral, 1 or 2, refers to the type of epoxy. Two epoxies are being investigated. The first one is a two-component epoxy that performed exceptionally well under previous studies for strengthening of R/C beams (1). Among the features of this epoxy are its high energy absorption, resistance to high humidity, salt spray, cold and hot environments, and economy. The epoxy has a consistency similar to cement paste with a pot life of approximately ½ hour. It is fully cured in room temperature in four hours. A dual-component dispense tool was used to achieve a uniform mixture of the epoxy as it was being applied to the wall and fabric. The second adhesive being studied is also a two-component epoxy which cures at room temperature. This epoxy has a lower viscosity than the first one and can be easily spread over the wall surface with a trowel.

The letter M designates the type of mortar used in the study which consisted of portland cement:lime:sand ratios of 1:¼:3. To simulate the effect of a weaker mortar which may be found in some older structures, one specimen was constructed with a mortar designated with M^* having ratios of 1:¼:5, respectively. The next numeral, 1, 2, or 3, refers to the type of fabric used. Three different fabrics of various strength (i.e. thickness and weave) have been used to investigate the possibility of achieving various modes of failure, such as tension failure of fabric, or compression failure of brick, etc. The last letter (F or S) refers to the overall roughness of the wall where the fabric is attached. The intent was to investigate the effect of the surface finish on bonding of the composite fabrics. In both cases, the fabric was epoxied to the smooth surface of the brick. In one case, however, the mortar joint was flush with the outside surface of the wall (F); in the other case, a small amount of mortar extruded from the joints (S).

All specimens were cast with new clay bricks. However, because the age of bricks may influence their bonding characteristics to the epoxy, one specimen (1M2S-1) was cast with reclaimed old bricks. The results for six beams which have been retrofitted and tested are presented here.

Materials

As mentioned earlier, two types of mortar were used in this study. Two- by four-inch cylinders of the mortar were tested at 28 days and the compressive strength was calculated as 4650 and 4100 psi for Type M and M^* mortars, respectively. Prisms were also constructed with the new brick and Type M mortar. The 28-day strength of the prisms was calculated as 1870 psi. The prisms failed by compression failure of the bricks; consequently, the slight change in the mortar strength did not have a significant effect on the overall strength of the specimens.

Three types of fabrics were used. The first one was a fiberglass fabric with an acrylic polyvinyl finish which comprises about 6-10% of the product weight. The fabric weighs 5.6 oz/yd^2 and had a visual 2x4 yarns/in. construction in the machine (warp) and cross-machine (fill) directions. According to the manufacturer, the tensile strength of the fabric as determined by ASTM-D579 3-inch jaw separation at a cross-head speed of 12 in./min. was 220x270 lbs/in. in the weak and strong directions, respectively. This fabric was epoxied to the specimens with the strong direction being parallel to the length of the beam. The second and third fabrics were unidirectional E-glass. Five samples of each fabric were tested by the manufacturer in accordance with the out strip method of ASTM-D1682. The results indicated that the second fabric had 11.3 yarns per inch and a tensile strength of 1422 pounds per inch. The corresponding numbers for the third fabric were 10 and 855.

Test Results

The beam specimens were subjected to four-point bending as shown in Fig. 2. Before discussing the results, it is interesting to note that when placed horizontally in the testing frame, the test specimens would normally fail under their self weight of approximately 125 pounds. Therefore, prior to strengthening, the specimens had to be handled very carefully. Plots of load vs. midspan deflection for the beam specimens are presented in Fig. 2. The first fabric, used in Specimen 2M1S, was relatively weak. Nonetheless, the specimen carried a maximum load of 700 lbs. and a deflection of 0.27 in. The ultimate load was governed by tension failure of the fabric. Based on this test it was decided to utilize stronger fabrics in the remaining tests.

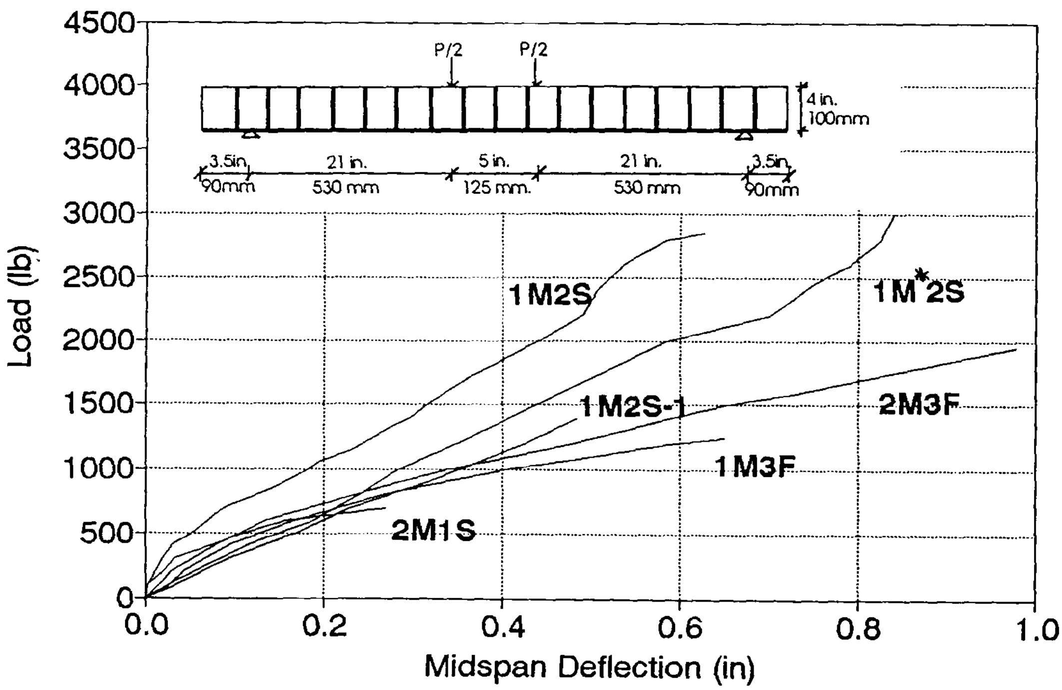
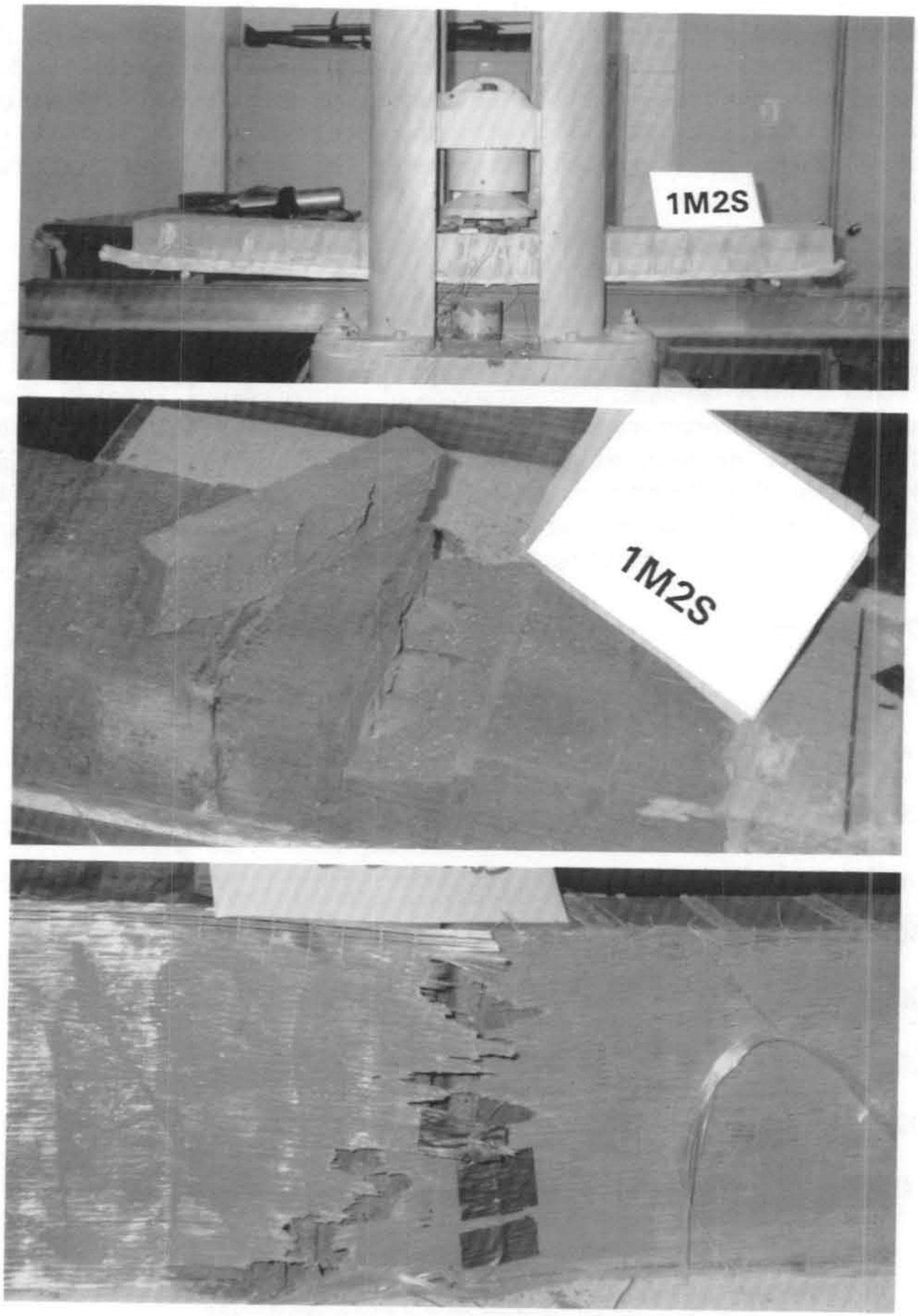


Fig. 2. Load vs. deflection for beam specimens

The influence of the strength of the fabric can be readily seen by comparing Specimens 1M2S and 1M3F, both retrofitted with the same epoxy (i.e. Type 1). The thicker fabric in 1M2S resulted in a failure load of 2850 lbs. and a deflection of 0.63 in. Failure was initiated by compression crushing of the bricks near the top of the beam, followed suddenly by diagonal cracking of the beam in the shear span (Fig. 3). Specimen 1M3F had a smaller stiffness due to the thinner fabric used. This specimen reached a maximum load of 1320 lbs. and a deflection of 0.65 in. At that point, the fabric failed in tension (Fig. 3).



(a)

(b)

(c)

Fig. 3. Beam 1M2S a) during and b) at conclusion of test; c) Beam 1M3F at conclusion of test

The performance of the second epoxy was superior to that of the first one. This is evident from comparison of the results for Specimens 1M3F and 2M3F. Both specimens were retrofitted with the lighter E-glass fabric. The performance of Specimen 1M3F was discussed above. Specimen 2M3F had a higher stiffness and reached a load of 1950 lbs at a deflection of 0.98 in., or 1/48 times the span. Both specimens failed by tension failure of the glass fabric. However, the additional load carried by Specimen 2M3F is attributed to the type of epoxy used in this specimen.

Comparison of Specimens 1M2S and 1M2S-1 can reveal information on the performance of the two types of brick used. Specimen 1M2S, constructed with new brick, had a larger stiffness and failed at a load of 2850 lbs. Specimen 1M2S-1, which was constructed with old reclaimed brick, failed at a load of 1400 lbs and at a deflection of 0.48 in. Due to the large thickness of the fabric used, both of these specimens failed by compression failure of brick. Although no prism tests were performed for the reclaimed brick, it is believed that the lower strength of this brick resulted in the lower failure load for the specimen.

The effect of the mortar strength appeared to be negligible in these specimens. Specimen 1M2S with the stronger mortar failed at a load of 2850 lbs. while its companion specimen with weaker mortar, $1M^{*}2S$, failed at a load of 3000 lbs. Both of these specimens were retrofitted with the thicker fabric and failed by compression failure of the masonry. In masonry prism tests, it was observed that failure was initiated by compression failure of the brick rather than the mortar. Consequently, the slight difference in the strength of the mortar in these two specimens did not change the mode of failure and the maximum load carried by both specimens were comparable.

Examination of the specimens during and after the tests indicated that none of them exhibited any visible sign of slip or bond failure at the epoxy/fabric interface.

In addition to the flexural tests described above, shear tests are also being conducted on specimens confined with a very thin composite fabric, having a strength of 50 and 70 lb/in. in the two orthogonal directions. The fabrics are attached to both sides of the specimens with a resin which becomes transparent after curing. Thus, it is very difficult to distinguish the fabric on the specimen. One test result is presented in Fig. 4. The specimen failed by formation of a longitudinal crack parallel to the line of action of the compressive force. However, at that point, the share of load carried by the fabric increased, resulting in a more ductile behavior. The improved behavior shown in Fig. 4 is greatly influenced by the strength of the fabric and is being studied.

FURTHER STUDIES

For both flexural and shear strengthening of walls, the connection of the fabric to the framing elements can be achieved by epoxy or a combination of epoxy and mechanical connectors such as steel angles and bolts. While a great deal of data is available on strength of epoxies in tension, little is known on their performance under tensile stresses perpendicular to the bond surface. The strength and ductility of these connections has a significant effect on the overall success of this technique. Another concern is the long-term durability of epoxies, specially when subjected to adverse environmental conditions. These topics are under investigation at the University of Arizona.

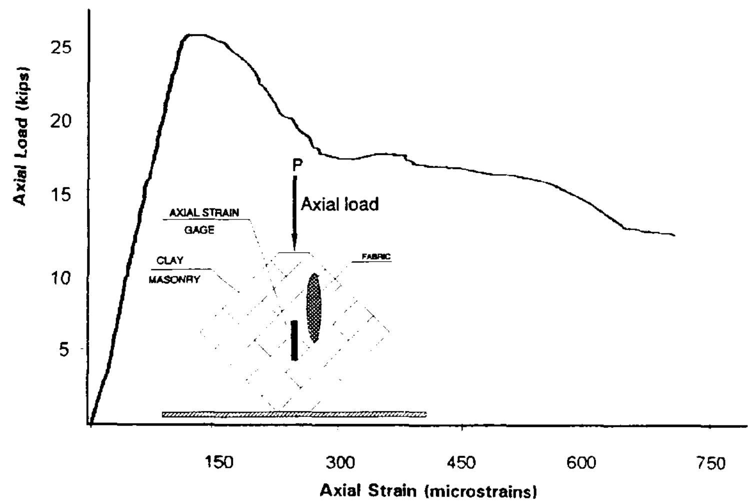


Fig. 4. Load vs. axial strain for the retrofitted panel

CONCLUSIONS

The test results indicate that retrofitting of unreinforced masonry structures with composite fabrics is a very effective technique for increasing the flexural strength and ductility of these elements. The specimens tested, carried loads more than twenty times their own weight and exhibited large deflections, in excess of 1/50 times the span. The strength of the fabric controlled the mode of failure. When lighter fabrics are used, the maximum load is that causing tension failure of the fabric. When stronger fabrics are used, the fabrics maintained the integrity of the specimen until the compressive strength of the brick was reached. The limited data also point to the effectiveness of this technique in increasing the strength and ductility of URM subjected to shear forces. Considering the ease of application of this method, it appears to have a great potential for seismic retrofitting of masonry structures.

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PHYSICAL AND ANALYTICAL MODELING OF BRICK INFILLED STEEL FRAMES

J.B. Mander[1], L.E. Aycardi[2] and D.-K. Kim[2].

INTRODUCTION

The behavior of infilled frames have been studied for the past four decades, yet no consensus has emerged leading to a unified approach for either their design or strength and ductility evaluation. The major parameters found to be important affecting the behavior of infilled frames are: strength, stiffness, hysteretic energy absorption characteristics, boundary conditions, distribution of strains and stresses within the infill panel, induced forces on the frame, initial lack of fit, openings and types of construction. One of the purposes for this study was to experimentally investigate the inelastic behavior of brick masonry infilled frames so that improved modeling can be developed for (i) the design of new structures with infilled frames; (ii) using infills to retrofit existing seismically vulnerable frames; and (iii) evaluation of strength and ductility capability of existing infilled frames before and after retrofitting.

IN-PLANE EXPERIMENTAL STUDY

The in-plane experimental research involved the testing of three clay brick masonry infilled frame sub-assemblages constructed from bolted steel frames, and tested under quasi-static cyclic loading. Full details of the in-plane part of the experimental research are summarized in Ref. [1]. Specimen 1 was tested, then repaired with ferrocement and retested. Specimen 2 was initially retrofitted with ferrocement, then tested. Specimen 3 was tested similar to Specimen 1, except an enhanced ferrocement overlay was used which included diagonal rebars. Fig. 1 shows a typical structural frame in which infill walls have been placed. It is generally the first and/or second story infill that is of concern under lateral earthquake loading as high story shears may cause distress in those elements. To model such critical regions under lateral story drifts (Fig. 1 (a)) a symmetrical substructure has been abstracted from the frame (Fig. 1 (b)). Under lateral load the substructure is doubly antisymmetric as shown in Fig. 1 (c). This idealized form of behavior was the starting point in the physical modeling scheme adopted in this study. The outer half-bays which may also contain infills, were replaced with pin-jointed diagonal braces whose stiffness was similar to the infill itself. Thus the boundary conditions within the test panel are similar to the prototype construction, where the plastic hinges form at the beam ends (or joint connections) and diagonal compression struts form within the infill.

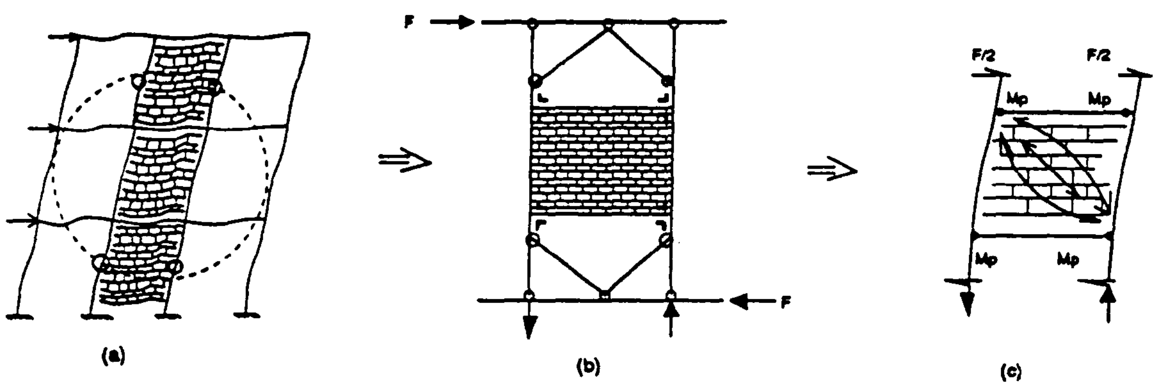

Fig. 1: Brick Infills in (a) Structure Under Lateral Loading (b) Experimental Subassemblage (c) Boundary Conditions of Subassemblage.

Each test specimen consisted of a steel frame with a central bay infilled with bricks. Beams were connected to the columns by bolted semi-rigid (top and bottom seat) connections. The strength of these connections was so designed such that their capacity was about 50% of the connecting members. Thus under lateral loading frame yielding was concentrated in the angles preserving the principal members from being damaged. Single wythe clay brick masonry infills were laid snug-fit in the central bay of each specimen. Structurally engineered ferrocement overlays were used to either repair or retrofit each specimen. The ferrocement overlays consisted of a mortar-like matrix with sand passing a No. 8 sieve mixed with a water : cement : sand ratio of 0.5:1:2 . A 13 *mm* thick ferrocement overlay was added to one side of the repaired and retrofitted infills (Specimens 1 and 2 respectively), as shown in Fig. 2 (a).

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A $13\,mm \times 13\,mm$ galvanized steel wire reinforcing mesh was fixed in the center of the overlay by means of 6 mm diameter concrete anchor bolts. The anchor bolts had a tensile pull-out strength of $4.5\,kN$ from the bricks. This anchorage system was designed to allow some relative in-plane panel movement when the coating separates from the infill. Based on results from tests on Specimens 1 and 2, it was concluded that a thicker overlay and a more densely spaced anchor bolt pattern should further enhance the energy absorption capacity. Thus for Specimen 3, a $25\,mm$ overlay with two layers of mesh, one diagonal and one vertical, was adopted. The anchor bolt placement is shown in Fig. 2 (b). A pair of $10\,mm$ reinforcing bars were also placed along each diagonal in order to lessen the concentration of large diagonal cracks observed in Specimens 1 and 2.

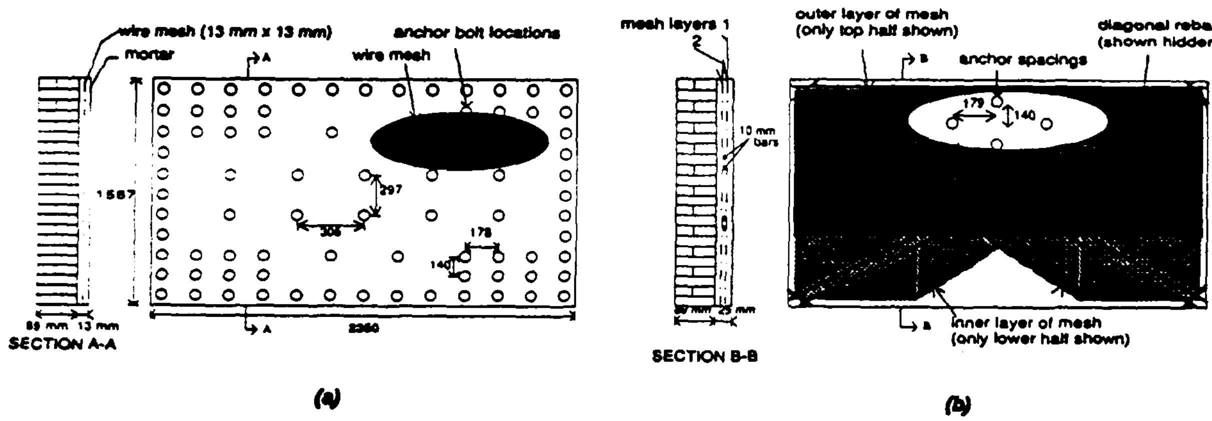


Fig. 2: Ferrocement Overlays used in (a) Specimens 1 and 2, and (b) Specimen 3.

The specimens were tested by applying lateral load at the top beam with a $1100\,kN$ actuator which was connected to a stiff reaction frame. The specimens were tested under cyclic lateral load in drift control with a cyclic sine wave frequency of $0.01\ Hz$ and a data recording frequency of $1\ Hz$. The displacements were measured using displacement transducers attached to the top and bottom of the steel beam surrounding the infill and the top and bottom beams of the test frame. The joint rotations were monitored by using linear potentiometers.

Specimen 1 was tested firstly as an ordinary frame at increasing amplitudes of cyclic loading. Fig. 3 (a) shows the lateral load-drift results for this initial phase of testing. It can be seen that the hysteretic curves show good energy dissipation characteristics, with only a modest drop in strength on the second cycle of loading.

The second phase of testing Specimen 1 involved repairing the infill by coating the bricks with the $13\,mm$ thick ferrocement overlay, and then retesting. This consists of two complete cycles of reversed load at an interstory drift amplitude of $\pm1.5\%$. Fig. 3 (b) shows the load-interstory drift response. The purpose of the ferrocement retrofit was two fold: to provide some out-of-plane membrane stiffening action to inhibit fall-out; and to provide some additional in-plane energy dissipation capacity. The same displacement history to was used as for Specimen 1, except two additional cycles were applied at $\pm1.5\%$ drift. Compression cracks were first observed during the $\pm0.75\%$ drift cycles. Diagonal tension cracks appeared at the center of the ferrocement panel during the $\pm1.0\%$ drift cycles. It was at this stage the composite infill panel commenced to walk-out of the steel frame during loading cycling. During the first $\pm1.5\%$ drift amplitude the cracks at the center of the infill panel widened considerably exposing the reinforcing mesh due to out-of-plane buckling between the concrete anchors along the compression diagonal.

Specimen 1 was repaired with a ferrocement overlay in a similar fashion to the retrofit of Specimen 2. Comparing the results from this test shows that the presence of the ferrocement provided only slight strengthening and additional energy dissipation. However it was evident that damage to the brick infill was deferred by way of the ferrocement.

Specimen 3 was tested in three phases. Phases I and II consisted of an ordinary brick infilled frame with and without external diagonal rebars tightened to take tensile loads under lateral loading. Fig. 4 (a) shows the experimental load-drift results for Phase I testing where under negative loading, one pair of rebars were active. It can be seen that under reversed loading the tensile contribution from the diagonal rebars added $80\,kN$ to the apparent shear strength capacity of the panel system. If the component of lateral load contributed by the diagonal rebars at yield is equal to about $56\,kN$, then it is evident that the diagonal tension in these bars also provided some confining action to the diagonal compression strut, thus enhancing the strength capacity of the masonry infill.

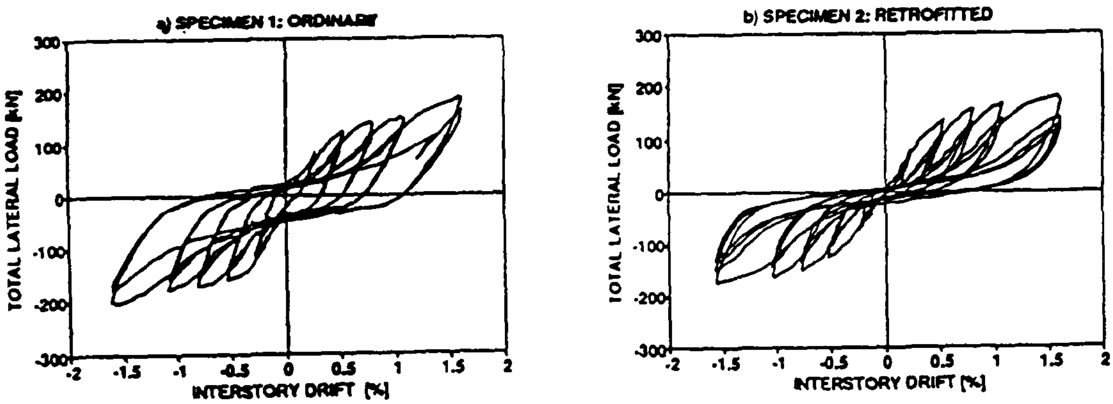


Fig. 3: Load-Interstory Drift Response (a) of Ordinary Infilled Frame and (b) Retrofitted Infill

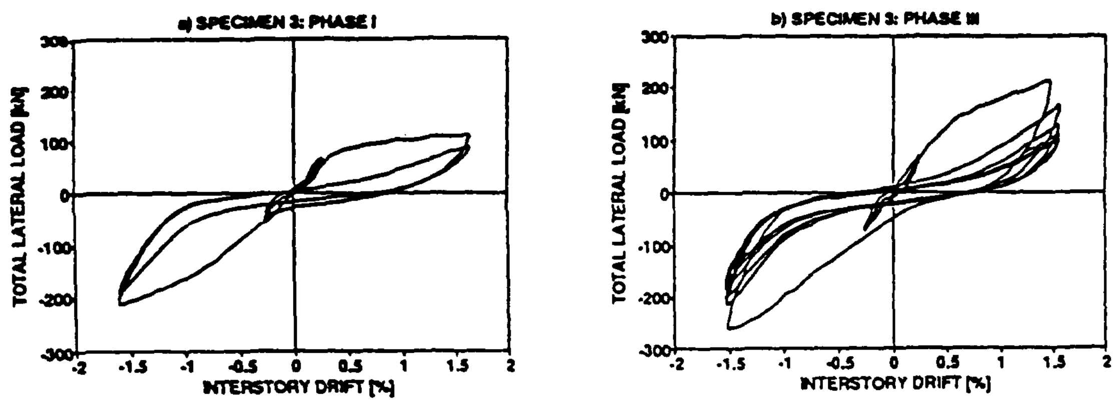


Fig. 4: Experimental Load-Drift Results from Specimen 3, Phase I and III.

The Phase III portion of testing Specimen 3 involved repairing the infill by coating the bricks with the $25\,mm$ thick ferrocement overlay that included new diagonal reinforcement within the coating, and then retesting. The purpose of including the diagonal rebars was three-fold: to provide some additional lateral load capacity by direct tension; to provide some confining action to the bricks as observed in Phase I; and to finely distribute the diagonal tension cracks across the infill. The retest results are shown in Fig. 4 (b). Comparing Phase III results with the previous two results shows that the enhanced ferrocement overlay increased the lateral load capacity by $100\,kN$. There was also a considerable increase in hysteretic energy on the first cycle of loading. Although the strength capacity continued to decay with subsequent cycles of lateral load, the shape of the hysteretic loops appear to have stabilized.

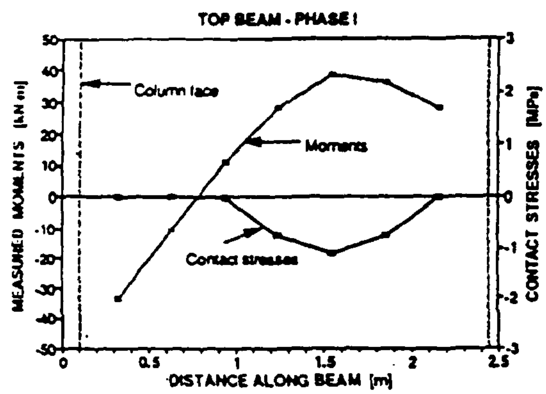


Fig. 5: Distribution of Contact Stresses and Moments

IN-PLANE ANALYTICAL MODELING STUDY

Contact stresses between the brick infill panel and the steel beams are calculated from the implied moments for each sub-test of the specimens for the final $\pm 1.5\%$ drift cycle. The finite difference method which employed forward, central and backward differences at appropriate nodes of the beams' strain gauge pairs was used to obtain inferred contact stresses. Fig. 5 shows a plot of the implied moment and corresponding contact stress distribution.

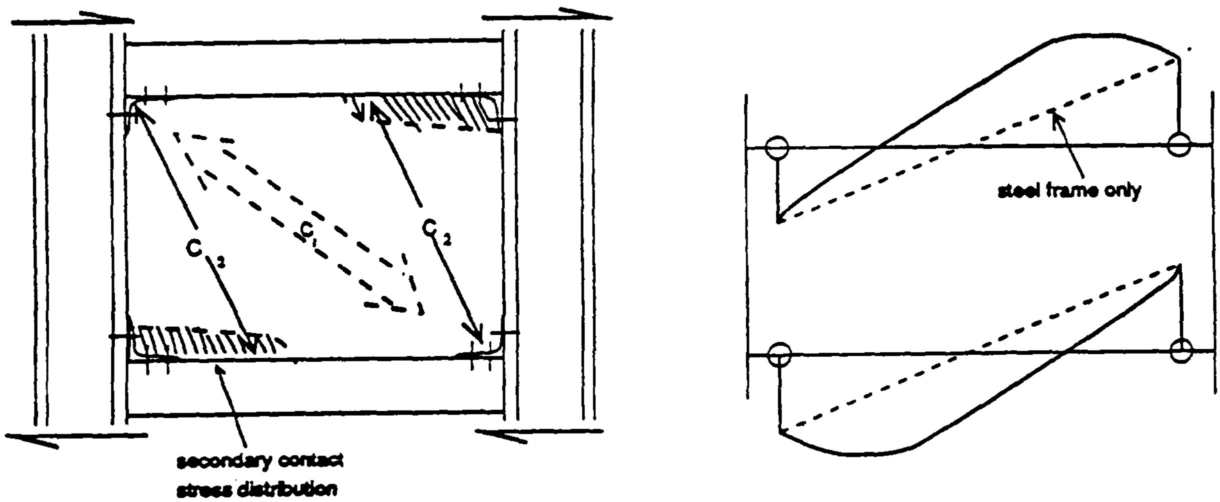
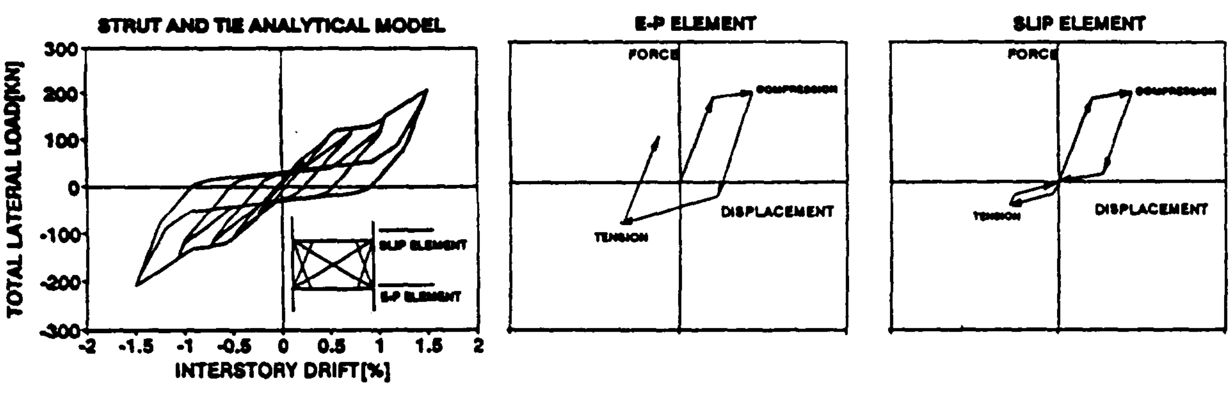


Fig. 6: Formation of Secondary Strut Mechanism

It should be noted that the stresses are tension positive, and the bending moments are plotted on the tension side of the beam. The stresses induced in the mid-span vicinity of the beam are due to the formation of a secondary strut mechanism. The initial primary strut mechanism leads to high stress concentrations at the corners of the infill. Following a few loading cycles, at low drift amplitudes, it is evident that the infill looses its tension strength at the interior of the panel and is less able to sustain the corner-to-corner diagonal strut. Thus, secondary struts form as shown in Fig. 6 which are governed by Coulomb shear friction across the mortar interfaces; the strut capacity being dependent on the sliding friction between the bricks and steel beam.

Computational modeling of the in-plane force deformation behavior of the infills was performed using the non-linear program DRAIN-2DX [2]. Strut forces C_1 and C_2 were modeled using the inelastic-link element option. The semi-rigid top and seat angle steel beam to column connections were modeled using a bilinear beam element, and the end diagonal steel braces were modeled to include bolt slackness. From the results for Specimen 1 presented in Fig. 7, it is evident that this strut and tie approach is very effective in modeling the in-plane hysteretic performance of the infill frame system.



(a) **(b)** **(c)**

Fig. 7: (a) Predicted Force-Displacement Response Using DRAIN-2DX; (b) Beam Element Behavior; (c) Strut/Tie Element for Compression-Tension Behavior of Diagonal Truss Members

Out-of-Plane Experimental Study

Two specimens have been tested in the out-of-plane direction. The specimen configuration was the same as those of the in-plane tests shown in Fig. 1. The first specimen was an undamaged specimen that was shaken on the shaking table with a 15 to 1 Hz sine sweep excitation. A maximum response acceleration of $10g$ was observed, for a constant input acceleration amplitude of $0.3g$ at a response frequency of 5.0 Hz. It was difficult to fail this specimen, but after considerable extra shaking at a constant acceleration amplitude of $0.4g$, the specimen became unstable at the 5 Hz frequency with a maximum response acceleration of $6.5g$. In order to inflict some damage in the panel, the second specimen was tested first of all in-plane under five cycles of quasi-static lateral loading at an interstory drift amplitude of $\pm 1.5\%$. This initial in-plane testing produced diagonal cracking in the panel as well as a loss of bond between the steel framing and the brick infill panel. The specimen was then shaken out-of-plane using several constant amplitude 15 to 1 Hz sine sweep motions. Due to the damage inflicted previously by the in-plane testing, a maximum response acceleration of only $5.0g$ was observed for the $0.3g$ input amplitude. Instability subsequently resulted for a $0.5g$ input acceleration amplitude at a maximum observed acceleration of $6.5g$ was observed at a frequency of 7.9 Hz. Fig. 8 presents the dynamic acceleration response at the center of the infill to a $0.3g$ amplitude input acceleration.

It is evident that some loss of strength results due to damage incurred in the in-plane direction, but the out-of-plane strength of the infill is still very substantial.

Using an approach similar to that developed for the in-plane direction described above, work is progressing applying strut and tie modeling techniques for out-of-plane behavior. Fig. 9 shows a strut and tie idealization for out-of-plane dynamic response. The strength and orientation of the struts are determined from large-displacement compression membrane theory. Static models have been successful in predicting maximum response loads and work is now proceeding to model the experimental dynamic response using the link elements in the DRAIN-2DX computer program [2].

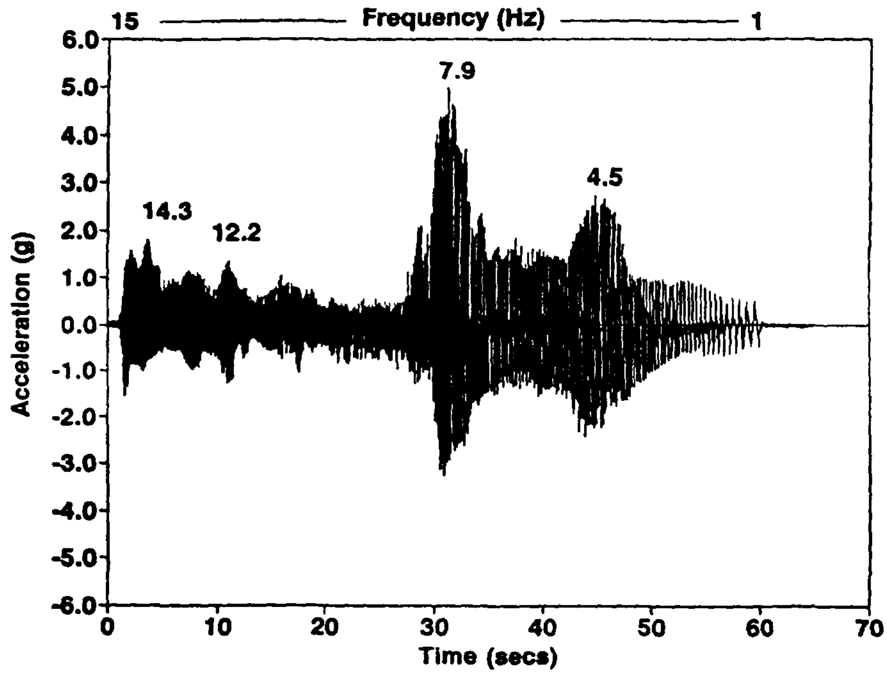


Fig. 8: Acceleration Response at the Center of the Panel to a $0.3g$ Amplitude Input Acceleration

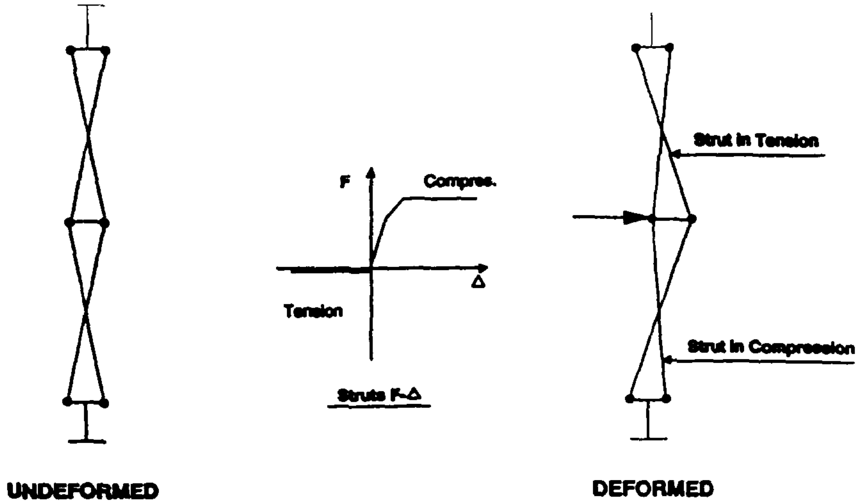


Fig. 9: Strut and Tie Idealization for Out-of-Plane Dynamic Response

CONCLUSIONS

Based on the research conducted to date in this study, the following conclusions have been made:

1. Unreinforced clay brick masonry infills, within steel frames, behave in a moderately ductile fashion under in-plane lateral loads. However, bricks are loosened within the frame during load cycling such that this may leave the infill vulnerable to fall-out from out-of-plane loads. Nevertheless, if fallout of the infill is not a problem, unreinforced clay brick masonry infills can act as ductile lateral load resisting elements in multi-story frames.
2. Although the experiments on ordinary brick infills demonstrated a reasonable ductility capability, by the end of testing the panels were quite loose within their frames.
3. Using an enhanced ferrocement overlay on the infill panel, which also contains diagonal reinforcing bars as reinforcement, provides an improved ductility capacity for the infill panel. An enhanced overlay should improve the general seismic performance of such an infilled wall system. The diagonal reinforcement provides additional energy dissipation capability and adds some strength. Tension cracks are dispersed along each diagonal with this class of ferrocement overlay. The diagonal reinforcing bars also help to prevent out-of-plane buckling of the ferrocement at the center of the panel. Such rehabilitated infills could be used in the lower story of a multi-story frame where plastic hinging would normally be expected to occur in structural wall elements under earthquake loading.
4. Infill shear strength assessments can be made by bounding the initial and final shear capacities for masonry and ferrocement mortar. Respective initial and final masonry (and ferrocement mortar) capacities of $0.167\sqrt{f'_m}$ and $0.05\sqrt{f'_m}$ *(MPa)* may be assumed.
5. Due to the relative crudeness of the above-mentioned strength assessments refined strut and tie modeling techniques can be adapted to better understand the interplay between the primary-secondary strut forces (C_1 and C_2 in Fig. 6) and the resulting distribution of stresses in the beams.
6. Strut and tie modeling using the DRAIN-2DX program is capable of making a good representation of the observed in-plane hysteretic response.
7. For the present infills which had a height to thickness ratio of 18 , failure was difficult to achieve under out-of-plane shaking. Damage incurred by concurrent in-plane displacements reduces the strength somewhat, but the residual out-of-plane capacity is still substantial. Strut and tie modeling, in conjunction with compression membrane theory, is capable of predicting ultimate out-of-plane failure modes. Work is currently in progress to develop inelastic dynamic out-of-plane response analysis techniques.

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PERFORMANCE OF MASONRY-INFILLED R/C FRAMES UNDER IN-PLANE LATERAL LOADS: EXPERIMENTS

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ABSTRACT

Eleven tests were conducted on 1/2-scale, single-story infilled frame specimens to study the influence of the relative strengths of the infill panels and the bounding frames and the frame aspect ratio on the performance of masonry-infilled R/C frames. It was observed that specimens with stronger infills exhibited a higher load resistance and a better energy-dissipation capability. However, their post-peak resistance dropped more rapidly as the displacement increased. In summary, infill panels tend to improve the lateral resistance of R/C frames.

INTRODUCTION

Masonry infills can be frequently found in existing R/C and steel frame structures, in the form of interior or exterior partition walls. The influence of infill panels on structural performance has been controversial, and there are no code provisions or rational guidelines available for the design and safety assessment of such structures. Even though a number of studies (1-3) have been conducted on infilled frames, experimental data and analysis methods which can be used to assess the performance of such structures are still very limited. The main objectives of this study are to assess the performance of existing concrete masonry-infilled R/C frames, to identify critical parameters that may affect the performance of this type of structures, and to develop analysis methods that can be used to assess their performance. This paper summarizes the experimental program and major experimental observations. The finite element analysis method developed in this study is presented in a companion paper (4).

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TEST SPECIMENS

A six-story, three-bay, reinforced concrete moment-resisting frame was selected as a prototype structure. The height/length ratio is about 1/1.5 for each bay. The structure was designed to carry a live load of 50 psf (2.39 kPa). A "weak" frame design and a "strong" frame design are chosen. The design of the weak frame is governed by a lateral wind pressure of 26 psf (1.24 kPa), while that of the strong frame is governed by the equivalent static forces stipulated for Seismic Zone 4 in the 1991 edition of the Uniform Building Code. The test specimens are 1/2-scale models representing the interior bay at the bottom story of the prototype frame, and the design details for a typical weak frame are shown in Fig. 1. The beam-column joints in the strong frame have closely spaced horizontal ties to prohibit shear failure. Both frames were designed in accordance with ACI 318-89 provisions. The weak frame design is also used for other specimens that have a height/length ratio of 1/2 as well as for a two-bay frame. For infill panels, 4x4x8-in. (0.1x0.1x0.2-m) hollow and solid concrete masonry blocks are used in respective specimens. These are considered to be "weak" and "strong" infills, respectively.

TESTING PROCEDURE

As shown in Table 1, a total of eleven tests were conducted at this stage of the project. The test setup is shown in Fig. 2. Two different vertical load distributions were simulated. One had vertical loads applied onto the columns only, and the other had 1/3 of the vertical loads applied on the beam and 2/3 on the columns. The total vertical load was kept to 66 kips (294 kN) in all tests. Two types of in-plane lateral load/displacement histories were selected. One is monotonic and the other is cyclic. As noted in Table 1, some of the tests were conducted on frames that had been tested before. They had been repaired with epoxy injection and retrofitted with new panels. Strain gages and displacement transducers were installed to monitor the strains in the reinforcing bars and the deformations of the specimens. Material tests were conducted on the reinforcing steel, and concrete and masonry samples for each group of frame specimens constructed. These include the modulus of rupture and split-cylinder tests of concrete, the compression tests of concrete cylinders, and the compression tests of masonry units, mortar cylinders and cubes, and masonry prisms. Additionally, direct shear tests were conducted on single mortar joints to obtain their cyclic shear behavior under different compression forces.

TEST RESULTS

Influence of Panel Strength. An infill panel can increase both the lateral stiffness and load resistance of a reinforced concrete frame by a substantial amount as shown in Fig. 3. The stronger the panel is, the larger is the increase. The strength of Specimen 9, which had a strong infill, is about 57% higher than that of Specimen 8, which had a weak infill. However, the drop of post-peak resistance with respect to displacement is more rapid with the strong infill than that with the weak infill. This is more evident under cyclic loads than under monotonic loads as indicated by the load-displacement envelope curves of Specimens 4 and 5 in Fig. 4. This can be partly attributed to the brittle shear failure that was induced in the columns by the strong infill and partly to compression failure of the infill itself. The strength of Specimen 5 is 71% higher than that of Specimen 4. The damage pattern of Specimen 5 is shown in Fig. 5. The behavior of Specimen 4 was dominated by the compression failure of the infill as well as the horizontal sliding of the mortar joints. The latter was not very significant in Specimen 5. Furthermore, from the load-displacement hystereses of Specimens 4 and 5, it can be observed that the strong infill leads to a much better energy dissipation that the weak infill.

Influence of Column Stiffness/Strength. The specimens with the strong frame had a substantially higher load resistance than those with the weak frame. The strength of Specimen 7, which had a strong frame and a strong infill, is 67% higher than that of Specimen 5, which had a weak frame and a strong infill, whereas the strength of Specimen 6, which had a strong frame and a weak infill, is only 29% higher than that of Specimen 4, which was a weak frame-weak infill combination. The theoretical load carrying capacities of the bare weak frame and strong frame designs are 21.4 and 28.5 kips, respectively, i.e., the strong frame has a capacity 33% greater than the weak frame. In the case of a weak infill, where the sliding shear failure in the panel was the dominant mode, the frame and panel actions were more or less independent and their strengths were additive. In the case of a strong infill, the resistance depends on the shear strength of the columns and the diagonal compression mechanism of the infill. The latter depends on the relative stiffnesses of the frame and panel (5). The strong frame had a longer contact length between the frame and the panel, and thereby, a more effective compression mechanism; and it also had a higher shear strength, which prohibited shear failure.

Influence of Aspect Ratio. By comparing the load-displacement envelopes of Specimens 10 and 11 with those of 4 and 5 in Fig. 4, it is interesting to note that the frame aspect ratio has little influence on both the strength and the ductility of a specimen. In the case of strong infills, the frame with the lower aspect ratio appeared to be slightly stronger than the one with the higher aspect ratio. However, it must be noted that the total vertical loads were the same in these tests.

CONCLUSIONS

Results of this study indicate that infill panels can significantly enhance the load resistance capabilities of reinforced concrete frames. They can be potentially used to strengthen existing moment resisting frames. Even though a strong infill could cause brittle shear failure in columns, they provide a better energy-dissipation capability and are more effective in enhancing the load resistance of a frame as a result of the frame-panel interaction.

ACKNOWLEDGMENTS

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Table 1. Tests of Masonry-Infilled R/C Frames

Test No.	Type of Frame	Type of Masonry Units	Panel Aspect Ratio	Lateral Load	Vertical Load Distribution
1	weak	no infill	0.67	monotonic	columns only
2	weak - repaired (1)*	hollow	0.67	monotonic	columns only
3	weak - repaired (2)*	solid	0.67	monotonic	columns only
4	weak	hollow	0.67	cyclic	beam & columns
5	weak	solid	0.67	cyclic	beam & columns
6	strong	hollow	0.67	cyclic	beam & columns
7	strong	solid	0.67	cyclic	beam & columns
8	weak - repaired (4)*	hollow	0.67	monotonic	beam & columns
9	weak - repaired (8)*	solid	0.67	monotonic	beam & columns
10	weak	hollow	0.48	cyclic	beam & columns
11	weak	solid	0.48	cyclic	beam & columns

* Prior test number

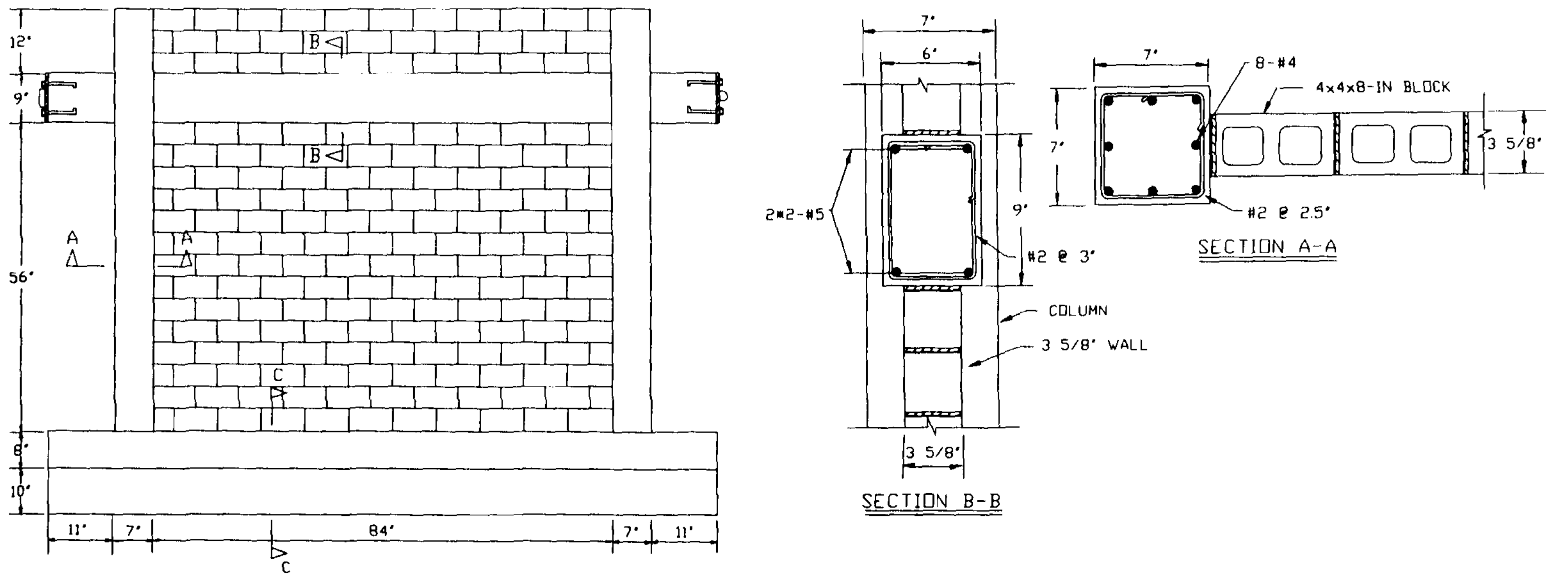

Figure 1. Design of a Typical Infilled Frame Specimen
(Weak Frame; 1 in. = 0.0254 m)

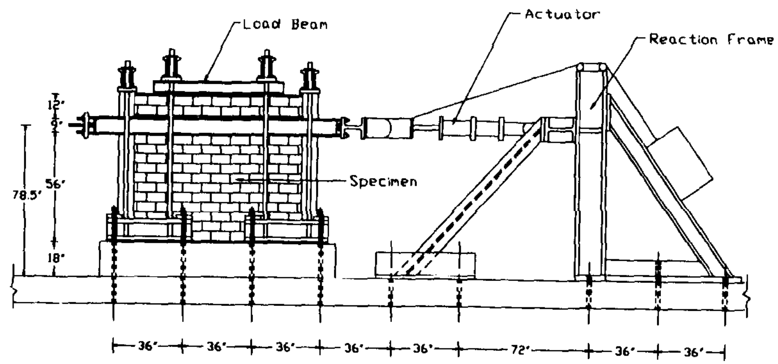

Figure 2. Test Setup and Loading Apparatus (1 in. = 0.0254 m)

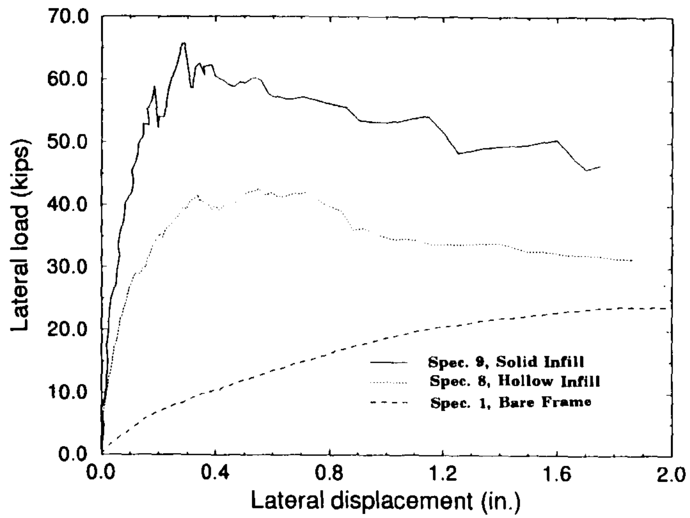

Figure 3. Tests of Weak Frames with Monotonic Loads (1 in. = 0.0254 m; 1 kip = 4450 N)

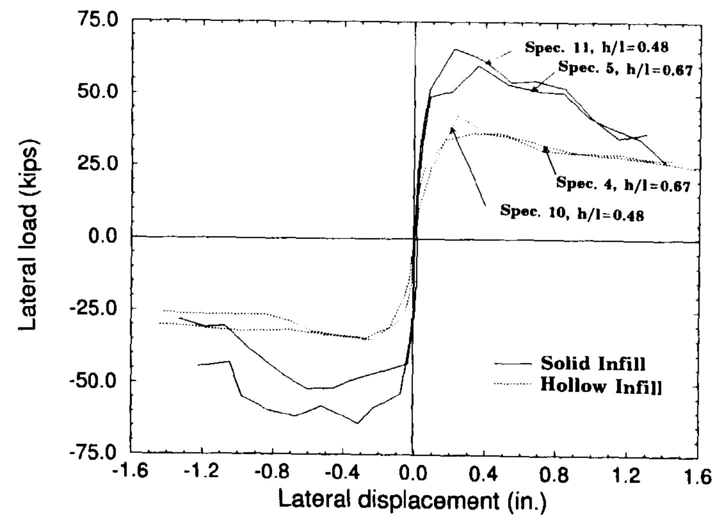

Figure 4. Load-vs.-Displacement Envelopes from Cyclic Tests (Weak Frames; 1 in. = 0.0254 m; 1 kip = 4450 N)

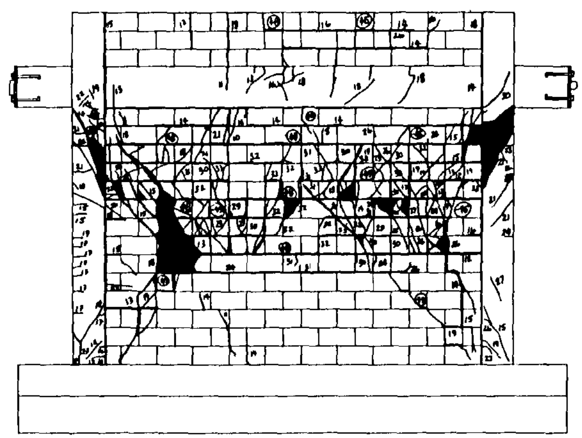

Figure 5. Damage Pattern of Specimen 5 (Weak Frame-Strong Infill)

OUT-OF-PLANE RESPONSE OF UNREINFORCED MASONRY INFILL FRAME PANELS

James A. Hill[1]

ABSTRACT

This paper presents an interim report of the results of an out-of-plane testing program sponsored by the National Science Foundation Grant BCS 9102347. The program to date has consisted of the destructive testing of infill panels in two buildings and included three configurations of unreinforced masonry walls and one hollow clay tile wall partition. In all cases, the loads were applied as point forces distributed by a bearing plate. The loads were generated using hydraulic jacks which were alternately loaded unloaded and reloaded in a manner which allowed the load deflection characteristics to be monitored under successive load cycles. This paper presents a comparison of tested results with data calculated using a linear three hinged arch theory. Subsequent phases will correlate tested data with analytical results obtained with both linear and nonlinear finite element programs.

OUTLINE OF EXPERIMENTAL RESEARCH

The ultimate goal of the test program was to generate full scale test data from actual buildings which could be used to formulate requirements for retrofit standards. To economically achieve these results the test rig was designed to provide basically a point loading distributed over a flat plate to the centroidal area of the infill panel. The rig was designed in a modular manner so that with minimum modification it could be made available for testing in other buildings. The load application rig consisted of two hydraulic rams for the three experiments conducted in the first series of tests and was simplified to a single ram for the tests in the second series of tests. In both cases the load was applied with hydraulic jacks activated by a hand pump with a calibrated pressure transducer. Instrumentation to measure wall movement consisted of a series of string gauges located at appropriate positions on the panel. The first series of tests were conducted at a location within the building which provided a relatively stiff confinement for the infill. The infill in this building was confined within the frame but it have a small gap at the top (approx. 1/16" to 1/8"). The tests conducted on the second series of tests consisted of loading a 22 inch wide 13 inch thick vertical window mullion and a 4 inch thick hollow clay tile partition wall. The window mullion spanned between two reinforced concrete spandrel beams.

INITIAL SERIES OF TESTS

Three panels were identified for testing. The first panel selected for test was a solid rectangular panel. The rectangular shape was selected to minimize the number of assumptions required to

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analytically correlate model performance. The second configuration represented a solid panel with one way arching and the third panel configuration was selected to represent panels with substantial openings. First tests were performed on a rectangular solid 13" unreinforced clay brick infill panel with a floor-to-floor height of 10'-6" and an overall length of 20 ft . The loading was applied in a monotonic manner recording deflections at each load. The wall was unloaded and allowed to return to a zero load state after completion of each load cycle. The process was continued with each new load cycle taken to a higher value. Loading was increased up to a load of 9,000 lbs. (an equivalent lateral load of 0.38g) at which point the tests were stopped because the test rig was felt to be at maximum safe capacity. The rig was subsequently strengthened and the test procedure repeated with the maximum load of approximately 22,000 lbs.. Again the rig was showing evidence of lateral buckling and no further loads could be safely applied.

During these tests there was no evidence of wall failure. A plot of the load deflection history for a representative location presented in Fig. 1 and compared with estimated deflections obtained using the three hinged arch model. While data was obtained using the actual line loads for easier interpretation the results are presented here in terms of equivalent lateral load. Similar results are presented in Figs. 2 and 3 for a vertical one way span condition and a horizontal one way span configuration respectively. In both of these cases testing was continued until the wall was failed. During the high load cycles of the vertical test panel strong arching action was observed. The arching caused such a large floor to floor deflection that the 2"x4" struts containing the instrumentation gauges to fall from their wedged positions. At the time of substantial wall movement load cracking noises were noted coming from the interior of the wall. While some cracking was observed along the header courses the cracking noise was later attributed to the shearing of the headers across the wythes.

TESTS CONDUCTED AT SECOND BUILDING

Prior to conducting the out-of-plane tests, both flat jack and shear tests were conducted on the unreinforced masonry wall panels at representative locations in the building. While the push tests were not used directly, they did establish a quality measure for the wall panels that indicated above average quality. Results of a representative flat jack test demonstrating the nonlinear nature of the modulus is presented in Fig. 4. Following the material tests a vertical mullion was selected for the first out-of-plane test. The selected element spanned 85" vertically between reinforced concrete spandrel beams, was 21.25" wide and 13" thick. The wall was cyclically loaded with a single jack attached to the center of the vertical span. The loading was continues until substantial wall failure occurred. The cracking of the header courses was also noted during the later phases of these tests. Results of this test along with the predicted response is presented in Fig. 5.

A hollow-clay-tile partition wall was also selected for testing. The wall measured 9'-11" long spanned 12'-8" from floor to floor and was bounded on one edge with a column. The wall itself was a 4" thick unit with plaster on each side. Both measured and predicted response is presented in Fig. 6. During the application of the load a classic yield line failure pattern for a plate in bending was observed. Final failure for this test was from punching shear, and not from bending. The wall could have perhaps demonstrated a larger capacity if a uniform load was applied. Since material tests were not conducted for this wall, correlation places a great deal of dependence upon the assumed parameters.

CONCLUSIONS

All wall panels exhibited substantial resistance to out-of-plane loading and evidence of arching action was noted during the test procedures, particularly during the third panel test in building one. Here, large floor-to-floor displacements (approx. 1/4") were generated during period of high out-of-plane loading. The degree of floor deflection generated by the arching was quite remarkable considering the size of the spandrels and the general stiffness of the slab assembly.

In all cases, the three hinged arch theory provided reasonable predictions of the wall response in the linear deformation range. Out-of-plane capacity was substantial in the linear range. The onset of nonlinear behavior appeared to be preceded by shear failures of the header bricks within, resulting in a debonding of the wythes rather than a crushing failure in the outer wythe.

Additional studies are planned with more sophisticated predictive algorithms.

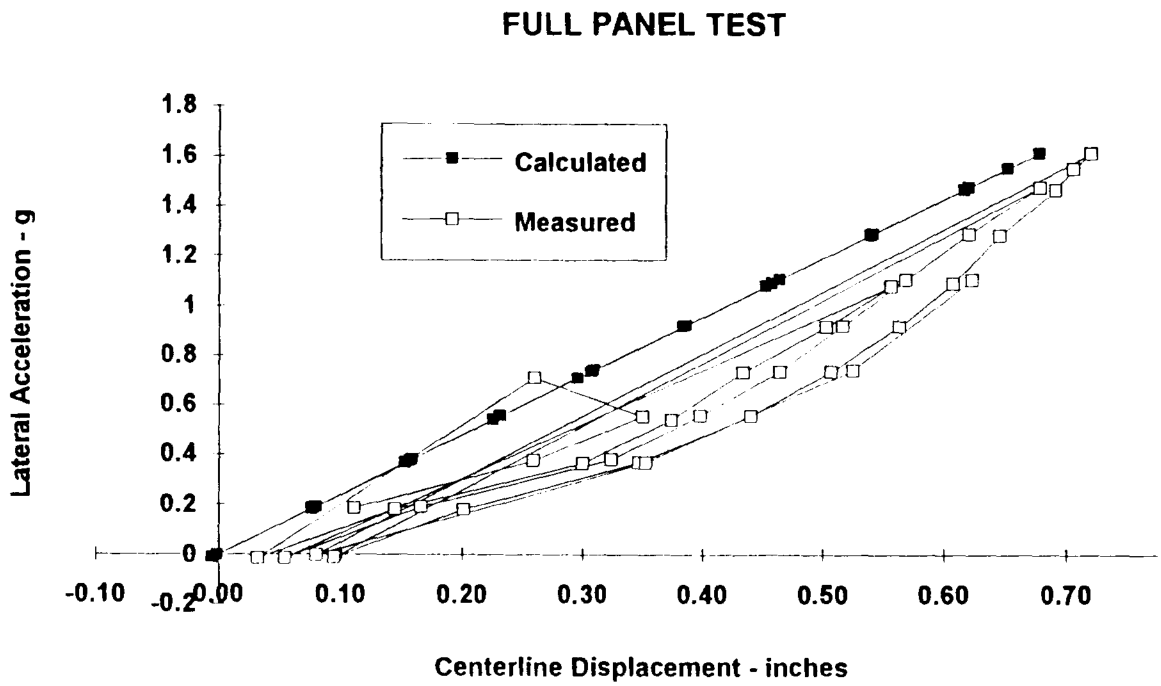

Figure 1 Full Panel Test Bldg. 1 Elastic Response

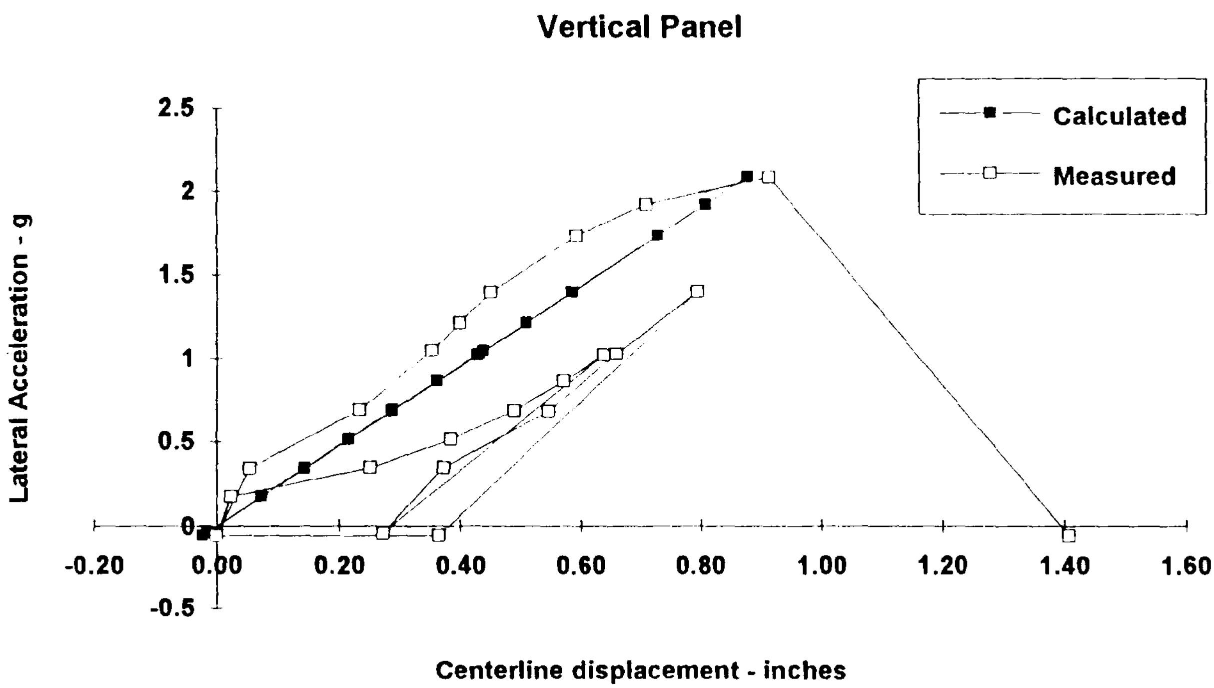

Figure 2 One Way Vertical Arching Bldg. 1

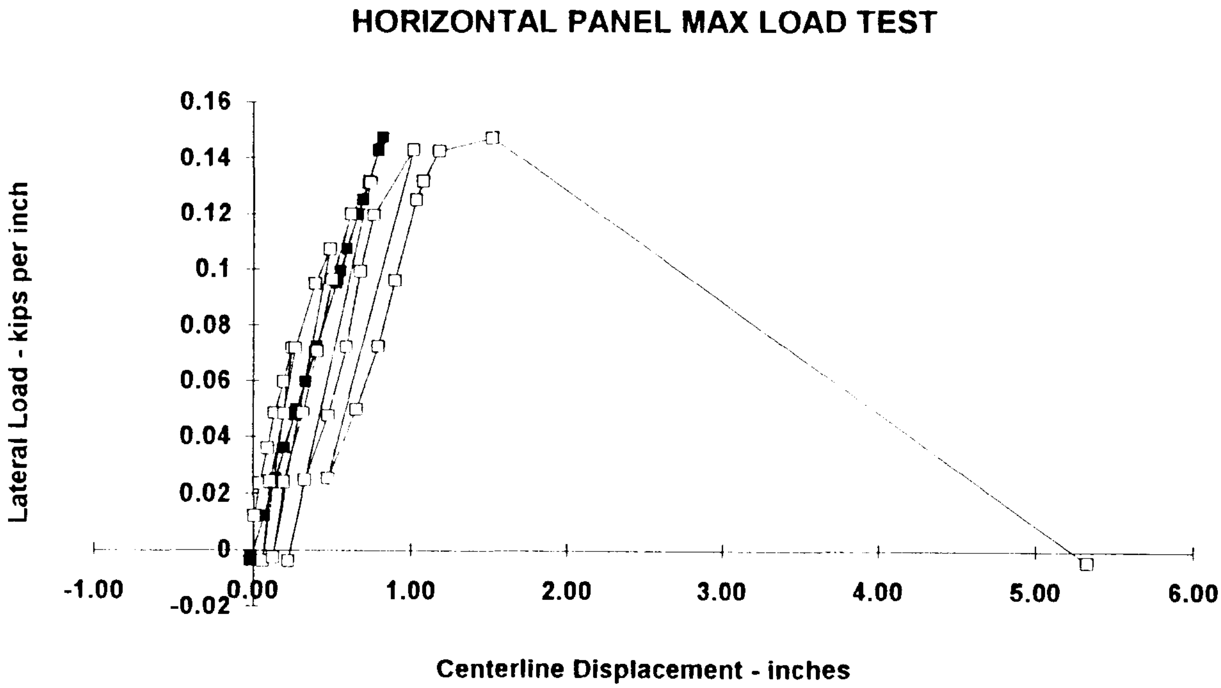

Figure 3 One Way Arching Horizontal Span Bldg. 1

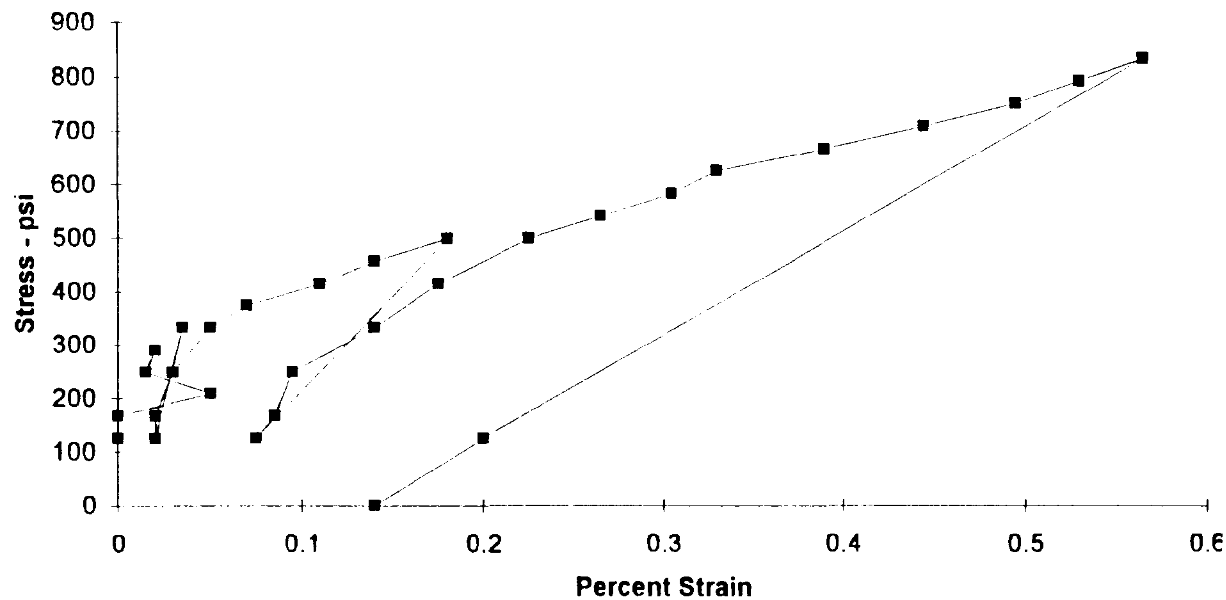

Figure 4 Typical Flat Jack Test Results for Bldg. 2

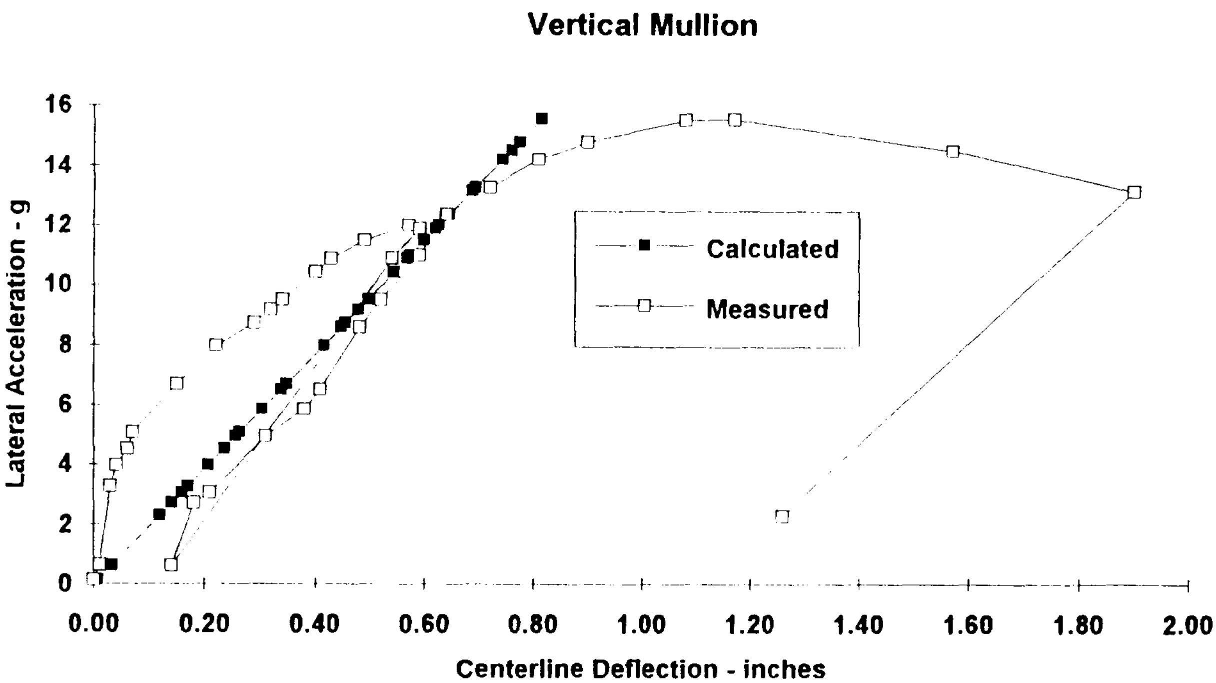

Figure 5 Response of Vertical Mullion to out-of-plane Loads

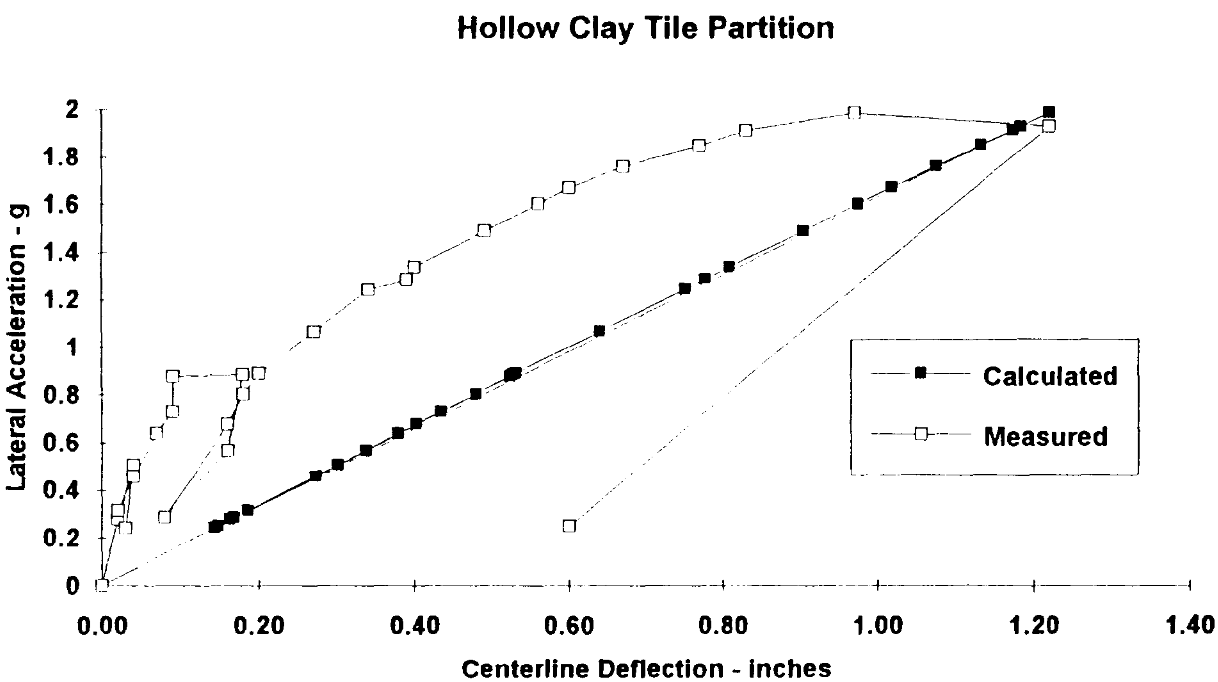

Figure 6 Lateral Response of Hollow Clay Tile Partition

THE INFLUENCE OF MODELING ASSUMPTIONS ON THE PREDICTED BEHAVIOR OF UNREINFORCED MASONRY INFILL STRUCTURES

Nabih Youssef*

ABSTRACT

The results of an analytical study are presented showing how modeling assumptions influence the predicted behavior of unreinforced masonry (URM) infill panels. A masonry panel was analyzed using a nonlinear finite element program (FEM/I). Various assumptions were made in the modeling of the panel, i.e. boundary condition, frame-masonry interface, etc., and the effect of each assumption was investigated. The results of this study suggest that the nonlinear finite element analysis of URM panels is not significantly affected by most of the modeling assumptions investigated.

INTRODUCTION

The structural behavior of unreinforced masonry used as infill has a significant influence on the overall seismic behavior of infilled frame structures. The evaluation of the seismic capacity of existing structures, and of the adequacy of retrofitting measures must take into account the strength and stiffness contribution of the infill.

Methods which consider the infill masonry as an equivalent strut had been discussed during the 1970's by several researchers (3). However, recent advances in the development of nonlinear finite element software has resulted in thc resurgence of the equivalent strut analogy. Using nonlinear finite element analysis, the infill is modeled to determine its nonlinear force-deformation relationship (2). The masonry infill is then modeled using an equivalent diagonal strut to brace the structural frame in an elastic model of the building. The stiffness properties of the equivalent strut are determined based on the force-deformation relationship of the infill at a target displacement.

OUTLINE OF PARAMETRIC STUDIES

The object of this study was to investigate the influence of modeling parameters on the force-deformation relationship of masonry infills analyzed using nonlinear finite element analysis methods.

Proto-Type Model

A typical infill panel was selected from architectural drawings. The panel and 1/2 of the adjacent panel, on either side, was modeled in the analysis (Fig. 1). The wall is made up of two wythes of

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masonry and one wythe of terra cota. The confining frame consists of steel beams and columns encased in concrete. Frame member sizes were obtained from structural drawings.

A proto-type model of this panel was developed (Fig. 2). Modeling assumptions that were considered in the proto-type include:

• Frame Members - The steel frame is concrete encased. The steel and concrete material properties were assumed to be inelastic. Beams were modeled by flange and web elements. Flange elements were 3 inches wide and its thickness was computed by equating moments of inertia. The thickness of the web element was set equal to that of the beam section. The column was modeled as an element of constant thickness, due to its weak axis orientation. The thickness of this element was computed by equating moments of inertia. The concrete encasement was modeled by concrete elements that overlaid the steel frame. The thickness of these elements was computed by equating areas.

• Beam-Column Connections - The confining steel frame was constructed using semi-rigid beam-column connections (wind clips). These connections were modeled as hinges.

• Frame-Infill Interface - Gap elements were used to model the frame-infill interface. These elements are strong in compression and weak in tension.

• Vertical Restraint Condition - Nodes at the top of the panel were vertically restrained.

Permutations of this model were made by individually altering each of the modeling assumptions. The new assumptions are simplifications to the model, these include:

• Frame Members - All frame members were modeled as elements of constant thickness, computed by equating moments of inertia.

• Beam-Column Connections - Connections were assumed to be rigid.

• Frame-Infill Interface - The frame-infill interface were not explicitly modeled.

• Vertical Restraint Condition - Nodes at the top of the panel were not vertically restrained.

Analytical Procedure

The models were analyzed using the nonlinear finite element program FEM/I (1). In the analysis the frame/panel was pinned at the bottom and incremental displacements applied at the top. Models of the frame without masonry infill were also developed and analyzed. From the analyses force-deformation relationships were obtained. To obtain the effect of the infill alone, the influence of the frame was subtracted from the force-deformation response of the frame/panel model (Fig. 3). The force-deformation characteristics of each of the permutated models were compared to that of the proto-type.

Results

It was found from the analysis that the modeling of the frame members did not significantly influence the analytical behavior of the infill panel (Fig. 4), provided that the moments of inertia of the members are accounted for. However, in cases where the panel is tall and slender or where the column is oriented in its strong axis, the effects of frame member modeling may be greater.

The effect of the beam-column connections is shown in Figure 5. As can be seen, modeling the connection as hinged or fixed does not significantly affect the force-deformation characteristics of the infill.

Modeling of the frame-infill interface does not have a significant effect on the force-deformation response of the panel (Fig. 6). The figure shows that the strength of the panel is increased when the frame-infill interface is modeled. This increase in strength, approximately 10%, is within the computational uncertainty of the analysis.

As expected the vertical restraint condition at the top of the panel has a significant influence on the behavior of the infill (Fig. 7). The 'true' restraint condition of the panel is neither fixed or free, therefore this condition cannot be adequately modeled using FEM/I. A multi-story model may approximate this condition at the lower and intermediate levels.

To investigate the appropriate constraint condition further a 3-story frame-wall model was developed (Fig. 8). This model incorporated all earlier finding of this study, i.e. connections were assumed rigid, frame-infill interfaces were not modeled, etc.. This model was excited by prescribed displacements applied at the top level and a uniform load distribution applied to the lower levels. The force-deformation relationship of the bottom floor was obtained by considering the base shear and the displacement at the top of the level. This relationship was compared to that of a single story panel modeled with and without vertical restraints (Fig. 9). As can be seen from the figure the single story model with no vertical restraint does not adequately predict the behavior of the infill panel. The vertically restrained single story model adequately approximates the response of the infill for deformations less than 0.35 inches.

A permutation of the proto-type which incorporated all results of this study was developed and analyzed. Figure 10 shows that the simplified model adequately approximates the force-deformation behavior of the proto-type.

CONCLUSIONS

A proto-type model of a typical masonry infill panel was developed and analyzed using the nonlinear finite element method. This model accounted for semi-rigid beam-column connections, frame-infill interfaces, variation of frame member thickness and the vertical restraint of the panel. Permutations of this model were developed and analyzed to investigate the influence of modeling assumptions on the predicted behavior of masonry infill panels.

The results of this study indicate that the beam-column connections and frame-infill interfaces have no significant impact on the analysis. Frame members can be modeled as elements of constant thickness, provided that the moments of inertia of the members are accounted for. However, for tall and slender panels the modeling of frame members may influence its force-deformation relationship. Restraining the vertical displacement of a panel approximates the behavior of an infill panel that is overburdened. A model incorporating these simplifications was shown to adequately approximate the force-deformation behavior of the proto-type.

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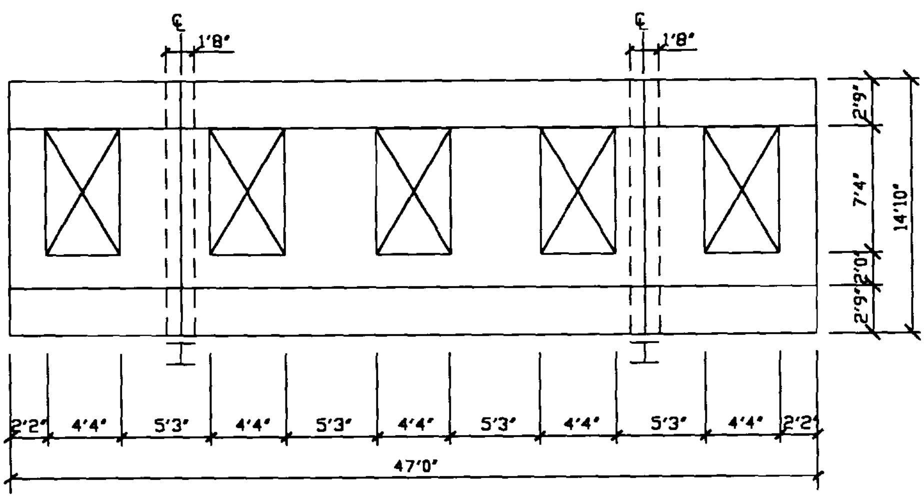

Figure 1. Typical Infill Panel.

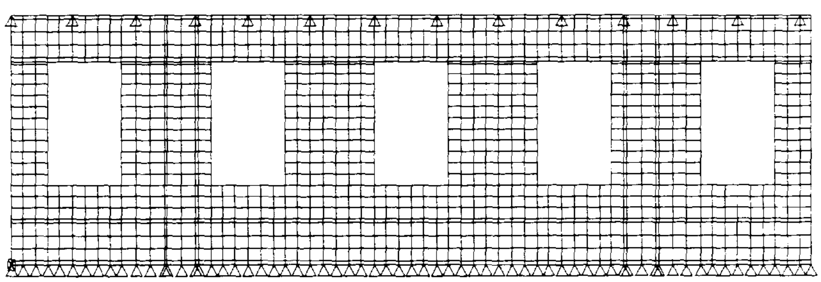

Figure 2. Proto-type Model.

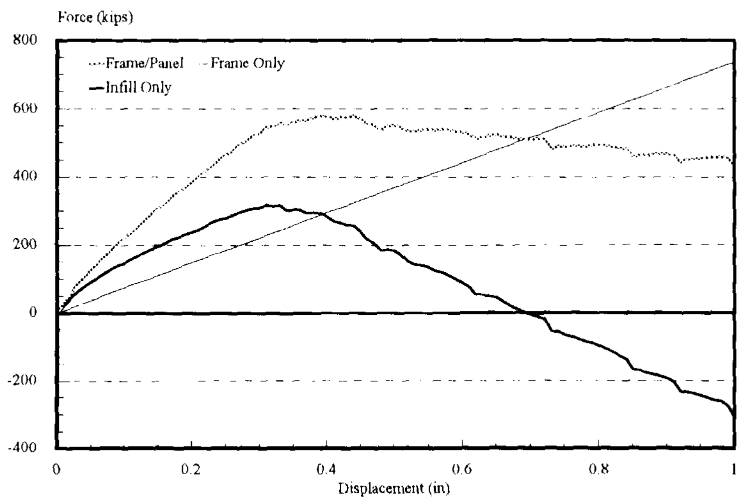

Figure 3. Force-Deformation Plot of Frame/Panel, Frame Only and Infill Only.

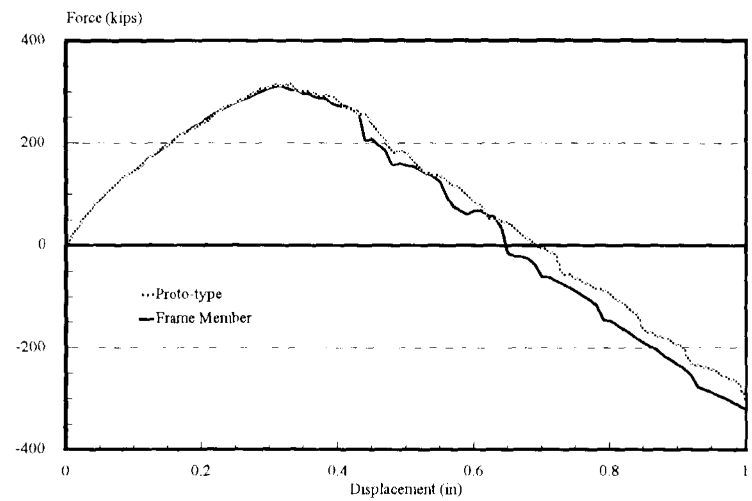

Figure 4. Force-Deformation plot of Proto-type and Constant Thickness Frame Element Model .

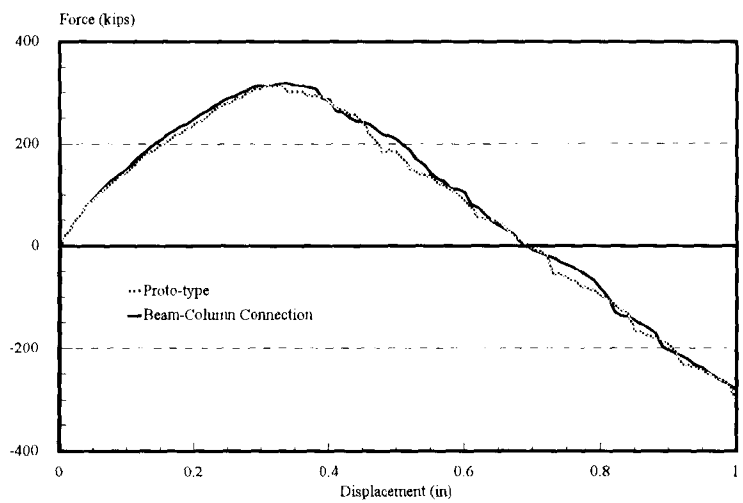

Figure 5. Force-Deformation plot of Proto-type and Rigid Connection Model.

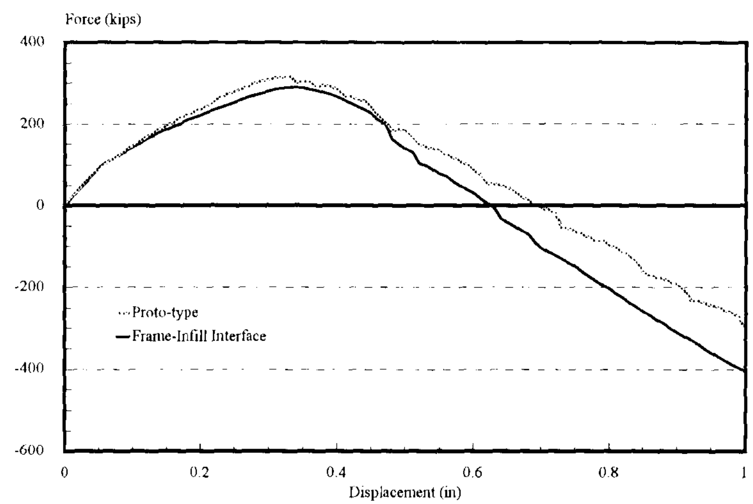

Figure 6. Force-Deformation plot of Proto-type and No Gap Element Model.

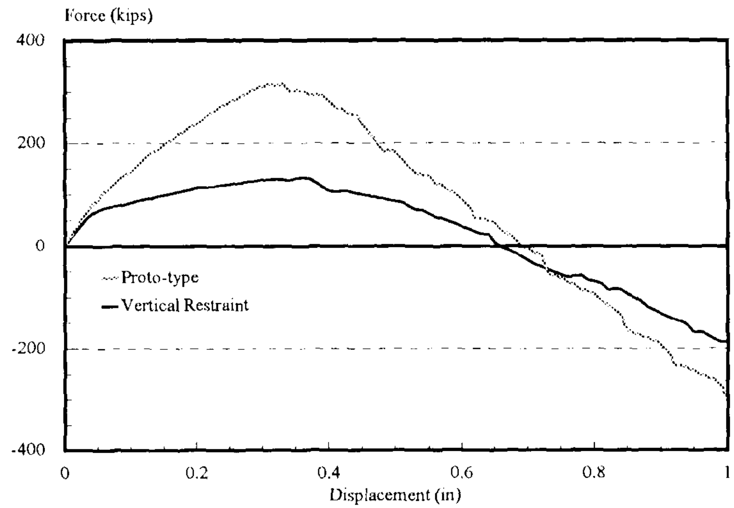

Figure 7. Force-Deformation plot of Proto-type and Vertically Unrestrained Model.

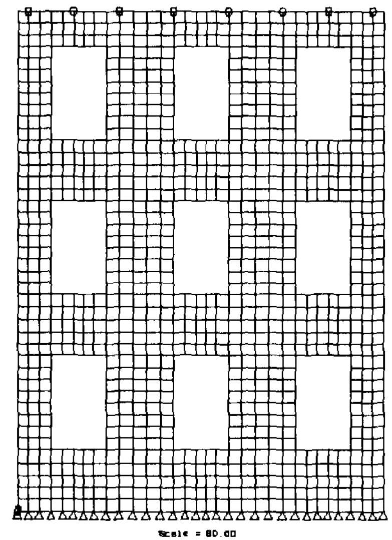

Figure 8. 3-Story Model.

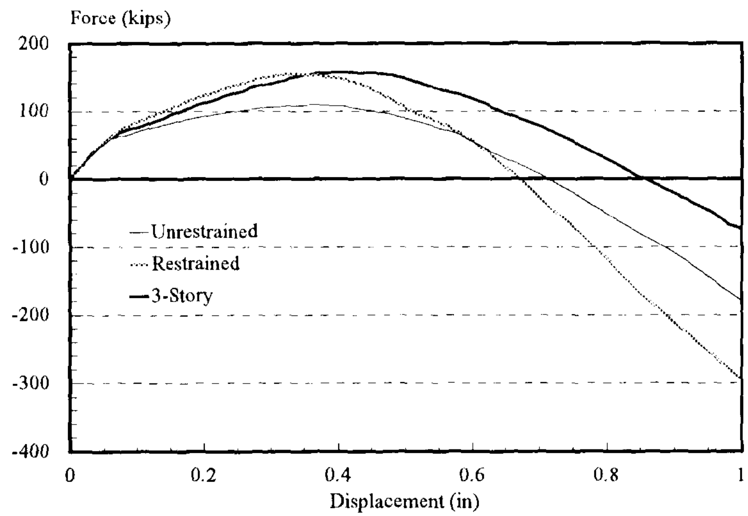

Figure 9. Force-Deformation plot of Vertically Restrained and Unrestrained Single Story Model and 3-Story Model.

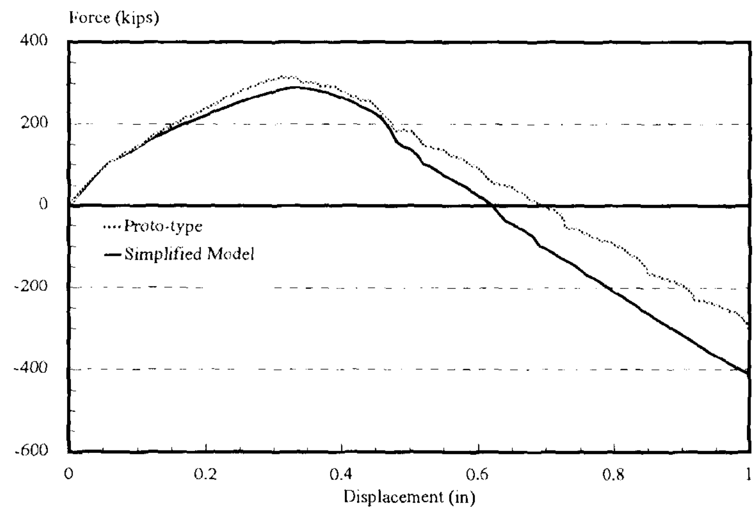

Figure 10. Force-Deformation plot of Proto-type and Simplified Model.

PERFORMANCE OF MASONRY-INFILLED R/C FRAMES UNDER IN-PLANE LATERAL LOADS: ANALYTICAL MODELING

A.B. Mehrabi[1] and P.B. Shing[2]

ABSTRACT

Finite element models considering the fracture behavior of R/C frames, masonry units, mortar joints, and frame-panel interface have been developed to study the lateral load resistance of masonry-infilled R/C frames. The models have been validated with the results of tests conducted on 1/2-scale, single-story, R/C frames infilled with concrete masonry units. An excellent correlation has been obtained between the experimental and numerical results. This analysis method can be used to evaluate the influence of different design parameters on the performance of infilled frames.

INTRODUCTION

Masonry infills can be frequently found in existing R/C and steel frame structures, in the form of interior or exterior partition walls. The load resistance mechanisms of an infilled frame is quite complex, involving the interaction of the frame and the infill. Depending on the relative strength and stiffness of the frame and infill, a number of failure mechanisms are possible, some of which are summarized in Fig. 1. It is, therefore, difficult to have a simple analytical model that can account for all these mechanisms and, thereby, be used to assess the resistance of these structures. In this paper, a finite element analysis method that can be used to evaluate the performance of these structures is presented. In this method, the fracture behavior of concrete frames and masonry units is modeled in a smeared fashion, while the tensile and shear behavior of mortar joints and frame-panel interface is modeled with interface elements. The numerical results are compared with exper-

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imental data obtained from 1/2-scale frame specimens that were infilled with hollow and solid concrete masonry units (3). Some of the comparisons are presented here.

FINITE ELEMENT MODELING

Two types of elements are used to model the fracture behavior of concrete and masonry members in an infilled frame. The tension and compression behavior of the concrete frame and masonry units are essentially modeled with smeared crack elements. The fracture of the mortar joints, the separation of the frame-panel interface, and the shear cracks in the concrete columns are modeled with interface elements.

For the smeared crack element, an elastic-plastic plane-stress model based on the von Mises yield criterion and associated flow rule, combined with a Rankine-type tension cutoff, is used. The von Mises criterion is used to simulate the compressive fracture of masonry. Tensile cracks occur when the tension-cutoff surface is reached. This transforms the material behavior from elastic-plastic to nonlinear orthotropic with the axes of orthotropy parallel and perpendicular to the crack. A rotating crack formulation is adopted in the following analyses. Appropriate post-peak softening rules are incorporated for tension and compression. The details of the model can be found in Ref. 2. Reinforcing steel is considered as an elastic-hardening plastic material. It is represented by a smeared overlay and discrete bar elements, with the strain compatibility between the steel and concrete assumed.

For the interface element, a plasticity-based constitutive model (1) is adopted. In this model, the relative normal and tangential displacements between the two contact surfaces of an interface are decomposed into an elastic component and a plastic component. The failure surface of the interface model is represented by a hyperbolic curve expressed in the following form.

$$F(\underline{\sigma}, \underline{q}) = \tau^2 - \mu^2(\sigma - s)^2 - 2r(\sigma - s) = 0 \tag{1}$$

in which $\underline{\sigma} = \{\sigma \quad \tau\}^T$, where σ and τ are the normal and shearing stresses at the interface, and $\underline{q} = \{\mu \quad s \quad r\}^T$, where μ is the slope of the asymptotes of the yield surface, s is the tensile resistance of the interface, and $-r$ is the radius of curvature at the vertex of the hyperbola. The evolution of the internal variables $\underline{q}$ is governed by a set of work softening rules to model the development of mode-I, mode-II, or mixed-mode fracture. Shear dilatancy is also account for by a non-associated flow rule. The above constitutive model is implemented in an isoparametric interface element.

NUMERICAL RESULTS

The finite element models are validated with experimental results reported in a companion paper (3). Specimens 8 and 9 of the aforementioned experimental study are considered. They were subjected to monotonic in-plane lateral loads. While the R/C frames of Specimens 8 and 9 had an identical design, the former had a hollow concrete masonry infill and the latter had a solid concrete masonry infill. The finite element mesh and numerical results are shown in Fig. 2. The frame-panel interface and mortar joints are modeled with interface elements, which have been calibrated with results of direct shear tests conducted on similar materials. The concrete frames and masonry units are modeled with smeared crack elements, with the compressive strengths obtained from corresponding material tests. However, because of the lack of experimental data, the material parameters governing the tension softening of concrete and masonry have to be determined and fine tuned based on the global response of the specimens. As shown in Fig. 2(a), interface elements are also used to capture diagonal shear cracks in the columns in a discrete fashion. This is needed in order to overcome the shortcomings of smeared crack elements (2). The shear reinforcement in the frames is modeled by a smeared overlay, while the flexural reinforcement is modeled by discrete bar elements. Nine-node quadrilateral elements are used for the concrete frames and 4-node quadrilaterals are used for the infills.

The cracking and failure patterns obtained for Specimen 9, which had a strong panel, are shown in Figs. 2(a) through (c). It can be observed that the failure mode involves the horizontal sliding of the masonry units, corner crushing, and the shear failure of the windward column. In the case of Specimen 8, which had a weak panel, corner crushing is more significant, while no shear failure occurs in the columns. These are very similar to the failure modes observed in the tests. The numerical load-displacement curves are compared to experimental results in Fig. 2(d). It can be observed that the finite element analyses tend to overpredict the actual strengths of the specimens by a small amount.

CONCLUSIONS

The finite element models presented here have shown an excellent correlation with experimental results. They capture both the failure mechanisms and load resistance of infilled frames. Such models can be used to evaluate the influence of different design parameters on the performance of infilled frames, as well as to evaluate the load resistance capabilities of existing infilled structures.

ACKNOWLEDGEMENTS

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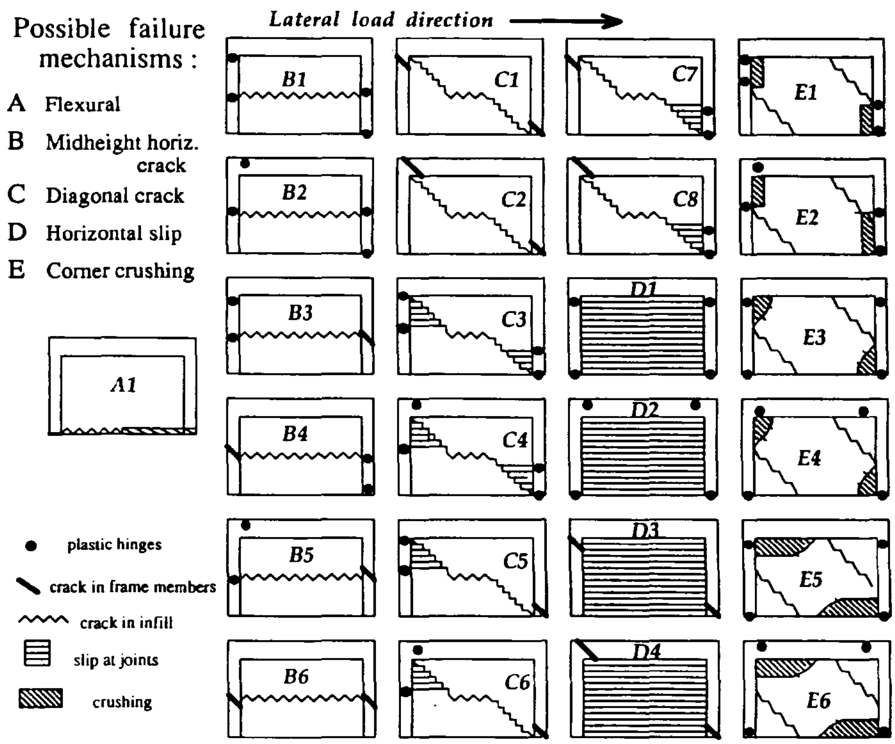
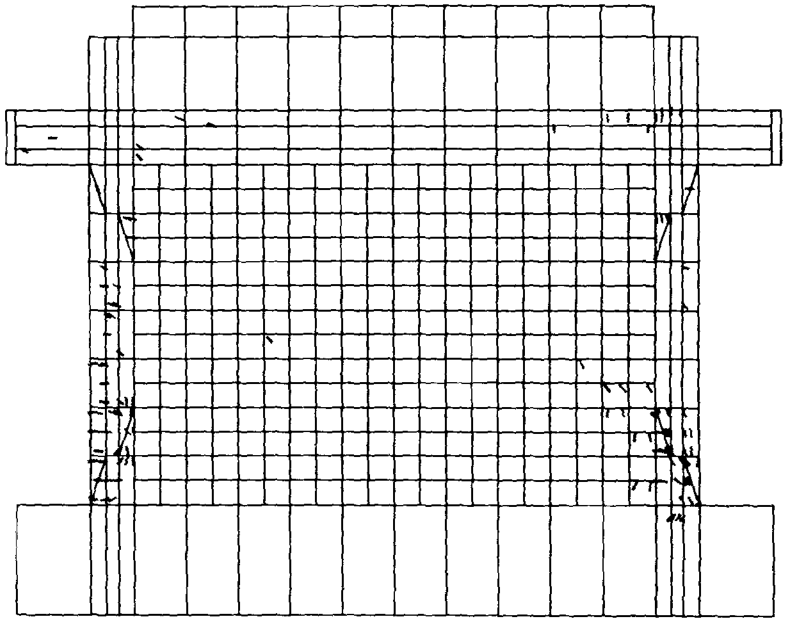
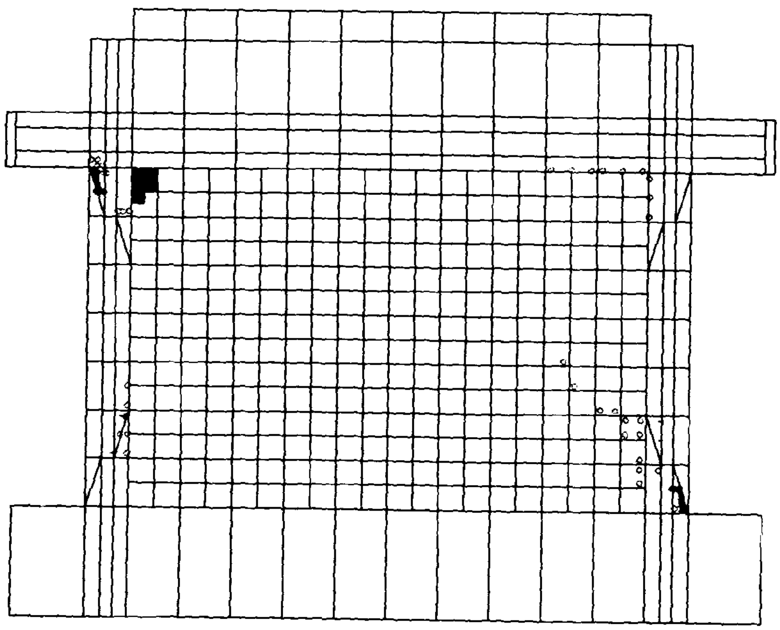


Figure 1. Failure Mechanisms

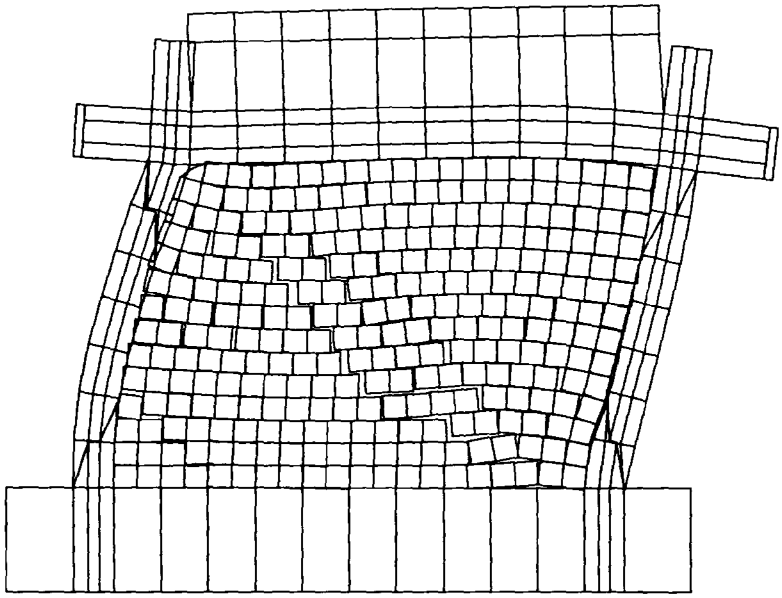


(a) Crack Pattern (Specimen 9)

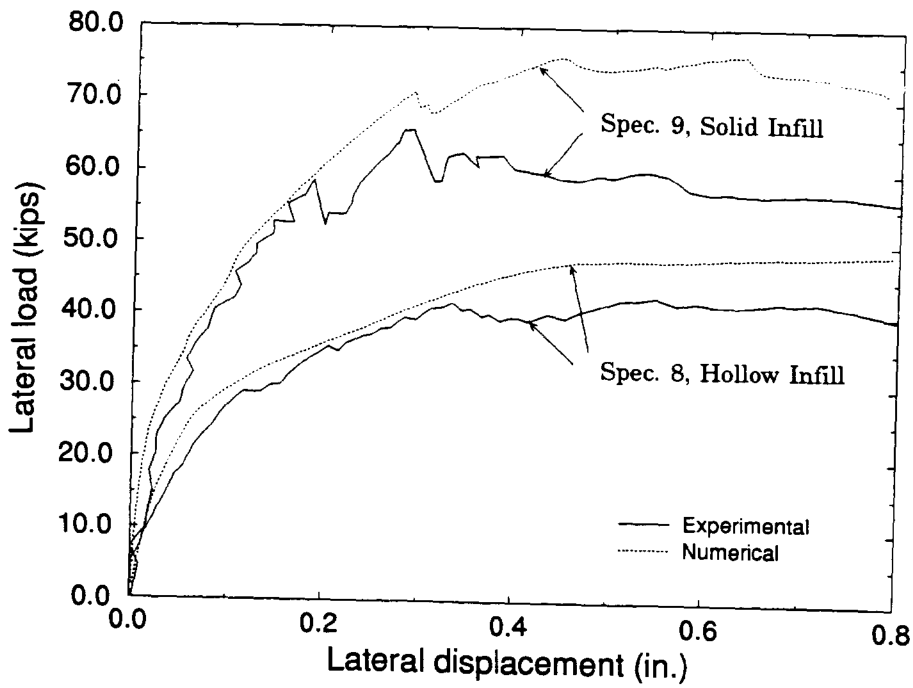


(b) Compression Failure (Specimen 9)

Figure 2. Finite Element Analyses (1 in. = 0.0254 m; 1 kip = 4450 N)



(c) Deformed Mesh (Specimen 9)



(d) Lateral Load vs. Lateral Displacement

Figure 2. Finite Element Analyses (1 in. = 0.0254 m; 1 kip = 4450 N)

EVALUATION AND MODELLING OF INFILLED FRAMES

Peter Gergely[1], Richard N. White[2] and Khalid M. Mosalam[3]

ABSTRACT

Several reduced-scale models of single story, one and two bay Semi-Rigidly Connected Steel frames infilled with unreinforced masonry walls have been tested under lateral cyclic loading. The observed modes of failure include corner crushing and progressive cracking of the infill walls, depending primarily on the relative strengths of the concrete blocks and the mortar joints. These modes were accompanied by sliding and gap formation between the frame members and the infill walls. The information gathered during these experiments provides a data-base for developing and evaluating analytical models for the seismic performance of infilled gravity load designed frames.

INTRODUCTION

A coordinated experimental and analytical research program has been under way for several years with the sponsorship of NCEER. The main participating institutions are Cornell University and SUNY/Buffalo. The primary goal of this research program is to develop analytical models for the estimation of the seismic response of infilled frames. The frames are either Semi-Rigidly Connected Steel (SRCS) or Lightly Reinforced Concrete (LRC). Both types are designed for gravity loading only or for small wind forces. These frames are typical in many buildings throughout the world. In such buildings, infill walls are made of UnReinforced Masonry (URM) concrete blocks or clay bricks.

The present research program considers two types of analytical models: a relatively simple multi-strut model and a more complex finite element procedure. The accompanying experimental program using reduced-scale models serves the purpose of validation and calibration of these analytical models.

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[3]Graduate Research Assistant, Cornell University.

ANALYTICAL MODELS

The Multi–Strut Model

Part of the experimental program concentrated on verifying a previously developed multi–strut model known as the compression–only six struts model (1). In this relatively simple model, three parallel struts (one diagonal strut and two off–diagonal struts) act simultaneously in compression only. In this load transfer mechanism, the diagonal strut may lose much of its load carrying capacity if the masonry crushes at the corners. The two off–diagonal struts then pick up the force and can directly transfer forces to the frame members away from the corners. The main advantage of this model is its capability to model post–crushing behavior and the formation of plastic hinges in the frame members.

Unfortunately, the multi–strut model has several drawbacks. It is difficult to model the force transfer and slip along the frame/wall interfaces. The force transfer is complicated by the presence of door or window openings and it is not yet clear whether the multi–strut model can be modified for these cases.

The Finite Element Model

To faithfully model the interface conditions between the wall and the bounding frame (sliding and gaps) and the vast variety of possibile geometrical configurations of wall panels with openings (sizes and locations), it is mandatory to employ a method where the wall is treated as a continuum. The Finite Element (FE) method is an appropriate candidate for this. The adopted FE system (DIANA[1]) can model detailed material properties such as cracking and plastification. Also, important geometrical discontinuities such as interface conditions (initial gaps, contact, and sliding) can be treated.

The FE model was used to analyze three frames: two sixth–scale models of LRC three story single bay frames with and without URM infill walls which were tested at Drexel University (2) and a fourth–scale model of SRCS single story two bay frame with URM infill walls which was tested at Cornell University. In this model, bilinear relationships are assumed for the normal and tangential forces and displacements at the the interface. All material parameters were identical to those determined from separate quality control experimental programs conducted at Drexel and Cornell Universities. More details of the FE model of these frames can be found in (4) and (5).

The results of preliminary analyses of both Drexel LRC and Cornell SRCS model frames are shown in Figs. 1 and 2, respectively. The FE model predictions are quite good and indicate the stiffening effect of the infill. For Drexel model frames, the analysis shows that the infilled frame failed at a lower story drift than the bare one. For Cornell model frame, the analysis was based on uniform initial gap between the

[1]DIsplacement ANAlyser (DIANA) is the finite element code being developed at TNO Building and Construction Research in cooperation with other institutes and universities in the Netherlands.

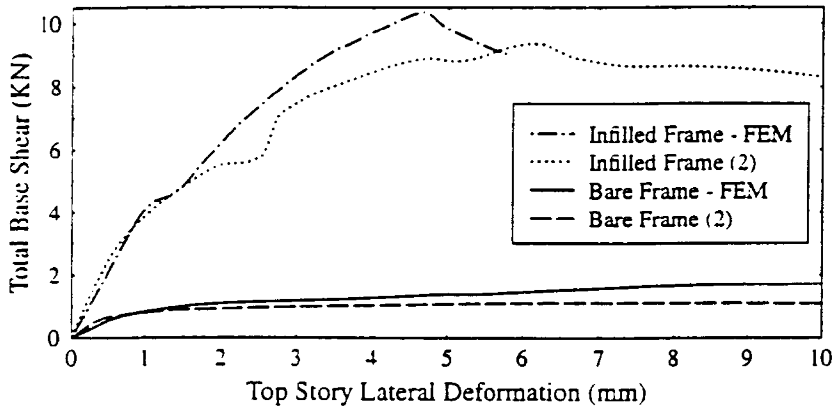

Fig. 1: Response of Drexel LRC bare and infilled frames.

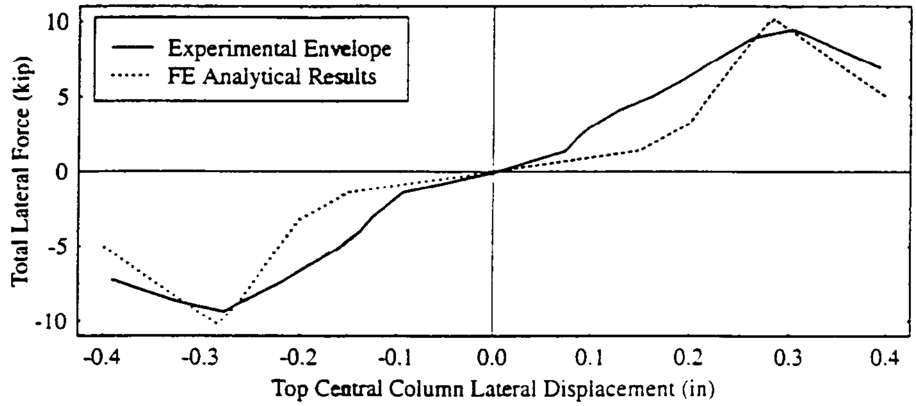

Fig. 2: Response of Cornell SRCS infilled frame.

frame members and the walls. From the experimental results, it is obvious that this assumption overestimates the initial values of these gaps.

More refinements are being pursued at Cornell University for the FE model. These refinements will utilize the experimental measurements to estimate the parameters of the material models. Also, adaptive FE mesh adjustment based on minimizing the discretization error is another refinement to be considered in the FE model.

EXPERIMENTS

A series of reduced-scale experiments have been planned to study the interaction of infills and steel or concrete frames designed only for gravity loads and to provide data for the confirmation and calibration of the analytical models. Three $\frac{1}{4}$-scale single-story frames have been tested to date, one single-bay and two two-bay SRCS frames. Reversed cyclic loads with increasing amplitudes were applied with several cycles (normally three) at each load level. Displacement measurements were relative along the assumed struts and along the interface in the normal and tangential directions, and absolute for the columns and several points in the walls to obtain the displacement field

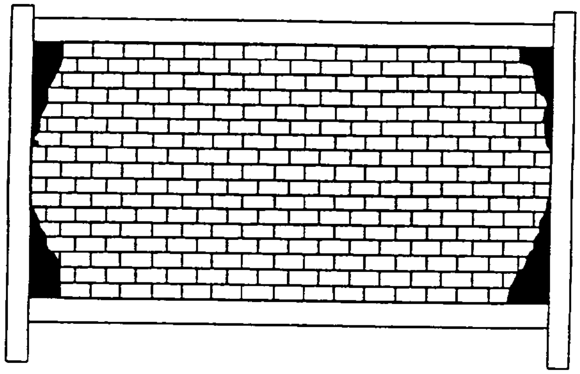

Fig. 3: Failure mode of the first two experiments.

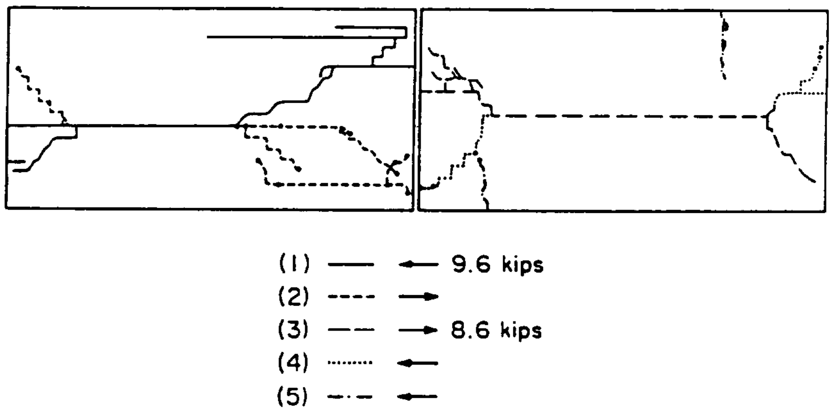

Fig. 4: Cracking sequence of the second two–bay infilled frame.

needed for the parameter estimation. Strain measurements consisted of strain rosettes on the walls to estimate principal strain directions and values, and strain gages at selected points in the frame members to estimate the member curvatures.

The failure mode in the first two experiments (one single–bay and one two–bay) was dominated by crushing of the corners of the infills (Fig. 3). The extent of the crushing zone was about 0.3 times the bay panel height, which agrees with the predicted contact zone between the wall and the frame. In the third experiment where a two bay frame was tested, corners did not crush, but there was extensive horizontal and diagonal cracking (Fig. 4). In this third test, a wall grid of absolute displacement measurements was recorded for later use in the analytical model to estimate several important material parameters. The reason for the difference in behavior between the first two experiments and the third one, is the relatively low strength concrete blocks and the high strength mortar in the first two tests. In all three tests large gaps developed between the infill and the frame, along with sliding of the wall. Crushing and cracking progressed as a result of cycling at a load level close to the failure load, and indicated the sudden nature of the failure mode.

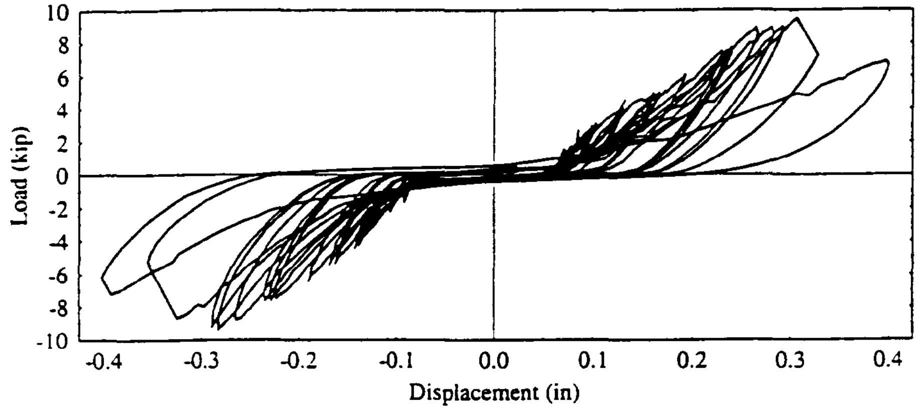

Fig. 5: Load–displacement relationship of the second two–bay infilled frame.

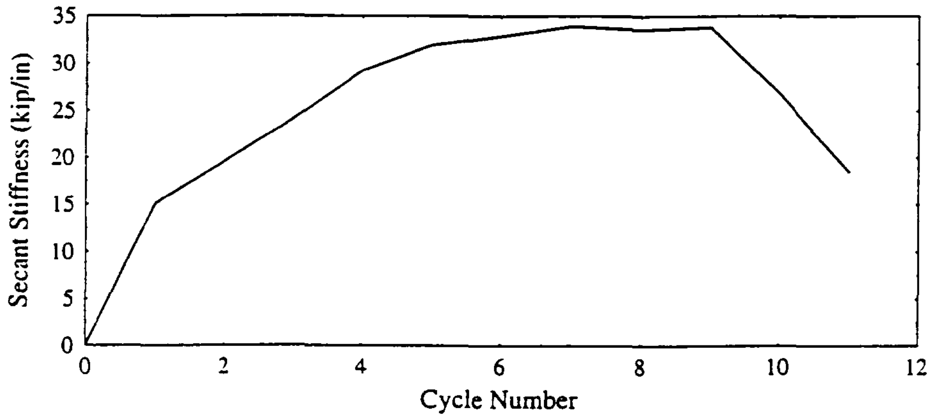

Fig. 6: Stiffness change with load cycles of the second two–bay infilled frame.

The measured top lateral displacement for the third experiment is plotted against the applied lateral load in Fig. 5. It should be noted that the capacity of the two–bay frame was about twice that of the one–bay frame. It is reported by Liauw and Lo (3) that the strength of two–bay infilled frames is about 1.5 to 1.6 of the corresponding strength of single–bay infilled frame, not twice. This disagreement can be attributed to the difference in the aspect ratios (2 in the present tests and 1.5 in (3)) and the fact that the loading in (3) was concentrated at the side of the frame rather than being at the middle of the beam a in the present tests. As long as the applied displacement is less than the gap between the wall and the columns, the frame acts as if it were bare one. Once walls bear against the frame members, stiffness increases significantly as illustrated in Fig. 2. The stiffness of the models reduced to about half of the peak stiffness in the second two–bay frame as illustrated in Fig. 6.

These tests confirmed the applicability of the multi–strut model for infills without openings. The principal strain direction decreased from an initial angle of 40^o to about 23^o as the cycling progressed as illustrated in Fig. 7. Strain measurements along frame members showed that the bending moments were very small initially. However, these bending moments increased steadily in the tests with corner crushing.

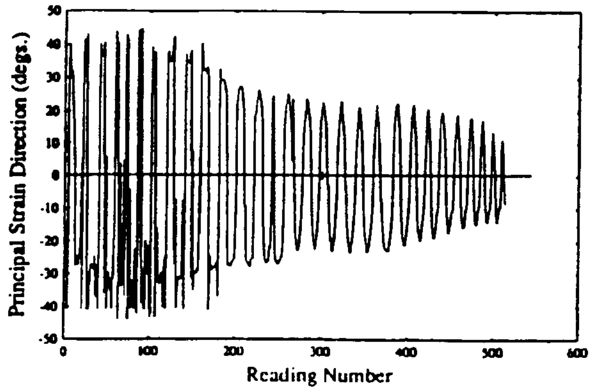

Fig. 7: Principal strain direction of the first two–bay infilled frame.

CONCLUSIONS

The presented FE approach captures the complex behavior of infilled LRC or SRCS frames. Due to the the generality of the FEM, accurate treatment for complicated geometrical configurations such as infill walls with openings is possible.

Based on the conducted reduced scale testing, the compression-only six struts model is justified. The complexities and uncertainties inherent in the infill materials justifies the use of approximate material modeling.

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SIMULATION OF THE RECORDED RESPONSE OF UNREINFORCED (URM) INFILL BUILDINGS

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ABSTRACT

The Strong Motion Instrumentation Program of the California Department of Mines and Geology (CSMIP) has obtained records of the response of four buildings with unreinforced masonry (URM) infills. The response was to the Landers, Upland and Sierra Madre earthquakes. The objective of this research was to replicate by computer analysis the CSMIP records. Three dimensional elastic computer models were prepared from data obtained from the original construction documents. The URM infills were modeled as diagonal braces in the frame. The stiffness properties of the infills were determined by a nonlinear finite element analysis.

INTRODUCTION

The Strong Instrumentation Program of the California Department of Mines and Geology (CSMIP) has instrumented buildings with unreinforced masonry infills. Four of these buildings were shaken by the Landers earthquake. Two of these buildings had been shaken by the 1990 Upland and the 1991 Sierra Madre earthquakes.

These buildings are: .

- A six-story commercial building in Pasadena (CSMIP Station No. 24541) constructed in 1906. It has a steel frame infilled with unreinforced brick masonry. The maximum acceleration at the basement level was 0.195g during the Sierra Madre earthquake and 0.04 g during the Landers earthquake.
- A six-story commercial building in Pomona (CSMIP Station No. 23544) constructed in 1923. It has a reinforced concrete frame with unreinforced brick masonry infills. The maximum acceleration at the basement level was 0.13g for the 1990 Upland earthquake and 0.07g for the 1992 Landers earthquake.
- A nine-story office building in Los Angeles (CSMIP Station No. 24579) that is L-shaped in plan. It was constructed in 1923 and has a reinforced concrete frame with unreinforced masonry infills. The maximum acceleration at the basement level was 0.05g during the Landers earthquake.
- A twelve-story commercial/office building in Los Angeles (CSMIP Station No. 24581) that was constructed in 1925. It has a concrete encased steel frame and unreinforced brick masonry infills. The maximum acceleration at the basement floor level was 0.04 g during the Landers earthquake.

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These buildings have very significant vertical and plan irregularities. The lateral resistance was provided by the frames and the unreinforced masonry that is infilled into the frame. The masonry is multi-wythe brick laid in lime, Portland cement, and mortar. Cast stone, terra cotta and brick veneer wythes are a part of the masonry infills. The material properties of the masonry were estimated by comparison with masonry that had been tested by the flat jack method.

The problem was to simulate the recorded response of these buildings to the motions recorded at the lowest level. The mechanical properties of the structural materials were estimated and effects of systems such as stairs that are continuous between floors and interior partitioning, were neglected.

RESEARCH PLAN

The existing structural systems, the mass of the building and the geometry of the system was determined by review of the existing drawings. The weight and center of gravity of each story level above the base of the building was estimated. Elevations of each column-beam line and sketches of the infilled bays were prepared. The size and location of all openings within the infilled bays were noted on the elevations.

The exterior elevations were used to determine "typical" infills. The parameters for establishing "typical" infills were:

- Moment of inertia and area of the confining frame members.
- Story height and length of the infilled bay.
- Location of the openings relative to the frame and number and size of the openings.

The initial compressive modulus of elasticity, the tensile cracking stress, the strain associated with peak compressive stress and the peak compressive stress were chosen by experience and/or visual evaluation of the exposed masonry. The force-displacement relationship for each of the "typical" infill panels was calculated by use of a nonlinear finite element program developed by R.D. Ewing, A. El-Mustapha and J. Kariotis (2). An effective stiffness of a pair of diagonal braces within the bay of the infilled frame was substituted for the unreinforced masonry. This effective stiffness was determined by the following process:

- For each typical infill bay configuration, the confining frame and the masonry was analyzed by the nonlinear FEM.
- The confining frame was analyzed without any infill.
- The force-displacement relationships of the infilled frame and the frame alone was differenced.
- The area and modulus of elasticity of the equivalent diagonal braces was calculated to provide an effective system stiffness at the story displacement as determined by evaluation of the CSMIP displacement data.

The process of obtaining a best-fit computer replication was an iterative process. The viscous damping used in the linear-elastic model was established using the best available data. The computed periods of the linear-elastic model were compared to estimated periods extracted from the CSMIP data. Rotational periods for the SAP model and for the CSMIP data were compared. The parameters that were modified to improve the fit were the effective stiffness of the frame members, the effective stiffness of the diagonal struts that represent the infills and the percent of critical damping. These parameters are variables as the materials properties of the concrete frames, the stiffness of the beam-column connections of the steel frames and the material properties of infills are estimated, not quantified by physical testing.

The CSMIP Station Nos. 24579 and 23544 have reinforced concrete frames. Station No. 23455 has a severe plan irregularity below the second floor level and a lesser degree of plan irregularity from the second floor to the roof level. A mass irregularity is at the roof level. The lateral resistance at the east and south is provided by the concrete frame and minimal infills. Station No. 24579 is an L-shaped building that has a single story garage structure constructed in the portion of the property not occupied by the nine-story building. Reinforced concrete walls separate the occupancies. These reinforced concrete infills were analyzed by methods identical to those used for unreinforced masonry infills.

The CSMIP Station No. 24541 and 24581 have structural steel frames and multi-wythe brick masonry infills. Station No. 24541 has a severe plan and stiffness irregularity below the second floor. The south and east street fronts have only frames to resist lateral displacements. The west wall below the second floor is infilled with a small window in each bay. The north end is highly perforated with openings. Above the second floor the infilled walls at the perimeter of the light well add stiffness, especially in the north-south direction. The exterior walls have more symmetry in plan above the second floor except that the east and south walls are thicker. This moves the probable rotational center of the building above the second floor in the opposite direction from the probable location below the second floor. Station No. 24581 is nearly symmetrical in plan in the north-south direction. A plan irregularity exists in the east-west direction. The floor beams are encased in concrete. The columns of both buildings are encased in brick or clay tile. The floor beams in Station No. 24541 support a clay tile arch system topped with an unreinforced concrete slab.

All beams that frame into the building columns were included in the model. All beam-column joints were considered rigid. This assumption was used for the structural steel systems regardless of the detailed connection. The analyses of CSMIP Station No. 24581 found that the stiffness of the steel beams in the frame must be adjusted to less than 100% to account for the flexibility of the beam-column connection. The diagonal members were given pinned-ends to eliminate any contribution to flexural stiffness. Eighty percent of the stiffness determined from the FEM analysis was used as the initial elastic stiffness. This was chosen to estimate the stiffness on reloading to a stabilized force-displacement envelope. The base of the building was taken as the top of the first floor. This assumption was made as reinforced concrete perimeter walls are below this level. All columns were considered fixed at this level. This assumption and the assumption of a fixed base building, that is no rotation of the building on the supporting soils, increased the effective stiffness of the computer model of the building over that of the existing building. There are three critical unknowns as to the dynamic response of these buildings. These are:

- Translational stiffness on the x and y axes.
- Rotational stiffness at levels of plan irregularity.
- Damping that occurred during the recorded time.

Matching of the CSMIP time-displacement records would require that all three of these critical unknowns be calculable. The translation and torsional stiffness was calculated for the computer model using "typical" infilled bays. The damping force used in the linear-elastic model was a viscous damper that functions full time during the time-history analysis. The percentage of critical damping is calculated for the structural stiffness of each mode. The dynamic damping force is related to the response velocity. The actual damping is hysteretic and does not have a damping force acting opposite to the loading force on a loading cycle. The real damping is due to nonlinear cyclic distortion of the masonry infill. The maximum damping ratio used in these analyses was five percent of critical damping.

A limited quantity of data is presented in this paper for CSMIP Station No. 24541. The data for all buildings analyzed is presented in a similarly titled report (1).

RESULTS OF THE ELASTIC ANALYSIS OF CSMIP STATION NO. 24541

Landers Earthquake

This building has one significant translational line of resistance below the second floor. All modes with significant mass coupling are torsional. The torsional stiffness above the second floor greatly exceeds the torsional stiffness below the second floor. The stiffness of the infill panels was taken directly from the FEM analyses. No reduction in stiffness of the infill due to cyclic loading was taken. The stiffness of the structural steel frame was not deducted from the infilled system stiffness. The material properties used to model the masonry were identical to that used for the other three buildings. It is possible that the estimated materials properties exceeds those that would be determined by testing. The SAP model generally over estimated the dynamic displacement at the second floor level and under estimated the displacements at the roof. Five percent damping was used for all modes. Six modes of response were used in the SAP model. The relative displacements shown in Table No. 1 have a reasonable agreement. The displacement-time record shown in Figure 1 is out of phase. The difference appears to be related to the frequency of rotational modes.

Sierra Madre Earthquake

The comparison of measured and calculated displacements is shown in Table No. 2. The stiffness model used for these predictions is the same as used for predicting the displacements caused by the Landers earthquake. The quality of the predictions when plotted in time vs. displacement, Figure 2, is better in phase relationship.

CONCLUSION

The elastic three-dimensional analyses successfully predicted the maximum values of the relative displacement of four buildings with URM infills. The comparative time-histories shows that the technical limitations of the elastic model to replicate nonlinear behavior limits the matching of displacement records to a small segment of time. Variables used to improve the fit of the calculated data to the recorded data included damping, reduction of the stiffness of concrete frames from uncracked stiffness, reduction of the stiffness of the equivalent diagonal braces from that determined by the nonlinear finite element analysis and reduction of the stiffness of beams in a steel frame due to flexibility of the beam-column connection. There is technical substantiation for the values used in these studies. Additional research is needed to establish most probable values of element stiffness but the methodology used in this research has been shown to be adequate.

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TABLE NO. 1

COMPARISON OF DISPLACEMENTS FOR STATION NO. 24541, LANDERS EQ.

DIRECTION	FLOOR & LOCATION	SENSOR	CSMIP DATA MAX. INCHES	SAP DATA MAX. INCHES
N-S	W. 2nd Fl.	1	0.24	0.26
N-S	E. 2nd Fl.	2	0.90	1.34
E-W	N. 2nd Fl.	11	0.80	1.05
E-W	S. 2nd Fl.	12	0.75	0.85
N-S	W. Roof	3	1.40	1.06
N-S	E. Roof	4	2.00	1.97
E-W	N.W. Roof	5	2.50	2.00
E-W	N.E. Roof	6	2.50	2.00
E-W	Mid Roof	7	2.02	1.57
E-W	S. Roof	8	1.35	1.35

TABLE NO. 2

COMPARISON OF DISPLACEMENTS FOR STATION NO. 24541, SIERRA MADRE EQ.

DIRECTION	FLOOR & LOCATION	SENSOR	CSMIP DATA MAX. INCHES	SAP DATA MAX. INCHES
N-S	W. 2nd Fl.	1	0.20	0.20
N-S	E. 2nd Fl.	2	0.90	0.86
E-W	N. 2nd Fl.	4	0.50	0.75
E-W	S. 2nd Fl.	12	0.50	0.60
N-S	W. Roof	3	1.60	0.84
N-S	E. Roof	4	1.60	1.15
E-W	N.W. Roof	5	1.50	1.52
E-W	N.E. Roof	6	1.50	1.52
E-W	Mid Roof	7	0.98	1.02
E-W	S. Roof	8	0.80	0.91

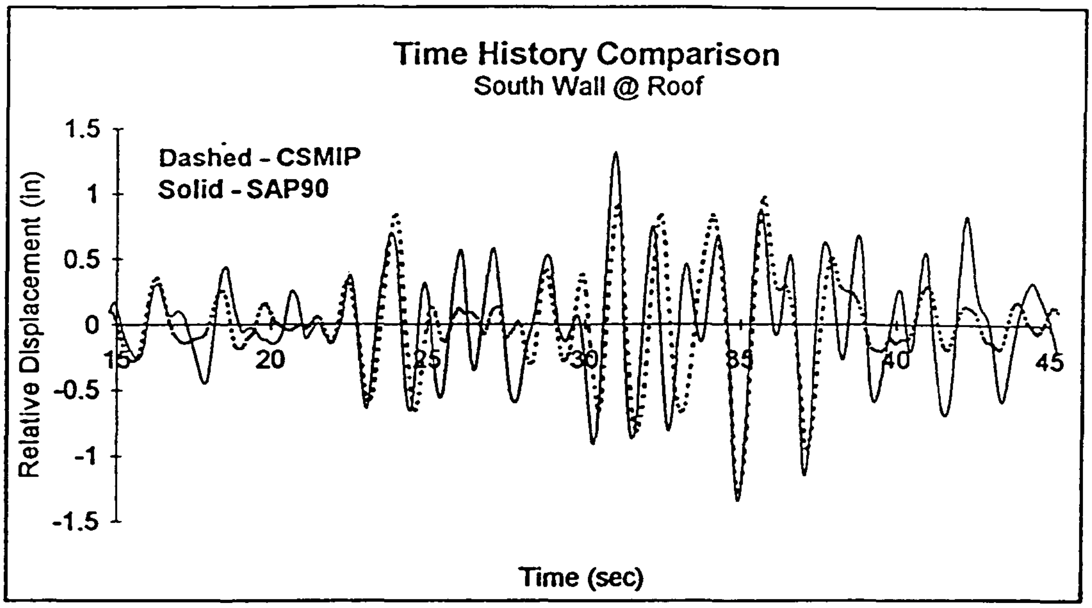

FIGURE NO. 1 - RESPONSE OF SENSOR NO. 8 AT ROOF, LANDERS EQ.

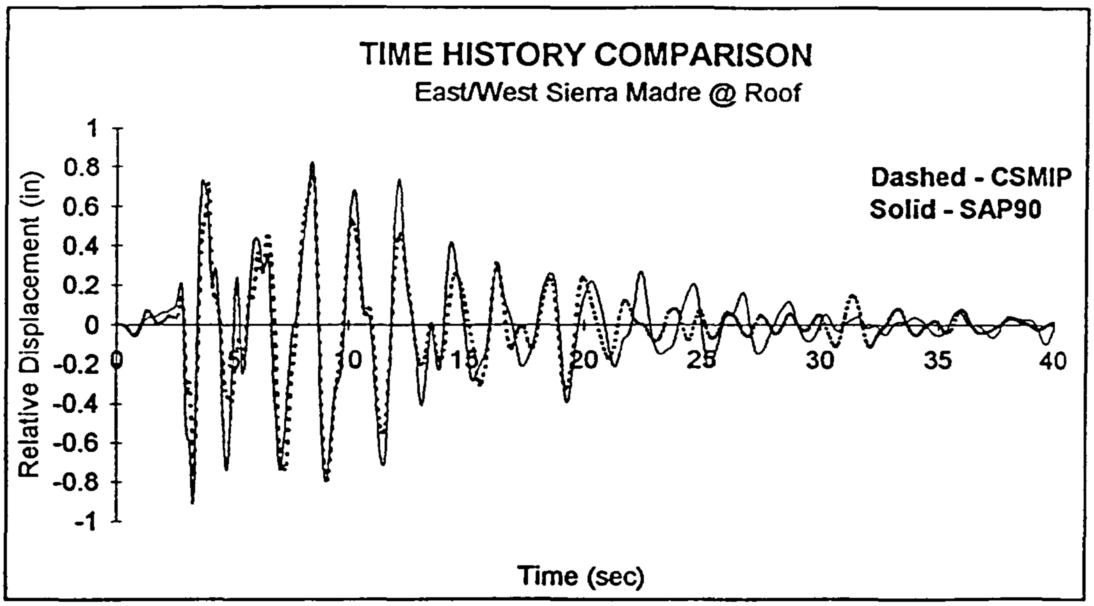

FIGURE NO. 2 - RESPONSE OF SENSOR NO. 8 AT ROOF, SIERRA MADRE EQ.

NUMERICAL MODELING OF CLAY TILE INFILLS

Roger D. Flanagan[1], Michael A. Tenbus[1], and Richard M. Bennett[2]

ABSTRACT

An analysis methodology for large complex industrial facilities with unreinforced structural clay tile infilled frames is presented. The method is based on a nonlinear equivalent strut to model the in-plane behavior, and is used in conjunction with typical equivalent static or response spectra techniques. Out-of-plane behavior is examined on a panel by panel basis by comparing panel accelerations to the capacity considering panel arching. The development and validation of the methodology is based on over twenty large-scale experimental tests.

INTRODUCTION

Structural clay tile was a popular building material for the first half of the century. Many existing structures have unreinforced structural clay tile, often acting as an infill. The infill can enhance the seismic behavior of the structure. However, it is difficult to quantify the actual effects of the infill, and evaluate the safety and capacity of the structure. This paper summarizes preliminary recommendations that were developed for the analysis of a large complex industrial facility that had numerous buildings with simply connected steel framing and unreinforced structural clay tile infills. The recommendations are based on an extensive test program, as described briefly below.

Thirteen cyclic in-plane racking tests were performed on large-scale clay tile infill specimens to determine the behavior. The tests were designed to evaluate a variety of engineering conditions including varying frame stiffness, single and double wythe construction, varying panel aspect ratio, offset panels, weak column/infill interface, and corner openings. The behavior was characterized first by diagonal panel cracking followed by corner crushing at ultimate capacity. Significant postpeak strength was observed indicating continued energy absorption capability. Details of the in-plane testing are contained in Flanagan et al. (1993b).

The stability of infill panels under out-of-plane inter-story drift was evaluated by testing two specimens (Flanagan and Bennett, 1992). There was little relative movement of the infill panels with respect to the steel framing. Infills constructed snugly to the framing, but without ties or other reinforcing, were shown to remain stable when subjected to cyclic out-of-plane drift displacements within the range that a typical infill might experience. The specimens were then loaded in-plane to failure, with only minimal degradation of in-plane stiffness and strength.

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Three infill panels were tested out-of-plane with a uniform lateral loading to simulate the inertial effects of the panel normal to its plane. Three different wall thickness were tested, and the ultimate capacities appeared to vary with the square of the panel thickness. The behavior was dominated by arching action of the masonry (Flanagan and Bennett, 1994).

Two additional specimens were tested using sequential in-plane racking and out-of-plane pressure loadings. One specimen showed a 15% reduction in out-of-plane capacity after being first loaded to approximately 75% of ultimate in-plane capacity. Approximately the same ultimate in-plane capacity was observed in a specimen that was first loaded with an uniform out-of-plane pressure to 75% of ultimate. Finally, one specimen was loaded with simultaneous in-plane and out-of-plane forces. The in-plane drift was held as uniform lateral pressure load-unload cycles were applied. Significant out-of-plane capacity was observed at high levels of in-plane drift. Although some interaction of in-plane and out-of-plane forces exists, the interaction did not appear to be significant at low to moderate levels of loading.

Supporting material property tests included unit tile testing (Flanagan et al, 1993a), and uniaxial compressive testing of 2′ by 4′ prisms (Boyd, 1993). Average gross compressive strengths parallel to the tile cores were 436 psi and 372 psi for 8" single wythe and 13" multi-wythe prisms respectively. Average gross compressive strengths normal to the tile cores were 810 psi and 332 psi for 8" and 13" prisms respectively.

ANALYSIS METHODOLOGY

Seismic analyses of complex industrial facilities are typically performed using three dimensional equivalent static or response spectra techniques. The methodology presented for seismic evaluation of clay tile infills is also applicable for simpler conditions where two dimensional analysis or cruder approximate techniques are employed.

In-Plane Methods

The in-plane behavior of the infill is modeled using an equivalent compression strut. Because the stiffness of the infill is a function of the load magnitude, the strut model is necessarily nonlinear. The conditions required for a compression strut to develop must be present. Openings, interface gaps, and other discontinuities may affect development of a compression diagonal. Equivalent strut methods are typically formulated without consideration of vertical forces. Thus, vertical loads should be omitted during lateral load analysis and superimposed later.

Attention to detail in modeling the compression strut is also important. For instance, the use of a nonsymmetric strut may result in erroneous axial forces in members at reentrant corners of buildings. It is recommended that a compression only strut be used in each direction. If a compression only element is not available, a tension-compression truss member may be used with half the strut area in each diagonal direction.

The compression strut formulation suggested is based on work by Stafford-Smith and Carter (1969). The method has been extended to include nonlinear behavior of the infill. The contribution to lateral stiffness may be computed using Equations 1-4, see Figure 1. The length of column bearing on the infill, α (in), is given by

$$\alpha = \frac{\pi}{C\lambda} \tag{1}$$

where

$$\lambda = \sqrt[4]{\frac{E_m\, t\, \sin 2\theta}{4\, E_s\, I_c\, h'}} \tag{2}$$

in which E_m is the gross elastic modulus of the masonry (psi), E_s I_c the flexural rigidity of the column (psi, in^4), θ the slope of the infill diagonal to the horizontal, t the infill gross thickness (in), and h' the infill height (in). The width of the equivalent strut, w (in), is

$$w = \frac{\alpha}{\cos\theta} \tag{3}$$

and the area of the equivalent strut, a (in^2) is

$$a = wt \tag{4}$$

It is suggested that the length of contact, α, be limited to 20% of the panel height (h'). The parameter C varies with the in-plane drift displacement, see Table 1.

For specimens in the range of height tested, ultimate in-plane capacity of the clay tile infills was limited by a 1" in-plane drift displacement. This corresponds to an in-plane drift of approximately 1%. An upper limit of 50 kips for single wythe panels and 60 kips for double wythe panels should also be used for the in-plane capacity of the infills (maximum horizontal component of equivalent strut force). Stiff frames with a greater length of contact along the infill will be limited by this criteria and will reach ultimate capacity at a lower in-plane displacement.

Beyond peak capacity the in-plane strength of the infill is assumed to reduce to 75% of peak at an in-plane drift of 1.5 times the displacement at peak. Postpeak testing of the infilled frames indicates significant capacity well beyond a displacement of 1.5 times that reached at peak capacity. However, this displacement serves as a practical limit in predicting repairable damage levels of the masonry as well as an indicator of potential damage to the columns.

Out-of-Plane Methods

Out-of-plane story drifts due to the relative top-bottom displacements of orthogonal walls may be evaluated based on an allowable drift ratio of 1%. Panels drifts exceeding 1% may be stable but require more detailed evaluation.

Out-of-plane inertial loads are evaluated for individual panels based on the maximum acceleration computed for a given elevation. Since the frequency of the panel is typically significantly higher than the frequency of the structure, the panel acceleration is approximately the nodal acceleration. If the panel and structural frequencies are similar, magnification factors can be determined similar to methods used for equipment evaluation. The panel load is computed as the mass times the maximum of the peak accelerations at the supporting nodes. This is compared to the static arching capacity. A convenient method of estimating the panel capacity is the empirical equations presented by Dawe and Seah (1989). These equations were developed from tests using concrete masonry units, but have shown good correlation with the tests using structural clay tile. The masonry strength used in the calculation is from the prism tests with the compressive load normal to the cores, which is the approximate primary behavior mode of the panels (e.g., vertical arching).

ILLUSTRATIVE EXAMPLE

A three dimensional modal analysis of a prototypical industrial facility was performed using the previously described analysis methodology. Although simplifications have been made in the details of the structure, this prototypical building is of the same general size, mass, eccentricity and construction as many existing facilities. Seismic input used in the analysis was a 0.15g Newmark-Hall response spectra with 10% damping. The spectra was used for both horizontal acceleration inputs and the vertical input was assumed to be 2/3 of the horizontal. The modal analysis results for different spectra directions and modes were combined using the square root of the sum of the squares method (SRSS). Several iterations were required to account for the nonlinear strut behavior, but convergence was quite rapid.

Plan and elevation views of the structure are shown in Figure 2. The building includes three full stories with a penthouse creating part of a fourth story. W16x67 columns are used throughout with W24x76 floor beams and W16x45 roof purlins. The steel frame is infilled with structural clay tile around the perimeter of the building and in two interior bays. An open area with no masonry is assumed in the two bays comprising the reentrant corner of the building. Infill walls in the first story are 13" double wythe construction and the upper stories are 8" single wythe construction. The double wythe walls were assumed to weigh 52 psf and the single wythe walls 32 psf. Total floor loads were taken as 120 psf and roof loads taken as 30 psf. Floor diaphragms are 6" concrete slabs modeled with plate elements. Roof diaphragms were considered completely flexible and not modeled, except for some light diagonal horizontal bracing.

The fundamental frequency is primarily an E-W mode which picks up 90% of the E-W mass at 1.16 Hz. The primary N-S frequency is at 2.19 Hz, and picks up 78% of the N-S mass. Other significant horizontal modes were at 2.11 Hz (twisting mode with 10% of the N-S mass), 3.34 Hz (twisting mode with 8% of the E-W mass), and 6.0 Hz (second N-S mode in the penthouse portion with 9% of the N-S mass).

Numerous modes were needed to pick up the mass in the vertical direction. The most mass, 24%, was picked up at 11.9 Hz. There was a concern with the validity of these modes due to the use of the equivalent struts for the infills. The problem was rerun with no diagonal equivalent struts. The results for the vertical modes and frequencies remained essentially the same, indicating the vertical response was primarily in the columns, and not the struts.

Typical E-W results are given in Table 2 for column line 3, bay CD. Using the in-plane drift displacements or C values, the masonry damage levels may be roughly correlated with those in Table 1. The lower stories are near ultimate capacity and some tile cracking in the corners would be expected, whereas only minor diagonal mortar joint cracking would be expected in the top floors. Similar forces and behavior were seen in the N-S direction, although the story drifts tended to be smaller due to the bending of the columns about their strong axes. The higher column stiffness also caused the panels in the N-S direction to experience higher forces at smaller displacements.

With respect to out-of-plane forces, the drifts were small enough so that no problems due to inter-story drift are expected. The maximum nodal acceleration of 0.39g occurred in the N-S direction in column A1 at level 1045. The capacity of the panel using the arching equations is 0.44 psi, or 2g. Since this is approximately 5 times the actual nodal acceleration, no problems with out-of-plane panel stability are expected.

CONCLUSIONS

A rational method for seismic analysis of steel frames infilled with structural clay tile is presented. The methodology has been validated with a significant amount of large-scale experimental results. The method accommodates standard techniques for dynamic analysis while incorporating the nonlinear behavior of masonry infills in a computationally efficient manner.

ACKNOWLEDGEMENTS

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Table 1 Typical Values of C for Varying In-Plane Drift Displacements

C	Displacement (in)	Typical Infill Damage
5	0.0-0.05	None
7	0.05-0.2	Diagonal Mortar Joint Cracking
11	0.2-0.4	Off Diagonal Mortar Joint Cracking
14	0.4-0.6	Banded Diagonal Mortar Joint Cracking
16	0.6-0.8	Corner Mortar Crushing and Tile Cracking
18	0.8-1.0	Tile Faceshell Splitting (Corner Regions)
-	1.5 x Disp @ Ultimate	Tile Faceshell Spalling

Table 2 Summary of In-Plane Forces for Column Line 3 Bay CD

Elevation	Story	C	Total Δ (in)	Story δ (in)	Strut Force (k)	Strut Allowable (k)
1060			1.36			
	4	7		0.08	8.3	58.3
1045			1.28			
	3	7		0.15	16.5	58.3
1030			1.13			
	2	14		0.54	28.4	58.3
1015			0.60			
	1	16		0.6	46.6	70.0
1000			0			

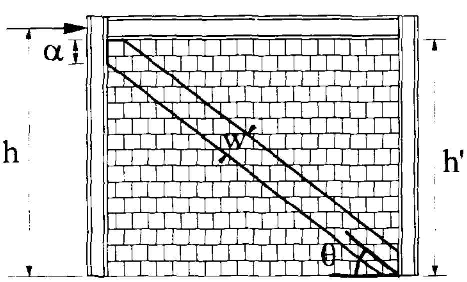

Figure 1 Compressive Strut

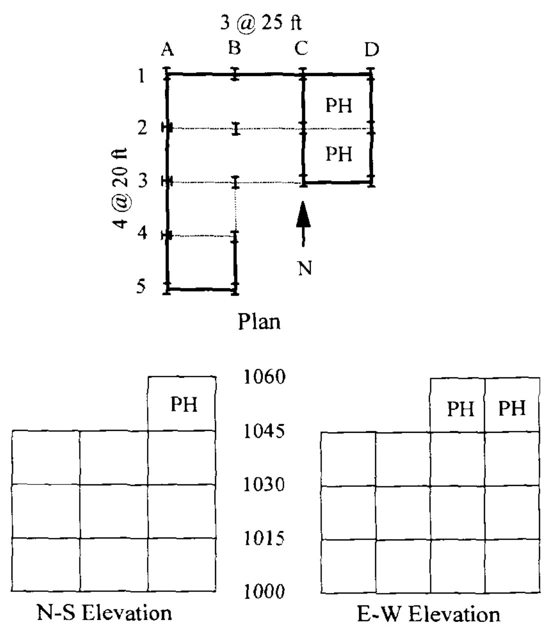

Figure 2 Illustrative Example

Section II

Design Criteria and Case Studies

Public Policy vs. Seismic Design: Cost and Performance Criteria for Seismic Rehabilitation of URM Infill Frame Buildings

The Oakland Experience During Loma Prieta - Case Histories

Structural Framing Systems: 1890-1920, Implications for Seismic Retrofit

Impact of Infilled Masonry Walls on the Response of Buildings in Moderate Seismic Zones

PUBLIC POLICY VS. SEISMIC DESIGN: COST AND PERFORMANCE CRITERIA FOR SEISMIC REHABILITATION OF URM INFILL FRAME BUILDINGS

Randolph Langenbach[1]

ABSTRACT

This paper explores the public policy issues raised by the current concerns over the safety of older buildings constructed with masonry infill frames. In recent years, there has been a vast range of costs of seismic upgrade projects involving large and significant downtown infill-frame buildings. Frequently even the most costly of such projects are accompanied by a strongly held rationale that the work is required to meet the minimum acceptable standards for life safety and limitation of property damage. The characteristics of the infill--frame system itself tends to accentuate these debates because of the fact that the infill masonry can crack at a very low force level, even though the total construction system, consisting of the masonry bounded by the structural frame, has proved to be far more ductile and resilient than masonry is when taken alone. FEMA, as a funding source for both post-disaster relief and hazard mitigation, has found itself at the center of the debates over repair and upgrade solutions involving this system when it tries to balance the regulatory requirements for code conformance and cost-effectiveness.

INTRODUCTION

The Loma Prieta Earthquake has brought the issue of infill frames into the forefront, both in terms of historic preservation and in terms of seismic safety public policy. Masonry infill frame construction in the United States hitherto had not been considered to be a major seismic hazard, but the 1989 Earthquake delivered a punch which left widespread damage uniquely concentrated in infill-wall buildings. With the waves extending out from an epicenter distant from the Bay Area, the soft alluvial soils under San Francisco and Oakland responded with longer period vibration. This motion then resonated with the mid-rise frame buildings in the downtowns of both cities causing extensive cracking in the walls and fallen bits of terra cotta ornament. Only in rare cases was damage sustained in the structural frames as well as in the masonry. Most often, it was the cracking of the masonry which got people alarmed, and which led to the closure of many buildings.

Historically, if the kind of cracking which was observed in many of these buildings had been observed following a major earthquake, people would have most likely considered the damage minor. Repairs consisting of spackle and paint would have been undertaken, and the buildings would have continued in use. For example, at the Ferry Building in San Francisco, the evidence of shifted masonry and cracked arches from the 1906 Earthquake is still visible.

Following the 1989 earthquake, however, expectations have been greater - or at least significantly different. Cracks are no longer as acceptable, perhaps because at all levels of decision making, from the engineers in the field, to the public policy leaders, masonry cracks are no longer as well understood. Masonry is not now an accepted building material, and

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masonry buildings have been singled out by officials in the State of California as being particularly dangerous. This contemporary condemnation has tended in the public's mind to spill over onto infill frame structures, stigmatizing many substantial and historically significant downtown buildings. After years of training in reinforced concrete, where cracks can indicate incipient structural failure, engineers have often questioned the stability of infill-wall buildings when only minor cracking (loss of stiffness), as opposed to material crushing (structural degredation), was observed after an earthquake.

In Oakland, which has the largest concentration of infill-wall buildings which were closed following the 1989 earthquake, a repairs ordinance was enacted within the first two months after the quake. This ordinance requires that, in addition to the repair of the earthquake damage, a full seismic upgrade to the Uniform Building Code be carried out before many of the affected buildings could be reoccupied. In almost every case, this requirement could not be met for economic reasons unless there was a large infusion of government funding. At the time it seemed wise public policy. Now in retrospect, four years after the earthquake, most of these buildings continue to sit empty and deteriorating, their owners often in bankruptcy, and with no hazard mitigation of any kind. The question of whether these buildings were so dangerous as to require such a drastic requirement has not been completely answered, but the public policy issues this situation raises are exactly the kind of public policy issues that FEMA has found itself embroiled in since the Loma Prieta earthquake.

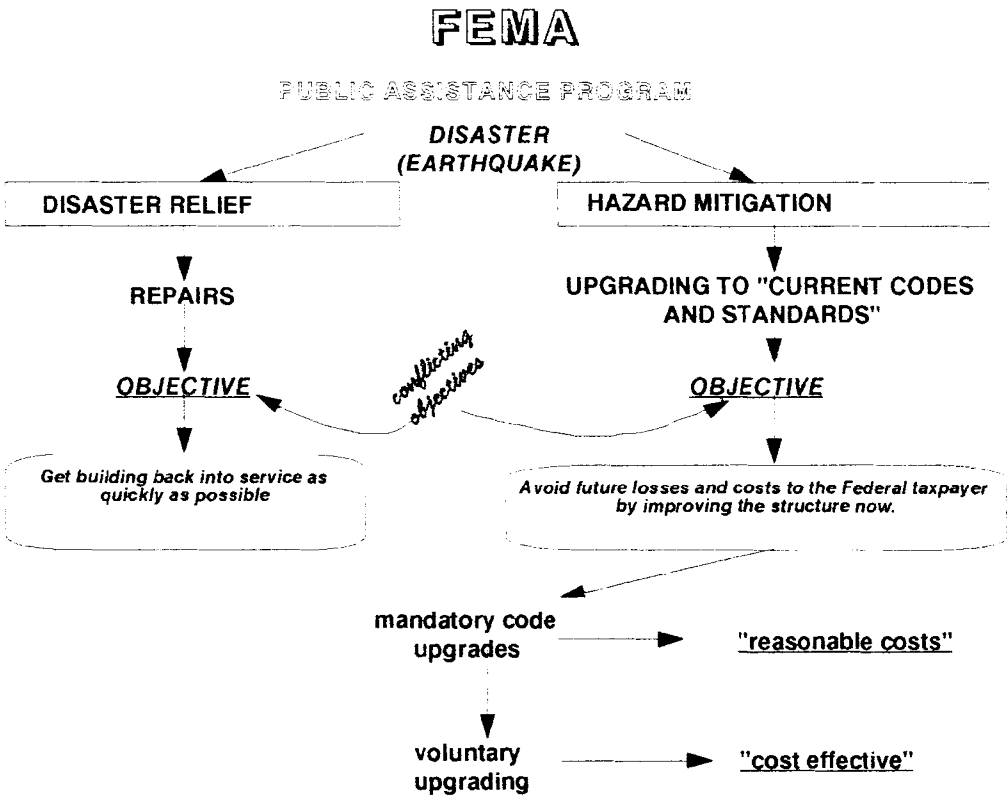
FEMA PROGRAM CONSTRAINTS

The FEMA Public Assistance program provides Federal funding assistance to State and local governments, and selected non-profit organizations, to help repair disaster damaged buildings under their ownership or control. The primary objective is to get the affected buildings back in service as rapidly as possible, when, without outside government support, there may not be enough resources to allow a local government to deal with damage to its own infrastructure, much less to be ready to help the private sector with the task of rebuilding.

The second objective of the public assistance program is to provide funding for hazard mitigation improvements to the damaged structures, Such funded improvements are intended to reduce or avoid future disaster damage. The underlying justification for the funding of improvements, in addition to repairs, is that the reduction of future damage will benefit the public and save the goverment money over time.

Following a disaster, these hazard mitigation objectives become commingled with the disaster relief objectives. Code upgrades as well as straight repairs are eligible for funding. In addition, the Regional Director has the discretion to award more money as a hazard mitigation grant for discretionary upgrades, even when applicable codes or local ordinances do not require the work.

Disaster relief and hazard mitigation, therefore, sometimes can be conflicting goals. The objective of using limited resources to repair and reopen damaged governmental buildings as quickly as possible is not the same as that of spending large sums of money on selected individual structures to avoid potential losses which may be way in the future. At the very least, such seismic upgrading takes a great deal of time as well as money, and down-time does not benefit distressed communities.



The damage to infill-wall buildings in the 1989 earthquake has served to bring this conflicting public policy goal into high relief. This is especially true because of the unique characteristics of this construction system, and the lack of agreement within the engineering community on economical upgrade strategies.

THE BUILDING CODE

Since the Loma Prieta Earthquake, public policy analysts have had to learn more about the building codes than they have ever wanted to learn. For FEMA, the building code has become the focus of many disputes because of the provision in the regulations that provides for funding eligibility based on the costs of meeting current code requirements.

No matter how laudable the reasoning behind this legislation, the use of the building code as a funding eligibility measure uses the code for something it was never intended for. The code sets a legal minimum standard - it does not provide a measure or any guidance in analyzing how far a given design may exceed that standard. In normal practice, cold economics tends to provide the kind of discipline which keeps designs from becoming excessive, but in Federally funded projects, there is a premium gained locally by making a project as expensive as can be remotely justified. Since the Federal government is not in a position to be in control of the design process on a state and local project, the Federal agencies involved in funding such projects cannot easily explore alternatives to that which is presented by a local applicant for funding. In the case of infill frame buildings, the problem is particularly acute simply because the prevailing codes, which are used to set Federal funding amounts, do not recognize infill frame construction.

THE SEISMIC UPGRADE COST SPREAD FOR MASONRY INFILL FRAME BUILDINGS

Masonry infill-wall buildings usually comprise the major structures in the economic life of a city. This construction technology evolved beginning in Chicago during the 1880's, gaining reputation as the "Chicago Frame" which has formed the basis for highrise construction. The common present day practice of hanging the exterior skin of a building as panels attached to the frame with flexible connections, rather than enclosing a building by laying masonry directly between and around the frame, is of relatively recent origin. It has only been possible to separate the skin from the frame after certain products, such as long lasting flexible caulking compounds, became available which could allow the building to accommodate significant movement in the skin membrane itself. The stiffer masonry walled frame buildings were able to accommodate small movements within the mortar joints.

Prior to the Loma Prieta Earthquake, few engineers and public officials had singled out this earlier highrise construction type as being uniquely hazardous. Seismic safety concerns had been focused on unreinforced masonry bearing wall buildings, as well as other construction types such as non-ductile (bare) concrete frame and tilt-up construction. With the 1989 earthquake, this has all changed. Concern over the safety of masonry infill-wall buildings has increased, despite the fact that no infill-frame buildings collapsed, or to my knowledge have ever collapsed because of degredation of the masonry infill in any earthquake in the United States. This added concern may in part be a result of the Mexico Earthquake of 1985 and other earthquakes which have revealed serious problems with a different type of infill-wall construction common in more recent reinforced concrete buildings in other countries.[2]

The Loma Prieta earthquake, the enactment of the Oakland *Repairs Ordinance*, and the enforcement of code trigger provisions such as San Francisco's building code section 104, has forced building owners to consider alternative,ways of upgrading, as well as repairing, these types of buildings. The vast extremes in approach and cost are revealed by two project examples in Oakland: (1) the Oakland City Hall, and (2) the Oakland Medical Building, (1904 Franklin St.)

Oakland City Hall, a 15 story steel frame & infill-wall building, is being repaired and seismically upgraded using base-isolation. It is currently under construction, with the total project cost, including interior improvements, estimated at $76 million. The FEMA / OES grant is $54 million for both the hard and soft costs of the repairs and seismic upgrade work itself. The Oakland Medical Building, which is a 10 story non-ductile concrete frame and infill-wall building, has been repaired and upgraded for a construction cost of $500,000, and a total project cost of $650,000, including engineering peer review costs.

The cost per square foot of the Oakland City Hall project total, including the interior renovation is a whopping $530/sq. ft. (this figure includes interior tenant improvements.) This includes an accounting for the 42,000 square foot reduction from the original 185,000

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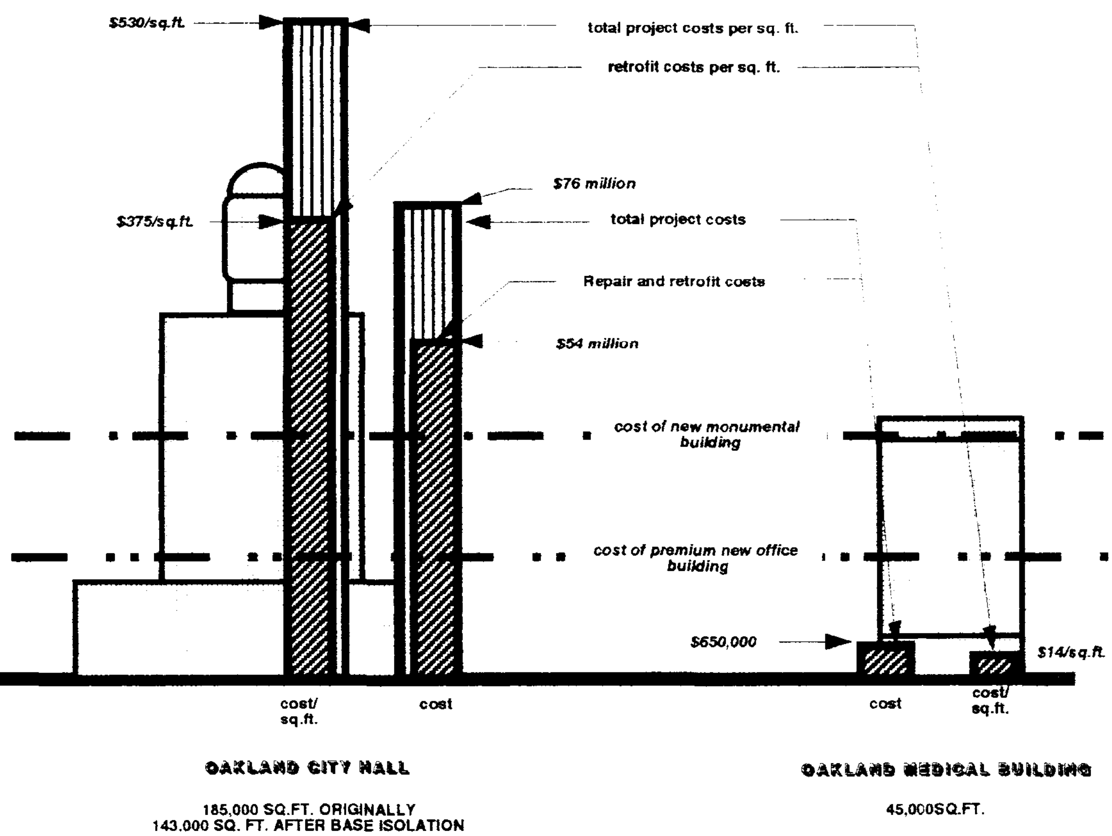
[2]There are many examples of concrete frame buildings, usually of the 1950's, 60's and 70's, in other countries which have collapsed. These are usually substantially different in their construction from the early 20th century American infill construction system, and have proven to be much more hazardous in earthquakes than the early 20th Century American infill-wall buildings have proven to be.

square foot size of the building which results from the loss of the basement and two floors of the tower because of the new truss work. For the FEMA portion of the work, the costs are \$375/sq. ft., still significantly higher than the cost of a new building.

The costs for the Oakland Medical Building, (on which the author worked as a consultant,) are \$14/sq. ft. for the total project, and \$11/sq. ft. for the construction hard costs (Some of the interiors of the public spaces were upgraded, but most of the tenant spaces did not have to be disturbed.) In addition, the Oakland Medical Building was upgraded without displacing any of the tenants, whereas any calculation of the total overall costs of the City Hall project should also include the off-site rental of space for a several year period in addition to the costs shown here.



Even without the rental and moving costs which would increase the spread, this represents a difference between the two projects of 35 times. A logarithmic difference in construction costs between a high and a low for projects involving the retrofit of infill-wall masonry (with the low end building being of more vulnerable non-ductile concrete construction) does show a spread in costs which is of serious concern for public policy analysts, politicians, and government employees dealing with other buildings of this type in the future. And yet, both projects were justified as cost-effective solutions to conformance with the requirements of the Oakland *Repairs Ordinance.*

There are clearly a number of factors involved here. A city hall would normally merit a more premium and expensive upgrade than a low-rent private office building; the city hall has more exterior and interior ornament and construction anomalies; and the city hall is a publicly funded project which can support costs which would not provide a return on investment in the private sector, if the work is found to be in the public interest. However, all of this does not explain such an extreme difference in costs when the prevailing codes and life-safety are the stated objectives for both of the projects.

The soft costs, including the design fees, testing, peer review, and inspection fees for these two projects are also significant. For the Oakland Medical Building, the soft costs of $150,000 come to almost 30% of the construction cost. In this case, the less costly project is not necessarily less complex from a design standpoint. It may be an example where for this type of work, owners must become aware that large savings in construction cost may require comparatively high design fees on a percentage basis because of the need to carry out complex engineering analysis to gain the savings. In the case of the City Hall, the costs, however, are also very high. The soft costs for the FEMA eligible portion of the work are $10 million, almost 20% of the construction cost. This results from the fact that the base-isolation system also requires extensive analysis. In addition, the architectural fees are higher because of the extent and complexity of the widespread gutting and remodelling of the monumental historical building.

MASONRY INFILL FRAME CONSTRUCTION

Despite the other differences between these two example projects, it is important to explore whether there is anything inherent about masonry infill-wall construction which tends to aggravate this difference, and which could conceivably justify the higher cost for publicly owned buildings. Masonry infill has one important characteristic which does tend to confuse the normal earthquake damage analysis - infill masonry cracks at relatively low seismic force levels. Since the masonry is an important part of the lateral resisting system in an infill-wall building, the initial cracking is identified as damage to the structural system. This has to be looked at not only in terms of engineering science, but also from a public policy standpoint. The philosophy behind the standard building codes is that in small earthquakes all structural damage should be avoided; in medium earthquakes, only minor structural damage is expected; and in major earthquakes, collapse is averted, but significant structural damage may be anticipated. The debate which has come to surround public policy with infill frame buildings is over the relative structural significance of that initial cracking in terms of the stability of the building as a whole. This is because the infill masonry is brittle, and when it is cracked, the common perception is that the building as a whole is at risk, based on the excedence of the elastic limit of one of its constituent structural materials.

The problem with this approach that the true strength of infill masonry as a system is not reached until the cracking becomes so severe that the infill masonry begins to be crushed and fall out of the frame. This cracking includes the compression failure of the masonry "struts" as well as the tension failure evidenced at the initial onset of cracking. Since the forces required to crush the masonry are much greater than to crack it in tension, the strength of the steel or concrete frame with masonry infill system is much greater than that of the masonry alone. As a result, the kind of cracking seen in most of the Loma Prieta simply does not represent <u>any</u> loss of ultimate strength because of the fact that, unless the ultimate strength of the masonry and frame system is exceeded (as evidenced by crushing as well as cracking of the masonry,) that capacity still exists. The presence or absense of minor tension cracks does not change this fact.

Regulators are faced with a dillemma when dealing with infill frame buildings with cracked infill because building code triggers are not always based on the ultimate strength of a structural system, especially when the elastic strength of one of its companant materials is so low. Infill frame buildings undergo a change of stiffness in relatively minor earthquakes which is largely permanent, and the visible cracking of plaster and masonry finishes can require widespread repairs, which arguably trigger some codes or ordinances. Since the ultimate strength of the building cannot be utilized until the elastic strength of the masonry infill has been exceeded across the interior and exterior walls of a large portion of the building, a great deal of cracking can be observed in a building which has none-the-less still retains its full existing seismic capacity. Since the ultimate strength of the building is not a function of the elastic strength of the masonry itself, any code trigger based on the initial cracking can stand in the way of economical repairs and divert funds from other structures with greater evidence of risk.

The result of this kind of gap between the brittle behavior of a component material in a system, and the ductile performance of the system taken as a whole, is that the simplistic application of prevailing codes does not lead to a unified engineering or public policy answer. This is futher complicated by the obvious fact that much cosmetic damage and even falling hazard risk may exist in a particular case during the initial brittle cracking phase of the building's response to an earthquake. A public official may make the decision that a building subject to any possibility of this must be strengthened to move the initial yield point of the masonry into the major earthquake category of the code. This is usually extremely costly to do. In certain cases, it may in fact only be possible by using a system like base isolation.

CONCLUSION

Arguments over upgrade requirements and vast cost differences between proposed building code based solutions places a Federal agency like FEMA into a dilemma. The agency must either (1) spend on the upgrade of a single structure an extraordinary sum of money, sometimes way out of proportion to the costs of the repairs and even the costs of new construction, or (2) find some way both to explain and to justify a level of interior and exterior surface cracking which will be in excess of that expected under current code design for new public buildings. This situation is further complicated by the fact that many of these buildings are of historic significance with historic finishes on the masonry walls expected to crack if the building is left to respond to an earthquake without a stiffening upgrade or base-isolation.

To deal with this public policy problem, FEMA has recently begun using benefit/cost analyses of proposed seismic upgrade projects. The purpose of this study is to try to determine what the present day value of future benefits is for different levels of upgrade based on both future physical damage and potential for loss of life, either from collapse or falling debris. When all of this is put into financial terms, based on broadly accepted values, this is expected to be able to help define a limit to Federal involvement based on the value of future public benefit from the constructed project.

While this may seem like a fair way to arrive at a public policy decision, it cannot alone solve the problem. There are two problems which remain outstanding. One is that individuals may disagree over what the expected future damage and loss of life for a given earthquake may be, which can have a significant impact on the computed benefits. If the damage in a medium to major earthquake can include collapse of the non-upgraded structure, then the benefits of upgrading are much greater than if collapse is not expected except at a level of shaking in excess of that ever experienced historically.

The second problem is with the upgrade designs themselves. While the benefit/cost analysis may show that a given upgrade scheme may not be cost beneficial, it does not solve the problem of what else to do with a damaged building which has been determined to need at least some form of upgrade. That debate continues.

FEMA is mandated by its regulations to address the issue of cost-effectiveness of the work which the agency funds. This has proved very difficult to do in the contentious climate which has surrounded the issue of the upgrade of infill-wall buildings. When people disagree on the fundamentals of the expected performance of the structural system of the subject buildings, there is not enough common ground on which to reach consensus. Life safety is as holy as a goal as it is vague as a benchmark. All of this only reinforces the need not only to fully understand this historically important structural system from a scientific and academic standpoint, but also to develop building code provisions specifically for infill-wall construction which can be easily and consistently applied in the future.

THE OAKLAND EXPERIENCE DURING LOMA PRIETA - CASE HISTORIES

Sigmund A. Freeman[1]

ABSTRACT

The City of Oakland, California, has a substantial inventory of mid- to high-rise steel frame buildings with unreinforced brick infill forming the exterior walls. Many of these buildings suffered significant damage during the October 17, 1989, Loma Prieta Earthquake. These buildings were built about the 1920s and their designs are similar to buildings built throughout the United States. Data obtained from observations and evaluations of brick infill steel frame buildings are summarized. A generalized case study is given to illustrate damage assessment techniques and methods of evaluation.

INTRODUCTION

These buildings were generally not designed with earthquake forces in mind; wind being the governing factor for lateral forces. Not much is known on how these buildings will perform in a major earthquake. They are difficult to evaluate by conventional analytical procedures because of the uncertainty of how the brick walls interact with the structural steel frames. Reported damage from the Loma Prieta earthquake to brick-infilled steel-framed buildings ranged from very minor to significant. The types of damage included cracked interior partitions and ceilings, spalled ornamental exterior terra-cotta and stone, and cracked and dislodged exterior brick. Although most of the buildings remained open, many were closed because of potentially hazardous conditions. The post-earthquake damage assessments and structural evaluations give us an opportunity to gain knowledge on how these types of buildings work.

BUILDING DESCRIPTION

The type of building discussed here is a steel frame structure with brick infill exterior walls. The buildings generally range in height from 6 to 7 stories for hotels to over 15 stories for office buildings. Floor areas range from 5,000 to 10,000 square feet. Most of the buildings have some shape irregularities such as flat-iron, L- or U-shaped in plan, light wells on the sides, light courts in the center, tall first stories, and open storefronts on one or two sides.

The typical building has a complete three-dimensional steel frame that carries the gravity loads. The beams are generally rolled sections and the columns are generally riveted built-up sections.

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The exterior perimeter beams typically are offset from the columns' center lines so that the web of the beam lines up with the outer flange of the columns. An idealized representation of the brick-infilled steel frame is shown in Fig. 1. The riveted beam-column connections are primarily designed for gravity loads with some nominal moment resistant capacity for lateral forces. The moment resistant capacity is generally provided by two or four rivets attaching the top flange clip angle and the beam seat.

The infill masonry generally consists of three wythes of brick. Two interior wythes are supported by the top flange of the beam. The exterior wythe often includes decorative terra-cotta elements and its method of support varies for different buildings. In some cases a steel ledger plate cantilevers out from the steel beams. In other cases, it is supported by the keying action of header bricks interlocked with the interior wythes. Whereas the brick is generally tightly fitted above and below by the steel beams, the details of the brick interfacing with columns on each side are not usually well known.

The floor framing systems for these buildings may be wood or concrete. When wood is used, the floors generally have two layers of wood sheathing. When concrete is used, some of the structural steel framing may be encased in concrete fireproofing. Interior partitions may be plastered stud walls or hollow clay tile. Ceilings are lath and plaster hung from the floor framing above.

STRUCTURAL EVALUATION

Lateral Force Resisting System

The lateral force resisting system of an existing brick infill steel frame building does not conform to structural systems in current codes (5). If the steel frame alone is considered to be the structural system, it has deficiencies in connection details, strength, drift, and compatibility with the brick. If the brick is considered to act as a shear wall it is deficient because of lack of continuity and because it is unreinforced.

However, if we consider the interaction between the brick infill and the steel frame we can begin to develop a mathematical model that will define the lateral force resisting system. There are still some obstacles to fully understanding how the system works. For example, how does the brick interface with the steel frame? Is there horizontal shear transfer between beam and brick by bond or friction, or is the column confined enough by the brick to be fully engaged? What are the vertical reactions at the beam-column connections? How do the window openings affect the load paths through the brick? What is the difference between a brick pier that encases a steel column and an intermediate pier that is not on a column line? Fig. 2 shows an elevation of a typical exterior wall with some representative load paths.

Until better data is available, it appears reasonable to assume that the wider piers act as compression struts that fail in diagonal tension as shown in Fig. 2. For evaluation purposes, we have taken the area of a horizontal plane through the piers as an effective shear area. It is then assumed that the loads transfer to the steel frame and that the steel columns work axially to resist overturning. In addition to the exterior infill frame system, consideration must also be given to the horizontal diaphragms and to possible participation of interior partitions. If hollow clay tile partitions exist, they can play a major part in resisting lateral forces because of their rigidity. Lath and plaster partitions, to a lesser extent, can also participate in the lateral force resisting system; especially in cases where the diaphragms are flexible (e.g., wood flooring instead of concrete). Although these procedures may appear crude, they provide reasonable correlations between the performance observed for these types of buildings during the Loma Prieta earthquake and the data obtained from ground motion records.

Loma Prieta Earthquake Experience

The results of the studies covered by this paper are primarily based on data from buildings in the downtown area of the City of Oakland.

The strong motion instrument located closest to the downtown area recorded peak ground accelerations of roughly 10% of gravity in a north-south direction and 20% of gravity in an east-west direction. These values were consistent with other recordings in the Bay Area. Response spectra that were developed from these recordings are shown in Fig. 3 in the ADRS format (i.e., spectral acceleration vs. spectral displacement response spectra where the periods are plotted as radial lines)(4). It can be seen that the acceleration and displacement responses are relatively large at the 1.0 to 1.5 second period range for 5% damping (close to the value of the UBC design spectra for S_2 soil normalized to peak ground acceleration of 0.40 g.). However, these peak values drop off substantially for 10% and 20% damping and the displacements reduce significantly beyond the 2.0 second period to a peak value of about 5½ inches.

Soon after the earthquake the City of Oakland adopted an Emergency Order for Abatement of Hazardous Structures Associated with Earthquake Damage. This emergency order requires that a Damage Assessment Report (DAR) be prepared for earthquake damaged buildings. The DAR is to include an estimate of the loss of capacity to resist lateral forces that the building sustained during the earthquake. If the loss of capacity is more than 10%, the order requires a total seismic upgrade.

The capacity of the building is determined in a rational, consistent manner, taking into account relative strengths and rigidities of the various materials. This includes materials not conforming to current codes if it is apparent that their participation was significant in resisting the forces caused by the earthquake. The results of the evaluations of both the pre-earthquake and post-earthquake structure should be consistent with the observed earthquake damage. In order to be rational, the values of the strengths of the elements have to reasonably represent the in-place materials. Sample guidelines were established to aid engineers. For example, a tentative recommendation of 75 psi was suggested for the shear capacity of infilled brick.

Sample Evaluation

As a result of studies of several damaged buildings some general observations can be made. Each of the studied buildings had some shape irregularities of the types described earlier. Upon evaluation of the load paths and the overall response characteristics, a pattern of the sequence of damage could usually be established. In this sample all buildings were about seven stories in height. Periods of vibration were estimated to be about 0.5 to 0.6 seconds in their pre-damaged state.

When buildings were evaluated on the basis of a code type allowable shear stress of 12 psi for brick infill, the resulting pre-earthquake equivalent base shear capacities ranged from about 2% to 4% of the weight of the building. For the steel frames, with their nominal moment resisting capacities, the base shears coefficients generally ranged from 1% to 3%. However, the brick infill was much stiffer than the frames and major loss of brick would have to occur before the very flexible steel frames would participate. If ultimate strengths, such as 75 psi for the brick infill, were used in the evaluation, the base shear equivalent would be about 15% to 25% of the building weight. In its cracked (or damaged) state the periods of vibration were estimated to be about one (1) second. Damping values were assumed to be 5% of critical damping for the uncracked state and 20% for the cracked state. Using this data with the response spectra in Fig. 3, approximations of the building capacities in terms of peak ground accelerations can be made as shown below. The procedure is The Capacity Spectrum Method (1,2,4).

	C_B	T	B	S_A	A_G
Uncracked	.02	0.5	5%	.025	.01g
Uncracked	.04	0.6	5%	.05	.02g
Cracked	0.15	1.0	20%	.20	0.13g
Cracked	0.25	1.0	20%	.30	0.20g

C_B = base shear coefficient; T = fundamental period of vibration; B = percent of critical damping
S_A = spectral acceleration = $C_B/0.8$ (approximate)
A_G = peak ground acceleration (acceleration at zero period of response spectra) obtained from ratio of S_A value above to S_A value in Fig. 3

This indicates that a peak horizontal ground acceleration of 1% or 2% of gravity could result in brick stresses of 12 psi; however, it would take a ground acceleration of 13% to 20% to reach an ultimate cracking strength of 75 psi. On the assumption of 10% peak ground acceleration in the north-south direction and 20% in the east-west direction during the Loma Prieta earthquake, it appears that one would expect minor or no damage in the north-south direction and significant cracking in the east-west direction of the building. This agrees with the observations.

REPAIR AND UPGRADE

If a building loses more than 10% of its pre-earthquake capacity, Oakland requires that the entire structure be made to substantially comply with the structural requirements of the current code (5). However, some allowances are made because of the difficulties of bringing an existing building up to current standards. This includes the application of the equivalent of the 1991 UCBC for unreinforced masonry bearing wall buildings (6) and the provision that the total design base shear coefficient need not exceed 0.133. The repair ordinance also states that the building official may approve an alternative procedure if it can be demonstrated by rational analysis that the modified structure provides the intended level of safety. This last provision allows for innovative solutions and the use of performance criteria or limit state procedures for seismic upgrading of existing structures.

CONCLUDING REMARKS

There is still a lot to learn on how steel frames with brick infill perform when subjected to major earthquakes. If we take the opportunity to learn from data obtained from past earthquakes, we can improve our design procedures and criteria for upgrading existing buildings. This paper is a modified version of reference 3.

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2. Freeman, S.A., "Code Designed Steel Frame Performance Characteristics", *Dynamics of Structures Proceedings, Structures Congress '87*, American Society of Civil Engineers, Orlando, Florida, August 1987.
3. Freeman, S.A., "Behavior of Steel Frame Buildings with Infill Brick", *Proceedings of the 6th Canadian Conference on Earthquake Engineering*, Toronto, Ontario, Canada, June 12-14, 1991.
4. Mahaney, J.A., T.F. Paret, B.E. Kehoe, S.A. Freeman, "The Capacity Spectrum Method for Evaluating Structural Response During the Loma Prieta Earthquake", *1993 National Earthquake Conference*, Memphis, TN, May 3-5, 1993.
5. UBC 1988. Uniform Building Code, International Conference of Building Officials, Whittier, California.
6. UCBC 1991. Uniform Code on Building Conservation, International Conference of Building Officials, Whittier, California.

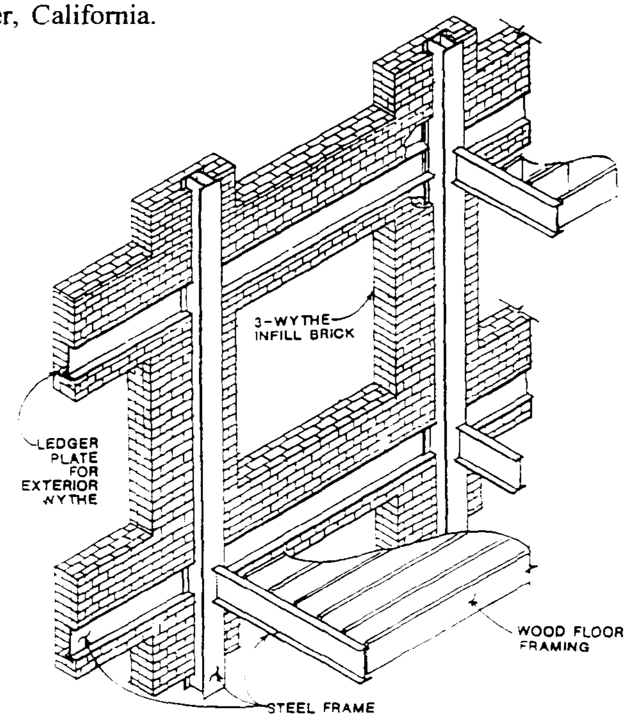


Figure 1. Schematic representation of brick infilled steel frame

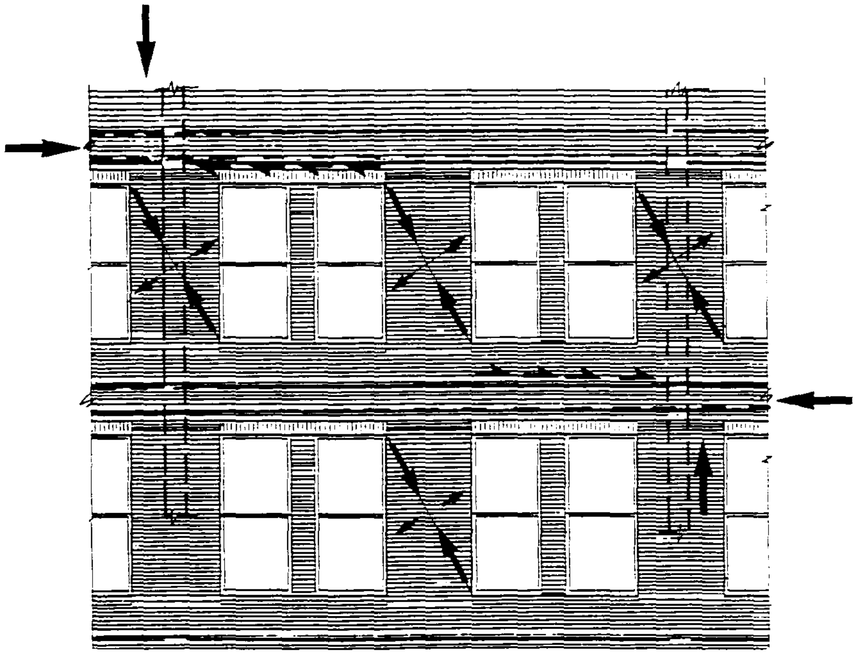


Figure 2. Partial wall elevation with representative load paths

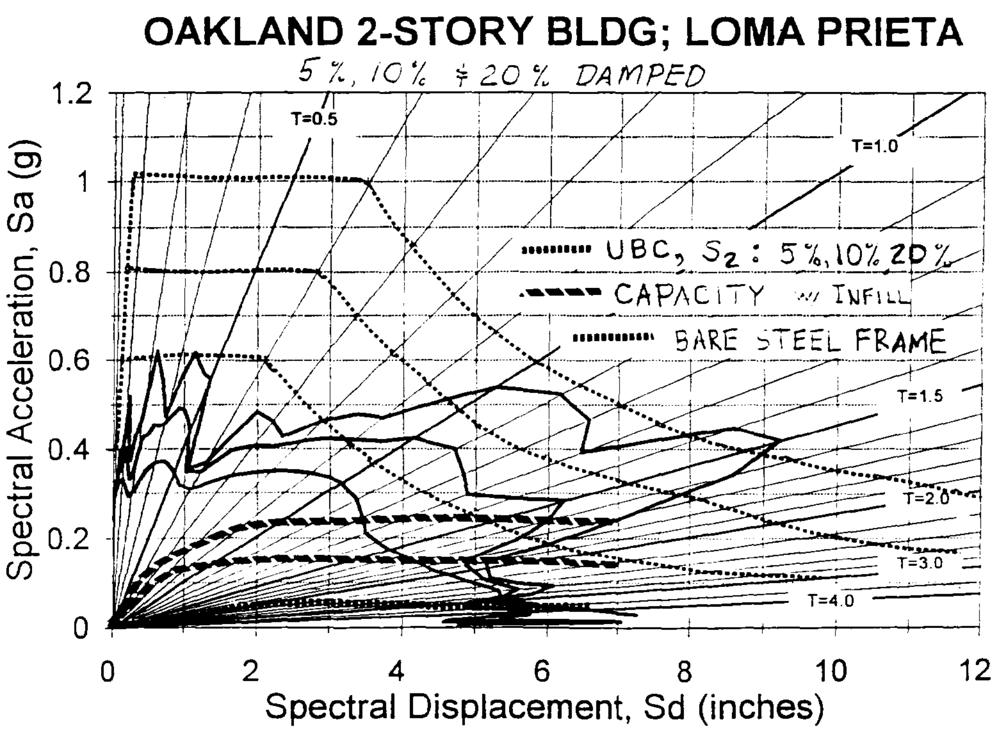


Figure 3. Capacity spectrum method using Oakland, October 17, 1989 response spectra

Structural Framing Systems: 1890-1920, Implications for Seismic Retrofit

Melvyn Green[1]

Abstract

With the advent of wrought iron and steel taller structures could be constructed. Along with this came a need for fireproof floor systems. Numerous floor framing systems were developed to meet this need. This paper reviews the preliminary findings relative to the type, advantages and disadvantages of these systems. A preliminary discussion of the seismic rehabilitation issues needing research are reviewed.

Intent

In support of the American Society of Civil Engineer's role in the FEMA/BSSC Seismic Rehabilitation Guidelines Project, we have been assembling information on old building systems used on major structures. The intent of this is to provide the ATC-33 development teams, as well as the engineering community, information on the type of building constructions that may be encountered so they may consider the implications for a building's seismic rehabilitation. This preliminary effort includes building systems used during the period of 1890 through approximately World War II.

With increasing urbanization in the nineteenth century, land values in cities began to justify the construction of multi-story buildings. The principal available building material for such buildings was load-bearing masonry, usually brick, with heavy timber structure for floors and roof. The practical limit of this construction type is about nine stories. With the invention of the passenger elevator in 1853, and of widespread electrical distribution in the 1880's, taller, and thus more profitable, buildings were theoretically habitable. But the problem of constructability remained. Wall thicknesses and weights became excessive with increased height, and floor construction was a problem. There were no methods for long spans. There were also no methods to obtain a reasonable span with fireproof or non combustible construction. Engineers and manufacturers struggled to solve this problem. New possibilities were opened up as a result of the availability of rolled wrought iron beams in the 1870's, with steel following in the late 1880's (Wrought iron was available through about 1910). As the nineteenth century drew to a close, the need for taller buildings combined with a rapidly evolving technology to produce an unprecedented burst of inventiveness. The result is a combination of traditional systems such as a tile arch floor and patented systems using unique reinforcing or decking. As we worked with the floor systems we also reviewed cast iron, wrought iron and steel columns. There were parallel activities to develop more economical and stronger columns.

Some of these systems have evolved into today's construction methods. Others have fallen out of favor, usually for economic reasons but some because they performed in an unsatisfactory manner. There is no way to estimate the number of buildings constructed with the systems shown. Based on the catalogs and publications, they were apparently widely used. Most of these buildings will be in the eastern half of the country. However we are aware of such buildings in the Pacific Northwest.

[1]Structural Engineer, Melvyn Green and Associates, Inc. Torrance, CA 90503

The potential of misunderstanding or incorrect assumptions is increased in the case of seismic rehabilitation for both horizontal and vertical elements. An engineer, without understanding the implications, could use a cast iron column as a bending element. Cast iron is a brittle material with no ductility.

A parallel activity to this project is a summary of early building code loads and allowable design loads. However this report is not ready for review at this time.

Approach and Work Completed

The work involved reviewing a number of publications of the era, Sweet's and other construction information services and Architectural Graphic Standards.

Information was obtained about the construction details of the various systems. Additional information was reviewed to determine the advantages and disadvantages of the systems. This was then organized in a manner that briefly summarizes the available information and notes the source of the data. The reporting format is shown in Figures .

Observations

There are a number of interesting observations and concerns that engineers should be aware of in the systems reviewed.

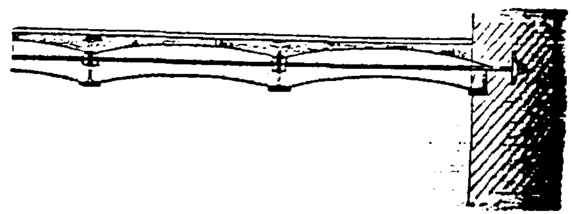
- The system in place may be quite different than that perceived from field observations. The research identified a number of hollow floor systems, with hollow clay or gypsum units. An inexperienced observer could consider this as solid system with a greater load capacity than the actual system.
- The weight of the floor may be quite different than assumed.
- Lateral load behavior of such floors may be very different than assumed. For example, does a masonry arch floor behave as a solid diaphragm or should one consider the bracing strut approach in determining strength?
- Masonry floors are compression members. Are they subject to collapse potential due to loss of compression under lateral loads?
- There is a concern over the use of cinder concrete aggregates. The New York schools have had a problem with the deterioration of floors constructed with cinder aggregates.

Seismic Rehabilitation Implications and Research Needs

The following preliminary research needs were identified:

- Determine if masonry floors act as struts, similar to the recent work on walls. Or do they behave as a solid diaphragm under lateral forces?
- Stability of masonry floors under lateral forces. Are the masonry arch and infill floors stable under large seismic loads?
- Investigative techniques are desirable to determine the composition of floors.

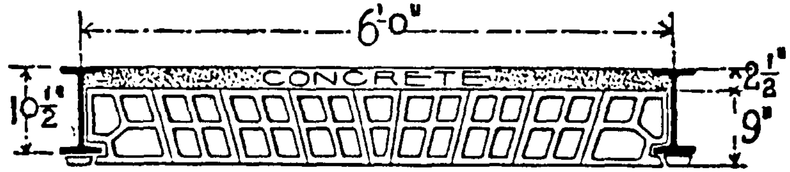
Detail 1



Description	Advantages	Disadvantages	Implications for Seismic Safety
Brick Floor Arch - Arches are sprung between steel beams and resting on the bottom flanges. Joints are filled with grout. The bricks of one line should break joints with those of the next adjoining. If there is more than one row, the joints or one row should also break joints with those of the row above or below. Arches need not be over 4 in. thick for spans between 6 and 8 ft, provided the haunches are filled with a good cement and gravel concrete, put in rather wet. The rise of the arch should be about 1/8th the span, or 1 1/2 in. to the foot. The most desirable span is between 4 and 6 ft. The building laws of many cities provide that when the spans exceed 5 ft the arches must be increased in thickness, generally to 8 in. Tie rods should always be provided.	Strongest type of arch for its span	Weight Lower flange of beam exposed Appearance considered unattractive in its time Expensive	

Kidder-Parker, p. 950

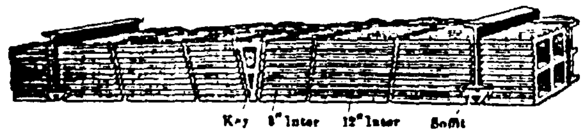
Detail 10



Description	Advantages	Disadvantages	Implications for Seismic Safety
Side Construction Flat Tile Arch - True arch, wherein all parts are in compression, constructed of shaped tiles. In the side construction arch, the voids run parallel to the supporting beams. This was the earliest flat tile arch developed.	Flat shape means that false ceilings and extensive filling of the upper side with concrete are not necessary. Side construction gives better fireproofing to steel.	Relatively weak. Side construction skewbacks (pieces adjacent to beams) could not be cut and had to be ordered to fit steel.	

Freitag, p. 91

Detail 11



Description

End Construction Flat Tile Arch - In this construction the sides an voids of the individual tile run at right angles to the beams, so that the pressure on the webs is endwise of the tile. The individual tiles in end construction are commonly made rectangular in shape, advancing by 1 in from 6 to 15 inches in depth. The total depth of flat arch construction should not be less than 6 in. and never less than 1 1/2 in. per foot of span. The length and width also of the blocks may be varied but the standard size is 12 in. for both dimensions. The number or partitions or webs in the blocks varies with the size of the tile and also with the strength desired.

Advantages

Stronger than side construction arch

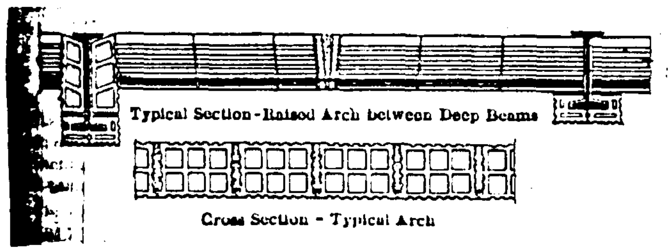
Disadvantages

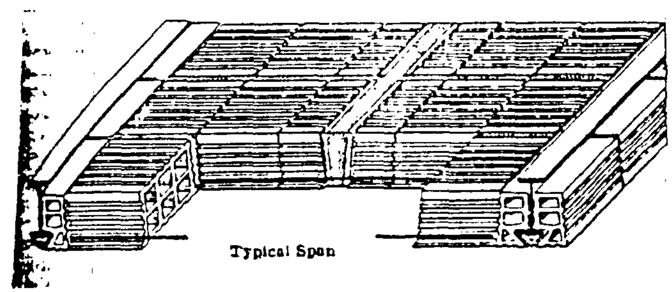
Harder to construct properly than side construction arch

Implications for Seismic Safety

Kidder-Parker, p. 958

Detail 13





Description	Advantages	Disadvantages	Implications for Seismic Safety
Natco "New York" Reinforced Tile Floor Arch - Combination Flat Tile Arch reinforced with 1/4 or 3/8 in round rods, placed in the mortar ribs, one rod to each rib, the size of the steel depending upon the span of the arch and the load to be carried. Finally, the tile being set, the ribs and keys are grouted with a mortar of one part Portland cement to three parts sand. The skewbacks should be carefully bedded against the beams with mortar and clip tile used with raised arches should also be set with mortar joints. Light cinder concrete was used to level the floor up to the tops of the beams. The arch key was made of mortar.	This detail was particularly used with wide spans in shallow beams.		

Kidder-Parker, p. 961

IMPACT OF INFILLED MASONRY WALLS ON THE RESPONSE OF BUILDINGS IN MODERATE SEISMIC ZONES

Samy A. Adham [1]

ABSTRACT

Observations of building damage during earthquakes in highly seismic areas indicates that infilled walls may have a negative impact on the integrity of these buildings. Infilled walls can result in stiffening these buildings, thus attracting more earthquake forces. Infilled walls will resist shear forces, however, under numerous loading cycles associated with large duration earthquakes may fail, resulting in a sudden transfer of shear forces to the columns. Such sudden transfer generally results in brittle failure of these columns and sudden collapse of the infilled building. In addition, such infilled walls, if not properly anchored to the main structural system, may fail in the out-of-plane mode, thus creating a considerable hazard to life safety and building contents. Little or no attention has been given to impact of infilled walls on the response of buildings in zones of moderate seismicity. Infilled masonry walls can be advantageous in these zones where earthquakes would generally have a shorter duration with a small number of loading cycles. The presence of the infilled masonry walls would enhance the shear resistance of a building especially where the integrity of the main carrying structural frame is questionable. It will also increase the stiffness of a flexible frame building founded on soft soils.

The large inventory of infilled masonry buildings in moderately seismic zones of the United States indicates that such types of structural systems should be evaluated using a philosophy that would allow for taking full advantage of the infilled walls in resisting earthquakes. The recent magnitude 5.9 earthquake in Egypt provided a significant lesson to be learned on the response of infilled walls in moderately seismic zones and it's impact on the survival of these buildings.

INTRODUCTION

On October 12, 1992, an earthquake of M_b 5.9 (M_s 5.4) that resulted in excess of 500 deaths, thousands of injuries, collapse of few structures and damage to many structures in the Cairo metropolitan area (population 12,000,000), Figure 1. The Earthquake Engineering Research Institute dispatched a team of earthquake engineers and scientists to investigate the effects of this event (Youssef, et al 1992).

Seismological and Geological Observations

[1] Senior Associate, AGBABIAN ASSOCIATES, Pasadena, California

In as much as this earthquake came as a surprise to the world community, Egypt and it's surrounding countries have a history of earthquake events in Egyptian writings that extend back to 2200 B.C. A great number of these earthquakes are unknown to engineers as their descriptions are scattered in various seismological, historical and geographical manuscripts. Historically, Egypt has experienced eleven earthquakes in the magnitude range five to six between the years 796 and 1992.

The earthquake had a focal depth of 24 Km with no observed surface faulting. However, tension cracks were observed in semi-consolidated alluvium that might be related to a small amount of near surface ground failure. Settlement of up to 1.5 m was reported about 25 Kilometers south of Cairo.

The highest intensity observed was Modified Mercalli Intensity VIII in the village of Manshiat Fadil and represents the epicentral intensity of the earthquake (Figure 2). Intensity VIII corresponds to the fall of ordinary masonry walls and complete collapse of adobe structures. This type of damage was also associated with liquefaction near this village. Intensity VII was common in many villages of the Nile Valley and generally corresponds to cracks in ordinary masonry walls and the fall of adobe walls and masonry parapets that weren't reinforced.

Observations of Earthquake Damage

From the nature of the observed damage, significant amplification of earthquake motion probably occurred in the alluvial deposits of the Nile Valley around Cairo and vicinity. Such amplified motion with long period characteristics at the deep alluvium sites near the banks of the Nile would generally be expected to have a detrimental effect on high-rise buildings built in this area. However, little damage occurred to these buildings (Figures 3 and 4).

Types of Infilled Masonry Wall Buildings and Their Structural Response

1. Reinforced concrete skeleton with unreinforced masonry infill walls (low-rise, 2-5 stories). In general, reinforced concrete framed buildings with unreinforced masonry infill performed well in this earthquake. Particularly the masonry framed and tight mortar-jointed buildings with boundary reinforced concrete columns, slab below and beam above, with relatively short height, large redundancy (exterior walls and interior partitions), and very low shear stresses. However, when quality of concrete or reinforcing were not adequate, masonry infilled walls provided a redundant path to carry both the vertical load and shear forces, Figure 5.

2. Reinforced concrete framed buildings mid-rise (5-20 stories) and high rise (20-45 stories) with unreinforced masonry infilled walls. The damage observed to these types of buildings appears to be mainly from random/special circumstances encompassing some of the following conditions: a. Highly irregular stiffness distribution in the structural system. High-rise reinforced concrete buildings with masonry infill above the second or third level, with ground floor and mezzanines of higher stories and no masonry (variable, slender, column length for commercial and retail use). Many of these lower level modifications are done as afterthoughts without any evaluation of seismic force and drift demands, and consequently without any proper shear or confinement reinforcement. b. Reinforced concrete systems with deep spandrel beams and short columns without appropriate shear

reinforcement. c. Excessive overloading coupled with soft and weak stories at lower levels due to illegal (not properly engineered) additions. (The Heliopolis collapsed, 14 story structure that was designed to 8 stories). d. Inadequate reinforced concrete detailing, (column ties, beam-column joints, location and length of lap splices, development length, etc.) e. Inappropriate locations and poor workmanship at column cold joints just below beam soffits that could cause problems if subjected to strong ground motions. f. Some buildings exhibit poor construction materials [aggregate, poor concrete mix, some defective cement, underformed reinforcing bars and ties] overall construction and workmanship. Poorly enforced code [public schools and affordable housing], coupled with poor maintenance and defective plumbing and sewage systems.

Some of these buildings suffered crushing of reinforced concrete corner columns and deterioration of some interior columns (Figure 6 through 8). However, the presence of masonry infilled walls in these buildings provided an alternative system that beside resisting shear forces, also carried part of the vertical load. These infilled masonry walls also provided stiffness to these high-rise buildings, thus avoiding resonance between tall buildings, soft sites, and the long period earthquake.

CONCLUSIONS

Infilled masonry walls should be considered as enhancement to buildings in moderately seismic zones. Their presence will increase the shear capacity of the building. It will also improve the stiffness of buildings, particularly those founded on soft soils. However, care should be exercised in properly repairing these buildings if they suffer any cracks during a moderate earthquake. Such a repair would restore the original shear capacity of these buildings and avoid brittle failure of these buildings in future earthquakes.

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Youseff, Nabih, Adham, Samy, Celebi, Mehmet, and Josephine Malilay, "Preliminary Report of Earthquake Reconnaissance, October 12 Dashour Egypt Earthquake," EERI, December 1992.

U.S. Geological Survey, "Reconnaisance Report on the 12 October 1992 Dashour, Egypt Earthquake," Administrative Report to U.S. OFDA, Agency for International Development, December 14, 1992.

Figure 1: Collapse of Buildings in Cairo

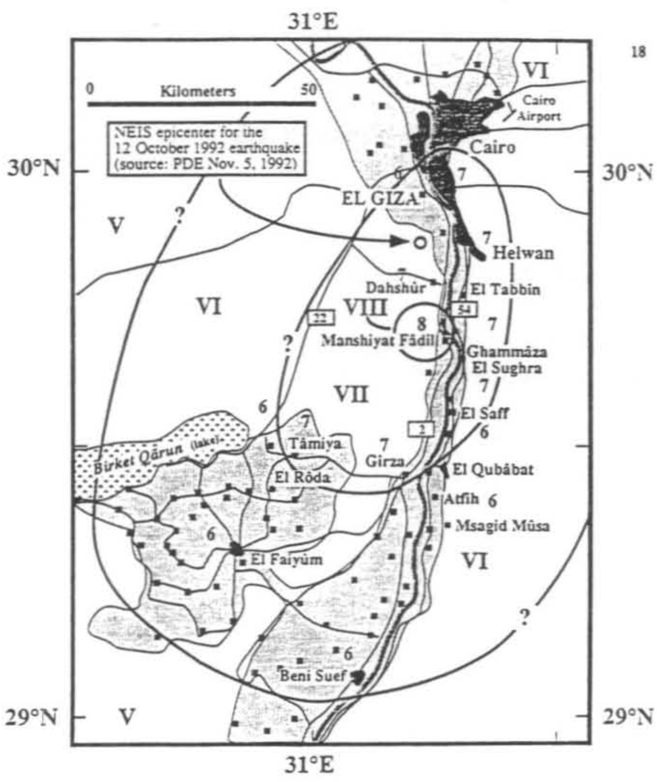

Figure 2: Modified Mercalli Intensity (MMI) U.S.G.S. December 14, 1992

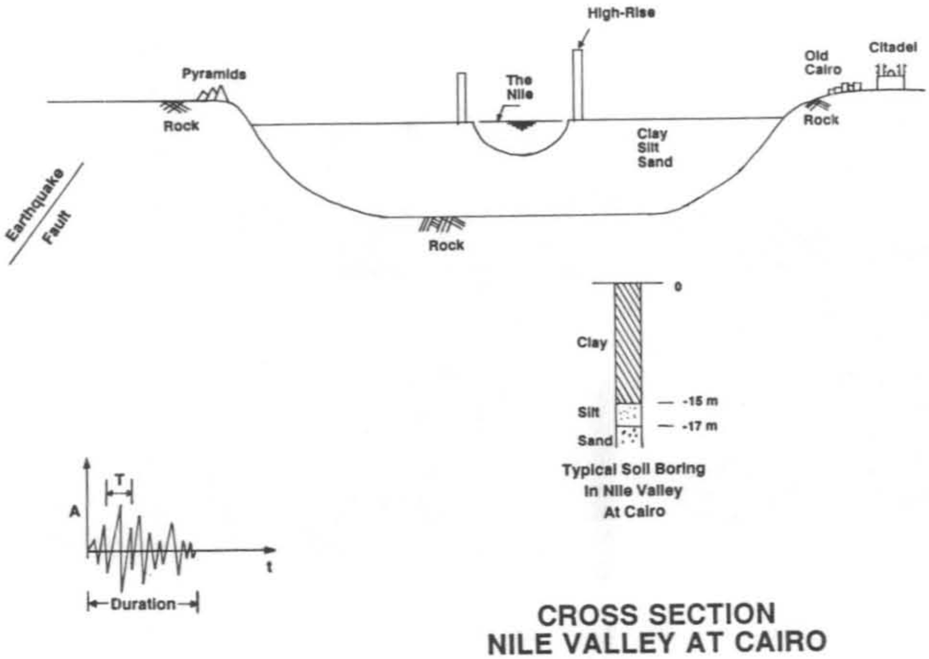

Figure 3: Cross Section Nile Valley at Cairo

Figure 4: Reinforced Concrete High-Rise Building with Masonry Infilled Walls had Little or No Damage

Figure 5: Infilled Masonry Walls Prevented Severe Damage to the Two Story School Near the Epicenter

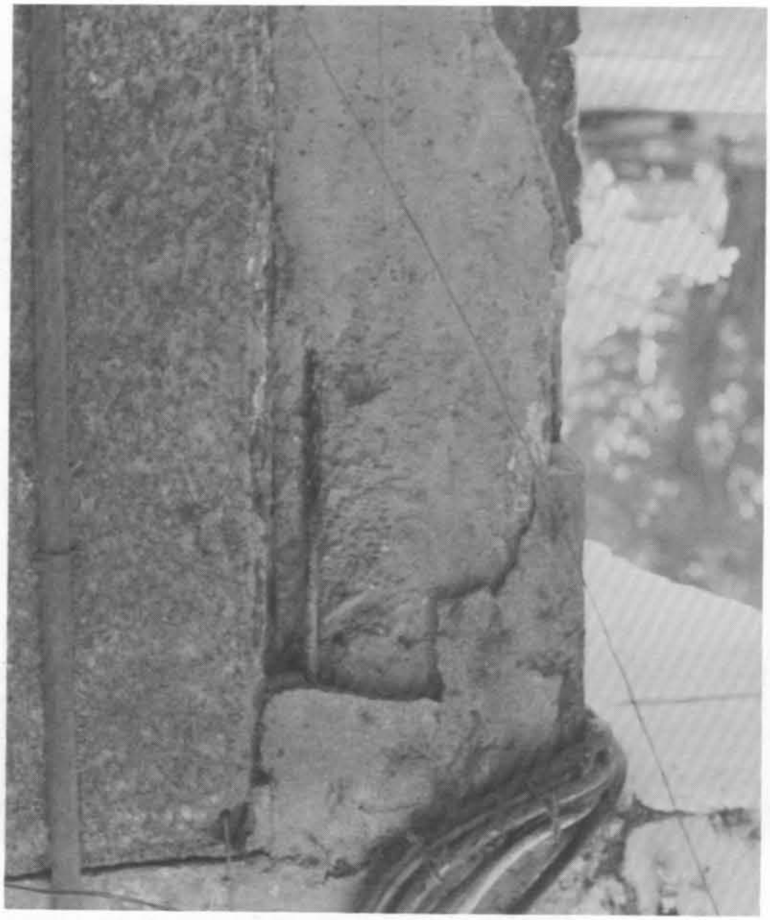

Figure 6: Crushing of Reinforced Concrete Corner Columns of a Seven Story Reinforced Concrete Frame Building with Infilled Walls

Figure 7: Twenty-two Stories Reinforced Concrete Building with Infilled Masonry Walls Suffered Crushing of Corner Columns

Figure 8: Crushing of Reinforced Concrete Corner Columns

Section III

Conference Information

Final Program

List of Participants

Attendance at Discussion Groups

Final Program

NCEER Workshop on Seismic Response of Masonry Infills

February 4 and 5, 1994
Holiday Inn Golden Gateway, 1500 Van Ness Avenue, San Francisco, CA 94109

Sponsored by the National Center for Earthquake Engineering Research, The Masonry Society and the Earthquake Engineering Research Institute

Friday, February 4

10:00 am-10:15 am Welcome and Introductions

10:15 am-1:15 pm Synopses of Research Projects (12 @ 15 minutes each)

Laboratory Studies

"Seismic Retrofit of Flat-Slab Buildings with Masonry Infills,"
A. Durrani*, Rice University, Houston, Texas

"Out-of-Plane Strength Evaluation of Infill Panels,"
R. Angel* and D.P. Abrams, University of Illinois at Urbana-Champaign

"Out-of-Plane Strength of Masonry Infills Retrofitted with Fiber Composites,"
M. Ehsani* and H. Saadatmanesh, University of Arizona

"Physical and Analytical Modeling of Brick Infilled Steel Frames,"
J.Mander*, L.E. Aycardi and D.-K. Kim, State University of New York at Buffalo

"Dynamic Testing of an Infilled Frame Structure,"
R.Flanagan*, Martin Marietta Energy Systems, Oak Ridge, Tennessee

"Performance of Masonry Infilled R/C Frames under In-Plane Lateral Loads: Experiments,"
M. Schuller, Atkinson-Noland & Associates; A.B. Mehrabi, University of Colorado; J.L. Noland*, Atkinson-Noland & Associates; and P.B. Shing, University of Colorado

Field Studies

"Out-of-Plane Response of Unreinforced Masonry Infill Frame Panels,"
J.A. Hill*, James A. Hill & Associates, Signal Hill, California

*presenting author

Friday, February 4
10:15 am-1:15 pm (continued)

Analytical Studies

"The Influence of Modeling Assumptions on the Predicted Behavior of URM Masonry Infill Structures,"
N.Youssef and Owen Hata*, Nabih Youssef & Associates, Los Angeles

"Performance of Masonry Infilled R/C Frames under In-Plane Lateral Loads: Analytical Modeling,"
A.B. Mehrabi and P.B.. Shing*, University of Colorado at Boulder

"Evaluation and Modeling of Infilled Frames,"
P. Gergely*, R.N. White and K.M. Mosalam, Cornell University, Ithaca, New York

"Simulation of the Recorded Response of Unreinforced Infill Buildings,"
J.Kariotis*, Kariotis & Associates, South Pasadena, California; T.J. Guh, Delon Hampton & Associates, Los Angeles; G.C. Hart, Hart Consultant Group, Santa Monica; J.A. Hill, James A. Hill & Associates, Signal Hill; and N. Youssef, Nabih Youssef & Associates, Los Angeles.

"Numerical Modeling of Clay Tile Infills,"
R.D. Flangan, M.A. Tenbus, Martin Marietta Energy Systems, Oak Ridge, Tennessee, and R.M. Bennett*, University of Tennessee, Knoxville

1:15 pm-2:30 pm **Lunch**

2:30 pm-4:30 pm **Discussion Groups I**

Discussion Group IA:
Modeling Global Response of Building Systems with Masonry Infills
Discussion Group IB:
Modeling of Infill Panel Behavior: Normal and Transverse Loadings

4:30 pm-5:00 pm **Break**

5:00 pm-6:15 pm **Design Criteria and Case Studies (5 @ 15 minutes each)**

"Public Policy vs. Seismic Design: Cost and Performance Criteria for Seismic Rehabilitation of URM Infill-Frame Buildings,"
R. Langenbach*, FEMA, Washington D.C.

"The Oakland Experience During Loma Prieta - Case Histories,"
S. Freeman*, Wiss, Janney and Elstner, Emeryville, California

"Structural Framing Systems: 1890-1920, Implications for Seismic Retrofit,"
M. Green*, Melvyn Green & Associates, Torrance, California

Friday, February 4
5:00 pm-6:15 pm (continued)

"Impact of Infilled Masonry Walls on Seismic Response of Buildings in Moderately Seismic Zones,"
S. Adham*, Agbabian Associates, Pasadena, California

"Masonry Infill Damage in the Northridge Earthquake,"
J.A. Hill, James A. Hill & Associates, Signal Hill, California
(an impromtu presentation with no paper)

6:20 pm **Adjourn for Day**

Saturday, February 5

8:30 am-10:30 am **Discussion Groups II**

Discussion Group IIA:
Criteria for Rehabilitation of Infills and Infill Systems
Discussion Group IIB:
Criteria for Evaluation of Infills and Infill Systems

10:30 am-11:00 am **Break**

11:00 am-12:00 pm **Summaries from Discussion Groups**

12:00 pm-1:00 pm **Formulation of Workshop Resolutions**

1:00 pm **Adjourn**

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ATTENDANCE AT DISCUSSION GROUPS

Discussion Group IA:

Modeling Global Response of Building Systems with Masonry Infills

Roger Flanagan, *moderator,* Jim Noland, *recorder,* Dan Abrams, Samy Adham, John Clappison, Ahmad Durrani, Sig Freeman, Tom Hale, Afshar Jalalian, John Kariotis, Randolph Langenbach, Dawn Lehman, Frank McClure, John Meyer, Robert Preece, Dan Shapiro, Steve Short, Chris Tokas, James Wong.

Discussion Group IB:

Modeling of Infill Panel Behavior: Normal and Transverse Loadings

John Mander, *moderator*, Benson Shing, *recorder*, Dan Abrams, Jim Amrhein, Richard Angel, Richard Bennett, Kent David, Mohammad Ehsani, Ken Fricke, Mel Green, Peter Gergely, Jim Hill, Onder Kustu, Gayle Johnson, Ted Pruess, Steve Sweeney, Sven Thomasen, Joe Uzarski, Ted Zsutty.

Discussion Group IIA:

Criteria for Rehabilitation of Infills and Infill Systems

Dan Shapiro, *moderator*, Randolph Langenbach, *recorder,* Dan Abrams, Samy Adham, Richard Angel, Mohammad Ehsani, Sig Freeman, Ken Fricke, Peter Gergely, Subhash Goel, Mel Green, Afshar Jalalian, Gayle Johnson, John Kariotis, Dawn Lehman, Frank McClure, Ted Pruess, Steve Sweeney, Sven Thomasen

Discussion Group IIB:

Criteria for Evaluation of Infills and Infill Systems

Ahmad Durrani, *moderator,* Dick Bennett, *recorder,* Dan Abrams, Kent David, Roger Flanagan, Jim Hill, John Mander, John Meyer, Jim Noland, Bob Preece, Benson Shing, Steve Short, Joe Uzarski

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