11 WCEE

Copyright © 1996 Elsevier Science Ltd
Paper No. 501. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

HYSTERETIC BEHAVIOR OF UNREINFORCED MASONRY PIERS STRENGTHENED WITH STEEL ELEMENTS

DURGESH C. RAI

Department of Civil and Environmental Engineering, The University of Michigan,
Ann Arbor, MI 48109-2125, USA

ABSTRACT

This paper is concerned with the seismic strengthening of URM walls in which the in-plane lateral resistance is significantly weakened by openings for doors and windows. The system of wall piers and spandrels thus created by perforations largely controls the seismic behavior of the wall. Steel structural elements have been used to strengthen perforated walls. For the "rocking-critical" masonry wall piers, the overall hysteretic behavior can be significantly improved by installing a steel framing system consisting of vertical and horizontal elements around the wall — without any braces. Vertical elements provide the necessary hold-down forces to stabilize the rocking piers. The stabilized piers "rocked" through a number of cycles of large displacements (up to 2.5%)without crumbling or shattering, displaying a ductile response. The degradation of their hysteretic response was observed at story drifts in the vicinity of 1% to 1.5%, primarily due to crushing of the spandrel portion of the wall. The strengthened system has excellent strength, stiffness and ductility, despite the brittleness of the masonry. Considerable load sharing was observed at almost all load stages between the existing masonry and the added steel elements. The controlled amount of masonry cracking added to the damping of the system without serious risk to the structure's vertical load carrying capacity. FE analyses predicted the envelope response of the test specimens very accurately. A simple mechanics based model was developed to predict the load-deflection behavior of a stabilized rocking pier. Because of reasonable agreement between the experimental data and the model, this model can be used to design the strengthening system in a more rational way.

KEYWORDS

Unreinforced masonry; piers; seismic strengthening; rocking-critical.

INTRODUCTION

Unreinforced masonry (URM) buildings resist lateral seismic forces through in-plane shear capacity of their peripheral walls. This ability to resist lateral forces is significantly weakened by large openings for doors and windows. The system of wall piers and spandrels thus created by perforations largely controls the seismic behavior of the wall. Steel structural elements have been used to strengthen perforated walls, but are designed without much regard for ductility or beneficial effects of masonry-bracing interaction. As a result, such schemes not only tend to be uneconomical but also their performance during severe seismic inputs is questionable.

e masonry wall piers are usually slender enough to qualify as "rocking-critical," that is, they develop horizontal cracks at the top and bottom at the outset of the loading and then continue to rock about their bases for further loading. For these of rocking piers, the limiting shear resistance is determined by the moment stability of disturbing in-plane inertial forces and restoring gravity (overburden) forces acting on the pier and can be given by:

$$V_R \approx P(1-2k)\,(D/H) \tag{1}$$

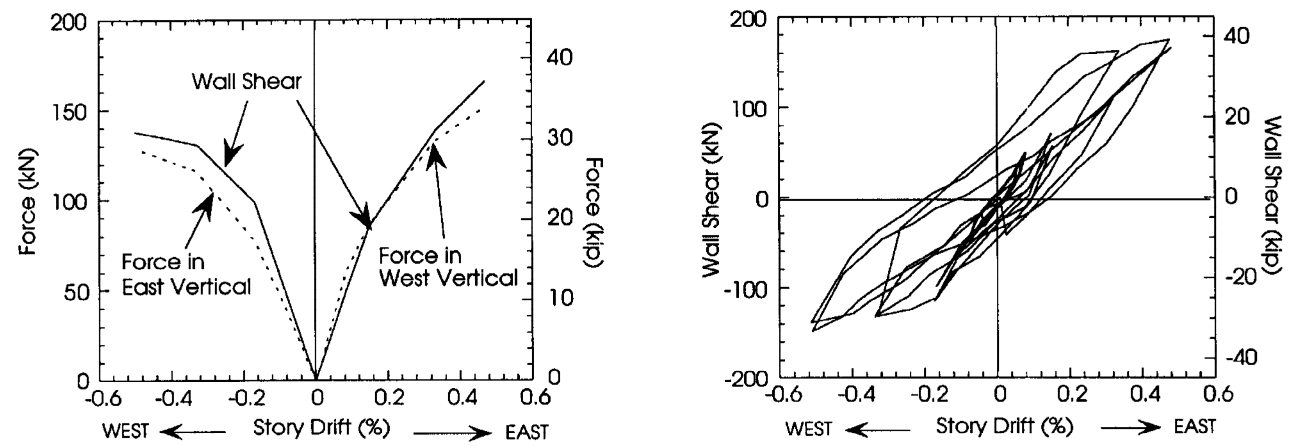
where kD is the compression contact length at ultimate, V_R is the rocking shear capacity of a pier of height H and width D in the wall plane. Clearly, very large displacements equal to $(1\text{-}2k)D$ will be required to cause instability of the rocking piers by overturning. The size of the compression block diminishes at large displacements and a value of 0.9 for the factor $(1\text{-}2k)$ is shown to match experimental results (ABK 1985).

The overall hysteretic behavior of these rocking-critical masonry piers can be significantly improved by installing a steel framing system consisting of vertical and horizontal elements around the wall, without any braces. Vertical elements provide the necessary hold-down forces, thus, stabilizing rocking piers under compression which in turn enhances their shear resistance. The main objectives of the paper are to describe:

i. experimental and FE investigation of hysteretic behavior of URM piers "stabilized" by steel elements;
ii. development of an analytical method to describe their load-deformation characteristics; and
iii. formulation of a rational procedure to design such a system in order to ensure improved seismic performance.

EXPERIMENTAL INVESTIGATION

In the initial phase of the study, a half-scale model of a wall with openings was tested under cyclic loading. The strengthening steel frame consisted of chevron braces and vertical elements at the two ends of the wall pin-connected at both ends to a shear-transferring horizontal element adequately anchored to the spandrel portion of the wall. The masonry was a four-pier wall system of old bricks and type N mortar to simulate the masonry with in-place "push test" shear strength values representative of older buildings. The piers with an aspect ratio of 1.9 are expected to be rocking-critical as indicated by the UCBC General Procedure (1991). Details of this investigation are described elsewhere (Rai *et al.*, 1994, Rai, 1996). The test clearly demonstrated that the shear resistance of rocking-critical piers was dramatically increased (about sixteen times) due to hold-down forces provided by the verticals. As a result, verticals carried large axial forces as shown in Fig.1(a) where these forces are compared with the shear resisted by the wall. The axial forces in verticals increased the overburden P experienced by the piers, thus causing a proportional increase in shear capacity as suggested by (1). Overall hysteretic behavior of the wall was greatly improved as seen in Fig. 1(b). However, no clear evidence of interaction between the wall and the braces was observed, i.e, they appeared to act independent of each other.



(a) Forces in steel verticals

(b) Hysteretic behavior

Fig. 1. Comparison of wall shear with axial forces in the steel verticals and hysteretic behavior of wall piers.

Hysteretic Behavior of Stabilized Rocking Piers

It was evident that the behavior of rocking piers under the action of lateral forces is largely influenced by the two force quantities: shear force and the effective overburden experienced by individual piers. Further tests were conducted on individual stabilized piers to determine the relationship between these two force quantities, their variation with the imposed cyclic displacements and with the arrangement of piers (exterior vs. interior piers) and finally, how the steel verticals responded to rocking piers.

Test Specimens. Test specimens were composed of an URM wall pier in a steel frame consisting of only vertical elements and shear transferring horizontal elements. In one specimen the pier was centrally located in the steel frame (interior pier specimen) as shown in Fig. 2.and in the other, it was located asymmetrically next to a vertical (exterior pier specimen). The dimensions of the pier were approximately 0.53 m (21 in.) wide and 1 m (39 in.) high. The pier was set on five courses of sill masonry and was topped by four courses of spandrel masonry. Reclaimed bricks from 60-80 year old building were used to create the pier masonry, in the running bond with a header course located every fourth course, using a type N mortar mix. The average reference material properties of the masonry are summarized in Table 1.

Table 1: Average reference properties of masonry materials used in the study

1	5-Brick Standard Prism Compression Test		
	Peak Stress f'_m MPa (psi)	Modulus E_m, MPa (ksi)	Strain at Peak Stress ε'_m
	Avg. = 7.31 (1 060), COV = 20%	Avg. = 950 (138), COV =26%	Avg. = 0.0107, COV = 29%
2	*Compressive strength of Mortar (1:1:6):* Avg. = 9.86 MPa (1 430 psi), COV = 19%		
3	*Compressive Strength of Bricks*: Avg. = 17.0 MPa (2 460 psi), COV = 11%		
4	*Push-Over Test Value:* Avg. = 0.76 MPa (110 psi), COV=24%		

Test Set-up, Instrumentation and Loading History. The wall pier was mounted inside a four-hinged loading frame as shown in Fig. 2. A simple array of instruments consisting of linear potentiometers, LVDTs, strain gages and universal load cell was employed to measure the force and deformation quantities. The load cell situated in the loading arm measured the total horizontal shear resisted by the specimen pier. Linear potentiometers and LVDTs sensed the displacements at the top of the masonry pier, shear deformation of the pier when arranged in a diagonal pattern, and the rotation of the pier. The strain gages mounted on the verticals measured surface strains used to obtain axial strains, thus leading to the estimate of the axial forces. The specimens were subjected to the same loading history: a simple multi-step loading history of symmetric cycles of increasing displacements in predetermined steps. The specimens were loaded pseudo-statically through two cycles of reverse loadings performed at story drifts of 0.05, 0.15, 0.30, 0.45, 0.75, 1.15, 1.75, 2.25, 3% etc.

Overall Behavior under Cyclic Loading. The first flexural cracking at the top and bottom of the pier was observed at about 0.20% story drift. The piers then continued to rock until severe crushing of the toe and sill masonry at 2.25% story drift prevented any further loading. The observed base shear-displacement hysteretic response for the interior pier is shown in Fig. 3(a). The specimen resisted shear force increasingly until a maximum of 63.9 kN (14.4 kip) at a story drift of 1.5%. However, there was a gradual decrease in the stiffness, especially beyond excursions greater than 1% of story drift. This softening was due primarily to the toe crushing and damage done to the sill masonry under high compressive stresses. Substantial cracking in the sill masonry was observed during excursions of 2.25% of story drift, when shear resistance dropped to approx. 60% of the peak shear strength.

Similar hysteretic behavior was observed for the exterior pier specimen, as shown in Fig. 3(b). However, the average shear resistance was noticeably different for each direction of loading. Two different sets of flexural cracks developed — one set for each direction of loading — at different elevations in the pier. The pier continued to rock for subsequent loading excursions, as expected, after the formation of flexural cracks at 0.15% story drifts. No new cracks developed until the loading cycle of 0.75% story drift, when cracking was observed in the sill masonry near the toe. More cracking occurred in the sill masonry with each cycle of loading, and a sig-

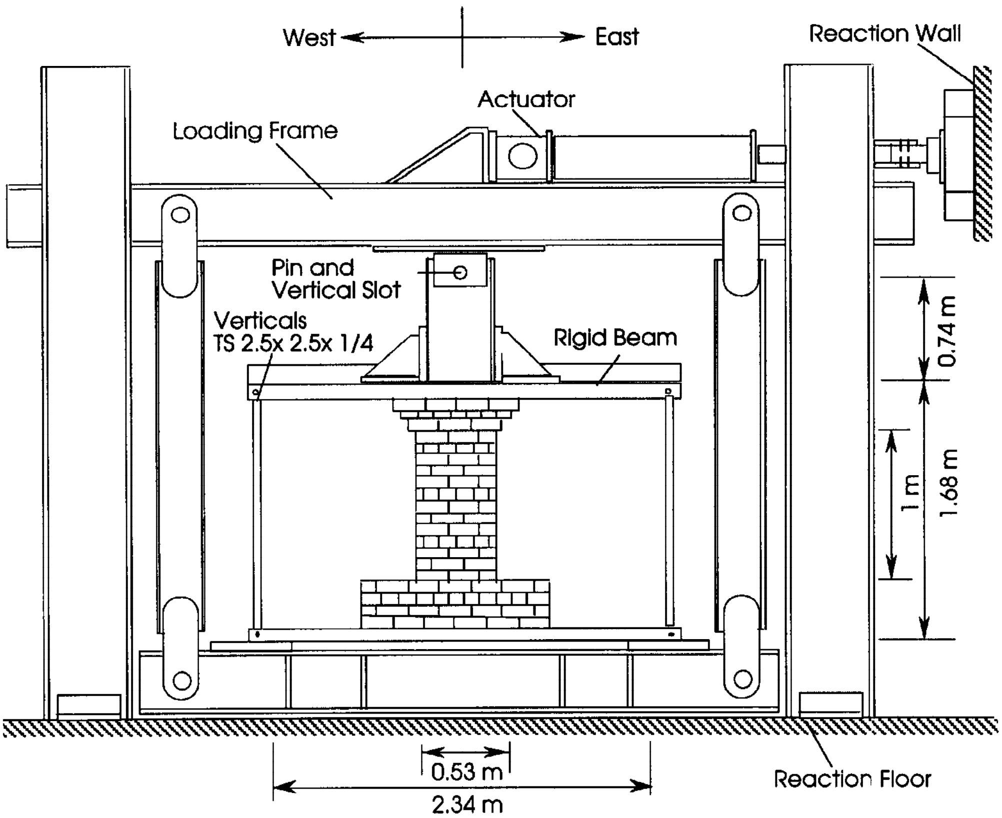


Fig. 2. Interior pier specimen as positioned in the loading frame.

nificant decrease in shear resistance was noted at 3.0% story drift. The peak strength was registered at 89.0 kN (20.0 kip) for westward excursion and at 74.8 kN (16.8 kip) for eastward excursion; these are markedly different values in each direction. This asymmetric behavior was due to the non-symmetric cracking pattern of the specimen, probably because of the inability of the stepped spandrel masonry to provide equally effective constraints in both directions. The specimen eventually failed by nearly vertical splitting in two parts — a characteristic failure pattern of brick masonry under compression — at a very large displacement of 4.5% story drift.

Nearly 42% of the total energy was dissipated consistently over the entire loading regime by the rocking piers through inelastic activity and through minor cracking in the sill masonry. Both steel verticals experienced ten-

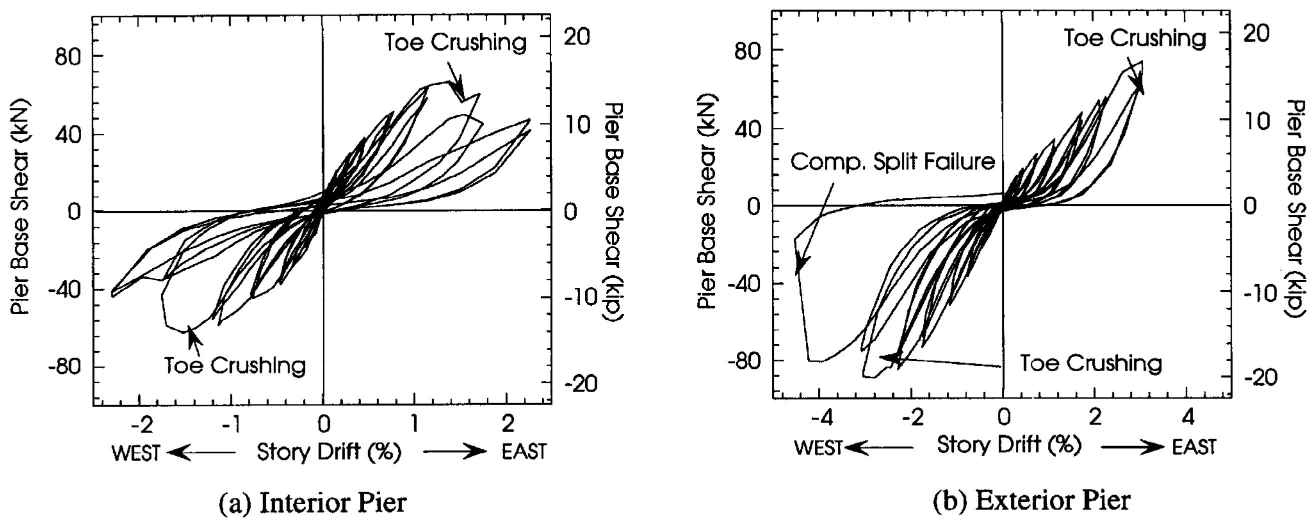


Fig. 3. Overall hysteretic response of the stabilized piers

sion as the rocking piers "wedged-up" the spandrel beam subjecting the rocking pier to additional compression enhancing the lateral resistance. The pier compression and shear resistance was observed to be linearly related to the lateral resistance in the rocking regime.

ANALYTICAL INVESTIGATION

Finite Element (FE) Based Study

Finite element (FE) approach was adopted to provide information at the "microscopic" level in order to ascertain the key parameters needed in a relatively simple mechanics based approach. The study was restricted to only monotonic loading because of exorbitant computational cost and complexities involved with FE analysis for cyclic loadings. A general purpose computer program, ABAQUS (Hibbit et al. 1995) was used for its ability to model the various sources of nonlinearities expected in a complex system of masonry and steel elements. A two-dimensional continuum in the state of plane stress was considered for all the masonry. However, one-dimensional axial and flexural elements modeled various steel members. The discretization of masonry was achieved by 25.4 or 76.2 mm (1 or 3 in.) square sized, 4-noded isoparametric CPS4 type elements of the ABAQUS element library. Steel verticals were modeled as truss elements of type C1D2 while the top horizontal steel member and the vertical stub were modeled as beam elements of type B21 of the element library. Beyond the elastic regime, the material behavior of the masonry was modeled with the material option *CONCRETE, which is used to define the properties of plain concrete outside the elastic range. Since this option captures most of the characteristics of a brittle material in terms of its non-linear stress-strain relationship, cracking, and failure, it has been used by other researchers to model masonry material with satisfactory results.

A Simple Mechanics-Based Model

A simple mechanics based model was developed to provide an approximate but complete picture of the load-deformation of a rocking pier under a set of stabilized forces.The approach is similar to the one attempted for face-loaded walls by McDowell et al. (1956). This model assumes that lateral resistance of the wall piers is due entirely to in-plane forces set up in the pier as a result of the masonry's tendency to crush at the end supports against sill and spandrel masonry. It is further assumed that the laterally loaded wall pier has already developed flexural cracking on the tension sides at the ends where elastic moments were maximum. On being further deflected, the wall pier pushes against the support masonries, assumed to be unyielding, thus, creating clamping forces P_u at the ends as indicated in Fig. 4. During subsequent loading, the wall pier remains rigid and rotates about the end support without any translation. This rotation is resisted by the internal force couple (P_u times r_u) which develops as a result of the crushing of masonry at these locations.

The stabilizing moment $P_u\, r_u$ is a function of the pier geometry, masonry material properties, contact area and lateral deflection Δ. The pier is assumed to be a uniform solid body of height H, width D, and thickness t. Masonry material is assumed to have an elasto-plastic stress-strain relationship in compression, that is, it is elastic up to strain ε_c corresponding to crushing stress f_c and for compressive strains greater than ε_c the stress remains constant at f_c. The stress drops to zero whenever strains decrease in the plastic range and a permanent set equal to the plastic strain is maintained afterwards. In tension, the material is assumed to have no resistance.

The kinematic relation implied by the assumed rotation of the pier results in the following required strain-displacement along the contact area, which is fundamental to this model:

$$\varepsilon_y = (u/s)\,(1 - us/2 - 2y/D) \tag{2}$$

(2) is combined with the assumed stress-strain relation of the masonry to give a stress distribution along the contact area. The clamping force P_u is then evaluated as the resultant of this stress distribution by integrating across the width of the wall. It clearly depends on the non-dimensional deflection parameter u. With P_u known at a given u, the internal force couple M_u is given as the product of P_u and r_u, where r_u is the moment arm and can be approximated as,

$$r_u = D(1 - us - 2\bar{y}/D) \tag{3}$$

where $\bar{y}$ locates the centroid of the stress distribution where the resultant P_u acts. Knowing the moment resis-

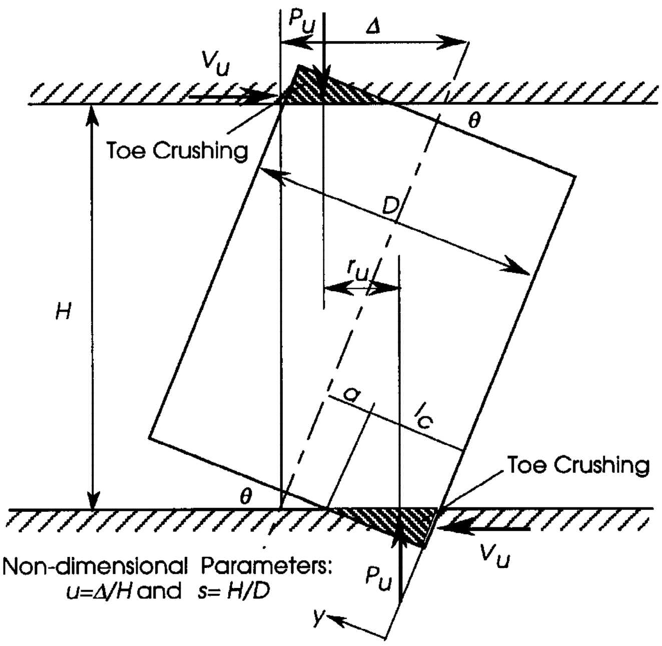


Fig. 4. Idealized deflected position of the rocking pier.

tance M_u, the lateral load-deflection $(V_u$-$u)$ relation can be easily obtained by equating it to the externally applied moment, V_u times H, i.e.,

$$V_u = M_u/H = (P_u r_u)/H \tag{4}$$

The stress distribution at the contact is linear elastic until the stress at the extreme edge fiber reaches the peak stress of f_c; and the lateral displacement at which this occurs is:

$$u_1 = (1-\sqrt{1-2\varepsilon_c s^2})/s \tag{5}$$

For $u > u_1$ the stress distribution is elasto-plastic and with further increase in deflection, strains ε_y continue to increase and more contact area crushes at stress f_c. This continues until the unloading begins at a fiber already crushing at f_c. The displacement u_2 at which this unloading commences the first time at a fiber is:

$$u_2 = \sqrt{2\varepsilon_c} \tag{6}$$

In most practical situations, the acceptable deflection limit will be smaller than u_2. For $\varepsilon_c = 0.005$, a typical value for the masonry, u_2 will correspond to 5% of story drift if the pier height is half the story height. The final expressions of P_u and V_u, defining the load-deformation characteristic of rocking piers in the range $0 \le u \le \sqrt{2\varepsilon_c}$, are given below:

for $0 \le u \le u_1$

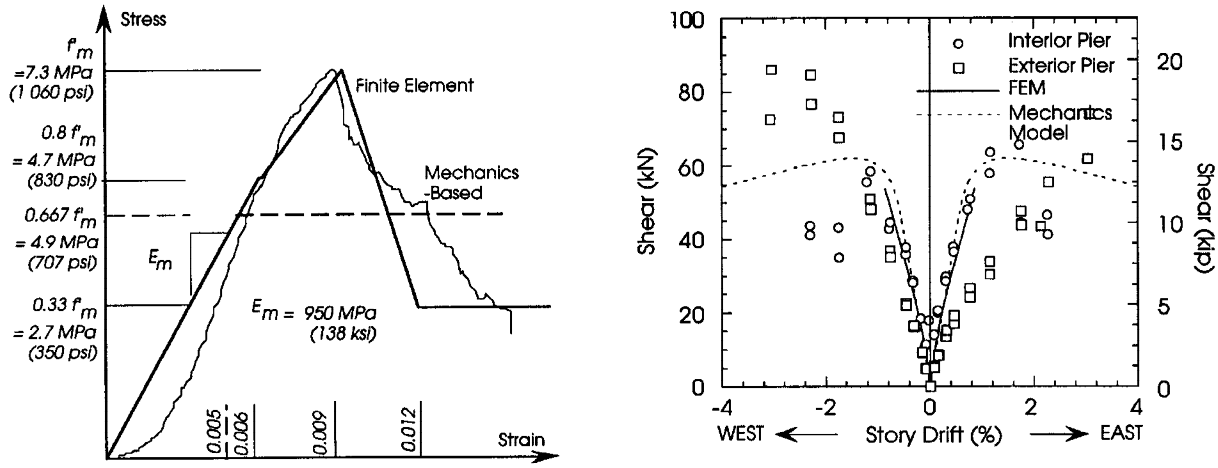
$$P_u = \frac{AE_m}{4s}\left[u\left(1-\frac{us}{2}\right)^2\right];\; V_u = \frac{AE_m}{6s^2}\left[u\left(1-\frac{us}{2}\right)^2\left(1-\frac{5us}{4}\right)\right] \tag{7}$$

for $u_1 \le u \le u_2$

$$P_u = \frac{Af_c}{2}\left[1-\frac{us}{2}-\frac{s\varepsilon_c}{2u}\right];\; V_u = \frac{Af_c}{4s}\left[1-2us-\frac{\varepsilon_c u^2}{2u}+\frac{3u^2s^2}{4}-\frac{\varepsilon_c^2 s^2}{3u^2}\right] \tag{8}$$

Comparison of Analytical Predictions with Experimental Results

In Fig. 5, the experimental envelope values of base shear for exterior and interior piers is compared with those predicted by the FE analysis and the mechanics-based model. Most of the parameters needed for the analysis were obtained from the compression tests conducted on the masonry prisms and the observed non-linear compressive behavior was approximated as shown in Fig. 5(a). Default values (assumed for plain concrete material) were used for the parameters which were not available for the masonry. Reasonable agreement between



(a) Compressive behavior of masonry

(b) Envelope shear response of piers

Fig. 5. Idealized compressive behavior of masonry and comparison between experimental and predicted shear envelope of rocking piers.

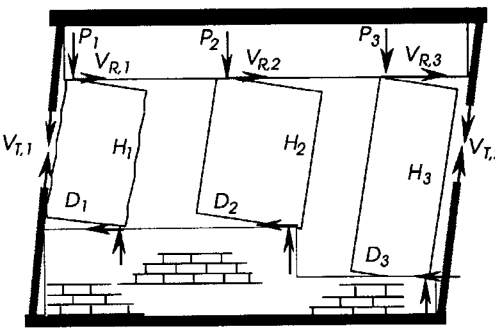
the analytical results and experimental data was found. Mechanics model does capture the general shape of load-deformation curve and the ultimate loads sustained by them before substantial cracking commenced.

Load-sharing Mechanism Between Rocking Piers and Verticals

Analytical studies confirmed the experimental observation that restoring (stabilizing) moment of a rocking-critical pier can be greatly enhanced by the verticals in a steel frame, The steel frame engages in resisting lateral loads only after the piers have begun rocking — verticals practically have no effect on pier's behavior in pre-rocking stage. The rocking motion of the pier is associated with a vertical movement of its corner (toe) butting against the support masonries and the steel verticals resist the motion by restraining this vertical movement. As a result, the verticals are subjected to tension forces which in turn are reacted by the compression forces of the rocking piers. It is the flexural action of the spandrel beams and the horizontal steel member which transfers the effect of verticals (i.e. compressive forces to the piers). These in-plane compressive forces in the piers create a couple which resists the lateral motion of the pier. The load from the verticals to the piers is transferred via the flexural action of the spandrel beams and the horizontal steel member. The in-plane compression forces in the piers create a force couple which resists the lateral motion of the pier. The axial forces in verticals, the pier compression and the lateral resistance increase nearly linearly as the rocking pier undergoes a near rigid body rotation. The large pier compression causes very high stresses in the toe region of the pier where damage to the masonry is first observed. This basic resistance mechanism is not affected by the location of the pier in the frame. However, the axial force in each of the steel verticals is related to the arrangement of the piers in the frame.

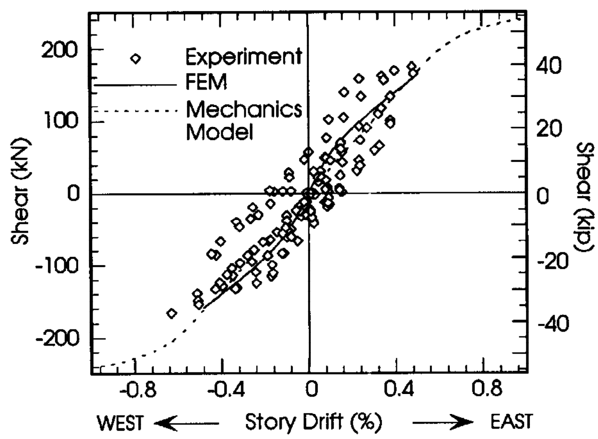
SHEAR BEHAVIOR OF A SYSTEM OF URM ROCKING PIERS

Since rocking is a story phenomenon, the simple addition of shear resistances of individual piers gives the total story shear. The above investigation shows that the total shear is shared among various rocking piers in a ratio of the relative rocking rigidity of each pier, which is given by *(PD/H)* for each pier. The compression in each pier can then be determined by (1) and is assumed to be working at a point 0.05D from the edge of the pier. From there the axial forces in the steel verticals can be obtained easily by moment equilibrium, as shown in Fig. 6(a). In Fig. 6(b), the measured lateral resistance of the four-pier system is compared with those predicted by the theoretical models mentioned. For mechanics based model, the shear-displacement curves were obtained from (7) and (8) and then simple addition of force quantities gave the response for the overall system. Once again, a close match verifies the validity of analytical approaches.



$$P_i = \frac{V_i}{(1-2k)\,(D_i/H_i)};\ \sum V_{R,i} = V;\ \sum V_{T,1} = \sum P_i$$

(a) System of rocking piers



(b) Shear response of four-pier wall system

Fig. 6. Shear behavior of a system of rocking pier.

CONCLUSIONS

This experimental as well as analytical study clearly indicates that in-plane seismic performance of URM wall piers can be considerably improved by a steel frame consisting of vertical and horizontal elements alone. The strengthened system not only had excellent strength, stiffness and ductility but also controlled the damage to brittle wall piers and thus provided safety against sudden failure. The post-elastic behavior of the integrated system is more ductile despite the brittleness of masonry. This strengthening scheme utilizes the considerable load sharing which occurs almost at all load stages between the existing masonry and the added steel elements. The controlled amount of masonry cracking adds to the damping of system without serious risk to the structure's vertical load carrying capacity. The steel framing system seemed to be compatible with the stiffness of the masonry piers and helped to maintain their structural integrity even at large displacement levels. The failure mode of interior wall piers did not change due to the steel frame. They behaved in rocking mode during the entire loading history. The loss of stiffness and strength was observed due to major crushing of the base masonry. Finite element analyses using ABAQUS successfully predicted the experimental response of the piers and verified the mechanism of interaction between masonry and steel verticals. The simple mechanics based model predicted the load-deflection behavior of the stabilized rocking piers reasonably well and can be used to determine the capacity of URM "rocking" wall piers at a given displacement level, which is needed in order to design the strengthening system.

REFERENCES

ABK (1984). Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings. *Topical Report 08, National Science Foundation, Contract No. NSF-C-PFR-78-19200,* Applied Science and Research Applications, Washington, DC, 20550.

Hibbit, Karlson and Sorenson (1995). ABAQUS User's Manuals, Version 5.4, Hibbit, Carlson and Sorenson Inc., Providence, RI.

McDowell, E. L., McKee, K. E. and Sevin, E. (1956). Arching action theory of masonry walls. *J. Struct. Div., ASCE, ST2,* Mar., 915/1-915/18.

Rai, D. C., Goel, S. C., and Holmes, W. T. (1994). Seismic strengthening of URM buildings with ductile steel bracing. *Proc. of ASCE Structures Congress,* Atlanta, April 24-28.

Rai, D. C. (1996). Seismic strengthening of unreinforced masonry buildings with steel elements. Ph. D. Dissertation, Dept. of Civ. Engrg., The University of Michigan, Ann Arbor, MI, Jan.

UCBC (1991).Uniform Code for Building Conservation. International Conference of Building Officials, Whittier, California.