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Strength and behavior of unreinforced masonry elements

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ABSTRACT: Results of a series of experiments on lateral strength and behavior of unreinforced masonry elements reveal that masonry walls or piers need not be considered brittle. Measured behavior of the walls showed that unreinforced masonry can possess considerable capacity for inelastic deformations, and need not be limited in strength by forces which induce initial flexural or diagonal tensile cracks. Behavior under reversed and repeated cyclic forces was not influenced appreciably by previous damage, and could be represented for each half cycle with analogous behavior under monotonically increasing forces. A study of the mechanics in cracked masonry walls is presented which serves as the basis for a suggested evaluation procedure.

1 INTRODUCTION

Evaluation of lateral strength in unreinforced masonry buildings is many times clouded because of uncertainties associated with estimating shear or flexural strength of individual walls or piers. Furthermore, an incomplete depiction of inelastic behavior for such elements aggravates the situation since the nature of story shear redistribution to various elements in a building structural system is not well understood.

For lack of better information, unreinforced elements are usually assumed to be brittle. Lateral strength is limited by allowable stresses, and no inelastic action is considered. This assumption infers that, upon initial cracking, all lateral force resisted by a particular masonry element is transferred to adjacent elements which in turn reach their limit when overloaded. Much like buttons on a shirt, when the first one tears the rest follow quite suddenly. This assumption is very severe because the lateral strength of a system is limited to the strength of its weakest element. If masonry elements are actually not brittle, then the lateral strength of a system may be thought of as the combined strengths of all of its masonry elements.

Most seismic codes prescribe an equivalent static lateral base shear for unreinforced masonry buildings on the basis that the structure has no capacity for inelastic deformation. For example, R factors are commonly taken as 1 or 2 for various codes of practice. This limitation, along with the ignorance of any story shear redistribution, minimizes the seismic capacity for a structure to something far less than it actually may be.

The ultimate limit state for an unreinforced masonry wall or pier is commonly associated with first cracking whether it be a result of flexural tension or diagonal tension. No redistribution of stress within the element is assumed after initial cracking. When an evaluation criteria is based on this limit state, lateral strength may be unreasonably restricted. This is particularly so for older construction with lime mortars where flexural tensile strength is assumed close to zero.

This paper demonstrates through the use of experimental and analytical analyses that unreinforced masonry elements do not necessarily fail immediately soon after cracks develop, and as a result, they can behave with considerable deformation capacity. The series of experiments described in this paper demonstrate the ability of unreinforced walls to resist up to three times their initial cracking strengths. In addition to this residual strength, the series of unreinforced walls were seen to possess a gradual softening until the ultimate limit state was reached rather than cracking abruptly. A simple analytical solution is presented to estimate post-cracking ultimate strength of masonry flexural elements, and an evaluation procedure based on it is given.

2 BEHAVIOR UNDER MONOTONIC LOADS

A series of unreinforced brick walls were tested at the University of Illinois at Urbana-Champaign. The test walls were extracted from a building constructed in 1917, and were transported across the street to the Newmark Laboratory for testing. Each wall consisted of four wythes of unreinforced brick which were interlaced with header courses. Despite the age of the masonry, it was apparent that the mortar was quite strong and that cement must have been used in the mix. Experimental procedures and results are presented in Abrams and Epperson (1989) and Epperson and Abrams (1989, 1990).

Lateral forces were applied within the plane of each wall while a constant vertical compressive stress was maintained. The tests were run by controlling the lateral force applied at the top of each test wall. A summary plot for the five wall specimens is shown in Fig. 1 where aver-

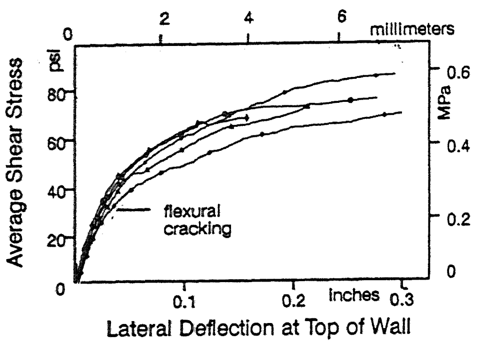

Figure 1. Measured behavior of test walls.

age shear stress (total lateral force divided by gross wall area) is plotted versus the lateral displacement at the top of each wall. The aspect ratio of all specimens was similar. Vertical compressive stress was varied for each specimen within a range of 0.52 to 0.99 MPa (76 to 143 psi).

Though the walls were unreinforced, they had a long life after cracking. Flexural cracks were observed at the

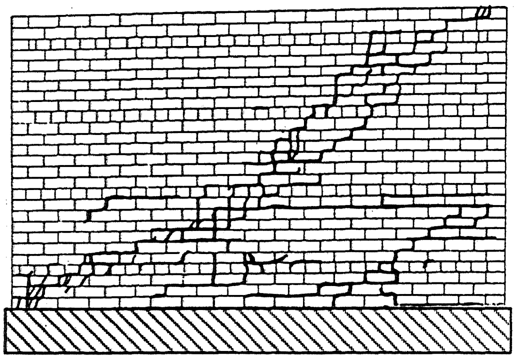

Figure 2. Typical crack pattern under monotonic loads.

heel region of each wall at a lateral load that was nominally 40% of the ultimate load. As the resultant of vertical compressive stress tended to shift towards the wall toe after cracking, the lateral stiffness gradually diminished. The ultimate limit state was precipitated by sliding along bed joints in the vicinity of the central portion. This was followed immediately by diagonal compressive splitting at the toe. Lateral deflections at ultimate were nominally 15 times that at initial flexural cracking. Ultimate lateral drifts were nominally 0.4% for all test walls.

3 BEHAVIOR UNDER CYCLIC LOADS

Basic resistance mechanisms are most easily understood and developed for structural elements that are subjected to lateral forces that increase monotonically until failure occurs. During an earthquake, however, buildings sway back and forth and lateral shears and deformations follow many repeated and reversed cycles. Therefore, it is of interest to know the relation between lateral strength and behavior under monotonically increasing loads to that for loads which cycle.

Two test walls were built using reclaimed Chicago common brick with Type N mortar, and subjected to cyclic lateral forces. Though an attempt was made to mimic materials used in the 1917 walls, it was impossible to replicate load-deflection behavior precisely because environmental aging effects could not be simulated. The old walls were subjected to seventy years of freeze-thaw cycles. As a result, delamination of the mortar bed joints occurred which lowered the flexural tensile strength normal to the joints. The new walls were tested a few months after construction, and were subjected to nearly no temperature or humidity variations. Therefore, only general correlations in behavior can be made for old and new specimens.

The length-to-height aspect ratios of the two walls was varied as well as the vertical compressive stress, so that two basically different behavioral modes could be observed. The first test wall had an aspect ratio of 2.0 and was subjected to a vertical stress equal to 0.52 MPa (75 psi). The second wall had an aspect ratio of 1.5, and was subjected to a stress equal to 0.34 MPa (50 psi).

The two test walls were subjected to a simple series of lateral forces from a twin pair of hydraulic actuators. One actuator was operated in displacement control while the other was controlled to apply an identical force as the first.

More information on testing of these two walls can be found in Shah and Abrams (1992).

3.1 *Hysteretic shear behavior*

The first wall being somewhat stocky failed in shear with no flexural cracking (Fig. 3). At a lateral force equal to

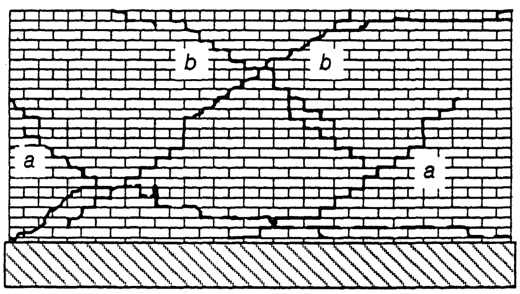

Figure 3. Observed crack pattern for Wall 1.

approximately 62% of the ultimate load, a stair-step diagonal crack (marked with the letter *a* in Fig. 3) was observed. A second diagonal crack (*b*) was observed just as the test wall was reaching its ultimate load.

Reversing the lateral force for the first specimen resulted in an identical crack pattern as for the earlier half cycle. It appeared as though the previous loading and damage had little to do with the subsequent behavior. When previously opened cracks in the mortar joints closed, behavior emulated that for an uncracked joint. Thus, it appears as though behavior under monotonically increasing forces, or deflections, may be representative of that for the cyclic loading case.

The lateral force-deflection hysteresis loop for the first test wall is shown in Fig. 4. The measured relation reveals

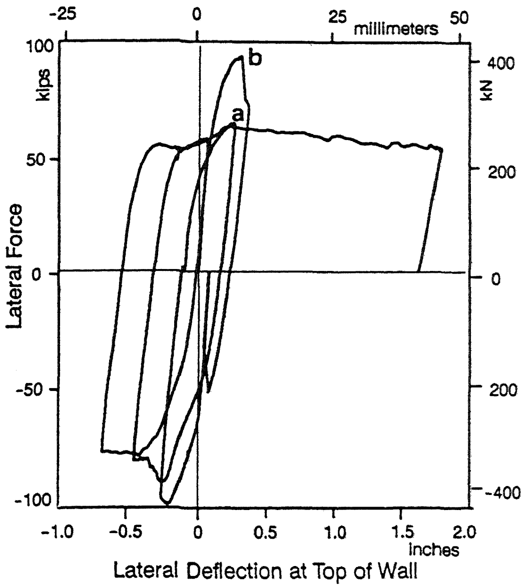

Figure 4. Measured behavior of Wall 1.

symmetry with respect to reversed loading cycles corroborating the finding that cyclic behavior may be uncoupled into monotonically increasing load components. The striking feature, however, is the fact that the wall could resist considerable lateral force while deflecting to extremely large deflections. Lateral drifts as large as seven times the drift at peak loads were observed. The test wall could have shown a larger deformation capacity that this if the stroke limit of the loading actuators had not be exceeded.

Though stair-stepped diagonal cracks opened the mortar head joints considerably (as much as 25mm or one inch), the bed joints remained closed as shown in Fig. 5. The large post-cracking strength had to be attributable to friction acting along the bed joints. Thus, the vertical compressive stress made a significant difference on the apparent ductility of the element.

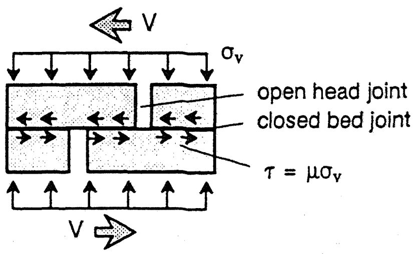

Figure 5. Shear transfer across stair-stepped crack.

3.2 Hysteretic flexural behavior

The second test wall being less stocky than the first, and subjected to a lighter vertical compressive stress, cracked initially in flexure. A horizontal crack along the bed joint immediately above the bottom course was observed along approximately two-thirds of the base (crack *a* in Fig. 6).

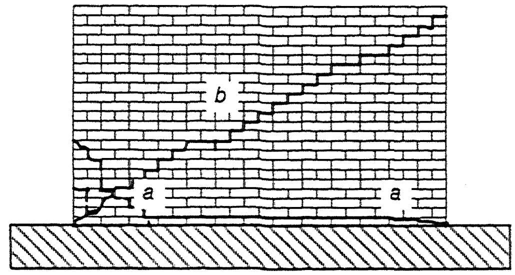

Figure 6. Observed crack pattern for Wall 2.

When the lateral force was reversed, the flexural crack occurred on the opposite side of the wall, and was then continuous along the entire length of wall.

Lateral force-deflection behavior for Wall 2 is shown in Fig. 7. Peak lateral strength was limited by crushing of

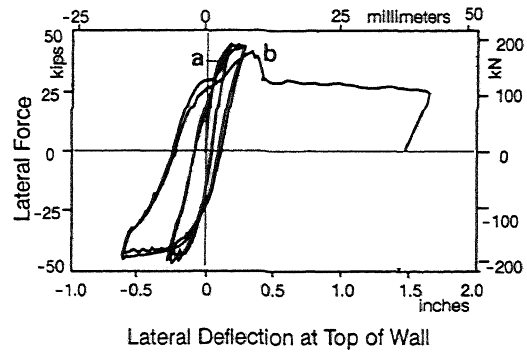

Figure 7. Measured behavior of Wall 2.

masonry at the wall toe. Vertical splitting cracks, and spalling of bricks was observed at the toe much like that for a prism of masonry subjected to concentric vertical compression.

Like the previous test wall, behavior for loading in one direction did not appear to be influenced by previous damage in the other loading direction. The flexural crack simply closed as the force was reversed. No reductions in stiffness were observed when the force was reversed suggesting that the open crack had closed during unloading as a result of the vertical compressive stress. Ultimate lateral strengths in each direction of loading were quite similar as might be expected. Previous flexural cracking did not influence the compressive strength at the toe.

Only until the very last half cycle of loading did a diagonal shear crack occur (crack *b* in Fig. 6.) It is interesting to note that this diagonal crack did not occur at the peak loads of the earlier cycles, but after the same maximum lateral force had been reached several times (noted with point *b* in Fig. 7). The shear crack was a result of the lateral deflection rather than the lateral force. It is surmised that with increased lateral deflections, the flexural crack grew in length, and shear stress continued to be redistributed to the ever shrinking compression zone near the toe of the wall. Thus, the concentration of shear stress increased though the lateral force did not until diagonal cracking occurred.

Because the total vertical force was held constant during the test duration, the total restraining frictional force along the base of the wall remained the same though the resultant vertical force had to shift across the wall to near the toe. This implies that flexural cracking did not tend to reduce the overall shear strength which is the reason why the diagonal tension strength could be reached well after flexural cracks were first observed.

The second test wall also revealed a surprising capacity for inelastic deformation. Like the first test wall, the second continued to resist lateral force well after flexural and shear cracks were dominant. It was again obvious that a frictional mechanism had developed along the compressed bed joints (Fig. 5) because 25mm wide (1 inch) separations in head joints were observed.

In summary, in-plane behavior of the two test walls suggested that unreinforced masonry elements may resist sizeable lateral forces while deforming well beyond the linear range of response. This was found true for walls controlled by both flexure or shear. If used in a building system, the test walls would have a high potential for dissipating energy, as well as shedding lateral force to other elements while resisting lateral forces in the range of their ultimate strength.

4 STATICS OF CRACKED MASONRY WALLS

Both series of test walls showed that substantial increases in lateral strength could exist past initial cracking whether it be for older walls tested with forces that increased monotonically to failure, or for newer walls subjected to repeated and reversed cyclic lateral forces. The reason for this can be explained in terms of simple statical equilibrium concepts.

An unreinforced wall can resist moment after cracking in flexure through the force couple that is generated as the centroid of the vertical force resultant shifts toward the compression toe. Lateral strength can be derived on the basis of statical equilibrium of external forces as shown in Fig. 8. Summing moments of forces about the centroid of the vertical force resultant at the base gives the equilibrium equation of Eq 1.

Assuming linear material behavior in compression, the length of the compressed zone at the base, *d*, can be determined to be three times the distance from the extreme compression fiber to the location of the resultant force. The compressive edge stress can then be determined as twice the average stress over this triangularly stressed zone as given by Eq. 2 where the term *Hh/P* has been substituted for the eccentricity, *e*, according to Eq. 1.

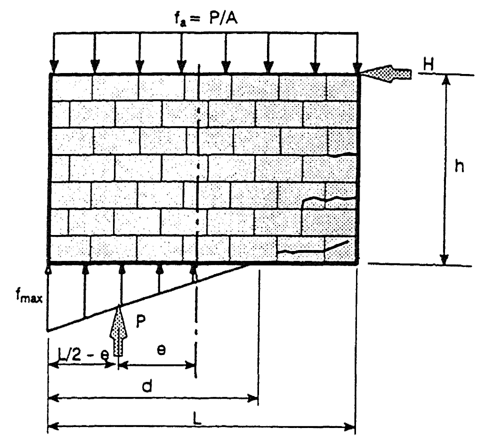

Figure 8. Free body diagram of wall cracked at base.

$$Hh = Pe \quad \text{or} \quad e = \frac{Hh}{P} \qquad [1]$$

$$f_{max} = \frac{2P}{3b\left(\frac{L}{2} - \frac{Hh}{P}\right)} \qquad [2]$$

The limiting stress, f_{max}, is based on the assumption that compressive stress will be linear with strains. Any nonlinearities in compressive behavior would shift the resultant vertical force towards the wall toe, and thus increase the moment capacity. Thus the formulation is slightly conservative in this regard.

5 SUGGESTED EVALUATION PROCEDURE

5.1 Limiting flexural stress

A safe limit on lateral strength can be estimated by equating the compressive edge stress, f_{max}, (Eq. 2) to some allowable value, F_a. Since the edge stress is a composite of both axial and flexural stresses, and the fact that the stress gradient is rather small for most in-plane walls, the lower allowable axial stress value is suggested over the higher allowable flexural stress value, F_b. Values for F_a prescribed by the appropriate building code may be used.

Substituting F_a for f_{max} and H_a for H in Eq. 2, and solving for the allowable lateral force capacity gives:

$$H_a = \frac{P}{3bhF_a}\left(\frac{3bL}{2}F_a - 2P\right) \qquad [3]$$

Further expansion and normalization by the area, *A*, (*b* times *L*) results in Eq. 4 for the average shear stress, f_{va} (H_a over the gross area, *bL*) that is limited by toe compression.

$$f_{va} = \frac{H_a}{A} = f_a \left(\frac{L}{h}\right) \left[\frac{1}{2} - \frac{2}{3}\frac{f_a}{F_a}\right] \qquad [4]$$

where f_a is the nom ıal vertical stress $= \frac{P}{bL}$

It is interesting to note that the aspect ratio, L over h, can be uncoupled from the other parameters in Eq. 4, and that the average shear stress is then linearly related to the vertical compressive stress for a given ratio of f_a / F_a. The relation between f_{va} and f_a is shown in Fig. 9 for various ratios of f_a / F_a. The aspect ratio is taken as 1.0. Limiting shear stresses for walls of other aspect ratios can be determined simply by multiplying ordinate values by the wall aspect ratio.

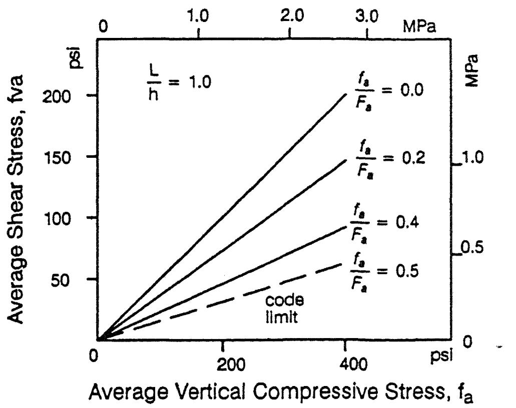

Figure 9. Ave. shear stress vs. ave. compressive stress.

The bottom line in Fig. 9 for the ratio f_a to F_a equal to 0.5 represents the case where axial and flexural stresses are equal, or the case of no net tensile stress. Any combination of shear stress, f_{va}, and compressive stress, f_a, that results in a point below this line indicates a case where the vertical compressive stress is sufficiently high relative to the shear stress that the wall base will be stressed in compression along its entire length. Thus, this line represents a lower limit for Eq. 4. It can also be construed to represent the allowable lateral force that is limited by American codes of practice on the basis of a limiting flexural tension stress (in this case for a F_t value equal to zero).

Depending on the ratio of f_a to F_a, the lateral force capacity of a wall can be as high as three times that limited by current codes on the basis of a limiting flexural tensile stress. This is a clear demonstration that flexural cracking should not represent an ultimate limit state for lateral capacity of unreinforced masonry walls. Since the formulation is based on the concept of statical equilibrium, there should be little question of its applicability.

5.2 *Limiting shear stress*

If post–cracking behavior is to be relied on for flexure as described in the previous section, then some allowance must be made to account for the reduction in shear strength with flexural cracking.

It is assumed that shear strength will reduce directly with the length of uncracked wall. This assumption is based on the consideration that zero shear stress can be transferred across an open flexural crack. Only the uncracked portion of the wall is considered effective in resisting shear. Because flexural cracks will be the longest at the base, reductions in shear strength are evaluated at this location. Thus, the portion of wall that is considered effective in resisting shear consists of a rectangular area a distance *d* long by *h* high as shown in Fig. 10. Though this assumption may not be realistic, it is conservative and permits an engineer to rely on accepted code values for allowable shear stress.

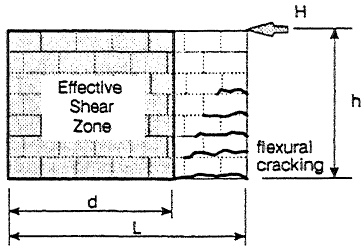

Figure 10. Assumed effective shear zone.

To simplify the problem, and to be consistent with previous assumptions, the length of uncracked wall at base, *d*, is determined for the special condition when F_t is equal to zero. For horizontal shear forces, *H*, that are small

$$\text{if } H < \frac{PL}{6h} \text{ then } d = L \qquad [5]$$

$$\text{if } H \geq \frac{PL}{6h} \text{ then } d = \frac{3L}{2} - \frac{3Hh}{P} \qquad [6]$$

enough so that cracking will not occur at the base, *d* is simply equal to the full length of wall, *L*, (Eq. 5). For forces that are large enough to crack a wall, the distance *d* is a fraction of the overall length according to Eq. 6. This equation is derived simply by multiplying the distance between the compression toe and the centroid of the vertical force resultant by three. The distribution of compressive stress at the wall base is again assumed to be triangular.

The fraction of the wall length that is uncracked can be expressed by dividing each side of Eq. 6 by *L*, and substituting f_v / f_a for H/ P.

$$\frac{d}{L} = \frac{3}{2} - 3\left(\frac{h}{L}\right)\left(\frac{f_v}{f_a}\right) \qquad [7]$$

Wall shear stress that is limited by toe compression, f_{va}, was found to be related to the ratio of f_a to F_a in Eq. 4. If it is assumed that flexure will govern over shear, then f_{va} is equal to f_v, and the ratio of f_v / f_a from Eq. 4 may be substituted in Eq. 7 as is done in Eq. 8.

$$\frac{d}{L} = \frac{3}{2} - 3\left(\frac{h}{L}\right)\left(\frac{L}{h}\right)\left[\frac{1}{2} - \frac{2}{3}\frac{f_a}{F_a}\right] \quad [8]$$

Reduction of Eq. 8 gives the following simple expression for the fraction of uncracked wall;

$$\frac{d}{L} = 2\left(\frac{f_a}{F_a}\right) \qquad \text{for } f_a \leq 0.5F_a \quad [9]$$

The relation is limited to only those cases when f_a is less than one-half of F_a. For values of vertical compressive stress equal to this limit, the wall would be compressed along its entire length ($d = L$). The limiting ratio of f_a to F_a equal to 0.5 is shown with the broken line in Fig. 9.

Since it is common to express shear strength in terms of the product of some allowable shear stress, F_v, and the gross web area, any reductions in the effective shear area attributable to flexural cracking will be stated in terms of an equivalent reduction in the allowable shear stress rather than a reduction in the available shear area. Thus, allowable shear stress, F_v, is expressed as a fraction of the allowable shear stress for a fully uncracked wall, F_{vo}. It is assumed that the allowable stress reduces in the same proportion as the wall cracks. Thus, a very simple estimate of allowable shear stress that accounts for effects of flexural cracking can be obtained from Eq. 11.

$$\frac{F_v}{F_{vo}} = \frac{d}{L} = 2\left(\frac{f_a}{F_a}\right) \qquad \text{for } f_a \leq 0.5F_a \quad [10]$$

Code values of allowable shear stress, F_{vo}, can thus be reduced by multiplying by a factor equal to twice the ratio of f_a to F_a to account for the extent of flexural cracking.

5.3 Proposed evaluation procedure

Using Eq. 4, the ultimate lateral flexural capacity of a cracked wall can be easily estimated in terms of the vertical compressive stress, f_a, and the allowable compressive stress, F_a. Capacity based on a code allowable value of shear stress, F_{vo}, and the extent of flexural cracking can then be checked using Eq. 10.

As a verification of this procedure, lateral capacity of the series of laboratory test walls was estimated, and compared with actual strengths (see Abrams, 1992 for a complete description). Instead of using allowable compressive stresses in Eq. 4, tested prism compressive strengths were input so that a direct correlation with measured ultimate strengths could be made. Likewise, measured results of nondestructive in-place shear tests were used in Eq. 10 in lieu of allowable shear stresses. Predicted strengths ranged from 89% to 104% of measured test values suggesting that the procedure was indeed valid.

6 CONCLUSIONS

Results of two series of laboratory experiments illustrated that unreinforced masonry elements may behave differently than that which is commonly assumed. Test walls constructed over seventy years ago and subjected to monotonically increasing lateral forces, as well as test walls which were newly constructed and subjected to reversed cyclic loads, revealed that unreinforced masonry elements may:

1. be significantly stronger than their strengths at initial cracking.
2. have substantial deformation capacity past initial cracking.
3. be characterized in terms of their behavior under monotonically increasing forces/deformations even though they are subjected to cyclic loads.

An evaluation procedure has been suggested for estimating safe limits of in-plane lateral strength of cracked, unreinforced masonry walls. A simple formula has been derived for estimating permissible shear stress based on toe compressive stress. Another expression has been given to determine reductions in shear strength attributable to flexural cracking. The proposed method was shown to provide excellent correlations with measured lateral strengths.

7 ACKNOWLEDGEMENTS

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8 REFERENCES

Abrams, D.P. 1992. Safe limits for lateral capacity of cracked URM walls. *Proc. of Sixth Canadian Masonry Symposium*, University of Saskatchewan, 12pp.

Abrams, D.P., and Epperson, G.S. 1989. Evaluation of shear strength of unreinforced brick walls based on nondestructive measurements, *Proceedings of Fifth Canadian Masonry Symposium*, University of British Columbia, pp. 817–826.

Epperson, G.S., and Abrams, D.P. 1989. Nondestructive evaluation of masonry buildings, Advanced Construction Technology Center, *Report No. 89–26–03*, University of Illinois at Urbana-Champaign, 208 pp.

Epperson, G.S., and Abrams, D.P. 1990. Evaluating lateral strength of existing unreinforced brick masonry piers in the laboratory, *Proceedings of Fifth North American Masonry Conference*, Univ. of Illinois, pp. 735–746.

Shah, N. , and Abrams, D.P. 1992. Cyclic load testing of unreinforced masonry walls. Advanced Construction Technology Center, University of Illinois at Urbana-Champaign.