

DISCUSSION OF PAPER 'REINFORCED MASONRY SHEAR WALLS: CYCLIC LOAD TESTS IN CONTRAFLEXURE'

by S.J. Thurston and D.L. Hutchison
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Discussion by M.J.N. Priestley

The authors are to be congratulated for a useful addition to the sparse literature on the seismic performance of masonry structures. Although in general agreement with conclusions reached by the authors, the writer disagrees with interpretation of some of the results presented, and wishes to make some comments of a general nature.

1. Loading System

The paper presents a novel loading system which forced the pier units into double bending, thus simulating conditions in a pier forming part of a pierced masonry shear wall. Part of the reason for the tests was to investigate the significance of this loading system compared with the more commonly used simple vertical cantilever which, it was felt, might not be suitable for simulating inelastic action of piers in double bending.

It should be pointed out that dissipating seismic energy through plastic hinge formation in piers is exactly equivalent to forming hinges at top and bottom of columns in multistorey reinforced concrete frames. This involves the formation of a soft storey with high local ductility demand in the piers (or columns) for only low to moderate ductility capacity of the overall structure^(A1). Consequently, the draft masonry code¹ discourages this form of construction by requiring 'near-elastic' design.

Secondly, it should further be pointed out, that the loading system, though placing the wall in equal double bending as claimed, applies its loads in a fashion far removed from that occurring in practice. Scaling dimensions from the authors Test set-up (Fig. 2), since actual dimensions are not given in Fig. 2, results in the approximate vertical and horizontal load components applied to each end of the top load beam as shown in Fig. A1. It will be seen that with the direction of loading as shown ('push' loading), the top beam is placed in tension, and a very large shear load is applied very close to the tension end of the wall. Vertical reactions simultaneously require a high tension load (approximately 0.88P) to be applied at the same end of the wall. It is clear that if the reinforcement in the wall is uniformly distributed rather than concentrated at the wall end, premature yielding of extreme tension reinforcement must occur. Since the top beam is in axial tension its ability to distribute these large concentrated loads to the reinforcement without deformation, is reduced.

Clearly the situation at the top of the wall differs markedly from that at the wall base. In particular, shear transfer at the wall top under 'push' loading is disadvantageous. It is felt that this system of load application was largely responsible for premature degradation and failure at

the wall top experienced in many of the walls, and that segregation and settlement of grout (the explanation offered by the authors) is a secondary effect.

2. Partially Filled Walls

The paper is particularly useful in providing some much needed information on the behaviour of partially filled walls. However, not too much emphasis can be placed on the rather poor performance of the walls, particularly under ductile response, because of extreme violation of draft code¹ requirements, as discussed below.

Unit 4: The authors quote draft code¹ provision for walls to be designed with horizontal steel at 2.4 m centres provided the building is Class III in seismic zones B or C, and claim that Unit 4 thus complies with code requirements. However, other limitations of the draft code also apply: namely, the structure must be designed for 'near elastic' response (i.e. $S = 4$), and design shear stresses must not exceed values given in Table 3.1⁽¹⁾ for shear carried by masonry. For Grade A masonry, this latter provision would limit the maximum ideal shear stress at ultimate, v_i , to 0.3 MPa. With the effective depth of the wall estimated at $0.8 l_w$, or 1.28 m, this would limit the shear to be carried by Unit 4 to

$$\begin{aligned} V_{\max} &= v_i b_w \cdot d \\ &= 0.3 \times 0.14 \times 1.28 \text{ MN} \\ &= 54 \text{ kN based on the gross section} \end{aligned}$$

However, since the wall is partially grouted, the width should be taken as the net width of the face shells, further reducing the allowable ultimate shear force to about 25 kN.

By comparison, the flexural capacity of the wall corresponds to a shear of 168 kN, or 1.8 MPa based on net wall width. Obviously shear failure is expected at a load substantially lower than that associated with flexural yielding. In the light of these observations, the behaviour of Unit 4, with a maximum load of about 105 kN is felt to be very satisfactory.

Unit 2: Duplicated Unit 4, but with horizontal steel of D12 @ 1 m centres. Although this will help to provide basketing reinforcement, the spacing of horizontal reinforcement is larger than $l_w/2$, and therefore it cannot be considered effective as shear reinforcement. Nevertheless, the wall was able to exhibit ductile action because of the very low shear stress associated with flexural yielding.

Unit 5: Duplicated Unit 4, but with horizontal steel of D16 @ 500 centres on average. This should have resulted in an ideal shear strength of 234 kN. However, it appears that shear failure occurred at

low ductility, with a maximum load of about 150 kN. It appears from the photograph of failure (Fig. 9) that this premature failure may have been a direct result of the technique of load application, as the shear failure is concentrated in the upper few courses of the wall.

3. Displacements and Ductility Factors

The writer has had some difficulty in interpreting the authors definition of 'yield' displacements, and reconciling various data pertaining to ductilities and theoretical displacements and stiffnesses.

Theoretical deflections are apparently based on a measured modulus of elasticity of 6.66 GPa, found from compression tests on 3-course prisms. This value appears very low, and the writer wonders whether the results come from measurements of test machine ram displacement, which will invariably include machine flexibility and platten take-up, rather than from measurements over the central (say) half of the prism height, using extensometers. If, as the writer suspects, it was the former, unrealistically low values for E will result. In recent measurements over the central 3 courses of 5-course prisms (A2), and in measurements of tall masonry shear walls under axial load applications (A2, A3), the writer obtained values for E in the range 25-35 GPa.

The writer would welcome comment from the authors on his attempts to reconcile Table 3 with the text. It is stated that yield displacement Δ_y was found by factoring up the theoretical displacement at first yield Δ'_y (based on $E = 6.66$ GPa) by the ratio of ultimate to first-yield moment. Thus:

$$\begin{aligned}\Delta_y &= \Delta'_y \frac{M_u}{M_y} \\ &= \Delta'_y \frac{P_u}{P_y}\end{aligned}$$

It is not stated whether theoretical displacements were based on uncracked or cracked-section properties, nor whether shear deformations were considered.

Consider Unit 8. Based on the stated theoretical yield load of $P_y = 151.7$ kN, and the theoretical ultimate load of $P_u = 222.2$ kN:

$$\Delta'_y = \frac{P_y h^3}{12 EI} + \frac{1.2 P_y h}{A(E/2.3)}$$

$$\Delta_y = \frac{222.2}{151.7} \Delta'_y = 1.465 \Delta'_y$$

with $h = 2.4$ m, $l_w = 1.6$ m, and $b_w = 0.14$ m, the writer obtains the following values:

Uncracked Section:

flexure only : $\Delta'_y = 0.55$ mm \rightarrow $\Delta_y = 0.80$ mm
Flexure and shear: $\Delta'_y = 1.22$ mm \rightarrow $\Delta_y = 1.79$ mm

Cracked Section:

flexure only $\Delta'_y = 1.90$ mm \rightarrow $\Delta_y = 2.78$ mm
flexure and shear: $\Delta'_y = 2.57$ mm \rightarrow $\Delta_y = 3.76$ mm

For the last estimate, which should be the most realistic, shear deformation has been

based on an uncracked section, and is thus underestimated substantially. Assuming only the compression zone is effective in resisting shear deformation results in a very high value for Δ_y (approximately 6.8 mm) because of the low value for E.

None of the calculated values agree with the value of $\Delta_y = 2.45$ mm given in Table 3. Moreover, it appears that despite the statement in the text that Table 3 lists Δ_y rather than Δ'_y , it is in fact the latter value that is included, since the stiffness K_y in Table 3 is found by dividing the yield load P_y by the listed displacement.

Further confusion results when attempting to reconcile the yield displacements and ductilities with the values of Table 3. Again, choosing Unit 8 as an example, extrapolation of the straight line from the origin through the load-deflection point at $0.75 P_y$ on Fig. 12 gives yield displacements of 3.2 mm in one direction and 4.1 mm in the other, for an average of 3.65 mm. The experimental value listed in Table 3 is 5.06 mm. It appears this may have resulted from extrapolation of the slope of the load-deflection curve at $0.75 P_y$ to P_u . If this is the case it does not correspond to the method used for theoretical calculation, nor to the common definition for experimental determination of Δ_y (which results in $\Delta_y = 3.65$ mm, as above). Ductility values for Unit 8 are apparently based on some theoretical value for Δ_y , rather than the experimental value, as is commonly the case, but scaling from Fig. 12 results in $\Delta_y = 2.05$ mm, which does not agree with Table 3, nor any of the values calculated by the writer. Similar discrepancies appear to exist for other units tested.

4. Shear Friction

The writer is unable to agree with the authors comments about shear friction. Prior to visible cracking, it is unlikely that either the vertical or horizontal reinforcement places a significant role in shear transfer, just as reinforcement in concrete beams is ineffective until the concrete cracks.

Of more importance, however, is the weight given by the authors to shear friction as an ultimate mechanism, and the elevation of code shear friction equations to status as a 'theory'. The following points are made

- (1) Shear friction is a means whereby shear can be transferred across cracks when the clamping reinforcement does not exceed yield strain. The clamping force is supposed to result in a frictional force of $\mu A_s f_y$ where $A_s f_y$ is the yield force of the clamping reinforcement. Clearly for this to occur the surfaces have to be in contact.

Where the reinforcement crossing the potential crack is at yield and crack widths are high, shear friction is ineffective. Hence the concrete design code (A4) does not consider shear friction as an effective mechanism in plastic hinge zones. At critical cracks shear is transmitted by compression-shear transfer in the

compression zone. Work by Hamid and Drysdale (A5) has recently shown that very high effective shear stresses are possible in the compression zone.

- (2) The value for μ advanced by the draft masonry code (i.e. $\mu = 0.7$) is a conservative figure obtained by code committee consensus after reducing values accepted for concrete which are also largely the result of conservative code committee consensus, rather than based on exhausted experimental data. Thus the close agreement between the authors' eqn. 6 and 7 has no theoretical significance.
- (3) Applying axial load to a wall, or column, while no doubt improving shear friction transfer (if appropriate), influences the ultimate load capacity of walls designed for flexural action according to well established principles of structural mechanics, familiar to any designer who has designed a column. To suggest that the increase in strength results from shear friction is, at best, misleading. Fig. A2 shows two walls subject to the same axial preload P_e , one wall being 5 times as high as the other, but otherwise having the same section dimensions and reinforcement ratio.

According to the authors both walls have the same increase in strength due to preload, of approximately $0.7 P_e$. In fact, clearly the squat Wall 2 will have an increase in lateral load capacity five times that for the slender Wall 1.

It is again emphasised that the flexural strength of masonry wall units, with or without 'preload' is predictable by normal flexural strength theory for reinforced concrete, modified for masonry material properties, provided adequate shear strength is ensured by normal design procedures.

References

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- A2. PRIESTLEY, M.J.N. and ELDER, D. McG., 'Seismic Behaviour of Slender Masonry Shear Walls', Department of Civil Engineering, Research Report No. 82-4, University of Canterbury, March 1982, 116 pp.
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- A5. HAMID, A.A. and DRYSDALE, R.G., 'Concrete Masonry Under Combined Shear and Compression Along the Mortar Joints', Journal ACI, Vol. 77, No. 5, Sept-Oct. 1980, pp. 312-320.

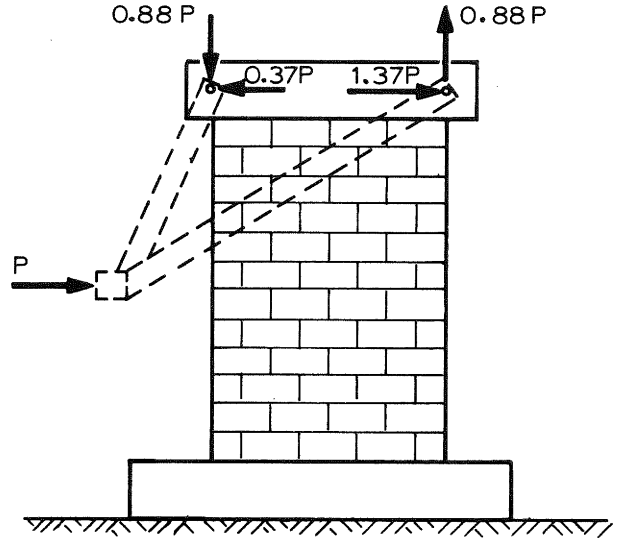


Fig. A1 APPROXIMATE APPLIED LOADS FROM AUTHOR'S TEST RIG

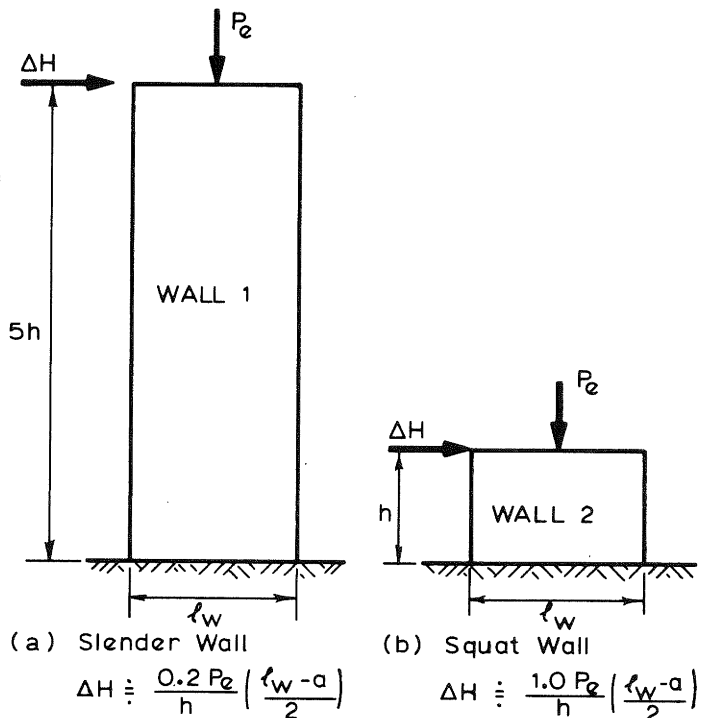


Fig. A2 INCREASE IN LATERAL LOAD CAPACITY DUE TO AXIAL LOAD

AUTHORS' REPLY TO DR PRIESTLEY'S COMMENTS

We would like to thank Dr Priestley, internationally regarded in RHM research, for his detailed comments to our paper.

His criticism of our loading frame is threefold:

In the first place, fig. A1 leads Dr Priestley to conclude that most of the applied shear is transferred to the RHM in the vicinity of the 'pin'. In fact, the axial stiffness at the top loading beam is such that a 400 kN horizontal load applied to one end would extend the beam if restrained only at the other end by less than 0.1 mm. The twin 381 x 102 channels into which the applied load is directly transferred are bonded by stiffening plates to the concrete beam subsequently poured against them and to each other by ties. The 0.1 mm beam elongation represents the maximum total width of vertical crack permitted at the top of wall by the loading system for an applied load at 400 kN. Hence the beam enhances rather than reduces the performance of the top of the wall in shear.

Dr Priestley's second criticism appears to centre around the question of inadequate flexural stiffness of the beam with the effect that the applied load is transferred directly into the RHM. The degree to which this happens can be tested by a simple analysis: We fixed the beam at one pin and applied a vertical load at the other end. The beam was supported at 200 centres on springs for which it is assumed that strain goes linearly to zero over half the wall height (i.e. from top beam to point of contraflexure). To be very conservative, we assumed that the wall was uncracked and hence spring properties were large (280 kN/mm average stiffness). The deviation from linear slope of the beam caused by the 'pull' of the RHM springs amounted to less than 0.1% or 10^{-8} mm. We therefore conclude that an essentially linear strain distribution was imposed on the wall by the applied couple and thus the test rig allows good simulation of seismic loading. The excellent agreement between theoretical and measured flexural wall strengths also provides confidence in the rig.

In the third place, most previous masonry wall tests, in particular by Priestley, ^{7,8} were loaded as a simple cantilever with a heavy top beam. The test results from these low walls were extrapolated to predict the performance of multistorey shear walls. It was thought that the top beam, (at the location of point of contraflexure in a building), may have inhibited cracks which would have propagated from the top of the wall, and thus artificially enhanced the wall strength. The main purpose of the elaborate rig used was to investigate this concern rather than merely to test piers in double curvature as suggested by Priestley. It was concluded that the simple cantilever rig is adequate

where the walls fail in flexure, but not where failure is by diagonal shear cracking. Further, earthquakes have not read our design codes, and thus even if piers have been designed elastically, many will experience significant inelastic deformations during large seismic excitation. This is our justification for post yield testing of unit 4.

Unit 2 was, in fact, reinforced substantially differently from Unit 4 as shown in Fig 3. Notwithstanding the large spacings, it is clear from comparison of these units that the presence of horizontal reinforcement between floors serves to inhibit the development of large diagonal cracks.

We agree that the measured value of masonry modulus of elasticity E_m (6.6 GPa) is low for the reasons suggested by Priestley. This was discussed in a correction for Ref 11 circulated on 10 March 1982. However, for Unit 8 for example, doubling E_m only reduces the theoretical yield deflection by 7% and increases the theoretical yield and ultimate moments by 4.6% and 0% respectively. Calculations for theoretical loads and deflections are shown in Appendix B of Ref 11. With the exception of Unit 9, the theoretical yield deflections in Table 3 were based on the yield rather than ultimate load for the cracked section, and include shear deflections. Thus for Unit 8 the yield deflection at ultimate load =

$$\Delta y \cdot P_u/P_y = 2.45 \times 222.2/151.7 = 3.59 \text{ mm}$$

which is in agreement with Priestley's calculated 3.76 mm, as Priestley used the mid-cell rather than measured steel locations to calculate I . The $DF = 1$ deflections in the graphs were based on the theoretical deflections of the units at first yield due to flexure alone rather than experimental values because, as most units were virtually uncracked during the cycles to $3/4$ yield load, the $DF = 1$ deflections could not be extrapolated. This answers all of Priestley's Section 3 questions.

We did not intend to present shear friction as an acceptable ultimate mechanism, but merely played devil's advocate by showing that the good agreement between experimental and theoretical wall strengths could equally be predicted by code shear friction equations. Clearly if all bars have yielded sufficiently to hold a crack open wide enough across the entire wall width so that no aggregate interlock can occur then the shear must be resisted by steel dowel action. However for practical deformations the cracks will not be this wide and as discussed for Unit 7 shear friction theory provided a better agreement with experimental results than dowel action equations. Further, with Unit 4 which has no horizontal steel, because dowel action only provides a shear strength of 47 kN which was less than half the measured wall strength, shear friction must have played a significant role.