DISCUSSION SECTION

"SEISMIC RESISTANCE OF BRICK MASONRY WALLS"

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Bulletin of the N.Z.N.S.E.E., Vol. 7, No. 4, December, 1974

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The authors have made a worthy contribution in this paper :-

i) by providing the cyclic load-deflection relationships for both RBM construction and the larger size of hollow-cell brick masonry,

ii) by clarifying the common misconception regarding the relative effectiveness of horizontal and vertical steel in resisting shear (qualified below for inelastic cyclic conditions) and demonstrating that high average shear stresses can be sustained by adequately reinforced brick cantilever walls, and

iii) in showing that effective confinement to the crushing toe zone of a masonry panel is indeed not only possible but also a viable proposition.

However, several statements warrant comment.

1. The authors make continual reference to "non-standard conditions at the base of walls," in the Canterbury tests by Williams and Scrivener. It is true that the steel base condition was not typical but with such varying practice in the field, what is "Standard"? Potential variation within the concrete base allows for poorer reproducibility of important boundary conditions. Nevertheless, accepting the existence of this "non-standard" condition, the influence of the angle shear connectors (25 mm high) would have been confined to the lowest masonry course. Their existence was solely to prevent premature shear failure by sliding between the lower mortar bed and steel plate. If this was the natural failure mode then it would be suppressed in this course but would prevail in the course above. In fact failure by vertical tensile splitting (crushing) often occurred in the second course. Thus the test panel may be regarded as having one course less and mounted on a one unit high masonry base as was explained in the more thorough report of the work. (1) This writer believes that concern for the "non-standard conditions at the base of the walls" is unfounded and that the condition as adopted had very little effect on the masonry wall behaviour.

2. In the last paragraph of Section 7 it was suggested that differences in behaviour between the PACRA and Canterbury tests could be attributable to the above described base condition. However, this reader would rather suspect that two different types of construction are being compared and that a general lack of structural integrity in the RBM due to bond failure between the outer brittle unreinforced skins and the reinforced grout curtain could account for the greater severity of load degradation in the PACRA walls compared to the filled hollow-cell reinforcible masonry construction of the Canterbury tests.

This leads to the question of whether the confining plates are as effective in the hollow-cell panels as in the RBM walls? Few details are given for the test specimens CBU-1 and CBU-2 and it is difficult to determine the relative improvement achieved. The plates are obviously very effective in improving the cyclic behaviour for RBM but maybe they only provide the inherent lack of tie across the two unreinforced masonry skins, so essential for satisfactory inelastic cyclic behaviour as experienced in the crushing zone. Load-deflection cycles for specimens CBU-3 and CBU-4 are very similar to those obtained in the Canterbury tests.

3. The situation without applied bearing load may not necessarily be the most critical case. Although it "produces conditions where minimum flexural and shear strength result" under monotonic loading, a number of investigators have demonstrated that high bearing loads are associated with greater element deterioration and reduced ductility during cyclic loading.

4. While acknowledging that results from the Canterbury tests are too few to be conclusive, the results do suggest that contrary to expected behaviour or similar behaviour in reinforced concrete, testing at a higher frequency may have produced different results. This same phenomenon has been experienced by Mayes and Clough, (2) in comparing equivalent reinforced concrete masonry pier specimens tested under similar pseudo static (0.02 Hz) and dynamic (3 Hz) loading cycles.

5. The paper should not leave the impression that shear reinforcing should be concentrated in the horizontal direction at the expense of vertical steel as this could lead to undesirable results, in the writer's opinion. The objective for good design is to provide sufficient shear reinforcing to ensure a primary flexural failure prevails which results in horizontal cracking along the base courses. The upper limit on the shear

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capacity of a cantilever wall after initial post elastic cyclic deformation is governed by shear transfer in the reaction corner, by dowel action of the reinforcing bars and aggregate interlock along the base section, provided the element has been adequately reinforced for shear carrying functions. Subsequent to the first full cycle the resistance to shear along the base is reduced to the stage where sliding shear, analogous to that in reinforced concrete plastic hinge zones (3), may become the predominant response mechanism as noted in the paper. "If full depth cracks exist the shear force will be carried mainly by dowel action of the reinforcement and by greatly diminished aggregate interlock shear." (The latter is generally less effective in masonry than in reinforced concrete). "If the shear force to be transferred ... is very large ... this may lead to failure by sliding shear along a continuous crack across the critical section ...." (3). In such a case vertical shear reinforcement will be beneficial in increasing shear resistance by the dowel action mode. Of course it will increase the flexural strength and a balance is required. But as it would be unwise to precipitate failure by sliding shear this possibility should not be overlooked.

It would be interesting to see the second and subsequent cycles of walls F11 and F12 to determine the effectiveness of horizontal reinforcing for cyclic shear resistance.

The above comments reflect a variation in interpretation on several aspects of this work but do not question the validity of its major findings as stated initially in this discussion.

REFERENCES

1. Williams, D., "Seismic Behaviour of Reinforced Masonry Shear Walls", Ph.D. Thesis, University of Canterbury, Christchurch, New Zealand, 1971.
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3. Park, R. and Paulay, T., "Ductile Reinforced Concrete Frames - Some Comments on the Special Provisions for Seismic Design of ACI 318-71 and on Capacity Design," Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 8, No. 1, March, 1975.

AUTHORS' CLOSURE TO DISCUSSION:

The Authors wish to thank Dr. Williams for his discussion, and feel that his comments deserve reply. Dealing with his paragraphs in order:

1. Although considerable variation may occur in the design of reinforced concrete foundation beams the critical condition appears to be the degree of restraint of lateral expansion at the heel and toe of the wall. This will depend on the material properties in these areas rather than the structural effects of size and reinforcing content. The high modulus of elasticity of the steel base in Williams' test is such that it would restrain lateral expansions of the most critical mortar course in the same fashion as a very thick confining plate. Consequently, as evidenced by Williams' tests, tensile splitting failure cannot initiate at the critical course. The authors believe that conditions at the next course can never duplicate the conditions at the base, and therefore Williams panels cannot be considered as being equivalent to panels having one course less, for the following reasons. The maximum moment sustained by the panel was dictated by the conditions at the base course. Moments at the next course up were restricted to a maximum of about 93% of ultimate as shown in Fig. D1, (a) and (b). With the inevitable spread in the length of the compression zone with height above the base, peak compression stresses at the first course would be substantially lower than at the base course. Splitting failure would be inhibited until strain-hardening of the tension steel was sufficient to increase the moment at the first course to that which would normally induce failure at the base course. As illustrated in Fig. D1 (c) this implies a substantial increase in displacement, and therefore ductility, at failure in Williams' tests.

The authors also disagree with the contention by Dr. Williams that the angle shear connectors would have no influence on the sliding shear failure mode. The fundamental flexural crack inevitably forms at the base course due to poor bond, and results in a very smooth crack surface. This crack generally relieves any tendency for wide crack opening at the next few courses, and consequently sliding shear resistance on the bottom course is a fraction of that at higher courses. In the event that a wide crack did develop at the first or second course, the crack surface would be more irregular than the basecrack, due to the random crack path through the grout cores. In this case some aggregate interlock could be expected, with increased sliding shear resistance. This behaviour was very noticeable in the authors' testing with virtually no sliding occurring at any course above the base.

2. The authors tested both RBM and filled hollow-cell units. Bond between the grout core and bricks of the RBM construction was in fact better than obtained for the filled hollow cell units, where adhesion between the face shells and grout was poor. Face shells in filled hollow cell construction tended to shear-off under sliding shear.

Units CBU-1 and CBU-2 suffered severe structural degradation on load reversal. Behaviour was significantly improved by the inclusion of confining plates in units CBU-3 and CBU-4. In these units structural integrity was maintained at the toe and heel until very large ductilities were obtained. Unlike comparable tests referred to by Dr. Williams, no load degradation occurred, as only small increases in deflection were required to re-establish the ultimate moment capacity.

3. Dr. Williams is correct in stating that high bearing loads may result in reduced ductility during cyclic loading. However, the ultimate moment capacity is dramatically increased, and thus for a given base shear, ductility requirements are reduced.

4. The authors are in agreement with Dr. Williams that additional research is required to establish the significance of frequency of cycling on the behaviour of masonry shear units.

5. Dr. Williams reasoning appears at fault in this section. If (say) 30% additional vertical reinforcing is provided to increase the sliding shear resistance by 30%, then the ultimate moment capacity, and hence the shear force at ultimate moment is also increased by very close to 30%. This is due to the inevitably small length of the compression block at ultimate in comparison to the wall length. Thus the ratio of shear force to available dowel strength is the same as before adding the vertical steel, assuming that the added vertical bars have the same diameter as the original flexural steel. The end result would be an increase in both vertical and horizontal steel (the latter to carry the higher shear force) without increasing the competence of the wall. The authors thus fail to understand how the 'balance' between flexural strength and shear resistance referred to by Dr. Williams can be obtained.

Finally, Walls F11 and F12, as mentioned in the paper suffered flexural, not shear, failures. Classical toe collapse accompanied by severe load degradation occurred at each end under load reversal without the development of open shear cracks. For a more appropriate example of the effectiveness of horizontal reinforcing for cyclic shear resistance, Dr. Williams is referred to the results for Walls F13 and F14 which were both subjected to higher shear stresses than either of walls F11 and F12.

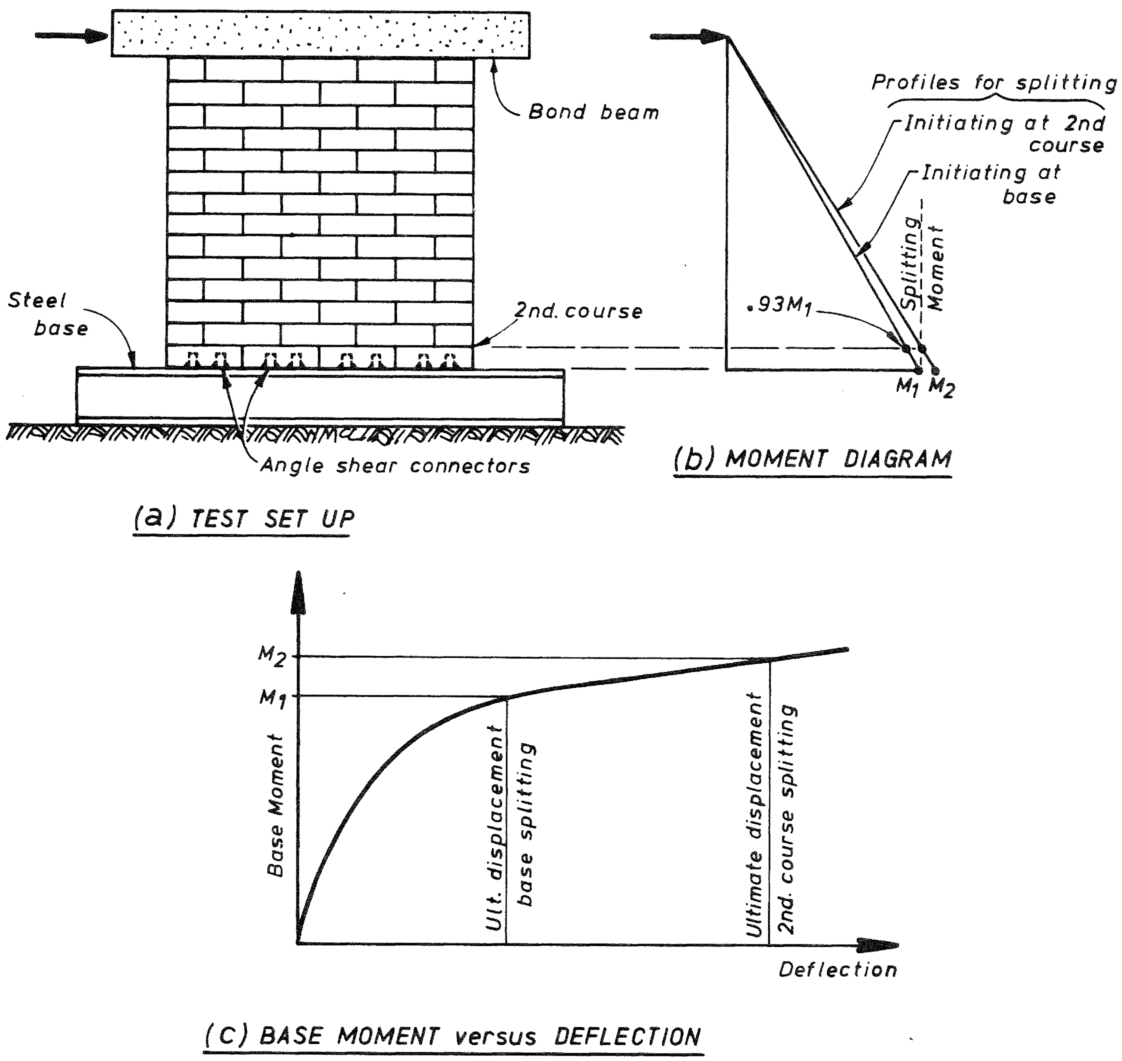

FIGURE D1: MECHANISM OF DUCTILITY ENHANCEMENT BY THE NON-STANDARD BASE CONDITION IN WILLIAMS' TESTS.