DISCUSSION

"SEISMIC RESISTANCE OF REINFORCED CONCRETE MASONRY SHEAR WALLS WITH HIGH STEEL PERCENTAGES" - M.J.N. Priestley

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Dr Priestley's tests have shown that reinforced masonry cantilever walls, with high percentages of reinforcement distributed at close centres, soundly constructed without significant voids and deficiences in the region of the construction joint, can take high shear loads and may be expected to dissipate significant amounts of seismic energy.

Insofar as it has been the policy in the writer's office for many years to require closely spaced and uniformly distributed reinforcement in two directions, the performance of the walls is gratifying indeed. No issue is taken with the theoretical considerations in Dr Priestley's paper but concern is expressed at the statement that current under-capacity factors and shear stresses allowed are both too low. The problem is not one of theory but of practice.

The panels in the test were all constructed in the yards of a masonry company and even though there may have been no conscious effort to employ more than normal care, the small size of the panels made construction far easier than under real, typical site conditions.

Soundness in reinforced hollow masonry is far more difficult to achieve than with most other structural materials including reinforced concrete and lack of soundness even more difficult to discover. Neither the present, let alone Dr Priestley's proposed under-capacity factors for the design of reinforced hollow masonry, realistically represent the effect of the variability of the completed construction, nor are there adequate statistics available to quantify them. Most practising engineers, however, have intuitively known for many years the difficulties which prevent contractors in duplicating the construction quality in the panels tested by Dr Priestley.

As the prevention of premature sliding shear failure is so important, the writer initiated a programme of randomly coring various structures around the country at the wall base construction joint to evaluate their condition when built under normal site conditions.

The results and analysis of this investigation will be offered for publication in this magazine in the near future.

The specifications for the construction evaluated were in line with the latest SANZ draft, DZ 4210, Masonry Buildings material and workmanship code, which follows long time Ministry of Works and Development practice: - the use of cleanouts in high lift construction, fluid but well designed grouts with checks on spread values, the vibration and subsequent revibration of grout. On observing some of the cores taken, a prominent firm of consultants also carried out investigations of a similar nature.

At this stage it should be pointed out that the results have been very discouraging and that a significant proportion of the cores revealed serious deficiencies in the base joint region. These included voids, inadequate or complete lack of joint preparation, mortar droppings in a considerable quantity preventing grout adhesion, the presence of sand placed to facilitate removal of mortar droppings but not subsequently removed and preventing grout bond, etc.

A programme of controlled construction is now being undertaken by one of the Ministry of Works and Development district offices to evaluate the difficulties in achieving sound construction particularly at this joint region. It is important that the construction standard that can be achieved consistently with reasonable normal site methods and supervision procedures be determined and design parameters be related to it.

Unless practical construction and supervision techniques can be shown to achieve a standard comparable to that in Dr Priestley's walls, and with a reasonably high degree of reliability, and until the effect of deficiencies in the construction joint are better understood, it would be imprudent to use design stresses anywhere near as high as proposed.

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The author wishes to thank Mr. Glogau for his words of caution about the translation of results from a controlled test programme into the less ideal situation of actual constuction practice. Mr. Glogau disagrees with recommendations in the paper to (a) increase the flexural undercapacity factor from 0.65 to 0.85 and to (b) increase the maximum allowable design shear stress from 0.62 MPa to 1.25 MPa, with higher values permitted provided expected structure ductility is reduced by designing to higher lateral load coefficients.

The following comments are offered:

1. Mr. Glogau asserts that the panels were unrealistically 'sound' due to small panel size, despite attempts to use no more than normal care. It should be noted that the walls were laid, not by a qualified block-layer, but by a general tradesman. It should further be noted that since the walls were nominal 150 mm rather than the more common 200 mm widths the extremely high steel percentages adopted made grouting much more difficult than would normally be the case. General construction practice deliberately did not conform to the high standards required by the MWD. Despite these factors, as pointed out in the paper, all walls substantially exceeded the theoretical flexural capacity, based on measured material properties. If the recommended under-capacity factor of 0.85 was adopted, the minimum excess of actual strength over design strength would have been 53%. These results indicate a certain amount of leeway to take account of any artificial strength due to unforseen 'soundness' of the test panels.

2. The author is not aware of any experimental evidence to support Mr. Glogau's statement that "... the present undercapacity factors (do not) .. realistically represent the effects of the variability of the completed construction". It appears that Mr. Glogau is advocating a reduction in the flexural undercapacity factor. It is, however, accepted that the quality of masonry constrution is very variable, and the author would not advocate the use of $\phi = 0.85$ for inadequately supervised, or unsupervised construction. A current masonry test programme at the University of Canterbury is investigating, amongst other variables, the influence of workmanship on strength and ductility, and it is intended to offer the results, in the form of a paper, to the Bulletin in the near future.

3. As mentioned in the paper, and agreed by Mr. Glogau, the critical region for shear is the base-joint area. Shear within the wall can be adequately controlled by properly designed horizontal steel. It is important to realise that during inelastic cycling, a wide crack will occur along the base course, regardless of the extent of joint preparation, and base slip will be resisted solely by dowel action until sufficient lateral load has been developed to cause the compression end of the base crack to close. This mechanism will not be improved by limiting the shear force. Consider two geometrically identical walls, height H and base length ℓ, the second designed to carry twice the shear load of the first. The amount of distributed vertical steel required is as follows:

Wall 1: Shear $V_1 = V$, Moment $M = V.H$

$$\text{Vert. steel} \quad As_1 \doteqdot \frac{2VH}{\ell . f_y} \qquad (1a)$$

Wall 2: Shear $V_2 = 2V$, $As_2 \doteqdot \dfrac{4VH}{\ell . f_y}$ (1b)

Dowel Resistance will be proportional to the number of vertical bars times the section modulus of the vertical bars

i.e. $F_{D1} \propto N_1 \, d_{b1}^{\ 3}$ (2a)

$$F_{D2} \propto N_2 \, d_{b2}^{\ 3} \qquad (2b)$$

where d_{b1}, d_{b2} are the diameter of the vertical steel.

There are two extremes for providing the extra vertical steel required for wall 2.

(a) Use the same size bars, i.e. $d_{b1} = d_{b2}$.

Then $N_2 = 2N_1$

Clearly from equations (1) and (2),

$$\frac{F_{D1}}{V_1} = \frac{F_{D2}}{V_2}$$

The ratio of dowel strength to flexural strength is constant, and it seems reasonable to expect the two walls to perform equally well (or equally poorly, as the case may be) in limiting base slip.

(b) Use the same number of bars, i.e. $N_1 = N_2$

Then $d_{b2} = \sqrt{2}\, d_{b1}$

From equations (1) and (2)

$$\frac{F_{D2}}{V_2} = \sqrt{2}\,\frac{F_{D1}}{V_1}$$

In this case relatively better resistance to base slip is developed by the more heavily loaded wall.

4. It is suggested that as engineers we expect, and frequently get, poorer quality workmanship for masonry structures than would be tolerated for (say) reinforced concrete construction. The doubts expressed by Mr. Glogau must remain unless the average standards of construction can be improved. It is unfortunate, however, that the high quality work required by some designers and obtained regularly from some competent contractors should be subject to limitations based on sub-standard construction by other contractors.

"SLENDERNESS EFFECTS IN EARTHQUAKE RESISTING FRAMES" - A. L. Andrews

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Mr. Andrews has presented a novel means by which energy principles may be used to assess the degree to which deflection control limits P-δ effects in frames. Mr. Andrews concludes that, although zone A is satisfactory, some tightening of control is needed for zones B and C. However, it is not clear that zone A is satisfactory because these conclusions are based on his equation (15) which is itself based on a "top-heavy" triangular distribution

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