DISCUSSION

"DESIGN OF SHEAR WALLS FOR SEISMIC RESISTANCE" - I. C. Armstrong

Bulletin of N.Z. National Society for Earthquake Engineering, Vol. 10, No. 1, March, 1977.

R. L. WILLIAMS*

Mr. Armstrong has presented a useful and interesting paper and, as the author suggests, research is needed into low walls at capacity in order to understand the load mechanism, cracking patterns and damping factors.

Mr. Armstrong illustrates his example of shear in low walls by dividing his wall into elements separated by 45° cracks and then considers the forces across the crack which appear to be mainly the nominal wall steel.

My own philosophy in the detailing of low shear walls is to ensure that the area at and adjacent to the top of the wall contains sufficient reinforcement to transfer all applied code loads back to the leading edge of the wall and to tie the elements together at that level. Arch action will then carry these forces down to the base.

I would consider that the distribution steel (nominal .0025 vertical and horizontal) is mainly to control any sudden loss of strength from cracking. Damping would be principally elastic together with yielding of steel across, hopefully, well distributed fine cracks in the wall. Particular attention would have to be paid to trimming of openings and, where possible, diagonal trimmers at the corners should be used to minimise large local cracks.

In square or intermediate walls, Mr. Armstrong suggests that one should consider capacity moments and then design the shear reinforcement for the shear developed without the contribution of the concrete. This seems to me to be unreasonably conservative. If this standard of design is being used, surely the wall would qualify as Class 4 or 5 (S = 1 or 1.2). The design code load for Class 6 (S = 1.6) is already 60% greater than for most ductile walls and the more usual practice of designing the wall shear reinforcement at code levels and ignoring the contribution of the concrete would provide a substantial shear reserve for loads above this relatively high code level. I would, however, suggest that as very high trusts can be developed at the toe of such walls and that under cyclic capacity loading progressive deterioration of this area may result, it would be desirable to confine say the first four bars of the wall as for a column for a short distance above the base. This detailing should minimise buckling of this steel under cyclic loading to yield and enhance the compressive ductility of the wall thus minimising damage.

In the section under coupled shear walls, Mr. Armstrong suggests that "the designer should aim for full yield capacity to be reached first in all coupling beams". While I fully support this view, early yielding of the tension wall hinge may be hard to avoid due to planning constraints which depend almost entirely on the geometry of the walls and coupling beams, I suggest that some early yielding may be acceptable as the rotation of this hinge is controlled by the capacity of the whole structure which is principally in the coupling beams. This is a much more favourable situation than in, say, cantilever walls where the rotation of the base hinge is controlled by its own strength which will not increase greatly with further rotation. I therefore suggest that where the walls are detailed for ductility then some yielding of the tension wall base hinge may be acceptable prior to all coupling beams reaching full yield capacity providing the ductility demand on the wall is only small at the time that capacity is reached.

I. C. ARMSTRONG

Mr. Williams briefly discusses his approach to detailing low shear walls. Clause 3.3.4.3 of the code applies to these walls, which should be suitably detailed to ensure that under earthquake attack a distributed system of controlled cracking will form in the web, so that significant amounts of seismic energy are dissipated in the shear mode and premature shear failure (e.g., by uncontrolled opening of a single critical diagonal crack) is precluded. Compliance with the code here requires specific design of web reinforcement in proportions adequate to control seismic performance in the shear mode, as indicated in the paper, and not simply a nominal provision of distribution steel at the 0.25% level (i.e., minimum web reinforcement) as Mr. Williams advocates. It should be noted that, far from being principally elastic behaviour, walls designed to S = 1.6 may reach their elastic limit when subject to approximately one-quarter El Centro 1940 N-S ground accelerations, and to survive El Centro-type motions with 0.33g ground accelerations they should continue to resist reversals at an overall displacement ductility factor of between 2 and 3 depending on seismic zone.

Experimented studies of flanged low

* Ministry of Works and Development, Hamilton.

shear walls by Barda[7,8] appear to confirm the author's approach. Under horizontal load reversals no yielding was observed in the vertical flange steel, while distributed vertical web reinforcement reached yield at loadings near the ultimate shear strength of the walls and was effective in contributing to the level of shear resistance sustained, principally by maintaining equilibrium of web strut elements by resisting overturning (Element 2 in the paper). Barda found that horizontal and vertical web reinforcement were effective in producing a well-distributed crack pattern and in controlling crack widths. Parallel cracks at inclinations between 37 and 43 degrees formed a series of web struts which sustained very high compressive forces (equilibrated by adequately designed vertical web bars) in transferring shear through the web to the base.

In reversals of horizontal loading increased gradually up to the ultimate shear strength of the wall, a proportion of the total shear appears to have been resisted by the contribution of the concrete. After reaching ultimate, the load-carrying capacity in shear decreased during subsequent reversals, presumably due to progressive degradation of concrete mechanisms (e.g., aggregate interlock). This decrease may have been relatively rapid in some test walls but little information has been published on this phase of wall performance. When fully developed, web struts depend upon the overturning resistance provided by distributed vertical web reinforcement for their continued effectiveness in transmitting shear forces. Therefore, this author considers that until further test information is available, distributed web reinforcement should be provided to resist the full code specified seismic loading without reliance upon the contribution of the concrete.

The intermediate group of category 6 shear walls covering the range $/ \leq h_w/l_w < 2$ includes a variety of practical wall cases, e.g., with or without substantial flanges or significant axial loads, and consequently the mode of earthquake resistance may vary considerably. Either a shear mode (i.e., diagonal web cracking) or ductile flexural yielding of tension reinforcement may dominate behaviour, or both mechanisms may combine to share in seismic energy dissipation. Since actual behaviour is difficult to predict reliably, the code requirement that all these walls be detailed to adequately control diagonal web cracking at code lateral loads corresponding to S = 1.6 should ensure acceptable performance of the wall in respect of contribution of the shear mode.

In some intermediate shear walls it appears likely that, notwithstanding extensive controlled web cracking, the principal energy dissipating mechanism may be that of yielding of flange reinforcement near the base. Since this behaviour would be very similar to that of tall ductile shear walls, a correspondingly similar capacity design approach is suggested so as to ensure that the principal energy dissipating mechanisms should be maintained in a stable manner and are not precluded by non-ductile failure mechanisms. For example, in a wall of rectangular section with low axial load where designed cantilever moment and web shear steel have comparable resistance, the contribution of concrete may enhance diagonal web shear resistance and cause the base section to hinge at flexural capacity. However this in turn could result in early failure due to sliding shear, a situation which could be predicted and corrected if capacity checks were made during design.

Intermediate wall cases where such ductile hinging is likely to be attainable may be relatively few. Capacity design checks referred to by Mr. Williams are suggested only for these cases, and such checks should involve only a minor amount (if any) of additional horizontal web reinforcement in the base area. The standard of design remains at the level required by the code (S = 1.6), and hence concern regarding undue conservatism should be removed.

The passage relating to coupled shear walls quoted by Mr. Williams is not a suggestion by the author, but reflects a particular code requirement (Cl. 3.3.4.1 and commentary). It is part of a discussion on code intent based on the commentary and reference (3). Mr. Williams' contribution, contrary to the present code requirement, is a discussion on the code itself and is understood to be the subject of recent analytical studies by the University of Canterbury.

From the particular stand-point of the code, using available background information, the seminar paper attempted to elucidate some design aspects of the code provisions and intent. More detailed guidelines on strength design and capacity design of shear walls have yet to be formulated to resolve areas of uncertainty. The author thanks Mr. Williams for his contribution and is appreciative of the opportunity to clarify aspects of the paper including tests on low shear walls.

ADDITIONAL REFERENCE

8. Barda, F., Hanson, J. M. and Corley, W. G., "An Investigation of the Design and Repair of Low-Rise Shear Walls." Fifth World Conference on Earthquake Engineering (Session 3A), Rome, 1973.

"MOMENT REDISTRIBUTION IN CONTINUOUS BEAMS OF EARTHQUAKE RESISTANT MULTISTOREY REINFORCED CONCRETE FRAMES" - T. Paulay

Bulletin of N.Z. National Society for Earthquake Engineering, Vol. 9, No. 4, December, 1976.

R. L. WILLIAMS

Professor Paulay has made an interesting and worthwhile contribution to the art of moment redistribution, a subject of increasing importance in ductile reinforced concrete moment resisting frames where equalisation of top and bottom steel in the beam at plastic hinges has been shown to significantly improve their performance under cyclic loading.

Professor Paulay however omits to show the next logical step of capacity design. That is to adjust his diagram so that it