DISCUSSION SECTION

"CAPACITY DESIGN OF REINFORCED CONCRETE FRAMES FOR DUCTILE EARTHQUAKE PERFORMANCE

By I.C. Armstrong

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T. PAULAY:

Mr. Armstrong presented a timely paper which was also followed with considerable interest during the seminar held at the University of Auckland in August 1972. It draws attention to many aspects of earthquake resistant design, only very recently recognised, and hence not incorporated even in the more progressive overseas codes.

Little factual knowledge exists in relation to several issues raised in the paper and it is understandable that the author, in his design recommendations, approached these with considerable conservatism. Many outstanding problems mentioned are high priority items of the research programs planned by Professor Park and the writer at the University of Canterbury. Unfortunately, only a limited amount of experimental work can be undertaken because of the lack of a large capacity testing machine in New Zealand. In a recent pilot project[1] many aspects of the concurrent response of frames to seismic excitation along their two principal axes, to which Mr. Armstrong paid particular attention, has been studied. It is intended to submit some of the findings of this study for possible publication in future issues of this Bulletin.

At this stage the writer would like to discuss a few issues related only to the design of beams and columns for shear.

Mr. Armstrong states that some uncertainty exists as to the magnitude of the maximum earthquake induced shear across beams and columns, such uncertainty being greater where members have small span to depth ratios. Therefore he recommends that, until test evidence becomes available, the calculation of induced shears should contain a margin. He recommends that shears, associated with plastic hinging, be obtained from calculated hinge capacity moments using a reduced member length (L-d), i.e. the clear span L reduced by 0.5d for each plastic hinge existing in the member, as shown in Fig. 4 and Fig. 7 of the paper.

In fact there is no uncertainty of what the maximum possible shear could be. The maximum hinge moment capacities, with due allowance for possible overstrength, can be uniquely determined at the face of the column. For the type of loading shown in Fig. 4, these hinge moment intensities cannot be exceeded anywhere along the clear span without violating the laws of statics. Consequently the moment induced shear must be based on the clear span so that the appropriate equation of Fig. 4 becomes:

$$V = \frac{M_1^{pc} + M_2^{nc}}{L} + 1.05\ V_D + 1.27\ V_{LR}$$

The length of the plastic hinge, a debatable quantity, does not enter the issue at all! For a short and relatively deep member, for which Mr. Armstrong suggests a possibly larger degree of uncertainty, his proposition would result in an overestimation of the moment induced shear force by

50%	if	L/d	=	3.0
100%	if	L/d	=	2.0
200%	if	L/d	=	1.5
infinity	if	L/d	=	1.0

It is emphasised that beam hinge moments, with an absolute maximum value at column faces, and the corresponding constant beam shear are related to each other by the laws of statics only. Thus there can be no uncertainty. A fact borne out also by experimental evidence[2].

The non-ductile nature of a shear (diagonal tension) failure does warrant precautions, such as suggested by Mr. Armstrong, to ensure that such failures will not occur. The current approach to shear strength, as embodied in the 1971 ACI Building Code, subscribes to this philosophy and consequently is conservative enough. In the light of experimental evidence additional provisions for shear strength margins do not seem to be warranted. Four points, not specifically mentioned in codes but related to seismic performance in shear, are relevant to propositions of the paper.

(1) The contribution of the concrete towards shear resistance, v_c, can always be relied upon in those regions of a member where the stress in the flexural reinforcement can not approach yield level, i.e. between plastic hinges. This also applies to cyclic loading.

(2) As a corollary to the above, the maximum possible induced shear forces across plastic hinges must be resisted entirely by web reinforcement. The web steel must operate below yield strength level. The state of the cover, i.e. spalling, is not likely to affect the shear strength of plastic hinges.

(3) The nominal shear stress, v_u, across a plastic hinge should not approach the maximum values permitted by codes, such as $v_{u,max} = 10\sqrt{f'_c}$ psi, when using stirrups only. It is to be remembered that between the last stirrup in a beam and the fac of the column nearly the whole of the shear will have to be transmitted by the concrete in the compression

zone. Under reversed cyclic loading a sliding shear failure can occur if the nominal shear stress is too high. Even a drastic increase in hinge hooping is not likely to be able to prevent sliding.

(4) Axial compression or tension, in the case of columns, does not adversely affect the orientation of diagonal cracks and hence the resisting mechanism of the web reinforcement.

REFERENCES

(1) Row, D.G., "The Effects of Skew Seismic Response on Reinforced Concrete Frames", M.E. Report, University of Canterbury, Christchurch, Feb. 1973, p.113.
(2) Paulay, T., "Coupling Beams of Reinforced Concrete Shear Walls", Proceedings ASCE, Vol. 97, No.ST3, March 1971, pp.843-862.

AUTHOR'S REPLY:

Dr. Paulay appears to agree with the author that shear failures should not occur and precautions are required. Any difference seems to be a matter of degree.

There can of course be no difference of opinion with regard to statics as applied to a particular model. The analyst may prescribe his theoretical model completely and state the structural actions to be taken into account. He can then proceed to an exact solution with the degree of confidence so evident in Dr. Paulay's discussion. However, the selection of a fully representative model of the real structure, subject to the maximum forces associated with large inelastic lateral displacements, is far from certain. A responsible designer, realizing that prevention of non-ductile shear failure is critical, must, unlike the theoretician, bring considerable conservation to his shear design. Otherwise, he risks the possibility that his building and its occupants may eventually become the subject of a "full-scale test" to destruction. His concern is not lessened by the scarcity of applicable test results or by the fact that many vital parameters have not yet been assessed and most of the test results have been obtained from relatively small scale models.

A few of the sources of uncertainty which necessitate adequate margins for shear design are:

(a) The assignment of values to beam and column hinge capacity moments and possible axial loads. Notwithstanding design allowances, significant overstrength may still exist in the real structure.

(b) Effects which may exceed the allowances made in the design - e.g. vertical earthquake (including magnifications).

(c) Effects not usually considered in analysis or design - e.g. $P - \Delta$ effect in columns. (This is understood to be causing concern overseas and appears to increase as stiffness degrades during hinging. Its significance is not yet clear. Tentatively at this stage, the Author thinks that one effect is that in columns under axial tension an additional shear equal to $P\Delta/h$ is imposed. Thus it appears that Dr. Paulay's maximum possible shear might not be so certain after all.)

(d) The status of present methods for predicting shear capacity of reinforced concrete members - e.g. the almost complete absence of experimental evidence on the ability of rectangular columns to resist skew shears and also on the shear strength of circular columns.

If margins are thus accepted as being necessary, they can be provided in various ways, any one method being perhaps better suited to compensate for some sources of uncertainty and less suited for others. The author considers the use of a reduced span length particularly useful for column shear design where greater uncertainties necessitate increased margin, and this method is also generally suitable for beam shear design. However, for beams, as stated in the Author's address at the University of Auckland Seminar, a percentage margin (say 15% to 20%) would serve equally well. If other ways for providing margins are proposed they would be considered on their merits.

Dr. Paulay appears to have misunderstood the extent to which the reduced span length is applied in providing margins for shear design. Firstly, for column axial load and moment design the full clear beam span (not the reduced span) is used to obtain column axial force. Secondly, the paper applies to frames having members of normal cross-section proportions, not deep members. This should remove Dr. Paulay's concern related to deep membered frames.

The subject of capacity design is in its infancy and as more information becomes available design procedures will continue to develop, and margins may possibly reduce if appropriate. It is hoped that the Universities will guide designers and in due course formulate recommendations covering areas of concern. The Author thanks Dr. Paulay for his perceptive discussion, and is glad of this opportunity to clarify aspects of the paper.