DISCUSSION

"P-DELTA EFFECTS IN MEDIUM HEIGHT MOMENT RESISTING STEEL FRAMES UNDER SEISMIC LOADING"

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This paper describes a series of structural steel moment resisting frames, which were designed for the dual purpose of supporting gravity loads and providing seismic resistance. A structural ductility factor of 3 was used to determine the design earthquake actions. To assess the seismic performance series of numerical integration time history analyses were made using five different ground motions. One of the conclusions of this study was that the effects of P-delta actions were small for "design level" earthquakes and that limiting the interstorey drift limits so they do not exceed those proposed in the 1986 draft loadings code provides adequate protection against these effects. For a number of reasons the writer believes that this conclusion needs to be qualified.

From the report on which this paper was based [13] it is clear that the equivalent static method of analysis was used to determine seismic actions. Several 6, 12 and 18 storey frames were considered. Fundamental periods of vibration ranged from 1.14 seconds for the stiffest 6 storey frame, to 3.67 seconds for the most flexible 18 storey frame. In the draft code (DZ4203-86), [10] to which these frames were designed, a form of modal analysis is required for all structures which have a fundamental period in excess of 1.0 second. A subsequent draft of the code (DZ4203-91) increased the limit to 2 seconds. A comparison of different methods of analysis indicates that the overturning moment obtained with the equivalent static method is typically 1.25 times the corresponding value found with the modal response spectrum approach. Consequently the choice of the equivalent static method meant that the frames were conservatively designed compared with the minimum requirements of the draft code.

From the report [13] it appears that the required strength of the beams was found without allowing for any significant redistribution of the bending moments. As illustrated in Fig. A this approach can lead to the lateral strength of the structure being considerably greater than that required by the loadings code. In Fig. A (a) the initial gravity bending moments acting on a beam are shown. In (b) the design seismic actions are illustrated. The design seismic shear, V_e, is equal to the sum of bending moments acting on the beam at the column faces divided by the clear span. In (c) the gravity and seismic actions are combined with the latter values being increased until the first plastic hinge forms. At this stage the seismic shear in the beam just exceeds the design action value, V_e. However, with prismatic beams a considerable reserve in strength exists as the bending moments on the right hand side of the beam are well below the plastic strength. By the natural process of redistribution as the seismic shears are increased the bending moments can change to these shown in (d) and (f). Two different cases are possible. With light gravity loads the second hinge forms at the column face, as illustrated in case 1 in (d), while with a higher gravity loading the second hinge forms in the span, as shown for case 2 in (f). At this stage considering case 1 the seismic shear in the beam has increased to V_e', which is equal to twice the plastic flexural strength of the beam divided by the clear span. When the seismic actions are removed, as shown in (e), it can be seen that the redistribution involved in step (d) has changed the distribution of the gravity bending moments. If lateral seismic actions are again increased in either direction both plastic hinges form simultaneously when the beam seismic shear reaches $\pm V_e'$. The lateral displacement versus seismic beam shear, which is proportional to the storey shear, is shown in (g). As illustrated the first inelastic excursion in the earthquake leads to redistribution of the gravity actions and the subsequent inelastic behaviour depends upon the strength V_e'.

From the paper and the report [13] the writer found that the overturning strength of the frames ranged from about 2.1 to 2.6 times the strength required by the code with a structural ductility factor of 3. This means that the frames had an actual strength which corresponded to structural ductility factors in the range of 1.1 to 1.5. With such values under "design level earthquakes" very limited inelastic behaviour could be expected beyond the small amount associated with the redistribution of the gravity bending moments. With this interpretation the results of the analyses are in line with the conclusions drawn by many other researchers [a. b. c]. This is that P-delta effects are negligible in structures which respond elastically or with a small ductility demand.

In Figs. 15, 17 and 19 in the paper the plastic hinge rotations in the beams have been plotted as a function of interstorey drift. The rotations are shown to be almost linear with this value. For several of the six and 12 storey frames a number of the plastic hinge rotations are incorrect due to an error in the modelling. It was assumed that plastic hinges would form in the beams at the column faces, as was illustrated in case 1 in Fig. A (d). Such plastic hinge zones are described as "reversing", because such a zone is subject to positive inelastic rotations when the structure deforms inelastically in one direction and this is followed by negative inelastic rotations when the direction of inelastic deformation reverses.

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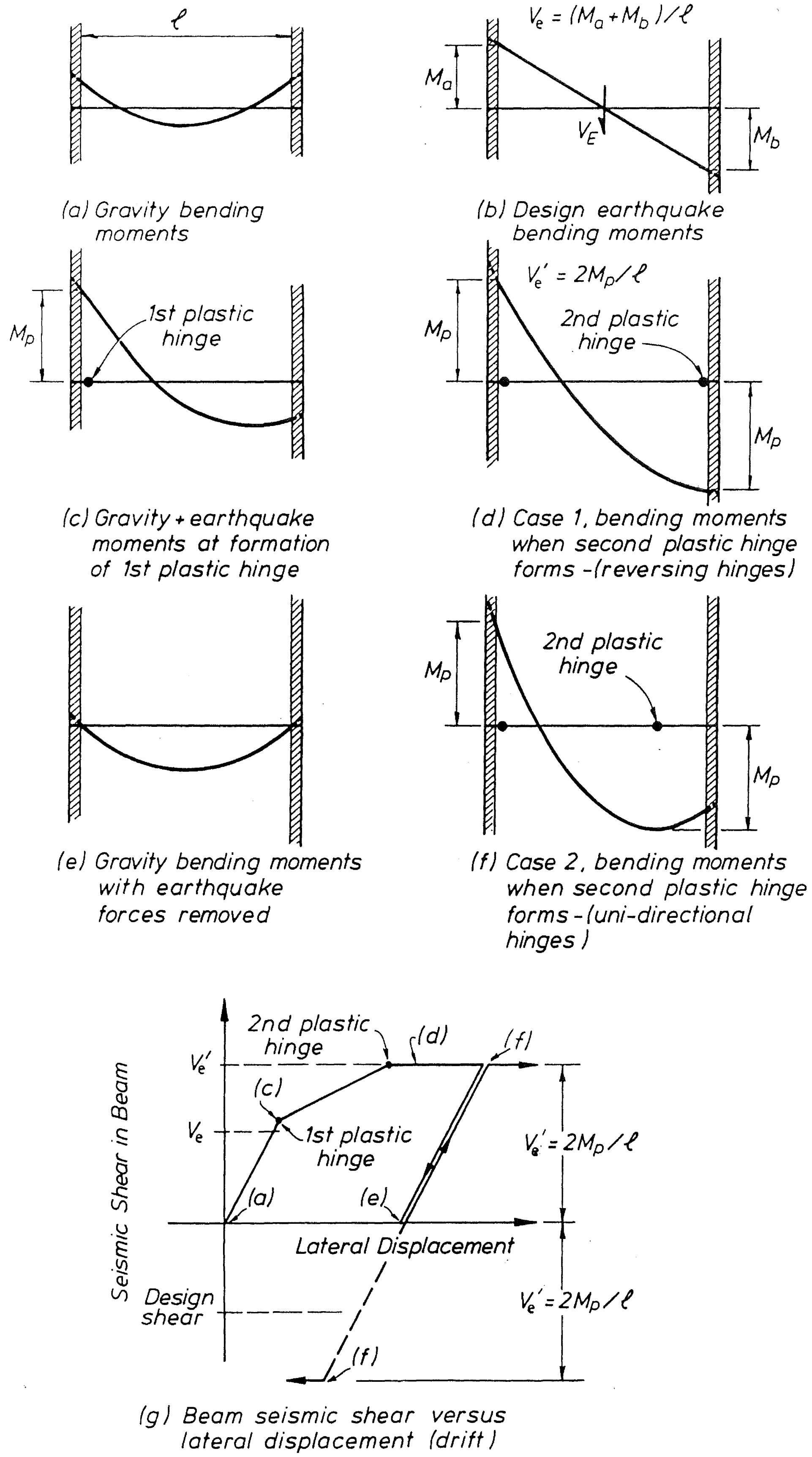

Fig. A Redistribution of bending moments in beams and its influence on seismic shear resistance.

However, from the gravity loads and member strengths used in the analyses it can be shown that in some beams while negative moment plastic hinges form at the column faces positive moment plastic hinges form in the span of the beam as illustrated in Fig. A (f). In these "uni-directional plastic hinges" develop [d]. As the direction of inelastic rotation in these zones does not reverse, the net rotation progressively increases with each inelastic displacement of the storey. With this type of plastic hinge the maximum rotation is a function of the magnitude and number of inelastic excursions sustained and consequently it is a function of both the intensity and duration of the earthquake. Typically plastic hinge rotations sustained by this type of hinge are several times the corresponding values in reversing hinges. If this phenomenon has been taken into account the linear relationships shown in Fig. 15 and 17 would have provided a poor fit to the data.

The writer considers that:

1. The results of the analyses confirm previous research findings that P-delta effects in elastically responding structures and in structures where the ductility demand is small (structural ductility factor, $\mu \leq 1.5$) are negligible.

2. Previous research has shown that the significance of P-delta actions increase with ductility demand. The frames in which the P-delta effects were assessed had lateral strengths in excess of twice the strength required by the code. This arose from the conservative method of analysis that was used and from the redistribution of bending moments which occurs with plastic hinge formation. Due to this high strength and the associated reduced ductility demand no conclusions can be drawn from this work on the adequacy or otherwise of the interstorey drift limits in the draft code (DZ4203-86) for controlling P-delta effects.

3. In carrying out numerical integration time history analyses care is required in selecting the positions of potential plastic hinge zones. Failure to select the positions correctly can lead to -

   (a) a very substantial under-estimation of the plastic hinge rotations, as occurred in some beams in this project, and

   (b) in some situations, though not in the frames considered in the paper, it can lead to a substantial over-estimate of the lateral strength of the structure.

References

(a) Montogomery, C.J., "Influence of P-delta effects on seismic design", Canadian Journal of Civil Engineering, Vol. 8, 1981, pp 31-43.

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(c) Chung, B.T., "Dynamic behaviour of multi-storey buildings", University of Auckland, School of Engineering Report, No. 492, Feb. 1991, pp. 170.

(d) Fenwick, R.C. and Davidson, B.J., "Moment redistribution in seismic resistant frames", Proceedings of Pacific Conference on Earthquake Engineering, Wairakei NZ, 1987, Vol. 1, pp 95-106.

AUTHOR'S REPLY

The frames analyzed in the study were design to the provisions of NZS4203 using capacity design, and, as such, they are not the minimum code stiffness frames considered by Professor Fenwick. These frames are certainly stiffer and stronger than those designed to the limits of the latest draft of DZ4203. The 1991 version of DZ4203 post dates the work of the original paper by several years, and the other variations of the original equivalent static method of analysis have been suggested since the work was carried out.

While the comments of Professor Fenwick about moment redistribution and plastic hinge rotations may be correct, most designers and analysts do not usually undertake the amount of detailed analysis implied by Professor Fenwick in his discussion. At the present time, most dynamic analysis techniques consider hinging only at the member ends, and consideration of hinges within the span would require the beam to be represented by a minimum of three sub-elements,[1] thus increasing the analysis costs and the complexity of the model. However, this problem is currently being addressed by computer software developments.

The authors agree with Professor Fenwick's final consideration (1) and the first part of (2), but do not accept his view expressed in the second part of (2). Considerations (3) may be correct in general, but in most current analytical approaches this would lead to considerable increase in effort in order to investigate each multi-storey frame as suggested.

[1] Tompkins, D.M. "The seismic response of reinforced concrete multi-storey frames," Dept. of Civil Engineering Research Report No. 80-5, University of Canterbury, Christchurch, New Zealand, February 1980.