DISCUSSION

SEISMIC RESPONSE OF LOW-RISE BUILDINGS

P.J. Moss, A.J. Carr and A.H. Buchanan

(Vol. 19, No. 3, September 1986, pp.180-199)

G.C. Clifton*

The principal conclusions reached by the authors in this indepth study on the seismic response of low-rise buildings are interesting and support the results of similar research conducted in New Zealand and overseas.

Firstly the conclusion that the inelastic response is not strongly affected by the shape of the hysteresis loop supports the conclusions reached by Stewart et al at the University of Canterbury and Iwan at the California Institute of Technology in similar research. Iwan has proposed a simple empirical formula relating equivalent inelastic damping to the initial elastic damping and expected displacement ductility demand, which is applicable to a wide range of hysteresis loops. The formula is:

$$G = Go + 5.87\ (\mu - 1)^{0.371}$$

where: Go = initial elastic damping (percentage)
G = equivalent inelastic damping (percentage)
μ = displacement ductility factor

Secondly the equal displacement concept is shown to be inappropriate for use over the range of structural periods for which it is considered applicable in NZS 4203. A more accurate approach in relating the level of inelastic design force to the displacement ductility demand is to apply the equal energy concept over the acceleration controlled region of the spectrum (approximate period range of 0.2 at 0.8 seconds), the equal displacement concept over the displacement controlled region (periods exceeding 2.5 seconds) and to use a linear interpolation between these regions. The elastoplastic response of the steel portal frames with periods of 0.6, 1.0, 1.5 and 2.0 seconds under several earthquakes (shown in Fig. 18 of this paper) is more accurately described by this approach than by application of the equal displacement concept alone.

It is unfortunate, however, that the steel portal frame member sizes selected in this study were not representative of sizes used in practice. However they were determined from the design procedure formulated for the project, which was intended to generate a structure with a given period and is not practical for design office use. I am not familiar with sizes for timber portal frames but would consider that similar comments would apply to the range of timber frames tested.

* HERA Structural Engineer

The portal frames used in this study had pinned bases and no applied vertical loads. It would have been interesting to have modelled both the actual restraint from a "pinned" base and the dead load and compared the inelastic response with that of at least some of the frames studied.

In the paper comment was made that the normal design deflection limits were exceeded by many of the portal frames. Some of the column sizes chosen for the more ductile (R = 4) long period frames were such that the frames would have been very flexible, however the lateral deflection limits specified in NZS 4203 are more severe than those normally applied to portal framed structures and in addition some account is usually taken of the stiffening effect of the cladding in reducing the calculated deflection of the bare frame alone. It would be interesting to know what the authors considered appropriate deflection limits for the portal frames.

Finally I would like to comment on K-braced steel frames. Contrary to the author's comments, their seismic behaviour is well understood, although it is much less than ideal. They exhibit pinched hysteresis loops, with less energy absorbtion and dependable ductility capacity than cross-braced frames, unless the braces are quite stocky. Guidance is available as to the appropriate level of seismic design load to use for a given brace slenderness ratio and the frame used in the study was part of a complete K-braced seismic resisting system of a two storey office building designed for an SM factor of 2.6, giving a base shear coefficient of 0.26. Despite the high seismic base shear coefficient, the brace sizes were dictated by the need to keep the slenderness ratio to a maximum of 80 in the lower storey and 135 in the upper storey, while the sizes of the rest of the members were determined by non-seismic considerations. The K-braced seismic resisting system used was very cost-effective for this structure.

Despite the fact that many of the portal frames used in this study were not representative of typical frames designed in practice, the authors have produced some very valuable results regarding limitations of the equal displacement concept in relating inelastic and elastic response and the relative insensitivity of the inelastic response of single storey structures to the shape of the hysteresis loop.

AUTHORS' REPLY

The authors thank Mr. Clifton for his comments on their paper. While they have not compared the Iwan formula with the results of the frames analysed in the paper, other studies being conducted at the University of Canterbury into base-isolated buildings have shown generally reasonable agreement. More recent work [1] has shown a much better method of estimating the maximum response than that of considering 'equal displacement' or 'equal energies'. This is to evaluate the 'effective period' of the non-linear structure together with the 'effective damping' and then use these values on the appropriate elastic spectra.

The member sizes chosen for the frames reported on in the paper were selected to ensure that the frames had the desired range of natural periods of free-vibration as we were concerned in investigating simple structures with periods in the range of 0.5 to 2.0 seconds and where the inelastic behaviour was essentially a single yield mechanism. If all the strength and stiffness requirements of the seismic code were followed then the frames would have fallen into a very narrow period range and would have been unlikely to yield under the seismic excitation. The considerations were the period, the yield force level (ductility) and the type of hysteresis loop being basically bi-linear for the steel frames and somewhat pinched for the timber frames with nail-plates at the knees.

The authors have not doubt that the K-braced frame was cost-effective but from the point of view of the analyses being attempted the frame had too many of its strength and stiffness characteristics dominated by other than seismic strength considerations to easily fit into the form of study conducted on the portal frame. It became exceedingly difficult to independently arrange the required stiffness and predictable yield strength under a variety of different seismic actions for a multi degree of freedom structure.

Reference

1. Turkington, D.H., "Seismic Design of Bridges on Lead-Rubber Bearings", M.E. Report, Dept. of Civil Engineering, University of Canterbury, March 1987.