DISCUSSION

"SEISMIC DESIGN OF SOUTH BRIGHTON BRIDGE - A DECISION AGAINST MECHANICAL ENERGY DISSIPATORS" - M.J.N. Priestley and M.J. Stockwell

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The results and conclusions reported in this paper are misleading because the energy dissipation system selected was unsuitably matched to the elastic characteristics of the elastomeric bearing system. To obtain a worthwhile attenuation of response for all patterns of ground motion, the inelastic strain energy represented by half the area of the hysteresis loop for the energy dissipator system should be equal to, or greater than, the elastic strain energy stored in the elastomeric bearing system at its maximum displacement during the same cycle. In contrast, from figure 4 and figure 2(b) in the paper, the inelastic strain energy dissipated in a half cycle in the bridge model appears to be only about 40% of the elastic strain energy stored in the elastomeric bearing system at its maximum displacement in the cycle. From the first sentence of paragraph 2.1 in the paper, it is apparent how the incorrect selection of dissipator characteristics came to be made. The authors adopted a dissipator yield force of 5% of the superstructure weight because this was the value suggested by Skinner and McVerry for their building isolation system. However, the authors have overlooked the fact that this value was suggested for use with laminated rubber mounts having an effective rubber height of 6 inches, which were selected to give a rigid building a period of 2.0 seconds. The bridge models which were analysed in the paper had a period of the order of 1.3 seconds. The ratio of the stiffness of the rubber mounts to the weight of the building is inversely proportional to the square of the response period. Consequently, the ratio of lateral elastomeric stiffness ÷ superstructure weight for the bridge analysed in the paper was approximately 2½ times the corresponding ratio for the building to which Skinner and McVerry were referring. It is evident that the ratio of dissipator yield force to bridge superstructure weight should also have been chosen as 2½ times that suggested for Skinner and McVerry's building. Alternatively, the laminated rubber mounts for the bridge could have been reporportioned, to give the bridge a response period of 2.0 seconds. This would have given the additional benefit of putting the bridge period on a lower response portion of the Bucharest acceleration response spectrum, as well as giving a better ratio of inelastic strain energy to elastic strain energy in each cycle of response.

Hence, what the authors have done in the paper is analogous to putting the shock absorber system designed for a light car on to a heavy truck, and then putting forward the conclusion, after subsequent tests on the vehicle, that shock absorbers are of no benefit to heavy trucks. It is obviously necessary to carry out more analysis investigations on a different bridge model, based on a dissipator system correctly matched to the rubber mounts, before conclusions on possible benefits to bridge structures can be arrived at. The system with an inadequate level of dissipator yield force would be particularly ineffective in the case where the maximum response was due to the superimposing of a single large pulse, because there would be only the energy dissipation over a quarter of a cycle to attenuate the first maximum response displacement.

It is possible that investigations on a bridge model incorporating correctly matched dissipators may still indicate that there is no economic advantage in using dissipators on a typical standard bridge. One would expect less economic advantage to show in a bridge than in a building, because dissipators in a building confer significant additional benefits on the non-structural components, as well as the structure. Separation requirements can be reduced, because they do not have to cater for large inelastic structural displacements, and damage to non-structural components will also be reduced. One shouldnot overlook one further advantage of using dissipators namely that for earthquakes significantly larger than the design earthquake, correctly matched dissipators and rubber mounts can give a structure substantial reserve capacity to respond to such an earthquake without large inelastic deformations of the structure. On the other hand, a monolithic structure will respond to such an earthquake with large inelastic deformations. For example for earthquakes of 1.5 El Centro and 1.5 Bucharest, Figs 5 and 6 respectively indicate that the monolithic structure will undergo large inelastic excursions. On the other hand, an isolated structure with the bearings proportioned to give a response period of 2.0 seconds and matched dampers can be expected to go through both such earthquakes without moments exceeding yield level. (This will be apparent from fig. 9 which shows that even the Bucharest spectral value for 2.0 seconds is about 50% of the El Centro value for the period of monolithic structure. Hence 50% x 1.5 x the maximum moment peak for the monolithic model in figure 5 will give the upper limit for the response moment in the base isolated structure of 2.0 seconds period. This will,

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of course, be reduced by the attenuating action of the dissipators.)

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The authors thank Mr. McKenzie for his contribution, but cannot subscribe to his conclusions. We believe that it is his arguments rather than our analyses which are misleading, for the following reasons.

Mr. McKenzie states that the isolation system chosen was incorrectly sized, and that either a more flexible bearing system, or a stronger damper, should have been provided.

In fact, the bearings were designed on the basis of reference A, to the recommended maximum compression stress of 5.5 MPa under HN loading[2]. Considerably higher compression stresses were allowed under HO loading[2]. From commercially available bearings, a shear stiffness per bearing in the range 640 - 4000 kN/m was possible, with the realistic range being 640 - 1500 kN/m. The value adopted, at 910 kN/m, is close to the most flexible bearing available. Adopting the most flexible commerically available bearing would have resulted in a 15% increase in the natural period, which cannot be expected to cause any fundamental change in characteristics.

The alternative suggested by Mr. McKenzie, of strengthening the dissipator (to give it a yield strength of 12.5% of the structural weight) is clearly impractical. With the 5% dampers as analysed, peak pier moments were higher than required for a ductile pier design. Utilising a stronger damper would have had two undesirable effects - stiffening the elastic system and decreasing the elastic natural period, which Mr. McKenzie acknowledges to be undesirable, and further increasing the peak seismic response accelerations of the superstructure. The consequence would have been to require expensive increases in foundation design, and increases in pier longitudinal reinforcement. The concept of needing to provide additional protection against seismic forces as a result of incorporating a base-isolation system appears to us to be philosophically unacceptable.

Mr. McKenzie states that reproportioning the b aring/damper stiffness to a natural period of 2.0 sec. will result in satisfactory performance under 1.5 x Bucharest (and 1.5 x El Centro), whereas large inelastic displacements will result from the monolithic design. Dealing with this last point first, we are at a loss to comprehend how these large inelastic displacements occur. Table 1 in the paper indicates that very moderate curvature ductility factors of about 4.2 and 3.0 can be expected from the monolithic design under 1.5 x El Centro and 1.5 x Bucharest respectively. Mr. McKenzie's comments on the response cf the isolated 2.0 sec. period design are suspect as they are based on the post-yield stiffness. The response spectra approach is based on the elastic stiffness, which will result in a much lower period than 2.0 sec. because of the damper elastic stiffness.

We regret that the Bucharest 2% damping response spectrum shown in Fig. 9 was a crude approximation based on an early analysis of the accelerogram. Since writing the paper the more accurate corrected spectra have become available from the U.S. Geological Survey at Menlo Park(B). The response spectra from this source for 0-20% damping are shown in Fig. A for the Bucharest 1977 S - N component used in our analyses. It is significant that the improved performance claimed by Mr. McKenzie for a 2.0 sec. period structure is not apparent in Fig. A. Even for 10% equivalent viscous damping (which is rather more than could be expected for the damped system), peak response at 2.0 sec. is expected to be 0.45 g, using the approach adopted by Mr. McKenzie ignoring damper stiffness. The structure ductility would then be expected to be in the vicinity of 2.0. However, since approximately 90% of the structure yield displacement is provided by bearing displacement, while all the inelastic displacement will now be provided by pier plastic displacements, the curvature ductility corresponding to the structure ductility of 2.0 will be very high (about 20 in this case, depending on the estimated plastic hinge length). This behaviour is illustrated in Fig. B. Clearly under 1.5 x Bucharest, the curvature ductilities will be much greater (approximately doubled).

Finally, it is of interest to examine the likely superstructure displacements for a 2.0 sec. period design. From the U.S.G.S. source(B), at 10% damping the peak superstructure displacement under 1.0 x Bucharest 1977 S - N is estimated to be 450 mm. This is vastly in excess of the capabilities of existing bearings, yet Mr. McKenzie wishes us to believe that the system would respond elastically in satisfactory fashion under 1.5 x Bucharest.

We attempted to indicate in our paper that base-isolation is not a panacea for seismic ills. Seismic response may well be improved by base-isolation, but equally, performance may be adversely affected if the earthquake characteristics include high energy in the long-period range.

REFERENCES

A. "Provisional Rules for Rubber Bearings in Highway Bridges". M.O.T. Memorandum 802, London.

B. "Analyses of Bucharest 3/4/77 Earthquake Accelerogram". Unpublished report. USGS Seismic Engineering Branch, Menlo Park.

"A CONSIDERATION OF P-DELTA EFFECTS IN DUCTILE REINFORCED CONCRETE FRAMES" - T. Paulay.

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Mr. Andrews, who has seen a draft of Professor Paulay's paper, has suggested that his comment might be useful in promoting constructive comment of aspects of the draft concrete code if published now while comments are still being received by SANZ. Professor Paulay has agreed to this procedure but will reply to the comment, and to any other contributions, in a subsequent Bulletin.