USING A LEAD-BASED DAMPER TO INCREASE NEAR-SOURCE GROUND MOTION RESISTING CAPACITY OF EXISTING BASE-ISOLATED STRUCTURES

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SUMMARY

We use a model of a 4-storey reinforced-concrete moment-resisting frame isolated by lead-rubber bearings to investigate the performance of a typical early design of base-isolated structure when subjected to impulsive near-source ground motions. The base of the isolated building has a gap of 150mm between the concrete slab above the lead rubber bearings and a buffer stop, and the maximum displacement capacity of the lead rubber bearings is assumed to be about 250mm. The Sylmar (1994 Northridge earthquake) record results in a very large ductility demand on the upper structure because of base-buffer impact when the buffer stiffness is large, and an unacceptably large bearing displacement when the buffer stiffness is small. Various methods of increasing the capacity to resist near-source motions are investigated. Increasing the buffer gap size to the maximum displacement capacity of the bearings does not greatly reduce the ductility demand on the upper structure, but adding lead based dampers proves to be a very good way of controlling both the bearing displacements and the inter-storey drifts of the isolated structure. A disadvantage of the additional dampers is that the peak storey accelerations at the base and the top floor increase significantly because of high-frequency modal responses. The retrofitted structure also responds satisfactorily when subjected to near-source motions which exhibit the “backward directivity” effect and to “conventional” ground motions such as the 1940 El Centro record. The combined system is expected to be cost effective because of the reduced displacement demand on the bearings

INTRODUCTION

In early designs of base-isolation systems buffers or retaining structures were often used to limit the isolator displacements as a precaution against rare large ground motions. This was the case for the first building in the world to be seismically isolated, the William Clayton building in Wellington, New Zealand, which was designed and constructed in late 1970's. The isolators were lead rubber bearings (LRBs) (Robinson 1982) and a seismic gap of 150mm was allowed at the basement level to accommodate what were the state-of-art earthquake ground motions of the time. Buffers were provided to restrain the building should the base-isolator displacement exceed 150mm (Skinner et al. 1993). In the 1980s another base-isolated building in Wellington, the Wellington Central Police Station, was designed with a maximum basement-level displacement of 350mm, again with a buffer to restrain the building should the displacement exceed the design maximum. Lead extrusion dampers (LEDs) (Robinson & Greenbank, 1976) provided the necessary level of damping.

Over the last 10 years or so many near-source records have been obtained from large earthquakes, for example, the Lucene and Joshua Tree records from the 1992 Landers earthquake (M_w=7.2) and the Sylmar record from the 1994 Northridge earthquake (M_w=6.7). A common feature of several of the records is a long period velocity pulse of very large amplitude. Such a pulse can impose very large displacement demands on intermediate and long period structures, including base-isolated buildings (Hall et al.1995).

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We use a frame model (Figure 1) designed for 1.5 times the NS component of the 1940 El Centro record to demonstrate that buffer impact can be expected during excitation by one of the recently-recorded near-source motions containing the forward directivity effect, and that the impact will have a detrimental effect on the building. The 15% damped displacement spectrum at about 2s period for the Sylmar record for example is about twice that of the above design motion, and it is a great challenge to retrofit the model structure so that it will be able to cope with such a record. The maximum bearing displacement under the excitation of the design motion is about 110 mm and we use a buffer with a gap of 150 mm. We assume also that the bearings are designed for a maximum displacement of 150mm, but that they can undergo a displacement of 250mm without severe damage on the grounds that the early designs of LRBs were generally very conservative. We also show that retrofitted LEDs will provide the additional damping needed for the structure to be suitably protected against attack from possible near-source ground motions that it was not designed for.

Two types of lead-based dampers, the lead extrusion damper (LED) developed in 1980s and improved recently (Robinson & Greenbank, 1976, Cousins and Porritt, 1993), and the penguin vibration damper (PVD) recently developed by Robinson Seismic Ltd. (Zhao et al, 1999) can be used to provide additional damping for structures. Both dampers have large initial stiffnesses and nearly rectangular hysteresis loops. Other useful features are that the PVD starts to absorb a considerable amount of energy at 1/1000 the maximum design displacement and that the LED is an economical damper when large displacements are required. These properties make the dampers, especially the LED, suitable for reducing the displacement demand on a base-isolated building during an impulsive near-source ground motion. The improved design by Cousins and Porrit has better hysteresis behaviour (Figure 2), is less expensive to manufacture than the original version and ongoing design improvements are expected to further reduce manufacture cost. PVDs are a relatively new technology, while LEDs have been used to protect bridges and buildings in New Zealand for nearly 20 years. Neither device has yet been tested by a major earthquake.

RESPONSE OF AN ISOLATION SYSTEM COMBINING LRBS AND LEDS

We use the Sylmar record from the 1994 Northridge earthquake to investigate the large-event response of the early frame model (Figure 1). Firstly, we calculate the response of the isolation system without additional LEDs and we present the peak storey displacements, peak inter-storey drifts and peak storey accelerations in Figure 3. Because the buffer stiffness is unknown we model a range of buffer stiffnesses. Figure 3 shows that to limit the maximum bearing displacement to the assumed bearing displacement capacity of 250mm the buffer stiffness has to be larger than the total LRB initial stiffness of 60MN/m. For all values of buffer stiffness the inter-storey drifts are too large, and so the structure is likely to suffer severe damage or could even collapse. The storey accelerations also are far too high. The inter-storey drifts and storey accelerations at most floors increase with increasing buffer stiffness, except that severe non-linear hinging action in the frame causes the storey accelerations to be attenuated at the roof for large values of buffer stiffness.

The maximum impact force on the buffer is very sensitive to the buffer stiffness and increases with increasing buffer stiffness. For a buffer stiffness of 10 MN/m, the maximum buffer impact force is 1.9 MN, increasing to 5.6 MN at a buffer stiffness of 60 MN/m, and to 7.8MN at a buffer stiffness of 150 MN/m. Relatively large tensile forces could be expected in the outmost LRBs as a result of such large impact forces, but the peak tension force is only 333 kN (tension stress of 0.93 Mpa) for a buffer stiffness of 150 MN/m. According to the experimental results of Tyler [1991], this level of tension stress is significantly less than the maximum allowable tension stress for an LRB.

We also find that increasing the buffer gap is not a worthwhile measure. The maximum storey displacements (Figure 4(a)) generally increase with increasing buffer gap. When the buffer gap is larger than about 425mm there is no impact between the base of the structure and the buffers and so the displacement profile is almost a straight line. The maximum roof displacement occurs at a buffer gap of 300mm. The maximum inter-storey drifts first increase and then decrease with increasing buffer gap Figure 4(b). When the buffer gap increases from 150mm to 200mm the inter-storey drifts increase by about 7-17mm. From 200mm to 250mm the inter-storey drifts for the first storey continue to increase but those for the top storey decrease. Further increase in buffer gap above 250mm results in reduction in inter-storey drifts for all levels. When the buffer gap is 400mm or larger the maximum inter-storey drifts are of the order of 30-50mm, a level that can be expected to be accommodated by the structure without the formation of plastic hinges in the columns. The maximum storey accelerations are shown in Figure 4(c). When the buffer gap is larger than 425mm and no impact occurs, the maximum storey accelerations distribute along the building height more or less uniformly being about 0.5g for the bottom 4 storeys and 0.65g at the roof. The maximum storey accelerations generally increase with decreasing buffer gap. The largest storey accelerations occur at a buffer gap of

150mm. When the buffer gap is less than 400mm, accelerations larger than 0.8g develop at both the roof and basement floor and severe hinge yielding occurs in both beams and columns. The building is unlikely to survive such a large level of ground shaking even if the bearing displacement is within its capacity. The response parameters in Figure 4 suggest that, for a stiff buffer, buffer-base impact can have a detrimental effect on a structure even for a relatively large buffer gap.

We next present the case of a combined LRB and LED system (Figure 1) with a buffer stiffness of 10 MN/m. Three records are used as excitation. We show later that from a cost effectiveness point of view this is an attractive way of retrofitting the structure.

The response of the isolated structure fitted with 2 LEDs at the base is excellent (Figure 5). For a near-source record with forward directivity effect, the Sylmar record, the maximum bearing displacement is about 220mm, which is within the assumed bearing displacement capacity. The peak inter-storey drifts fall within in a range of 18-25 mm, 0.45-0.6% of the storey height and so the structure responds essentially elastically. The inter-storey drifts are significantly smaller even than the case shown in Figure 4 for a buffer gap of 430 mm where there is no buffer-structure impact. The peak storey accelerations are about $5m/s^2$ at the middle three levels and $6\text{-}7m/s^2$ at the base and the roof. Compared with the case for a buffer gap of 430mm, the accelerations at all floors except for the base are close to those shown in Figure 4(c). Although the accelerations are quite high for a seismically isolated structure, they are significantly smaller than the peak accelerations of the exciting (Sylmar) record.

A building located close to an active fault is likely to experience not just impulsive motions of the Sylmar-type but also other types of strong to moderately strong earthquake motions. The NS component of the 1940 El Centro record is a good example. There is also the possibility of the “backward directivity” effect, as present in the Joshua Tree record from the Landers earthquake. It is important to investigate how well an isolated structure will respond to other likely types of excitation, when the structure is designed for near-source ground motions which exhibit the forward directivity effect. Figures 5(b) and (c) show that adding damping devices, unsurprisingly, increase both the maximum inter-storey displacements and the storey accelerations under the excitation of the 1940 El Centro record. The increases are especially significant for the storey accelerations and the total base shear for the structure with dampers is 1.7 times that of the structure without dampers. Although not modelled here explicitly, the large initial stiffness of the dampers is also expected to result in a significant increase in the high-frequency responses (Skinner et al. 1993) because the first isolated frame period becomes quite short.

Figure 5 also shows the response parameters of the model under the excitation of the Joshua Tree record. Again, as expected, the performance of the isolated model without additional dampers is better than with dampers, but the structure responds essentially elastically and plastic hinges form only in the top floor and roof beams without plastic hinge reversal. The behaviour of the structure with dampers is therefore acceptable.

We believe that in future it will be possible for an optimum design to be achieved so that all vital structural response parameters can be optimized under both near-source ground motions and the other types of ground motions.

For the frame structure shown in Figure 1, to achieve a satisfactory performance under the excitation of Sylmar record using LRBs only, the maximum design displacement for the LRBs would have to be larger than 425mm. Such a large displacement would require an LRB at least 380mm high and 1000mm in diameter. By using a combination of LRBs and LEDs, the size of the LRB can be reduced to about 600mm diameter and 200mm height. The cost saving achieved by reducing the size of the LRBs will be offset by the cost of the additional LEDs, but the response of the structure to extreme near-source ground motions is greatly improved

Buffer stiffness remains an important factor in the performance of the structure fitted with LEDs. The peak storey accelerations are quite sensitive to the buffer stiffness and increase with increasing buffer stiffness. However, the peak inter-storey drifts, which control the hinging action of beams and columns, are much less sensitive to changes in buffer stiffness than in the case of a structure protected with LRBs only. This is desirable because there are large uncertainties in evaluating buffer stiffness.

CONCLUSIONS

The following conclusions can be drawn from this study:

1. Some early lead rubber bearing isolation systems are not adequate for protecting structures against the large velocity pulses that have been recorded close to the sources of some recent earthquakes. However

additional damping, provided for example by lead-extrusion dampers, can bring the performance of the isolation system to a satisfactory level.

2. In our example the additional damping reduces the displacement demand on the lead rubber bearings from 430 mm to 220mm, when the structure is subjected to near-source ground motions with large velocity pulses. For this particular case, inter-storey drifts and storey accelerations are slightly smaller with the additional damping than without it, but much smaller than those resulting from impact on stiff buffer systems.

3. The reduction of maximum bearing displacement means cost reductions for LRBs and connections of service facilities. According to the current market prices, the cost saving in LRBs is likely to be higher than the cost of the additional dampers. The combined system is, therefore, likely to be cost effective, and the performance of the system is less sensitive to the increase of displacement demand imposed by ground shaking than is a simple LRB system.

4. For a combined isolation system of LRBs and LEDs, the peak inter-storey drifts of the structure, which control the amount of hinging action of beams and columns, are not greatly sensitive to changes in buffer stiffness. This is desirable because the evaluation of buffer stiffness is usually very imprecise.

5. The structure fitted with additional dampers also performs satisfactorily when subjected to either near-source ground motions containing the backward directivity effect or (1.5 times) the NS component of the 1940 El Centro record.

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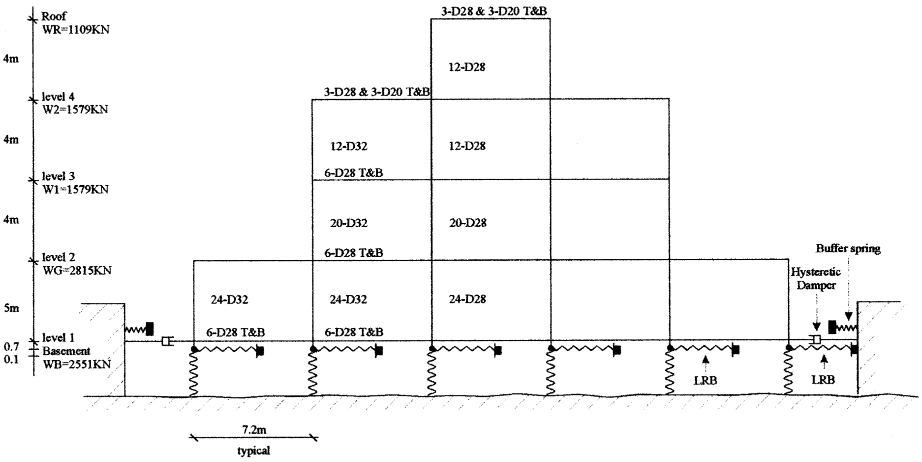

Figure 1: The frame model used in our study. The vertical and horizontal springs represent the vertical and horizontal stiffnesses of the LRBs and the gap springs model the buffer stiffness. The gap between the base of the structure and the buffer walls is 150mm, the assumed displacement capacity of the LRBs is 250mm, and the buffer stiffness is unknown. The buffer gaps and the heights of the LRBs are not to scale.

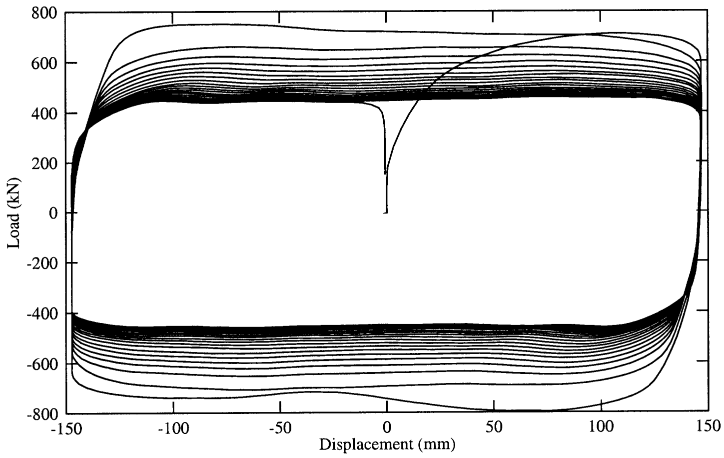

Figure 2 Hysteresis loops for a lead-extrusion damper subjected to 20 cycles of sinusoidal displacement. This is an extreme test because during a real earthquake it is likely that only a few cycles of full-stroke displacement will be required.

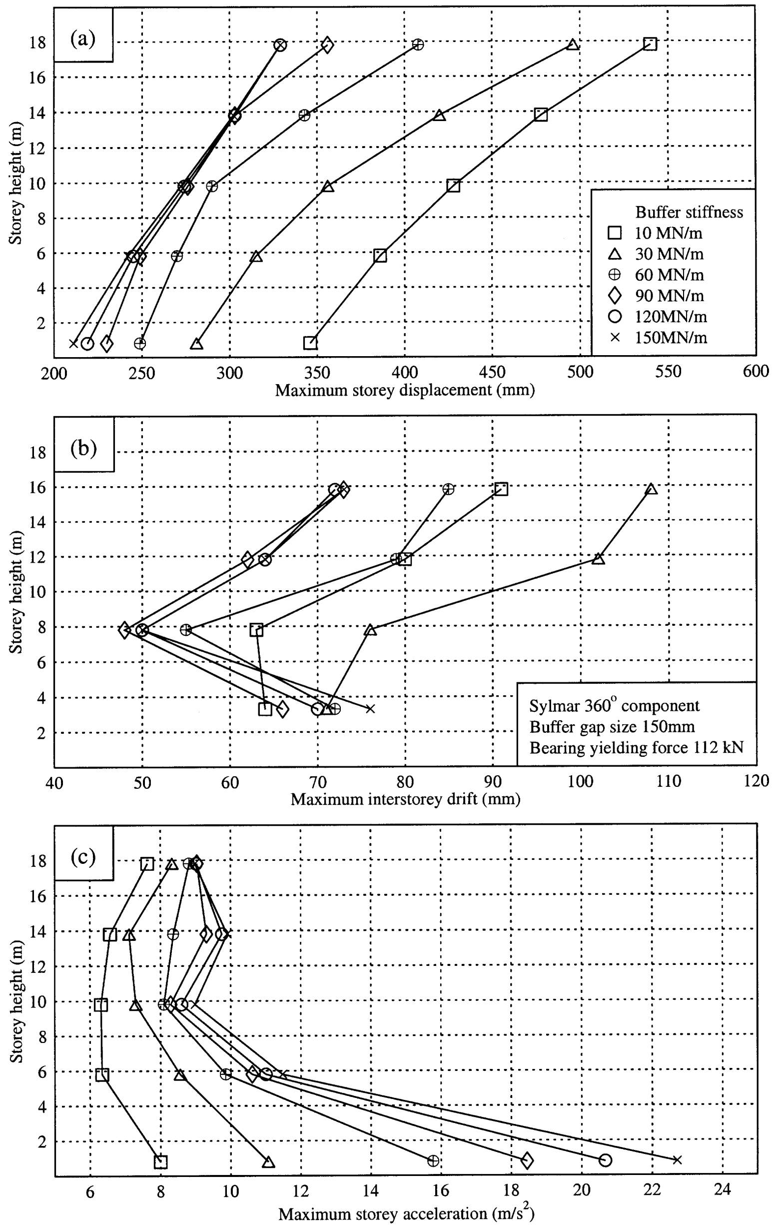

Figure 3: Effect of buffer stiffness on the response of the frame model of Figure 1, WITHOUT the hysteretic dampers, to the Sylmar record from the 1994 Northridge earthquake: (a) storey displacement, (b) inter-storey drift and (c) storey acceleration. The Sylmar record exhibits the near-source forward-directivity effect.

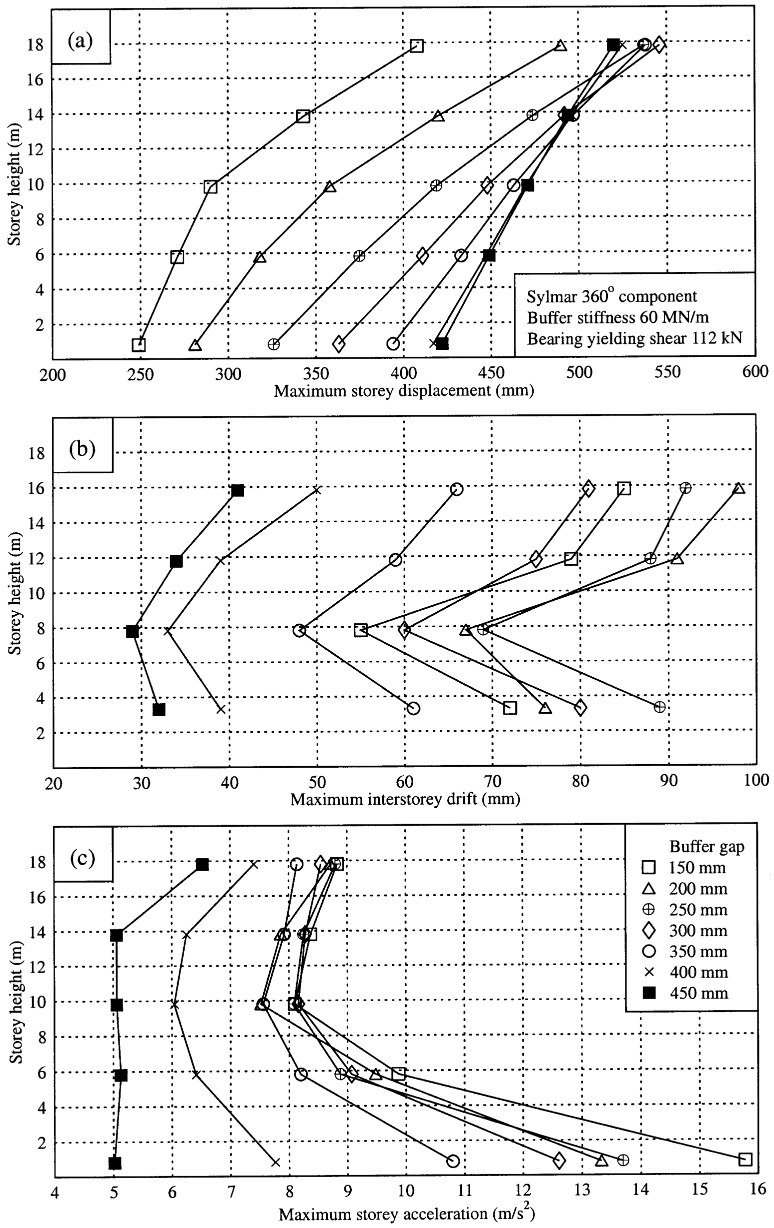

Figure 4: Effect of buffer gap on the response of the frame model of Figure 1, WITHOUT the hysteretic dampers, to the Sylmar record from the 1994 Northridge earthquake: (a) storey displacement, (b) inter-storey drift and (c) storey acceleration. Buffer impact ceases to occur at a gap of about 425mm.

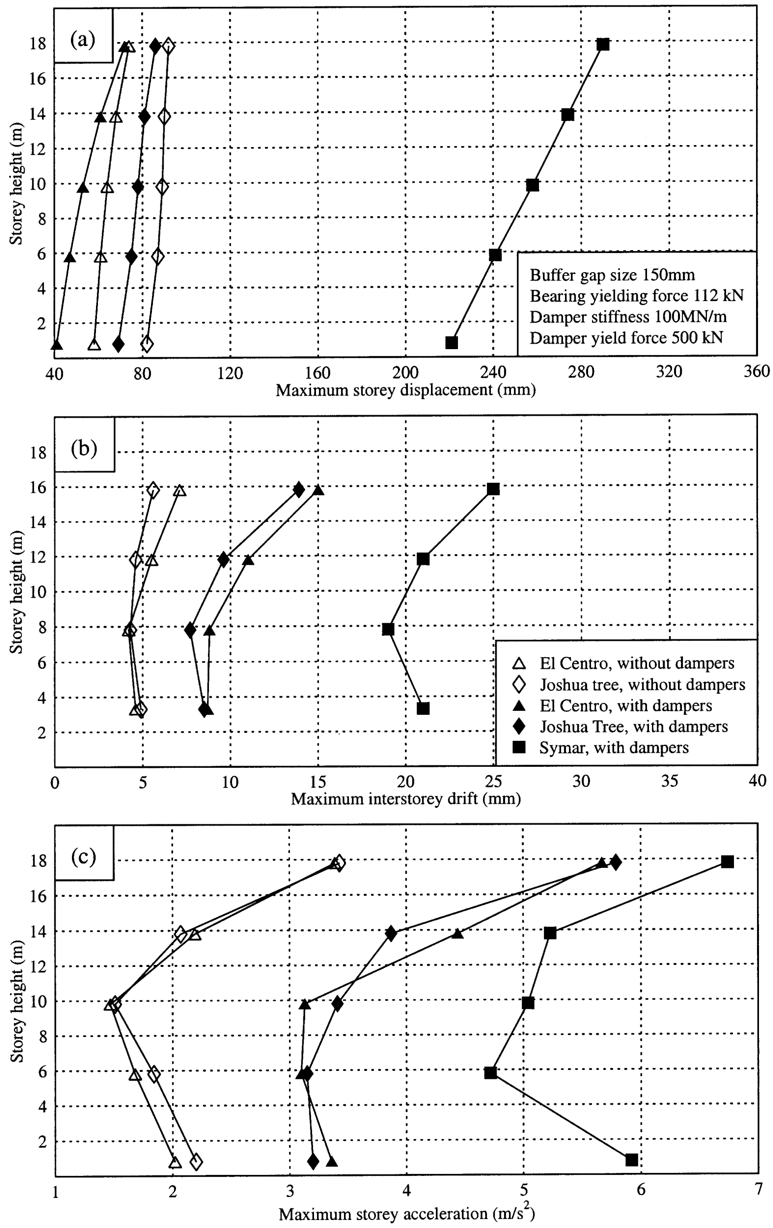

Figure 5: Response of the frame model of Figure 1, WITH the two hysteretic dampers, to the NS component of the Sylmar record, the NS component of the 1940 El Centro record and the fault normal component of the Joshua Tree record from the 1992 Landers earthquake: (a) storey displacement, (b) inter-storey drift and (c) storey acceleration.