RESPONSE OF SEISMICALLY ISOLATED BUILDINGS WITH BUFFERS SUBJECTED TO NEAR-SOURCE GROUND MOTIONS AND POSSIBLE ALTERNATIVE ISOLATION SYSTEMS

J. X. Zhao[1]

SUMMARY

The response of a seismically isolated building with lead rubber bearings (LRB) to near source ground motions from large earthquakes was investigated. The building was assumed to have a buffer to limit the maximum bearing displacement in a rare event of large magnitude and the buffer gap was assumed to be only 150mm (the level of maximum isolator displacement used in the 1980s). The structure was assumed to be designed for 1.5 times the NS component of the 1940 El Centro record. The 15% damped (an amount of damping which is close to the equivalent damping ratio for an seismically isolated building at its isolator design displacement) displacement spectrum of the design motion is only 40% that of the Sylmar County Hospital Parking Lot record from the 1994 Northridge earthquake ($M_w = 6.7$) in the period range around the first modal periods of both isolated and un-isolated structure used in the present study. Among the near-source records that are available, the near-source Sylmar record from the 1994 Northridge earthquake was found to have a very large displacement demand in a period range of 2.0 - 3.0s and this record is thought to be a better representation of the expected near-source motions than the 1.5 times the 1940 El Centro record.

Structure-buffer impact was found to impose very large inter-storey drifts and produce very large storey accelerations, when the building was subjected to the excitation of the Sylmar record. The structure-buffer impact was found to be detrimental to the structural response if the structure was not designed to provide inelastic deformation capacity, and the structural response did not improve when the gap was increased to 200-250 mm, the expected maximum displacement capacity of the LRBs used in the building. An alternative isolation system of LRBs and hysteretic dampers was investigated and found to be adequate for resisting near-source motions. A large initial damper stiffness and relatively small buffer stiffness (compared with the total initial stiffness of LRBs) were found to be effective in reducing inter-storey drifts and storey accelerations at floors except for the base and roof of the structure. A disadvantage of such a system is the relatively large base and roof accelerations. The system has relatively large inter-storey drifts and storey accelerations compared with an isolated structure using LRBs only when the structure was subjected to either the 1940 El Centro type ground motions or the Joshua Tree type ground motions with backward directivity effect. Such an isolation system would still enable the structure to respond essentially elastically under the excitation of the Sylmar record even though the isolated structure was designed for a much lower level of ground shaking.

As the upper structure of a seismically isolated building is usually designed to respond essentially elastically, the detailing used in the design of a reinforced concrete structure to provide inelastic deformation capacity was generally uncommon and was not fully accounted for in the present study.

1. INTRODUCTION

The first building in the world seismically isolated with lead rubber bearings (LRB) [1] was the William Clayton building in Wellington. The building was designed and constructed in late 1970's, and at that time the state-of-art knowledge for earthquake ground motions predicted a maximum possible base-isolator displacement of less than 150 mm [2] at this site. A seismic gap of 150 mm around the building basement was provided and retaining walls were also provided to restrain the building to a certain extent in cases where the base-isolator displacement exceeded 150 mm. In the early 1990s another base-isolated building in Wellington, the Wellington Central Police Station, was designed to have a maximum basement displacement of 350 mm with a buffer to restrain the building if the building basement displacement exceeded the maximum

[1] *Institute of Geological & Nuclear Sciences, P.O. Box 30-368, Lower Hutt, New Zealand (Member).*

design displacement [3]. Lead extrusion dampers (LED) [4] were used to provide damping in this building. The seismic gap provided for a third base-isolated building, the Wellington Newspaper Press, is 480 mm [5]. Though buffers were provided for these base-isolated buildings and structure-buffer impact is likely for those structures with large design isolation bearing displacements, there was few published systematic studies on how a base-isolated building would respond when the designed function of its buffer is realized under a severe near-source ground shaking.

In the last 10 years or so, many near source records have been obtained from large earthquakes, for example, the Lucerne 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, typically 1m/s. A recent study also indicates that the fault-normal component of a near source record has much larger long period motion amplitudes than the fault-parallel component if the fault rupture is propagating towards the site [6]. Such characteristics of near source motions may impose very large displacement demands on intermediate and long period structures [7].

Recently, attention has been focused on the effect of long-duration velocity pulses on long period structures, including base isolated buildings [7, 8]. Preliminary results indicate that the displacement demand of such near-fault ground motions on a base-isolated structure will be too large to be accommodated by many existing base-isolated buildings [7]. In such cases, the base of the building will impact the buffer and the dynamic behaviour of the seismically isolated structure will be drastically different from that of the building without base-buffer impact. Most base-isolated buildings are designed to work elastically or to undergo only small nonlinear deformation in the upper structure. When building base-buffer impact occurs, it is very likely that the impact load applied at the base of the isolated structure could induce large inelastic deformations which could cause moderate or severe damage to the structure. The large impact load could also induce large floor accelerations and therefore diminish one of the main benefits of seismic isolation.

In the present study, the building site was assumed to be close to an active fault that is capable of producing a large earthquake of magnitude 7 - 7.6. The ground motions at the building site are assumed to have the typical characteristics of many near fault records and are expected to impose a very large displacement demand on all long period structures, including base-isolated structures. It is impossible to accurately estimate the ground motions generated by any fault but the near source records from the 1994 Northridge earthquake and the 1992 Landers earthquake may be good representative motions. The large uncertainty in estimated near source ground motions means that buffers are ideally required for an isolated structure, and designers then have to ensure that structure-buffer impact does not have a detrimental effect on the response of the structure.

There has been relatively little published research on the response of base-isolated structures with buffers. Hall *et al.* [7] investigated the effects of near source motions on base-isolated structures with buffers using artificial ground motions generated by a fault model. Tsai [9] presented a detailed study on the response of a base-isolated building model with a buffer subjected to the NS component of the El Centro 1940 record. Though many useful results were presented in that study, the conclusions derived may not directly apply to a base-isolated structure with buffers subjected to near-source motions. Markris [8] also investigated a single mass base-isolated structure model subjected to various types of velocity pulse functions and suggested a semi-active viscous damper as a possible device to provide additional damping and to resist the long period pulse motions. Davidson *et al.* [10] investigated the response of the William Clayton building to two near source records. The model used by Davidson et al. consisted of a frame model for estimating the response of the structure without a buffer and with/without added viscous and hysteretic damping. They also investigated base-buffer impact action using a single mass model. The added viscous damping in their study was achieved by increasing the contribution of mass proportional damping. The base-buffer impact was investigated separately, incorporating a nonlinear (yielding) gap element to model the buffer.

A frame model similar to that investigated by Megget [2] and Davidson et al. [10] will now be used to estimate the response of a generic existing seismically isolated building to impulsive near source motions. In their model, the retaining walls around the building basement were assumed to act as a buffer though they were not designed for such a function (Les Megget, personal communication 2004). The likely range of the stiffness for the retaining walls to act as a buffer is difficult to estimate and a number of values were assumed to illustrate the effect of the buffer stiffness on the response of a seismically isolated building, and the conclusions derived from the present study are intended to provide a possible guide in considering building-buffer impact effect, instead of for this particular building. The frame model includes gap elements so that the response of the building and the ductility demand during base-buffer impact can be estimated. The effect of adding additional damping will also be investigated, with various retrofitting scenarios being presented.

The computer model for the frame structure is shown in Figure 1. The isolation system has an initial isolation period of 0.8s and post-yield period of 2.0s and the fixed-base first modal period is slightly over 0.4 s. This gives the post yield stiffness ratio of 0.16 for the LRB. The total nominal yield shear of the LRB was taken as 7% of the total structural weight according to Skinner et al. [11]. In practice buffer stiffness would be difficult to estimate and a range of values are investigated in the present study. Megget [2] gives the reinforcement details for the columns and the nominal yield moments for the beams and the yield moment-axial force interaction curves were calculated for the columns. The computer program PCANSR (Maison [12]) was used and a small time step of 0.002s for the time domain integration was employed in the dynamic analyses. The time step was found adequate for the analyses.

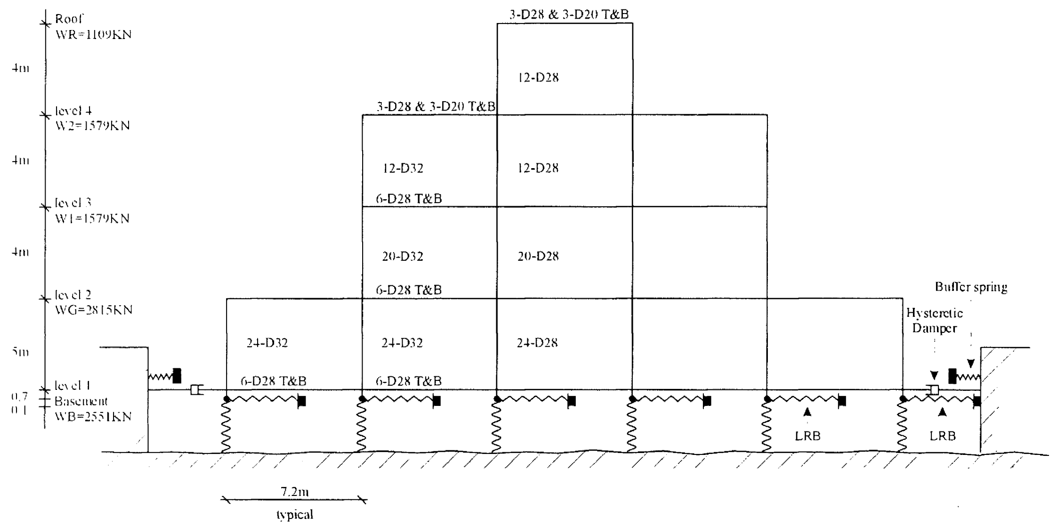

Figure 1. The frame model, similar to that used by Megget [2] and Davidson [10], investigated in the present study.

In an ideal energy dissipation mechanism, columns should not be allowed to yield because a sway mechanism will develop when all beams yield (LRBs will not be able to restrain the rotation of the columns). The capacity design approach requires that the sum of the maximum moments developed at all beam ends, including overstrength, at a node will be considerably less than that of the columns [13]. This is to ensure that the columns will respond elastically. In an analysis, the designed beam over strength moment will be exceeded when very large inter-storey drifts are imposed on a frame, because constant post yield stiffness is used in the modelling. In a real situation, this is not likely to occur because the post-yield stiffness ratio is unlikely to be constant, and therefore the yielding of columns in an analysis simply means that the estimated beam end rotation is too large. Because many seismically isolated buildings are designed to respond essentially elastically with a strength reduction factor of only 2 being allowed (UBC 1997 [14]), we assumed that the frame model used in this study would suffer considerable damage if a significant number of plastic hinges formed in the columns. In a computer code such as PCANSR using initial stiffness proportional damping, the yielding of beams causes the moment in elastically responding columns unintentionally to increase by a considerable amount and thus false yielding of columns can be reported, as shown by Bernal [15]. Using tangent stiffness proportional damping can overcome this problem but iterations are usually terminated because of convergence difficulties. As a way of avoiding such problems, nodal moment will be evaluated when a column yielding is reported, and if the sum of the beam end moments at that node is less than the sum the column yield moments the column will be assumed to respond elastically still. For the simplicity of assessing nodal moments, rigid end length for both beams and columns is assumed to be zero. This assumption would be more close to reality where considerably large joint deformation develops (real beam-column joints of reinforced concrete structures would deform significantly, Paulay and Priestley [13]). The design nominal yielding moments for beams and columns are not adjusted in this paper.

In Japan and USA, many new seismically isolated structures have increasingly large displacement requirements on isolation devices where a known active fault is close by. Because of large uncertainties in the estimate of possible future ground motions, buffers are likely to be provided in most cases as a safe-guard and buffer-structure impact is still likely. There is a need to investigate the effect of the buffer impact on the response of these new seismically isolated structures. The large displacement requirement inevitably increases the cost of seismic isolation and so an alternative isolation system for resisting near-source ground motions need to be investigated. From economic considerations, some of the criteria for assessing the merit of a conventional isolation system may needs to be modified so that optimum isolation systems for resisting both near-source motions and the other types of ground motion can be obtained.

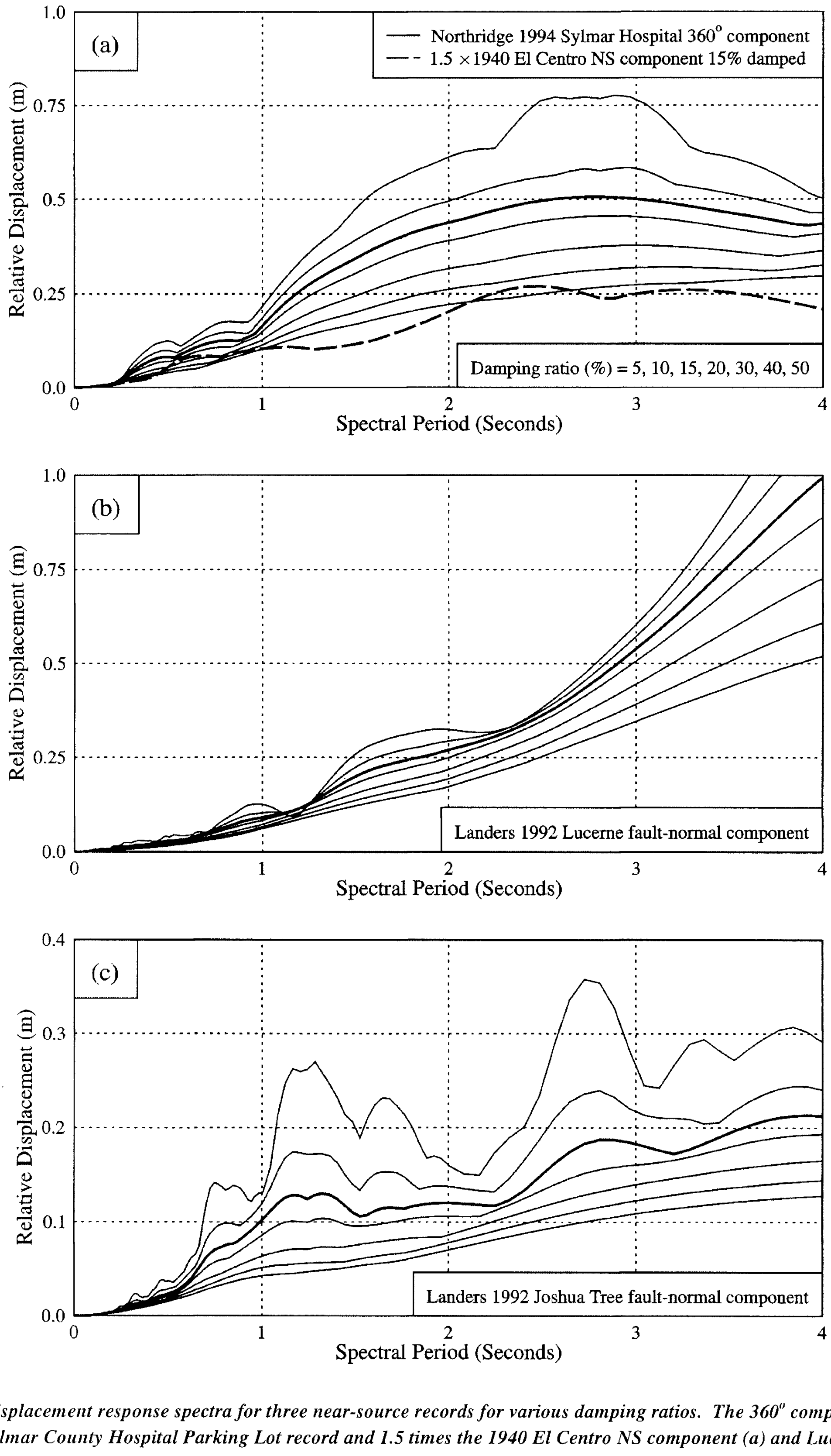

Figure 2. Displacement response spectra for three near-source records for various damping ratios. The 360° component of Sylmar County Hospital Parking Lot record and 1.5 times the 1940 El Centro NS component (a) and Lucerne (b) records contain forward directivity effect and the Joshua Tree (c) record has backward directivity effect. The 15% damped spectra (thick solid line) indicate the likely displacement demands for LRB isolation systems.

The frame model used in the present study was almost identical to that of the William Clayton building with the same design ground motion (1.5 times the NS component of the El Central 1940 records), but there are several major differences between the frame model of this study and the structure of the William Clayton building. The William Clayton building was designed in a conventional manner with full ductility requirements even though the building was seismically isolated. The seismic load distribution over the building height was assumed to be invert of triangular as in a conventional building design and the capacity design approach and the detailing were also used to ensure that any energy dissipation in the structure would be at the beam ends rather than in the columns [2]. The detailing in William Clayton building allowed inelastic deformation up to a global ductility ratio of 4 (Les Megget, personal communication). In the present study, the benefits of ductile deformation of the structure were not explicitly accounted for. Caution must be exercised in relating the results of the present study to the seismic response of the William Clayton building.

2. NEAR-SOURCE RECORDS

A number of near-source records obtained at the time of the present study were used. The Sylmar record of the 1994 Northridge earthquake was found to have a larger displacement demand for a structure with a period of 2.0 seconds than most other available near-source records, including most near-source records from the recent Taiwan M_w= 7.6 earthquake. The Sylmar record will be used to investigate the forward directivity effect on the response of base-isolated structures and Joshua Tree record from the 1992 Landers earthquake will be used to investigate the backward directivity effect.

For a base-isolated structure, displacement spectra can be taken as the best indication for the isolator displacement demand if structure base-buffer impact does not occur. The displacement spectra for the stronger of the two horizontal components of the three near source records are shown in Figures 2(a) - (c) for various critical damping ratios. Figure 2 shows that the Sylmar record has a larger spectral displacement at a period of 2.0s than the other two records. For a base-isolated structure with an equivalent first modal period of 1.5s, to limit the maximum structural displacement relative to the ground to within 250 mm, a damping ratio of 30% is required for the Sylmar record, 10% for the Lucerne record and only 5% for the Joshua Tree record. However, for a structure with a natural period of 4.0s, the Lucerne record has an extremely large displacement demand, 750 mm for 30% damping.

The displacement response spectrum for the design motion (1.5 times NS component of the 1940 El Centro record) for a damping ratio of 15% is shown in Figure 2(a). In the period range of 1 - 2s and below 0.5s, the design displacement spectrum (it will be shown later that 15% damping is appropriate for an isolation system using LRBs) for the frame model is only about 40% of that of the Sylmar record. It would be a great challenge to retrofit either an un-isolated or an isolated structure for such a large difference in displacement demand.

3. RESPONSE OF THE FRAME MODEL WITH VARIOUS BUFFER STIFFNESS

The response of the isolated structure-buffer system subjected to the Sylmar record is investigated first. The gap for the buffer is 150 mm and the buffer is modelled by two gap elements on each side of the building basement floor as indicated in Figure 1. The gap element in the PCANSR program is specified by a gap and stiffness after the gap of the element is closed. Yielding of the gap element is not accommodated in this program. Note that in practice a large uncertainty is involved in assessing the stiffness of a buffer system and a series of values for the buffer stiffness are investigated. The results give some insights on the influence of buffer stiffness on the isolation system responses.

Three most important response parameters will be presented in this paper: i.e., the maximum storey displacement, the maximum inter-storey drift ratio as a percentage of storey height and the maximum storey total acceleration. The maximum plastic rotations for some members will also be reported as an indication of possible damage. An appropriate isolation system would ensure that the maximum isolator displacement is within the displacement capacity of the isolation devices and the service system of the building, that the maximum inter-storey drift ratio is within the design allowance, and that optimal storey accelerations are obtained.

The maximum storey displacement of the model is shown in Figure 3(a) for various values of buffer stiffness. To limit the isolator displacement, a relatively large buffer stiffness is required. The expected maximum displacement capacity of LRBs with a design displacement of 150 mm is about 200-250 mm (Skinner *et al.* [11]) because of the conservative nature in the early designs of LRBs. A maximum design displacement of 250 mm was assumed for the LRBs in the frame structure in Figure 1. For example, if the isolator displacement is restrained to be less than 250 mm, the buffer stiffness has to be equal to or larger than the total initial stiffness of the six LRBs in the model frame (60.6 MN/m). If the buffer stiffness is larger than about 1.5 times the total bearing initial stiffness, its effectiveness in further reducing displacements diminishes. Figure 3(a) also shows that the storey displacement profile changes drastically with varying buffer stiffness when the buffer stiffness is less than 1.5 times the total bearing stiffness, possibly due to high frequency modal response.

The maximum inter-storey drift ratios are shown in Figure 3(b). The inter-storey drift ratios are generally very large, 1-2.7% of the storey height depending on the buffer stiffness. Such large values of inter-storey drift are very difficult for the structure to accommodate and so plastic hinges were formed in all beams and some columns with very large plastic rotations.

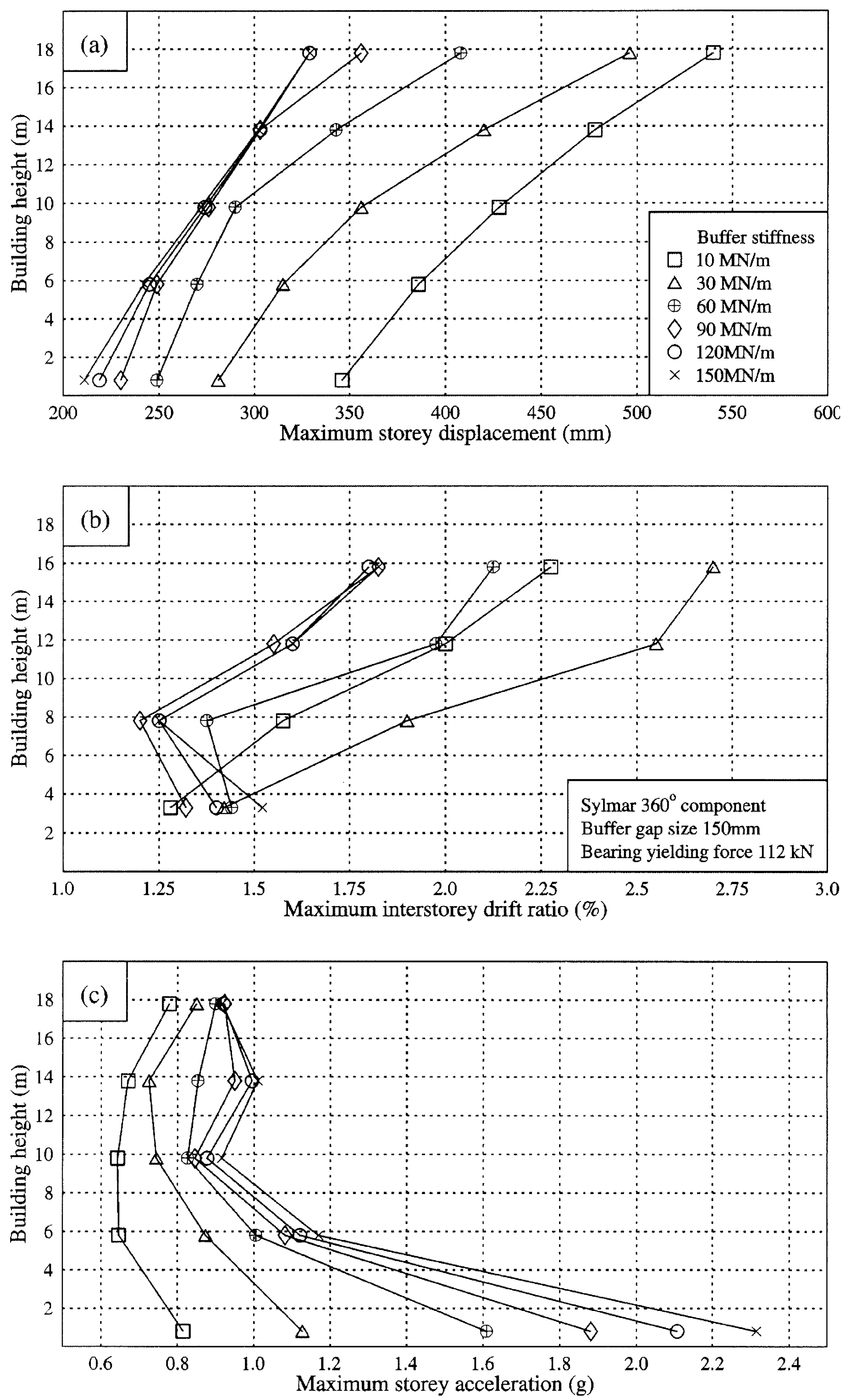

Figure 3. Response parameters of the frame model of Megget [2] subjected to the 1994 Northridge earthquake Sylmar record for various values of buffer stiffness: (a) storey displacement, (b) inter-storey drift ratio and (c) storey acceleration. The expected displacement capacity for the LRBs in the Megget model were 200-250 mm, and the buffer stiffnesses were unknown.

The change of inter-storey drift ratio with buffer stiffness is complicated. For the top 3 storeys, the largest inter-storey drift ratio occurs when the buffer stiffness is about 1/2 the total bearing initial stiffness and then the inter-storey drift ratios decrease with increase in buffer stiffness, for buffer stiffnesses up to about 1.5 times the total bearing initial stiffness, and then become insensitive to further increases in buffer stiffness. When the buffer stiffness is 1/6 of the bearing stiffness, the maximum storey drifts decrease considerably compared with those at a buffer stiffness of 1/2 the total bearing stiffness. When the buffer stiffness is larger than 1.5 times the total bearing stiffness, the inter-storey drift ratios have a very small increase with increasing buffer stiffness. The inter-storey drift for the first storey generally increases with increasing buffer stiffness, but not monotonically. The complicated change of inter-storey drift ratios with buffer stiffness indicates that the buffer impact induces large high frequency mode responses.

The maximum storey accelerations are shown in Figure 3(c). Storey accelerations generally increase with increasing buffer stiffness. Participation of high-frequency modes can be clearly observed when the buffer stiffness is large. The level of acceleration in all cases is unlikely to be acceptable for a base-isolated building. The maximum storey accelerations for the cases of large buffer stiffness generally decrease with height up the building and the roof accelerations are only weakly dependent on buffer stiffness, because of nonlinear deformation of the structures [9].

The maximum impact force applied on the buffer largely depends on the buffer stiffness assumed 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 7.8 MN at a buffer stiffness of 150 MN/m.

Because of the large impact forces, relatively large tensile forces are also generated at the outmost LRBs. The maximum tension forces in the LRB are 152 kN (0.43 MPa in tension stress) for a buffer stiffness of 10 MN/m, 270 kN (0.75 MPa) for a buffer stiffness of 60 MN/m and 333 kN (0.93 MPa) for a buffer stiffness of 150 MN/m. According to the experimental results of Tyler [17], this level of tension stress is significantly less than the maximum allowable tension stress for a LRB, which is over 3 MPa.

In terms of inter-storey drift ratios, an isolated structure with a buffer stiffness of 90 MN/m produced the best results with inter-storey drift ratio ranging from about 1.25 - 1.75%. This large level of deformation results in the plastic rotations up to 0.019 radians in the roof beam, and exceeding 0.01 radians for most other beams. Plastic hinges formed in nearly all columns with the maximum plastic rotations being over 0.007 radians but not at the same time.

It can be concluded that, if a frame structure is not designed to deform in-elastically to provide a reasonable level of ductility, it is unlikely to accommodate the large displacement demand caused by such a high level of ground shaking. Buffer stiffness has a very large effect on the structural response. Hence, because in practice it is likely to be extremely difficult to accurately estimate the buffer stiffness, the uncertainty in the structural response would be too large if a buffer was to be relied on as the mechanism to limit the maximum isolator displacement. For example, the buffer in the William Clayton building would be likely to fail because of the high impact forces [10]. If the yielding of the buffer is accounted for, it is unlikely that the bearing displacement will be limited within its maximum capacity, resulting in uncertain bearing performance.

4. RESPONSE OF THE FRAME MODEL WITH VARIOUS BUFFER GAPS

Although an increase in buffer gap to the maximum bearing displacement capacity is not expected to sufficiently improve the structural performance, the buffer gap is an important parameter governing the base-isolation system and needs to be investigated. In this section, the response of the model is evaluated for various values of buffer gap, assuming that the bearing displacement capacity is not a limiting factor. The buffer stiffness is assumed to be 60 MN/m, bearing in mind that the true level of the buffer stiffness is likely to be very difficult to estimate.

The maximum storey displacements (Figure 4(a)) generally increase with the increase of buffer gap. When the buffer gap is larger than about 425 mm, impact between structure base and buffers does not occur and the displacement profile is almost a straight line. The maximum roof displacement occurs at a buffer gap of 300 mm. The maximum inter-storey drift ratios are shown in Figure 4(b). When the buffer gap increases from 150 mm to 200 mm, the inter-storey drift ratios increase by about 0.18 - 0.43%. When the buffer gap increases from 200 mm to 250 mm, the inter-storey drift ratios for the top storey reduce but those for the first storey increase slightly. Further increase of buffer gap from 250 mm results in further reduction in the inter-storey drift ratios for all levels. When the buffer size is 400 mm or larger, the maximum inter-storey drift ratios are of the order of 0.75 - 1.25%, a level without the formation of plastic hinge 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 150 mm. When the buffer gap is less than 400mm, accelerations larger than 0.8g developed at both the roof and basement floor and severe hinge yielding occurred in both beams and columns. The building is unlikely to survive such a large level of ground shaking without collapse, even if the bearing displacement is within its capacity.

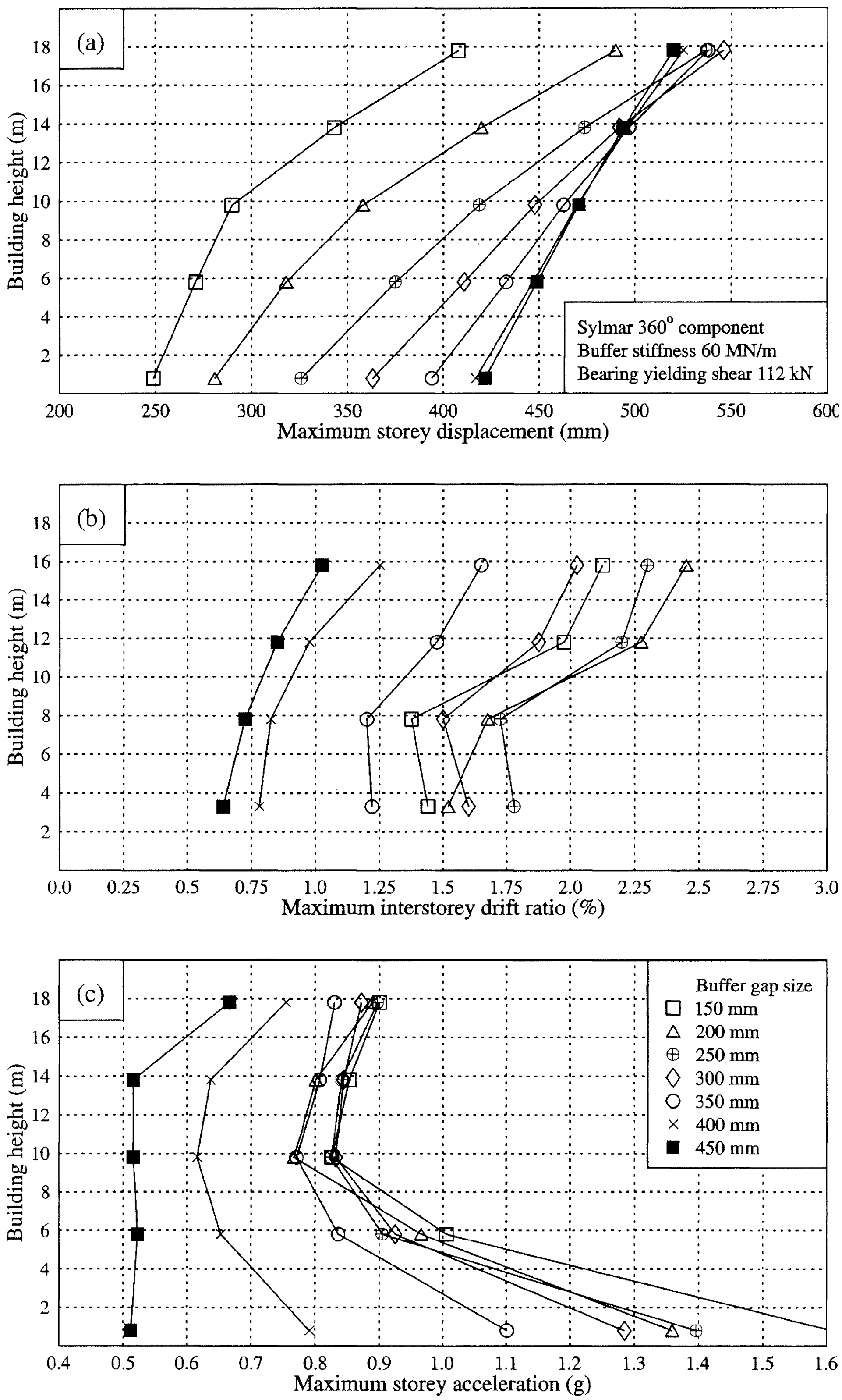

Figure 4. Response parameters of the frame model of Megget [2] subjected to the 1994 Northridge earthquake Sylmar record for various buffer gaps: (a) storey displacement, (b) inter-storey drift ratio and (c) storey acceleration.

Note the relatively large roof acceleration in Figure 4(c). This is presumably caused by the relatively large roof weight and small storey lateral stiffness [2].

It is interesting to note that, for a buffer gap of 400 mm, the buffer does not significantly reduce bearing displacement but does impose somewhat large inter-storey drift ratios and significantly larger storey accelerations than the case without structure-buffer impact, for the structural configurations investigated here.

5. RESPONSE OF THE FRAME MODEL WITH ADDITIONAL HYSTERETIC DAMPERS

In the last section it was clearly shown that extending the buffer gap up to the maximum bearing design displacement will not improve the building response under a near source record such as the Sylmar record, and also that the isolator displacement would be likely to exceed the maximum displacement capacity of the LRBs. Alternative isolation systems will have to be investigated.

The post yield stiffness of a LRB is usually quite significant, typically 0.15 times the initial stiffness of the bearing. In some situations, the high value of the LRB post yield stiffness can lead to a decrease in the effective damping ratio (defined in Equation 2.13 by Skinner et al. [11]) provided to a base-isolated structure with increasing bearing displacement. The damping ratios of a bi-linear hysteretic damper are shown in Figure 5 as a function of bearing displacement amplitude, with the ductility factor being defined as the ratio of maximum displacement to the nominal yield displacement of the damper. A typical ductility factor for a LRB would be 20-30 and the damping ratio provided by the LRB to the base-isolated structure about 10 - 15%. Some other types of hysteretic damper, such as the Penguin Vibration Damper (PVD) [18] and Lead Extrusion Dampers [4], have very small post-yield stiffnesses and thus provide an almost constant damping ratio at large damper displacements. The combination of an LRB and such a kind of hysteretic damper as an alternative method will also have an advantage in providing the required amount of damping without detrimentally increasing the total base shear transmitted into the base of the structure.

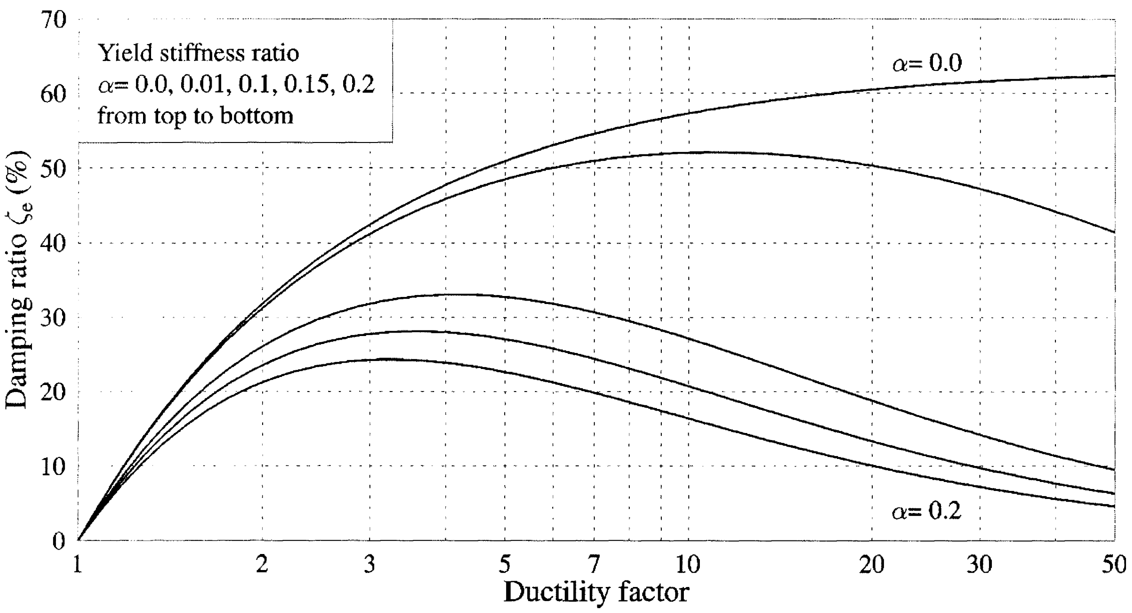

Figure 5. Equivalent damping ratios of a bi-linear damper for various values of post-yield stiffness ratio.

The added dampers are modelled as two nonlinear truss members at each side of the frame model. The members are each assumed to have an initial stiffness of 100 MN/m and a post yield stiffness ratio of 0.1% as indicated by experimental results (Monti *et al.* 1998 [18]). Note that although the initial stiffness (for the bi-linear model) of a PVD can be as high as 2000 MN/m for a device with a small displacement capacity (20 mm), for a PVD with a displacement capacity of 200-250 mm or an LED, the initial stiffness would be smaller than those published [18]. The structural member connecting the damper to the ground and the structure base also would reduce the overall initial stiffness for the damper model. Even so, the high initial stiffness of the damper is still high and is likely to induce large high-frequency response in a seismically isolated structure [11]. This effect will be investigated in this paper. The buffer stiffness is taken as 60 MN/m and it will be shown that the structural responses are not too sensitive to the buffer stiffness if an appropriate isolation system is taken.

a) Influence of Damper Yield Force

The most sensitive parameter for the structural response was found to be the damper yield force. The responses of the frame model fitted with hysteretic dampers are shown in Figure (6).

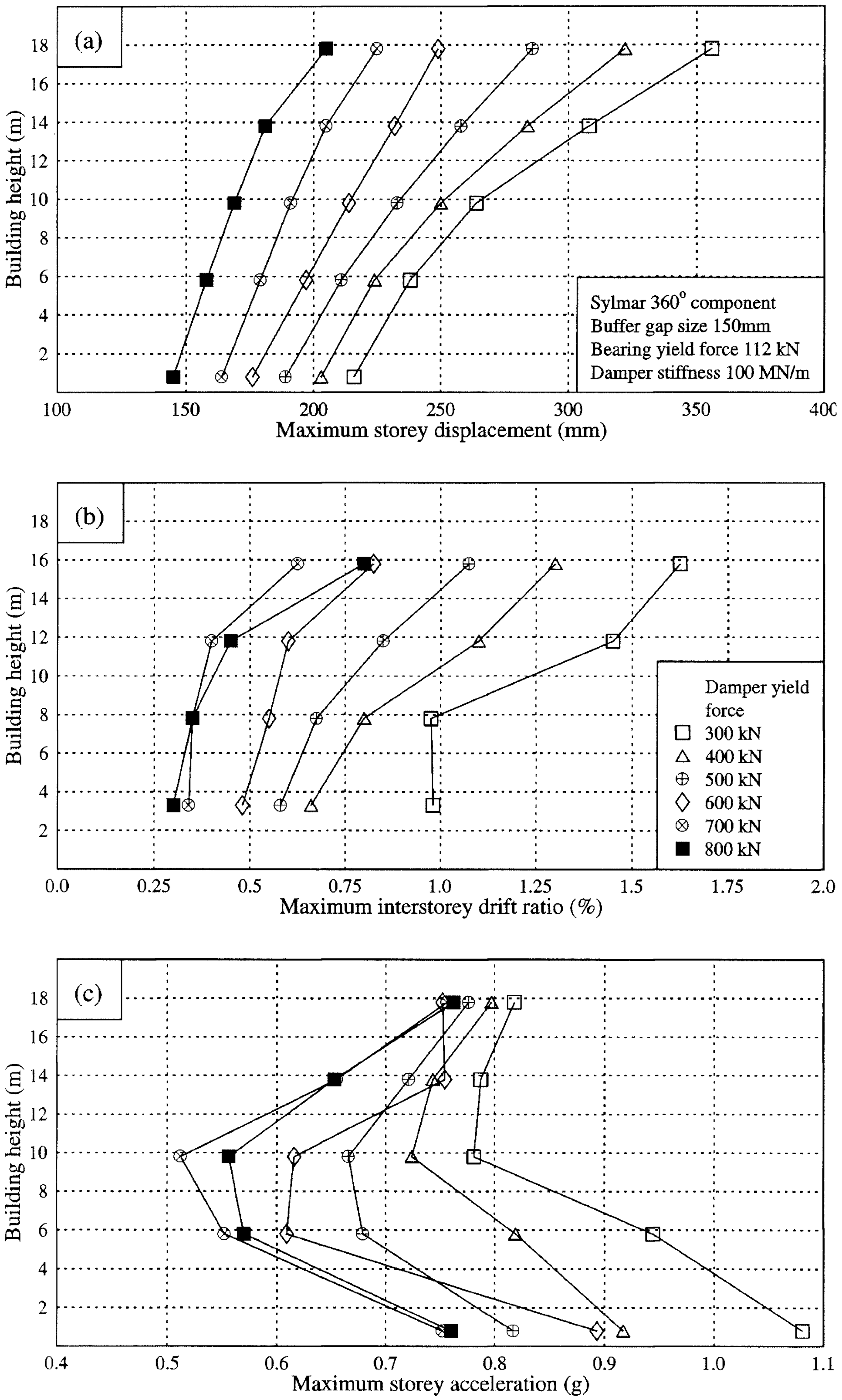

Figure 6. Response parameters of the frame model fitted with two hysteretic dampers subjected to Sylmar record from the 1994 Northridge earthquake for various values of yielding force: (a) storey displacement, (b) inter-storey drift ratio and (c) storey acceleration.

Figure 6(a) shows that the maximum storey displacements decrease quickly with the increase of damper yield force, and when the yield force reaches to 800 kN base-buffer impact will not occur. Even with a damper yield force of 300 kN, the maximum bearing displacement is much less than the expected bearing displacement capacity of 250 mm, even though storey accelerations at lower floors are still too large. Figure 6(b) shows that maximum inter-storey drift ratios reduce rapidly with increasing damper yield force, and that the optimum inter-storey drift ratios are reached at a damper yield force of 700 kN with the inter-storey drift ratios being about 0.3% for the bottom 3 storeys and 0.7% for the top storey. Figure 6(c) shows that the storey accelerations generally decrease with increasing damper yield force. When the damper force increases to 700 kN, the maximum storey accelerations are about 0.5 - 0.65g from level 2 to level 4, while the acceleration is about 0.75g at the basement floor and the roof. The large accelerations at the basement and the roof are presumably caused by high frequency modal responses, which result from the large initial stiffness of the isolation system [11] with dampers, and the impact forces.

The structure is likely to suffer only minor damage if 2 dampers with a yield force of 700 kN are used to retrofit the frame. All columns responded elastically except that plastic hinges were formed at the bases of the columns of the top storey with a small amount of plastic rotation. Plastic hinges were formed at nearly all beam ends but plastic hinge reversal was not recorded for most beams except for the top floor and roof beams. The maximum plastic rotation was less than 0.005 radians for most beams and was 0.0075 radians in one hinge only. To put these values into perspective, Boardman and Kelly [19] suggested that a reinforced concrete structural system would suffer significant damage if plastic hinge rotations exceed approximately 0.007 radians and could collapse at a plastic rotation beyond 0.02 radians (the underline assumption is that the reinforced concrete structure was designed to have a ductility ratio up to 2 only).

For the frame retrofitted with 2 dampers each having a yield force of 500 kN, plastic hinges formed in the columns of the top two floors of the middle bay. The maximum plastic rotations exceeded 0.007 radians in the roof and top floor beams but remained significantly less than 0.02 radians.

(b) Effect of buffer gap

A possible way of overcoming the above problems is to use a combination of installing hysteretic dampers and increased buffer gap to reduce the inter-storey drift ratios and storey accelerations. The responses of the structural model for a buffer gap of 200 mm are shown in Figure 7. Figure 7(a) shows that the isolator displacement is less than or close to 250 mm for all levels of the specified damper yield force. When the damper yield force is larger than or equal to 600 kN, the maximum isolator displacement is less than or very close to the buffer gap. At a damper yield force of 500 kN, the smallest inter-storey drift ratio shown in Figure 7(b) is reduced to 0.5% from 0.57% for a buffer gap of 150 mm shown in Figure 6(b). The inter-storey drift of the first storey changes by a relatively small amount when the buffer gap is extended. For a damper yield force of 300 kN, extending the buffer gap can reduce the maximum storey drift by only 10 - 15%. Figure 7(c) shows that the maximum storey accelerations are reduced by about 20 - 30% compared with those shown in Figure 6(c) with similar vertical distributions for both cases. Further extending the buffer gap to 250 mm results in little improvement in the smallest inter-storey drift ratio for a damper yield force larger than 400 kN though the optimum inter-storey drift ratios are reached at a damper yield force of 400 kN. For a damper yield force of 300 kN, extending the buffer gap from 200 mm to 250 mm results in a large reduction of maximum inter-storey drift but the isolator displacement is larger than the expected bearing displacement capacity of 250 mm (Skinner *et al.* pp 295 [11]). The storey accelerations at level 2 to level 4 are reduced considerably but the structural base and roof accelerations are changed little when the buffer gap is increased from 200 mm to 250 mm.

For a buffer gap of 200 mm, if 2 dampers with a yield force of 500 kN are used, plastic hinges form in the base of the columns of the top floor only with maximum plastic rotations less than 0.002 radian and without plastic hinge reversal. Plastic hinges form in all beams and the maximum plastic rotations at beam ends are between 0.007 - 0.01 radians in the top floor and roof beams. The maximum impact force at the buffer is 1.7 MN. If 2 dampers with a yielding force of 600 kN are used, there are no plastic hinges formed in the columns and the maximum plastic rotations of beam ends are only 0.005 radians. The buffer impact force in this case is negligible.

c) Effect of Damper Initial Stiffness

Both friction dampers and PVDs have very large initial stiffnesses while a lead extrusion damper also has a considerable initial stiffness. The stiffness contributed by hysteretic dampers to the overall stiffness of the isolated structure would then largely depend on the stiffness of the structural members that connect the dampers to the structural joints and the ground. There would be a considerably large uncertainty in estimating damper stiffness because it would involve the evaluation of the stiffness of the connections as well as the foundation supporting the dampers.

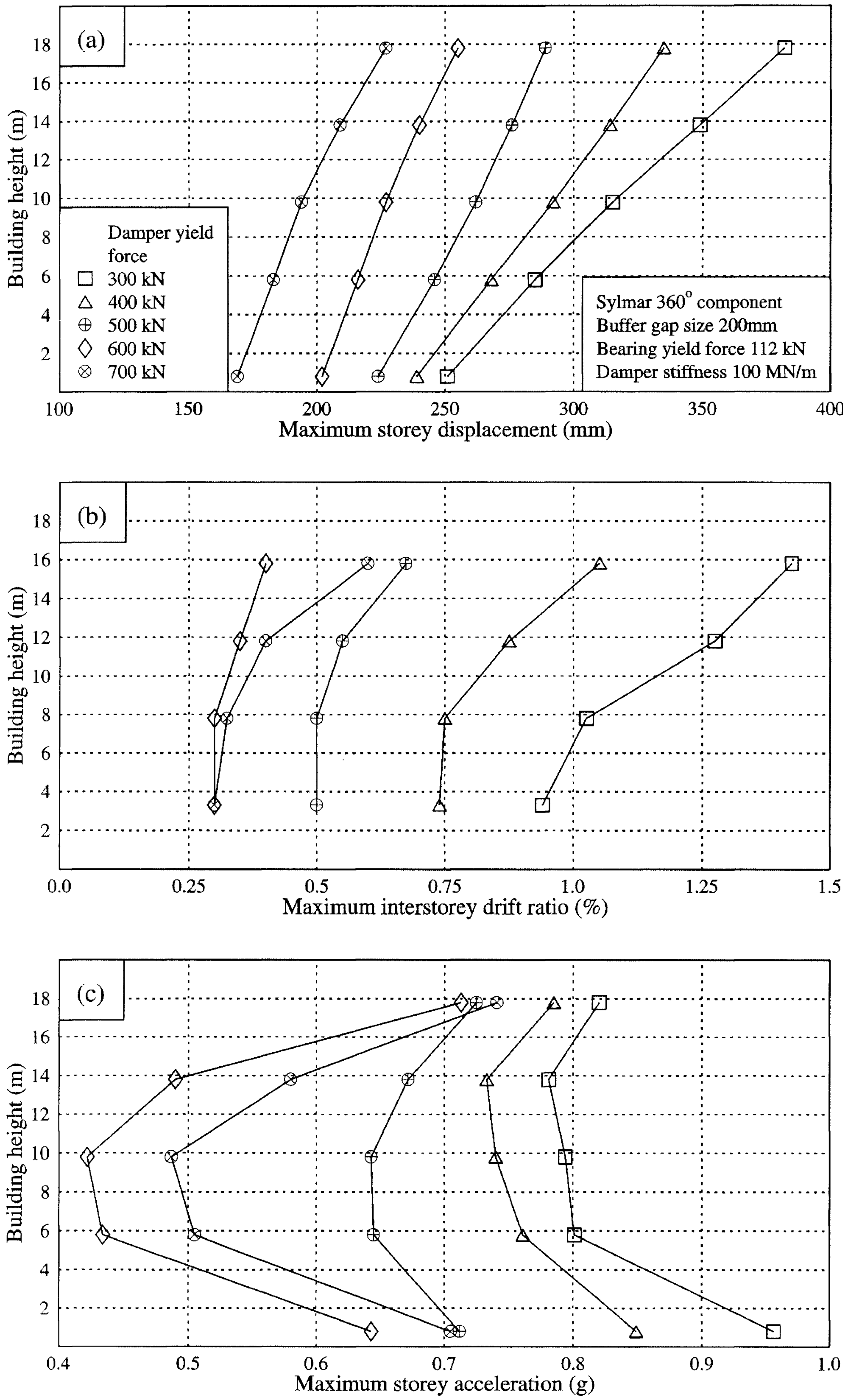

Figure 7. Response parameters of the frame plus damper model from Figure 6 with a buffer gap of 200 mm: (a) storey displacement, (b) inter-storey drift ratio and (c) storey acceleration.

The effects of the damper initial stiffness (the combination of damper and connection member stiffness) are shown in Figure 8 for 4 different values of initial damper stiffness and a buffer gap of 150 mm. The damper yield force is taken as 500 kN and the buffer stiffness is taken as 60 MN/m. Figure 8 shows that the storey displacements and the inter-storey drift ratios are not very sensitive to the damper initial stiffness, probably because the storey displacements are largely controlled by the buffer stiffness. Storey accelerations change relatively little for the damper initial stiffness increasing from 100 MN/m to 200 MN/m but the acceleration at the base of the structure changes considerably. The large roof and base accelerations suggest the dominance of high-frequency responses because of energy transformation from low frequency modes to high-frequency modes [11]. The storey accelerations increase considerably when the damper stiffness decreases from 100 to 50 MN/m, suggesting that damper stiffness should be larger than the buffer stiffness. For a buffer gap of 250 mm, the damper initial stiffness has to be larger than or equal to 50 MN/m in order to limit the maximum bearing displacement to about 250 mm, as shown in Figure 9. For the cases where buffer impact does not occur, the increase of damper initial stiffness from 100 MN/m to 200 MN/m reduces storey displacements and inter-storey drift ratios by a small amount. The storey accelerations for the middle three floors are slightly above 0.4g, virtually independent of initial damper stiffness. The change of initial damper stiffness also has a relatively small effect on the roof acceleration which is between 0.6 - 0.7g for all cases. The increase of damper initial stiffness from 100 MN/m to 200 MN/m has only a small effect on the base acceleration. For all the cases shown in Figure 9, no plastic hinges form in columns, and for the cases that the damper initial stiffness is larger than or equal to 50 MN/m no plastic hinge reversal in beams was observed.

The optimum case shown in Figure 9 is the frame installed with 2 dampers of 500 kN yielding force and 100 MN/m initial stiffness and with a buffer gap of larger than 230 mm for the Sylmar record. This case exhibits much better overall performance than the case of simply extending the buffer gap to about 430 mm. This is, however, not necessarily the case for a new seismic-isolated structure designed for large near-source ground motions.

d) Effect of Buffer Stiffness

Buffer stiffness was shown to have a very large effect on the response of a base-isolated structure using LRB when subjected to a strong near source ground motion. The effect of buffer stiffness is expected to reduce after hysteretic dampers are installed in the building and this effect is investigated here.

For a buffer gap of 150 mm, the frame installed with 2 dampers having parameters as shown in Figure 10 is investigated. The results show that the effect of buffer stiffness is still significant but much smaller than for the cases shown in Figure 3 where dampers were not used. Storey displacements reduce with increasing buffer stiffness, and storey accelerations increase. The smallest inter-storey drift ratios were achieved at a buffer stiffness of 10 MN/m where the bearing displacement is slightly over 220 mm. There is no clear change of pattern for inter-storey drift ratios with buffer stiffness, reflecting that high-frequency response depends on the match of equivalent modal frequencies of the structure-buffer system and the dominant high-frequency components of the ground motion.

For a buffer stiffness of 10 MN/m, columns responded essentially elastically and the maximum plastic rotations in the roof beam were 0.0075 radians. For the worst case of inter-storey drift ratios (a buffer stiffness of 60 MN/m), plastic hinges formed in the middle bay columns of the top two storeys only and plastic hinge reversal was observed with a maximum plastic hinge rotations being less than 0.002 radians. Plastic hinges formed in all beams and the maximum plastic hinge rotations were slightly over 0.01 radians.

The storey accelerations of the middle 3 storeys were nearly constant for each case and increased with increasing buffer stiffness. The smallest storey accelerations were about 0.5g for the middle 3 storeys for a buffer stiffness of 10 MN/m.

If the buffer gap is extended to 200 mm, structural performance of the frame can be significantly improved, as shown in Figure 11, compared to the results of Figure 10. Figure 11 shows that the storey displacements of the top 3 storeys are not sensitive to the buffer stiffness, and the variation in base floor displacement is less than about 15 mm for a change of buffer stiffness from 10 MN/m to 150 MN/m. The maximum bearing displacements are all smaller than 235 mm. The inter-storey drift ratios increase progressively with increasing buffer stiffness. For a buffer stiffness of 10 MN/m, all columns responded elastically and plastic hinge reversal was not recorded in almost all beams. For a buffer stiffness of 150 MN/m (the worst case), plastic hinges formed in the middle bay columns in the top two storeys with maximum plastic rotations of only 0.0013 radians. The maximum plastic rotations in the beams were less than 0.01 radians. The storey accelerations for the middle three storeys progressively increase with increasing buffer stiffness with the lowest storey accelerations being 0.46g. The base and roof accelerations are virtually the same for buffer stiffnesses of 10 and 30 MN/m and increase with the buffer stiffness when the buffer stiffness is larger than 30 MN/m.

Tsai [9] concluded that a controlled and gently increasing buffer stiffness is required to avoid large storey accelerations. The results presented in the present study suggest that a small value of buffer stiffness is required for reducing storey accelerations in the middle three storeys. It seems that a buffer with controlled and gently increasing stiffness would be also ideal for the isolation system of LRBs combined with hysteretic dampers. This kind of buffer needs to be activated at a displacement considerably smaller than the maximum bearing displacement.

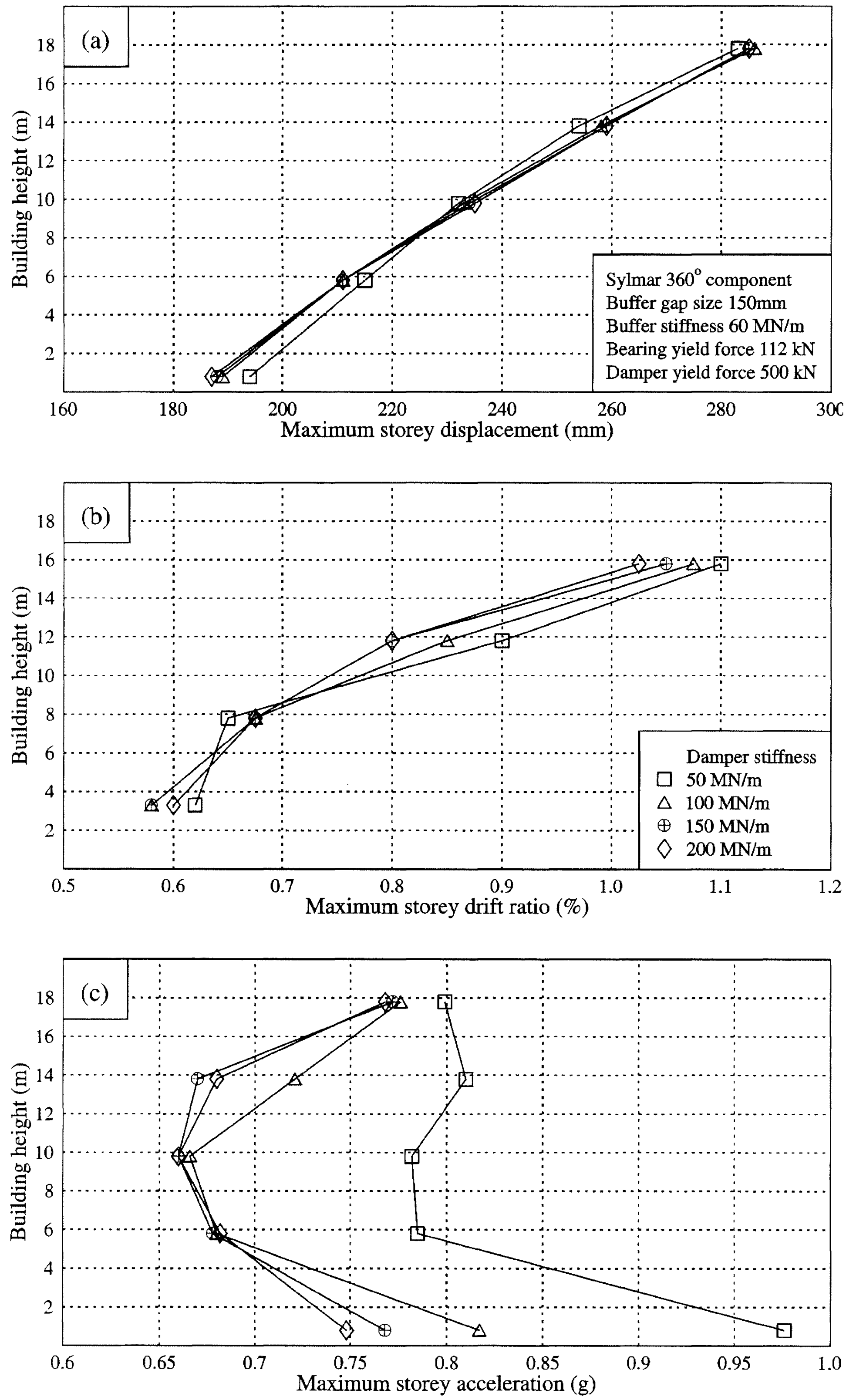

Figure 8. Response parameters of the frame model fitted with two hysteretic dampers subjected to Sylmar record from the 1994 Northridge earthquake for various values of damper initial stiffness: (a) storey displacement, (b) inter-storey drift ratio and (c) storey acceleration.

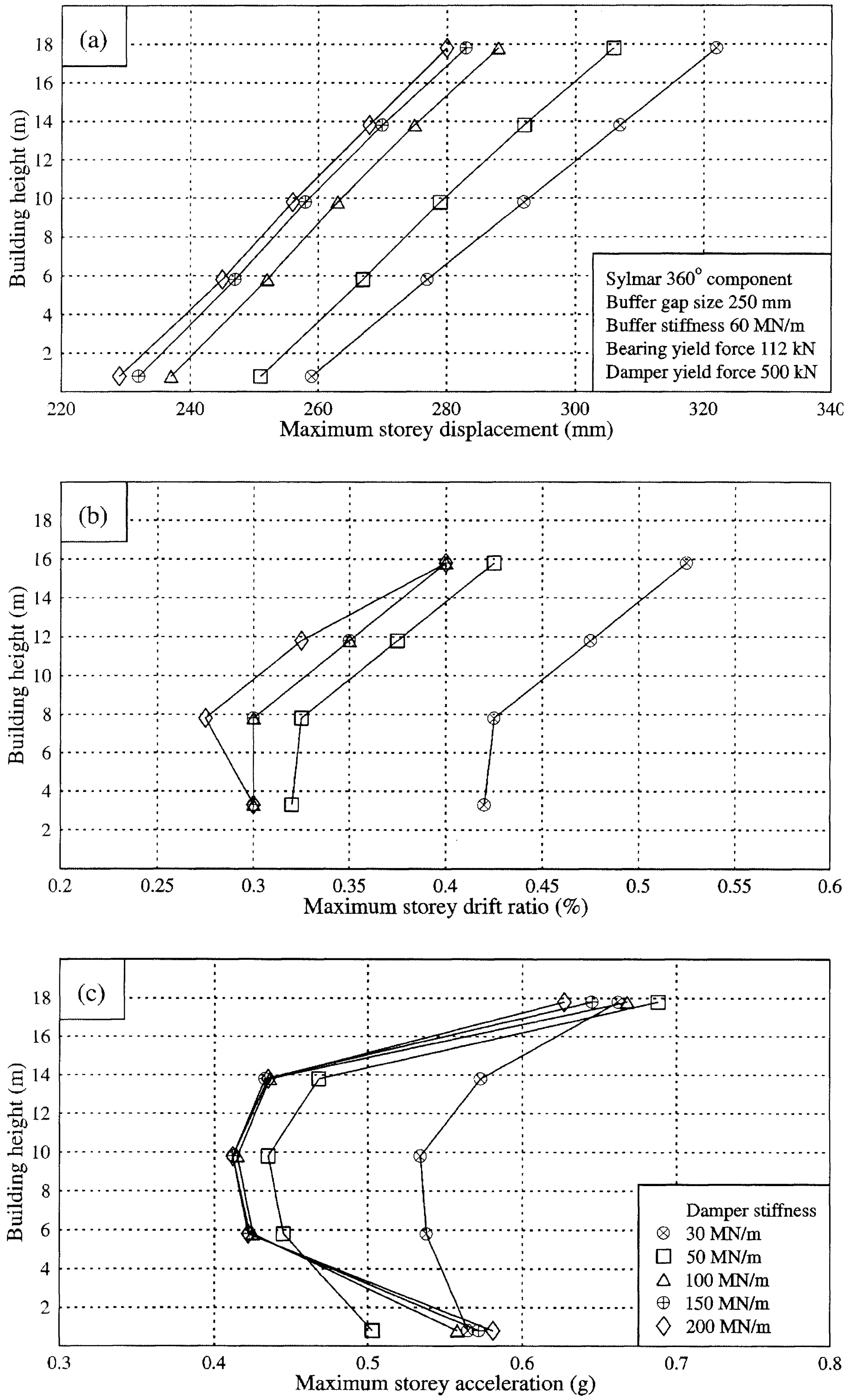

Figure 9. Response parameters of the frame plus damper model from Figure 8 with a buffer gap of 250 mm: (a) storey displacement, (b) inter-storey drift ratio and (c) storey acceleration.

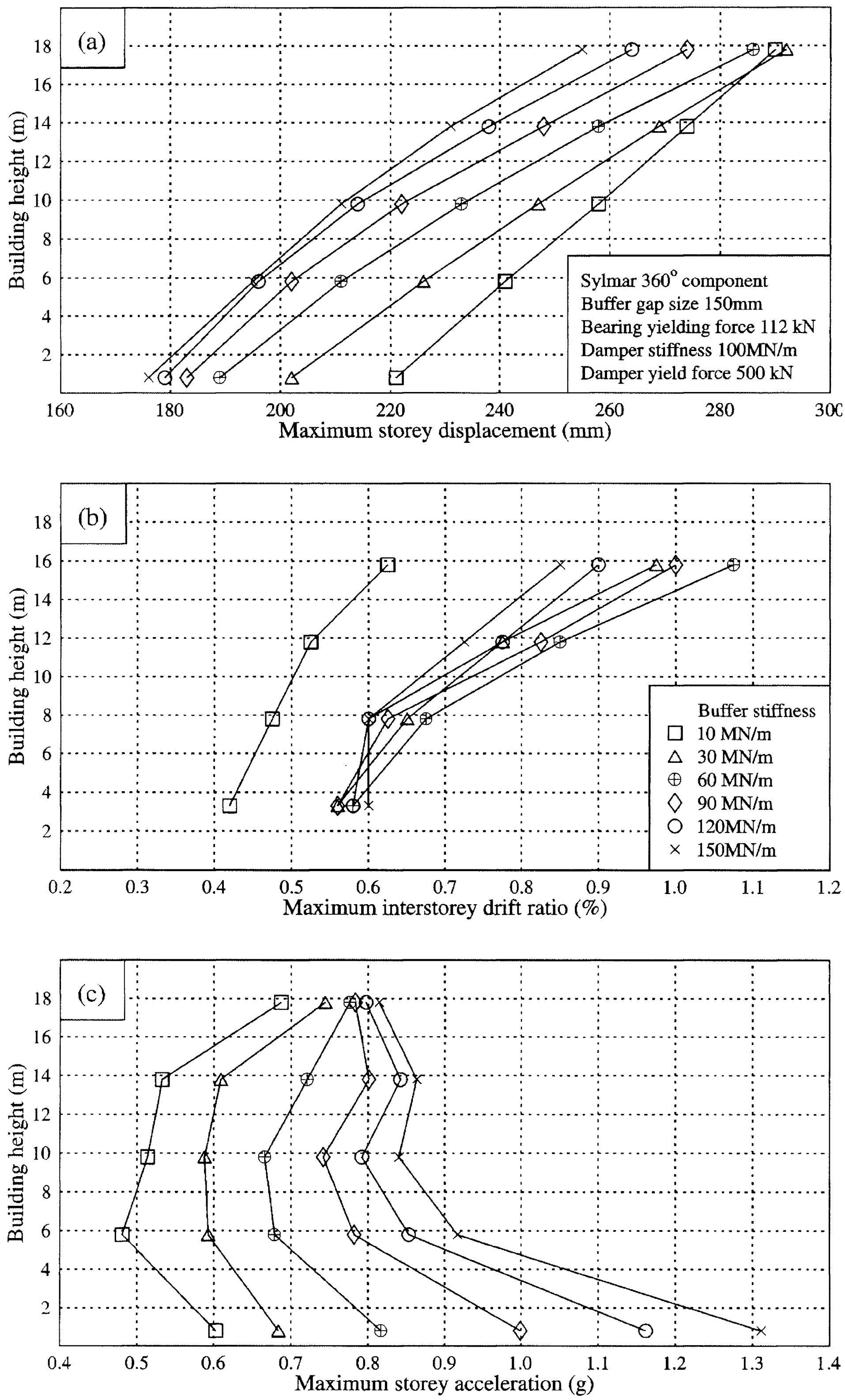

Figure 10. *Response parameters of the frame installed with two hysteretic dampers subjected to Sylmar record from the 1994 Northridge earthquake for various values of buffer stiffness: (a) storey displacement, (b) inter-storey drift ratio and (c) storey acceleration.*

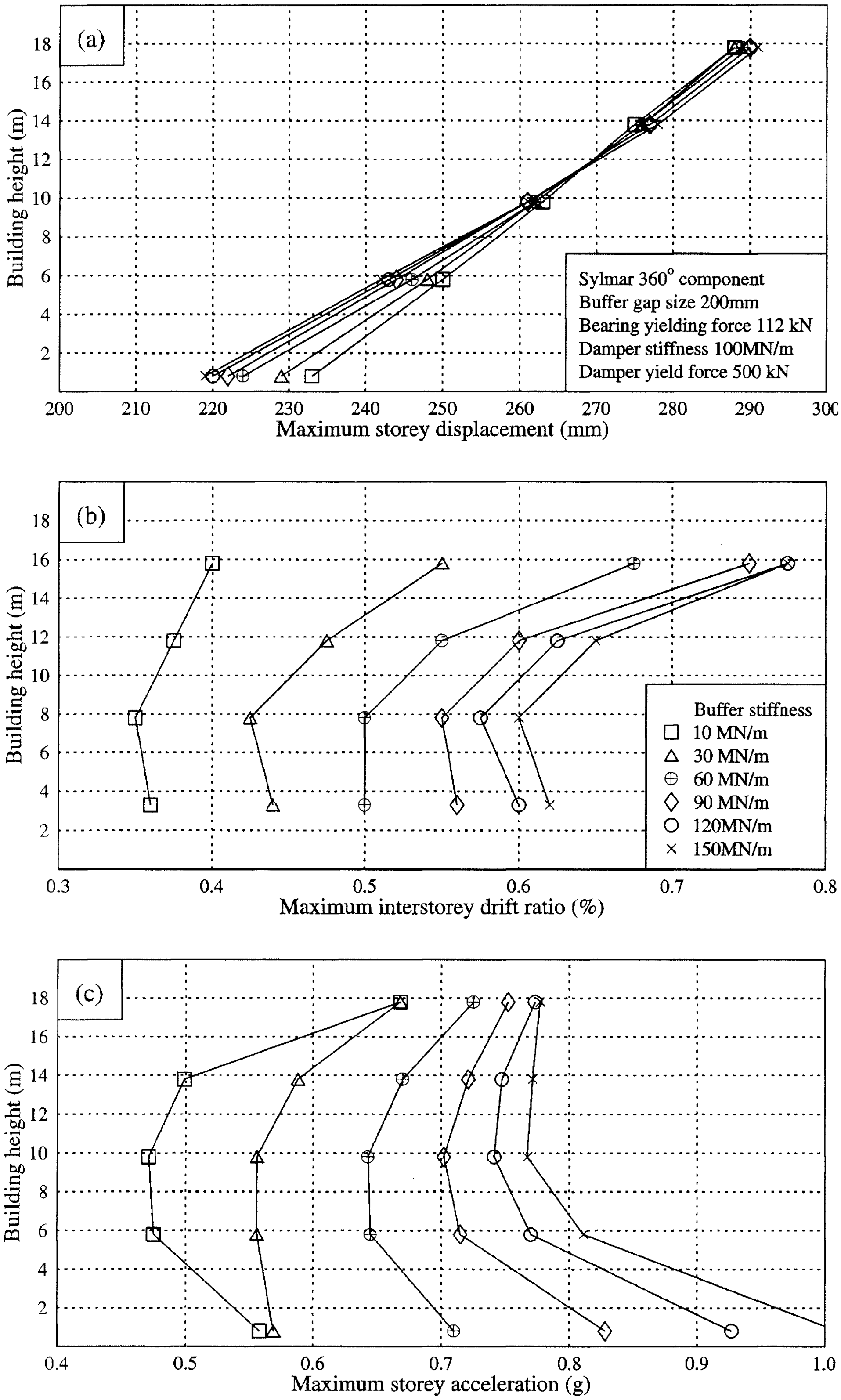

Figure 11. Response parameters of the frame with results shown in Figure 10 with a buffer gap of 200 mm: (a) storey displacement, (b) inter-storey drift ratio and (c) storey acceleration.

6. COMPARISON OF RESPONSES OF THE FRAME MODEL WITH AND WITHOUT BASE ISOLATION AND UNDER DIFFERENT TYPES OF GROUND MOTIONS

Though the building was not designed without base isolation and was not designed for such high level ground shaking as the Sylmar record, the frame model of Figure 1 was analysed for the case without base isolation for comparison purpose.

The maximum inter-storey drift ratios are 2.65, 2.15, 1.43 and 0.64% for the floors from the top and are about 20% worse than the case of isolated building case with a buffer size of 150 mm and a buffer stiffness of 60 MN/m. The storey accelerations are close to 1.0g for all floors, which is much smaller than that recorded in the Northridge earthquake at the Sylmar County hospital in the vicinity of the site where the Sylmar ground record was obtained. The accelerations recorded in the hospital building were 1.5g at the roof and just over 1.0g at the third and the fourth floor [20]. Note that the relatively low level of storey accelerations calculated for the fixed-base frame model is a result of extremely large plastic deformation developed in the structure, which would be suffer severe damage or to collapse under this level of ground shaking. In the context that the storey accelerations estimated for the fixed base building are likely to be much lower than those of the building if designed to withstand such a high level of ground shaking, it is interesting to compare the storey accelerations for the fixed base and isolated case.

The acceleration time histories are shown in Figure 12 for the fixed base case and one of isolated buildings investigated in this paper. The case of the isolated building is the model with a buffer gap of 200 mm and a buffer stiffness of 10 MN/m as shown in Figure 11. Figure 12 (a) shows that, for the isolated case, the peak roof acceleration is still smaller than for the fixed base case even though the fixed base structure would develop severe damage. Figure 12(b) shows that the isolated structure produced much smaller accelerations than the fixed base case and this is the case for all the floors of the middle three levels. Figure 12(c) shows that the large ground floor peak acceleration (Figure 11(c)) relative to those of the middle three floors occurs for an extremely short period of time because of a sharp acceleration pulse. Apart from the acceleration pulse, the ground floor acceleration is similar in amplitude to those of the middle three floors. The peak accelerations of the isolated building are slightly over 50% of the excitation peak acceleration of 0.9g for most floors, and about 70% of it for the roof.

It is also interesting to investigate how an isolated structure would respond to an excitation without near-source long period pulses, when the structure is designed for near-source ground motions with forward directivity effect. This is of practical importance as a building located close to a fault is also very likely to experience earthquake motions generated by a fault 20 km or more away from the structure. The isolated frame structure model investigated in the present study was therefore subjected to the 1940 El Centro record (NS component). The results are presented in Figure 13.

Under the excitation of the El Centro record with scale factors of 1.0 and 1.5, the bearing displacements for the structure fitted with dampers are much smaller than for the case without dampers, as shown in Figure 13(a). The reduction of bearing displacement due to the dampers increases with increasing level of excitation. This is because the damper used in the present study has an almost elasto-plastic behaviour and the damping ratios contributed to the structure by dampers increases with increasing damper displacement, while the damping ratio contributed by LRB decreases, as shown in Figure 5. However, Figures 13(b) and (c) shows that additional damping devices actually increase both maximum inter-storey displacements and storey accelerations under the excitation of the 1940 El Centro record, and the increases are especially significant for storey accelerations. The large initial stiffness of the damper possibly causes significant increase of high-frequency responses [11] because the first isolated frame period becomes quite short. However, a reduction of 50% in the damper initial stiffness results in little reduction of inter-storey drift ratios and storey accelerations under the excitation of the El Centro record, because the isolated first modal period is still too short compared with that of the fixed-base frame. The total base shear for the structure with dampers is 1.7 times that of the structure without dampers for the excitation scale factor of 1.0, and 1.4 times for the scale factor of 1.5. The increase of the base-shear is possibly the main reason for the increase of maximum inter-storey drift ratios and storey accelerations. The large initial stiffness of the dampers is necessary to limit the maximum bearing displacement when the structure is under the excitation of a near-source ground motion as shown in Figure 9(a).

The relatively large roof accelerations in Figure 13(c) are presumably caused by the relatively large roof weight and small storey lateral stiffness [2].

For the worst case in Figure 13, the structure with dampers under 1.5 times El Centro 1940 record, plastic hinges formed in beams only with the maximum plastic hinge rotations being 0.007 radians.

It is possible that an optimum design can 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.

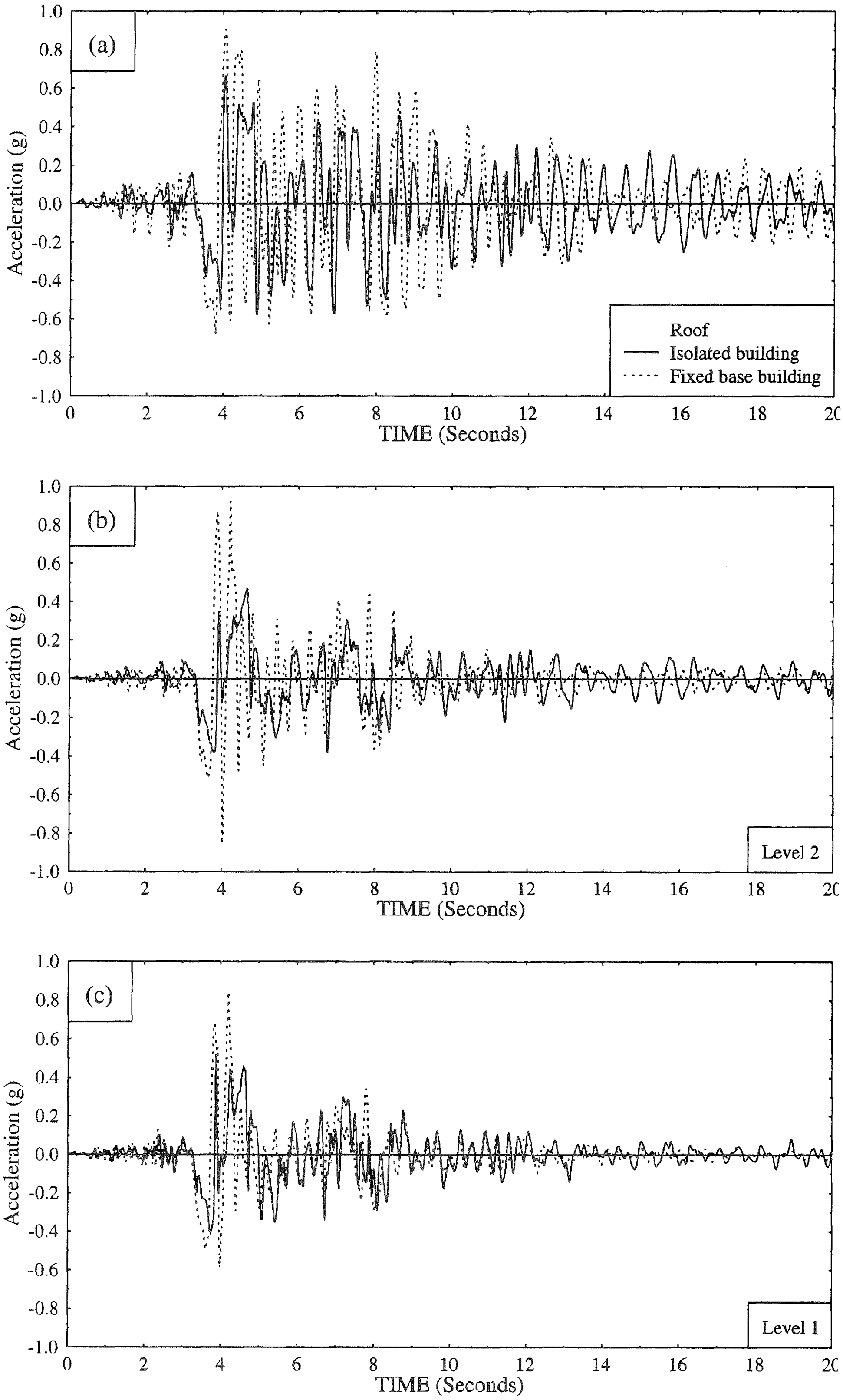

Figure 12. Comparison of acceleration time-histories for the fixed-base structure and seismically isolated structure with LRBs and additional hysteresis dampers.

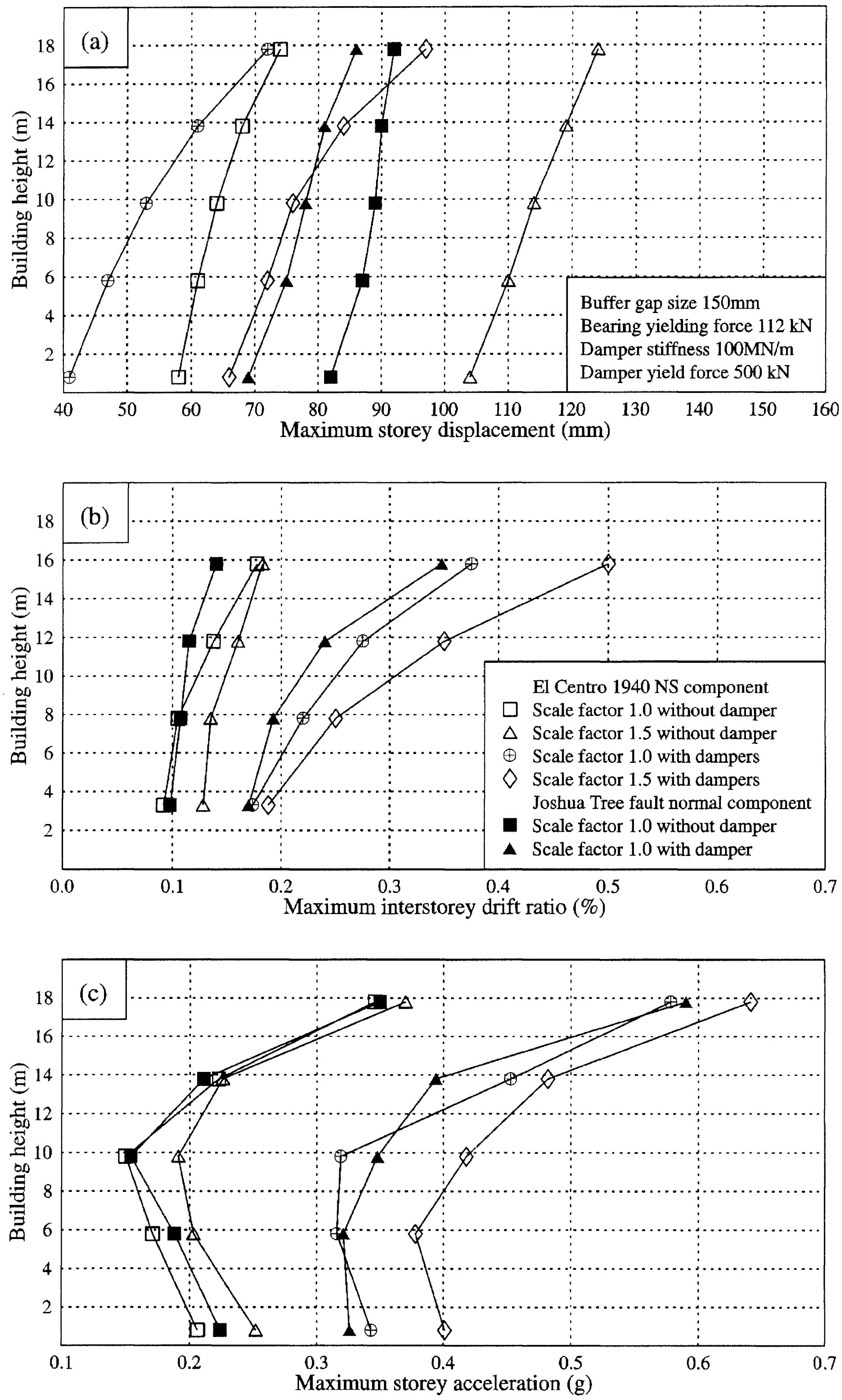

Figure 13. The response parameters of the frame installed with two hysteretic dampers subjected to the NS component of the 1940 El Centro record and the fault normal component of the Joshua Tree record from the 1992 Landers earthquake: (a) storey displacement, (b) inter-storey drift ratio and (c) storey acceleration.

For a structure close to an active fault, there is a large probability that the structure will experience earthquake motions subjected to the "backward directivity" effect, as present in the Joshua Tree record from the Landers earthquake. For a seismically isolated structure designed for resisting seismic motions with forward directivity, the performance of the building when subjected to backward directivity, especially the storey accelerations, must also be acceptable. Figure 13 shows the response parameters of the model under the excitation of the Joshua Tree record. As expected, the performance of the isolated model without additional dampers is much better than that with dampers. However, the performance of the isolated structure with dampers is still much better than that of the fixed-base structure. The structure responded essentially elastically and plastic hinges formed only in the top floor and roof beams without plastic hinge reversal. The maximum plastic hinge rotations were only 0.0045 radians in the roof beam.

7 CONCLUSIONS

Responses of a seismically isolated building with a buffer are investigated when the building is subjected to near-source ground motion Sylmar record obtained in the 1994 Northridge earthquake (M_w= 6.7). Near-source ground motions with forward directivity effects from a very large earthquake have an extremely large displacement demand unlikely to be accommodated by many existing seismically isolated buildings with buffers. Structure-buffer impact changes the response characteristics of a seismically isolated building and the effect can be detrimental to the performance of the structure. The following major conclusions can be drawn from this study.

1. When the buffer gap is not large enough, structure-buffer impact imposes very large inter-storey drift ratios and storey accelerations. The effect is expected to be detrimental to the response of an isolated structure, and could result in possible severe damage to, or even collapse of, the structure if the structure is not designed using capacity design techniques;

2. Buffer stiffness has a large effect on the response of an isolated structure, if impact occurs. The analysis in the present study suggests that, for a lead rubber bearing (LRB) isolation system with a buffer gap of 150 mm, the buffer stiffness has to be close to or larger than the total LRB initial stiffness in order to limit the bearing displacement within 250 mm under the excitation of Sylmar record. At this level of buffer stiffness, structure-buffer impact imposes unacceptable levels of inter-storey drift ratio and storey acceleration. Storey accelerations generally increase with the increase of buffer stiffness and the changing pattern of inter-storey drift ratios with buffer stiffness is complex. When buffer stiffness is equal to the total bearing stiffness, extending the buffer gap to about 400 mm, the buffer fails to reduce bearing displacement but imposes considerably large inter-storey drift ratios and storey accelerations, compared with the case of no structure-buffer impact (a gap of 430 mm);

3. Nonlinear deformation developed in a structure tends to reduce the large accelerations in the upper storeys of the structure caused by structure-buffer impact, consistent with the findings of Tsai [9];

4. Hysteretic dampers with small post-yield stiffness are found to be effective in providing additional damping to a seismically isolated structure using LRB. For a structure with a buffer gap of only 150 mm, additional dampers with a total yielding force about twice the total yield force of LRBs are sufficient to protect the structure from significant damage even though the structure was designed for a much lower level of ground shaking. The most effective isolation system investigated in this paper for an existing isolated structure is to extend the buffer gap to 200 mm, to limit the buffer stiffness to about 1/6 of the total LRB initial stiffness and to add additional hysteretic dampers with a total yielding force of 1 MN. The maximum bearing displacement for this configuration is slightly over 230 mm, within the expected displacement capacity of the existing LRBs. The structure responds essentially elastically and the peak storey accelerations in the middle 3 storeys are 0.4 - 0.5g under an excitation with a PGA of 0.9g, a deamplification of 50%;

5. Damper yielding force level has a large effect on the response of an isolated structure. Inter-storey drift ratios and peak storey accelerations at floors other than the base and roof reduce significantly with the increase of damper yield forces;

6. When a buffer gap is small compared with the maximum bearing displacement without the restraint of a buffer, damper initial stiffness has little effect on the bearing displacement or inter-storey drift ratios. When damper stiffness is larger than total bearing initial stiffness and buffer stiffness, storey accelerations of the upper structure are not sensitive to the damper initial stiffness. If damper initial stiffness is smaller than total bearing initial stiffness and buffer stiffness, storey accelerations of the structure increase considerably;

7. When a buffer gap is relatively large, maximum bearing displacement, inter-storey drift ratios and storey accelerations decrease considerably with the increase of damper initial stiffness. When damper initial stiffness is larger than the total bearing stiffness and the buffer stiffness, the effect of damper initial stiffness is relatively small;

8. When LRBs and hysteretic dampers are combined as an isolation system, the effect of buffer stiffness, which is

difficult to estimate accurately in practice, is much less than to an isolation system with LRBs only. However, the effect is still considerable and the storey accelerations decreases significantly with the decrease of buffer stiffness;

9. An isolation system tuned for resisting near-source motions with forward directivity effect in the present study performs quite well under the excitation of the 1940 El Centro type of ground motion or a near-source motion with backward directivity effect. The structure still responds essentially elastically. The inter-storey drift ratios, base shear and storey accelerations are larger for the structure with dampers than those of the structure without dampers. It is possible, however, an optimum seismic isolation system can be obtained for both types of seismic excitation for a newly designed building;

10. For the frame structure used in the present study to resist a large near-source motion such as the Sylmar record, two hysteretic dampers with a yield force of 500 kN is required per frame. The buffer stiffness also needs to be limited under 10 MN/m. The building would respond considerably better than a case of replacing the existing LRBs with large bearings capable of 430 mm displacement with a corresponding buffer gap. The cost effectiveness of the two methods needs to be investigated;

11. The relatively high level of accelerations at the base and the roof is the disadvantage for the isolation system proposed for the frame structure. However, the storey accelerations would be still much smaller than a fixed base structure even when extensive damage is allowed to dissipate seismic energy in a fixed-base structure. The large base acceleration occurs only for a very short period of time due to high-frequency responses;

12. The results presented in this paper and presented by Tsai [9] indicate that a buffer with controlled and gently increasing stiffness is an ideal safety guard for a seismic isolated structure using the combination of LRBs and hysteretic dampers. The buffer needs to be activated at a displacement significantly smaller than the maximum bearing displacement capacity.

13. Caution must be exercised in applying the results of the present study to any existing seismically isolated buildings in New Zealand. Due to conservative precautions in the early design of seismically isolated structures in New Zealand, for example, a conventional design with detailing that can provide a global ductility ratio of 4 for the William Clayton building, the performance of these seismically isolated structures could be significantly better than the findings suggested in the present study. A further study to account for the ductile performances of these structures may be warranted.

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