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COMPARATIVE PERFORMANCE OF BUCKLING-RESTRAINED BRACES AND MOMENT FRAMES

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SUMMARY

There has been a trend towards the use of technologies that are capable of minimizing inter-story drift and floor acceleration as a means of achieving performance-based seismic design. Some technologies focus more on drift reduction, while others focus more on acceleration reduction. For example, viscous dampers provide the ability to dissipate energy while minimizing inelastic deformations in the structural frame, reducing both drifts and accelerations. Buckling-restrained braces (BRBs) are braced frame structural system components with braces that do not buckle under anticipated axial loads but do yield in both tension and compression, thus acting as energy dissipators. However, the BRBs also stiffen their parent structures, which can reduce drift but increase acceleration. A new BRB under development has similar attributes to steel BRBs that have already been employed in buildings, but it uses aluminum as the dissipating element. It adds stiffness and damping, but the stiffness contribution is only 1/3 the stiffness of steel BRBs if the same area of material is used. It also employs light-weight composite materials to increase brace radius of gyration, thus increasing compression buckling loads.

This paper will provide a comparison of the maximum inter-story drifts and floor response spectra for 3 and 9 story buildings using both steel and aluminum BRBs designed with an R-factor of 7. The buildings had the same dimensions as those used in the SAC project [1] but were redesigned using the 1997 UBC. In addition the aluminum damper was designed as an energy dissipation element using the 2000 NEHRP (2) provisions and its performance is compared to the performance of a standard moment frame designed using the 1997 Uniform Building Code. Each of the building models was analyzed using a total of 20 time-histories that were selected from those developed as part of the SAC program for Los Angeles [2]. One set each of 5 time-histories was representative of the mean spectra for the 100, 500 and 2500 year return period design events for Los Angeles, and the remaining 5 are representative of severe near-fault

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ground motions. The results of each set of 5 time histories were averaged. Due to space limitations only the 100 and 500 year results are presented.

The design of the steel and aluminum BRBs used an R-factor of 7. The yield level of the steel BRBs was 45 ksi and that of the aluminum BRBs was 30 ksi. Buildings incorporating an R-factor of 3.5 and several other yield stresses of the BRBs were studied but are not reported herein. In addition the aluminum damper was also designed as an energy dissipation element using the 2000 NEHRP provisions (2) for energy dissipation elements. The building was designed as moment frames using 75% of the moment frame base shear, and the aluminum dampers were designed to satisfy the moment frame drift requirements.

INTRODUCTION

The original introduction of the buckling restrained brace (BRB) in the US was as a brace element that was a good energy dissipation element, and the early projects used the draft NEHRP energy dissipation design procedures. This meant the BRB was in the same category as viscous dampers, friction and hysteretic damping elements. The code provisions governing the design of these components are still evolving. Their use requires more sophisticated analytical effort plus prototype testing and production testing, as well as peer review of the design and testing of the devices. The proponents of the BRB technology in the US quickly realized that this positioning of the product was a mistake. They repositioned them as brace elements that do not buckle so they would be covered by the steel design provisions, which were less stringent with regard to peer review and test requirements. The BRBs are now established in the code design process as an alternate to the eccentric braced frame (EBF) with the advantage that they will not damage the floor during an earthquake and if necessary can be replaced more easily than the floor beam link in an EBF. As such, a designer can now use an R-factor of 7 to get the required yield force in the brace and, provided a brace of a similar size has been tested, no prototype or production tests are required.

For energy dissipation elements (viscous and hysteretic dampers) the trend is to design a moment frame using up to 75% of the required base shear for a full moment frame and design the energy dissipation element to reduce the inter-story drifts to at or below the code minimums. Generally moment frames are controlled by the code drift limits, and rarely does strength govern the design. In the early application of energy dissipation elements the moment frame had to satisfy the code minimum base shear and drift limits. Thus the addition of energy dissipation elements provided a further reduction in inter-story drifts and thereby enhanced performance. The newer provisions for energy dissipation elements are permitting the moment frame to be strength designed for 75% of the design base shear of a standard moment frame without any concern for inter-story drifts. The energy dissipation elements are then used to satisfy the code drift limits.

A new damper or buckling restrained brace is in the development process by KaZaK Composites Inc. It essentially consists of an aluminum damping core element with lightweight composite filler material surrounded by a stiff outer shell to prevent buckling when loaded in compression. The KaZaK damper is intended to provide significant weight reduction for retrofit applications. Similarly to other buckling restrained braces it adds stiffness and damping to a structure but the stiffness contribution is only 1/3 the stiffness of steel dampers if the same area of material is used as the elastic modulus E of aluminum is 1/3 that of steel. This property provides additional design flexibility. The deformation at the yield point is proportional to the yield strength (F_y) and inversely proportional to modulus of elasticity (E). The lower elastic modulus of aluminum compared to steel will increase the deformation before yield occurs for a structure of same strength. The wide range of available F_y of aluminum alloys will allow further tailoring of yield strain levels for a given frame.

Compared to pure viscous dampers which only provide added damping, the KaZaK damper combines both stiffness and damping components.

Load-deformation hysteresis curve for a KaZaK damper (reduced scale prototype with gage length of approximately 24 inches) is presented in Figure 1. A cyclic pseudo-static test corresponding to the standard protocol SAC Basic Loading History was used (6 cycles at 0.33% and 0.50%, 4 cycles at 0.66%, 2 cycles at 1.0%, 1.33% and 2.0% brace strains). The prototype specimen exhibits stable hysteretic behavior – resulting in good energy dissipation – during cycling at increasing amplitudes up to ±2% brace strain. An increase in slope was noted in compression for strain levels beyond -1%, corresponding to the onset of buckling or barreling of the brace core element which resulted in a significant load bearing contribution of the potting compound and outer shell. After completing the SAC test sequence, the same specimen was tested in low-cycle fatigue and failure occurred after 12 cycles to ±2% brace strain.

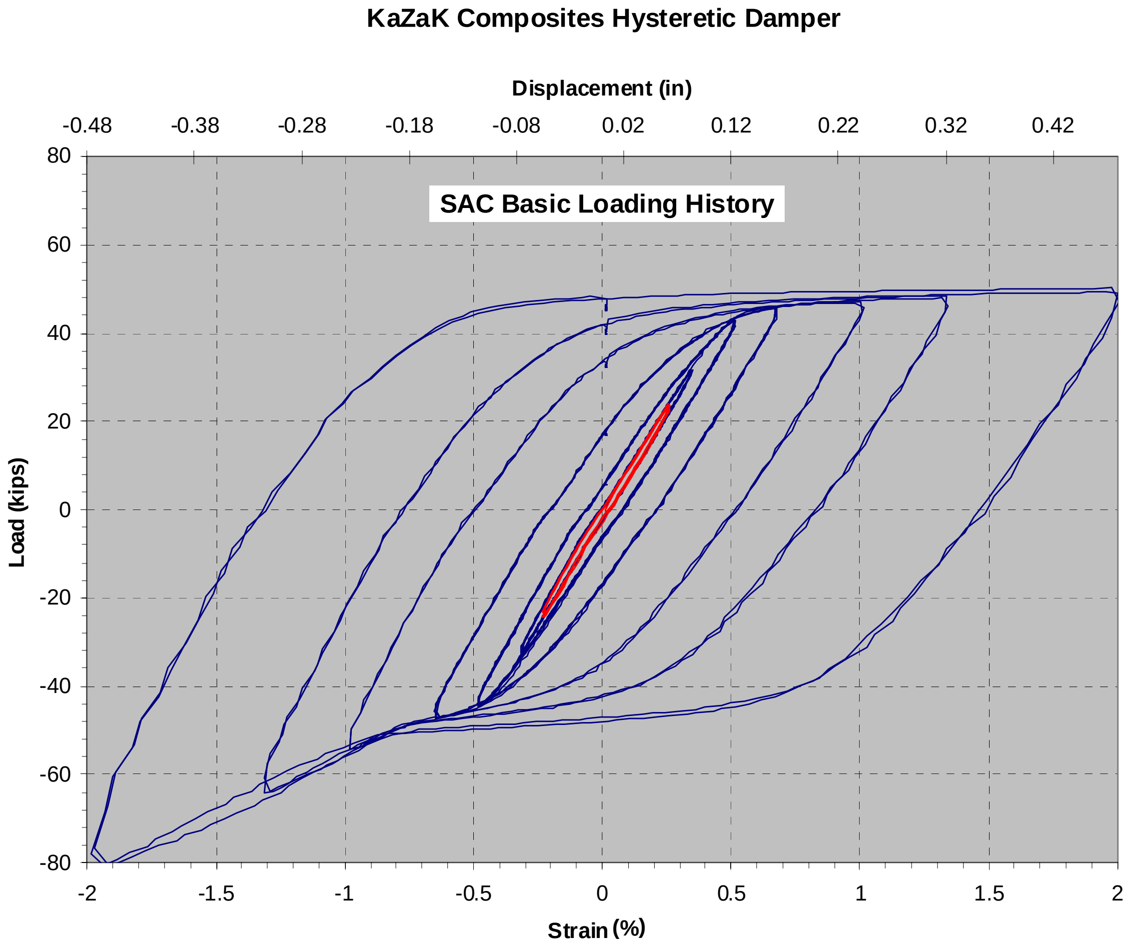

Figure 1 – Load-Deformation Hysteresis of KaZaK's Reduced-Scale Prototype Aluminum Damper: SAC Basic Loading History Test to a Maximum of ±2% Brace Strain (Corresponding to ±3% Interstory Drift).

3 AND 9 STORY BUILDING CONFIGURATIONS

The 3 and 9 story building configurations had similar floor plans with 30ft. bay spacing as shown in Figure 2. The 3 story building had equal story heights of 13ft. The 9 story building had a 1st story height of 18ft, with the remaining floor heights at 13ft. Figures 3 and 4 show the moment and braced frame configurations. The required number of moment frame bays was significantly greater than the required number of braced frame bays due to drift limitations and the new ρ factor provisions for special momentresisting frames in the 1997 UBC. For the 3 story building, only two exterior bays of braces or dampers on each exterior perimeter were necessary. For the 9 story building, 3 bays of bracing and 2 bays of dampers were necessary.

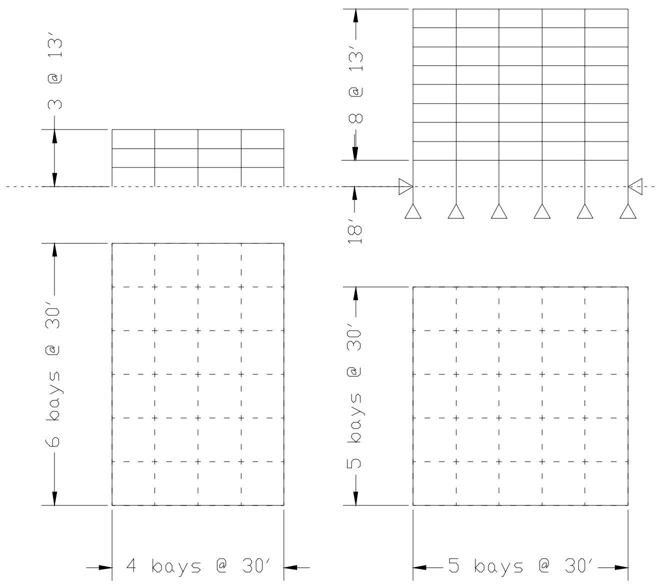

Figure 2 – General Building Configurations

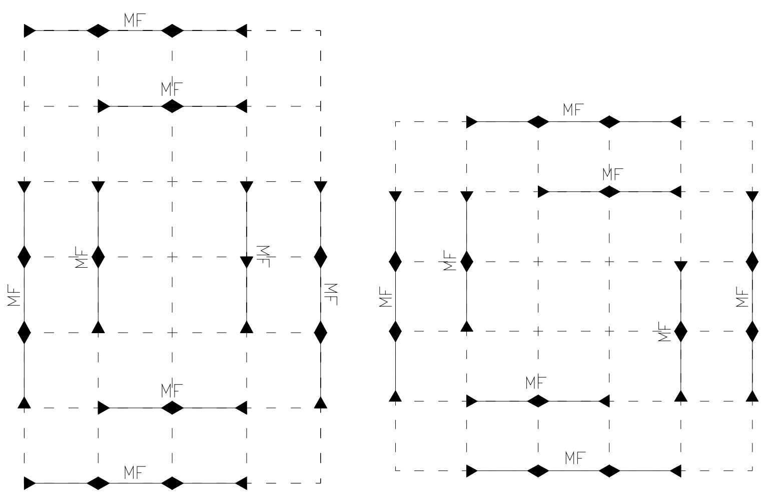

Figure 3 – 3 and 9 Story Moment Frame Configurations

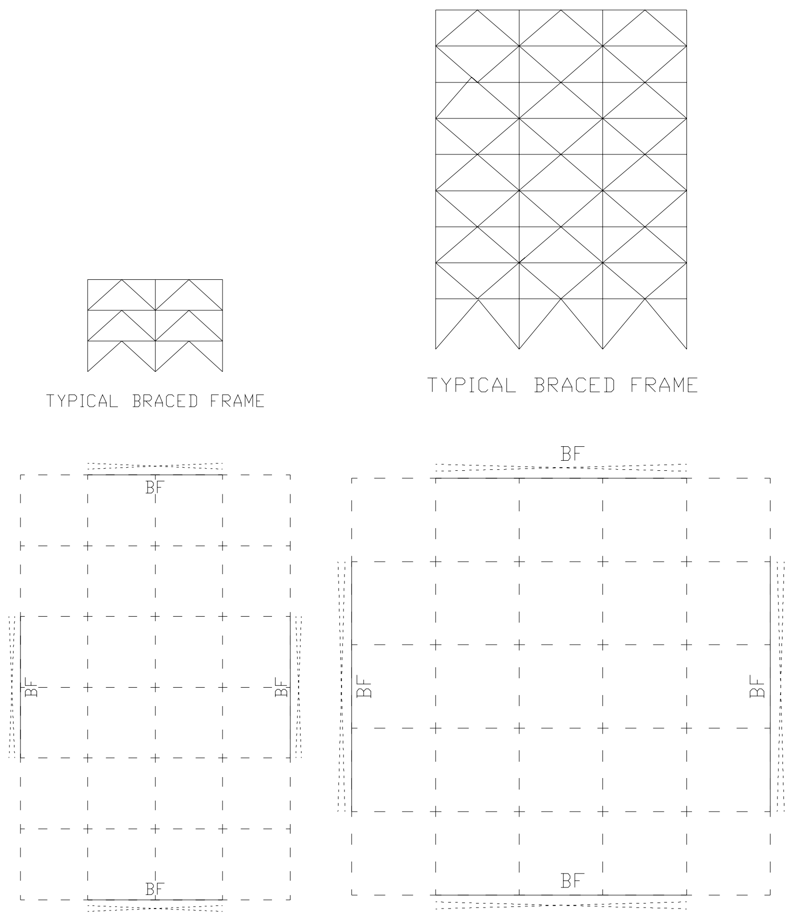

Figure 4 – 3 and 9 Story Braced Frame Configurations

The list below summarizes the designs performed for both the 3 and 9 story buildings and indicates the nomenclature used in this report:

- Moment frame (9MF or 3MF) controlled by 2% drift requirement. Figure 3 shows the moment frame configurations used. Moment frames extend the full building height.
- Braced frame with an R-factor of 7 and with 45 ksi yield strength steel BRBs (9BF-R7-45 or 3BF-R7-45S ksi). BRBs are trending towards higher yield stress materials after originally being introduced by Nippon steel as 22 ksi yield strength elements. Figure 4 shows the bracing configurations used.
- Braced frame with an R-factor of 7 and with 30 ksi yield strength aluminum BRBs (9BF-R7-30A or 3BF-R7-30A ksi). This design has approximately half the stiffness of the steel BRB and a yield displacement of twice the steel BRB. The plans and elevations used were identical to the steel BRBs described above.
- Moment frame (9MF-KD or 3MF-KD) designed to resist only 75% of the calculated base shear (V). KaZaK dampers were used to satisfy the 2% drift requirements. (For the 9 story building, the top 4 stories used 110 kip capacity dampers, next 4 stories used 220 kip dampers, and the first story used 260 kip dampers.) There were 2 bays of dampers on each perimeter of the plan.

The buildings were modeled using the 3D RAM PERFORM computer program recently developed by Professor Graham Powell of Berkeley and sold by RAM International. All of the building designs were done with RAM STEEL. Nonlinear time history analyses were performed on each building with the non-linear element being the actual BRB and its immediate surrounding frame members. In addition all results are compared with a full non-linear analysis of the moment frame design.

Each of the models was analyzed using a total of 20 time histories selected from those developed for Los Angeles as part of the SAC program. One set of 5 time histories is representative of a 50% probability of exceedance in 50 years design event for Los Angeles (100 year return period), 5 time histories are representative of a 10% probability of exceedance in 50 year design event for Los Angeles (500 year return period), 5 time histories are representative of a 2% probability of exceedance in 50 year design event for Los Angeles (2500 year return period) and 5 are representative of severe near fault ground motions. The results of each set of 5 are averaged. Due to space limitations only the more frequent 100 year and the 500 year design event results are presented.

DISCUSSION OF RESULTS

The results obtained from the 3 and 9 story building are quite different, especially from a performance based design perspective, and will be discussed separately.

There are two equally important variables that should be assessed when evaluating the seismic performance of a structural framing system. The first and almost universal variable is the inter-story drift. This is a code design parameter and is something most engineers focus upon during the design process. From a damageability perspective it is a measure that impacts damage to the framing system, building façade and windows, partitions, piping and ductwork. Intuitively it would be anticipated that the braced frame would have the best inter-story drift performance as it is a stiffer framing scheme. The other key parameter from a performance perspective is the floor acceleration as characterized by the floor response spectra. This is almost never assessed as part of the design process because it requires a time history analysis to obtain it. From a damageability perspective it is the measure that impacts damage to the ceiling and lights, electrical and mechanical equipment, elevators and the building contents.

For the 9 story building, the steel BRB (9BF-R7-45S) provides the best drift performance for both the 50% in 50 year and 10% in 50 year events. The more flexible aluminum BRB (9BF-R7-30A), as expected, causes an increase in the inter-story drifts with an unusual higher mode effect at the upper floor levels. The moment frame (9MF) and the 75% moment frame with aluminum dampers (9MF-KD) have similar drifts for the two design events with the moment frame producing slightly better drift performance. Compared to the steel braced frame, the drifts of both moment frames are higher. When the results of the four systems are compared based on the peak floor accelerations and floor response spectra the moment frame with the aluminum damper has the best performance closely followed by the moment frame. Both of these systems are measurably better than the two braced frame systems especially for the frequent 50% in 50 year event. As a consequence there appears to be a trade-off between better drift and acceleration related performance with the steel and aluminum braced frame systems providing better drift performance and the moment frame and the moment frame with the aluminum damper providing better acceleration performance.

For the 3 story building, the braced frames have larger inter-story drifts at the 1st floor level (especially as the magnitude of the design event increases) and although not shown herein, the 1st story drifts get as high as 5.5% for the 2% in 50 year event. The drift performance of the steel BRBs is again slightly better than the aluminum BRBs and is also better than the two moment frame designs at the 2nd and 3rd levels but significantly worse at the first floor level especially for the longer return period design events. When assessing the acceleration-related performance, the two braced frames (3BF-R7-45S and 3BF-R7-30A) have better performance than the two moment frame systems, with the aluminum BRB and the aluminum damper system providing slightly better performance than the steel BRB and moment frame respectively for both 50% in 50 year and 10% in 50 year design events.

SUMMARY AND CONCLUSIONS

The paper presents the comparative seismic performance of a new aluminum energy dissipation element that is designed first as a buckling restrained brace element using an R-factor of 7 and compared with the performance of steel buckling restrained brace design using the same R-factor. A second design was performed in which the aluminum damper was designed as an energy dissipation element using the 2000 NEHRP Energy Dissipation design requirements. In this case the moment frame is designed using 75% of the required design base shear for a full moment frame and the aluminum dampers are sized to satisfy the drift limits of 2%. The performance of this design is compared to a full moment frame design that satisfies the strength and drift requirements of the 2000 NEHRP provisions.

In assessing the performance of the new aluminum damper there are two considerations. First, if the element is designed as a buckling restrained brace it would compete and be compared with the steel BRB. For both the 3 and 9 story applications the trends are similar in that the steel BRB produces better drift performance and the aluminum BRB produces better acceleration-related performance. Second if the aluminum damper is designed as a damper and its performance compared to that of a moment frame, then the moment frame provides slightly better drift performance and the aluminum damped moment frame system has slightly better acceleration related performance. From a performance based design perspective these differences in drift and acceleration-related performance are not likely to have a significant impact on the total cost of earthquake damage for a given design event.

Another interesting comparison is the difference in performance between the braced frame systems and the moment frame and aluminum damped moment frame system. For the 3 story building it is clear that a building designed as a braced frame system with an R-factor of 7 produces better combined drift and acceleration performance than the 3 story moment frame systems for the 50% in 50 year and 10% in 50 year events. For the 9 story building the opposite is true. The combined drift and acceleration related performance of the moment frame systems is better than the performance of the braced frame systems.

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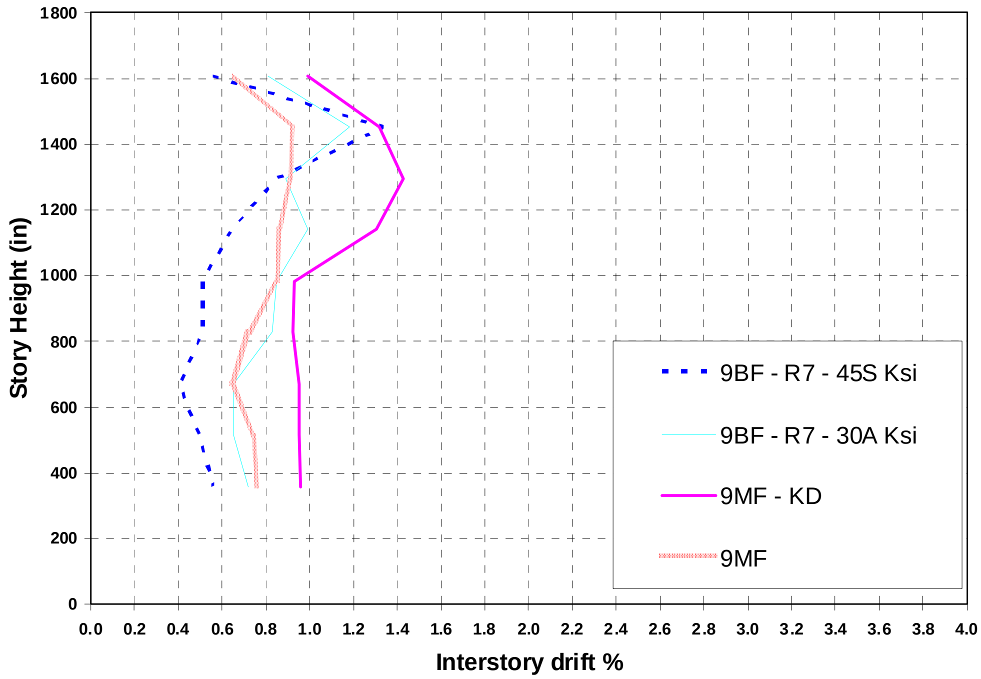

Figure 5 – Average inter-story drift for 9 story building for 50% in 50 year event

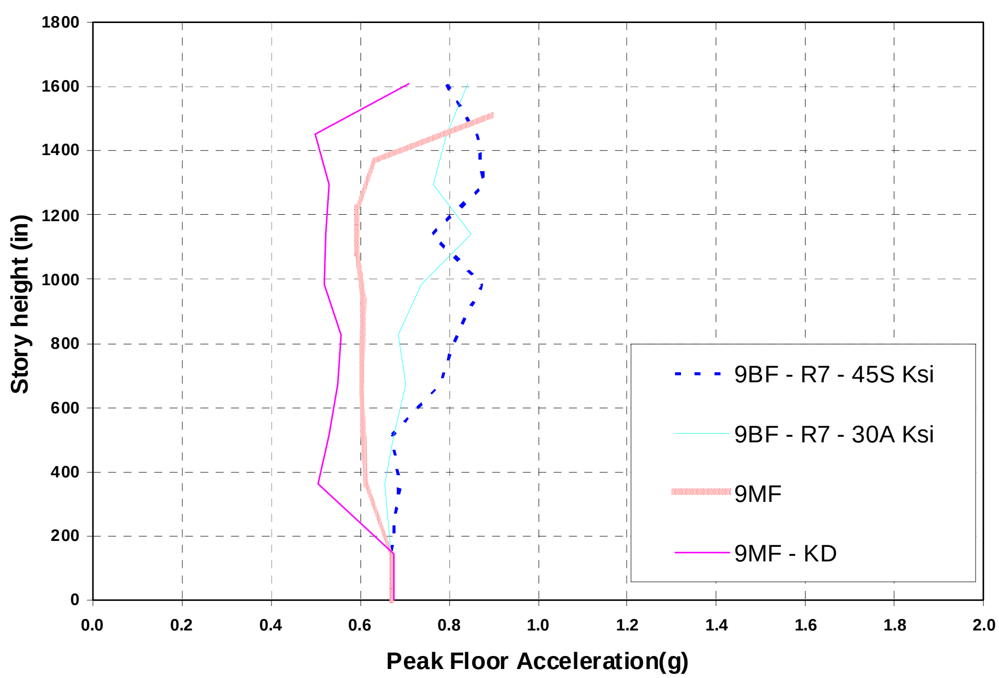

Figure 6 – Average peak floor acceleration for 9 story building for 50% in 50 year event

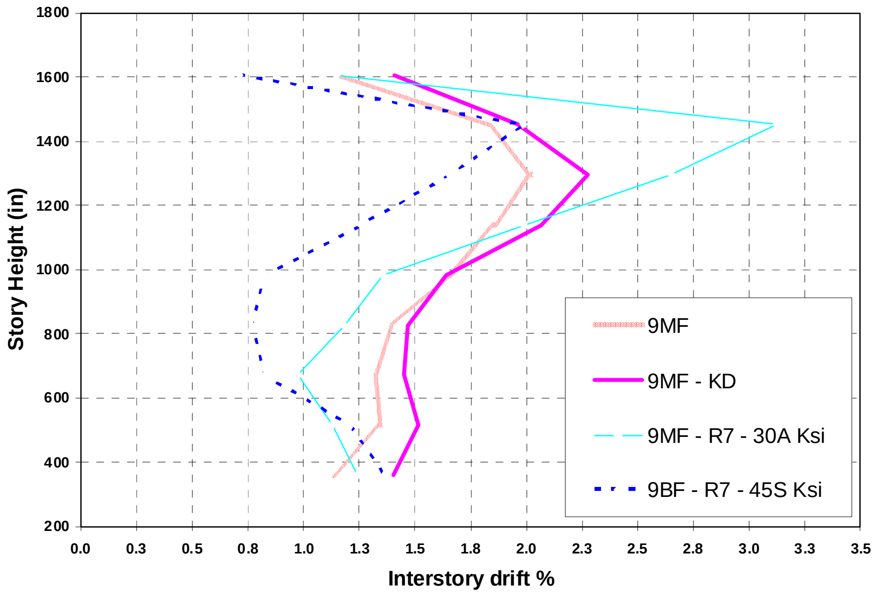

Figure 7 – Average inter-story drift for 9 story building for 10% in 50 year event.

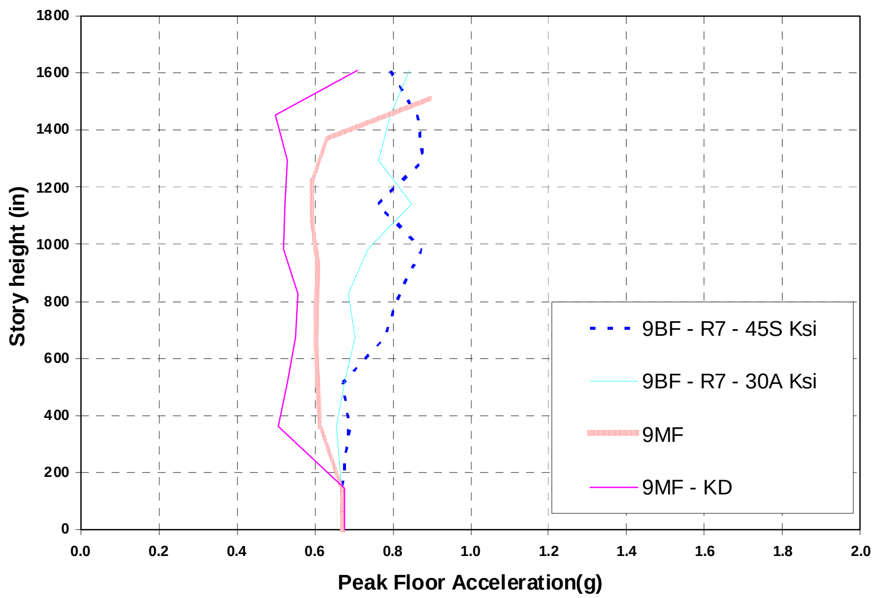

Figure 8 - Average peak floor acceleration for 9 story building for 10% in 50 year event

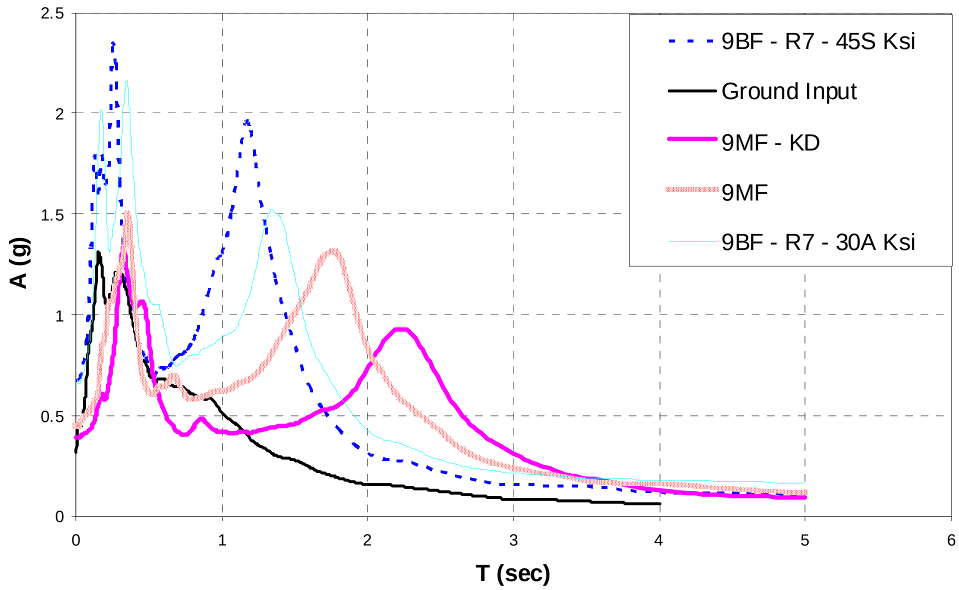

Figure 10 - Average 8th Floor Response Spectra for 9 story building for 50% in 50 year event.

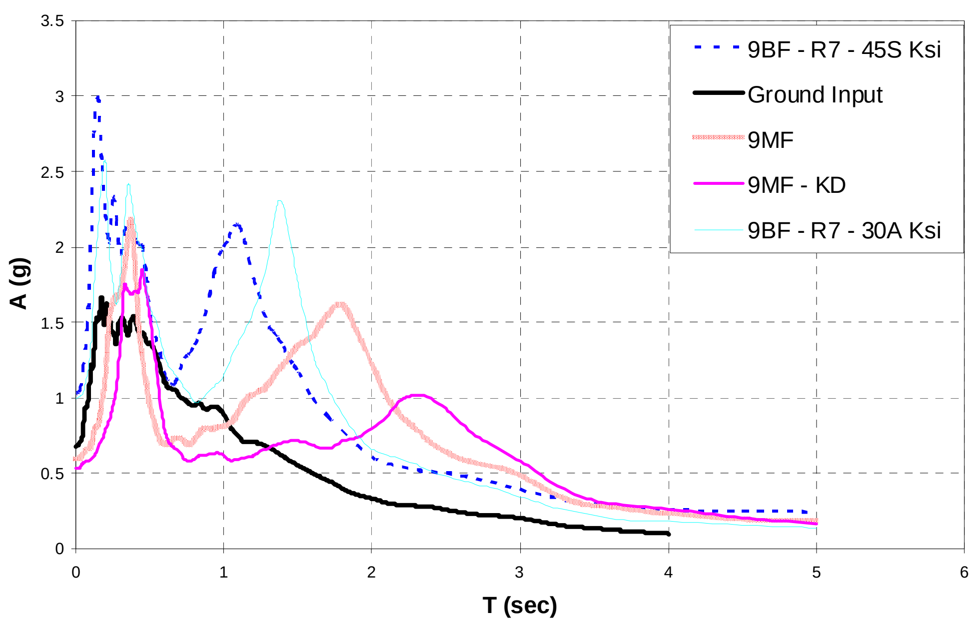

Figure 11 – Average 8th floor response spectra for 9 story building for 10% in 50 year event

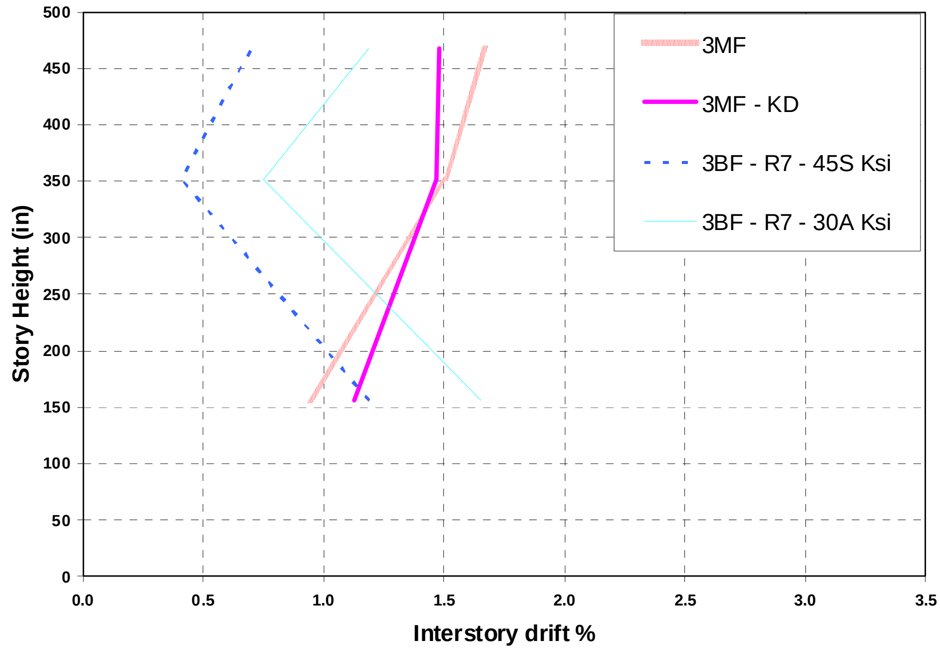

Figure 11 – Average inter-story drift for 3 story building for 50% in 50 year event

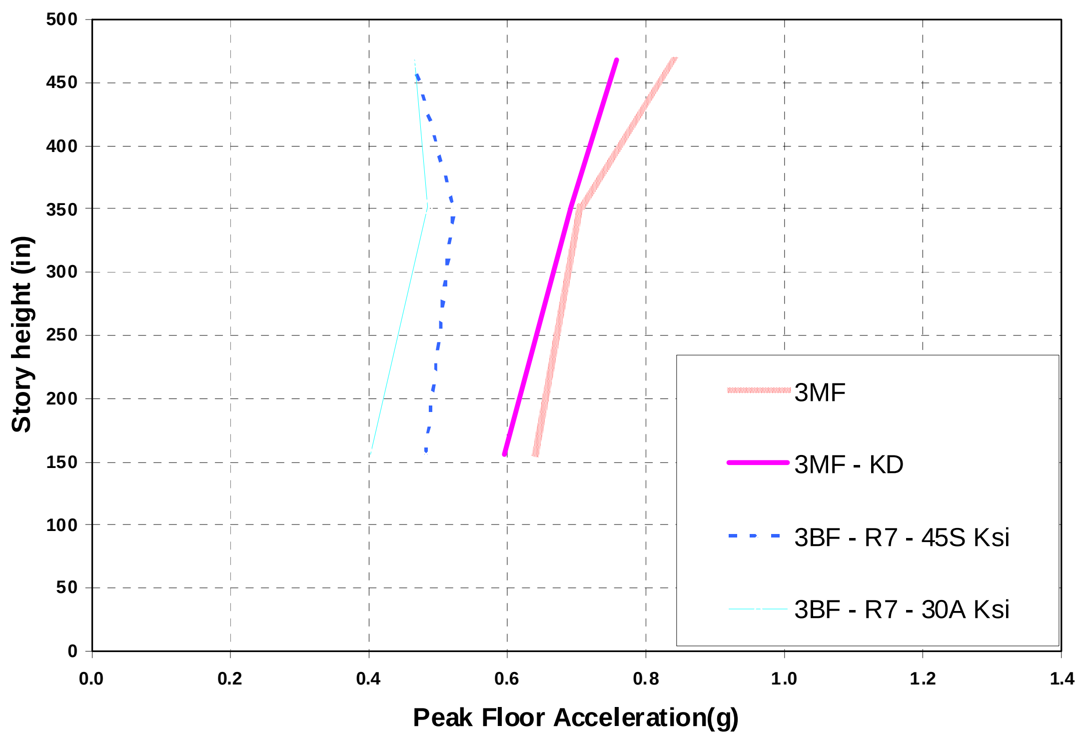

Figure 12 - Average peak floor acceleration for 3 story building for 50% in 50 year event

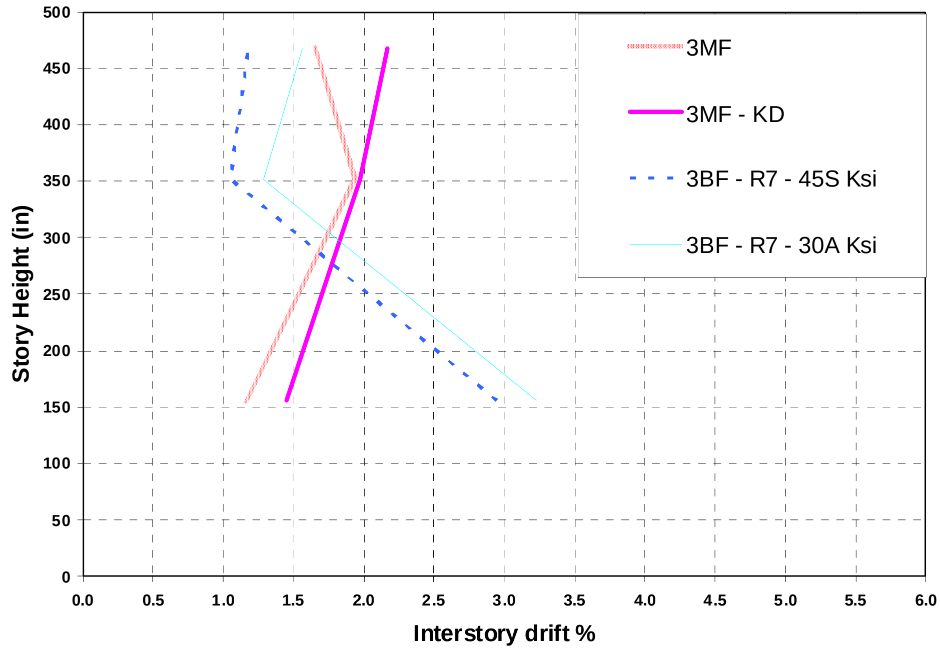

Figure 13- Average inter-story drift for 3 story building for 10% in 50 year event

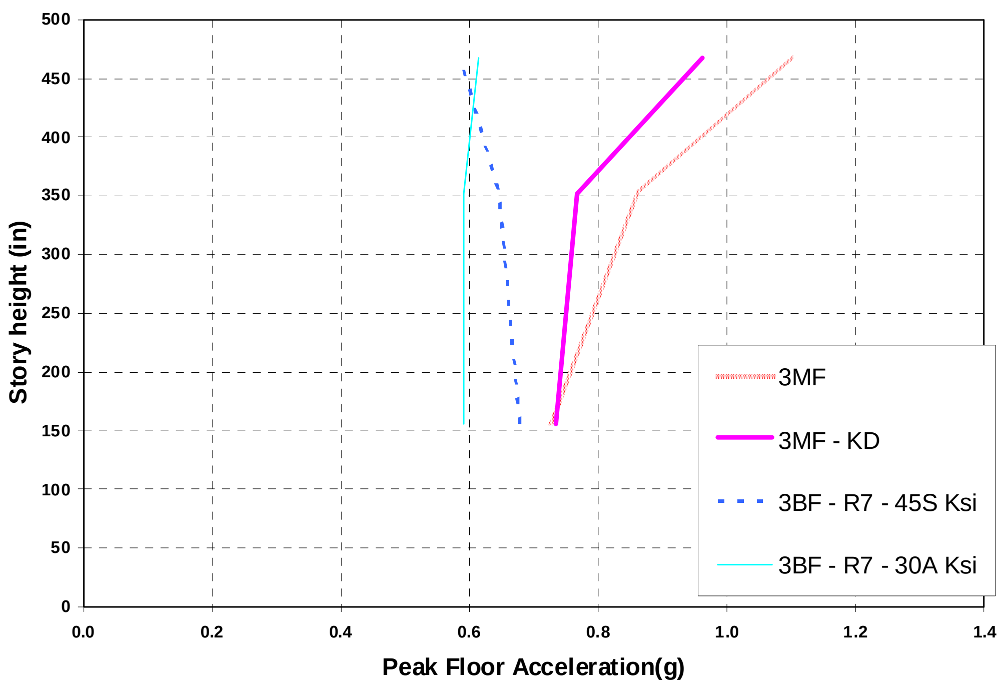

Figure 14 - Average peak floor acceleration for 3 story building for 10% in 50 year event

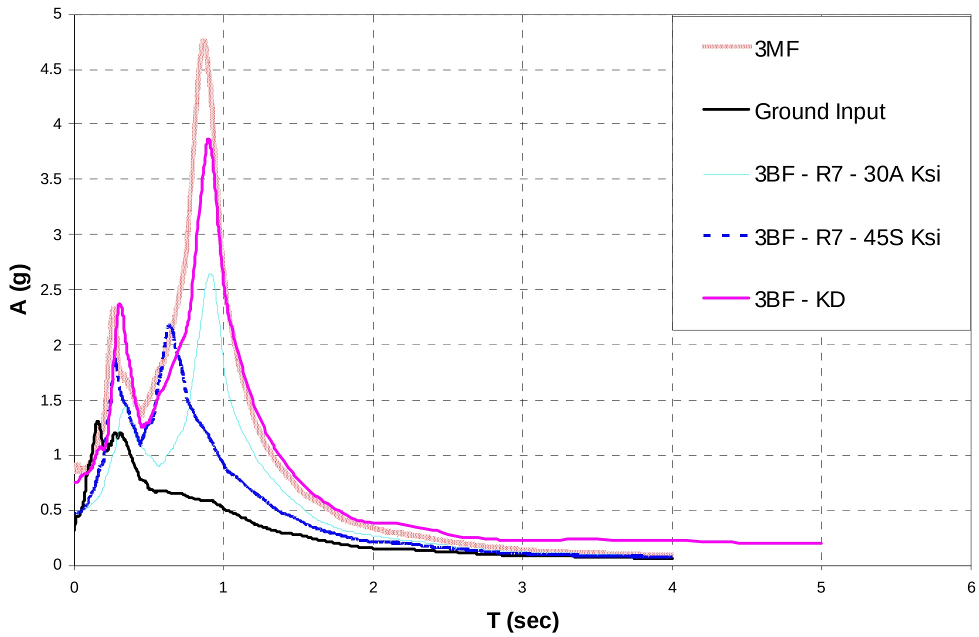

Figure 15 – Average 3rd floor response spectra for 3 story building for 50% in 50 year event

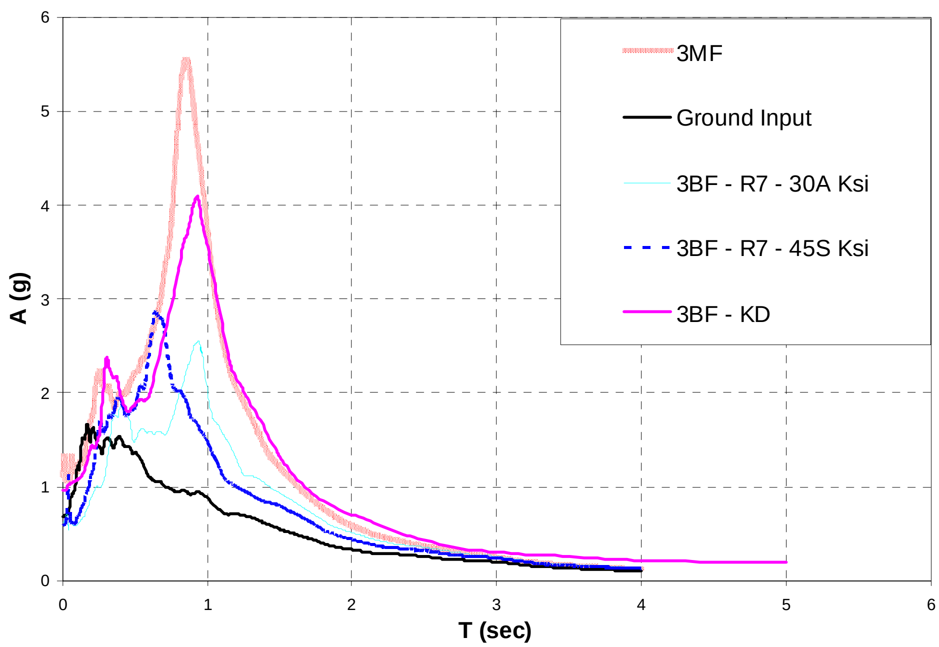

Figure 16 - Average 3rd floor response spectra for 3 story building for 10% in 50 year event