SYSTEM PERFORMANCE OF STEEL MOMENT RESISTING FRAME STRUCTURES

Helmut KRAWINKLER[1]

SUMMARY

As part of the SAC effort to find answers to the increased risk posed by recently observed connection failures in steel moments resisting frame (SMRF) structures, a series of coordinated analytical studies has been performed on the seismic performance of SMRF structural systems, investigating structures of different height (3, 9, and 20 stories), located in regions of different seismicity (Los Angeles, Seattle, and Boston), designed with different types of connections, and subjected to sets of ground motions representing different hazard levels. These studies have delivered a comprehensive set of information on the seismic performance of SMRF structures, insight into many important aspects that significantly affect seismic behavior, and quantitative information on, as well as procedures for the evaluation of, the effects of connection fractures on seismic safety. This paper focuses on the objectives and conclusions from several of the studies of this coordinated research project on the seismic performance of steel frame structures.

INTRODUCTION

The objectives of the studies carried out as part of the SAC system performance effort were to

- evaluate the range of interstory drift demands that can be expected in SMRF structures under different types of ground motions and at various hazard levels,
- appraise the effects of analytical models of various complexities on seismic demand predictions,
- quantify the effects of strength and stiffness deterioration on seismic demands,
- identify and quantify all issues that may have a significant impact on seismic performance,
- evaluate the effects of connection fractures on seismic performance, and
- assess the feasibility of using partially restrained connections as alternatives to welded rigid connections.

Five focused and coordinated analytical studies have been carried out at several universities to fulfill these objectives. The focus of the information presented here is on the main conclusions drawn from the individual studies, Some of the conclusions are taken verbatim from publications of other participants in this research, and some are the author's interpretation. The author takes full responsibility for the wording of the conclusions.

The analytical performance evaluation studies are based on designs of 3-, 9-, and 20-story model buildings, which follow local code requirements for Los Angeles (UBC'94), Seattle (UBC'94), and Boston (BOCA'93). The plan views and elevations of the model buildings are shown in Figure 1. Sections and loading and geometry details are presented in Gupta and Krawinkler 1999. All buildings were designed for a basic live load of 2.4 kPa (50 psf). The seismic mass for half of the 3-, 9-, and 20-story buildings, which are represented in the analytical models, is 1,475,000, 4,505,000, and 5,537,000 kg, respectively. All three design offices selected perimeter SMRFs as the structural system. The locations of the SMRFs are shown in solid and bold lines in the plan

[1] *Dept. of Civil and Environmental Engineering, Stanford Univ., Stanford, CA 94305-4020, e-mail: hk@ce.stanford.edu*

views. Designs with pre- and post-Northridge connection details were performed. Only analysis results of designs with fully welded pre-Northridge connections are discussed in this paper.

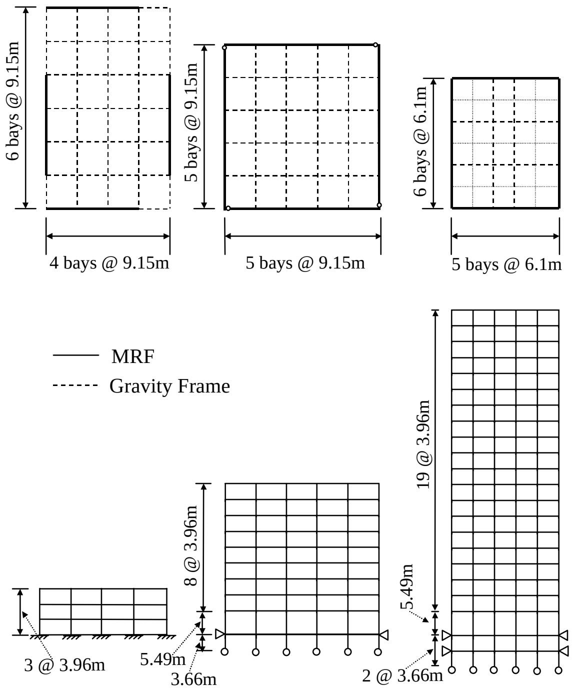


Figure 1. Plan Views and Elevations of the SAC Model Structures

For the three locations (Los Angeles, Seattle, Boston), sets of 20 records each were assembled that represent probabilities of exceedance of 2% and 10% in 50 years (2475 and 475 year return periods, respectively) [Somerville et al. 1997]. For LA an additional set for a probability of exceedance of 50% in 50 years (72 years return period) was assembled. From here on, these sets are denoted as 2/50, 10/50, and 50/50, for brevity. Most of the time histories are recorded pairs (two orthogonal components) that are scaled such that, on average, their spectral values match with a least square error fit to the USGS mapped values at 0.3, 1.0, and 2.0 sec., and a predicted value at 4.0 sec., for the appropriate return period and location. These periods are selected because they cover the range of interest for the flexible steel structures analyzed in this project. In addition, sets of records representing near-fault conditions and soft soil conditions were utilized to evaluate the effects of ground motion frequency characteristics. Median and individual spectra of the ground motions used in the studies are presented in [Gupta and Krawinkler, 1999].

SEISMIC PERFORMANCE OF DUCTILE SMRF STRUCTUES

A baseline study was concerned with the assessment of the seismic performance of "ductile" structures, in which it is assumed that no stiffness or strength degradation occurs and no connections will fracture. Most analyses were performed with 2-dimensional models. Point plastic hinges with a bilinear moment–rotation relationship with 3% strain hardening are used to model inelastic deformations at the end of flexural elements. M-P interaction is accounted for in the columns. A dummy column with appropriate vertical loads is placed in parallel with the SMRF in order to account for P-delta effects tributary to the interior gravity support system. Various analytical models are employed to evaluate the response sensitivity to modeling assumptions. Detailed results of studies that utilized static pushover analysis and time history analysis with the ground motion record sets associated with different hazard levels are presented in [Gupta and Krawinkler, 1999].

A few selected results are shown in Figures 2 to 4. Figure 2 shows global pushover curves for two different analysis models (M1 = centerline model, and M2 = model that incorporated panel zone strength and stiffness) of the three 20-story buildings. These curves illustrate the differences observed in global behavior and the large importance of P-delta effects, which in some cases lead to a rapid decrease in lateral load resistance at relatively small drifts. Figure 3 shows pushover curves for four models of the Seattle 3-story structure, which will be discussed later. Figure 4 presents the median and 84^{th} percentile of the maximum story drift demands at various hazard levels (50/50 for LA only, 10/50, and 2/50) for model M2 of all model structures.

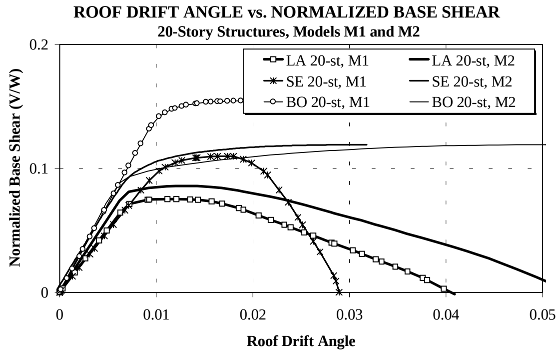


Figure 2. Pushover Curves for 20-Story Structures

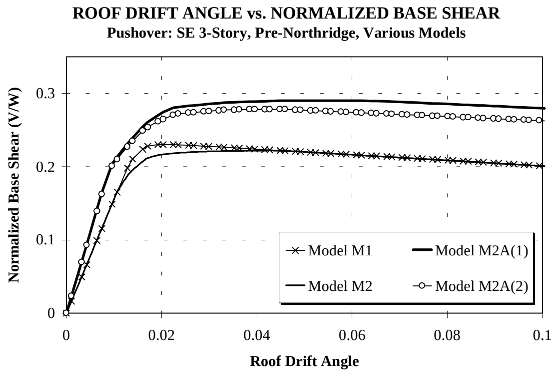
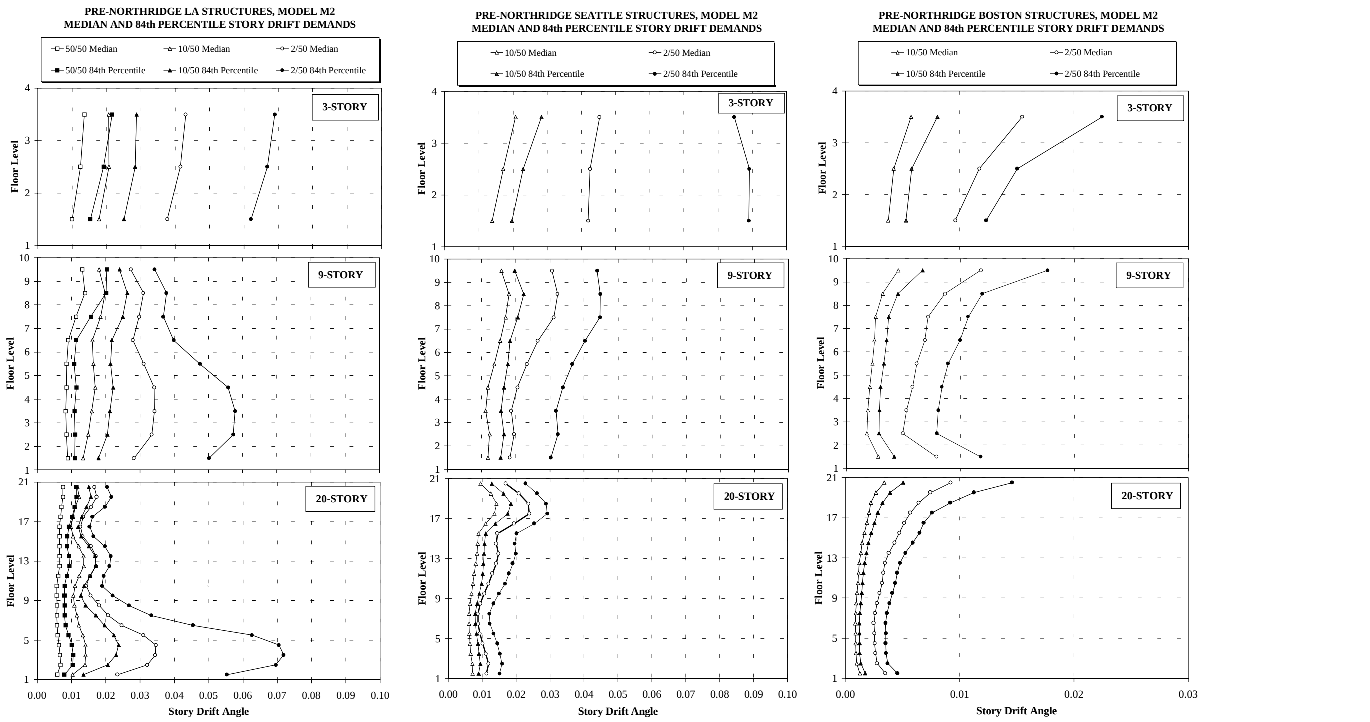


Figure 3. Pushover for 4 Models of Seattle 3-Story



LA Structures **Seattle Structures** **Boston Structures**

Figure 4. Statistical Values of Story Drift Demands at Various Hazard Levels

A few selected conclusions of this extensive study are (see Gupta and Krawinkler, 1999 and 2000 for details):

- Analytical modeling assumptions (M1 vs. M2 vs. an M2 model that incorporates also gravity frames [M2A]) may significantly affect the structural response for cases in which a change in mechanism occurs or the structure is driven into the range of negative post-yield stiffness.
- The expectation that beams attract most of the plastic deformation demands does not hold true if weak panel zones exist, which is a common condition for existing SMRF structures.
- The "strong-column" concept, as implemented in present design guidelines and codes, does not prevent the development of plastic hinges in the columns. Considering the distinct potential for column plastic hinging, it is essential to use only compact columns with proper lateral bracing.
- Structure P-delta effects may greatly influence the seismic response if the ground motion drives the structure into the negative post-yield stiffness range in any one story. If this possibility exists, the response may become very sensitive to modeling assumptions and structure and ground motion characteristics. For instance, for a specific long duration ground motion the maximum story drift of the Seattle 3-story models shown in Fig. 3 varied from 33% for model M1 to 18% for model M2 to 6.5% for model M2A(1) [Gupta and Krawinkler 2000].
- The performance of code designed SMRF structures is deemed to be in fair agreement with general expectations for conventional design events (10/50 ground motions) and more frequent events. For rare events (2/50 ground motions) larger than expected inelastic drifts may be experienced and the potential for unacceptable performance is not negligible.

- A pushover analysis is very helpful in assessing structural behavior and in estimating drift demands. It is particularly useful for estimating sensitivity to P-delta effects, as the analysis is able (in most cases) to identify and capture the drift range in which a negative post-yield story stiffness develops.

Effects of Special Ground Motion Characteristics and 3-D Effects

The effects on seismic demands of near-fault and soft soil ground motions and of vertical accelerations, as well as the sensitivity to 3-D effects (3-D modeling and orthogonal components of ground motion) were evaluated in a study at the University of Washington [MacRae, 1999, and MacRae and Mattheis, 2000]. Most of the near-fault records used in this study are unscaled versions of the records used in the LA 2/50 set. Only a few conclusions drawn from this study are summarized here:

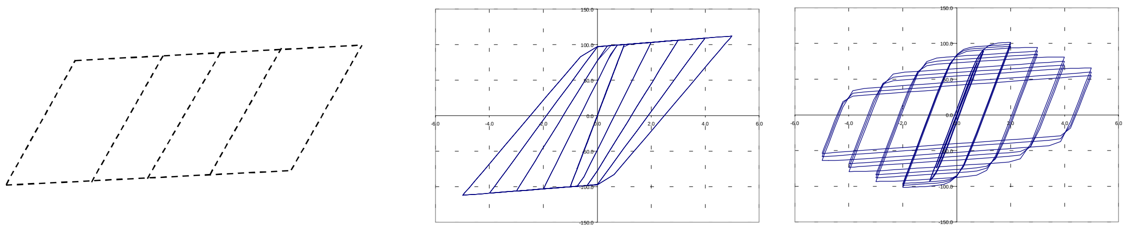
- Near-fault strike-normal components cause much larger drifts than strike-parallel components, and the largest drifts usually occur in the lower stories. The median drifts of the strike-normal components are larger than those of the 2/50 records.
- Near-fault records cause residual drifts that are, in average, larger than 50% of the maximum possible residual drift (maximum drift minus elastic drift).
- Column axial forces may be significantly affected by vertical acceleration. The SRSS method for predicting maximum column forces for combined horizontal and vertical motion is found to be non-conservative.
- 3-D analysis indicated small torsional effects. Drifts in the principal direction of shaking increased due to shaking in the orthogonal direction which resulted in flexural yield interaction at the column base.
- For shaking in the principal directions of the structure, the 3-D model sometimes experienced smaller drifts and other times larger drifts than the equivalent 2-D model.

EFFECTS OF HYSTERETIC CHARACTERISTICS ON STORY DRIFT DEMANDS

The baseline studies were performed with bilinear element hysteretic models that exhibit ductile and stable (non-degrading) cyclic behavior regardless of the level of deformation. Tests on beam-to-column assemblies show that significant degradation in stiffness and deterioration in strength may occur at beam plastic hinge locations within the level of rotations expected in severe earthquakes. This occurs especially in post-Northridge designs in which the plastic hinge is forced away from the connection in order to protect the beam flange welds. A typical test result for a beam with a cover plated connection together with a superimposed bilinear model is shown in Fig. 5(a).

The sensitivity of seismic performance to hysteretic characteristics was investigated in a separate project [Naeim et al., 1999]. Deteriorating models of the types shown in Figs. 5(b) and (c), as well as models with significant pinching of loops were employed in the time history analysis of all pre-Northridge structures. The results of this study are still in the evaluation state, and only the following preliminary observations in regard to the effects of hysteretic characteristics on story drifts can be made at this time:

- The effects are small for all but very severe ground motions that demand large plastic rotations. Thus, the following observations pertain to the latter cases only.
- Even severe pinching of hysteresis loops seems to have only a small or at worst a moderate effect for all structures.
- Severe stiffness degradation of the type shown in Fig. 5(b) seems to have an effect whose pattern is difficult to establish. For very severe ground motions the effect appears to be very large for the 3-story LA and Seattle buildings.
- Strength deterioration of the type shown in Fig. 5(c) appears to amplify story drifts by only about 10 to 20%. This result is surprising and may be a function of the details of the deterioration model used in this study. For instance, the post-elastic stiffness of the model never deteriorates.



(a) Test Results versus Bilinear Model **(b) Severe Stiffness Degradation** **(c) Severe Strength Deterioration**

Figure 5. Hysteretic Behavior with Degrading Characteristics

EFFECTS OF CONNECTION FRACTURES ON SMRF STORY DRIFT DEMANDS

A pivotal project of the system performance effort was concerned with the effect of connection fractures on drift demands [Cornell and Luco, 1999, and Luco and Cornell, 2000] and with a methodology for a probabilistic assessment of the effects of connection fractures on structural safety [Luco and Cornell, 1998]. Only the first aspect is briefly summarized here.

Nonlinear dynamic analysis was performed on the LA and Seattle structures, using the 10/50 and 2/50 ground motion sets and centerline analytical models (M1). The drift demands for structures with fractured connections ("brittle" cases) were compared with demands for ductile structures (no fractures permitted) in order to assess the effects of connection fractures. The moment-rotation model shown in Fig. 6 was used to represent pre- and post-fracture behavior at connections. This model [Foutch and Shi, 1996] assumes that the bending strength after fracture drops to a fraction (M_{red}) of the plastic bending strength M_p, and that under reversal the full bending strength is recovered once a crack closes. Fracture may occur at the bottom flange only (BFO case) or at both the top and bottom flanges (TBF case). Fracture may occur at an assigned plastic rotation (θ_f), which may be different for positive and negative bending, or it may occur in the elastic range at an assigned fraction of the bending strength M_p.

A brittle base case, which considers bottom flange fractures only (BFO) and is used as a reference case, is defined as follows: 25% of the beam bottom flanges fracture elastically (with random distribution of location) at a moment equal to $0.75M_p$, and all other bottom flanges fracture at a plastic rotation of 0.015. The post-fracture strength is assumed to be $0.3M_p$. Several variations to this base case were investigated, with the following variations being most relevant:

- Similar to BFO base case, but 75% of the fractures occur elastically at $0.75M_p$
- Similar to BFO base case, but plastic fractures occur at a plastic rotation of 0.005
- Similar to BFO base case, but plastic fractures occur at a plastic rotation of 0.03
- TBF case, with bottom flange base case and top flange fractures occurring at a plastic rotation of 0.045
- TBF case, with bottom flange base case and top flange fractures occurring at a plastic rotation of 0.015

Selected results of the nonlinear time history analyses are shown in Figs. 7 to 9. These results serve to support some of the following conclusions drawn from this study:

- The effect of BFO fractures is benign in most cases. As Fig. 7 shows, even for the severe 2/50 ground motions the effect is not significant in the median. The residual story shear strength after fracture appears to be sufficient to prevent large amplification of the story drift – in most cases.
- The effect of BFO fractures becomes important only for cases in which the drift demand is already very large for the ductile structure. Figure 8 shows the percentage of cases of "extreme" story drift for the ductile structures and the BFO base cases, using the LA 2/50 set of ground motions. Extreme is defines as the maximum story drift exceeding a value of 0.10. In fact, the percentage of extreme story drift at least doubles from the ductile case to the brittle case for all model structures that experience extreme drifts.
- Even when it is assumed that 75% of the bottom flange connections fracture elastically at $0.75M_p$, the effect on story drift does not change by much compared to the base case.
- Decreasing the connection plastic rotation capacity for the BFO case from 0.015 to 0.005 has less than 15% effect on drift demands at the median and the median plus sigma level. An even smaller effect (benefit) is obtained when the capacity is increased to 0.03.
- Decreasing the post-fracture strength for the BFO base case from $0.3M_p$ to $0.1M_p$ increases the median and 1-sigma drifts by less than 20%.
- Permitting top flange fractures in addition to the base case bottom flange fractures (TBF cases) may have a significant effect on the maximum story drifts. Except for "extreme" story drift cases, the effect is important only if the plastic rotation capacity for top flange fractures is relatively small (0.015).
- Based on results of the type shown in Fig. 9, Luco and Cornell [2000] conclude that the ductile case story drift demands can be used to anticipate at what plastic rotation capacity TBF connection fracture is expected to significantly increase the story drift demands.
- Luco and Cornell [2000] conclude that for "mild" ground motions (of the 10/50 type) the anticipated (median) effect of connection fractures is minimal because the demands are not large enough to induce more than a few fractures. Under "moderate" ground motions (most of the 2/50 records) BFO fractures are expected to have a relatively small effect on the median drift demands. The effect of TBF fractures is also expected to be small unless the plastic rotation capacity associated with top flange fracture is smaller than about the story drift demands in the ductile case minus 0.01 (elastic rotation). If the latter occurs, a significant number of top flange fractures can be expected and the story drift demands may increase significantly. For "rogue" records, which cause very large story drifts in the ductile case, even the BFO

base case may cause extreme story drifts. TBF cases will only exacerbate the situation, particularly if the plastic rotation capacity for top flange fractures is small.

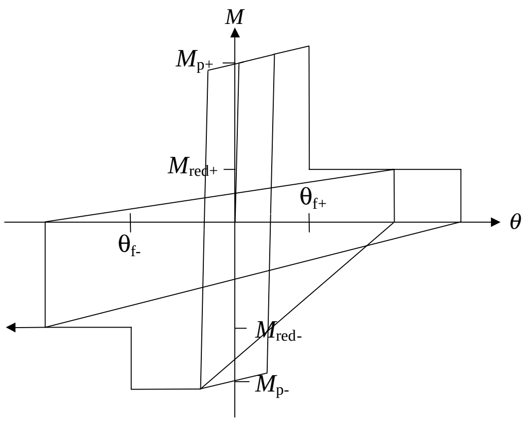


Figure 6. Hysteresis Model for Connection with Top and Bottom Flange Fractures [Luco & Cornell, 2000]

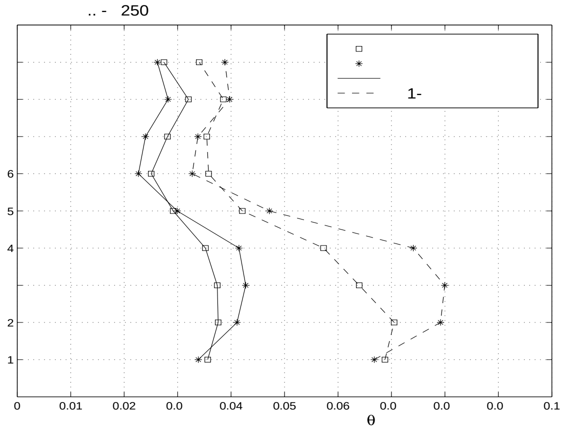


Figure 7. Effect of Connection Fractures on Story Drift Demands for LA 9-Story Structure, 2/50 Records, BFO Base Case [Luco & Cornell, 2000]

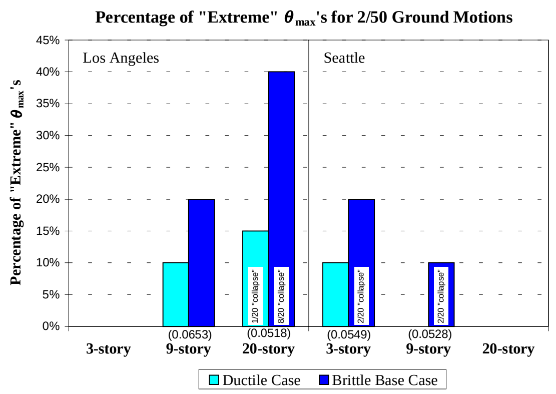


Figure 8. Percentage of Extreme Story Drifts for 2/50 Record Sets [Luco & Cornell, 2000]

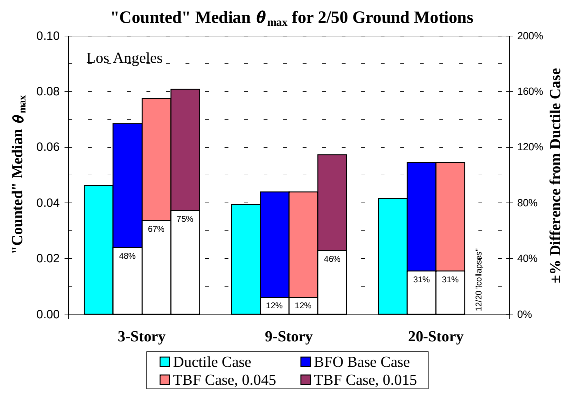


Figure 9. Median Effect of Connection Fractures on Story Drift, LA 2/50 Records [Luco & C., 2000]

PERFORMANCE OF FRAMES WITH PARTIALLY RESTRAINED (PR) CONNECTIONS

The objective of this study [Maison and Kasai, 1998, Kasai et al., 1999] was to assess the viability of construction with partially restrained (PR) connections in regions of moderate and high seismicity. Case studies of a 3-story building in LA and a 9-story building in Seattle were carried out. The buildings have the same layout as the base case buildings shown in Fig. 1, and are designed in accordance with the 1994 UBC.

As is customary for most frame structures with PR connections, all frames in the NS and EW directions were of the PR type, thereby greatly increasing the number of moment resisting connections compared to the base case structures with FR perimeter frames, and eliminating simple connections. For the LA 3-story building a combination of “composite” PR connections (bottom seat angles but only composite slab on top), and top and seat angle connections were used, and for the Seattle 9-story building stiffer T-stub connections were used for all connections. . In all cases the connection strength was much smaller the beam bending strength, usually less than half of M_p. A typical hysteresis model used in the analytical study [Maison et al., 2000] is shown in Fig. 10.

The focus of the study was on assessing the feasibility of PR designs and evaluating differences in seismic performance. Drifts were evaluated and compared to FR designs at various hazard levels, and emphasis was placed also on an assessment of collapse safety. The latter assessment has the same caveat as the base case study, namely that no strength or stiffness degradation is assigned to the element models.

Global pushover curves for the two structures, and their base case counterparts with FR connections, are shown in Fig. 11. Comparisons of median values of drifts of PR and FR structures are shown in Fig. 12 at the 50/50, 10/50, and 2/50 hazard levels for LA, and the 10/50 and 2/50 hazard levels for Seattle.

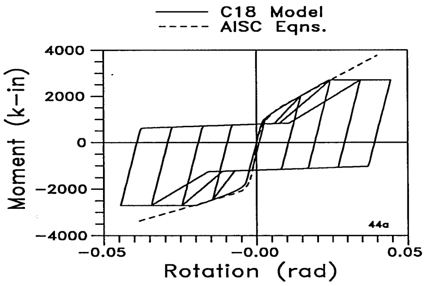


Figure 10. Typical Hysteresis Model for a Comp. PR Connection [Maison & K., 1999]

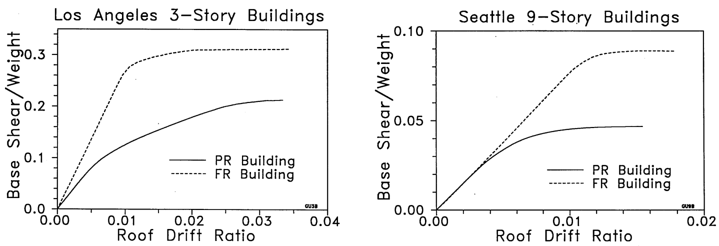
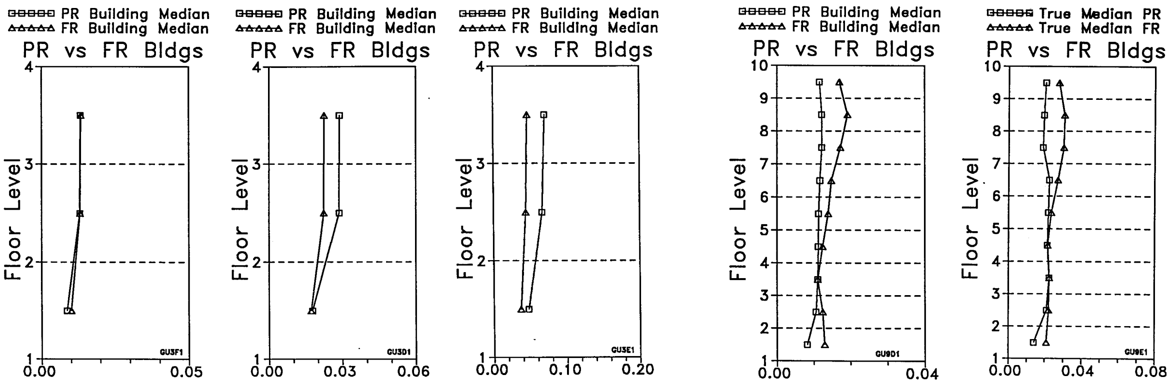


Figure 11. Global Pushover Curves for Structures with PR and FR Connections [Maison & Kasai, 1999]



LA-3, 50/50 LA-3, 10/50, LA-3, 2/50 SEATTLE-9, 10/50 SEATTLE-9, 2/50

Figure 12. Comparison of Median Drifts of Structures with PR and FR Connections [Maison & Kasai, 1999]

From these figures, and other results presented in the references, the following main conclusions are drawn.

- There is significant potential to design PR structures whose seismic performance is comparable to that of FR structures.
- It appears to be feasible to design PR structures for about the same elastic stiffness as FR structures.
- Even if the “yield strength” of PR structures is significantly lower than that of FR structures, the drift demands at high and low hazard levels may be comparable to those of FR structures.
- At a low hazard level (50/50 in LA) the low strength of PR structures may lead to notable plastic rotation demands. However, these demands are concentrated in the PR connections, which can tolerate limited plastic demands without the need for repair.
- The plastic rotation demands for PR connections are usually larger than those for FR connections (even if the drift demands are the same), and may be very large during very severe events. This calls for careful deformation-based design of PR connections and for extensive testing of such connections (particularly for deep beams).
- The design of PR structures requires much attention to issues that are of less concern in FR structures. The plastic rotation capacity of PR connections is very sensitive to design detailing. The rotation demands will depend on the connection strength, whose relationship to the beam bending strength is one of the basic variables in PR design.
- In many aspects PR structures behave differently than FR structures, and their seismic response depends on many more parameters. For instance, column restraint depends on connection stiffness, which in turn affects stability considerations. Dynamic story shears may be a multiple of the shears obtained from the static pushover analysis. It was also found that in a few cases the story drifts of PR structures become much larger than those of FR structures, and that dynamic instability was approached in a case in which the FR structure exhibited stable behavior.

- PR structures show much promise as feasible systems in regions of high seismicity, but there are many issues that need to be better understood to provide confidence in their design that is comparable to that for FR designs [provided that connection fractures are prevented in the latter].

CONCLUSIONS

Conclusions have been presented throughout this paper. In the context of global seismic performance, the following statements summarize the author's perspective:

- At high performance levels (e.g., serviceability), the performance predictions confirm the expectation of only little structural damage. The postulated fracture scenarios had no noticeable effect on performance.
- At the code design performance level (life safety, 10/50 hazard level), performance of ductile structures was also in line with expectations. . The postulated fracture scenarios had some, but not a very strong, effect on performance.
- Under rare events (2/50 hazard level), performance (in this case collapse safety) becomes sensitive to P-delta effects, deterioration in hysteretic behavior, and connection fractures. Providing adequate collapse safety remains the most challenging aspect of design of SMRF structures.

ACKNOWLEDGEMENTS

This work was conducted pursuant to a contract with the Federal Emergency Management Agency (FEMA) through the SAC Joint Venture. The opinions expressed in this paper are those of the author. No warranty is offered with regard to the results, findings and recommendations contained herein, either by FEMA, the SAC Joint Venture, the individual joint venture partners, their directors, members or employees, or the author of this publication.

REFERENCES

Cornell, C.A. and Luco, N. (1999), "The Effect of Connection Fractures on Steel Moment Resisting Frame Seismic Demands and Safety", SAC phase II project report, *NISEE – UC Berkeley*.

Foutch, D.A. & Shi, S. (1996), "Connection Element (Type 10) for DRAIN-2DX", *University of Illinois*.

Gupta, A., and Krawinkler, H. (1999). "Seismic Demands for Performance Evaluation of Steel Moment Resisting Frame Structures", *John A. Blume Earthquake Engineering Center Report No. 132*, Department of Civil Engineering, Stanford University.

Gupta, A., and Krawinkler, H. (2000), "Dynamic P-delta Effects for Flexible Inelastic Steel Structures", *ASCE Journal of Structural Engineering*, Vol. 126.

Kasai, K., Maison, B.F., and Mayangarum, A. (1999), "Effects of Partially Restrained Connection Stiffness and Strength on Frame Seismic Performance," *SAC phase II project report, NISEE – UC Berkeley*.

Luco, N. and Cornell, C.A. (1998), "Effect of Random Connection Fractures on the Demands and Reliability for a 3-Story Pre-Northridge SMRF Structure," *Proceedings of the 6th U.S. National Conference on Earthquake Engineering*, Seattle, Washington.

Luco, N. and Cornell, C.A. (2000), "Effects of Connection Fractures on SMRF Seismic Drift Demands," *ASCE Journal of Structural Engineering*, Vol. 126.

MacRae, G. A. (1999). "Parametric Study on the Effect on Seismic Demands of Ground Motion Intensity and Dynamic Characteristics," *SAC phase II project report, NISEE – UC Berkeley*.

MacRae, G.A., and Mattheis, J.E. (2000), "Inelastic Behavior of a Three-Dimensional Steel Moment Resisting Building to Near-Fault Ground Motions," submitted to *ASCE Journal of Structural Engineering.*

Maison, B.F., and Kasai, K. (1999), "Seismic Performance of 3 and 9 Story Partially Restrained Moment Frame Buildings," *SAC phase II project report, NISEE – UC Berkeley*.

Naeim, F. Skliros, S., Reinhorn, A.M., and Sivaselvan, M.V., (1999). "Effect of Hysteretic Deterioration Characteristics on Seismic Response of Moment Resisting Steel Structures," *SAC phase II project report, NISEE – UC Berkeley.*

Somerville, P., et al. (1997). "Development of Ground Motion Time Histories for Phase 2 of the FEMA/SAC Steel Project," *SAC Background Document, Report No. SAC/BD-97/04, EERC – UC*