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INFLUENCE OF DIFFERENT TYPES OF FULLY RESTRAINED CONNECTIONS ON THE RESPONSE OF SMRF STRUCTURES

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SUMMARY

The behavior, response, and performance of code-compliant designs employing a) standard pre-Northridge beam-to-column connections, b) cover-plate connections, and c) reduced beam section connections are discussed. The element and system level demands are presented for 9-story structures designed per recent code provisions. These demands are estimated from nonlinear time history analysis for sets of ground motions representative of a 2475 year return period. The system level behavior, expressed in terms of story drift demands, is not significantly affected by the choice of connection type, except in isolated cases. The element level behavior, expressed in terms of element plastic deformation demands, is sensitive to the connection type and to subjective decisions made by the design engineer. Capacity estimates are obtained from data collected from experimental tests for the different connection types. These estimates are preliminary but provide insight into expected behavior of different types of connections.

INTRODUCTION

The confidence in steel moment resisting frame (SMRF) structures with fully restrained (FR) welded beam-to-column connections was greatly shaken when during the 1994 Northridge earthquake more than 150 SMRF structures suffered damage, primarily in the form of brittle fracture of the welded beam-to-column connections. One of the reasons for this unexpected failure mode was the development of high stress states at the column face, which, coupled with deficiencies in the weld metal toughness and in the workmanship of the welds, among other factors, resulted in the brittle fractures. Emergency guidelines published in 1995 [FEMA 267, 1995] provide recommendations to mitigate this problem through the use of notch-tough weld metal, more stringent quality control, relocation of the zone of beam plastification away from the column face in order to reduce the stresses at the column face, and other improvements.

The focus of this paper is on an evaluation of element and system level seismic demands for designs employing three different fully restrained (FR) connection types. The first type (pre-Northridge) employs the standard beam-to-column connection detail prevalent prior to the Northridge event, which is the type damaged in the event. This type is used as the reference. The second type (post-Northridge_1) shifts the zone of plasticity away from the column face by increasing the strength near the column face by welding cover-plates to the beam flanges over a small length of the beam. The third type (post-Northridge_2) uses the reduced beam section (RBS) concept for forcing the beam plastification away from the column face.

The behavior and response of 9-story model structures designed for the SAC steel project is studied. Three seismic locations are considered, representing different seismic zones in the US. The ground motions used for the study represent a hazard level with 2% probability of being exceeded in 50 years [referred to as 2/50; return period of 2475 years]. Drift demand estimates are obtained from nonlinear time history analyses under the assumption that all connections remain ductile and do not fracture. Capacity estimates are obtained from data of 104 tests, which provide indicative values for the expected drift capacity of the different connection types.

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The results of the study demonstrate that the choice of the connection type (along with other issues such as material strength and subjective design decisions) greatly affects the distribution of plastic deformation demands between the elements at a connection. The demands may be concentrated in either the beams or the panel zones, or may be distributed amongst the two types of elements. Provided that connections do not deteriorate or fracture, the system level demands (e.g., story drifts) are not significantly affected by the choice of connection type, except in isolated cases. From a performance perspective, on average, the estimated demands compare favorably with the capacities for post-Northridge designs. Pre-Northridge connection capacities, even for joints welded using electrodes with improved notch-toughness (e.g., E70TG-K2 or E71T-8), likely are inadequate to sustain the demands imposed at the 2/50 hazard level.

STRUCTURES, MODELING, AND GROUND MOTIONS

The 9-story structures used in this study have been designed as part of the SAC steel project. These buildings conform to local code requirements; the 1994 Uniform Building Code (UBC) in Los Angeles (LA) and Seattle (SE), and the 1993 National Building Code (BOCA) in Boston (BO). The floor plan and elevation of the structures are shown in Figure 1. All buildings are office buildings designed for gravity, wind, and seismic loads, with a basic live load of 2.4 kPa (50 psf). The mass and gravity loading are the same for the different seismic regions. The structural system for all buildings consists of steel perimeter moment frames and interior gravity frames with simple (shear) connections. Member sections, loading, and geometry details for the structures are given in [Gupta and Krawinkler, 1999].

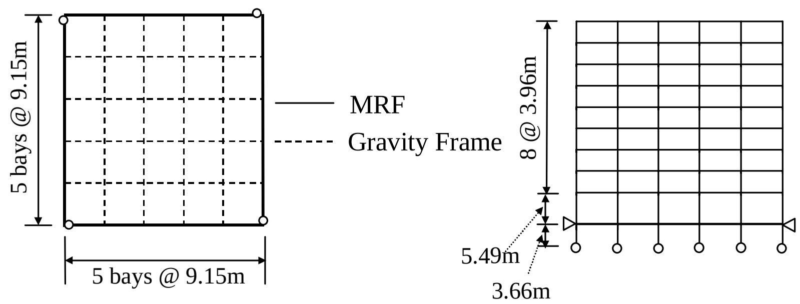


Figure 1. Plan View and Elevation of the 9-story SAC Model Structures

Pre-Northridge and post-Northridge_1 designs are studied for all three locations, while for Seattle the post-Northridge_2 design is studied in addition. The beam and column sections are very similar for the pre- and post-Northridge designs, resulting in only small variations in lateral stiffness and first mode period (see Table 1) for the different designs. The analytical evaluation is carried out using two dimensional bare frame models (referred to as model M2) to represent the buildings. These models include only the perimeter moment resisting frames, use clear span dimensions, and account for the strength and stiffness of the panel zones. Strength and stiffness degradation in elements, and fracture of welded connections are not considered. Structure P-delta effects are included in the analysis. Inelastic behavior of beams and columns is represented with a bilinear point hinge model with a constant strain hardening of 3%. The panel zone shear force - shear distortion relationship is assigned a trilinear model, as given in [Krawinkler, 1978]. The post-Northridge designs are modeled as shown in Figure 2. The plastic hinge is assumed to form at a distance of $d_b/4$ (d_b being the depth of the beam) away from the edge of the cover-plates for the cover-plate connections, and at the center of the reduced section for the RBS connections. Details of the analytical modeling are given in [Gupta and Krawinkler, 1999].

Table 1. First Mode Periods for Pre-and Post-Northridge Designs

	LA	Seattle	Boston
Pre-Northridge	2.24	3.06	3.30
Post-Northridge_1 (Cover-Plates)	2.21	3.06	3.17
Post-Northridge_2 (RBS)		3.10	

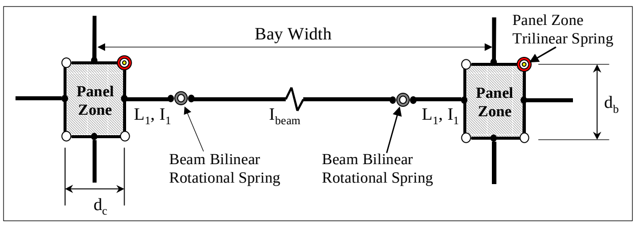


Figure 2. Analytical Model for Post-Northridge Bay

The analytical models of the structures are subjected to sets of 20 ground motion records representative of a 2475 year return period hazard level [2/50] at the three seismic locations [Somerville, 1997]. The median elastic strength and displacement demand spectra, for 2% damping, for the sets of ground motions used in this study are shown in Figure 3 and Figure 4. Individual spectra of the ground motions are presented in [Gupta and Krawinkler, 1999].

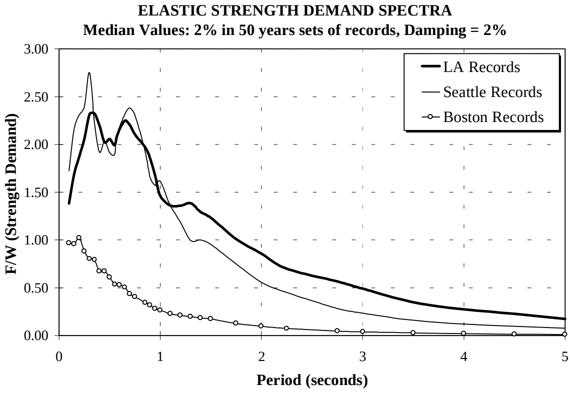


Figure 3. Median Strength Demand Spectra

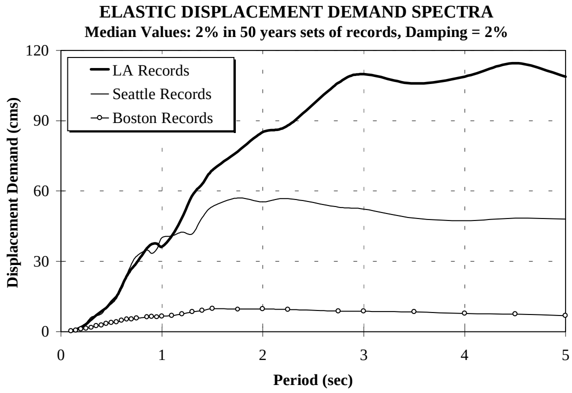


Figure 4. Median Displacement Demand Spectra

SEISMIC BEHAVIOR OF DIFFERENT DESIGNS AT THE SYSTEM LEVEL

The first mode period for the different designs, as given in Table 1, indicates that the change in connection type does not significantly affect the stiffness of the structure. The closeness of the fundamental periods can be attributed to

1) the small effect the addition of cover-plates [or the reduction in beam flange area in the RBS design] has on the overall stiffness of the structure, and
2) only minor changes in the member sizes are required to incorporate the FEMA 267 design recommendations for the relocation of zones of plastification away from the column face.

The second point deserves elaboration. Adding cover plates will increase the shear and moment demands at the connections, which in turn will increase the strength demands on the columns because of the strong column requirement. For the case study structures this made little, and in most cases no, difference in the column sizes compared to the pre-Northridge designs. Reducing the beam section (RBS designs) did not affect member sizes compared to pre-Northridge designs because beam sizes were governed by stiffness rather than strength.

The largest difference in post- versus pre-Northridge designs is noted in the Boston structures for the following reason. The pre-Northridge design has very weak panel zones because neither UBC'94 nor BOCA'93 has a minimum strength requirement (beyond the basic ASD requirement) for panel zones in structures located in seismic zones 1 and 2. The design of the post-Northridge structures conforms to FEMA 267 requirements, wherein a minimum panel zone strength is required. Conformance to FEMA 267 resulted in much higher demands on panel zone shear strength, which was met by either providing doubler plates or heavier column sections and resulted in larger lateral stiffness and strength.

The effect of the different connection types on the strength of the structure can be evaluated through a static pushover analysis. Figure 5 shows global pushover curves for the different LA and Boston designs, and Figure 6 shows global pushover curves for the various Seattle designs. The pushover analysis is based on a NEHRP'94 k=2 [parabolic] load pattern. As expected, the curves indicate that designs employing the cover-plate connection have higher global strength than the pre-Northridge designs, whereas the design employing the RBS connection has lower strength. The large difference in strength of the Boston designs is attributed to the panel zone design issue discussed in the previous paragraph.

The effect of the different connection types on the system level dynamic response is illustrated through Figures 7 and 8, which present the story drift demands for the various designs in the different seismic locations, for the 2/50 sets of ground motions. Median [calculated as the exponent of the mean of the natural log values of the data set] and 84th percentile [calculated as the median multiplied by the exponent of the standard deviation of the natural log values of the data set] values are presented. Additional information on the behavior and response of 3-story and 20-story structures, and for various hazard levels, is given in [Gupta and Krawinkler, 1999].

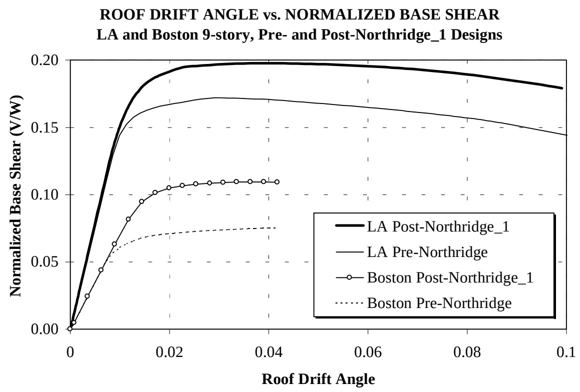


Figure 5. Pushover Curves for LA and Boston Designs

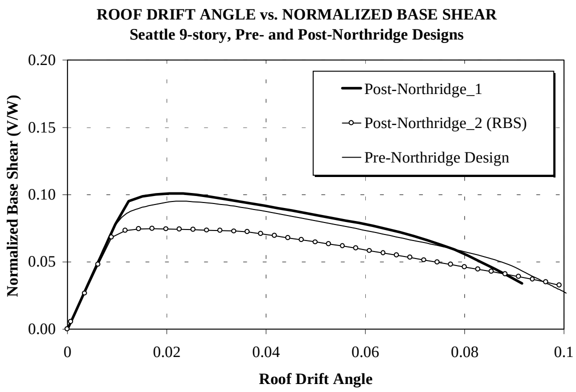


Figure 6. Pushover Curves for Seattle Designs

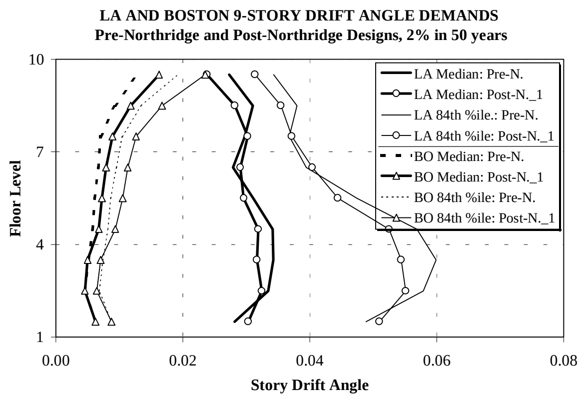


Figure 7. Story Drifts for LA and Boston Designs

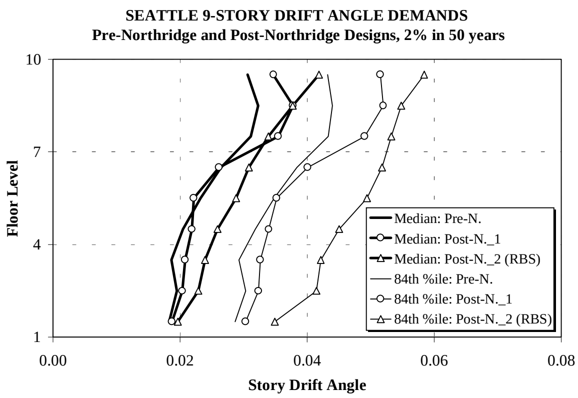


Figure 8. Story Drifts for Seattle Designs

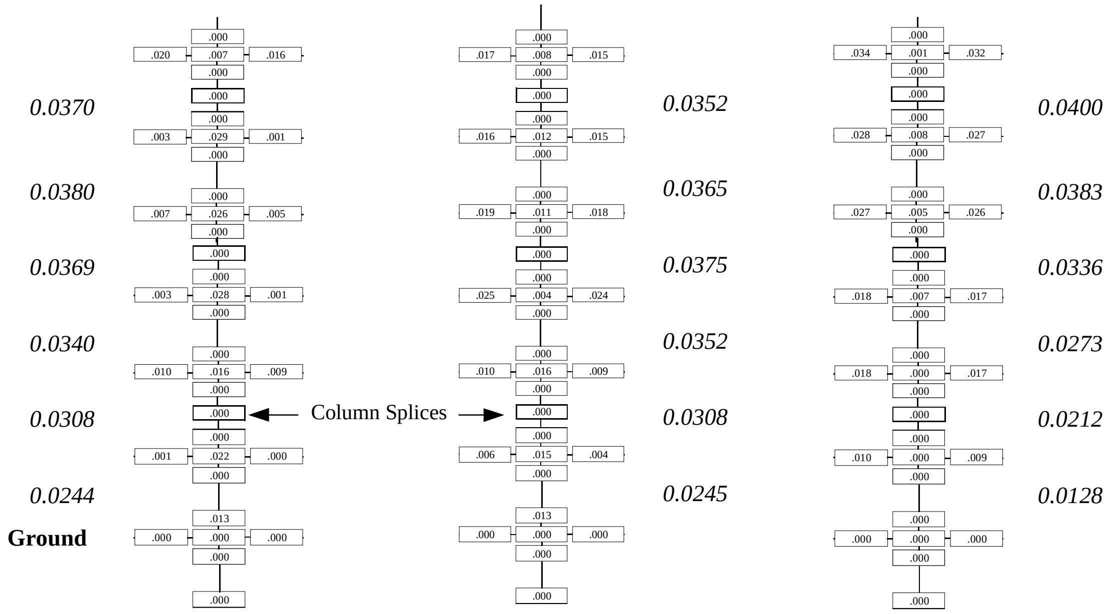
Figure 7 and Figure 8 illustrate the following important behavior characteristics:

1) The difference in story drift demands is not significant between the pre-Northridge and post-Northridge_1 designs, at both the median and 84th percentile levels, even though the global strength of the designs is quite different. This observation holds true across the three seismic regions.
2) The reduced beam section design for the Seattle structure follows the same drift pattern and exhibits similar demands as the pre-Northridge and post-Northridge_1 designs - at the median level. The 84th percentile demands are, however, larger. The differences arise on account of a few outlier values, caused by the reduction in strength and the resulting increased importance of P-delta effects, as seen in the global pushover curve presented in Figure 6 [Gupta and Krawinkler, 2000].
3) Drift demands for the LA and Seattle structures are comparable, even though the spectral displacement demands at the first mode period are significantly lower for Seattle as compared to LA [Figure 4]. The distribution of demands over height is, however, very different. The differences are on account of design considerations [wind design controls the Seattle structure] and on account of ground motion characteristics [the LA records have distinct near-field characteristics while the Seattle records are characteristic of subduction zone events].
4) The drift demands are large for the bare frame models of the LA and Seattle structures, especially at the 84th percentile level.

It must be stressed that the drift demands shown in Figures 7 and 8 are obtained from analytical studies that rely on a great number of simplifying assumptions. For a given ground motion the actual demands depend, among others, on 3-D effects, hardening and degradation characteristics of the elements, and contributions of structural and nonstructural elements not considered in the model. Thus, drift demand values of the type presented in Figures 7 and 8 are indicators rather than accurate predictions, which is important to recognize when demands are related to capacities in an effort to predict performance.

SEISMIC BEHAVIOR OF DIFFERENT DESIGNS AT THE CONNECTION LEVEL

The relative strength of the beam, column, and panel zone at a connection controls the state of stress at the welded joint. If the beam is the weak element, it develops a plastic hinge and the joining medium has to transfer the beam bending strength, with appropriate strain hardening, to the column. This implies transfer of very high horizontal normal stresses and strains generated by axial yielding and strain-hardening of the beam flange, and of high shear stresses due to concentration of shear in the beam flange near the column face. If the column is the weak element, it develops a plastic hinge, which implies very high vertical normal stresses and strains in the column flange near the weld root. If the panel zone is the weak element and yields in shear, neither the beam nor the column may reach their yield strength. This implies a state of stress and strain that may be controlled by localized yielding of the column flange due to large shear distortions in the panel zone. Each of these stress/strain states will affect the likelihood of fracture in a different manner.



(a) Pre-Northridge Design (b) Post-Northridge_1 Design (c) Post-Northridge_2 Design

Figure 9. Story and Element Deformation Demands at 3% Roof Drift, for Seattle 9-story Designs

Figure 9 presents the element plastic deformation demands [panel zone shear distortion in the center box of the joint, beam plastic rotations at the right and left of the joint, and column plastic rotations above and below the joint] at an interior column line for the first six stories of the three Seattle designs, at the roof drift of 3% under a static pushover. The story drifts are shown in italics. The distribution of story drift demands over height is similar for the pre-Northridge and post-Northridge_1 designs, while the post-Northridge_2 design shows a different pattern over height, with the bottom stories experiencing relatively low demands.

Figure 9 shows that even for stories experiencing very similar drift demands, e.g., stories 4 through 6, the distribution of demands at the connection level may be very different, which results in different states of stress at the welded joint. In this example, the demands are concentrated primarily in the panel zones for the pre-Northridge design, shared between the beams and panel zones for the cover-plated post-Northridge design, and concentrated in the beams for the RBS design. The designs for the Seattle 9-story structures are from one engineering office, which demonstrates the variability in the element deformation demands from one design to another. This variability is further compounded by inherent variability in material strength and by dynamic redistribution of forces, which may result in column plastification. For many of the structures studied, column plastification was observed even though the structures conformed to the “strong-column” concept [Gupta and Krawinkler, 1999].

The discussion so far was concerned with demand predictions, without regard to the deformation capacity at connections. Nondegrading connection behavior was assumed. The discussed example indicates that it may be inappropriate to correlate connection capacity with a single deformation parameter such as the plastic rotation of the beam. At this time insufficient knowledge exists to assess the relative importance of beam plastic rotation and panel zone distortion on connection behavior. For these reasons it is appropriate (or least inappropriate) to correlate connection capacity with a parameter that integrates over the elements framing into a connection. The story drift angle is a suitable parameter for this purpose. In recent experiments within the SAC steel project this deformation parameter is used to control the loading history and to provide a measure for connection capacity [SAC/BD-97/02, 1997]. The following short discussion on connection capacity employs story drift as the capacity parameter.

It is tempting to draw quantitative conclusions on performance by comparing the drift capacity estimates presented next with the demand estimates discussed before. Such a comparison is presumptuous because of the many connection parameters represented in the data bases from which the capacity estimates are obtained and the lack of a consistent definition of acceptable connection performance associated with the drift capacity. The presented capacity data should be viewed as indicative only. Moreover, the only implication of the drift demand exceeding the drift capacity is that a connection exhibits unacceptable performance, which is very different from claiming that the structure exhibits unacceptable performance.

CAPACITY ESTIMATES FOR DIFFERENT CONNECTION TYPES

Prior to the Northridge earthquake and especially following the Northridge earthquake, many different types of connections have been tested, which provide a wealth of information on which capacity estimates can be based. Compilations of experimental test results are available in the literature, e.g., [SAC 96-01, 1996], SAC website, which provides a comprehensive collection of connection test data, and individual reports and databases.

Capacity estimates used in this study are based on one such database, which includes information from the SAC database and from various technical papers and reports available in the open literature. The information on the following four connection types is evaluated:

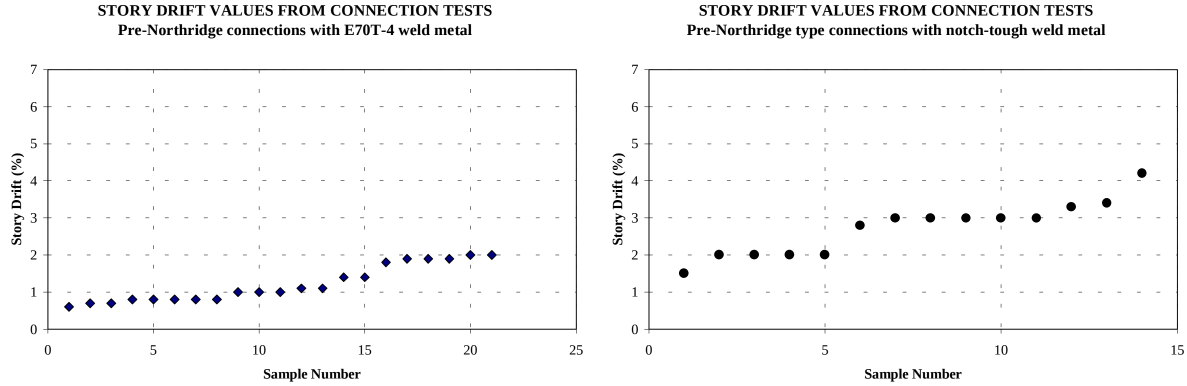
1) Pre-Northridge connections, with the E70T-4 weld metal [no minimum specified CVN value] used for the beam flange-to-column flange welded connection.
2) Pre-Northridge type connections, but using filler metals with specified minimum CVN toughness values, e.g., E71T-8 (45ft-lb at –20 degrees F), E70TG-K2 (35 ft-lb at –20 degrees F), etc. Emerging guidelines [AISC, 1997] require the use of filler metals with a minimum CVN value of 20 ft-lb at –20 degrees F.
3) Connections employing the use of cover-plates (connection type used in post-Northridge_1 designs). In this data set, only connection tests having both top and bottom cover plates are considered. No distinction is made on the basis of the filler metal used for the CJP welds, nor is a distinction made based on whether the beam flange is welded to the column flange or not.
4) Connections employing the use of reduced beam sections (connection type employed in post-Northridge_2 designs). In this data set, only connection tests having both top and bottom RBS are considered. No distinction is made based on the geometry of the RBS designs (e.g., circular cuts, tapered cuts, etc). Connections employing cover-plates in conjunction with RBS are included in this data set as the cover plates are used to reduce the stress level at the column face [FEMA 267].

It must be noted that not all connection tests falling under the four categories listed above have been included in the data sets on account of either incomplete information or based on the authors' judgement (e.g., repaired connections are not included in the pre-Northridge connection categories). The results presented here should only be considered as indicative of the capacity values. The values will change as additional information is incorporated. Furthermore, the categorization considered for the connection tests is very broad and groups together many factors that affect the connection response to varying degrees. Among these factors are member sizes, sizes of doubler plates, use of continuity plates, single-sided or double sided connection tests, presence or absence of floor slab, material properties, type of shear tab (bolted, welded), test geometry, testing procedure, among others.

Figure 10 presents maximum story drift data from connection tests of the pre-Northridge type connections, i.e., direct welding of the beam flange to the column flange with no additions or modifications. The mean story drift for pre-Northridge type connections with E70T-4 weld metal is calculated to be 1.2% with a COV of 0.41. The corresponding values for the pre-Northridge type connections with notch-tough weld metal are 2.7% and 0.27.

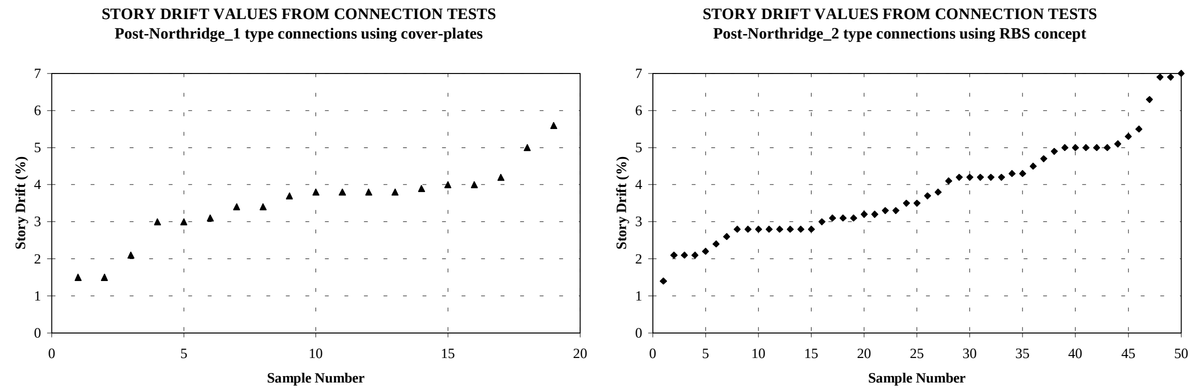
Figure 11 presents similar data for the post-Northridge type connections considered in this database, i.e., connections either using the cover-plate approach or the reduced beam section concept. The mean story drift value for the cover-plate connections is calculated to be 3.5% with a COV of 0.29. The corresponding values for the connections employing the reduced beam section concept are 3.9% and 0.34. The mean value of the RBS types would increase if reduced beam sections with tapered cuts were eliminated.

The presented results show that significant improvement can be achieved by forcing the critical beam section away from the column face, by either using cover plates or reduced beam sections. From the connection types considered here, the relatively best performance is achieved by using reduced beam sections with circular cuts.



(a) **(b)**

Figure 10. Distribution of Story Drift Values for Pre-Northridge type Connections; a) E70T-4 Weld Metal, b) Notch-Tough Weld Metal



(a) **(b)**

Figure 11. Distribution of Story Drift Values for Post-Northridge type Connections; a) Cover-plate connections, b) Connections with RBS

CONCLUSIONS

Based on these case studies, and additional results obtained by the authors, the following specific conclusions can be drawn concerning the behavior, response, and expected performance of steel moment resisting frame structures employing different types of fully-restrained connections. In the first two conclusions, which provide a comparison of demands with pre-Northridge designs, the assumption is made that no connection fractures occur in the pre-Northridge designs.

- The addition of cover-plates has only a small influence on the stiffness of the structure, but results in an appreciable change in the strength. The effect on the system level demands is, however, not significant even under very severe ground shaking. This is seen by the pre-Northridge and post-Northridge_1 designs having almost the same drift demands in all three seismic locations.
- The reduced beam section concept results in a reduction in the global strength (in effect, a reduction in the "overstrength") of the structure, compared to designs without connection modifications. The stiffness often is not much affected because similar (or identical) member sizes can be used in many cases since the beam sizes are usually controlled by drift considerations. The effect on the system level response is expected to be small unless the change in strength distribution (absolute and over height) results in a change in mechanism formation in the structure or induces a change in the basic load-deformation characteristics of the structure. Such changes may significantly affect the response to very severe ground motions and may lead to very large drift demands.

- The choice of the connection type, and many other issues including subjective design decisions, control the distribution of deformation demands at the connection level, and consequently the state of stress at the beam-to-column weld. The plastification may be localized in the beams, in the panel zone, or distributed between the two types of elements. Column plastic hinging is a distinct possibility even if the “strong column” concept is considered in design.
- Employing notch-tough weld metals will improve the performance of pre-Northridge type connections, but experimental data obtained so far does not provide confidence that good performance (story drift on the order of 4%) can be achieved with notch-tough weld metals such as E70TG-K2 or E71T-8.
- The best performance of the connections considered is observed for the RBS designs with circular cuts. The data set presented here includes tests on tapered RBS or RBS with sharp corners, which have been shown to fare poorly due to stress concentrations as compared to RBS with smooth circular cuts.

The observations and results presented in this paper need to be interpreted within the context in which they are obtained. The demand estimates are obtained from 2-dimensional analysis of 3-dimensional structures, with many judgmental assumptions inherent in the modeling. These include the neglect of many structural and nonstructural elements that contribute to strength and stiffness, the assumption of point plastic hinges, constant strain hardening of 3% in all inelastic elements irrespective of the magnitude of inelastic deformations, and disregard of cyclic hardening, weld fractures, stiffness degradation, and strength deterioration. Furthermore, only a small set of specific designs has been considered in this study. The capacity estimates are derived from a database that is quite extensive but is still under development, and contains data that lack a consistent definition of acceptable performance. Also, there are inherent assumptions associated with grouping the various connection test results.

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REFERENCES

AISC (1997). “Seismic Provisions for Structural Steel Buildings,” *American Institute of Steel Construction, Inc.*, April 15, 1997.

FEMA 267 (1995). “Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures”, *Federal Emergency Management Agency*.

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.

Krawinkler, H. (1978). “Shear design of steel frame joints,” *Engineering Journal*, AISC, Vol. 15, No. 3.

SAC Background Document 97/02 (1997). “Protocol for Fabrication, Inspecting, Testing, and Documentation of Beam-Column Connection Tests and Other Experimental Specimens,” *SAC Background Document, Report No. SAC/BD-97/02, NISEE – UC Berkeley*.

SAC Technical Report 96-01 Part 1 and Part 2, (1996). “Experimental Investigation of Beam-Column Subassemblages,” *SAC Document, Report No. 96-01, 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, NISEE – UC Berkeley*.