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Nonlinear Analysis of Steel Frames with Ductile Connections

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Abstract

One of the typically damaged steel beam-column connection during the Northridge earthquake of 1994 was bolted-web, welded-flange (BWWF), now known as “pre-Northridge” connection. Proposed post-Northridge connections are more ductile and capable of dissipating more energy, thus improving the behavior of steel moment resisting frames (SMRF). The post-Northridge connections with added ductility will be denoted in the paper as BWWF-AD. However, the estimation of the nonlinear response of SMRF in the presence of BWWF-AD is not simple. These are essentially partially restrained (PR) connections. The behavior of PR connections is generally represented by moment-relative rotation (M-θ) curves. Along with material and geometric nonlinearities, the presence of PR connections adds another major source of nonlinearity and must be considered appropriately for accurate seismic response analysis. Structural Seismic Design Associates (SSDA) proposed a unique proprietary SlottedWeb™ connection and tested several full-scale models. The nonlinear response of two frames in the presence of BWWF and ductile BWWF-AD connections were calculated and studied. Ten actual recorded earthquake time histories, six of them were recorded during the Northridge earthquake of 1994, were used to calculate the response. Several important observations and design recommendations are made.

Introduction

One of the typically damaged steel beam-column connections during Northridge Earthquake of 1994 was bolted-web, welded-flange (BWWF) connection. The BWWF connections shown in Fig.1 were fabricated with the beam flanges attached to the column flanges by full penetration welds (field-welded) and with the beam webs bolted (field-bolted) to single plate shear tabs. Many of these BWWF connections fractured in a brittle and premature manner.

The post-Northridge research was focused on the cause of the damage and to develop connections that can improve the overall response, ductility and the quality of the future designs. Structural Sesimic Design Associates (SSDA) proposed a very unique proprietary slotted web (SlottedWeb™) beam-column connection and tested several full-scale models. SSDA test results using ATC-24 test protocol showed substantial increase in ductility among other beneficial effects in comparison with BWWF connections. These connections are called here as bolted-web, welded-flange with adequate ductility (BWWF-AD). The authors were given access to some of the actual SSDA test results. Using the four parameters Richard model, the authors first proposed a mathematical model to represent moment-relative rotation (M-θ) curves for BWWF-AD connections. The model can generate M-θ curves for other beam-column assemblies not tested by SSDA. This was very beneficial in modeling the SAC steel frames of this study since it is not feasible to test every individual beam-column connection in every structure.

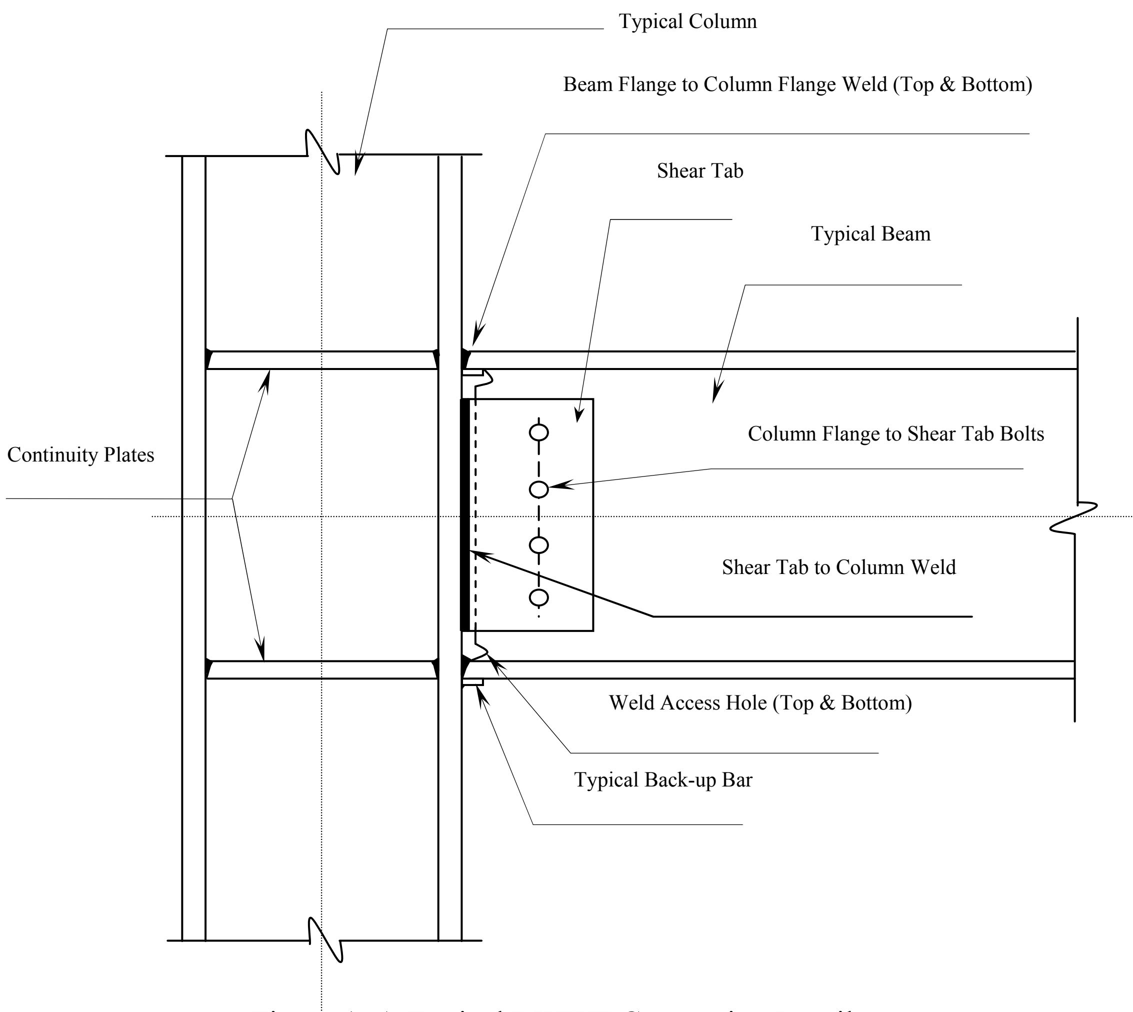

Figure 1: A Typical BWWF Connection Detail

Description of Frames

Two Steel Moment-Resisting Frames (SMRFs), as shown in Figure 2, are considered and their seismic responses are calculated and evaluated in the presence of different connections. These frames are the North-South SMRF of the SAC Steel Project 3-sroty and 9-story Los Angeles (LA) model buildings designed according to the equivalent lateral force method suggested in the 1997 NEHRP Provisions. These buildings are presented in FEMA-355F (2000) and are used in many studies by SAC Steel Project. These frames are referred to as 3-story and 9-story frames here.

As shown in Figure 2, based on the geometry of both of the SAC frames, they can be classified as "regular" in elevation. The 3-story frame consists of four bays and three stories above the ground level with a floor height of 3.96 m (13 ft) for all of the floors and bay width of 9.14 m (30 ft) for all of the bays. The 9-story frame consists of five bays, nine stories above the ground level and a basement. The height of the first floor is 5.49 m (18 ft) above the ground level, and the other floors are each 3.96 m (13 ft) in height, with the exception of the basement that is 3.66 m (12 ft) below the ground level. The sizes of the members of both frames are presented in Table 1.

Using the dynamic characteristics of the frames, the natural periods of vibration *T* are calculated as 0.21 seconds for the 3-story frame and 1.23 seconds for the 9-story frame. For comparison, *T* is also calculated as 0.53 seconds for the 3-story frame and as 1.41 seconds for the 9-story frame using FEMA 350, Equation 4-1, pp.4-15. The natural periods of vibration are calculated according to FEMA 350 as:

$$T = 0.028 h_n^{0.8} \tag{1}$$

Where T is the fundamental period (in seconds) in the direction under consideration, and h_n is the height (in feet) to the roof level above base.

In comparison to the calculated values from dynamic characteristics of the frames, the natural periods of vibration calculated using FEMA 350 equation are 60 percent and 13 percent higher for the 3-story and the 9-story frame, respectively.

In this study, 120 analyses were performed. The frames were modeled with three different types of connections as FR, BWWF, and BWWF-AD. The actual and the scaled time histories of ten different earthquakes were applied to the base of the frames, and the responses are calculated using the assumed-stress based FEM discussed in Mehrabian [2], and Reyes and Haldar [5]. Each column and each beam (girder) of the frame is modeled as one element except at the location of the connections. The connections are modeled as a kind of beam-element as discussed in Haldar and Zhou [4].

Modeling of Beam-Column Connections

In a realistic analysis and design of steel structures, knowledge of the moment-relative rotation (M-θ) response curves of each connection is essential. The information on M-θ curves of the connection becomes even more essential in the nonlinear seismic analysis of SMRF. Due to the nonlinear behavior of M-θ curves from the very beginning of the loading, these curves must be modeled as accurately as possible using a nonlinear mathematical model. Furthermore, the initial stiffness of each connection and the change in the stiffness during the loading, unloading, and reloading of the frame need to be calculated accurately.

M-θ curves used for modeling the BWWF-AD connections of the frames of this study were developed from the experimental data of full-scale ATC-24 laboratory tests provided by Structural Seismic Design Associates (SSDA). Among several available mathematical models, Richard 4-parameter model is employed for modeling the M-θ curves because it is adaptable, relatively simple, easy to differentiate and it does not pose any numerical difficulty in the algorithm used in this study. The four parameters of this model relate to the physical properties of the connection and are relatively simple to calculate based on the elastic and plastic stiffness of the connections. The four parameters of this model were calculated for the best fit of Richard Equation to the M-θ curves of the tested connections as presented in Mehrabian [2]. The accuracy of this model is found to be very good for BWWF-AD connections. Using a procedure described elsewhere [2], M-θ curves for the untested BWWF-AD connections used in modeling the frames of this study were developed. For some of the connections of the modeled frames, the calculated M-θ curves are shown in Figure 3. The M-θ curves for modeling the "pre-Northridge" BWWF connections of the frames of this study were generated using PRCONN computer program.

To provide a consistency in matching the initial stiffness of the BWWF and BWWF-AD connections, BWWF connections are modeled as double web angle, top and seat (DWATS). Mehrabian [2] showed that the initial stiffness of the BWWF-AD connections matches the initial stiffness of DWATS connections, but the ductility and the energy absorption capacity of BWWF-AD connections are much higher. In designing the DWATS connections, top and bottom angle lengths are assumed to be 22.86 cm (9 in) to match the least width of the beam flanges of the connected beams. Assuming no web adjustment length, the neutral axes of the beam and the connected web angles coincide. The top and bottom angles and the web angles thickness is 1.27 cm (0.5 in), and 8 bolts with the bolt diameters of 2.22 cm (7/8 in) are used in the web connection. The angles are assumed to be made of Grade 50 steel to match the grade of steel used in designing the frames.

The frames of this study are modeled twice with BWWF and BWWF-AD connections. In the first case, the connections are assumed to behave as FR or fully rigid. In this case, the full bending moment induced in the beams (or girder) are transmitted to the columns. The T Ratio is defined as M_b / M_{fix} and it is at least 0.9 in this case for all of the connections as discussed in Reyes and Haldar [5]. M_b is defined as the beam-end-moment and M_{fix} is defined as the fixed-end-moment. In the second case, it is assumed that BWWF and BWWF-AD connections behave as PR. In this case, not all of the bending moments induced in the beams (girders) are transmitted to the columns. The T Ratios for these cases are less than 0.9. The T Ratios in this case are calculated using the elastic beam-line concept as presented else where in Mehrabian [2]. This ratio is only valid at yield. In reality, during an earthquake, after a member reaches its yield strength, it is very difficult to calculate the inelastic T Ratio. During an earthquake while the magnitude of seismic loading is changing, the T Ratio is changing depending on the magnitude of the loading. The Calculated T ratios using the elastic beam-line concept may underestimate the real values of T ratios.

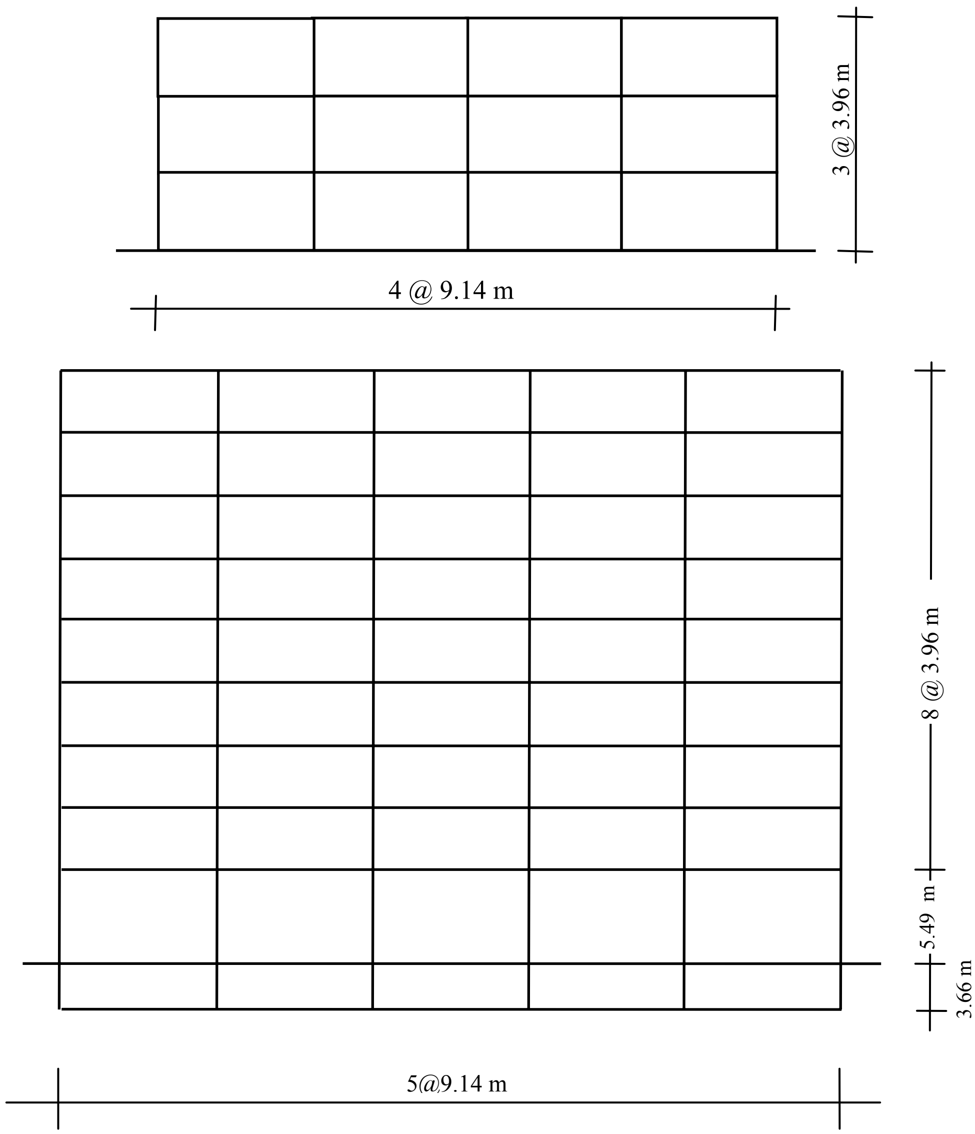

Figure 2: N-S SMRF of SAC 3- and 9-story Model Building Designed to 1997 NEHRP Provisions

Description of the Earthquakes

Two suits of ten different actual time histories of the earthquakes with different scale factors were used in this study. Mehrabian [2] described these earthquakes in detail elsewhere. These acceleration time histories are chosen in such a way that they represent the natural randomness in the frequency content, epicentral distance and ground acceleration. These time histories were recorded in different stations across Southern California. Six of these earthquakes were recorded during the Northridge earthquake of 1994, and four of them were recorded during El Centro earthquake of 1940, San Fernando earthquake of 1971, and Whittier Narrows earthquake of 1987. As a sample, Figure 4 shows the two horizontal acceleration time histories recorded at

Malibu Station during the Northridge earthquake 1994. All design levels ground accelerations are for firm soil sites. No attempt was made to consider soft soil sites. The first 16 seconds of each time-history is used in the analysis. Referring to the time history of the earthquake shown in Figure 4, in each analysis, the component of the earthquake corresponding to the larger Peak Ground Acceleration (PGA) is applied to the frame in the direction of the stronger axis of the frame. A method of rationalizing the appropriate scale factors for earthquakes corresponding to the natural periods of the vibration of the frames is discussed below.

Table 1: Member Sizes of Frames

3-story			
Story	Columns		Beam
	Exterior	Interior	
0 - 1	W14X257	W14X283	W30X99
1 - 2	W14X257	W14X283	W33X118
2 - 3	W14X257	W14X283	W21X68
9-story (with basement)			
Story	Columns		Beam
	Exterior	Interior	
basement - 0	W30X235	W30X261	W36X150
0 - 1	W30X235	W30X261	W36X150
1 - 2	W30X211	W30X235	W33X141
2 - 3	W30X211	W30X235	W33X141
3 - 4	W30X173	W30X211	W33X141
4 - 5	W30X173	W30X211	W33X141
5 - 6	W30X148	W30X173	W30X116
6 - 7	W30X148	W30X173	W27X94
7 - 8	W30X148	W30X148	W27X94
8 - 9	W30X148	W30X148	W18X50

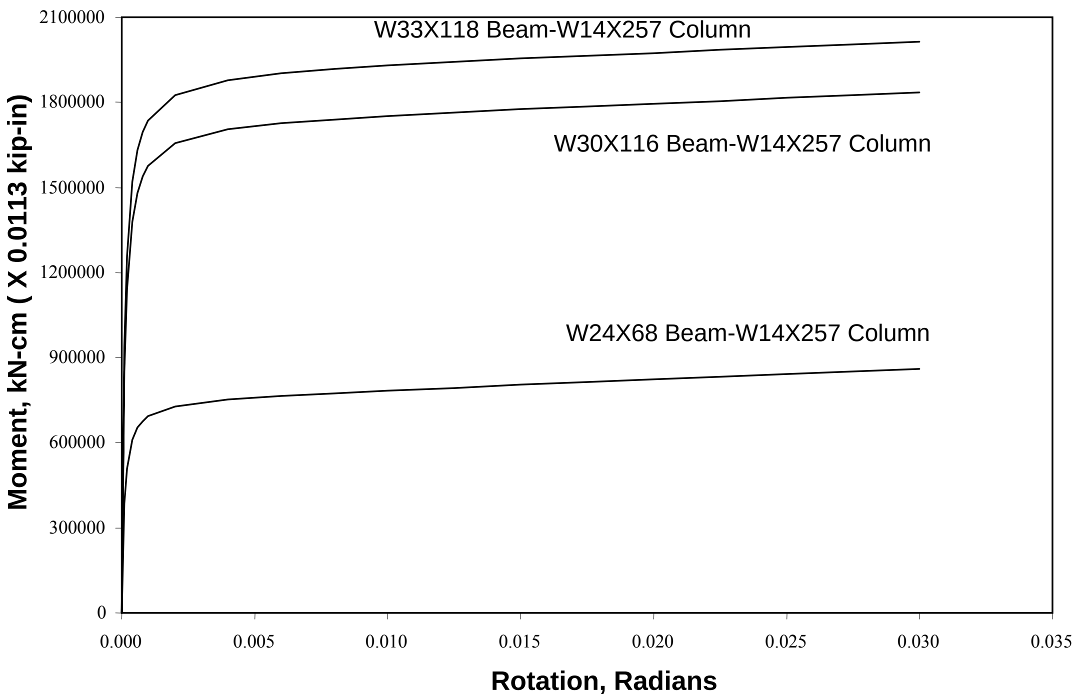

Figure 3: Moment-Rotation Curves for BWWF-AD Connections of 3-Story Frame

Actual time histories are used first to calculate the response of the frames. As such, in some cases, no significant response was calculated. To study the response of the frames comprehensively, time histories are then scaled according to the natural periods of vibration of the frames, to represent the earthquakes with 2% probability of occurrence in 50 years presented in FEMA-355F. The scales are adjusted accordingly to match the PGA of 5% Damped Los Angeles Response Spectra given in the FEMA-355F publication. As such, for the 3-story frame with a natural period of 0.53 seconds calculated using Equation 1, time histories are scaled in such a way to represent a PGA of 1.6g (1570 cm/sec.^{2}). For the 9-story frame with a natural period of vibration of 1.41 seconds, time histories are scaled in such a way to represent a PGA of 1.2g (1177 cm/sec.^{2}). Appropriate scales of the earthquakes used in the analysis of each frame of this study can be found in Mehrabian [2].

In this study, for the ease of reference, actual earthquakes and scaled earthquakes are referred to as AE and SE, respectively, with the earthquake number follows immediately. For examples, actual earthquake 1 is referred to as AE1 and scaled earthquake 5 is referred to as SE5.

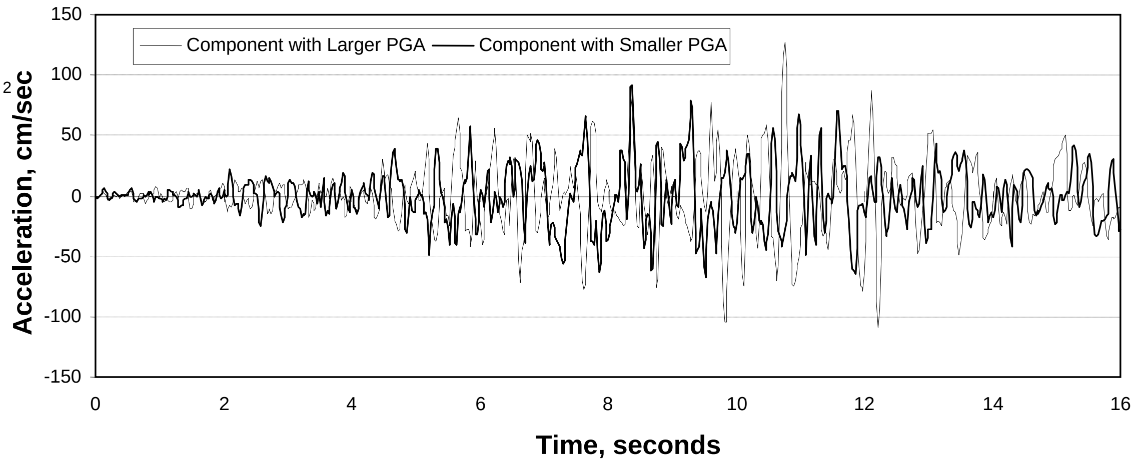

Figure 4: Horizontal Components of Northridge '94 Earthquake - Malibu Station

Responses of the Frames

A. Maximum Lateral Displacement

One of the main concerns in seismic design of the steel PR frames is the maximum lateral displacement. Engineers may fear that increasing the flexibility of the frame by including PR connections will inherently create large lateral displacements during a seismic event. To address this issue in this study, responses of the frames are evaluated in terms of the maximum lateral displacement of the frame.

MAXLD is defined here as the maximum top lateral displacement of the frame. For the 3-story and the 9-story frames this would be the maximum lateral displacement of the 3rd story and the 9th story, respectively. The response of the frames in terms of MAXLD subjected to scaled earthquakes (SE) are presented for the 3-story and 9-story frames. The response of the frames subjected to actual earthquakes (AE) are not discussed here since in most cases, no significant response is produced. MAXLDs are calculated separately for the frame modeled as FR, PR with BWWF connections, and PR with BWWF-AD connections.

For the 3-story frame, the type of the model used to represent the connections as PR or FR does not seem to have much impact on the maximum top displacement of the frame with BWWF-AD. This maybe attributed to the high elastic stiffness and ductility of the BWWF-AD connections in comparison with other types of flexible connections. Since no plastic hinge is formed in the frame in this case, it is reasonable to assume that the connections' high elastic stiffness made them to behave essentially as FR connections. However, maximum top lateral displacements of the frame modeled as PR with "pre-Northridge" BWWF connections are much larger compared to the FR case, and the PR case with BWWF-AD. For the 9-story frame, using SE3, 7, 8, 9, and 10, scaled up to a PGA of 1.6g, MAXLD of PR frames with BWWF connections are larger comparing to the frames modeled with the other connections. MAXDL is smaller for PR frame with BWWF-AD connections of SE2, 3, 4, 7, 8, and 10. In other cases, MAXDL is identical or almost identical.

Overall, it has been observed that when the ground shaking is not strong enough to produce significant lateral displacement in the frames, the response of the frames of this study slightly improves using PR frames with BWWF-AD. In this case, sometimes PR frames with BWWF connections produce larger lateral displacements. In most cases, when ground shaking produces larger lateral displacements of the frames, the response of PR frames with BWWF-AD improves over its counterpart with BWWF connections, and FR frame by reducing the lateral displacements of the frames. This is due to the large elastic stiffness and adequate ductility of the BWWF-AD connections. In this case, the lateral displacements of PR frames with BWWF connections increased some times up to 100 percent. This may be one of many reasons for the poor performance of frames with BWWF connections during the Northridge earthquake of 1994.

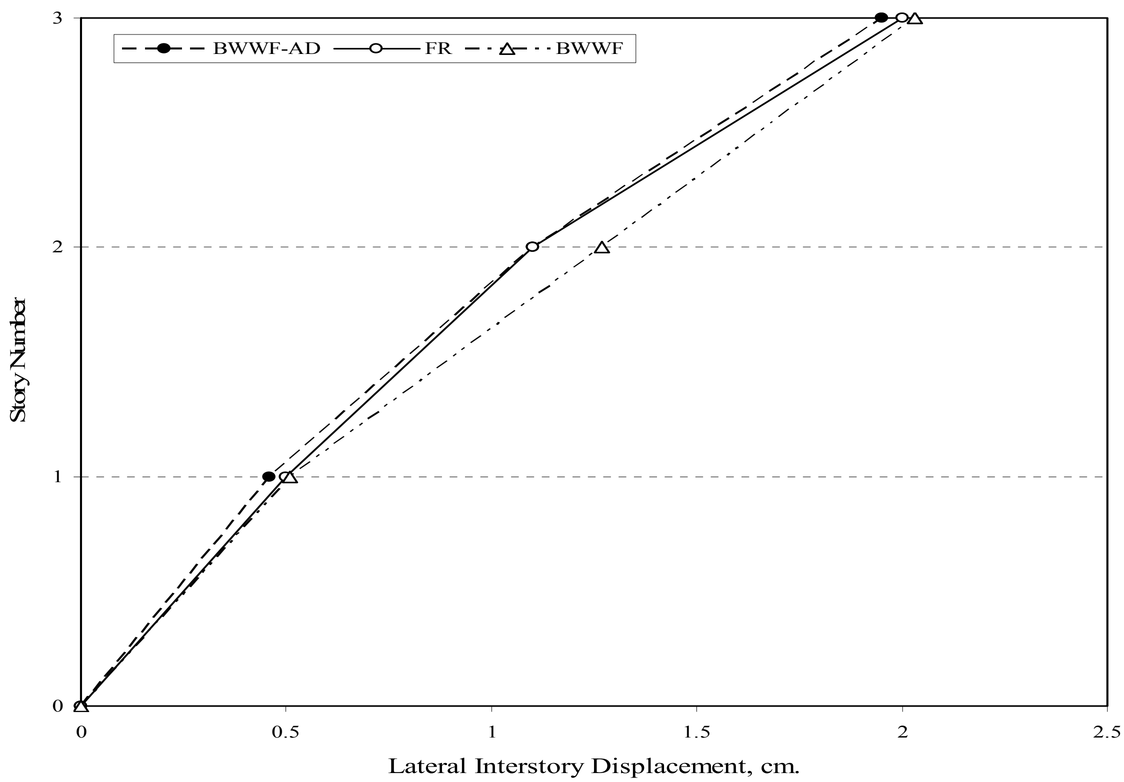

Figure 5: Lateral Displacement Skeleton Curve for the 3-Story Frame for Earthquake 6 (PGA = 1.6g)

B. Lateral Interstory Displacement

Lateral interstory displacement (abbreviated here as ID) of a frame is also an important parameter in the design and the analysis of the steel PR frames and is addressed in this study. The calculated IDs of the 3-story frame of this study are absolute relative values. At each story, the absolute ID is calculated relative to the displacement of the story below and above that particular story. Thus, the summation of all of the IDs of the frame is the maximum displacement of the frame at the top level. For the 3-story frame, ID skeleton curves are plotted for PR frames with BWWF-AD and BWWF connections and for FR frames for each earthquake to gives a clear picture of the variation of IDs at different stories. Samples of these curves are depicted in Figures 5 and 6 for the 3-story and 9-story frames, respectively. For the 9-story frame, ID is presented in terms of the accumulated lateral displacements to provide yet another insight.

In summary, a relatively large lateral ID is noticeable between the 5th and the 6th stories of the 9-story frames for all earthquakes. Referring to Table 1 of member sizes of this frame, there is a noticeable change in the sizes of both interior and exterior columns from the 5th to the 6th story. For the PR and FR frames studied, in most cases where strong shaking of the frames was not observed, the IDs of the PR frames modeled with BWWF-AD are slightly less than or almost identical to FR frames. In some cases of insignificant shakings with no large lateral displacements produced in the frames, it is difficult to interpret the results accurately. In these cases, although no large IDs produced in the frames, IDs are larger for PR frames with BWWF connections in comparison to other frames. As with the cases of no significant shaking, in cases where significant shaking was produced in the frames without the formation of any plastic hinge, PR frames with BWWF-AD connections underwent similar or slightly less IDs compare to FR frames. The large initial elastic stiffness, adequate ductility, and energy absorption capacity of BWWF-AD connections contribute to these effects. Furthermore, the IDs for PR frames with BWWF connections are larger for many earthquakes in these cases. In some cases, IDs are up to three times larger. Inadequate ductility and low energy absorption capacity of BWWF connections are more pronounced in these cases.

C. Maximum Connection Rotation

Steel frames designed with PR connections require sufficient ductility since it is expected that the connections provide the majority of the inelastic behavior during an earthquake. The SSDA experimental data used in this study to model the BWWF-AD connections have shown that these connections provide sufficient ductility using the full-scale ATC-24 beam-column test assembly as presented by Richard et al. [7]. Test data indicates that BWWF-AD connections developed large maximum angle of rotation at the connection and have stable hysteretic loops. In designing the frames with PR connections, the maximum angle of rotation that a connection undergoes is an important parameter in accessing the ductility of the connection and the frame.

The response of each frame in terms of absolute maximum angle of rotation of the connections (abbreviated here as MAXCR) is discussed in this section. MAXCRs for the 3-story and the 9-story frames modeled with PR and FR connections are compared for each ground motion. MAXCR is calculated using the maximum accumulative angle of rotation of the connections at each story. These values are plotted along the height of the frames at each story as shown in sample Figures 7 and 8 for the 3-story and 9-story frames, respectively.

For the 3-story frames, MAXCRs of the frames do not exceed the angle of rotation of 0.0125 radians for any of the applied earthquakes. Using the actual time histories of the earthquakes, no significant difference in MAXCR can be observed for the 3-story frames except for AE1 and AE4. For ground motions of AE1, MAXCR of the frame with BWWF connections is significantly larger than the other frames. For AE1, MAXCR is identical for the FR frame and the PR frame with BWWF-AD connections. For this earthquake, MAXCR is significantly larger for the PR frame with BWWF connections as shown in Figure 7. Using the scaled earthquakes with PGA of 1.6g for the 3-story frame, MAXCR is almost identical in all cases except for SE9 and SE 10. In the case of applied SE9, MAXCR is smaller for the PR frame with BWWF connections than the other two frames. For SE10 where no significant shaking produced, MAXCR is larger for the PR frame with BWWF connections and are identical for the other frames.

It is generally expected that a well-detailed steel connection with sufficient ductility will be able to accept angle of rotations of up to 0.03 radians without failure. The calculated MAXCR for the connections of this study is well below that level although they are much larger in PR frames with BWWF (pre-Northridge) connections. It is observed here that MAXCR is governed by different ground motions. Overall, it is observed that the calculated MAXCR of the connections of the PR frames modeled with BWWF-AD and subjected to the ground motions of the earthquakes of this study was less than FR frames. Again, this is some what contrary to the general assumption that the PR frames will have larger rotation than FR frames.

Generally, the elastic stiffness of the partially restrained (PR) or semi-rigid steel connection is less than other types of steel connections classified as fully restrained (FR) or rigid. There is also a variation of the elastic stiffness among steel connections classified as PR. However, as discussed in Mehrabian [2], the BWWF-AD connection used in modeling the PR connections of this study has much larger elastic stiffness in comparison with other types of PR connections commonly used. They can also develop full plastic moment capacity of the beam before failure. The initial elastic stiffness (in the absence of formation of any plastic hinge in this study) and ductility of BWWF-AD connections contributed to their identical or improved response in comparison to FR connections.

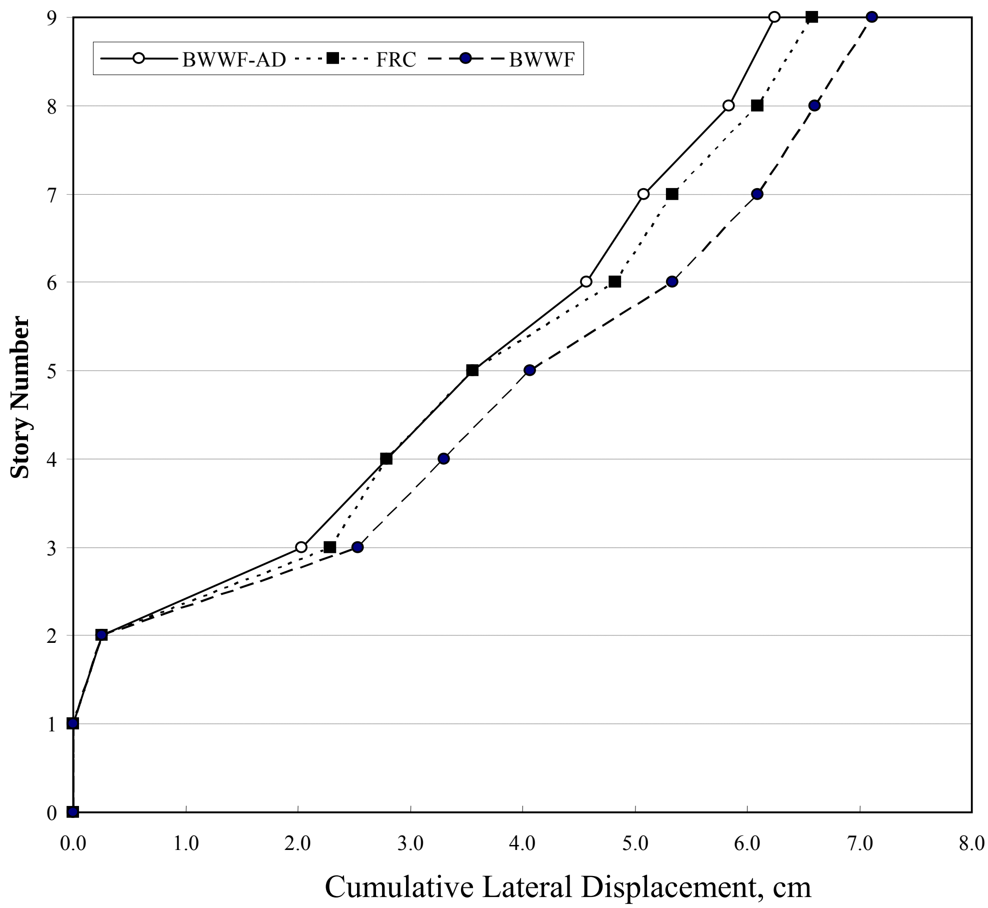

Figure 6: Lateral Displacement Skeleton Curve for 9-Story Frame (Earthquake 3, PGA = 0.39g)

Concluding Remarks

Essentially, in comparison with FR frames, the presence of BWWF-AD connections in PR frames, in most cases, had no significant effect on the maximum top lateral displacements of the frames or the interstory displacements. No significant effect on the maximum connection rotation or drift could be observed. In a few cases when the ground shaking was significant enough to produce larger lateral displacements, the responses where improved when BWWF-AD connections were present in PR frames by reducing the lateral interstory displacements, the maximum connection rotation and drift.

BWWF-AD connections used in this study have large elastic stiffness, ductility, and energy absorption capacity in comparison with other types of PR connections. When the ground shaking was not significant enough to produce large lateral displacement in the frame, the responses of SMRF with BWWF-AD connections were not significantly affected when they are modeled as PR in comparison with modeling them as FR. In some cases, when ground shaking was significant enough to produce large lateral displacements, the presence of BWWF-AD connections improved the responses of the frames.

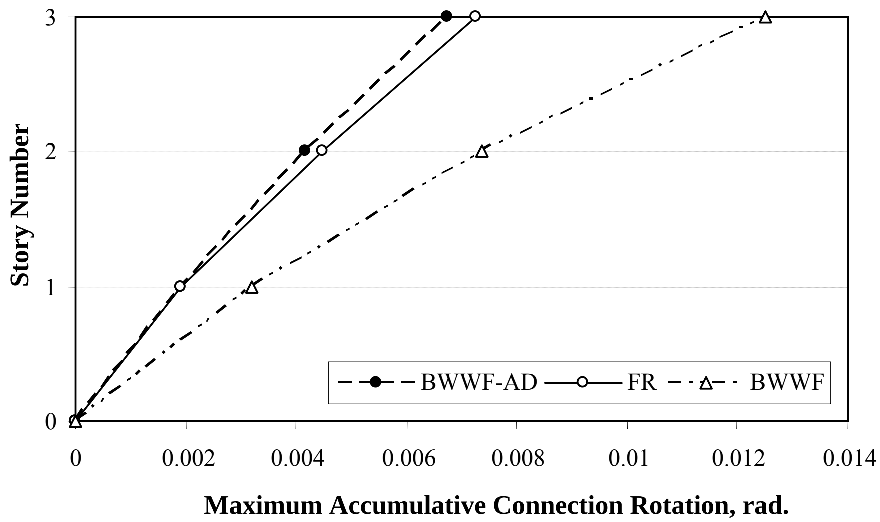

Figure 7: Absolute Maximum Connection Rotation for 3-Story Frame for Earthquake 1 (PGA = 0.35g)

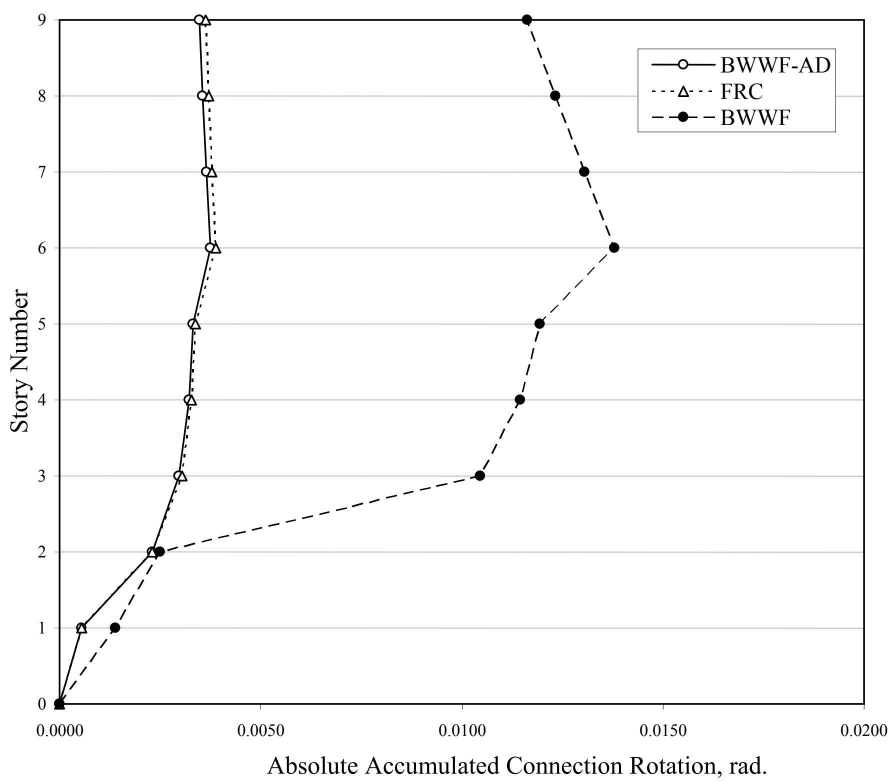

Figure 8: Absolute Maximum Connection Rotation Curve for 9-story Frame for Earthquake 7 (PGA = 1.2g)

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Biography

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