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Seismic response analysis of steel frames with post-Northridge connection

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Abstract. The seismic behavior of two steel moment-resisting frames, which satisfy all the current seismic design requirements, are evaluated and compared in the presence of pre-Northridge connections denoted as BWWF and an improved post-Northridge connections denoted as BWWF-AD. Pre-Northridge connections are modeled first as fully restrained (FR) type. Then they are considered to be partially restrained (PR) to model their behavior more realistically. The improved post-Northridge connections are modeled as PR type, as proposed by the authors. A sophisticated nonlinear time-domain finite element program developed by the authors is used for the response evaluation of the frames in terms of the overall rotation of the connections and the maximum drift. The frames are excited by ten recorded earthquake time histories. These time histories are then scaled up to produce some relevant response characteristics. The behaviors of the frames are studied comprehensively with the help of 120 analyses. Following important observations are made. The frames produced essentially similar rotation and drift for the connections modeled as FR type and PR type represented by BWWF-AD indicating that the presence of slots in the web of beams in BWWF-AD is not detrimental to the overall response behavior. When the lateral displacements of the frames are significantly large, the responses are improved if BWWF-AD type connections are used in the frames. This study analytically confirms many desirable features of BWWF-AD connections. PR frames have longer periods of vibration in comparison to FR frames and may attract lower inertia forces. However, calculated periods of the frames of this study using FEMA 350 empirical equation is longer than those calculated using dynamic characteristics of the frames. This may result in even lower design forces and may adversely influence the design.

Key words: seismic analysis; response; nonlinear; steel frames; post-Northridge and pre-Northridge connection; beam-column connection; drift; PR and FR connection; connection rotation; stiffness; damage.

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1. Introduction

Extensive damage to beam-column connections in steel moment-resisting frames (SMRFs) during the Northridge earthquake of 1994 and Kobe earthquake of 1995 forced the profession to reexamine seismic design practices existed before these events. One of the typically damaged steel beam-column connections during the Northridge earthquake of 1994 was bolted-web, welded-flange (BWWF) connection. As conceptually illustrated in Fig. 1, BWWF connections were fabricated with the beam flanges attached to the column flanges by full penetration welds (field-welded) and with the beam web bolted (field-bolted) to single plate shear tabs (Richard and Radau 1998). BWWF connections are generally referred to as “pre-Northridge” connections. BWWF connections were commonly used in the construction of SMRFs prior to the Northridge earthquake; however, many of them fractured in a brittle and premature manner during the earthquake. Following the Northridge earthquake, Structural Engineering Association of California (SEAOC) recommended not to use BWWF connections.

The post-Northridge steel connection research emphasized on identifying the causes of the damage, and to develop new steel beam-column connections that could improve the overall response, ductility and the quality of future designs. In the post-Northridge design practices of steel connections, the thrusts are to make the connections more flexible than the pre-Northridge connections and to move the location of formation of any plastic hinge away from the connection. Providing more ductility to increase the energy absorption capacity of the connection during an earthquake is another major objective in the post-Northridge steel connection design. Several improved connections can be found in the literature including cover plated connections (Engelhardt and Sabol 1995), spliced beam connections, side-plated connections, bottom haunch connections, connections with vertical ribs, and connections with a reduced beam sections (RBS) or Dog-Boned (FEMA 350-3). Other forms of “post-Northridge” connections are also available in the literature (FEMA 350-3).

Seismic Structural Design Associates, Inc. (SSDA) proposed a unique proprietary slotted web (SSDA SlottedWeb^{TM}) moment connection and tested several full-scale models (Richard *et al*. 1997). This connection is referred to hereafter as bolted-web, welded-flange with adequate ductility (BWWF-AD) and is conceptually illustrated in Fig. 2. As illustrated, top and bottom slots are cut in the beam web at the toe of the beam flange fillet from the end of the beam to a shop drilled 2.70 cm (1-1/16 in) in diameter slots termination holes. Three slot dimensions of the connection, labeled as L_{s1}, L_{s2}, and t_s, are also illustrated Fig. 2. These dimensions play a pivotal role in the design performance of BWWF-AD connections. L_{s1} is the slot length (top and bottom), measured from the beam end (web edge) to the center of the termination hole of slot. The slot length depends on nominal beam flange width, flange thickness, steel grade (yield strength), and nominal beam depth. The distance of L_{s2}, measured from the end of weld access hole of the beam to the center of the slot termination hole, is calculated by subtracting a maximum of 3.81 cm (1.50 in) or less from the slot length depending on the size of weld access hole. The slot width, t_s, shown in Fig. 2 varies from 0.32 cm (1/8 in) to 0.64 cm (1/4 in) depending on the width of the shear plate. A detailed design method and rationale for SSDA SlottedWeb^{TM} moment connection including the design of shear plate can be found in SSDA Beam Slot Connection Manual (1999) and is not presented here due to lack of space.

SSDA test results using ATC-24 test protocol and analytical studies showed substantial increase in the connection’s strength and ductility among other beneficial effects in comparison with BWWF connections (SSDA 2001). These tests met the plastic rotation criterion of 0.030 radian as required in the AISC (American Institute of Steel Construction) Seismic Provisions for Structural Steel Buildings, dated April 15, 1997. These tests also met the required SAC (a joint venture of the Structural Engineers

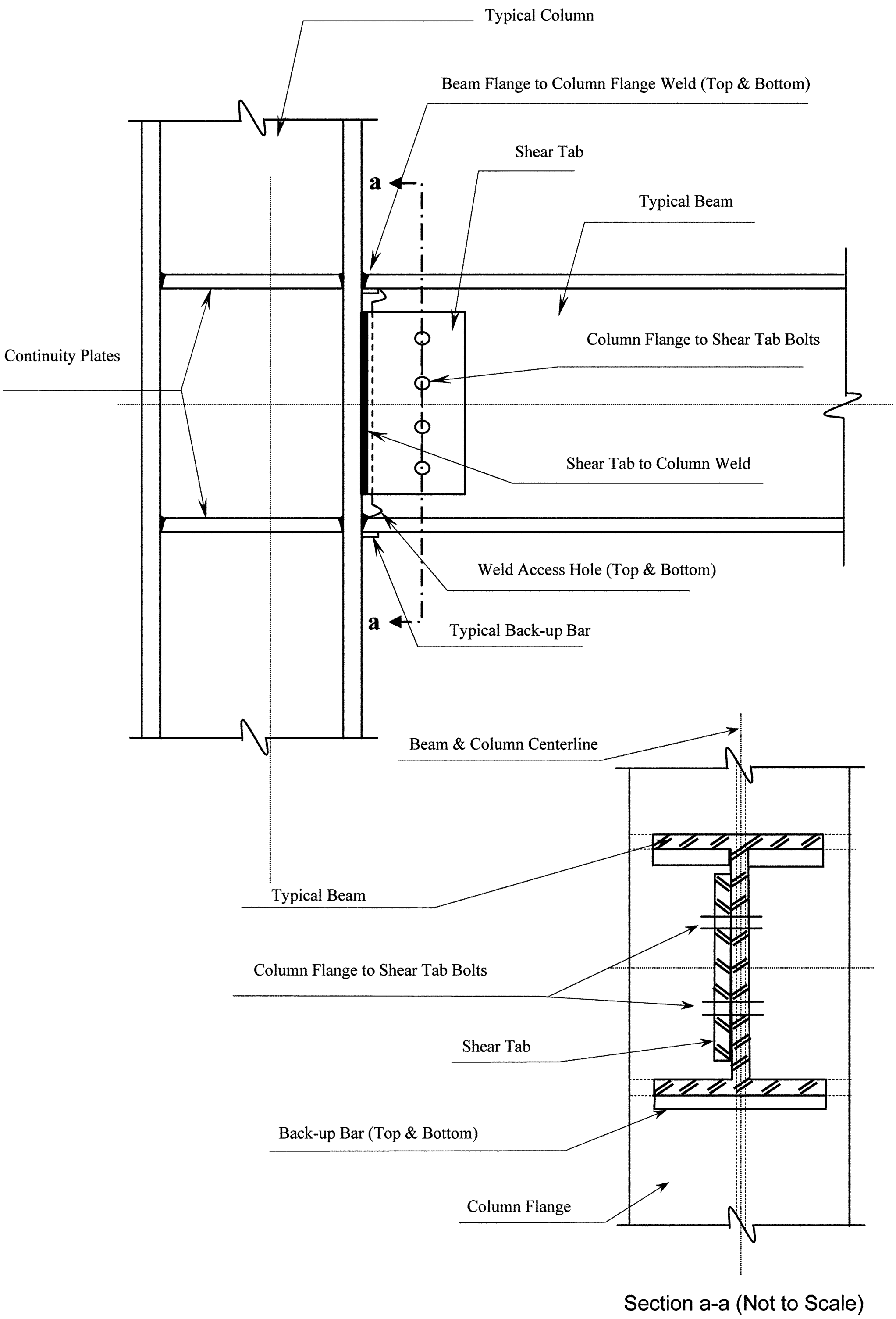

Fig. 1 A typical BWWF connection detail (Not to Scale)

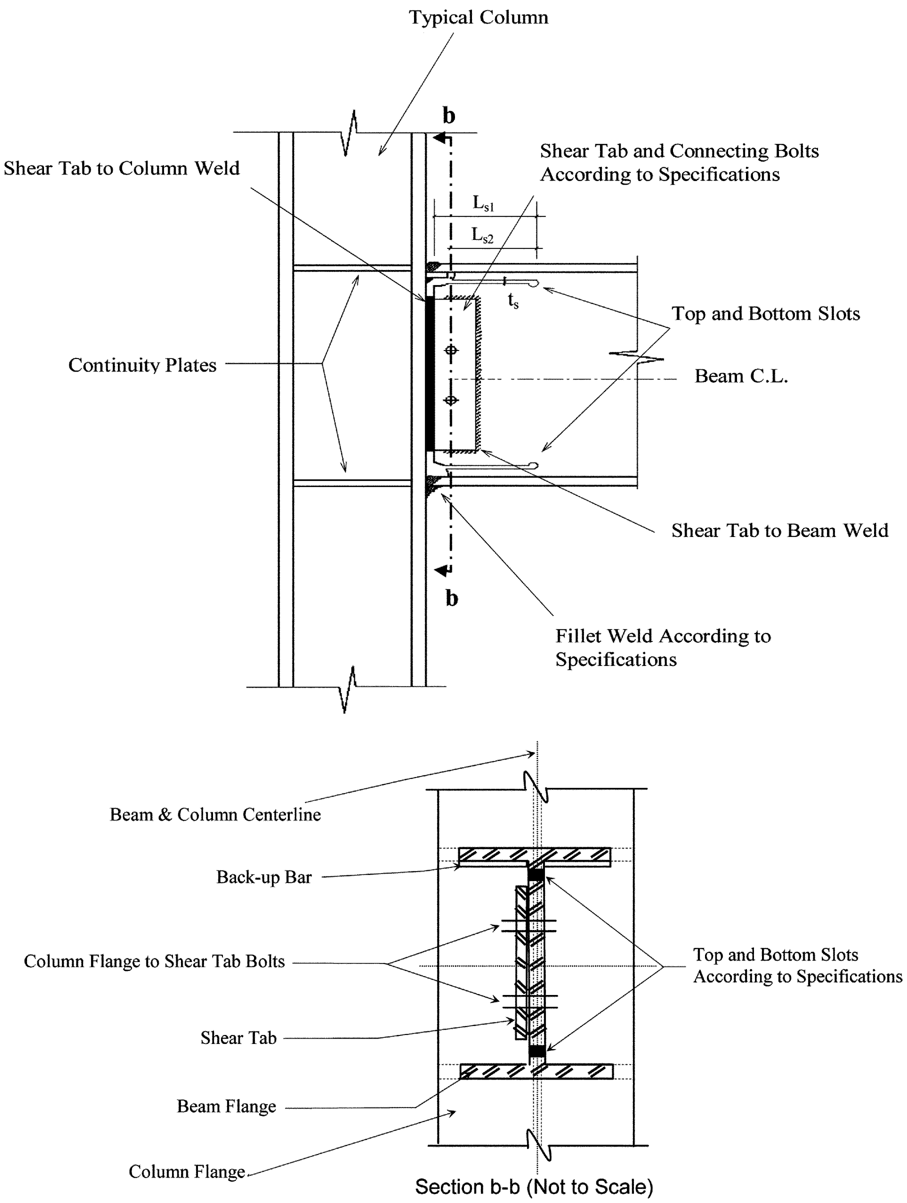

Fig. 2 A BWWF-AD (SlottedWeb™) connection detail (Not to Scale)

Association of California, the Applied Technology Council, and the Consortium of Universities for Research in Earthquake Engineering) and FEMA (Federal Emergency Management Agency) criterion of 0.040 radian of drift angle capacity for Special Moment Frames (SSDA 2001). ATC-24 protocol tests indicate that these connections develop full plastic moment capacity of the beam and do not reduce the elastic stiffness of the beam. They also developed stable hysteretic loops during testing. BWWF-AD connections 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 that must be considered appropriately for seismic response analysis, particularly for strong motion earthquakes.

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 (Mehrabian 2002). The model can generate M-θ curves for other beam-column assemblies not tested by SSDA. This extension was very beneficial in modeling steel frames in the presence of different BWWF-AD connections that can be expected in structures.

Following the Northridge earthquake, the SAC Steel Project proposed several benchmark SMRFs that satisfy all the current seismic design guidelines. Two such SMRFs are considered in this study and their behaviors are investigated in the presence of BWWF and BWWF-AD connections. These frames have been used in many research projects in the past and their descriptions will be given in the following sections. However, as mentioned earlier, the estimation of nonlinear seismic response of SMRFs in the presence of BWWF-AD is not simple. Of particular interest is to study the overall rotational deformation of the connections and its effect on the drift of the SMRFs since drift is an important design parameter. For the analysis purpose, both BWWF and BWWF-AD connections are FR type. However, as reported in the literature (Mehrabian 2002, Reyes and Haldar 1999), steel connections are PR type with various rigidities. Some general important questions related to the topics are addressed in this study such as: how would the seismic performance of frames is altered if BWWF connections are modeled as PR type as opposed to FR type? In seismic analysis, how would the response of a frame be affected if the connections were modeled as BWWF-AD? What would be the implications of modeling a frame with BWWF-AD as a frame containing PR connections?

2. Description of the frames

Two SMRFs, shown in Fig. 3, are considered and their seismic responses are evaluated in the presence of different connections. These frames are the North-South SMRFs 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 (National Earthquake Hazards Reduction Program) Provisions. These buildings are presented in FEMA-355F and are used in many studies by the SAC Steel Project. These frames are referred to as Frame 1 and Frame 2 in this study.

As shown in Fig. 3, based on the geometry, both frames can be classified as “regular” in elevation. Frame 1 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 a bay width of 9.14 m (30 ft) for all of the bays. Frame 2 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 given in Table 1 and are shown in Fig. 3.

Using the dynamic characteristics of the frames, the fundamental period of vibration T is calculated to be 0.21 seconds for Frame 1 and 1.23 seconds for Frame 2. According to FEMA 350, Eq. (4-1), the fundamental period of vibration can be calculated as:

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

where T and h_n are the fundamental period of vibration (in seconds) in the direction under consideration

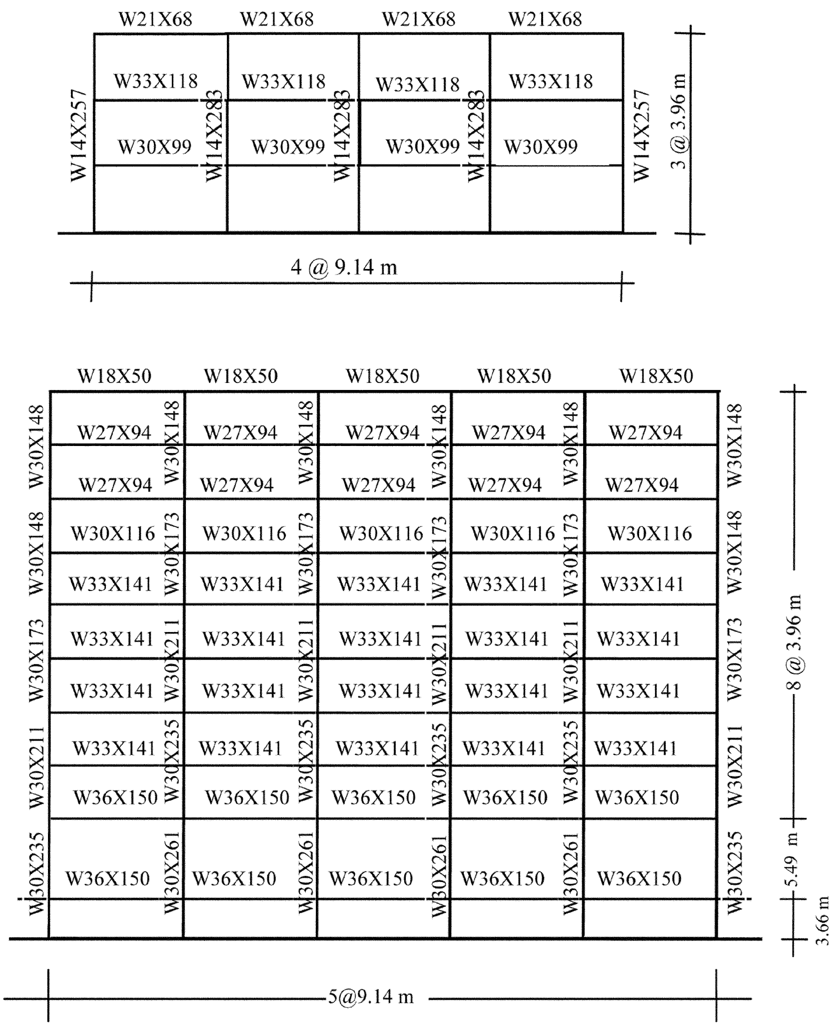

Fig. 3 N-S SMRFs of SAC 3- and 9-story model buildings designed according to 1997 NEHRP provisions

and height (in feet) to the roof level above base, respectively.

For comparison, using Eq. (1), T is calculated to be 0.53 seconds for Frame 1 and 1.41 seconds for Frame 2. The theoretical T values and the corresponding empirical values obtained by Eq. (1) are quite different. In fact, the FEMA 350 equation gives the fundamental periods to be 60% and 13% higher than the theoretical values for Frame 1 and Frame 2, respectively.

To meet the objectives of the study, the behavior of the two frames in the presence of three different types of connections are considered. The connections are considered to be i) fully restrained (FR), ii) partially restrained (PR) and modeled as BWWF, and iii) PR and modeled as BWWF-AD. The frames are excited by ten different recorded earthquake time histories. The time histories are also scaled to force the frames to behave differently. Thus, 120 analyses are performed. The nonlinear responses of the frames are evaluated by using a sophisticated computer program developed by the research team (Haldar and Nee 1989, Gao and Haldar 1995, Reyes and Haldar 1999, Mehrabian 2002). The program

Table 1 Member sizes for Frames 1 and 2

Frame 1 (3-story)			
Story	Columns		Beam
	Exterior	Interior	
0 – 1	W14×257	W14×283	W30×99
1 – 2	W14×257	W14×283	W33×118
2 – 3	W14×257	W14×283	W21×68
Frame 2 (9-story and basement)			
Story	Columns		Beam
	Exterior	Interior	
Basement - 0	W30×235	W30×261	W36×150
0 – 1	W30×235	W30×261	W36×150
1 – 2	W30×211	W30×235	W33×141
2 – 3	W30×211	W30×235	W33×141
3 – 4	W30×173	W30×211	W33×141
4 – 5	W30×173	W30×211	W33×141
5 – 6	W30×148	W30×173	W30×116
6 – 7	W30×148	W30×173	W27×94
7 – 8	W30×148	W30×148	W27×94
8 – 9	W30×148	W30×148	W18×50

is based on the assumed-stress based finite element method (FEM). It is capable of evaluating the seismic response in time-domain considering all major sources of nonlinearity. The program is very efficient and has been verified extensively. The complete formulation of the algorithm cannot be presented here due to lack of space. Only the essential elements of the algorithm are briefly discussed below.

3. Nonlinear seismic response analysis

The nonlinear dynamic equation of motion can be expressed in the iterative form as:

$$\mathbf{M}\ddot{\mathbf{D}}^{(n)}_{(t+\Delta t)} + \mathbf{C}_t\dot{\mathbf{D}}^{(n)}_{(t+\Delta t)} + \mathbf{K}^{(n)}_{(t+\Delta t)}\Delta\mathbf{D}^{(n)}_{(t+\Delta t)} = \mathbf{F}^{(n)}_{(t+\Delta t)} - \mathbf{R}^{(n-1)}_{(t+\Delta t)} - \mathbf{M}\ddot{\mathbf{D}}^{(n)}_{g,(t+\Delta t)} \tag{2}$$

where **R**, **C**, and **K** are the mass, damping and the global tangent stiffness matrices, respectively, $\ddot{\mathbf{D}}$, $\dot{\mathbf{D}}$, and **D** are the acceleration, velocity, and the relative displacement vectors, respectively, $\Delta\mathbf{D}$ is the incremental displacement vector, **F** is the external load vector, **R** is the internal force vector and $\ddot{\mathbf{D}}_g$ is the ground acceleration vector. Superscript (n) and subscript ($t + \Delta t$) indicate the iteration number and the time, respectively.

The global mass, tangent stiffness matrices and internal force vectors are developed by assembling information on all the elements using the standard finite element concept. Each column and each beam of the frame is modeled as one element except at the location of the connections. The connections are modeled as a kind of beam-column element as discussed in Haldar and Nee (1989). Explicit expressions for the tangent stiffness matrix and the internal force vectors are developed for each beam-column element for the nth iteration at time t. The nonlinear elastic tangent stiffness matrix for a beam-column element, $\mathbf{K}^e$, can be expressed as:

$$\mathbf{K}^e = \mathbf{A}_{\sigma do}^T \mathbf{A}_{\sigma\sigma}^{-1} \mathbf{A}_{\sigma do} + \mathbf{A}_{ddo} \tag{3}$$

where $\mathbf{A}_{\sigma\sigma}$, $\mathbf{A}_{\sigma do}$ and $\mathbf{A}_{ddo}$ are the elastic stiffness property, transformation, and the geometric stiffness matrices, respectively. Similarly, the internal force vector of an element level $\mathbf{R}^e$ can be expressed as:

$$\mathbf{R}^e = -\mathbf{A}_{\sigma do}^T \mathbf{A}_{\sigma\sigma}^{-1} \mathbf{R}_{\sigma} + \mathbf{R}_{do} \tag{4}$$

where $\mathbf{R}_{do}$ and $\mathbf{R}_{\sigma}$ are the homogeneous part of the internal force vector and the deformation difference vector, respectively. The explicit expressions for all the terms in Eqs. (3) and (4) can be found in the literature (Gao and Haldar 1995) and are not given here due to lack of space.

For plane structures, the effect of torsion on the connection deformation can be ignored. In modeling the connections of SMRFs as beam-column elements, the effects of shear and axial forces are small in comparison to the bending moment and can be neglected. Therefore, only the effects of bending moment are considered.

4. Connection modeling

The comprehensive properties of a PR connection are commonly described by the relationship between moment transmitted by the connection (M) and the relative angle of rotation between connecting members (θ), generally referred to as M-θ curves. In a realistic seismic analysis of steel frames, the information on the moment-relative rotation (M-θ) curve of each connection is essential. The information on M-θ curves of connections becomes even more essential in the nonlinear seismic analysis of SMRFs. Due to the nonlinear behavior of M-θ curves from the very beginning of the loading, M-θ curves must be modeled as accurately as possible using a nonlinear mathematical model. Furthermore, the initial stiffness of each beam-column connection and the changes in its stiffness during the loading, unloading, and reloading of the frame need to be calculated accurately.

As mentioned earlier, M-θ curves for the BWWF-AD connections used in this study were developed from the experimental data of full-scale ATC-24 laboratory tests provided by SSDA. Among several available mathematical models, Richard 4-parameter model is employed for modeling the M-θ curves in this study. The model 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 Richard 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 are evaluated using the actual test data (Mehrabian 2002). The accuracy of this model is found to be good. Using a procedure described by Mehrabian (2002), M-θ curves for the untested BWWF-AD connections can also be generated. The M-θ curves for some of the connections used in this study are shown in Fig. 4.

The M-θ curves for modeling the BWWF connections are generated using PRCONN computer program (Richard 1993). For a user specified connection, PRCONN generates M-θ curve for a connection using the Richard equation. To generate M-θ curves using PRCONN, the BWWF connections are modeled as single web angle, top and seat (SWATS) connections. Mehrabian (2002) showed that the elastic stiffness of the BWWF-AD connections matches the elastic stiffness of SWATS connections, but the ductility and the energy dissipation capacity of BWWF-AD connections are much higher. In

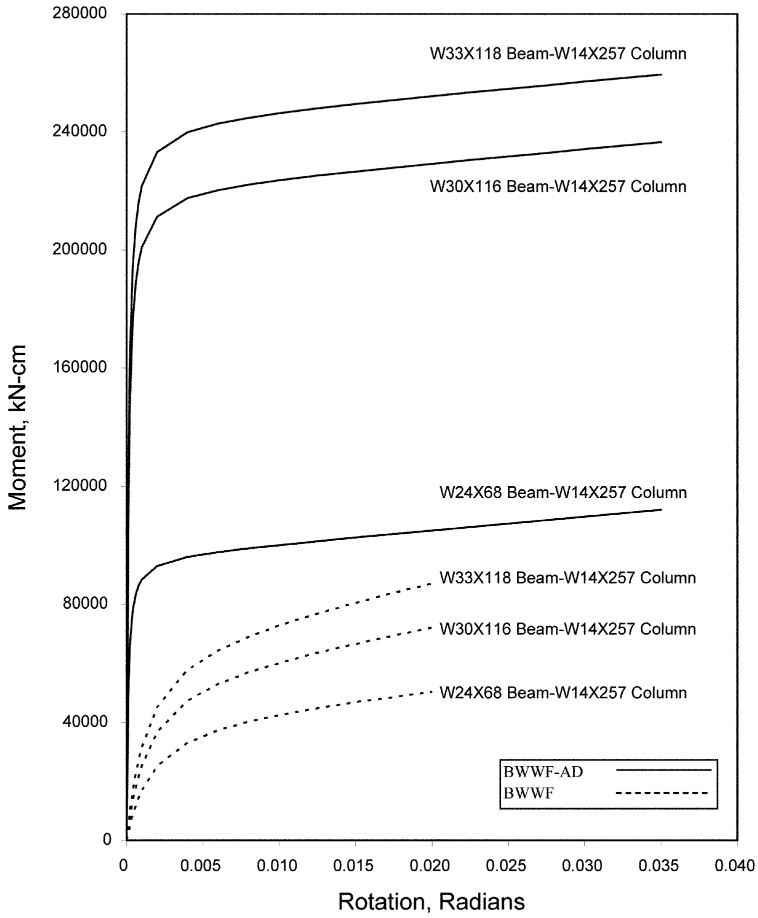

Fig. 4 Moment-rotation curves for BWWF and BWWF-AD connections for Frame 1

designing the SWATS connections, top and bottom angle lengths are assumed to be 22.86 cm (9 in) to match the least width of the beam flanges. Assuming no web adjustment length, the neutral axes of the beam and the connected web angle coincide. The top and bottom angles and the web angle thickness is 1.27 cm (0.5 in), and eight 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.

Initially, all the connections in the two frames are assumed to be of FR type, a commonly used practice in the profession. In this case, the full bending moment induced in the beams is transmitted to the columns. The T ratio is defined as M_b / M_{fix}, where M_b is the beam-end-moment and M_{fix} is the fixed-end-moment. If the T ratio is at least 0.9, the connection can be considered as FR type (Reyes and Haldar 1999). All FR type connections used in this study satisfy this criterion and have T ratios greater than 0.9. It should be noted that the T 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. This is discussed in detail by Astaneh in 1989. The calculated T ratios using the elastic beam-line concept may underestimate the real values of T ratios. As mentioned earlier, first all the connections are considered to be FR type. Next, both BWWF and BWWF-AD connections are modeled as PR type.

5. Description of the earthquakes

Two suits of ten different actual acceleration time histories with different scale factors are used in this study. These earthquakes are described in detail elsewhere (Mehrabian 2002). The 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 at different stations across Southern California. Six of these earthquakes were recorded during the Northridge Earthquake of 1994, and four of them were recorded during the El Centro earthquake of 1940, San Fernando earthquake of 1971, and Whittier Narrows earthquake of 1987. As an example, Fig. 5 shows the two horizontal components of acceleration time histories recorded at Malibu Station during the Northridge earthquake of 1994. All design level 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 histories shown in Fig. 5, the component of the earthquake containing the larger peak ground acceleration (PGA) is applied to the frame in the direction of the stronger axis of the frame. A method for rationalizing the appropriate scale factors is discussed next.

Actual acceleration time histories are used first to calculate the responses of the frames. However, in some cases, they did not produce any significant response. To study the response of the frames comprehensively, time histories are then scaled according to the natural period of vibration of the frames, to represent the earthquakes with 2% probability of occurrence in 50 years as presented in FEMA-355F. The scales are adjusted accordingly to match the PGA of Response Spectra for 5% damping for Los Angeles, as suggested in FEMA-355F. For Frame 1 with a fundamental period of vibration of 0.53 seconds calculated using Eq (1), the time histories are scaled in such a way to represent a PGA of 1.6 g (1570 cm/sec.^{2}). For Frame 2 with a fundamental period of vibration of 1.41 seconds, the time histories are scaled in such a way to represent a PGA of 1.2 g (1177 cm/sec.^{2}). Mehrabian (2002) discussed the appropriate scale factor to be used in the analysis for each earthquake elsewhere. Table 2 shows the appropriate scale factors for Frames 1 and 2.

For the ease of discussion, the actual earthquakes and scaled earthquakes are referred to as AE and SE, respectively, followed by the earthquake record number. For examples, actual earthquake record no. 1 is referred to as AE1 and scaled earthquake no. 5 is referred to as SE5.

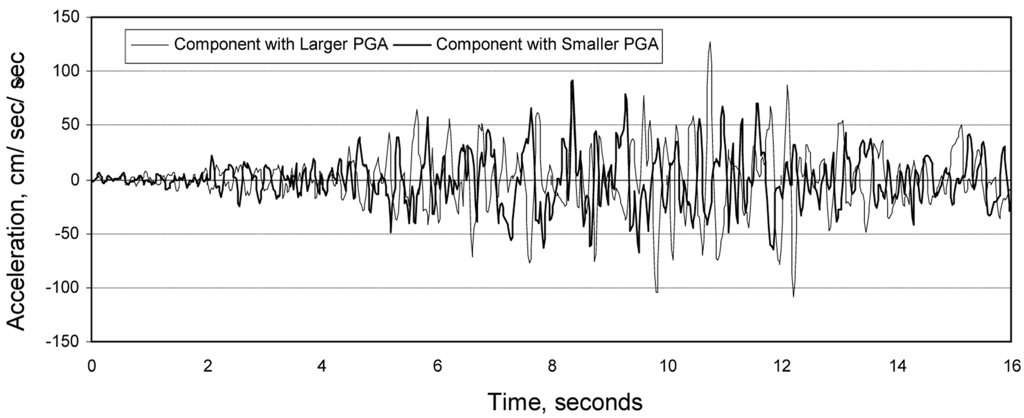

Fig. 5 Horizontal components of Norhtridge '94 earthquake - Malibu Station

Table 2 Summary of scaled earthquakes (SE) for Frame 1 (PGA = 1.6 g) and Frame 2 (PGA = 1.2 g)

Earthquake		Station	Scale factor	
			Frame 1	Frame 2
SE1	El Centro '40	San Fernando Valley	4.59	3.44
SE2	Northridge '94	Northridge - 17645 Saticoy Street	3.54	2.65
SE3	Northridge '94	Canoga Park - 7769 Topanga Canyon Rd	4.12	3.09
SE4	Northridge '94	Santa Monica - City Hall	1.81	1.36
SE5	Whittier Narrows '87	Northridge - 17645 Saticoy Street	10.68	8.01
SE6	Northridge '94	Malibu - Point Dume	12.36	9.27
SE7	San Fernando '71	Los Angeles - 1150 South Hill	18.92	14.18
SE8	Northridge '94	New Hall - Pico Canyon Rd.	3.82	2.86
SE9	Northridge '94	Simi Valley - 6334 Katherine Rd.	2.20	1.65
SE10	Northridge '94	Beverly Hills - 12520 Mulhollan Dr.	2.72	2.04

6. Response of the frames

The responses of the two SMRFs excited by all the earthquake time histories are evaluated using the computer program discussed earlier. Initially, all the connections are considered to be pre-Northridge BWWF type. Following the general practice, they are first modeled as FR type. Then, they are modeled as PR type representing their realistic behavior. Finally, they are considered to be post-Northridge BWWF-AD type PR connections. As mentioned earlier, all PR connections are represented by the four-parameter Richard model. The four parameters for BWWF and BWWF-AD connections are presented in Tables 3 and 4.

Table 3 Parameters of Richard equation for Frames with pre-Northridge BWWF connections

Beam	Column	Parameters of Richard equation			
		K, kN-cm/rad	K_p, kN-cm/rad	M_o, kN-cm	N
W18×50	W30×148	1.03E+07	2.94E+05	1.96E+04	1.4
W24×68	W14×257	2.51E+07	5.56E+05	4.16E+04	1.1
W24×68	W14×311	2.51E+07	5.56E+05	4.16E+04	1.1
W27×94	W30×173	3.65E+07	8.48E+05	5.57E+04	1.1
W27×94	W30×148	3.65E+07	8.48E+05	5.57E+04	1.1
W30×116	W14×257	3.95E+07	9.19E+05	5.65E+04	1.1
W30×116	W14×311	3.95E+07	9.19E+05	5.65E+04	1.1
W30×116	W30×148	4.38E+07	1.02E+06	6.22E+04	1.1
W30×116	W30×173	4.38E+07	1.02E+06	6.22E+04	1.1
W33×118	W14×257	5.08E+07	1.14E+06	6.79E+04	1.1
W33×118	W14×311	5.08E+07	1.14E+06	6.79E+04	1.1
W33×141	W30×211	5.19E+07	1.21E+06	6.88E+04	1.1
W33×141	W30×173	5.19E+07	1.21E+06	6.88E+04	1.1
W36×150	W30×235	5.89E+07	1.38E+06	7.41E+04	1.1
W36×150	W30×261	5.89E+07	1.38E+06	7.41E+04	1.1

Table 4 Parameters of Richard equation for Frames with post-Northridge BWWF-AD connections

Beam	Column	Parameters of Richard equation			
		K, kN-cm/rad	K_p, kN-cm/rad	M_o, kN-cm	N
W18×50	W30×148	6.85E+08	4.52E+05	6.18E+04	1.0
W24×68	W14×257	1.00E+09	4.52E+05	9.64E+04	1.0
W24×68	W14×311	1.00E+09	4.52E+05	9.64E+04	1.0
W27×94	W30×173	1.89E+09	4.52E+05	1.94E+05	1.0
W27×94	W30×148	1.89E+09	4.52E+05	1.94E+05	1.0
W30×116	W14×257	2.14E+09	4.52E+05	2.21E+05	1.0
W30×116	W14×311	2.14E+09	4.52E+05	2.21E+05	1.0
W30×116	W30×148	2.56E+09	4.52E+05	2.69E+05	1.0
W30×116	W30×173	2.56E+09	4.52E+05	2.69E+05	1.0
W33×118	W14×257	2.34E+09	4.52E+05	2.44E+05	1.0
W33×118	W14×311	2.34E+09	4.52E+05	2.44E+05	1.0
W33×141	W30×211	3.94E+09	4.52E+05	4.20E+05	1.0
W33×141	W30×173	3.94E+09	4.52E+05	4.20E+05	1.0
W36×150*	W30×235*	3.95E+09	4.52E+05	4.52E+05	0.8
W36×150*	W30×261*	3.95E+09	4.52E+05	4.52E+05	0.8

*Actual SSDA test data are used

6.1. Connection rotation

In designing the frames with PR connections, the angle of rotation that a connection undergoes as the load increases is an important parameter in assessing the ductility of the connection and the overall seismic performance of the frame. A SMRF designed with PR connections require sufficient ductility since it is expected that the connections provide the majority of the inelastic behavior during an earthquake. One way to assess the ductility of PR frames is to study their time-domain response behavior in terms of the overall angle of rotation of the connections.

The responses of the frames in terms of the absolute maximum angle of rotation of the connections, denoted hereafter as α, is considered first. α represents the maximum accumulative angle of rotation of the connections at each story. α values for the frames are plotted along the height of the frames. Two such plots are shown in Figs. 5 and 6 for Frames 1 and 2, respectively.

For Frame 1, α does not exceed 0.0125 radian for any of the earthquakes considered in this study. No significant difference in α can be observed for the frame modeled with three different types of connections and excited by either the actual or the scaled earthquakes. Only for AE1, α for the PR Frame 1 with BWWF connections are significantly larger than the other two cases as shown in Fig. 6. For AE1, α is almost identical when the connections are modeled as FR type and PR type represented by BWWF-AD. Similar observation is made for Frame 1 excited by scaled earthquakes with PGA of 1.6 g where α is almost identical regardless of the types of connection used to model the frame.

The response behavior of Frame 2 excited by actual earthquakes shows that in most cases, α for FR frames is similar to PR frames modeled with BWWF-AD connections. However, α is larger for PR frames with BWWF connections. Similar observations are made for Frame 2 excited by scaled earthquakes. One example is shown in Fig. 7 for SE7 with PGA of 1.2 g where α is much larger for Frame 2 with BWWF connections in comparison with FR frames and PR frames with BWWF-AD

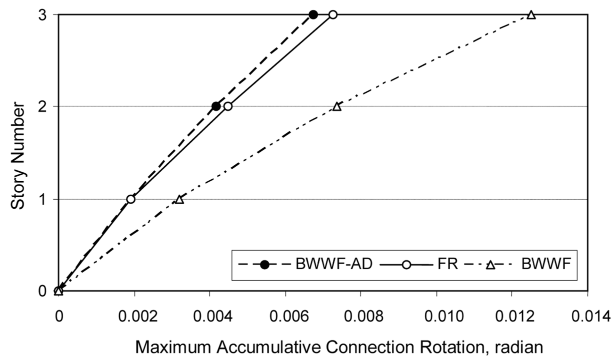

Fig. 6 Absolute maximum connection rotation for Frame 1 for AE1 (PGA = 0.35 g)

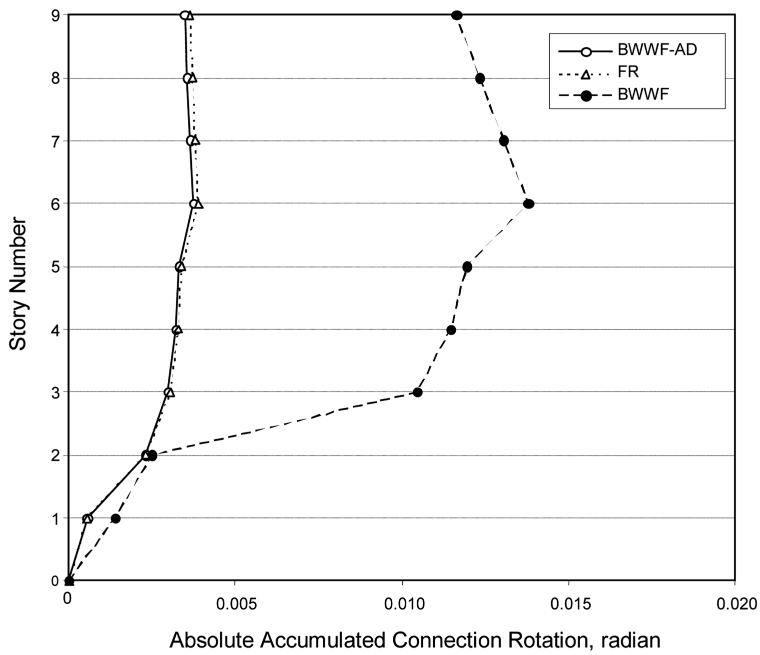

Fig. 7 Absolute maximum connection rotation curve for Frame 2 For SE7 (PGA = 1.2 g)

connections. Similar observations are made for Frame 2 excited by other scaled earthquakes. Additional information on response behavior of the frames excited by actual and scaled earthquakes can be obtained from Mehrabian (2002) and is not given here due to lack of space.

It is generally expected that a well-detailed steel connection with sufficient ductility will be able to tolerate a large angle of rotation, of the order of 0.03 radians. The calculated α values for both frames excited by different earthquakes in the presence of three types of connections are less than this value, although they are much larger in PR frames with BWWF connections. It can be observed that α depends on the ground motion being considered. Overall, it is observed that the calculated α of the PR frames modeled with BWWF-AD is similar or slightly less than that of if they are modeled as FR type for all the ground motions considered in this study. This observation contradicts the general assumption that the frames with PR connections may experience larger angle of rotation than the frames with FR connections. Similar observation was made by Nader and Astaneh (1991) in laboratory investigations.

Generally, the elastic stiffness of a PR connection is expected to be less than the FR connection. Certainly, the elastic stiffness can vary for different PR connections reflecting the presence of different amount of rigidity. However, Mehrabian (2002) observed that the BWWF-AD connections considered in this study have much larger elastic stiffness compared to other types of commonly used PR connections. BWWF-AD connections can also develop full plastic moment capacity of the connecting beam before failure. For the PR frames studied, the large elastic stiffness and ductility of BWWF-AD connections, in the absence of formation of any observed plastic hinge in this study, contribute to their improved response over BWWF.

6.2. Drift analysis and comparison with IBC 2000

Roof drift or simply drift is referred to as the maximum lateral top story displacement of the frame divided by the frame height. Drift is a critical parameter in seismic design of steel PR frames. Excessive drift can cause the frame to go beyond its drift demand and may cause instability and collapse. It has been a concern among the designers, albeit unjustifiably, that the presence of PR connections may increase the flexibility of a steel frame and may cause large drift when subjected to strong motion earthquakes. Therefore, it is very desirable to study the effect of the presence of different types of connections in steel frames on drift evaluations.

As before, drifts are calculated for both frames in the presence of three types of connection, i.e., all the connections are either FR type or PR type represented by BWWF and BWWF-AD. Drift is also calculated for each frame using the provisions given in the International Building Code (IBC 2000). According to IBC 2000, Section 1617, in simplified analysis procedure for seismic design of buildings, the design drift Δ shall be taken as one percent of the story height. This is a conservative drift limit and is assumed as a reference value. It is plotted along with the calculated drift value for Frames 1 and 2 excited by scaled earthquakes in Figs. 8 and 9, respectively. Research has shown that drift limits, although primarily related to serviceability, also improve frame stability and seismic performance because of the resulting additional strength and stiffness (AISC 2002). Only drifts due to scaled earthquakes are discussed here since more meaningful interpretation of the results could be extracted. Additional information on drift response of the frames excited by actual earthquakes can be found in Mehrabian (2002) and is not given here due to lack of space.

As shown in Fig. 8, for Frame 1 subjected to SE8, 9, and 10, the drifts are identical or slightly less for the PR frames with BWWF-AD relative to the drift response of other frames. For SE2 through SE7, drifts are identical for the FR and PR frame with BWWF-AD. In all these six cases, PR frames with BWWF connections exhibited larger drifts with the case of SE2 and SE4 almost twice as large. For Frame 2 excited by scaled earthquakes drifts are identical for the FR frames and PR frames with BWWF-AD. In a few cases, PR frames with BWWF connections exhibited larger drifts. As shown in Fig. 9, for Frame 2 subjected to SE5, drifts are the same regardless of the connection type used. In other cases of scaled earthquakes, drifts are almost identical for the FR frames and PR frames with BWWF-AD connections. In three cases, drifts for PR frames with BWWF connections are larger in comparison with other frames. Under the application of SE7, drift is much larger for the PR frame with BWWF, even exceeding the IBC 2000 allowable drift.

In summary, in cases of applied actual and scaled earthquakes of this study, no significant differences have been observed between the calculated values of drift for both Frames 1 and 2 where connections are modeled as FR type and PR type represented by BWWF-AD. In general, for frames excited by

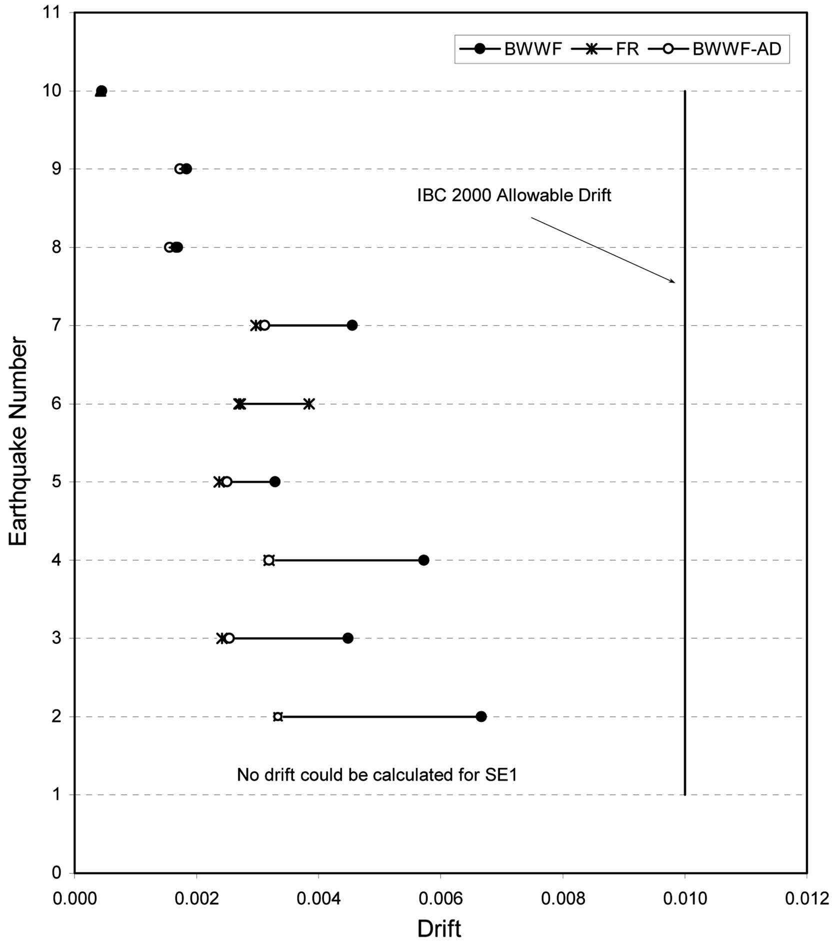

Fig. 8 Comparison of drift for Frame 1 using scaled earthquakes

actual earthquakes the calculated drift is found to be smaller than 0.006. When actual earthquakes are applied, the calculated drift is smaller in general in comparison to the applied scaled earthquakes. However, for scaled earthquakes where strong shaking of frames were observed, the drift is larger for the PR frames modeled as pre-Northridge BWWF connections in comparison with the other frames. In two cases, the frames excited by SE1 even developed very large lateral displacements suddenly, exceeding the IBC prescribed conservative drift limit; no drift could be calculated as noted in Figs. 8 and 9. In one case, for PR Frame 2 with BWWF connections the drift exceeded the conservative allowable design drift prescribed in IBC 2000 by 5 percent. This is an indication that modeling the steel frames with pre-Northridge BWWF connections as FR type may underestimate the actual drift of the frame and may have adverse affects on the seismic performance of the frame. The study substantiates that the inclusion of BWWF-AD connections in the design of SMRFs reduces the frames' drift in comparison with modeling the frames with BWWF connections. For the PR frames of this study with BWWF-AD connections, the drift is similar to FR frames and does not exceed the conservative drift limit of IBC 2000. This study analytically confirms many desirable features of BWWF-AD connections.

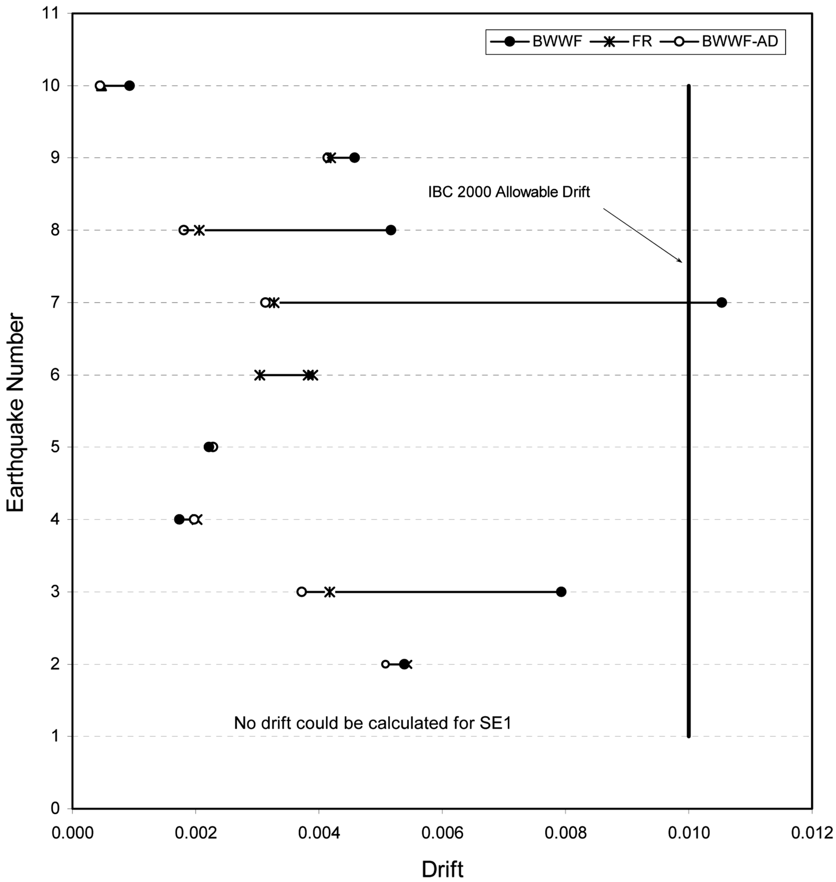

Fig. 9 Comparison of drift for Frame 2 using scaled earthquakes

7. Conclusions

Seismic responses of two SMRFs, in terms of overall connection rotation and drift, in the presence of an improved post-Northridge connection are calculated. Responses of the same frames with pre-Northridge connection are also calculated for comparison. Connections are mathematically modeled using the Richard model developed from the actual ATC-24 full-scale laboratory test data. Using a sophisticated nonlinear time-domain finite element program developed by the authors, the seismic responses of two benchmarked frames are evaluated in the presence of all major sources of nonlinearity. Ten recorded earthquake time histories are used for the response analyses. These time histories are then scaled up to produce some response characteristics.

With the help of 120 analyses, the following important observations can be made. The realistic representation of a connection behavior significantly influences the nonlinear seismic behavior of steel frames. In general, frames with pre-Northridge BWWF connections modeled as FR type, produce larger rotation and drift relative to modeling them as PR or post-Northridge BWWF-AD type. Many BWWF connections damaged prematurely during Northridge Earthquake of 1994. The frames with FR and BWWF-AD types connections behave similarly in most cases indicating that the presence of slots in the web of beams in BWWF-AD is not detrimental to the overall response behavior. In cases where the ground shaking produced significantly larger lateral displacements of the frames, the responses are improved if connections are modeled as BWWF-AD or FR type. However, modeling the frames with BWWF connections as FR type may underestimate the actual drift and may have adverse affects on the design. This study analytically confirms many desirable features of BWWF-AD connections. PR frames have longer periods of vibration in comparison to FR frames and may attract lower inertia forces. However, calculated periods of the frames of this study using FEMA 350 empirical equation is longer than those calculated using dynamic characteristics of the frames. This may result in even lower design forces and may adversely influence the design.

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