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Effect of connection modeling on the seismic response of steel braced non-moment resisting frames

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Abstract. Non-moment beam-to-column connections, which are usually referred to as simple or shear connections, are typically designed to carry only gravity loads in the form of vertical shears. Although in the analysis of structures these connections are usually assumed to be pinned, they may provide a small amount of rotational stiffness due to the typical connection details. This paper investigates the effects of this small rotational restraint of simple beam-to-column connections on the behavior and seismic response of steel braced non-moment resisting frames. Two types of commonly used simple connections with bolted angles, i.e., the Double Web angle Connection (DWC) and Unstiffened Seat angle Connection (USC) are considered for this purpose. In addition to the pinned condition - as a simplified representation of these connections - more accurate semi-rigid models are established and then applied to some frame models subjected to nonlinear pushover and nonlinear time history analyses. Although the use of bracing elements generally reduces the sensitivity of the global structural response to the behavior of connections, the obtained results indicate considerable effects on the local responses. Namely, our results show that consideration of the real behavior of connections is essential in designing the column elements where the pin-connection assumption significantly underestimates design of outer columns of upper stories.

Keywords: simple connections; steel braced frames; nonlinear analysis; seismic response; semi-rigid connections

1. Introduction

One of the basic assumptions of conventional structural analysis of steel frames is that beam-to-column connections behave either as ideally pinned or fully rigid. The use of an ideally pinned condition implies that no moment will be transmitted between connected members. On the other hand, the use of a fully rigid condition implies that no relative rotation will occur between connected members; therefore, the angle between the beam and the column remains ideally unchanged. However, experimental investigations show that most connections in practical cases respond between these simplified extremes (Nader and Astaneh-Asl 1996, Elnashai *et al*. 1998, Shen and Astaneh-Asl 1999, Shi *et al*. 2004, Chen, *et al*. 2017, Kong and Kim 2017). Simple or non-moment connections exhibit some rotational stiffness, while moment connections possess some degree of flexibility and consequently, the simplified representations may lead to unrealistic predictions of the response and strength of steel structures. One of the practical solutions to improve the accuracy of the structural analysis is to consider the actual rotational behavior of joints in the form of moment-rotation relationships (Lui and Chen 1987, Awkar and Lui 1999, Kim and Choi 2001, Diaz *et al*. 2011, Zohra and Abd Nacer 2018).

When a moment M is applied to a connection, it rotates by an angle θ. The rotation represents some changes in the angle between the beam and the column from its original configuration. Fig. 1 shows schematically the moment-rotation (M-θ) behavior of a connection. These types of M-θ curves are extensively obtained from experiments. If the direction of an applied moment is reversed, the connection will be unloaded and follow a different path which is almost linear with a slope equal to the initial slope of the loading curve (Chen and Lui 1987, Shen and Astaneh-Asl 2000, Sekulovic *et al*. 2002, Hadianfard 2012). This loading and unloading characteristic of the connections must be properly modeled in order to predict the response of the frame reliably.

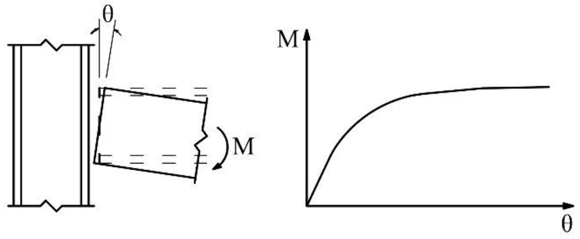

Fig. 1 Relative rotation of beam to column

So far, in the case of steel beam-to-column connections, most studies are limited to evaluate the flexibility of steel moment frames with fully restrained (rigid) or partially restrained (semi-rigid) connections. On the other hand, usually referred to as simple or shear connections, beam-to-column connections of non-moment frames are often assumed to be pinned for the purpose of structural analysis

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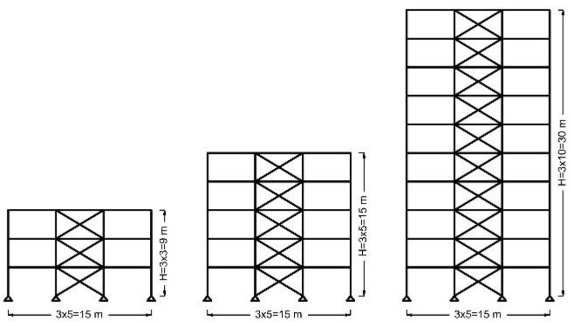


Fig. 2 Typical geometry of 2-D frame models used in this study

Table 1 Cross-sections of structural members in different models

Story	Outer columns	Inner columns	Beams	Braces
3-story model				
3	HE100B	HE120B	IPE360	Box 80×80×10
2	HE120B	HE180B	IPE360	Box 90×90×10
1	HE140B	HE220B	IPE360	Box 100×100×10
5-story model				
5	HE100B	HE120B	IPE360	Box 80×80×10
4	HE120B	HE160B	IPE360	Box 100×100×10
3	HE140B	HE220B	IPE360	Box 120×120×10
2	HE160B	HE280B	IPE360	Box 120×120×10
1	HE160B	HE340B	IPE360	Box 120×120×10
10-story model				
10	HE100B	HE120B	IPE360	Box 80×80×10
9	HE120B	HE180B	IPE360	Box 100×100×10
8	HE140B	HE220B	IPE360	Box 120×120×10
7	HE160B	HE280B	IPE360	Box 120×120×10
6	HE180B	HE340B	IPE360	Box 140×140×10
5	HE200B	HE500B	IPE360	Box 140×140×10
4	HE220B	HE650B	IPE360	Box 140×140×10
3	HE220B	HE900B	IPE360	Box 160×160×10
2	HE240B	HE1000B	IPE360	Box 160×160×10
1	HE320B	HE1000B	IPE360	Box 180×180×10

(AISC 360-10 2010). This is a simplified representation of these connections because they may provide some small rotational restraints due to the typical connection details; thus, several experimental and theoretical works have been carried out to assess the actual behavior of these simple or shear connections under gravity or earthquake loads (Nader and Astaneh-Asl 1996, Astaneh-Asl *et al*. 2002, Gong 2009, Pirmoz *et al*. 2009). However, static lateral load-displacement analyses have demonstrated that the provision of bracing elements effectively reduces the sensitivity of the global structural response to differences in connection behavior (Lui and Chen 1988). Furthermore, a numerical static analysis of a 3-story building under only gravity loads has shown that the assumption of pinned joints for modeling of simple beam-to-column connections (which are known to present small bending resistance) is safe regarding the resistance or stability of the structure and frame displacements (Braham and Jaspart 2004).

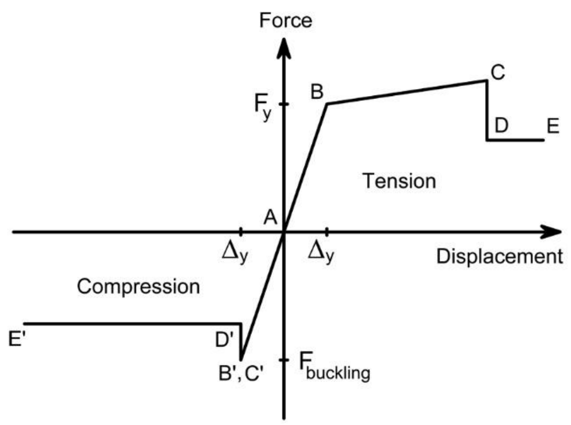
The present study further investigates the effects of the small rotational stiffness of two common types of simple beam-to-column connections with bolted angles, i.e., the Double Web angle Connection (DWC) and Unstiffened Seat angle Connection (USC), on the behavior and seismic response of steel braced non-moment resisting frames. Numerical models that include both nonlinear behavior of connections and inelastic behavior of frame elements are developed for the purpose of structural analysis. Connection flexibility is modeled by a nonlinear rotational spring in the structural models and both global and local structural responses are compared with those obtained from ideally pinned alternatives. The objective of this paper is not to study the connection behavior in details, but it tries to determine whether or not the ideally pinned representation of these connections can lead to the unconservative results in seismic demand analysis of steel braced non-moment resisting frames.

2. Description of the structural models and assumptions

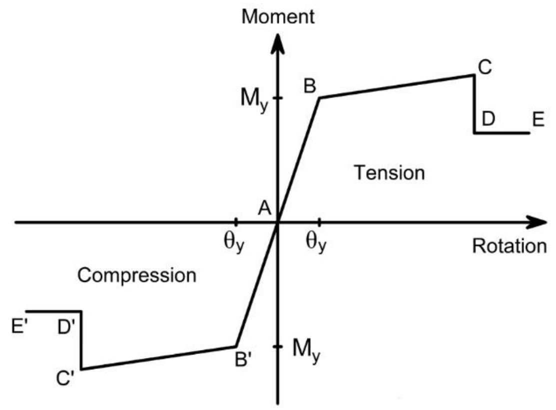
In this study, several ordinary concentrically braced frames, which are composed of three, five, and ten stories, are modeled as 2-D building frames (Fig. 2). The lateral resistance of each frame is only provided by the bracing system. Material properties of mild steel used in the frames are: F_y=2400 kgf/cm^2, F_u=3700 kgf/cm^2 and E=2×10^6 kgf/cm^2; where F_y, F_u, and E are the yield stress, tensile strength and modulus of elasticity, respectively.

Structural members of the frames are designed to carry gravity and lateral loads according to the specifications of AISC (2010). In this design procedure, three types of prismatic frame sections are assumed: beams with IPE cross-sections, columns with HEB (IPB) cross-sections and braces with box cross-sections. Final designed cross sections for different models are presented in Table 1.

After this design process, plastic hinges that are assumed to be lumped at the middle and ends of beam elements, at the middle of bracing elements, and at the ends of column elements are assigned to incorporate plastic behavior. The definition of plastic hinges in structural members is taken according to FEMA-356 (2000). Hinge properties are defined as force-displacement or moment-rotation curves. These properties for axial force (P) hinges and axial force-bending moment (P-M) hinges can be computed on the basis of the element material and cross-section properties according to FEMA-356 criteria. Fig. 3 shows the schematic representation of different plastic hinge status considered in this paper in which the region between points A and B represents the elastic range; from B to C is the plastic range that is followed by a sudden drop in strength (C to D) and then failure at point E. Moreover, it should be noted that section capacity of bracing elements in tension and compression can be different because of buckling under axial compression and this phenomenon is considered in the definition of axial hinges for bracing



(a)



(b)

Fig. 3 Plastic hinge definition: (a) Force vs. displacement curve, (b) Moment vs. rotation curve

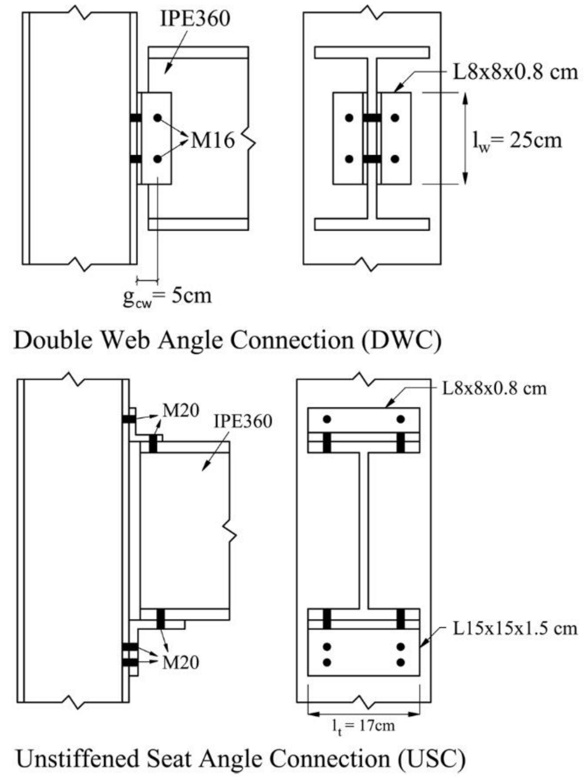


Fig. 4 Used connections in this study

elements.

In addition to the ideally pinned representation of beam-to-column connections, a more accurate semi-rigid model of these connections is established and then applied to the structural models. Furthermore, the fully rigid assumption is used as another extreme case to investigate the sensitivity of the structural responses to the variations in the connection behavior. However, the simple supports of the considered frames are assumed to be pinned in all structural models. In order to estimate seismic demands, nonlinear pushover and nonlinear dynamic analyses are performed on the models.

3. Modeling of connections

The two types of shear (simple) beam-to-column connections used in the present study are double web angle connections (DWCs) and unstiffened seat angle connections (USCs) without web angles. These connections are widely used in practical cases for steel non-moment resisting frames. Designed beam-to-column connections with ASTM A325 bolts for the models considered here are shown in Fig. 4. It should be noted that these types of connections in braced non-moment frames are usually designed to carry only gravity loads in the form of vertical shears and consequently, the associated design parameters depend only on the length of span and load distribution. Therefore, details of these connections in the different models considered here and in the different stories of each model are the same.

A USC is made with a seat angle and a top angle, as illustrated in Fig. 4. While both angles are designed to resist the vertical shear load in a DWC, only the bottom (seat) angle is designed to carry the entire end reaction of the supported beam in a USC according to the recommendations of the Steel Construction Manual (AISC, 2011) and the top (stabilizing) angle is selected as a minimum one to prevent the beam from rolling over and to assist in a safe erection. However, the effects of both top and seat angles are considered in the analysis models.

The M-θ relationship of a connection is typically obtained via a curve fitting to the experimental data using simple expressions. Numerous tests on connections have been performed in the past, resulting in a rather large body of M-θ data. Thus, various moment-rotation relationships have been derived using the available data for modeling semi-rigid connections. These relationships vary from linear to exponential forms, though they are intrinsically nonlinear. In the present study, the nonlinear M-θ properties of the connections are adopted from the three parameter power model of Kishi and Chen (Chen and Kishi 1989, Kishi and Chen 1990). This model is suitable for practical purposes since its key parameters are physically meaningful and can be determined analytically with acceptable accuracy (Liew *et al*. 1993). Although most M-θ relationships have been devoted to moment connections in the form of fully restrained or partially restrained (semi-rigid) ones, the Kishi-Chen power model would be applicable to simple (shear) connections of non-moment

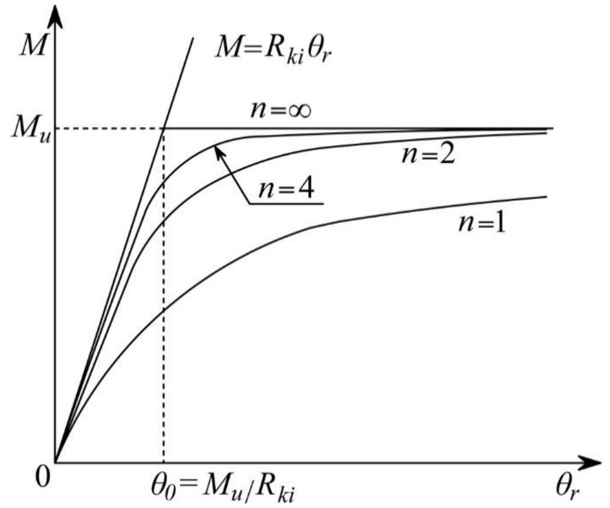


Fig. 5 Three-parameter power model according to Eq. (2)

resisting frames with tiny rotational restraints.

The generalized equation of the applied model has the following form

$$m = \frac{\theta}{(1+\theta^n)^{1/n}} \quad \text{for } \theta > 0 \text{ and } m > 0 \tag{1}$$

The non-dimensional moment and rotation parameters in this equation are defined as $m = M/M_u$ and $\theta = \theta_r/ \theta_0$, where

M = moment on the connection
M_u = ultimate moment capacity of the connection
θ_r = relative rotation between beam and column
θ_0 = M_u/R_{ki} , the reference plastic rotation
R_{ki} = initial connection stiffness
n = shape parameter

Eq. (1) with substituted values of m and θ has the form

$$M = \frac{R_{ki}\theta_r}{[1+(\theta_r/\theta_0)^n]^{1/n}} \tag{2}$$

The connection tangent stiffness R_{kt} at an arbitrary rotation $|\theta_r|$ can be evaluated by differentiating Eq. (2) with respect to $|\theta_r|$. When the connection is loaded, the connection tangent stiffness is

$$R_{kt} = \frac{\mathrm{d}M}{\mathrm{d}|\theta_r|}\Big|_{|\theta_r|} = \frac{M_u}{\theta_0(1+\theta^n)^{1+1/n}} \tag{3}$$

and when the connection is unloaded, we have

$$R_{kt} = \frac{\mathrm{d}M}{\mathrm{d}|\theta_r|}\Big|_{|\theta_r=0|} = \frac{M_u}{\theta_0} = R_{ki} \tag{4}$$

Variation of M as a function of θ_r is shown in Fig. 5 for various values of the shape function n. If n is infinity, the model becomes a bilinear curve that has initial connection stiffness R_{ki} and ultimate moment capacity M_u. Kishi *et al.* (1991a, 1991b) have carried out statistical analyses of several test data to obtain equations for the shape parameter of various connection types. These equations for double web angle connections and unstiffened seat angle connections are given, respectively, as

$$n = 1.322\log\theta_0 + 3.952 > 0.6 \tag{5}$$

$$n = 2.003\log\theta_0 + 6.070 > 0.4 \tag{6}$$

The initial stiffness, ultimate moment capacity and shape parameters of the connections considered here are listed in Table 2. After obtaining the moment-rotation curve for a beam-to-column connection, the rotation capacity of the connection should be recognized. The connection classification system proposed by Bjorhovde *et al.* (1990) allows the rotation capacity of a connection to be determined once the moment-rotation curve is estimated either from tests or by analytical approaches. In the present study, this connection classification system is utilized to limit the rotation capacity of the connections as depicted in Fig. 6.

Table 2 Moment-rotation parameters of connections

Type of connection	M_u (tonf.m)	R_{ki} (tonf.m/rad)	n
DWC	2.996	1080.576	0.600
USC	3.884	2279.732	0.524

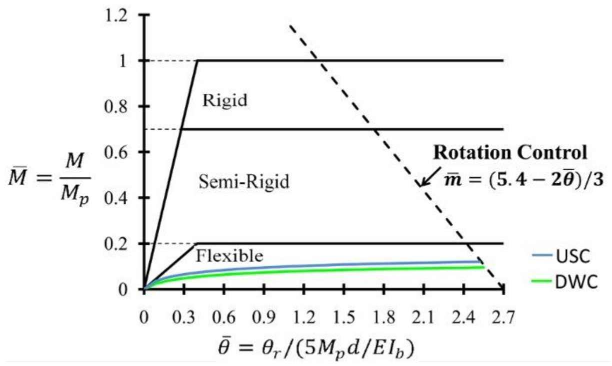


Fig. 6 Classification of connections due to Bjorhovde *et al.*'s system

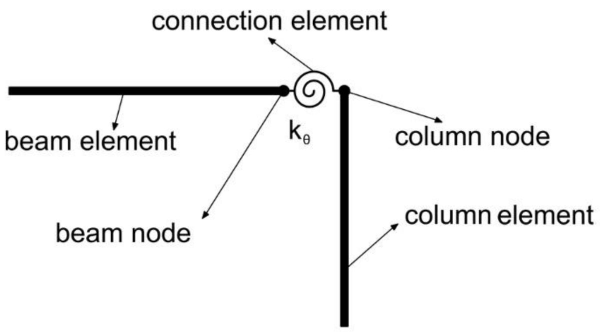
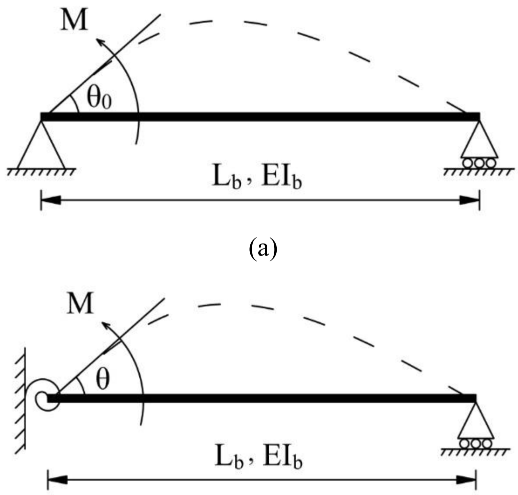


Fig. 7 Modeling of a flexible beam-to-column connection

Table 3 Fundamental period of vibration of models (s)

Model	Ideally pinned	DWC	USC	Fully rigid
3-story	0.3104	0.3102	0.3102	0.3027
5-story	0.4953	04951	04951	0.4714
10-story	1.0052	1.0048	1.0048	0.9063



(a)

(b)

Fig. 8 Definition of end fixity factor

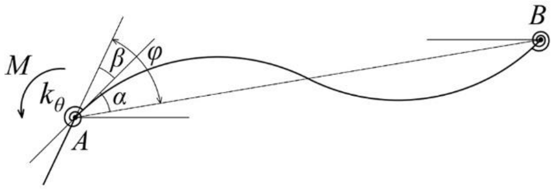


Fig. 9 Rotation of a beam with semi-rigid connection

For the purpose of structural analyses, the nonlinear moment-rotation behavior of the connections is modeled as a multi-linear plastic spring with a kinematic hardening in SAP2000 software package, by Computers and Structures, Inc. (2009). Assigning the nonlinear spring for *M-θ* behavior of the beam-to-column connection is shown in Fig. 7.

Fundamental periods of vibration of the structural models considered here with different behavior of connections are computed and listed in Table 3. It is obvious that a stiffer connection results in a more lateral stiffness, which in turn results in a lower fundamental period. However, differences in the obtained results between the ideally pinned model and more accurate models of DWC and USC are negligible, while they are not slight when comparing these 3 groups with the fully rigid model.

4. End fixity factors of connections

End fixity factor is an effective parameter to quantify the flexibility of a connection. For an ideally pinned connection, the value of end fixity factor is zero and for a fully rigid connection, this factor is unity. For a semi-rigid connection, this value is between zero and unity. End fixity factor (r) is defined according to the following equation,

$$r = 1 - \frac{\theta}{\theta_0} \tag{6}$$

Table 4 Maximum and minimum end fixity factors in the presented models

Model	DWC		USC	
	min	max	min	max
3-story	3.9%	4.1%	4.6%	4.9%
5-story	3.9%	4.2%	4.7%	5.0%
10-story	3.6%	4.5%	4.4%	5.4%

where θ_0 is the rotation of a beam element with two pinned ends under applied moment M (see Fig. 8(a)), and θ is the rotation of the same beam element with an actual semi-rigid end (see Fig. 8(b)). Thus, with k_θ the effective stiffness of the connection spring and EI_b/L_b the flexural stiffness of the beam, the end fixity factor for a flexural member is given as

$$\theta_0 = \frac{ML_b}{3EI_b} \quad , \quad \theta = \frac{ML_b}{3EI_b + k_\theta L_b}$$
$$r = 1 - \frac{\theta}{\theta_0} = \frac{1}{(1 + 3EI_b / k_\theta L_b)} \tag{7}$$

There exists another approach to define the end fixity factor. According to Fig. 9, end fixity factor is defined as the ratio of the rotation of the beam end with semi-rigid connection (α) to the rotation of both beam end and connection (i.e., $\varphi=\alpha+\beta$)

$$\alpha = \frac{ML_b}{3EI_b} \quad , \quad \beta = \frac{M}{k_\theta}$$
$$r = \frac{\alpha}{\varphi} = \frac{\alpha}{\alpha + \beta} = \frac{1}{(1 + 3EI_b / k_\theta L_b)} \tag{8}$$

Analytical results of end fixity factors for the presented models under gravity loads are summarized in Table 4. As expected, numerical results show that DWCs and USCs, which are typical beam-to-column connections in simple framing construction, have rather small fixity factors. However, in frame models with the same length of spans and number of stories, end fixity factors of USCs are greater than DWCs.

According to this table, an increase in height of the frames (or the number of stories) results in a slight increase in ranges of end fixity factors. For example, in the frame with USCs, the end fixity factors lie between 4.6% and 4.9% for the 3-story frame model and between 4.4% and 5.4% for the 10-story frame model. Although details of each type of these connections in all models and stories are the same (see Fig. 4), effective stiffness of connections can be different because the effective stiffness of a connection, which is related to the rotation of the connection, can be changed due to the nonlinear *M-θ* behavior of the connection.

5. Nonlinear pushover analysis

In the nonlinear static analysis reported herein, the structures are first subjected to full gravity loads then

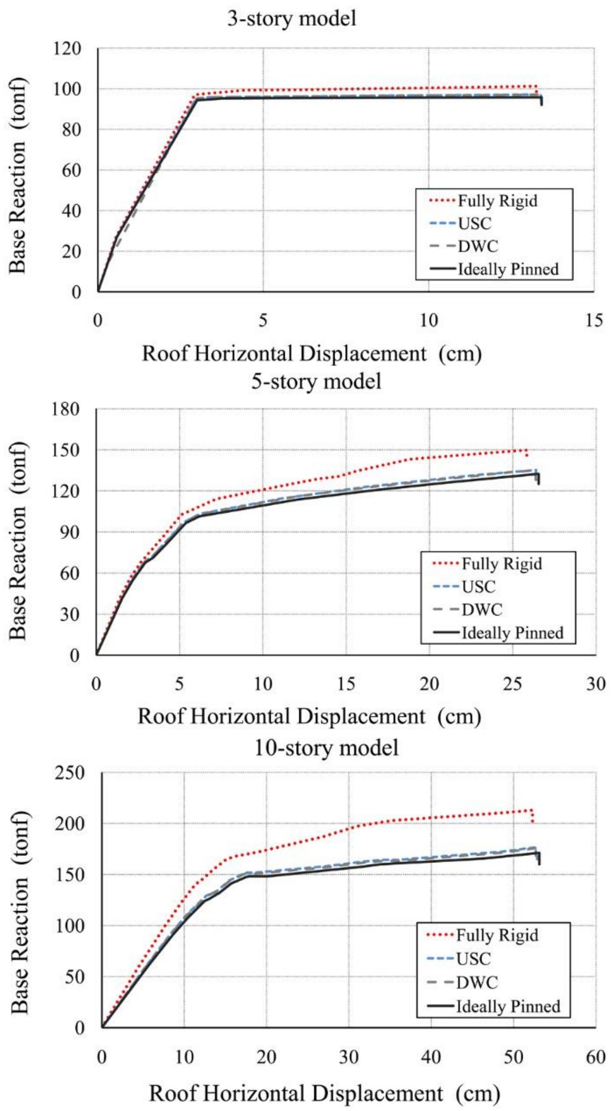


Fig. 10 Capacity curves of the models

Table 5 Lateral load capacity of different models due to pushover analyses (tonf)

Model	Ideally pinned	DWC	USC	Fully rigid
3-story	95.76	97.09	97.18	101.21
5-story	132.42	135.09	135.29	149.77
10-story	171.11	175.95	176.65	212.95

followed by the lateral loads. In this paper, a triangular lateral load pattern, which is representative of the forces associated with the first vibration mode in low-and mid-rise regular buildings, is applied for static pushover analysis. In Fig. 10, the base shear vs. roof displacement curves of the frame models due to nonlinear pushover analyses are shown. In general, some slight differences in the behavior of the frames between the ideal and real models can be seen, especially in the inelastic regimes. According to Fig. 10, when the connection moment capacity increases, the initial stiffness and ultimate lateral load capacity of the frames increase as well. In other words, considering actual behavior of beam-to-column connections in braced steel

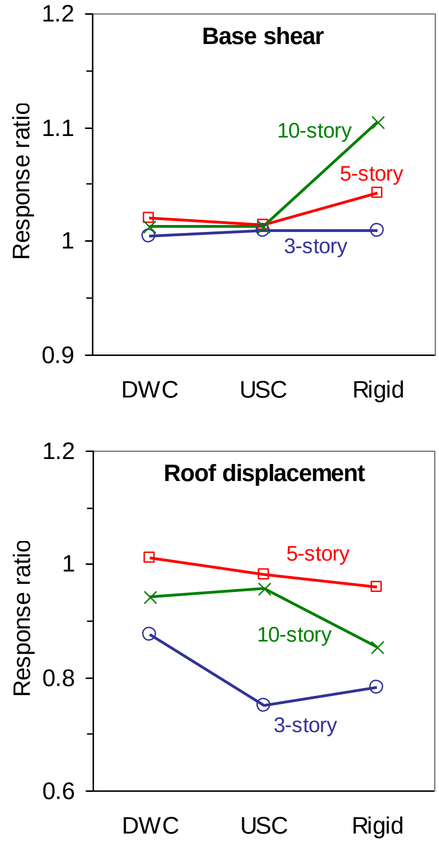


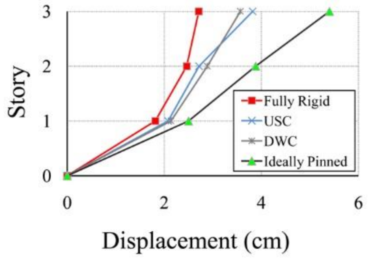
Fig. 11 Base shear and roof displacement response ratios for different frame models

Table 6 Characteristics of selected ground motion records for time history analyses

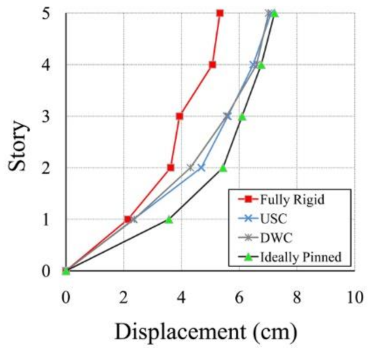
No.	Earthquake	Date	M_w	PGA (g)	Site conditions
1	San Fernando	02/09/1971	6.6	0.210	USGS (C)
2	Whittier Narrows	10/01/1987	6.0	0.221	USGS (C)
3	Northridge	01/17/1994	6.7	0.356	USGS (C)
4	Kocaeli	08/17/1999	7.4	0.358	USGS (C)
5	Loma Prieta	10/18/1989	6.9	0.367	USGS (C)
6	Parkfield	06/28/1966	6.1	0.367	USGS (C)
7	Westmoreland	04/26/1981	5.8	0.368	USGS (C)
8	Imperial Valley	10/15/1979	6.5	0.425	USGS (C)
9	Erzikan	03/13/1992	6.9	0.496	USGS (C)
10	Cape Mendocino	04/25/1992	7.1	0.590	USGS (C)

frames instead of assuming ideally pinned behavior slightly increases the ultimate lateral load capacity.

It is a well-established fact that instability due to buckling of bracing members often limits inelastic response in braced frames. Ultimate lateral load capacity or maximum base shear capacity of the frames is obtained from the peak points of pushover curves shown in Fig. 10. These values are summarized in Table 5. As the results show, the differences in lateral load capacity between the ideally pinned connection cases and the more accurate cases of flexible connections are not noticeable (maximum



(a)



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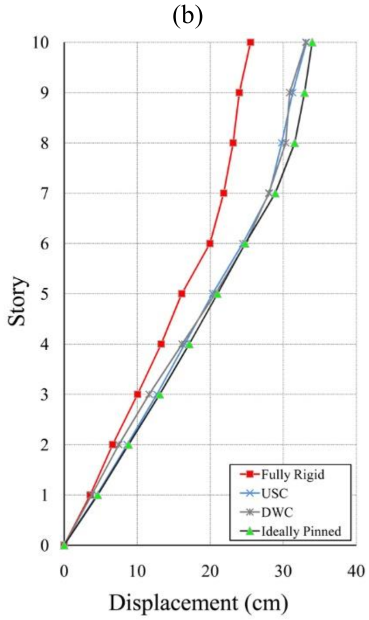

(c)

Fig. 12 Lateral displacement envelopes of (a): 3, (b): 5, and (c): 10-story frame model under Kocaeli earthquake

increase up to 3% in the 10-story frame model with USCs). There is no doubt that the rigid connection assumption results in a higher increase in the capacity of the frame models in comparison to simple connections. This increase is found to be up to 24% in the 10-story frame model.

6. Nonlinear time history analysis

6.1 Selected ground motions

Characteristics of the selected earthquake records for time history analyses are shown in Table 6. These ground motions have different frequency contents and intensities. The peak ground accelerations (PGA) for the selected earthquake records are between 0.2 g and 0.6 g.

6.2 Global responses

Horizontal roof displacements and total base shear demands of the braced frame models are computed in this section based on the nonlinear time history analyses. These global response parameters obtained for the cases of ideally pinned connection, flexible connections and fully rigid connection are compared together. In order to better understand the numerical results, a dimensionless parameter representing the ratio of the peak response of a model with flexible DWCs or USCs or rigid connections to the peak response of the corresponding model with ideally pinned connections is defined and computed for the roof displacements and base shears. This response ratio greater than, equal to, or less than unity means that the peak response of the model with DWCs/USCs/rigid connections is larger than, equal to, or smaller than that is obtained from the corresponding model with ideally pinned connections. Median values of the response ratios obtained from the nonlinear time history analyses of selected earthquake records are shown in Fig. 11. Since each of the selected ground motions has a different frequency content and different intensity levels, the response ratio of a particular model under different ground motions can vary considerably. However, Fig. 11 shows that the median values of response ratios have similar trends for the different models.

From this figure, it can be observed that the base shear response ratios for 3, 5, and 10-story models with DWSs and USCs are between 1.00 and 1.02, indicating a negligible increase of base shear demand with more accurate modeling of simple beam-to-column connections of steel braced non-moment frames. That means small rotational restraints of these types of simple connections do not considerably affect the base shear demand while bracing members with relatively high lateral stiffness and strength have been employed. Increasing the rotational stiffness of the beam-to-column connections to the upper bound, i.e., fully rigid condition, the base shear response ratio by keeping it between 1.01 and 1.11 for the models with different stories.

From Fig. 11, it can also be observed that the frame models including flexible DWCs/USCs or rigid connections often have smaller lateral roof displacements when compared with the corresponding models having ideally pinned connections. The roof displacement response ratios for the models with different stories having flexible DWSs, USCs and rigid connections are between 1.01 and 0.88, between 0.98 and 0.75, and between 0.96 and 0.85, respectively. Therefore, our results show that small rotational restraints inherent to the simple connections influence displacement demands more than base shear demands especially in the 3-story frame model. Comparison of the results for inter-story displacements is somehow similar to the roof displacements. For instance, the maximum lateral inter-story displacements of frames

Table 7 Response ratios of beam elements in the frame models with DWCs and USCs subjected to San Fernando earthquake

Model	Story	$f_b(\max)_{DWC}$ / $f_b(\max)_{pinned}$			$f_b(\max)_{USC}$ / $f_b(\max)_{pinned}$		
		Left beam	Middle beam	Right beam	Left beam	Middle beam	Right beam
3-story	3	0.96	0.94	0.97	0.95	0.92	0.96
	2	0.96	0.94	0.96	0.94	0.92	0.95
	1	0.97	0.97	0.96	0.96	0.95	0.96
5-story	5	0.98	0.95	0.97	0.97	0.93	0.95
	4	0.97	0.94	0.96	0.96	0.92	0.94
	3	0.96	0.93	0.96	0.95	0.90	0.95
	2	0.96	0.93	0.96	0.94	0.91	0.94
	1	0.97	0.96	0.97	0.96	0.95	0.96
10-story	10	0.99	0.97	1.00	0.99	0.95	0.99
	9	1.00	0.96	1.00	0.99	0.97	0.99
	8	1.00	0.94	0.99	0.99	0.94	0.99
	7	0.99	0.92	0.99	0.99	0.97	0.99
	6	0.98	0.92	0.98	0.98	0.94	0.99
	5	0.98	0.93	0.98	0.97	0.90	0.97
	4	0.97	0.93	0.97	0.96	0.91	0.96
	3	0.97	0.93	0.97	0.96	0.91	0.96
	2	0.96	0.96	0.96	0.95	0.91	0.95
	1	0.95	0.93	0.95	0.94	0.92	0.94

according to nonlinear dynamic analyses under Kocaeli record are plotted in Fig. 12.

6.3 Member demands

6.3.1 Beam elements

It is a well-established fact that the effective factor in the design of a beam element in the steel braced frames is the maximum stress due to the bending moment that usually occurs in the middle of the beam. It should be noted that bending moments of beams in frames with ideally pinned connections is independent of lateral loads. However, in the case of flexible beam-to-column connections, beam elements take part in lateral resistance of frames.

Generally, in the braced simple steel frames with equal spans, beam sections are the same (see Table 1). However, the seismic demands of these similar elements in models with different stories and also in different stories of a model are not the same under a particular ground motion excitation. Thus, a large number of outputs can be obtained for such a local (member) response under several strong ground motions. Similar to the previous section, a response ratio for each beam element is defined as the ratio of the maximum stress in the particular beam of a model with flexible connections (DWCs or USCs) to that obtained from the corresponding model with ideally pinned connections. The obtained numerical results are summarized in Table 7 for the San Fernando earthquake. As expected, the results show that maximum demands in beam elements slightly decreases by considering small rotational restraint of beam-to-column connections (in both cases of DWC and USC).

Table 8 Response ratios of column elements in the frame models with DWCs and USCs subjected to San Fernando earthquake

Model	Story	Stress ratio (DWC/Pinned)				Stress ratio (USC/Pinned)			
		Outer left column	Inner left column	Inner right column	Outer right column	Outer left column	Inner left column	Inner right column	Outer right column
3-story	3	1.42	1.18	1.20	1.30	1.63	1.14	1.18	1.50
	2	1.24	1.17	1.19	1.28	1.23	1.10	1.15	1.32
	1	1.18	1.16	1.24	1.37	1.13	1.10	1.19	1.33
5-story	5	1.40	1.03	1.02	1.45	1.39	1.13	1.11	1.52
	4	1.34	1.02	0.90	1.30	1.35	1.03	1.02	1.27
	3	1.13	1.01	0.89	1.11	1.14	1.00	0.88	1.15
	2	1.28	1.06	0.95	1.25	1.29	1.06	0.95	1.26
	1	1.32	1.05	0.96	1.40	1.32	1.05	0.96	1.43
10-story	10	1.37	0.93	0.87	1.46	1.39	1.14	0.97	1.53
	9	1.18	0.93	0.87	1.14	1.23	1.04	0.97	1.20
	8	1.00	0.95	0.97	0.99	1.25	0.95	0.97	1.26
	7	0.90	0.98	1.04	0.90	1.27	0.98	0.99	1.32
	6	0.93	0.93	0.93	0.93	1.17	0.93	0.93	1.20
	5	1.10	0.96	0.88	1.10	1.19	0.97	0.90	1.18
	4	1.09	0.95	0.89	1.09	1.12	0.95	0.90	1.13
	3	1.06	0.94	0.90	1.06	1.09	0.95	0.92	1.09
	2	1.06	0.95	0.93	1.06	1.08	0.96	0.94	1.09
	1	1.05	0.96	0.96	1.05	1.07	0.97	0.96	1.06

The response ratios for all beams of different models are between 1.00 and 0.90. Similar observations are also made for the other selected earthquake records. Thus, considering ideally pinned behavior for simple beam-to-column connections of steel braced non-moment resisting frames is a conservative assumption in determining seismic force demands in beam elements.

6.3.2 Column elements

The effective factors in the design of a column element in steel braced frames are bending moment and axial force. Again, a response ratio for each column element is defined as the ratio of the maximum normal stress due to bending moments and axial forces in a model with flexible connections (DWCs or USCs) to that obtained from the corresponding model with ideally pinned connections. The numerical results are presented in Table 8 and depicted in Fig. 13 for the 3, 5, and 10-story models under the San Fernando earthquake. Most of these values, especially in the outer columns, are greater than one, which indicates that maximum demands in column elements increase due to considering partial rigidity of the simple beam-to-column connections. In other words, by considering small rotational stiffness of the flexible connections, some portion of the applied moment transfers to the columns and increases the seismic demand of the column. Therefore, the ideally pinned representation of simple beam-to-column connections in steel braced non-moment resisting frames is not always a conservative assumption for the design purpose of columns.

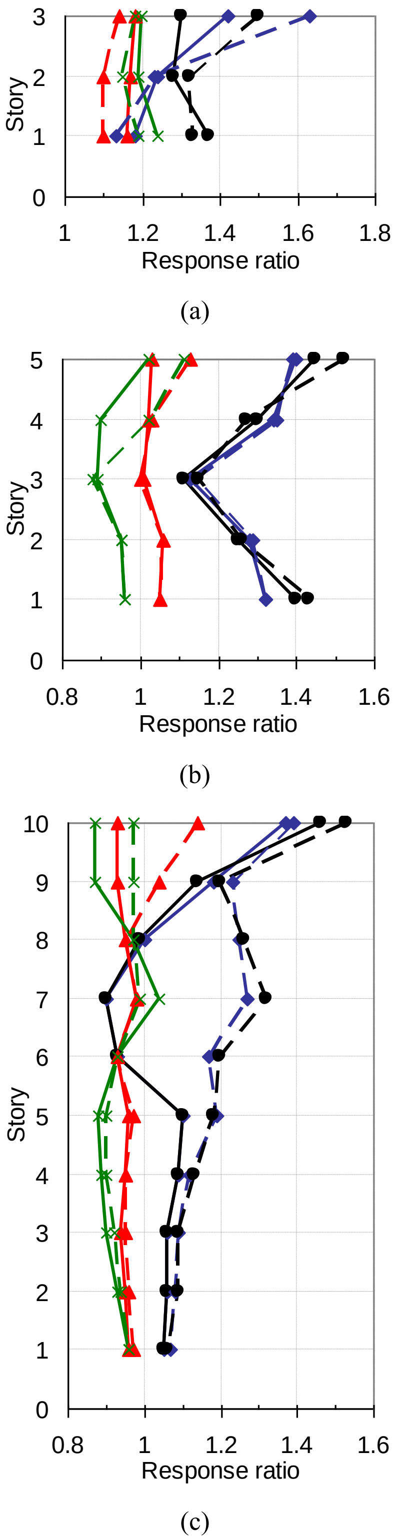

Fig. 13 Response ratios of outer left columns (blue lines), inner left columns (red lines), inner right columns (green lines), and outer right columns (black lines), in the (a): 3, (b): 5, and (c): 10-story frame model with DWCs (solid lines) and USCs (dashed lines) subjected to San Fernando earthquake

As it can be seen from Table 8, differences between results of flexible connections and ideally pinned connections for outer columns, especially in the upper stories, are remarkable. This is due to the fact that the inner columns are connected to beams from the two sides with an almost equal amount of transferred moments and as a result, the bending moments of these columns are balanced. On the contrary, outer columns are connected to beams from one side only and the bending moments of these columns are not balanced. In addition, in the columns of upper stories, transferred bending moments of beams are rather large in comparison to axial forces. This can be viewed as the main finding of the present work and should be considered in a design procedure when applicable. It is recommended that the outer columns in upper stories of steel braced non-moment resisting frames to be designed with rather larger safety factors when the ideally pinned representation of beam-to-column connections is used for the purpose of structural analysis.

Table 9 Response ratios of bracing elements in the frame models with DWCs and USCs subjected to San Fernando earthquake

Model	Story	$f_a(\max)_{DWC}$ / $f_a(\max)_{pinned}$		$f_a(\max)_{USC}$ / $f_a(\max)_{pinned}$	
		"/" Shaped brace	"\" Shaped brace	"/" Shaped brace	"\" Shaped brace
3-story	3	1.00	1.00	1.00	0.99
	2	1.00	1.00	1.00	1.00
	1	1.00	1.01	1.00	1.01
5-story	5	1.00	0.99	1.00	0.99
	4	0.98	1.00	0.99	1.00
	3	0.98	0.98	0.98	0.99
	2	1.02	1.01	1.01	0.98
	1	1.00	1.04	1.00	1.04
10-story	10	0.99	1.00	0.99	1.00
	9	0.99	1.00	1.00	0.98
	8	0.96	0.95	0.97	0.94
	7	0.97	0.98	0.99	1.01
	6	0.93	0.94	0.94	0.95
	5	0.96	0.94	0.95	0.93
	4	0.94	0.98	0.96	0.97
	3	0.93	0.99	0.94	0.99
	2	0.93	0.99	0.94	0.99
	1	0.94	0.99	0.94	1.00

6.3.3 Bracing elements

The effective factor in the design of a bracing element is the maximum axial force. Similar to the previous sections, a response ratio for each brace element is defined as the ratio of its maximum stress due to axial force in a model with flexible beam-to-column connections (DWCs or USCs) to that obtained from the corresponding model with ideally pinned connections. The obtained results are presented in Table 9 for the 3, 5, and 10-story models under the San Fernando earthquake. It can be seen from this table that considering partial rigidity of connections (in both cases of DWC and USC) does not significantly affect the maximum demands in bracing elements.

7. Conclusions

Because of the very small rotational stiffness of simple (or shear) beam-to-column connections of non-moment resisting frames, they are usually assumed to be pinned for the purpose of structural analysis. This study evaluates the effects of modeling of these connections behavior on the seismic responses of steel braced non-moment resisting frames using nonlinear pushover and nonlinear time history analyses. For this purpose, several frame models having two common types of simple steel beam-to-column connections (i.e., the double web angle connection and unstiffened seat angle connection) were developed considering nonlinear behavior in the connections and structural members.

According to our numerical results, end fixity factors of the connections range between 3.6% and 5.4%, indicating that DWCs and USCs have rather small rotational stiffness. However, partial fixity of the connections in the frame models with USCs is slightly greater than those with DWCs, indicating greater rotational stiffness of the unstiffened seat angle connection in comparison to the double web angle connection.

Results of the applied nonlinear pushover analyses show that when the rotational stiffness of the connection increases, the ultimate lateral load capacity of the frame also increases; however, the differences between the ideally pinned connection cases and the more accurate cases of flexible connections are not noticeable. Moreover, on the basis of the results of the applied time history analyses, it is evident that the flexible connections can affect the dynamic behavior of steel frames. In the presence of bracing members with relatively high lateral stiffness, negligible increases of base shear demands are seen due to the more accurate modeling of simple beam-to-column connections of steel braced non-moment frames, while displacement demands are reduced to some extent.

It is however found that details of connection modeling can strongly affect the seismic demands of some structural members. The assumption of ideally pinned connections leads to somewhat overestimated force demands in beams and braces, which can be ignored; but for columns, these demands are underestimated, especially in outer columns of upper stories. Thus, the ideally pinned assumption for beam-to-column connections of steel braced non-moment resisting frames is not always a conservative assumption for the purpose of structural design. It is recommended to pay attention to these effects of joint modeling of simple beam-to-column connections in the structural analysis.

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