The 2015 World Congress on
Advances in Structural Engineering and Mechanics (ASEM15)
Incheon, Korea, August 25-29, 2015

Seismic Evaluation of a Steel Moment Frame with Cover Plate Connection Considering Flexibility by Component Method

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Abstract

Cover plate (CP) moment resisting connections have been recognized as a very practical steel moment connections for steel structures due to their seismic performance. Since almost every moment connections in moment resisting frames are considered as being fully-rigid and the certain behavior of this connection is neglected, this study aims at investigating the seismic performance of a steel moment frame considering influence of flexibility of its connections. This paper deals with a spring-stiffness model called "the component method" to predicting the real behavior of steel moment connections (specially their moment-rotation curves).

The behavior of the frame with cover plate moment connections in both cases of including as well as excluding the flexibility of the connections is compared using nonlinear static pushover, and incremental dynamic analyses. In all models, P-Delta effects along with material and geometrical nonlinearities were included in the analyses. Results revealed that taking the realistic behavior of beam column connection into account results in almost different outcome.

Keywords. Cover plate, Beam to box column connection, Component method, Flexibility.

1. INTRODUCTION

The behavior of steel moment resisting frames is greatly affected by the properties of the beam to column connections, specially in seismic zones and understanding the strength and rigidity of connection regions is necessary for the efficient design of steel building systems. Before the Northridge earthquake, moment frames with full penetration welds acquired reputation as an ideal seismic resistant systems. However, the Northridge earthquake has revealed brittle fracture near the beam to column flange groove welds in this connection. The behavior of steel connections continues to be an issue of interest in

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the area of steel structures. The nonlinear behavior of steel frames joints is experimentally proved even in the case of welded connections. A practical method for later concept is considering the real behavior of the connections in moment resisting frames. Cover plate (CP) moment resisting connections is one of those connections which supposed to reveal rigid behavior in beam to box column connections. A considerable number of literatures have been published on the seismic behavior of beam to column connections considering the real behavior of the joints.

Ang and Morris (1984) studied the performance of unbraced steel frames with semi rigid connections considering the flexibility of connections in the models. Azizinamini and Radziminski (1989) investigated on static and cyclic performance of semi rigid steel beam to column joints. Results revealed that the behavior of the moment-rotation of the joint depend on the length and the thickness of the angles. Nader and Astaneh-Asl (1991) studied the behavior of a structure with rigid, semi rigid, and pinned connections under different seismic records. Their results had shown that frames with semi rigid connections had a better performance rather than rigid joints. Lui and Lopez (1997) investigated on dynamic analysis and response of semi rigid frames. Mazroi *et al*. (1999) studied the behavior of the beam to box column connections with cover plates. Awkar and Lui (1999) investigated the behavior of a five and eight story frame with rigid, semi-rigid and pinned connections. Results have shown that the connection flexibility tends to reduce frame stiffness and hence increase vibration periods, specially in lower modes. While it increases damping and when properly designed, steel frames with semi rigid connection are very efficient against resisting forces generated from ground motion because of their ability to dissipate seismic energy through large inelastic deformation and damping. Natural period and base shear in the frames with semi rigid connection can be controlled. Da Silva *et al*. (2001) investigated on steel joints at elevated temperatures.

McMullin and Astaneh-Asl (2003) studied on seismic behavior of semi rigid connections in column-Tree moment resisting frame. Urbonas and Daniūnas (2005) studied on the behavior of semi-rigid steel beam-to-beam joints under bending and axial forces. The results showed that tensile and compression forces respectively decreased and increased the rotational stiffness of the joints. Ghassemieh *et al.* (2013) studied on flexibility of end-plate steel joints using finite element method when the connection is subjected to seismic excitation. Ghassemieh *et al*. (2014) investigated on seismic response of end-plate moment connections adopting the flexibility of the joint.

The purpose of this research is to investigate on seismic performance of a seven story frame with CP connection by considering the actual behavior of connection. First, considered sample was modeled using component method for predicting the flexibility of the connection, then for the evaluation of obtained results, the connection modeling by component method was verified by the connections designed by Saneei Nia *et al* (2013). Then pushover analyses and incremental dynamic analyses (IDA) of 7-story frame with different behavior for connections are conducted in order to compare the behavior of the frame with realistic rigidity of the connection with the rigidity normally used in the idealistic behavior.

2. COMPONENT METHOD

The originality of this 'Component Method' (Eurocode 3, 2000), contains a simplified mechanical model composed of springs and rigid links, whereby the connection is simulated by an appropriate choice of rigid and flexible components is illustrated in Fig. 1; These components represent a specific part of a connection that, reliant on the type of any loading, make an identified contribution to one or more of its structural properties. A beam to box column joint using the cover plate (CP) connection can be divided into three major zones (i.e. tension, shear and compression zones). For each component, the initial stiffness and ultimate capacity is determined and assembled to form a spring model which is adopted to simulate the rotational behavior of the whole joint (Faella *et al*. 2000). In current method, (M-θ) curve is obtained.

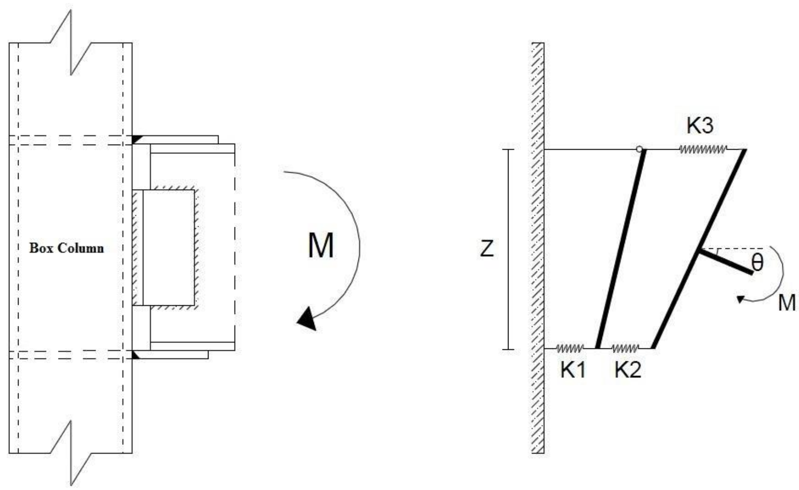


Fig. 1 Mechanical model for cover plate beam to box column connection

3. MICRO BEHAVIOR OF CP CONNETION

In this study, a steel moment frame model, as proposed by Saneei Nia *et al.* (2013), including seven-story structural frame was considered as benchmark example in order to investigate the effect of flexibility of the connection. Floor plan of the seven-story model, along with elevation of the frame is depicted in Fig. 2

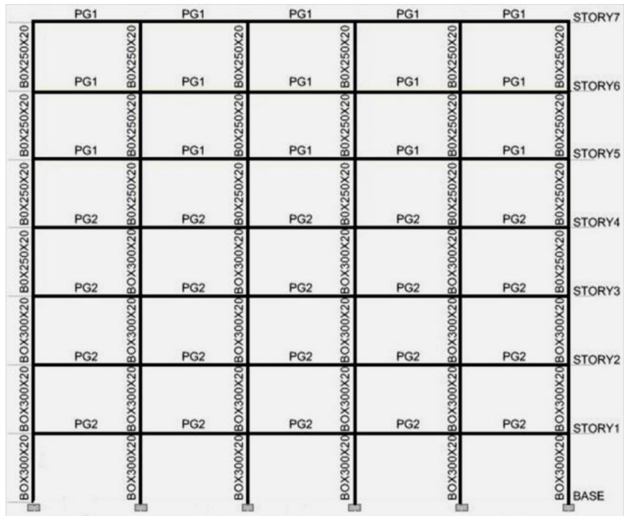


Fig. 2 Plan and elevation view of selected frame (Saneei Nia *et al*, 2013)

Details of the designed beams and columns are provided in Table 1. In addition, Fig. 3 and Table 2 provide the information on geometrical details of the cover plate for the designed specimens. All the connections for the above mentioned frames are designed in accordance with the design steps of the prequalified connections in the Iranian Building Code No.10 (IBC).

Table 1. Beams and columns properties (Saneei Nia *et al*, 2013)

Beam	b_f (mm)	t_f (mm)	h_b (mm)	t_w (mm)
PG1	150	10	320	8
PG2	200	15	300	8
Column	b_c (mm)	t_c (mm)	h_c (mm)	t_w (mm)
Box 300x20	300	20	300	20
Box 250x20	250	20	250	20

Table 2. Dimensions and characteristics of the specimens

Specimens	Column	Beam	Top Plate (mm)		Seat Plate (mm)		Continuity Plate Thickness (mm)	
			b_t	t_t	b_s	t_s	t_{ct}	t_{cs}
TSP1	Box 300x20	PG2	240	35	240	25	35	25
TSP2	Box 250x20	PG2	240	35	240	25	35	25
TSP3	Box 250x20	PG1	160	30	180	20	30	20

In all specimens of above, the yield and ultimate tensile stresses were considered as 240 and 370 MPa, respectively. In addition Modulus of elasticity and inelastic stiffness slope to idealize mechanical behavior is considered as 2.1×10^5 MPa and 1%, respectively. The location of the specimens in the frame has been shown in Table 3.

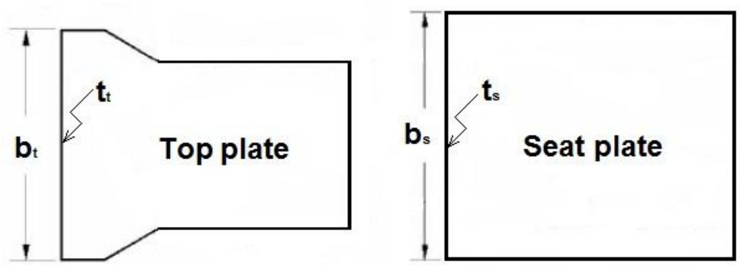


Fig. 3 Cover plate details

Table 3. Location of specimens in the frame

Specimen	Story level
TSP1	1,2,3
TSP2	4
TSP3	5,6,7

At first modeling of the connection by component method was verified by a steel moment frame model, as tested by Saneei Nia *et al.* (2013). The information of material properties of the specimens and their details is provided in Table 3.

Table 3. Dimensions and characteristics of the specimens (Saneei Nia *et al*, 2013)

Dimensions (mm)	Beams			Dimensions (mm)	Columns		
	S	M	L		S	M	L
b_f	160	240	240	b_c	300	400	500
t_f	15	15	20	t_c	15	20	25
h_b	330	330	380	h_c	300	400	500
	Top plate				Seat plate		
b_t	180	240	280	b_s	260	280	300
t_t	30	30	35	t_s	15	20	25

In this research, in order to validate and consider the nonlinear behavior of the beam to box column connection modified Ibarra-Krawinkler deterioration model is used (Karimi and Ghassemieh, 2013). The parameters of the moment-rotation of the connection are presented by Eq. (1-7); as follows:

$$\frac{M_y}{M_p} = 2.03\left(\frac{h}{t_w}\right)^{-0.029} \times \left(\frac{L}{d}\right)^{-0.205} \times \left(\frac{b}{t}\right)_{tp}^{-0.012} \times \left(\frac{b}{t}\right)_{sp}^{-0.026} \times \left(\frac{F_{y,pl}}{240}\right)^{0.013} \quad (1)$$

$$\frac{M_c}{M_y} = 1.59\left(\frac{h}{t_w}\right)^{-0.0689} \times \left(\frac{b_f}{2t_f}\right)^{-0.053} \times \left(\frac{b}{t}\right)_{tp}^{0.018} \times \left(\frac{b}{t}\right)_{sp}^{-0.02} \quad (2)$$

$$\theta_p = 46.74\left(\frac{h}{t_w}\right)^{-1.525} \times \left(\frac{b_f}{2t_f}\right)^{-0.448} \times \left(\frac{L}{d}\right)^{-0.364} \times \left(\frac{d}{280}\right)^{-0.4} \quad (3)$$

$$\theta_{pc} = 8129.33\left(\frac{h}{t_w}\right)^{-1.997} \times \left(\frac{d}{280}\right)^{0.675} \times \left(\frac{b}{t}\right)_{tp}^{-0.483} \times \left(\frac{b}{t}\right)_{sp}^{-0.771} \times \left(\frac{F_{y,pl}}{240}\right)^{-0.868} \quad (4)$$

$$\theta_c = \theta_y + \theta_p \quad (5)$$

$$M_r = 0.4 M_c \quad (6)$$

$$\theta_u = 0.08 \quad (7)$$

where, M_p is the expected moment resistance, M_y the effective moment resistance, M_c the maximum moment resistance, M_r the residual resistance, $(\frac{h}{t_w})$ the ratio of the beam height to its web thickness, L length of the beam, d depth of the beam, $(\frac{b}{t})_{tp}$ the ratio of the top plate width to its, $(\frac{b}{t})_{sp}$ the ratio of the seat plate width to its thickness, $F_{y,pl}$ is yield stress of the plates, $(\frac{b_f}{2t_f})$ the ration of the flange width to twice its flange thickness, θ_y the yield rotation, θ_p the plastic rotation, θ_{pc} the post maximum rotation, θ_c the maximum rotation and θ_u is the ultimate rotation. The various components contributing to the overall response of the cover plate beam to column connection is shown in Table 4.

Table 4. Various components used in modeling

Column web in shear	k_1
Column web in compression	k_2
Column web in tension	k_3

Results of the three connections (Small, Medium and Large) that are modeled by component method are presented in Table 5.

Table 5. Deterioration parameters of the connections

	Modeling specimens		
	Small	Medium	Large
M_y (kN-m)	356.36	569.42	875.36
θ_y (rad)	0.0092	0.0114	0.0111
M_c (kN-m)	392.46	627.11	964.04
θ_c (rad)	0.0413	0.0357	0.0288
M_r (kN-m)	156/98	250.84	385.62
θ_u (rad)	0.0600	0.0600	0.0600

Moment rotation curve resulting from component method is obtained and the accuracy of the results is confirmed with the experimental results as illustrated in Fig. 4

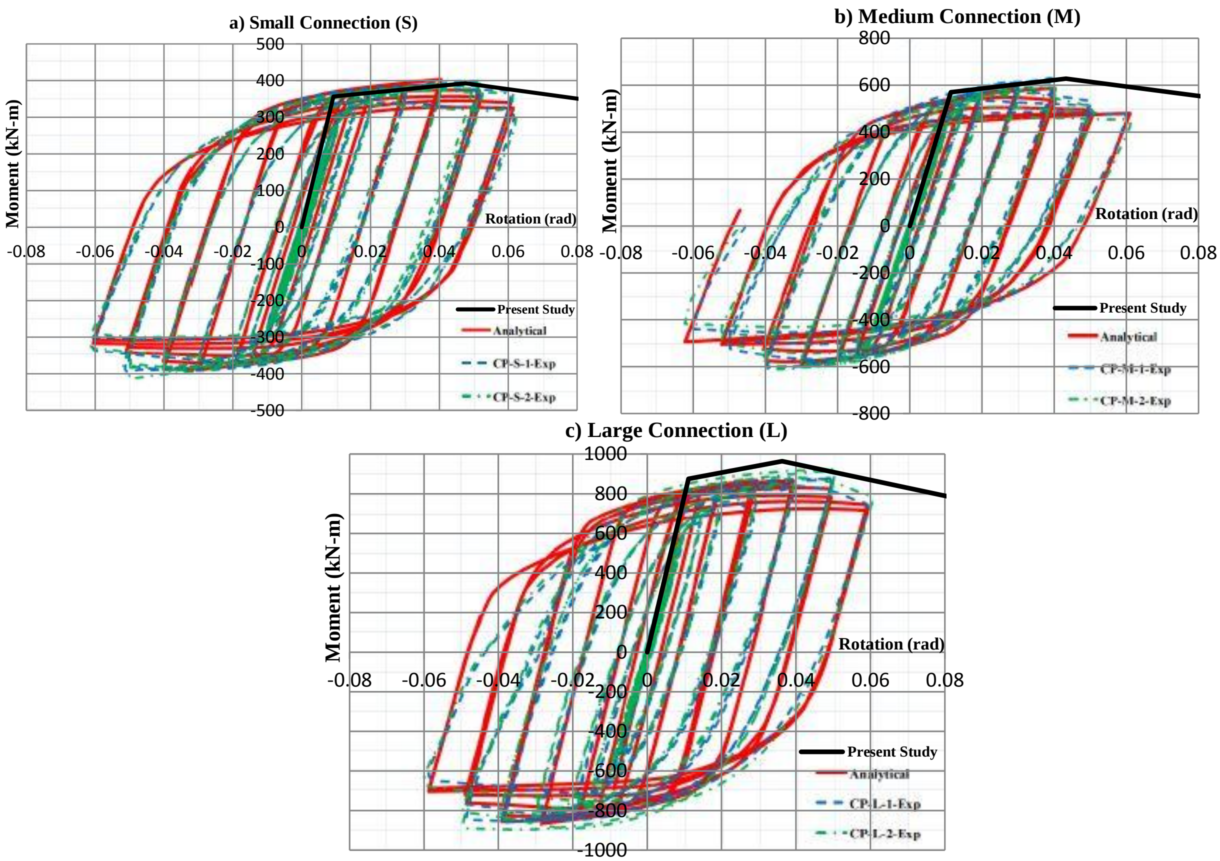


Fig. 4 Moment rotation curve a) Small specimen b) Medium specimen c) Large specimen

After verification, the proposed connections (TSP1, TSP2 and TSP3) were modeled by using component method and the moment rotation curves of the connections were illustrated in Fig. 5 Results have been presented in Table 6.

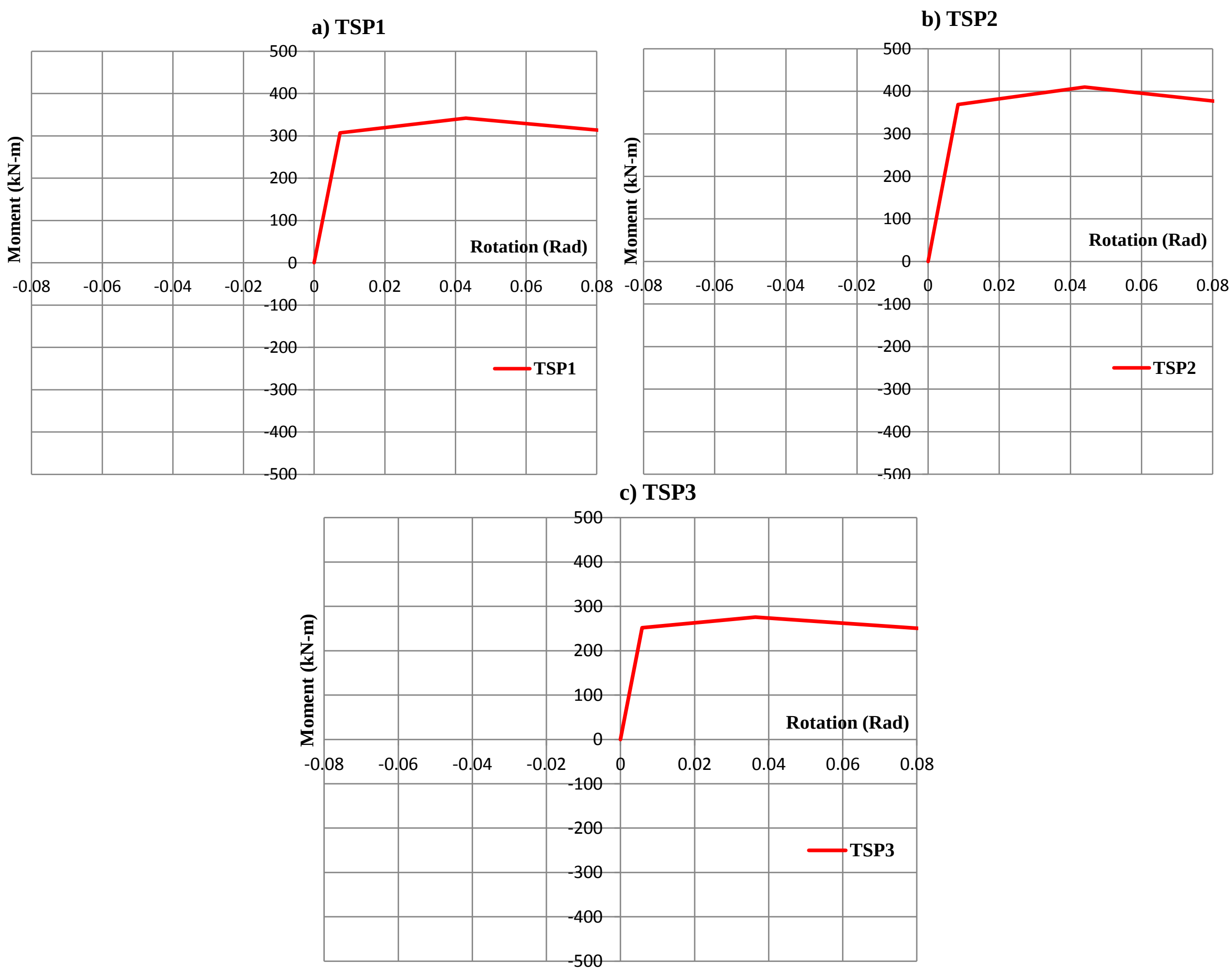


Fig. 5 Moment rotation curve using component method a) TSP1 b) TSP2 c) TSP3

Table 6. Deterioration parameters of the model

	Modeling connections		
	TPS1	TPS2	TPS3
M_y (kN-m)	307.36	368.84	251.54
θ_y (rad)	0.0073	0.0084	0.0058
M_c (kN-m)	341.79	410.15	275.86
θ_c (rad)	0.0429	0.0440	0.0363
M_r (kN-m)	136.71	164.06	110.34
θ_u (rad)	0.0600	0.0600	0.0600

4. MACRO BEHAVIOR OF CP CONNETION

In order to model the flexibility of the connections, bilinear behavior for the connections is utilized. It has three specific parameters which are initial stiffness of the connection k_y, post buckling stiffness k_p and yield moment M_y as shown in Fig. 6 In addition, the element Zero length in OPENSEES (Mazzoni *et al*, 2005) was selected to define the above mentioned element and the material Steel01 was chosen. Validation of the model has been proved by (Ghassemieh and Bahadori, 2015).

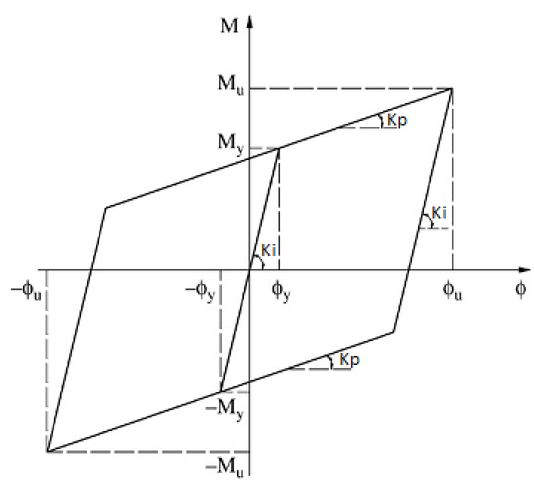


Fig. 6 Bilinear modeling of the connections

4.1. Modal Analysis

By performing the modal analysis, the periods of the 7-story frames are obtained and they have been presented in Table 7. Applying the flexibility of the connection to the model made periods corresponding to the first modes increase while its effect on higher modes is reduced as illustrated in Fig. 7 As seen, when flexibility of beam to column connections is considered, mode shapes of higher modes is comparably similar.

Table 7. Comparison of the periods in all modes

Mode No.	Rigid connections	Connections with flexibility	Error (%)
1	0.897	0.992	9.57
2	0.308	0.329	6.38
3	0.161	0.171	5.84
4	0.105	0.108	2.78
5	0.073	0.074	1.35
6	0.058	0.063	7.93
7	0.057	0.057	0

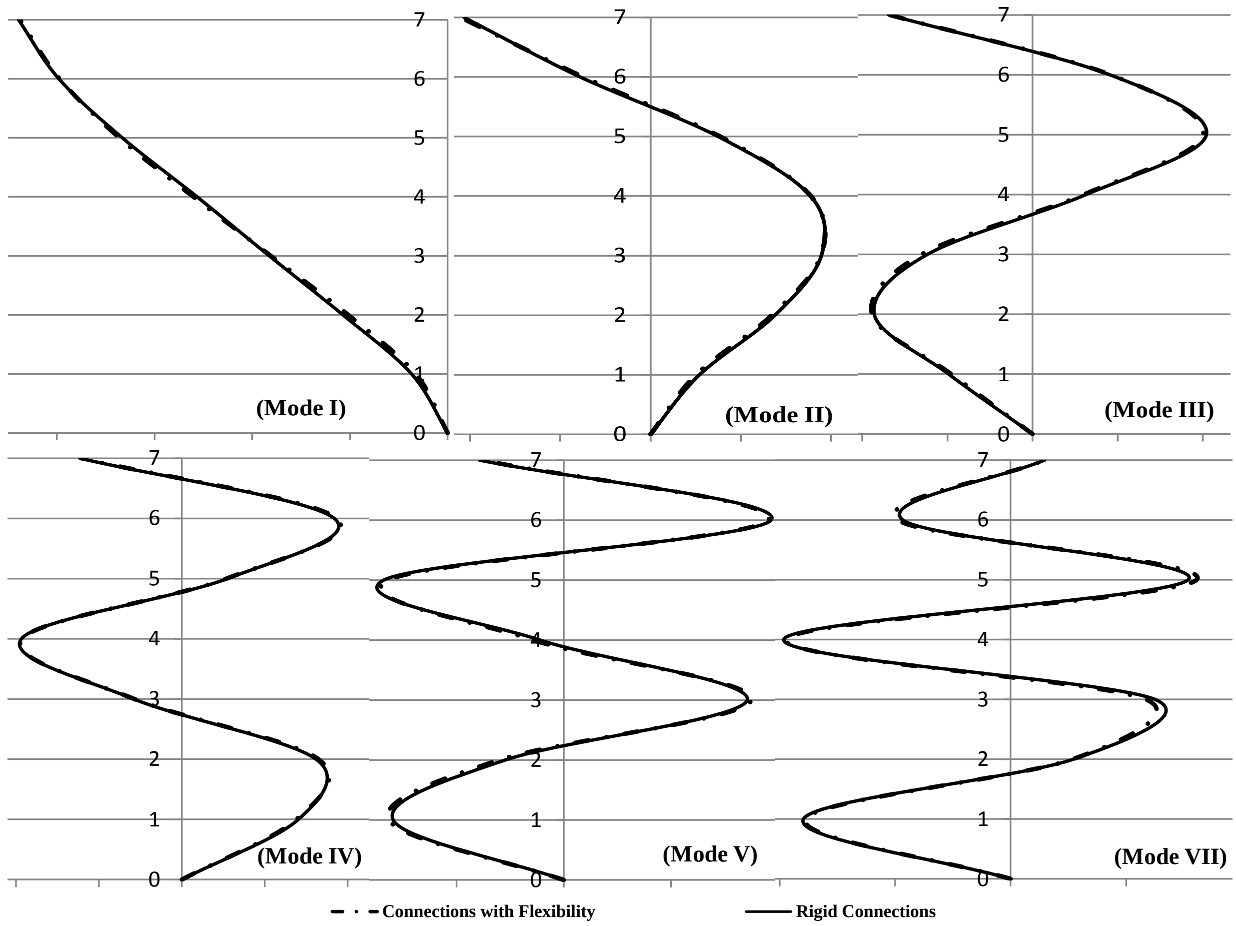


Fig. 7 Mode shapes of the frame

4.2. Nonlinear static analysis (Pushover)

Pushover nonlinear analysis for the seventh story frame with different connections is performed in order to evaluate the actual behavior of the frames. Modal and uniform load patterns were used for performing the analysis (FEMA, 2000a). The first period of the two frames was implemented for the modal analysis. The results of pushover analysis in terms of normalized base shear with respect to the weight of the frame versus roof drift angle and the inter-story drifts are presented in Figs. 8 to 10.

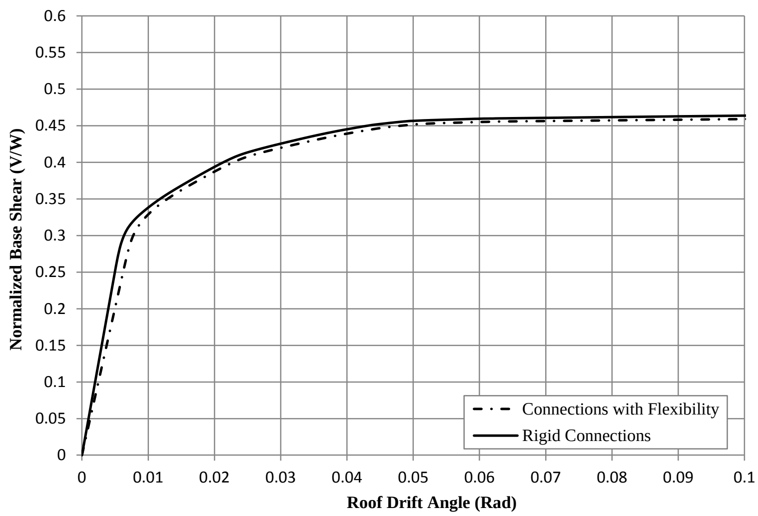


Fig. 8 Pushover response curves (Modal load pattern)

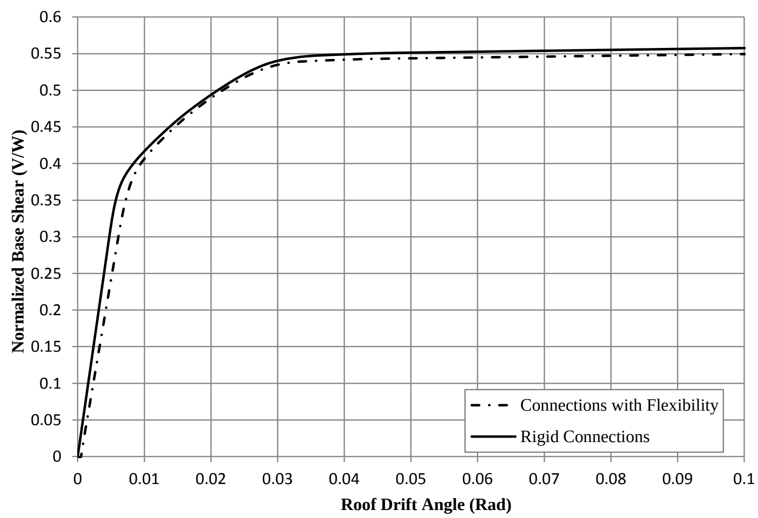


Fig. 9 Pushover response curves (Uniform load pattern)

As illustrated in Figs. 8 and 9 the normalized base shear for rigid and with flexibility connections in elastic zone have considerable difference under modal and uniform load pattern. By applying the flexibility of the beam to column connection in the structural analysis, the initial stiffness decreased. By decreasing rigidity of connection, the base shear decreases. Frames with rigid connection subjected to uniform load pattern reveal higher ultimate strength than frames with flexible connections. It can be drawn that structure response is dependent upon the load pattern, but modal load pattern has shown more logical results than uniform load pattern in stiffness and strength degradation zone. In addition, as demonstrated in Fig. 10 the inter-story drifts in the frame with flexible connections is greater than the frame with fully rigid connections.

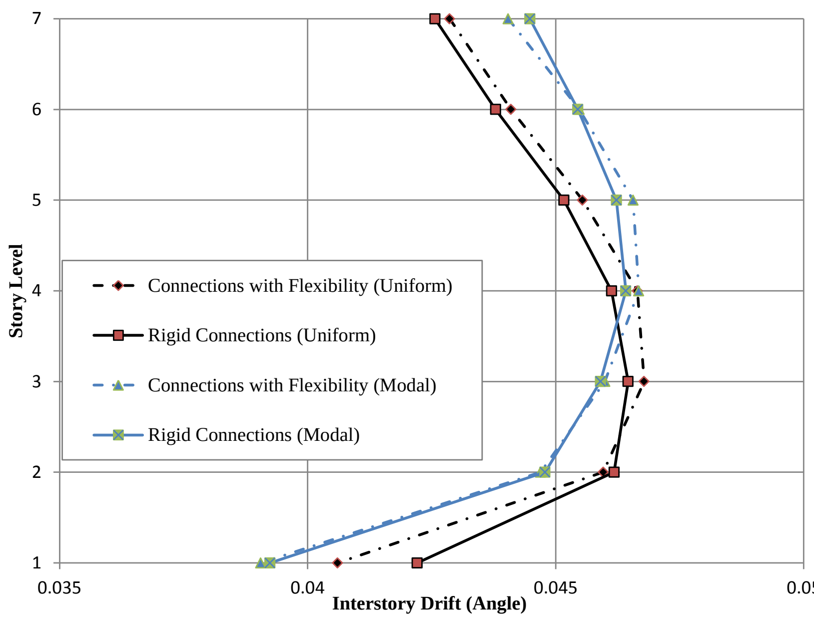


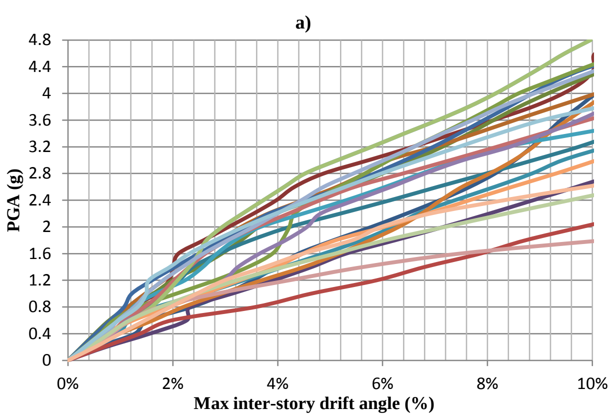
Fig. 10 Inter-story drifts (Modal and uniform load pattern)

4.3. Incremental dynamic analysis (IDA)

Evaluation of the performance of a structure requires a method that monitors the structure behavior from linear elastic region to yielding stage and until it collapses. Incremental Dynamic Analysis (IDA) is a widely used approach to evaluate the performance of structures (Vamvatsikos and Cornell, 2002). In this method, a set of ground motion records are chosen, each record is scaled into multiple intensity levels to cover the whole range of structural response from elastic behavior all the way through yielding and then to dynamic instability. Twenty two earthquake records were selected for this study as shown in Table 8 and PGA versus maximum inter-story drift angle for frame with rigid and flexible connections were drawn in Fig. 11

Table 8. Details of selected ground motions

Number	Ground motion name	Station	***PGA***
1	*Imperial Valley 1979*	*Chihuahua*	*0.25*
2	*Imperial Valley 1979*	*Chihuahua*	*0.27*
3	*Northridge 1994*	*Hollywood Storage*	*0.23*
4	*San Fernando 1971*	*Lake Hughes #1*	*0.15*
5	*San Fernando 1971*	*Hollywood Stor Lot*	*0.21*
6	*Super Stition Hills 1987*	*Wildlife Liquefaction Arrey*	*0.13*
7	*Super Stition Hills 1987*	*Wildlife Liquefaction Arrey*	*0.13*
8	*Super Stition Hills 1987*	*Plaster City*	*0.19*
9	*Landers 1992*	*Barstow*	*0.14*
10	*Cape Mendocino 1992*	*Rio Dell Overpass*	*0.39*
11	*Coalinga 1983*	*Parkfield - Fault Zone 3*	*0.16*
12	*Imperial Valley, 1979*	*El Centro Array #12*	*0.14*
13	*Loma Prieta, 1989*	*Anderson Dam Downstream*	*0.24*
14	*Loma Prieta, 1989*	*Agnews State Hospital*	*0.16*
15	*Loma Prieta, 1989*	*Anderson Dam Downstream*	*0.24*
16	*Loma Prieta, 1989*	*Coyote Lake Dam Downstream*	*0.18*
17	*Loma Prieta, 1989*	*Sunnyvale Colton Ave*	*0.21*
18	*Imperial Valley, 1979*	*El Centro Array #13*	*0.12*
19	*Imperial Valley, 1979*	*Westmoreland Fire Station*	*0.07*
20	*Loma Prieta, 1989*	*Sunnyvale Colton Ave*	*0.21*
21	*Imperial Valley, 1979*	*Westmoreland Fire Station*	*0.11*
22	*Loma Prieta, 1989*	*Hollister Diff. Array*	*0.27*



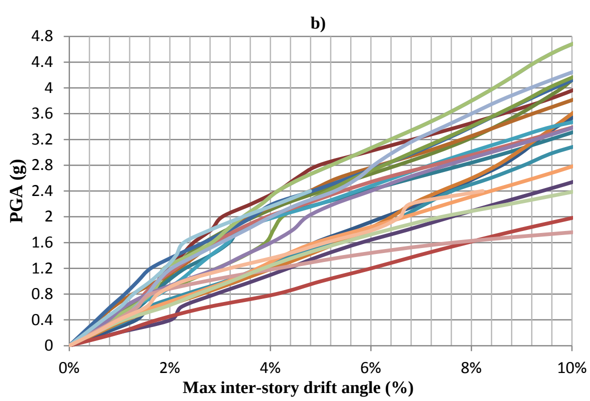


Fig. 11 IDA curves a) fully rigid connections b) considering flexibility

The IDA curves consist of a set of scaled ground motion records known as Intensity Measure (IM) and a series of the structural response known as Damage Parameter (DP). In this study, peak ground acceleration (PGA) and maximum inter-story drift angle (θ_{max})

were considered as IM and DM, respectively. All 22 records were analyzed with 24 (per 0.2g) scaled factors. In Performance Based Earthquake Engineering it is important to define a limit state or performance level for the structure. FEMA 350 (FEMA, 2000b) has defined two structural performance levels as the recommended criteria, the Collapse Prevention (CP) and the Immediate Occupancy (IO) structural performance levels and as stated the maximum allowable rotation for IO and CP levels are 2% and 10%, respectively. By definition, a multi-record IDA study is resulted from a collection of single-record IDA studies. For this aim, Sample median (50%), 16% and 84% fractals are calculated as shown in Fig. 12

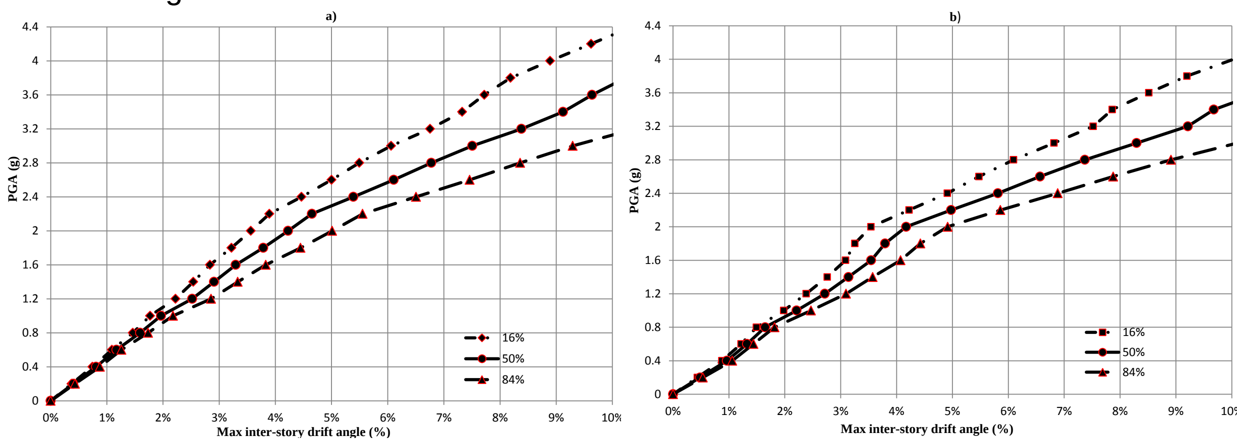


Fig. 12 Multi-IDA curves a) fully rigid connections b) considering flexibility

Table 9 provides information for the difference between the required PGA to pass IO level ($\theta = 2\%$) and CP level ($\theta = 10\%$) for 16%, 50%, and 84% data, respectively between the two cases of study.

Table 9. Minimum PGAs (g) required to pass defined performance levels

	IO level			CP level		
	Rigid	with Flexibility	Differences (%)	Rigid	with Flexibility	Differences (%)
16% of data	0.92	0.82	10.86	3.14	3.0	4.45
50% of data	1.04	0.92	11.53	3.76	3.44	8.51
84% of data	1.12	1.04	7.40	4.32	4.0	7.40

As seen in Table 9, the difference between the required PGA to pass CP level was less than 10% while it increased in IO level up to 12%. This difference between the behavior of the structural frame with connection modeled as fully rigid and connection modeled by considering the flexibility revealed that the influence of the flexibility of beam to column moment connections should not be overlooked.

5. CONCLUSION

In this study, the moment-rotation behavior of beam to box column with cover plate was investigated by component method and the correctness of this method was investigated by using experimental samples available in literature. Comparing obtained results from component method and experimental tests have shown a good agreement between the predicted curves and the real behavior of the connection obtained from tests. The results from the pushover analysis revealed that the initial stiffness and the ultimate strength of the cover plate frames with fully rigid connections are more than their counterpart frame with considering the flexibility of connections. In addition, considering the flexibility of the connection in the behavior of the structural frame made the period of the structure increases. Moreover, the maximum inter-story drifts in frame with considering the flexibility of the connections experience greater values than frames with fully rigid connections. And finally, conducting the incremental dynamic analysis revealed significant difference between the two seven story frames in terms of performance levels and indicated that overlooking the flexibility of beam to column moment connections may lead to inaccurate conclusions.

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