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Effect of semi-rigid connections in improvement of seismic performance of steel moment-resisting frames

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Abstract. Seismic performances of dual steel moment-resisting frames with mixed use of rigid and semi-rigid connections were investigated to control of the base shear, story drifts and the ductility demand of the elements. To this end, nonlinear seismic responses of three groups of frames with three, eight and fifteen story were evaluated. These frames with rigid, semi-rigid and combined configuration of rigid and semi-rigid connections were analyzed under five earthquake records and their responses were compared in ultimate limit state of rigid frame. This study showed that in all frames, it could be found a state of semi-rigidity and connections configuration which behaved better than rigid frame, with consideration of the base shear and story drifts criterion. Finally, some criteria were suggested to locate the best place of the semi-rigid connections for improvement of the seismic performance of steel moment-resisting frames.

Keywords: semi-rigid connections; dual steel moment-resisting frames; seismic behavior, base shear; story drift

1. Introduction

For decades, consideration of the welded rigid connections of the steel frames as a ductile structural system was the common method in building design. But, due to the earthquakes in Northridge (1994) and Kobe (1995), many steel moment-resisting frames with welded rigid connections failed at the beam to column connection area. A key reason for this shortcoming was low ductility and stress concentration in welded area of the connections (Di Sarno and Elnashai 2002). Therefore, finding appropriate alternatives for welded rigid connections has become a challenge for researchers. Liu and Deyuan (2005), proposed an analytical model for design of the semi-rigid connections between steel beams and RC walls in high-rise hybrid buildings, based on the analysis of a typical structure. Rafiee *et al*. (2013) developed the Big Bang-Big Crunch (BB-BC) optimization algorithm for optimal analysis of steel frames with semi-rigid connections. The algorithm used to obtain the minimum total cost which comprises total member plus connection costs by selecting suitable sections. Türker *et al*. (2009) presented an investigation into the determination of the quality of the semi-rigid connections when considering changes in dynamic characteristics of the steel structures. The investigations involved three scaled models:

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columns with box cross-sections, columns with rectangular cross-sections, and a 2D frame. Liu and Lu (2014) investigated the effect of semi-rigid connections on structural performance of the link which suspended the floors with their supporting structure. They presented a new method to design the dynamic response of the suspended building structures. Fan *et al*. (2012) carried out tests on fifteen joints to study the mechanical behavior of socket joints and bolt-ball joints as the typical semi-rigid joint systems which are widely used in spatial structures. Valipour and Mark (2013) developed the formulation for a force-based 1-D compound-element that captures both material and second order P- nonlinearities in steel frames. They verified the accuracy of the formulation by some numerical examples on the nonlinear static, cyclic and dynamic analysis of steel frames. Shi *et al*. (2012) compared different measurement methods in the research literature on beam-to-column joint rotation in steel frames, and proposed improved approaches for the experimental quantification of the rotation of welded joint and welded-flange-bolted-web (WFBW) joints. Galvão *et al*. (2010) studied the free and forced nonlinear vibrations of slender frames with semi-rigid connections. Special attention was given to the influence of static pre-load on the natural frequencies and mode shapes, nonlinear frequency–amplitude relations, and resonance curves. Also, Mirza and Uy (2011), studied the behaviour of the composite beam-column flush end-plate connections subjected to low-probability, high-consequence loading. The semi-rigid bolted connections have been considered as a suitable alternative in frame design. These connections have less moment capacity, more ductility and more plastic rotation capacity with regard to the welded rigid connection. The high plastic rotation capacity of the semi-rigid connections enables them to deform non-elastically without similar damages which occurs in the welded connections.

More story drifts in semi-rigid frames is one of the main obstacles for use of semi-rigid connection, especially in tall buildings. Some researchers believe that the use of the combined rigid and semi-rigid connection (dual frame) can reduce story drifts, besides having the advantages of the semi-rigid connections (Dubina *et al*. 1998). In the dual frames, semi-rigid connections supply the required levels of the ductility. Also, the rigid connections can supply the most lateral stiffness and absorb the seismic energy levels to prevent from higher lateral displacements. By using the semi-rigid connections the stiffener members can be eliminated, therefore the constructional costs would be decreased (Dubina *et al*. 2000). Most of the semi-rigid frames are manufactured by bolted connections which are easily repairable after any probable structural failing, with a low cost (Dubina *et al*. 1998).

Dabina *et al*. (2000) studied behavior of semi-rigid frames in seismic areas on a three span six-story two-dimensional frames. The main comparison parameters were the formation of the failure mechanism and bending moment of the elements. They concluded that the semi-rigidity of the connections led to improvement of the frame behavior, but increased the story drifts.

Kishy *et al*. (1996) studied application of dual frames in a four-span eight story building. They considered the semi-rigid connections as a criterion for comparing the construction costs. By analyzing different frames with various configurations of the semi-rigid connections, they showed that with appropriate selections of the semi-rigid connections and also controlling the amount of the drifts, the construction costs significantly could be decreased with increasing the number of the semi-rigid connections.

Bullent and Shen (2003) studied the behavior of the dual frames and showed that the initial stiffness of the rigid connection restricted the story drifts. On the other hand, they approved that the ductility, energy dissipation capacity and the stable cyclic behavior of the semi-rigid connections could prevent the stress concentration in the rigid connections during an earthquake.

Therefore, it decreased the ductility demand of the elements. As another study, Bullent and Shen (2003) examined the behavior of five and ten story dual buildings. In the buildings, external frames were rigid and internal frames were pinned or semi-rigid. They showed that an optimized design could be reached using the dual frames in seismic areas, and the base shear and bending moment of columns and connections would be drastically decreased.

In this paper, the dynamic behavior of the dual frame systems are investigated and compared with the rigid and the semi-rigid frames. For this purpose, non-linear responses of three building groups (included of three, eight and fifteen story frames) with rigid, semi-rigid and combined configurations under the dynamic loads of five different earthquakes are determined. Then, the results are compared in ultimate limit state of rigid frame. In this state, the effect of the rigidity of connection of frame is evaluated during their dynamic behaviors. Drain -2Dx software is applied for non-linear dynamic analysis of the frames (Parkash *et al*. 1993). Base shear, story drifts, ductility of the elements and connections and soft story mechanism, are employed as criteria to investigate the frame behavior.

2. Frames collapse criteria

As a comparison criterion, the ultimate limit state of rigid frame was considered for investigation of the nonlinear responses of the frames. The rigid frames analyzed under 5 earthquake records for each building groups, while the scale of spectrum were increasing gradually to determine the failure rate acceleration. Then the dynamic response of the other frames found out under the obtained failure rate acceleration. Allowable story drifts was limited to 3% of the story height to prevent from structural damages. Based on the strong column weak beam concept, the steel structures should be designed in a manner that the plastic hinges form in beams and the base of the first story columns. Also, the soft story mechanism should not form. The third criterion for frame collapse was the plastic rotation of connections. According the Euro code, the connections are permitted to rotate plastically less than the 0.03 radian.

The forth collapse criterion was the beam and column ductility. The sections of the beams and columns should support the required rotation during the plastic hinge formation, avoiding buckling. Plastic rotation capacity of the beams and columns can be measured as follow (Gioncu and Petcu 1997)

$$\theta_u = (1 + R_{av})\frac{M_p L_{sb}}{EI} \tag{1}$$

Where, θ_u is the plastic rotation capacity of members, M_p is the plastic moment capacity of beam, L_{sb} is the standard beam length, E is elasticity modulus and I is the moment of inertia. Rotation capacity of beam and columns (*Rav*) is acquired via Eqs. (1) and (2), respectively

$$R_{avB} = \frac{423\times10^4 t_f\left[0.8 + 0.2\left(f_{yw}/f_{yf}\right)\right]}{(b - 0.5t_w - 0.8r)L_{sb} f_{yw}} \tag{2}$$

$$R_{avC} = \frac{165b(1 + 44.2n_p)(\bar{\lambda}\frac{b}{t_f}\sqrt{f_y})^{-1.25+0.9n_p}}{b - 0.5t_w - 0.8r} \tag{3}$$

In Eqs. (2) and (3), t_f and t_w are the thicknesses of flanges and web, f_{yw} and f_{yf} are yield tensile strength of flange and web, b is half-width of flange and axial force ratio n_p and slenderness factor λ are defined as bellow

$$\bar{\lambda} = \sqrt{\frac{N(L_{sb})^2}{\pi^2 E_b I_b}} \tag{4}$$

$$n_p = \frac{N}{N_p} \tag{5}$$

Where, N is the axial force and N_p plastic strength capacity of the column. The axial forces of the columns N is determined by static analysis.

3. Frames and connections modeling

The characteristics of the investigated frames are shown in Fig. 1 and Table 1 (Reyes-Salazar and Haldar 1999). Beams and columns of the frames were made by the A36 and G50 steel with 250 and 345 MPa yielding stress. Dead load, live load and damping coefficient were considered equal to 4.9, 2.5 KN/m^2 and 5%, respectively. Also, span length and story height were 7.32 and 3.66 meter, respectively.

To investigation of the connection effect on the frame behavior, 3 types of semi-rigid connections were considered, in addition to rigid connection, as shown in Table 2. The positions of

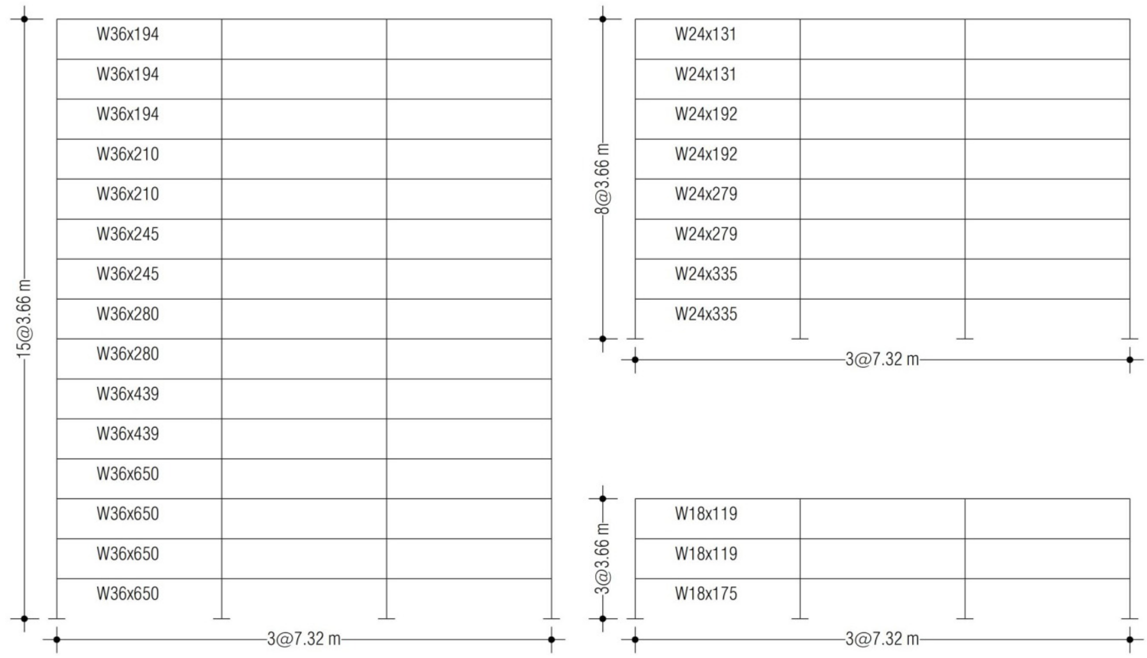


Fig. 1 Geometry of the frames (Reyes-Salazar and Haldar 1999)

Table 1 Specification of three, eight and fifteen story frames elements

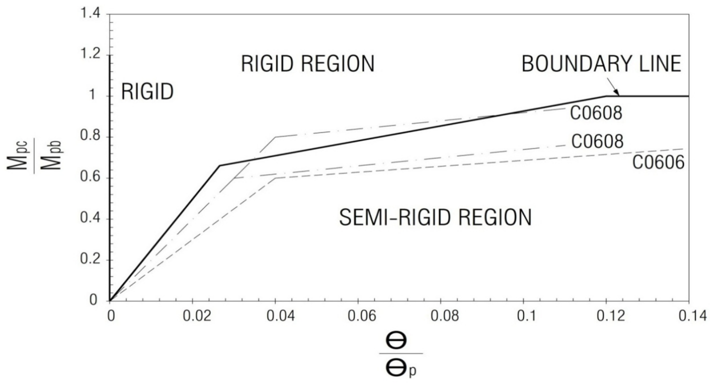
Frame	Story	External column	Internal column	Beam
3 story	1	W14 × 211	W14 × 283	W18 × 175
	2-3	W14 × 145	W14 × 211	W18 × 119
8 story	1-2	W14 × 370	W14 × 550	W24 × 335
	3-4	W14 × 257	W14 × 370	W24 × 279
	5-6	W14 × 211	W14 × 257	W24 × 192
	7-8	W14 × 193	W14 × 211	W24 × 131
15 story	1-4	W14 × 665	W14 × 730	W36 × 650
	5-6	W14 × 455	W14 × 665	W36 × 439
	7-8	W14 × 426	W14 × 455	W36 × 280
	9-10	W14 × 398	W14 × 426	W36 × 245
	11-12	W14 × 342	W14 × 398	W36 × 210
	13-15	W14 × 311	W14 × 342	W36 × 194

Table 2 Specification of the connections

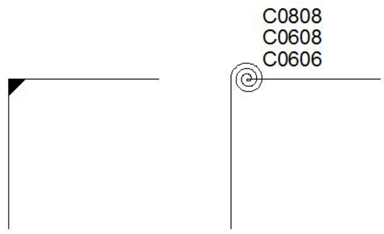
Connection	Strength of connection	Connection rigidity
RIGID	1.2 $M_{pl,beam}$	∞
C0808	0.8 $M_{pl,beam}$	0.8 K_{sup}
C0608	0.6 $M_{pl,beam}$	0.8 K_{sup}
C0606	0.6 $M_{pl,beam}$	0.6 K_{sup}

the connections based on the Euro code classification system are shown in Fig. 2. In this figure, k_{sup}= 25 EI/L_b (rigidity of connection) is a boundary line between semi-rigid and rigid region and $\theta_p = M_{pb} \Big/ \left(\frac{E.I}{5d_{be}}\right)$ is the rotation of the beam with the length of $5d_{be}$ under the plastic moment capacity (Dubina *et al*. 1998). In these equations, L_b is the length of the beam and d_{be} is the depth of the web.

The frames were studied in three main groups with 3, 8 and 15 stories. Each group contained a rigid connection frame, a frame with semi-rigid connection and also the dual frames with different combinations of the rigid and semi-rigid connections. Twenty three different configurations of semi-rigid connections in 3 story frames are shown in Fig. 3 schematically. With consideration of 3 various connection types (C0808, C0608 and C0606) for the semi-rigid connections, the number of the cases were 70 frames. The 8 story frames contained a fully rigid frame, 24 dual frames and a semi-rigid frame, as shown in Fig. 4. Each frame, shown in Fig. 4 was studied in three types of frames, by assuming the joints with fully rigid connections in first story, rigid connections in first and second stories and rigid connections in first and eight stories. Moreover, the 15 story frames contained a fully rigid frame, 24 dual frames and a semi-rigid frame, as shown in Fig. 5. Each frame, shown in Fig. 5 was studied in three types of frames, assuming the joints with fully rigid connections in first and second stories, rigid connections in 1~4 stories and rigid connections in first, second and 14th and 15th stories.



(a)



(b)

Fig. 2 (a) The position of the connections based on the Euro code classification system; (b) Different rigid and semi rigid forms

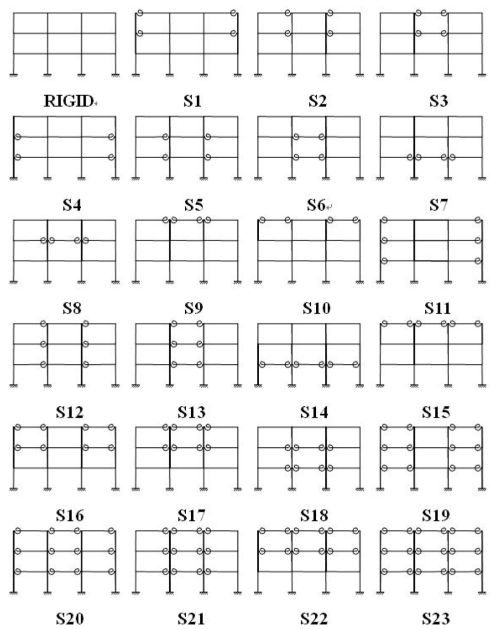


Fig. 3 Dual and rigid 3 story frames

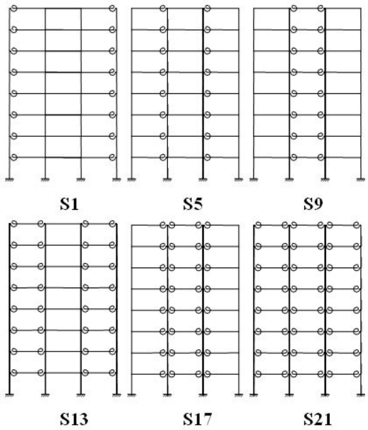


Fig. 4 Dual and semi-rigid 8 story frames

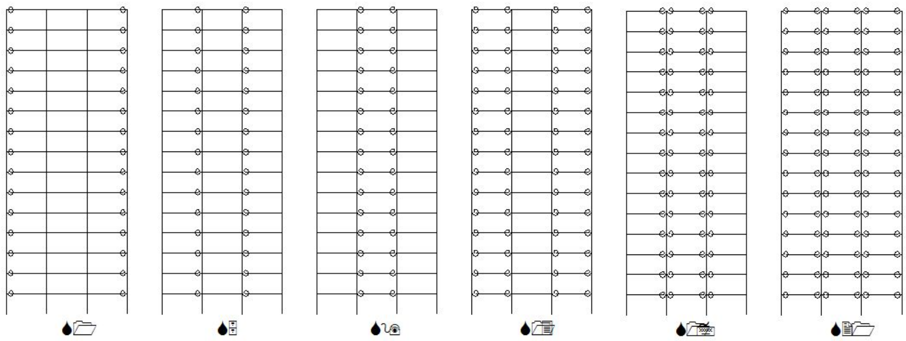


Fig. 5 Dual and semi-rigid 15 story frames

Non-linear dynamic analysis of frames was carried out using the Drain-2Dx computer program (Parkash *et al*. 1993). This program is able to perform this analysis based on the time history method. For the sake of considering the material nonlinearity effects, the bilinear stress-strain diagram was used. Also, the ground motions were applied to supports in the form of the acceleration time histories. The P-Δ effects were ignored because of the disregard of the vertical accelerations. Element No. 2 was used for modeling the beam and columns. This element is able to model the plastic hinges that form only in two oppose ends. Element No. 4 was used to model the connections. In the Drain software, the type No. 2 is a simple inelastic beam column element and is able to model the plastic hinges that form only in two oppose ends. To define this element, the amount of the initial stiffness (E = 2.1E6 kg/cm^2), the strain hardening ratio (0.1) and yield stress are necessary in addition to cross section area, moment of inertia and etc. The type No. 4 is an element with zero length as a simple inelastic connection, which allows for translational as well as rotational force transfer. Bilinear model is used to describe the M-θ behaviour of the connections. To define this element, the introducing of the amount of the initial stiffness, the strain hardening ratio and yield strength are necessary (Erbay *et al*. 2004).To define the connection element, the initial rigidity, yield strength, and ratio of secondary stiffness to initial stiffness was assumed equal

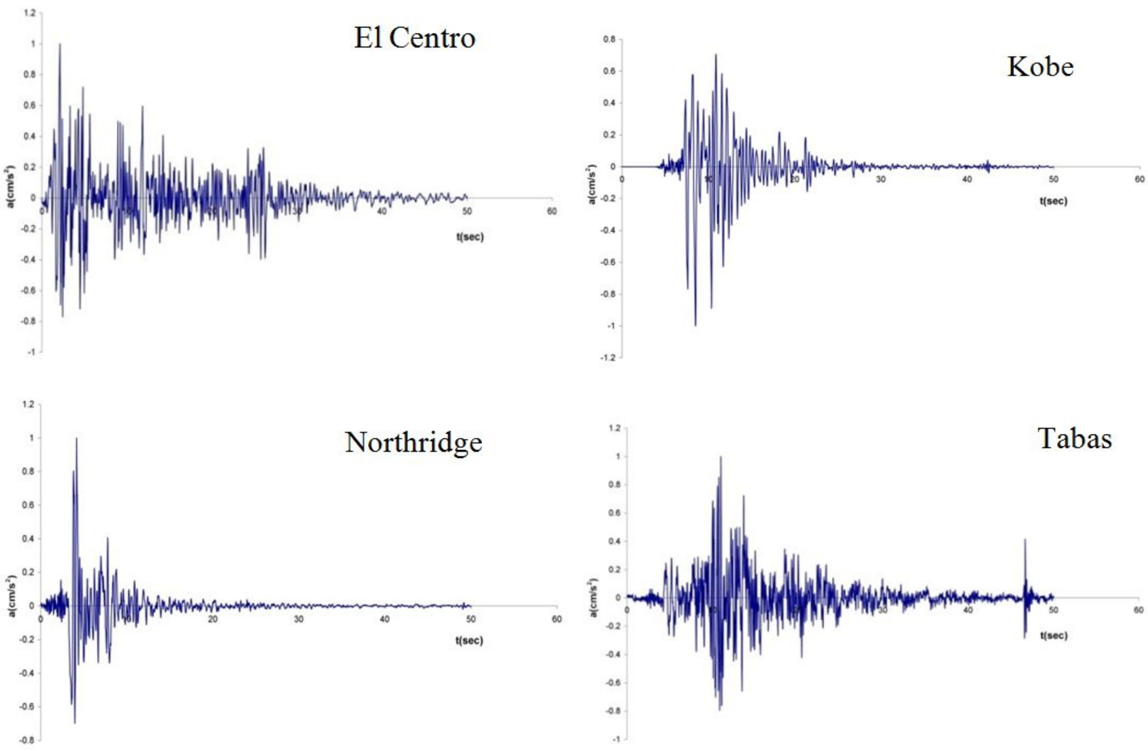


Fig. 6 Acceleration time: El Centro (1940) SE component, Kobe (1995) NS component, Northridge (1994) 360deg component, Tabas (1979) 344 component

to 0.1. The building masses were modeled as lumped mass. Acceleration time histories were selected so as to represent different types of ground motions. The following records were used: El Centro (1940) SE component, Kobe (1995) NS component, Northridge (1994) 360deg component, Tabas (1979) 344 component and Taft (1952) E21N component (as shown in Fig. 6). The abovementioned frames, were subjected to scaled acceleration records of increasing magnitude in order to determine the different structural states.

4. Results and discussion

The behavior of 3 group frames devaluated in ultimate limit state of rigid frame subjected to 5 selected records. The distribution of plastic hinges and collapse mechanism of frames were inspected under the changes of the rigidity, strength and configuration of the semi-rigid connections. To this end, the collapse acceleration of the rigid frame was calculated for each frame group, under the five earthquake records. Then, by applying this collapse acceleration to other frames of the group, their dynamic responses were determined. For measuring the ultimate limit state of the rigid frame, the criteria of the allowable story drifts, soft story mechanism and plastic rotational capacity of the connections were considered based on the (Dubina *et al*. 2000). The obtained results are presented in next sections.

4.1 Frame periods

The rigidity of connections was the dominant parameters to determine the period of the mode shapes, and the analyzing process was not sensitive to the moment capacity. The first period of three story frames are shown in Table 3. The differences between first period of dual frames and rigid frames are illustrated in Fig. 7. In this figure, the effects of the position, rigidity and amount of the semi-rigid connections on first period are shown.

Table 3 First period of the dual and semi-rigid frames (sec)

3 story frame				3 story frame			
RIGID		0.526				Connection rigidity	
		Connection rigidity				0.6 K_{sup}	0.8 K_{sup}
		0.6 K_{sup}	0.8 K_{sup}		S12	0.542	0.538
	S1	0.535	0.533		S13	0.540	0.537
	S2	0.535	0.533		S14	0.549	0.543
	S3	0.534	0.532		S15	0.532	0.531
	S4	0.541	0.538		S16	0.544	0.540
	S5	0.540	0.537	Frame	S17	0.544	0.540
Frame	S6	0.538	0.536		S18	0.554	0.548
	S7	0.540	0.537		S19	0.559	0.551
	S8	0.540	0.536		S20	0.559	0.552
	S9	0.530	0.529		S21	0.559	0.551
	S10	0.530	0.529		S22	0.554	0.548
	S11	0.543	0.539		S23	0.577	0.565

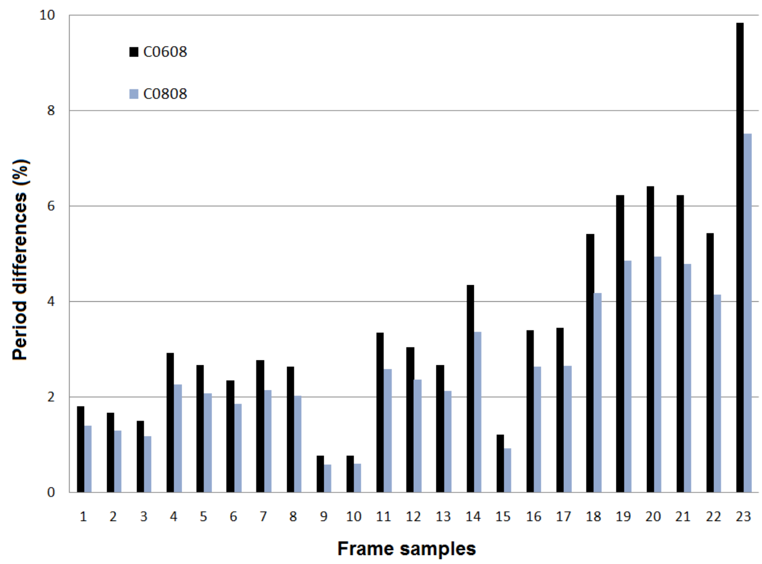


Fig. 7 Variance percentage between mode shapes of the dual frames with 3 story rigid frames

The similar behavior was concluded for 8 and 15 story frames. These results implied that the shortest period was observed for the rigid frame. Also, the frame period increased by increasing the number of semi – rigid connections, and the maximum period was for the semi-rigid frame. By assuming the rigid connections in lower stories, the periods of these frames approach to period of rigid frame that led to similar behavior for frames.

Generally, frame period increased with increasing the number of semi-rigid connection and decreasing of the rigidity. In addition, the position of the semi-rigid connections is the significant factor for the frame behavior. The number of the semi-rigid connections in S1~S10, S11~S15, S16~S18, S19~S22 and S23 were 4, 6, 8, 12 and 18, respectively.

As a general rule, by assuming the rigid connections in lower stories and simultaneously in the exterior spans of the upper stories and as well as the semi – rigid connections in interior spans of upper stories, there is no considerable change in dual frame periods. Consequently, the construction cost will decrease. This fact can be used as a suitable way to find an optimized draft

Table 4 The ratios of base shear and maximum story drift of the dual for three story frames and C0606 connection

Record	Frame	3 story frame	
		C0606	
		V_{max}	Drift
El Centro	S1	0.927	1.058
	S2	0.938	1.054
	S3	0.949	1.044
	S4	0.929	1.102
	S5	0.950	1.069
	S6	0.955	1.049
	S7	0.980	1.076
	S8	0.916	1.098
	S9	0.956	1.014
	S10	0.967	1.000
	S11	0.910	1.092
	S12	0.932	1.066
	S13	0.945	1.041
	S14	0.963	1.092
	S15	0.938	1.018
	S16	0.871	1.105
	S17	0.871	1.121
	S18	0.883	1.115
	S19	0.851	1.097
	S20	0.842	1.113
	S21	0.844	1.111
	S22	0.829	1.156
	S23	0.750	1.107

with suitable seismic behavior for the frames.

4.2 Seismic behavior of dual frames

The ratios of base shear (V_{max}) and maximum story Drift of the dual frames divided by the amounts for the rigid frame are shown in Table 4 for three story frames and C0606 connection, under El Centro record. The ratio of plastic rotation of members of the dual frames divided by the amount for the rigid frame is shown in Table 5 for eight and fifteen story frames. In Table 4, λ is the collapse acceleration scale factor of the rigid frame.

Frame optimization plan is selected considering criteria such as the base shear, story drifts, ductility of frame elements and probability of the soft story mechanism. Generally, with respect to the above mentioned criteria, a configuration can be found for all frames in which that the dual frames behave better than the rigid frame. The resultant optimized plan depends on the frame

Table 5 Ratio of plastic rotation of elements for semi-rigid frames to rigid frames in El Centro Earthquake

	8 stories			15 stories		
	C0606	C0608	C0808	C0606	C0608	C0808
S1	0.6723	0.6987	0.8010	0.9234	0.9199	0.9249
S2	0.6922	0.7229	0.8191	0.9367	0.9329	0.9299
S3	0.7534	0.7846	0.8571	0.9416	0.9368	0.9313
S4	0.6942	0.7253	0.8236	0.9515	0.9461	0.9395
S5	0.6686	0.6545	0.7365	1.1286	1.0824	1.0349
S6	0.6909	0.6769	0.7513	1.1267	1.0825	1.0314
S7	0.7372	0.7163	0.7814	1.1272	1.0825	1.0327
S8	0.6954	0.6802	0.7553	1.1381	1.0903	1.0404
S9	0.7696	0.7795	0.8278	1.1421	1.1160	1.1142
S10	0.7914	0.8021	0.8458	1.1405	1.1159	1.1123
S11	0.8358	0.8450	0.8703	1.1455	1.1211	1.1147
S12	0.7967	0.8064	0.8503	1.1655	1.1377	1.1223
S13	0.6565	0.6330	0.6308	1.1747	1.1220	1.0726
S14	0.6664	0.6473	0.6483	1.1707	1.1192	1.0613
S15	0.7073	0.6922	0.6907	1.1693	1.1229	1.0643
S16	0.6702	0.6492	0.6562	1.1952	1.1427	1.0796
S17	0.5672	0.5996	0.6807	1.1222	1.1130	1.1094
S18	0.5846	0.6250	0.7031	1.1125	1.1082	1.1001
S19	0.6416	0.6825	0.7354	1.1115	1.1101	1.0991
S20	0.5862	0.6282	0.7087	1.1411	1.1355	1.1165
S21	0.1684	0.2237	0.4025	0.3434	0.4313	0.5603
S22	0.1915	0.2501	0.4129	0.3328	0.4366	0.5545
S23	0.2567	0.3201	0.4717	0.3556	0.4477	0.5474
S24	0.1954	0.2501	0.4203	0.3643	0.4503	0.5608

geometrical and mechanical properties, rigidity and strength of the connections. Considering the Tables 4 and 5 it has been observed that the amounts of the story drift, base shear and plastic rotation of the elements are related to stiffness, rigidity and strength of the connections and also the pattern of locating of the semi rigid connections.

4.2.1 Effect of rigidity and strength of connections

Fifteen story semi-rigid frame (S21), three story and eight story dual frames (S10 and S8) showed the best performance. In these cases, the maximum base shear was less than the rigid frame. Also, the maximum story drift was less than the similar value for the rigid frame, except the fifteen story semi-rigid frame (S21).

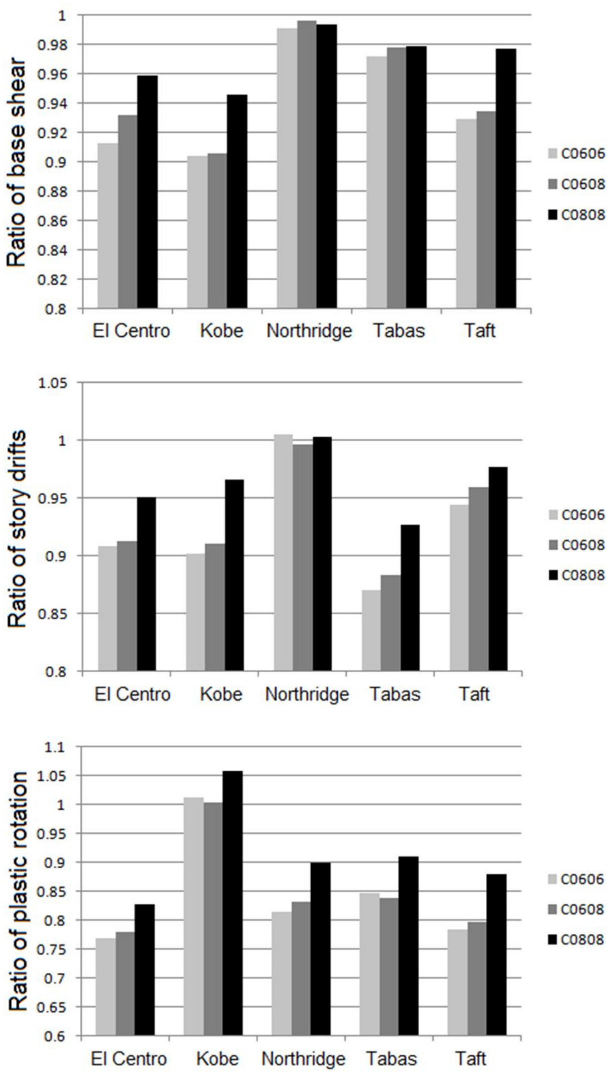


Fig. 8 Ratios for 8 story S9 dual frame to rigid frame: (a) the ratio of base shear; (b) the ratio of story drifts; (c) the ratio of plastic rotation of elements

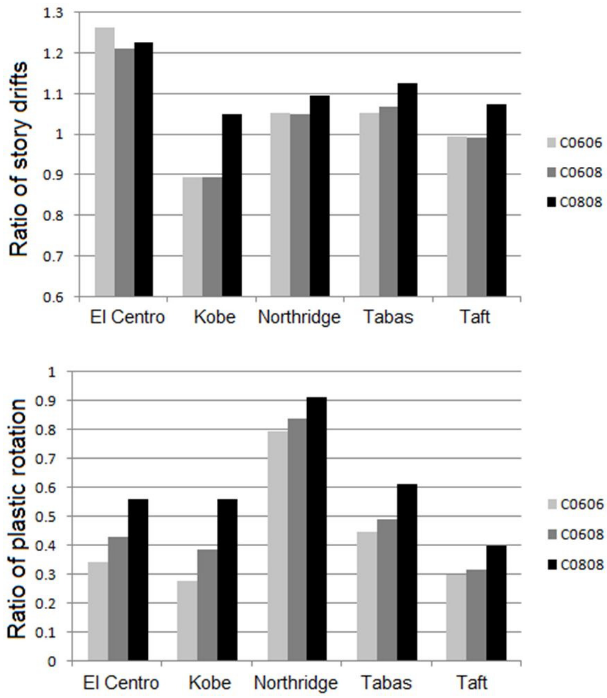


Fig. 9 Ratios for 15 story S21 semi-rigid frame to rigid frame: (a) the ratio of story drifts; (b) the ratio of plastic rotation of elements

The ratios between the base shear, story drifts and member plastic rotation of the eight story dual frame (S9) and the rigid frame are shown in Fig. 8. The behavior was the same in three and fifteen story frame. The ratio between the story drifts and member plastic rotation of fifteen story semi-rigid frame (S21) and rigid frame are shown in Fig. 9.

The limitations for story drifts didn't satisfy for S21 frame. Maximum story drifts of this case with C0606 connection under El Centro earthquake were 1.26 times more than the similar value for rigid frame while the plastic rotation of the element was 0.43 times less than the similar value for the rigid frame.

The semi-rigid connections can cause to transfer the plastic hinges from the elements to the connections. Therefore, the plastic rotation of the elements would decrease. By decreasing the plastic rotation of the elements, the probability of the local bulking and soft story mechanism formation would decrease. Decreasing of the connection strength (from 0.8 M_{pl} in CO808 to 0.6 M_{pl} in CO608) would cause decreasing of the base shear and story drifts, as shown in Fig. 8.

By decreasing the strength connection and transferring of the plastic hinges from elements to connections, the connections will play an important role in absorbing the lateral displacement. In this case, the plastic rotation of the elements will decrease.

In opposed to the parameter of the connection strength, the connection rigidity has no considerable effect on the frame behavior.

Generally, the connections with low-strength show better performance under the seismic loads.

Based on the Euro code, simultaneously decrease in rigidity and strength of semi-rigid region leads to decrease the base shear, plastic rotation of the elements and story drifts.

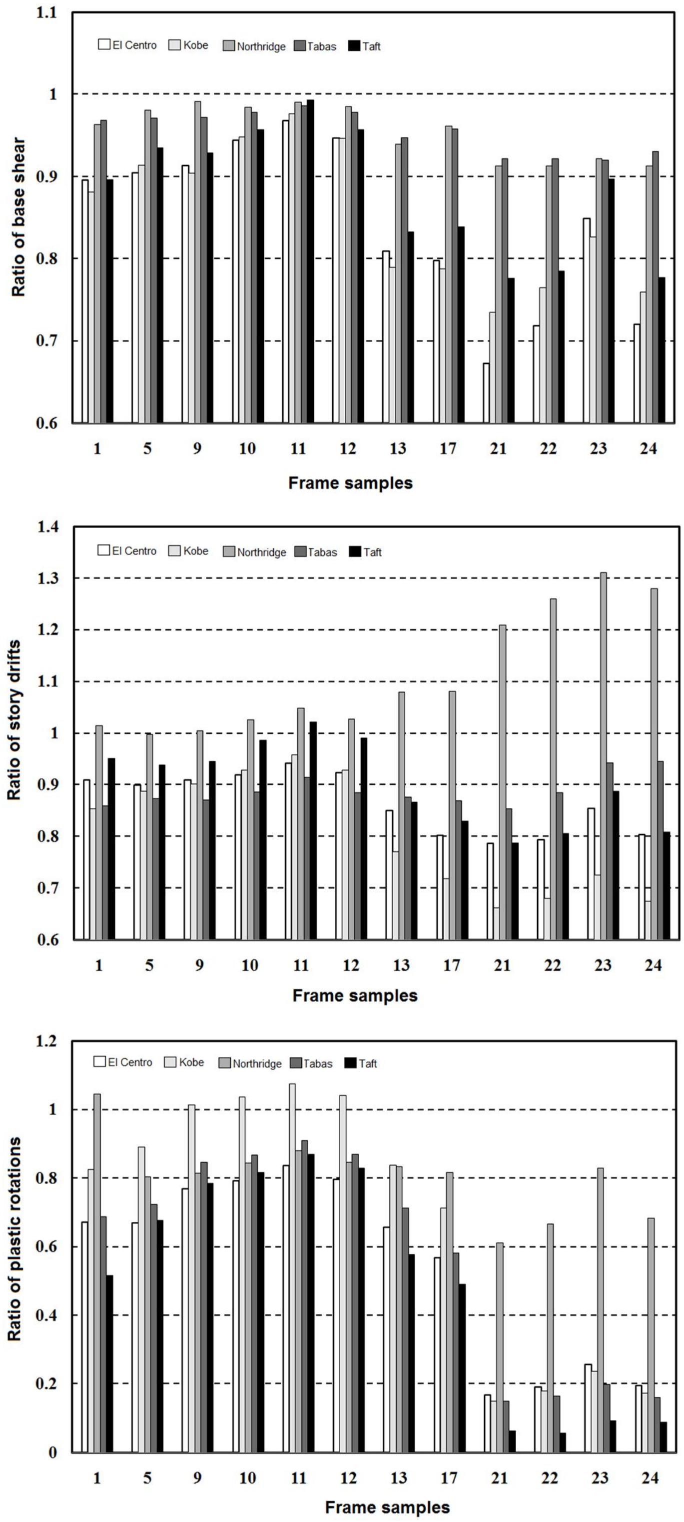


Fig. 10 Variance of (a) base shear; (b) story drifts; (c) plastic rotation of elements for eight-story dual frames

In the case of fifteen story frame, the limitation of the story drift wasn't satisfied. In these cases in order to satisfy the story drift limitation, an increase in beams and columns size is required, but on the other hand the construction cost of the connections will reduce. Therefore, in order to judge about the construction cost of the rigid and dual-frames in tall buildings, it is needed to compare the cost of the increase in the size of the elements and decrease in the cost due to using the semi-rigid connection.

4.2.2 The effect of the positions and number of the semi-rigid connections

For a certain dual frame, the most appropriate seismic behavior was observed for the frames with the CO606 connections. In this section, the effects of the change in the number and position of the semi-rigid connection on the dual frame behavior are investigated using CO606 connections.

By increasing the number and the changes of the connection position, the behavior of the dual frames (base shear, story drifts and maximum plastic rotation of elements) under the all abovementioned earthquake records will change according to the certain pattern, as shown in Fig. 10.

The dual frame behavior was investigated under the El Centro earthquake and the results were generalized for the other earthquake records. The semi-rigid frames behaved better in comparison with the rigid frames, by supplying more ductility and less base shear. The main defect of the semi-rigid frames was the much observed story drifts, especially in tall buildings. This problem could be addressed by increasing the size of the structural elements and/or using the rigid connections in some joints. In this paper, the second solution was investigated (as shown in Fig. 11).

By transferring the semi-rigid connections from exterior spans to the interior spans led to decrease the lateral stiffness and the base shear and increase in story drifts and ductility demand of the elements. Therefore, using the semi-rigid connections in exterior spans is more desirable (e.g., more observed base shear and ductility demand for the eight story S9 frame in comparison with S1).

With the same number of the semi-rigid connections, using of these connections in lower

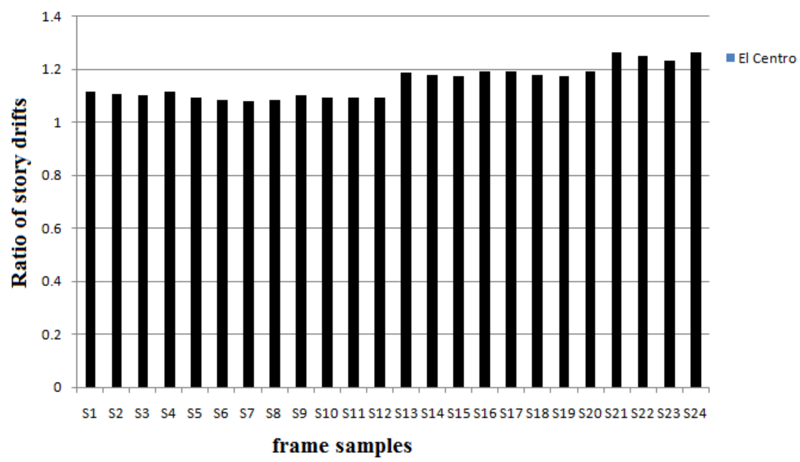


Fig. 11 Variance of story drifts for fifteen-story dual frames (El Centro)

stories led to more flexibility and less base shear and ductility demand (e.g., 2.6% less observed base shear and 6.8% less story drift and 23.4% more ductility demand for S15 frame in comparison with S14).

The concentrated rigid or semi-rigid connections in a local region of the frame led to disturbance in the distribution of the internal forces and deformations. On the other hand, the appropriate distribution of the rigid and semi-rigid connections, in all over the frame, led to better behavior (e.g., less observed story drifts and ductility demands for three story S10 frame with comparison of S9). With the appropriate positioning of the semi-rigid connections, the increase in the number of the connections not only led to unsuitable behavior but also could improve the frame behavior.

In the case of the three story S11 frame, increasing in the number of the semi-rigid connections in comparison with the S4 frame and using them in appropriate location, the frame behavior was improved (2.04% and 0.91% decrease in the base shear and story drifts were observed, respectively). Also, by increasing the number of the semi-rigid connections, the construction cost decreases. Therefore, it should be tried to find the best pattern for the distribution of the semi-rigid connections in the dual frame systems.

5. Conclusions

In this paper, seismic behavior of dual frames was investigated and compared with rigid and semi-rigid frames, as an alternative system for the frames. The semi-rigid frames behave better in comparison with the rigid frames, by supplying more ductility and less base shear. The main defect of the semi-rigid frames is the much observed story drifts, especially in tall buildings. e.g., all connections of the S21 frame are semi-rigid and the story drifts have been increased in comparing with the rigid frames, as seen from the Fig. 8(a). In this research using the rigid connections was inspected to address this problem, as shown in Fig. 11.

In ultimate limit state, the frame behavior was affected by moment capacity of connection more than the connection rigidity, and second parameter didn't have any considerable effect on the frame behavior. The connections with low-strength showed better performance under the seismic loads. By decreasing the strength connection and transferring the plastic hinges from elements to connections, the connections play an important role in absorbing of the lateral displacement. Moreover, the problem of the brittle fracture of welded connections would be obviated. Ductility of connection increased due to more rotatable semi-rigid connection. While, a small amount of the rotation could be leaded to onset of the damage in the rigid connections. Also, by decreasing the number of plastic hinges in beams and columns, ductility demand and probable local buckling would be decreased.

By checking the different arrangement of rigid and semi-rigid connections, it is shown that the model of dual frames behaves generally better than the rigid frame. Also, there is a state of semi-rigidity and connections configuration which behaves better than the rigid frames due to internal forces, story drifts and plastic rotation of elements.

By assuming the rigid connections in lower stories and the exterior spans, the frame behavior would be improved. Using the joints with fully rigid connections in lower stories or upper stories has undesirable effects on the frame behavior.

The appropriate distribution of the rigid and semi-rigid connections, in all over the frame, leads to better behavior.

In tall dual frames, in order to satisfy the story drift limitation, an increase in beam and column sizes is required. In these cases, in order to judge about the construction cost of the rigid and dual frames, it is needed to compare the cost of the increasing in the sizes of the elements and decreasing in the cost due to using the semi-rigid connections. Also, in order to find the appropriate places for the semi-rigid connections, an optimizing method is required.

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Nomenclature

N and N_p	force and plastic strength capacity of the column
λ	slenderness factor
L_{sb}	standard beam length
f_{yw}	yield tensile strength of flange
t_w	thicknesses of web
f_{yf}	yield tensile strength of web
t_f	thicknesses of flanges
M_p	plastic moment capacity of beam
B	half-width of flange
E_b	elasticity modulus of beam
Rav	Rotation capacity
θ_u	Plastic rotation capacity