STUDY THE INFLUENCE of NEAR-FAULT EARTHQUAKE on MOMENT-RESISTANT FRAMES with SEMI-RIGID CONNECTIONS

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ABSTRACT :

While fragile fractures have been spotted in welded connections due to recent earthquakes, semi-rigid connections have ductile behavior and high energy absorption. The purpose of this paper is to study the behavior of semi-rigid frames under near-field earthquake records. Three moment resisting frames of 3, 6 and 12 stories were designed according to UBC97 provisions. The design was carried out for rigid and semi-rigid connections with and without considering the near-field coefficients. For semi-rigid frames, the connections were designed as top and seat bolted angle connections with double web angles (TSAW). Near-field and far-field earthquake records were applied to the rigid and semi-rigid frames and the results of nonlinear dynamic analysis showed that in the case of rigid frames, beams and even columns yielded under near-field records and the connections fractured. But in the case of semi-rigid frames, connections have less strength compared to those of columns and beams. The connections absorbed the earthquake energy by proper rotations and in this manner, stress in beams and columns decreased. It was concluded that the use of semi-rigid frames with slight strength designed by considering the near-field coefficients can be suitable for near-field regions. It should be noted that the lateral drift should be controlled in these frames.

KEYWORDS: semi-rigid Frame, Semi-rigid Connections, Near-field, Earthquake

1. INTRODUCTION

Near-field earthquakes apply a large amount of energy to structures in a short period of time. In other words, they contain low frequency impulsive motion in the velocity time history. Impulsive characteristics of near-field earthquakes cause high rotary ductility to be needed in some stories and connections. During 1994 Northridge and 1995 Kobe earthquakes, fragile failure of connections, sudden destruction of buildings and soft storey failure were observed. Since then, to prevent the failure of rigid connections, several new connections are proposed to improve the behavior of moment resisting steel frames in high-seismic regions. One of these connections is the connections with high-strength bolts. Many of bolted connections which are commonly called semi-rigid connections are much more ductile than corresponding welded connections.

In seismic design of structures, rigid welded connections are used in moment resisting frames. The majority of economical bolted connections are not used, since they have high flexibility compared to rigid welded connections and large deformations will occur in the frame. This design method is for static loading. In dynamic loading the response of structures may be completely different. A structure with semi-rigid connections have higher vibration period and this will change the amount of energy absorbed by the structure and will increase the damping of the structure. Shake table experimental tests and numerical analysis confirm that during an earthquake, the drift of a moment resisting structure with semi-rigid connections is not essentially higher than those of a moment resisting frame with rigid connections. Same results were obtained by studying the behavior of semi-rigid composite connections with bolted angle and concrete slab of roof.

It should be noted that the changes in ground motion acceleration will affect the stiffness of structures and behavior of its connections. Shen et al. studied some frames with different connections and heights under foundation excitation. They concluded that semi-rigid frames have higher drift than rigid frames under weak and moderate excitations, but for strong excitations, the results were not clear. Semi-rigid connections have high ductility and stable hysteresis loops and they do not need expensive site welding. Moreover, considering the problems of rigid connections in moment resisting frames under near-field earthquakes, the use of semi-rigid connections in moment resisting frames may improve the structure behavior.

2. LITERATURE REVIEW

Many experimental and numerical studies have been carried out to study the behavior of seat angle connections. These investigations confirmed that nonlinear behavior of seat angle connections with large deformations can be studied by finite element method considering the pretension force in bolts and slippage effects. Furthermore, the applicability of power model with three parameters is acceptable to estimate $M-\theta$ curve of connections and can be used for nonlinear analysis of steel frames with semi-rigid connections.

In recent studies, simple mathematical methods are proposed to obtain moment-rotation curves by curve fitting with experimental results. Three-parameter power model is proposed by Richard et al. and is used by Kishi et al. to predict moment-rotation behavior of connections under uniform loading. This model is presented by equation 1.

$$M = \frac{K_o \theta}{\left[1 + \left(\frac{\theta}{\theta_o}\right)^n \right]^{\frac{1}{n}}} \tag{1}$$

Where K_0 is initial stiffness of connection, n is shape parameter, $\theta_0 = \frac{M_u}{K_0}$ is plastic rotation and M_u is ultimate moment capacity. According to the above equation, M-θ stiffness of two connection types, top and seat bolted angle connection (DWA) and top and seat bolted angle connection with double web angle (TSWA) is shown in figure 1a. For simulation of inelastic behavior of connections under cyclic loading, independent hardening model can be used which is shown in figure 1b.

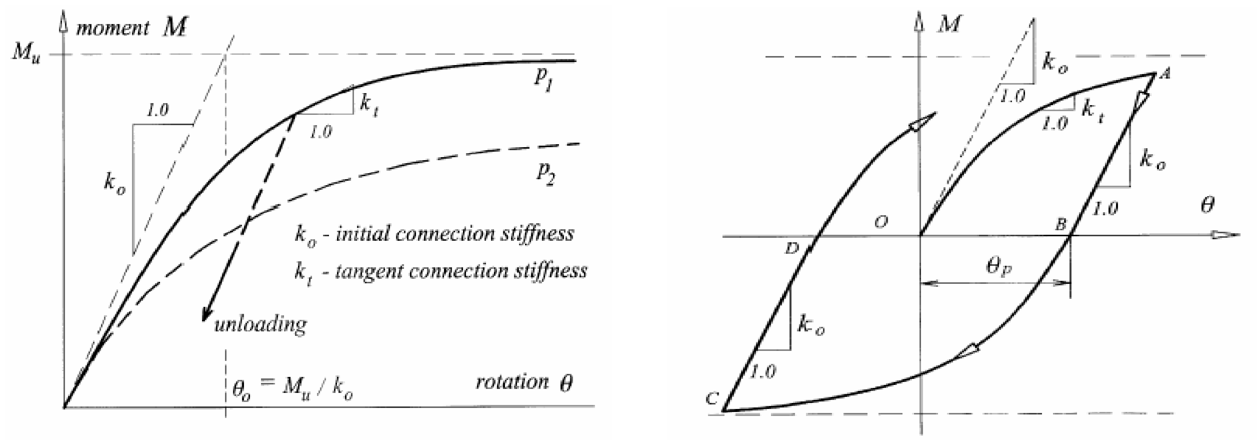


Figure1: a) Three-parameter power model, b) Independent hardening model

3. SPECIMEN MODELING

In this study, three frames of 3, 6 and 12 stories were modeled. The structures assumed to be residential and have common moment-resisting system. The frames were designed in two methods, one by assuming the connections to be semi-rigid and in second method the connections were assumed to be rigid. Moreover, very frame was designed in two methods, by considering the near-field coefficients and also by common design method. An UBC97 provision is used for design in which the near-field condition is simulated by some coefficients.

The height of stories is assumed to be 3.2 meters. Three and six-storey frames have 3 spans and 12-storey frames have two spans. All of the frames were assumed to be placed in high-seismic region on hard soil. Uniform dead load of beams for stories and the roof are 2000 and 1800 kg/m respectively. Live load is assumed to be 600 and 450 kg/m for stories and the roof respectively.

ETABS 2000 software is used for design of the frames. UBC97-ASD provision is used in this software for design. For considering the near-field region effect, UBC97 coefficients are used as 1.2 for N_a and 1.6 for N_v. IPB profiles are used in all of the frames.

In design of semi-rigid frames, the stiffness of beams at two ends of them is decreased to consider the stiffness of semi-rigid connections. The ratio of connection-beam stiffness is assumed to be 0.67 for all of the frames which is shown by parameter r and can be calculated by equation 2.

$$r = \frac{1}{1 + \dfrac{3EI}{K_0 L}} \tag{2}$$

where E, I, L, and k_0 are the elasticity modulus, moment of inertia, beam length and initial stiffness of the connection respectively. The vibration periods of rigid and semi-rigid frames are tabulated in table 1.

Table 1: The periods of rigid and semi-rigid frames

	Frame designed for far-field earthquake			Frame designed for near-field earthquake		
	3-Storey	6-Storey	12-Storey	3-Storey	6-Storey	12-Storey
Rigid	0.68	1.11	1.99	0.66	0.91	1.48
Semi-rigid	0.719	1.24	2.08	0.67	0.998	1.85

Ram-Perform software is used to model the frames and study nonlinear dynamic behavior of structures under earthquake records. This software is effective and applicable for performance based design of structures with nonlinear methods. Beams, columns and semi-rigid connections were modeled by introducing moment rotation curve and were designed for different design levels of immediate occupancy, life safety and collapse prevention according to seismic retrofit provisions.

3.1 Semi-rigid connection design

In this study, top and seat bolted angle connections with double web angles were used as semi-rigid connection since this kind of connection has good flexibility and high energy absorption capability. Three-parameter power constitutive model (Richards and Abbott) were used which can be presented by equation 1. In this constitutive model, there are three unknowns which are the initial stiffness (K), ultimate moment (M) and shape coefficient (n). The expected initial stiffness of connection is known. These three unknowns can be calculated by substituting the beam dimensions, web angle dimensions, flange angle dimensions, diameter and grade of bolts in formulations proposed by Kishi-Chen. The calculated value of connection initial stiffness can be checked by expected initial stiffness. Finally, this stiffness will reach to desired stiffness by changing the dimensions of angles. The properties of designed connections are summarized in table 2 and the moment-rotation curves for these connections are shown in figure 2.

Table 2: Properties of the designed TSAW connections

Connection name	Beam section	Angle dimensions	Length (mm)	Angle dimensions	Length (mm)
ConB18	IPB180	L 100*200*12	200	L 80*80*10	140
ConB20	IPB200	L 100*200*12	220	L 80*80*10	140
ConB22	IPB220	L 100*200*14	240	L 80*80*10	160
ConB24	IPB240	L 100*200*14	260	L 100*100*12	180
ConBv22	IPBv 220	L 100*200*16	280	L 100*100*14	160
ConBv24	IPBv 240	L 100*200*20	300	L 100*100*14	200

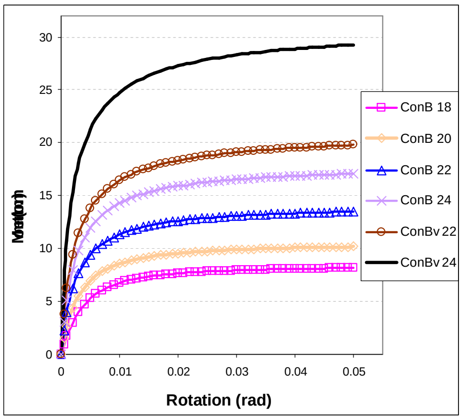


Figure 2: moment-rotation curve for TSAW connections

3.2. Selection of earthquake records

Extensive range of records were selected to investigate seismic behavior of structures and earthquake records characteristics like energy content, peak acceleration and duration of earthquake are considered for the selection. Furthermore, near-field records were selected considering a/v ratio, acceleration and velocity pulse, number of pulses, acceleration spectra shape and particularly the velocity spectra. The selected records were normalized using energy method and area under acceleration curve method. The properties of far-field records and near-field records are tabulated in tables 3 and 4 respectively. The spectra of selected records are compared to the UBC97 spectra and shown in figure 3 and 4.

Table 3: Properties of far-field records

earthquake	summary	Distance (km)	PGA (g)	PGV (cm/s)	a / V
Duzce, Turkey 1999	Duzce	134.9	.0177	1.853	0.95
Kocaeli, Turkey 1999	Kocaeli	78.9	0.256	26.26	0.97
Landers 1992	Lander	151.1	.033	4.76	0.69
Northridge 1994	Nor-far	84.2	0.113	6.99	1.61

Table 4: Properties of near-field records

earthquake	summary	Distance (km)	PGA (g)	PGV (cm/s)	a / V
Northridge 1994	Newhall	7.1	0.317	93.49	0.34
Erzincan, Turkey 1992	Erzincan	2	.442	65.29	0.67
Superstitn Hills(B) 1987	Super	0.7	0.539	86.95	0.62
N. Palm Springs 1986	Palm	8.2	0.599	61.55	0.97

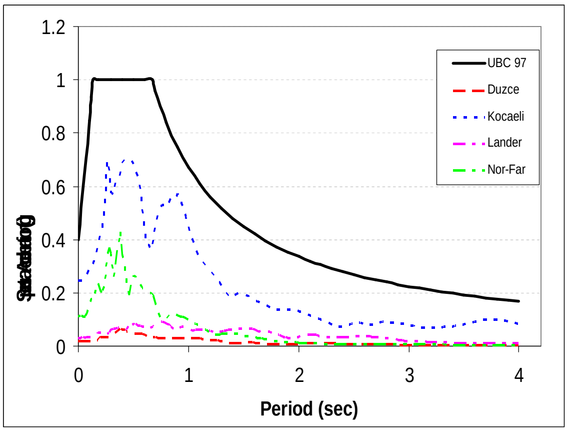


Figure 3: Far-Field earthquake acceleration spectra

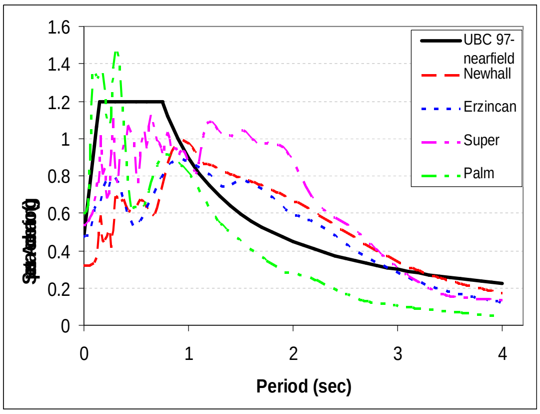


Figure 4: Near-Field earthquake acceleration spectra

4. RESULTS OF NONLINEAR DYNAMIC ANALYSIS

4.1 Comparison between rigid and semi-rigid frames designed for far-field earthquakes under far-field earthquakes

The frames designed for far-field earthquakes are actually the structures which have not been designed for near-field earthquakes. The place of occurrence of plastic hinges in short rigid frames are at beams and bottom of ground level columns. In semi-rigid frames, plastic hinges occur at semi-rigid connections, since these connections has less stiffness compared to columns and beams and absorb most of earthquake energy. A comparison between the shear envelopes of rigid and semi-rigid frames showed that semi-rigid frames have always fewer shears than those of rigid frames. In 3 and 6-storey frames, this decrease exists in all of the structure stories. In 12-storey frames, the shear is decreased in all of the stories except the three highest stories.

Using semi-rigid connections will decrease the structure strength and also decrease the absorbed forces by structure. The drift envelope shows that the drift of lowest stories of semi-rigid frame is decreased compared to those of rigid frames but the drift of highest stories in semi-rigid frames is more than those of rigid frames. Changing the frame system from rigid to semi-rigid will transfer the energy absorption from lower stories to higher stories. Moreover, semi-rigid frames in highest stories have more deformations compared to those of rigid frames.

The maximum ratio of beam and column rotation to yield rotation and maximum rotations of connections to yield rotation is calculated for semi-rigid frames under far-field earthquakes. The results showed that high ductility is required for lower stories in rigid 3 and 6-storey frames compared to those of higher stories. In 12-storey rigid frame the need for ductility is high in beams and columns of intermediate stories. In semi-rigid frames, high ductility is required in connections and this ductility is distributed in majority of stories.

4.2 Comparison between rigid and semi-rigid frames designed for far-field earthquakes under near-field earthquakes

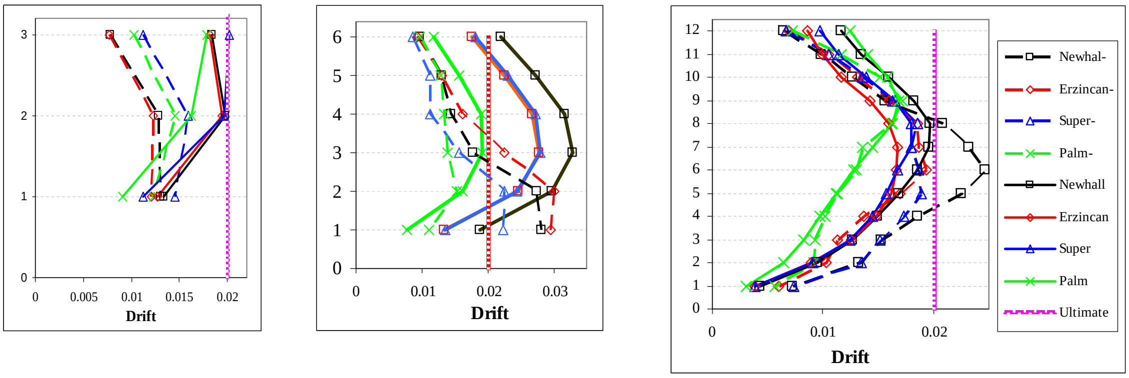
Plastic hinges were occurred in beams of lower stories and bottom of ground floor columns in 3 and 6-storey rigid frames. In 12-storey rigid frame, the plastic hinges were placed at beam of intermediate stories and the structural members in these locations have passed the IO limit state. In contrast, columns of 3, 6 and 12-storey semi-rigid frames did not reach to IO limit state except in the case 6-storey semi-rigid frames under Newhall earthquake record. The connections of semi-rigid frames have passed IO limit state and even in 6-storey semi-rigid frame they have passed the LS limit state.

A comparison between the shear envelope of rigid and semi-rigid frame stories under near-field earthquake records showed that semi-rigid frames have generally lower or equal base shear to rigid frames. The drift envelope showed that for semi-rigid frames, the drift of lower stories is decreased but the drift of intermediate and higher stories are increased compared to those of rigid frames. In 3-storey frame the drift of ground floor is decreased by 10% but the drift of second storey is increased by 100%. In the case of 6-storey frame, the drift of ground floor is decreased by 40% but the drift of third to last stories is increased by 75 to 90%. In 12-storey frame, the drift of ground floor is decreased by 42%, the drift of sixth storey is decreased by 6% and the drift of last storey is increased by 55%.

In 6-storey rigid frame, the drifts of ground and first stories is very high, but in the 6-storey semi-rigid frame the drift of intermediate stories is high and the drift of both rigid and semi-rigid frames have passed the ultimate limit (2% drift), so it can not be said that the drift led to the failure of structure when the system is changed from rigid to semi-rigid frame. A comparison is made between the drift of rigid and semi-rigid frames in figure 5. The solid lines are corresponding to semi-rigid frame and dashed lines are corresponding to rigid frames. Comparison between the required stiffness of members showed that ground and first storey beams and also the bottom of ground floor columns in 3 and 6-storey rigid frames are high. In 12-storey rigid frame the beams in intermediate stories should provide medium ductility. In contrast, in semi-rigid frames connections should provide high ductility and this is possible by using top and seat bolted angle connections with double web angles which can provide high rotations. Also the connections will have large rotations, there will be no damage in the columns and the structure will survive. In semi-rigid frames, the required ductility is distributed in connections of all stories but in rigid frames, the required ductility is concentrated in beams and columns of some specific stories. The maximum ratios of rotation to yield rotation for the connections of 6-storey semi-rigid frame are tabulated in table 5. These values are presented in table 6 for rigid 6-storey frame under near-field earthquake.

Table 5: Maximum values of $\theta y / \theta$ for connections of 6-storey semi-rigid frame under near-field records

story	Θy	Newhall	Erzincan	Super	Palm
6	0.004	4.45	3.375	3.5	1.825
5	0.004	5.475	4.375	4.5	2.875
4	0.004	6.475	5.25	5.425	3.575
3	0.004	7.35	6.15	6.175	4.175
2	0.004	7.225	6	5.95	3.85
1	0.004	5.85	4.55	4.55	2.75



3-storey 6-storey 9-storey

Figure 5: Comparison between the drifts of 3, 6 and 12-storey rigid and semi-rigid frames under near-field earthquake records

Table 6: Maximum values of θy / θ for columns and beams of 6-storey semi-rigid frame under near-field earthquake records

	Rigid frame columns					Semi-rigid beams				
story	Θy	Newhall	Erzincan	Super	Palm	Θy	Newhall	Erzincan	Super	Palm
6	0.0052	0.06	0.54	0.42	0.46	0.0088	0.84	0.82	0.80	0.85
5	0.0052	0.33	0.54	0.48	0.48	0.0088	1.58	1.67	1.39	1.59
4	0.0052	0.48	0.79	0.69	0.67	0.0073	1.92	2.05	1.56	1.84
3	0.0045	1.13	1.02	0.82	0.70	0.0066	2.27	2.88	1.85	1.95
2	0.0045	1.49	1.07	0.87	0.77	0.0066	3.03	4.17	2.97	2.24
1	0.0045	5.64	5.96	4.16	1.67	0.0073	4.21	4.75	3.66	2.22

4.3 Comparison between rigid and semi-rigid frames designed for near-field earthquakes under near-field earthquakes

A comparison between IO and LS performance level of rigid and semi-rigid frames showed that in rigid frames, beams and columns have passed IO level but not reached LS limit state and even the columns has more value than beams in IO state. In contrast, in semi-rigid frames, columns did not reached IO limit stated and the connections always passed IO state by absorbing earthquake energy but did not reach to LS state. A comparison between shear envelope of stories in rigid and semi-rigid frames under near-field records showed that shear value is always fewer in semi-rigid frames in all of the stories. This maximum amount of decrease occurred at 12-storey building (40 to 50 %).
The drifts of bottom stories in semi-rigid frames are lower than those of rigid frames, but in higher stories, the drift of semi-rigid frames are higher. Moreover, semi-rigid frames have more deformations than rigid frames.
The maximum required ductility of rigid frames is concentrated at beams and columns of lowest stories which are several times greater than those of top stories. In semi-rigid frames, the required ductility is distributed among all of the stories and the capacity of all stories is used to resist near-field earthquake.

5. CONCLUSION

A comparison between semi-rigid and rigid frames under far-field earthquakes showed that changing the system from rigid to semi-rigid transfers the energy absorption from bottom stories to intermediate and top stories.

Furthermore, the drift of bottom stories in semi-rigid frames is less than those of rigid frames, but the drift at top stories in semi-rigid frames is more and semi-rigid frames has less base shear compared to base shear of rigid frames.

In the case of near-field earthquakes, the energy absorption is concentrated in specific stories and plastic hinges occur at the bottom of ground floor columns, ends of beams and rigid connections. In contrast, in semi-rigid frames the columns do not participate in energy absorbing and large rotation in semi-rigid connections will absorb the earthquake energy. In this case, the energy absorption is not limited to some specific stories. Consequently the behavior of frames is proper and ductile fracture will occur. However, the drift of structure is increased in this condition.

Study of behavior of semi-rigid frames designed for near-field earthquakes showed that most of structure stories participate in energy absorption. The use of near-field coefficients for increasing the base shear of structures mentioned in UBC97 provision is verified to be proper for the frames of this study and these base shear increase has significant effect in the case of 6-story frames. However, the behavior of structure is not assured to be proper just by applying base shear coefficient. Consequently, using semi-rigid connections with slight strength which are designed for near-field earthquakes can be proper, if the lateral drift be controlled.

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