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Research Article

SEISMIC BEHAVIOR EVALUATION OF STEEL MOMENT-RESISTING FRAMES WITH T-STUB CONNECTIONS UNDER EARTHQUAKE RECORDS

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ABSTRACT
Previous conventional steel frame designed to consider the beam-to-column connections in steel frames as rigid or pinned, because design of this type of connection was easy. In general, the connection that welds the beam directly to the flange of the column is considered to be fully rigid. However, the connection that fastens the beam to the column with some angles and/or a plate, bolts or rivets, displays a nonlinear behavior and lies somewhere between the fully rigid and perfectly pinned conditions. Semi-rigid connections behavior can be shaped and have a good ability to absorb energy. In this paper considered four steel moment frames 3, 6, 9 and 12 floors with rigid and T-stub semi-rigid connection and UBC 97 Regulations for analyzing. A typical T-stub connection cut from a 'W' section to the required dimensions was used. Behavior of semi-rigid connections with the M-θ curve in design of steel moment frames used. ETABS software was used for the initial analysis of frames. PERFORM 3D software used for modeling and analyzing of nonlinear dynamical behavior of structures under earthquake records. Non-linear dynamic analysis results showed in steel rigid frame, more beams and columns have surrendered, while the in steel semi-rigid frame stresses in beams and columns reduced. This is due to the entering of semi-rigid connections the non-linear range. Hence, if we control displacement of the semi-rigid frame, the connection looks well and the frame stable against lateral loading.

Keywords: *T-Stub Semi- Rigid Connection, Steel Moment Frame, Time History, Perform 3D, Plastic Joint*

INTRODUCTION
Vulnerability of welded moment connections in steel moment-resisting frames subject to severe cyclic loading was demonstrated during the 1994 Northridge Earthquake. Low ductility in welded rigid connection area was one of major reasons of this destruction. Since then, numerous alternative connections have been proposed for the retrofit and the new design of steel moment frames in high seismicity areas. Among the proposed connections are those with high-strength bolts. The issues in the bolted connections as compared to the welded connections are related to the stiffness, complex behavior, ductility, analytical model, as well as construction cost. Many bolted connections, often called semi-rigid connections, are considered much more flexible than their welded counterparts. This causes some concerns about the overall stiffness of moment-resisting frames with bolted beam-to-column connections. The inelastic behavior of a bolted connection is intrinsically more complex than that of a welded connection simply because more components, such as bolts, angles and plates, are introduced into the congested connection zone. Thus, a bolted connection tends to behave in a complicated manner with a variety of failure modes. Nevertheless, if designed properly, the bolted connection may have high ductility and cyclic energy dissipation capacity since it eliminates the brittle failure that was observed in the welded connection (Shen, 2000).

T-Stub Bolt Connection Experiments and Background
Nair *et al.*, (1974) conducted 16 T-stub tests under cyclic and fatigue loading. The effects of bolt load-deformation characteristics, T-stub geometry, and the other connection detailing geometric variables were investigated. It was concluded that the prying forces, which result from the flange deformations, cause substantial reduction in ultimate load capacity and fatigue strength. Empirical equations predicting the ultimate strength of T-stub connections were also presented (Nair *et al.*, 1974). Leon & Swanson in 2000,

during six specimen laboratorial showed action of prying T-Stub under cyclic loading and the change in geometric specification and mechanical joints in M-θ curve with complete curve (Swanson, 2001). Sridhar in 2004, did two specimen perfect testing on T-Stub connection in Cincinnati University. He put this connection in effect of combining moment and shear in order to reach M-θ curve of samples and compared his result with Leon & Swanson (Sadasivan, 2004).

Geometric Characteristics of T-Stub Connections

A typical T-stub is generally cut from a 'W' section to the required dimensions. A T-stub beam-to-column connection has two T-stubs that are used to transfer moment. In addition to this, shear tabs or double web angles are used to transfer pure shear on to the column. A schematic of the connection when the top T–Stub is subjected to tension and the bottom T-Stub to compression is shown in Figure 1. This implies that the bolts on the column side of the connection are subjected to alternate tensile and compressive forces.

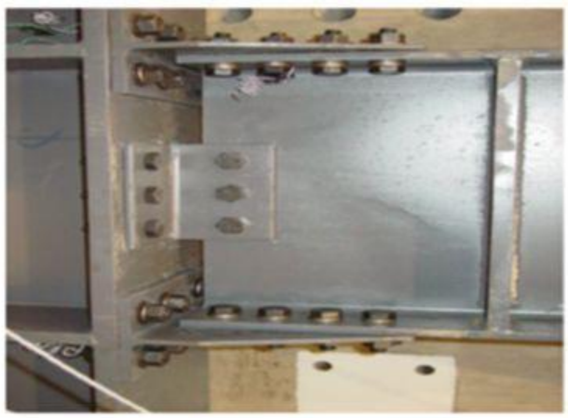

Figure 1: Typical T-Stub Connection (Sadasivan 2004)

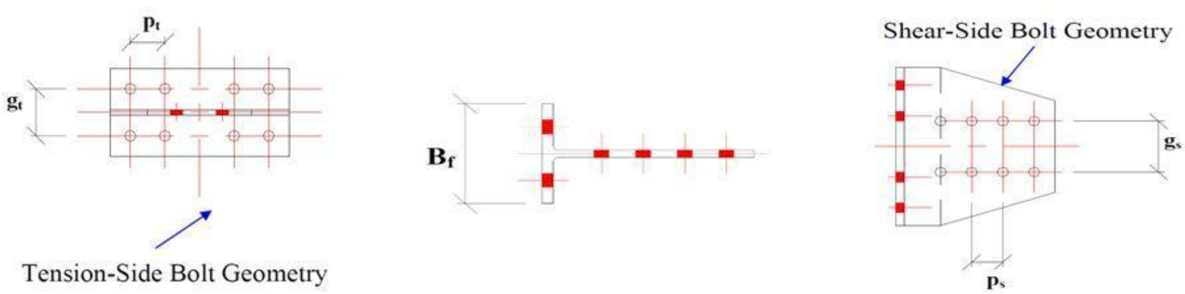

Figure 2: Typical T-STUB Geometry (Sadasivan 2004)

Three-Parameter Power Model for Semi-Rigid Connection

The first step a using of mathematical phrase for define M-θ curve came back to 1936, and then in 1998 Kishi & Chen represented a practical method for using in designing structure with top and seat double-angle semi-rigid connection. In recent studying simple mathematic method are created for reaching M-θ curve instead experimental result.

Three sample mathematic model suggested by recent researchers were (Power series 1986, Exponential 1987, Ramberg-Osgood equation 1943). Power series equation for first time suggested by Richard & Abbott 1975 and a generalized form of the equation suggested by Kishi & Chen (1986), as follows, (chen, 2000).

That M_{po} = connection moment, θ=connection rotation, K_e = initial Stiffness, M_u = ultimate moment, and n = shape parameter.

$$M_{Po} = \frac{K_e \theta}{\left[1 + \left(\frac{K_e \theta}{M_u}\right)^n\right]^{\frac{1}{n}}} \quad (1)$$

Thus, the three parameters characterizing this model are K_e, M_u and n. For different values of the shape parameter, n, the moment-rotation curves for this model are shown in Figure 3. As is evident from the curves shown, a high value of n will result in a steeper curve, and in the limit (i.e., when $n \rightarrow \infty$) it represents an elastic-perfectly plastic curve.

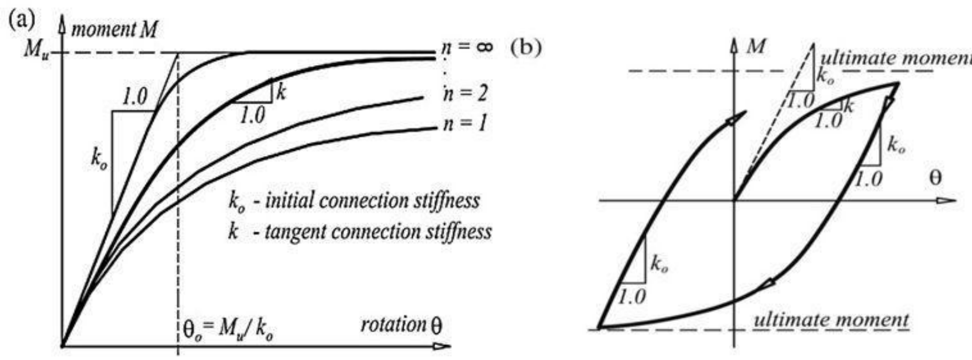

Figure 3: Connection modeling. (a) Richard–Abbott model, (b) independent hardening model (Chen, 2000)

The prediction equations obtained from Mr. Sridhar for the dependent variables are presented below (Sadasivan, 2004). Three specify parameter generalized Power series equation Richard & Abbott, as follow:

$$M_u = 838.35\left(t_f{}^2 t_w f_y\right)^{0.24}\left(\frac{g_s}{p_s}\right)^{0.28}\left(\frac{B_f}{D_{beam}}\right)^{0.012}\left(t_{fb}\, b_{fb}\, D_{beam}\right)^{0.33} \quad (2)$$

$$K_e = 119713.89\left(t_f{}^2 t_w f_y\right)^{0.25}\left(\frac{g_s}{p_s}\right)^{0.84}\left(\frac{B_f}{D_{beam}}\right)^{-0.16}\left(t_{fb}\, b_{fb}\, D_{beam}\right)^{0.72}\left(n_s\, d_b\right)^{-0.65} \quad (3)$$

$$n = 0.099\left(t_f{}^2 t_w F_y\right)^{-0.28}\left(\frac{g_s}{p_s}\right)^{0.69}\left(\frac{B_f}{D_{beam}}\right)^{-0.16}\left(n_s\, d_b\, F_y\right)^{1.48}\left(\frac{M_u}{K_e}\right)^{0.99} \quad (4)$$

B_f: Breadth of flange of the T-Stub
d_b: Bolt diameter
h_b: Bolt hole diameter
n_t: Number of bolts on the tension side
g_t: Spacing between rows of bolts on tension side
p_t: Center-to-center spacing of bolts along a row on tension side
n_s: Number of bolts on the shear side
g_s: Spacing between rows of bolts on shear side
p_s: Center-to-center spacing of bolts along a row on shear side

Investigated Structure

The design procedure was based on the AISC seismic provisions for structural steel buildings (AISC, 2005), minimum design loads for buildings and other structures: SEI/ASCE 7-05 (ASCE, 2005). In this paper, steel moment frames is modeled with 3, 6, 9, 12 stories as well as a bay length of 4 m were designed. The frames designed in two complete state with full rigidly and semi-rigid connection. A 2D four bay frames and the height of every model structure was fixed at 3.2 m. This structure is in seismically area and type of soil is SC. In all models a uniform dead load 750 kg/m^2 and live load 200

kg/m^2 are assumed. Behavior of semi-rigid connection analysis is showed that these connections are commonly weakness under near the fault records. Hence, because of the need to impose condition of structure in near fault zone, some of criteria were considered that in UBC97 regulation, establish these circumstances with N_V and N_a coefficient is possible (UBC, 1997). Where I=1 (seismic important factor) and C_a, C_v is the seismic coefficients factor and N_a, N_v is Near-Source Factor. Lateral loading systems frame is ordinary moment frame and according to UBC97 regulation R=4.5 (response modification factor). For Initial designing frames with semi-rigid connection, stiffness of connection was equal to the stiffness of beam.

This according to FEMA 335 C, connections stiffness (k_e) are assumed equal ($\frac{6EI}{L}$) (FEMA 355C, 2000). For nonlinear analysis of structures after scaling of earthquake pair records, mentioned structures was evaluated with nonlinear time history analysis method with PERFORM 3D software according to FEMA 356 prestandard (FEMA 356, 2000)

Time History Analysis

For evaluating frames used the nonlinear time history analysis. To investigate the seismic behavior of structures under earthquake records were used the earthquake of Kobe 1995 and the Northridge 1994 and Loma Prieta 1989. The earthquake records are presented in the following table as shown in table 1.

Table 1: Earthquake records used in the structure dynamic analyses

Ground motion record (year)	Station	Magnitude	Peak ground acceleration (g)
Loma Prieta 1989	16 LGPC	6.9	0.605
Northridge (1994)	90014 Beverly Hills - 12520 Mulhol	6.7	0.617
Kobe 1995	Takarazuka	6.9	0.694

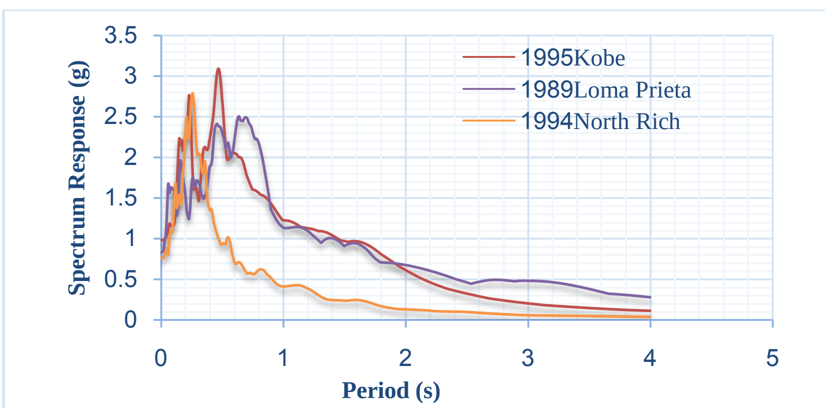

Figure 4: Response Spectra of Loma Prieta, Kobe and Northridge

Design of Model Structures and Analysis Modelling

Semi Rigid Connection

Semi-rigid connection have different kinds, in this paper use T-Stub connection because this connection have good flexibly and suitable absobtion energy. M-θ curve plot for all connection and the first stiffness checked with expected stiffness in all designing. For connection modeling used semi-rigid moment connection that M-θ curve connection is given by a third line curve to perform 3D software see figure (5). Also, assumed T-stubs failed by net section fracture of the T-Stub stem (Sadasivan, 2004), Semi-rigid model parameter in software displayed in table (2).

Table 2: Modeling Parameters and Acceptance Criteria for T-Stub connection (FEMA 356, 2000)

Component/Action	Modeling Parameters			Acceptance Criteria				
	Plastic Rotation Angle, Radians		Residual Strength Ratio	Plastic Rotation Angle, Radians				
					Primary		Secondary	
	a	b	c	IO	LS	CP	LS	CP
Partially Restrained Moment Connections								
c. Tension failure of spilt tee stem (Limit State 3)	0.012	0.018	0.800	0.003	0.008	0.010	0.010	0.015

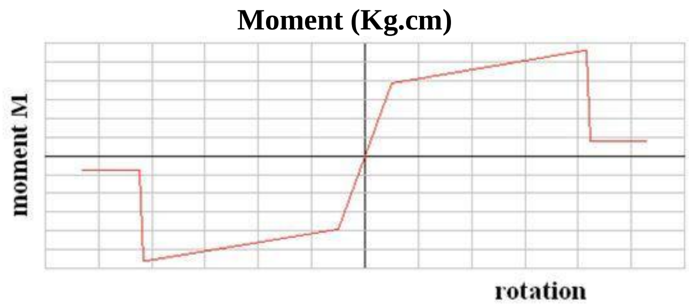

Figure 5: Third line semi-rigid connection curve in software (Manual Ram Perform 3D, 2000)

Beam and Column

In lieu of relationships derived from experiment or analysis, the generalized load-deformation curve shown in Figure 6, with parameters a, b, c, as defined in Tables 2 and 3, were used for components of steel moment frames.

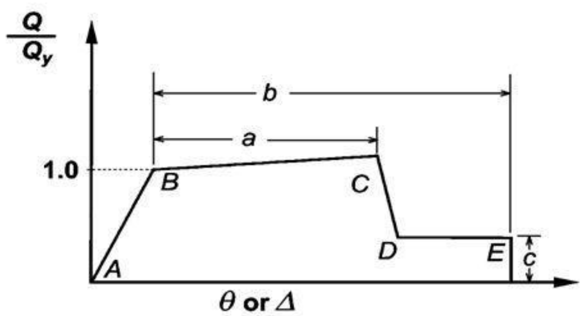

Figure 6: Force–deformation relationships of structural members (FEMA 356, 2000)

Table 2: Modeling Parameters and Acceptance Criteria for Beam (FEMA 356, 2000)

	Modeling Parameters			Acceptance Criteria				
	Plastic Rotation Angle, Radians		Residual Strength Ratio	Plastic Rotation Angle, Radians				
					Primary		Secondary	
Component/Action	a	b	c	IO	LS	CP	LS	CP
Beams—flexure								
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{418}{\sqrt{F_{ye}}}$	$9\theta_y$	$11\theta_y$	0.6	$1\theta_y$	$6\theta_y$	$8\theta_y$	$9\theta_y$	$11\theta_y$

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Modification of this curve shall be permitted to account for strain-hardening of components as follows: (a) a strain-hardening slope of 3% of the elastic slope shall be permitted for beams and columns unless a greater strain-hardening slope is justified by test data; and (b) where panel zone yielding occurs, a strain-hardening slope of 6% shall be used for the panel zone unless a greater strain-hardening slope is justified by test data, table (2), (3). Here, the nonlinear dynamic time history analysis were conducted by considering the behavior of members in life safety structural performance level as suggested by the Federal Emergency Management Agency (FEMA 356, 2000).

Table 3: Modeling Parameters and Acceptance Criteria for Column (FEMA 356, 2000)

	Modeling Parameters			Acceptance Criteria				
	Plastic Rotation Angle, Radians		Residual Strength Ratio	Plastic Rotation Angle, Radians				
					Primary		Secondary	
Component/Action	a	b	c	IO	LS	CP	LS	CP
Columns—flexure [2, 7]								
For $P/P_{CL} < 0.20$								
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{300}{\sqrt{F_{ye}}}$	$9\theta_y$	$11\theta_y$	0.6	$1\theta_y$	$6\theta_y$	$8\theta_y$	$9\theta_y$	$11\theta_y$

RESULTS AND DISCUSSION

Result of Nonlinear Dynamic Analysis

Comparison of frames performance levels shown, that in 3 and 6 story frames, the performance of plastic joints of structures with T-Stub semi-rigid connections is more critical than the performance of plastic joints of structure with fully rigid connections. Also Comparison of T-Stub connections shown that the frames with T-Stub semi-rigid connections have more appropriate performance than other rigid connections. Important point in the 9 and 12 story frames with T-Stub semi-rigid connections is desired response of the structures columns due to depreciate a lot of energy with beams. So, less number of the plastic joints in the columns for T-Stub connection frames did pass from (LS) Limit. Because the T-Stub semi rigid connections depreciate a lot of energy. Semi-rigid frames with this connection have a better answer than the rigid frames. Also, the T-Stub connections prevented the concentration energy in specific stories by proper distribution of energy in different stories. This shown better performance of frames as follow:

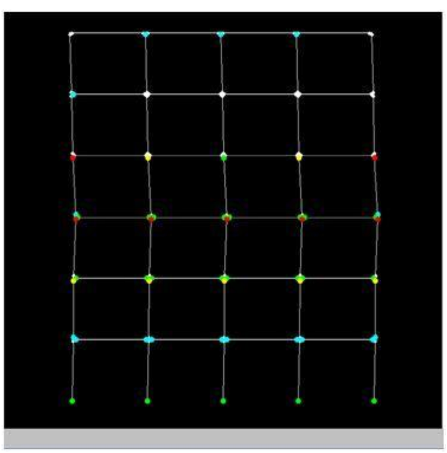

Figure 7: Six story frame with fully rigid connections under kobe earthquake

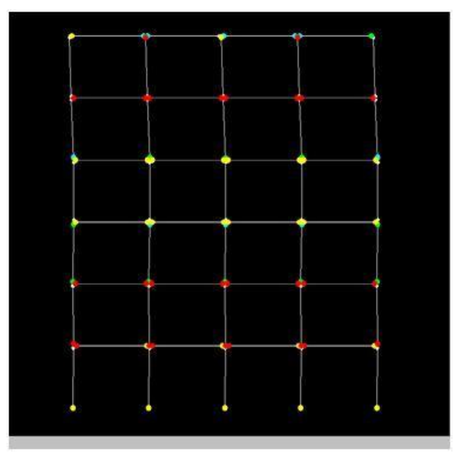

Figure 8: Six story frame with T-STUB semi-rigid connections under kobe earthquake

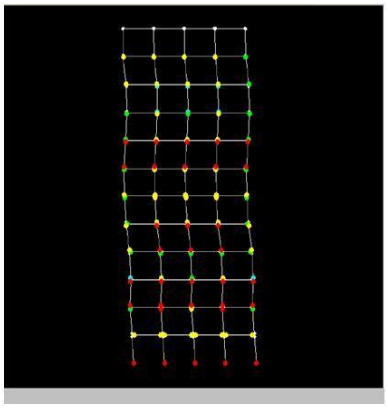

Figure 9: Twelve story frame with fully rigid connections under kobe earthquake

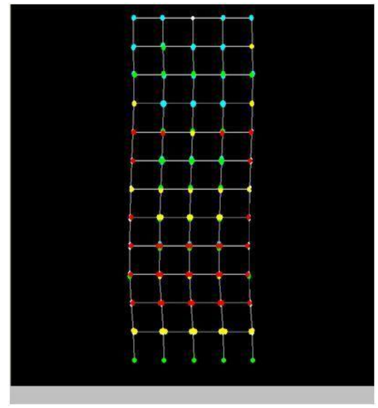

Figure 10: Twelve story frame with T-STUB semi-rigid connections under kobe earthquake

Result of Hysteresis Curves

Hysteresis curve show that frames with rigid connection in low stories the level under hysteresis curve the beams have many quantities and consequently the incoming energy is more depreciate, however when at the frames with low height (3, 6 stories) move toward frames with high height (9, 12 stories), area under hysteresis chart rigid frame beams is reduced and the area under hysteresis diagram frames beams with semi-rigid increased. This shown better performance of frames as follow:

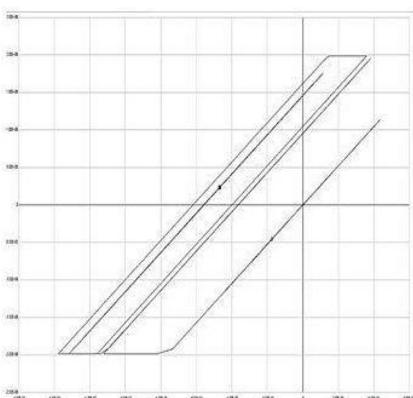

Figure 11: Hysteresis curve of 9 story frame with fully rigid connection

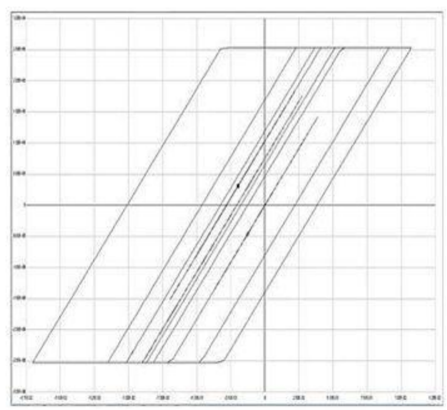

Figure 12: Hysteresis curve of 9 story frame with semi- rigid connection

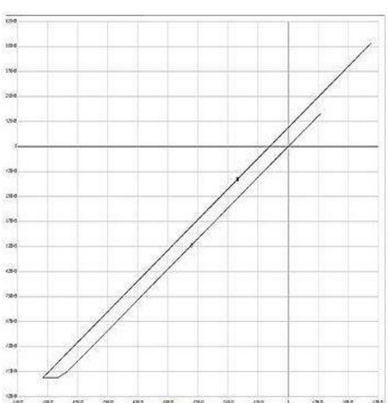

Figure 13: Hysteresis curve of 12 story frame with fully rigid connection

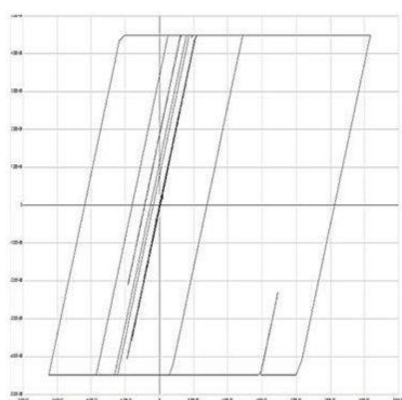

Figure 14: Hysteresis curve of 12 story frame with semi- rigid connection

Conclusions

Comparing rigid and semi-rigid frames show that drift in semi-rigid frames, in comparison with rigid frames decrease in low and Increase in high story. Increasing drift frame is high stories because changing frame system from rigid to semi-rigid system that cause transfer energy absorption from low story to high levels. Comparing semi-rigid and rigid frames shearing push show that frames with semi-rigid connection have low shearing, this is because existence of semi-rigid connection cause energy absorbing and this energy spread in all structure.

In analysis the influence of semi-rigid connections, on the coefficient of behavior, it can be said that with the performance of connection by T-stub semi-rigid model, in general the coefficient of behavior is increases. Also, in analysis the influence of increasing the story on the coefficient of behavior, the result is that how height of structures increases, the coefficient of behavior is Increase. Compare dissipating energy by the various components of rigid and semi-rigid frame under earthquake records showed that energy distributed by the beam in a semi-rigid frame with increased levels will increase and also use of semi-rigid connections was reduced stresses in beams and columns. This is due to the performance of semi-rigid connections and entering the connection of nonlinear area.

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