

Seismic Energy Loss in Semi-rigid Steel Frames Under Near-Field Earthquakes



Vijay Sharma , M. K. Shrimali, S. D. Bharti, and T. K. Datta

Abstract A comparative study is carried out to estimate the seismic energy losses between the semi-rigid steel frames, modeled in two different approaches and rigid frames. For this purpose, three variant of earthquakes is considered, namely, far-field and near-field with forward directivity and fling step effect. These earthquakes are scaled to a peak ground acceleration (PGA) level of 0.4 and 0.6 g. The seismic energy loss is evaluated along with other seismic response parameters. The responses parameters of interest are maximum roof displacement, base shear, the total number of formation of plastic hinges with their square root of the sum of square (SRSS) values of maximum hinge rotations, and the energy dissipation in the form of modal damping and link hysteretic energy. For this numerical simulation study, a five-story rigid frame is designed as per Indian standard provisions as an illustrative problem. A nonlinear response history analysis is performed using the SAP2000 platform to evaluate the desired responses. The results of present work reveal that (i) the seismic energy dissipation significantly more in semi-rigid connected frame with plastic link as compared to elastic link; (ii) the energy dissipation in the form of plastic hinges are substantial in rigid frames as compared to semi-rigid frames with plastic and elastic link, plastic link model provides comparable loss in seismic energy with rigid frames; and (iii) the significance of seismic energy loss depends on earthquakes type, PGA level, degree of semi-rigidity and connection type.

Keywords Semi-rigid · Near-field · Far-field · Energy dissipation

1 Introduction

The efficiency of steel moment frames (SMF) is considerably better as compared to other civil engineering structures, especially in major seismic prone areas due to their strength and high ductility. The efficacy of moment frames is dependent significantly

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on beam-column connections. The beam-column connections are crucial in seismic events due to the cyclic nature of ground motions. Conventionally, the steel beam-column connections are designed as rigid with extremely high stiffness to compel the safety consideration of high stiffness as well as sufficient over-strength irrespective of the economy of construction. The 1994 Northridge and 1995 Kobe earthquake events were significantly damaged the several SMF buildings, and the beam-column welded connections were severely affected. These events diverted the attention of seismic analysts toward rigid to semi-rigid (SR) connections. During the seismic event, the huge amount of stocked energy dissipation is carried out in the form of plastic hinges, mainly in flexural members. Depending upon the stocked seismic energy, the number and rotations of the plastic hinges could be significant, resulting in a high level of damages. The application of SR connections considerably reduces the plastic hinge formation and enhances the performance level (lower down the collapse prevention 'CP' to life safety 'LS' or immediate occupancy 'IO' as described in ASCE 41-17 [1]). Thus, most of the current standards, like Indian, American and European codes incorporated the three types of the beam-column connections, i.e., fully restrained/rigid, partially restrained/ semi-rigid and flexible/ pinned connection [2–4].

Diaz et al. [5] reviewed comprehensively on the development of SR connections and classified the SR connections based on moment-rotation behavior in different categories such as experimental, analytical, numerical, mechanical, and informational. Enhancing the global hysteretic energy can be attained either by improving the hysteretic path of structural components (member and connection) as provided in FEMA 355D [6] or actuating the enhanced number of locations for plastification before the structural collapse. The second measure is implied in the current practices in AISC 341-16 [4] and Eurocode 8 [7]. The seismic performance of SMF with SR connections was investigated by various authors in the past [8, 9]. Aksoylar et al. [10] investigated the hysteretic moment-rotation behavior of low-rise semi-rigid frames. Abolmaali et al. [11] experimentally investigated the energy dissipation in welded/ bolted connections. Sekulovic and Nefovska-Danilovic [12] investigated the seismic performance of the multistory SR frames under different peak ground acceleration (PGA) levels and observed that the earthquake energy is primarily dissipated in beam-column connections and flexural plastic hinges. The efficacy of tuned mass dampers for multi-mode seismic response control and energy dissipation for different structures was investigated by various researchers [13–16]. Recently, Lemonis [17] analyzed the seismic performance of SMF, especially in this regard to energy dissipation in joints and beams.

The previous studies focused on far-field seismic excitations to assess the structural seismic performance. Fewer studies were carried out considering the near-field ground motions, and primarily these were constrained to rigid frames only [18]. The seismic behavior of SR frames and energy dissipation characteristics in these studies are not comprehensively studied for near-field seismic excitations. This paper aims to investigate and compare the seismic behavior of the 5-Story SMF SR frame with rigid frames. The beam-column connections in SR frames are modeled in two ways, namely, the multi-linear elastic link and multi-linear plastic link element. The seismic demand parameters and the energy dissipation are evaluated by a nonlinear

response history analysis (NRHA) for three variety of ground motions, viz., the far-field and the near-field with forward directivity and fling effects. For each earthquake, two PGA levels (design and high level) are considered. The seismic demand parameters included the roof displacement, maximum base shear, formation of a total number of plastic hinges, the SRSS of maximum plastic hinge rotations and the energy dissipation in the form of connection/link hysteretic energy and modal damping energy.

2 Theory

The standard software SAP2000 [19] platform is used to perform the nonlinear response history analysis (NRHA) to compare the responses in rigid and semi-rigid frames. The modelings of frames are explained in subsection with some attentive measures.

2.1 Implementation of Semi-rigid Connection Link Element in SAP2000

The semi-rigid beam-column connections in this study are modeled in two different approaches. The two-jointed zero-length link element is used to represent the two types of SR connections with different hysteretic behaviors. The SR connection with a multi-linear elastic link element (MLE) exhibits the isotropic hysteretic behavior, whereas the kinematic hysteretic behavior is exhibited by SR connections with multi-linear plastic link (MLP) elements (see Fig. 1). The considerable amount of seismic

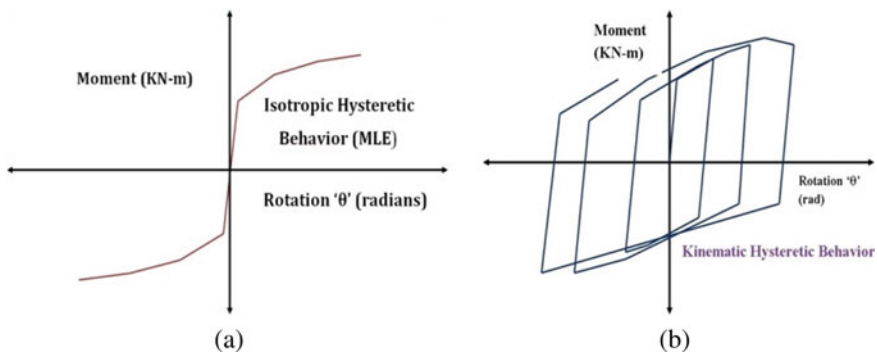


Fig. 1 Semi-Rigid Connection Behavior. **a** Isotropic Hysteretic Behavior for Elastic Link. **b** Kinematic Hysteretic Behavior for Plastic Link

energy is dissipated in cyclic loading in the MLP link element. The MLP shows the kinematic hardening behavior, and it is pertinent for ductility in connections.

Figure 2a explains the typical generic moment-rotation ($M-\theta$) curve adopted for SR connections. The three parameters, namely, stiffness parameter (k), flexural strength parameter (s), and ductility parameter (μ), decide the shape and values of the $M-\theta$ curve. The acceptance criteria for these parameters are based on AISC 341-16 for seismic strengthening of beam-column SR connections. The flexural strength of the connection is chosen in such a way that the ratio of yield moment capacity ($M_{y,c}$) of connection to the plastic moment capacity ($M_{p,c}$) of connections is maintained at 0.67. The flexural strength of connection at the column end should be 0.8 multipliers of plastic moment capacity of the adjoining beam ($M_{p,b}$) to satisfy the story drift limit (greater than 0.04 rad), recommended in ASCE 341-16. The degree of semi-rigidity in connection is explained by parameters ‘ k ’ and ‘ s ’ shown in Eq. 1 (a, b). Chan

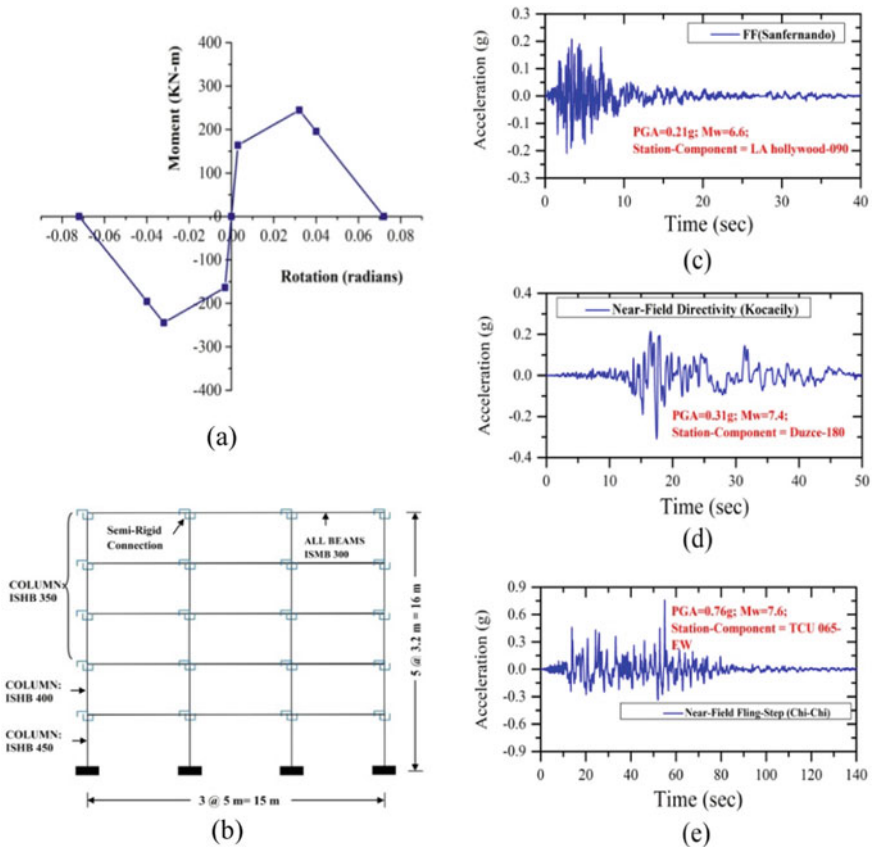


Fig. 2 a Typical moment Rotation Curve for Semi-rigid connection for $k = 15$, $s = 1.5$; b 5-Story Semi-rigid frame, and Ground Motion Records: c Far-Field, d Near-Field Directivity and (e) Near-Field Fling Step

et al. [20] prescribed the ductility limit to a minimum value of 0.04 rad for partially restrained connections for seismic strengthening. The connection parameters are

$$k = \frac{S_i}{\frac{EI_b}{L_b}}; s = \frac{M_{p,c}}{M_{p,b}} \quad (1)$$

where S_i is initial connection stiffness; EI_b/L_b is the flexural strength of the adjoining beam.

Three types of nonlinearity are considered in modeling, namely, connection nonlinearity in R3 direction in both types of link connection, material nonlinearity for flexural plastification in the form of plastic hinges as per ASCE 41-17 criterion and geometric nonlinearity considering second-order P- Δ effects. The panel zone behavior of joints is excluded in this work.

2.2 Nonlinear Response History Analysis (NRHA)

Nonlinear response history analysis (NRHA) is carried out for numerical simulation using the Hilber-Hughes-Taylor time integration approach with default values ($\gamma = 0.5$, and $\beta = 0.25$) and second-order P- Δ effects are also considered. The 5% proportional Rayleigh damping is accounted for considering the first and second modes of vibration. In all, 30 NRHA simulations were performed for the 5-Story frame with different degrees of semi-rigidity.

3 Numerical Simulation

In this study, the numerical simulations are performed on a 5-Story frame with different degrees of semi-rigidity and link properties. The 5-Story steel frame has a uniform height of 3.2 m in each floor and 5 m bay width in both directions (see Fig. 2b). The building consists of special moment frames (SMF) with rigid beam-column connections, satisfying the primary requirements of the capacity design concept. The column sections are selected in such a way that the ratios of plastic moment capacity of columns are maintained 1.2 times to the plastic moment capacity of the adjoining beam (Strong Column-Weak Beam SCWB). The gravity loads on the particular beams consist of 20KN/m floor dead load, 15 KN/m roof dead load and 4 KN/m as live load, uniformly distributed on all floors. The SMF is designed for Indian seismic code requirements as per IS 1893-2016, IS 800-2007, and IS 875-1987 [2, 21, 22]. The seismic design parameters comprised of zone factor ($Z = 0.36$; Zone V), medium soil condition, importance factor ($I = 1$ for multistory frames) and response reduction ($R = 5$ for SMF) factor. Three types of earthquakes are selected as shown in Fig. 2 (c-e) for simulation, namely, San Fernando as Far-field (FF), Kocaeli

as Near-field with the forward directivity (NFD) and Chi-Chi TCU065 as near-field with fling step (NFF) effects. A typical internal frame is chosen for analysis. Two types of beam-column connections, rigid and two sets of semi-rigid with a multi-linear elastic link and plastic link elements are considered for numerical simulation. The degree of semi-rigidity is described by two parameters: k and s (see Eq. 1). The semi-rigid frames are entitled as (i) SR-1E and SR-1P ($k = 5$; $s = 1.2$); (ii) SR-2E and SR-2P ($k = 15$; $s = 1.5$) in which 'E' stands for multi-linear elastic link whereas 'P' stands for multi-linear plastic link elements. The responses from SR frames are compared with rigid frames. The material nonlinearity in frames are modeled as concentrated default plastic hinge, defined in ASCE 41-17 at both ends of flexural members (M3 hinges for Beams and P-M3 hinges for columns in SAP2000).

4 Numerical Results and Discussions

The performance of 5-Story steel frames with different degrees of semi-rigidity, including rigid frames, is investigated for three types of ground motions at design and high PGA (i.e., 0.4 and 0.6 g) level. The seismic energy dissipation in SR frames with multi-linear elastic and plastic links are compared with rigid frames. The seismic demand parameters for comparison included maximum roof displacement, the maximum base shear, the total number of plastic hinges formed (NH), square root of sum of square values of maximum plastic hinge rotations (SRSS) and energy dissipation in link hysteretic energy and modal damping.

4.1 Energy Dissipation in Rigid and Semi-rigid Steel Frames

Figure 3 represents the variation of link hysteretic and modal damping energy in the SR frames for three types of seismic excitations at a high PGA level of 0.6 g. It is apparently visualized from the figure that the modal damping energy is more in all types of earthquakes as compared to the link hysteretic energy. Further, modal damping energy is higher in near-field fling step earthquakes. It is also noticed that the elastic link frames exhibited less link hysteretic energies as compared to plastic link connected SR frames in all cases. The link energy is decreased with an increase in the degree of semi-rigidity ($k = 15$ to 5). The reverse pattern is observed in modal damping energy.

Figure 4 shows the distribution of different types of energy in two types of SR connection modeling (multi-linear elastic and plastic) for the NFF type earthquake at the PGA level of 0.6 g. It is seen from the figure that, the multi-linear elastic link model, the model energy stored most of the input energy, very less amount of energy is dissipated in the form of other energies. As a result, the energy dissipation by way of the formation of plastic hinges is small in case of multi-linear elastic link SR frames with less number of plastic hinges (see Table 1). It is also observed that in

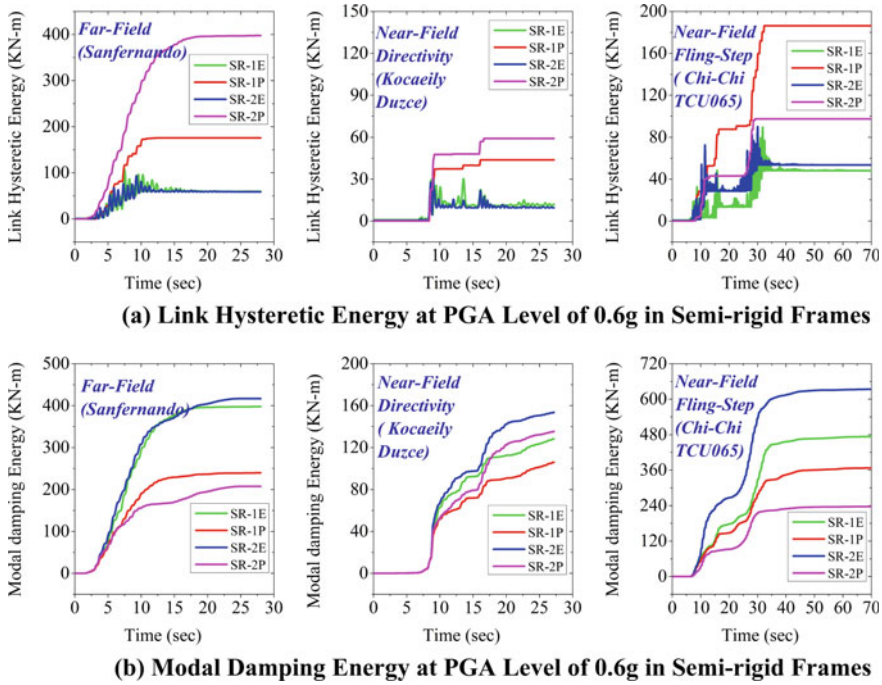


Fig. 3 Variation of **a** Link Hysteretic Energy, and **b** Modal Damping Energy under FF, NFD and NFF ground motion at a PGA level of 0.6 g for 5-Story Semi-rigid frames

the case of multi-linear plastic link SR connection, the most of energy is consumed in both modal damping and link hysteretic energy, not much energy is available to be dissipated by way of formation of plastic hinges. As a result, the formation of a number of hinges in SR connection with the plastic link element, there is a significant reduction.

Table 1 shows that the number of plastic hinges increased with an increase in the strength factor ‘s’ (1.2–1.5). Further, in fully rigid (FR) frames, the near-field earthquakes stocked more energy as compared to the far-field earthquake at the same PGA level (see Fig. 5). It is clearly seen from Fig. 5 that modal energy together with other energies consumes only a part of total energy; much energy is left, which gets dissipated in the formation of plastic hinges. Thus, a large number of plastic hinges formed in the rigid frame (see Table 1).

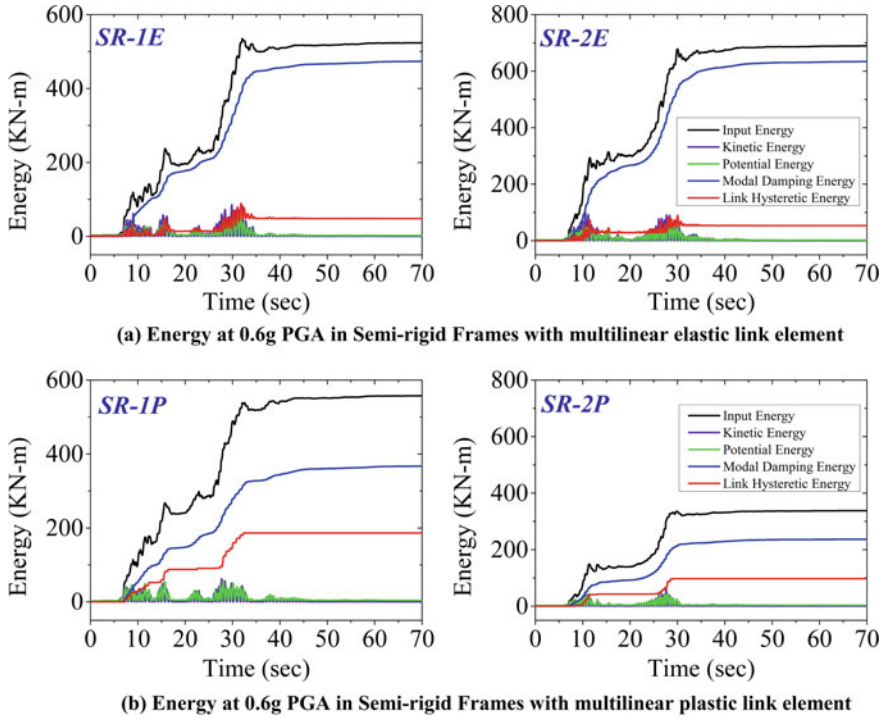


Fig. 4 Variation of Energy in SR frames with Multi-linear **a** Elastic Link, and **b** Plastic Link under the Chi-Chi TCU065 (NFF) earthquake at PGA level of 0.6 g

Table 1 Variation of total number of plastic hinges

Frame id	Total number of plastic hinges (NH)							
	FF		NFD				NFF	
	0.4 g	0.6 g	0.4 g	0.6 g	0.4 g	0.6 g	0.4 g	0.6 g
SR-1E	4	8	3	4	2	6		
SR-1P	4	6	0	4	0	4		
% Difference [^]	0.00	25.00	100.00	0.00	100.00	33.33		
SR-2E	0	18	0	9	4	18		
SR-2P	0	4	0	4	4	6		
% Difference [^]	0.00	77.78	0.00	55.56	0.00	66.67		
FR	16	27	21	30	26	31		

[^]Percentage Difference = $((NH_{SR-Elastic} - NH_{SR-Plastic}) / NH_{SR-Elastic}) \times 100$

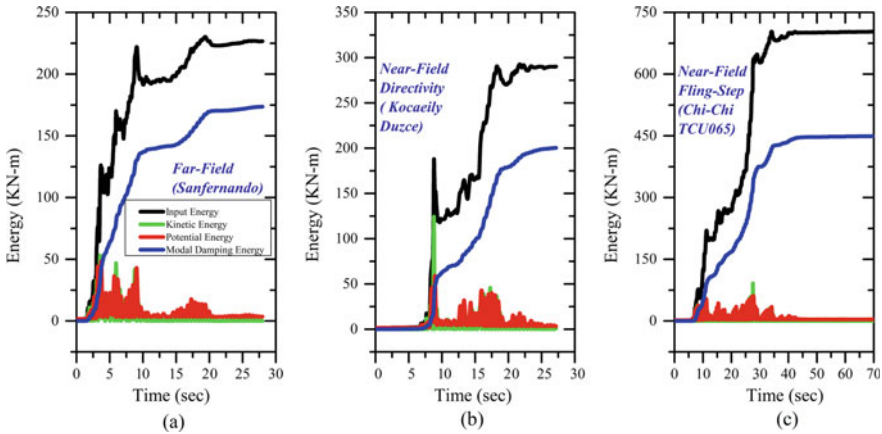


Fig. 5 Variation of Energy in Rigid frame at PGA 0.6 g for a FF, b NFD, and c NFF

4.2 Inelastic Excursion in the Semi-rigid and the Rigid Frames

Earlier section explained that the less number of plastic hinges formed in SR frames as compared to FR. Thus, the damages in terms of plastic hinges incurred at flexural member ends are less in the SR frames. However, considerable inelastic excursion occurs in plastic link connections dissipating energy through a hysteretic cycle in SR frames as discussed in earlier subsection. Table 1 shows a large number of plastic hinges formed in rigid frames, and it is increased with PGA level and type of earthquakes, maximum in near-field with fling step earthquakes. This number is considerably reduced in SR frames as energy is dissipated more in link hysteretic loops as compared to SR frames.

Table 2 shows the SRSS of maximum plastic hinge rotations in the rigid and semi-rigid frames. The SRSS values are considerably higher in FR as compared to SR frames. It is again observed that more seismic energy is dissipated in the plastic hinges in FR frames as compared to SR frames, as it would be expected.

4.3 Variation of Maximum Roof Displacements in Semi-rigid and Rigid Frames

Table 3 describes the variation of roof displacement in rigid and semi-rigid frames. It is observed from the table that the way of SR modeling affects significantly on roof displacement. The plastic link SR connection dissipates seismic energy considerably, so the maximum roof displacement in all cases is less as compared to elastic SR connection and reduced with decrease in the degree of semi-rigidity which means

Table 2 Variation of SRSS of maximum plastic hinge rotations in radians

Frame id	SRSS (radians)					
	FF		NFD		NFF	
	0.4 g	0.6 g	0.4 g	0.6 g	0.4 g	0.6 g
FR	0.01058	0.03175	0.01771	0.03854	0.03389	0.05000
SR-1E	0.00292	0.01232	0.00032	0.00346	0.00007	0.00700
SR-1P	0.00059	0.00397	0.00000	0.00304	0.00000	0.00481
<i>% Difference^^</i>	<i>79.67</i>	<i>67.76</i>	<i>100.00</i>	<i>12.13</i>	<i>100.00</i>	<i>31.30</i>
SR-2E	0.00000	0.00772	0.00000	0.00356	0.00360	0.01363
SR-2P	0.00000	0.00216	0.00000	0.00356	0.00129	0.00756
<i>% Difference^^</i>	<i>0.00</i>	<i>71.98</i>	<i>0.00</i>	<i>0</i>	<i>64.33</i>	<i>44.49</i>

^^Percentage Difference = ((SRSS_{SR-Elastic} - SRSS_{SR-Plastic})/SRSS_{SR-Elastic}) X 100

Table 3 Variation of roof displacement in semi-rigid frames

Frame id	Roof displacement (mm)					
	FF		NFD		NFF	
	0.4 g	0.6 g	0.4 g	0.6 g	0.4 g	0.6 g
SR-2E	183.63	313.05	183.63	241.89	270.09	354.28
SR-2P	179.56	228.90	170.15	239.04	220.59	291.27
SR-1E	325.83	463.03	173.59	270.71	254.56	358.48
SR-1P	264.56	344.66	173.62	271.74	230.24	291.88
FR	160.75	198.87	190.30	224.96	207.73	272.93

lowest in FR and highest in SR-1E case. The % change in maximum values are increased with PGA values from the design level to high PGA level for all cases.

4.4 Variation of Maximum Base Shear in Semi-rigid Frames

Table 4 represents the percentage variation in the maximum value of base shear in SR elastic and plastic frames for three types of earthquakes. It is clearly observed that the maximum value of base shear reduced in plastic SR frames, and it is increased with the PGA level in all cases. As the PGA increased, more energy is attracted and the more amount of energy is dissipated in the multi-linear plastic links (due to kinematic hysteretic behavior) as compared to multi-linear elastic links (due to isotropic hysteretic behavior).

Table 4 Variation of maximum base shear in semi-rigid frames

Frame id	Percentage difference in maximum base shear (KN)					
	FF		NFD		NFF	
	0.4 g	0.6 g	0.4 g	0.6 g	0.4 g	0.6 g
SR-2E	615.67	792.82	615.92	781.35	680.46	788.29
SR-2P	538.98	665.51	582.74	702.79	654.03	754.81
% Difference [#]	12.46	16.06	5.39	10.05	3.89	4.25
SR-1E	608.77	755.07	603.79	757.59	496.10	630.55
SR-1P	589.72	707.45	581.93	676.69	490.86	612.64
% Difference [#]	3.13	6.31	3.62	10.68	1.06	2.84

Percentage Difference = ((Base Shear_{SR-Elastic} - Base Shear_{SR-Plastic})/Base Shear_{SR-Elastic}) X 100

5 Conclusions

The seismic energy dissipations and seismic demand parameters of 5-Story steel rigid and semi-rigid (SR) frames are examined at two peak ground acceleration (PGA) levels (designated as design level and high level) for three variants of earthquakes, viz., the far-field and near-field with the forward directivity effect and fling step effects. For the simulation study, a nonlinear response history analysis is carried out to obtain the seismic energy along with other responses. The semi-rigid frames are simulated in two different ways, namely, semi-rigid connection with a multi-linear elastic link and plastic link connections. The degree of semi-rigidity is taken as a prime parameter. The results of numerical simulation achieve the following outcomes:

- i. The way of the dissipation of seismic energy in the rigid and semi-rigid frames are in different ways. The modal energy in the plastic hinges shared the maximum of input energy in rigid frames. Whereas for semi-rigid frames, the seismic energy is dissipated in the modal damping energy along with link hysteretic energy in plastic hinges.
- ii. For semi-rigid frames, the modal energy consumes the maximum share of input seismic energy in connection with an elastic link, and the link hysteretic energy is considerably less as compared to modal energy. On the other end, in plastic semi-rigid connections, most of the input energy is dissipated in the form of link hysteretic and modal energy along with a very little share of other energies.
- iii. The number of plastic hinges formed and their SRSS of maximum plastic hinge rotations are significantly higher in the rigid frames and considerably less in semi-rigid frames.
- iv. The maximum values of roof displacement are observed in semi-rigid frames due to less strength and stiffness values as compared to the rigid frames.
- v. The base shear is less in semi-rigid frames as compared to rigid frames. Especially, the semi-rigid frames with plastic link provide less base shear than elastic

semi-rigid frames, and these difference decreases with increase in the degree of semi-rigidity.

- vi. The multi-linear plastic link semi-rigid frames are performed considerable better during earthquakes as compared to rigid frames and semi-rigid frames with multi-linear elastic links.

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