ASPS Conference Proceedings 1: 781-787 (2022)

Proceedings of

12th Structural Engineering Convention-An International Event (SEC 2022)

Available at https://asps-journals.com/index.php/acp

# Numerical Study of Hybrid Steel Frames under Far-field Earthquakes

Vijay Sharma [1,] *, Mohit Bhandari [2], M.K. Shrimali [3], S.D. Bharti[3], T.K. Datta [4]

[1] Department of Applied Mechanics, Assistant Professor, Government Engineering College, Palanpur 385 001, India
[2] Department of Civil Engineering, Assistant Professor, Chandigarh University, Chandigarh, 140 413, India
[3] National Centre for Disaster Mitigation and Management, Professor, Malaviya National Institute Technology, Jaipur, 302 017, India
[4] National Centre for Disaster Mitigation and Management, Adjunct Professor, Malaviya National Institute Technology, Jaipur, 302 017, India,

**Paper ID - 060320**

**Abstract**

The 1994 Northridge and 1995 Kobe earthquakes have shown the susceptibility of severe damage of welded moment-resisting frames, especially at the beam to column connections subjected to severe earthquake ground-motions. Since then, an alternative of welded connections called bolted or semi-rigid connections is considered for designing or retrofitting of steel moment frames. From the last few years, the semi-rigid connections combined with rigid connections, termed as hybrid frames have become a promising approach to design the steel frames for enhanced seismic performance. In the present paper, a numerical study is carried out to investigate the seismic performance of the hybrid steel frames. For this purpose, a ten-story high-rise rigid steel frame is analyzed and designed for seismic requirements as per Indian Standards. The rigid connections of frames are replaced by semi-rigid connections with a moderate degree of semi-rigidity. The six different patterns (locations of semi-rigid connections) of hybrid frames are considered here to compare the seismic responses with those obtained from the rigid connected steel frame. The nonlinear time-history analyses are executed using the SAP2000 software at two scaled PGA level in compliance with design basis earthquake and maximum considered earthquake. An ensemble of five time-histories of far-field earthquakes is considered for the numerical study. A wide range of seismic response parameters, namely, the maximum value of base shear, top-story displacement, inter-story drift ratio, the total number of plastic hinges is extracted for the suggestion of a suitable pattern for hybrid frames. The major findings from the present numerical study show that the pattern and number of semi-rigid connections in the hybrid frame considerably affect the seismic performance of steel frames. It is also observed from the seismic responses that the hybrid frames are better than the rigid frame at high PGA levels.

*Keywords*: Seismic performance, Hybrid Frames, Far-field, Semi-rigid connection, Nonlinear time-history analysis

## 1. Introduction

The welded moment-resisting frames is a traditional approach to design the seismic-resistant steel building frames in the last few decades. The construction and fabrication costs of moment frames are comparatively more as compared to the other civil engineering construction practices. But these frames are preferred due to their ductile nature and faster construction. During the Northridge-1994 and Kobe-1995 earthquakes, hundreds of building frames were severely damaged at the beam-to-column connections and it was found difficult to retrofit. The main reason for this deficiency was the stress concentration and ductility in the welded zone of connections [1].

As a consequence, an alternative to welded connections has become a key concern for various researchers. Several researchers found that the efficacy of frames was increased by using the semi-rigid connections (SR) in place of rigid connections. Recently, Sharma et al. [2] carried out a comprehensive study on the behavior of semi-rigid steel frames and investigated the efficacy of SR frames as compared to rigid frames. Various studies showed the improved performance of semi-rigid frames in the form of energy dissipation under severe earthquakes with different degrees of semi-rigidity [3-8]. Experimental investigations also showed that SR connections have good ductility and the ability to dissipate a large amount of seismic energy without failing. Díaz et al. [9] carried out a state-of-the-art review of SR connections that categorize the connection in various models like numerical, experimental, mathematical, informational based on their moment-rotation (M-θ) curve. The most common SR connections models are the Frye-Morris polynomial model, Kish-Chen power model, Bjorhovde model [10-12].

It is also found that the complete replacement of rigid connections with SR connections increased the top-story displacements and flexibility of steel frames which increase the vulnerability of frames under severe shaking. Therefore, most of the design standards eliminated the use of semi-rigid or partially restrained connections for high seismic zones. In the dual frames, the rigid connections provide the required lateral stiffness and the lower down the seismic energy level

*Corresponding author. Tel: +919409143690; E-mail address: vrsgec2011@gmail.com





Published online: December 19, 2022
doi:10.38208/acp.v1.582

to maintain the higher lateral displacement under the limit. On the other side, the SR connections fulfill the ductility requirement. Dubina et al. [13] suggested that stiffener members can be eliminated or reduced in the hybrid frames, which results in lower construction costs. Since SR connections are bolted connections so it can be repaired easily after damage induced due to any severe shaking [14]. Earlier, few researchers investigated the seismic performance of hybrid frames with a reduced number of SR connections with the rigid connections [15-17]. Abolmaali et al. [15] presented the concept of hybrid frames using in the 9- and 20-story SAC benchmark frames and identified the energy dissipation capabilities. Several different patterns and locations are identified for SR connections for improved performance, especially for those 9- and 20-story SAC frames. Feizi et al. [16] studied the behavior of steel frames with a combination of rigid and SR connections under dynamic loadings and found that hybrid frames performed relatively better in terms of base shear and drift values. Recently, Bayat et al. [17] suggested the different patterns for 10, 15, and 20-story steel frames and compared the base shear, story drift, and roof accelerations with the responses obtained from respective rigid frames.

Various studies focused on the SR frames due to their stable behavior and high ductility to dissipate seismic energy. However, very less attention was paid to the seismic performance of hybrid steel frames. The present study concentrates to identify the most suitable pattern and location of SR connections and evaluates the seismic performance of a 10-story hybrid frame in the form of inter-story drift, base shear, top-story displacements. The inelastic excursions of hybrid frames are measured in the form of the total number of plastic hinges obtained under two PGA level of the ensemble of the far-field earthquakes. For each earthquake, two PGA levels depicting the design basis earthquake (DBE) at PGA=0.2g and maximum considered earthquake (MCE) at PGA=0.5g are considered.

## 2. Methodology

The present study aims to investigate the seismic performance of high-rise steel rigid and hybrid (i.e., combination of rigid and semi-rigid or partially restrained connections) moment frame. A 10-Story rigid steel moment frame is analyzed and designed using the SAP2000 software [18]. Similar sections are used for corresponding hybrid frames. A nonlinear time-history analysis (NTHA) is performed to evaluate the seismic response. The modeling of semi-rigid connections defined in SAP2000 is explained in the next sub-section.

### *2.1 Modeling of SR connection and its implementation in software*

Various moment-rotation (M-θ) models are available in the existing literature to model the properties of semi-rigid (SR) connections. The degree of semi-rigidity in SR beam-column connections is broadly dependent on three parameters, namely, stiffness ($\alpha$), strength ($\beta$), and ductility ($\mu$) of adjoining members [19, 20]. The SR connections used in the present study are based on the AISC 341-16 recommendations for special moment frames (SMF) satisfying the seismic requirements. The ratio of the yield moment ($M_{yc}$) capacity to the plastic moment capacity of connections ($M_{pc}$) in the generic model is chosen as 2/3. The flexural strength or ultimate moment capacity of connections at the column ends should be a minimum of 80% of the connecting beam plastic moment capacity ($M_{pb}$) to satisfy the story drift limit (minimum 0.04 rad) of connection as per AISC 341-16. The moderate degree of semi-rigidity in connections are defined by two dimensionless parameters ($\alpha$=10 and $\beta$=1.5) and the connection ductility. For SR connections, the ductility of connections should not be less than $\theta_u \geq 0.04$ rad [21]. The parameters are taken as follows:

$$\alpha = K_i / (EI_{beam})/L_{beam}) \tag{1}$$

$$\text{and} \quad \beta = M_{pc}/M_{pb} \tag{2}$$

Where $K_i$= Initial Connection Stiffness;

$EI_{beam}/L_{beam}$ = Stiffness of connected beam members

In SAP2000, the SR connections modeled as two jointed zero-length multi-linear plastic link elements (MLP) with rotational nonlinearity in the R3 direction. The material nonlinearity in the structure is modeled like a concentrated default flexural plastic hinge as per ASCE 41-17 [22], and geometric nonlinearity is incorporating through P-Δ consideration. The kinematic hysteresis behavior of MLP link for energy dissipation in cyclic loadings is shown in Figure 1.

### *2.2 Analytical Approach*

A numerical model of steel SMF is analyzed and designed as per seismic requirements of Indian provisions. The nonlinear time-history analysis (NTHA) is performed to examine the seismic performance of rigid and hybrid steel frames. The flexural beam member is modeled like an elastic member with default moment type (M3) plastic hinges at the ends and default P-M3 type hinges are employed in column ends which considers the interaction of

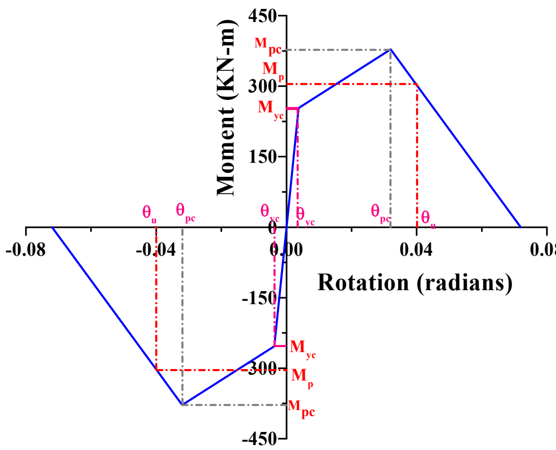


Fig. 1 Moment-rotation curve of SR connection

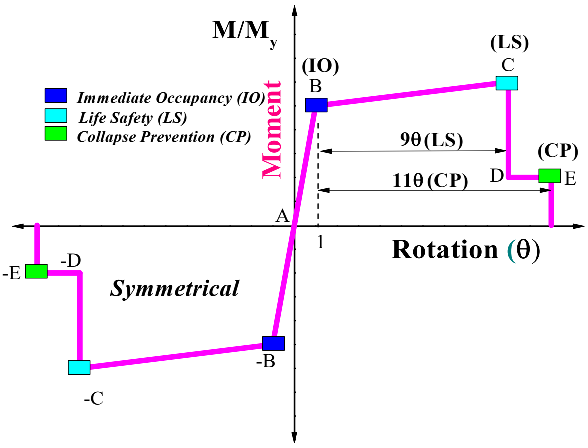


Fig. 2 Hysteretic plot of plastic hinges (ASCE 41-17)

axial force with bending moments. The default hinge-properties are described in Fig. 2 as per the ASCE-41 [22] acceptance criterion. Three performance levels are defined in the curve, immediate occupancy (IO), life safety (LS), and collapse prevention (CP) and the values are obtained from Table 9.6 of ASCE-41. The secondary P-delta effects are considered to carry the geometric nonlinearity. Therefore, in this study, three types of nonlinearity, namely, the connection nonlinearity, material nonlinearity (in form of concentrated default flexural hinges), and geometric nonlinearity are taken.

For the NTHA in SAP2000, the Hilber-Hughes-Taylor direct integration scheme is used for numerical simulation with default values. The Rayleigh proportional damping is considered with the 5% damping for the first and second modes of vibration.

## 3. Numerical Study

For the numerical study, a 10-Story frame with rigid and semi-rigid connections are considered. The 10-Story frame has an identical height of 3.2 m each with three bays of 5 m each in both directions. The building comprises of special moment resisting frames with rigid beam to column connections. The sections are selected to ensure the capacity design concept, i.e., the stiffnesses of columns are 1.2 times the connected beam stiffness (strong column-weak beam; SCWB) as shown in Fig 3. The frames are subjected to gravity load resulted from 150 mm thick concrete slab along with floor finish and 225 mm thick partition wall load. The effective gravity load consists of 20 kN/m as dead load, 15 kN/m as roof load, and 4 kN/m as live load, uniformly distributed over beams. The SMFs are designed for seismic strengthening as per Indian standards IS-800 [23], IS 1893-Part-1 [24], and IS 875 [25]. The seismic design parameters considered for design are zone factor (Z=0.36, Zone V), soil type (medium soil), importance factor (I=1 for high rise commercial building), and response reduction factor for frame type (for SMRF, R=5). An ensemble of five earthquake records as listed in Table 1 with their properties. The far-field records are chosen from the FEMA P695[26] report.

The six different patterns for hybrid frames and rigid are shown in Fig. 4 and the natural period of time for the first three vibration modes are represented in Table 2.

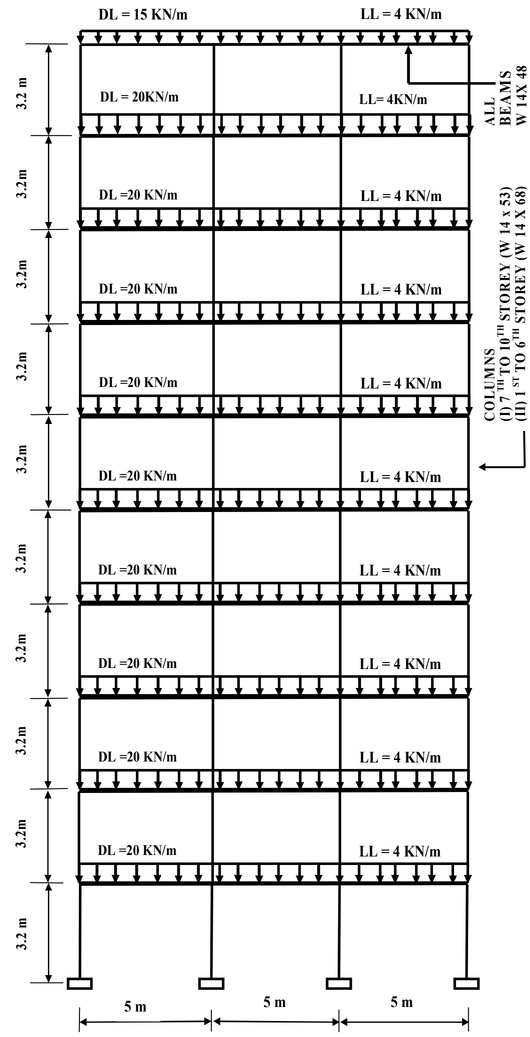


Fig. 3 10-Story rigid frame with gravity load

Table 1- Properties of earthquake records

| No. | Event Year | Station-Component | $M_w$* | PGA (g) | PGV (cm/s) | PGD (cm) | $R_{jb}$# (km) |
|---|---|---|---|---|---|---|---|
| 1 | 1995 | Kobe-Nishi-000 | 6.9 | 0.51 | 37.28 | 9.53 | 25.2 |
| 2 | 1992 | Landers-TR | 7.3 | 0.42 | 42.35 | 13.84 | 19.74 |
| 3 | 1978 | Tabas-Ferdows-L | 7.4 | 0.093 | 5.4 | 2.24 | 89.76 |
| 4 | 1987 | Superstition hill-POE-270 | 6.5 | 0.45 | 35.72 | 8.81 | 22.25 |
| 5 | 1971 | San Fernando-90 | 6.6 | 0.21 | 18.87 | 12.42 | 124.38 |

***Mw: Magnitude; #$R_{jb}$: Closest Distance**

Table 2- Natural period and number of SR connection used in different hybrid steel frames

| Sr. No. | Pattern Name | N $ | Natural period of time (sec) T1 | T2 | T3 |
|---|---|---|---|---|---|
| 1 | HF10_01 (Interior/Middle Bay) | 20 | 1.907 | 0.623 | 0.35 |
| 2 | HF10_02 (Cross Diagonal) | 28 | 1.94 | 0.633 | 0.356 |
| 3 | HF10_03 (Exterior/ Outer Bay) | 40 | 2.01 | 0.654 | 0.367 |
| 4 | HF10_04 (Middle Stories 3-7) | 30 | 2.014 | 0.636 | 0.357 |
| 5 | HF10_05 (Stoup) | 28 | 1.938 | 0.633 | 0.356 |
| 6 | HF10_06 (Exterior Diagonal) | 20 | 1.919 | 0.627 | 0.353 |
| 7 | RFR10 (Rigid Frame) | 0 | 1.823 | 0.687 | 0.338 |

**$ N= Number of SR connections**

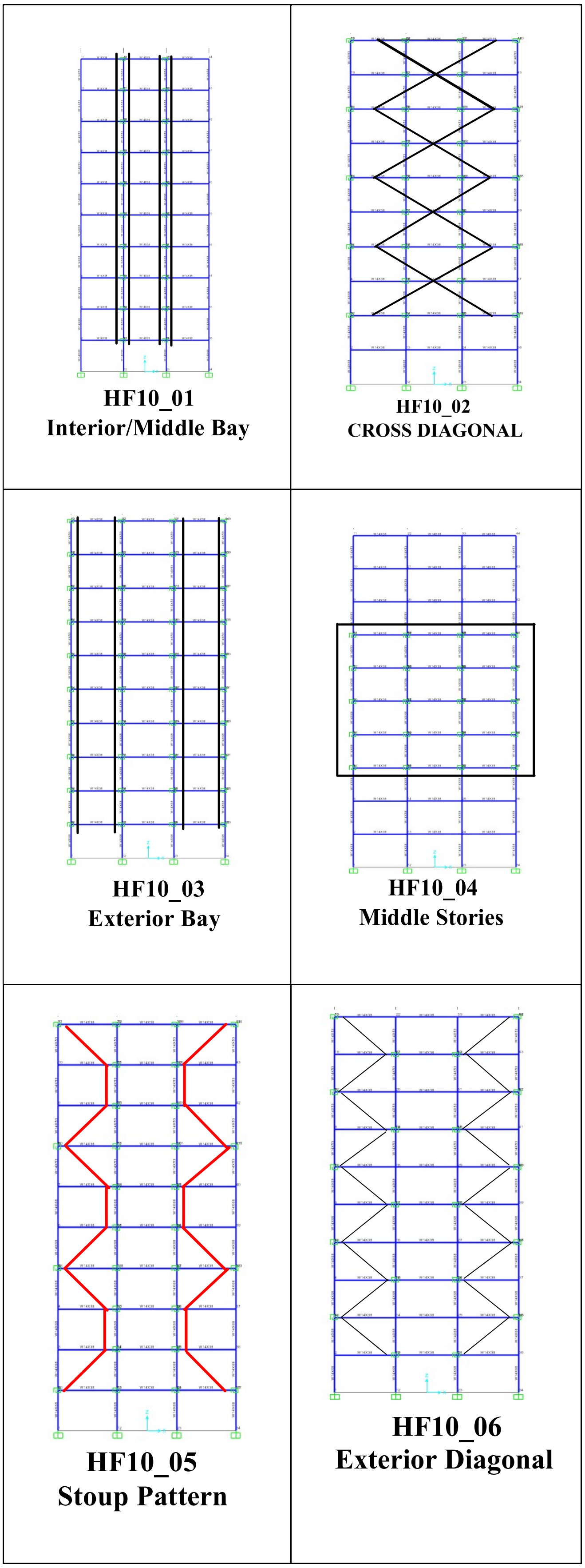


Fig. 4 Different pattern for 10-Story Hybrid pattern

## 4. Results and Discussion

The seismic performance of hybrid steel frames under an ensemble of far-field earthquakes are investigated at two PGA levels in compliance with the design basis earthquake, DBE (=0.2g), and maximum considered earthquake, MCE (=0.5g). For this study, the six different patterns (arrangement of semi-rigid connections) for hybrid frames are taken and the most suitable pattern/s for enhanced seismic performance are suggested. The response quantities or seismic demand parameter for comparison are maximum values of inter-story drift ratio (MIDR), top story displacement, base shear, the total number of plastic hinges, and the square root of the sum of the squares (SRSS) of the maximum plastic hinge rotations. Further, the ranking is carried for the most suitable pattern for each seismic demand parameter.

### *4.1 Comparison of maximum values of inter-story drift ratio (MIDR)*

The variation of MIDR along the story height for rigid and six hybrid frames at two PGA levels (0.2g and 0.5g) are shown in Fig 5. It is observed from the figure that the MIDR values along with the height in the hybrid frames with semi-rigid (SR) connection in the middle or interior bay (HF10_01) and exterior diagonal (HF10_06) are closely correlated with the rigid frames. The maximum values of MIDR in all cases are observed in the lower stories (mainly at the second or third story) level. The mean values of MIDR are represented in Table 3 and the suitability of frames is ranked (1 for most suitable and 6 for least recommendation). It is also noticed that the MIDR values are under permissible values, i.e., < 1% (even at higher PGA level (MCE PGA=0.5g). It is observed that the MIDR values are increased with an increase in the number of SR connections (20 in case of HF10_01 and HF10_06 to 40 for HF10_03). The MIDR is the most promising criteria for seismic performance of frames, and the hybrid frames fulfill the effectiveness under high earthquake levels.

Table 3- Variation of the mean value of the maximum inter-story drift ratio

| Pattern/ Frame ID | Mean value of MIDR | | Rank |
|---|---|---|---|
| | PGA 0.2g | PGA 0.5g | |
| HF10_01 | 0.343 | 0.858 | ***1*** |
| HF10_02 | 0.356 | 0.890 | ***3*** |
| HF10_03 | 0.392 | 0.979 | 6 |
| HF10_04 | 0.371 | 0.930 | 5 |
| HF10_05 | 0.364 | 0.911 | ***4*** |
| HF10_06 | 0.351 | 0.878 | ***2*** |
| **RF10** | **0.312** | **0.774** | |

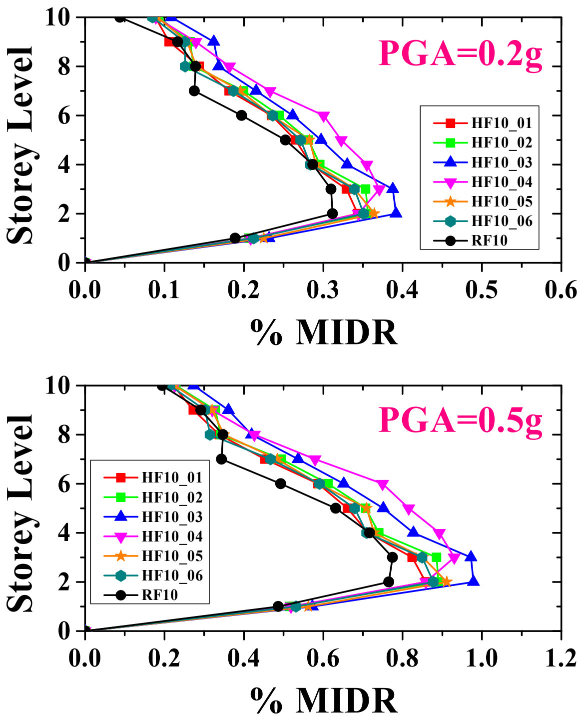


Fig. 5 Variation of MIDR along height in hybrid frames

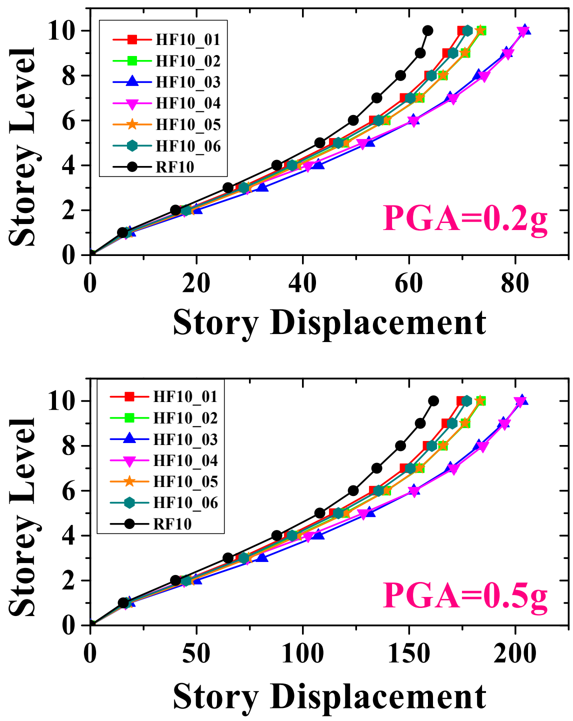


Fig. 6 Variation of story displacement with story level for hybrid frames

*4.2 Comparison of story displacement*

Fig. 6 shows the variations of story displacement with story level for hybrid frames with a different pattern as compared with the rigid frame at two PGA levels. It is noticed that the story displacement increases with an increase in the number of SR connections, maximum in the HF10_03 hybrid frame (40 SR connections). The pattern of SR connections also significantly affect variation in story displacement. It is observed that the hybrid frame HF10_04 with 30 SR connections represents nearly the same story displacement with HF10_03 with 40 SR connections. Table 4 represents the maximum and mean values of story displacements for hybrid frames. The table shows that the pattern with interior bay and external diagonal performs well as compared to other patterns with the rigid frame. On this basis, the HF10_01 is ranked 1 and suggested as the most suitable pattern.

Table 4- Ranking of different hybrid patterns based on the maximum value of story displacement

| Pattern/ Frame ID | PGA = 0.2g | | PGA = 0.5g | | RANK |
|---|---|---|---|---|---|
| | Mean | Max | Mean | Max | |
| HF10_01 | 69.98 | 92.59 | 174.56 | 231.64 | *1* |
| HF10_02 | 73.6 | 97.36 | 183.81 | 243.57 | 4 |
| HF10_03 | 81.75 | 105.82 | 203.07 | 264.71 | 6 |
| HF10_04 | 81.46 | 103.4 | 202.16 | 258.37 | 5 |
| HF10_05 | 73.41 | 97.2 | 183.37 | 243.16 | *3* |
| HF10_06 | 70.97 | 94.03 | 177.15 | 235.24 | *2* |
| RF10 | 63.52 | 84.94 | 161.47 | 212.52 | |

*4.3 Comparison of base shear*

Table 5 shows the variation of maximum values of base shear at two PGA levels for hybrid steel frames with different patterns. The maximum and mean values of base shear is represented here; the mean values are the average of base shear responses obtained from five different far-field earthquakes and the maximum is the maximum of peak values of base shear obtained in five different earthquakes. It is noticed that minimum base shear is noticed in the pattern with SR connections in the interior or middle bay for both conditions (i.e., maximum and mean values) and it is ranked 1. The pattern with exterior diagonal (HF10_06) also holds good.

Table 5- Variation of base shear (kN) in hybrid frames

| Pattern/ Frame ID | PGA = 0.2g | | PGA = 0.5g | | Rank |
|---|---|---|---|---|---|
| | Mean | Max | Mean | Max | |
| HF10_01 | 202 | 233 | 506 | 582 | *1* |
| HF10_02 | 206 | 245 | 514 | 611 | *3* |
| HF10_03 | 206 | 262 | 508 | 642 | 5 |
| HF10_04 | 210 | 264 | 524 | 655 | 6 |
| HF10_05 | 206 | 245 | 514 | 611 | *4* |
| HF10_06 | 203 | 236 | 507 | 591 | *2* |

Table 6: Inelastic excursion in different hybrid steel frames

| Frame Model | NH | | SRSS (X 10-3) | | Rank |
|---|---|---|---|---|---|
| | Mean | Max | Mean | Max | |
| HF10_01 | 3 | 6 | 0.51 | 1.58 | 2 |
| HF10_02 | 4 | 5 | 0.96 | 1.78 | 3 |
| HF10_03 | 5 | 9 | 1.8 | 3.1 | 6 |
| HF10_04 | 2 | 4 | 0.81 | 2.73 | 4 |
| HF10_05 | 5 | 7 | 1.1 | 2.33 | 5 |
| HF10_06 | 2 | 3 | 0.11 | 0.44 | 1 |

### *4.4 Inelastic excursion and plastic hinge formation in hybrid steel frames*

Table 6 shows the inelastic excursion in the hybrid frames with different patterns in the form of the total number of plastic hinges and SRSS of maximum plastic hinge rotations at 0.5g PGA level. The earlier research studies showed that a significant reduction of the formation of plastic hinges observed in the case of frames with SR connections as compared to the rigid frames. Therefore, the present table simply compares the number of hinges and SRSS values in hybrid frames. It is noticed that the less number of hinges are observed in the exterior diagonal pattern and similarly maximum and mean values are also least in the HF10_06 case. At PGA of 0.2g, there are no plastic hinges formed, which means the structure is still in the elastic range. Therefore, it is not shown here. The pattern with SR connection in the interior bay is ranked 2 on the basis of inelastic excursion that occurred in the frame.

## 5. Conclusions

The seismic performance of high-rise ten-story moment-resisting steel rigid and hybrid steel frames are examined at two scaled peak ground acceleration (PGA) level (denoting the design basis earthquake 0.2g and maximum considered earthquakes) for an ensemble of five far-field earthquakes. For the numerical simulation, six different arrangements/patterns of semi-rigid connections are considered to define the hybrid frame. The nonlinear time-history analyses at two PGA levels for rigid and hybrid frames are performed to extract the wide range of seismic responses. The seismic responses include the maximum values of inter-story drift ratio (MIDR), story displacement, base shear, the total number of plastic hinges, and the SRSS values of maximum plastic hinge rotations. The most suitable patterns based on these response quantities are ranked among the considered hybrid frames and comparisons are made concerning responses obtained from the rigid frames. A moderate degree of semi-rigid connection is used for the entire analysis. The major findings from the numerical study are as follows:

1. The number of SR connections affect the performance of hybrid frames. The responses such as the MIDR, story displacement, the total number of plastic hinges increase with an increase in the number of SR connections.
2. The distribution of SR connection or pattern significantly affects the seismic performance. Even at a high number of SR connections, the pattern considerably reduces the seismic response. The exterior or outer bay (HF10_03) has more number of SR connections, but the story displacement at all story levels are observed nearly the same values as obtained in pattern with SR connections in the middle bay (i.e., HF10_04).
3. The trend of variations in the mean values of MIDR along the height and story displacement at different stories does not vary substantially with an increase in PGA levels. This might be due to the averaging effect of responses.
4. The maximum MIDR is observed at the bottom (at second and third story level) story levels for both PGA levels.
5. The pattern with the SR connections in the interior or middle bays are the most suitable pattern (HF10_01) as compared to other cases and ranked at 1 for most of the cases.
6. The inelastic excursion measured in the SRSS of maximum plastic hinge rotation showed that HF10_02 (pattern with SR connections in the exterior diagonal bay) is the best among all other patterns. For the inelastic excursion, the HF10_01 performs also considerably well as compared to other patterns.

**Disclosures**

Free Access to this article is sponsored by SARL ALPHA CRISTO INDUSTRIAL.

## References

[1] A.S. Elnashai, L. Di Sarno, Fundamentals of earthquake engineering, Wiley New York2008.

[2] V. Sharma, M.K. Shrimali, S.D. Bharti, T.K. Datta, Behavior of semi-rigid steel frames under near- and far-field earthquakes, Steel and Composite Structures-An International Journal 34(5) (2020) 625-641.

[3] V. Sharma, M. Shrimali, S. Bharti, T. Datta, Behavior of semi-rigid frames under seismic excitations, 16th Symposium on Earthquake Engineering, Indian Institute of Technology, Roorkee, 2018, pp. 1-10.

[4] V. Sharma, M. Shrimali, S. Bharti, T. Datta, Seismic energy dissipation in semi-rigid connected steel frames, 16th World Conference on Seismic Isolation, Energy Dissipation and Active Vibration Control of Structures, Saint Petersburg, Russia, 2019.

[5] V. Sharma, M.K. Shrimali, S. Bharti, T. Datta, Evaluation of responses of semi rigid frames at target displacements predicted by the nonlinear static analysis, Steel and Composite Structures-An International Journal 36(4) (2020) 399-415.

[6] V. Sharma, M.K. Shrimali, S. Bharti, T. Datta, Sensitivity of lateral load patterns on the performance assessment of semi-rigid frames, Technologies for Sustainable Development: Proceedings of the 7th Nirma University International Conference on Engineering (NUiCONE 2019), November 21-22, 2019, Ahmedabad, India, CRC Press/Balkema, Taylor and Francis Group, Schipholweg, 107C, 2316XC, Leiden, The Netherlands, 2020, pp. 62-67.

[7] V. Sharma, M.K. Shrimali, S. Bharti, T. Datta, Behavior Of Mid Rise Semi-Rigid Connected Steel Frames Under Near-Field And Far-Field Earthquakes, SEC18: Proceedings of the 11th Structural Engineering Convention – 2018, Jadavpur University, Kolkata, India, 2018.

[8] V. Sharma, M. Shrimali, S. Bharti, T. Datta, Seismic Energy Loss in Semi-rigid Steel Frames under Near-field Earthquakes, in: S.K. Saha, Mukherjee, Mousumi (Eds.), Recent Advances in Computational Mechanics and Simulations Volume-I: Materials to Structures, Springer Singapore2021, pp. XVI, 621.

[9] C. Díaz, P. Martí, M. Victoria, O.M. Querin, Review on the modelling of joint behaviour in steel frames, Journal of constructional steel research 67(5) (2011) 741-758.

[10] M.J. Frye, G.A. Morris, Analysis of flexibly connected steel frames, Canadian journal of civil engineering 2(3) (1975) 280-291.

[11] W.-F. Chen, N. Kishi, Semirigid steel beam-to-column connections: Data base and modeling, Journal of Structural Engineering 115(1) (1989) 105-119.

[12] R. Bjorhovde, A. Colson, J. Brozzetti, Classification system for beam-to-column connections, Journal of Structural Engineering 116(11) (1990) 3059-3076.

[13] D. Dubina, A. Stratan, G. De Matteis, R. Landolfo, Seismic performance of dual steel moment-resisting frames, International Conference on Behaviour of Steel Structures in Seismic Areas, Rotterdam, Netherlands, 2000, pp. 569-576.

[14] D. Dubina, A. Stratan, F. Dinu, Suitability of semi-rigid steel building frames in seismic areas, European Conference on Earthquake Engineering, Balkema, Rotterdam, Netherlands, 1998.

[15] A. Abolmaali, A. Kukreti, A. Motahari, M. Ghassemieh, Energy dissipation characteristics of semi-rigid connections, Journal of Constructional Steel Research 65(5) (2009) 1187-1197.

[16] M.G. Feizi, A. Mojtahedi, V. Nourani, Effect of semi-rigid connections in improvement of seismic performance of steel moment-resisting frames, Steel and Composite Structures 19(2) (2015) 467-484.

[17] M. Bayat, S.M. Zahrai, Seismic performance of mid-rise steel frames with semi-rigid connections having different moment capacity, Steel and Composite Structures 25(1) (2017) 1-17.

[18] SAP2000v21, Integrated Software for Structural Analysis and Design, Computers and structures Inc, Berkeley, CA, USA (2019).

[19] B. AISC-360, AISC 360-16, Specification for Structural Steel Buildings, Chicago AISC, 2016.

[20] ANSI/AISC-341, 341 Seismic provisions for structural steel buildings, 2016, p. 60601.

[21] S. Chan, P. Chui, Nonlinear static and cyclic analysis of steel frames with semi-rigid connections, Elsevier2000.

[22] ASCE-41, ASCE 41-17: Seismic Evaluation and Retrofit Rehabilitation of Existing Buildings, Proceedings of the SEAOC, 2017.

[23] IS-800, General Construction in Steel-Code of Practice (Third Revision) Bureau of Indian Standards, New Delhi, 2007, pp. 1-143.

[24] IS-1893, Criteria for earthquake resistant design of structures, Part 1 General Provisions and Buildings (Sixth Revision), Bureau of Indian Standards, New Delhi, 2016, pp. 1-44.

[25] IS-875, Part 1: DEAD LOADS — UNIT WEIGHTS OF BUILDING MATERIALS AND STORED MATERIALS, Bureau of Indian Standards, New Delhi, 1987.

[26] FEMA-P695, Quantification of building seismic performance factors, FEMA P695, Federal Emergency Management Agency, 2009.