Connection Rotation Demand in Special Moment Frame Buildings under Seismic Actions

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SUMMARY
The first story in buildings is usually flexible compared to upper stories as the first story columns are extended up to the foundation level and same column sections are used for the first few stories. Nonlinear time history analyses of a six and a twenty-story special moment frame building for a suite of earthquake ground motion show that inelastic actions are concentrated in such relatively soft first stories. Increasing the first story stiffness by changing the first story columns helps distribute inelastic actions in higher storeys of the six-story building. However, this is not sufficient for the twenty-story building. Further, incremental dynamic analyses results show that connection rotation demands are significantly larger than current maximum connection rotation design capacity of 0.04 radians for earthquake intensities with PGA greater than 0.5g.

Keywords: soft storey; nonlinear time history analysis; incremental dynamic analysis

1. INTRODUCTION

Moment resisting frame (MRF) is commonly used as lateral load resisting system in low to mid-rise buildings to resist wind and seismic actions, in addition to supporting gravity loads. MRFs are often chosen over structural wall system or dual system in steel buildings because of the simplicity in behavior and design, as well as economy. Special moment frames with additional proportioning and detailing requirements are used to resist strong earthquake induced effects through cyclic inelastic actions during the shaking. In particular, special steel MRFs provide higher energy dissipative capacity through plastic actions in the form of flexural yielding of beams and columns, and shear yielding of panel zones. This is in tune with earthquake-resistant design philosophy that aims to prevent collapse of buildings during severe shaking while allowing ductile structural damage to dissipate the input energy.

Still, behaviours of buildings with MRFs in past earthquakes show that damage is usually concentrated at few stories and the ideal distribution of plastic hinges along the height of the building is not achieved. Such situations arise due to to excessive demand being imposed on few hinges, while on the other hand, hinges are not formed at many places. This eventually leads to brittle failure of connections that are critical components of MRFs. Also, all connections in a MRF are designed for the same target rotation capacity. Thus, while the target capacity is utilized at some connections, it remains unused at many more. Further, the target rotation capacity is set based on estimates of demand using the idealized condition of distributed inelasticity while the actual demand can be significantly different if the ideal sway mechanism is not mobilized under actual earthquake ground shaking. Thus, it is important to investigate actual demands on hinges in real buildings during earthquake ground motions. This study aims to provide rational estimate of seismic connection rotation demand, an important design parameter in steel MRFs.

2. SEISMIC DEMAND ON CONNECTIONS IN MRFs

Conventionally, seismic demands are quantified by response parameters that are useful for engineering design decision-making, such as global deformations like roof and story drifts, or local deformations such as plastic hinge rotations. Seismic demands, also known as engineering demand parameters, of interest are parameters that correlate best with the various types of structural, nonstructural, and contents damage. For structural damage, relevant local parameters are shear distortions in joints and rotations at plastic hinges [Gupta and Krawinkler, 1999].

But, these response parameters are dependent on primarily (i) input ground motion characteristics, and (ii) the design of beams, columns and panel zones. For ordinary ground motions, which are motions recorded at a distance of more than about 15 km from the fault rupture, the peak ground acceleration (PGA) is commonly used as frequency characteristics are not very sensitive to magnitude and distance, and a single intensity measure (IM) is adequate to describe the intensity of such motions. However, this is not always true with near-fault ground motions (with the directivity) that impose quite different demands on structural members and connections, particularly along the height of the building. It is seen that first yielding, and thus maximum ductility demand, usually occurs in the upper stories in relatively long period structures when the period of the pulse of a near-fault motion is close to the fundamental period of the building, while maximum ductility demand occurs in the bottom stories in relatively short period structures [Krawinkler et al, 2003].

The stiffness and strength of beams, columns, and in particular the joint panel zones (JPZs), significantly affect the distribution of ductility demand along the height of buildings [Krawinkler and Popov, 1982; Tsai and Popov, 1988]. There are three broad approaches to design of JPZs, namely, weak design that allows ductile shear yielding of JPZ to dissipate energy, strong design that ensures that JPZs remain elastic while ductile flexural plastic hinges are mobilised in the beams, and balanced design that allows controlled energy dissipation in both JPZs and beams [Bertero et al., 1972; Krawinkler, 1978; Englekirk, 1999].

Another important aspect that affect moment frame behaviour is strong column weak beam (SCWB) design that helps distribute inelasticity, and hence, ductility demand over a larger number of members across the structure, and mostly in beams [Roeder, et al., 1993; Schneider, et al., 1993]. To achieve this, connections need to be properly designed to have adequate plastic rotation capacity without strength degradation. It is expected that connections in special (SMFs), intermediate (IMFs) and ordinary moment frames (OMFs) would withstand plastic connection rotations of 0.03, 0.02, and 0.01radians, respectively [AISC, 1997]. The current approach is to ensure that connections retain their strength till interstory drift angle of 0.04 and 0.02 radian is attained in SMFs and IMFs, respectively, and that the connections remain elastic in OMFs [FEMA 350, 2000; AISC, 2002, AISC 2005; IS 800, 2007; AISC 2010]. In summary, reliance is placed on inelastic rotation capacity at beam-to-column connections in moment frames for resisting earthquake shaking. Accordingly, most connection types developed and pre-qualified are required to have maximum rotation capacity of 0.04 radian, or plastic rotation capacity of 0.03 radian. These correspond well with the expected performance levels in terms of acceptable structural, nonstructural, and content damage levels. Thus, few connections are available that have significantly larger rotation capacity. Consequently, seismic demand exceeding the capacity limit would cause brittle connection failures; there is a need to ascertain maximum intensity measure (say, PGA) of earthquake shaking that such connections are capable of resisting.

3. NUMERICAL STUDY

Two benchmark buildings, a six story and a twenty story office buildings with peripheral moment resisting frame system are chosen for the numerical study (Fig. 3.1) [Tsai and Popov, 1988]. The beam and column sections of the two buildings are listed in Table 3.1. The buildings are modelled and non–linear time history analyses are performed of the building models using structural analysis program PERFORM 3D [PERFORM V4, 2006]. Lineal frame members with nonlinear hinge properties are used for modelling beams and columns [FEMA 273, 1997]. Strong JPZ design is adopted in order to assess upper bound seismic demand on connections, but panel zones are explicitly modelled to capture both strength and stiffness contribution using established procedure [*e.g.*, FEMA 355C, 2000]. Incremental dynamic analysis (IDA) is carried out of the two building models using five earthquake ground motions as listed in Table 3.2. The earthquake records are scaled to PGAs of 0.16g, 0.24g, 0.36g, 0.5g, 0.6g, 0.7g, and 0.8g, where g is acceleration due to gravity. Here, the first three values represent the PGAs of the highest three seismic zones of India [IS 1893 Part 1, 2002]. Thus, PGA is used as in intensity measure and joint plastic rotation as the damage measure in the IDA.

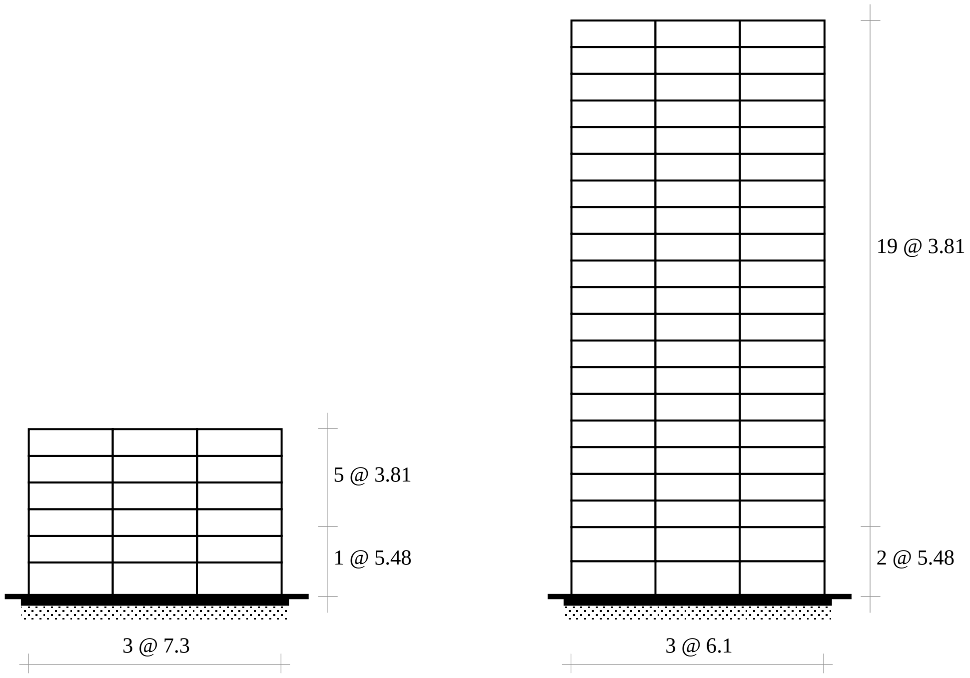


Figure 3.1. Elevation of six and twenty story study buildings

Table 3.1. Beam and column sections of the two buildings

	6 story building (named OM6)			20 story building (named OM20)		
Story	Beam	Column		Beam	Column	
		Outer	Inner		Outer	Inner
20 / Roof	…	…	…	W27×84	W14×109	W21×122
19						
18				W30×99	W14×132	W24×146
17						
16				W30×99	W14×159	W24×146
15						
14				W30×116	W14×176	W24×162
13						
12				W30×116	W14×211	W24×162
11						
10				W30×116	W14×257	W24×162
9						
8				W30×116	W14×283	W27×178
7						
6 / Roof	W21×68	W14×99	W18×86	W30×116	W14×311	W27×178
5						
4	W24×94	W14×132	W21×122	W30×116	W14×342	W27×178
3						
2	W27×94	W14×159	W24×146	W30×116	W14×370	W30×191
1						
Basement	…					

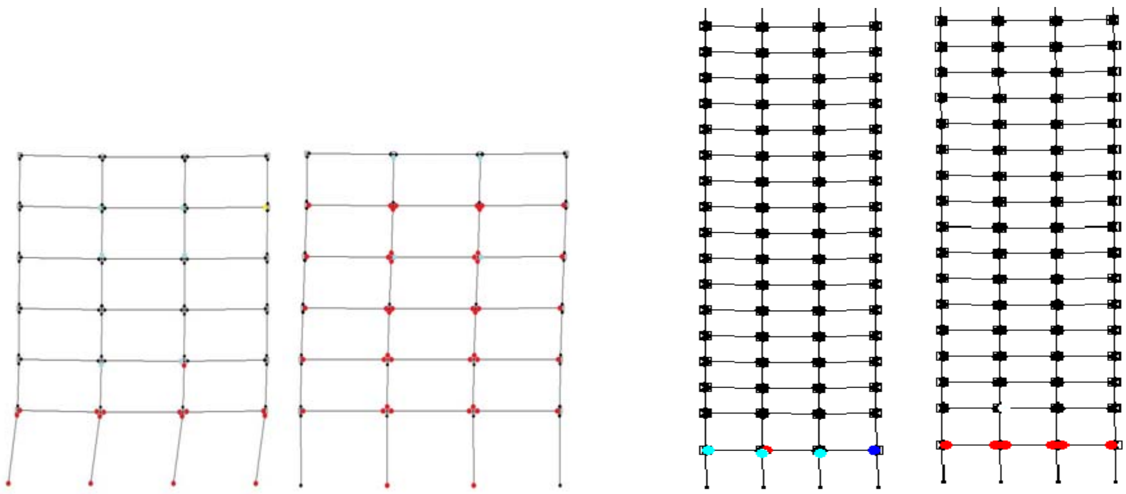
Table 3.2. Suite of five ground motion records used in IDA

Earthquake Name	Year	Component	PGA (g)	Station
Elcentro	1940	N00S	0.3129	Imperial Valley
Parkfield	1966	N65E	0.4759	Cholame
Loma Prieta	1989	S90W	0.4794	Corralitos
Northridge	1994	N52E	0.6038	Sylmar CA
Chamoli, India	1999	N70W	0.1987	Gopeshwar

Inelasticity is concentrated in the lower story in both six and twenty story *original* buildings (with model names OM6 and OM20) owing to higher flexibility of the lower story/stories. This is caused due to larger column lengths compared to upper stories while using same column section. This is a common practice in most building designs. This proves that sway mechanism with distributed inelasticity along the building height at beam ends is difficult to achieve using the current practice; columns being stronger than the beams by certain margin (SCWB design) do not necessarily ensure the same.

To eliminate concentration of inelasticity in the lower (bottom) stories, the columns in the lower stories are re-designed and larger sections are used such that the bottom story stiffness is comparable to the upper story stiffness. These revised design models are named as RM6 and RM20 for the six-and twenty-story buildings, respectively. The revised column sections are W14×398 and W24×370 in model RM6 (in place of W14×159 and W24×146 in the first two stories of building model OM6; Table 3.1) and W27×368 and W36×439 in model RM20 (in place of W14×370 and W30×191 in the first three stories of building model OM20; Table 3.1).

Nonlinear dynamic analyses results show progression of inelasticity along the building height in the revised models (RM6 and RM20). An example of comparative formation of plastic hinges is shown in Fig. 3.2 of six and twenty story buildings under Parkfield motion with PGA scaled to 0.36g. The spread of inelasticity is more in the six story building compared to that in the twenty story building with the revised larger columns. This indicates that storey stiffness is critical for distribution of inelasticity in frame buildings and more flexible taller storeys limit the same. Also, the revised models help is sustaining larger duration of earthquake shaking as seen from the average program run-time (of all five ground motions at each intensity level) listed in Table 3.3. This also indicates larger energy dissipation capacity of the revised models (RM6 and RM20) [Charan, 2011].



Original Building Model OM6 | Revised Building Model RM6 | Original Building Model OM20 | Revised Building Model RM20

Figure 3.2. Plastic hinges at beam and column ends under Parkfield motion with PGA scaled to 0.36g

Table 3.3. Suite of five ground motion records used in IDA

Building Model	Average analysis run-time at different intensity measures (sec)			
	0.36g	*0.6g*	*0.7g*	*0.8g*
OM6	32.26	23.81	16.71	15.95
RM6	40.20	32.21	32.19	31.96
OM20	32.03	31.94	24.06	24.06
RM20	40.20	32.05	32.01	31.98

Another important observation is the magnitude of cumulative plastic rotation demand mobilized at the beam ends. The prescriptive maximum total rotation capacity of 0.04 radians, or plastic rotation capacity of 0.03 radians, is exceeded at shaking intensity of about 0.5g to 0.6g in all the analysis models, implying possible connection failures. Thus, the current design provisions may become inadequate under severe earthquake shaking. Fig. 3.3 shows the variation of plastic rotation demand mobilized at interior and exterior joints (16%, 50% and 84% fractile response curves for all five ground motions) *versus* peak ground acceleration at first story in the six story building models (OM6 and RM6) from IDA. Similar results are obtained for the twenty story building [Charan, 2011].

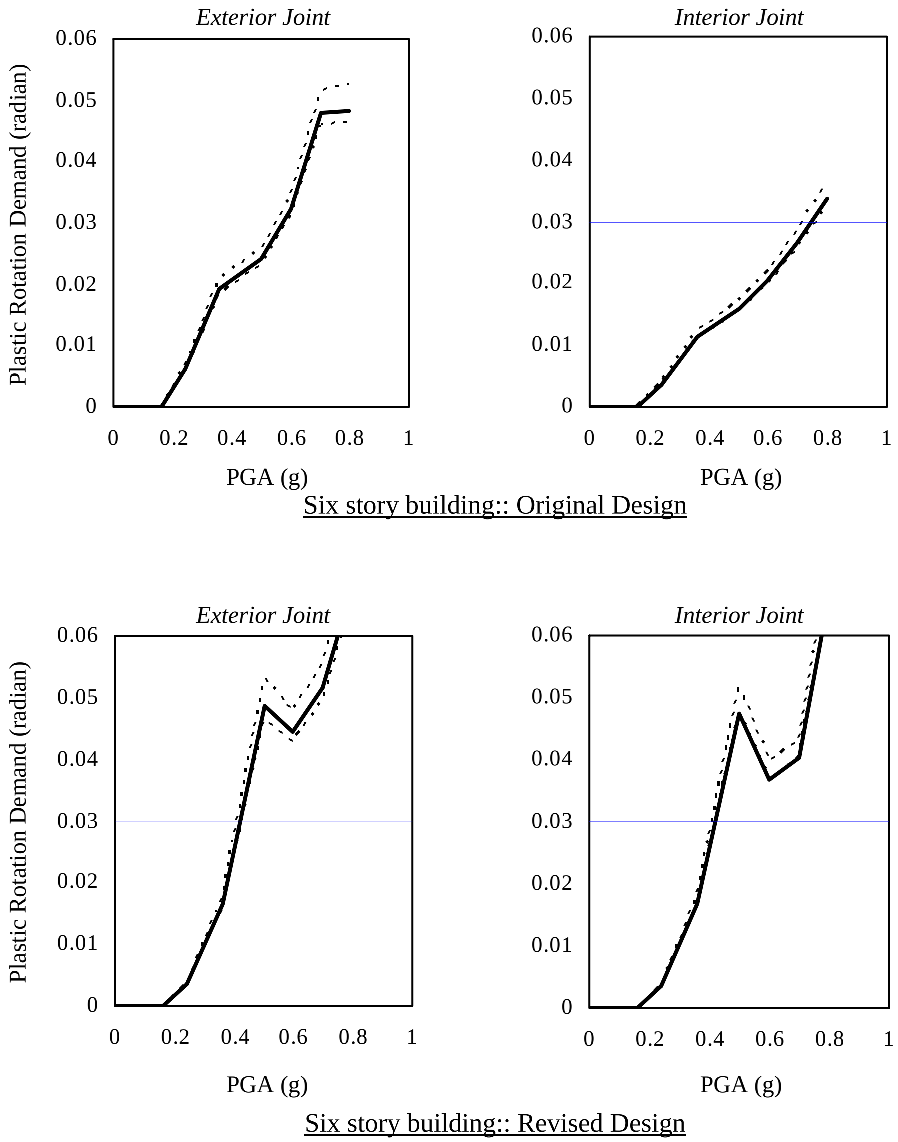


Figure 3.3. Plastic rotation demand on connections at first story under Parkfield motion with PGA scaled to 0.36g

Average plastic rotation demands imposed are shown in Fig. 3.4 at each of the first five stories in the two buildings (all four building models, namely OM6, RM6, OM20, and RM20) under the five earthquake motions. Inelasticity mobilised is low in upper stories even under strong shaking with PGA as high as 0.8g; inelastic connection rotation demand is concentrated at the relatively flexible lower stories. Inelastic connection rotation demand increases at all stories with increase in story stiffness in the lower stories. However, the effect is localised to few stories only in the taller twenty story building.

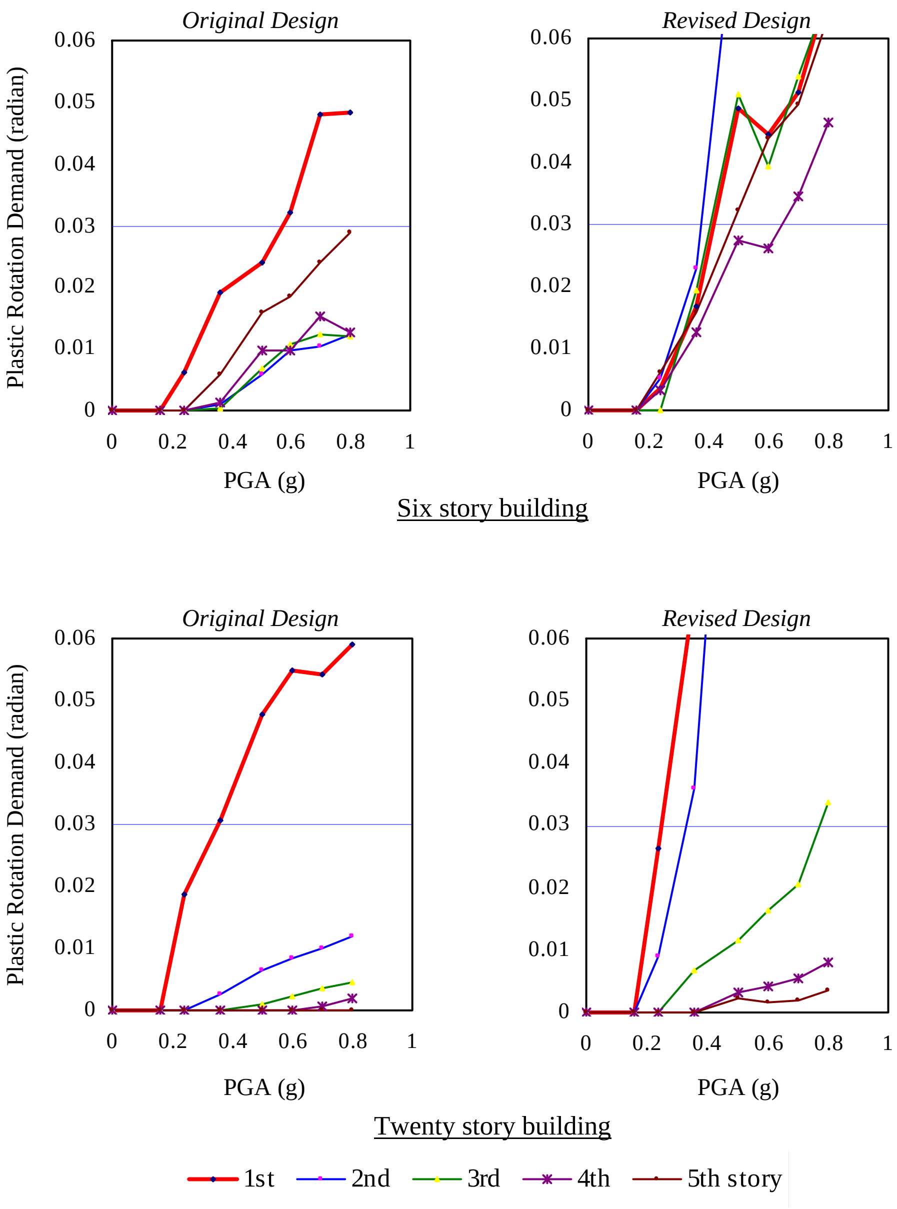


Figure 3.4. Average plastic rotation demand on connections at first five stories under Parkfield motion with PGA scaled to 0.36g

4. CONCLUSIONS

Buildings with moment frames designed according to the building codes are expected to deform into inelastic range during severe earthquakes, thereby dissipating input energy. It is important that such inelastic actions are distributed to as many different elements as possible that possess stable energy absorbing characteristics. Connections, which exhibit brittle failure modes, are required to have sufficient plastic rotation capacity (minimum of 0.03 radian) to be used in special moment frames.

This study provides answers to two important questions, namely (i) how well the inelastic action is distributed to different elements in designed structures, and (ii) what is the approximate range of seismic demand that is imposed on these connections under different intensities of earthquake ground shaking.

The following salient conclusions are drawn from the investigations undertaken as part of this study:

1. Using the same column section for first few stories makes the bottom (ground) story flexible compared to the stories above because of larger effective story height (including that due to extension of columns up to the foundation level). It is observed that inelastic connection rotation demand is concentrated at such flexible story beams and columns while the demand at higher floors is comparatively less, and often nil even at severe intensity of shaking.
2. Inelastic connection rotation demand is fairly distributed over the stories in six-story building when story stiffness is made uniform by using higher column sections. But, the effect is limited till few stories above, and thus, does not work well for taller twenty-story building used in this study. Thus, proper sway mechanism with distributed inelasticity in all floor beams may never be realized in actual mid-rise buildings.
3. Inelastic connection rotation demand depends on input ground motion. Still, in general, inelastic rotation demand on most connections in the study moment frames reaches the limit of 0.03 radians in the range of 0.36g to 0.5g PGA level. Thus, buildings may require alternate lateral load resisting systems (*e.g.*, bracings, walls) to resist stronger seismic shaking.

REFERENCES

AISC. (1997). Seismic Provisions for Structural Steel Buildings, American Institute of Steel Construction, Inc., Illinois, USA.

AISC. (2002). Seismic Provisions for Structural Steel Buildings, American Institute of Steel Construction, Inc., Illinois, USA.

AISC. (2005). Seismic Provisions for Structural Steel Buildings, American Institute of Steel Construction, Inc., Illinois, USA.

AISC. (2010). Seismic Provisions for Structural Steel Buildings, American Institute of Steel Construction, Inc., Illinois, USA.

Bertero, V.V., Popov, E.P. and Krawinkler, H. (1972). Beam-Column Subassemblages under Repeated loading. *Journal of Structural Engineering*, ASCE, **98:5**, 1137-1159.

Charan, R. (2011). Seismic Demands on Steel Moment Resisting Frames. Master of Technology Thesis, Department of Civil Engineering, Indian Institute of Technology Madras, India.

Englekirk, R.E. (1999). Extant Panel Zone Design Procedures for Steel Frames are Questioned. *Earthquake Spectra*, EERI, **15:2**, 361-369.

FEMA 273. (1997). NEHRP Guidelines for the Seismic Rehabilitation of Buildings. SAC Joint Venture, Federal Emergency Management Agency, USA.

FEMA 350. (2000). Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings. SAC Joint Venture, CA, USA.

FEMA 355C. (2000). State of the Art Report on Systems Performance of Steel Moment Frames Subject to Earthquake Ground Shaking. SAC Joint Venture, CA, USA.

Gupta, A. and Krawinkler, H. (1999). Prediction of Seismic Demands for SMRFs with Ductile Connections. SAC Phase II Steel Project Report, SAC/BD 99/06, SAC Joint Venture, CA, USA.

IS 800. (2007). *General Construction in Steel – Code of Practice*. Bureau of Indian Standards, New Delhi.

IS 1893, Part 1. (2002). *Criteria for Earthquake Resistant Design of Structures.* Bureau of Indian Standards, New Delhi.

Krawinkler, H. (1978). Shear in Beam-Column Joints in Seismic Design of Steel Frames. *Engineering Journal*, AISC, Third Quarter, 82-91.

Krawinkler,H., Medina,R. and Alavi, B. (2003). Seismic drift and ductility demands and their dependence on ground motions. *Engineering Structures*, **25**, 637-653.

Krawinkler, H. and Popov, E.P. (1982). Seismic Behavior of Moment Connections and Joints. *Journal of Structural Division*, Proceedings of the ASCE, **108:2**, 373-391.

PERFORM. (2006). Components and Elements for PERFORM 3D and PERFORM COLLAPSE Version 4, 2006, Computers and Structures, Inc., USA.

Roeder, C.W., Schneider, S.P. and Carpenter, J.E. (1993). Seismic Behavior of Moment-Resisting Steel Frames: Analytical Study. *Journal of Structural Engineering*, ASCE, **119:6**, 1866-1884.

Schneider, S.P., Roeder, C.W. and Carpenter, J.E. (1993). Seismic Behavior of Moment-Resisting Steel Frames: Experimental Study. *Journal of Structural Engineering*, ASCE, **119:6**, 1885-1902.

Tsai, K.C. and Popov, E.P. (1988). Steel Beam-to-Column Joints in Seismic Moment Frames. Report No. UCB/EERC-88-19, Earthquake Engineering Research Center, College of Engineering, University of California at Berkeley.

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