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PRELIMINARY INVESTIGATION ON THE OVERSTRENGTH AND FORCE REDUCTION FACTORS FOR INDUSTRIAL RACK CLAD BUILDINGS

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ABSTRACT: Rack Clad Building (RCB) is a type of warehouse structure built entirely using steel storage racks. Now-a-days, these structures are being built in many places around the world including high seismic zones. However, there exists no seismic design guideline for these structures. In this study, the overstrength and ductility related force reduction factors have been investigated for these structures. Several RCB moment resisting frames were modeled and analyzed in S-Frame structural analysis and design software using incremental dynamic time history and nonlinear static analysis procedures. The effects of frame height on the force reduction factors have also been investigated. The preliminary values of the force reduction and overstrength factor were found to be 2.06 and 1.06 respectively. The values obtained in this study are conservative and within the limit proposed by previous researchers.

1. Introduction

In recent years, RCB structures have become common around the world. The primary building block of the RCB structures are steel storage racks, which could be found in hardware stores and supermarkets. The RCB structures have some advantages compared to traditional steel structures. They can be easily disassembled and moved to a new location if required. As no welding or riveting is required, these structures can be constructed very rapidly. Unlike steel storage racks, RCB structures have to resist environmental loads. These structures have to withstand the full force of wind and earthquake; otherwise, they might collapse and cause severe personal and property loss. As this structural system is relatively new compared to the traditional steel structures, very little effort has been put into the development of a seismic design guideline for them. The Canadian standard CSA A344.2 (2005) provides some guidelines to design steel storage racks, but no similar guidelines exist for designing rack clad building structures. Also in the United States, FEMA-460 (2005) and the Rack Manufacturing Institute (RMI 2012) provide seismic guidelines for designing steel storage racks but not for RCBs.

In this study, pushover and incremental dynamic time history analyses have been carried out to calculate the two most important parameters for seismic design, e.g. the overstrength and ductility related force reduction factor. These two factors are used in the National Building Code of Canada (NBCC 2010) to reduce the elastic base shear demand by taking advantage of structural reserve strength (Overstrength) and the building's capacity to dissipate energy by going into a nonlinear range of response (Ductility).

2. Force-Based Design

Most building codes around the world recommend force-based design (FBD) procedure for designing buildings against earthquake loading. The NBCC (2010) also recommends force-based seismic design methods and provides a list of ductility and overstrength factors for different types of reinforced concrete frames (Mitchell et al. 2003). Force-based design is primarily based on estimated seismic demand force that is calculated using empirical equations or simple response spectrum analysis. In NBCC (2010), the design seismic base shear demand for a structure is calculated by dividing the elastic base shear demand (V_E) by the product of the ductility related force reduction factor (R_d) and the overstrength factor (R_o) also known as the response modification factor (R) (Lee et al. 1999). This is the most important parameter for current force-based seismic design. The reduction of base shear is based on the observation that a well-detailed structure is able to sustain lateral force in excess of its design strength and can undergo a large amount of deformation without collapse (Kim and Choi 2005). The reduction of the elastic base shear demand is calculated by the following equation available in NBCC (2010).

$$V_d = \frac{S(T_a)M_v\, I_E W}{R_d R_o} \tag{1}$$

Where,

V_d is the design base shear demand

$S(T_a)$ is the spectral acceleration demand

M_v is a factor to account for higher mode effects

I_E is the importance factor

W is the seismic weight of the structure

The numerator of equation 1 is the elastic base shear demand (V_E). The higher the value of the factors "R_d" and "R_o" the lower the base shear demand is, hence leading to more economic design.

At present, the RCB structures are also designed using equivalent lateral force procedure, which uses the above-mentioned two factors. As there are no design guidelines for RCB structures, engineers normally use experience and practical judgment in guessing the values for these factors (Haque and Alam 2013).

Figure 1 provides the definition of these factors and their correlation. This figure also shows a representative nonlinear behaviour of a frame structure in terms of base shear versus roof top displacement. In the same figure, an idealized bilinear elastic-perfectly plastic representation of this nonlinear behaviour is shown. This idealization was done as per FEMA 356 (2000) recommendations. A notable parameter in this figure is ductility (μ), which is defined as the ratio of maximum displacement (Δ_{max}) to the displacement (Δ_y) corresponding to the global yield (V_y) of the idealized bilinear curve. Here the factor 'R' is the ratio between the elastic base shear demand (V_E) and the design base shear (V_d). A higher value of 'R' signifies higher energy dissipation capacity (Kim and Choi, 2004). In this figure the ductility related force reduction factor (R_d) is indicated by the ratio between elastic base shear (V_E) and yield base shear (V_y). The overstrength factor is defined as the ratio between the yield base shear (V_y) and the design base shear (V_d).

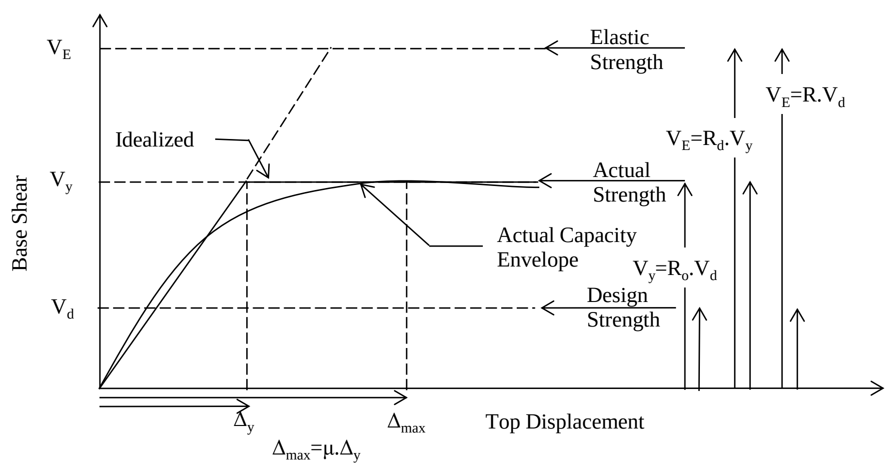

Figure 1: Relationships between force reduction (*R_d*), overstrength (*R_o*), response modification factor (*R*) and displacement ductility (μ) (adapted from Mwafy and Elnashai, 2002)

2.1. Response modification factor calculation in literature

The standard method for calculating the force reduction factor and the overstrength factor can be found in literature. Asgarian and Shokrgozar (2009) calculated response modification factors for buckling restrained braced frames (BRBF) for the Iranian building code. They carried out nonlinear static analyses to calculate the over-strength factor and incremental dynamic collapse analyses to calculate the force reduction factor. They found out that the force reduction factor decreases very rapidly with the increase of frame height but the overstrength factor decreases very little for the same change of height.

Annan et al. (2009) presented the values and calculation method of overstrength factor for modular steel building (MSB), a type of structure for which structural members are made in factories and brought to the site and joined together. They carried out their research in the context of Canadian building design code.

A more comprehensive analysis for determining the force reduction factor ("*R*") can be found in Mwafy and Elnashai, (2001). They carried out a study for the calculation of force reduction or "*R*" factor "supply" for a wide range of medium rise reinforced concrete buildings. Both inelastic pushover and incremental dynamic collapse analyses were carried out for the calculation of the "*R*" factor.

2.2. Current standard in rack industry

The Rack Manufacturing Institute (RMI 2012) recommends a response modification factor (R) of "Six" for the down-aisle moment-resisting frame (MRF) and "Four" for the braced frame in the cross-aisle direction. The values suggested in RMI (2012) are independent of structural steel types (hot-rolled or cold-formed) and the degree of connection flexibility. On the contrary, Euro code 8 does not explicitly mention rack structures or buildings. However, for members made of cold formed steel with semi-compact class 3 sections, it assumes a value of behavior factor (Response modification factor) less than or equal to 2 for seismic design. Euro Code 8 does not provide any recommendation for slender class 4 sections with perforations that are used generally in rack columns. Beattie (2006) proposed a design guideline for steel storage racks where he suggested that for both cross-aisle and down-aisle direction, the maximum ductility used for design should be 1.25 and in no cases should it go beyond 3.0 for down-aisle direction except detailed study suggests otherwise.

3. Analytical Assumptions

The following studies were taken into consideration during finite element modeling of the RCB frames. Filiatrault et al. (2006a) developed a simple analytical model to capture the behavior of steel racks under seismic excitation in the down-aisle direction. In the analytical model, they assumed that the beam-to-column joint and the column-to-baseplate connections have lower moment rotation stiffness than those of the beams and columns. As a result, during lateral loading, only the connections experience a nonlinear response. On the other hand, the beams and columns essentially remain elastic. Filiatrault et al. (2006a) verified this assumption using shake table testing of the rack structures. As the beam-to-column and the column-to-baseplate joints have significantly lower stiffness than the connected members, the connectors serve as weak links or structural fuses during seismic loading. Consequently, these connections go into a nonlinear range of response well before the beams or columns reach their yield capacity. Due to the presence of such flexibility at the joints, the possibility of the beam or columns failing due to warping is significantly low.

4. Lateral Force Resisting System

The basic components of steel storage racks are shown in Figure 2(a). The frame system consists of upright (column) posts with openings at a regular interval for connecting the beams on one side and the braces on the other side, which can be seen in Figure 2(b). They rely on portal frame action in the down-aisle direction and frame action in the cross-aisle direction to resist lateral loads (Beattie 2006). Often braces and cables are used in the down-aisle direction to reduce the horizontal deflection due to lateral load.

The moment resisting frame systems used in the down-aisle direction of steel storage racks generally incorporate teardrop beam-to-upright connections (Figure 2(b)). Although teardrop connections appear to be similar to those of the steel moment-resisting frames, they behave differently than the connection systems commonly used in buildings (Filiatrault et al. 2006a). Figure 3(a) shows a typical cross section of an RCB column. The RCB columns are generally made of 1.8mm to 3mm thick cold formed steel. The commonly used shapes are known as Ω sections. Beams are generally rectangular box sections with thicknesses varying from 1.5mm to 1.8mm. The beam depths usually range from 72mm to 150mm and width around 50 mm. Braces are generally made of small channel sections.

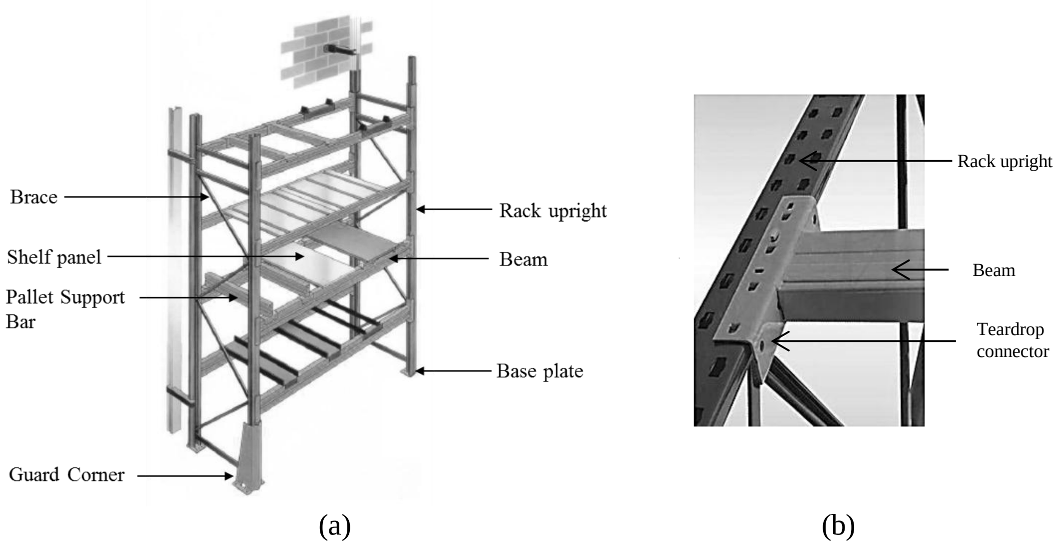

Figure 2: (a) Basic components of a steel storage rack (b) Teardrop connector (adapted from Saar Lagertechnik GmbH (2010))

The inelastic rotation capacity of the beam-to-upright connections of rack structures is significantly high; for example, the connection hysteresis adapted from Beattie (2006) for this study has exceeded 0.068 radians. Figure 3(b) shows the simulated moment rotation hysteresis of a rack beam-to-column joint presented by Beattie (2006). The hysteresis shows significant pinching and strength degradation, which is very different from the beam-to-column joints of a typical steel structure.

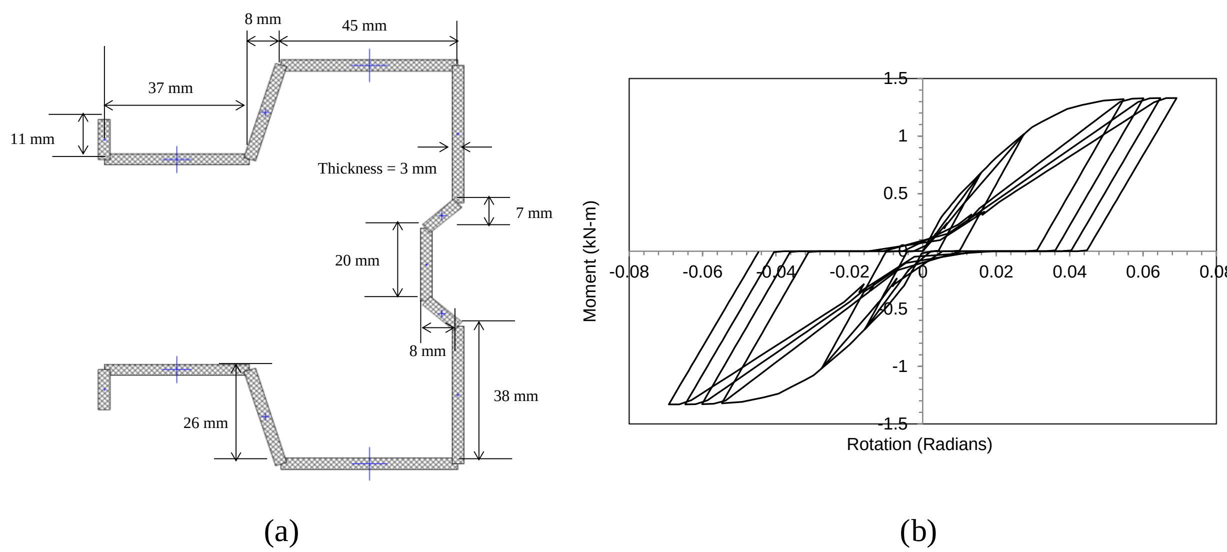

Figure 3: (a) Cross section of the rack column (b) Rack beam-to-column connection hysteresis simulated using pivot hysteresis model

Filiatrault et al. (2006a) found out from experimental study that the maximum rotation capacity of rack connectors could be as high as 0.2 radians. In contrast, building moment-resisting connections have inelastic rotation capacities of around 0.04 radians for special moment-frame systems. As these structures have low story heights with long fundamental periods compared to general building structures, the rotational demand at the beam-to-column joints also becomes very large in order to withstand strong earthquake ground motions.

5. RCB Frame Modeling and Pushover Response

Figure 4 shows the selected RCB frames for force reduction factor calculation. In this study the beam-to-column joint moment-rotation hysteresis has been adopted from the experimental results of Beattie (2006), which were simulated in S-Frame (2014) by the pivot hysteresis model (Dowell et al, 1999) as shown in Figure 3(b). More details on the model validation can be obtained from Haque and Alam (2013).

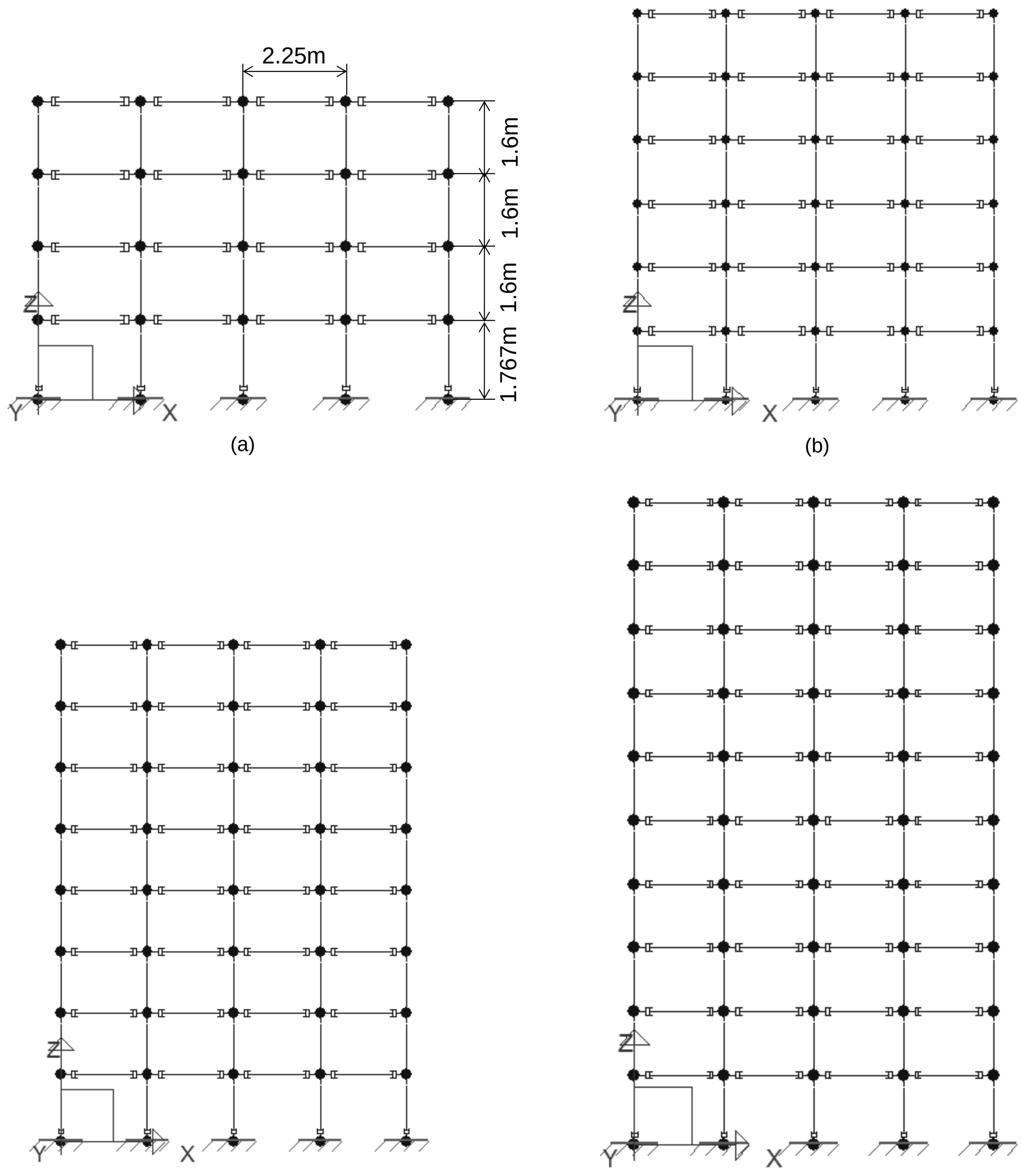

Figure 4: RCB frames modeled for R_d and R_o calculation (a) four (b) six (c) eight and (d) ten story

Material properties and some finite element model details are as follows: the selected frames have similar story heights where the first story height is 1.767 m and the subsequent stories are 1.6 m. Each bay is 2.25 m wide. Modulus of elasticity of steel was set as 200 GPa. A uniformly distributed load of 2.37kN/m was applied on the beams. Columns are connected to the ground with semi rigid springs with the same moment rotation hysteresis as the beam-to-column joints.

The seismic load of the frame was calculated by taking the contribution of full dead load. For storage load, two load reduction factors were used for the storage load: one was the area reduction factor (0.8) and the other one was for disconnection of pallets from the shelves during seismic excitation (0.66). Their multiplication results in a value of 0.536 (Beattie 2006). The calculated seismic load of the frames under the "DL+0.536LL" load case is 87.7kN, 131.5kN, 175.16kN, and 218.7kN for the 4, 6, 8 and 10 storied frames, respectively. Columns used in the RCB frame have similar cross sectional dimensions as shown in Figure 2(c) and all the beams are 50mmx100mmx1.5mm rectangular hollow section. All of these frames have similar beam and column dimensions. The only thing that varied is the beam-to-column connector stiffness, and were designed using tentative R_o and R_d values approximated from pushover analysis using Riddell et al. (1989) method.

The RCB columns contain perforations at regular intervals. Modeling a large frame using perforated column section requires the use of shell elements, which is very time consuming. In order to avoid this, the RCB models used in this study were built using beam elements. As omega section is not available in S-Frame (2014) software, a general section was created and all sectional properties were directly provided in that section. These section properties (Table 1) were collected from Haque and Alam (2013).

Using the calculated section properties and the simulated connector backbone curve (Figure 3(b)), pushover analyses were carried out on the RCB frames. Figure 5 shows the base shear vs. roof displacement curves for four, six, eight and ten storied RCB frames. The straight-line curves drawn along these curves are the bilinear idealizations, which were used for the calculation of overstrength factors. The areas under the actual and idealized curves were made approximately same using trial and error procedure. These calculated values are presented in Table 2.

Table 2 presents the values of the global yield base shear and base shear at first structural yield along other important parameter values. The base shear force at the first structural yield was calculated by observing connector moment values simultaneously with the base shear forces from each step of the pushover analysis. From the ratio of yield base shear and base shear at first yield, the overstrength factor values were calculated.

Table 1: Mechanical properties of the RCB column section

Mechanical Property	**Value**	**Unit**
Cross Sectional Area, A	1053.26	mm2
Torsional Constant, J	3117.16	mm4
Moment of Inertia around 3 axis, I_{33}	1651176	mm4
Moment of Inertia around 2 axis, I_{22}	1029853	mm4
Shear area in 2 direction, A_{s2}	406.16	mm2
Shear area in 3 direction, A_{s3}	587.53	mm2
Radius of gyration around 3 axis, r_{33}	39.64	mm
Radius of gyration around 2 axis, r_{22}	31.26	mm

Table 2: Overstrength and Ductility Factor Calculation from Pushover Analysis

Description	**Four Storied**	**Six Storied**	**Eight Storied**	**Ten Storied**
Base Shear at Global Yield, V_y (kN)	6.1	4.50	2.85	2.46
Yield Displacement, Δ_y (m)	0.17	0.18	0.19	0.19

Ultimate Displacement, Δ_u (m)	0.47	0.46	0.38	0.36
Base shear at First Yield, V_{1y} (kN)	5.37	4.17	2.75	2.46
Overstrength Factor, $R_o=V_y/V_{1y}$	1.14	1.08	1.04	1.00
Ductility= Δ_u/Δ_y	2.76	2.57	2.00	1.89

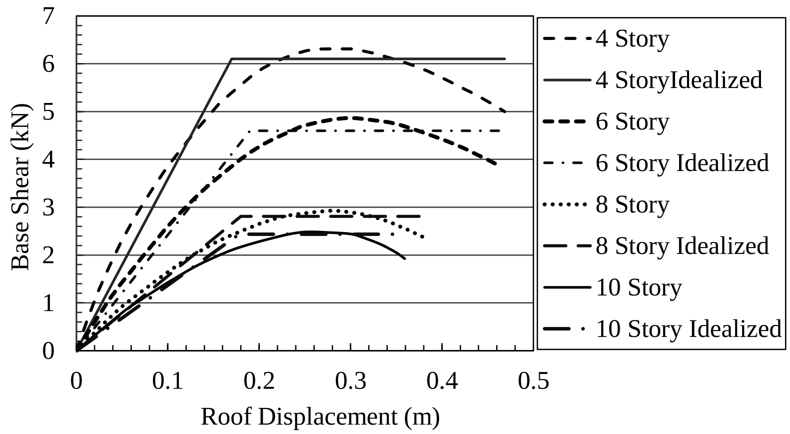

Figure 5: Over strength factor calculation for four bay wide RCB moment resisting frames using pushover analysis

6. Force Reduction Factor Calculation

At the first step, the RCB frames were designed against the code specified response spectrum. The tentative R_o and R_d values were calculated using pushover analysis and Riddell et al. (1989) method. During the design process, it was observed that, the selected beams and columns had adequate reserve strength and did not require any change in section size but the beam-to-column connectors required design revisions. The connector design was revised as per demand and practical moment rotation limits were checked using Prabha et al. (2010) model. The revised connector designs are presented in Figure 8. It can be observed that with the increase in story height both stiffness and strength of the connectors need to be increased in order to satisfy the increasing moment demand at the beam-to-column joint.

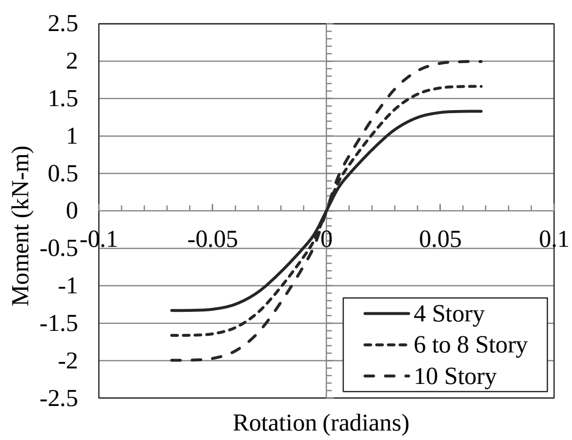

Figure 8: Beam-to-column moment rotation backbone curves for four, six, eight and ten storied frames

The force reduction factor is defined as the ratio between the elastic base shear ($V_{elastic}$) and the inelastic base shear capacity ($V_{inelastic}$) of a structure. The inelastic base shear capacities of RCB frames were determined using incremental dynamic time history analysis. A set of ten earthquake records was selected and was scaled to match the Vancouver soil class ‘C’ response spectrum using SeismoMatch

(2013) software. This software uses the wavelet algorithm proposed by Abrahamson (1992) and Hancock et al. (2006). The matched spectra are shown in Figure 9. At the next step, these earthquake records were scaled using different scaling factors. Finally, these scaled records were used to carry out non-linear dynamic time history analyses on the RCB frames. During the incremental dynamic analyses (IDA), the base shear and the roof top displacements were recorded for each record. These analyses were carried out for gradually increasing scale factors until the frame finally became unstable under lateral load. The final base shear forces before global instability were recorded as collapse base shear and denoted as $V_{inelastic}$. This final scaled earthquake was used to run a linear dynamic time history analysis of the same frame and the maximum base shear was recorded. This base shear was denoted as $V_{elastic}$.

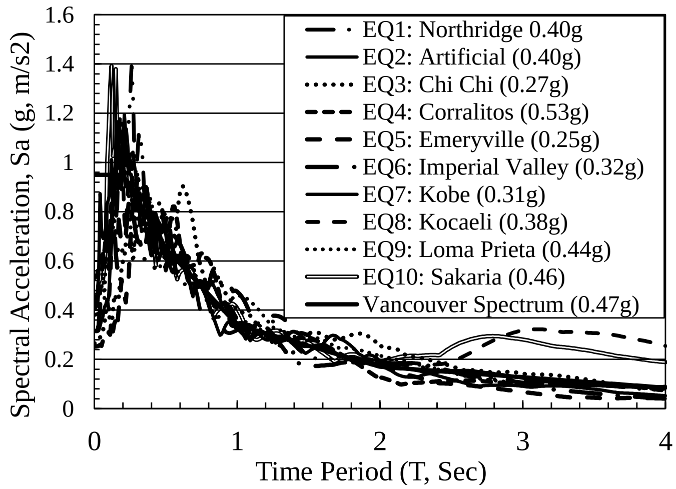

Figure 9: Response spectra of ten earthquake records matched with Vancouver response spectrum (PGA values of the matched records shown inside bracket).

The ratio between $V_{elastic}$ and $V_{inelastic}$ is denoted as the force reduction factor (R_d) and presented in Figure 10. From this figure, it can be observed that the values of force reduction factors reduce with the increase in story height with exceptions found in the Northridge, Artificial and Kocaeli earthquake records. The average R_d value for four, six, eight and ten storied frames were 2.58, 2.49, 1.65 and 1.53, respectively

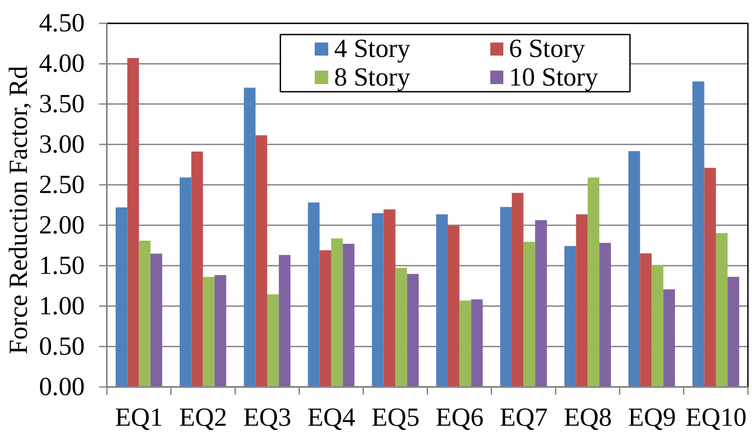

Figure 10: Force Reduction Factors (R_d) for Different Earthquake Records

7. Performance Evaluation

To evaluate the efficacy of the R_d and R_o factor calculated above, the above-mentioned four frames were again designed using the factors calculated in this study. The Vancouver soil class "C" response spectrum was used for the design. After the design was complete, the frames were analyzed using nonlinear time history analysis procedure. Previous ten earthquake records were selected for this analysis. After performing the time history analysis, roof drift data were collected from the middle point of the roof. Roof drift ratio is an important parameter in seismic design. It is necessary to control high roof drift ratio in order to avoid structural hammering, non-structural damages, and merchandise falling from the racks. Building code provided roof drift ratios ensure that none of these incidents will take place. The maximum roof drift values are presented in Figure 10. It can be seen from the figure that for different earthquake records the roof drift values vary quite significantly ranging from 142mm to 401mm for the four frames. The average roof drift value was found to be 243mm which is approximately 3.7% of the story height. This average roof drift ratio is significantly high compared to traditional residential or factory buildings but according to RMI (2012) this is normal for rack type storage structures due to their high beam-to-column connector flexibility. Furthermore, during shake table testing of racks Filiatrault et al. (2006b) observed that the transient interstory drift values experienced by the racks were very high (3.8%-9.1%) compared to general steel structures. The residual interstory drift values were also very high (0.5%-2.6%); however, the racks did not lose their vertical load carrying capacity at these high drift values. Similar result was also observed for six, eight and ten storied RCB frames. The empty places in this figure (e.g. EQ8 Kocaeli) are due to the instability of the RCB frames that occurred during the time history analysis. It can be observed that with an increase in frame height, the roof drift ratios from nonlinear time history analysis become lower and closer to 2.5%.

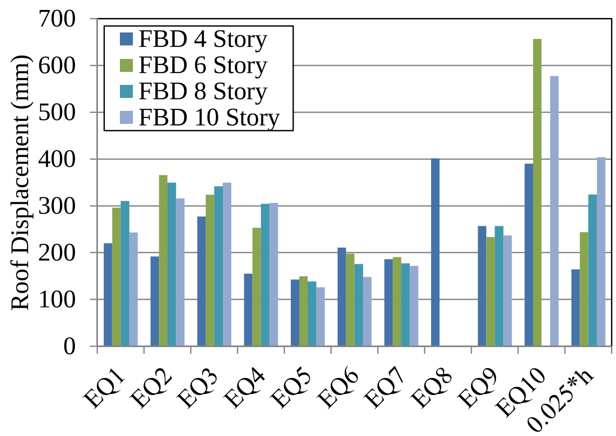

Figure 10: Roof drift of the RCB frame from NLTHA

8. Conclusion

Several RCB frames were modeled with different frame heights. Pushover and incremental dynamic time history analyses were carried out on these frames using nonlinear beam-to-column and column-to-baseplate connector moment-rotation hysteresis. The overstrength and force reduction factors were calculated from pushover and incremental dynamic analyses, respectively. It was observed that both overstrength and ductility related force reduction factors decrease with the increase of frame height. The average R_d value obtained from the dynamic analysis is 2.06 and Ro value from nonlinear static analysis is 1.06. Although rack connections show a larger deformation capacity than traditional SMRF systems, their ductility is lower because of their very high yield deformation. Furthermore, energy dissipated by the beam-to-column joint is much less compared to the traditional connection because of the pinched hysteresis loop and high stiffness degradation. For these reasons, the calculated R_d factors from IDA analyses were lower than traditional SMRF systems. It was also observed that the ductility capacity of the structure reduces with the increase of frame height. The average ductility value obtained for these frames was 2.3.

9. Acknowledgements

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