$\beta_{EDP|IM} = \sqrt{\frac{\sum_{i=1}^{N}(ln(EDP)-ln(aIM^{b}))^{2}}{N-2}}$ (2), N= number of total simulation cases

With the probabilistic seismic demand models and the limit states corresponding to various damage states, it is now possible to generate the fragilities (the conditional probability of reaching a certain damage state for a given *IM*) using Equation 3 (Nielson 2005).

$$P[LS|IM] = \varphi[\frac{ln(IM)-ln(IM_n)}{\beta_{comp}}] \quad (3)$$

where, $\varphi[]$ is the standard normal cumulative distribution function and

$$ln(IM_n)= \frac{ln(S_c)-ln(a)}{b} \quad (4)$$

$ln(IM_n)$ is defined as the median value of the intensity measure for the chosen damage state, a and b are the regression coefficients of the *PSDM*s, and the dispersion component is presented in Equation 5 (Nielson 2005).

$$\beta_{comp}=\frac{\sqrt{\beta_{EDP|IM}+\beta_c^2}}{b} \quad (5)$$

where, S_c is the median and β_c is the dispersion value for the damage states of the column and buildings.

DESIGN OF COLUMNS AND FRAME STRUCTURE

This section briefly describes the configuration of high strength RC column (HSRC) and normal strength RC column (NSRC) used in this study. The column is an interior column of the ground floor of a twelve storey building which has five bays in both directions with the same bay length of 5m (16.4ft) each. This RC building was analyzed as per NBCC (2005) and the columns were designed according to CSA A23.3-04 (2004) as ductile moment resisting frames. The building is located in the city of Vancouver, and its seismicity is obtained from NBCC (2005). The columns have been designed with the maximum moment and shear forces developed during the analysis considering all possible load combinations specified in NBCC (2005). The size of longitudinal rebars and spacing of transverse reinforcement was selected following current code requirements CSA A23.3-04 (2004).

The cross sections of both HSRC and NSRC are shown in Fig. 1. The HSRC is a 600 mm (23.5 inch) square column reinforced with 16-No.7 (22 mm diameter) high strength steel, f_y= 800 MPa (116 ksi) and high strength concrete, f_c= 90 MPa (13 ksi). The NSRC is a 700 mm (27.5 inch) square column reinforced with 20-No.8 (25 mm diameter) regular strength steel, f_y= 450 MPa (65 ksi) and normal strength concrete, f_c= 35 MPa (5 ksi). The total height of the column section was 3500 mm (11.5 ft). To ensure flexural dominated behaviour and avoid shear failure, an aspect ratio (cantilever height to equivalent column diameter) of 5.1(for HSRC) and 4.45 (for NSRC), a longitudinal reinforcement ratio of 1.7% (for HSRC) and 2% (for NSRC), and lateral steel meeting the current seismic provisions were used. The column rotation is fixed at the top but movement is free.

The same building is also considered for fragility assessment of building structure built using HSMs. Fig.2 shows the plan and elevation of the twelve story building considered in this study. The building was designed for both high strength and normal strength materials. The high strength moment resisting frame (HSMRF) was designed using high strength steel, f_y= 800 MPa (116 ksi) and high strength concrete, f_c= 90 MPa (13 ksi) and the normal strength moment resisting frame (NSMRF) was designed using regular strength steel, f_y= 450 MPa (65 ksi) and normal strength concrete, f_c= 35 MPa (5 ksi). The reinforcement of the building has been detailed as per Canadian standards (CSA A23.3-04) and Tables 1 and 2 show the member sizes and the reinforcement detailing of the columns and the beams, respectively. The 20 columns located along the perimeter of the buildings are designated as C2, and the remaining interior 16 columns are designated as C1.

FINITE ELEMENT MODELING

The analytical model of the columns and buildings have been developed using the SeismoStruct nonlinear analysis program (SeismoStruct 2011). For simplicity, the buildings were modeled as 2D moment resisting frames (MRFs). Only one interior frame (shown with dotted line in Fig. 2b) was modeled to represent the building. Nonlinear static pushover and incremental dynamic time-history analyses have been performed on the columns and buildings to determine their performances. The program has the ability to figure out the large displacement behaviour and the collapse load of framed structures accurately under either static or dynamic loading, while taking into account both geometric nonlinearities and material inelasticity (Pinho et al. 2007). Inelastic beam elements have been used for modeling the beam and the columns. The fibre modeling approach has been employed to represent the distribution of the material nonlinearity along the length and cross-sectional area of the member. Each fibre has a constitutive relationship, which can be specified to represent unconfined concrete, confined concrete, or longitudinal steel reinforcement. The confinement effect of the concrete section is considered on the basis of reinforcement detailing.

To develop the analytical model, Menegotto-Pinto steel model (Menegotto and Pinto 1973) with Filippou (Filippou et al. 1983) isotropic strain hardening property is used for reinforcing steel material. High strength concrete has been modelled using a uniaxial nonlinear constant confinement for high strength model, developed and programmed by Kappos and Konstantinidis (1999), that follows the constitutive relationship proposed by Nagashima et al. (1992). On the other hand, the normal strength concrete was modelled using the non-linear variable confinement model of Madas and Elnashai (1992) that follows the constitutive relationship proposed by Mander et al. (1988). Although the analysis has been performed using a freely available software, the authors have verified the software with several experimental results that consist of static and dynamic loading of structures. For instance, static pushover test of RC bridge bent by Billah (2011), shake table test of a 3-storey moment resisting steel RC frames by Alam et al. (2009), shake table test of an SMA RC column by Alam et al. (2008), quasi-static reversed cyclic loading test of SMA reinforced concrete beam-column joint by Alam et al. (2008), and SMA-FRP hybrid RC beam-column joint by Billah and Alam (2012).

VALIDATION WITH EXPERIMENTAL RESULT

Matamoros (1999) conducted a series of experiments on high strength concrete columns subjected to shear reversals into the nonlinear range of response. The applied load was displacement controlled and the load history is shown in Fig. 3. The load was applied in such a way that it forced the column well into the nonlinear range. Fig.4 shows the hysteretic behaviour of one of the eight column specimens (Specimen-C10-05S) adopted in that study. The concrete compressive strength of the column was 69.6 MPa (10 ksi) and longitudinal steel had an yield strength of 586 MPa (85 ksi). Fig. 4 also depicts the predicted load-displacement behavior of the numerical model, which seems to be fairly accurate as compared to the experimental results of Matamoros (1999). The cumulative energy dissipation was calculated 23.75 kN-m (17.52 kip.ft) from the predicted load-displacement curve, whereas the experimental result was 24.27 kN-m (17.9 kip.ft), which is only 2.2% higher than that of the calculated result. The maximum shear force at the column base was predicted as 67.72 kN (15 kip) which was only 0.5% lower than that of the experimental result of 68.06 kN (15.3 kip).

SELECTION OF GROUND MOTION

In order to establish a relationship between earthquake ground motion and structural damage, data set comprising of inputs (ground motion records) and outputs (damage) is necessary. Nevertheless, uncertainty arising from a number of sources is present in the modeling and performance assessment of RC structures, which require careful consideration while selecting models of the structure and input ground motions. The nonlinear time history analyses take the nonlinearity of the members into account, and responses of the structures are subsequently dependent on the characteristics of earthquake ground motions. So, the uncertainty characteristics of the earthquake ground motions regarding ground type, intensity and frequency contents have a great effect on nonlinear time history responses of members. Moreover, it is important to properly select input motion parameters to correlate with structural damage.

Selection of proper Intensity Measure (*IM*) plays a vital role in establishing fragility relationship. For better accuracy in *PSDM*, Luco and Cornell (2007) suggested three criteria for selecting an appropriate *IM*, i.e. efficiency, sufficiency and computability. One of the most commonly used *IM* is the spectral acceleration at the first-mode period, $S_a(T_1)$ or simply S_a. Bazzurro & Cornell (2002) demonstrated that as an *IM,* $S_a(T_1)$ tends to be less than ideal for tall and long period buildings. Several alternatives of *IM* include *PGA*, Peak Ground Velocity (*PGV*), Arias Intensity (*AI*) etc. as proposed and developed by numerous researchers for instance, Giovenale (2003) and Mackie and Stojadinovic (2007). Mackie and Stojadinovic (2007) and Padgett and DesRoches (2008) suggested that the peak ground acceleration (*PGA*) is the optimum choice to describe the severity of the earthquake ground motion because of its efficiency, practicality, sufficiency, and hazard computability. However, as a large value of *PGA* is not always followed by severe structural damage, other intensity measures such as peak ground velocity (*PGV*) (Nielson 2005), peak ground displacement (*PGD*), time duration of strong motion (T_d), spectrum intensity (*SI*), and spectral characteristics can also be considered. In this study *PGA* is used (Huo and Hwang 1996; Ji et al. 2009) as the *IM* because of its efficacy, utility and adequacy in vulnerability assessment.

A suite of 20 far field ground motions are used in this study to develop fragility curves for the HSRC, NSRC, HSMRF and NSMRF. The 20 far field ground motion histories used in the study were obtained from the FEMA P695 (ATC-63) far-field ground motion set. The characteristics of the earthquake ground motion records are presented in Table 3 and Fig. 5. All these ground motions have low to medium *PGA* ranging from 0.24g to 0.73g with epicentral distances more than 10km. In this study only one horizontal component of ground motions were considered. The strong horizontal component having higher *PGA* was selected and used in this study.

MODELING AND TREATMENT OF UNCERTAINTY

Assessment of the seismic fragility of reinforced concrete structures is associated with large uncertainties arising from uncertainties in material properties, geometric configurations and inherent uncertainties in ground motions (Nielson, 2005).Gardoni et al. (2002) classified the uncertainties affecting the structural performance into two categories, i.e. aleatory and epistemic uncertainty. Aleatory uncertainty refers to the inherent randomness of the system and epistemic uncertainty stems from the lack of knowledge, ignorance and coarse modeling. Inherent randomness in material properties is one of the main sources of uncertainty in RC structure. In this study the compressive strength of concrete and the yield strength of both longitudinal and transverse reinforcement have been considered as random variables. Past investigations (Ellingwood, 1977; Mirza et al., 1979) suggest that variability in concrete compressive strength can be characterized by a normal distribution. In this study normal distribution is employed to represent the variability in concrete strength. For representing the uncertainties in steel strength a log-normal distribution is assumed based on previous research (Ellingwood, 1977, Ghobarah, 1998). Moreover, there exist a number of geometric and analytical modeling parameters which are potentially variable in RC structures. One group of uncertain variable in geometric parameter can be the geometry and dimension of columns and aspect ratio. In this study, column dimension is considered as a random variable and it is assumed to follow a normal distribution with a mean value equal to the nominal value plus 1.6 mm and constant standard deviation equal to 6.4 mm to represent the in situ cast conditions (Mirza and MacGregor 1979). It is a common approach to consider normal distribution to represent the uncertainty associated with column dimensions (Lu et al. 2005; Aydemir and Zorbozan, 2012).One of the key design input in high-rise building is its intrinsic damping (Willford et al. 2008). In this study damping ratio is considered as the modeling uncertainty. The uncertainty in the damping ratio is modeled using a normal distribution (Fang et al. 1999). The parameters for this distribution are calculated considering the typical range of damping ratios for tall buildings – 0.02 to 0.05 (Satake et al. 2003). Table 4 shows the means, coefficients of variation (COV), probability distribution and ranges of variations of the random variables considered in this study. Uncertainties inherent in the earthquake loading are also considered by using a suite of 20 far field ground motions.

Sampling based methods are most widely used for the uncertainty analysis in seismic fragility assessment. Monte Carlo simulation method is one of the most powerful tools for uncertainty analysis but it requires large amount of sample for sufficient accuracy. On the other hand, Latin Hypercube Sampling (LHS) provides the flexibility of constrained sampling scheme instead of random sampling (Ayyub and Lai, 1989). In this study the LHS approach is used to account for the uncertainties associated with estimating the parameters discussed above. In this approach individual probability distributions are assumed for each parameter and the probability distribution range of each random variable is divided into ten intervals having similar probability. This allowed augmenting the base column and the building model by sampling upon the various significant modeling parameters to generate 10 statistically different yet nominally identical column and building samples, respectively (Pan 2007). Each column and building sample is again paired with two randomly selected ground motions. This created a total of 20 column and building samples which reflect the uncertainty in ground motion and material properties. An incremental dynamic time history analysis is performed for each column and building sample by scaling each ground motion to ten intervals. These created a total of 200 data set for columns and buildings and the demand parameter is calculated for each data set. The values of the demand parameter calculated for all ten combinations of the random variables are then used for probabilistic description of the demand for a given intensity of ground motion.

CHARACTERIZATION OF DAMAGE STATES

Defining a quantitative or qualitative measure for identifying the seismic damage level is an important step in fragility assessment (Erberik et al. 2003). Damage states (*DS*s) are often related to the structural capacity of a member or system and discrete in nature as they are labelled with various limiting values of considered damage index (Zhang and Huo, 2009). Damage states for buildings or members should be defined in such a way that each damage state indicates a particular level of functionality. For analytical fragility functions structural capacities or limit states must have a relation with the damage state in terms of damage index (*DI*). Different forms of *EDPs* are used to measure the *DS* based on global level and member level evaluation. ATC 40 and FEMA 356 defined limit states based on global behavior (inter-story drift) as well as element deformation (plastic rotation). SEAOC (1995) defined five performance levels based on transient drift (%) while Chryssanthopoulos et al. (2000) used only two limit states. On the other hand Wen et al. (2003) defined three quantitative limit states for defining performance levels. Considering all these available damage measures, FEMA 356 provides global level and member-level limit states for three performance levels for seismic evaluation. FEMA 356 defines the performance levels as immediate occupancy (IO), life safety (LS), and collapse prevention (CP). The limit states for the global-level evaluation are defined by the maximum inter-story drift, whereas for member level evaluation the limit sates are based on member plastic rotation. Table 5 shows the different performance levels and associated limit states used in this study for global (building) and member (column) level evaluation. The inter-story drift was calculated as the relative lateral displacement between floors expressed as a percent of the story height at that floor (FEMA356). For member level evaluation, plastic rotation of column was calculated following the equation proposed by ATC 40

$$\theta_p = (\varphi_{ult} - \varphi_y)\, L_p \qquad (6)$$

where, φ_{ult} = ultimate curvature, θ_{p} = plastic rotation, φ_y = rotation at yield and L_p= plastic hinge length, calculated according to Paulay and Priestley (1992) equation:

$$L_p = 0.08\, L + 0.022 d_b f_y \qquad (7)$$

where, L is the length of the member in mm, d_b represents the bar diameter in mm and f_y is the yield strength of the rebar in MPa.

Finally, the limit state capacities for the columns and frames are also presented in terms of median (S_c) and lognormal standard deviation (β_c) in Table 6 There is also uncertainty associated with each median (S_c) which must be defined. This uncertainty is given in the form of a lognormal standard deviation or dispersion (β_c). In this study the value of lognormal standard deviation or dispersion (β_c) is taken as 0.3 (Wen et al. 2003).Wen et al. (2003) derived the fragility curves for RC frames using a β_c value of 0.3 to quantify the dispersion in the drift capacity. The 0.3 dispersion is not a specific value, but was considered reasonable for this study based on the report by Wen et al. (2003). Previous studies (Ramamoorthy et al. 2006; Baiet al. 2009; Ramamoorthy et al. 2008) have also shown that this value of β_c represents a reasonable estimate of the uncertainty associated with limit state capacity.

DEVELOPMENT OF FRAGILITY CURVES

In this study probabilistic seismic demand models (*PSDM*s) are used to derive the fragility curves which help express the effect of HSMs on the seismic demand placed on the MRFs and columns. The demand parameter considered in this study is the column plastic rotation and inter-storey drift (%) for the column and building, respectively. The *PSDM*s are developed by analyzing the demand placed on the structure through a regression analysis. *PSDM*s are constructed from the peak response of the column and building obtained from the *IDA*.

Fig. 6 shows the *PSDM*s for the HSRC and NSRC for the considered far field ground motions. For generating the *PSDM*s a suite of suitable ground motions representing a broad range of values for the selected *IM*

(*PGA* in this study) was chosen. After the development of analytical models of HSRC and NSRC, *IDA* was carried out. From each analysis the peak responses (column plastic rotation) were calculated and plotted against the *IM* for that ground motion. Finally a regression analysis was carried out to estimate *a*, *b* and $\beta_{EDP|IM}$. The regression equation is shown in Figure 6, which represents the values of the regression parameters *a* and *b*. In the regression equation, the coefficient multiplied with *x*, indicates the parameter *b* and the next parameter indicates *ln(a)* from which the other regression parameter *a* was calculated. Once the two regression parameters *a* and *b* are obtained, $\beta_{EDP|IM}$ can be calculated using equation 2.

The impact of different materials on the demand models is compared in Table 7. The parameters listed represent the regression parameters from Equation 1 along with the dispersion. From this table it is evident that the NSRC yields an increase in the dispersion in the demand ($\beta_{D|IM}$) while HSRC exhibited reduction in the dispersion in the demand. On the other hand the NSRC increases the median value of the demands placed on the columns, exhibited by an increase in the parameters affecting the intercept (*ln(a)*) of the regression model. It revealed that HSMs were effective in reducing the column plastic rotation. This can be attributed to the higher yield strength of high strength steel which eventually reduced the plastic rotation demand in the HSRC. Higher yield strength allowed the HSRC to undergo large curvature before yielding which eventually resulted in lower plastic rotation (Eq.6). This difference is reflected in the median value of the demands placed on the two types of columns in which the plastic rotation in HSRC is considerably lower.

Evaluation of the fragility curves offers a valuable insight on the effectiveness of HSMs in reducing the probability of damage considering the column plastic rotation. Fig. 7 presents the fragility curves of the two different columns for plastic rotation as the *EDP*. The fragility can be directly estimated from the limit state capacity of each damage state as well as the parameters for the *PSDM*s obtained from the regression analysis. Utilizing these parameters, the fragility curves were generated using equation 3. These figures facilitate the comparison of the relative effectiveness of HSMs over NSMs in high-rise construction, and aids in expressing the effect of HSM in reducing the damage probability. Evaluation of the fragilities (shown in Fig.7) indicates that for all the damage states from IO to CP, the NSRC possesses more vulnerability as compared to the HSRC. Higher yield curvature value was observed in HSRC which eventually reduced the plastic rotation demand in HSRC. Previous researchers (Restrepo et al. 2006) have demonstrated that bridge piers reinforced with high strength steel can undergo large curvature before yielding. Moreover, the concrete compressive strength has influential effect on the plastic rotation capacity. Lopes and Bernardo (2003) experimentally demonstrated that for similar reinforcement ratio, the plastic rotation capacity increases with the increasing compressive strength. All these advantages of HSMs rendered the HSRC less vulnerable. The practical implication of the major reduction in plastic rotation is that the structure is more likely to remain serviceable after the earthquake when HSMs are used.

Finally, the fragility curves for the HSMRF and NSMRF were developed based on the FEMA 356 limit states as shown in Table 5. The PSDMs developed for the moment resisting frames are depicted in Fig. 8 and the corresponding regression parameters are listed in Table8.Comparison of the fragility curves provides valuable insight on the effect of material characteristics in exceeding the probability of FEMA limit states. Fig. 9 shows the fragility curves obtained for HSMRF and NSMRF. From Fig. 9 it can be observed that both the MRFs have very similar probability of exceeding IO under a given level of ground shaking. On the other hand, in LS and CP level the NSMRF portrays more vulnerability as compared to the HSMRF. This can be attributed to the higher drift capacity of HSMRF which eventually reduced the probability of damage under a given earthquake intensity. Structural members designed for seismic resistance using HSMs results in smaller initial stiffness and greater yield displacement (Restrepo et al. 2006) which reduced the probability of yielding of HSRC and reduced the vulnerability. Given the fragility curves, the seismic vulnerability of the two RC frames can be evaluated based on the given seismic event scenarios. If the design *PGA* for the location of the structure is 0.7g, the probability of exceeding the FEMA limit states can easily be determined from these fragility curves. For instance, the probabilities of exceeding the FEMA limits for the NSMRF are 91% for IO, 61% for LS, and 46% for CP whereas those values for the HSMRF are 88%, 49% and 30% for IO, LS and CP, respectively.

EFFECT OF UNCERTAINTIES ON FRAGILITY ASSESSMENT

In this study LHS technique was used to account for the uncertainties associated with the considered random variables. This sampling technique created ten nominally identical but statistically different set of columns and MRFs. Therefore, it is necessary to determine the sensitivity of these random variables on the fragility curves. In this study all the uncertain parameters were combined together to evaluate the probability of exceeding a certain damage state under different ground motion intensity. For brevity, the effect of uncertainties has been discussed only for the column fragility developed based on FEMA limit states. With 10 sample sets and each paired with two ground motions a total of 20 samples were generated. Again *IDA* was carried out by scaling each ground motion to ten intervals and generating 200 data sets. Fig. 10 depicts the effect of uncertainty in the probability of exceeding the IO, LS, and CP limit states in the case study columns (HSRC and NSRC). In order to consider the uncertainty envelopes, 95% (upper bound), 50% (median) and 5% (lower bound) confidence fragilities were computed. As can be seen in the figures, uncertainty considerations introduce significant variations to the probability of exceeding a certain limit state at a given ground motion intensity. It can be observed that in IO limit state, the fragility curves of all confidence level were almost identical but the fragility curves of LS and CP limit states were much different at different confidence levels. For example, at PGA of 1g the probability of HSRC exceeding the IO limit state varies between 94% to 99%, whereas the probability of HSRC exceeding the LS limit state varies from 13% to 63%. Interestingly, the difference in the fragilities of HSRC and NSRC become more prevalent at higher confidence level for all three limit states. This can be attributed to the higher variation in the material uncertainty that strongly affected the failure probabilities and thus resulted in wide variation in fragilities of HSRC and NSRC.

CONCLUSIONS

This study utilizes analytical simulation method to conduct seismic fragility assessment of RC columns and frames using high strength and normal strength materials considering material, geometric, and ground motion uncertainty. Within the scope of this study, a performance-based seismic assessment of RC columns and frames considering qualitative and quantitative limit states has been carried out using probabilistic framework. Through the process, the impact of HSMs on the probabilistic seismic demand models, vulnerability of the RC columns and frames were evaluated. The impact of HSMs on *PSDM*s was illustrated to express the shift in the plastic rotation demand of columns and the inter-storey drift demand of the MRFs resulting from the use of different strength materials.
Based on the analysis the following conclusions can be drawn:

- The numerical results in general show that both the columns and frames made with HSMs are less susceptible to seismic vulnerability compared to those of NSRC and NSMRF.
- The material properties seem to contribute significantly in the variability of structural response.
- Application of HSMs in columns of high rise building increased the yield curvature which eventually reduced the plastic rotation demand thereby rendered the column less vulnerable.
- Analyses of the fragility curves reveal that the effectiveness of different types of materials in mitigating probable damage can be measured using fragility curves for a given damage state of interest.
- 95%, 50% and 5% confidence intervals on the fragility estimates were developed to reflect the inherent epistemic uncertainty in the predicted values. Fragilities of HSRC and NSRC appear to vary widely at higher confidence level.
- Consideration of uncertainty has significant impact on the seismic fragility assessment. This fact emphasizes the importance of careful selection of uncertain variables.

Considering the performance and the relatively low seismic vulnerability of HSRC and HSMRF, the application of HSMs in RC building seems to offer several potential benefits. The fragility curves derived in this study will provide an insight into the vulnerability derivation of high rise structures using HSMs.

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Table 1- Column size and reinforcement arrangements

Building ID	Floor Level	Description	Column ID	
			C1	C2
High strength moment resisting frame (HSMRF)	1st to 8th floor	Size (mm x mm)	600x600 (23.5x23.5 inch)	600x600 (23.5x23.5 inch)
		Main reinforcement	16-No.7 (22mm)	12- No.7 (22mm)
	9th floor to roof	Size (mm x mm)	550x550 (21.65x21.65 inch)	550x550 (21.65x21.65 inch)
		Main reinforcement	16-No.6 (19.5mm)	12-No.6 (19.5mm)

Normal strength moment resisting frame (NSMRF)	1st to 8th floor	Size (mm x mm)	700x700 (27.5x27.5 inch)	700x700 (27.5x27.5 inch)
		Main reinforcement	20-No.8 (25mm)	18- No.8 (25mm)
	9th floor to roof	Size (mm x mm)	650x650 (25.6x25.6 inch)	650x650 (25.6x25.6 inch)
		Main reinforcement	20- No.7 (22mm)	18-No.6 (19.5mm)

Table 2- Beam reinforcement details

Building ID	Floor Level	Description	Beam
High strength moment resisting frame (HSMRF)	1st to 7th floor	Size (mm x mm)	300x600 (12x24 inch)
		Main reinforcement	4-No.6(19.5mm)
	7th floor to11th floor	Size (mm x mm)	300x600 (12x24 inch)
		Main reinforcement	4- No.5(16mm)
	Roof	Size (mm x mm)	250x600 (10x24 inch)
		Main reinforcement	3- No.5(16mm)
Normal strength moment resisting frame (NSMRF)	1st to 7th floor	Size (mm x mm)	350x700 (13.75x27.75 inch)
		Main reinforcement	6- No.7(22mm)
	7th floor to11th floor	Size (mm x mm)	350x700 (13.75x27.75 inch)
		Main reinforcement	6- No.6(19.5mm)
	Roof	Size (mm x mm)	300x600 (12x24 inch)
		Main reinforcement	3- No.6(19.5mm)

Table 3- Characteristics of the far field ground motion histories

EQ No	Earthquake			Epicentral Distance (km)	PGA (g)	PGV (cm/s.)
	M	Name	Station			
1	6.7	Northridge	Beverly Hills	13.3	0.42	58.95
2	7.3	Landers	Yermo Fire Stn	86	0.24	52
3	6.7	Northridge	Canyon Country	26.5	0.41	42.97
4	7.3	Landers	Coolwater	82.1	0.28	26
5	7.1	Duzce, Turkey	Bolu	41.3	0.73	56.44
6	6.9	Loma Prieta	Capitola	9.8	0.53	35
7	7.1	Hector Mine	Hector	26.5	0.27	28.56
8	6.9	Loma Prieta	Gilroy array#3	31.4	0.56	36
9	6.5	Imperial Valley	Delta	33.7	0.24	26
10	7.4	Manjil, Iran	Abbar	40.4	0.51	43
11	6.5	Imperial Valley	El Centro array#1	29.4	0.36	34.44
12	6.5	Superstition	El Centro Imp. Co.	35.8	0.36	46