Performance Based Seismic Design of Building Using Non-Linear Response History Analysis

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***Abstract*—In modern years, the term Performance Based Design is being used as a popular buzzword in the field of earthquake engineering, with the structural engineer taking keen interest in its concepts due to its potential benefits in assessment, design and better understanding of structural behavior during strong ground motions. The basic idea of Performance Based Design is to conceive structures that perform desirably during various loading scenarios. Furthermore, this notion permits the owners and designers to select personalized performance goals for the design of different structures. However, there is a need to emphasis that some minimum level or minimum acceptable criteria are required to be fulfilled by all structures.**

In the context of seismic design, it has been realized that the increase in strength may not enhance safety, nor reduce damage. The distribution of strength through the building rather than the absolute value of design base shear is now considered of importance, as endorsed by the capacity design principles. At the same time, the objective of most codes is to provide life safety performance during large and infrequent earthquakes. However, recent earthquakes have shown that structures may suffer irreparable or too costly to repair damages. Besides, inelastic behavior, indicating damage, is observed even during smaller earthquakes. It seems that Performance Based Design concepts, which consent multi-level design objectives, could provide a framework to improve the current codes; by obtaining structures that perform appropriately for all earthquakes.

Current research developments in seismic structural behavior indicate that the most suitable approach of achieving the performance objectives is by performing a damage-controlled design. The most important is to perform an evaluation process easy to be applied but that gets the main features that considerably influence the performance objective. Various recommendations are made in order to implement this ideology into the design procedure.

***Index Terms*— Damage, earthquake engineering, Performance Based Design, Safety.**

I. INTRODUCTION

Analysis and design of the structure of tall buildings to resist lateral loads due to earthquake is a factor of prime importance especially in cities like Mumbai and Delhi which are classified as Zone III or above as per earthquake zoning given in the codes. It is not possible to design a building to resist any future expected earthquake without any distress – since such design would not be economically viable. Hence, the philosophy of earthquake resistant design is that a building be able to sustain an earthquake expected to occur frequently in its life span without much damage and be able to resist earthquake of higher magnitudes which may rarely occur in its life span, with some distress, deformation but without total collapse. The building structure is designed to provide ductility arising from inelastic material behaviour and overstrength resulting from the additional reserve strength in the structure over and above design strength. With this ductile behaviour the building can resist earthquakes causing larger loads than code designed values with some distress, deformation but without collapse.

II. DESIGN OF CODE EXCEEDING BUILDINGS

For this the code recommends finding actual response of the building under various Hazard Levels and ensures the minimum performance objective is satisfied based on the usage of the building.

The response of the building using linear and non-linear time-history analysis under various expected earthquake ground motions is calculated and studied. The actual performance of the building as regards its lateral deflections, drifts, formation of plastic hinges can be studied from the results of the analysis. It can then be checked if its performance as to deflection, drifts, strains in its members are within limits in which case the building design could be acceptable although it does not satisfy some of the criteria in the code. Such design of a building based on its actual performance under earthquake loads is called Performance Based Structural Design or PBSD and is now being used to design several, especially tall, buildings which are otherwise code exceeding buildings.

India Standard IS 16700 – 2017 recommends Performance Based Seismic Design to be carried out for code exceeding building as per table 12. Currently guidelines for PBSD for RCC building is under draft state and will soon form a part of Indian Standards.

Use of flat slab and flat plate systems without the mandatory use of (1) backup Perimeter moment frames (taking 25% of base shear, in addition to 100% base shear carried by shear walls), (2) conservative "R" factor of 3 (instead of 4 for shear wall system) and (3) stringent drift limitations (Refer Fig. 1.1)

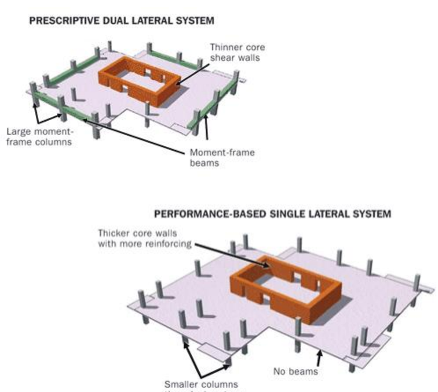


Figure 1.1 illustrates a dual structural system based on prescriptive codal provisions compared to a one developed using PBSD

Performance Levels: As per ASCE 41-17 the performance of a building's structure and nonstructural components together in a specified earthquake ground motion is defined as a Building Performance Level.

Hazard Levels: As per ASCE 7 -16 the Maximum Considered Earthquake (MCE) is defined as a Risk Targeted (MCE modified by risk coefficients) PSHA level of shaking having a 2 percent probability of exceedance in 50 years (a 2,475-year return period) with a deterministic cap at the 84th percentile level of the governing fault for a 5 percent critical damping ratio quantified in the maximum direction.

Performance Objectives

Building Type	Performance level	Seismic Hazard Level		
		SE	DBE	MCE
Ordinary Buildings	Structural	IO	LS	CP
	Non-Structural	O	NC	
Hospital Buildings	Structural	IO		LS
	Non-Structural	O		PR
Important Service and Community Buildings (other than Hospitals)	Structural	IO		LS
	Non-Structural	O	PR	LS

Ground Motions: One of the most important aspects of PBSD is the determination of demand on the structure. An accurate evaluation and representation of the seismic hazards is very critical. These hazards include the level of ground shaking for structural design and liquefaction, ground deformations, loss of bearing, and slope stability hazards that may impact the performance of foundations.

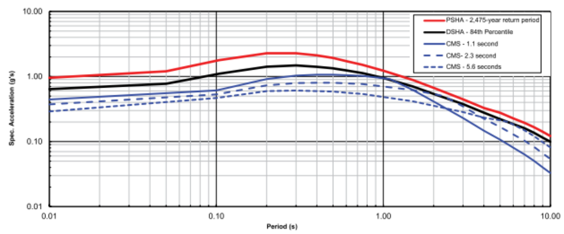


Figure 1.2 Example of Site-Specific Spectra

ASCE 7-16 recommends a minimum of 11 ground motions be selected for modification and application to the Non-Linear Response History Analysis (NRHA) model. According to the standard, an advantage of using this larger number of motions is that if unacceptable response is found for more than one of the 11 motions, this does indicate a significant probability that the structure will fail to meet the 10% target collapse reliability for Risk Category I and II structures. Ground motions can be selected from the following database:

•European Strong Motion Database (ESD) (http://www.isesd.hi.is/ESD_Local/frameset.htm)

•COSMOS database for worldwide Earthquake data (https://strongmotioncenter.org/vdc/scripts/default.plx)

•K-NET and KIK-NET NIED strong motion seismograph network, Japan (http://www.kyoshin.bosai.go.jp/)

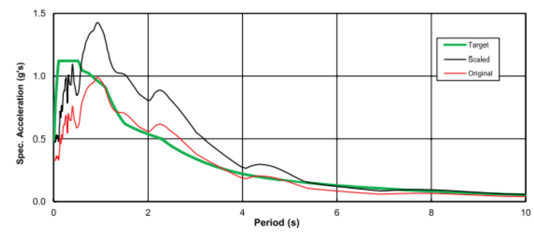
•PEER (Pacific Earthquake Engineering Research Center) NGA database (https://ngawest2.berkeley.edu/)

•PESMOS (Program of Excellence in strong motion studies, IIT Roorkee, India) (http://pesmos.com/)

Ground Motion Modification:
ASCE 7 recommends two methods of modifying ground motions to make them compatible with the targeted spectrum; Amplitude scaling and Spectral matching. Also the period range should have an upper bound greater than or equal to twice the largest first mode period (2T1) in the principal horizontal direction of response; the lower bound period should be established such that the period range includes at least the number of elastic modes necessary to achieve 90% mass participation in each principal horizontal direction. The lower bound period should not exceed 20% of the smallest first-mode period (0.2T2) of the two principal horizontal directions of response.

Amplitude Scaling: This is a relatively simple approach in which a single scalar is used to modify the spectral values of the original time series. An example of spectrally-scaled time series is presented in Figure 1.3.

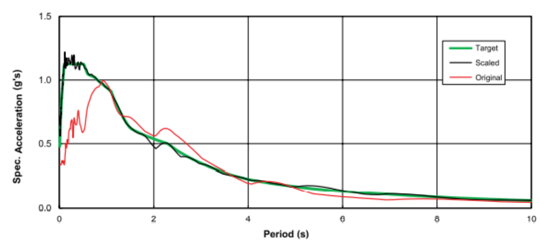
Figure 1.3 Spectrally scaled (amplitude scaling) time



series

Spectral Matching: In this approach, the shape of the response spectrum of the original time series is modified to match a target response spectrum (see Figure 1.4).

Figure 1.4 spectrally matched Time series



II. OBJECTIVE OF THESIS

The objective is to study the seismic performance of the building which is designed as per Indian standards with certain exceptions of stringent clauses from IS 1893(Part 1)-2016. The first step involves preliminary sizing of the structure using linear analysis methods such as Response Spectrum Analysis for SLE hazard. The second step involves verification of the structural behaviour at MCE hazard with 11 suits of ground motions using nonlinear analysis. There are two types of nonlinear analysis which can be performed

1. Static Nonlinear Analysis which is known as Pushover analysis and
2. Dynamic Nonlinear analysis which is known as Nonlinear Response History Analysis (NRHA).

The aim is to use NRHA model to study the seismic performance.

III. THESIS DENITION

The main aim of the thesis is to conduct The Performance Based Seismic Design of a building by using Non-Linear Response History Analysis.

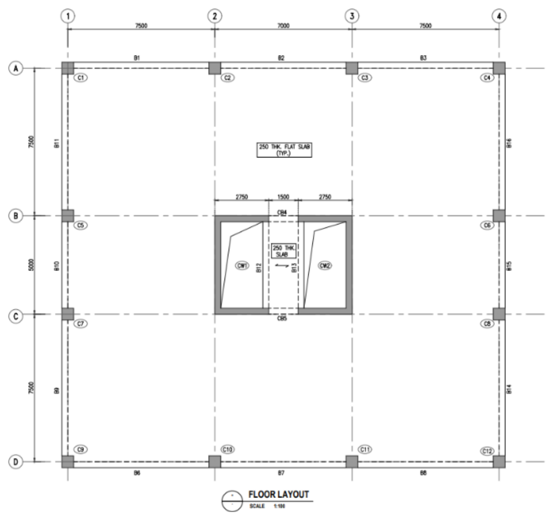


Figure 3.1 Floor Layout

Description	
No of Floor	G+15+Terrace
Flr to Flr height	3.0 m
Height of Structure	48 m
Loading	
Dead Load	
Floor Finish (kN/m^2)	1.5
Ext. Wall (kN/m)	7.2
Int. Wall (kN/m2)	1.5
Parapet (kN/m)	4.5
Live Load	
Floor Live (kN/m^2)	4
Roof Live (kN/m^2)	1.5
Material	
Concrete	
Grade in Beams/Slab	M30
Grade in Columns	M40
Steel	
Grade in Beam	Fe 500
Grade in Columns	Fe 500
Seismic Data	
Zone	III
Location	Mumbai
Response Red Factor	4
Soil Type	Hard

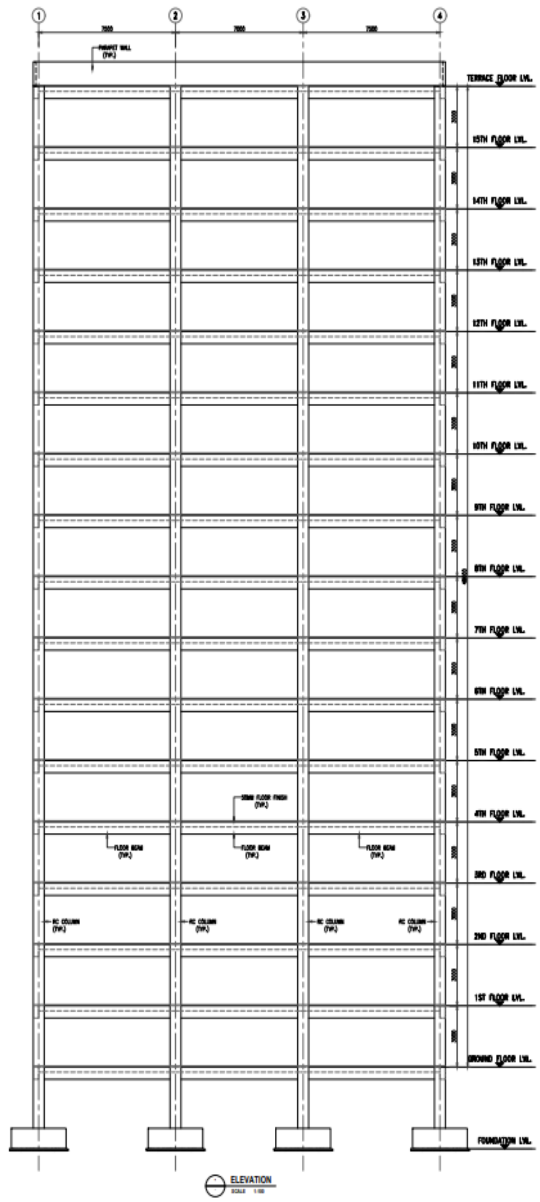


Figure 3.2 Elevation

IV. MODELLING AND ANALYSIS

- Linear structural analysis and design of the building

The initial proportioning of a building consists of a complete design process whereby all members of the seismic force-resisting system are designed and detailed. Linear design is done using DE (Design Earthquake) as per the all the provisions of the code to demonstrate code-equivalency. The building is designed to resist dead, live, wind and other loadings as per the codal provisions. After initial proportioning of the building is carried out, the next step is performing a SLE evaluation. SLE evaluation typically corresponds to the Immediate Occupancy performance level for structures with I = 1 (I is the Importance Factor as per IS 1893 (Part 1) - 2016). The purpose of the evaluation is to validate that the building's structural and non-structural components retain their general functionality during and after such a frequently possible event.

expected material properties should be used for analysis in PBSD. Figure 4.1 shows the expected material properties recommended by PEER/TBI. Alternatively, project-specific values based on adequate material testing can be used.

Material	Expected strength	
Reinforcing Steel	Expected Yield Strength, f_{ye}, psi	Expected Ultimate Strength, f_{ue}, psi
A615 Grade 60	70,000	106,000
A615 Grade 75	82,000	114,000
A706 Grade 60	69,000	95,000
A706 Grade 80	85,000	112,000
Structural Steel* **		
Hot-rolled structural shapes and bars		
ASTM A36/A36M	1.5 f_y *	1.2 f_u **
ASTM A572/A572M Grade 50	1.1 f_y	1.1 f_u
ASTM A913/A913M Grade 50, 60, 65 or 70	1.1 f_y	1.1 f_u
ASTM A992/A992M	1.1 f_y	1.1 f_u
Plates		
ASTM A36/A36M	1.3 f_y	1.2 f_u
ASTM A572/A572M Grade 50, 55	1.1 f_y	1.2 f_u
Concrete	$f'_{ce} = 1.3 f'_c$ †	

* f_y is used to designate the specified (nominal) yield strength of steel materials in this Guideline. It is equivalent to f_y or f_{yt} used in ACI 318 and F_y used in AISC (2006) standards.

** f_u is used to designate the specified (nominal) ultimate strength of steel materials in this Guideline. It is equivalent to F_u used in AISC (2006) standards.

*** For steel materials not listed, refer to Table A3.1 of ANSI/AISC 341-16

† f'_c = specified compressive strength. Expected strength f'_{ce} is strength expected at approximately one year or longer. Note that the multiplier on f'_c may be smaller for high-strength concrete, and can also be affected by (1) use of fly ash and other additives, and/or (2) local aggregates.

Figure 4.1 Expected Material Properties recommended in PEER/TBI

- Effective Member Stiffness:

In linear and nonlinear analysis, section properties are reduced to account for cracking and damage to the members using section/stiffness property modifiers. According to CTBUH (2017), property modifiers provided in literature are based on experimental testing; the application of property modifiers can have significant impact on members forces and should be carefully considered for each project.

For steel members and components and reinforcement bars the elastic modulus shall be taken as 200000 Mpa.

For reinforced concrete components, effective stiffness should be based on recommendation by PEER/TBI and LATBSDC. Table 4.3 is an excerpt of the recommended effective stiffness for reinforced concrete members from PEER/TBI.

- Modelling for Nonlinear Structural Analysis at MCE level

Nonlinear Response History Analysis (NRHA) can be used to evaluate earthquake demands when the structure deforms significantly beyond the elastic range. NRHA is required for MCE level evaluation of structures. The goal of a performance-based assessment using NRHA is to simulate the building response as realistically as possible to obtain an unbiased measure of its performance. Nonlinear modelling for performing NRHA is much more detailed and appropriate care should be given to all modelling assumptions. Much more detailed information can be obtained from NRHA which leads to a refined understanding of the building and opens up the possibility of making design changes to improve efficiency. For example, in shear wall type of structures, it is possible to identify whether a single concentrated plastic hinge forms at the base or much more distributed yielding is expected. Based on this information it may be possible to provide improved detailing in these high deformation areas and provide more efficient reinforcement detailing elsewhere.

Table 4.1 Reinforced Concrete Effective Stiffness Values (source, PEER/TBI)

Component	Service Level Linear Models			MCE Level Nonlinear Models		
	Axial	Flexural	Shear	Axial	Flexural	Shear
Structural Walls[1] (in-plane)	1.0EcAg	0.75EcIg	0.4EcAg	1.0EcAg	0.35EcIg	0.2EcAg
Structural Walls (out-of-plane)	--	0.25EcIg	--	--	0.25EcIg	--
Basment Walls (in-plane)	1.0EcAg	1.0EcIg	0.4EcAg	1.0EcAg	0.8EcIg	0.2EcAg
Basement Walls (out-of-plane)	--	0.25EcIg	--	--	0.25EcIg	--
Coupling Beams with conventional or diagonal reinforcement	1.0EcAg	0.07(L/h)EcIg ≤ 0.3EcIg	0.4EcAg	1.0EcAg	0.07(L/h)EcIg ≤ 0.3EcIg	0.4EcAg
Composite steel / reinforced concrete coupling beams	1.0(EA)trans	0.07(L/h)(EI)$_{trans}$	1.0EsAsw	1.0(EA)trans	0.07(L/h)(EI)$_{trans}$	1.0EsAsw
Non-PT transfer diaphragms (in-plane only)[3]	0.5EcAg	0.5EcIg	0.4EcAg	0.25EcAg	0.25EcIg	0.1EcAg
PT transfer diaphragms (in-planeonly)[3]	0.8EcAg	0.8EcIg	0.4EcAg	0.5EcAg	0.5EcIg	0.2EcAg
Beams	1.0EcAg	0.5EcIg	0.4EcAg	1.0EcAg	0.3EcIg	0.4EcAg
Columns	1.0EcAg	0.7EcIg	0.4EcAg	1.0EcAg	0.7EcIg	0.4EcAg
Mat (in-plane)	0.8EcAg	0.8EcIg	0.8EcAg	0.5EcAg	0.5EcIg	0.5EcAg
Mat[4] (out-of-plane)	--	0.8EcIg	--	--	0.5EcIg	--

[1] Values are relevant where walls are modeled as line elements. Where walls are modeled using fiber elements, the model should automatically account for cracking of concrete and the associated effects on member stiffness.

[2] (EI)trans is intended to represent the flexural rigidity of the cracked transformed section. It is acceptable to calculate the transformed section properties based on structural mechanics or to use (EI)trans = EcIg/5 + EsIs per ACI 318.

[3] Specified stiffness values for diaphragms are intended to represent expected values. Alternative values may be suitable where bounding analyses are used to estimate bounds of force transfers at major transfer levels. For diaphragms that are not associated with major force transfers, common practice is to model the diaphragm as being rigid in its plane. Flexural rigidity of diaphragms out of plane is usually relatively low and is commonly ignored. The exception is where the diaphragm acts as a framing element to engage gravity columns as outrigger elements, in which case out-of-plane modeling may be required.

[4] Specified stiffness values for mat foundations pertain to the general condition of the mat. Where the walls or other vertical members impose sufficiently large forces, including local force reversals across stacked wall openings, the stiffness values may need to be reduced.

- Modelling for Nonlinear Structural Analysis at MCE level

Nonlinear Response History Analysis (NRHA) can be used to evaluate earthquake demands when the structure deforms significantly beyond the elastic range. NRHA is required for MCE level evaluation of structures. The goal of a performance-based assessment using NRHA is to simulate the building response as realistically as possible to obtain an unbiased measure of its performance.

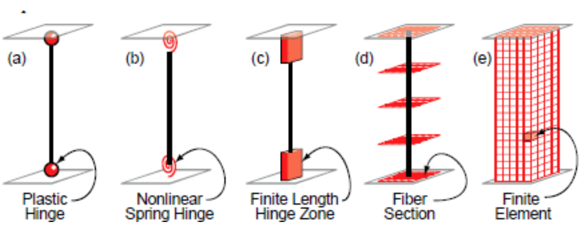


Figure 4.2 Range of Nonlinear Model Types (source, NIST GCR 17-917-46v1)

- Important Modelling Parameters

For deformation-controlled elements which use lumped-plasticity models, a nonlinear force-deformation relationship (known is PBSD parlance as the Backbone Curve) is used. Backbone curves are established based on physical testing. Generic backbone curves can be generated using NIST GCR 17-917-45 or ASCE 41-17.

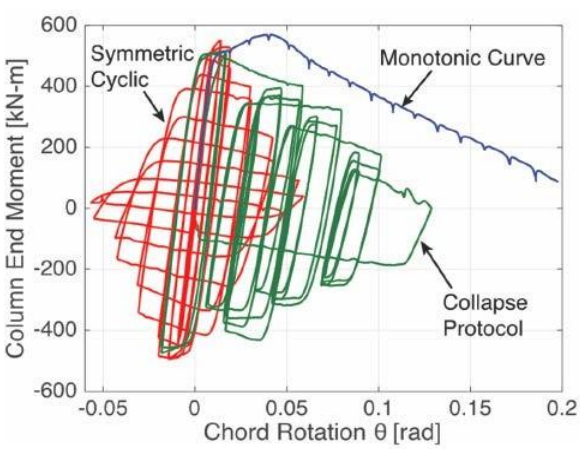


Figure 4.3 Force-Deformation Response of Steel Moment Frame subjected to different Loading Protocols (source, NIST GCR 17-917-45 after Suzuki et. al. (2015))

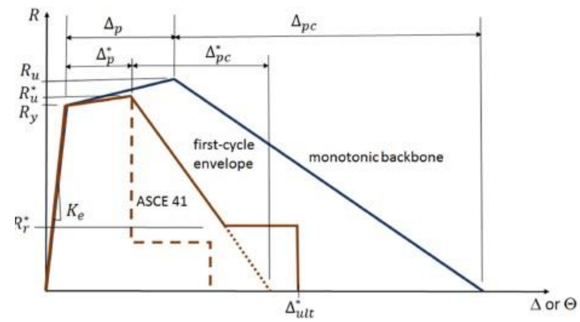


Figure 4.4 Idealised Backbone Curves from NIST GCR 17-917-45 compared to ASCE 41-17 (source, PEER/TBI)

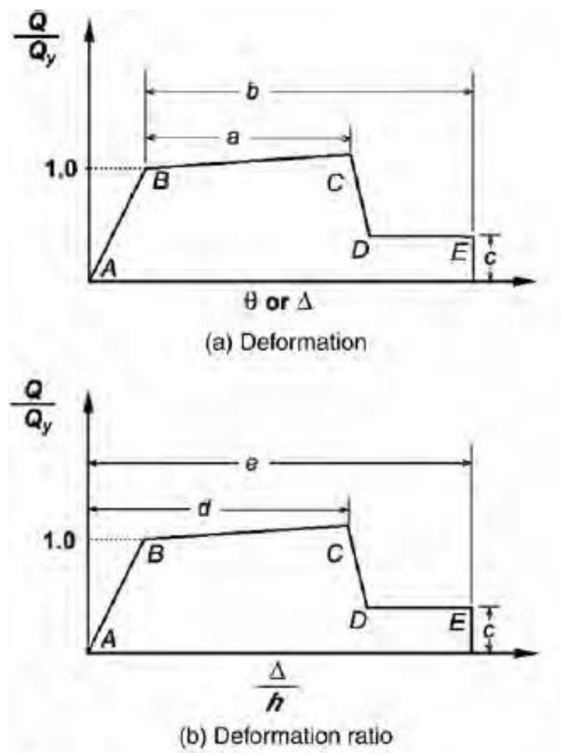


Figure 4.5 Idealised First-Cycle Backbone Curve of ASCE 41

- Reinforced concrete Components

Reinforced Concrete Beams and Columns

According to PEER/TBI, the moment-rotation response of beams and columns can be developed using parameters in NIST GCR 17-917-45 or ASCE 41-17. NIST GCR 17-917-45 provides parameters for establishing both the monotonic backbone and first-cycle envelope curves for concrete beams and columns that generally conform to the design requirements for Special Moment Frames (SMF) in ACI-318.

Reinforced Concrete Slab-Column Frames

According to PEER/TBI, concrete slabs and slab–column connections should be modelled using guidelines of PEER/ATC 72-1 or ASCE 41-17. ASCE 41-17 recommends using either an effective beam-width model or an equivalent-frame model to model slab-frames and slab-column connections. The slab-frames and slab-column connections are mainly modelled to capture the ‘micro-outriggering’ effect they have on columns and to satisfy the framing rotation limits. Figure 4.6 illustrates the concept of using effective beam widths for analytical modelling of slabs.

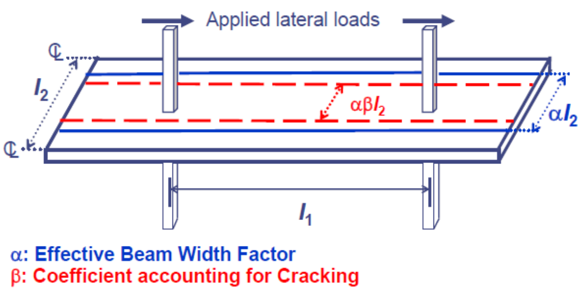


Figure 4.6 Effective Beam Width for Analytical Modelling

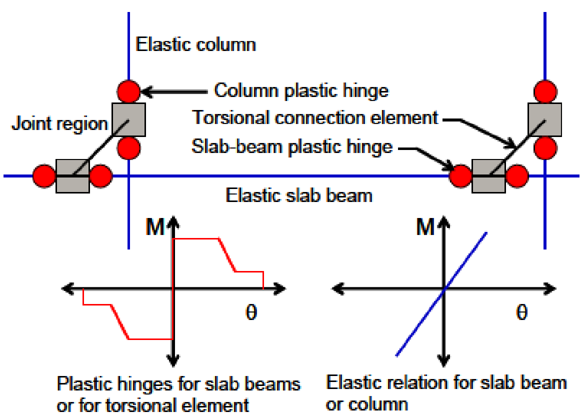
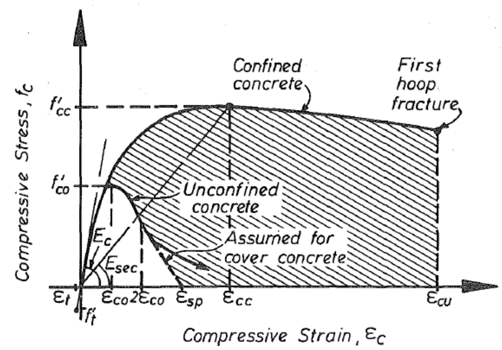


Figure 4.7 Slab-Column Frame and Connection Modelling as per PEER/ATC 72-1

Shear Walls

CTBUH (2017), PEER/TBI and LATBSDC recommend using ‘fiber’ elements as per PEER/ATC 72-1 for modelling shear wall axial and flexure behaviour of flexure dominated shear walls.

As per recommendations of LATBSDC, stress-strain curves for concrete as defined by Collins and Mitchell (1997) can be used for un-confined concrete; for confined concrete models described by Mander et. al. (1988) or Saatchioglu and Razvi (1992) are



acceptable.

Figure 4.7 Stress-Strain Curve for Confined and Un-Confined Concrete (source, Mander et. al.1988))

Coupling Beams

Most medium to tall buildings resist horizontal loading (earthquake or wind) through interior reinforced concrete shear walls. Such horizontal actions usually govern structural design of the cores, which in turn control the deformability of the building. In order to provide access to elevators or other facilities, shear walls usually have aligned openings (Figure 4.8). The part of the wall above the opening, connecting two vertical wall piers, is called a coupling beam.

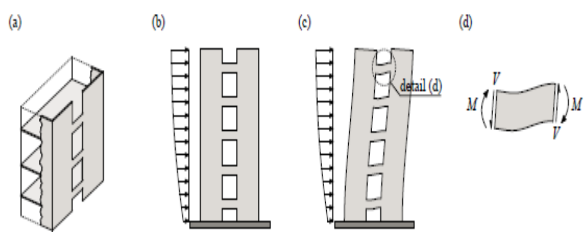


Figure 4.8 Coupling Beams in Shear Walls: Illustration of their Loading, Deformations and Internal Forces (source, Brena (2009))

Beam-Column Joints

According to PEER/TBI, shear behaviour in concrete beam-column joints should be modelled as elastic, force-controlled action using guidelines in NIST GCR 17-917-45 or ASCE 41-17.

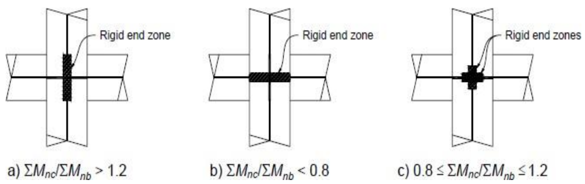


Figure 4.9 Rigid End Zones for Beam Column Joint Modelling based on Relative Capacity of Beams and Columns (source, Elwood et. al. (2007)

V. RESULTS AND DISCUSSION

- Linear Elastic Model and Acceptance Criteria for SLE Evaluation

A three-dimensional model of the structural system is prepared in ETABS for SLE evaluation. The analysis model includes all lateral force-resisting elements, primary gravity system elements, P-Delta effects are included. Expected material properties are utilized for realistic estimates of stiffness and section property modifiers are applied as discussed in Section above Accidental eccentricity is not considered for serviceability evaluation; however, torsion sensitivity study is performed. Rigid diaphragms are used to model the typical floor.

Response spectrum analysis is performed Response parameters, including forces and displacements from the response spectrum analysis are used evaluate acceptable performance Figure 5.1 shows the linear elastic model (developed in ETABS) used for SLE evaluation.

- Results from Analysis and SLE Evaluation

A summary of the periods and mass participation of the first three modes of the building is provided in Table 5.1.

Table 5.1 Time Periods and Modal Mass Participation Ratios from SLE Model

Mode	Time Period	Mass Participation (UX)	Mass Participation (UY)	Mass Participation (RZ)
1	2.21 s	0 %	64.19 %	0 %
2	1.89 s	67.62 %	0 %	0 %
3	1.59 s	0 %	0 %	77.4 %

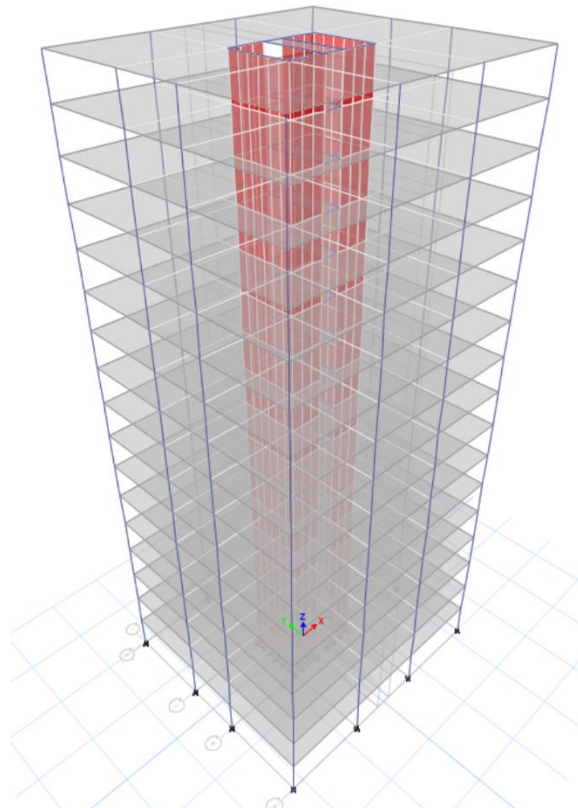


Figure 5.1 Linear Elastic Model prepared in ETABS for SLE Evaluation

The drift levels are within the acceptable range (refer Figure 5.2). The SLE level design to capacity ratios of (1) flexure in shear walls, (2) PMM interaction in columns are shown in Figure 5.3 and Figure 5.4 respectively.

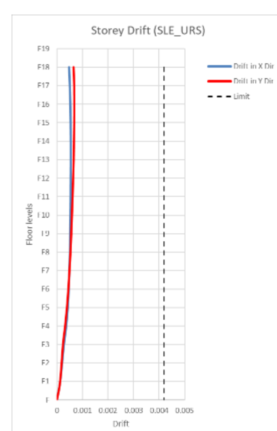


Figure 5.2 Story Drifts in X- and Y- Direction at SLE Hazard Level

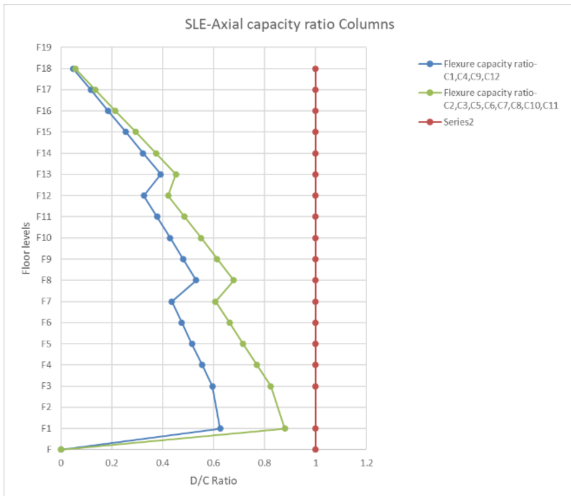


Figure 5.3 Flexural D/C Ratios of Shear Walls at SLE Hazard Level

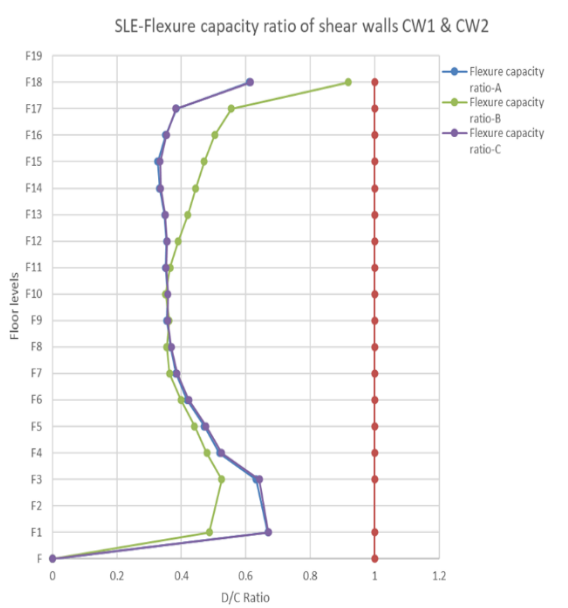


Figure 5.4 PMM D/C Ratios of Gravity Columns at SLE Hazard Level

- Maximum Considered Earthquake (MCE) Evaluation
- Ground Motions

Figure 5.13 shows the maximum direction response spectra from the selected ground motions (11 selected time series) matched to the target response spectrum. These selected and modified ground motions are used in NRHA for MCE evaluation.

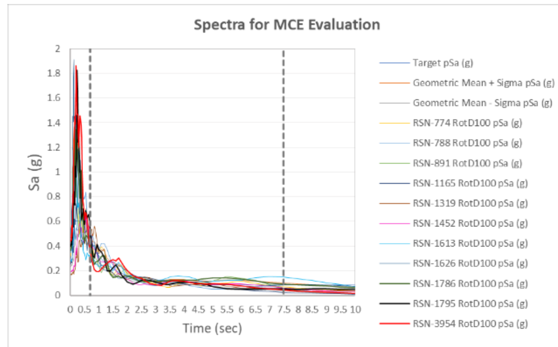


Figure 5.13 Maximum Direction Ground Motion Spectra matched to Target Spectrum for MCE Evaluation

- NRHA Model and Acceptance Criteria for MCE Evaluation

ETABS is used for constructing the three-dimensional NRHA model. ETABS recently introduced fiber models, direct integration nonlinear time history analysis and various output tools that can simplify PBSD. ETABS and SAP2000 have been widely used for nonlinear modelling incorporating plastic hinge type nonlinearity in frame elements which use pre-defined backbone curves.

Nonlinear models adopting an event-to-event solution strategy using direct integration require high computational efficiency and hence only elements which are likely to affect the dynamic response are modelled in ETABS.

Figure 5.5 shows the 3D analysis model, the equivalent frame modelling to represent slabs at the typical floor level and the rigid diaphragm extent applied to nodes.

- Damping

Equivalent viscous damping value of 2.5% is considered in the NRHA model as per recommendations of PEER/TBI and LATBSDC. The damping is specified as a combination of modal damping (2.4%) and stiffness proportional Raleigh damping which contributes the remaining 0.1%. A sensitivity study was also performed using a combination of modal damping (2%) and mass and stiffness proportional Raleigh damping (0.5%). The center of mass (CM) deflection history at the roof level for both these damping options are similar. The remaining damping resulting from energy dissipation due to inelastic action is represented in the model by the backbone curves and nonlinear material constitutive relationships.

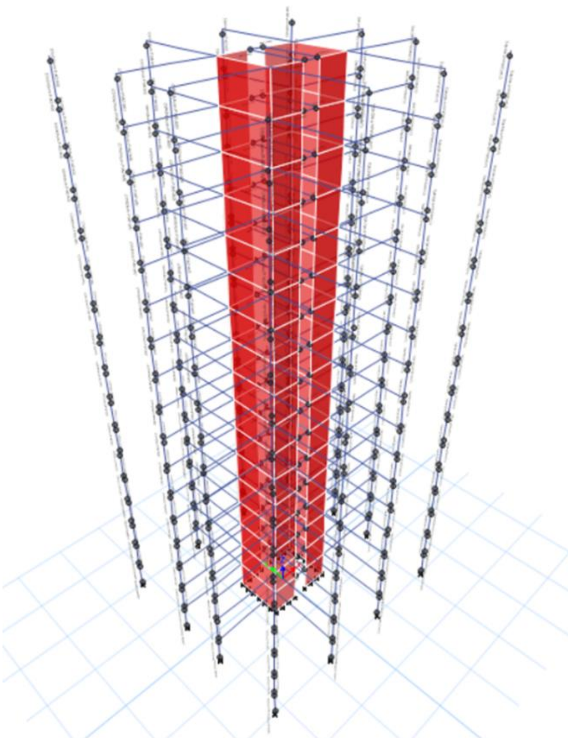


Figure 5.5 Simplified 3D model used for NRHA

- Results from Analysis and Global MCE Evaluation

A summary of the periods and mass participation of the first three modes of the building is provided in Table 5.2.

Table 5.2 Time Periods and Modal Mass Participation Ratios from MCE Model

Mode	Time Period	Mass Participation (UX)	Mass Participation (UY)	Mass Participation (RZ)
1	2.82 s	0 %	0 %	74.24 %
2	2.43 s	69.62 %	0 %	0 %
3	2.41 s	0 %	67.44 %	0 %

The X- and Y- direction average of maximum base shears from eleven ground motions for the MCE hazard are shown in Figure 5.6 and Figure 5.7. The graphs also compare the code- based seismic/wind and SLE level base shears with those from MCE level hazard. The MCE story shears are approximately 1.75 to 2.5 times the code-based and SLE story shears.

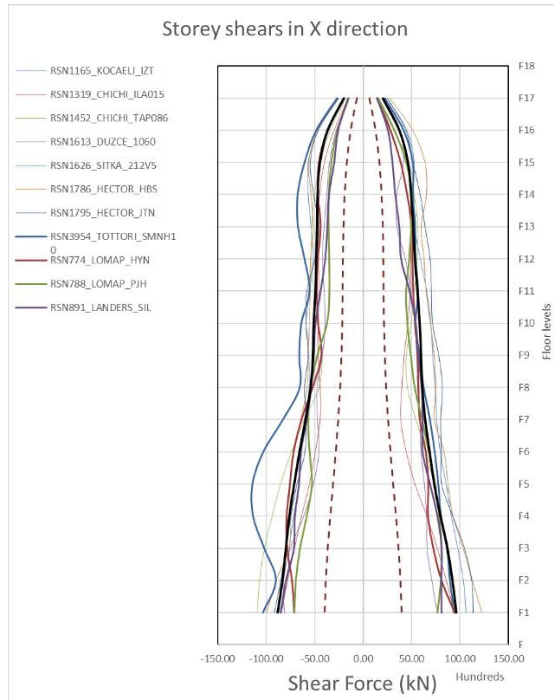


Figure 5.6 Comparison of Cumulative Story Shears in X- Direction

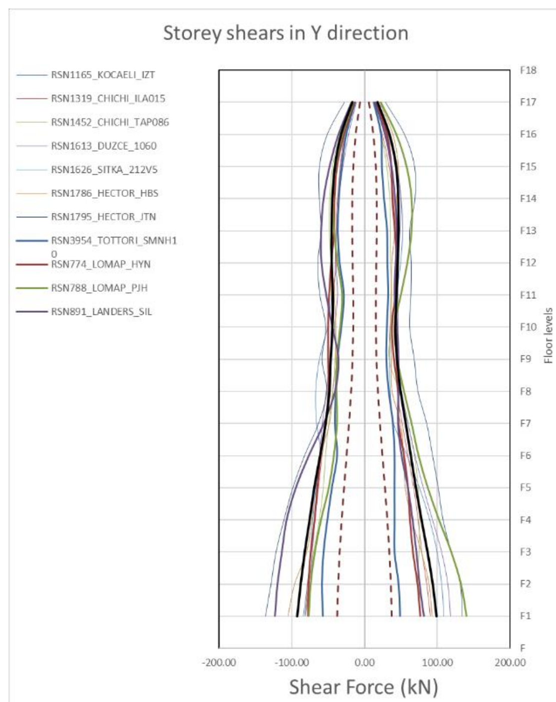


Figure 5.7 Comparison of Cumulative Story Shears in Y- Direction

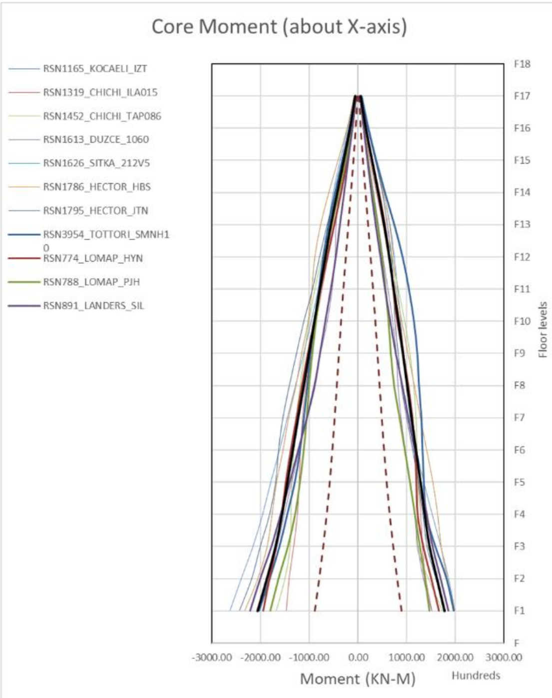


Figure 5.8 Comparison of Shear Wall Core Moments about X- Axis

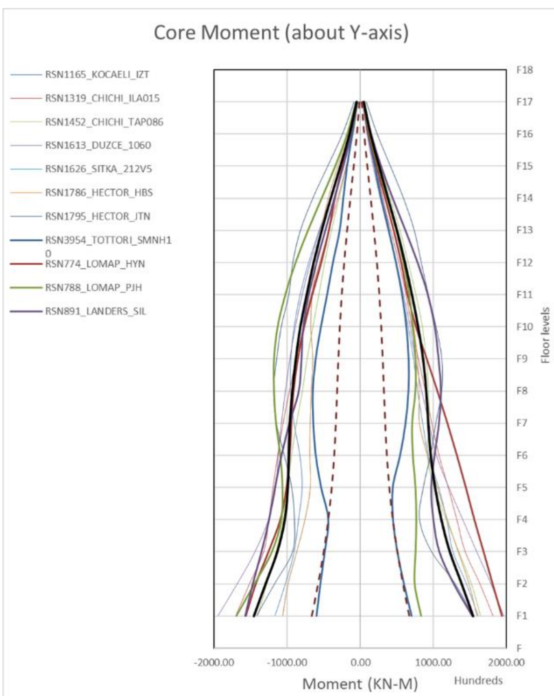


Figure 5.9 Comparison of Shear Wall Core Moments about Y- Axis

The X- and Y- direction average of maximum shear wall core moments from eleven ground motions for the MCE hazard are shown in Figure 5.8 and Figure 5.9. The graphs also compare the code-based seismic/wind and SLE level core moments with those from MCE level hazard. The difference between the MCE core moments and the SLE and code-based core moments (obtained from elastic analysis and used for design) is significantly less than the difference in the story shears; this is primarily because in the NRHA model, a chunk of the building overturning moment is taken up by tension-compression moment couple of the slab-column micro-outriggers. One can expect very little nonlinearity in the shear walls at MCE hazard based on the comparison of core overturning moments.

The X- and Y- direction average of maximum center of mass displacements from eleven ground motions for the MCE hazard are shown in Figure 5.10 and Figure 5.11.

The average and maximum drifts in both X- and Y- directions are evaluated for the global acceptance criteria for MCE level. The average drifts from eleven ground motions are comfortably below the acceptable level of (0.03/I) i.e. 0.025; the maximum drifts from any ground motion do not exceed the acceptable level of 0.045 (refer Figure 5.12 and Figure 5.13).

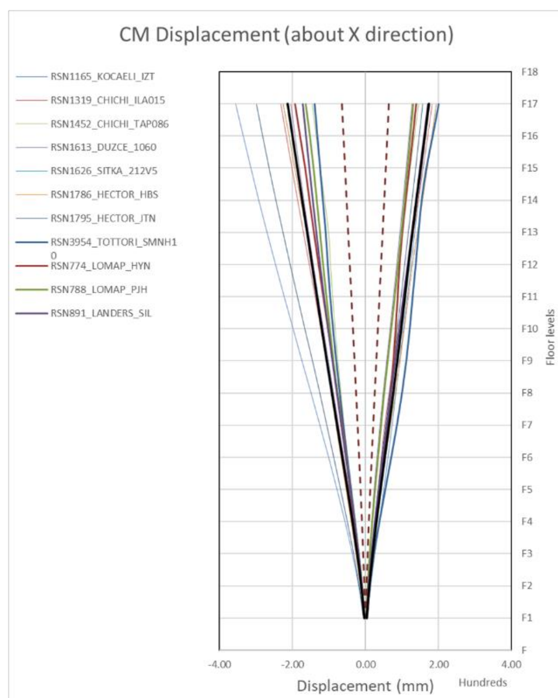


Figure 5.10 Diaphragm Center of Mass (X-Direction) Displacement at MCE Hazard

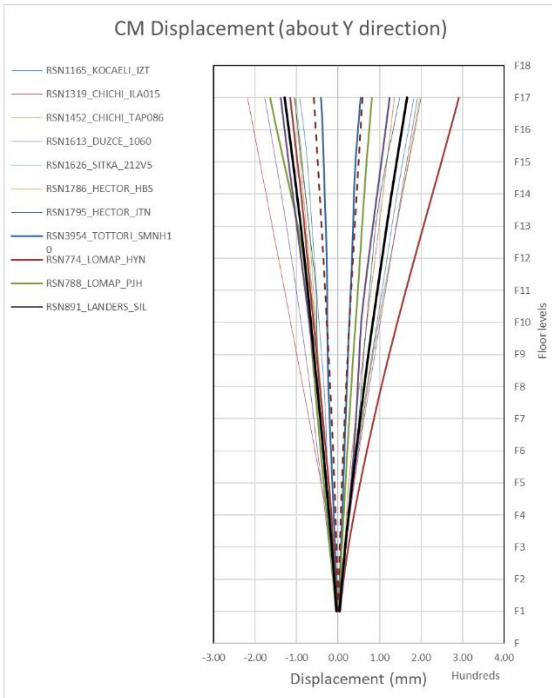


Figure 5.11 Diaphragm Center of Mass (Y-Direction) Displacement at MCE Hazard

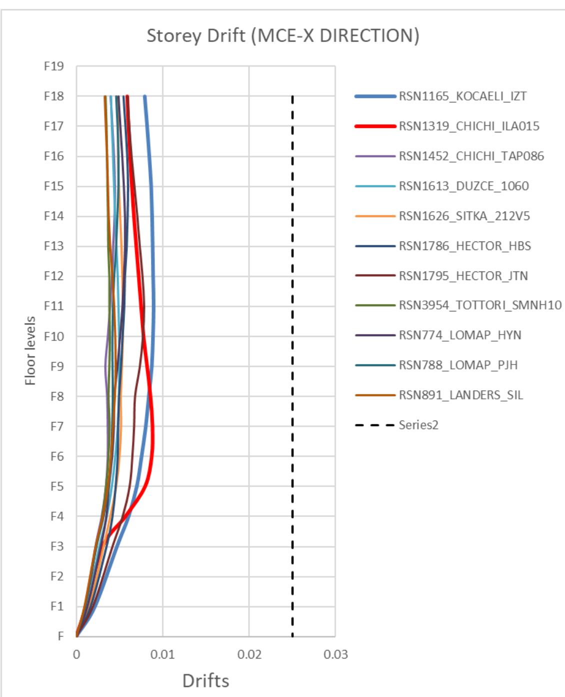


Figure 5.12 Story Drifts (X- Direction) at MCE Hazard

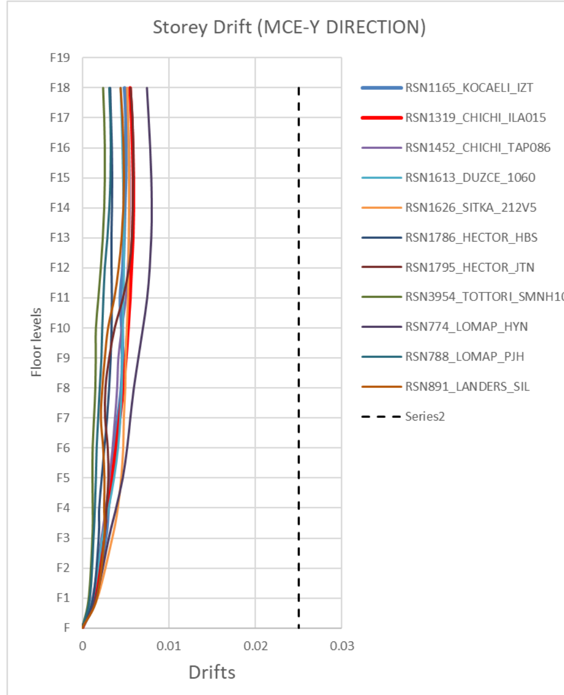


Figure 5.13 Story Drifts (X- Direction) at MCE Hazard

VI. SUMMARY & CONCLUSION

The current building designed as per Indian standard in EQ Zone III with Response reduction factor as 4 instead of 3 as suggested by IS1893 for flat slab building i.e. code exceeding building.

From the result it can be seen that the building drift, deflection is well within permissible limit at MCE level.

It is also observed that strain limits at MCE level are well within limit.

Ductile shear wall panels meet collapse prevention criteria of the codes.

Except for the shear stress in the wall panel all other aspect of codes are meet within the current design.

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