SEISMIC COLLAPSE SAFETY ASSESSMENT

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Abstract - *Development level of the country is mainly indicated by its buildings and infrastructure. Many of the natural calamities result in failure of the structures to resist forces and thereby collapses occur. Structural collapse during earthquakes being a disastrous state, which is unacceptable even under very rare earthquake conditions, protection against it becomes a serious concern. Seismic collapse safety assessment, a branch of earthquake engineering, focuses on probabilistic approach to estimate safety margins for designed structures. In this paper, the collapse safety assessment methodology provided by FEMA P695 is applied to performance groups of structures, designed according to Indian standards. It considers cases of Reinforced concrete special moment resisting framed type two-dimensional index archetype models, designed based on Indian configurations. The considered archetype frames are assessed in two phases, by Static Pushover Analysis followed by Incremental Dynamic Analysis using Seismosoft 2018 products. Acceptable performance is defined by collapse prevention objectives, both for performance group as a whole and for each index archetype under the performance group. The main focus is to apply seismic collapse safety assessment methodology to archetypes under consideration and to check, whether they possess acceptable collapse safety margin or not.*

Key Words*: Collapse safety margin, collapse margin ratio, static pushover analysis, incremental dynamic analysis, seismic performance factors, response modification coefficient, overstrength factor, deflection amplification factor*.

1. INTRODUCTION

Collapse of the structure can be defined as the condition at which a structure, or a significant portion of it, is unable to support its gravity loads during a seismic excitation. Local collapse may occur, for instance, if a vertical load-carrying component fails in compression, or if shear transfer is lost between horizontal and vertical components (e.g., shear failure between a flat slab and a column). Global collapse will occur if local collapses propagate or if an individual storey displaces sufficiently so that the second-order P-delta effects fully offset the first-order storey shear resistance and dynamic instability (sidesway collapse) occurs. For many years, the structural engineering profession has paid little attention to collapse prediction. The primary reason being, the comfort exists in elastic design principles and that collapse protection is believed to be adequately considered by good detailing criteria, partial consideration of capacity design concepts and by placing limits on maximum permissible storey drifts. Looking to the present status of collapse prediction and advantages gain by the engineering profession at this time, it must be said that we have a long way to go to predict collapse with confidence, and it must be realized that collapse prediction is not a deterministic process, instead it is the probabilistic one. Collapse prevention is one of the objectives of a performance-based design, and one of its promises is the assurance of an adequate safety margin against collapse under the expected maximum seismic load. Although this collapse prevention issue has attracted large interest among earthquake engineers and researchers during several past decades, no standard method has been introduced. During the past few years FEMA has published a guideline for quantification of building Seismic Performance Factors which include Response modification factor, overstrength factor and displacement amplification factor.

To achieve one of the main objectives of the seismic design philosophy, it is important to assess margin of safety against structural collapses. Objective of this study is to apply the methodology of collapse safety assessment provided by FEMA P695 to the archetype structures designed using Indian codes and common configurations in particular locality. It covers design and development of archetype configurations which are based on different ranges of parameters common in building practices in India. The archetype framework bridges the gap between performance predictions for a single specific building and the generalized predictions needed for quantification of the performance of a full class of structures. The designed frames considered for assessment are reinforced concrete special moment resisting framed [SMRF] type two-dimensional index archetypes, which are the space frames, in which the ration of gravity load tributary area to the lateral load tributary area comes out to be unity. Two performance groups each containing three archetypes of 4 stories,8 stories and 12 stories are considered for the study and checked if they pass the expectations of safety margin against collapse or not.

Many studies have enrolled the seismic performance factors, spectral shape factors, selection and scaling of ground motions, soil structure interaction consideration. The literature shows considerable research in this field. Some of correlated works are, [Liel A.et al.][1] (2007) conducted detailed assessments of the collapse performance of both modern reinforced concrete (RC) special moment frames

(SMF) and existing RC non-ductile moment frames also, described approaches for evaluating the effects of modeling uncertainties. [Roberto Villaverde][10] (2007) presented a comprehensive review of the analytical methods that were available to assess the capacity of building structures to resist an earthquake collapse, pointed out the limitations of those methods, described past experimental work in which specimens were tested to collapse, and identified what was required for an accurate evaluation of the seismic collapse capacity of a structure and the safety margin against such a collapse.

[Deierlein G. et al.][9] (2008) described the Applied Technology Council project (ATC-63) to develop a methodology to assess seismic design provisions for building systems. The approach to evaluate the collapse safety of a set of archetype buildings, whose designs reflect the key features of the seismic design requirements have been described. [Haselton C.et al.][4] (2010) described an example application of the newly developed FEMA P695 (ATC-63) methodology for assessing collapse performance. This methodology is applied to assessing the collapse performance of code-conforming reinforced concrete (RC) special moment frame (SMF) buildings. This process showed that RC SMF buildings have pass the methodology and are deemed to have acceptable collapse safety. [Ali Reza Manafpour and Maryam Tohidian][2](2017) tackled the issue of evaluation of collapse safety margin for structures design based on modern seismic code requirements. They considered RC frame structures designed according to Iranian seismic standard. Incremental Dynamic Analysis was carried out using 22 natural ground motion records to RC moment resisting frames with 3, 6 and 10 stories considering two types of soil classifications. It was concluded that the collapse margin is generally reduced with the increased height of the structure. [Haselton C. et al.][5] (2011) has shown that rare high-intensity ground motions have a peaked spectral shape that should be considered in ground-motion selection and scaling. [Vamvatsikos D. et al.][6] (2002) introduced Incremental dynamic analysis (IDA), a parametric analysis method that has recently emerged in several different forms to estimate more thoroughly structural performance under seismic loads.

2. COLLAPSE SAFETY ASSESSMENT

Seismic collapse assessment is a branch of earthquake engineering which includes adequate prediction of the seismic hazard, ground motion selection, identification of possible modes of collapse, modeling of cyclic component deterioration, appropriate consideration of hysteretic and viscous damping, quantification of modeling and parameter uncertainties, and nonlinear dynamic analyses. Federal Emergency Management Agency (FEMA) commissioned the Applied Technology Council (ATC) under the ATC-63 Project to develop the FEMA P695 (ATC-63) document, which contains a formalized assessment methodology for quantifying structural collapse safety under seismic loading.

2.1 Applied Technology Council (ATC)assessment methodology

This methodology mainly includes development of representative models, consideration of the fully defined ground motion data and methods of analysis that are generically applicable to all seismic-force- resisting systems. It begins with consideration of an idealized model which reflects salient design features that affect the collapse response of the structural system.Fig.1 shows the flowchart of ATC-63 Methodology for system performance assessment. This paper includes index archetype structural models. These index archetype models should be capable of capturing the important behavioral effects in beams, columns and beam-column joints that govern collapse behavior. The three-bay configuration is judged to be the minimum necessary to capture overturning forces in columns and a mix of interior and exterior columns and joints.

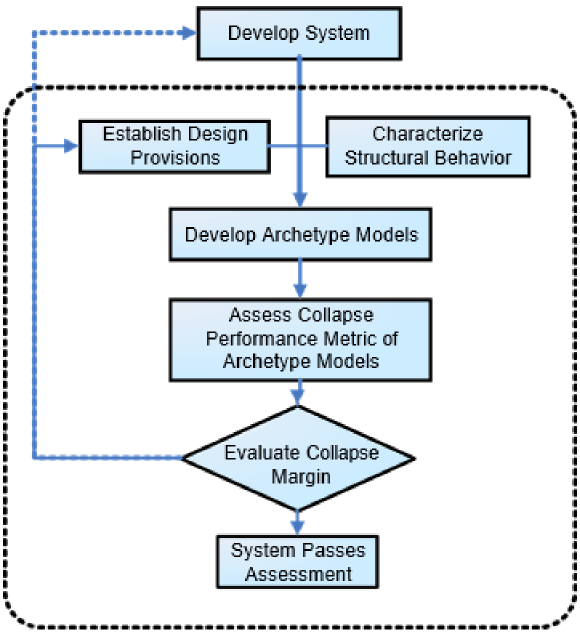

Figure -1: Flowchart of ATC-63 Methodology for system performance assessment

Present work includes assessment for two-dimensional index archetype models of actual buildings. Fig. 2 shows the index archetype ideal model for moment frame buildings. These two-dimensional models, not accounting for torsional effects, are considered acceptable because most reinforced concrete special moment frame buildings that are regular in plan will not be highly sensitive to torsional effects, and the goal is to verify the performance of a full class of buildings, rather than one specific building with a unique torsional issue.

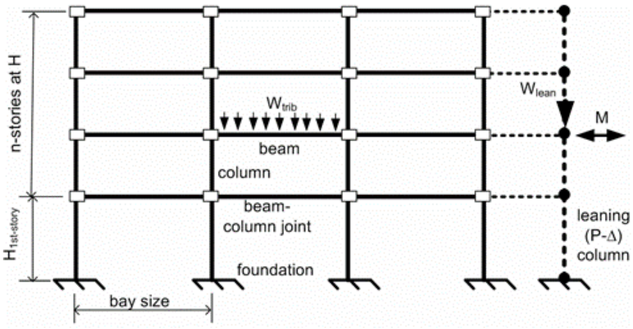

Figure -2: Index archetype analysis model for moment-frame buildings

3. METHODS OF ANALYSIS AND PROPOSED WORK

The considered archetype frame models should be assessed through two analysis methods, Nonlinear static pushover analysis and Incremental dynamic analysis, one followed by the other, with evaluation of expected parameters through each one of them required for this methodology of collapse safety assessment. Table 1 shows the list of parameters to be obtained from these analysis methods. Products of Seismosoft-2018 are being used for analysis.

Table -1: Data to be reported from analysis procedures

Method of analysis	*Information to be reported*
Nonlinear Static Pushover Analysis	1. Fundamental period of vibration (T_1) 2. Fully yielded strength (Vmax) 3. Static overstrength factor (Ω) 4. The effective yield ($\delta_{y.eff}$) 5. Ultimate roof displacement (δu) 6. Period based ductility (μ_T) 7. Spectral shape factor (SSF)
Incremental Dynamic Analysis	1. MCE ground motion intensity (S_{MT}) 2. Median collapse intensity (S_{CT}) 3. Collapse Margin ration (CMR) 4. Adjusted collapse margin ratio (ACMR) 5. Acceptable collapse margin ratio

3.1 Non-linear Static Pushover Analysis

The pushover analysis is a nonlinear static method which is used in a performance-based analysis. Local nonlinear effects are modelled and the structure is pushed until a collapse mechanism is developed. With the increase in the magnitude of loads, weak links and failure modes of the buildings are found. It gives an idea of the maximum base shear that the structure is capable of resisting and the corresponding inelastic drift. Fig.3 provides the definitions of the terms needed to be found out through non-linear static pushover analysis.

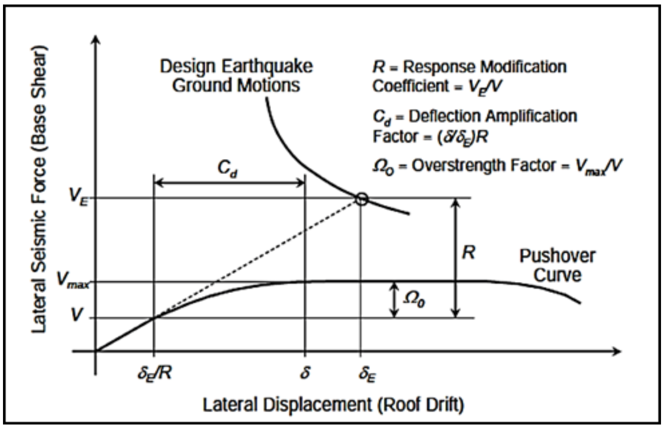

Figure -3: Illustration of Seismic Performance Factors

Overstrength factor(Ω) can be defined as the ratio of the maximum base shear resistance of the fully-yielded system to the design base shear.

$$\Omega = V_{max} / V \quad (1)$$

The period-based ductility for a given index archetype model (μ_T) is defined as the ratio of ultimate roof drift displacement (δu) to the effective yield roof drift displacement ($\delta_{y.eff}$).

$$\mu_T = \delta u / \delta_{y.eff} \quad (2)$$

3.2. Incremental Dynamic Analysis

Incremental dynamic analysis (IDA)is a computational analysis method of earthquake engineering for performing a comprehensive assessment of the behavior of structures under seismic loads. It includes a series of non-linear time history analysis with a suite of ground motions, during which the ground motions' intensities are increased using a specified scale factor. In this, considered structural model is subjected to one (or more) ground motion record(s), each scaled to multiple levels of intensity, until the structure reaches a collapse point, thus producing one (or more) curve(s) of considered response parameter versus intensity level. Scale factor can be defined as positive scalar which multiplies to ground motion to increase the intensity. Scale factor can be increased in a constant steps or distinct steps. Intensity measure (IM) can be defined as positive scalar which depends on the unscaled ground motions and it is increased monotonically with scale factor. IM can be increased by multiplying the scale factors to the ground motion. Damage measure (DM) can be defined as positive scalar which is also known as a Structural State Variable. DM characterizes more structural response which is subjected to prescribed seismic load. Current work includes assessment of collapse safety for two dimensional frames which replicates actual configurations of buildings of a particular locality in India.

FEMA P695 methodology suggests, for collapse safety assessment, IDA should be conducted under the factored gravity load combination and input ground motions from the Far-Field record set. For two-dimensional analyses, forty-four ground motion records (22 pairs- 44 individual components) should be applied as independent events to calculate the median collapse intensity. In the proposed work, by performing IDA, we are interested in getting the values for median collapse intensity (S_{CT}), MCE-intensity (S_{MT}) and thereby collapse margin ratio (CMR). Fig.4. explains the definition of CMR.

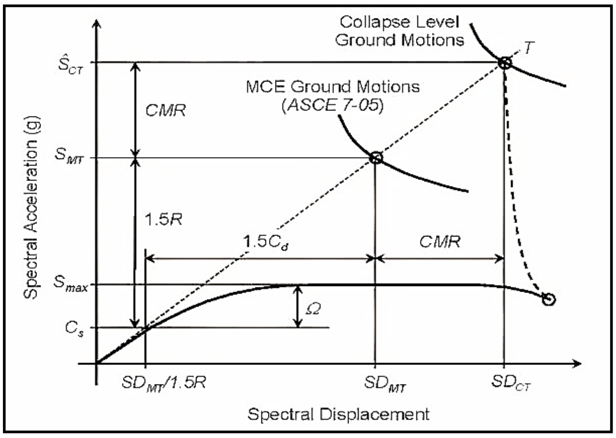

Figure -4: Definition of CMR.

Median collapse intensity *(S_{CT})* can be defined as the as the spectral intensity when half of the ground motions cause the structure to collapse and the MCE intensity*(S_{MT})* can be directly obtained from the response spectrum of MCE ground motions for considered seismic design category at the fundamental period, *T* of the structure or it can also be obtained from FEMA P695 provisions, based on fundamental period of considered structure. Collapse margin ratio can be defined as the ratio of median collapse intensity and MCE-intensity as,

$$CMR = S_{CT} / S_{MT} \tag{3}$$

[Baker and Cornell][11] (2006) have shown that rare ground motions in the Western United States, such as those corresponding to the MCE, have a distinctive spectral shape that differs from the shape of the design spectrum used for structural design in ASCE/SEI 7-05. The most direct approach to account for spectral shape would be to select a unique set of ground motions that have the appropriate shape for each site, hazard level, and structural period of interest. But this is not possible in a generalized procedure which includes assessing the performance of group of structures together, with a range of possible configurations, located in different geographic regions, with different soil site classifications.

To remove this limitation, simplified spectral shape factors, SSF, which depends on fundamental period and period-based ductility, are used to adjust collapse margin ratios. Tables 7-1a and 7-1b of FEMA P695 provides values of suitable SSF based on period-based ductility and the fundamental time period (calculated using equation 5-5 of FEMA P695 provision). Thus, the collapse margin ratios are needed to be adjusted by the appropriate SSF as,

$$ACMR = SSF \times CMR \tag{4}$$

For assessment, last step is to check whether adjusted values of collapse margin ratio are greater than the acceptable values for performance. Acceptable values of adjusted collapse margin ratio are based on total system collapse uncertainty, β_{TOT}, defined as square root of the sum of squares of record-to-record (β_{RTR}) uncertainty, design requirements-related (β_{DR}) uncertainty, test data-related (β_{TD}) uncertainty, and modeling (β_{MDL}) uncertainty, as

$$\beta_{TOT} = [(\beta_{RTR})^2+(\beta_{DR})^2+(\beta_{TD})^2+(\beta_{MDL})^2]^{1/2} \tag{5}$$

Acceptable values of adjusted collapse margin ratios can be calculated based on total system collapse uncertainty using table 7-3 of FEMA P695 provision. Acceptable performance is defined by the following two basic collapse prevention objectives:

- The probability of collapse for MCE ground motions is approximately 10%, or less, on average across a performance group.
- The probability of collapse for MCE ground motions is approximately 20%, or less, for each index archetype within a performance group.

Acceptable performance is achieved when, for each performance group, adjusted collapse margin ratios, ACMR, for each index archetype meet the following two criteria:

- The average value of adjusted collapse margin ratio for each performance group exceeds $ACMR_{10\%}$.

$$ACMR_i \geq ACMR_{10\%} \tag{6}$$

- Individual values of adjusted collapse margin ratio for each index archetype within a performance group exceeds $ACMR_{20\%}$.

$$ACMR_i \geq ACMR_{20\%} \tag{7}$$

3.3. Proposed Work

For this research work, six regular reinforced concrete special moment framed three dimensional buildings were designed, modelled using Indian codes like IS 456:2000 and IS 800:2000 in *Etabs-2017* software. They were analyzed by

Equivalent static analysis, to get sustainable design, proper section properties and reinforcement details, using IS 1893(Part 1):2016 in Etabs-2017 software. Present work includes assessment for two-dimensional index archetype models of actual buildings. The representative archetype frames are then modelled in *Seismostruct-2018*, with the design parameters obtained from basic analysis in Etabs-2017 and *Non-linear static pushover analysis* is carried out firstly to get required parameters like overstrength factor, period-based ductility, etc. as mentioned in table 1. After processing over the results obtained through Pushover analysis, selection of ground motion data was done using *PEER NGA database* and scaling of ground motions in done using *Seismomatch-2018.*The modelled archetype frames were analyzed by *Incremental dynamic analysis* with far-field record set including twenty-two record pairs (44 individual horizontal components) as provided *in FEMA P695 Appendix-A*, to get the parameters like MCE ground motion intensity, median collapse intensity, etc.as mentioned in table 1.Thus by obtaining all the values for parameters by analysis and plotting the results by graphical means, the goal was to check, whether the considered structures are possessing acceptable collapse margin of safety or not, using the provisions of FEMA P695. Various properties of models are as follows,

TABLE 2. MATERIAL PROPERTIES

Group	Material	Grades
PG1	Concrete Grade	M20
	Steel	Fe500
PG2	Concrete Grade	M25
	Steel	Fe500

TABLE 3. BUILDING GEOMETRY

Group	Plan Area	Typical Story height	Ground story height
PG1	12m X12 m	3m	4.5m
PG2	18m X 18m	3m	4.5m

For all the six archetypes, Soil B-medium or stiff soils was taken as per IS 1893(part-1):2016 for initial design considerations and to consider the same effects of soil type further, in this assessment methodology, Site class D (stiff soil) was chosen as soil type, as per ASCE/SEI 7-10.The initial scale factor for IDA analysis was taken to be 1.3S_{MT} as per the FEMA P695 provisions (i.e. Scale factor 2 for 1A and 2A and scale factor 2 for remaining four archetypes). All the frames are modelled and analyzed using Seismostruct-2018 a product of Seismosoft. The pictorial view of frames is shown through figures from Figure 5 to Figure 10.

TABLE 4. BASIC CONFIGURATIONAL PROPERTIES

Group	ID	Grouping criteria			Number of stories	Height (m)
		Bay length (m)	Design load level			
			Live load	Seismic design category		
PG1	1A	4m	3	SDC D_{max}	4	13.5
	1B				8	25.5
	1C				12	37.5
PG2	2A	6m	4	SDC D_{max}	4	13.5
	2B				8	25.5
	2C				12	37.5

TABLE 5. SECTIONAL PROPERTIES

ID	Column Size (mm)	Beam Size(mm)
1A	400 x400	400 x400
1B	450 x 450	450 x 450
1C	500 X 500	500 X 500
2A	450 x 450	450 x 450
2B	600 X 600	600 X 600
2C	600 X 600	600 X 600

TABLE 6. REINFORCEMENT DETAILS

ID	Column		Beam	
	Longitudinal	Shear	Longitudinal	Shear
1A	8-32 Ø	8 Ø@220c/c	8-16 Ø	8 Ø@60c/c
1B	14-32 Ø	8 Ø@90c/c	8-20 Ø	8 Ø@50c/c
1C	16-32 Ø	8 Ø@110c/c	8-20 Ø	8 Ø@50c/c
2A	14-32 Ø	8 Ø@75c/c	6-25 Ø	8 Ø@50c/c
2B	16-40 Ø	8 Ø@50c/c	6-32 Ø	8 Ø@50c/c
2C	22-36 Ø	8 Ø@50c/c	6-32 Ø	8 Ø@50c/c

For the calculations of overstrength factors and the ratio of distribution of incremental loads for Non-linear static pushover analysis in Seismostruct-2018, there was a need of values of design base shear, initially. Through this process of manual calculations, one more finding of fundamental time period of archetypes was calculated using equation 5-5 of FEMA P695.To calculate the design base shear value equation 12.8-1 of ASCE 7-10 was used. These findings are tabulated in Table 7.

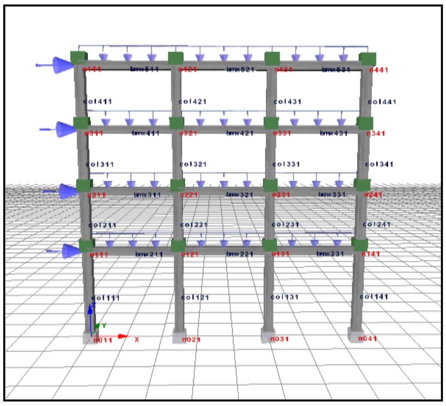

Figure-5. Archetype 1A

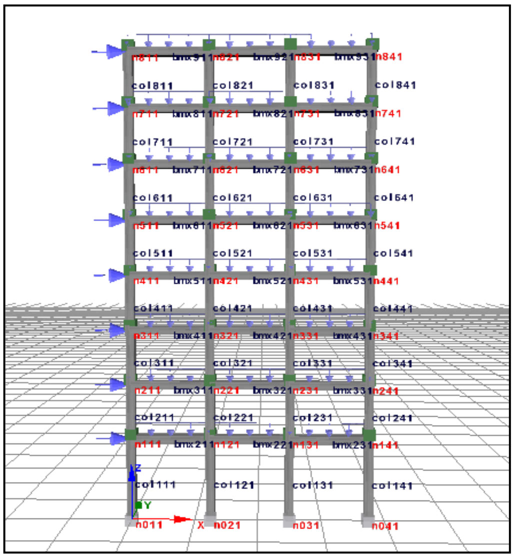

Figure-6. Archetype 1B

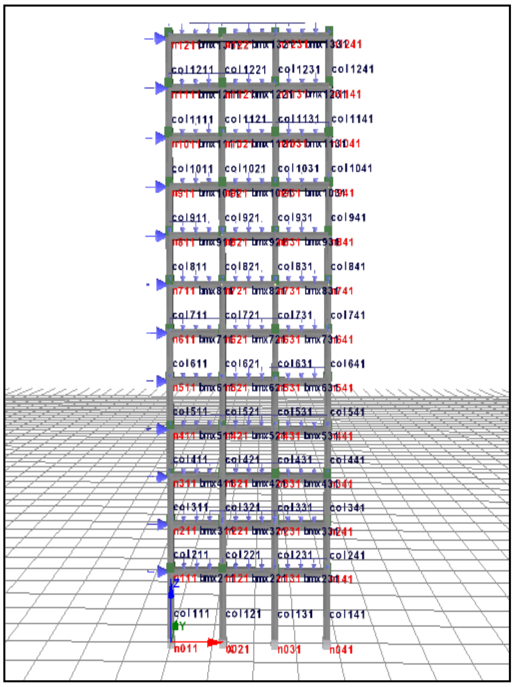

Figure-7. Archetype 1C

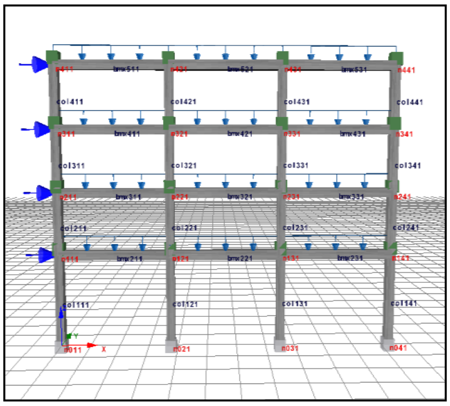

Figure-8. Archetype 2A

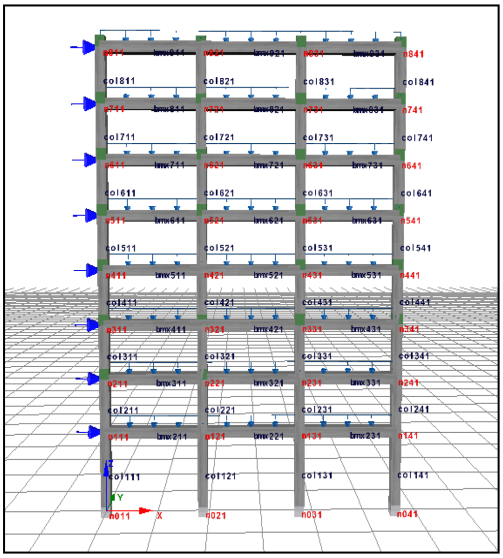

Figure-9. Archetype 2B

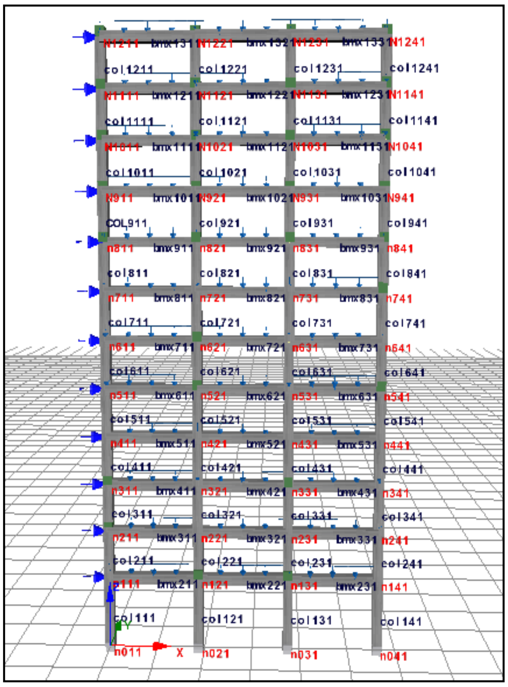

Figure-10. Archetype 2C

TABLE 7. CALCULATIONS FOR BASE SHEAR

ID	S_{M1} [g]	S_{D1} [g]	R	I_e	Cu	Ta	T	Cs	W (KN)	Vb (KN)
1A	0.9	0.6	8	1.5	1.4	0.4	0.56	0.201	1600.8	321.59
1B	0.9	0.6	8	1.5	1.4	0.8	1.12	0.100	3380.18	339.53
1C	0.9	0.6	8	1.5	1.4	1.2	1.68	0.067	5392.5	361.10
2A	0.9	0.6	8	1.5	1.4	0.4	0.56	0.201	3528.23	708.80
2B	0.9	0.6	8	1.5	1.4	0.8	1.12	0.100	8193.6	823.02
2C	0.9	0.6	8	1.5	1.4	1.2	1.68	0.067	12813.23	858.03

4. RESULTS AND DISCUSSION

After the manual calculations for design base shear and theoretical fundamental time period Nonlinear static pushover analysis can be performed. Once Nonlinear static pushover analysis as well as the Incremental dynamic analysis are completed, the pushover and IDA curves are plotted, to get values of certain parameters, graphically. Pushover curves are plotted between *base shear* and *roof drift ratio.* Whereas for IDA curves, Intensity measure (IM) was selected as the *first mode spectral acceleration (Sa(T1,5%))* and the Damage measure (DM) was selected to be *Maximum story drift ratio*. These pushover curves and multi-record IDA curves are presented here for all six archetypes, one by one.

4.1. Curves for Archetype 1A

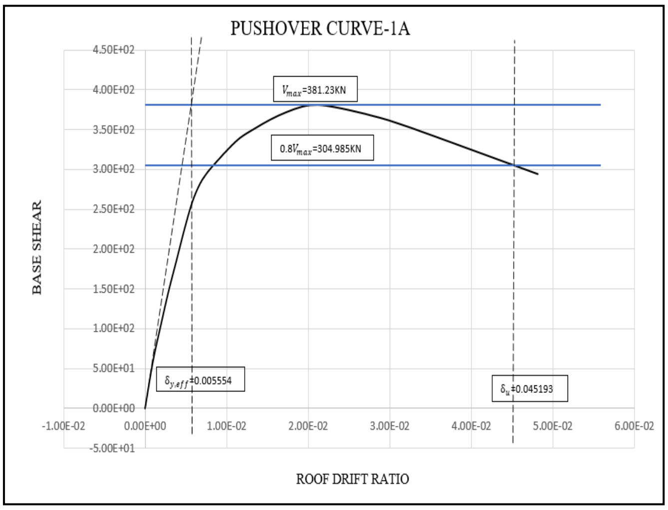

Figure-11. Pushover curve for 1A

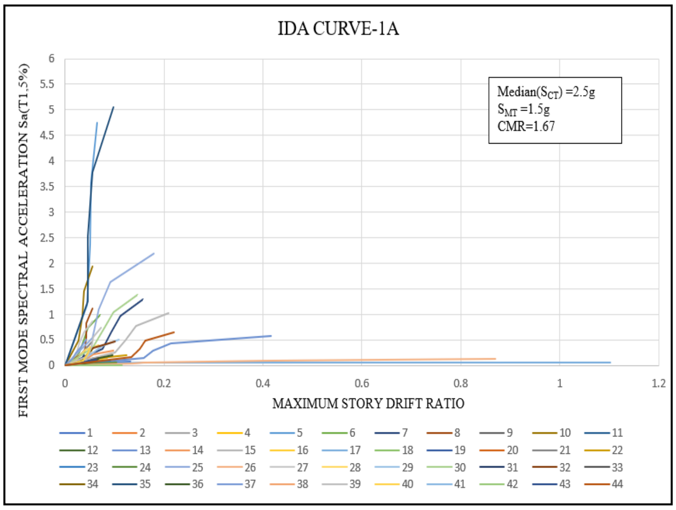

Figure-12. Multi-record IDA curve for 1A

4.2. Curves for Archetype 1B

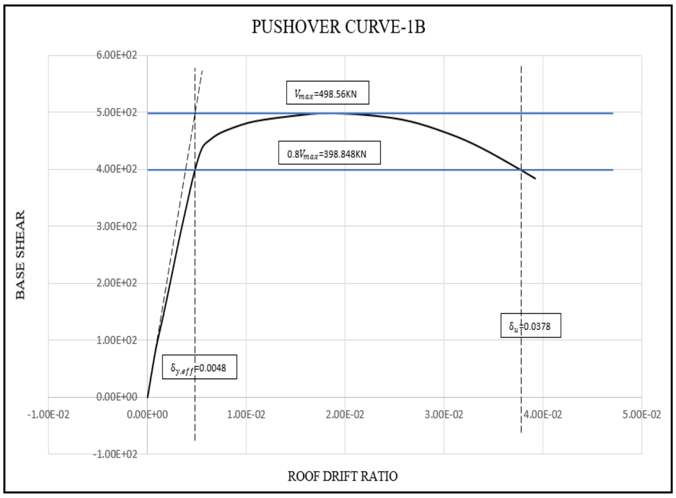

Figure-13. Pushover curve for 1B

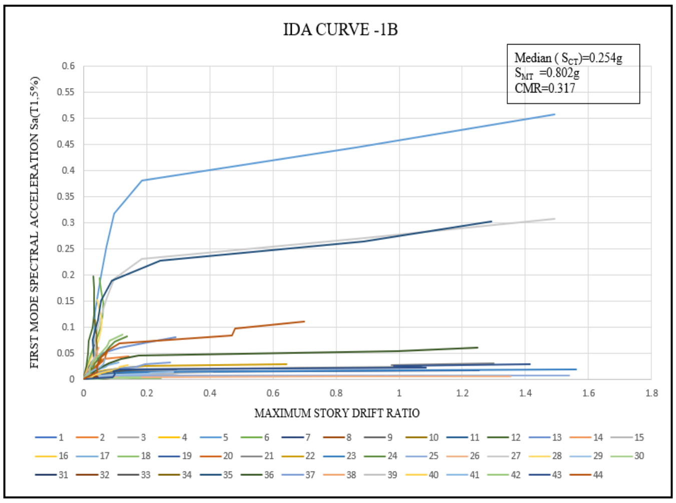

Figure-14. Multi-record IDA curve for 1B

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4.3. Curves for Archetype 1C

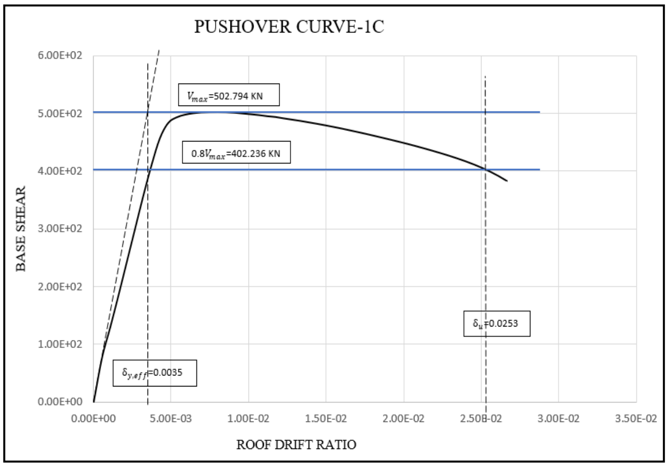

Figure-15. Pushover curve for 1C

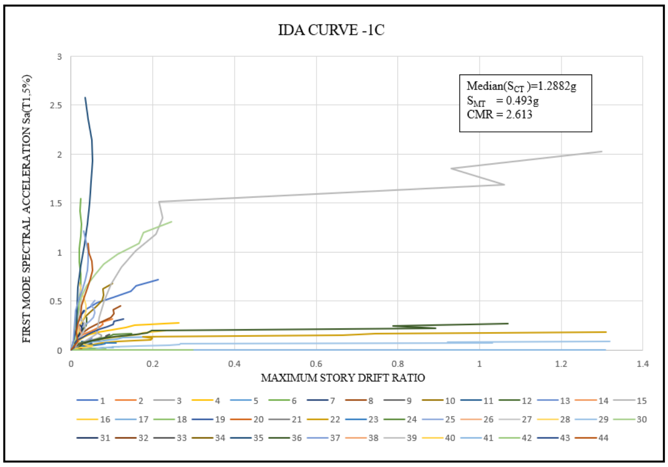

Figure-16. Multi-record IDA curve for 1C

4.4. Curves for Archetype 2A

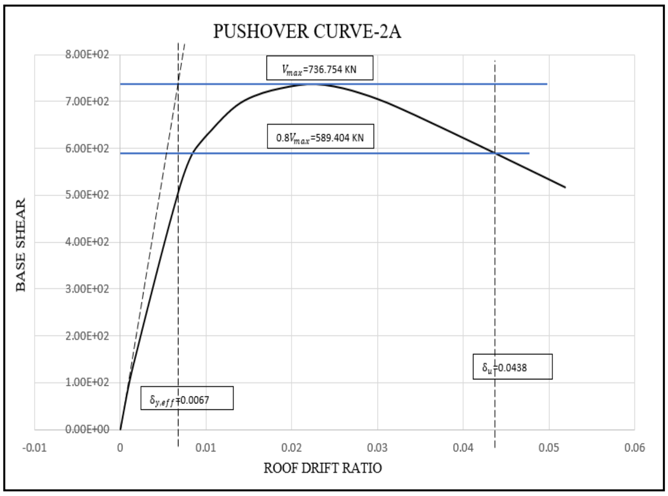

Figure-17. Pushover curve for 2A

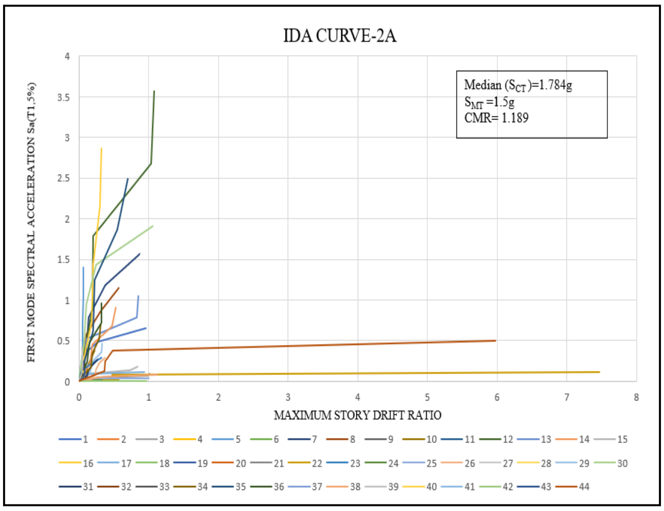

Figure-18. Multi-record IDA curve for 2A

4.5. Curves for Archetype 2B

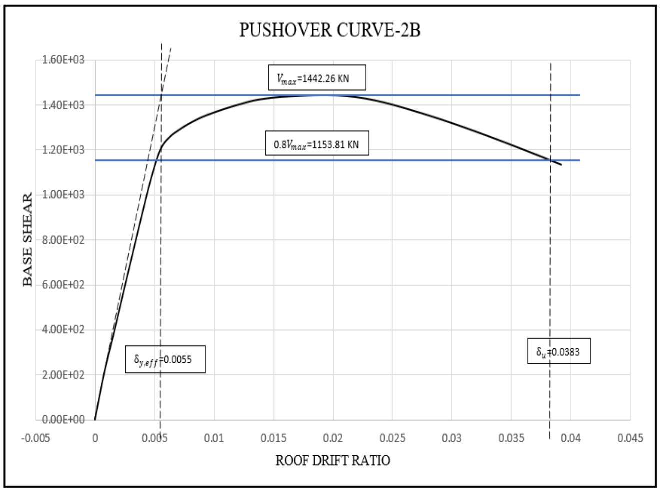

Figure-19. Pushover curve for 2B

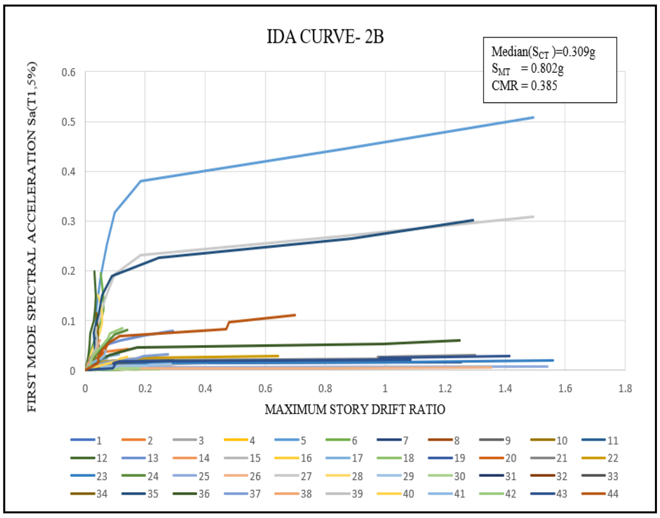

Figure-20. Multi-record IDA curve for 2B

4.6. Curves for Archetype 2C

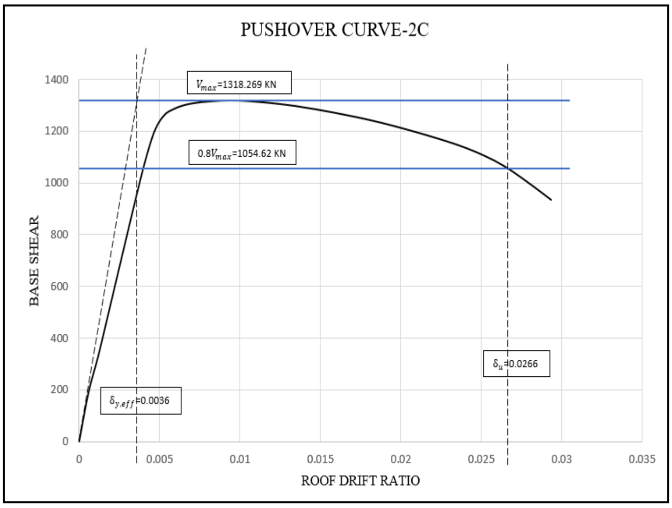

Figure-21. Pushover curve for 2C

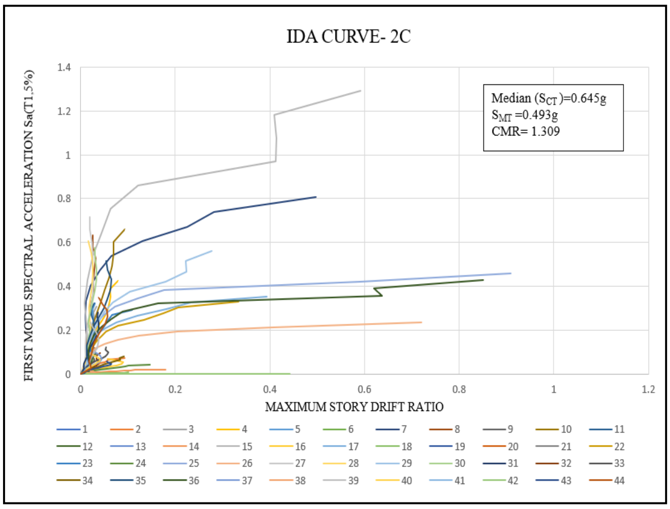

Figure-22. Multi-record IDA curve for 2C

Excel-2016 was used to extract the values of the parameters obtained through analysis procedures carried out in Seismostruct-2018 and to draw the respective curves. The values for maximum base shear for each of the archetype and the values of median collapse intensity *(S_{CT})* for each archetype are obtained from the respective Nonlinear static pushover curves and the respective IDA curves for corresponding archetypes, as shown here from fig.11 to fig. 22. The required values for MCE ground motion intensities *(S_{MT})* were obtained from the response spectrum for seismic design category SDC D_{max} as given in ASCE/SEI 7-10.The total collapse uncertainty *(β_{TOT})* was taken to be 0.475 for good model quality and superior quality of test data for period based ductility greater than or equal to 3, as per table 7-2b of FEMA P695.The further calculated values for all the required parameters like collapse margin ratios, adjusted collapse ratios, acceptable values for collapse margin ratios, etc. are mentioned below in table 8 and table 9.

TABLE 8. PARAMETERS OBTAINED AFTER PUSHOVER ANALYSIS

ID	V_{max} (KN)	Ω	δu	δy.eff	μ_T	T_1	SSF	S_{MT}
1A	381.2317	1.185	0.0452	0.0055	8.136	4.51	1.34	1.5
1B	498.5604	1.468	0.0378	0.0048	7.870	11.72	1.49	0.802
1C	502.7944	1.392	0.0253	0.0035	7.257	3.35	1.57	0.493
2A	736.7544	1.039	0.0438	0.0067	6.516	5.39	1.31	1.5
2B	1442.257	1.753	0.0383	0.0055	6.927	10.03	1.45	0.802
2C	1318.269	1.536	0.0266	0.0036	7.413	4.16	1.58	0.493

TABLE 9. PARAMETERS OBTAINED AFTER IDA

ID	S_{MT} [g]	S_{CT} [g]	CMR	ACMR	B_{TOT}	ACMR$_i$	$ACMR_{10\%}$	$ACMR_{20\%}$
1A	1.5	2.5	1.67	2.2378	0.475		1.84	1.49
1B	0.802	0.254	0.317	**0.4725**	0.475	2.2748	1.84	1.49
1C	0.493	1.288	2.617	4.1142	0.475		1.84	1.49
2A	1.5	1.784	1.189	1.5528	0.475		1.84	1.49
2B	0.802	0.309	0.385	**0.5583**	0.475	**1.3931**	1.84	1.49
2C	0.493	0.645	1.309	2.0683	0.475		1.84	1.49

The calculations and results show that Archetypes 1A,1C,2A and 2C passed the performance criteria of approximately 20% or less probability of collapse for MCE ground motions for individual structure within a performance group but Archetypes 1B and 2B are failed to pass the same criteria. The performance group-1 seem to be passing the criteria of approximately 10% or less probability of collapse for MCE ground motions on average across a performance group but performance group-2 seem to be failing for the same criteria. Although Archetype 1B found out to be failing for the criteria for individual structure within a performance group, Performance group-1 passed the criteria for average of a performance group. Results clearly given the idea about mid-rise buildings (8 storied) are not passing the criteria of this methodology.

5. CONCLUSIONS

This study includes the designing and modelling of the six regular 4-story, 8story and 12-story reinforced concrete special moment framed three dimensional buildings according to Indian codes and performing seismic collapse safety assessment methodology over the two-dimensional index archetype space frames, which are the representative models of actual three-dimensional buildings designed earlier, by means of performing the Nonlinear Static Pushover Analysis and Incremental Dynamic Analysis. The FEMA P695 methodology was used for collapse safety assessment of the structures through Incremental Dynamic Analysis. Seismic design category SDC D_{max} and soil class D (ASCE/SEI 7-10) were considered for this study. As two-dimensional structures are considered for the actual work, all forty-four ground motion records (twenty-two pairs) are applied as independent events to calculate the median collapse intensity (S_{CT}) for each index archetype model. Thereby, the collapse safety margins were calculated for the considered structures. The acceptance collapse criteria included 20% probability of collapse for an individual archetype and 10% probability of collapse in average for all archetypes in a performance group. After getting the basic parameters from the analyses and further manual calculations by following the provisions of FEMA P695, exact results were obtained. Following conclusions can be figured out from the results obtained.

1. The methodology followed in this work as per the provisions of FEMA P695, employs the performance-based concepts which provides a more consistent and scientifically based method to assess collapse safety of buildings.
2. This work illustrated how the FEMA P695 methodology of collapse safety assessment can be applied iteratively to buildings designed based on Indian codes.
3. It is observed that short-period $(T \leq Ts)$ having, 4-story archetype structures have shown sufficient margin of safety against collapse.
4. Long period having $(T \geq Ts)$ 8-story archetype structures are found out to be possessing very less values for margin of safety against collapse. As developing country like India has more mid-rise buildings, this methodology itself explains its importance for the need of collapse safety assessment of buildings in India.
5. 12 story archetype structures having long-period $(T \geq Ts)$, have shown sufficient margin of safety against collapse, almost 50% more than that of 4-story archetype structures of the same performance group.
6. Although Archetypes 1B and 2B found out to be failing for the criteria for individual structure within a performance group, performance group-1 passed the criteria for 10% probability of collapse for MCE ground motions on average across a performance group, whereas performance group-2 failed to pass the criteria for average of a performance group.
7. The structures having more bay length values (i.e. structures in performance group 2) possess lesser adjusted collapse margin ratios than those with comparatively lower bay lengths (i.e. structures in performance group 1).
8. Based on this study and according to general methodology of FEMA P695, it is also concluded that the assumed response modification factor of R=8 for space frames is acceptable.
9. This work demonstrated that seismic collapse safety assessment methodology is useful to check whether the existing structures are having acceptable seismic collapse safety or not.
10. Present work illustrated that this seismic collapse safety assessment methodology is also helpful to develop system design provisions that result in acceptable collapse safety of a newly proposed structural system.

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REFERENCES

[1] Abbie Liel, Curt Haselton, Gregory Deierlein, Jack Baker (2007), "Assessing the Seismic Collapse Risk of Reinforced Concrete Frame Structures, Including the Effects of Modeling Uncertainties", Special Workshop on Risk Acceptance and Risk Communication-2007, Stanford University.

[2] Ali Reza Manafpour and Maryam Tohidian (2017), "Collapse Safety Margin in Iranian Seismic Design Code: Case Studies of RC Frame Structures", JSEE Vol.19, No. 1,201

[3] ASCE, 2010, Minimum Design Loads for Buildings and Other Structures, ASCE 7-05, American Society of Civil Engineers, Reston, Virginia.

[4] C.B. Haselton, A.B. Liel and G.G. Deierlein (2010), "Example application of the FEMA P695 (ATC-63) methodology for the collapse performance evaluation of reinforced concrete special moment frame systems",2010EQConf-ATC63.

[5] Curt B. Haselton, Jack W. Baker, Abbie B. Liel, and Gregory C, (2011), "Accounting for Ground-Motion Spectral Shape Characteristics in Structural Collapse Assessment through an Adjustment for Epsilon", 332 / Journal of Structural Engineering © ASCE / March 2011.

[6] Dimitrios Vamvatsikos and C. Allin Cornell (2002), "Incremental dynamic analysis", Earthquake Engng Struct. Dyn. 2002; 31:491–514.

[7] F. Behnamfar & S. Haghollahi (2012), "Collapse Performance Evaluation of Reinforced Special Moment Frame Systems Considering Soil–Structure Interaction", WCEE 2012.

[8] FEMA P695, June 2009, "Quantification of building seismic performance factors", prepared by Applied Technology Council, Redwood city, California.

[9] Gregory G. Deierlein, Abbie B. Liel, Curt B. Haselton, Charles A. Kircher (2008), "ATC 63 Methodology for Evaluating Seismic Collapse Safety of Archetype Buildings", ASCE SEI 2008, ATC-63Session.

[10] Roberto Villaverde (2007), "Methods to Assess the Seismic Collapse Capacity of Building Structures: State of the Art", Journal of Structural Engineering © ASCE / January 2007 /133(1): 57-66

[11] Jack W. Baker and C. Allin Cornell (2006), "Spectral shape, epsilon and record selection", Wiley InterScience, Earthquake Engng Struct. Dyn. 2006; 35:1077–1095.

[12] Seismosoft. (2018). Seismostruct v5.2.1– A computer program for static and dynamic nonlinear analysis of framed structures. Italy: retrieved from http://www.seismosoft.com.

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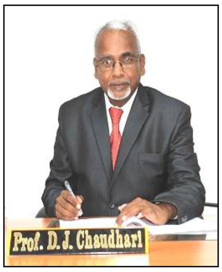

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