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Evaluation of Seismic Performance Factors for Concrete Filled Steel Tube Diagrid Structural System

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Abstract

Seismic performance factors are used in current building codes and standards to estimate strength and deformation demands on seismic force resisting systems that are designed using linear methods of analysis, but are responding in the nonlinear range. Many recently evolved seismic force resisting systems have never been subjected to any significant level of earthquake ground shaking. As a result, the seismic response characteristics of many systems, and their ability to meet seismic design performance objectives, are both untested and unknown. Therefore, it is necessary to determine the seismic performance factors of new seismic force resisting systems proposed for inclusion in building codes that will result in equivalent safety against collapse during an earthquake when properly implemented in the seismic design process. In this study, the response modification factor, overstrength factor and period based ductility for concrete filled steel tube (CFST) diagrid structural system are evaluated. To quantify these factors, the rational procedure introduced in Federal Emergency Management Agency (FEMA) P695, which is based on low probability of structural collapse and encompasses nonlinear static and dynamic analyses, is used. To this end, performance group consisting of 4, 8, 16 and 24 storey diagrid structures with 50° angle of external braces is considered. Nonlinear static analyses are performed to obtain overstrength factor and period-based ductility. Incremental dynamic analyses are then performed to assess collapse margin ratio of the archetypes. For modelling and numerical analysis, Open System for Earthquake Engineering Simulation (OpenSees), an open source software is used.

Keywords: Seismic performance factors, FEMA P695, Nonlinear static analysis, Incremental dynamic analysis, OpenSees.

1. Introduction

Diagrid is a type of space truss. It comprises of inclined members on the periphery of the building, forming a diagonal grid on the periphery. Diagonal grid creates a series of triangulated trusses, due to the intersection of diagonal grid and periphery beams, which resist lateral loading by truss action. Diagrid structure eliminates the vertical columns on the periphery making the structure almost column free. It requires only internal columns for resisting the gravity loads and increases the availability of space inside the building by accommodating the services in core. Diagrid structure saves approximately 20% of steel as compared to conventional frame structure [11].

The inclined members in diagrid structure resists the lateral load by axial action because of its truss configuration whereas in conventional lateral load resisting system, like framed tube, lateral load is resisted by bending and shear action. The angular configuration of diagonal members allows multiple load paths which makes diagrid structural system less vulnerable to progressive collapse. Study shows that progressive collapse of diagrid structure does not take place until removal of more than 11% of inclined members [11].

The latest introduction in diagrid structure is use of concrete filled steel tube diagonal. This composite section provides more axial strength and stiffness than conventional steel tube diagrid structure and provides better performance of diagrid joint under seismic load. It also reduces the consumption of costly steel to achieve economy in construction and because of concrete, size of steel tube reduces which makes its handling on the construction site easy.

Seismic performance factors, such as response modification factor (R), overstrength factor (Ω_0), period based ductility (μ_T) and deflection amplification factor (C_d) of new seismic force resisting systems are evaluated based on standard methodology provided by FEMA P695. The FEMA P695 methodology is primarily consistent with the life safety parameter. This is achieved by assessing a very less probability of structure under the maximum considered earthquakes. FEMA P695 methodology involves three main steps in order to determine the seismic performance factors: (1) to characterize system behaviour and define appropriate index archetypes (structural system with different storey height and configuration); (2) to prepare nonlinear model and perform nonlinear static and dynamic analysis; and (3) to evaluate presumed seismic performance factors [10].

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1.1. Nonlinear analyses

To simulate the collapse performance of a system, nonlinear static and dynamic analyses shall be conducted using nonlinear models. At first, by pushover analysis, the overstrength factor (Ω_0) and period based ductility (μ_T) of the archetype models are evaluated. Then, nonlinear dynamic analyses are performed to characterize the collapse properties of the seismic force resisting system. Median collapse capacity (S_{CT}) and collapse margin ratio (CMR) are two key parameters of each model which are obtained from the results of dynamic analyses. The CMR is defined as the ratio of S_{CT} to the spectral intensity of the maximum considered earthquake (MCE) at the fundamental period of the structure (S_{MT}) [2].

1.2. Performance evaluation

Performance evaluation process for each model consists of three main steps as: (1) adjusting CMR of the model by spectral shape factor (SSF); (2) determining acceptable collapse margin ratios based on total uncertainty of the model; and (3) checking performance of the model by comparing the adjusted collapse margin ratios (ACMR) with the acceptable values of collapse margin ratio. If ACMR becomes greater than acceptable CMR, seismic performance of the model is suitable, and the employed R in designing the model is valid. If not, the seismic performance of the model is not acceptable and whole process should be repeated by using a different R or modifying the design requirements [2].

2. Development of structural system

In this study, performance group consisting of 3D models of 4, 8, 16 and 24 storey CFST diagrid is considered. Typical floor plan of CFST diagrid building and elevation of building considered in study are shown in Fig. 1. The inclination of diagonal columns on periphery of building is considered as 50°. Every archetype has uniform storey height of 3.5 m. The sizes of beam, column and CFST diagonal members that are considered in modeling the diagrid building are mentioned in Table 1 and 2. Cross-section of column and CFST diagonal are shown in Fig. 2 and 3 respectively.

The column section comprises of double I-section and cover plates at top and bottom. The I-section and thickness of cover plate adopted in column section are ISMB 350 and 15mm, ISMB 500 and 10mm, ISMB 600 and 20 mm for 8, 16 and 24 storey CFST diagrid building respectively.

Table 1: Gravity load resisting members

	4 storey	8 storey	16 storey	24 storey
Beam B1	ISMB600	ISMB600	ISMB600	ISMB600
Beam B2	ISMB500	ISMB500	ISMB500	ISMB500
Column C1	ISMB600	380mmx380mm	520mmx520mm	640mmx640mm

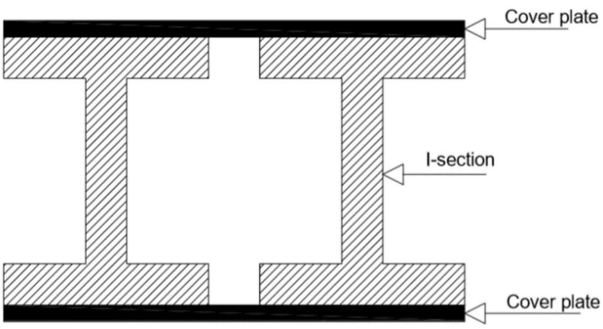


Fig. 2 Cross-section of column (for 8, 16 and 24 storey diagrid building)

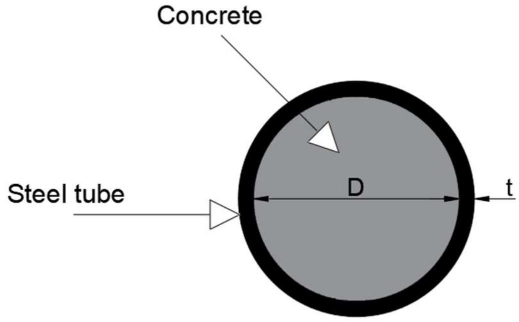
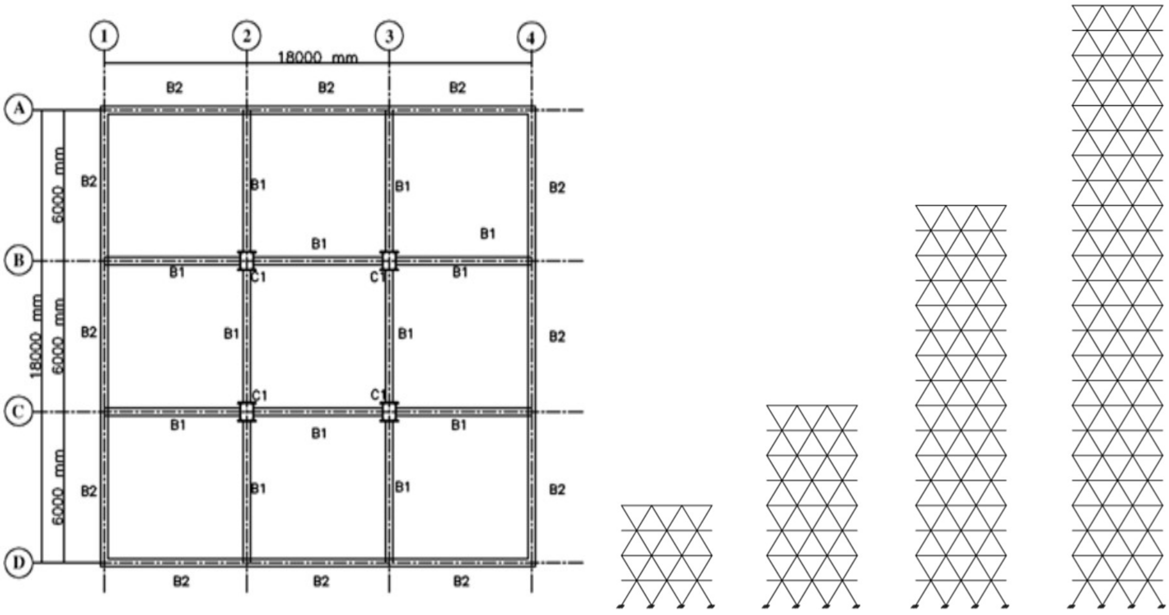


Fig. 3 Cross-section of CFST diagonal

Table 2: Diagonal CFST size

Storey No.	4 storey		8 storey		16 storey		24 storey	
	D (mm)	t (mm)	D (mm)	t (mm)	D (mm)	t (mm)	D (mm)	t (mm)
1−2	170	5	240	6	305	7.5	355	8.5
3−4	140	4.5	210	6	290	7	340	8
5−6	-	-	185	5.5	270	6.5	325	8
7-8	-	-	150	4.5	250	6	310	7.5
9-10	-	-	-	-	225	6	285	7.5
11-12	-	-	-	-	200	5.5	275	7
13-14	-	-	-	-	175	5	255	6.5
15-16	-	-	-	-	140	4.5	240	6
17-18	-	-	-	-	-	-	230	5.5
19-20	-	-	-	-	-	-	200	5
21-22	-	-	-	-	-	-	180	4.5
23-24	-	-	-	-	-	-	150	4



(a) (b)

Fig. 1 (a) Typical plan of CFST diagrid building (b) Elevation of 4, 8, 16, 24 storey CFST diagrid building

3. Nonlinear model development

For numerical modeling and analysis of CFST diagrid structures, considered in study, Open System for Earthquake Engineering Simulation (OpenSees) software is used [16]. The steel and concrete materials are defined as hysteretic steel material Menegotto-Pinto with isotropic hardening (Steel02) and Concrete02 material respectively and are assigned to each fiber of the element. Main parameters of Steel02 material are defined as: the yield stress [fy = 310 MPa (for diagonal elements) and fy = 250 MPa (for column elements)], the young's modulus (E = 200 GPa), strain hardening ratio (b = 0.02), three parameters (R0 = 18.5, cR1 = 0.925, cR2 = 0.15) to control the transition from elastic to plastic branches, four parameters (a1 = 0.0005, a2 = 0.01, a3 = 0.0005, a4 = 0.01) to account for cyclic isotropic hardening. Cross section of beam elements is defined by elastic section. Cross section of column and diagonal elements are defined by quadrilateral and circular fiber section respectively. The patch command is used to generate number of fibers over a cross-sectional area. The beam, column and diagonal elements are modeled using nonlinear Beam-Column element with the command "element dispBeamColumn" which considers the spread of plasticity along the element. The diagonal elements are pinned at the base and to simulate the effects of rigid floor, all nodes of each level are constrained by the command "rigidDiaphragm". The P-Delta coordinate transformation command is used to include the P-Δ second-order effects in modeling the central gravity columns. Newmark method with parameters of γ = 0.5 and β = 0.25 and Rayleigh damping theory with 5% damping ratio are applied for dynamic analysis. Flow of modeling of CFST diagrid structures in OpenSees is shown in Fig. 4

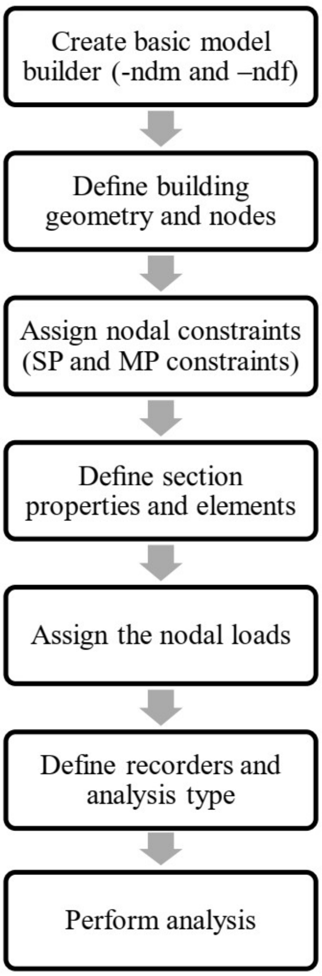


Fig. 4 Flow of modeling

4. Nonlinear static (pushover) analyses:

Following the FEMA P695 guidelines, overstrength factor (Ω_0) and period based ductility (μ_T) of the archetypes are evaluated using pushover analysis. For that purpose, first the gravity load combination given by Eq. (1), in which D and L represent the values of dead and live loads respectively, is applied to the model. Gravity loads are assigned as distributed load to all the elements using the element load command. The gravity loads are applied as a plain load pattern with a constant time series since the gravity loads always act on the structure.

$$1.0D + 0.25L \tag{1}$$

Then, lateral loads are distributed to the frame using Eq. (2) as per IS 1893 (Part 1): 2016. Lateral loads are applied to all the frame nodes in a given floor. A plain load pattern with a linear time series is used for lateral load application so that loads increase with time. Fig. 5 shows the FEMA P695 suggested method to determine the overstrength (Ω) and ductility (μ_T) coefficients as defined by Eq. (3) and Eq. (4):

$$Q_i = \frac{W_i h_i^2}{\sum W_i h_i^2} \times V_B \tag{2}$$

$$\Omega = \frac{V_{max}}{V} \tag{3}$$

where, V_{max} and V are the maximum base shear and design base shear, respectively

$$\mu_T = \frac{\delta_u}{\delta_{y,eff}} \tag{4}$$

where, δ_u is ultimate roof displacement corresponding to the 80% of maximum base shear ($0.8V_{max}$) and $\delta_{y,eff}$ is the effective yield roof displacement calculated as:

$$\delta_{y,eff} = C_0 \times \frac{V_{max}}{W} \times \frac{g}{4\pi^2} \times (\max(T,T_1))^2 \tag{5}$$

where, C_0 is coefficient of fundamental mode displacement, V_{max}/W is the ratio of maximum base shear obtained from pushover analysis to the seismic weight of the structure, g is the gravitational constant, T is the fundamental period of the structure calculated using codal provision and T_1 is the fundamental period of the structure obtained using eigenvalue analysis. The coefficient C_0 is calculated as:

$$C_0 = \phi_{1,r} \times \frac{\sum m_x \phi_{1,x}}{\sum m_x (\phi_{1,x})^2} \tag{6}$$

where, m_x is the mass of building at floor x; and $\phi_{1,x}$ ($\phi_{1,r}$) is the ordinate of the fundamental mode at floor x (roof), and N is the number of stories. Fig. 6 represents pushover curves for archetypes and the summary of results is presented in Table 3.

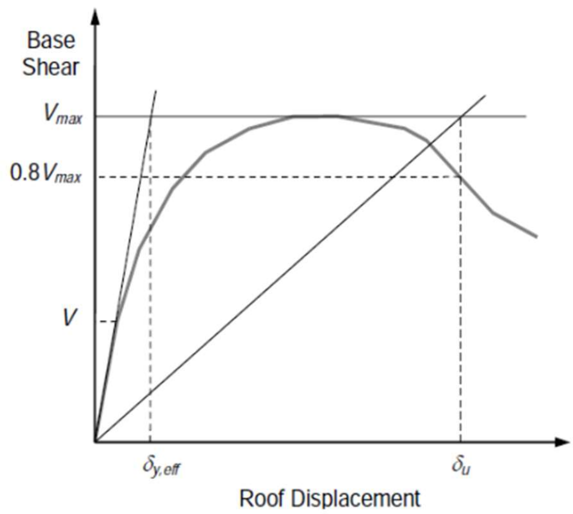


Fig 5. Idealized pushover curve [10]

5. Nonlinear dynamic analyses

Incremental dynamic analysis (IDA) proposed by Vamvatsikos and Cornell [12] is utilized under a suite of multiple scaled ground motion records to quantify the median collapse capacity (S_{CT}) as well as collapse margin ratio (CMR) for each archetype. S_{CT} is defined as the ground motion intensity at which half of the ground motions in set causes collapse of the structure.

In this step, time history analysis is performed with increasing scale factor until the structure reaches the collapse limit. For each scale factor, maximum story drift and spectral acceleration corresponding to 5% damping is noted and graph of maximum story drift against spectral acceleration is plotted. The input ground motions applied to CFST diagrid structures in this study consist of 9 individual far-field earthquake records. Table 4 provides a summary of ground motion parameters used in IDA. In this suite of records, the minimum distance from the fault is considered as 11.1 km and maximum distance of 26.4 km. The site is considered with stiff soil and shear velocity considered from 180 to 360 m/s. Range of earthquake magnitude is considered M6.5 to M7.6. Time history data is collected from PEER ground motion data base [15].

After performing the incremental dynamic analyses, the collapse margin ratio as a primary parameter to evaluate collapse safety of the CFST diagrid archetypes, is calculated as the ratio of median collapse intensity (S_{CT}) to the MCE intensity (S_{MT}) as:

$$CMR = \frac{S_{CT}}{S_{MT}} \tag{7}$$

Fig. 7 shows IDA curves for all time histories in which which median collapse intensity (S_{CT}) and MCE intensity (S_{MT}) are shown. The median collapse intensity (S_{CT}) is obtained from fragility curve corresponding to 50% collapse probability and the MCE intensity (S_{MT}) is obtained from Table 6-1 of FEMA P695 [10] or MCE design spectrum suggested by FEMA P695 for the seismic design category D_{max} at the fundamental period of each archetype.

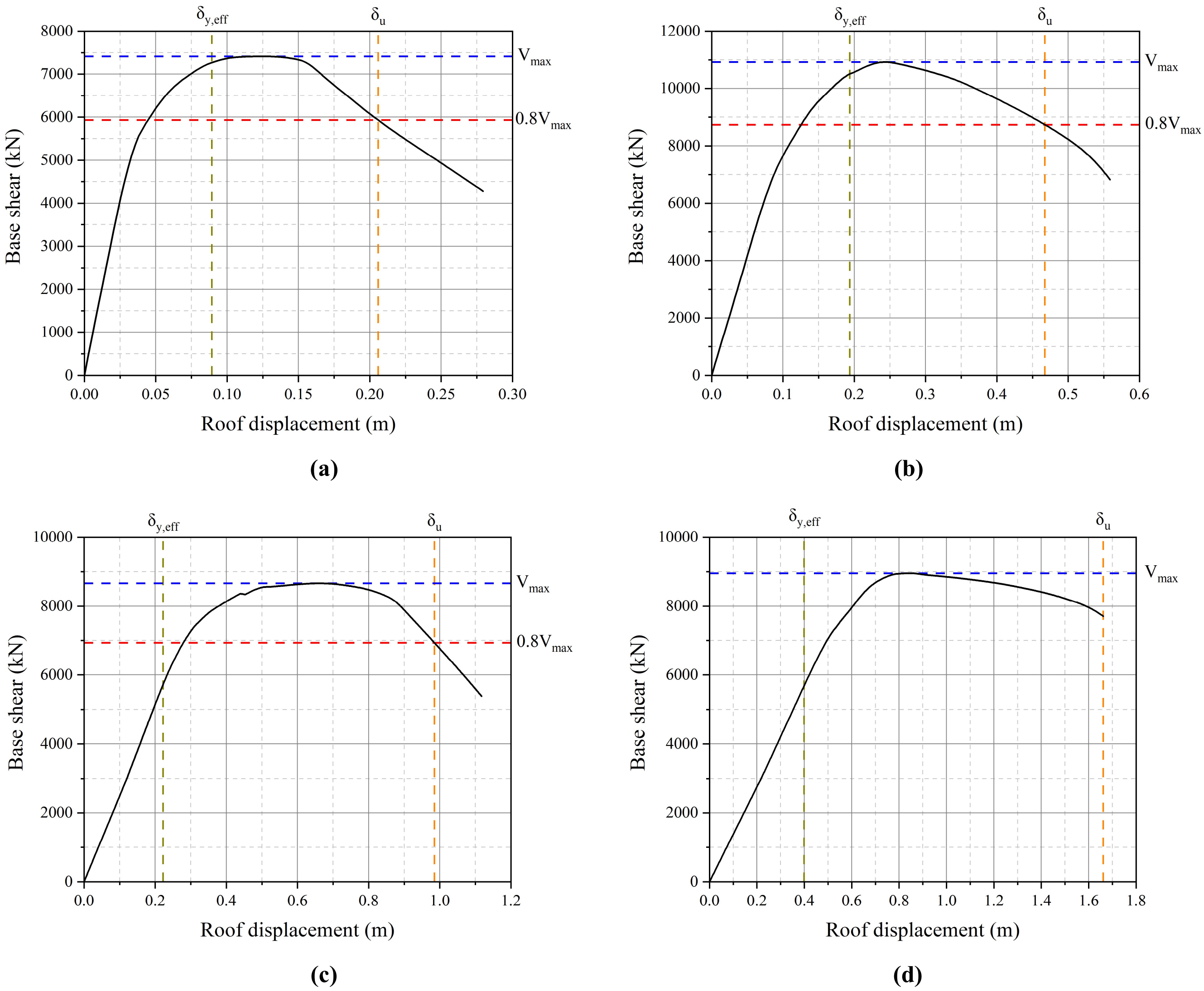


Fig. 6 Pushover Curve: (a) 4-storey, (b) 8-storey, (c) 16-storey, (d) 24-storey

Table 3: Summary of nonlinear static analyses

	4-storey	8-storey	16-storey	24-storey
V_{max} (kN)	7412.476	10922.211	8664.403	8956.06
V (kN)	791.623	972.51	1193.065	1478.446
Ω	9.36	11.23	7.26	6.06
δu (m)	0.205	0.467	0.985	1.661
δy,eff (m)	0.089	0.193	0.222	0.4
μ_T	2.304	2.41	4.43	4.16

Table 4: Ground motion records used in incremental dynamic analysis

ID	Earthquake name	Year	Station	Record no.	Magnitude	PGA (g)
1	Imperial Valley-06	1979	Compuertas	167	6.53	0.584
2	Imperial Valley-06	1979	Delta	169	6.53	0.697
3	Imperial Valley-06	1979	El Centro Array #12	175	6.53	0.4
4	Superstition Hills-02	1987	Westmorland Fire Station	728	6.54	0.96
5	Superstition Hills-02	1987	El Centro Imp. Co. Cent	721	6.54	0.626
6	Loma Prieta	1989	Sunnyvale - Colton Ave.	806	6.93	0.725
7	Loma Prieta	1989	Agnews State Hospital	737	6.93	0.503
8	Landers	1992	Desert Hot Springs	850	7.28	0.596
9	Landers	1992	Yermo Fire Station	900	7.28	0.592

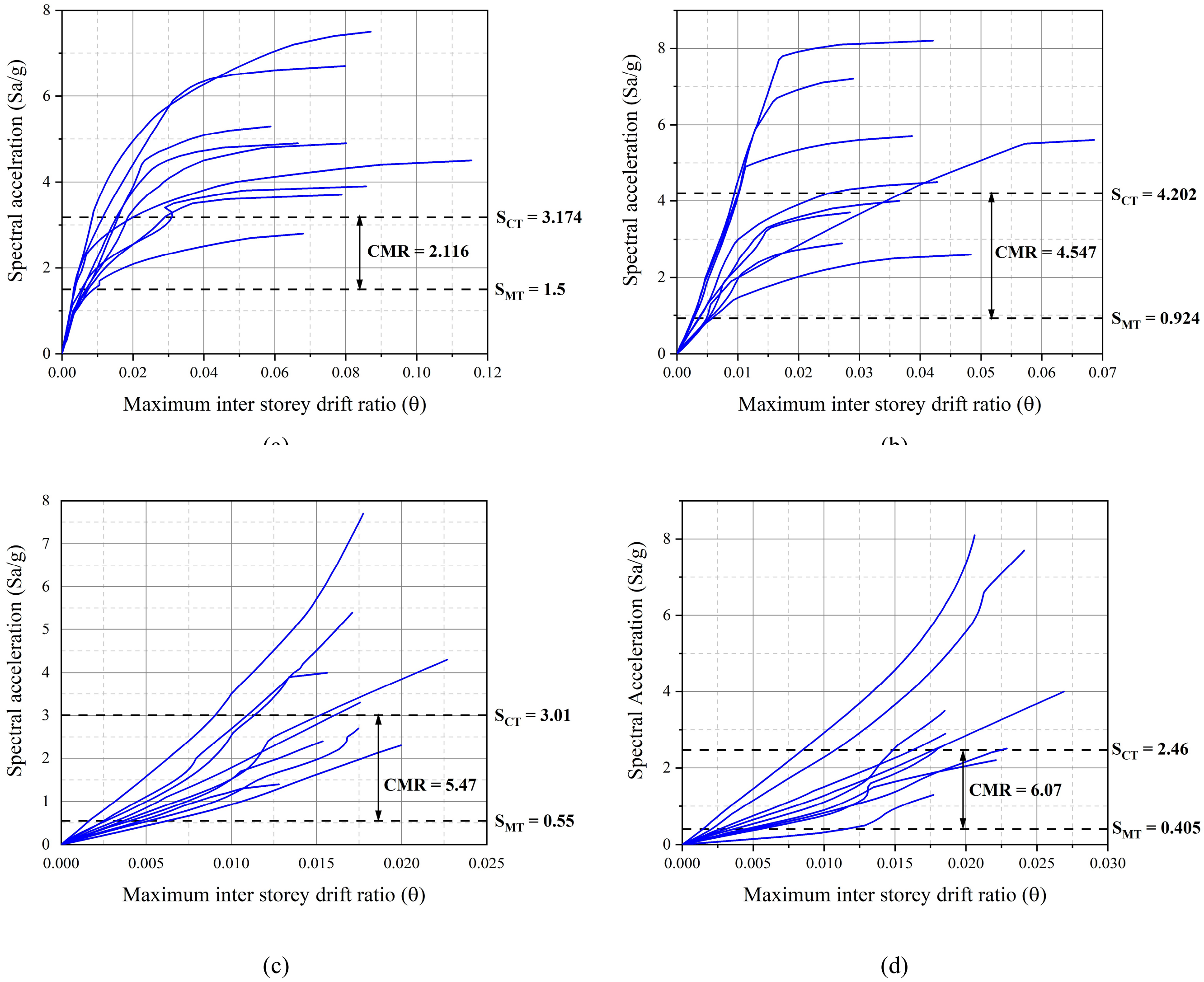


Fig. 7 IDA curves: (a) 4-storey (b) 8-storey (c) 16-storey (d) 24-storey

As per FEMA P695 methodology, the calculated collapse margin ratio is then multiplied by a factor called Spectral Shape Factor (SSF), which consider the shape of the spectrum of rare ground motions, to obtain adjusted collapse margin ratio (ACMR) for each archetype. Thus, the adjusted collapse margin ratio for each index archetype is calculated as:

$$ACMR = CMR \times SSF \quad (8)$$

The SSF is a function of fundamental period of structure (T), period based ductility (μ_T) and the seismic design category. The SSF value is determined from the Table 7-1 given in FEMA P695. Different sources of uncertainties affect the collapse capacity of archetypes. Systems with higher levels of collapse uncertainty will require larger collapse margins in order to limit the collapse probability to an acceptable level at the MCE intensity. It is required to consider the effects of all significant sources of uncertainty in collapse assessment process. The sources of uncertainty in the collapse assessment are record-to-record (RTR) uncertainty, design requirement (DR) uncertainty, test data (TD) uncertainty, and modeling (MDL) uncertainty.

RTR uncertainty (β_{RTR}) is due to the variability in the response of archetypes under the diverse ground motion records. The value of RTR uncertainty is related to the period based ductility corresponding to Eq. (9). This value is assumed to a constant value of 0.4 for structures with period based ductility (μ_T) greater than 3.0.

$$0.2 \leq \beta_{RTR} = 0.1 + 0.1\,\mu_T \leq 0.4 \quad (9)$$

DR uncertainty (β_{DR}) is defined as the completeness and robustness of the design requirements that is used to design archetypes. TD uncertainty (β_{TD}) is related to the completeness and robustness of the test data, which is used to prepare analytical models of archetypes. MDL uncertainty (β_{MDL}) is related to the numerical modeling of archetypes and how well structural collapse behaviour is captured.

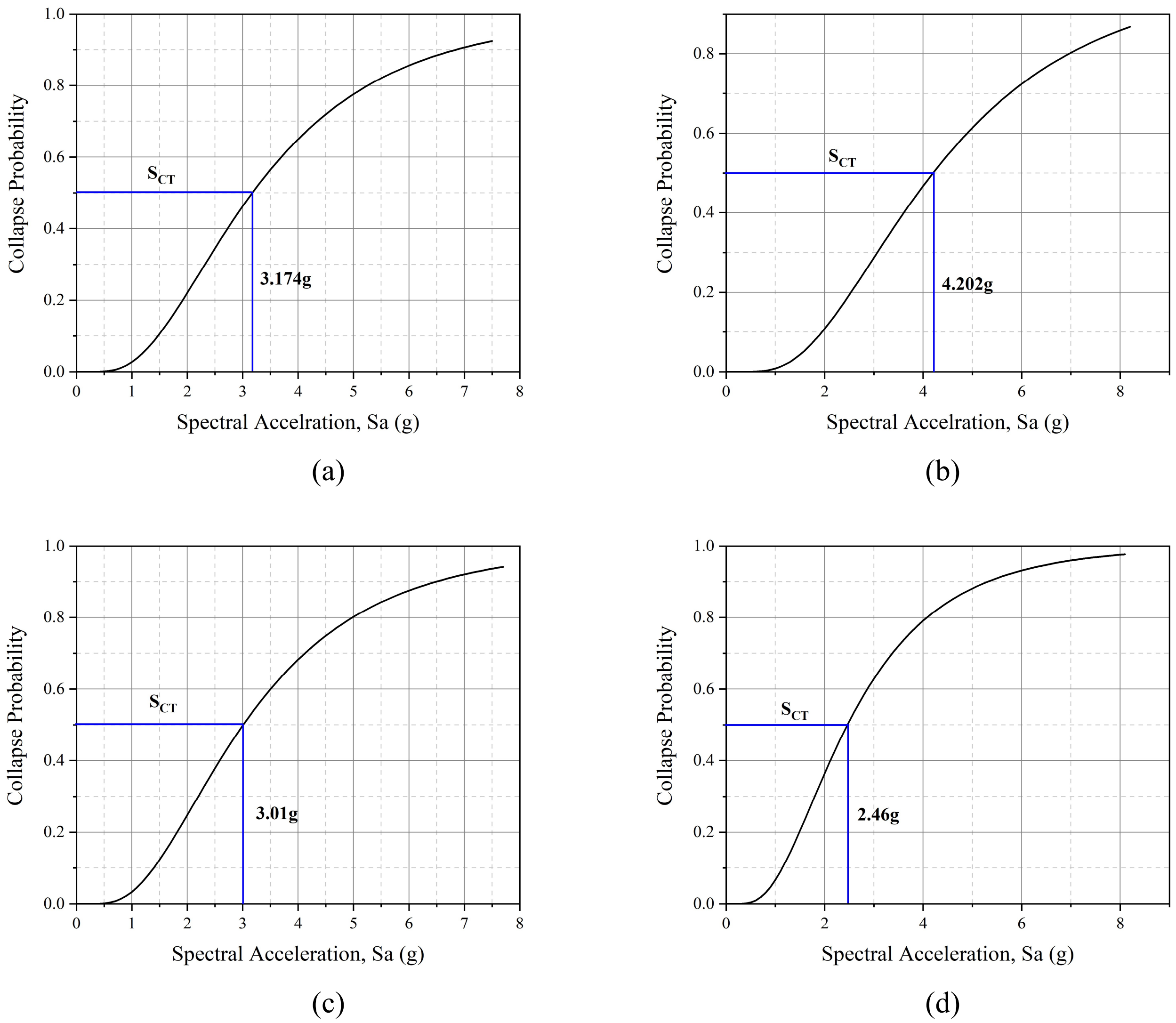


Fig. 8 Collapse Fragility curves: (a) 4-storey (b) 8-storey (c) 16-storey (d) 24-storey

For quantification of three above mentioned uncertainties, FEMA P695 introduces four quality ratings: A-superior, B-good, C-fair and D-poor; which are quantified as 0.1, 0.2, 0.35 and 0.5, respectively. In this study, DR uncertainty is considered to be good ($\beta_{RTR} = 0.2$), TD uncertainty is considered to be good ($\beta_{TD} = 0.2$) and MDL uncertainty is considered to be fair ($\beta_{MDL} = 0.35$). The total collapse uncertainty (β_{TOT}) is obtained by combining all four uncertainties and is calculated using Eq. (10). Selecting the aforementioned uncertainties, the total collapse uncertainty has been computed as $\beta_{TOT} = 0.6$

$$\beta_{TOT} = \sqrt{\beta_{RTR}^{2} + \beta_{DR}^{2} + \beta_{TD}^{2} + \beta_{MDL}^{2}} \qquad (10)$$

Using the results of incremental dynamic analysis, collapse fragility curve are derived through a cumulative distribution function which relates the ground motion intensity to the probability of collapse. The two parameters that define lognormal collapse fragility are the median collapse intensity (S_{CT}) and total collapse uncertainty (β_{TOT}). Fig. 8 shows the collapse fragility curve for each archetype.

6. Evaluation of Response Modification Coefficient

FEMA P695 defines two basic collapse prevention objectives for acceptable performance:

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

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

- the average value of adjusted collapse margin ratio for each performance group exceeds ACMR10%

Table 5: Collapse performance evaluation of CFST diagrid system

	S_{CT} (g)	S_{MT} (g)	CMR	SSF	ACMR	Avg. ACMR	$ACMR_{10\%}$	$ACMR_{20\%}$
4-storey	3.174	1.5	2.116	1.155	2.445	6.06	2.16	1.66
8-storey	4.202	0.924	4.547	1.206	5.485			
16-storey	3.01	0.55	5.47	1.423	7.787			
24-storey	2.46	0.405	6.07	1.41	8.557			

$$\overline{\mathrm{ACMR}} \geq \mathrm{ACMR}_{10\%} \quad (11)$$

- individual values of adjusted collapse margin ratio for each index archetype within a performance group exceeds ACMR20%

$$\mathrm{ACMR_i} \geq \mathrm{ACMR}_{20\%} \quad (12)$$

where the $ACMR_{10\%}$ and $ACMR_{20\%}$ are the acceptable values of adjusted collapse margin ratio and are presented in Table 7-3 of FEMA P695.

Final collapse assessment and comparison with acceptance criteria are summarized in Table 5. According to this table, the computed values of $ACMR_i$ and $\overline{\mathrm{ACMR}}$ are greater than the acceptable values of ACMR given in FEMA P695. So it can be inferred that the presumed Response Reduction factor (R) equal to 5 is acceptable.

7. Conclusions

Based on static nonlinear analysis and incremental dynamic analysis of CFST Diagrid structural system for evaluation of seismic response factor, following conclusions are derived:

- Overstrength factor for 4, 8, 16 and 24 storey CFST diagrid building are 9.36, 11.23, 7.26 and 6.06 respectively. This shows that as the height of CFST diagrid building increases, overstrength factor decreases (except for 8-storey).
- Period based ductility for 4, 8, 16 and 24 storey CFST diagrid building are 2.304, 2.41, 4.43 and 4.16 respectively. This shows that as the height of CFST diagrid building increases, period based ductility increases upto 16 storey building.
- From the fragility analysis, the value of median collapse intensity (S_{CT}) for 4, 8, 16 and 24 storey CFST diagrid building are obtained as 3.174, 4.202, 3.01 and 2.46 respectively.
- Adjusted collapse margin ratio (ACMR) for 4, 8, 16 and 24 storey CFST diagrid building are 2.445, 5.485, 7.787 and 8.557 respectively which satisfies the acceptable values of ACMR given in FEMA P695 ($ACMR_{10\%}$ = 2.16 and $ACMR_{20\%}$ = 1.66).
- Incremental dynamic analysis results indicate that the value of response reduction factor (R) equal to 5 satisfies the acceptance criteria and hence it can be used for designing the CFST diagrid structural system with the range of height and diagonal angles considered in this study

Disclosures

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