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Seismic Performance Evaluation of Post Fire affected Unconfined and Confined Reinforced Concrete – SMRF Structures

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ABSTRACT

Fires are relatively likely events in urban locations in India. The seismic zone map of Indian sub-continent emphasizes that more than 60% of the land is prone to moderate to severe earthquake. This paper presents the seismic performance of code-conforming (IS456-2000 and IS1893-2016) post fire affected special moment resisting frame (SMRF) structure with and without the effect of confinement. Simulation of stress vs strain behavior of unconfined and confined concrete is achieved with the Mander's stress strain model and the same has been modified by incorporating the thermal properties of materials as per BS EN 1992-1-2:2004 for elevated temperature conditions. The seismic performance of structural components in terms of target performance levels for various elevated temperatures were studied with nonlinear static analysis using SAP2000. The results reveal that the base shear strength of the unconfined and confined structure drastically reduced to 68% and 62% respectively at 800^{o}C w.r.t ambient condition. The seismic performance at various high temperatures were assessed by converting the capacity curves to capacity spectra and superimposed with the code conforming demand spectra.

1. INTRODUCTION

A potential but infrequently studied hazard is the sequential occurrence of fires and earthquakes. Fire mishaps are a very typical occurrence among the many that occur during the structure's lifetime for a variety of reasons. The consequences of a fire disaster on a reinforced concrete structure can be catastrophic. Based on the severity and duration of a fire, the structure is vulnerable to a variety of minor and major damages. The examination of post occupancy of fire-affected structures necessitates more investigation. India being a land with several regions of potential seismic zones, the

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assessments of these fire affected structures situated in these vulnerable zones receive immediate concern. Seismic analysis of these post fire damaged structures has become a key tool in the determination of their seismic performance. The structural behavior beyond the yield limit and up to its ultimate capacity could be investigated with the non-linear static analysis procedure. Ramaraju *et.al*. (2012), studied the seismic performance evaluation of existing RC buildings as per past codes of practice. A seismic performance evaluation of a 2D framed structure is included in the study. The current paper validates the seismic performance evaluation of the given 2D structure and extends the research to 3D structure under design ground motions as per IS 1893: 2016 and incorporates the effects of temperature varying from ambient to 800°C with an interval of 100°C. The design of SMRF structures and the non-linear analysis has been carried out as per the guidelines of IS:456-2000, IS:1893-2016, IS:13920-2016, ATC-40, FEMA356, FEMA 440. The effect of temperature on the mechanical and thermal properties of concrete and reinforcing steel have been incorporated into the study with the aid of BS EN 1992-1-1,2:2004. The effect of confinement in enhancing the core compressive strength of concrete has been modelled with reference to studies conducted by Mander et.al., (1988). The current paper emphasizes on the comparative study of the evolution of stress vs strain models over the years and adopting a suitable one for the modelling in SAP2000.

The current study incorporates a code conforming seismic study of G+6 storied reinforced concrete structure located in the city of Vadodara. The structure is analyzed both as an unconfined structure and as a confined one to signify the importance of enhancement of the structure's behavior when the effect of confinement is considered. The role of longitudinal and confining reinforcement in defining the core compressive strengths has been verified with several existing stress vs. strain models. The effect of temperature however can be crucial as the mechanical and thermal properties of the materials that contribute to the elemental behavior as a whole can be dependent directly on the temperature effects. The post fire affected nature of the buildings is being modelled in SAP2000 V23 by isothermal heating throughout the structure which is achieved by defining the degraded material properties at various elevated temperatures. The material behaviors at every 100°C rise in temperature up to 800°C were studied by programming the corresponding in order to have a better understanding. Siliceous type of aggregates has been considered in the concrete mixes that do not contribute to major loss of masses at elevated temperatures unlike calcareous aggregates. The seismic performance of structural components in terms of target performance levels for various elevated temperatures were studied with nonlinear static analysis. Default hinges have been used to study the damage mechanisms simulated in the frame elements. A 100% dead load combination and a 50% live load combination has been adopted to define the mass source for the structure. Base shear vs. displacement graphs for the various elevated temperature cases with and without the action of confinement effect have been plotted discretely. These capacity curves are then transformed into capacity spectra as per the guidelines prescribed in ATC-40. Similarly, the demand imposed upon the structure by the earthquake is also plotted as a demand curve which is then converted to a demand spectrum with the same source of reference ATC-40. The seismic performance of the structure is then evaluated as per the three prominent performance levels viz., immediate occupancy, life safety and collapse prevention. This is achieved by superimposing the capacity spectra over the acceleration response demand spectra

(ADRS) of the structure for the given design ground motion. The point of intersection describes the performance point of the structure.

2. DESCRIPTION OF THE STRUCTURE

A typical G+6 storied commercial structure of base plan area 22.5x22.5m located in the seismic zone III (Z=0.16) is considered for the study. The floor design is divided into three bays in each direction, with a center-to-center distance of 7.5 meters in both directions. The type of soil located in the region is Soil type II corresponding to IS 1893-2016. The zone and soil type as corresponding to UBC 1997 guidelines are zone 2A and stiff soil profile respectively. The plan and elevation of the building are as shown in Figure1. The building is analyzed and designed as per the load cases and combinations as mentioned in Indian Standard codes, IS:456-2000, IS:1893-2016. The section properties with reinforcement details are as pictured in Figure 2. In the present study Normal Strength Concrete of grade M25 were used for sub-structure and M30 for the super-structure. Reinforcing steel of yield strength 415N/mm^2 are used as longitudinal and transverse reinforcements.

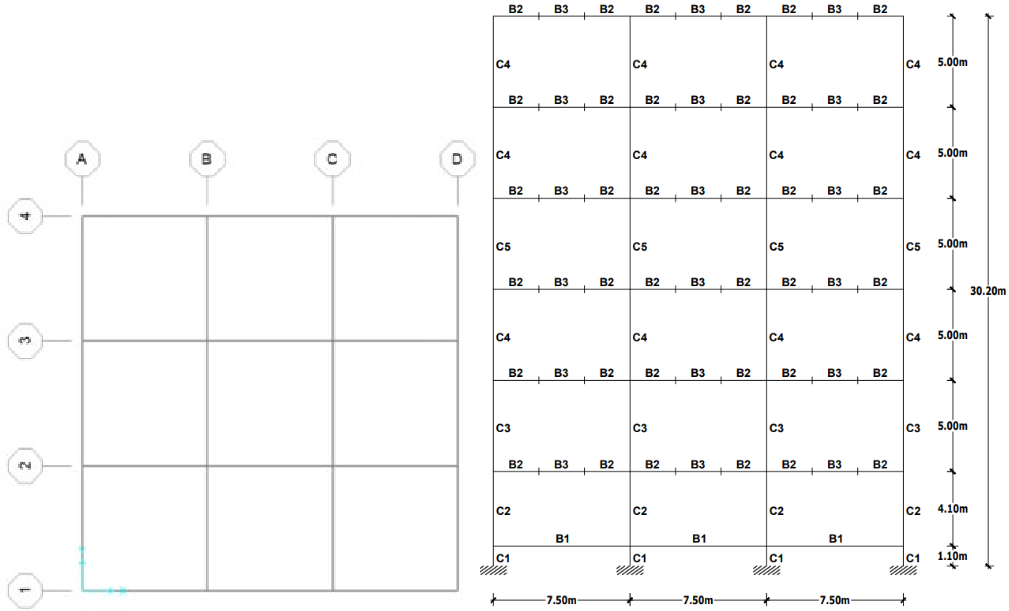

Figure 1. Base plan and Elevation of the building

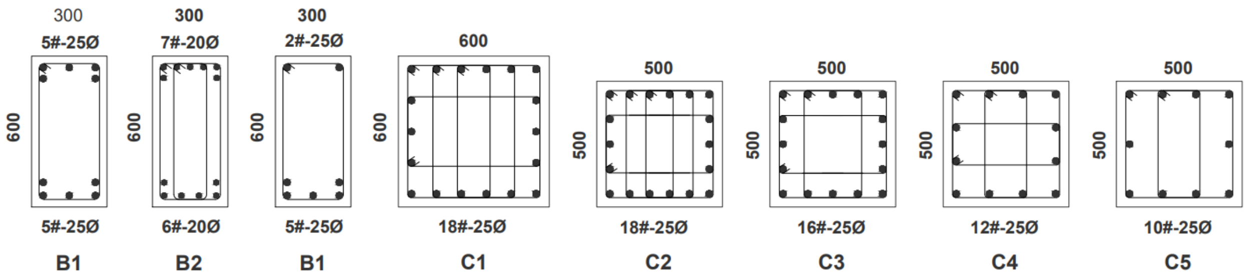

Figure 2. Section properties of the frame elements

(Note - All dimensions are in millimeters and all transverse reinforcements provided are of 8mm φ @100mm c/c spacing)
The structure under investigation comprises of typical beam-column RC frames with tie beams and no shear walls. Slabs of 100mm thickness have been assigned. The ductile detailing of the frame elements has been carried out according to IS:13920-2016 in order to model realistic member behavior in pushover analysis. It is to be also noted that the building under analytical investigation is free of any vertical or torsional plan irregularities (viz., soft storey, floating or stub columns, any setbacks, re-entrant corners, geometrical shape irregularities). A master joint viz., a semi rigid diaphragm is created at every storey to connect all the constrained joints that are rigid in their own planes. These diaphragms which are horizontal elements play a major role in connecting all the vertical lateral load resisting elements rigidly thereby preventing their out of plane deformations. Thus, the transfer of lateral loads is achieved with these elements.

Table 1 - Distribution of lateral forces and determination of base shear

Storey level	Seismic weight, W_i	Storey height, h_i	$W_i h_i^2$	Design lateral force, V_i
	kN	m		kN
Ground floor	2271.40	1.1	2748.39	3.93
First floor	6130.17	4.9	147185.50	210.57
Second floor	6373.17	5	159329.38	227.95
Third floor	6373.17	5	159329.38	227.95
Fourth floor	6373.17	5	159329.38	227.95
Fifth floor	6373.17	5	159329.38	227.95
Terrace floor	5782.01	5	144550.38	206.81
	$\Sigma W_i = 39676.29$		931801.77	$1333.12 = V_{base}$

3. VALIDATION PROBLEM

The methodology adopted for numerical modelling in SAP2000 has been validated with results reported in reference of Ramaraju *et.al*, (2012). The structure validated is a three bay, six storey 2D framed structure. The load patterns and calculations were also simulated according to the Indian Standard codes. The load vs deformation characteristics have been validated accordingly. The corresponding demand and capacity spectra for the validation are shown in the Figure 3.

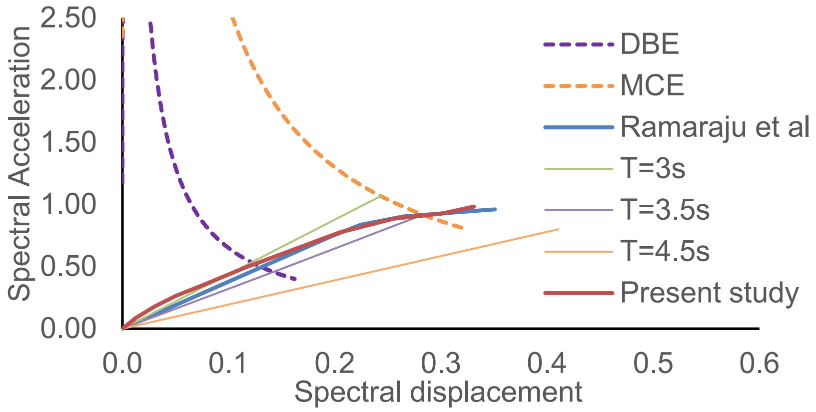

Figure 3 – Performance point determination for validation problem

4. STRESS vs. STRAIN BEHAVIOR OF CONFINED AND UNCONFINED CONCRETE

The effect of confinement due to transverse reinforcement on the core compressive strengths of the elements has not been given significant importance in the structural designs. The following study provides an insight on the enhancement of confined strengths in structural members when the transverse reinforcement is considered. In this paper an effort has been made to compare the stress vs. strain behavior proposed by Hognestad (1951), Kent and Park (1971), Mander *et al.*, (1988), Chung *et a*l., (2002) which is reflected in Table 2. Based on the comparative study, the factors effecting confined compressive strengths such as volumetric ratios of confining steel, stirrup effectiveness coefficient, materials grades and amount and orientation of reinforcement provided given, among the others. The study was further extended to elevated temperatures which involved the definition of material degraded parameters for various levels of fire exposures at elevated temperatures. User defined stress-strain models were fed as inputs to the model.

Table 2 - Summary of Stress vs. Strain Models

Model	Proposed Equations
Hognestad (1951)	$f_{cc} = f_c'(2x-x^2)$ where $x = 2\varepsilon_c/\varepsilon_{c0}$ and $\varepsilon c0 = 0.002$
Kent and Park (1971)	$f_{cc} = f_c'(2x-x^2)$, when $\varepsilon_c < \varepsilon_{c0}$ (Ascending part of graph) where $x = 2\varepsilon_c/\varepsilon_{c0}$ For descending part of the graph, $f_{cc} = f_c'(1-Z(\varepsilon_c-\varepsilon_{c0}))$ Unconfined concrete, $Z = 0.5/(\varepsilon_{50u}- \varepsilon_{c0})$ Confined concrete, $Z = 0.5/ (\varepsilon_{50u} + \varepsilon_{50h} - \varepsilon_{c0})$
Mander et al. (1988),	$f_c = x\ r\ f_{cc}/(r-1+x^r)$ where $x = \varepsilon_c/\varepsilon_{cc}$ and $r = E_c / (E_c - E_{sec})$ $f'_{cc} = f'_c\,(-1.254 + 2.254\sqrt{1+\frac{7.94fi}{f\prime c}} - 2\frac{fi}{f\prime c})$

Chung et al. (2002),	$f_c = x\ r\ f_{cc}/(r-1+x^r)$
	where $x = \varepsilon_c/\varepsilon_{cc}$ and $r = E_c/\ (E_c - E_{sec})$
	$f_{cc} = 1+\Delta K$
	where $\Delta K = 326.4\rho_s^{1.17}.\ f_{hcc}^{0.267}.\lambda^{3.168}.\ f_c^{-0.65} + 0.104$
	For descending part of the graph, $fc = -D\varepsilon_c + f_{cc} + D\varepsilon_{cc}$
	where $D = 0.15f_{cc}/(\varepsilon_{0.85}-\varepsilon_{cc})$
	$\varepsilon_{cc} = 0.0015\rho_s^{0.56}.f_{hcc}^{0.457}.\lambda^{0.503}.f_c^{0.258} + 0.00269$
	$\varepsilon_{0.85} = 0.212\text{x}10^{-9}\rho_s^{0.5}.\ f_{hcc}^{3.514}.\lambda^{-3.06}.f_c^{-0.36} + 0.00270$

The Hognestad (1951) and Kent – Park's (1971) model summarizes that confining the concrete with rectangular or spiral hoops barely attributed to any improvement in the confined compressive strength of concrete. as a result, for a given grade of concrete, the unconfined and confined strength remained the same as per the theory suggested by the two theories. Unlike Hognestad, Kent and Park, Mander served a different conceptual visualization of the role of confinements. The presence of rectangular or spiral hoops to confine the strength of concrete have proven indeed to enhance the core compressive strengths of concrete. The spacing of stirrups and the orientation of the longitudinal reinforcements have played a key role in defining the confined core area of the given elemental cross section. Chung et al., also conducted a detailed study for the confined concrete. The confinement effect has been defined by Mander in terms of a effectively confined concrete area whereas Chung defined it in terms of effectively confined distance from the face of the rectangular area as shown in Figure 4. Chung reported that with the increase of axial loads in any given column, the core concrete is divided into the confined and unconfined areas. The provision of effective confinement of the core concrete leads to an increase in the confined area and thereby a large increase in strength and ductility of the column. He further assumes that the separation between the confined concrete and the unconfined concrete is in the form of a series of arcs spanning between the longitudinal bars which has a similarity with that of Mander's model.

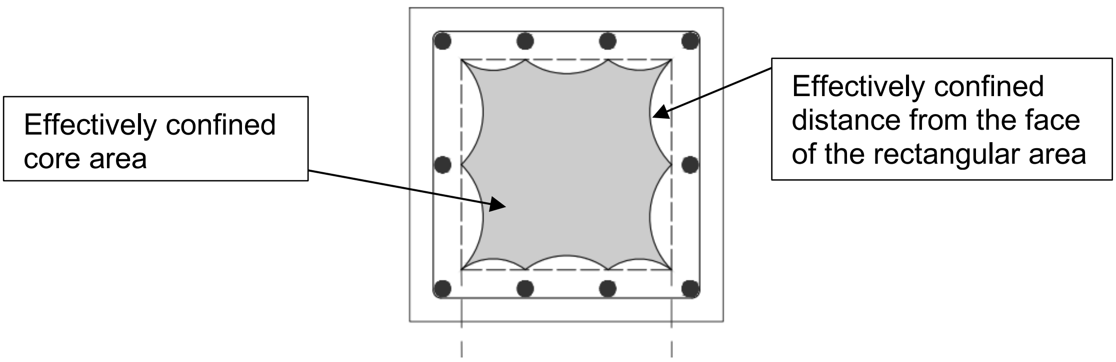

Figure 4 – Effectively confined area and distance as per Mander et al., and Chung et al., respectively.

The comparison in the stress strain behavior of confined and unconfined concrete as per theories proposed by Kent – Park, Mander, Chung for a given C1 column are as given in Figure 5.

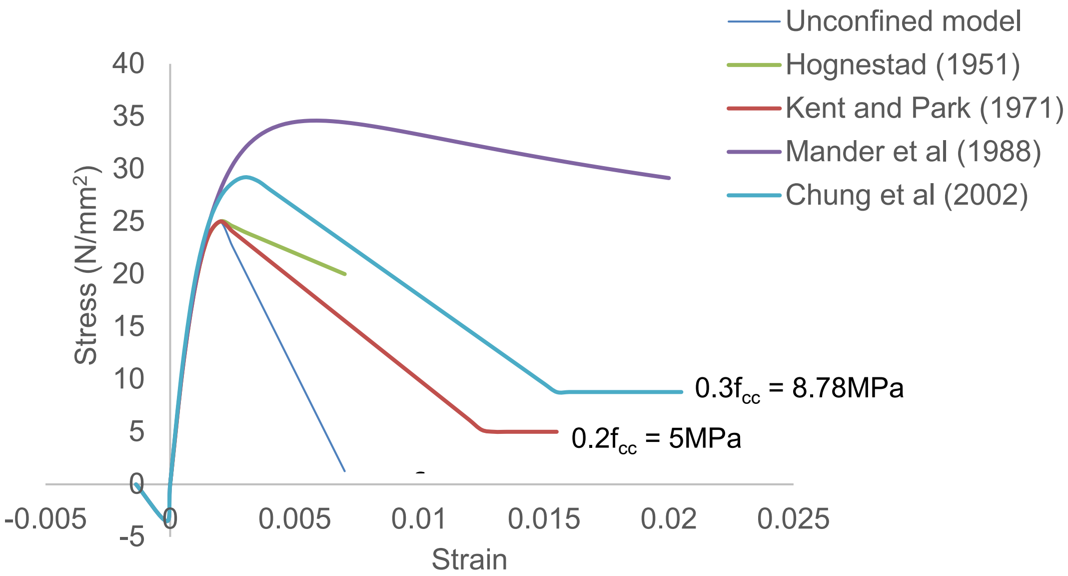

Figure 5-Comparison of stress vs. strain behavior as per Kent-Park, Mander and Chung

The following study uses the Mander's stress strain behavior in the structural modelling. Figure 6 depicts the stress strain behavior of 5 different column sections who have varied reinforcing patterns given and M25 grade of concrete. The spacing between the stirrups was 100mm c/c.

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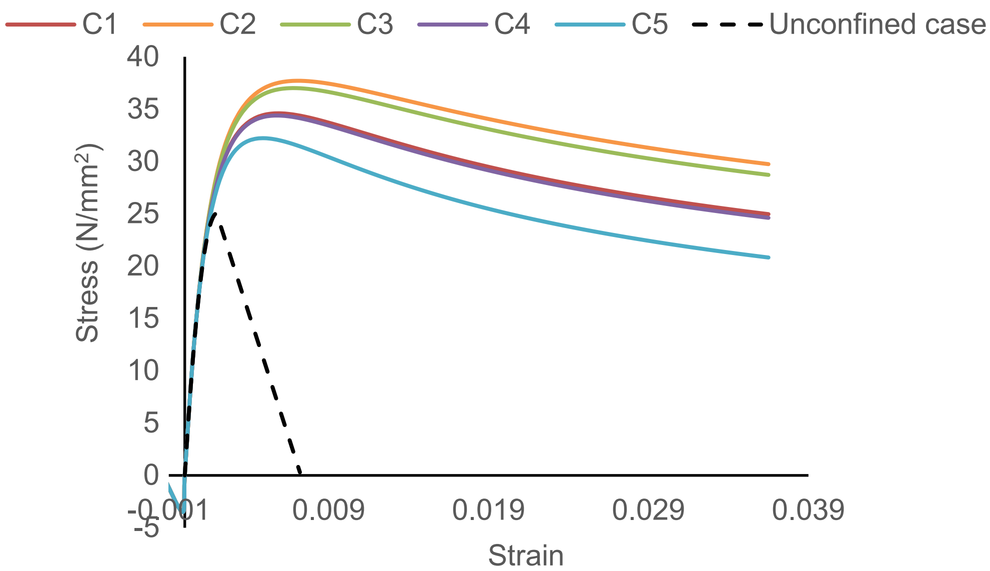

Figure 6 – Stress vs. strain variation for all given columns

5. EFFECTS OF TEMPERATURE ON MECHANICAL PROPERTIES OF CONCRETE AND REINFORCING STEEL

Materials have shown divergent properties when exposed to different atmospheric conditions. The behavior of the materials widely depends on multiple factors like the type of environmental exposure, duration of exposure, the vulnerability capacity of the material to the given atmospheric condition, which may cause deterioration of the property of the material which in turn affects the serviceability of the structure. Various environmental hazards include earthquakes, tsunamis, fire, cyclones, floods, landslides, etc., and unnatural accidents like blast accidents, mining activities, etc., This study mainly deals with post fire affected seismic performance of a reinforced concrete structure for which the material characteristics at elevated temperatures were investigated.

Various researchers have reported the effect of temperatures on the degenerated material properties such as compressive strength, tensile strength, thermal elongation, modulus of elasticity, thermal conductivity, density, etc., The present study adopts the mathematical model as described in BS EN 1992-1-1,2:2004 to study the deteriorated concrete and reinforcing steel properties on exposure to elevated temperatures. The post fire effect has been simulated by considering uniform (isothermal) heating throughout the structure. The code specifies concrete properties for elevated temperatures in terms of two discrete type of aggregates namely siliceous and calcareous aggregates. The current study describes the structure being located at India and hence the siliceous type of aggregates has been incorporated according to the nature of aggregates being used in the corresponding construction industry. It is to be also noted that the mass loss for these aggregates at higher temperatures are insignificant. The study expresses the variation of material properties for every 100^{o}C rise in temperature up to a temperature of about 800^{o}C as provided in Table 3 and 4.

Table 3 - Mechanical and thermal properties of concrete at elevated temperatures.

	As per BS EN 1992-1-2:2004				
Temperature	Compressive strength	Elasticity Modulus	Density	Thermal elongation	Thermal conductivity
^{o}C	N/mm^2	N/mm^2	N/mm^3	(w.r.t length at 20^{o}C)	Wm/K
Ambient	25	25000	2.50E-05	1.00E-04	1.32
100	25	25000	2.50E-05	7.00E-04	1.23
200	23.75	24367	2.45E-05	1.80E-03	1.11
300	21.25	23048.9	2.41E-05	3.10E-03	1.00
400	18.75	21650.6	2.37E-05	4.90E-03	0.91
500	15	19364	2.35E-05	7.20E-03	0.82

600	11.25	16770.5	2.33E-05	1.02E-02	0.75
700	7.5	13693.1	2.30E-05	1.40E-02	0.69
800	3.75	9682.45	2.28E-05	1.40E-02	0.64

Table 4 - Mechanical and thermal properties of reinforcing steel at elevated temperatures.

Temperature	As per BS EN 1992-1-2:2004		
	Yield strength	Elasticity Modulus	Thermal elongation (w.r.t length at 20°C)
°C	N/mm^2	N/mm^2	
Ambient	415	200000	1.00E-04
100	415	180000	1.00E-04
200	415	160000	2.30E-03
300	415	140000	3.70E-03
400	415	120000	5.20E-03
500	323.70	62000	6.80E-03
600	195.05	26000	8.40E-03
700	95.45	18000	1.00E-02
800	45.65	14000	1.10E-02

The following data are incorporated while modelling the beam and column elements of the structure. The stress vs strain behavior of the members has shown a decreasing trend w.r.t. increasing temperature. The serviceability of the structure also thereby tends to reduce. The effects of elevated temperatures have been simulated by modifying Mander's stress vs strain model with respective thermal parameters as per BS EN 1992 -1-2:2004. The following graph depicts the behavior of one column, C1 for various elevated temperatures.

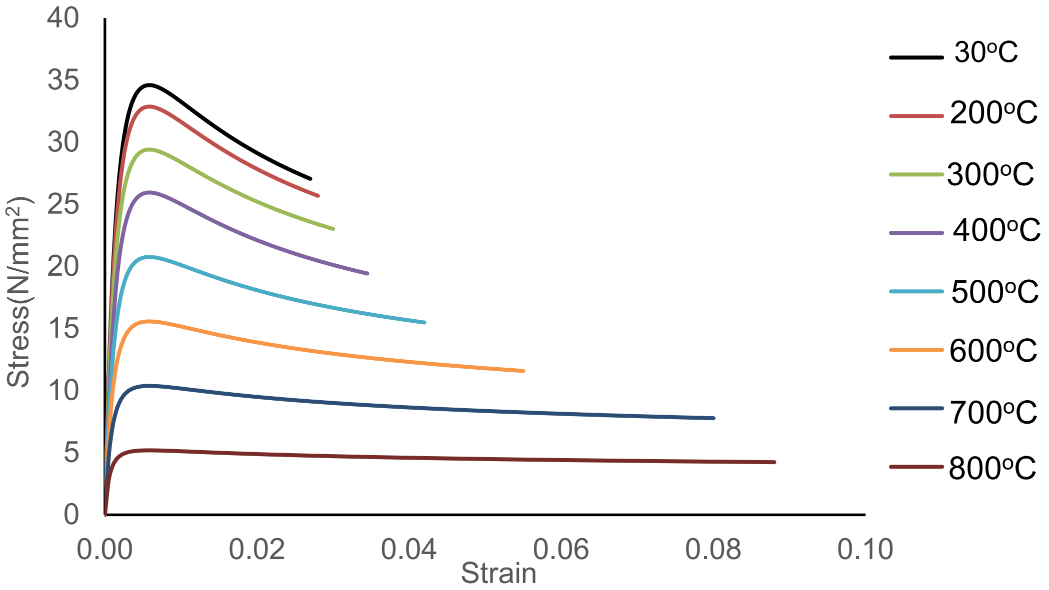

Figure 7. Stress vs. strain variation for elevated temperatures.

6. MODELLING INELASTIC BEHAVIOR OF STRUCTURE

Analytical models developed in the study comprise the complete three-dimensional behavior of the building which generally include the mass distribution, strength, stiffness and deformability though a wide range of global and local displacements. Although an elastic analysis facilitates identification of the initial yielding nature of the structure, it lacks the ability to trace the damage mechanisms that develop in the structure progressively beyond yielding. Actual behavior of the conventional buildings based on the severity of the experienced seismic intensity involves the study of the inelastic response of the structural system which describes the reliability of the structure to absorb and dissipate the earthquake energy. Thus, the seismic analysis and evaluation of the structure in its inelastic range gains prime priority when it's behavior beyond the elastic range majorly determines the actual response of the structure for a given seismic activity.

The non-linear modelling and analysis of the structure is carried out by considering the simultaneous effects of gravity and lateral loads. The lateral loads are applied in such a way that they represent predominant distributions of lateral inertial loads during a critical earthquake response. These lateral loads during a design ground motion tend to be lumped at floor levels and thereby the application of lateral loads in increments to the structure up to the code specified target displacements facilitates the tracking of inelastic mechanisms developed. These inelastic behaviors at the elemental level i.e., the frame elements are simulated with hinges to trace the extent of damage that is caused within them. The following study uses the default hinges available in the software for the assignment of the same as provided in Figure 6. These hinges have been assigned at a relatively measured distance from either ends of the support. Details of these hinges in the various performance levels of the structure have been depicted in Table 4. Care has also been taken to analyze the simultaneous effect of gravity loads acting through these lateral displacements, commonly referred to as a P-Δ effects.

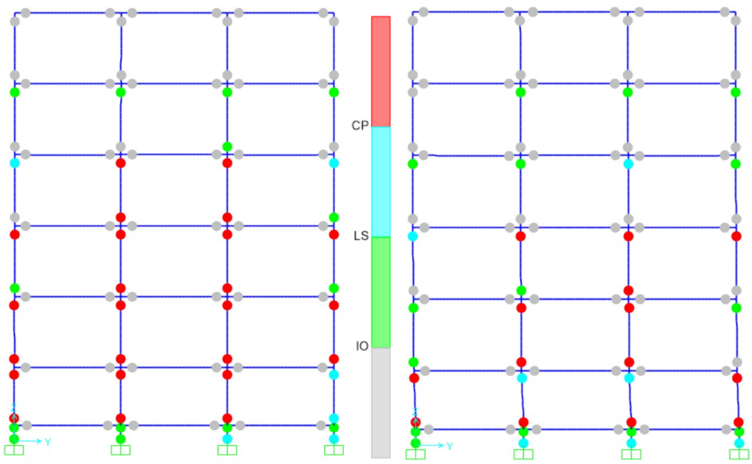

Figure 8– Hinges developed in the frame elements for Pushover along X direction

7. PERFORMANCE BASED EVALUATION OF THE BUILDING

At the beginning of any evaluation process of a project, it becomes ultimately necessary to decide the performance-based objectives for the current structure. It is based on these performance objectives, a wide range of performance levels can be targeted in the structural design, ranging from the onset of damage to collapse. The building performance on its whole is a combination of the structural performance of the structural components and the non-structural components during a seismic hazard. The two basic approaches to design a building to achieve its performance objectives include displacement-based design which measures displacement quantities to judge the performance acceptability and force-controlled design which measures the design base shear strength to evaluate the structural performance levels. Accordingly, the major performance levels include Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). The seismic hazards are broadly classified to be of three types based on the probability of the occurrence namely Serviceability earthquake (SE), Design Basis earthquake (DBE) and Maximum Considered earthquake (MCE). It is the type of seismic hazard that governs the criteria to which structural design need to be carried out.

7.1 DISPLACEMENT COEFFICIENT METHOD

The pushover is carried out by defining the target displacement of the structure which is 4% of the roof drift. The corresponding roof displacements of the structure against the incremental lateral loads given to the structure are plotted which depict the capacity of the structure. This plot of Base shear vs roof displacement identifies the capacity curve of the building. Figure 8 and 9 depicts the capacity curve of the building for various elevated temperatures for unconfined and confined structures respectively. Table 5 depicts the various salient features of the capacity curves at elevated temperatures for ultimate base shear capacity.

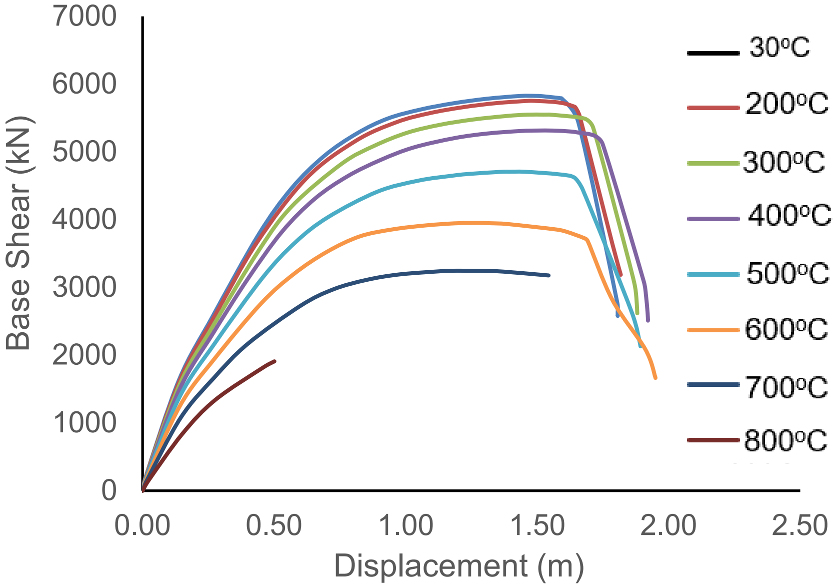

Figure 9 – Capacity curves for the unconfined structure at elevated temperatures.

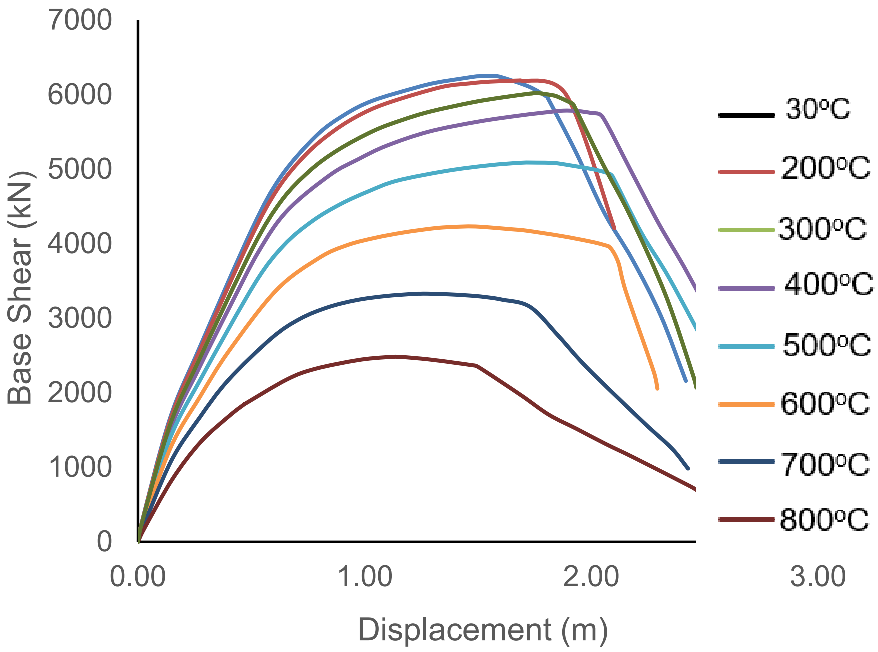

Figure 10 – Capacity curves for the confined structure at elevated temperatures.

Table 5 – Salient features of Capacity Curve

Cases	T	Disp.	$V_{ultimate}$	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	> E	Total
	ºC	m	kN				unitless					
Unconfined	30	1.461	5831.09	222	302	94	64	104	136	0	0	564

Confined		1.597	6293.07	206	292	86	84	102	143	0	8	564
Unconfined	200	1.436	5661.51	225	302	95	59	108	137	0	0	564
Confined		1.681	6190.96	196	282	88	69	125	146	0	12	564
Unconfined	300	1.132	5205.82	239	311	99	77	77	112	0	0	564
Confined		1.851	6120.84	199	281	85	62	136	156	0	9	564
Unconfined	400	0.967	4574.33	267	323	119	56	66	93	0	0	564
Confined		2.159	6046.72	203	285	85	58	136	149	0	18	564
Unconfined	500	0.768	3789.71	294	328	132	72	32	45	0	0	564
Confined		1.822	5086.64	221	296	83	57	128	147	0	9	564
Unconfined	600	0.711	3224.66	301	322	127	97	18	24	0	0	564
Confined		1.503	4235.35	252	318	77	52	117	143	1	0	564
Unconfined	700	0.713	2628.17	305	305	120	134	5	16	0	0	564
Confined		1.349	3383.89	267	354	66	34	110	125	0	0	564
Unconfined	800	0.559	1871.62	359	305	139	109	11	5	0	0	564
Confined		1.469	2372.72	290	348	72	38	106	132	0	0	564

Note: T: Temperature exposed, Disp: Roof Displacement, $V_{ultimate}$- Ultimate Base Shear, A-B: No of hinges in Operational range, B-IO: No of hinges in Operational and Immediate Occupancy range, IO-LS: No of hinges in Immediate Occupancy and Life Safety range, LS-CP: No of hinges in Life Safety and Collapse Prevention range, CP-C: No of hinges in Collapse Prevention and ultimate capacity range, C-D: No of hinges in ultimate capacity and residual strength range, D-E: No of hinges in residual strength and failure range, >E: No of hinges that have undergone complete failure.

7.2 CAPACITY SPECTRUM METHOD

The two key elements that play a crucial role in determining the structure's behavior is the demand imposed by the earthquake and the capacity inherited within it to resist the seismic demand. The essence of push over analysis of the structure is to simply make a comparison between the demand that the earthquake poses on the structure to the capacity of the structure to satisfy the demand. The non-linear static procedures utilize the displacements to compare the seismic demand to the capacity of the structure. Thus, the ductile behavior (confined) of the structure is investigated and the corresponding ductile behaviors for the same structure at various elevated temperatures is also reported in this paper.

Based on the type of seismic hazard, a demand curve which signifies the spectral acceleration over various time periods for a default 5% damping scenario is plotted. The demand and the capacity curves obtained are then transformed to the corresponding spectra viz., a graph of spectral acceleration vs spectral displacements (ADRS format) by the use of modal mass coefficient, modal participation factors all of which correspond to the first mode of the structure. Table 6 briefs about the determination of these factors.

Table 6 – Determination of modal factors

Floor levels	Floor weights, W_i	Amplitudes at different levels for mode 1, φ	$W_i*\varphi$	$W_i*\varphi^2$

	kN	M		
Terrace	5782.01	0.0234	135.29	3.16
5	6373.17	0.0218	139.24	3.04
4	6373.17	0.0188	119.94	2.25
3	6373.17	0.0144	91.74	1.32
2	6373.17	0.0089	56.96	0.50
1	6130.17	0.0032	19.81	0.06
	ΣW_i = 37404.89		$\Sigma W_i*\varphi$ = 563.01	$\Sigma W_i*\varphi^2$ = 10.35

Modal mass coefficient, α = 0.81 and Modal participation factor for the first mode, PF_1 = 60.61(derived as per ATC-40 and IS:1893-2016)
Roof level amplitude corresponding to first mode, $\varphi_{roof,1}$ = 0.023m
These values have been used to convert the capacity and demand curves to their corresponding capacity spectrum and Acceleration demand response spectrum (ADRS) respectively. The two spectra on super-imposition give rise to a point of intersection which is commonly referred to as the performance point. The location of this performance point helps in understanding the structural behavior and also in proposing the desired retrofitting strategies. The performance points obtained for the current study problem w.r.t. Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE) for an Importance factor of 1.5 are shown in Figure 10. The following study follows the guidelines as mentioned in ATC 40 to derive the two spectra, thereby obtaining the performance point. Equations 4 and 5 depict the conversions to obtain the necessary co-ordinates as per ATC guidelines.

$S_d = \delta_i / (PF \times \varphi_1)$ ………………………………………………………………………… (4)

$S_a = V_i / (W \times \alpha)$ …………………………………………………………………………….. (5)

From the graph it has been observed that the performance of the structure at high temperatures is presented.

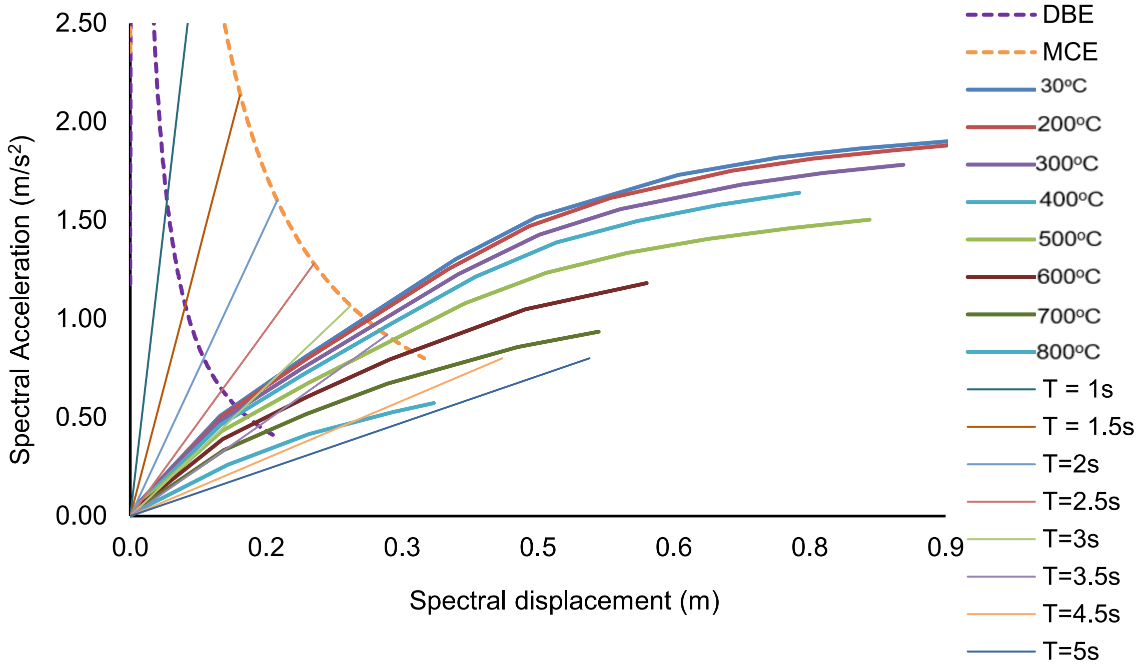

Figure 11 – Capacity spectra for DBE and MCE for elevated temperatures

8. CONCLUSIONS

In the present study seismic performance evaluation of post fire affected unconfined and confined Special Moment Resisting Frame structures as per the provisions of IS:456-2000, IS:1983-2016, IS:13920-2016, ATC-40, FEMA356, FEMA 440 and BS EN 19921-2:2004 is carried out. The post fire effect has been simulated by considering isothermal heating throughout the structure. The thermo-mechanical effect has been simulated by incorporating thermal parameters as per BS EN 19921-2:2004 in well-established Mander's stress vs strain model for reinforced concrete. The thermal effects on material characteristics were also modelled for various elevated temperatures such as 200ºC, 300ºC, 400ºC, 500ºC, 600ºC, 700ºC and 800ºC. The damage mechanism in beam and column members were carried out by modeling these members with default hinges (M3 for beams and P-M2-M3 hinges for columns) at their ends. The analysis was carried out by considering the default hinges as defined in SAP2000 V23. It was ensured that all the structural elements underwent uniform exposure to the elevated temperatures. The performance of the structure was assessed in terms of various performance levels as described in FEMA 356, FEMA 440 and ATC 40. From the results of the non-linear static analysis (Push over analysis), various capacity curves were plotted for elevated temperatures. These plots of base shear vs. roof displacement serve as a key tool in estimating the amount of damage the structure has undergone and the possible retrofitting strategies that could be implemented. In addition to the capacity curves, the demand spectra viz., a graph of spectral acceleration vs, spectral displacement (ADRS) was also plotted to simulate the demand imposed on the structure by a seismic action. Accordingly, demand spectra for Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE) were plotted. To identify the performance point of the structure, the capacity curves were transformed into corresponding capacity spectra with the help of mass coefficient and modal participation factors as per ATC 40 and then super imposed on the demand spectra. The points of intersection define the performance point of the structure. It was observed that the global structural behavior for elevated temperatures followed a gradual reduction in capacity with increase in temperature levels. Accordingly, it was observed that the base shear strength for 800°C reduced to about 62% and 68% of the original value for confined and unconfined cases respectively. Also, from the capacity and demand spectra curves, it was observed that no performance point was achieved at 800°C.The structure has undergone a drastic reduction in capacity (strength and stiffness) thereby making the structure irrepairable or unfit for any repair through various retrofitting strategies.

NOTATIONS

f'_c - the unconfined compressive strength of concrete
f_{cc} - confined compressive strength of concrete
f_{hcc} – tie stress
f_i - Lateral Confining pressure
E_c – Tangent Modulus of Concrete
E_{sec} – Secant Modulus of Concrete
hi - height at i^{th} floor level
PF_1 - Modal participation factor for the first mode
V - Base shear
W_i – Seismic weight corresponding to structure at i^{th} floor level
W – total seismic weight of the structure
α - Modal mass coefficient
δ - roof displacements
ε_c - longitudinal compressive concrete strain corresponding to f_c
ε_{cc} - longitudinal compressive concrete strain corresponding to f_{cc}
ε_{co} - longitudinal compressive concrete strain corresponding to f'_c
ε_{50u} - strain corresponding to 50% of the maximum unconfined compressive strength
ε_{50h} - strain corresponding to 50% of the maximum confined compressive strength
$\varepsilon_{0.85}$ - longitudinal compressive concrete strain corresponding to $0.85f_{cc}$
ρ_s – Volumetric ratio of Confining steel
λ – confinement distance ratio
φ_1 – roof level amplitude at first mode

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