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EFFECT OF ELEVATED TEMPARATURE ON THE SEISMIC PERFORMANCE OF AN RC BUILDING

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Abstract: A number of catastrophic incidents across the world have indicated that extreme fire and earthquake could trigger an event to initiate the progressive collapse of reinforced concrete (RC) structures. For the majority of the fires in reinforced concrete (RC) structures, structural damage was observed due to the deterioration of the materials, such as concrete and steel reinforcement. However, traditional design guidelines cannot provide any reliable estimate for the performance of the structures after a fire event, and therefore, the level of safety against the risk of limit states (i.e., collapse) is simply unknown. Furthermore, probable future earthquakes may cause unanticipated strength and stiffness degradation to these types of fire-damaged structures, which may be critical in terms of the lateral dynamic instability in earthquake prone areas. This instability may be manifested in reinforced concrete structures as a side-sway collapse, caused by the loss of a lateral-force-resisting capacity or substantial increases in peak displacement demands. Hence, research on behaviour of RC structures under elevated temperature and earthquake is important. However, limited studies have been undertaken in the seismic analysis of RC buildings exposed to elevated temperature. This thesis presents performance of an elevated temperature effected (i.e 300°C and 500°C) a G+6 storey RC building during a code specified earthquake. The considered RC building frame is designed for load cases as per IS456 1964 and IS 456 2000 and IS:1893-2016. The building is analysed using SAP 2000 to predict the sensitivity of elevated temperature effected RC structure during an earthquake event. The scenario of elevated temperature with reduced material properties is studied and the load redistribution and plastic hinge formation is also investigated. From the study it is concluded that there is a necessity of enhancing the lateral strength of existing buildings designed as per IS 456: 1964 to resist earthquake. It is also concluded that the building frame designed as per IS 456: 2000 exposed to 500°C is highly vulnerable during any future earthquake.

Index terms- Reinforced concrete, Elevated temperature, Earthquake, Plastic hinge.

1. INTRODUCTION

According to world fire statistics, about 1 million deaths were caused by fires between 1993 and 2014, and approximately 40% of all fires around the world originated from structures. As per the ADSI (Accidental Deaths & Suicides in India) - 2019 report, there were 11,037 fire accidents reported across the country in 2019. In most of these fire incidents, building structures have experienced minor to major damage due to fire exposure, such as degradation of material properties due to elevated temperatures and damage to structural elements due to thermal expansion. After any building fire incident, an inspection is required for the assessment of the structural loss and damage and evaluation of the building residual capacity. As the result of such a survey, the decision is made to either repair or demolish and rebuild the structure. Assessing the extent and gravity of fire damage on reinforced concrete buildings is a crucial task in order to plan the rehabilitation or the demolition of their structures. Unfortunately, the fire resistance capacity of reinforced concrete is very difficult to assess, as concrete itself is a composite material with components characterized by different thermal properties, but also because moisture and porosity have a great influence on mechanical behaviour. In addition, steel reinforcements, unlike concrete, are very sensitive to the effects of high temperatures caused by fire. Therefore, application of a reliable approach for damage evaluation plays an important role in the safety and subsequent cost of the repair or reconstruction of the fireexposed structures. This would include assessment of both the short-term and long-term response of the structures after a fire. Major earthquakes in urban areas often lead to building fires. Such earthquakes frequently damage buildings' water sprinkler systems and diminish or strain firefighting capabilities. In such scenarios, buildings get exposed to fire for a long period of time. Consequently, RC frame temperature may reach as high as the maximum fire temperature, which subsequently cools slowly to ambient temperature.

Generally concrete exhibits good natural fire resistance due to its non-combustible nature and lower thermal conductivity but exposure to high temperature causes thermo-mechanical and physicochemical reactions which leads to reduction in strength, degradation of material properties and deterioration of concrete by spalling. The realistic fire behaviour of building structures depends on several parameters, the most important of which are: the structural configuration and layout, fire intensity, duration, and spread, structural loading and boundary conditions, fire protection distribution, and structural details. Therefore, a better understanding of system response due to fire-induced progressive collapse can provide substantial insight that could potentially lead to advances towards developing performance-based design provision that can result in safe and economical repairing or retrofitting of buildings to improve the performance under future earthquake.
Reinforced concrete (RC) frame structures constructed prior to 1980s generally do not meet the requirements of seismic design code due to the deficient design which only took gravity loads into consideration and ignored the lateral actions such as earthquake and winds. For instance, some old buildings were designed as sub-standard RC frames with strong beams but weak columns, which might result in harmful brittle failures such as joint shearing and column hinging. In addition, building fire is one of the most frequent disasters that will happen in RC buildings. Although the post-fire RC frames can generally be retrofitted using proper strengthening techniques such as enlarging of joint area with newly cast concrete and fibre reinforced cementitious composites, adhesive bonding or near surface mounting of fibre reinforced polymers (FRP), bonding or anchoring of steel plates, etc. they may encounter an earthquake attack in their subsequent post-fire service life thus fail and even collapse. Therefore, for these sub-standard RC frames, not only the researches on their residual bearing capacity is necessary, but also those focusing on their post-fire seismic performance is of great significance. This thesis presents seismic performance of a G+6 storey RC office building pre- exposed to elevated temperature effect, which is likely to trigger during a fire accident. The structure considered is designed for two load cases as per IS 456-1964 and IS 456-2000 and IS 1893-2016

2. LITERATURE REVIEW

The following section provides relevant background material on the experimental studies carried out on the material properties of concrete at elevated temperature.

Qifang Xie. (2019)[6] studied the effects of high temperatures on the surface characteristics, weight loss rate, and residual mechanical properties (strength, initial elastic modulus, peak, and ultimate compressive strains) of carbonated and uncarbonated concrete. 75 prism specimens were prepared and divided into four groups (three carbonated groups and one uncarbonated group). Specimens were tested under different temperatures (20, 300, 400, 500, 600, and 700°C), exposure times (3, 4, and 6 hours), and cooling methods (water and natural cooling). Accelerated carbonation experiments were conducted according to the Standard for Test Methods of Long-term Performance and Durability of Ordinary Concrete. The test results indicated that the weight loss rates of the carbonated concrete specimens are slightly lower than that of the uncarbonated ones and that the cracks are increased with raising of temperatures. Surface colours of carbonated concrete are significantly changed, but they are not sensitive to cooling methods. Surface cracks can be evidently observed on carbonated specimens when temperature reaches 400°C. Residual compressive strength and initial elastic modulus of carbonated concrete after natural cooling are generally larger than those cooled by water. Heating time over 3 h has little effect on the weight loss rate, residual compressive strengths, and initial elastic modulus of carbonated and uncarbonated concrete. Residual compressive strengths and initial elastic modulus of carbonated concrete decrease with the gradual raising of exposure temperatures and overall perform better than the uncarbonated concrete. The peak and ultimate compressive strains of both carbonated and uncarbonated concrete specimens increase after heating, but the values of the latter are greater than that of the former. Also, established a constitutive equation to predict the compressive behaviours of carbonated concrete under high temperatures.

I. Hager (2013)[16] presents a comprehensive review on the cement concrete behaviour at a high temperature. Changes in the mechanical properties of concrete caused by heating have been presented in this paper. Tested "hot" mechanical properties, including stress-strain relationship, compressive strength, and modulus of elasticity, decline with the increase of temperature. The possible explanations of the observed gradual decrease of mechanical properties have been discussed in detail. Moreover, the important role of water was put forward, both in the context of the changes of concrete properties as well as in the occurrence of the spalling phenomenon. It has also been revealed that water also plays an important role in a transient thermal strain development. By drying material, the extent of the phenomenon is significantly reduced, or even eliminated, at up to 400◦C. Above this temperature, the mismatch of thermal deformations between the aggregates, which expand, and cement paste, which undergoes shrinkage, prevails and results in the development of cracks. Significant cracking continues, thus altering the material mechanical properties.

Netinger. (2011)[23] investigated the fire resistance of concrete made with some locally available, potential "fire-resistant" aggregates, such as diabase, steel slag, crushed bricks and crushed tiles. The specimens of measurements 4×4×16 cm^3 were kept in moulds for 24 h and, after demoulding, were kept in water at room temperature of about 20 ± 2^0C until testing. At the age of 28 days, the specimens, with moisture content within the limits of 3–5%, were exposed to high temperatures in a previously heated test furnace. The residual mechanical properties (compressive and flexural strengths) of these concretes after natural cooling were compared with the residual mechanical properties of concrete made with commonly used river and dolomite aggregates. The replacement of natural concrete aggregates with brick and steel industry waste materials was justified, not only in terms of increased fire resistance, but also in terms of more responsible waste disposal.

Venkatesh Kodur. (2011)[24] presents the effect of temperature on the thermal properties of different types of HSC. Specific heat, thermal conductivity, and thermal expansion are measured for three concrete types, namely, HSC, self-consolidating concrete (SCC), and fly ash concrete (FAC), in the temperature range from 20–800°C. The effect of steel, polypropylene, and hybrid fibers on thermal properties of HSC and SCC is also investigated. Results from experiments show that SCC possesses higher thermal conductivity, specific heat, and thermal expansion than HSC and FAC in the 20–800°C temperature range. Data generated from tests is utilized to develop simplified relationships for expressing different thermal properties as a function of temperature. The proposed thermal property relationships can be used as input data for evaluating the response of concrete structures under fire conditions.

Chen. (2009)[25] presented experimental study on the compressive and splitting tensile strengths of concrete cured for different periods and exposed to high temperature. The effects of the duration of curing, maximum temperature and the type of cooling on the strengths of concrete were investigated. Experimental results indicate that after exposure to high temperatures up to 800^0C, early-age concrete that has been cured for a certain period can regain 80% of the compressive strength of the control sample of concrete. The 3-day-cured early-age concrete was observed to recover the most strength. The type of cooling also affects the level of recovery of compressive and splitting tensile strength. For early-age concrete, the relative recovered strengths of specimens cooled by sprayed water are higher than those of specimens cooled in air when exposed to temperatures below 800°C, while the changes for 28-day concrete are the converse. When the maximum temperature exceeds 800^0C, the relative strength values of all specimens cooled by water spray are lower than those of specimens cooled in air.

Tang. (2009) [26] conducted experimental investigation on the residual mechanical and fracture properties of normal and high-strength concrete (HSC) with and without fly ash after exposure to high temperatures ranging from 200 to 600°C. The fracture behaviour of concrete was investigated using the three-point bending notched beam test. The effects of concrete strength, temperature level and the presence of fly ash on the fracture and mechanical properties of concrete were investigated. Some studies on the aggregate–cement paste transition zone have been made by means of a scanning electron microscope (SEM). The mechanical properties of concrete specimens significantly decreased with increase in temperature level, the extent of which depended on the presence of fly ash in the mix. In general, fly ash concrete performed better in residual mechanical and fracture properties especially at relatively high temperature levels than concretes without fly ash. Also, fewer and finer microcracks were identified in fly ash concrete under the SEM study.

and multi-layer shell elements were used, in conjunction with appropriate material constitutive laws and elemental failure criteria under high temperature conditions. Extreme fire scenarios were also considered, based on the actual fire-induced progressive collapse events of the WTC towers and the Windsor Tower. The simulation results indicated that a progressive collapse of a super-tall building was triggered by the flexural failure of the peripheral columns, approximately 7 h after being exposed to fire. The bending deformations of the peripheral columns increased significantly, due to the outward thermal expansion of the upper floors and the inward contraction of the lower floors, a result of the fire-induced damage. The results also revealed that, when multiple stories are subjected to fire, the internal forces in the components are redistributed in the horizontal and vertical directions by way of the truss mechanism, leading to a maximum increase (of approximately 100%) of the axial forces in the columns. The identified the mechanisms of the fire-induced progressive collapse of a typical RC supertall building and provided an effective analysis framework for further research on the fire safety of tall and super-tall RC buildings.

3. RESEARCH METHODOLOGY

- Framing objectives
- Data collection and analysis
- Numerical method
- Software method
- Comparison of result
- Analysis and result

4. NEED OF THE STUDY

In seismic area, it is of very importance to evaluate the post-fire performance of a building to ascertain whether the load capacity and stiffness of the building and its components can satisfy the requirements of structure design codes. In recent years significant developments have been made in analysing the behaviour of steel and concrete framed structures at elevated temperatures. For high-rise buildings, seismic action is usually the decisive load, so it is particularly important that the post-fire seismic performance of the buildings can satisfy the requirements of seismic design. This is important because the majority of structural members are not designed for extreme conditions, combining gravity loads, lateral loads and aftershock loads. Consequently, the buildings which have been moderately damaged by a fire can rapidly be destroyed in a subsequent earthquake. These have demonstrated the significance of evaluating the performance of fire exposed structures under future earthquake. The objectives described in the previous section are fulfilled using various research tasks that encompass the scope of this study. Specifically, numerical models are developed and analysed using the finite element software Sap 2000. Two six-story structures designed on the basis of two different version of IS 456 and IS 1893-2016 are evaluated using non-linear pushover analysis under different elevated temperature. The properties of concrete and reinforcement at elevated temperature defined in European code (EN 1992-1-2:2004) are utilized in the simulations.

5. OBJECTIVES

The specific objectives of this research are summarized as follows:

- To evaluate the seismic performance of RC frame subjected to elevated temperature of 300°C and 500°C.
- To compare the seismic performance of RC frame designed for two load cases as per IS 456-1964 and IS 456-2000.
- To investigate the collapse pattern of fire exposed reinforced concrete members under an earthquake load as per IS 1893-2016.

6. RESULTS AND DISCUSSION

I. Pushover curve details for case 1 without exposure to elevated temperature

Step	Displacement (m)	Base Force (KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0.000091	0	182	0	0	0	0	0	0	0	182
1	0.01993	38.712	180	2	0	0	0	0	0	0	182
2	0.049942	89.456	176	6	0	0	0	0	0	0	182
3	0.080216	127.646	169	13	0	0	0	0	0	0	182
4	0.100216	146.282	169	13	0	0	0	0	0	0	182
5	0.131532	175.065	167	15	0	0	0	0	0	0	182
6	0.151532	192.269	167	15	0	0	0	0	0	0	182
7	0.171785	209.373	163	19	0	0	0	0	0	0	182
8	0.191785	224.35	163	17	2	0	0	0	0	0	182
9	0.211785	239.326	161	17	4	0	0	0	0	0	182
10	0.232478	251.914	154	22	6	0	0	0	0	0	182
11	0.257368	262.877	151	23	8	0	0	0	0	0	182
12	0.277368	271.401	151	21	10	0	0	0	0	0	182
13	0.304914	282.161	145	25	12	0	0	0	0	0	182
14	0.324914	288.627	145	25	10	2	0	0	0	0	182
15	0.348701	296.086	142	28	6	6	0	0	0	0	182
16	0.368701	301.891	139	31	6	6	0	0	0	0	182
17	0.399008	308.803	136	32	8	0	0	6	0	0	182
18	0.415285	310.094	132	35	8	1	0	6	0	0	182
19	0.435285	310.388	132	32	11	1	0	6	0	0	182

20	0.47236	310.858	131	28	12	5	0	6	0	0	182
21	0.49236	311.288	131	25	14	6	0	6	0	0	182
22	0.51236	311.718	131	22	17	6	0	6	0	0	182
23	0.519885	311.88	130	21	19	5	0	7	0	0	182
24	0.531375	311.894	128	21	21	4	0	8	0	0	182
25	0.548299	311.53	128	21	21	2	0	10	0	0	182

II. Pushover curve details for case 1 after exposure to 300°C

Step	Displacement (m)	Base Force (KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0.000141	0	182	0	0	0	0	0	0	0	182
1	0.020141	25.051	182	0	0	0	0	0	0	0	182
2	0.0319	39.779	180	2	0	0	0	0	0	0	182
3	0.057238	67.854	177	5	0	0	0	0	0	0	182
4	0.077238	89.183	176	6	0	0	0	0	0	0	182
5	0.098003	109.006	173	9	0	0	0	0	0	0	182
6	0.125919	129.304	169	13	0	0	0	0	0	0	182
7	0.145919	141.537	169	13	0	0	0	0	0	0	182
8	0.165919	153.771	169	13	0	0	0	0	0	0	182
9	0.204841	177.208	167	14	1	0	0	0	0	0	182
10	0.224841	188.532	167	13	2	0	0	0	0	0	182
11	0.244841	199.856	167	11	4	0	0	0	0	0	182
12	0.281572	219.074	165	9	8	0	0	0	0	0	182
13	0.301572	229.124	164	10	8	0	0	0	0	0	182
14	0.321572	239.127	162	9	11	0	0	0	0	0	182
15	0.356965	253.912	158	12	11	1	0	0	0	0	182
16	0.38517	263.071	156	14	9	3	0	0	0	0	182
17	0.40517	269.281	153	17	6	6	0	0	0	0	182
18	0.425546	275.276	152	17	7	4	0	2	0	0	182
19	0.448533	280.719	149	18	9	2	0	4	0	0	182
20	0.471186	284.151	148	18	9	1	0	6	0	0	182
21	0.5083	288.397	146	19	9	2	0	6	0	0	182

22	0.537681	291.424	144	19	9	4	0	6	0	0	182
23	0.573401	294.357	139	20	11	5	0	7	0	0	182
24	0.593401	295.021	137	22	11	5	0	7	0	0	182
25	0.600141	295.216	137	21	12	5	0	7	0	0	182

III. Pushover curve details for case 1 after exposure to 500°C

Step	Displacement (m)	Base Force (KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0.000207	0	146	6	0	0	0	0	0	30	182
1	0.012207	6.958	146	6	0	0	0	0	0	30	182
2	0.018027	10.332	143	9	0	0	0	0	0	30	182
3	0.03155	15.637	140	12	0	0	0	0	0	30	182
4	0.04355	18.851	140	12	0	0	0	0	0	30	182
5	0.05555	22.064	140	12	0	0	0	0	0	30	182
6	0.06755	25.277	140	12	0	0	0	0	0	30	182
7	0.07955	28.49	140	12	0	0	0	0	0	30	182
8	0.097659	33.147	139	13	0	0	0	0	0	30	182
9	0.109659	36.167	138	14	0	0	0	0	0	30	182
10	0.121659	39.159	138	14	0	0	0	0	0	30	182
11	0.133659	42.152	138	14	0	0	0	0	0	30	182
12	0.145659	45.144	137	15	0	0	0	0	0	30	182
13	0.157659	48.13	137	15	0	0	0	0	0	30	182
14	0.169659	51.117	137	15	0	0	0	0	0	30	182
15	0.181659	54.103	137	11	4	0	0	0	0	30	182
16	0.193659	57.09	137	11	4	0	0	0	0	30	182
17	0.205659	60.076	137	10	5	0	0	0	0	30	182
18	0.217659	63.062	137	9	6	0	0	0	0	30	182
19	0.229659	66.049	137	9	6	0	0	0	0	30	182
20	0.241659	69.035	137	9	6	0	0	0	0	30	182
21	0.257708	72.85	128	16	8	0	0	0	0	30	182
22	0.269708	74.739	128	16	8	0	0	0	0	30	182
23	0.281708	76.628	128	16	8	0	0	0	0	30	182
24	0.293708	78.517	128	16	8	0	0	0	0	30	182
25	0.305708	80.405	126	18	8	0	0	0	0	30	182
26	0.317708	82.111	124	19	8	1	0	0	0	30	182

27	0.329708	83.602	124	19	5	4	0	0	0	30	182
28	0.341708	85.094	124	19	5	4	0	0	0	30	182
29	0.353708	86.586	124	19	5	4	0	0	0	30	182
30	0.365708	88.077	120	23	5	4	0	0	0	30	182
31	0.375874	89.2	120	22	6	3	0	1	0	30	182

IV. Pushover curve details for case 2 without exposure to elevated temperature

Step	Displacement (m)	Base Force (KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0.000081	0	182	0	0	0	0	0	0	0	182
1	0.026748	98.802	182	0	0	0	0	0	0	0	182
2	0.033924	125.389	180	2	0	0	0	0	0	0	182
3	0.063466	220.298	175	7	0	0	0	0	0	0	182
4	0.096374	291.302	171	11	0	0	0	0	0	0	182
5	0.126763	344.078	166	16	0	0	0	0	0	0	182
6	0.165077	389.62	162	19	1	0	0	0	0	0	182
7	0.21102	429.078	159	15	8	0	0	0	0	0	182
8	0.237687	449.173	159	11	12	0	0	0	0	0	182
9	0.264354	469.268	159	9	13	1	0	0	0	0	182
10	0.304795	498.35	156	10	12	4	0	0	0	0	182
11	0.334213	515.41	155	8	13	6	0	0	0	0	182
12	0.364975	529.04	155	6	9	9	0	3	0	0	182
13	0.394714	540.185	155	6	8	8	0	5	0	0	182
14	0.411851	544.752	154	6	8	6	0	8	0	0	182
15	0.415905	545.21	154	6	7	6	0	9	0	0	182
16	0.426138	545.524	153	7	7	5	0	10	0	0	182
17	0.435045	545.443	152	8	6	6	0	10	0	0	182
18	0.443307	545.047	152	8	6	5	0	11	0	0	182
19	0.450264	544.512	150	10	5	5	0	12	0	0	182
20	0.481663	537.334	150	9	5	5	0	13	0	0	182
21	0.528274	521.518	148	10	6	2	0	16	0	0	182
22	0.566102	505.075	148	10	5	1	0	18	0	0	182

23	0.618296	478.048	146	8	8	2	0	18	0	0	182
24	0.644963	460.419	146	7	9	2	0	18	0	0	182
25	0.671629	442.756	146	6	10	2	0	18	0	0	182
26	0.698296	425.048	145	5	12	2	0	18	0	0	182

V. Pushover curve details for case 2 after exposure to 300°C

Step	Displacement (m)	Base Force (KN)	Ato B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0.000126	0	182	0	0	0	0	0	0	0	182
1	0.034393	81.496	182	0	0	0	0	0	0	0	182
2	0.050805	120.529	180	2	0	0	0	0	0	0	182
3	0.094829	211.993	176	6	0	0	0	0	0	0	182
4	0.148375	288.354	171	11	0	0	0	0	0	0	182
5	0.184325	329.951	168	13	1	0	0	0	0	0	182
6	0.223744	366.849	163	15	4	0	0	0	0	0	182
7	0.261918	394.912	161	13	8	0	0	0	0	0	182
8	0.320947	430.335	159	11	9	3	0	0	0	0	182
9	0.368616	455.343	159	8	9	4	0	2	0	0	182
10	0.410115	472.517	159	5	7	8	0	3	0	0	182
11	0.450564	486.004	157	6	6	6	0	7	0	0	182
12	0.467584	489.899	156	5	7	4	0	10	0	0	182
13	0.485894	491.803	155	6	7	2	0	12	0	0	182
14	0.520161	492.168	155	6	5	4	0	12	0	0	182
15	0.531069	492.285	155	6	5	3	0	13	0	0	182
16	0.546841	491.736	155	6	4	3	0	14	0	0	182
17	0.581107	489.931	155	5	4	3	0	15	0	0	182
18	0.603594	487.731	155	5	4	2	0	16	0	0	182
19	0.656689	479.316	153	6	4	2	0	17	0	0	182
20	0.701768	470.127	151	8	2	3	0	18	0	0	182
21	0.769525	450.148	149	9	3	3	0	18	0	0	182

22	0.820455	434.402	149	9	2	2	0	20	0	0	182
23	0.872545	417.19	148	7	5	2	0	20	0	0	182
24	0.880934	414.339	147	7	6	2	0	20	0	0	182

VI. Pushover curve details for case 2 after exposure to 500°C

Step	Displacement (m)	Base Force (KN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0.000205	0	162	0	20	0	0	0	0	0	182
1	0.012205	13.54	162	0	20	0	0	0	0	0	182
2	0.024205	27.079	162	0	20	0	0	0	0	0	182
3	0.036205	39.213	160	2	20	0	0	0	0	0	182
4	0.048205	50.327	159	3	20	0	0	0	0	0	182
5	0.060205	60.713	158	4	20	0	0	0	0	0	182
6	0.072205	69.329	155	7	20	0	0	0	0	0	182
7	0.084205	76.555	155	7	20	0	0	0	0	0	182
8	0.096205	83.782	155	7	20	0	0	0	0	0	182
9	0.108205	90.736	154	8	19	1	0	0	0	0	182
10	0.120205	97.431	154	8	17	3	0	0	0	0	182
11	0.132205	104.125	154	8	16	4	0	0	0	0	182
12	0.144205	110.819	154	8	16	4	0	0	0	0	182
13	0.156205	117.221	152	9	15	6	0	0	0	0	182
14	0.168205	123.165	152	9	15	6	0	0	0	0	182
15	0.180205	128.814	151	9	16	5	0	1	0	0	182
16	0.192205	133.659	149	10	16	6	0	1	0	0	182
17	0.195205	134.733	149	9	17	6	0	1	0	0	182
18	0.210205	139.158	148	10	17	4	0	3	0	0	182
19	0.222205	142.433	147	10	18	4	0	3	0	0	182
20	0.234205	145.093	146	10	18	5	0	3	0	0	182
21	0.252205	148.195	146	10	17	6	0	3	0	0	182
22	0.255205	148.705	146	10	17	6	0	3	0	0	182
23	0.255299	148.704	146	10	17	6	0	3	0	0	182
24	0.255486	148.74	146	10	17	6	0	3	0	0	182

25	0.25551	148.728	146	10	17	5	0	4	0	0	182

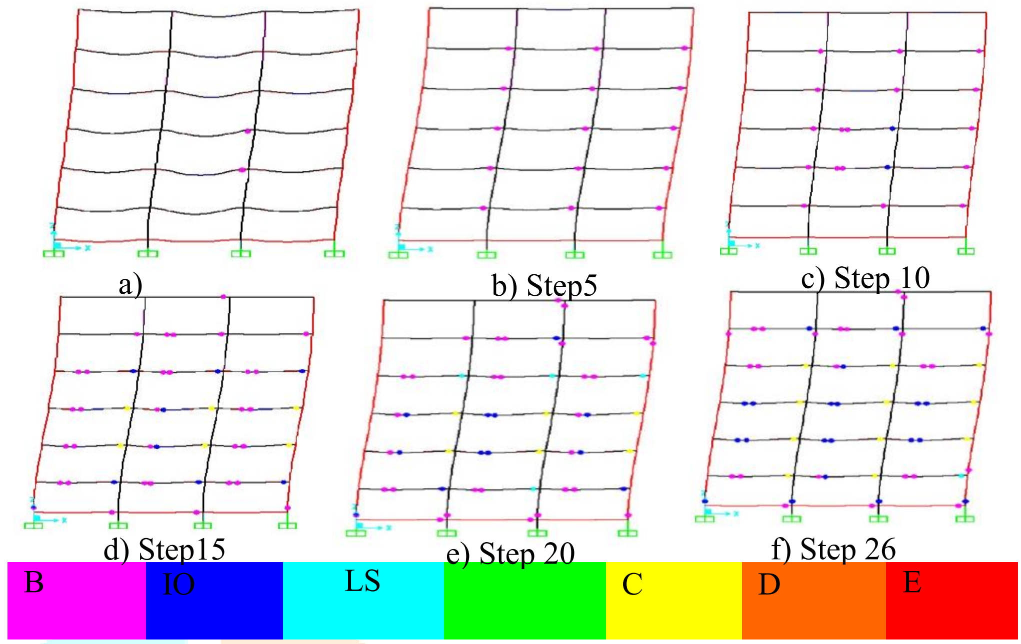

Fig. Plastic hinge formation in case 1 RC frame without exposure to elevated temperature

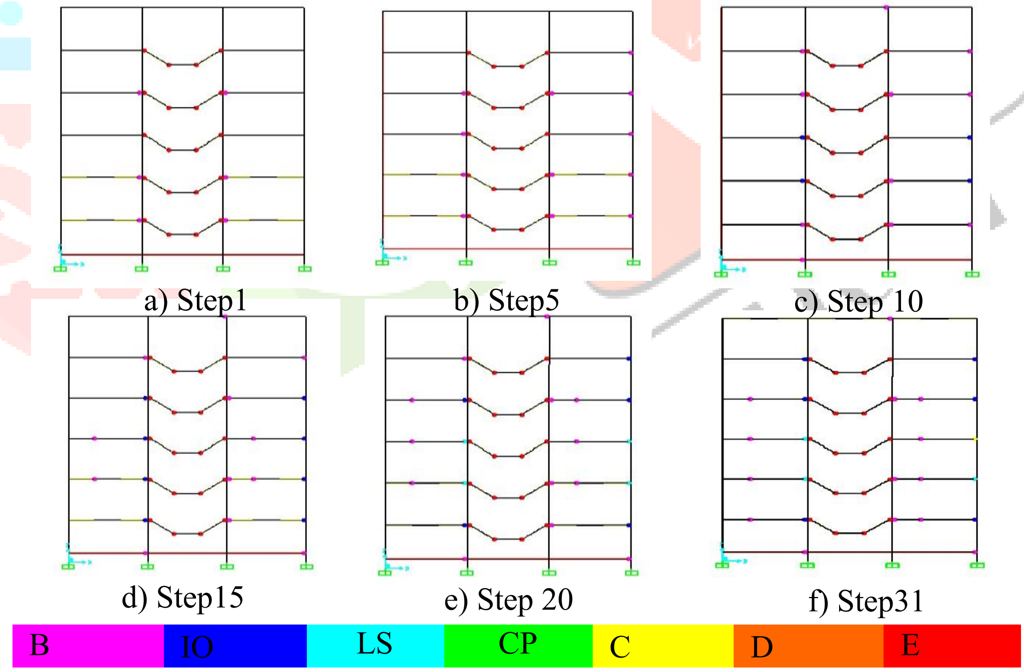

Fig. Plastic hinge formation in case 1 RC frame after exposure to 500°C

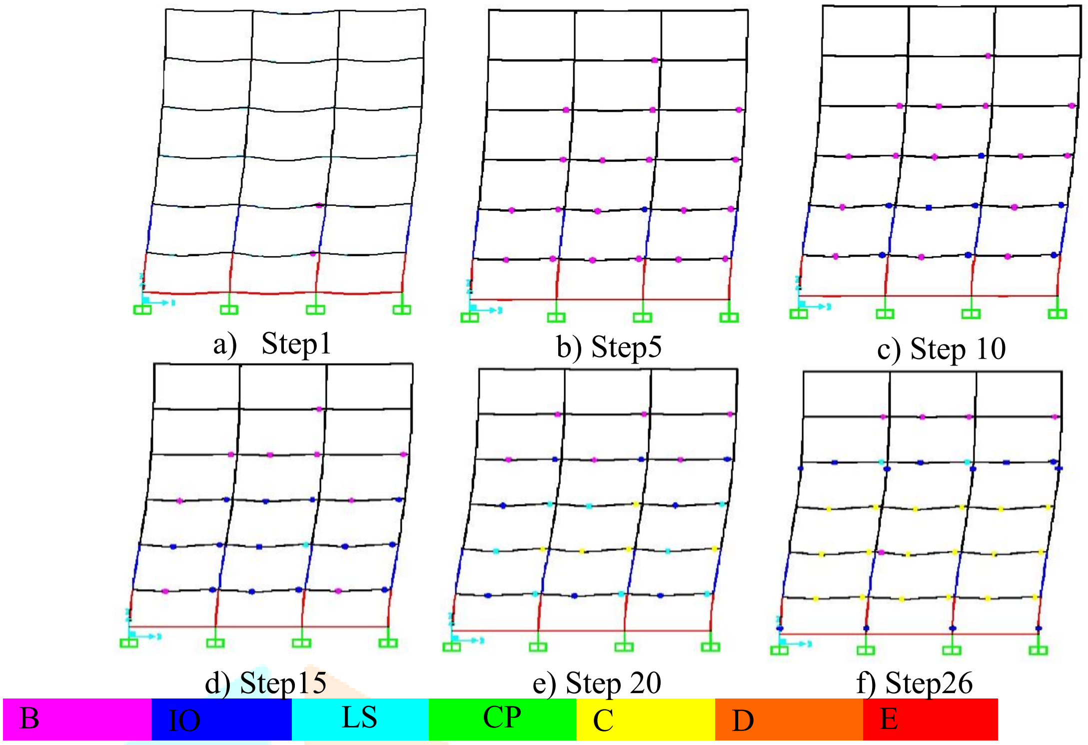

Fig. Plastic hinge formation in case 2 RC frame without exposure to elevated temperature

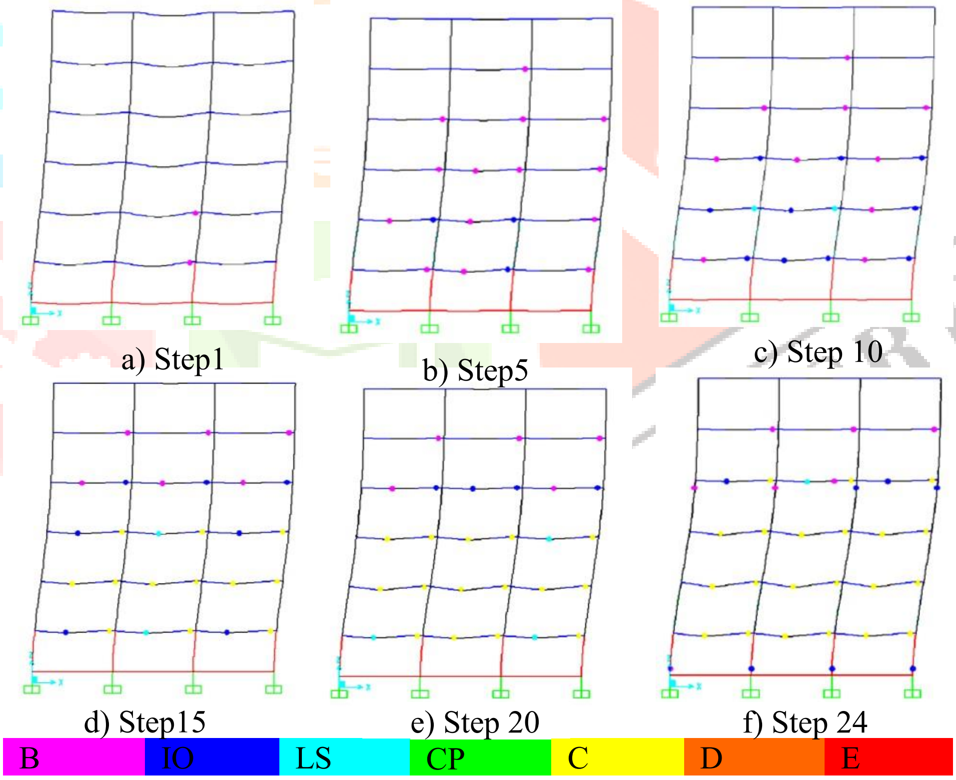

Fig.Plastic hinge formation in case 2 RC frame after exposure to 300°C

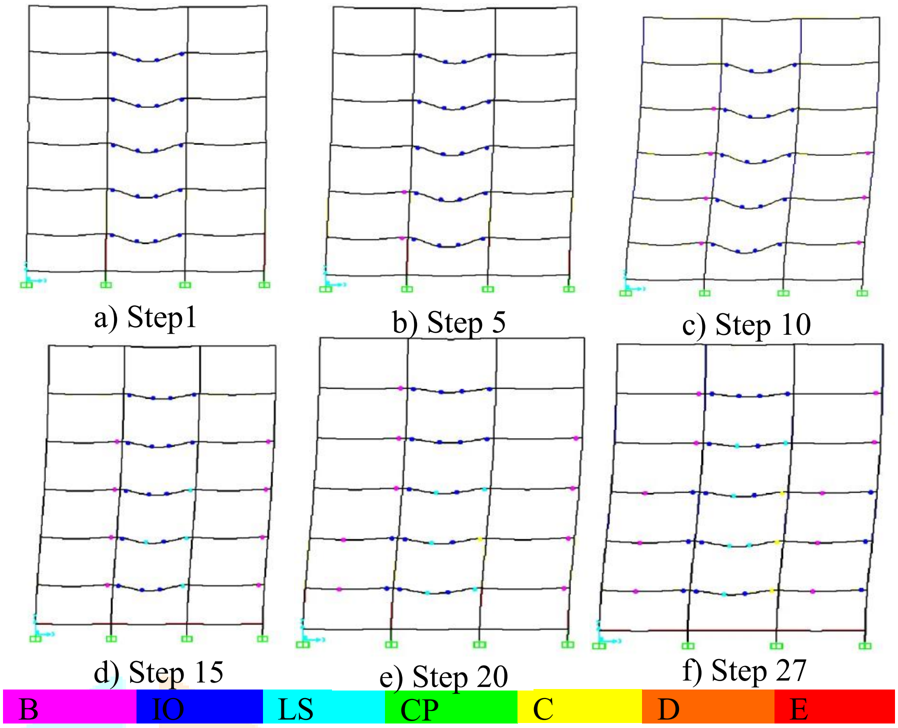

Fig. Plastic hinge formation in case 2 RC frame after exposure to 500°C

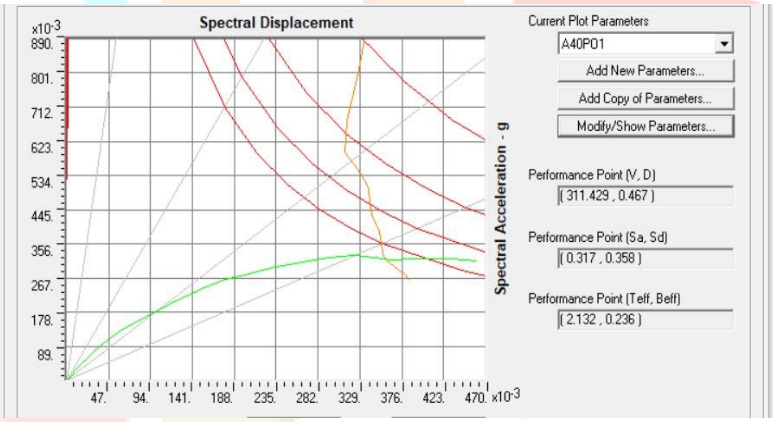

Fig. Capacity spectra for case 1 without exposure to elevated temperature

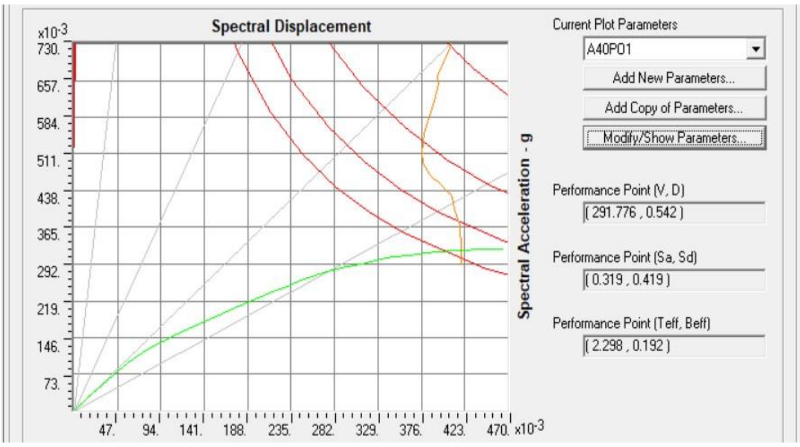

Fig. Capacity spectra for case 1 after exposure to 300°C

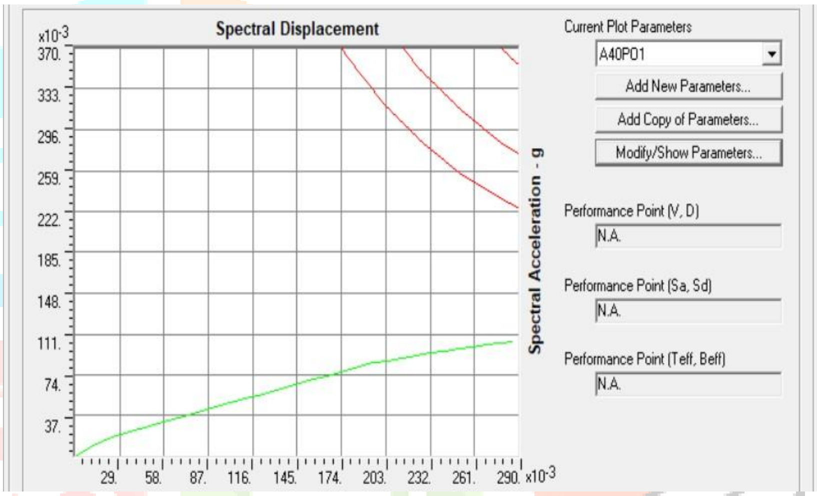

Fig. Capacity spectra for case 1 after exposure to 500°C

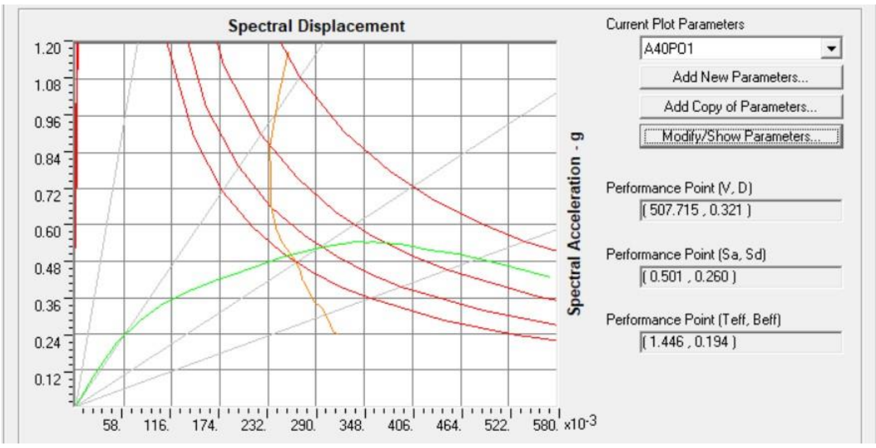

Fig. Capacity spectra for case 2 without exposure to elevated temperature

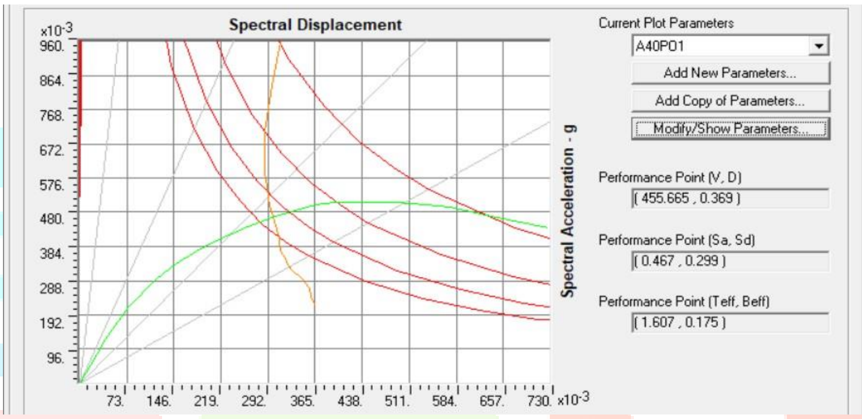

Fig. Capacity spectra for case 2 after exposure to 300°C

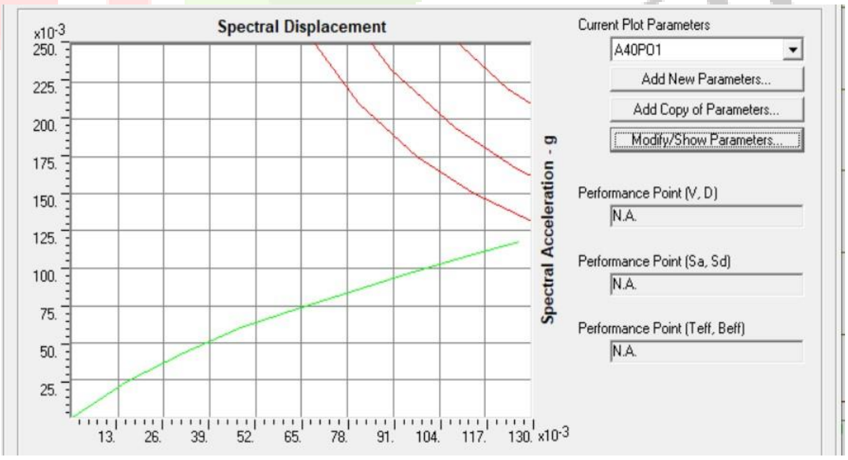

Fig. Capacity spectra for case 2 after exposure to 500°C

VII. Performance points (DBE)

Case	Temperature	Displacement (M)	Base shear (KN)
Case 1	20	0.467	311.429
Case 1	300	0.542	291.776
Case 1	500	-	-
Case 2	20	0.321	507.715
Case 2	300	0.369	455.665
Case 2	500	-	-

VIII.Vulnerability Index based on nonlinear static analysis

Case	Temperature (°C)	Vulnerability index
Case 1	20	0.059
Case 1	300	0.078
Case 1	500	0.208
Case 2	20	0.045
Case 2	300	0.048
Case 2	500	0.081

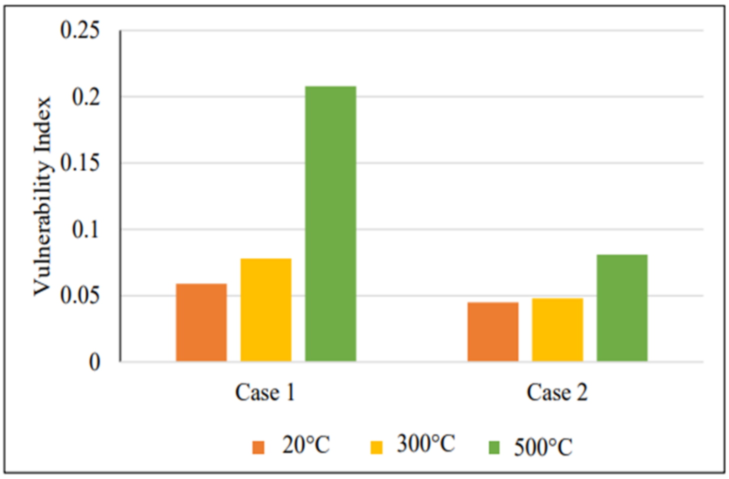

Fig. Vulnerability Index based on nonlinear static analysis

7. CONCLUSION

The nonlinear static analysis (Pushover Analysis) is carried out for a typical 6-storey office building at ambient and elevated temperature of 300°C and 500°C. In the present study, nonlinear stress–strain curves for confined concrete and default hinge properties are used. The elastic beam and column members are modelled as elastic elements with plastic hinges at their ends. The acceptance criteria with reference to the three performance levels such as IO, LS and CP are prerequisite for estimation of inelastic member as well as global structural behaviour. The present analysis involves force-controlled analysis under gravity load and the stressed structure, then is analysed for displacement-control option till target displacement is achieved. From the study the following conclusions are obtained:

- A significant variation is observed in base shear capacities and hinge formation mechanisms for building designed as per the load cases of IS 456-1964 and IS 4562000.
- The base shear in case 2 has a capacity of 545.129 kN, whereas for case 1, the base shear is found to be 311.6 kN. The base shear capacity decreases as the temperature increases.
- A drastic reduction in base force and displacement of about 72.73% and 40.14% is observed in case 2 frame exposed to 500°C which indicates M30 concrete is more vulnerable to elevated temperature.
- Ductile behaviour of the members is lost at elevated temperature which is indicated by the significant degradation in the ductility factors (μ) of the post-elevated temperature frames, and the reduction percentages for 300°C and 500°C were 33% and 40% for case 1 and 40% and 55%, respectively, for case 2 frame.
- It is observed that there is no significant reduction in base force and displacement for the frame exposed to 300°C compared to the frame without exposure to elevated temperature. The performance points obtained for frame without exposure to elevated temperature and frame exposed to 300°C are 311.139 and 292.809, respectively, for frame designed on gravity load and 507.851 and 455.054, respectively, for frame designed to resist seismic load.
- Based on the studies, it is concluded that RC frame exposed to 500°C is more vulnerable during an earthquake. No performance point is observed for frame exposed to 500°C for both the cases for DBE of IS: 1893–2002. The vulnerability index of RC frame designed as per IS 456:1964 and IS 456:2000 exposed to 500°C are 0.208 and 0.081, which is 71.6% and 44.4% higher than that of frame without exposure to elevated temperature.
- There is a necessity of enhancing the lateral strength of existing buildings designed as per the past code exposed to elevated temperature to resist a future earthquake.

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