THERMO-STRUCTURAL ANALYSIS OF RC COLUMNS

Bhavya Shree M
Student, Department of Civil Engineering
The National Institute of Engineering, Mysore, India

Dr. N.C. Balaji
Assistant Professor, Department of Civil Engineering
The National Institute of Engineering, Mysore, India

ABSTRACT
This paper investigates the axial load carrying capacity of the column at elevated temperatures 600°C to 1200°C at the interval of 200°C. Steel distribution in column design can be done in two ways, for the same percentage of steel i.e. 2-face reinforcement distribution and 4-face reinforcement distribution was done. Structural analysis using ETABS is performed for the both ways of column design for the safety check. Finite element software ANSYS mechanical APDL 15.0 is used to perform the thermal analysis to develop temperature profiles at elevated temperatures and Euro Code 2 suggests simplified methods, among that 500°C isotherm methods is utilised to calculate the reduced area after the fire exposure, then decrease in axial load carrying capacity is calculated as per IS 456 (2000). Calculated axial load carrying capacity of the column at elevated temperature is compared with the axial load carrying capacity of the column, which is not exposed to fire. Different fire exposure conditions like, one major dimensional face exposed to fire, two adjacent faces exposed to fire, only two major dimensional faces exposed to fire and all four faces exposed to fire, these are the different fire exposure conditions considered. Interaction curves which serves as failure envelop are developed for both the columns with two face steel reinforcement distribution and four face steel reinforcement distribution at both ambient and elevated temperatures.

Key words: ANSYS, 500°C Isotherm method

http://www.iaeme.com/IJCIET/issues.asp?JType=IJCIET&VType=9&IType=9

1. INTRODUCTION
Fire is considered as one of the most serious potential risks for buildings. In order to overcome against this risk fire safety in buildings has evolved as a main objective. These days
numerous researches are carried out keeping fire safety as main criteria. When RC structural systems remain exposed to elevated temperature for long time leads to loss in its strength and stiffness, structural system starts drastically losing its strength after 300°C temperature. Among the various structural elements, columns are the most important members in a building and its failure cause the sudden collapse of the entire structure. Columns are the most susceptible members in fire, since it can be exposed to a 1-, 2-, 3-, or 4-face fire attack, depending on the column orientation in the fire compartment. In a structure, columns are the primary structural elements which transfer the loads from superstructure to substructure. When the columns are exposed to high temperatures, change in material properties of concrete and reinforcing steel occurs. As the temperature increases yield strength and modulus of elasticity reduces, which leads to overall strength reduction of the column. If the column strength decreases than the applied load, then column will undergo crushing or buckling type of failure. Both thermal and structural behaviour is presented in this paper. Thermo-structural analysis is the combination of thermal analysis and structural analysis, in which the outputs of thermal analysis such as temperature profiles are incorporated to find structural components such as axial load bending moment, from which determination of the effect of loads on structure can be done. This type of analysis is called as thermo – structural analysis.

2. OBJECTIVES AND SCOPE OF THE WORK

Objectives of the work

- To study the structural behaviour of the RC columns exposed to the elevated temperatures
- To obtain the interaction curves for the columns subjected to elevated temperatures.

Scope of the work

- To determine the structural capacity of the RC column elements.
- To determine the temperature profiles of the RC column sections subjected to elevated temperatures of 600°C, 800°C, 1000°C and 1200°C for different reinforcement distributions.
- To determine the residual axial load carrying capacity of the RC column subjected to the elevated temperatures for different reinforcement distributions.
- To develop interaction curves for RC columns subjected to elevated temperature, having steel reinforcement on two and four face distribution for the same percentage of steel.

3. LITERATURE SURVEY

M. Mohamed Bikhiet et.al. ,(2014) [9] Carried out study on behaviour of reinforced concrete short columns exposed to fire, The study has given importance on the study of columns exposed to fire under axial load and to evaluate reduction in column compressive capacity after fire. This study concludes that the grade of concrete and steel has less effect in thermal criteria of failure but has a significant effect on axial capacity based on strength criteria. The ultimate failure loads for columns, which were exposed to fire, are smaller than columns, which were not exposed to fire. Columns not exposed to fire showed first crack load at nearly 80% of column failure load, while columns exposed to fire showed first crack load at about 50% of column failure load.

Aneesha Balaji et.al. ,(2015) [10] Focused on validation of Indian standard code provisions for fire resistance of flexural elements, the purpose of this research is to explain the simplified method, i.e., 500°C isotherm method. The method is customized for Indian conditions and a parametric study is done to find the fire rating for flexural elements. Fire ratings recommended in IS 456:2000 is compared with strength criteria by using the 500°C
iso therm method. It is also matched by thermal criteria obtained by heat transfer analysis of finite element model. In this study the code provisions are compared with thermal analysis and 500°C isotherm method. The 500°C isotherm method used in the present study is a simplified method which can be used for manual design of structural elements.

Aneesha Balaji et.al.,(2016) [11] Carried out study on behaviour of Reinforced Concrete Columns subjected to fire, the study was performed on columns of changed dimensional cross-section to examine the effect of eight parameters, namely the thermal boundary conditions, grades of concrete, grades of steel, types of aggregate, distribution of reinforcement on column faces, concrete cover, load eccentricity and support conditions. This study concludes that the bars distributed on four faces of the column cross-section provide some improvement in capacity and fire rating than those distributed on two faces for the same percentage of reinforcement.

Aneesha Balaji et.al.,(2016) [12] Carried out study on behaviour of Reinforced Concrete Columns of various cross-sections subjected to fire, the work focused on the effect of cross-sectional shape of column in fire resistance design. The various cross sections considered are Square, Ell (L), Tee (T), and Plus (+) shape, methodology involved is 500°C isotherm method and FEA (software used is ANSYS)

Author concluded that when geometry turn into complex, fire resistance decreases drastically. Comparing to the square shape, L, T and ‘+’ shapes shows sufficient decrease in fire resistance.

Sreelatha Vuggumudi et.al.,(2018) [13] Worked on Interaction diagrams for FRP strengthened RC rectangular columns with large aspect ratio, the objective of the work is to develop P–M interaction diagrams for RC rectangular columns having an aspect ratio greater than 2 externally strengthened using FRP composites without any shape modification and subjected to combined axial and lateral loading. This study concludes that the proposed semi-empirical solutions shall be used to predict the lateral load carrying capacity of FRP strengthened RC rectangular columns having aspect ratio greater than 2 and without shape modification of cross section.

4. METHODOLOGY
4.1. Structural Analysis
In this study, structural analysis is carried out for the plan shown in fig.1 using ETABS software, this structural system consists of 3 stories, each story height 3.2 m. Dimension of the structure is 10mx10m columns positions are as shown in fig.1, 3-D view of structural system is shown in fig.2.

Considering the building type as educational building, as per IS 875 part-2 building classification [ii], live load is taken as 3 kN/m² and floor finish and ceiling load is as per IS 875 part-1 table 2 is taken as 0.75 kN/m² and 0.25 kN/m² respectively. Load coming on the
Thermo-structural Analysis of RC Columns

central axial column is calculated and for 1% of steel, column is designed for two ways of steel distribution i.e. steel distribution on two faces of column and steel distribution on four faces of column and axial load carrying capacity is calculated as per IS 456(2000). The designed columns were modelled and analysed using ETABS, then it is checked for safety by comparing the load coming on the column (obtained from ETABS) and calculated axial load carrying capacity. This gives us structural performance of the RC column before fire.

4.2. Models and Fire Exposure Conditions
Model 1: Rectangular column section having dimension 500X250, with 2 face steel reinforcement distribution
Model 2: Rectangular column section having dimension 500X250, with 4 face steel reinforcement distribution

Fire exposure conditions- following are different cases of fire exposure conditions of central column, considered in the present study.
Case 1: Major dimensional axis face exposed to fire
Case 2: Two adjacent faces exposed to fire
Case 3: Two major dimensional axes faces exposed to fire
Case 4: All four faces exposed to fire

http://www.iaeme.com/IJCIET/index.asp  1454  editor@iaeme.com
4.3. Thermal Analysis
Thermal analysis deals with the study of material properties as they change with the temperature. In this study, thermal performance of RC column is found by performing thermal analysis for which the column designs were modelled in ANSYS mechanical APDL 15.0 and subjected to elevated temperatures of 600°C to 1200°C at the interval of 200°C, for different fire exposure conditions. From this we get temperature profiles as output, this temperature profiles gives temperature at each node of the column. From these temperature profiles reduced axial load carrying capacity is calculated by 500°C Isotherm method. This gives us the thermo-structural performance of the column.

4.3.1. Thermal Analysis by ANSYS
For modelling RC column SOLID70 and LINK33 were used for concrete and reinforcing steel respectively. SOLID70 is a 3-D element and brick shaped with 8-node, whereas LINK33 is a line element with 2-node, for both the elements temperature is the DOF at each node. Getting solution will be nodal solution from which we can get temperature at every node in temperature contours, which leads us to develop temperature profile. Fig.10 and Fig.11 shows temperature contour and temperature profile respectively.

4.3.2. 500°C Isotherm Method
4.3.2.1. Calculation of reduced cross sectional area of the column
500°C isotherm method is used to calculate the reduced cross sectional area of column due to fire. This method is based on assumption that concrete strength above 500°C is neglected, whereas concrete at a temperature below 500°C is assumed to retain its full strength, following are the steps involved:

- Evaluation of obtained temperature profile from the thermal analysis.
- Excluding the area of concrete outside the 500°C isotherm. Rest of the area is rounded up to form reduced area and it is approximated by rectangles.
- Temperature of the reinforced steel is determined from the obtained temperature profile. If any reinforcement falls outside the reduced area, despite this, they are included in calculation of the reduced axial load carrying capacity of the column section exposed to fire.

4.3.2.2. Calculation of reduced axial load carrying capacity of RC column section
After calculating reduced cross sectional area and reduced strength of steel and concrete, the procedure for column design given in the IS 456 2000 is used to calculate the reduced axial load carrying capacity. In clause 39.3 [IS 456: 2000] gives equation for axial load carrying capacity of short axially loaded columns, the same equation is substituted with reduced cross sectional area and reduced strength in order to get reduced axial load carrying capacity.

\[ P_{uol} = 0.4*(f_{ck}*A_c + f_{ck,0}*A_c') + 0.67*\sum (f_{y,0i}* A_{si}') \]  

(1)

\( f_{ck} \) = characteristic strength of concrete; \( f_y \) = characteristic strength of steel; \( A_c \) = Area of concrete whose temperature is lesser than 500°C; \( A_c' \) = Area of concrete whose temperature is greater than 500°C.
[Note: \[ \sum f_{y,0i} A_{si} = f_{y,01} A_{st1} + f_{y,02} A_{st2} + \cdots + f_{y,0i} A_{st'} \]]

\[ A_{si} = \text{area of steel bars having different temperatures}; \quad f_{y,0i} = \text{characteristic strength of steel at different temperature} \]

Let \( P_{uo1}' \) and \( P_{uo2}' \) be the reduced axial load carrying capacities of model 1 and model 2 at 600\(^{\circ}\)C respectively. Similarly above mentioned steps were performed at 800\(^{\circ}\)C, 1000\(^{\circ}\)C and 1200\(^{\circ}\)C.

4.3.2.3. Calculation of Percentage Decrease in Axial Load Carrying Capacity

After calculating reduced axial load carrying capacities of the column, which were exposed to elevated temperatures of 600\(^{\circ}\)C, 800\(^{\circ}\)C, 1000\(^{\circ}\)C and 1200\(^{\circ}\)C, percentage decrease in axial load carrying capacity of column exposed to elevated temperature, with respect to actual load carrying capacity of column at ambient temperature is calculated. Actual load carrying capacity of the column is calculated as per IS 456 (2000) clause 39.3.

4.3.2.4. Compression of Percentage Decreased in Axial Load Carrying Capacities in Model 1 and 2

Percentage decrease in axial load carrying capacity of column exposed to elevated temperatures 600\(^{\circ}\)C, 800\(^{\circ}\)C, 1000\(^{\circ}\)C and 1200\(^{\circ}\)C for both the model 1 and model 2 were compared for the load carrying capacity.

4.4. Interaction Curves

Apart from the axially loaded central column, uniaxial columns of the building were considered for developing interaction curves for ambient temperature and elevated temperature of 600\(^{\circ}\)C to 1200\(^{\circ}\)C at the interval of 200\(^{\circ}\)C. Interaction curve is a complete graphical representation of the design strength of uniaxial loaded columns. Interaction curve defines the different \((P_u, M_u)\) combinations for all possible eccentricities of loading. For column 2, 4, 6 and 8 interaction curves were developed.

5. RESULTS

The designed columns were checked for safety using ETABS which is found to be safe, load coming on the column was 1650 kN. Whereas calculated axial load carrying capacity of the column as per IS code provision, with two face and four steel distribution was found to be 1834.3 kN and 1927.93 kN respectively. Hence the column design is safe, following tables gives the results of the work.

**Table 1 Models and it’s descriptions**

<table>
<thead>
<tr>
<th>Type of the column</th>
<th>Model type</th>
<th>Fire exposure conditions</th>
<th>Temperature in °C</th>
<th>Models generated</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axially loaded central column</td>
<td>Model 1 Column with 2 face reinforcement ( A_s ) (required) 1250 ( \text{mm}^2 ); ( A_s ) (provided) 1256.63 ( \text{mm}^2 ) With axial load carrying capacity (no fire conditions) 1834.32 kN</td>
<td>Case 1 Major dimensional axis face exposed to fire</td>
<td>600</td>
<td>M1C11</td>
<td>M1C11, M1C12, M1C13 and M1C14 these are the model 1 columns for which only major dimensional axis face was exposed to fire at temperatures of 600(^{\circ})C, 800(^{\circ})C, 1000(^{\circ})C and 1200(^{\circ})C respectively</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>800</td>
<td>M1C12</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1000</td>
<td>M1C13</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1200</td>
<td>M1C14</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Case 2 Two adjacent faces exposed to fire</td>
<td>600</td>
<td>M1C21</td>
<td>M1C21, M1C22, M1C23 and M1C24 these are the model 1 columns for which two adjacent faces was exposed to fire at temperatures of 600(^{\circ})C, 800(^{\circ})C, 1000(^{\circ})C</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>800</td>
<td>M1C22</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1000</td>
<td>M1C23</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1200</td>
<td>M1C24</td>
<td></td>
</tr>
<tr>
<td>Case 3</td>
<td>Two major axes faces exposed to fire</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>--------</td>
<td>-------------------------------------</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>600</td>
<td>M1C31, M1C32, M1C33 and M1C34 these are the model 1 columns for which two major axes faces was exposed to fire at temperatures of 600°C, 800°C, 1000°C and 1200°C respectively.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>800</td>
<td>M1C32, M1C33 and M1C34</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1000</td>
<td>M1C33</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1200</td>
<td>M1C34</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Case 4</th>
<th>All four faces exposed to fire</th>
</tr>
</thead>
<tbody>
<tr>
<td>600</td>
<td>M1C41, M1C42, M1C43 and M1C45 these are the model 1 columns for which all four faces was exposed to fire at temperatures of 600°C, 800°C, 1000°C and 1200°C respectively.</td>
</tr>
<tr>
<td>800</td>
<td>M1C42, M1C43</td>
</tr>
<tr>
<td>1000</td>
<td>M1C43</td>
</tr>
<tr>
<td>1200</td>
<td>M1C44</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Case 1</th>
<th>Major dimensional axis face exposed to fire</th>
</tr>
</thead>
<tbody>
<tr>
<td>600</td>
<td>M2C11, M2C12, M2C13 and M2C14 these are the model 1 columns for which only major dimensional axis face was exposed to fire at temperatures of 600°C, 800°C, 1000°C and 1200°C respectively.</td>
</tr>
<tr>
<td>800</td>
<td>M2C12, M2C13</td>
</tr>
<tr>
<td>1000</td>
<td>M2C13</td>
</tr>
<tr>
<td>1200</td>
<td>M2C14</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Case 2</th>
<th>Two adjacent faces exposed to fire</th>
</tr>
</thead>
<tbody>
<tr>
<td>600</td>
<td>M2C21, M2C22, M2C23 and M2C24 these are the model 1 columns for which two adjacent faces was exposed to fire at temperatures of 600°C, 800°C, 1000°C and 1200°C respectively.</td>
</tr>
<tr>
<td>800</td>
<td>M2C22, M2C23</td>
</tr>
<tr>
<td>1000</td>
<td>M2C23</td>
</tr>
<tr>
<td>1200</td>
<td>M2C24</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Case 3</th>
<th>Two major axes faces exposed to fire</th>
</tr>
</thead>
<tbody>
<tr>
<td>600</td>
<td>M2C31, M2C32, M2C33 and M2C34 these are the model 1 columns for which two major axes faces was exposed to fire at temperatures of 600°C, 800°C, 1000°C and 1200°C respectively.</td>
</tr>
<tr>
<td>800</td>
<td>M2C32, M2C33</td>
</tr>
<tr>
<td>1000</td>
<td>M2C33</td>
</tr>
<tr>
<td>1200</td>
<td>M2C34</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Case 4</th>
<th>All four faces exposed to fire</th>
</tr>
</thead>
<tbody>
<tr>
<td>600</td>
<td>M2C41, M2C42, M2C43 and M2C44 these are the model 1 columns for which all four faces was exposed to fire at temperatures of 600°C, 800°C, 1000°C and 1200°C respectively.</td>
</tr>
<tr>
<td>800</td>
<td>M2C42, M2C43</td>
</tr>
<tr>
<td>1000</td>
<td>M2C43</td>
</tr>
<tr>
<td>1200</td>
<td>M2C44</td>
</tr>
</tbody>
</table>

Model 2
Column with 4 face reinforcement
Aₑ (required) 1250 mm²
Aₑ (provided) 1608.49 mm²
With axial load carrying capacity (no fire conditions) 1927.93 kN
After calculating the percentage decrease in axial load carrying capacity, compression of percentage decrease in axial load carrying capacity in model 1 and model 2 is done, following are the graphs showing comparative representation of model 1 and model 2 results at every case considered.

Case 1: Major dimensional axis face exposed to fire

**Figure 11**

**Figure 11** Graphical representation of % decrease in axial load carrying capacity of model 1 and model 2, at elevated temperatures for case 1 fire exposure conditions.

Case 2: Two adjacent faces exposed to fire

**Figure 12**

**Figure 12** Graphical representation of % decrease in axial load carrying capacity of model 1 and model 2 at elevated temperatures for case 2 fire exposure conditions.

Case 3: Two major axes faces exposed to fire

Case 4: All four faces exposed to fire

**Figure 13**

**Figure 14**
Figure 13 Graphical representation of % decrease in axial load carrying capacity of model 1 and model 2 at elevated temperatures for case 3 fire exposure conditions.

Figure 14 Graphical representation of % decrease in axial load carrying capacity of model 1 and model 2, at elevated temperatures with case 4 fire exposure conditions.

5.1. Interaction Curves
Apart from the axially loaded central column, uniaxial columns of the building were considered for developing interaction curves for ambient temperature and elevated temperature of 600°C to 1200°C at the interval of 200°C. In this section interaction curves for model 1 and model 2 at elevated temperatures were developed.

Model 1

Figure 15: P-M Interaction curve of model 1 at 25°C temperature

Figure 16: P-M Interaction curve of model 1 at 600°C temperature

Figure 17: P-M Interaction curve of model 1 at 800°C temperature

Figure 18: P-M Interaction curve of model 1 at 1000°C temperature

Model 2

Figure 19: P-M Interaction curve of model 2 at 25°C temperature

Figure 20: P-M Interaction curve of model 2 at 600°C temperature
6. CONCLUSIONS

Conclusions drawn from the present study are:

- For the same percentage of steel, model 2 (i.e. column provided with the 4 face steel distribution) exposed to elevated temperatures retains more axial load carrying capacity, than the model 1 (i.e. column provided with 2 face steel distribution) under same fire conditions.

- Different fire exposure conditions have considerable effect on the axial load carrying capacity of the column.

- As the temperature increases axial load carrying capacity of the RC column decreases.

- In case 4 (i.e. all four faces exposed to fire) at 1200°C load carrying capacity of RC column is completely zero, since the reduced strength of concrete and steel were taken as zero [as per euro code 2].

- From the interaction curves as well we can conclude that, column with four face steel distribution possesses higher compressive resistance, than the column with two face steel distribution

- As the temperature increases compressive resistance of the column decreases.

REFERENCES


