Constitutive Model of Concrete Frame Structure Under Localized Fire Simulations

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Abstract. In recent decades, special considerations have been given to the use of fire safety analysis on modern concrete buildings. The considerations include the stability of building which providing sufficient time for the occupants to evacuate and for firefighting. This paper is mainly concerned with the performance of concrete frame structure exposed to localized fire. From the simplified model, load carrying mechanism of concrete frame is identified and several design implications are discussed. Frame model in present analyses constructed of normal-strength reinforced concrete beams and high-strength reinforced concrete columns represented typical commercial office building. All elements of the frame are modeled using 3D tetrahedral coupled-field solid element defined by ten nodes with four degree of freedom. Temperature response predicted from previous researchers and loadings represented total design load at the fire limit state are applied. Constitutive model in a full thermal strain-stress analysis is employed on simple empirical formula. Three-dimensional finite element computer model using ANSYS considering transient effects gave reasonable results. The results indicate that behavior of individual isolated member accommodated by the current codes is different from the behavior of complete structure. Therefore, developing fire engineering analysis from the behavior of complete structures is strongly recommended.

Keywords: Concrete · Fire · Frame · Stress · Strain

1 Introduction

Concrete structures are traditionally seen to have a favorable position in building industry with regard to their performance of fire resistance. So that modern concrete structures may often require special consideration for their fire performance. The special consideration may include the ability of building to fulfill its assigned function in the event of fire as well as the stability of building which provides adequate time for the occupants to escape and for firefighting. In a large and complex building, this tends to be as more important as evacuation of the occupants to escape the building and firefighting operations take much longer [1].

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Reference [2] explained that in order to achieve sufficient standard of fire safety and fire performance of structural building construction, there are two principal ways can be applied in the practice. First is the simple application of the building codes and secondly is a fire safety engineering solution. The application of the building codes and standards gives little flexibility and require only a limited engineering approach. The second way gives flexible design ability to achieve a special performance of the building and needs greater skills involving engineering judgment. The integration should be managed from the design, construction, and continued maintenance [3].

Change in materials performance always happen when the structures are exposed to high temperature caused by the changes of structural behavior. Reinforced concrete materials when subjected to fire are anticipated to encounter a loss of strength. The loss of strength may or may not be recovered after cooling. The ability of recovery depends on the materials type, severity of the fire as measured by its temperature, and the duration of the exposure fire [4].

2 Structure and Material Model

2.1 Concrete Frame

Frame model constructed of normal-strength reinforced concrete continuous beam and high-strength reinforced concrete columns. The frame is assumed to be fixed at the bottom of all columns while the top of columns can move horizontally both in the X and Z and vertically in the Y directions. The frame has 4.00 m height and 21.50 m length which divided by four columns with the distance of each column is 7.50 m. The internal columns have dimension of 0.40 × 0.40 m and the external columns of 0.25 × 0.40 m. The dimension of 0.40 m × 0.40 m for the beam is chosen in order to match the performance of the slab behavior. To represent the steel-reinforcement, pairs of 0.01 m thick of steel have been layered longitudinal within the beam and vertical within the internal columns, and the 0.006 m thick are layered within the external columns. The steel layers have been positioned at the distance of 0.04 m from both sides of the top and bottom concrete surfaces. The 0.04 m distances from concrete surfaces represent the concrete cover to the reinforcement. The dimensions of steel layer were used so that the reinforcement ratio of concrete were not exceed to the maximum specified (4%) in the Standards [5–7]. Dimension of the element at concrete frame model in 3-D is illustrated in Fig. 1. And, the typical cross section of internal column as a structural member is presented in Fig. 2.

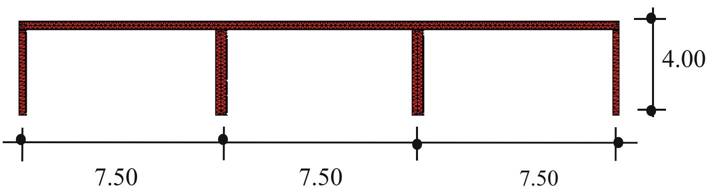


Fig. 1. Frame model of reinforced concrete

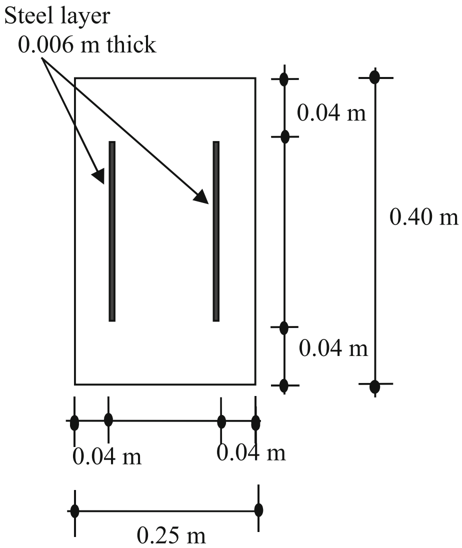


Fig. 2. Typical cross-section of the internal column

The use of the layered steel instead of steel reinforcement in thermal case is due to convenience in the model analysis used. Bending direction of the layered steel will have similar results obtained from the steel reinforcement. The temperature distribution throughout the element will have little different between the layered steel and the reinforcement. But, the effect to the reinforcing steel can be considered minor [8–10].

2.2 Element Type and Material Model

The Finite element analysis software has been performed using ANSYS with all elements of the reinforced concrete frame are modeled using 3-D tetrahedral coupled-field solid element. The coupled-field analysis is an analysis that takes into account the interaction between two or more disciplines of engineering. In this case, the coupled-field analysis is used for thermal-stress analysis [11–13].

The element is defined by ten nodes with four degrees of freedom (UX, UY, UZ, and TEMP) at each node. The total number of 38036 elements and 59080 nodes are used in the analyses with maximum size of the element is 130 mm. All materials defined with three different material models, i.e. Material Model Number 1 for layered steels, Material Model Number 2 for high-strength concrete of the columns, and Material Model Number 3 for normal-strength concrete of the beams [14].

The geometry, node locations, and the coordinate system for this type of element are shown in Fig. 3 and meshing of the element at concrete frame model is illustrated in Fig. 4.

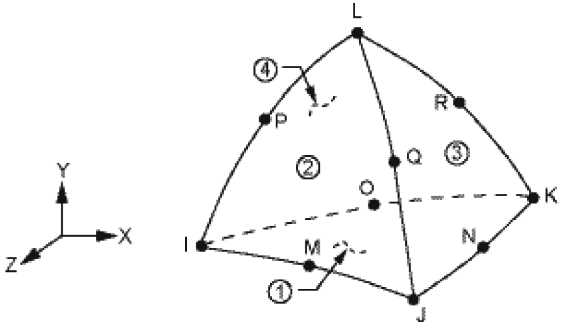


Fig. 3. Geometry of the element

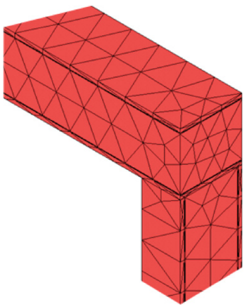


Fig. 4. Meshing of the elements at the top corner frame

2.3 Boundary Conditions and Loadings

The boundary conditions assumed that the connections between columns and beams are rigid and the beam can move in any directions. All of the columns are fixed in their foundations, therefore the underneath of the column are restrained against translations and rotations.

The loadings represent total design load at the fire limit state applied at an office building. In order to simulate loading from the above floor, the beams are assumed to receive vertical uniform design load over the fire compartment area of 3.25 kN/m^2. Loading from the weight of concrete slab of $2400 \times 9.81 \times 0.25 = 5.89$ kN/m^2 is added so that the total vertical uniform load would be 9.14 kN/m^2. Therefore, at 7.50 m runs a uniformly distributed load of $q = 68.55$ kN/m is applied.

At 4.00 m height columns, compressive forces of $P_1 = 462.5$ kN applied to internal columns and $P_2 = 925$ kN applied to external columns, respectively. As described in the previous explanation that the loadings of the columns are represent total design load at the fire limit state at an office building as well. The total loading of the concrete frame is delineated in Fig. 5.

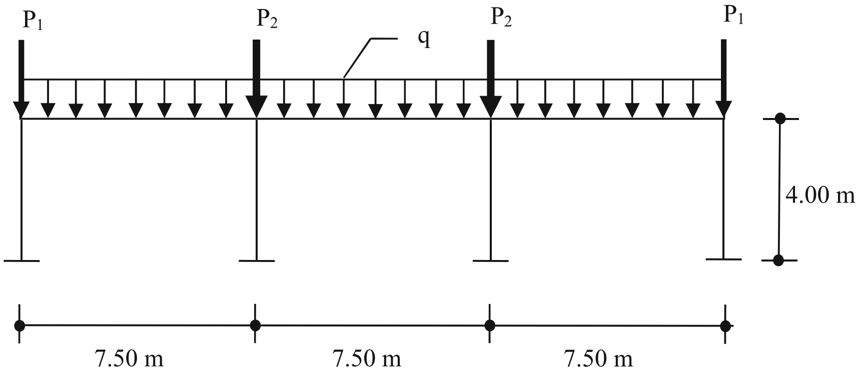


Fig. 5. Loading of the concrete frame

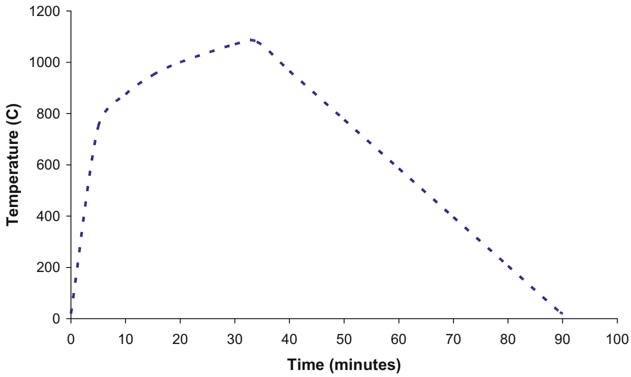


Fig. 6. Time-temperature of the firing compartment

3 Analysis Under Transient Strains

Reference [15] developed an empirical formula based on the plotted results of the average Young's modulus to obtain stress-strain relationship with transient strain included. The simple empirical formula of constitutive model can easily be incorporated into various commercial finite element analysis codes.

Time-temperature response of the analysis is predicted from the European code. The time-temperature curve of the firing compartment and the normalized stress-strain curve with transient strain included are presented in Fig. 6 and Fig. 7, respectively.

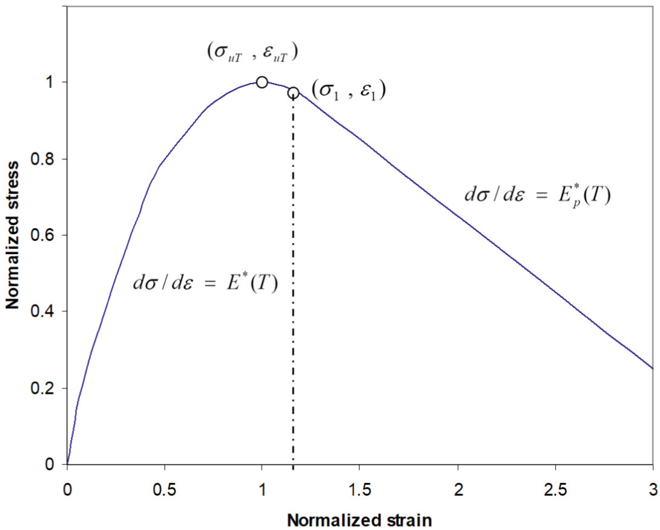


Fig. 7. Normalized stress-strain curve with transient strain included

Temperature-dependent of Young's modulus in the elastic ranges are suggested in the following equations:

$$E^*(T) = E_0\ \exp\left[-\frac{(T-20)^{0.65}}{25}\right] \tag{1}$$

For the Young's modulus in the plastic range the following equation is recommended:

$$E_p^*(T) = E_0^-\ \exp\left[k_p(T-20)^{2.15}\right] \tag{2}$$

where: E_0 = initial young's modulus,
E_0^- = -0.045 E_0, and
$k_p = 10^{-6}$.

4 Results and Discussions

The analysis carried out five different fire arrangements where Case 1 analyses the frame structure before fire. Follow by Case 2 when the fire exposes at one external room. Case 3, Case 4 and Case 5 exposes the fire at one internal room, both external and internal rooms, and all three rooms, respectively. In all cases the temperature distribution in the fired compartment is assumed to be uniform both in the vertical and horizontal direction. The five various fire arrangements of Cases 1 – 5 are presented in Fig. 8.

Summary of maximum/minimum stresses and displacements for the all cases are presented in the following Table 1 for the beams and Table 2 for the columns.

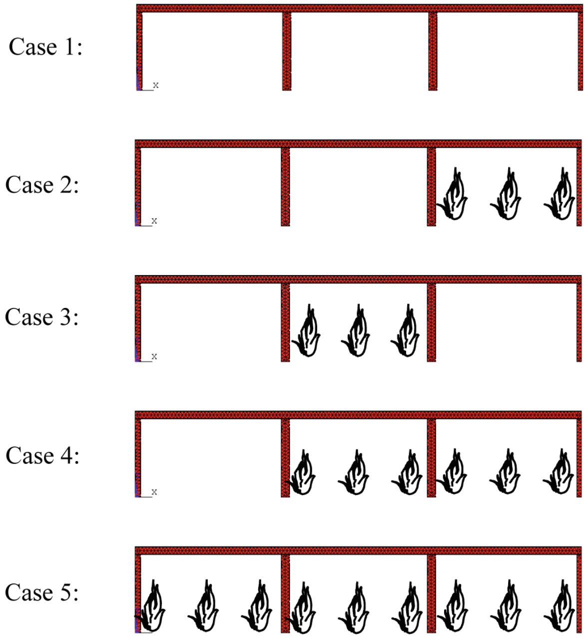


Fig. 8. Various fire arrangements of cases 1 - 5

Table 1. Stresses and displacements of the beams

	Case 1	Case 2	Case 3	Case 4	Case 5
Max. T (°C)	20	935	935	935	935
Max.Δx (mm)	3.5	21	11	33	27
Max.Δy (mm)	13	21	12	19	21
f'_{max} (MPa)	23.88	25.13	23.06	19.22	23.34
f'_{min} (MPa)	-17.23	-35.67	-41.39	-47.28	-39.34

Table 2. Stresses and displacements of the columns

	Case 1	Case 2	Case 3	Case 4	Case 5
Max. T (°C)	20	935	935	935	935
Max.Δx (mm)	3.5	21	11	33	27
f'_{min} (MPa)	-32.86	-60.79	-64.46	-66.50	-62.67

Where: Max T = maximum temperature of the element.
Max Δx = maximum horizontal displacement.
Max Δy = maximum vertical displacement.

f'_{max} = maximum tensile stress.
f'_{min} = maximum compressive stress.

4.1 Displacements

From Table 1 the maximum values of horizontal-displacement of 33 mm occurred in external column of the Case 4 and maximum vertical-displacement of 21 mm occurred in the external beam of the Case 2 and Case 5. The less significant values of the displacements are found in the Case 3 due to symmetrical loading where the fire only exposed at one room in the middle.

The maximum displacements in all cases are satisfied to the normal limiting conditions for the stability performance criterion in European Standard [7] where the failure criteria for load bearing capacity in terms of deflection for beam is ≥span/20 and for lateral column is ≥120 mm. However, the value of 33 mm lateral displacement of the column in Case 4 can be considered very significant since the initial displacement before fire was only 3.5 mm. This means that the lateral displacement of the column increased over 900% from the initial displacement before fire.

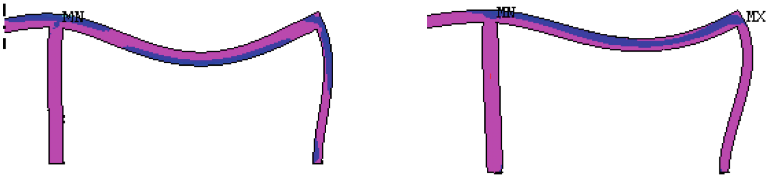
The large lateral displacement of the external column is caused mainly by thermal expansion of the heated beams. In fact, the European Standard [7] does not require the actual displacement to be calculated explicitly in ambient conditions and does not concern with deflection in an accidental situation. However, the span/depth ratios should be satisfied. In this case the beams span/depth ratio of 18.75 (calculated from 7500/400) is satisfied the standard.

4.2 Stresses

It is interesting to notice that from the Table 1, the highest maximum concrete compressive stresses of -47.276 MPa in the beam and -66.499 MPa in the column are both found in the Case 4 where the firing compartment localized at two rooms, i.e. the internal and external rooms of the frame. On the other hand, the lowest maximum concrete compressive stresses of -35.672 MPa in the beam and -60.799 MPa in the column are both occurred in the Case 2 when the fire exposed only at one external room. The improvement of compressive stresses in the beams from the condition before firing in the Case 1 to after firing in the Case 2, Case 3, Case 4, Case 5 are 207%, 240%, 274%, and 228%, respectively. While in the column the improvement of the compressive stresses are 185%, 196%, 202%, and 191%, respectively.

The tension stress is assumed to be taken by the steel reinforcement. The critical temperature at the steel reinforcement is less than 400 °C. Since the temperature reduced steel strength is 0.85 f_y, for all cases the steel strength are not significantly reduced. Therefore, further discussion would be focused mainly on the compression stresses which will be taken by concrete strength.

The average concrete temperature within the element can be determined from arithmetic mean of concrete temperature at the surface, quarter depth-temperature and center line temperature. In all cases, the critical average temperature is 402 °C on the beams and 433 °C on the column. Applying linear interpolation for strength reduction factor,



Tensile stress before fire on Case 1 Tensile stress after fire on Case 2

Fig. 9. Comparison of maximum principal stresses before and after fire of Case 1 and Case 2

the residual concrete strength reduction factors are 0.80 for the beam and 0.72 for the column. Assuming that concrete design strength of the beam $f_{cu} = 61$ MPa and column $f_{cu} = 103$ MPa, the temperature reduced permissible stresses, $\overline{f}_c$ of concrete beam and column would be:

- Beam: $\overline{f}_c = 0.80 \times 61 = 48.8$ MPa
- Column: $\overline{f}_c = 0.72 \times 103 = 74.2$ MPa

In order to identify how close each case to the failure, the values of compressive stresses given in Table 1 then be compared with the temperature reduced permissible stress. Fractional values of the applied stresses to the temperature reduced permissible stresses at the firing beams in Case 2, Case 3, Case 4 and Case 5 are $0.73\overline{f}_c$, $0.85\overline{f}_c$, $0.97\overline{f}_c$ and $0.81\overline{f}_c$, respectively. And the applied stresses at the columns in the Case 2, Case 3, Case 4, and Case 5 are $0.82\overline{f}_c$, $0.87\overline{f}_c$, $0.90\overline{f}_c$, and $0.84\overline{f}_c$, respectively.

For all cases, the applied stresses are found lower than the permissible stresses. These indicate that the structure will not collapse assuming there are no further beams and columns strength reductions due to concrete spalling.

4.3 Load Carrying Mechanism

The objective of this section is to study the mechanism by which the structure sustains the applied load at elevated temperature [16, 17]. The comparison is made between Case 1 at condition before fire and Case 2 at elevated temperature when the thermal expansion is taken into account. It has been shown in previous section that the thermal expansion effects play considerable role with regard to the significant increase of displacement achieved, especially the horizontal displacements of the beam.

Comparison of maximum principal stress before fire and under fire is represented in Fig. 9. The figures show the increases of horizontal displacements of the beams indicate that the load carrying mechanism has been modified by thermal expansion at elevated temperature. The location of maximum tensile stress distribution in the middle span of beam at elevated temperature is moved upward compared to the initial maximum tensile stress before fire. These indicate that sagging moment occurred after fire is higher than before fire. It should be noted that the moment capacity of the beam will also reduce in real structures under fire due to concrete spalling at the bottom of the beams.

5 Conclusions

Performance of concrete frame structures when exposed under localized fire scenarios have been discussed in present investigation. Three-dimensional finite element computer model using ANSYS considering transient effects gave reasonable results. Constitutive model with the transient strains included [15] is employed because this simple constitutive model can easily be incorporated into various commercial finite element analysis codes.

The tension stress is assumed to be taken by the steel reinforcement. The critical temperature at the steel reinforcement is less than 400 °C. In this condition, the temperature reduced steel strength is 0.85 f_y, thus, for all cases the steel strength are not significantly considered.

The results of the present modeling indicated that the behavior of complete structure is different from the behavior of individual isolated member. The results have good agreement with the previous researches in the similar studies [3, 18, 19]. Therefore, it is essential to develop new fire engineering methods from the study of complete structures rather than from isolated member behavior.

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