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MODELING FLEXURAL CAPACITY OF 3D REINFORCED CONCRETE CEILING PANEL SUBJECTED TO A STANDARD FIRE

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The summary. *In concrete structures, determining the fire endurance time is fundamental so as ensure that structural elements be able to withstand an aggressive fire for a reasonable time period. Slabs are the most susceptible to fire damage because of their large surface area and shallow depth. Analyzing their bearing capability after sustaining fire requires the knowledge of temperature distribution in the cross sections, as well as thermal properties of concrete and embedded steel. In this paper, a thermal structural analysis is carried out on 3D slab panels system following American Concrete Institute (ACI) building code [1] in conjunction with stress-strain relationships and ultimate stain data recommended by the Euro code. The consistency between analytical modeling and experimental results has confirmed the accuracy of the proposed model.*

Key words: *concrete, structural mechanics, experimental mechanics, nonlinear elasticity, failure*.

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МОДЕЛЮВАННЯ ЗГИННИХ ВЛАСТИВОСТЕЙ 3D ЗАЛІЗОБЕТОННОЇ СТЕЛЬОВОЇ ПАНЕЛІ ЗА СТАНДАРТНОГО ВИПРОБУВАННЯ ВОГНЕТРИВКОСТІ

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Резюме. *Тривалість збереження властивостей матеріалів під впливом вогню характеризує вогнетривкість бетонних конструкцій Бетонні плити є найбільш вразливими до впливу вогню конструкціями внаслідок їх значної площі та товщини. Аналіз їх тримкості потребує температурного розподілу по перерізу конструкції, а також теплових властивостей бетону та сталі. В даній роботі проведено тримірний температурний структурний аналіз системи панелей у відповідності до будівельних норм (ACI) та Єврокодів. Достовірність аналітичної моделі підтверджено експериментальними результатами.*

Ключові слова: *бетон, будівельна механіка, експериментальна механіка, нелінійна пружність, руйнування.*

Introduction. Recently, there has been an increasing interest in using new structural systems for green housing. Such systems allow savings in energy and building costs while possessing higher seismic stability as compared to tradition reinforced concrete and steel structures. Three dimensional panel systems have been used in various parts of the word to construct multistory buildings and houses since 10 years ago [1]. A geometric configuration of a typical 3D ceiling panel is shown in Fig. 1. The panel consists of two steel meshes on both sides of an expanded polystyrene core and connected together with a truss wire to provide a 3D system. Additional longitudinal reinforcement was added at the tension side to promote structural capacity. A major concern regarding the use of the new building system is its ability to withstand an aggressive fire. In this paper, the fire time endurance of a typical panel under a standard fire regime was determined both experimentally and analytically. Hence, two panel were tested under a standard fire concentrated at the middle third region of its span using methane gas portable blazers fasten to a steel frame. The heat regime recommended by the ACI code was followed [2]. While load was applied uniformly throughout the panel span by 200kN hydraulic jack, deflection measurements were acquired. The test was terminated at the moment of collapse designated by excessive deflection. To establish the temperature profile the temperature at the top and the bottom surfaces as well as at the inside surfaces of the concrete layers were measured during testing by type K thermocouples. The results from fire testing were used to validate the analytical analysis.

Modeling Flexural Performance.

When heat is transferred through a medium, the temperature varies with time and position. Under particular conditions, temperature varies in a parabolic manner with time; such a system is called a Lumped System [3]. The concept underlying the original lumped system is extended such that following separation, material temperature and mechanical characteristics do not vary with position. The temperature and mechanical properties of the unit are assumed to be everywhere the same as those at the center of the unit. In this work, the cross section of an RC slab is divided into N segments for analysis. Each segment is considered to have a uniform (but different) temperature and iso-properties, according to the lumped system concept.

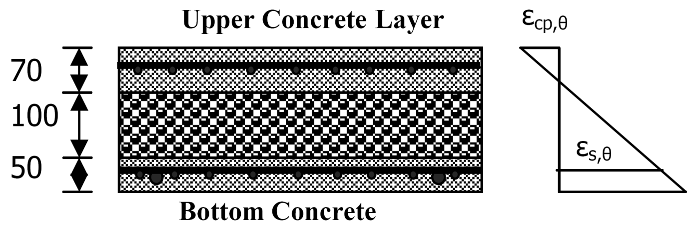

Figure 1. Cross section in the panel showing 4-mm steel meshes and 8-mm additional bottom steel (dimensions in millimeters)

The strain at extreme concrete compression fiber is first assumed, ε_{cp}. A distance from the extreme compression fiber to the neutral axis, c, is then defined. Combining the temperature distribution of the cross-section calculated from the temperature distribution model and the strain, a residual compressive stress at j element, $f_{j,\theta}$ can be obtained from the stress-strain relationships of concrete expressed mathematically in Eqns. 1. The values for which are given according to Euro code [4] as:

$$f_{j,\theta} = \frac{3\varepsilon_{cp}.f_{c,\theta}}{\varepsilon_{c1,\theta}(2+(\frac{\varepsilon_{cp}}{\varepsilon_{c1,\theta}})^3)} \qquad \varepsilon_{cp} \le \varepsilon_{c1,\theta}$$

$$f_{j,\theta} = f_{c,\theta}\left[1-\frac{\varepsilon_{cp}-\varepsilon_{c1,\theta}}{\varepsilon_{cu,\theta}-\varepsilon_{c1,\theta}}\right] \qquad \varepsilon_{cp} > \varepsilon_{c1,\theta} \tag{1}$$

Where $f_{c,\theta}$ is the ultimate compressive strength at a temperature θ, $\varepsilon_{c1,\theta}$ is the corresponding strain, and $\varepsilon_{cu,\theta}$ is the ultimate normal strain, given by Euro code [4].

It can assumed that the temperature, T_y, distribution to vary in a parabolic manner across the slab depth, hence we write:

$$T_y = (T_{bus} - T_{us})\frac{y^2}{0.07^2} + T_{us}$$

$$T_y = T_{es} + \frac{[T_{tll} - T_{es}]}{0.05^2} + (h-y)^2 \tag{2}$$

where, T_{es},T_{us}, T_{bus}, and T_{tbs} are the temperature at exposed, and unexposed surfaces, and at upper surface of the bottom layer, and the bottom surface of the top layer, respectively.

The tensile strength of the cross section of RC slabs can be written as:

$$T_{en} = \begin{cases} A_s f_{sy,\theta} & \text{when } \varepsilon_s \geq \varepsilon_{yr} \\ A_s f_{sr,\theta} & \text{when } \varepsilon_s < \varepsilon_{yr} \end{cases} \quad (3)$$

The strain in tension steel is related to strain in concrete based upon the strain diagram shown in Fig. 1 as follows:

$$\varepsilon_{s,\theta} = \varepsilon_{cp,\theta}\left(\frac{d}{c} - 1\right) \quad (4)$$

The stress in steel $f_{sr,\theta}$ is calculated based on the idealized stresses strain diagram and ultimate stain values given by the Eurocode [4]. The yielding strength of steel at elevated temperatures $f_{sr,\theta}$ is calculated based on the eqn. of Eurocode [3] given as:

$$k_{y,T} = \left[0.967\left(1 + \exp\left[\frac{T - 482}{39.19}\right]\right)\right]^{-\frac{1}{3.838}} \quad (5)$$

Where $k_{y,T}$ is the ratio of $f_{sy,\theta}$ (the yield strength at elevated temperature) to f_y (the yield strength at 20° C).

The compressive strength of the cross-section of RC slabs can be calculated by summing all the compressive strengths on the compressive side of lumped units as follows:

$$c = \sum_{j=1}^{c/\Delta y} f_{cj}^{T}.b.\Delta_y = \sum_{j=1}^{c/\Delta y} f_c^{/}.\sigma_j(\theta).b.\Delta_y \quad (6)$$

When the slab cross section is in static equilibrium, the residual ultimate moment of the slab M_u can be calculated according to ACI code [2] as:

$$M_u = A_s f_{yr}\left[d - \frac{\sum_{j=1}^{c/\Delta y} f_c^{/}.\sigma_j(\theta).b.\Delta_y.c_j.\Delta_y}{\sum_{j=1}^{c/\Delta y} f_c^{/}.\sigma_j(\theta).b.\Delta_y}\right] \quad (7)$$

If the sectional stress of the cross section is in static equilibrium, Eqns. (3) and (6) should be equal. If not, the assumed c value is suspected to be too small to satisfy the equilibrium. In this case, the value of c is increased and the calculation is repeated. If the equilibrium remains unsatisfactory with the adjusted c value, we assume that ε_{cp} is too small and ε is increased. This process continues until Eqns. (3) and (6) are equal. A computer program was developed to calculate the residual bending moment of reinforced concrete slabs after exposure.

Results and Discussion

The moment capacity for the ceiling panel under fire was determined based on equations 1 through 7 using the compressive strength of concrete, measured from obtained concrete cores, and the yield strength of reinforcing steel, determined experimentally, as well as the measured temperature profile across the panel depth. The obtained residual moment capacity versus exposure temperature was graphically presented in Fig. 2. The theoretical temperature at failure that corresponded to 50% residual moment capacity was about 780°C. This value is the same as the determined experimentally. The semi-linear behavior noticed after 600°C is referred to the fact that the decrease in the moment capacity is dictated by the reduction in steel yield strength under high temperatures: the concrete in the compression zone was subjected to relatively low temperature that had no effect on its mechanical behavior.

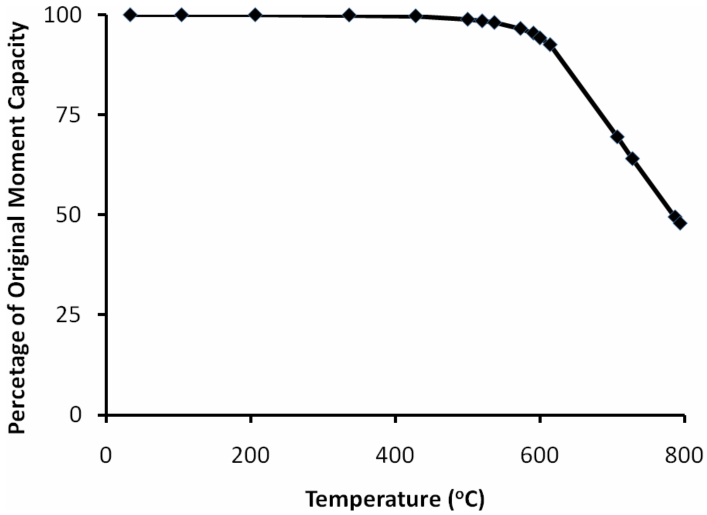

Figure 2. Reduction in Moment capacity versus temperature

References

1. Numayr K., and Haddad, R. Static and dynamic analytical and experimental analysis of 3D reinforced concrete panels. *Structural Engineering and Mechanics,* Vol. 32, pp. 399-406, 2009.
2. ACI 318R-02. Building Code Requirements for Structural Concrete and Commentary, American Concrete Institute, Michigan, 2002.
3. Çengel, Y. H. *Heat Transfer: a practical approach*, (International Edition), McGraw-Hill, USA, pp. 57-111, 1998.
4. Eurocode 3. Design of Steel Structures. *ENV 1993-1-2, General Rules –Structural Fire Design. European Committee for Standardization*, Brussels, Belgium, 1995.