NLFEA Fire Resistance of 3D System Ceiling Panel

Rajai Z. Al-Rousan
1 Department of Civil Engineering, Jordan University of Science and Technology, Irbid, Jordan
E-mail: reralrousn@just.edu.jo
2 Department of Civil and Infrastructure Engineering, American University of Ras Al Khaimah, Ras Al Khaimah, UAE

Abstract-
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, fire endurance a typical 3D ceiling panel is investigated analytically by using ANSYS in terms of time to failure and physical changes associated with exposure to failure. A nonlinear finite element (NLFEA) model was developed and validated by the experimental data before predictions were extended to investigate the significance of concrete density, concrete mechanical properties, and bottom concrete layer cover on the fire endurance time.

Keywords: Fire, Resistance, 3D System, Ceiling, Panel, Endurance Time, Thermal Analysis, NLFEA.

INTRODUCTION
People around the world expect that their workplaces and homes will be protected from the depredation of an unwanted fire. Therefore, it is extremely important to have sufficiently strong reinforced concrete (RC) structures to withstand fire damage to property and human life. The fire safety of RC structures largely depends on their fire resistance, which in turn depends on the fire resistance and combustibility of their main structural elements especially loaded flexural members. The post-fire residual load bearing capacities of these members are significant factors in determining the safety of the structure. Hence, the post and under fire residual mechanical properties of their reinforcing steel, concrete, and constituent materials are extremely important [1-6]. Analyzing the bearing capability of RC slabs after sustaining fire requires the understanding of temperature distribution in the cross sections. This is usually determined by the thermal properties of the material, such as the heat capacity and thermal conductivity. A simple thermal model, which is generally to all slabs with a rectangular cross section, has been assessed in a separate serious of studies which were also reported in a previous paper [7]. The modeling results achieved realistic concurrence with isothermal contours obtained by Lin [8], who analyzed the temperature distribution of pure concrete according to the time-temperature curve of standard fire. 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 world to construct multistory buildings and houses since 10 years ago [9]. A geometric configuration of a typical 3D ceiling panel is shown in Figure 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 was determined analytically using nonlinear finite element analysis (NLFEA). The NLFEA aimed at determining optimal key parameters that enhance fire endurance of the panels under study.

Figure 1: Typical panel showing steel meshes in concrete layers, steel truss, and polystyrene core

Nonlinear Finite Element Analysis (NLFEA)

Specimen details and materials properties
Figure 2 shows the typical dimensions and reinforcement layouts for the test specimens by Rami Haddad [10]. The 3D panel system was made from the welded truss of wire cross pieces, zinc coated steel wire mesh on inside and outside with a diameter of $\phi\ 3\ mm\ (3.1\ mm)$, and foamed polystyrene. Additional $\phi\ 8\ mm$ bars were added as shown in Figure 2. The panels were cast through shotcreting of the 50-mm concrete bottom layer or conventional placing of the 70-mm top concrete layer. The ceiling panels (400 $\times\ 220\times\ 1500\ mm$) were sliced from full-scale panels having 1200 mm width and 3000 mm length. The yield and ultimate strengths and elongation at failure for the reinforcing steel was determined experimentally. Corresponding results for $\phi\ 3$ and $\phi\ 8\ mm$
bars were (477 and 394 MPa), (752 and 509 MPa), and (2.4 and 1.4%), respectively. Concrete cores (75\(\Phi\)100 mm) were taken from the concrete top layer, cast especially for obtaining cores, before tested for compressive strength according to ASTM C42. The average compressive strength for 6 cores was 10 MPa.

**Figure 2:** Cross section in the panel showing 3-mm steel meshes and 8-mm additional bottom steel along with locations of thermocouples (dimensions in millimeters) [10]

**Fire endurance test**

The ceiling panels were loaded up to 50% of their nominal flexural and axial load capacity, before subjected to fire at their middle third as shown in Figure 3 [10]. The fire exposure is controlled to conform to the standard time-temperature curve recommended by the ASTM E119 [11] as shown in Figure 4.

![Figure 3: Schematics showing test setup [10]](image)

**Figure 3:** Schematics showing test setup [10]

**Modeling**

A NLFEA model was proposed and validated using the present data before used to predict fire endurance of a typical 3D panel system prepared from concretes of variable densities yet having same compressive strength. The analysis was carried out using the software package ANSYS (2011) [12] using various element types for concrete, reinforcing steel and the polystyrene core.

**Concrete**

The concrete was modeled by using a plane element PLANE55 which can be used as an ax-symmetric ring element with a 2-D thermal conduction capability. At each node, the element has four nodes with a single degree of freedom and temperature. The element is applicable to a 2-D, transient thermal analysis or steady-state, and can also compensate for mass transport heat flow from a constant velocity field. The element should be replaced by an equivalent structural element (such as PLANE42) if the model containing the temperature element is also to be analyzed structurally. In order to define the failure criterion of the concrete, the young's elastic modulus, compressive strengths, and ultimate tensile were used, along with an estimated Poisson’s ratio of 0.20. The condition of the crack face is defined in term of shear transfer coefficient (\(\phi_t\)); a common value of which was used at 0.2 [13, 14].

**Steel**

The reinforcement bars are indistinguishable in compression and tension with idealized elastic-plastic behavior. LINK32 element was used to model the reinforcement bars which is a uniaxial element with the ability to conduct heat between its nodes. At each node point, the element has a single degree of freedom, temperature, and is applicable to a 2-D (plane or axisymmetric), steady-state or transient thermal analysis. The steel supports and loading steel plates (100\(\Phi\)50mm) were modeled using PLANE42 element.
Polystyrene

The polystyrene was modeled using PLANE55 element with a 2-D thermal conduction capability.

Data entry

The elements PLANE55 and PLANE42 are defined by 4 nodes, while the element LINK8 is defined by 2 nodes, with the four elements having 3 degrees of freedom at each node reflecting translations freedom in the x, y, and z directions. Real constant of set 1 that defines the material number was used for the concrete (PLANE55 element) which include reinforcement type, its ratio in concrete, and orientation. The cross sectional areas of the main steel and stirrups were entered for the LINK32 element as real constant sets 2.

Nonlinear solution and failure criteria

Convergence approach was used to determine the appropriate mesh density for ceiling panel as shown in Figure 5. The bond between the steel reinforcement and concrete is assumed to be perfect. Newton-Raphson equilibrium method was used with load step increment of 10 seconds and nonlinear displacement convergence criteria of 0.01 mm. The results were generated for varying surface temperatures, across - depth distribution of which was determined based on the thermal conductivity coefficients of Table 1. The mechanical properties for concrete, reinforcing steel and polystyrene listed also in Table 1 were used to generate predictions for fire endurance.

Discussion and NLFEA Results

The NLFEA was employed to estimate endurance time for a typical ceiling panel loaded to 50% of its ultimate load capacity while exposed to a fire regime of Figure 6. The result indicates that 39 minutes was needed to fail the panel, which is consistent with the time measured experimental and analytical values at about 36 minutes. The temperature-time regime generated by the NLFEA showed excellent agreement with the standard regime recommended by the ASTM. Therefore, no time correction is needed to adjust for the deviation of the temperature-time regime employed from the standard curve.

Effect of concrete density

The results of Figure 7 gives a clear idea about the significance of using lightweight concrete in casting the present 3D ceiling panels. The curves pertaining to deflection and endurance time showed a decreasing trend as the density of concrete was increased and vice versa. It indicates that it is possible to double the endurance time for a ceiling panel by just replacing ordinary weight concrete by a lightweight concrete at a unit weight of about 1500 kg/m³ while keeping the strength the same. This would cost more as higher cement

Table 1: Mechanical Properties of Materials

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Bottom concrete layer, mm</th>
<th>Density, Kg/m³</th>
<th>Tensile Stress, MPa</th>
<th>Compressive Stress, MPa</th>
<th>Modulus of Elasticity, MPa</th>
<th>Poisson’s Ratio</th>
<th>Thermal conductivity, W/mK</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>50</td>
<td>1500</td>
<td>3.1</td>
<td>25</td>
<td>12400</td>
<td>0.2</td>
<td>0.47</td>
</tr>
<tr>
<td></td>
<td>1600</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.53</td>
</tr>
<tr>
<td></td>
<td>1800</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.56</td>
</tr>
<tr>
<td></td>
<td>1900</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.68</td>
</tr>
<tr>
<td></td>
<td>2000</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.68</td>
</tr>
<tr>
<td></td>
<td>2100</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.68</td>
</tr>
<tr>
<td></td>
<td>2200</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.68</td>
</tr>
<tr>
<td></td>
<td>2250</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.68</td>
</tr>
<tr>
<td></td>
<td>2300</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.68</td>
</tr>
<tr>
<td></td>
<td>2350</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.68</td>
</tr>
<tr>
<td></td>
<td>2400</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.68</td>
</tr>
<tr>
<td></td>
<td>2450</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.68</td>
</tr>
<tr>
<td></td>
<td>2500</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.68</td>
</tr>
<tr>
<td>55</td>
<td>2300</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.68</td>
</tr>
<tr>
<td>60</td>
<td>2300</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.68</td>
</tr>
<tr>
<td>65</td>
<td>2300</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.68</td>
</tr>
<tr>
<td>70</td>
<td>2300</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.68</td>
</tr>
<tr>
<td>75</td>
<td>2300</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.68</td>
</tr>
<tr>
<td>Reinforcing Steel (3 mm)</td>
<td>7800</td>
<td>394</td>
<td>394</td>
<td>200000</td>
<td>0.3</td>
<td>43</td>
<td></td>
</tr>
<tr>
<td>Reinforcing Steel (8 mm)</td>
<td>7800</td>
<td>509</td>
<td>509</td>
<td>200000</td>
<td>0.3</td>
<td>43</td>
<td></td>
</tr>
<tr>
<td>Polystyrene</td>
<td>1.05</td>
<td>34</td>
<td>----</td>
<td>3000</td>
<td>0.34</td>
<td>0.08</td>
<td></td>
</tr>
</tbody>
</table>
content is needed to compensate for the use of the lower strength and lighter aggregates.

Effect of concrete density
Figure 7 shows the effect of concrete density on deflection and time to failure for a typical ceiling panel. Inspection of Figure 7 reveals that the time to failure increased sharply with the increase of concrete density while the concrete density had a diminutive impact on the deflection which decreased with the increase of concrete density.

Effect of concrete cover
Figure 8 shows the effect of concrete cover on the deflection and time to failure for a typical ceiling panel. Inspection of Figure 8 reveals that the time to failure increased sharply with the increase of concrete cover while the concrete cover had a diminutive impact on the deflection which decreased with the increase of concrete cover.

Effect of concrete strength
Figure 9 shows the effect of concrete strength on the deflection and time to failure for a typical ceiling panel. Inspection of Figure 9 reveals that the concrete strength had a diverse strong impact on the deflection at failure and no impact on time to failure.

Residual moment capacity versus exposure temperature
The residual moment capacity versus exposure temperature was graphically presented in Figure 10. 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 10 indicates that a temperature of 780°C is needed to cause failure corresponding to endurance times of 42 and 36 minutes before and after correction, respectively. This means that the endurance times obtained experimentally and NLFEA results are matched.
CONCLUSION
The ceiling panel, simulated as described herein, achieved a fire resistance rating of about 36 minutes, when tested and simulated as bearing assemblies with the fire applied against one surface using a mobile furnace, in accordance with ASTM Method E119 [11]. The fire test for the ceiling panel was terminated when the panel started showing uncontrolled deflection and the load set at 50% of its flexural capacity started decreasing. The NLFEA was very helpful in determining the extent by which concrete density can help extending the endurance time of a ceiling panel under fire. It indicates that it is possible to double the endurance time for a ceiling panel by just replacing ordinary weight concrete by a lightweight concrete at a unit weight of about 1500 kg/m$^3$ while keeping the strength the same. This would cost more as higher cement content is needed to compensate for the use of the lower strength and lighter aggregates. The time to failure increased sharply with the increase of concrete cover while the concrete cover had a diminutive impact on the deflection which decreased with the increase of concrete cover. The concrete strength had a diverse strong impact on the deflection at failure and no impact on time to failure.

REFERENCES


