Modeling the Response of Composite Beam-Slab Assemblies Exposed to Fire

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1.0 ABSTRACT

This paper presents the development of a three-dimensional nonlinear finite element model for evaluating the response of composite beam-slab assemblies subjected to a combination of gravity and fire loading. The behavior of typical beam-slab assemblies with different shear connection types (welded-bolted shear tab and all-bolted double-angle connection), exposed to different fire scenarios, were modeled using ANSYS. The finite element model accounts for temperature dependent thermal and mechanical properties of constituent materials, connections, and composite action. Transient time domain coupled thermal-stress analysis is performed to obtain temperature distribution and deformation response of the composite beam-slab assembly. The finite element model is validated by comparing the predicted and measured thermal and structural response parameters of three composite beam-slab assemblies tested under fire conditions. The comparisons show that the proposed model is capable of predicting the fire response of beam-slab assemblies with good accuracy. Research from the analysis clearly shows that the composite action between the beam and slab significantly enhances the fire performance of composite beam-slab assemblies. It is concluded that the proposed finite element model could be used as a feasible tool to evaluate the fire response of composite floor systems.

Keywords: Finite element analysis, Composite floor systems, Connections, Fire resistance, Unprotected steel beams.

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2.0 INTRODUCTION

Composite floor assemblies are widely used in buildings due to numerous advantages they offer over other floor systems. Such advantages include reduced construction costs, higher load-carrying capacity, and better fire resistance. The steel deck present on top of the steel framing system, acts as formwork during concrete casting, and eliminates the need for external formwork thus leading to reduced labor and construction costs. Once concrete attains its design strength, the concrete slab acts compositely with the steel beams in carrying the applied loads. Hence, the overall load carrying capacity of composite floor assemblies is enhanced through composite action.

Numerous factors affect the behavior of composite floor assemblies under fire conditions i.e. fire scenario, degree of restraint at support and material properties. To study the behavior of similar assemblies in fire conditions, a series of full scale fire tests were conducted on an eight-storey building at the Cardington large building test facility [1]. Test data indicated that the measured load carrying capacity of the composite floor assemblies under fire conditions is different when compared to the nominal capacities calculated according to the conventional flexural theory [2]. Similarly, other experimental and numerical programs indicated that steel frame buildings, with composite floors, achieve higher fire resistance than predicted using elemental level fire resistance tests [2-9].

The enhanced fire resistance of composite floor assemblies was attributed to the phenomenon of tensile membrane action, which occurs when the slab experiences large deformations under fire exposure. In general, large deflections occur in floor assemblies exposed to fire due to the temperature induced degradation of strength and stiffness in steel beams supporting the concrete slab. The development of tensile membrane action helps in transferring

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forces from fire weakened steel beams to the RC slab as well as other cooler parts of the structures. Such behavior helps enhancing the overall load carrying capacity and fire resistance of the composite assembly.

Currently there is lack of understanding on the structural behavior of composite floor assemblies specifically those built with unprotected secondary beams and are subjected to non-standard fire conditions. Further, the interaction between fire induced internal forces and composite construction is often not accounted for when analyzing composite floor. To bridge this knowledge gap, a numerical study in the form of three-dimensional finite element modeling has been conducted herein. Detailed description on the finite element model of three composite floor slab assemblies, material constitutive laws, model validation and failure modes of the composite slab are presented in this paper.

3.0 FIRE BEHAVIOR OF COMPOSITE SLABS

A review of the current literature indicates that there have been few experimental and numerical studies on the fire behavior of composite floor assemblies with unprotected secondary beams. Experimental studies on composite floor assemblies began in 1990's after a series of fire incidents in high rise buildings, such as the One Meridian Plaza [10] and the Broad-gate Phase 8 fire [11]. Post-fire evaluation of the Broadgate building indicated that the structural integrity of the composite floor slab was maintained despite the fact that it experienced temperatures above 1000°C. The behavior of beams (acting compositely with the concrete slab) was strongly influenced by the restraint to thermal expansion provided by the surrounding cooler structure, i.e. RC slab. In addition, the composite slab influenced the overall stability of the structure by distributing loads (through membrane action) from fire-damaged steel members (beams) to surrounding stronger members (beams, columns).

These fire incidents stimulated researchers to study the performance of steel framed structures under realistic gravity and fire loading, along with different end restraint conditions [1, 12]. As a result, six fire tests on a full-scale steel frame building were conducted at Cardington, UK. Of these tests, four tests (Tests 3-6) involved measuring the fire response of composite floor systems. Results from the tests showed that (a) the structural integrity of the composite system was maintained though the slab and the connected secondary beams underwent large deflections, (b) secondary beams and composite deck in the composite floor assembly experienced temperatures in excess of 900°C and 1100°C, respectively, (c) local buckling occurred near the ends of secondary beams due to the axial restraint imposed by the connecting steel members (beams, columns) and composite slab, (d) shearing failure of bolts was not observed as the presence of slab decreased the magnitude of tensile forces experienced by the connections. Based on the experimental results, it was concluded that the interaction of the heated members (beams) with the cooler structure (concrete slab), enhanced the overall performance of the structural system.

Bailey et. al. [2] designed an ambient temperature experimental program to simulate and study the development of tensile membrane action in composite slabs under fire conditions. To circumvent the complexities associated with testing and measuring the tensile membrane action at elevated temperatures, the authors have modified the room temperature test set-up to represent the behavior of composite floor slab under fire conditions. To achieve this, the steel deck below the concrete was removed before the slab was loaded, leaving the concrete and anti-crack mesh unsupported. The rationale can be summarized based on the fact that in a fire test, the steel deck could reach temperatures above 1100°C thus losing most of its original strength. Consequently, testing the composite slab at ambient temperatures with a removed deck represents similar testing conditions (as closely as possible) experienced by testing the slab at elevated temperatures. Once the deck was removed, the composite slab was gradually loaded till failure. The authors observed the development of tensile membrane action in the slab and the failure load of the slab was more than twice the load calculated using the yield line theory. Based on these observations, the authors concluded that the tensile membrane action significantly enhances the performance of composite floor systems under fire conditions.

Wellman et al. [13] carried out fire tests to evaluate the fire behavior of thin composite floor slab assemblies with different connection configurations, and fire protection schemes on the secondary beams by subjecting them to standard and non-standard fire conditions. The authors observed that no failure of shear studs occurred despite the fact that the slabs were designed for a very low composite action (25-33%). None of the connection types (shear tab, double angle) failed during the heating or cooling phases of fire exposure though they experienced significant rotations and permanent deformations. Further, the removal of fire protection from secondary beams did not significantly influence the thermal response of connected girders and connections, however it increased the mid-span deflections associated with the secondary beams and shortened the failure time of the slab. Based on these observations, the authors concluded that the composite slab, through the development of tensile membrane action, plays a significant role in transferring loads from beams to girder under fire conditions. Also, the secondary beams in composite slab assemblies can be left unprotected provided a better load transferring mechanism is available (either through increased thickness of the slab or through the presence of continuous floor systems on either side of the composite slab).

Due to the huge costs and complexities associated with conducting full-scale fire tests, several researchers have developed numerical finite element (FE) models to simulate the fire behavior of composite floor assemblies. A performance-based analytical design approach for steel beams supporting a composite floor system was developed by Bailey [6]. The method predicts the failure envelope of the composite floor system by taking into account (1) the effect of membrane action of the slab and the beam acting compositely and (2) the membrane action of the slab due to the variation in its deflected shape (which is assumed to follow changing yield-line patterns as the composite floor system is heated in a fire). The author validated the approach by comparing the prediction from this method against tests data and existing design methods. Based on the results, the author observed that evaluating the membrane action of the slab based on lower yield-line mechanism predicted conservative estimates of the composite floor systems' failure envelope. In addition, the author concluded that steel beams, within a composite floor slab panel can be left unprotected due to enhanced performance of the slab through membrane action.

Gillie et. al. [3] developed and validated a finite element subroutine (FEAST), in ABAQUS, to analyze the fire behavior of composite slab tested as part of first Cardington fire tests. The model accounted for material and geometrical nonlinearities, thermal expansion, thermal curvature and non-linear thermal gradients within the RC slab. The concrete slab was discretized using 3-D shell elements while the beams and columns were modeled using beam elements, and the connections were assumed to be perfectly rigid. Based on the results of the analysis, the authors concluded that the development of axial forces and deflections in the composite slab assembly are

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strongly influenced by the effects of thermal expansion and strength (stiffness) degradation of steel and concrete. The gravity loading had a minor effect on the fire response of the composite slab. However, this study did not consider some of the important factors such as the local buckling phenomenon in beams, true connection behavior, cracking of concrete and stress concentrations near cracked regimes in the concrete material, which influenced the behavior of the slab in the Cardington fire tests.

Lamont et. al. [9] developed a numerical model to study the structural behavior of a steel-concrete composite frame subjected to a natural fire. The study was aimed at comparing the behavior of composite slab assemblies with and without fire protected edge beams. Based on the results obtained from the numerical model, the authors concluded that (a) the behavior of the slab is dominated by the catenary action of the beam when the edge beams are unprotected as opposed to tensile membrane action when the edge beams are protected, (b) when the edge beams are unprotected, the columns displace inwards towards the end of the fire indicating a possibility of runaway collapse, (c) the magnitude of tensile mechanical strains in the concrete slab are maximum when the edge beams are fire protected, (d) protected edge beams allow the development of tensile membrane action in the slab in addition to enhancing lateral support to the columns. The effects of unprotected secondary beams, cooling phase of fire, true connection behavior on the fire response of composite slab assembly were not considered in this study.

Fike et al. [14] carried out experimental and numerical studies to evaluate the effect of steel fiber reinforced concrete (SFRC) on the fire behavior of composite floor assemblies. The authors tested one composite slab, with fire protected girder and unprotected secondary beam by subjecting it to a design fire. The design fire had a 90 minutes growth phase analogous to the ASTME119

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standard fire followed by a decay phase of 5.6°C/min. Based on the test results, the authors developed and validated a finite element model of the composite slab using SAFIR. The model incorporated the nonlinear compressive, tensile strength behavior and high-temperature stress-strain relationships of SFRC. The validated model was used to study the effect of fire exposure, load ratio, thickness of the slab and the properties of SFRC on the fire behavior of composite slab assembly. Based on the results from test and numerical studies the authors concluded that the fire resistance of the composite slab assemblies (with unprotected secondary beams) can be significantly improved through the use of SFRC. The superior tensile strength and ductility property of SFRC aids the development of tensile membrane action in the composite slab assemblies. Furthermore, it is possible to achieve the required fire resistance of composite slab assemblies with fully unprotected secondary beams in the case of SFRC slabs.

The above studies offer significant insight into the behavior of composite floor systems exposed to fire. Nevertheless, most of the previous experimental and numerical studies were conducted with protected secondary beams, idealized connection behavior and the local beam instabilities such as buckling effects in the beam were not accounted for. In addition, the effects of interacting forces (shear, tensile and compressive) at beam-slab interface on the fire response of composite slab assemblies were not studied. To overcome some of these drawbacks, a numerical study was carried out to trace the fire response of composite floor assemblies using the commercially available finite element software ANSYS [15]. Detailed description of the numerical model and results are presented in the following sections.

4.0 NUMERICAL MODEL

For evaluating the realistic fire response of a composite slab assembly, several critical parameters are to be accounted for in the analysis. These include relevant geometric and material nonlinearities, temperature dependent constitutive property relations of concrete and steel and the level of composite action.

4.1 Composite Slab Assembly – General Considerations

Geometric nonlinearities may arise from existing boundary conditions, structural members' interactions as well as presence of imperfections during fabrication. Hence, special attention should be given when modeling geometric nonlinearities by using the actual/measured dimensions, applying appropriate restraints (boundary conditions) in terms of degrees of freedoms (d.o.f). In addition, interaction between different members such as composite action between the slab and steel girders, slab and shear studs and bolt and connection interface should be accurately modeled.

Similarly, material nonlinearities need to represent those existing in a composite beam-slab assembly. Ambient temperature material properties in conjunction with high-temperature strength and stiffness reduction factors (specified in codes and standards) can be used to simulate the high-temperature effects into a well-defined material model.

4.2 Discretization Details

In order to model the fire response of composite beam-slab assemblies, different element types were used in the development of the finite element analysis of the transient coupled thermal-stress problem. Both SOLID70 and LINK33 elements with thermal capabilities were used in the thermal analysis. On the other hand, the structural analysis was carried using SOLID65, SOLID45, LINK8, CONTA174 and TARGE170. The average of total number of elements in the beam-slab assembly was 89000.

The thermal elements were converted to structural elements upon successfully completing the thermal simulation. Such conversion is needed to account for the different material (mechanical) properties and responses in the stress analysis. The conversion of elements was completed as follows:

- For the concrete material, SOLID70 (3-D Thermal Solid) is converted to the structural element SOLID65 (3D 8-node Reinforced Concrete Solid).
- For the steel girders, beams, connections, bolts and insulation material, SOLID70 (3-D Thermal Solid) element is converted to the structural element SOLID45 (3D 8-node Structural Solid).
- For the steel mesh reinforcement, LINK33 (3-D Conduction Bar) is converted to LINK8 (3D 2-node Structural bar).

In the structural analysis, the contact behavior at the bolts-connections interface was modeled using CONTA174 and TARGE170 elements. The contact regions were defined as surface-to-surface areas that only permit sliding of the adjunct faces. In addition, Coulomb's frictional law was used to govern the amount of sliding in the connected components. The frictional model uses a coefficient of friction of 0.3 to represent the amount of friction present in the connections.

A typical composite beam-slab assembly along with its discretization is shown in Fig. 1. It can be seen from Fig. 1c that a finer mesh was adopted at regions where high stress/strain intensities were anticipated, i.e. connection region, bolts and loading supports.

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4.3 High-Temperature Material Properties

As explained above, temperature-dependent thermal and mechanical material properties should be specified as part of input data into ANSYS. The mechanical properties consist of Young's modulus, stress-strain relations, thermal expansion and creep which vary with temperature. The thermal properties that are to be specified includes density, specific heat, and thermal conductivity and these properties are taken from Eurocode 2 [16] and Eurocode 3 [17].

It is well established that the strength and stiffness of concrete and steel degrade with temperature (Eurocode 2, [16]; Eurocode 3, [17]). Therefore, concrete and steel material at elevated temperatures were assumed to vary according to Eurocode 2 [16] and Eurocode 3 [17] material models.

ANSYS [15] uses Williams and Warnke [18] constitutive material model formulation to define the plastic behavior of the concrete. The model takes into account the spread of plasticity of concrete in both compression and tension regimes. The compressive plastic behavior of concrete is defined using isotropic multi-linear compressive stress–strain curve that varies with temperature. The concrete tensile stress is taken as $0.62\sqrt{(f'_c)}$ where f'_c is the compressive strength of concrete. Once the concrete reaches its tensile rupture stress, a tensile stiffness multiplier of 0.6 is used to simulate a sudden drop of the tensile stress to 60% of the initial rupture stress. Then, the drop is followed by a linearly descending response to zero stress at a strain value of six times the rupture strain.

Additional parameters identified as the open and close crack shear transfer coefficients, (β_t and β_c) are required for the concrete constitutive model. Typical shear transfer coefficients are taken as zero when there is a total loss of shear transfer (representing a smooth crack) and 1.0

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when there is no loss of shear transfer (representing a rough crack). The values of β_t and β_c in the developed model are assumed to be 0.2 and 0.7, respectively. Upon the first crack, the concrete material is treated as an isotropic elastic material, then it transforms to an orthotropic material after the initiation of cracks. Once a concrete element cracks, the modulus of elasticity is set to zero in the direction parallel to the principal tensile stress direction [15].

The steel beams, girders and connections are modeled as elasto-plastic materials using Von-Mises plasticity yielding criterion. As explained above, temperature reduction factors were used to incorporate the degradation of the steel material properties at elevated temperatures. Since the insulation material has significantly low strength and stiffness, the strength contribution from the insulation is neglected. However, these properties of insulation are accounted for in the analysis and these properties are taken from the manufacture's data sheets.

4.4 Loading and Boundary Conditions

Under fire conditions, composite slab assemblies experience both thermal (fire) and mechanical loading simultaneously. Fire tests usually start by applying mechanical loading calculated as a percentage of the service load level. Once both loading and deflection levels stabilize, then thermal loading (fire exposure) starts. The application of both thermal and mechanical loading continues until failure of the composite assembly.

Due to the symmetry of the geometry, loading, and boundary conditions that are present in beam-slab assemblies, a quarter model was analyzed. The symmetrical boundary conditions were simulated by inserting vertical restrains perpendicular to the nodes of elements present in the outer most space of the two planes of symmetry. The main advantage of building quarter models is the tremendous reduction in the total number of elements and thus computational time required for the analysis. Figure 1 shows a representative sample of the developed finite element models of the tested composite slabs.

4.5 Failure Criteria and Numerical Convergence

In the finite element analysis, failure of the composite assemblies is assumed when one or a combination of the following failure criteria is met. The failure criteria were based on the assembly reaching a critical deflection limit or when deflection rate is exceeded following the the BS-476 [18] provisions. The BS-476 [18] standard states that failure of the composite slab will occur if (a) deflection exceeds ($\frac{L}{20}$), or (b) deflection exceeds ($\frac{L}{30}$), and the deflection rate exceeds ($\frac{L^2}{9000d}$), where (L) and (d) are the defined as the unsupported length and depth of the member, respectively. Excessive deflection (those exceed ($\frac{L}{20}$)) of beams and girders are calculated to be 106.7 and 198.1 mm, respectively. While, the second deflection limit ($\frac{L}{30}$), and deflection rate ($\frac{L^2}{9000}$) were calculated to be 71.1, 132.1 mm and 2 and 5.7 mm/min for beams and girders, respectively.

Numerically, convergence in the structural simulations is governed by the Newton-Raphson equilibrium iterations [15]. Divergence of the solution can occur in the thermal analysis if the temperature variance at each node between the equilibrium iterations is more than 0.5 °C. Similarly, the structural-based finite element model would diverge if a force convergence tolerance limit value of 0.1 (typical range 0.05 to 0.2) was to be exceeded.

5.0 MODEL VALIDATION

In order to examine the validity of the above developed finite element model, the beam-slab assemblies tested by Wellman et al. [13] are selected for validation. Then, predictions of temperatures, deflections and failure modes from three finite element analysis were compared against the experimental data of tested beam-slab assemblies FA-1, FA-2 and FA-3.

5.1 Description of the Slab Assemblies Selected for Validation

The composite floor assemblies consisted of concrete slabs placed on the top of two A992 steel W12×16 girders (3.96 m long) and three A992 steel W10×15 beams (2.13 m long). A 38.1 mm deep Vulcraft 1.5VLR metal deck was installed and fixed to the top of the steel girders and beams using 76.2 mm long, 15.9 mm diameter headed shear studs. The metal deck extended the floor plan to 3.96×4.57 m. Owing to limitation of the furnace size; the area exposed to the thermal actions was limited to 3.12×2.54 m. The heating area was centered over the floor plan and included the central interior beam, the beam-to-girder shear connections, and 2.54 m length of the girders. Figure 2 shows details of the furnace facility as well as tested composite slabs.

The concrete slabs used in FA-1, FA-2 and FA-3 assemblies had measured average concrete strengths of 45, 45 and 44 MPa, respectively. The steel girders and beams were made from A992 steel. The average yield strength of steel used in the W10×15 beam was 358 MPa, and the average yield strength of the W12×16 girder was 375 MPa. In the composite slabs FA-1 and FA-2, 35 mm thick ASTM A36 steel plates were welded to the girder web and bolted to the beam web by using two 19.05 mm ASTM A325 bolts. In addition, the connections used in beam-slab assembly FA-3 were two 101.6×101.6×6.35 mm ASTM A36 angle sections attached to the girder and beam webs by using two 19.05 mm ASTM A325 bolts at each angle leg. Further details on the connections are described by Wellman et al. [13]. The thermal conductivity, dry density and compressive strength of fire insulation were 0.086 m.K, 240 kg/m^3 and 112 kPa, respectively. Table 1 shows the designation, configuration and insulation thicknesses used in each assembly.

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The composite assemblies were exposed to a fire scenario comprised of both heating and cooling phases as encountered in the fire test. In the case of slab FA-1 the cooling phase of fire had uncontrolled cooling, while in composite slabs FA-2 and FA-3 a controlled cooling phase of 12.2°C/min was used. The cooling rate was based on the Eurocode [17] time-temperature curves developed for realistic compartment fire scenarios. The thermal simulation was carried out by applying the average temperature of the fire into the nodes of the bottom and sides of the exposed girders and beams.

The applied gravity loading had a magnitude based on the service-level loading for fire design (e.g., 1.2 Dead Load + 0.5 Live Load) (AISC, [20]). The loading was selected to produce a service-level bending moment capacity equal to 46% of the moment capacity of the partially composite W10 × 15 beams. Composite slab FA-1 was analyzed under a concentrated force of 133.4 kN, while slabs FA-2 and FA-3 were analyzed under a 111.2 kN concentrated force at the beam mid-span. The gravity loadings were applied as a series of nodal forces at their corresponding locations in the developed finite element model. In all three cases, the applied loading was kept constant until failure of the composite assemblies. Failure is said to occur when the strength (capacity), deflection or rate of deflection exceeds the limiting state. More discussion on the failure criteria is provided in subsequent sections. Full details of the fire tests and results from tests can be found else were [13].

5.2 Thermal Response

In order to validate the model, predicted temperatures from analysis are compared with the measured temperatures from fire tests on composite slab assemblies. Figures 3-5 show the predicted and measured nodal temperatures at various locations of the composite slab assemblies

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FA-1 (Fig. 3), FA-2 (Fig. 4), and FA-3 (Fig. 5) during the entire range of fire exposure. As shown in Fig. 2f , the thermocouples were located at the bottom flange, web and top flange of the beams as well as the unexposed surface of the concrete slab. As expected, temperatures at the bottom flange of the beam are higher than that in the web which in turn are higher than that in the top flange. The temperatures at the unexposed surface of the concrete slab are much lower than that in the beam. It can be seen from Figs. 3-5 that the interior beam in all three composite assemblies experienced significantly high peak temperatures (700-900˚C). However, the unexposed concrete surface experienced relatively low peak temperatures (65-160˚C).

Upon the start of the cooling phase, the furnace temperature reduced at a higher rate than those at the different structural members. Finally, all components reached their peak temperatures slightly after the peak fire temperature because of their different thermal inertias and associated thermal lag. The peak temperatures were also reduced in a similar trend at the cooling phase due the reasons mentioned above.

As shown in Fig. 3, temperatures at the beam's bottom flange, web and top flange of the composite assemblies of FA-2 and FA-3 dropped to 200°C after 80-100 minutes of the start of the cooling phase (approximately 180 minutes from the beginning of the fire test). Figures 4 and 5 show the temperature-time history in the different members of floor assemblies, FA-2 and FA-3, respectively. In both assemblies, it can be seen that the temperature distribution at the interior beam level is higher than that observed at the same assembly of FA-1 and this is mainly due to lack of insulation on the interior beams of FA-2 and FA-3. A common behavior noticed in assemblies FA-2 and FA-3 is that during the cooling phase, temperatures dropped slowly to below 200°C after approximately 270 min. In addition, Figs. 3-5 show a good correlation between the

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experimental and predicted temperatures results in both heating and cooling stages of fire exposure. Hence, the developed thermal finite element models can be used with confidence as a valid tool to predict the temperatures at any location within the composite floor assembly.

5.3 Structural Response

The structural response of the beam-slab assemblies is validated by comparing the predicted deflections from the model with measured values in fire tests. As indicated above, the structural model takes into account material and geometric nonlinearities as well as composite interaction between slab-beam and beam-girder interfaces.

Figure 6 presents a comparison of predicted and measured mid-span deflection for the middle interior beams in the three composite slabs FA-1 (Fig. 6a), FA-2 (Fig. 6b), and FA-3 (Fig. 6c), respectively. Similarly, Fig. 7 shows a comparison between the mid-span deflection results for the girders of composite slabs FA-1 (Fig. 7a), FA-2 (Fig. 7b), and FA-3 (Fig. 7c), respectively. Figures 6 and 7 show that the mid-span deflection increases steadily with the fire exposure time. Heating from the bottom and side surfaces of the beams and girders produced increasing deflections as a result of the temperature induced degradation of stiffness and strength of steel. The reduction of strength and stiffness on the tension side of the steel beam causes the neutral axis to shift upward, resulting in additional deflections. It should be noted that the concrete did not experience much loss of strength or stiffness since the slab did not experience significant high temperatures.

It is clear from Figs. 6 and 7, that the deflection at the mid-span of the interior beam was larger than those observed in the side girders. This can be attributed to the fact that the middle interior beam had a smaller cross section and was fully exposed to the fire while the girders were

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made of insulated larger steel sections, and partially exposed to fire. All girders had one hour fire rating, while the middle interior beams in composite slabs FA-2 and FA-3 did not have any fire protection. This had a major effect on increasing the mid-span deflection. Fig 6a shows that the measured deflection of the middle interior beam in assembly FA-1 was significantly less than those observed in the middle interior beams of assemblies FA-2 and FA-3. Insulating the interior beam in assembly FA-1 helped in reducing the severity of the fire exposure to beams. Further, the reduction of strength and stiffness in assembly; FA-1, occurred at a slower rate than those observed in assemblies FA-2 and FA-3.

Figs. 6 and 7 show clearly that there is a good agreement between the measured and predicted deflections in all three beam-slab assemblies. Hence it can be concluded that the developed structural finite element model was able to accurately capture the performance of the beams tested under ambient and elevated temperatures.

5.4 Failure Modes

The main failure mechanism of the tested beam-slab assemblies was runaway failure of the interior steel beams followed by a runaway failure of the steel girders. It was evident that the failure mechanism was common between the three assemblies regardless of the presence of fire protection on the interior beams (FA-1) and the type of fire scenario. As explained above, steel beams and girders experienced significantly high temperatures. Such temperatures initiated a significant loss of both strength and stiffness in steel. Hence, runaway failure took place. Figs 6 and 7 show that both beams and girders experienced large deflections (runaway) prior to the start of the cooling phase. Table 2 summaries the measured and predicted failure modes of the different composite beam-slab assemblies presented herein.

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To further validate the model, predicted failure modes from analysis were compared with observations collected from the fire test. Figures 8 and 9 show a comparison between the observed and predicted failure modes. Fig. 8 shows the observed and predicted final cracks patterns in the concrete slab after the fire exposure cooling phase and also at the end of fire test. It can be seen that the finite element model successfully captured the main crack distribution along the concrete slab. In the assembly FA-1 (Fig. 8a) a major diagonal shear crack was observed in the fire test and this was predicted by the analysis as well. The main shear crack started from the middle loading point to the end support. Similarly, concrete crack predictions of in assembly FA-2 also agrees with the observed (in the test) shear cracks distributions at several locations across the concrete slab as shown in Fig. 8b. An out of plane shear failure near the main girder was observed in the FA-3 assembly. Major stress concentration resulted in an accumulation of shear cracks near the same zone (Fig. 8c); hence the predictions of the FA-3 agree well with the experimental observations.

Furthermore, Fig. 8 shows that slab assemblies FA-2 and FA-3 experienced significant amount of cracking compared to the first slab assembly FA-1. This is an indication that the absence of the fire insulation on the middle interior beam caused large amount of deflections to the fire exposed steel members of FA-2 and FA-3 assemblies. As a result, both assemblies needed to rely heavily on the composite action by redistributing the applied loading to the composite concrete slab. Thus, more cracks are present in the concrete slab of FA-2 and FA-3.

It should be noted that none of the welded-bolted shear tab or all-bolted double-angle connection failed or fractured during the fire test. Fig. 9 shows the observed and predicted state of the bolts after the fire test for assembly of FA-2. However, the presence of permanent deformations, an indication of reaching the yield stress of the bolts, was eminent. It can be seen from Figs. 8 and 9 that the developed finite element models successfully managed to predict the different failure modes of the tested beam-slab assemblies.

5.5 Fire Resistance

The measured failure times of tested assemblies FA-1, FA-2 and FA-3 was 94, 110 and 85 min, respectively. The corresponding failure time from analysis was 94, 110 and 85 minutes for slab assemblies FA-1, FA-2 and FA-3, respectively, thus indicating good agreement in failure times. From practical point of view, the fire resistance of these composite assemblies was 90 min for slab assemblies FA-1 and FA-2 and 60 min for subassembly FA-3. It should be noted that although assembly FA-1 had an insulated interior beam, still it failed prior to FA-2 because the level of gravity loading was different in both cases. Further information of the application of loading can be found elsewhere [13]. It is clear from Table 2 that the deviation between the measured and predicted failure times is considered acceptable since it was found to be less than 5% in all three tested assemblies.

6.0 CONCLUSIONS

Based on the results of the above analysis, the following conclusion can be drawn:

1. The proposed finite element model can simulate thermal and structural response of composite beam-slab assemblies subjected to realistic fire loading scenarios.
2. Fire resistance rating of 60 to 90 min can be achieved in thin composite assemblies with unprotected secondary beams.
3. Under fire scenarios, unprotected secondary beams can experience high temperatures that initiate runaway failure. However, adequate composite action, facilitated from the

concrete slab through shear studs can, enhances the overall fire performance of such beam-slab assemblies.

4. The presence of concrete slab significantly enhances the fire performance of the composite beam-slab assemblies by transferring gravity load from the fire weakened interior beams to the adjacent girders. Hence the contribution of the concrete slab should be accounted for in fire resistance analysis.
5. Connections did not have significant influence on the overall fire resistance of beam-slab assemblies. No failure occurred in connection (fin plate, angle) despite being subjected to elevated temperatures and permanent deformations.

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LIST OF TABLES

Table 1 Designation, and insulation schemes of modeled composite slabs

Slab Assembly	***Fire insulation thickness on beam (mm)***	***Fire insulation thickness on girder (mm)***	***Fire scenario***	***Connection type***
FA-1	14.3	14.3	Design fire - 1	Shear tab
FA-2	-	14.3	Design fire - 2	Shear tab
FA-3	-	14.3	Design fire - 3	Double angle

Table 2 - Measured and predicted failure times to failure of composite beam-slab assemblies

Slab Assembly	***Failure time (min)***		***% Diff.***	***Mode of Failure (Test and Model)***
	Measured	***Predicted***		
FA-1	94	98	-4.2	Runaways of tension members
FA-2	110	115	-4.5	Runaways of tension members
FA-3	85	88	-3.5	Runaways of tension members

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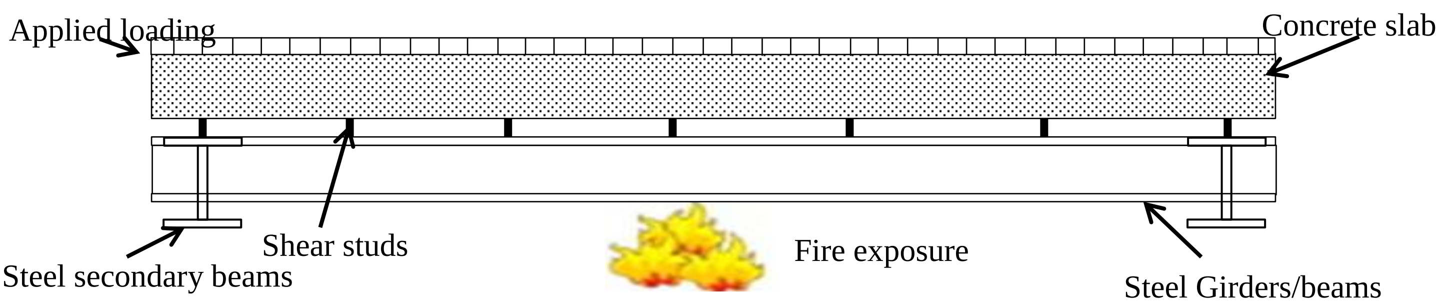
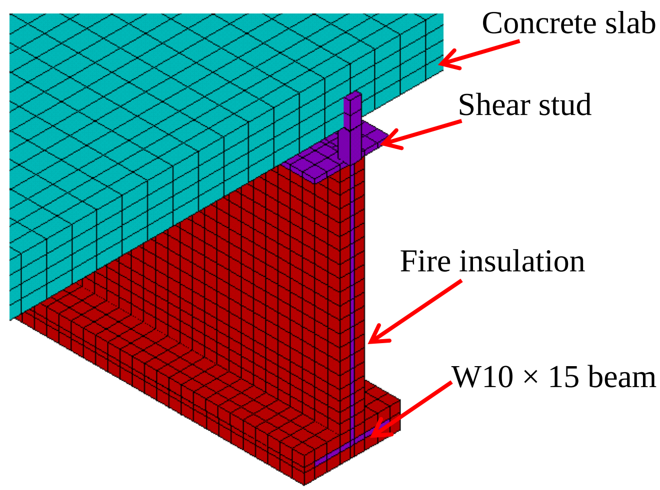
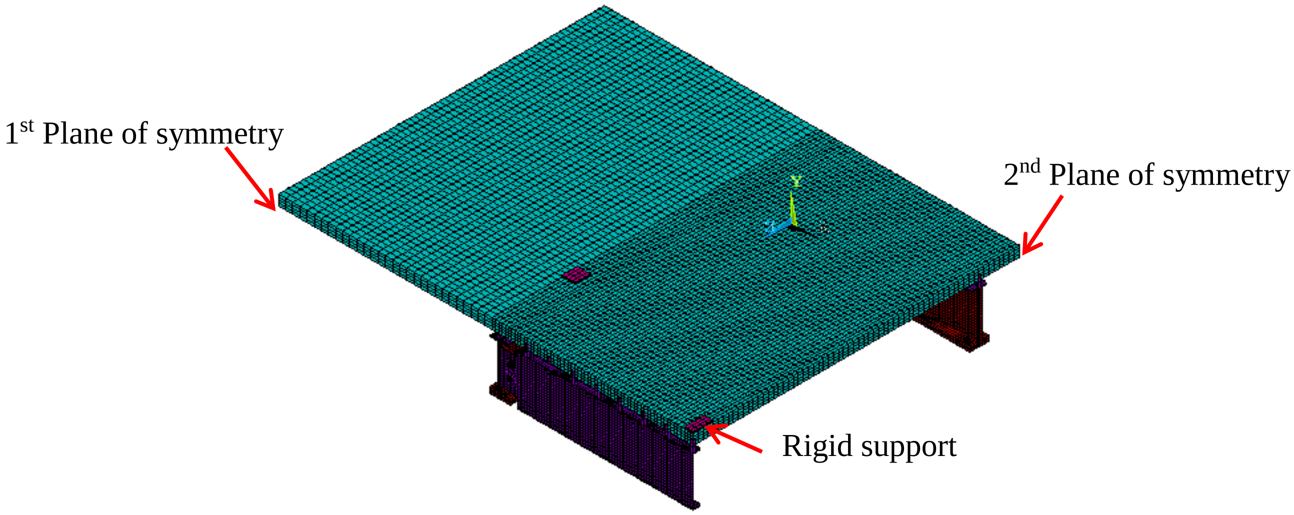


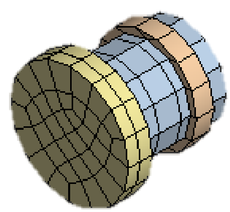
(a) Typical composite beam-slab assembly exposed to fire



(b) Details of the geometry of the model



(c) Quarter model of the assembly



(d) Discretization of bolts

Fig. 1: Discretization of typical composite slab for finite element analysis

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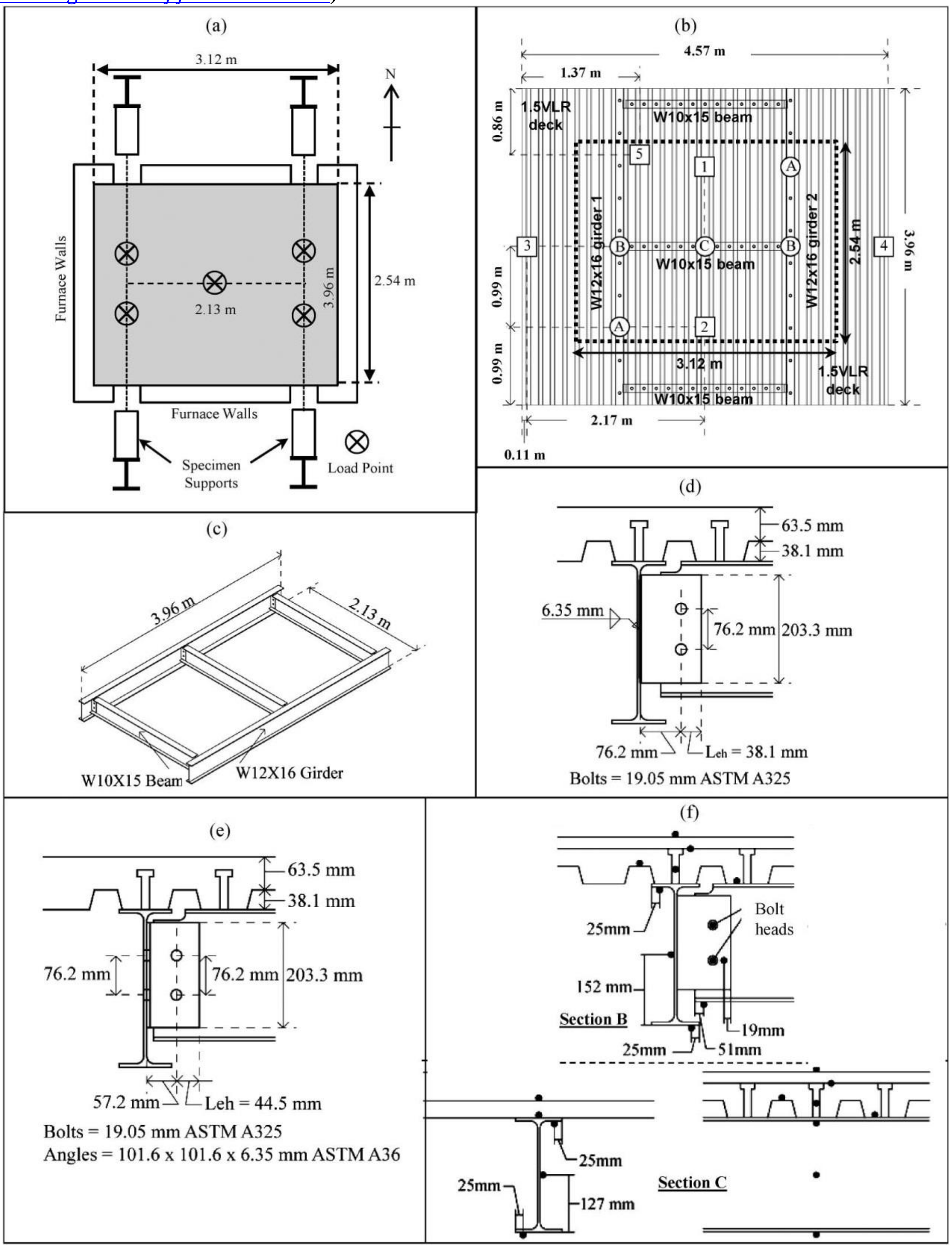

Fig. 2: Details of the furnace testing facility and tested assemblies: (a) furnace floor plan; (b) composite slab floor plan; (c) composite slab beam-girder assembly; (d) shear tab connection in composite slabs FA-1 and FA-2; (e) double-angle connection in composite slab FA-3; (f) thermocouple locations

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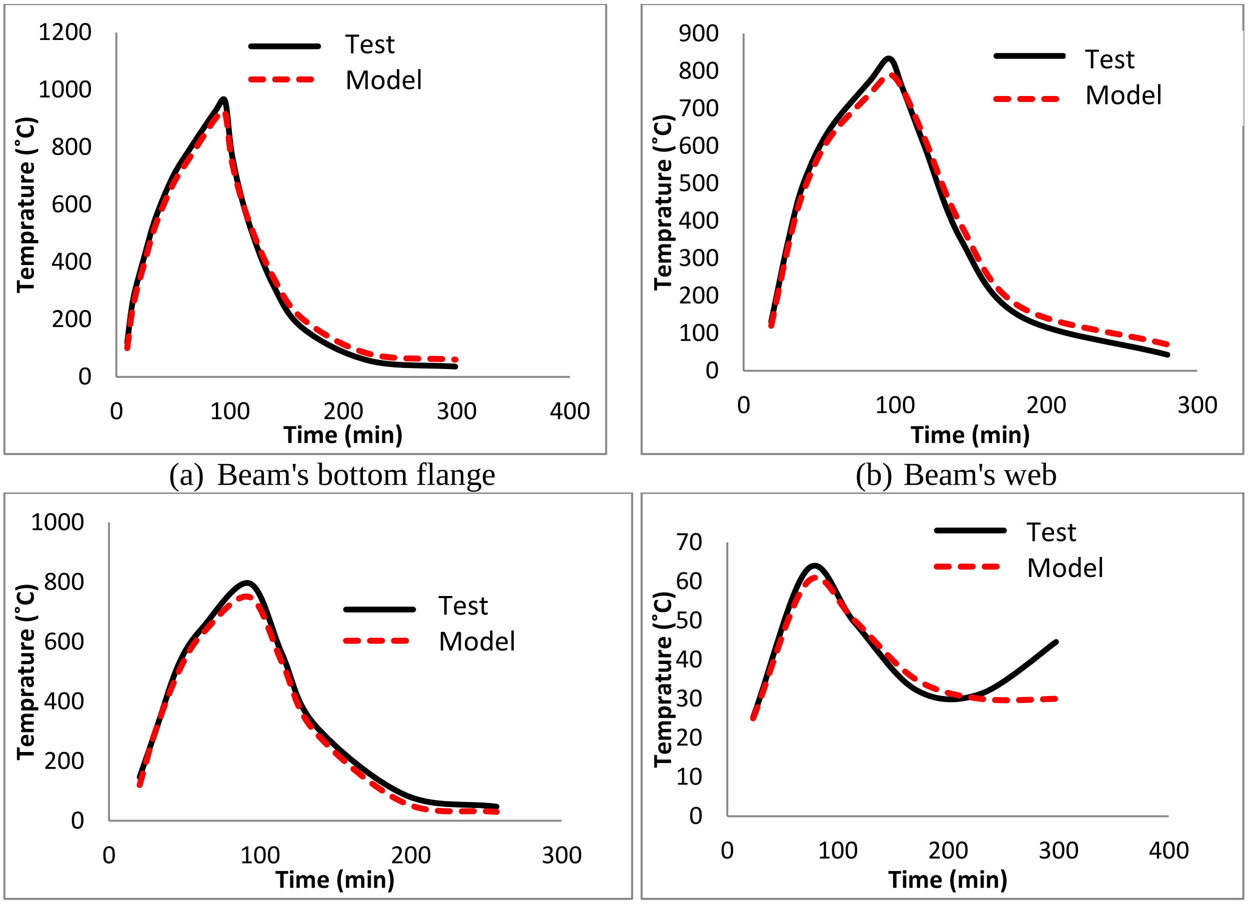
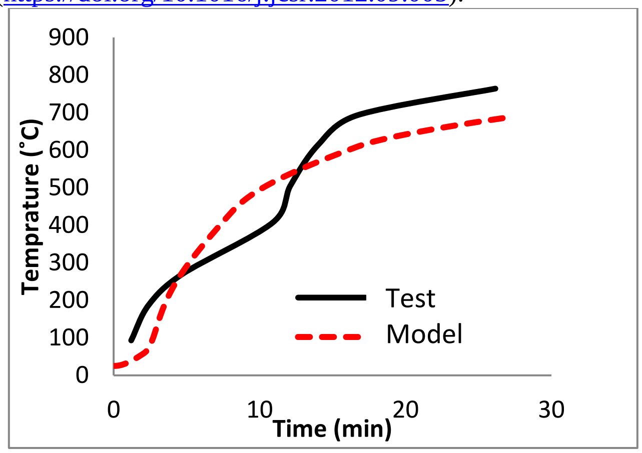
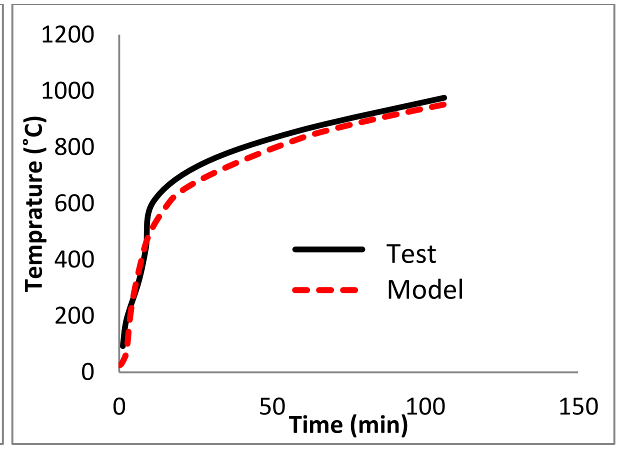


(a) Beam's bottom flange
(b) Beam's web
(c) Beam's top flange
(d) Concrete surface

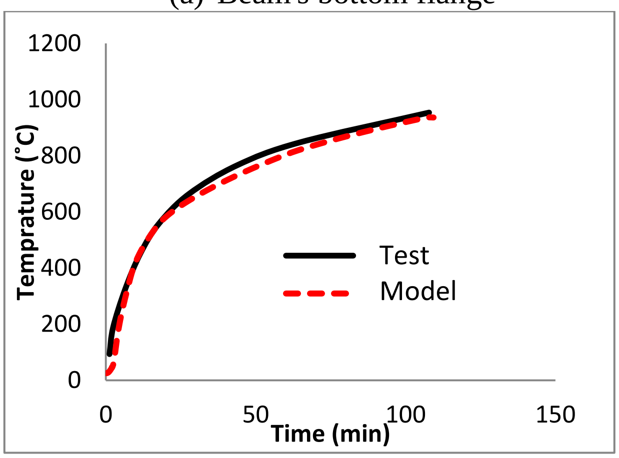
Fig. 3 Comparison of predicted and measured temperatures in beam-slab assembly FA-1



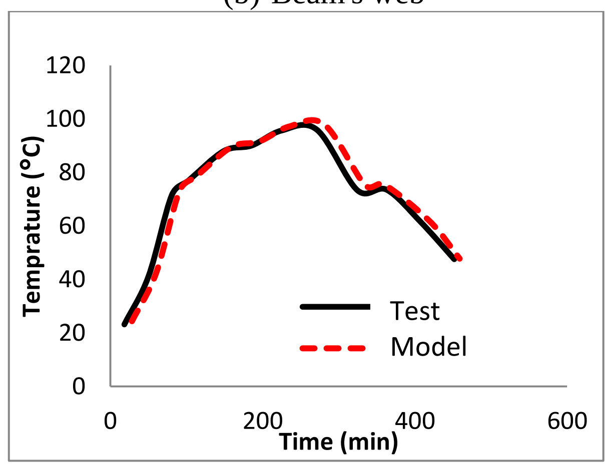
(a) Beam's bottom flange



(b) Beam's web



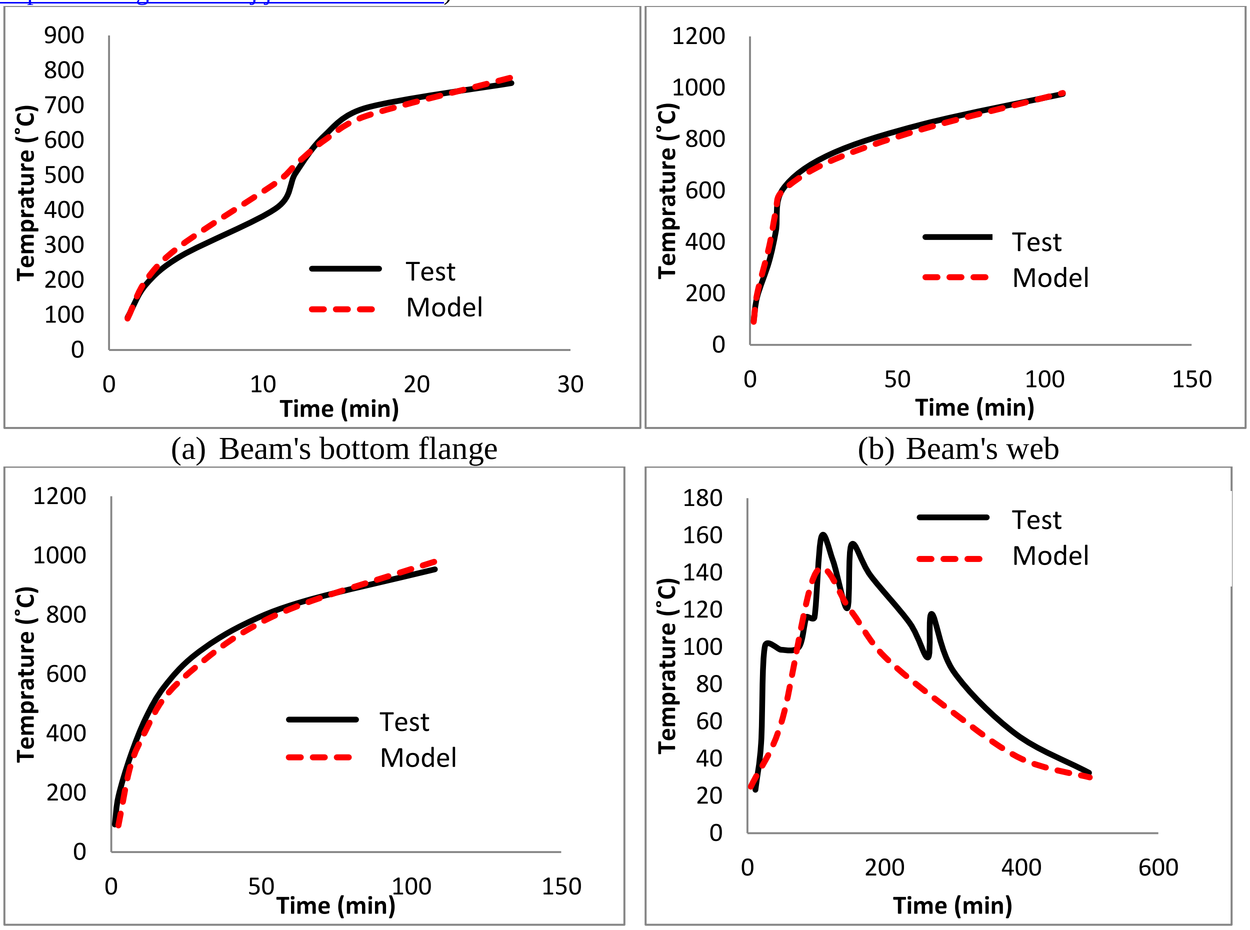
(c) Beam's top flange



(d) Concrete surface

Fig. 4 Comparison of predicted and measured temperatures in beam-slab assembly

FA-2

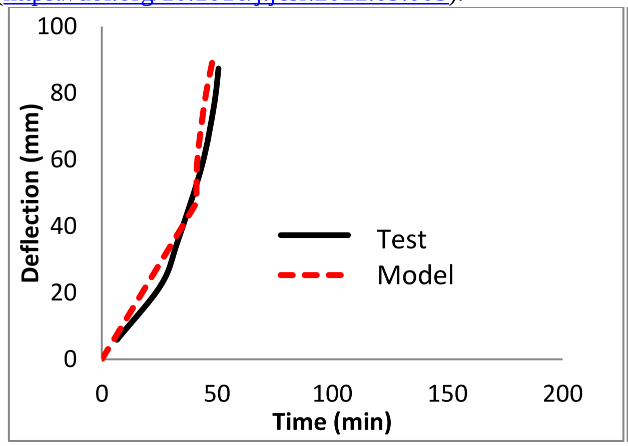


(a) Beam's bottom flange (b) Beam's web

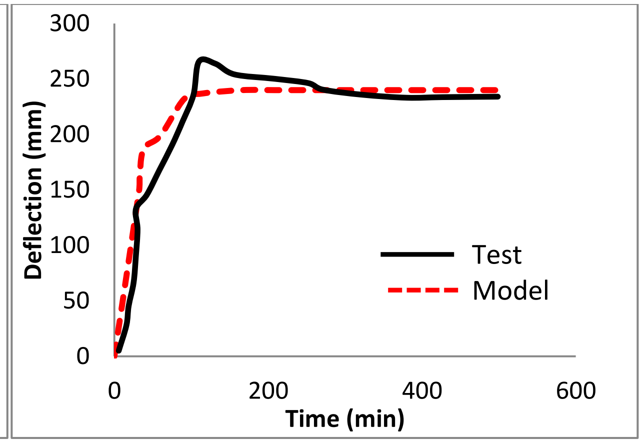
(c) Beam's top flange (d) Concrete surface

Fig. 5 Comparison of predicted and measured temperatures in beam-slab assembly

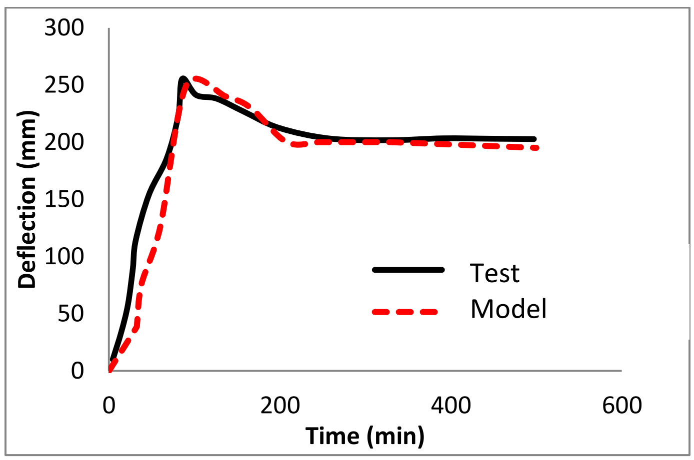
FA-3



(a) FA-1



(b) FA-2



(c) FA-3

Fig. 6 Comparison of predicted and measured mid-span deflections of interior beams of the composite beam-slab assemblies

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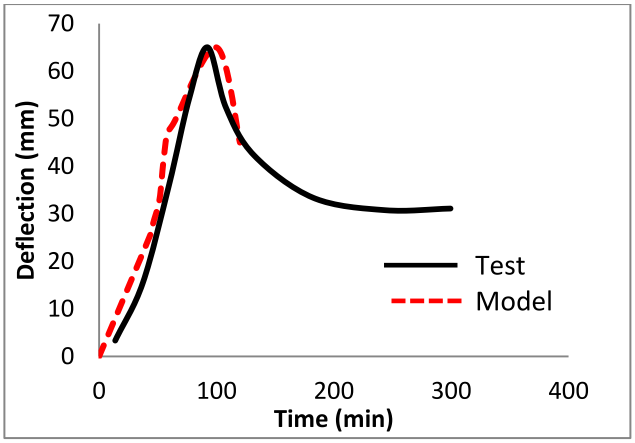
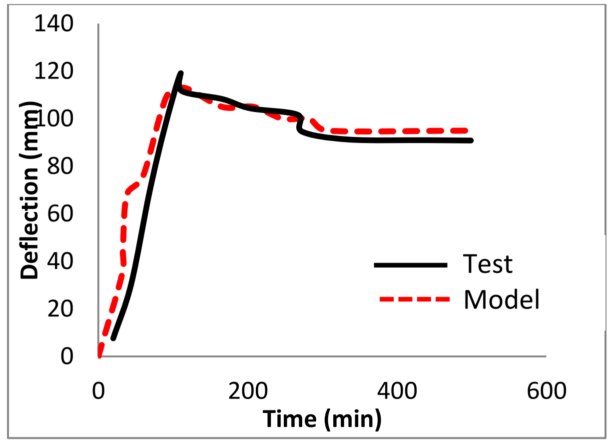
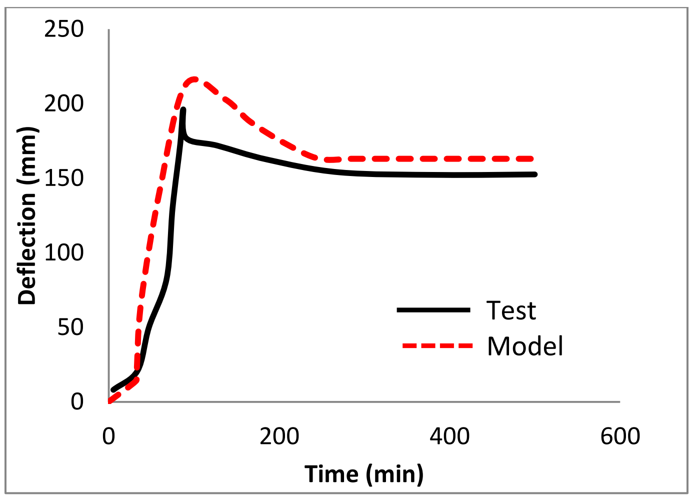


(a) FA-1

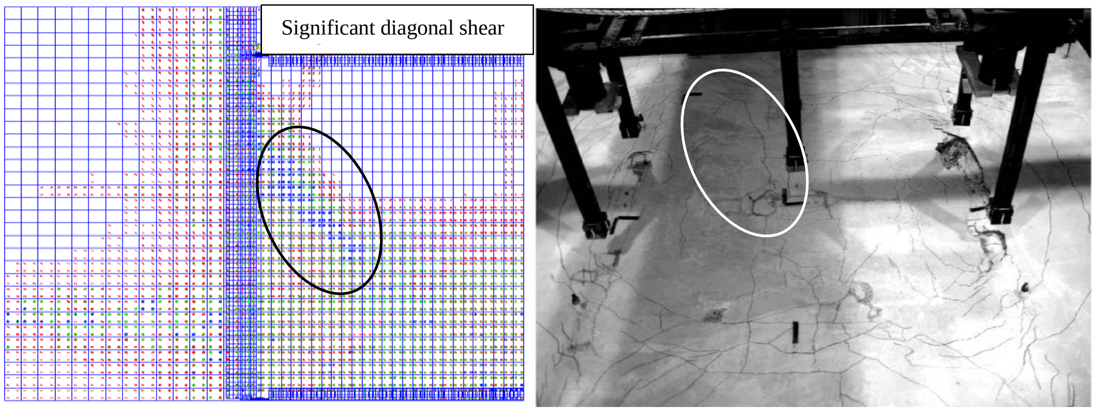


(b) FA-2

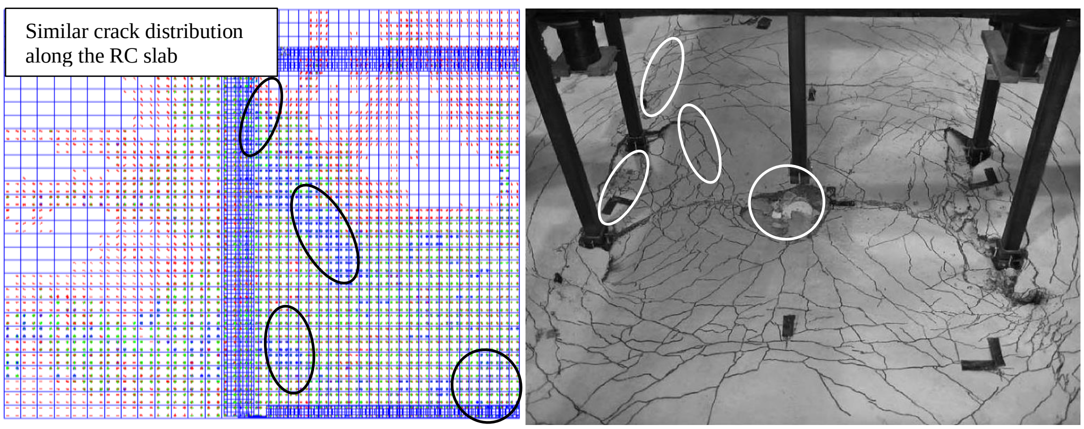


(c) FA-3

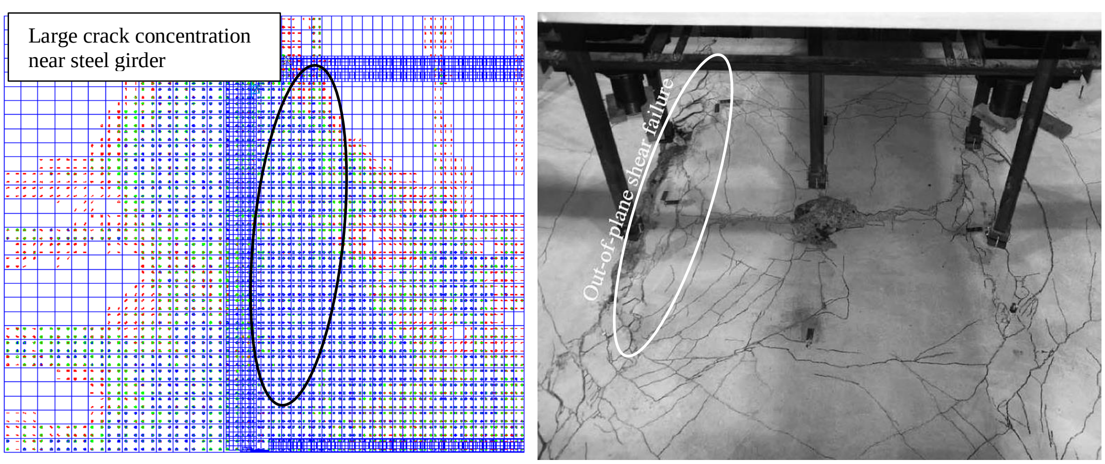
Fig. 7 Comparison of predicted and measured mid-span deflections of girders of the composite beam-slab assemblies



(a) FA-1



(b) FA-2



(c) FA-3

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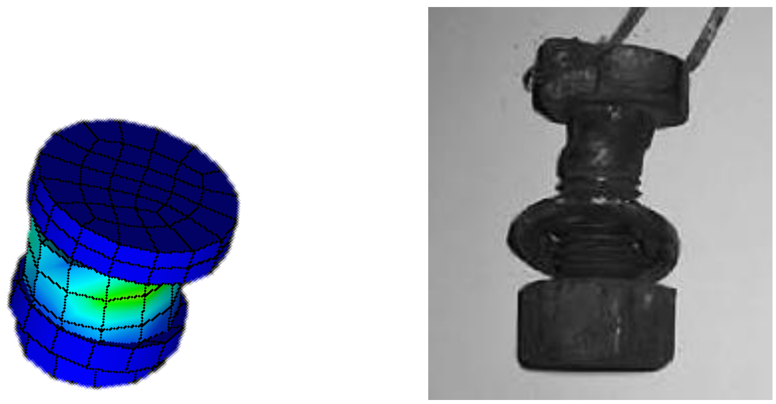
Fig. 8 Comparison of observed and predicted final crack patterns after cooling of the assemblies



(a) Predicted (model) (b) Observed (test)

Fig. 9 Comparison of state of the "bolts" after exposure to fire in beam-slab assembly FA-2