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Behavior of Composite Floor Assemblies Subject to Fire: Influence of Slab Reinforcement

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Abstract

A series of fire experiments are being conducted on the 2-story steel framed building with composite floors in the National Fire Research Laboratory. This paper presents a brief overview of experimental design and discusses some results of the first test conducted on 6.1 m by 9.1 m composite floor assembly exposed to a standard fire. The test floor slab was constructed with lightweight concrete cast on 7.6 cm deep formed steel deck units and welded wire fabric (60 mm/m^2) as the minimum shrinkage reinforcement required in the U.S. practice. The structural steel beams and girders were sprayed with a cementitious fire resistive material for a 2-hour restrained fire resistance rating. While the test floor assembly resisted mechanical loads (2.7 kN/m^2) applied using hydraulic actuators, its underside was exposed to a natural gas-fueled compartment fire that was equivalent to the standard gas temperature-time relationship. The study revealed that the test floor assembly could withstand an imposed load at a large vertical deflection on the order of span / 16, but the concrete slab failed due to limited ductility and strength prior to 2 hours in a standard fire. Simple post-test predictions were performed incorporating tensile membrane action in the floor slab to compare with the measured behavior. Limited comparisons discuss the contribution of steel decking and slab reinforcement to the load-carrying capacity of a floor assembly in fire.

Keywords

Composite floors, compartment fires, standard fires, fire resistance

1 Introduction

Fire safety design of steel-framed buildings in the United States is based on prescriptive fire-resistance ratings of individual load-bearing elements with insulation. In real fires, however, the actual fire resistance of composite floor assemblies can be largely influenced by restraints and stiffness provided by adjoining structures that often remain cool. If structural redundancy of a given composite floor assembly against fire attack is unknown, fire ratings mandated in building codes can result in high installation cost of passive fire protection systems in multistory buildings but do not necessarily guarantee increasing fire safety.

Over the last few decades, there have been active experimental studies to assess the fire resistance of composite steel frames in Europe. The Cardington fire test program [1–2], in particular, highlighted that the actual fire resistance of composite floor assemblies in a steel-framed building surpassed that of isolated floor assemblies used for standard fire testing. With the secondary load-carrying mechanisms, i.e., tensile membrane action, composite floor assemblies supported by primary structural steel frames can withstand the fire loading without collapse, thus secondary floor beams can remain unprotected. The load-carrying capability of composite floor systems observed in those tests was highly influenced by structural steel connections and steel reinforcement used in composite slabs. Connection details and high reinforcement ratios used in those tests are more acceptable in the European construction practice.

Although the methods for high-fidelity modeling have evolved, they still require validation with test data with quantified uncertainty in measurements. To date, there is lack of data describing the actual fire performance of full-scale composite steel frames designed in compliance with the United States building codes and specifications. Experimental tests studying the full-scale span and size of floor assemblies commonly used in common construction practice cannot be achieved using the standard testing method with furnaces.

Motivated by such research needs, a multi-year research project is being conducted at the National Institute of Standards and Technology (NIST) to study the fire behavior and design limit states of full-scale composite floor systems. The experiments conducted at

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NIST represent a major advance in full-scale experimentation of steel-concrete composite floor systems under fire and structural loading. The proposed test series capture a broad spectrum of geometric, design, and loading parameters relevant to current construction practice. This paper presents an overview of the first test of this program, design of the test fire, and some results of the test floor assembly in fire.

2 Test structure

Figure 1 shows a 3D rendering of the two-story steel-framed building constructed at the NIST National Fire Research Laboratory [3]. This 7.2 m tall prototype building has two bays by three bays floor plan which covers a nominal area of 18 m × 11 m. The fire test bay was located at the middle edge bay that was 6.1 m by 9.1 m in plan and 3.8 m in height above the strong floor. In this way, other surrounding bays acted as restraints to the test floor assembly while the continuity of the test floor slab was achieved. The second-story steel framing was to simulate the column continuous over two stories. Also, slab splices were designed to enable replacement of the test floor assembly new for subsequent tests. During the test, the slab continuity was acheived through steel reinforcement (No. 4 reinforcing bars) and shear connectors in the region of slab splices.

The test structure was designed according to the U.S. building codes and design specifications (e.g., [4–6]) to resist a construction live load of 0.96 kN/m^2, super imposed dead load of 0.48 kN/m^2, and a live load of 3.35 kN/m^2 at ambient temperatures, using the load combination of 1.2×dead load + 1.6×live load.

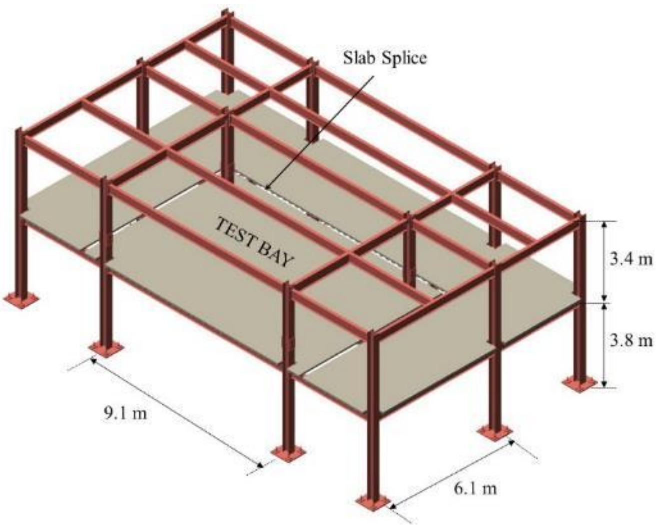


Figure 1 Test Structure-3D view

2.1 Structural frame

The structural steel gravity frames commonly used in the office buildings were designed to support composite floors. Figure 2 shows the layout of steel framing. In the test bay, a W16×31 shape was selected for three 9.1 m long beams, and a W18×35 shape was selected for the two 6.1 m long girders based on the serviceability criteria of floor vibration. All steel beams and girders were made of ASTM A992 [7] steel and were not cambered. The W12x106 shapes that were available in the laboratory as an erector set were used for columns and they were anchored to the strong floor using high strength post-tensioned bars.

Single-plate connections (shear tabs) were used for the test floor beam or girder connections. For the end connection of W16x31 beams (Figure 3), a 10 mm thick plate (ASTM A36 [8]) was bolted to the beam web using three 19 mm diameter bolts (Gr. A325 specified in the ASTM F3125 [9]) and welded either to the girder web or to a sacrificial plate on the column flange using a 6 mm fillet weld. For the girder-to-column connection (Figure 4), a 10 mm thick extended shear tab was bolted to the girder web using five 19 mm diameter bolts and welded to the web of the W12×106 column using a 6 mm fillet weld. Standard short-slot holes (21 mm in width and 25 mm in length) were used in shear tabs to allow the erection tolerances of ± 3 mm.

Sprayed fire resistive materials (SFRM) were applied to steel members exposed to fire. A medium density (ranging from 240 kg/m^3 to 350 kg/m^3) gypsum-based cementitious material was used. For a 2-hour fire resistance rating, the SFRM thickness was 17 mm for both primary beams and girders and 11 mm for the secondary beam.

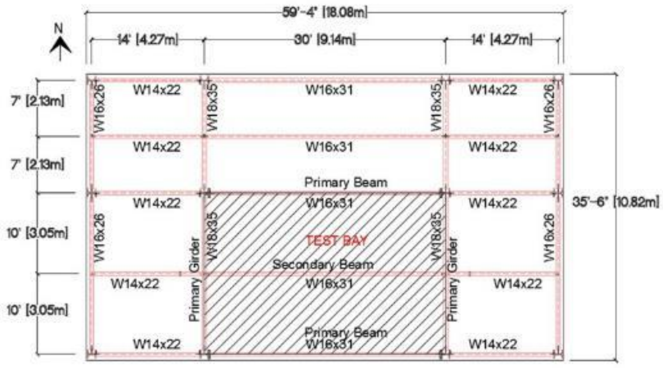


Figure 2 Steel frame layout-plan view

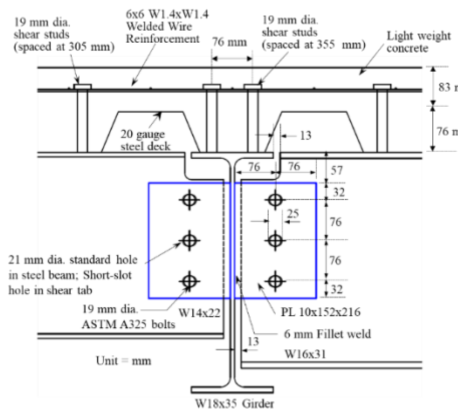


Figure 3 Shear tab connection between beam and girder

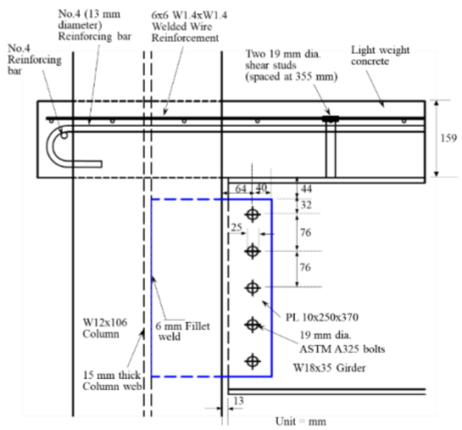


Figure 4 Extended shear tab connection between girder and perimeter column

2.2 Composite floor

The composite floor consisted of lightweight concrete slab cast on a 76 mm deep formed steel decking. Monofilament polypropylene microfibers, in the amount of 2.37 kg/m^3, were used in the concrete mix to reduce the likelihood of spalling [10]. An 83 mm thick slab above the steel deck was selected for a 2-hour fire rating with exposed steel deck. The ribs of the steel deck were oriented perpendicular to the W16x31 beams and parallel to the W18x35 girders. Figures 3 and 4 also illustrate the concrete slab details above beams and girders. Steel headed stud anchors (ASTM A29 [11]) with the diameter of 19 mm provided composite action of approximately 65% between the concrete slab and W16×31 beams or W18×35 girders.

Welded wire fabric (3.4 mm diameter cold formed plain steel wires spaced 150 mm) were placed at the mid-height of the topping concrete as required minimum area of shrinkage reinforcement specified in the design specification [6].

No.4 reinforcing bars (minimum yield strength of 414 MPa) with 180-degree hooks were placed below the heads of stud anchors welded on the edge beam to prevent premature failure in concrete slab due to lack of continuity in the south edge of the test bay.

2.3 Gravity loading setup

A total mechanical load of 125 kN was applied to the test floor assembly during the fire test. When combined with the weight of the test floor assembly and loading system, this load level was equal to 5.1 kN/m^2, the design load combination [4] with fire events (1.2×dead load + 0.5×live load). The applied load was equivalent to 20−30% of the ambient flexural capacity of the composite floor beams and 20−40% of the ambient shear capacity of shear connections.

Four hydraulic actuators mounted (in the basement) below the fire test bay were used to apply uniform loads on the test floor assembly. As shown in Figure 5, the actuator loads were transferred to purpose-built loading systems placed on top of the test floor assembly via water-cooled structural steel tubes running through the fire test bay and the strong floor. The applied loads were distributed at 24 points over the 6.1 m by 9.1 m test floor. A total of seventy-six water filled drums (2.1 kN each) were uniformly distributed over the surrounding floors, providing a gravity load of 1.2 kN/ m^2 (equivalent to 0.*5*×live load).

Figure 5 Photograph of the test floor **Source: NIST**

3 Test fire

3.1 Design objective

Standard fire testing [12] is aimed to develop uniform gas temperatures within a furnace by following the prescribed time-temperature curve. However, the size of specimens is often limited by that of furnaces available in fire testing facilities. In this test program, thus, the natural gas fuel delivery system [3] was used instead.

The key elements considered for design of the test fire include room geometry, ventilation, and fuel load. The fire exposure to the test floor assembly is designed to (i) be confined within a compartment, allow flame leakage through openings with restricted sizes and locations; (ii) produce the uniform gas temperatures in the upper layer of the compartment below the test floor assembly; and (iii) be controllable and repeatable with the use of existing natural gas fueled burners in the lab.

3.2 Fire confinement & burners

In order to confine a fire below the test floor assembly, the fire test compartment (10 m x 6.9 m x 3.8 m) was constructed with 3 m tall light-gauge steel walls protected by 50 mm thick type-C gypsum boards (Figure 6a). The upper portion of the test compartment (0.8 m in height) was confined by 50 mm thick ceramic blankets in order to allow deflection of the floor assembly without damaging the compartment wall during the fire test or generating restraint from the wall. The main ventilation opening (6 m wide, 1.5 m high) was in the south wall, while the slit on the opposite (north) wall was designed for air intake only. Four 1 m by 1.5 m natural gas burners, rated up to 4MW each, were distributed on the floor of the test compartment (Figure 5). The test fire conditions, ventilation openings and location of burners were designed using numerical simulations and mockup fire tests presented in Zhang et al [13] and briefly summarized herein.

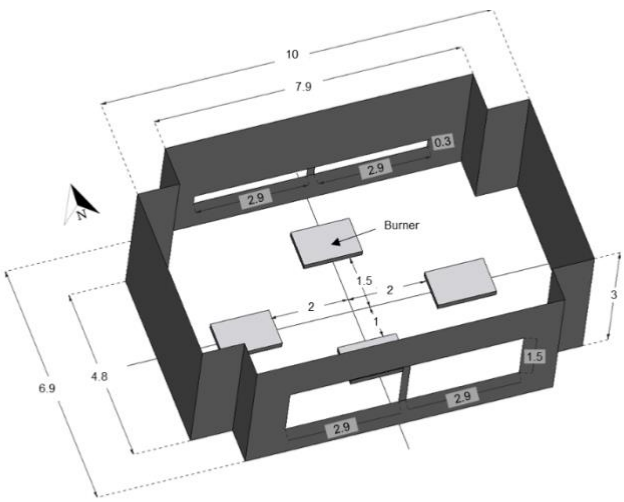
3.2.1 Fire Load

The maximum value of heat release rate (HRR) considered in this study was determined based upon knowledge gained in previous full-scale experiments [2, 14–15] with a similar compartment size. Surveys [Vassart, 2014] have found that the fuel loads in commercial and public spaces vary greatly. A typical office contains in the range of 420 to 655 MJ/m^2 of combustible material; a shopping center is in the range of 600 to 936 MJ/m^2; and a library can have fuel loads up to 2340 MJ/m^2. Thus, the natural gas burners used in this test program was designed to create a fuel load density (q_f)of approximately 1200 MJ/m^2 for a two-hour or longer fire exposure to attain significant structural failure. The peak intensity of the fire on a volumetric basis is 38 kW/m^3.

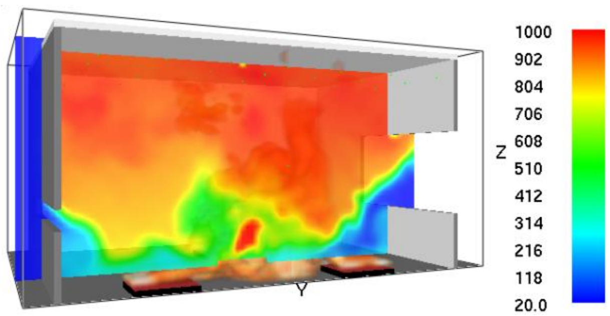
3.2.2 Opening factor

The test fire considered in this study is designed to maximize the upper layer temperature, to minimize the level of smoke, and to avoid excess fuel feeding a fire external to the bay. The ventilation is controlled by the total opening area, A_o, and the height of the opening, H_o. When scaled with the room volume, the opening area in this study, 0.034/m, is similar to the opening area/volume (A_o/V) used in the over-ventilated Cardington fire. The corresponding opening factor (F_o) is 0.045 m$^{1/2}$ where $F_o = A_o H_o^{1/2} A_t^{-1}$ and A_t(= 70 m^2) is the area of internal compartment boundaries including openings.

Once the initial values of fire load density (q_f) and opening factor (F_o) for the test fire were chosen, a zone model, CFAST [16], was used to compute the upper layer gas temperature varied with characteristics of openings (quantity, geometry, and distribution). With the opening geometry shown in Figure 6, CFAST results showed that the proposed *HRR*, 10MW was sufficient to produce the uniform upper layer temperature in the range of 1000 °C to 1200 °C. In addition, a computational fluid dynamics (CFD) model developed using Fire Dynamics Simulator (FDS) [17] indicated that with the proposed opening factor of 0.045 $m^{1/2}$, a fire appeared to be over-ventilated while well confined in the compartment. The predicted gas temperatures were deemed uniform (Figure 6b).



(a)



(b)

Figure 6 (a) Test compartment geometry and location of burners (unit = m), (b) predicted gas temperatures (unit = °C) using FDS model [18]

4 Results and Discussion

Almost 500 data channels were used to characterize the imposed fire and loading conditions as well as thermal and structural responses of the test floor assembly. This paper only focuses on gas temperature measured in the upper layer of the test compartment, the average temperatures in the test floor slab and the secondary beam, and slab deflections at selected locations. A full test report which contains the entire test data will be published in 2020. Refer to NIST Reports [10, 19] for the uncertainty in measurements reported in this paper.

4.1 Upper layer gas temperature

Figure 7 shows the HRR measured at the burner and the average upper layer gas temperature measured at 30.5 cm below the exposed surface of the test floor. The average upper layer gas temperature reached 950 °C at 60 min and a peak value of 1050 °C when the fire was extinguished at 107 min. At about 102 min, the fire was temporarily disrupted due to a network problem of data acquision system. As shown, measured upper layer gas temperatures were compared well with those predicted using FDS as well as the ASTM E119 [12] and ISO 834 [19] time-temperature curves.

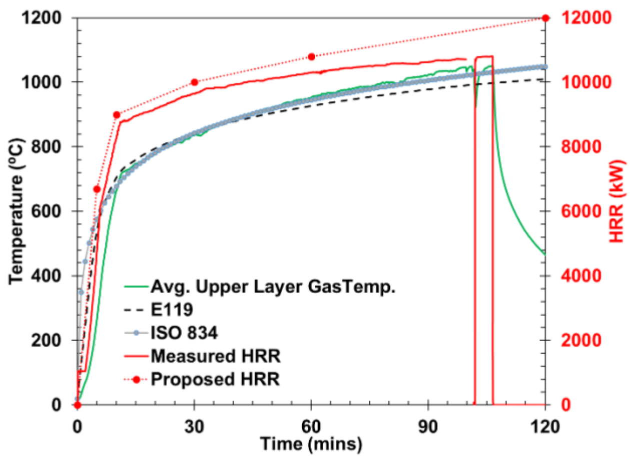


Figure 7 Upper layer gas temperature and burner HRR

4.2 Thermal response

Figure 8 shows locations of the embedded thermocouples used to measure the concrete temperature. Figure 9 shows the average temperatures measured at various depths within the concrete. The bottom concrete reached 700 °C to 800 °C at 107 mins. The error bars on these graphs represent the maximum standard deviation of measured temperatures across the test floor during heating and cooling phases. The temperature at mid depth of the topping concrete, where the welded wire fabric was located, varied due to the difference in concrete mass between the deep and the shallow sections. The temperature difference between these two sections was as high as 200 °C during the fire exposure. The temperature of the upper concrete continued to increase after the fire was extinguished. Figure 10 shows the temperature of the SFRM-coated secondary beam at midspan. Total expanded uncertainty (with a 95 % confidence interval) in measurements of the gas-phase, steel, and concrete temperatures are 8 % at 1110 °C, 4 % at 970 °C, and 6 % at 310 °C, respectively.

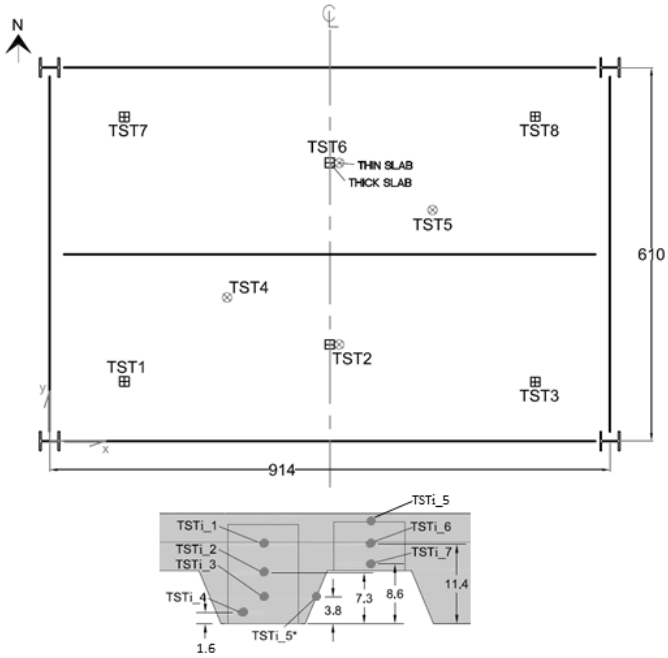
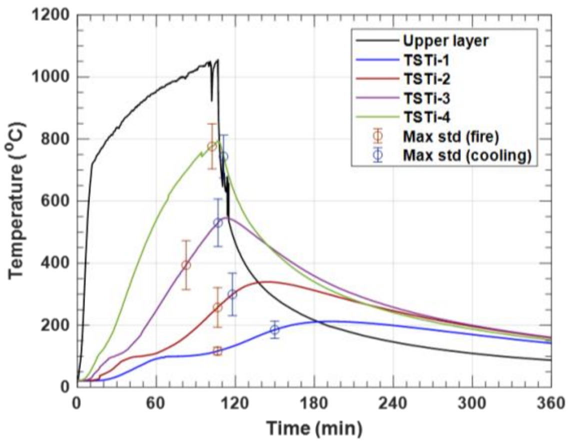
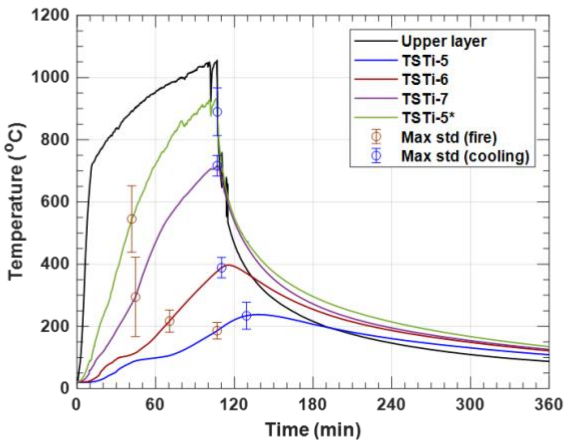


Figure 8 Locations of embedded thermocouples in the concrete slab (unit = cm)



(a)



(b)

Figure 9 Concrete slab temperature (a) through deep section, (b) through shallow section

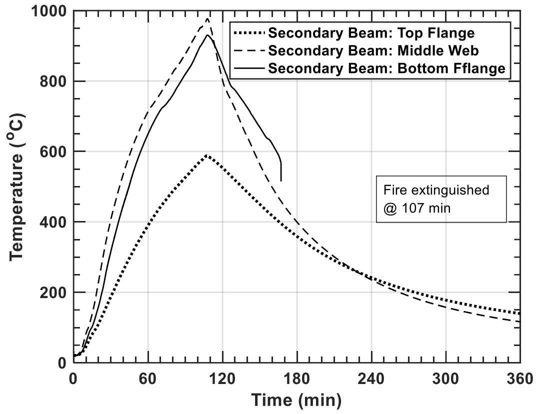


Figure 10 Temperature of the secondary (W16x31) beam at midspan

4.3 Slab deflection

Figure 11 shows locations of the vertical displacement measurements of the slab and Figure 12 shows the vertical displacements measured during the heating and cooling phases. Throughout the fire test, the south side of the slab (VD8 and VD10) deflected more than the north side (VD1 and VD3) due to the absence of the slab continuity in the south side. The displacements measured at VD5 was similar to that of VD8 until 60 min and became greater afterwards possibly due to local buckling at the end of the secondary beam and concrete fracture along the west and east girders. The mid-panel deflection (VD5) increased to 460 mm (equivalent to the ratio of L/20 where L = 9.1 m) at 92 min and to its peak value of 580 mm (L/16) without collapse when the fire was extinguished around 107 min. The total expanded uncertainty (with a 95 % confidence interval) in measurements of the vertical displacements is estimated 1 % at 580 mm.

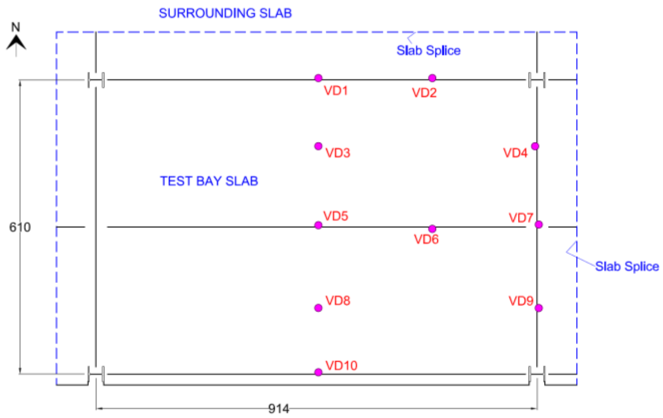


Figure 11 Location of vertical displacement measurements on the concrete slab

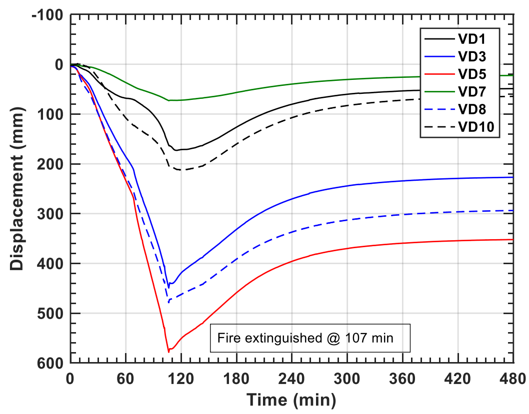


Figure 12 Vertical displacement of the test floor slab along the slab centrelines

4.4 Concrete fracture failure

Figure 13 shows photographs of the test floor slab after cooling, including concrete fracture along the perimeter of the test bay (west and east girders as well as the north primary beam) and along the secondary beam. Before reaching 30 min in fire, concrete cracked along the perimeter of the test bay. The cracks along the west and east side of the test floor were formed in the shallow section of the trapezoidal deck in the test bay next to the 6.1 m long girders. This is the critical section subjected to a large vertical shear and a hogging moment due to the mechanical loads on the test floor. The north longitudinal cracking was developed outside of the column grid. This cracking was developed between the beam centerline and the slab-splice plate. There was no crack along the south beam which had a free slab edge. Most of welded wire reinforcement across the perimeter cracks ruptured except for the interior column regions. Neither concrete cracking nor ruptures of reinforcement was observed in the slab splices used in this experiment.

Along the secondary beam, tensile concrete fracture started developing around the midspan at approximately 70 min and propagated towards the west and east directions. The steel deck locally fractured near the east end of this longitudinal crack (Figure 13), and small flames were visible on top of the slab until the fire was extinguished at 107 min. No collapse failure occurred, whereas most of welded wire reinforcement in the longitudinal crack ruptured.

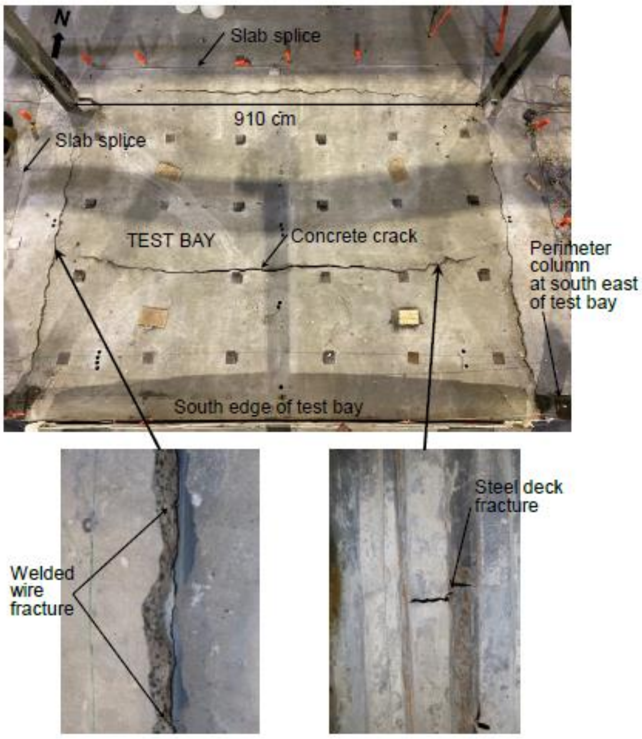


Figure 13 Concrete crack (after cooling) **Source: NIST**

4.5 Discussion on slab reinforcement

Steel reinforcement used in the test floor slab met the minimum shrinkage requirement of 59 mm^2/m specified in the U.S. design provisions [6], approximately equal to 0.075% of the concrete area above steel decking. However, this amount of slab reinforcement was not sufficient to resist tensile forces developed in the slab as the mid-panel deflection increased to 300 mm (approximately L/30) around 70 min of the fire exposure, prior to the required fire resistance rating of 2 hours.

This requirement of slab reinforcement used in a composite floor slab was much smaller than that used in European and New Zealand construction practice or their fire testing (Table 1). The shrinkage steel requirement for reinforced concrete structure [19] is 2.4 times greater than that used in composite floors with steel decking supported by steel floor beams.

In Test 7 [2] of the Cardington test program conducted on a 9 m × 6 m edge bay, similar to the test floor used in the present study, the mid-panel displacement increased to a maximum value of 900 mm (L/10) without collapse. Although the secondary beam remained unprotected and lost much of its flexural capacity during the test, the composite floor with the reinforcement of 142 mm^2/m resisted the imposed load (equivalent to 56% of live loads at ambient temperatures) through tensile membrane action.

Table 1 Shrinkage steel requirements in code provisions and reinforcement used in the present study and the Cardington test.

Reference	Reinforcement ratio (%)	Minimum area/length (mm^2/m)	Spacing (mm)
SDI [6] & Present study	0.075	59	150 typ.
ACI 318 [20]	0.18	165	305 typ.
Eurocode 4 [21]	0.1	80	–
Cardington [2]	0.17	142	200

The Slab Panel Method (SPM)[3], the Steel Construction New Zealand (SCNZ) design software program developed by Clifton et al. [22], was used to predict the behavior of the floor assembly tested in this study. This program incorporates an updated tensile membrane model from that proposed by Bailey [23-24] and the yield line theory by Park [25]. This program predicts the load carrying capacity of a heated slab panel including the secondary beam(s) using the principle of virtual work and also allows for inclusion of the edge reinforcement depending on its ductility. Developed as a design tool, it was used iteratively in this study to generate the results shown.

Figure 14 shows the measured bottom flange temperature of the secondary beam as a function of the mid-panel displacement compared with that predicted using the SPM. The temperature of the secondary beam beyond the fire exposure time longer than 107 min was estimated based on the linear regression of temperature data recorded between 60 min and 107 min. The profiled steel decking was included as a second layer of slab reinforcement (1182 mm^2/m) in addition to welded wire fabric (59 mm^2/m). As shown, the SPM conservatively estimates the mid-panel displacement at secondary beam bottom flange temperature under 930 °C. The SPM prediction implies that although the fire was extinguished in the test

[3] *A certain commercial entity identified herein is not intended to imply recommendation or endorsement by NIST, nor is it intended to imply that this entity is necessarily the best available for the purpose of this study.*

at 930 °C, the test floor assembly could resist at higher temperature without collapsing. The predicted maximum temperature at which the test floor assembly failed to resist the applied load was 1025 °C and the corresponding displacement was 627 mm (L/14.5).

Figure 15 shows a further comparison of demand-to-capacity ratio (DCR) as a function of the bottom flange temperature of a secondary beam, estimated using the SPM. The values of DCR greater than 1 imply failure of the floor assembly. The results presented are only applicable to a composite floor assembly similar to the tested specimen. Two different amounts of embedded reinforcement were compared, including 59 mm^2/m with cold-formed plain wires spaced at 150 mm, the same as the welded wire fabric used in the present study, and 230 mm^2/m with No. 3 hot-rolled reinforcing bars (diameter = 9.5 mm) spaced at 305 mm, which could provide larger ductility and satisfies the minimum reinforcement requirement for reinforced concrete (Table 1). The results suggest that a floor assembly with more reinforcement with larger ductility exhibit a superior performance with increasing temperature.

At elevated temperatures, the decking is not typically considered in routine design using the SPM because of the conservative assumption that there is no force transfer between adjacent sheets of decking within the slab panel. However, in this study, the decking was assumed to be continuous across the slab panel since the steel decking exhibited only local failure and maintained its overall integrity based on the post-test inspections. For comparison purposes, hence, the decking was included as reinforcement in the direction that trapezoidal deck ribs ran continuous. However, the influence of steel decking and slab reinforcement on the integrity of a floor system in fire will need to be verified through further studies.

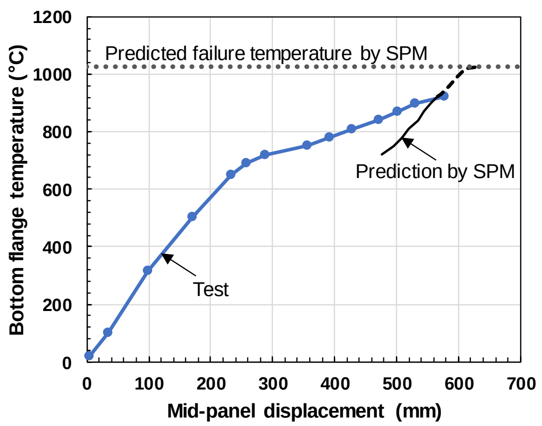


Figure 14 A comparison of the test result with the post-test prediction by SPM

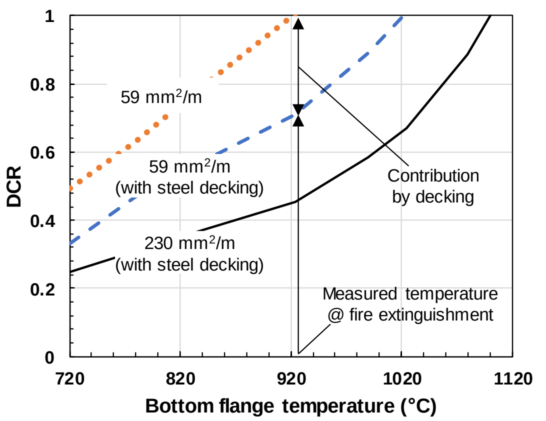


Figure 15 Estimated DCR ratio with respect to bottom flange temperature using SPM

5 Summary and Conclusions

This paper presents a brief overview of the first experiment conducted on a 6.1 m by 9.1 m composite floor assembly of a two-story test building constructed at NIST. Some test results are presented including upper layer gas temperature within the test compartment and responses of the floor assembly to a fire. Also, this paper discusses the influence of slab reinforcement on the behavior and load-bearing capacity of the tested floor assembly.

The experiments showed that the heated floor assembly continuously deflected downward with increasing temperatures. In the early phase of the fire loading, concrete fractured along the perimeter of the test bay. Then, a center longitudinal crack appeared on the floor slab around 70 min due to excessive tension. The mid-panel displacement reached about 580 mm (equivalent to L/16) without collapsing. Fire and mechanical loading were removed around 107 min. The area and ductility of reinforcement used in the test floor slab, the minimum value required in the U.S. construction practice, was not sufficient to delay tensile failure in the slab before 2 hours in a standard fire. According to the post-test prediction using the SCNZ's SPM, the test floor assembly could resist the imposed load slightly longer than 107 min or at higher temperatures due to additional load-carrying mechanism through steel decking although a further study is needed. The use of increased amount of ductile reinforcement (around 0.2-0.3% reinforcement ratio) could significantly improve the fire resistance of a composite floor assembly.

The test data and results partially presented herein can serve as technical information to better understand the behavior of a full-scale composite floor system for improvement of fire safety design provisions and for validation of predictive models used in structural fire engineering.

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