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FIRE RESILIENCE OF STEEL-CONCRETE COMPOSITE FLOOR SYSTEMS

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ABSTRACT

This paper presents the results of compartment fire experiments conducted on 9.1 m × 6.1 m steel-concrete composite floors in a full-scale, two-story, two-bays by three-bays steel gravity frame building to investigate the fire resilience of these widely used floor systems. A total of three composite floor specimens with varying slab reinforcement and fire protection schemes for the secondary beam were tested. The first experiment (Test #1) was designed to achieve a 2-hour fire resistance rating per current U.S. practice and create baseline data for the behaviour of the building. Test #2 and Test #3 were conducted to study the effect of enhanced slab reinforcement with larger area and ductility as well as unprotected secondary beams on the fire resilience of the composite floor systems. The 9.1 m × 6.1 m test floors were exposed to a compartment fire using natural gas fuelled burners while subjected to hydraulically applied gravity loads. The compartment test fire created upper-layer gas temperatures like those in standard fire resistance tests. The Test #1 specimen with steel wire reinforcement of 60 mm^2/m width exhibited mid-panel slab integrity failure at 70 mins of fire exposure. Test #2 and Test #3 showed that the use of deformed steel bars (230 mm^2/m) for the slab reinforcement maintained the structural integrity of the tested slab for more than two hours even with an unprotected secondary beam.

Keywords: Composite floor; compartment fire; steel frame building; large-scale fire experiment

1 INTRODUCTION

Fire safety design of composite floors in the United States (U.S.) is primarily based on prescriptive fire-resistance requirements determined using a standard fire testing method [1]. Standard fire testing of individual building elements does not evaluate the system-level fire resistance of full-scale composite floor assemblies with the realistic restraints from the surrounding structural assemblies. There is a lack of experimental data quantifying the fire performance of full-scale composite steel frames designed in accordance with U.S. building codes and specifications.

Over the last few decades, significant experimental research has been conducted in Europe to evaluate the fire resistance of full-scale composite floor systems [2-5]. In those studies, the fire performance of composite floor systems was found to be superior compared to the standard testing criteria used to determine fire resistance rating of composite beams. These tests showed that the fire performance of composite floors was influenced by the development of tensile membrane action of the reinforced concrete

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floor slab at large vertical displacements. The steel reinforcement ratio (equal to the area of steel reinforcement divided by the area of topping concrete) used in the floor specimens was 0.2 % to 0.3 %, which was found to be sufficient to meet the prescriptive fire resistance rating without fireproofing insulation of the secondary beams. In the U.S., a minimum shrinkage steel reinforcement ratio of 0.075 % is permitted for composite floor construction [6] and for the purpose of standard fire testing [7]. This reinforcement ratio is considerably lower than the steel reinforcement used in the European practice [8] as well as in the European experiments detailed above [2-5]. It is noteworthy that the fire resistance design in the U.S. does not consider the slab reinforcement as a factor to determine fire resistance.

Recently, the National Institute of Standards and Technology (NIST) conducted large-scale fire experiments using a two-story steel gravity frame designed and constructed following the U.S. practice. The fire experiments were designed to evaluate the system-level fire resistance, structural performance, and failure modes of the full-scale composite floor assemblies with the most realistic restraints from the surrounding structural assemblies. This test program included three experiments. The first experiment (Test #1) [9, 11] provided baseline data for the fire resistance and behaviour of the full-scale composite floor system designed to achieve a 2-hour fire resistance rating according to U.S. practice. The second experiment (Test #2) [12] was conducted to study fire resilience of the composite floor system with the enhanced slab reinforcement detailed to allow tensile membrane action to develop [10]. The third experiment (Test #3) was conducted to evaluate the effect of the secondary beam without fire protection when combined with the enhanced slab reinforcement. In the tests, all other conditions remained similar, including the specimen geometry, beam-end connections, test fire curves, and imposed gravity loads. Primary focus of this paper is to present comparisons of the test results from Test #1 and Test #3, evaluating the improved fire resilience of the composite floor with the enhanced slab reinforcement without fire protection on the secondary beam.

2 TEST PROGRAM

2.1 Test structure

Figure 1a shows the two-story steel gravity frame, two by three bays in plan, used for the fire tests. Composite floors were constructed on the first floor, whereas the second floor steel framing was erected to mimic continuous steel columns. The same wide-flange steel shapes were used for both first and second story steel framing as shown in Figure 1a. The baseplates of the columns were anchored to the laboratory strong floor with post-tensioned high-strength steel bars. Refer to the American Institute of Steel Construction (AISC) Steel Construction Manual [13] for the dimensions of the W-shapes. The test floor (687 cm × 1008 cm in plan) was situated in the south middle bay of the test building, with the footprint slightly greater than the column grid (610 cm × 914 cm). The fire test bay included 9.1 m long beams with W16×31 shapes and 6.1 m long girders with W18×35 shapes as shown in Figure 1a. The selection of these steel beams was governed by floor vibration criteria [14], which required larger sizes of steel beams compared to the beams designed only for flexural strength at ambient temperature.

The test floor assembly consisted of lightweight aggregate concrete cast on trapezoidal profiled steel decking (Figure 1b). The 7.6 cm deep deck flutes were oriented perpendicular to the 9.1 m long beams. The concrete topping placed over the top rib of the steel deck was 8.3 cm thick to comply with the 2-hour fire resistance rating. The floor slab was partially-composite with the steel beams via 19 mm diameter steel headed stud anchors. The resulting composite action was approximately 65 % of the yield strength of the steel beams. The test bay beams were supported by shear tab connections. Shear tabs (220 mm × 150 mm × 9.5 mm) with three 19 mm diameter structural bolts were used at the ends of the W16×31 beams. Extended shear tabs (370 mm × 250 mm × 9.5 mm), welded to the column webs and bolted to the girder webs using five 19 mm diameter bolts were used at the ends of the W18×35 girders.

For floor slabs, polypropylene microfibers (2.37 kg/m^3) were used in the concrete mixture to reduce thermally induced spalling. Concrete cylinders were cast concurrently with the placement of the concrete floor slab and cured under the same conditions as of the slab. The compressive strengths measured from the concrete cylinders shortly after the day of the fire testing were 67 MPa, 78 MPa, and 68 MPa in Test #1, Test #2, and Test #3, respectively. Purpose-built slab splices (along the blue lines in Figure 1a) were designed to reuse the same surrounding floors throughout the test program, and therefore only the fire-exposed floor assembly was reconstructed for each test. Details of the structural design and construction are provided in Choe et al. [9, 12].

The Test #1 concrete slab was reinforced with steel wire reinforcement of 60 mm^2/m width (6×6 W1.4×W1.4 mesh mat with 3.4 mm diameter plain steel wires spaced 152 mm in both orthogonal directions) as the minimum shrinkage reinforcement permitted in the U.S. (Figure 1b). Both Test #2 and Test #3 specimens were reinforced with No.3 deformed steel bars of 230 mm^2/m of slab width (9.5 mm diameter hot rolled bars spaced 305 mm in both orthogonal directions) determined by the Slab Panel Method [15] incorporating tensile membrane action [10]. In Test #2, mechanical couplers were used to splice the No.3 reinforcing bars to simulate continuous bars across the test bay. In Test #3, American Concrete Institute (ACI) [16] specified details were used to splice the No.3 reinforcing bars. The clear cover to the longitudinal and transverse wires from the top surface of the concrete were 41 mm and 38 mm in Test #1, respectively. They were 48 mm and 38 mm in both Test #2 and Test #3, respectively. The measured 0.2 % offset yield strength, the ultimate tensile strength, and percent elongation after fracture (measured as the ratio of the final elongation after testing to the initial gauge length) of the welded wire reinforcement were 760 MPa, 790 MPa, and 15 %, respectively. The values for the No.3 deformed bars were 480 MPa, 770 MPa, and 22 %, respectively. All the fire exposed beams in Test #1 and Test #2 were protected with sprayed fire resistive materials (SFRM) to meet the 2-hour restrained rating requirements in accordance with the Underwriters Laboratories (UL) directories [7]. A cementitious gypsum-based material with a density ranging from 240 kg/m^3 to 350 kg/m^3 was used for the SFRM. The measured thickness of the SFRM on the secondary beam was 13 mm ± 13 % in Test #1 and 14 mm ± 7 % in Test #2. The connections were over-sprayed with an SFRM thickness of at least 25 mm. In Test #3, however, the secondary beam and its end connections were left unprotected.

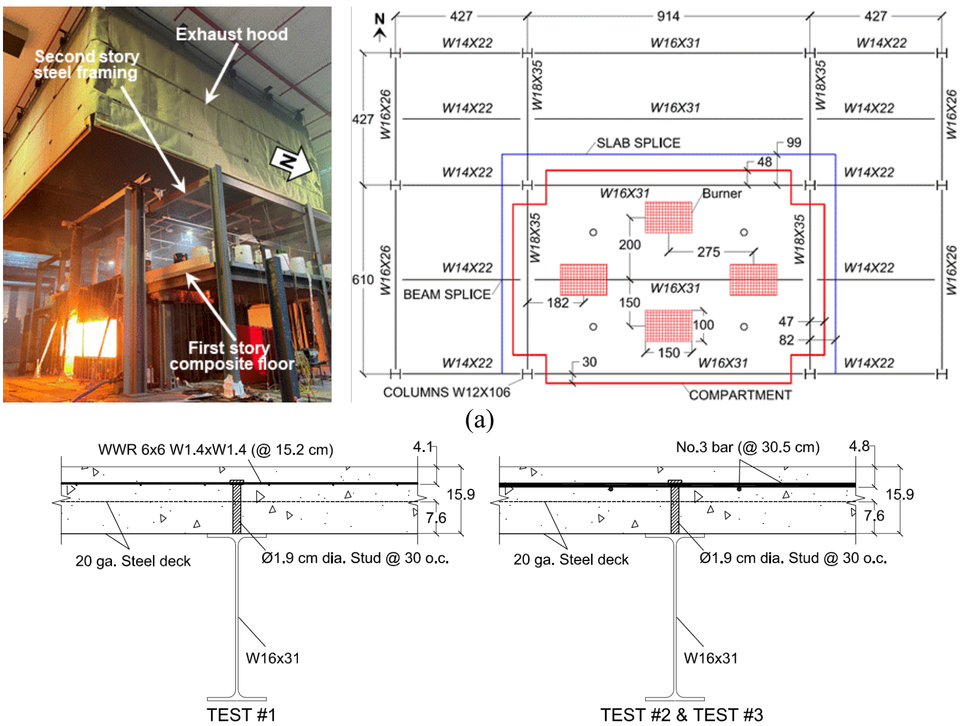


Figure 1. (a) Two-story test structure; (b) secondary beam composite sections (units in cm)

2.2 Test conditions

During the fire experiments, an imposed load of 2.7 kPa was applied to the test floor assembly using four hydraulic actuators and distributed to 24 loading points over the test floor (Figure 2a). The total gravity load including the floor self-weight was 5.2 kPa, which conforms to the gravity load demand determined from the American Society of Civil Engineers (ASCE) 7 [17] load combination for extraordinary events (1.2×dead load + 0.5×live load). The surrounding floors were loaded by water-filled drums (Figure 2a), providing an imposed gravity load of 1.2 kPa, equivalent to 50 % of the office live load considered in the structural design of the test structure. The test floor assembly was exposed to natural gas fire, with peak heat release rates in the range of 10.8 MW to 11.5 MW, using four 1 m × 1.5 m burners and confined under the slab using purpose-built compartmentation (Figure 1a and Figure 2b) to develop upper layer gas temperatures prescribed in American Society of Testing and Materials (ASTM) E119 [1]. The height of the fire compartment, the distance to the composite floor soffit above the top surface of the compartment floor, was 377 cm. Enclosing walls (along the red lines in Figure 1a) were constructed as non-load bearing walls made of sheet steel with gypsum board lining at the exposed surface. Steel columns were not directly exposed to fire except for the upper region where the floor beams and girders were joined in the test bay. The main ventilation opening on the south exterior wall was 150 cm tall × 582 cm wide. The four natural gas burners placed on the ground floor (Figure 2b) created fire exposure to the soffit of the composite floor in the test bay.

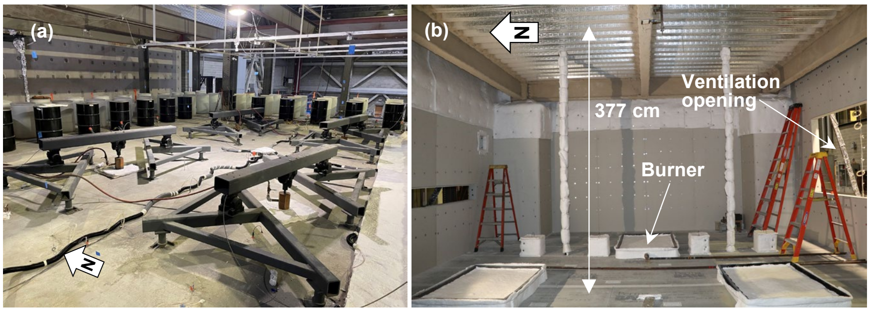


Figure 2. (a) Loading system on the top of the slab; (b) fire compartment

2.3 Instrumentation

The fire test conditions as well as the thermal and structural responses of the specimens were quantified using a variety of measurement systems. The mechanical load applied on the test floor assembly was controlled and measured using servo-hydraulic actuators and the load cells attached to them, respectively. The heat release rate of the test fires was quantified using both the fuel consumption and oxygen consumption calorimetry [18]. The gas temperature within the test compartment and temperatures of the floor specimens at various locations were measured using type-K thermocouples. Figure 3a through Figure 3c show a set of thermocouples mounted at the composite beam sections, secondary beam-to-girder connections, and within the composite floor slab in the test bay, respectively. Displacement transducers, installed outside the fire compartment, were used to measure deflections of the test floor and the surrounding structure. Figure 3d illustrates the selected locations of displacement measurements relevant to this paper, including the vertical displacement (VD) measurements at the centrelines of the test floor assembly and the horizontal displacement (HD) measurements of the test floor. HD4, HD6, HD9, and HD10 were at 15 cm above the top surface of the test floor and HD7 was at the mid-thickness of the topping concrete. The displacement values of HD4 and HD6 indicates thermally induced axial displacements of the connected steel beams. The data plots provided in this manuscript show the standard deviation of the averages of the measured values. The total expanded uncertainties, with a coverage factor of 2 for 95 % confidence level, in the individual measurements of mechanical load, burner heat release rate, temperature, and displacement were estimated to be 1 % at 125 kN, 1.4 % at 11.5 MW, 8 % at 1100 ºC, and 2 % at 655 mm, respectively. Details about the measurement systems and uncertainty in the measurements are presented in Choe et al. [9, 12].

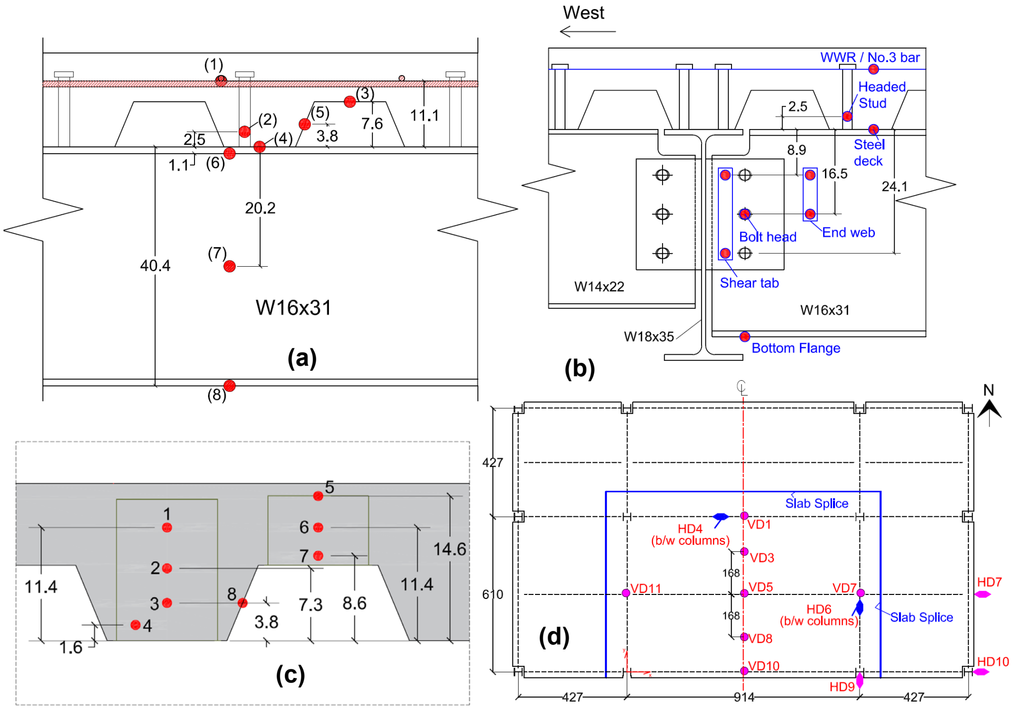


Figure 3. (a) Thermocouple locations at composite beam sections; (b) Thermocouple locations in beam-to-girder connection; (c) Thermocouple locations in concrete; (d) vertical (VD) and horizontal displacement (HD) measurements (units in cm)

3 RESULTS

Figure 4a shows the heat release rate (HRR) of natural gas burners measured based on fuel consumption and the total actuator load (Load) measured after the burner ignition. The test bay floor was imposed by a total actuator load of 125 kN. The HRR values in these tests were comparable. In Test #1, the fire and actuator loads were removed at 107 min due to slab fracture along the longitudinal centreline of the test bay. In Test #3, the actuator loading and the test fire were removed around 140 min due to significant integrity failure of the test floor slab. In Test #2, the fire was extinguished at 131 min due to significant damage of the south compartment wall. However, unlike in Test #1 and Test #3, the actuator loading was continued over a 2-hour cooling period. Details of the test results are presented in the subsequent sections.

Figure 4b shows the average upper layer gas temperatures measured using 12 thermocouple probes located 30.5 cm below the floor specimen soffit, comparable to the temperature-time curves prescribed in the standard fire testing standards (ASTM E119 [1] and International Organization for Standardization (ISO) 834 [19]). As shown in the figure, a small increase in the HRR value (by 0.5 MW) in Test #2 and Test #3 resulted in gas temperatures about 5 % higher than those measured in Test #1. In all three tests, the standard deviation in the upper layer gas temperature measurements at various locations was less than 50 ºC.

The Test #2 composite floor, which was reinforced with the No.3 reinforcing bars and had all the test bay beams protected with SFRM, tested under the same mechanical load and temperature time curve as in Test #1 and Test #3, did not exhibit structural integrity failure after 131 min of standard fire exposure. The mid-span of the SFRM-protected secondary beam reached a mid-panel vertical displacement of 455 mm at 131 min, compared to 530 mm vertical displacement of the unprotected secondary beam at the same fire exposure time in Test #3. The Test #2 floor slab reached a maximum vertical displacement of 465 mm at 145 min, during the cooling phase of the test. Given the superior performance of the Test #2 composite floor, the primary focus of this paper has been to the structural behaviour and failure characteristics of Test #1 and Test #3 composite floor assemblies with the aim to evaluate the improved fire resilience of the composite floor with the enhanced slab reinforcement and unprotected secondary beam. Refer to Choe et al. [12] for more details about Test #2.

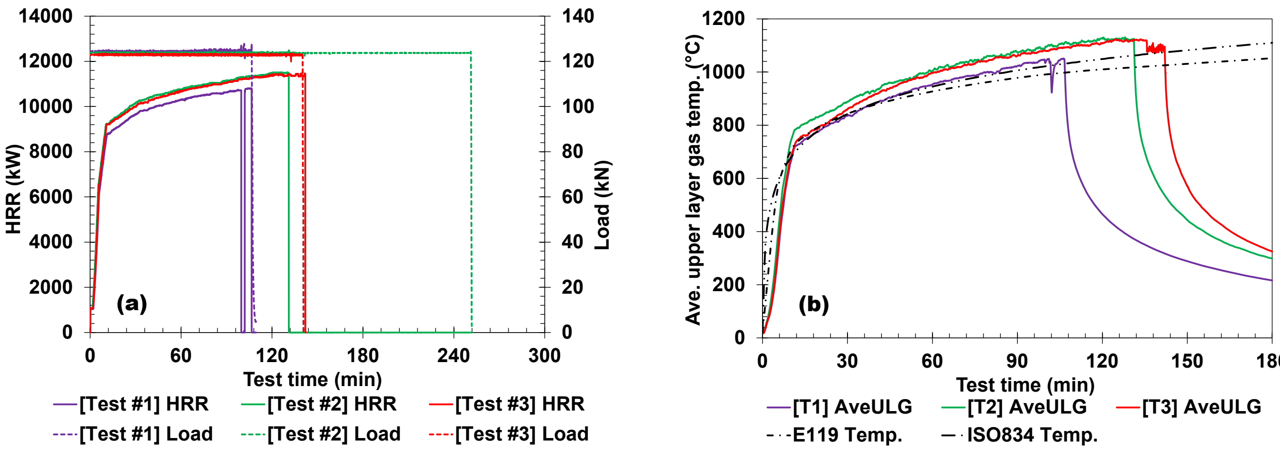


Figure 4. (a) Total actuator load and burner heat release rate (HRR); (b) average upper layer gas temperature (ULT)

3.1 Thermal response

Figure 5a shows the average temperatures of the composite beam sections along the SFRM-protected secondary beam (W16×31) in Test #1 and the unprotected secondary beam in Test #3. Refer to Figure 3a for locations and labels of the corresponding thermocouples. The error bars indicate the maximum standard deviation in temperature measurements at three different sections along the beam span. The web and bottom flange of the secondary beam in Test #1 reached a peak value of 860 °C at 107 min after ignition when the burners were shut off. In Test #3, they reached 1070 °C at 107 min. The temperatures of the unprotected secondary beam in Test #3 were close to the upper layer gas temperature during the test and reached a peak value of 1100 °C at 142 min when the burners were shut off. In Test #3, temperatures of the stud anchors and No.3 reinforcing bars above the secondary beam reached 720 °C and 500 °C, respectively. Figure 5b shows the average temperatures of various parts in the secondary beam-to-girder connection region. Refer to Figure 3b for locations and labels of the thermocouples mounted on these connections. The peak temperatures of end webs, bolt heads, and shear tabs were 640 °C, 500 °C, and 420 °C in Test #1. They were close to the upper layer gas temperature of 1100 °C in Test #3.

Figure 6 shows the averaged values of temperatures at discrete locations within the composite floor slab between steel beams, where the error bars indicate the standard deviation in temperature measurements across the test bay. Refer to Figure 3c for locations and labels of the thermocouples discussed herein. At 107 min in Test #1, the temperatures measured at location 1 (at deep section) and 6 (at shallow section) were 140 °C and 420 °C, respectively. At the same fire exposure time in Test #3, they were 130 °C and 440 °C, respectively, indicating similar temperature-time increment of the concrete slab in both tests.

The temperatures of the steel reinforcement at the deep and shallow sections of the slab placed between the test bay beams in Test #1 and Test #3 are shown in Figure 7. The average temperatures of the steel reinforcement at the deep and shallow sections of the slab in Test #3 were slightly higher than those measured in Test #1 at the same fire exposure time. This is because the bottom concrete cover of the steel reinforcement in Test #3 was smaller than Test #1, as shown in Figure 1b. The slab reinforcement at the deep and shallow sections of the slab in Test #3 reached 320 °C and 600 °C at 142 min, when the burners were shut off. For both tests, the average top surface temperature remained below the ASTM E119 limit.

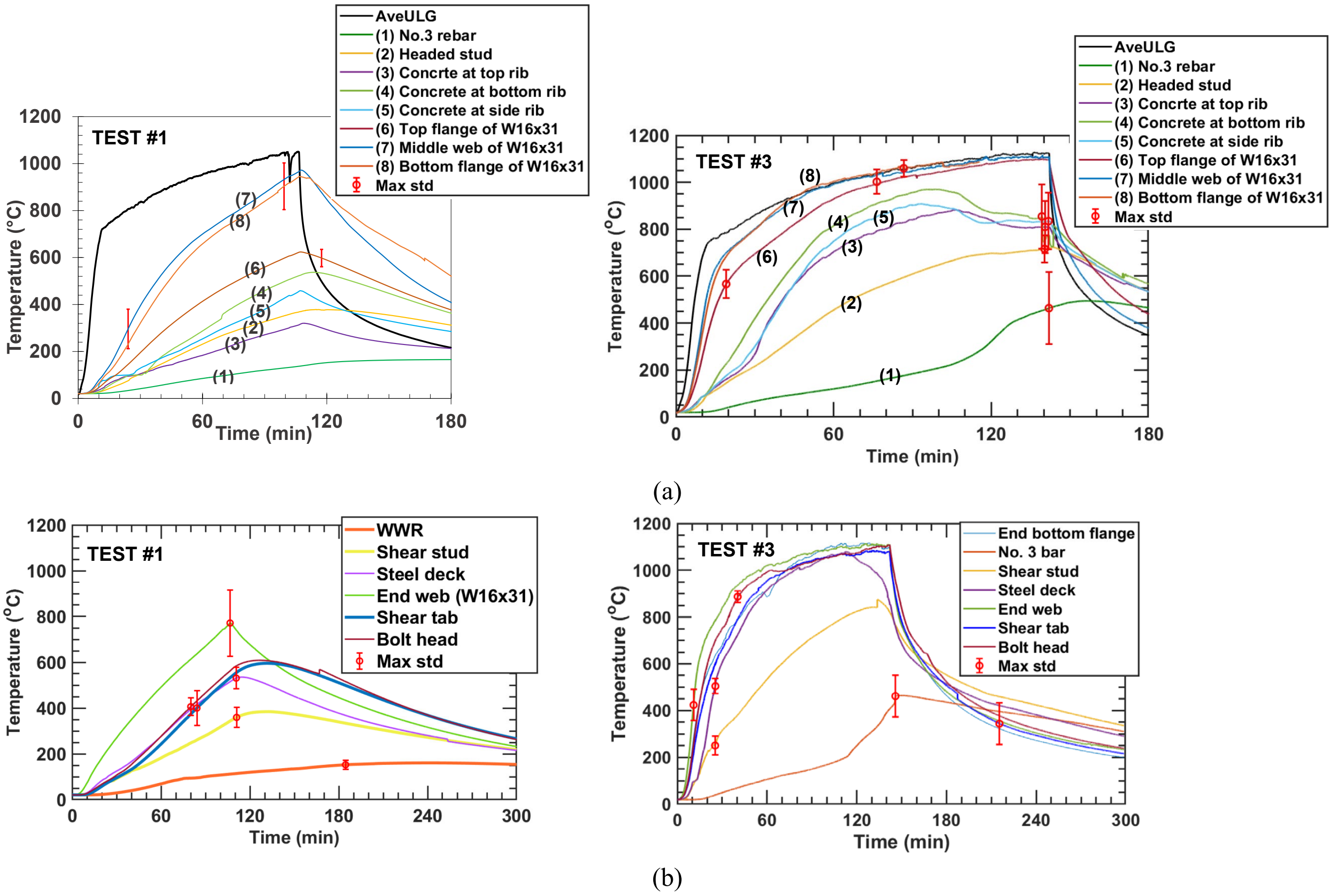


Figure 5. Measured temperatures (a) across secondary beam sections; (b) at secondary beam-to-girder connections

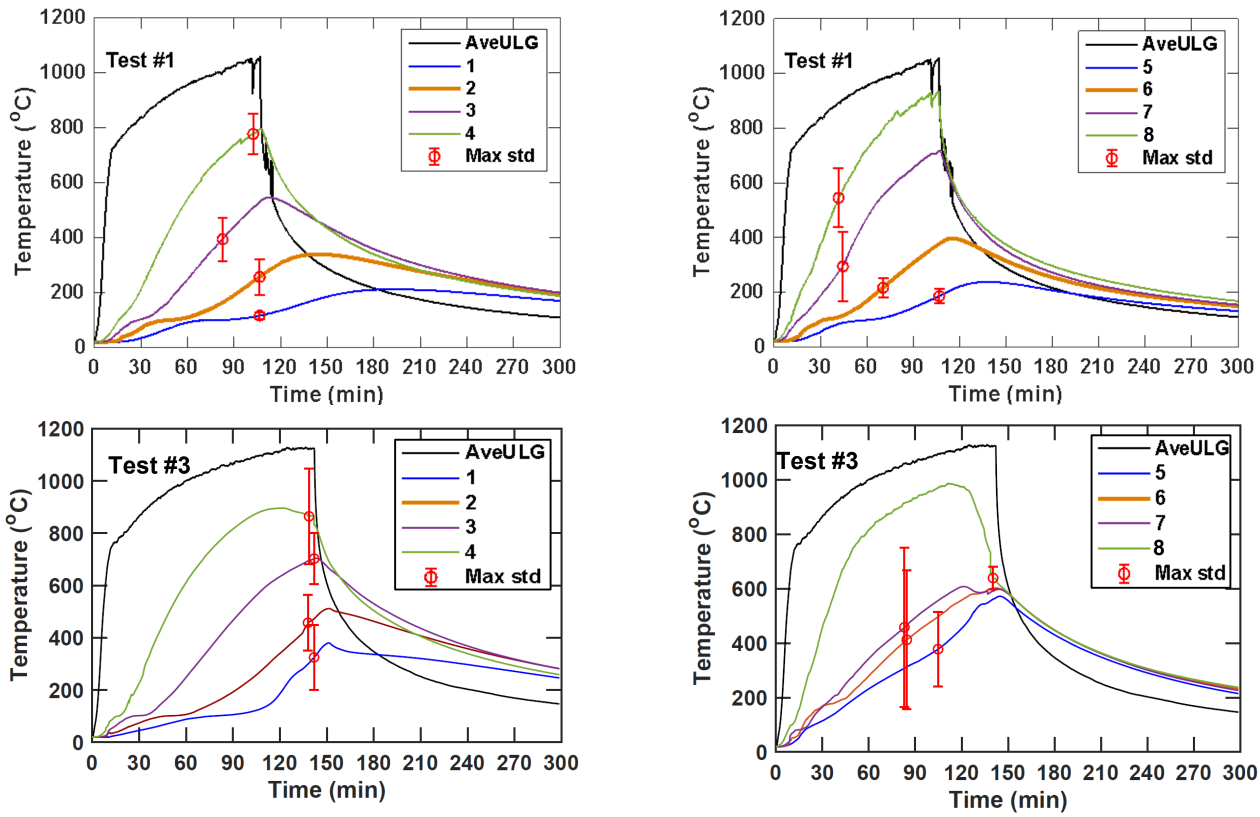


Figure 6. Measured concrete temperature in Test #1 and Test #3

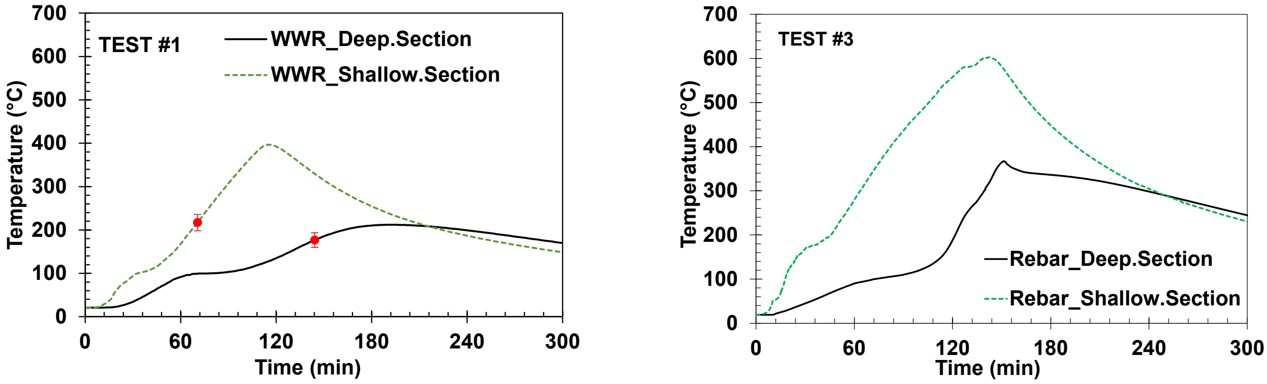


Figure 7. Measured slab reinforcement temperature in Test #1 and Test #3

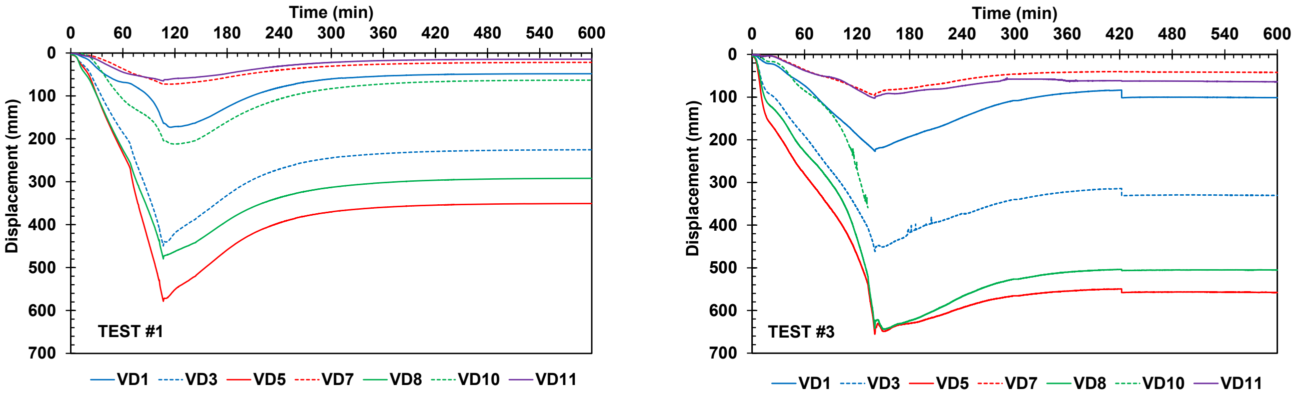
3.2 Structural response

Figure 8a shows the midspan vertical displacements of the test bay beams varying with the fire exposure time for Test #1 and Test #3. Refer to Figure 3d for the locations and labels of the vertical displacement measurements. In Test #1, the mid-panel vertical displacement (VD5) increased at an approximate rate of 4.2 mm/min until 70 min at which the bottom flange temperature of the secondary beam reached 700 °C. At 70 min after ignition of a test fire, full-depth longitudinal concrete cracking occurred in the middle of the test bay with rupture of reinforcement wires. The crack opening was widen as the wire reinforcement placed in the transverse direction of the test bay ruptured in tension, which was indicated by a noticeable change in the displacements after 70 min (Figure 8a). From 70 min to 107 min in fire, the VD5 value increased at an approximate rate of 7.2 mm/min to its peak value of 565 mm until the fire and mechanical loading were removed at 107 min.

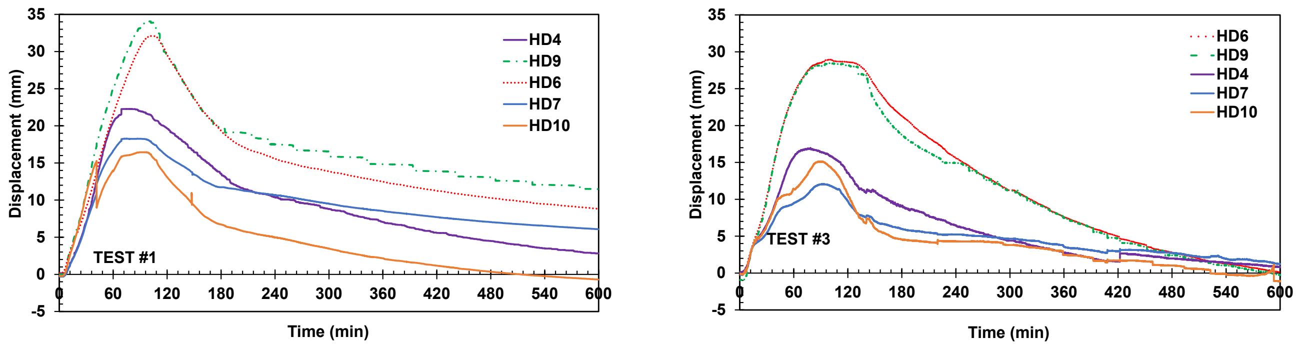
In Test #3, as shown in Figure 8a, the midspan vertical displacement of the secondary beam (VD5) increased at varying rates during the heating phase of the test: (a) 13.4 mm/min between 4 min and 14 min of fire exposure, (b) 3.1 mm/min between 14 min and 132 min, and (c) 13.9 mm/min between 132 min and 140 min. The first displacement rate between 4 min and 14 min indicates the significant loss of flexural strength of the secondary composite beam combined with thermal bowing due to more thermal gradient across the composite beam section as the unprotected secondary beam was heated to approximately 530 °C. The displacement rate change at 132 min corresponds to the occurrence of a transverse crack in the mid-panel region. VD5 reached a peak value of 655 mm (equivalent to the ratio of L/14 where L = 9.1 m, the length of the secondary beam) at 140 min when the actuator loading was removed. Around this time, flame penetration (through concrete cracks) was observed near the southeast corner of the test floor. Burners were shut off at 142 min. As the unprotected secondary beam lost its flexural strength in the early stages of the fire, tensile membrane action was activated in the Test #3 concrete slab which helped maintain the slab integrity longer than two hours.

Compared to Test #1, Test #3 exhibited larger mid-panel deflection at the early stages of the fire, before 73 mins of fire exposure, as the unprotected secondary beam was heated more rapidly. However, Test #3 exhibited smaller mid-panel deflections compared to Test #1 after 73 minutes due to the slab reinforcement of No.3 bars (230 mm^2/m) provided in Test #3 compared to the wire reinforcement (60 mm^2/m) provided in Test #1. At 107 min, the mid-panel displacement (VD5) reached 415 mm in Test #3 compared to 565 mm in Test #1. For both Test #1 and Test #3, the vertical displacement of the south edge beam (VD10) increased more rapidly than north edge beam (VD1) because of the connectivity of the north beam to the surrounding bay. At 107 min, VD10 and VD1 reached 205 mm and 165 mm in Test #1, respectively. They reached 195 mm and 165 mm in Test #1 at the same time, respectively.

Figure 8b shows the horizontal displacements measured in Test #1 and Test #3. In both tests, HD6 and HD9 values increased toward the south due to the restraint provided to the test floor by the north surrounding bay. Overall, the horizontal displacement values of the Test #3 floor specimen reinforced with No.3 bars (230 mm^2/m) were smaller than the Test #1 floor specimen lightly reinforced with wire mesh mats (60 mm^2/m), indicating the effect of slab reinforcement on the horizontal displacements. In Test #1, the values of HD6 and HD9 continuously increased with increasing temperatures, whereas those values in Test #3 seldom increased beyond 80 min.



(a)

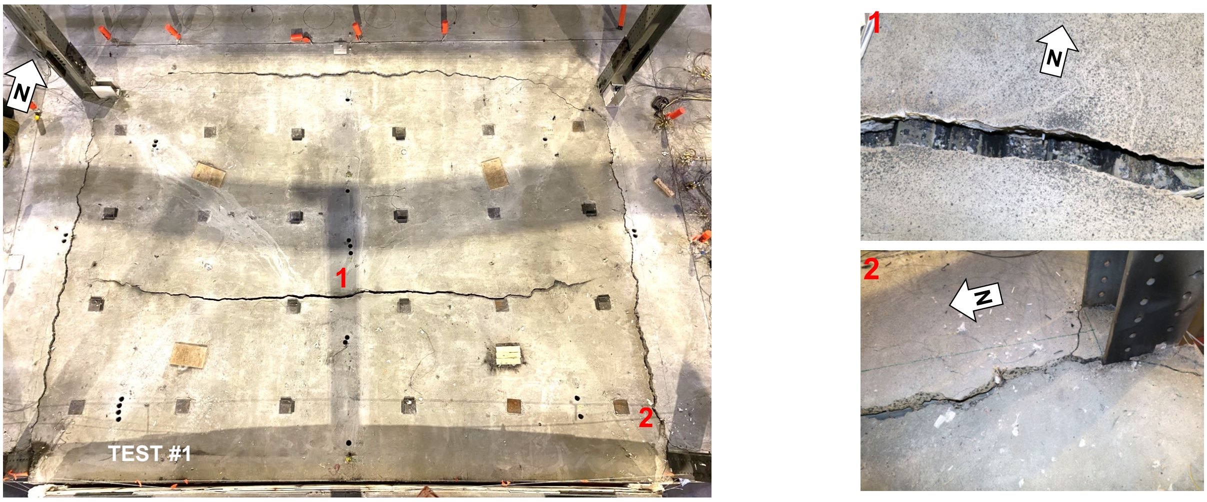


(b)

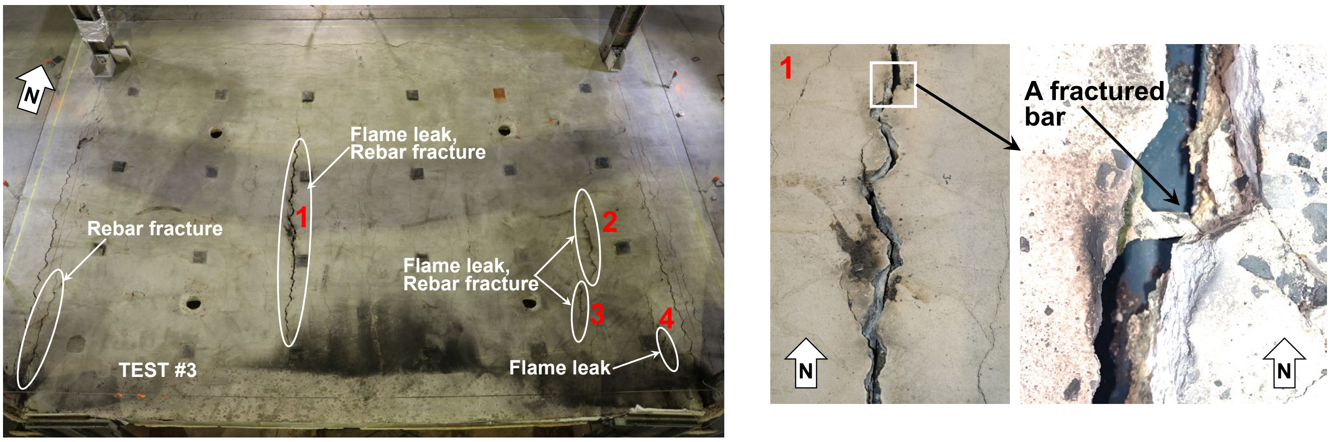
Figure 8. (a) Measured vertical displacements in Test #1 and Test #3; (b) Measured horizontal displacements in Test #1 and Test #3

Figure 9a and Figure 9b show the concrete crack patterns of the Test #1 and Test #3 floor specimens, respectively. As shown, the differences in the steel reinforcement scheme and in the fire protection of the secondary beam significantly influenced the structural integrity of the composite floor under fire effects. The Test #1 floor reinforced with the wire mesh mats exhibited large crack openings along the east, west, and north edges of the test-bay as well as in the mid-panel zone. Most of the wires within the enlarged cracks (shown in Figure 9a) were fully ruptured. The cracks at the east and west edges of the test bay appeared in the slab next to the top flanges of the girders inside the column grid causing the slab to lose its vertical edge support as well as continuity over the girders. The north edge cracks developed outside the test-bay column grid, close to the tip of the No.4 splice bars extended from the north surrounding floor. With the significant loss of vertical support and slab continuity over girders, the loaded floor slab appeared to be supported by the three longitudinal beams (9.1 m long W16×31) via headed studs with increasing temperatures leading to a tension load path primarily in the transverse direction. After Test #1, it was observed that most of steel deck units in the test bay did not rupture, which contributed to the tension load path during heating.

In Test #3, as shown in Figure 9b, the concrete slab exhibited a large transverse crack opening, approximately 105 cm west of the transverse centreline of the test bay. Most of the No.3 reinforcing bars ruptured in this crack opening. Flame leak above the slab through this transverse crack was observed after 132 min of fire exposure. The test floor exhibited cracks at other three locations (marked 2, 3, and 4 in Figure 9b) at 138 min of fire exposure although these cracks were smaller in size compared to the mid-panel transverse crack (marked 1). Concrete cracks at locations 2 and 3 also exhibited reinforcing bar fractures. The Test #3 specimen also exhibited cracks along the east, west, and north edges of the test-bay column grid (but within the footprint of the fire test compartment); however, only a few bars ruptured. Along the west edge crack, the first five bars from the south end exhibited bar fracture (Figure 9b) and the slab in that region lost continuity over the girders. Although Test #3 had the longer duration of fire exposure than Test #1, the No.3 bars controlled (1) the development of large cracks along the perimeter of the test bay, and (2) significantly delayed the development of mid-panel cracks by 60 min compared to Test #1.



(a)



(b)

Figure 9. Concrete crack patterns of the tested floors (a) Test #1; (b) Test #3

All beams in the test bay exhibited permanent global and local deformations after Test #1 and Test #3. The 9.1 m long north and south primary beams exhibited local buckling near the ends as well as twisting and lateral deformations. Along the secondary beam, severe local buckling occurred near the ends of the test bay secondary beam in Test #1 and in the W14×22 secondary beams of the surrounding bays (Figure 1) near their end connections to the test bay girders in Test #3. Relatively minor deformations were exhibited in the east and west girders. Figure 10a through Figure 10c show the failure of the beam-end connections in Test#1 and Test #3. In Test #1, the SFRM-protected shear tab connections at the ends of the longitudinal beams were seldom damaged (Figure 10a). In Test #3 where the secondary beam was unprotected including shear tab connections, all three bolts in the west end shear tab connection fractured during heating (Figure 10b). The west shear tab connection of the north primary beam exhibited bolt fracture during the cooling phase (Figure 10c). This failure is also indicated in Figure 8c where there is a sudden increase in HD4 value in Test #3 around 420 min (about 280 min into cooling). In Test #1 composite floor, no shear stud failure was observed along the test bay beams and girders. However in Test #3, the steel deck together with the concrete in the west half of the secondary beam separated from the steel beam during the test (as shown in Figure 10d) and most of the shear studs in the west half of the secondary beam as well as in the east end of the secondary beam exhibited stud fracture or large bending (Figure 10e), indicating the loss of composite action between the steel beam and concrete. This was because of the higher temperatures of the secondary beam and shear studs as well as due to the failure of the west end connection of the secondary beam in Test #3 compared to Test #1.

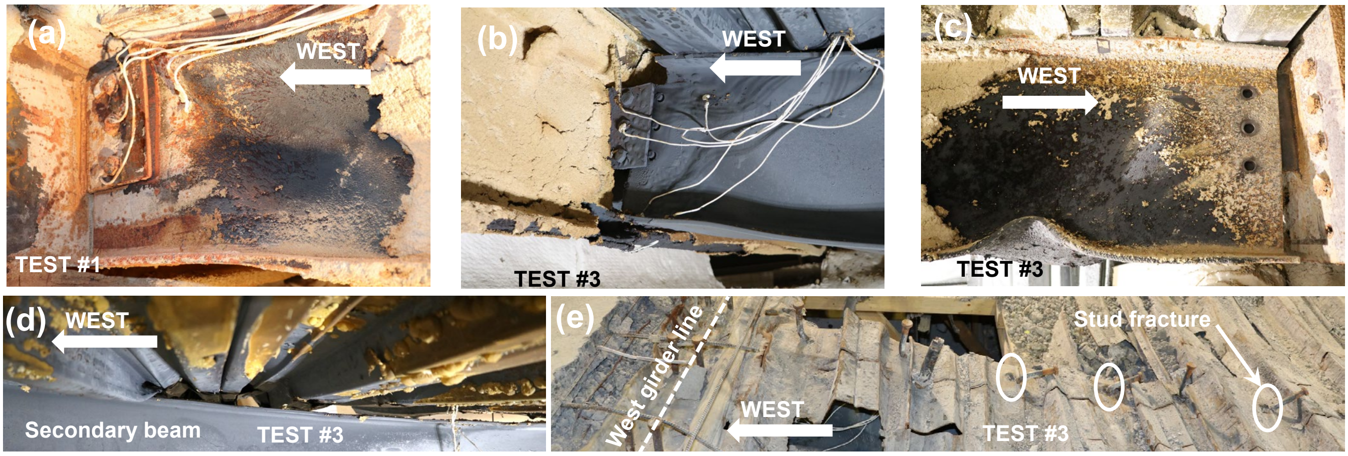


Figure 10. Photographs of test structures after completion of the experiments: (a) shear tab connection at the west end of secondary beam (Test #1); (b) shear tab connection at the west end of secondary beam (Test #3); (c) shear tab connection at the west end of north primary beam (Test #3); (d) Shear stud fracture (Test #3); (e) steel deck separation (Test #3)

4 CONCLUSIONS

The Test #1 specimen which used wire reinforcement of 60 mm^2/m width to meet the minimum shrinkage steel reinforcement permitted in the U.S. standard practice and SFRM to protect all the steel beams and achieve a 2-hour (120 min) fire resistance rating, exhibited slab integrity failure 70 min after fire ignition; i.e., before reaching the specified fire resistance rating period. This result highlights that the minimum required slab reinforcement currently permitted in the U.S. practice may not be sufficient to maintain the structural integrity of a composite floor assembly during a structurally-significant fire. Test #3 showed that the use of No.3 deformed steel bars (230 mm^2/m) for the slab reinforcement determined by incorporating tensile membrane action maintained the structural integrity of the tested slab for more than two hours (120 min) even when no SFRM was provided to the secondary beam or its connections. The additional structural capacity came from force redistribution provided by tensile membrane action of the reinforced concrete slab. The data from these experiments will help to explore engineered solutions to optimize the passive fire protection and slab reinforcement used in the steel-concrete composite floor systems for different structural and fire variables, a necessary step in the performance-based design of steel framed buildings subjected to fire.

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