Proceedings of the
Annual Stability Conference
Structural Stability Research Council
San Antonio, Texas, March 19-22, 2024

Structural fire analysis of steel-concrete composite floors designed with prescriptive and performance-based methods

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Abstract
An experimental program recently conducted at the NIST provided new data on the response of steel-framed composite buildings under standard ASTM 119 fire. One experiment tested the composite floor designed with prescriptive provisions for a 2-hour fire rating, including minimum area of shrinkage reinforcement of 60 mm^2/m. At approximately one hour, the floor developed a central breach leading to integrity failure. The second and third experiments used 230 mm^2/m of reinforcement, with the latter omitting the fire protection on the central steel beams. No failure occurred within the first 2 hours. This paper describes a numerical investigation of the NIST experiments and parametric analysis to gain further insights into the fire behavior of steel-concrete floors. Nonlinear finite element models coupling fiber-based beam elements and shell elements, with temperature-dependent plasticity and damage models, were validated against the tests. For test #1, predicted concrete damage and rebar fracture in the hogging moment area agree with the cracks observed experimentally. The model also captures the development of tensile membrane action in test #3, showing that instability of secondary steel members is bridged by alternate load paths in the composite floors. A sensitivity analysis allows identifying the minimum amount of reinforcement required for both protected and unprotected central beams configurations. The model is then used to run fire scenarios with decay phases and improve understanding of failure modes under realistic fires.

Keywords: steel-concrete composite floor, full-scale fire test, numerical modeling, performance-based fire design, concrete damage, tensile membrane action

1. Introduction

In the United States, prescriptive building codes and specifications set the minimum fire safety requirements to provide adequate structural integrity during a fire. These codes typically mandate passive fire protection on steel members, accounting for their diminished mechanical properties at high temperatures (Ma, et al., 2023). Recently, performance-based structural fire design (PBSFD) has garnered interest for its potential to offer flexible, resilient, and cost-effective solutions. However, its adoption in the United States remains an exception, despite existing research and application (Gernay, 2024, Jiang, *et al.*, 2022). Conducting a systematic comparative analysis of

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the fire behavior of composite floor systems designed according to the prescriptive and performance-based methods can be valuable. The results can support improvements of the fire requirements in the U.S. codes as well as the application of performance-based structural fire design for composite structures.

Designing composite floor systems with unprotected central beams is a recognized solution in PBSFD, utilizing the tensile membrane action (TMA) in the reinforced slab to maintain the load-bearing capacity of the floor at elevated temperatures (Bailey, 2004, Qi, *et al.*, 2024). Several experiments of TMA were conducted over the past three decades in Europe, Asia, and Oceania (Lim, *et al.*, 2004). The Cardington fire tests were the first to demonstrate in full scale the increased load-bearing capacity of composite concrete slabs via TMA (Newman and Simms, 1999, Wang, 2000). Subsequent tests, including the COSSFIRE tests (COSSFIRE, 2006), provided further evidence, showing that composite floor systems with unprotected central beams could endure a 2-hour exposure to the standard ISO fire curve without structural failure due to TMA. Additionally, the FICEB test (Vassart, et al., 2012) revealed that composite floor systems with cellular steel beams could also harness TMA, surviving a natural wood crib fire with a fuel load of 700 MJ/m^2 for over 90 minutes.

Until recently, no experiment had been conducted on the fire behavior of an entire composite floor system designed according to the U.S. practice. The National Institute of Standards and Technology (NIST) addressed this gap by conducting three full-scale fire tests. The first test was designed according to the prescriptive approach in the United States for a 2-hour fire rating, but the full-scale assembly failed to survive the 2-hour ASTM E119 fire due to integrity failure in the slab (Choe, *et al.*, 2021). The second and third experiments were designed with additional reinforcement in the slab, with the third one omitting the fire protection on the central steel beams, showing no signs of failure within two hours (Choe, *et al.*, 2022, Ramesh, *et al.*, 2023).

While the NIST tests have enabled an experimental comparison between prescriptive and performance-based design methods in fire performance, there remains a need to deepen understanding of the behaviors observed during these experiments and to supplement these findings with additional data on various design parameters. The experiments can serve to validate models which are then used to complete detailed analyses and provide comprehensive data at any point of the structure over time. Furthermore, validated models allow for the simulation of diverse scenarios and the analysis of different designs, circumventing the substantial costs associated with full-scale testing. The models can serve as a valuable complement to experimental testing, enabling a comprehensive comparative analysis of the prescriptive and performance-based design.

This study focuses on a comparative analysis of the fire response in composite floors designed via prescriptive and performance-based methods. The numerical models are validated against the recent NIST fire tests. The validated models are then used for conducting the comparative analysis, determining the minimum required rebar area to achieve a 2-hour fire resistance rating for the tested prototypes designed per the two methods, and simulate scenarios of natural fires.

2. Composite floor prototypes in the NIST tests

The structure is a two-story steel gravity frame with two bays by three bays in plan, for a total floor area of 18 m × 11 m. The overall height of the structure is approximately 7.2 m, as shown in

Fig. 1. Detailed information regarding the structural design, construction, and mechanical properties of the beams, columns, wire mesh, and rebars is available in the NIST technical Notes and publications (Choe, *et al.*, 2022, Choe, *et al.*, 2021, Ramesh, *et al.*, 2022, Ramesh, *et al.*, 2023).

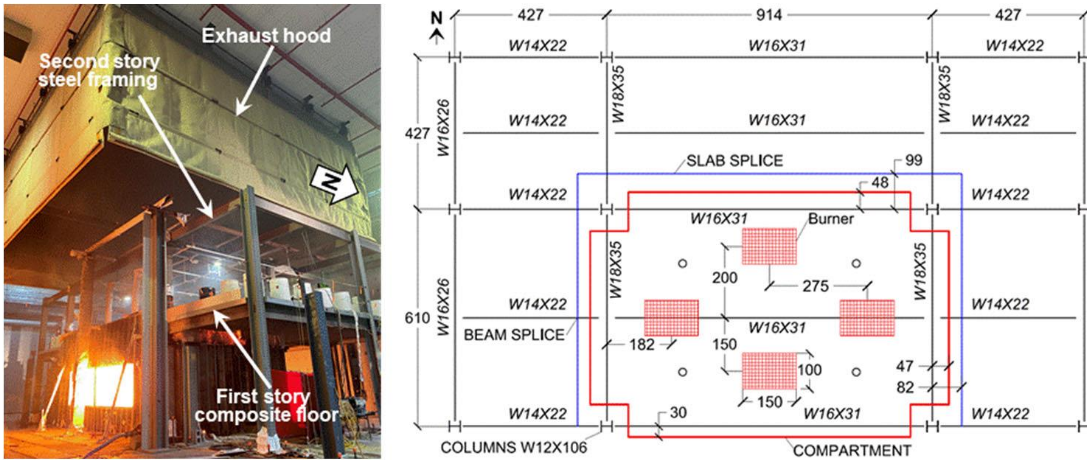


Figure 1: Two-story test frame and plan view of the test frame on the first floor. (Ramesh, *et al.*, 2022)

The structure was subjected to a mechanically applied gravity load, with the test bay experiencing a total gravity load of 5.2 kPa and the surrounding floors experiencing a uniform load of 3.7 kPa. Four natural gas burners were employed on the floor of the test compartment to create the test fire prescribed in American Society of Testing and Materials (ASTM) E119. The measured temperatures in the compartment show close agreement with the ASTM E119 curve (Ramesh, *et al.*, 2022).

The first experiment with 60 mm^2/m of wire mesh reinforcement (755 MPa strength) failed to maintain integrity for 2 hours of standard fire exposure though it was designed per U.S. prescriptive provisions for a 2-hour fire rating. A full-depth crack appeared above the central beam after approximately one hour of exposure leading to integrity failure. The second and third experiments were designed with 230 mm^2/m of steel bars reinforcement (480 MPa strength) in the concrete slab in lieu of the minimum 60 mm^2/m wire mesh. The third specimen had no fire protection on the central steel beam. The second and third experiments showed no failure within 2 hours of exposure. The second test displayed no major cracking, while the third test had its first significant crack after 132 minutes of fire exposure.

3. Numerical modeling strategy

Numerical models of the NIST tests are developed to conduct a detailed comparative analysis of composite floor systems between prescriptive and performance-based designs. The numerical simulations are conducted using the nonlinear finite element (FE) software SAFIR (Franssen and Gernay, 2017). The simulations follow a two-step process. First, thermal analyses are conducted to determine the transient temperature distribution within the structural members. Then, mechanical analyses are performed to obtain the structural response throughout the fire.

The thermal analysis uses two-dimensional beam and shell elements to construct the sections of beams and slab, respectively. Temperature-time curves obtained as the average temperature of the upper layer in the tests are applied at the boundaries. The steel deck, unprotected, is conservatively neglected in the model, and the profiled geometry is simplified using the method from Eurocode

EN1994-1-2 Annex D (EN1994-1-2, 2005), see Fig. 2. This approach has been used in previous numerical models of full-scale fire tests on composite floors (Vassart, *et al.*, 2012).

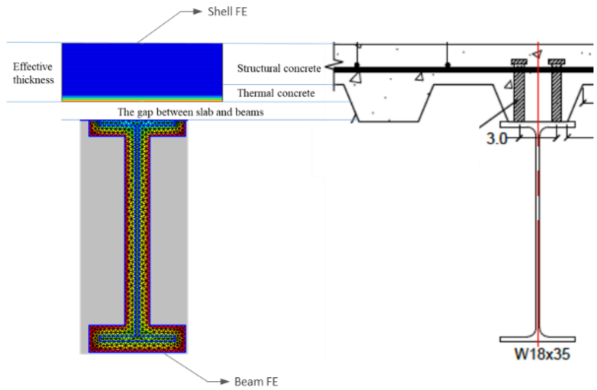


Figure 2: Modeling strategy for thermal analysis.

The mechanical analysis accounts for the material and geometrical non-linearities and large displacements. The steel reinforcement used is a hot rolled Class A reinforcement according to EN1992-1-2 (EN1992-1-2, 2004). For Class A reinforcement, the descending branch of the stress-strain curve starts at a strain of 0.05. The properties of the steel and concrete were determined through coupon tests. The concrete model used with the shell elements is an elevated temperature plastic-damage model with explicit transient creep (Gernay, *et al.*, 2013). The reduction of strength with temperature is in accordance with Eurocode. The model includes the steel columns and beams and the composite steel-concrete floor, with the same boundary conditions, material properties, and gravity loads as the test, see Fig. 3.

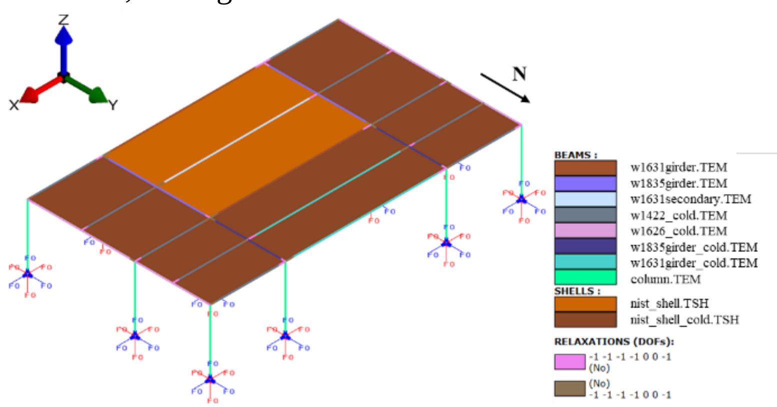


Figure 3: 3D FE model of the tested building in SAFIR (the fire-exposed part of the floor is in lighter color)

4. Validation of the numerical models

4.1 Test #1 (Prescriptive design)

Validation of the model is provided in terms of vertical displacements and damage patterns in the concrete slab. Fig. 4 plots the vertical displacements of the exposed floor in the test (solid lines) and simulation (dashed lines). The vertical line indicates the end of the heating at 106 minutes during the test. The gravity load was removed in the test after 106 minutes but maintained in the simulation. The simulation accurately predicts the peak displacement at the slab center (location 5), with a value of 599 mm closely matching the measured value of 579 mm. The simulation tends to overestimate the displacement of the slab between beams (location 3 and 8), probably due to the conservative omission of the steel deck in the model. Importantly, the simulation predicts significant damage and rebar strains (as discussed below) and stops converging early in the cooling phase, which aligns with the experimental integrity failure leading to removal of the load.

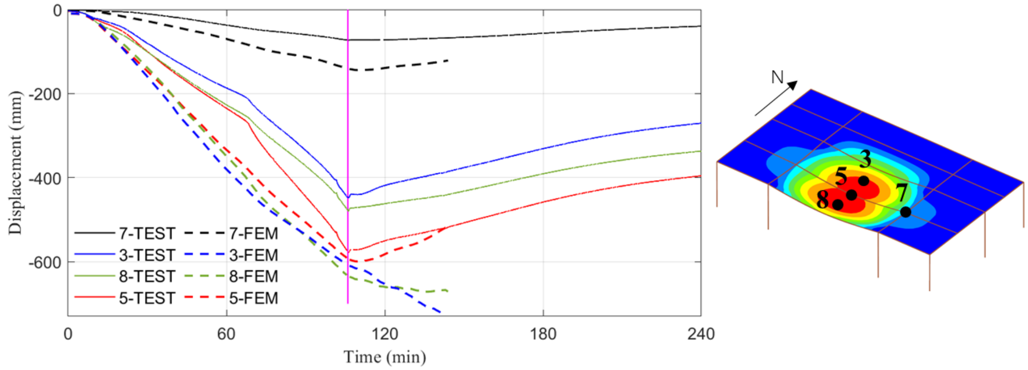


Figure 4: Comparison between the test (solid lines) and the simulation (dashed lines) for displacements in Test #1. The fire and mechanical load were removed at 106 min in the test.

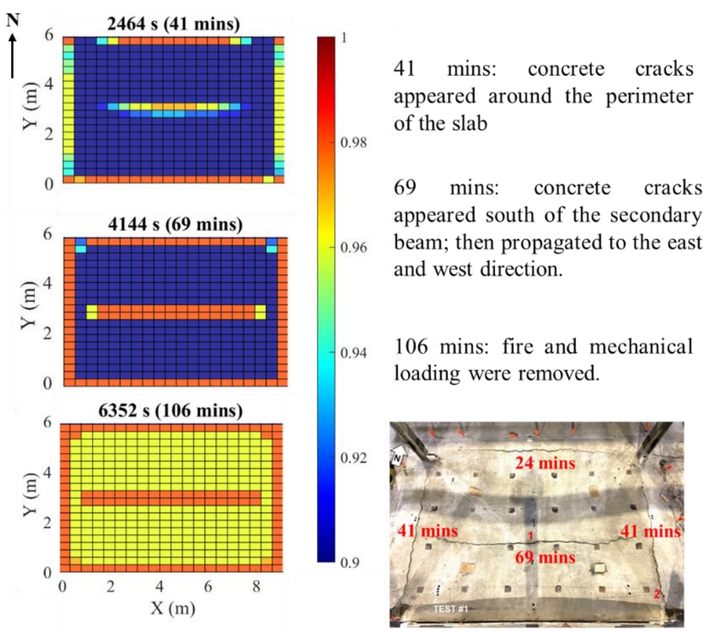


Figure 5: Computed tensile damage in the concrete slab and comparison with observations for Test #1.

The distribution of concrete damage in the simulation is compared with the test observations to further validate the numerical model. Fig. 5 plots the maximum tensile damage values of the concrete slab and the corresponding observations from the test. The damage value varies between zero and one, with one meaning that the material is fully damaged. The numerical model successfully captures the damage distribution, including the asymmetric behavior at the north and south slab edges. At 41 minutes, damage becomes pronounced around the north and south primary beams. At 69 minutes, the damage value exceeds 0.95 around the secondary beam and the perimeter beams, consistent with the test cracks. At that time in the test, the crack around the secondary beam was gradually developed to full-depth crack. At 106 mins, the fire and mechanical load was removed in the test due to the integrity failure. The simulation stops converging at

146 min due to the rebar failure. The experiment similarly exhibited rupture of the wire mesh and large opened cracks at that time, even though the mechanical load had been previously removed.

4.2 Test #3 (Performance-based design)

Removing the fire protection on the central beam(s) to activate the tensile membrane action (TMA) through large deformations is a well-known strategy in performance-based fire design. In the early stages of the heating phase, both the unprotected steel deck and the central steel beam experience rapid loss of stiffness and strength. Consequently, the slab undergoes substantial deflections. Nonetheless, the concrete slab and its reinforcement are capable of sustaining loads through a redistributed load path, as illustrated in Fig. 6. This phenomenon, known as tensile membrane action, significantly enhances the load-bearing capacity of the composite floor at large deflection, thereby improving the overall fire resistance of the structure (Bailey, 2004, Orabi, *et al.*, 2022).

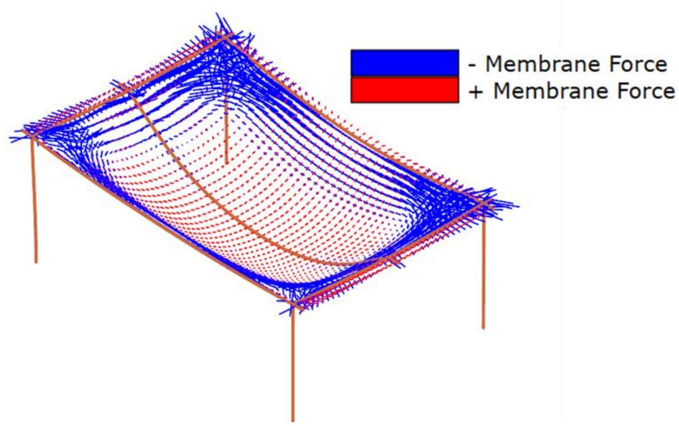


Figure 6: Distribution of membrane forces in a reinforced concrete slab undergoing TMA (SAFIR model).

In Test #3, the rebar area in the concrete slab was increased from 60 mm^2/m to 230 mm^2/m and the fire protection on the central beam was removed. Fig. 7 plots the vertical displacements of the exposed floor in the test (solid lines) and the FE simulation (dashed lines). The model can successfully converge until the end of the fire, and the predicted vertical displacements agree overall with the measured behavior during the experiment. The simulation predicts a maximum displacement at the slab center ("5") of 635 mm, aligning with the measured value of 655 mm.

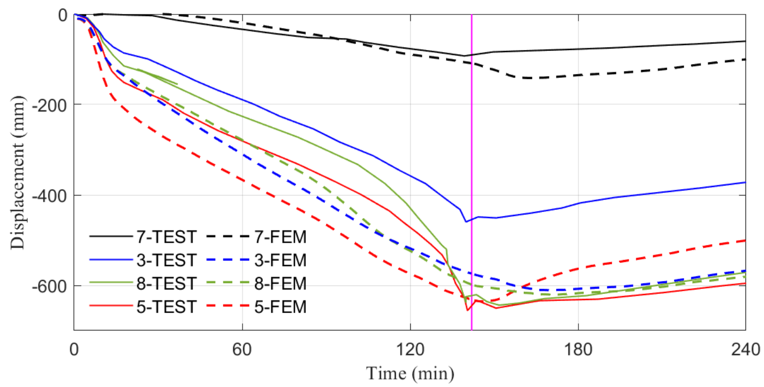


Figure 7: Comparison between the test (solid lines) and the simulation (dashed lines) for displacements in Test #3. The fire and mechanical load were removed at 142 min in the test.

The maximum tensile damage distribution in the simulation is compared with the concrete crack pattern observed in the test. During the heating phase, the tensile damage appears early in the center of the slab due to the activation of TMA. Tensile damage also develops over the peripheral beams, where hogging moments exist in the multi-bay design. In the test, concrete cracks were similarly observed in the slab center and around the perimeter of the slab.

5. Comparison between prescriptive and performance-based approaches

5.1 Vertical displacement

The vertical displacements at the slab center for prescriptive (test #1) and performance-based (test #3) designs are plotted in Fig. 8. In Test #1, displacement rates were considerably lower until 69 minutes of exposure compared to those in Test #3. However, after 69 minutes, the emergence of full-depth longitudinal concrete cracking results in a higher displacement rate. At 100 minutes, the displacement in Test #1 (before stopping the burners) starts to exceed that of Test #3. It is worth noting that the model does not capture the sudden shift of displacement rate caused by the apparition of large, discrete cracks (e.g., at 69 min in Test #1), as the continuum damage model results in a more progressive behavior. In contrast, Test #3 showed a rapid increase in displacement within the initial 15 minutes, attributable to the loss of strength in the metal deck and the unprotected beam at elevated temperatures. Subsequently, the displacement increased at a slower rate, owing to the activation of tensile membrane action. The model successfully predicts the shift in displacement rate observed during the transition to tensile membrane action in Test #3.

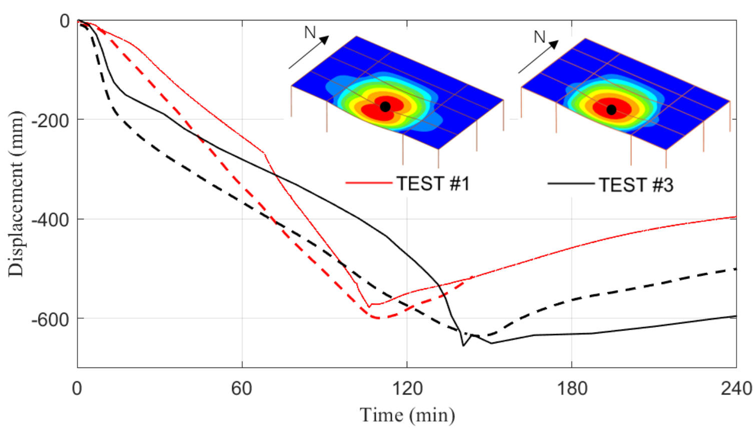


Figure 8: Comparison of vertical displacements at the center of the fire exposed composite floor for two of the NIST experiments: measurements (solid lines) versus SAFIR model (dashed lines).

5.2 Damage distribution and steel strain in concrete slab

A comparative analysis of the damage distribution pattern between the simulation of Test #1 and Test #3 reveals that leaving the central beam unprotected significantly reduces the hogging moment above the central beam during heating. This helps spreading the damage out from the slab center, resulting in a more evenly distributed damage pattern compared with the slab with protected central beam. This also reduces the strain in the steel reinforcement, as shown in Fig. 9. Compared with test #3, the peak strain in test #1 is significantly higher, surpassing the 0.05 threshold (indicative of the descending branch), especially at the north slab edge and mid-span of the slab. This corroborates the rupture of the wire mesh observed in the experiment. It indicates that the minimum rebar area of 60 mm^2/m permitted in the U.S. prescriptive approach may not be sufficient to maintain integrity for 2 hours of standard fire exposure in a full-scale assembly.

Fig. 10 plots the distribution of the membrane forces in the simulation of Test #1 and Test #3 at 30 mins of fire exposure. For Test #3, the compression ring in the exposed slab and central tension zone are characteristic of the development of TMA. The simulation of Test #1 reveals the composite behavior between the concrete slab and the steel beams, resulting in a large portion of the exposed concrete slab being in compression.

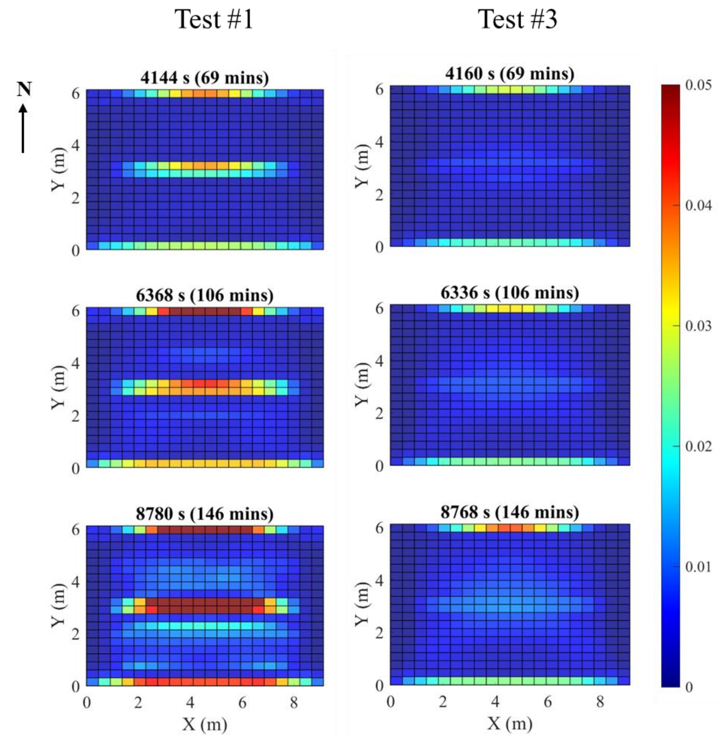


Figure 9: Computed average strain of steel rebar in the concrete slab (strain in the N-S direction).

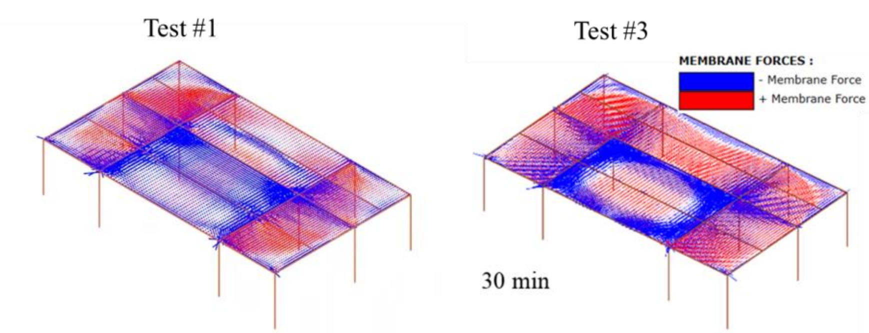


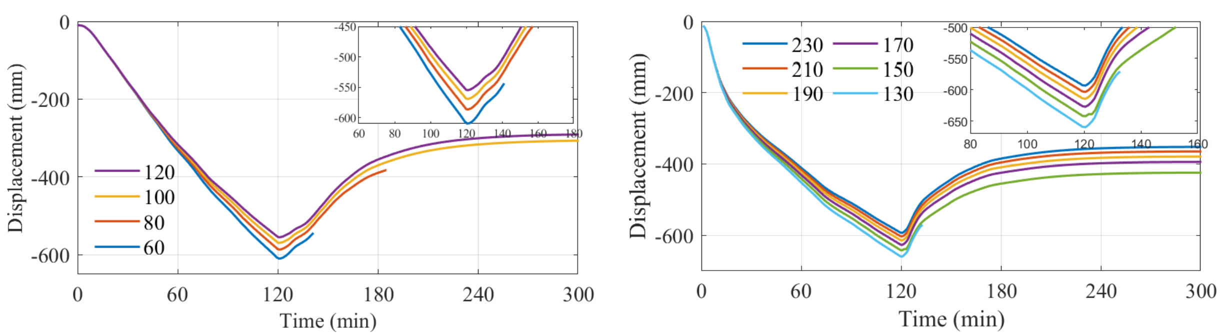
Figure 10: Distribution of the membrane forces for the two design methods at 30 min

5.3 Minimum required reinforcement

Both the test and numerical results underscore the significant impact of rebar area in the concrete slab on the fire response of the composite floor, which is also evidenced by the literature (Jiang and Li, 2018). Thus, a parametric analysis focusing on the rebar area is conducted to determine the minimum required rebar area for each of the two design methods. The objective is set as surviving two hours of standard fire exposure without rupture of reinforcement or loss of numerical convergence indicative of either a stability or integrity failure.

The failure of test #1 was attributed to an insufficient rebar area (60 mm^2/m), unable to withstand the hogging moment in the slab, as evidenced by both the test and simulation results. When the rebar area is increased from 60 mm^2/m to 100 mm^2/m, the simulation successfully converges to the end of the fire without any steel failure, as shown in Fig. 11 (a). Though the simulation can converge to 180 minutes with a rebar area of 80 mm^2/m, steel failure in the reinforcement is noted. Consequently, it is determined that the minimum required reinforcement for this slab with protected central beam is 100 mm^2/m of steel with 755 MPa strength. An increase in rebar area within composite floors is crucial to ensure that loads are effectively transferred across the concrete cracks, promoting a more uniform distribution of deformation rather than concentration at existing cracks. Note that the simulations use the same welded wire reinforcement as in Test #1, with a yield strength of 755 MPa. The average tensile damage distribution in the concrete slab for Test #1 with different rebar areas is plotted in Fig. 12. It indicates that an increased rebar area significantly reduces the peak damage values.

When the fire protection on the central beam is removed, the minimum required reinforcement is 170 mm^2/m of steel bars with 480 MPa strength to maintain the integrity and avoid failure, as shown in Fig. 11 (b). Though the simulation with a rebar area of 150 mm^2/m can converge to the end, steel failure is observed at 122 minutes till the end time, indicating the potential possibility of integrity failure during the cooling phase. It should be noted that the simulations of Test #3 use the same type of reinforcement as in the corresponding test, with a yield strength of 480 MPa. The minimum required rebar area in test #3 is equivalent to 108 mm^2/m with a yield strength of 755 MPa, based on the same total tensile strength of the wire mesh. This finding indicates that a composite floor with unprotected central beams has a comparable performance to one with protected central beams, requiring only an 8% increase in steel tensile capacity that would be need anyway to maintain integrity for 2 hours of exposure. Yet, leaving the central beam unprotected by leveraging TMA can bring savings in passive fire protection, which may have cost and/or sustainability benefits. It is important to note that these conclusions pertain to the context of the NIST floor prototypes and 2-hour ASTM E119 fire, which are based on U.S. design practices.



(a) (b)

Figure 11: Central beam displacement as a function of the rebar areas (mm^2/m) in the slab, for the designs of (a) NIST Test #1 and (b) NIST Test #3

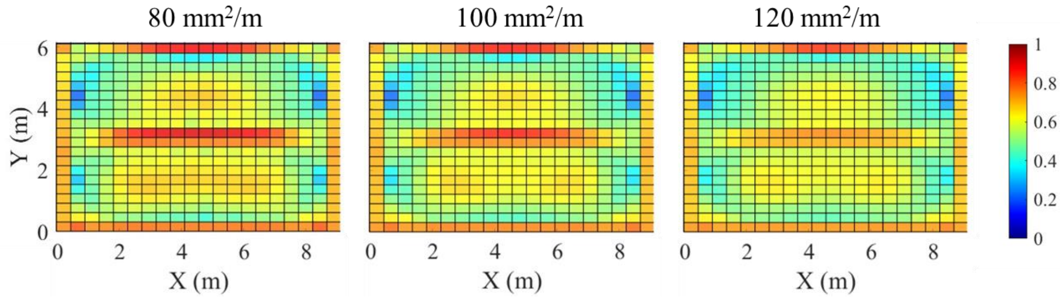


Figure 12: Average damage distribution in the slab of Test #1 with various steel reinforcement at 69 min.

5.4 Structural response under natural fire

This section compares the fire response of the composite floor designed according to the two approaches when subjected to natural fire. For this purpose, three scenarios are selected, comprising two fuel-controlled fires and one air-controlled fire. The time-temperature curves, obtained from the Eurocode parametric fire model, are plotted in Fig. 13 (a). The corresponding vertical displacements at the center of the composite floor are plotted in Fig. 13 (b). The prescriptive design has all steel beams protected and 60 mm²/m of wire mesh (755 MPa) in the slab. The PBD has the central beam unprotected and 230 mm²/m of steel reinforcement (480 MPa).

Maximum displacements are always observed during the cooling phase, which reveals the importance of considering the cooling phase. Under the moderate fire with 800 MJ/m^2, both designs survive to full burnout, with the prescriptive design exhibiting significantly lower peak and residual deformations. This suggests possibly fewer repairs needs after a moderate fire when all steel members are protected. However, under more severe fires, the performance-based design fares better. With an increased fire load of 1500 MJ/m^2, the simulations still converge to the end of the cooling phase with the PBD, while they stop early with the prescriptive design. The peak displacements before failure appear similar under both design methods for these severe fires, as the temperatures have time to rise in the central beam despite the fire protection. The simulations then stop for the prescriptive design specimen, in one case with failure of the wire mesh. Based on the modeling of the experiments discussed in Section 4, this behavior correlates with large crack openings and integrity failure in the lightly reinforced slab. These simulations under natural fire suggest higher resilience from the performance-based design in case of severe fires, owing to the robust alternate load path resulting from increased steel reinforcement in the slab.

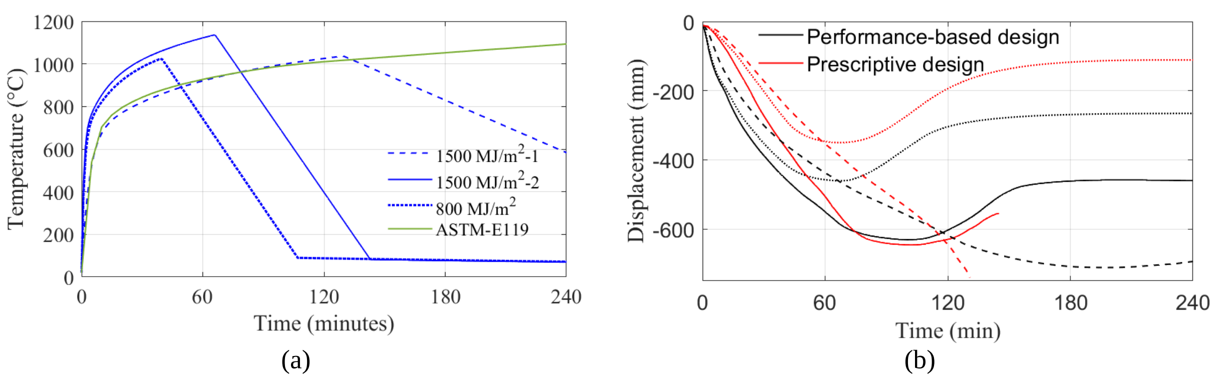


Figure 13: (a) Fire curve with different fire loads, and (b) corresponding slab center displacements.
Remark: in figure (b) dashed lines are for 1500 MJ/m^2-1 fire load, solid lines are for 1500 MJ/m^2-2 fire load, and dotted lines are for 800 MJ/m^2 fire load.

6. Conclusions

This paper described a comprehensive numerical analysis comparing the fire performance of composite steel-concrete floor systems designed with prescriptive and performance-based approaches. The study leveraged recent full-scale experiments conducted at the NIST to validate the numerical model. The numerical model served to provide a deeper understanding of the behavior of the assemblies during the experiments as well as, through parametric analyses, of the effects of steel reinforcement area and realistic fire. The main conclusions are summarized below.

Both the test and simulation results of the NIST Test #1 show that the minimum permitted welded wire reinforcement of 60 mm^2/m in the U.S. prescriptive approach may not be sufficient to maintain integrity for 2 hours of standard fire exposure in a full-scale assembly. The numerical model of test #1 successfully captures the peak displacement, the concrete damage, and the mesh rupture at the slab center. The numerical model of Test #3 similarly captures the behavior of the experiments and highlights the development of tensile membrane action in the composite floor.

The comparative analysis between the prescriptive and performance-based approaches reveals distinct patterns of damage distribution. In the prescriptive design method, damage tends to concentrate above the protected central beam due to the thermal-induced large hogging moment. Increasing the rebar area can significantly reduce the peak damage value at the slab center and peak strains in the reinforcement. Conversely, adoption of a performance-based design method with unprotected central beam can significantly reduce the hogging moment above the central beam and thus spreads the damage out from the slab center to avoid damage concentration. The latter design harness tensile membrane action, a robust alternate load path for composite slabs.

The parametric analysis on the slab reinforcement quantifies the minimum amount of steel rebars to maintain stability under 2 hours of ASTM E119 fire and throughout the subsequent cooling phase. The models determine that these assemblies require at least 100 mm^2/m of S755 and 170 mm^2/m of S480 reinforcement in the slab for a 2-hr standard fire for the design with and without fire protection on the central beam, respectively. In the latter the increased amount of steel in the slab can be offset by a reduction in fire protection material, given that the central beams do not require protection.

The comparative analysis under natural fire conditions highlights that the mechanical response during the cooling phase is critical, with peak displacements consistently observed during this phase. Under moderately severe fires, simulations suggest that both designs can survive to full burnout, with the prescriptive design resulting in lower residual deformations. However, under severe fires only the structure designed with the performance-based approach withstands the exposure until burnout, owing to the development of tensile membrane action. Substituting enhanced steel reinforcement and system-level load path for member-level fire protection design offers a more resilient response under extreme fire events.

Acknowledgments

This work was performed under the following financial assistance award 60NANB22D110 from U.S. Department of Commerce, National Institute of Standards and Technology. This support is gratefully acknowledged. The statements, findings, conclusions, and recommendations are those

of the authors and do not necessarily reflect the views of the NIST or the U.S. Department of Commerce.

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