Fire behaviour of concealed connections for steel gravity frame construction

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Abstract: Simple beam-to-girder and girder-to-column connections in steel-frame construction brace gravity columns and girders. However, during a fire, these connections are heated to temperatures at which the components (plates, bolts, welds) material properties significantly degrade. This degradation of material properties coupled with large axial force and rotational demands causes failure in these connections subsequently causing buckling of gravity columns and potential progressive collapse of a building. This study examines a new concealed connection for steel gravity framing that is located within the depth of the concrete on metal deck such that during a fire, the connection components are protected from increasing temperature and thermal degradation of material strength and stiffness. A three-dimensional finite element method model was developed to simulate the behaviour of a gravity-framing girder throughout a standard fire exposure. The results of the study demonstrate feasibility for using this connection throughout steel-framed buildings.

1. Introduction

Simple connections are used throughout the US to resist vertical shear demands of the gravity floor framing within steel-frame buildings. During a fire event, floor systems will have excessive deflections and beam-end rotations. When the floor systems undergo such behaviours, the members, and connections, are exposed to elevated temperatures. However, since these connections are only designed for vertical shear demands, they do not have the strength and stiffness to resist such demands.

Failure of gravity connections can cause large, unbraced lengths of gravity columns and of girders leading to local buckling or progressive collapse. Previous research on the behaviour of gravity floor framing in fires has shown that the axial strength capacity and rotational ductility of the connection can control the behaviour of these connections throughout a fire [1 – 4].

Historically, steel buildings commonly used cast-in-place concrete floors. It was then practical to encase the steel beam, girder, and connection in concrete. With the advent of the composite metal deck, this method is no longer used. Fire safety design today consists of applying fire protective material to structural steel members to prevent the steel from reaching critical temperature thresholds [5] as the strength and stiffness of structural steel degrades with increasing temperature. Sometimes concrete is placed on top or inside steel to prevent the steel from reaching these temperature thresholds as the concrete acts as a heat sink. Examples of this approach are concrete infilled steel tubes and metal deck concrete slabs.

With the emergence of the field of structural fire engineering, structural engineers can utilize structural engineering mechanics to design buildings for the demands imposed during a fire such that the structural framing system maintains load bearing capacity throughout a burnout fire scenario. This can be accomplished through two mechanisms: (1) designing fire protection material thickness to prevent the structural steel from experiencing temperatures such that its strength and stiffness degrade, (2) oversize structural steel members such that the steel frame can resist the demands imposed during a fire without the application of fire protection, or (3) a combination of (1) and (2). This paper will present a new concept for a gravity connection and steel framing to be used within steel floor systems such that the steel beams and girders do not require fire protection. This concept ensures that the top flange of the composite floor beams that are in contact with and act compositely with the concrete deck maintains load bearing capacity even when the bottom flange and the web of have negligible strength and stiffness due to elevated temperatures. For this approach, the connection of the beam or girder to the connected element (girder or column) occurs at the top flange of the beam or girder rather than the web. This location thereby conceals the connection within the depth of the concrete deck. The concrete deck above the top flange provides fire protection to the connection from the fire. The top flange and the web are then designed to have enough thickness to transfer the load in-service conditions. Stocky beam sections meet these thickness requirements. Open web steel joists use a similar approach with a connection at the top of the joist. To ensure that the approach works for fire and strength, engineers will design shear studs to develop the composite action between the top flange and the concrete.

Stocky wide flange sections that meet the above requirements have less depth but weigh more than the economical beam shapes. The reduction of depth of the structural sandwich depth has many advantages. Eliminating the steel fire protection material eliminates a wet trade, thus shortening the construction schedule. It also reduces the floor-to-floor height, reducing the conditioned air volume, the façade area, partitions, and pipes. This approach creates a more sustainable solution as the reduction in materials and time compensates for increased steel weight [6]. The strength design summarized in the next section considers that during and after a fire, the bottom flange and web do not resist any load and the top flange acts compositely with the concrete to resist the demands. This paper summarizes a preliminary study that explores the feasibility of this concept and further analysis and exploration needed.

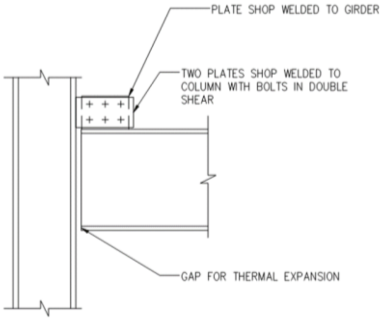
Specifically, the objectives of this study are to explore the feasibility of using embedded connections within steel-frame floors to improve the fire behaviour of steel-frame gravity floor framing. To meet this objective, the authors will explore: (1) quantify how embedding steel beam-to-column connections in a slab can change the thermal gradients through the connection regions of steel gravity floor framing throughout a fire scenario, and (2) how embedding connections within the concrete slab can change demands on the connection throughout a fire.

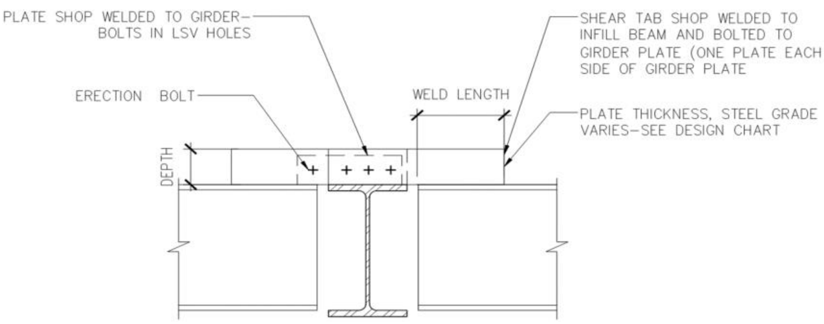
The following section of this paper will provide an overview of the design methodology for these connections and how heat transfer and stress-based analysis was performed on a composite beam using these new connections in a finite element (FE) software. The results of the FE analysis are then summarized along with recommendations for future work to better understand the fundamental behaviour of these connections throughout a fire scenario. This connection design and concept won the 2021 American Institute of Steel Construction (AISC) *SpeedConnection Steel Challenge* to develop innovative new gravity framing connections for steel construction.

2. Research methodology

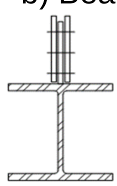
2.1 Design methodology

For this study, a typical U.S. office building layout was considered to focus on the behaviour of a single composite steel girder and its connection to the supporting column. The connection plate is welded in a fabrication shop to the top flange of the girder (Figure 1a). This plate is bolted at the construction site to a matching plate shop welded to the top flange of the supporting girder. When floor beams align on either side of the girder, the connecting bolts can be loaded in double shear (Figure 1b).





a) Girder-to-column connection b) Beam-to-girder connection



c) Section through beam-to-girder connection

Figure 1: Details of concealed connection

To analyse the connection, it is assumed that the shear load is delivered concentrically to the girder to avoid torsional effects and excessive eccentricity on the field bolted connection. The plate design is controlled by block shear at the bolted girder end or by shear and bending at the supporting beam end. The moment due to the eccentricity of the connection is developed by the welds between the plate and the beam. The beam itself must also be designed for the local shear and flexure where the load is transferred between the girder flange and the connection plate. To perform an initial analysis, the framing system under a fire condition can be simplified to a composite section consisting of the concrete slab and the beam top flange, assuming these elements retain their strength for the duration of the fire (Figure 2).

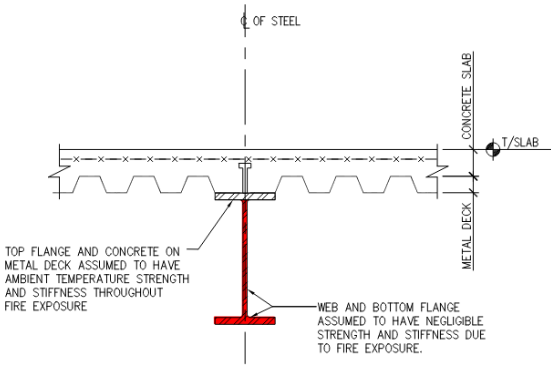


Figure 2: Post fire exposure girder configuration

For this study, the design considered is a composite steel girder spanning between steel columns as part of an office floor layout. The members were designed for typical commercial office strength and serviceability requirements. The section chosen is for the girder is a W14x82 spanning 9.1 m (30 feet) and supports infill beams spanning 12.8 m (42 feet) on either side. The floor slab is a 76.2 mm (3 inch) deep composite metal deck with 82.6 mm (3.25 inch) of lightweight concrete above the deck flutes. Shear studs are provided between the slab and the beam to develop 25% of the maximum girder composite action. The beams were designed for a service load case of 1.67 kPa (37 psf) superimposed load due to floor finishes, partitions, and hung mechanical equipment and 2.39 kPa (50 psf) live load for office occupancy. Two beams span into the girder along the length. For the fire load case, live load was assumed to be 0 kPa and deflection was not considered.

2.2 Modelling methodology

To meet the research objectives, a sequentially-coupled thermal-structural finite element (FE) analysis was performed using the commercial software Abaqus [7]. This analysis methodology consists of a two-dimensional (2D) nonlinear heat transfer analysis using heat transfer elements to calculate the time-temperature histories in each of the elements of the beam and connection section. The nodal temperatures calculated in the heat transfer analysis are used as input into a three-dimensional (3D) stress-based analysis to calculate the resulting stresses in the beam from the loading and fire scenarios.

The 2D heat transfer analysis was performed using temperature-dependent thermal properties for the steel and concrete [8, 9]. These properties include thermal elongation, thermal conductivity, specific heat, and density. The steel beam, concrete deck, and steel plate were simulated using 4-node linear heat transfer quadrilateral (DC2D4). The composite beam was exposed to a standard fire [5] for two hours. At the beginning of the fire, the top of the concrete deck was assumed to be at ambient temperature (20^{o}C). The beam was assumed to have no fire protection on the beam; therefore, the steel was directly exposed to the fire.

In the stress-based analysis the steel beam, connection elements, and the concrete slab were modelled as 8-node solid brick finite elements (C3D8R). These elements were used to simulate the thermal gradient through the steel beam and concrete slab and investigate how the presence of the connection may influence that thermal gradient. The shear studs were simulated as beam elements (B31) and connected to the top flange of the beam using wire connectors. The wire connectors were assigned temperature-dependent force-slip behaviour of the shear studs to simulate the force-slip behaviour in the longitudinal direction of the beam [10]. In the other directions, the shear stud behaviour was assumed to be rigid. Reinforcement within the concrete deck was simulated using 3D truss elements (T3D2). Bolts and welds were not modelled explicitly, rather the connection plates were tied to each other and to the top of the top flange of the girder. This assumption assumes there is no bolt slip during the fire.

The temperature-dependent mechanical properties of structural steel were simulated using Eurocode 3 [9] . Despite the yield stress of the steel being 689 MPa (100 ksi), previous research on high strength steel shows that strength reduction factors for high strength steel are comparable with those of mild steel [11]. Therefore, Eurocode 3 temperature-dependent stress-strain models were utilized to simulate the behaviour of the beam and connection throughout the fire scenario. In addition, Eurocode 2 stress-strain models were used to simulate the concrete behaviour along with concrete damaged plasticity model to simulate cracking of the concrete in tension [12, 13].

The loading for each of the simulations used the extreme load combination provided within Appendix 4 of the AISC Specification [14]. The beams are loaded in four-point bending with 133.4 kN (30 kip) point loads applied at the beam quarter points along the length of the beam. The load combination provided within Equation 1 was used to calculate the total load on the composite beam.

$$[0.9\ or\ 1.2]D + 0.5L + A_T + 0.2S \tag{1}$$

Where *D* is the dead load on the composite beam, *L* is the live load, A_T are the deformations and forces imposed on the structure due to the fire, and *S* is the snow load. Because the FE model itself will calculate the deformations and forces due to the fire, only the *D* and *L* were imposed on the composite beam.

3. Results

3.1 Heat transfer analysis results

The presence of the steel plate increases the temperature of the concrete deck throughout the fire exposure. At 60 minutes of fire exposure, the top of the concrete for a connection with a concealed plate exceeds the failure criteria for floors for a fire resistance rating is if the temperature of the unexposed surface exceeds 139°C [5]. Whereas for a conventional connection, this failure criteria is surpassed just before 180 minutes (Figure 3).

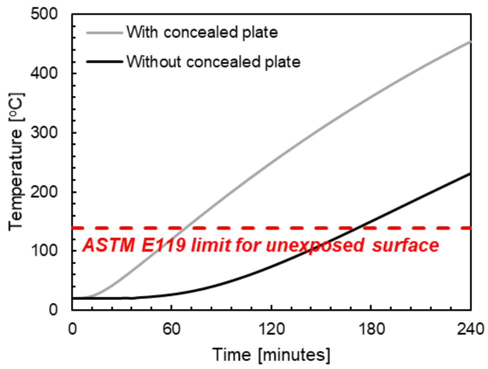
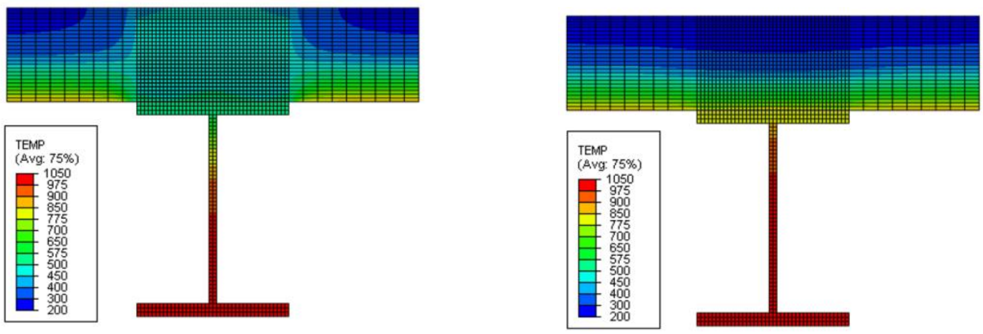


Figure 3: Temperature history of unexposed surface with and without concealed connection plates as compared with the ASTM E119 (2020) limit

The presence of a steel plate within the concrete deck for the connection can change the thermal gradient through the cross section of the beam at the connection (Figure 4). Eurocode 3 [9] Annex D provides an assumed thermal gradient through a composite beam at the connection given the bottom flange temperature away from the joint. This thermal gradient assumes a linear gradient through the cross section when the total depth of the beam is less than 400 mm, where the temperature of the beam web at the mid height of the depth of 85% that of the bottom flange temperature and the temperature at the top flange of the beam is 70% that of the bottom flange temperature. Previous researchers have shown that this thermal gradient is not consistent with experimentally measured temperatures [15]. The authors further demonstrated that this thermal gradient is not consistent with measured temperatures from the FE model (Figures 4 and 5) either with or without the presence of the steel connection plate in the concrete deck.

Figure 5 shows that the thermal gradient through the beam at the connection is a bi-linear curve where the slope of the thermal gradient from the bottom flange to the midheight of the web is different than that from the midheight of the web to the top flange. In addition, there is a larger difference in temperature between the bottom and top flanges when the steel connection plate is present in the concrete slab (black lines). When the steel connection plate is present in the concrete deck, the top flange temperature is on average 39% of the bottom flange temperature. Whereas when the steel connection plate is not present in the concrete slab, the top flange temperature is on average 76% of the bottom flange temperature. Lastly, the temperature of the beam web when no connection plate is present is on average larger than that of the bottom flange (by 1.3%) compared to when a steel connection plate is embedded in the concrete slab, the beam web temperature is on average 95% that of the bottom flange temperature. Regardless of whether the connection plate is present or not, the thermal gradients vary from those suggested by Eurocode 3, Annex D [9] .



a) with concealed steel connection b) without concealed steel connection

Figure 4: FE nonlinear heat transfer analysis results at 240 minutes of fire exposure

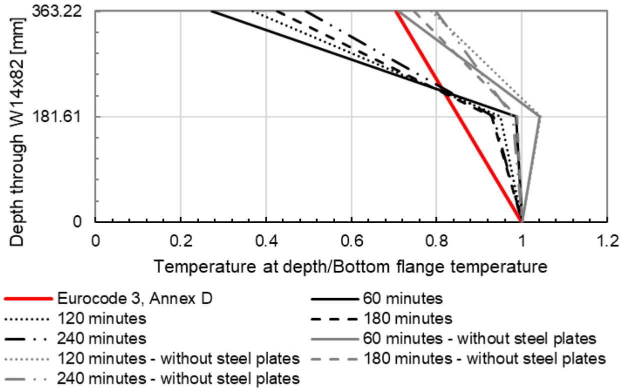
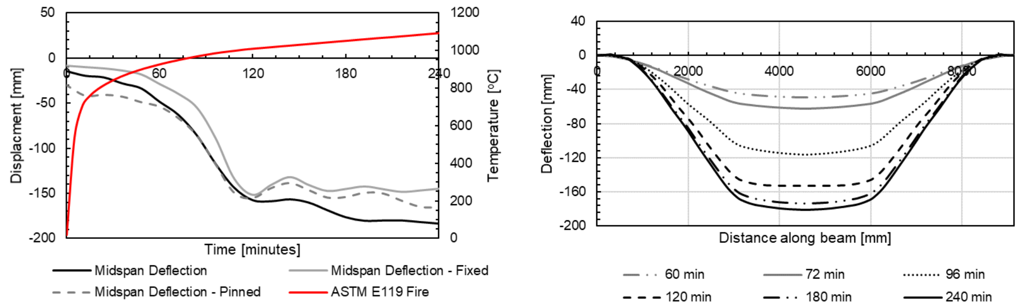


Figure 5: Thermal gradient through cross section with and without connection plate

3.2 Behaviour of composite beam with concealed connections in fire

The behaviour of the composite beam with concealed connections is more similar to a fixed connection behaviour (Figures 6 – 8). The deflected shape of the beam at 240 minutes shown in Figure 6 shows the lack of beam end rotation. Figure 6b shows the midspan deflection history throughout the fire, as the temperature increases, the beam deflection increases as well to a maximum deflection of 183 mm (7.2 inch) at 240 minutes of fire exposure. At 60 minutes, the deflection of the composite beam is equivalent to L/188, at 120 minutes the deflection is equivalent to L/58, and after 240 minutes of fire exposure the deflection is L/50. Comparing this midspan deflection to a beam with idealized fixed and pinned end connections shows that pinned versus fixed beam-end connections influence the gravity midspan deflection of the beam; however, do not have a significant impact on the midspan deflection of the beam throughout a fire.



a) midspan girder deflection b) deflected shape of girder

Figure 6: Beam deflections throughout fire scenario

The beam end rotations throughout the fire scenario, shown in Figure 7, are compared with a pure pinned beam end connection. The connection rotation histories shown in Figure 7 shows that the connection rotation throughout a fire is less than a pinned connection rotation. The maximum connection rotation is 0.00685 rad (0.36 deg) within the first hour of the fire scenario exposure. After 120 minutes of fire exposure, the connection rotation switches direction. Connection rotation continues to increase when the beam ends rotate in the opposite direction to a maximum rotation of -0.00685 rad (-0.39 deg). These values of connection rotation are significantly less than typically pinned connections during a fire [16] and of the beam with an idealized pinned beam end connection (0.015 rad).

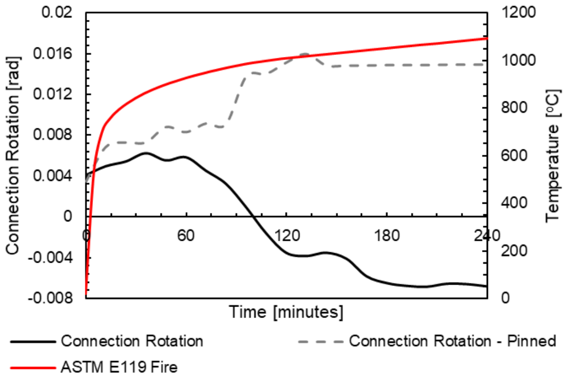


Figure 7: Connection rotation histories

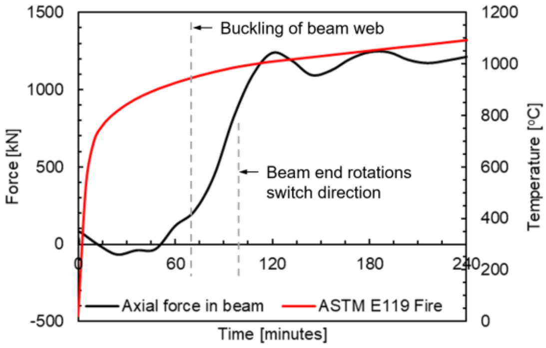


Figure 8: Axial forces in beam throughout the fire scenario

The beam end connections rotate in a positive direction until approximately 60 minutes. At that time the distance between the bottom flange of the beam and the column is 1.95 mm. The beam bottom flange temperature is 686°C, top flange temperature is 188°C, and the axial force in the beam is 118 kN (26.5 kip) in compression. At this time, the end shear studs (closest to the connections) fail and the slip of the shear studs begins to increase. In addition, the beam end rotation begins to decrease and the axial compression force in the beam increases. At 72 minutes of heating, the beam web buckles. At this time, the beam web temperature is 707°C, the beam top flange temperature is 216°C, and the axial force in the beam is 215 kN (48.3 kip). At 100 minutes the beam end rotation switches directions. Compression forces in the beam continue to increase until 120 minutes at which time the axial tensile force in the beam is 1237 kN (278 kip) when the beam bottom and top flange temperatures are 861°C and 317°C, respectively. Throughout the remainder of the fire, the web of the beam continues to buckle (Figure 9). At the end of the fire, there is no collapse of the beam. At the end of the fire exposure, the average temperature of the steel connection plate is 484°C and the beam top flange temperature is 515°C. At this average plate temperature, the stiffness of the steel has reduced by 42%; however, the yield strength and ultimate strength of the steel has only reduced by 20%.

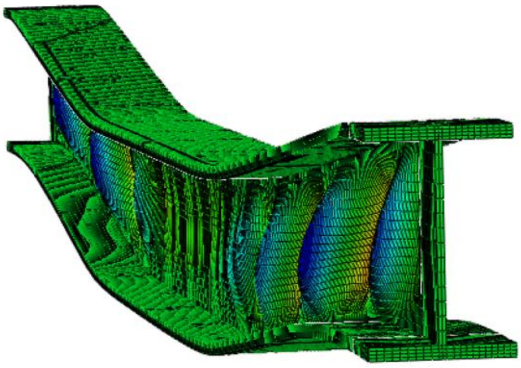


Figure 9: Damage state of beam at end of fire scenario (connection and concrete deck not shown for clarity)

4. Conclusions and future work

The development of a new connection and design methodology of steel floor framing for improved fire behaviour showed that changing the location of a steel gravity connection from the beam web to the top flange and using 689 MPa (100 ksi) steel and stocky beam sections can potentially lead to improved fire performance of steel floor framing. Embedding the connection within the steel deck significantly decreases the temperature of the steel connection elements thereby retaining more strength and stiffness of the steel connection elements throughout the fire scenario. In addition, the presence of the steel plates within the concrete deck creates a large thermal gradient through the composite beam, reducing the temperatures of the concrete on the unexposed face.

Using embedded steel plates for a connection will also create more restraint at the beam ends of the connection. Shear tab connections have demonstrated the ability to fracture and have local buckling during a fire. Additional rotational restraint of the beam end will result in an increased flexural capacity of the connection for the imposed flexural demands on simple connections during a fire.

The research study presented in this paper demonstrates the feasibility of using embedded steel plates for a simple connection in steel gravity floor framing. Additional research is necessary to further understand how varying geometric and dimensional parameters will influence the behaviour of these connections. Specifically, how varying parameters such as yield stress of beam or girder, percent composite action, fire demand influence the behaviour of this connection and the different demands when using of this connection for beam-to-girder connections in addition to beam-to-column connections. Further refinement of the numerical analysis is required to explicitly simulate the bolts in the connection and explore the influence of slotted holes on the axial force demand on the connection throughout the fire. More development of the connection detail is required to ensure that the temperatures on the unexposed surface of the concrete does not exceed 139^{o}C within two or three hours of standard fire [5] exposure so that this connection detail can be more widely adopted.

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