Fire Behavior of Composite Beams with Simple Connections: Benchmarking of Numerical Models

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ABSTRACT

This paper focuses on the development and benchmarking of 3D finite element models for predicting the behavior and failure of composite beams with simple (shear) connections subjected to gravity loads and fire conditions, which include both heating and cooling phases. The simple connections included are the shear-tab, single-angle, and double-angle connections commonly used in the U.S. engineering practice. All the structural components including the steel beam, concrete slab, shear connectors, and connection elements (bolts, plates, angles etc.) were modeled in detail using appropriate finite elements, contact interaction models, and material and component behavior at elevated temperature. The detailed finite element models were benchmarked using results from large-scale tests of composite beams and connections subjected to fire loading including heating and cooling phases. The numerical results including the midspan displacement histories and the connection rotation histories were compared with those measured experimentally by the authors. These comparisons indicate that the detailed models reasonably predict the experimental deformation histories and observed failure modes. The models were used to provide additional insight into the axial forces developed in the connections, particularly during the cooling phase, thus enhancing experimental findings. The benchmarked model is recommended for conducting analytical parametric studies to supplement experimental investigations of composite beams and simple connections under fire.

Keywords: Fire, Analysis, Connections, Composite Beam, Finite element analysis

1. INTRODUCTION AND MOTIVATION

Numerical models and simulations are valuable in research, particularly when the corresponding large-scale tests are complex, expensive, and time consuming. Structural behavior under realistic fire loading (including loading, heating, and cooling phases) can be complex, and therefore difficult to replicate and observe experimentally. Numerous connection fire tests have been performed in the U.S. and around the world. The fire tests performed at Cardington in the UK [1] showed shear-tab connection failure due to large axial forces induced in the connection. Yu et al. [2] performed fire tests on shear-tab (fin-plate) connections at elevated temperatures. In these tests, a combination of shear and axial (tying) forces were applied to the connections and remained constant throughout the experiment. The test

parameters included the number and grade of bolts. Isolated shear-tab connections were also tested by Hu and Englehardt [3]. These tests evaluated the effects of connection geometry and the grade of bolts on the bolt shear failure mode at elevated temperatures. These isolated connection tests provide insight into the behavior of shear-tab connections at elevated temperatures, but they do not focus on beam-to-column connections that are influenced by the geometry of the composite beam and concrete slab, or on their axial restraint to deformations during the heating and cooling phases of the fire.

Previous researchers have used numerical models and tools to investigate the behavior of steel structures including composite floor systems and simple connections subjected to fire loading. These models have produced observations and results that warrant verification through experimental testing. The modeling techniques used by the Sheffield group in the UK [4-6] to simulate the behavior of simple connections in fire have provided the basis for modeling composite floor systems in fire by researchers in the U.S. [7-10]. However, there are few numerical investigations that simulate both the simple connection and the composite slab behavior. For example, the numerical investigations performed by Agarwal et al. [7, 8] and Selamet and Garlock [9] on U.S. specific steel construction [11] have focused on the effects of fire loading on the behavior of simple connections, and thereby the overall structural system. These numerical investigations involved simplifications and idealizations. For example, Agarwal et al. [7, 8] utilized component based macro models for the connections, and limited failure modes to bolt shear and plate bearing. Shear-tab fracture was not included explicitly in the models. Selamet and Garlock [9, 10] utilized 3D finite element models to simulate connection behavior, but did not include the concrete slab, which is an integral part of steel buildings, and can improve the negative moment capacity of the composite beam system at the connections as discussed in [7, 8]. All these researchers [7-10] indicate that shear-tab connections can fail during the cooling phase of the fire; however, single-angle and double-angle connections may perform adequately.

The authors [12-14] have conducted experimental investigations to verify these findings for U.S. specific steel construction [11]. A series of composite beams with simple connections to steel columns were tested by subjecting them to combined thermal and mechanical loading. The tests included loading,

1 heating, and cooling phases, and provided valuable data regarding the structural fire behavior of
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4 composite beams with simple connections. Due to the limitations of the testing setup and available
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6 instrumentation, only the temperature histories, displacement histories, and connection rotation histories
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8 could be measured at specific points on the specimens during the tests. This paper presents the
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10 development and benchmarking of comprehensive 3D finite element models for predicting the behavior
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12 of the composite beams and simple connections tested by the authors [12-14]. These models are then used
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14 to supplement the experimental results, and gain additional insight into structural behavior and
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16 performance of the different types of simple connections tested. The benchmarked models will be used to
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18 conduct analytical parametric studies, and supplement the experimental database in a future paper.
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24 The development and benchmarking presented in this paper focuses on the tests conducted by the
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26 authors because they had access to comprehensive information regarding the testing conditions (loading
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28 and boundary), test setup, and specimen material and geometric properties, along with the observations
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30 and measurements from the experimental investigations. Additionally, the experimental investigations
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32 were quite diverse with different simple connection types, loading protocols and heating conditions, and
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34 observed failure modes. The intent for the benchmarked models presented in this paper is to expand the
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36 experimental database and conduct analytical parametric studies to evaluate the influence of additional
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38 geometric and material parameters that could not be considered experimentally due to practical and
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40 economic considerations. The results from the experimental and analytical parametric studies will be used
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42 eventually to develop structural performance based design guidelines or recommendations.
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46 The outline of the paper is as follows: The following section presents additional background
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48 leading up to the experimental and numerical investigations of the fire behavior of composite beams and
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50 simple connections. This is followed by discussion of the general details of the 3D finite element models
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52 including modeling techniques for the steel beams, concrete slabs, shear studs, and the components of the
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54 connections. The steel and concrete material properties are also discussed in detail, where the NIST
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56 developed σ - ϵ - T (stress-strain-temperature) [15] model is utilized for the steel components. The paper
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58 presents detailed comparisons of the numerical and experimental results for each test, and additional
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4 results and behavioral insight generated by the numerical models to supplement the experimental
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6 findings. Conclusions from the experimental and numerical results are presented along with
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8 recommendations for numerical modeling of experimental behavior.
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10 11 **2. BACKGROUND** 12

13 Agarwal et al. [7, 8] investigated collapse mechanisms of steel-frame buildings designed in
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15 accordance with US standards and codes [11] and subjected to realistic fires. Detailed FEM models were
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17 developed of the multi-story 3D building structures using temperature dependent thermal and mechanical
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19 material properties. These models accounted for large deformation theory and the various complexities of
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21 structural behavior including inelastic buckling of steel columns, yielding of steel beams, yielding of the
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23 metal deck and welded wire fabric shrinkage reinforcement in the slab, tensile cracking and compression
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25 inelasticity of the concrete slab, and partial composite action between the steel beam and concrete slab.
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27 The beam-end connections for the gravity floor systems were modeled as a series of springs that captured
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29 bolt shear and bolt bearing failure modes. These component models for connections were originally
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31 developed in the UK by researchers from the Univ. of Sheffield [2, 16], and adapted for U.S. connections
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33 by Agarwal et al. [7, 8]. The analytical investigations examined the collapse mechanisms of 3D steel-
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35 framed buildings with respect to fire location, fire intensity, and proximity of a fire to critical members in
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37 the building.
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42 The analytical results indicated that large negative moments develop in simple connections during
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44 the fire due to expansion and rotation deformations induced in the floor systems. These connections were
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46 designed according to the U.S. building code [11] to resist only the vertical loads from gravity systems at
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48 ambient conditions. The results of the parametric study performed by Agarwal et al. [7, 8] showed that
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50 bolt shear was a common failure mode of shear-tab connections during a fire. In each of the fires, the
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52 rotation of the beam-ends caused the bottom flange of the composite beams to come in contact with the
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54 gravity columns. This behavior caused large compressive forces in the bottom flange of the steel beams
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56 and large tensile forces in the bolts of the connections. In many cases, yielding of the composite beam
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58 bottom flange and the column flanges of the gravity frames was also observed.
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Selamet and Garlock [9] performed analytical studies to investigate the behavior of shear-tab and all-bolted simple connections during a fire event. This investigation resulted in suggested modifications to typical connections used in steel buildings to improve their fire performance. One of the suggested modifications was a larger gap between the beam-end and the connected member. This would allow for more beam-end rotation during a fire without developing contact between the bottom flange and the connected member. Delaying the contact would reduce the contact force (compression) and buckling of the beam bottom flange, and would lower the tensile forces in the connection, which eventually causes connection failure during the fire. The results of this study also showed that shear-tab connections may exhibit more rotational capacity during a fire, but they are susceptible to fracture during the cooling phase of the fire.

In addition to the analysis of shear-tab connections, Selamet and Garlock [10] investigated the behavior of single-angle and double-angle connections during fire events. These simple connections did not exhibit fracture during the cooling phases. All of the connections evaluated behaved well during the heating phase of the fire. The shear-tab connection models were benchmarked using the results of experimental work performed at Cardington. However, the single-angle and double-angle numerical models developed by Selamet and Garlock were not verified by experimental testing [10].

Several researchers have developed techniques and tools for modeling steel-framed building components under fire. For example, component based spring models were developed for steel connections from the extensive testing program at the University of Sheffield [2, 16]. These models were discussed in detail and adapted for US steel connections by Agarwal et al. [7, 8]. However, these component models are not the focus of this study because they are limited to bolt shear and plate bearing failure modes. These models do not adequately model fracture of the shear-tab connection or prying of single-angle or double-angle connections.

Huang et al. [17] developed techniques for modeling composite floor beams during a fire event. The modeling procedure was integrated in the analysis program VULCAN, which was developed to simulate the fire behavior of steel-concrete composite structures. The modeling procedure was benchmarked with a

number of composite beam tests performed at the University of Sheffield. The benchmarking analysis results showed that assuming full composite action between the steel beam and concrete slab resulted in conservative prediction of the midspan deflection at elevated temperatures. The comparisons between the experimental and analytical midspan deflections at elevated temperatures improved significantly when the models incorporated partial composite action between the concrete slab and steel beam. This analysis demonstrated the importance of modeling the behavior of shear studs at elevated temperature. These models, however, did not incorporate the influence of simple connections present in composite floor systems. The importance of modeling partial composite action between the concrete slab and steel beam was also confirmed recently by Selden [14].

3. GENERAL MODELING TECHNIQUES

3.1 Overview

The commercially available finite element program, ABAQUS [18], was used to model the behavior of the large-scale composite beam and simple connection specimens tested by the authors [12-14]. ABAQUS was selected for this study because it has the ability to incorporate: (i) temperature dependent material models, (ii) nonlinear heat transfer analysis, (iii) sequentially coupled thermal and structural analyses, (iv) modified Riks arc-length solution algorithm, and (v) implicit or explicit dynamic analysis of behavior and failure. The composite beams and simple connections were modeled using the sequentially coupled thermal-structural analysis approach in ABAQUS. Two finite element models were developed with identical finite element meshes: (1) one model with thermal finite elements for heat transfer analysis, and (2) the second structural analysis model with stress-based finite elements.

The thermal model was used first to conduct nonlinear heat transfer analysis and determine the thermal response of the structure along with nodal temperature histories ($T-t$ responses). These nodal temperatures were used to define the thermal loading during the structural analysis using the stress-based finite element model. The finite element meshes for both models were identical so that the results from the heat transfer analysis in terms of the nodal temperature histories ($T-t$) could be imported directly to define the thermal loading for structural analysis. ABAQUS includes compatible heat transfer and

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4 structural analysis elements that could be used for both these finite element models. For example, if C3D8
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6 is the 8-node solid element for structural analysis, then the corresponding 8-node heat transfer finite
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8 element is DC3D8. Additional details of the finite element mesh for conducting the sequentially coupled
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10 thermal-structural analysis are presented later in a following section along with details of the specimens.
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13 Temperature dependent material properties were specified for the finite elements based on previous
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15 research [19-21]. These material properties include thermal properties, namely, conductivity and specific
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17 heat, and mechanical properties, namely thermal expansion, density, and stress-strain relationships (σ - ϵ -
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19 T) for steel and concrete.
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22 **3.2 Material Models**

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24 The European Structural Code (Eurocode) provides guidance for modeling structural behavior in fire
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26 [20, 21]. Eurocode also provides more guidance than U.S. building codes for the consideration of
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28 structures in fire. Recently the National Institutes of Standards and Technology (NIST) has published a
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30 Best Practice Guidelines for Structural Fire Resistance Design of Concrete and Steel Buildings [15] and it
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32 was further refined by Luecke et al. [19]. These two documents present similar temperature dependent
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34 stress-strain (σ - ϵ) relationships for structural steel. This NIST document endorses Eurocode
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36 recommendations for temperature-dependent thermal and mechanical material properties of steel and
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38 concrete, namely, conductivity (λ), specific heat (c), thermal expansion (α), density (ρ), and the concrete
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40 uniaxial σ - ϵ - T relationship. However, the NIST document recommends an indigenously developed
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42 uniaxial σ - ϵ - T relationship. However, the NIST document recommends an indigenously developed
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44 uniaxial σ - ϵ - T relationship for steels in the U.S., which is presented in further detail in [15, 19]. Previous
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46 research has shown good correlation between experimental results and 3D FEM analyses using the NIST
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48 developed σ - ϵ - T relationship for steel [14]. Therefore in this study, the NIST developed
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50 σ - ϵ - T relationship was used for the steel components of the FEM model. The following subsections
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52 discuss and present equations for the σ - ϵ - T (temperature-dependent stress-strain) behavior of concrete and
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54 steel materials, and discuss the multiaxial plasticity models used in both material models.
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3.2.1 Concrete Material Model

The temperature dependent uniaxial stress-strain (σ - ε) relationship for concrete in compression was taken from Eurocode 4, Part 1-2 [21]. This σ - ε - T relationship is defined using the temperature dependent concrete compressive strength ($f_{c,\theta}$). The values for $f_{c,\theta}$ and $\varepsilon_{cu,\theta}$ are obtained from tables provided in Eurocode 4.

The concrete damage plasticity (CDP) model in ABAQUS was used to model the behavior of the concrete slab. This material model was developed by Lubliner, et al. [22], and later enhanced by Lee and Fenves [23] and used by Agarwal et al. [7, 8] and also by Selden [14]. This model accounts for two major failure modes for concrete: (i) concrete crushing in compression, and (ii) concrete cracking in tension. In compression, the model uses a multi-axial plasticity model with non-associated flow and isotropic scalar hardening and softening. In tension, the model uses smeared cracking with isotropic damaged elasticity concepts to account for the cracking behavior of concrete. Smeared cracking is modeled using the fracture energy approach developed by Hillerborg et al. [24], where tension stress vs. crack opening displacement (σ - u_o) relationships are used instead of conventional tension σ - ε relationships to address mesh dependence issues. The fracture energy (G_f) is the area under σ - u_o curve, and defined as the energy required to open unit area of the crack. The fracture energy of concrete and its variation with temperature are discussed in detail by Selden [14] who also used it to benchmark numerical models for predicting the behavior of composite beams during fire.

3.2.2 Steel Material Model

As part of the World Trade Center collapse investigations, NIST researchers characterized the elevated temperature stress-strain relationships of 17 different steels used in the World Trade Center towers. Through experimental testing and literature review, NIST researchers developed stress-strain curves that vary with temperature for different types of structural steels commonly used in US building construction [15]. The authors used these relationships (σ - ε - T) for modeling the steel beams and the simple connections in the composite beams subjected to fire.

Equations 1 and 2 provide expressions for the modulus of elasticity as it varies with temperature [19]. The constant term of E_0 was taken as an average modulus of elasticity of three steels tested at ambient temperature [19]. The values of e_1 , e_2 , e_3 , and e_4 are shown in Table 1.

$$E = E_0 \exp \left(-\frac{1}{2} \left(\frac{T^*}{e_3} \right)^{e_1} - \frac{1}{2} \left(\frac{T^*}{e_4} \right)^{e_2} \right) \quad (1)$$

where,

$$T^* = T - 20^\circ C \quad (2)$$

This temperature dependent stress-strain (σ - ε - T) relationship focuses on the variation of yield strength as a function of temperature. The model uses a 0.2% offset for the yield strength. Equation 3 provides the normalized value for yield strength as it varies with temperature [19]. This normalized value is with respect to the yield strength of structural steel at ambient temperature (S_y^0).

$$R = \frac{S_y}{S_y^0} = r_5 + (1 - r_5) \exp \left(-\frac{1}{2} \left(\frac{T^*}{r_3} \right)^{r_1} - \frac{1}{2} \left(\frac{T^*}{r_4} \right)^{r_2} \right) \quad (3)$$

Where the values of r_1 through r_5 are provided in Table 1. The retained strength, R , and the elastic modulus E can be combined to form the uniaxial stress-strain (σ - ε) relationship as a function of temperature. Equation 4 summarizes the relationship used in the FEM models.

$$\sigma = \left(RS_y^0 + (k_3 - k_4 S_y^0) \exp \left(-\left(\frac{T}{k_2} \right) k_1 \right) \right) \left(\varepsilon - \frac{RS_y^0}{E} \right)^n \quad (4)$$

Where the values of k_1 through k_4 and n are given in Table 1.

Eurocode [20] provides different σ - ε - T relationships for steel that implicitly account for rate-dependent effects such as creep. Figure 1(a) compares the Eurocode [20] and NIST [19] true σ - ε - T relationships at 200, 400, and 600°C. As shown, the NIST σ - ε - T relationships are stiffer in the elastic range, have higher yield stress, and more strain hardening than the corresponding Eurocode σ - ε - T relationships. This is because the NIST σ - ε - T relationships do not account for rate-dependent effects like

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4 creep at elevated temperatures. Figure 1(b) compares the yield stress retention factor for both the NIST
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6 and Eurocode σ - ε - T relationships. These retention factors were based on the yield stress calculated using
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8 0.2% offset strain for both relationships. This is uncharacteristic for Eurocode, where the retention factor
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10 is reported for 2% offset strain. This was modified in Figure 1(b) to provide an appropriate comparison
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12 between the two models. The NIST σ - ε - T relationships were used in this research to define the elevated
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14 temperature behavior of steel because rate dependent effects like creep are not significant for the
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16 maximum temperature achieved in the tests.
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20 Additionally, the ductile damage option in ABAQUS was used to model ductile fracture of steel
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22 material after reaching the ultimate strain. Based on prior research, and the steel σ - ε - T relationships, the
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24 damage initiation strain was specified to be 0.20 [25, 26], while the triaxiality for uniaxial loading was
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26 specified to be 1/3 [27]. Finite element deletion was enforced after damage initiation itself for
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28 conservatism and simplicity.
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31 3.2.3 Shear Stud Modeling

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33 Zhao and Kruppa [28] performed push-out tests to establish the behavior of shear studs at
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35 elevated temperatures. The experimental results were used to develop a series of equations for the
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37 interfacial shear force-slip-temperature behavior of shear studs with and without profiled deck. The
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39 equations developed are applicable to three categories of shear connectors: (i) shear studs with a flat
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41 concrete slab, (ii) shear studs with a profiled steel deck, and (iii) angle connectors with a profiled steel
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43 deck. Each of these force-slip curves have a linear portion, followed by a non-linear portion up to the
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45 maximum shear resistance of the connector, and finally a descending portion. Figure 2 shows the force-
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47 slip behavior as a function of temperature for shear connectors with a flat slab. This relationship is
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49 normalized with the shear capacity of the stud at ambient temperature (Q_n). As shown in Figure 2, both
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51 the strength and stiffness of the force-slip behavior decreases with increasing temperature.
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3.3 Finite Elements for Components

The steel beam, simple connection, and concrete slab were all modeled with 8-node solid brick elements (C3D8R). The steel welded wire mesh reinforcement was embedded in the concrete slab and modeled using 2-node linear 3D truss elements (T3D2). The shear studs in the composite beam specimens were welded to the top flange of the steel beam and embedded in the concrete slab. The fundamental behavior of the shear studs could be modeled using either: (i) solid elements, or (ii) beam elements with wire connectors. In the models discussed in the subsequent subsections, the shear studs were modeled using 2-node linear beam elements (B31) that were embedded in the concrete slab and attached to the top flange of the beam using a zero-length connector element. The force-displacement response of the connector element in the beam longitudinal direction was specified using the force-slip-temperature behavior developed by Zhao and Kruppa [28]. The connector elements were assigned to be rigid in the other directions.

Shear stud fracture was included in the model by implementing a slip displacement limit in the beam longitudinal direction. The limit was set equal to the slip displacement at the peak load of the force-slip models. For the case of the flat slab, this was 5.2mm. Due to the limitations of the software, this failure displacement criterion for the connector elements had to be independent of temperature. This resulted in a conservative fracture model for the studs because Zhao and Kruppa [28] observed that the slip displacement corresponding to peak load and failure increases with elevated temperature.

The simple connections were modeled using surface-to-surface penalty interactions with finite sliding between the steel surfaces of the connection. Penalty interactions are able to simulate the bearing behavior of bolts on bolt-holes throughout the test. The bolts were snug tightened (not pretensioned) during the experiments. For the simple connections using welds, a tie constraint was used between the connecting elements, rather than modeling the weld metal. The large-scale tests did not include the threads of the bolts within the shear plane of the connection, therefore the bolt threads were excluded from the 3D FEM models as well. The bolt head and nut were also included in the detailed model as shown later.

4. VERIFICATION WITH LARGE-SCALE COMPOSITE BEAM TESTS

The large-scale fire tests conducted by the authors [12-14] were used to verify both the thermal and structural modeling techniques for composite beams with simple connections. Relevant details of the tests are included in this paper as needed to discuss the analysis and benchmarking of the finite element models. The full scope of the tests and the results has been discussed in detail by the authors in [12-14]. Each of the tests used for verification have a heating, loading, and cooling phase of the experiment; and, each of these phases were used to verify the finite element modeling and analysis techniques. The specimens were 3.81m in length and consisted of a wide-flanged steel beams composite with an 88.9mm thick flat lightweight concrete slab. The steel beam in each of the tests was a W10x22 beam, except for the CB-6 specimen, which used a W12x22 beam.

During the testing of the composite beams with simple connections, digital cameras were used to track the horizontal and vertical displacement of targets on the concrete slab in the connection region. These measurements were then used to calculate the average rotation of the connection during the experiment. Vertical and horizontal displacements of the concrete slab in the FEM models were obtained at the same locations as in the experiment and the average rotation of the connection was determined from these displacements. The following sections discuss the comparisons between the experimental results and the FEM predictions in detail.

The axial force in the connection could not be measured during the test due to the limitations of the testing approach, sensors, and adverse (elevated temperature) conditions. The benchmarked models presented in this paper were used to obtain the axial force history in the connection during the loading, heating, and cooling phases of the tests. This axial force history provided insight into experimental behavior and the failure mode when compared with the estimated design strength of the connections.

4.1. Finite Element Mesh

The size of the finite element mesh in the connection regions was governed by the necessary cylindrical contact definition between the bolt shank and the bolt holes. This surface is defined as the bearing surface and is critical for load transfer from the composite beam to the connection. The contact

definition between the bolt head or nut and the beam web or shear-tab (or single-angle, double-angle) is necessary to simulate tightening of the bolts. If these two contact definitions were not critical for simulating connection behavior, then shell-elements rather than 8-node solid elements could be used in the model. Shell-elements offer better computational efficiency, however, cannot capture bearing surface contact definition. Therefore, 8-node solid elements were used to model all of the components of the large-scale composite beam tests. To adequately simulate plate deformation due to bolt bearing, the discretization through the thickness of the bolts holes and at the contact of the bolt head to the beam web or shear-tab (or single-angle, double-angle) was based on prior research [25, 26]. Based on this work, multiple elements were used through the thickness of the bolt holes. Figures 3b-d show the mesh size used in the connection region. Figure 3d shows that the mesh size gradually becomes coarser further away from the bolt holes. The area near the bolt holes is expected to undergo yielding and plastic deformation as the bolt bears on the bolt holes. Therefore, it is necessary to have a finer mesh around this area.

A coarser finite element mesh was used for the steel beam and the concrete slab. A mesh convergence study was performed to evaluate the mesh size in these areas. Two composite beam FEM models were created: (1) using a fine mesh, and (2) using a coarse mesh through the thickness of the slab and the thickness of the beam flanges. The FEM models were analyzed for the same loading, heating and cooling conditions and histories as the composite beam and simple connection specimens CB-2 through CB-6 tested by the authors [12-14]. The fine mesh model used six finite elements through the thickness of the concrete slab and four finite elements through the thickness of the beam flanges. The coarse mesh model used two finite elements through the thickness of the slab, and one finite element through the thickness of the beam flanges. The extremely fine mesh size in the connection region remained constant during both of these simulations. The resulting midspan deflections versus bottom flange temperature are shown in Figure 3a for the midspan and quarter point along the length of the beam. The figure shows insignificant differences between the responses predicted using both models and confirms that the coarse mesh model provides acceptable results for computational efficiency.

4.2. Structural Verification

The time-temperature (t - T) histories measured during the tests were used to define the thermal loading for the nonlinear heat transfer analysis of the FEM models of the corresponding specimens. The resulting temperature distributions and histories throughout the FEM models were comparable to those of the tested specimens. Loading was applied to the composite beam in the FEM model in the same manner as in the large-scale composite beam tests. Each test specimen was loaded at the midspan through a load spreader beam, which was supported on two 76.2mm diameter steel rollers. Two 152mm x 152mm steel plates were hydrostoned to the top of the concrete slab, and used to distribute the load from the rollers to the composite beam. Each plate was located 305mm from the centerline of the composite beam along the length.

Details of each of the large-scale composite beam specimens with simple connections are shown in Table 2. As shown in Table 2, each bolted connection specimen was 152.4mm in height, used 19mm diameter bolts, and provided the AISC *Specification* [29] minimum edge distances. Specimen CB-2 was attached to the columns using 6.4mm thick all-bolted double-angles. Specimens CB-3 and CB-4 used 6.4mm thick shear-tabs to connect the composite beam to the columns. The shear-tabs were bolted to the steel beam and welded to the column. Specimens CB-5 and CB-6 used an all-bolted or all-welded single-angle to attach to the columns. The single-angles used in these tests were 9.5mm thick. In Specimen CB-5, the 127mm angle leg was bolted to the steel beam while the 101.6mm angle leg was bolted to the column. In Specimen CB-6, both angle legs were welded.

4.2.1 Specimen CB-2

Specimen CB-2 consisted of a W10x22 steel beam composite with an 88.9mm thick and 914mm wide lightweight concrete flat slab. 37% partial composite action was achieved using 13mm shear studs spaced at 152mm on center throughout the length of the beam. The welded wire mesh reinforcement was a 6x6 W2.0 x 2.0 wire mesh reinforcement with a cover of 19mm. The composite beam was 3.81m long, and connected to W14x109 steel columns at each end using all-bolted double angle connections. The columns were covered with SFRM to provide 2 hours of fire rating. The specimen was designed for the

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4 composite slab to have a 1 hour fire rating. The entire Specimen CB-2 was modeled as shown in Figures
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6 3b-d. The connection was modeled including the interactions between all of the connection elements.
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9 The order, magnitude, and duration of thermal and mechanical loading applied to the model was
10 similar to those applied to the specimen during testing. Heating was applied to Specimen CB-2 prior to
11 loading at the midspan. The temperature of the underside of the concrete slab, both sides of the steel beam
12 web, and the underside of the steel beam bottom flange was increased at a rate of 7°C/min to a target
13 temperature of 500°C. At this time, concrete spalling was observed at the midspan of the specimen. The
14 heaters were shut off to protect the testing equipment. While the specimen was cooling naturally, the
15 loading was increased to failure, which still occurred at elevated temperature. The specimen failure
16 occurred at a load of 227kN, and the failure was governed by crushing of the concrete slab.
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26 Figures 4a and 4b show comparisons of the experimentally measured and FEM model predicted
27 vertical displacement versus average bottom flange temperature ($\Delta-T$) responses. Figure 4a shows the
28 midspan displacement, and Figure 4b shows the quarter point displacement of the composite beam.
29 Figure 4c shows comparisons of the experimentally measured and FEM model predicted load versus
30 midspan displacement ($P-\Delta$) responses for the composite beam. Similarly, Figure 4d shows comparisons
31 between the experimentally measured and FEM model predicted connection rotation histories. The
32 average connection rotation in the FEM models was determined in the same manner as in the
33 experiments, i.e., using the vertical and horizontal displacement of locations on the composite slab.
34 Figures 4a–d indicate reasonable agreement between the experimental data and the FEM model results.
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47 As mentioned earlier, the axial forces induced in the connections could not be measured during
48 the experiment due to limitations of the test setup and instrumentation. The results from the FEM model
49 in terms of the reaction forces could be used to determine the axial force induced in the connection.
50 Figure 5 shows the resulting axial force history in the double-angle connection during the loading,
51 heating, and cooling phases of the test. Positive values for the axial force in the plot represent
52 compression. Figure 5 includes the connection bolt shear strength calculated using the maximum
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temperature of the bolt shank (293°C) from the FEM simulation. This plot confirms that the axial force in the connection was less than the total shear strength of the bolts. This confirms the experimental observations that there was no fracture failure of the connection during the heating or cooling phases, and shear deformation of the bolts was not visible after the specimen was disassembled. The 3D FEM models also did not illustrate any fracture failure of the connection or shear deformation of the bolts.

4.2.2 Specimen CB-3

Specimen CB-3 was identical to Specimen CB-2 with the exception that shear-tab connections were used to attach the composite beam to the columns. The shear-tab was bolted to the composite beam with two 19mm diameter bolts, and welded to the columns. Specimen CB-3 was tested with load of 156kN applied to the midspan of the composite beam. The load was sustained while heat was applied to the underside of the concrete slab, both sides of the steel beam web, and the underside of the steel beam bottom flange at a rate of 7°C/min for the steel beam, and 4°C/min for the concrete slab. When the temperature of the steel beam reached the target temperature of 600°C on the bottom flange, a cooling curve was implemented. The specimen was cooled at a rate of 12°C/min on both the concrete and steel.

The FEM model simulation was performed in the same order as the experiment: loading, heating, cooling. The temperature measurements from the test were used to specify temperature histories at the corresponding nodes in the FEM models for heat transfer analysis. Figures 6a and 6b show comparisons between the experimentally measured and FEM model predicted vertical displacement versus average bottom flange temperature (ΔT) responses. Figure 6a shows the midspan displacement, and Figure 6b shows the quarter point displacement of the composite beam. Figure 6c shows the comparisons of the experimentally measured and FEM model predicted load vs. midspan displacement responses for the specimen. Similarly, Figure 6d shows the comparison between the experimentally measured and FEM model predicted average connection rotation histories. The targets on the south connection displaced from the camera field of view, therefore, the average connection rotation could not be determined for the south connection. Figures 6a–d indicate reasonable agreement between the experimental data and the FEM model results.

The shear-tab connection fractured during the cooling phase of the test. This fracture occurred in the shear-tab itself, rather than the bolts or welds. Similar fracture failure occurred in the FEM model during the cooling phase, as shown in Figure 7a. The time history of the axial force in the connection ($R-t$) was determined using the reaction forces predicted by the FEM model. The axial force time history ($R-t$) is shown in Figure 7b during the loading, heating, and cooling phases of the test. The positive values in Figure 7b indicate compressive axial forces, and negative values indicate tensile axial forces in the connection. The corresponding connection strengths associated with the limit state of bolt shear failure and weld failure (tensile rupture of base metal) are included in the figure. These connection strengths were calculated using the appropriate maximum temperatures predicted by the FEM model. As shown, the connection axial force during the loading and heating phase is in compression, but it does not exceed the connection strength associated with bolt shear failure. During the cooling phase, the connection axial tension force exceeds the connection strength associated with weld fracture failure (tensile rupture of base metal). Thus, the benchmarked FEM model could predict and supplement the experimental data by determining the axial force time history in the connection and the failure mode observed during testing.

Figure 7b shows the von Mises stresses in the connection region (shear-tab and bolts) at the maximum axial (compression) force during the heating phase, i.e., Point #1 in Figure 7a. As shown in Figure 7b, the von Mises stresses in the bolts are approaching but not exceeding the ultimate strength through the bolt section. Similarly, Figure 7c shows the von Mises stresses in the connection region at the maximum axial (tension) force during the cooling phase, i.e., Point #2 in Figure 7a. As shown in Figure 7c, the von Mises stresses in the shear-tab exceeded the steel yield strength, which eventually led to their fracture as shown in Figure 7a.

4.2.3 Specimen CB-4

Specimen CB-4 was identical to Specimen CB-3 with the exception of the loading, heating, and cooling phases. This specimen also used shear-tab connections to attach the composite beam to the steel columns. Specimen CB-4 was loaded with 111kN prior to heat being applied to the steel beam. Specimen CB-4 was tested twice: once with heat applied to the steel beam and the concrete slab (CB-4a), and once

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4 with heat applied only to the steel beam (CB-4b). The FEM simulation was performed for the CB-4b
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6 specimen, which included loading, heating, and cooling phases. Heating was applied to the steel beam at a
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8 rate of 7°C/min to achieve a target temperature of 700°C. At this temperature, cooling was implemented at
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10 a rate of 12°C/min. The concrete slab was allowed to naturally dissipate the heat applied to the steel beam.
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12 The actual temperature histories of the composite beam obtained during the experiment were used in the
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14 FEM heat transfer analysis.
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18 Figures 8a and 8b show the experimentally measured and FEM predicted displacement versus
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20 average bottom flange temperature (Δ - T) responses for the composite beam. Figure 8a shows the midspan
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22 displacement, and Figure 8b shows the quarter point displacement for the composite beam. Figure 8c
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24 shows comparisons between the experimentally measured and FEM model predicted load versus midspan
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26 displacement (P - Δ) responses. Similarly, Figure 8d shows comparisons between the experimentally
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28 measured and FEM model predicted connection rotation histories. Figures 8a–d indicate reasonable
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30 agreement between the experimental data and the FEM model results.
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34 Similar to Specimen CB-3, the shear-tab of Specimen CB-4 fractured during the cooling phase.
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36 This fracture failure was also observed in the 3D FEM model (similar to Figure 7a). The time history of
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38 the axial force in the connection (R - t) is shown in Figure 9a during the loading, heating, and cooling
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40 phases of the test. The corresponding connection strengths associated with the limit state of bolt shear
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42 failure and weld failure (tensile rupture of base metal) are included in the figure. These connection
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44 strengths were calculated using the appropriate maximum temperatures (357°C) predicted by the FEM
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46 model. As shown, the connection axial force during the loading and heating phase is in compression, but
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48 it does not exceed the connection strength associated with bolt shear failure. During the cooling phase, the
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50 connection axial tension force exceeds the connection strength associated with weld fracture failure
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52 (tensile rupture of base metal). Thus, the benchmarked FEM model could once again predict and
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Figure 9c shows the von Mises stresses in the connection region (shear-tab and bolts) at the maximum axial (compression) force during the heating phase, i.e., Point #1 in Figure 9b. As shown in Figure 9b, the von Mises stresses in the bolts are approaching but not exceeding the ultimate strength through the bolt section. Similarly, Figure 9d shows the von Mises stresses in the connection region at the maximum axial (tension) force during the cooling phase, i.e., Point #2 in Figure 9b. As shown in Figure 9c, the von Mises stresses in the shear-tab exceeded the steel yield strength, which eventually led to their fracture.

4.2.4 Specimen CB-5

Specimen CB-5 was identical to Specimens CB-2 through CB-4 with the exception that all-bolted single-angle connections were used to attach the composite beam to the columns. A load of 111kN was applied at the midspan of the composite beam, the steel was then heated at a rate of 7°C/min to a target temperature of 700°C. At this temperature, cooling was implemented at the rate of 12°C/min. The FEM simulation includes all of the three phases of the experiment, and uses the temperature data from the experiment to conduct heat transfer analysis for both the heating and cooling phases.

Figures 10a and 10b show comparisons of the experimentally measured and FEM model predicted vertical displacement versus average bottom flange temperature (Δ - T) responses. Figure 10a shows the midspan displacement, and Figure 10b shows the quarter point displacement of the composite beam. Figure 10c shows comparisons of the experimentally measured and FEM model predicted applied load versus midspan displacement (P - Δ) responses for the composite beam. Similarly, Figure 10d shows comparisons between the experimental measured and FEM model predicted connection rotation histories. Figures 10a–d indicate reasonable agreement between the experimental data and the FEM model results. As shown, the FEM model over predicts the midspan deflection of the composite beam at higher temperatures (500°C – 700°C) where there is more uncertainty with respect to the temperature dependent material σ - ϵ - T models.

The all-bolted single-angle connection did not fracture during the cooling phase of the test. Similarly, the FEM model also did not have any fracture failures in the connection during the simulation. The time history of the axial force in the connection ($R-t$) was determined using the reaction forces predicted by the FEM model. The axial force time history ($R-t$) is shown in Figure 11a during the loading, heating, and cooling phases of the test and compared with the bolt shear capacity of the connection. The figure includes the connection strength calculated using the bolt shear strength, where the maximum temperature of the bolt shank was estimated based on FEM model results. As shown, the axial force in the connection was in compression only. This was due to the prying action and deformations exhibited by the single angle during the test and shown in Figure 11b.

4.2.5 Specimen CB-6

Specimen CB-6 was similar to the other specimens with the exception that it consisted of a W12x22 steel beam, and all-welded single-angle connections were used to attach the composite beam to the steel columns. Specimen CB-6 was the last composite beam test performed in this testing series. This specimen was loaded and heated in the same manner as Specimens CB-4 and CB-5 where a load of 111kN was applied at the midspan. With the load sustained, heating was applied to the steel beam at a rate of 7°C/min to achieve a target temperature of 700°C. At this temperature, cooling was implemented at a rate of 12°C/min. The 3D FEM model of Specimen CB-6 was similar to those developed for other specimens and shown in Figures 3b-d. The welds in the connection were not modeled explicitly. Instead the single-angle was tied to the beam web and the column at the locations of the welds.

Figures 12a and 12b show comparisons of the experimentally measured and FEM predicted displacement versus bottom flange temperature ($\Delta-T$) responses for Specimen CB-6. Figure 12a shows the midspan displacement, and Figure 12b shows the quarter point displacement of the composite beam. Figure 12c shows comparisons of the experimentally measured and FEM model predicted applied load versus midspan displacement ($P-\Delta$) responses. Similarly, Figure 12d shows comparisons of the experimentally measured and FEM model predicted connection rotation time histories. Figures 12a–d

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4 indicate reasonable agreement between the experimental data and the FEM model results. The FEM
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6 model predicts the connection rotation behavior trend accurately, but the numerical values are slightly
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8 high. This is probably due to the uncertainty in experimental measurements, as well as the simplification
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10 in the model when simulating the welded connection.
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13 The all-welded single-angle connection did not fracture during cooling. Rather, prying of the
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15 single-angle connection was observed after the completion of the test. Similarly, the FEM model
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17 demonstrated prying of the all-welded single-angle connection. Figure 13a shows the connection after
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19 completion of the cooling phase of the simulation. The time history of the axial force in the connection
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21 ($R-t$) was determined using the reaction forces predicted by the FEM model. The axial force time history
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23 ($R-t$) is shown in Figure 13b during the loading, heating, and cooling phases of the simulation. The axial
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25 force in the connection is positive during the test indicating compression forces in the connection only.
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27 This was due to the prying action and deformations exhibited by the angle during the test and shown in
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32 33 **5. CONCLUSIONS AND RECOMMENDATIONS** 34

35 This paper presented the development and benchmarking of detailed 3D finite element models for
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37 predicting the behavior and failure of composite beams with simple connections subjected to combined
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39 gravity and thermal loads. The simple connections included shear-tab, single-angle, and double-angle
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41 connections, which are commonly used in U.S. building construction. These connections were modeled in
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43 detail including all the connecting elements (bolts, plates, angles, etc.) and the contact definitions between
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45 them. In addition the models included inelastic material behavior and force-slip behavior of the shear
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47 studs at elevated temperatures. The numerical analysis included temperature dependent thermal and
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49 mechanical material properties for concrete from Eurocode [20, 21]. The thermal and mechanical
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51 properties of the steel beam and connections elements were modeled using the temperature dependent
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53 model developed by NIST researchers [15, 19].
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57 The detailed FEM models were developed and analyzed using ABAQUS, and were benchmarked
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59 using the results from large-scale tests of composite beams with simple connections. The temperature
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histories measured during the tests were used to specify the thermal loading during nonlinear heat transfer analysis of the models. The resulting nodal temperature histories were specified as thermal loading while conducting nonlinear structural analysis of the models. The objective of the FEM models was to compare the experimental results with those predicted analytically. The comparisons focus on the: (i) measured displacement histories (at midspan and quarter points along the beam length), (ii) measured connection rotation histories, (iii) load-midspan displacement responses, and (iv) the observed connection failure mode. The comparisons indicate that the 3D FEM models predict the experimental behavior, results, and failure mode with reasonable accuracy. There are some differences between the analysis results and experimental measurements at temperatures greater than 500°C, which is attributed to the greater uncertainty in material behavior at higher elevated temperatures.

The benchmarked FEM models were also used to gain additional insight into experimental behavior, and supplement the experimental data obtained during tests. For example, the detailed models were used to determine the von Mises stresses in the connection region, and the axial force histories induced in the connections during the experiments. These stresses and axial forces in the connection region could not be measured due to the limitations of instrumentation and measurements at high temperatures and extreme conditions during the test. The axial force time histories from the FEM models indicated that: (i) the shear-tab connection would fracture during the cooling phase, but (ii) the all-bolted double-angle and single-angle connections would not fracture. This was also observed both experimentally and analytically. The shear-tab connection fractured due to the large axial tension forces during the cooling phase. The single- and double-angle connections had prying action deformations during the cooling phase, which relieved the axial tension forces, and therefore they did not fracture. The FEM analysis results indicated that the bolts would not fracture during the test, which was also observed experimentally.

The benchmarked models and approaches are recommended for conducting analytical parametric studies to expand the experimental database, and to enhance results and findings obtained by conducting large-scale experiments of composite beams with simple connections during fire. For example, the

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4 models are being used by the authors to examine the effects of different composite beam and connection
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6 parameters that could not be included in the experimental investigations due to practical considerations.
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8 These include effects of beam length, concrete slab thickness and type including profiled metal deck, steel
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10 reinforcement in the concrete slab, and the size and stiffness of the columns. Additionally, the
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12 benchmarked models can be used to investigate the effects of slab continuity and simple connections on
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14 the fire behavior single- and multi-bay floor assemblies. The results from these parametric studies will be
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16 included in future publications.
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30 are those of the authors and do not necessarily reflect the views of the sponsors.
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Table 1: Parameters used in uniaxial stress-strain relationship for steel [19]

Parameter	Value	Units
k_1	4.92	
k_2	575	°C
k_3	734	MPa
k_4	0.315	
n_1	4.51	
n_2	637	°C
n_3	0.329	
n_4	-423×10^{-4}	MPa ⁻¹

Table 2: Test matrix for simple (shear) connections of composite beams

Specimen	Connection Type	Beam Size	Diameter of Bolts mm (inch)	Size of Weld mm (inch)	Thickness of Simple Connection mm (inch)	Angle Dimensions mm (inch)
CB-2	All-bolted Double-angle	W10x22	19 (0.75)	N/A	6.35 (0.25)	127x127
CB-3	Shear-tab	W10x22	19 (0.75)	7.9 (0.31)	6.35 (0.25)	N/A
CB-4	Shear-tab	W10x22	19 (0.75)	7.9 (0.31)	6.35 (0.25)	N/A
CB-5	All-bolted Single-angle	W10x22	19 (0.75)	N/A	9.5 (0.375)	127x101.6
CB-6	All-welded Single-angle	W12x22	N/A	6.35 (0.25)	9.5 (0.375)	127x101.6

Figure 1: Stress-strain-temperature relationship using NIST [19] and Eurocode [20] (a) true stress-true strain relationship for 400°C, 500°C, and 600°C, and (b) retention factor for steel as a function of temperature at 0.2% strain offset

Figure 2: Temperature dependent force-slip behavior for shear connectors with flat concrete slab (based on Zhao and Kruppa [28])

Figure 3: 3D FEM model mesh details: (a) effect of mesh refinement on $P-\Delta$ responses at midspan and quarter point, (b) 3D FEM model of 19mm ASTM A325 bolts, (c) overall 3D FEM model for composite beam with simple connections, (d) zoomed-in view of FEM model of simple connection

Figure 4: Comparisons of experimentally measured and FEM model predicted responses for Specimen CB-2 (a) midspan displacement versus average bottom flange temperature, (b) quarter point displacement versus average bottom flange temperature, (c) applied load versus midspan displacement, (d) average connection rotation versus time

Figure 5: Axial force in connection from FEM model of Specimen CB-2

Figure 6: Comparisons of experimentally measured and FEM model predicted responses for Specimen CB-3: (a) midspan displacement versus average bottom flange temperature, (b) quarter point displacement versus average bottom flange temperature, (c) applied load versus midspan displacement, (d) average connection rotation versus time

Figure 7: Connection forces predicted by FEM analysis of Specimen CB-3: (a) fractured shear tab from FEM simulation at failure (Point #2) using element deletion of yielded finite elements, (b) axial force in connection versus time, (c) Von-Mises stresses in connection region at Point #1, (d) Von-Mises stress in connection region at Point #2 without element deletion

Figure 8: Comparisons of experimentally measured and FEM model predicted responses for Specimen CB-4: (a) midspan displacement versus average bottom flange temperature, (b) quarter point displacement versus average bottom flange temperature, (c) applied load versus midspan displacement, (d) average connection rotation versus time

Figure 9: Connection forces predicted by FEM analysis of Specimen CB-4: (a) fractured shear tab from FEM simulation at failure (Point #2) using element deletion of yielded finite elements, (b) axial force in connection versus time, (c) Von-Mises stresses in connection region at Point #1, (d) Von-Mises stress in connection region at Point #2 without element deletion

Figure 10: Comparisons of experimentally measured and FEM model predicted responses for Specimen CB-5: (a) midspan displacement versus average bottom flange temperature, (b) quarter point displacement versus average bottom flange temperature, (c) applied load versus midspan displacement, (d) average connection rotation versus time

Figure 11: Connection forces predicted by FEM analysis of Specimen CB-5: (a) axial force in connection region versus time, (b) prying of single-angle connection

Figure 12: Comparisons of experimentally measured and FEM model predicted responses for Specimen CB-6: (a) midspan displacement versus average bottom flange temperature, (b) quarter point displacement versus average bottom flange temperature, (c) applied load versus midspan displacement, (d) average connection rotation versus time

Figure 13: Connection forces predicted by FEM analysis of Specimen CB-6: (a) prying of single-angle connection, (b) axial force versus time for connection region

Figure1a

Figure1b

Figure2

Figure3a

Figure3b

Figure3c

Figure3d

Figure4a

Figure4b

Figure4c

Figure4d

Figure5

Figure6a

Figure6b

Figure6c

Figure6d

Figure7a

Figure7b

Figure7c

Figure7d

Figure8a

Figure8b

Figure8c

Figure8d

Figure9a

Figure9b

Figure9c

Figure9d

Figure10a

Figure10b

Figure10c

Figure10d

Figure11a

Figure11b

Figure12a

Figure12b

Figure12c

Figure12d

Figure13a

Figure13b