*Proceedings of the*
*Annual Stability Conference*
*Structural Stability Research Council*
*Grapevine, Texas, April 18-21, 2012*

# Enhancing the Fire Performance of Concrete-Filled Steel Columns through System-Level Analysis

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## Abstract
The use of concrete filling offers a practical alternative for achieving the required stability of steel Hollow Structural Section (HSS) columns under fire conditions. However, current methods for evaluating fire resistance of Concrete Filled Hollow Structural Steel (CFHSS) columns are highly conservative as they are based on an elemental approach without due consideration to structural interactions that occur in framed structural systems. To overcome this limitation, a system level fire resistance analysis was carried out by treating CFHSS columns as part of an overall structural frame. In this analysis, an eight story steel-framed building was modeled under a range of standard and performance-based fire scenarios (including multi-story progressive burn-out fires) to evaluate the contribution of various structural members/assemblies to overall fire resistance. One of the primary factors considered was the use of concrete filling in HSS columns as an alternative to standard W-shape columns. Results from the analysis indicate that the use of CFHSS columns, in place of W-shape columns, in a performance-based environment can fully eliminate the need for applied fire protection to columns, while providing the required level of structural fire resistance.

## Introduction
Steel Hollow Structural Sections (HSS) are very efficient in resisting compression, torsional, and seismic loads, and are widely used as compressions members in the construction of steel framed structures. Structural stability under fire exposure is one of the primary considerations in the design of high-rise buildings, and hence, building codes require fire protection for HSS columns to maintain overall structural stability in the event of fire. Providing such external fire protection to HSS columns involves additional cost, reduces aesthetics, increases weight of the structure, and decreases usable space. Also, durability of fire insulation (adhesion and cohesion of insulation to steel) is often a questionable issue, and hence, requires periodic inspection and regular maintenance, which in turn, incurs additional costs during the lifetime of the structure (FEMA 2002, NIST 2005)

Often these HSS sections are filled with concrete to enhance the stiffness, torsional rigidity, and load-bearing capacity of columns. The two components of the composite column complement

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each other ideally. The steel casing confines the concrete laterally, allowing it to act as in tri-axial compression and develop its optimum compressive strength, while the concrete, in turn, enhances resistance to elastic local buckling of the steel wall and global buckling of the column. In addition, the concrete filling in HSS columns insures a high fire resistance, without any external fire protection to the steel, thus increasing the usable space in the building and removing the need for application and maintenance of the fire insulation. Properly designed concrete-filled columns can lead economically to the realization of architectural and structural design with visible steel, but without any restrictions on fire safety (Kodur and Lie 1995a, Klingsch and Wuerker 1985, and Twilt et al. 1996).

Design provisions for achieving fire resistance through concrete filling have been incorporated into codes and standards (Bond 1975, Klingsch and Wittbecker 1988, and Kodur and Lie 1995b). These provisions are however unnecessarily prescriptive in nature since they are based on fire resistance tests and numerical simulations conducted on single columns under standard fire exposure, without due consideration to realistic fire exposures, structural interactions, load levels, and failure criterion. Thus, current provisions do not fully capture the inherent fire resistance that can be achieved through the use of CFHSS columns, and unnecessarily preclude the use of CFHSS columns in numerous applications such as airport, schools, and atriums where exposed steel is highly desired.

This paper presents a state of the art review on the fire resistance of CFHSS columns, the drawbacks of the current approach for evaluating fire resistance, and a brief overview of the factors to be considered in a performance-based design. Results from a numerical study on an eight story steel framed building incorporating CFHSS columns under fire exposure are presented. The analysis is carried out using finite element based computational model SAFIR, wherein the material and geometric non-linearity and stability-based failure criterion are considered. Results from the analysis are utilized to quantify the additional fire resistance that can be achieved in CFHSS columns through system-level analysis under a performance-based environment.

## State-of-the-Art

Alternate approaches for achieving fire resistance of CFHSS columns have been studied for the last three decades. Methods such as filling the HSS columns with liquid (water) and concrete, were studied by researchers (Kodur and Lie 1995a, Bond 1975, and Klingsch and Wittbecker 1988). However, the use of concrete-filling is the most attractive and feasible proposition developed by researchers.

### *Experimental and Numerical Studies*

The fire resistance tests on CFHSS columns were predominantly carried out at the National Research Council of Canada (NRCC), a few organizations in Europe, and more recently in China. The experimental program at NRCC consisted of fire tests on about 80 full-scale CFHSS columns (Kodur and Lie 1995a,b, Lie and Chabot 1992, Lie and Caron 1988, Lie et al. 1991). Both square and circular HSS columns were tested, and the influence of various factors, including type of concrete filling (PC, RC, and FC), concrete strength, type and intensity of loading, and column dimensions were investigated under the ASTM E-119 (ASTM 2001) standard fire exposure condition. The tests reported by other European and Chinese studies

(Klingsch and Wittbecker 1988, Grandjean et al. 1981, and FCSA 1989) are similar to NRCC tests, but the fire exposure was that of the ISO 834 (ISO 1975) or other equivalent standard fire; whose time-temperature curve is similar to that of ASTM E-119 (ASTM 2001).

The numerical studies, primarily carried out NRCC and more recently in the U.S. and elsewhere, consisted of development of mathematical models for predicting the fire behavior of circular and square CFHSS columns [16-18]. In these models, the fire resistance is evaluated in various time steps through cross-sectional analysis. The analysis consists of calculating the temperature of the fire to which the column is exposed, the temperatures in the column cross-section, its deformations, and strength during exposure to fire, and finally, deriving fire resistance of the column. Full details on the development and validation of these models are given in references (Lie and Chabot 1992, Kodur 1997, Lie and Chabot 1990, Lie and Kodur 1996, and Kodur and Lie 1997).

Data reported from NRCC tests and numerical studies can be used to illustrate the behaviour of concrete-filled HSS columns under fire conditions. Figure 1 shows the variation of the axial deformation as a function of time for 3 typical HSS columns filled with one of three types of concrete, namely: plain concrete (PC), steel fiber reinforced concrete (FC) and bar reinforced concrete (RC) (Kodur and Lie 1995b). The three columns had similar dimensions and loading conditions, and the results can be used to illustrate the comparative fire behaviour of the three types of concrete filling.

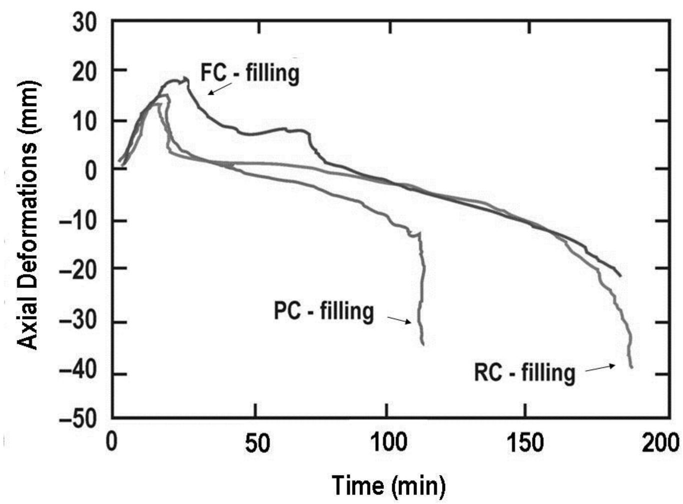


Figure 1: Axial deformation in CFHSS columns as a function of time

At room temperature, the load is carried by both the concrete and the steel. When the column is exposed to fire, however, the steel carries most of the load during the early stages because the steel section expands more rapidly than the concrete core. At higher temperatures, the steel section gradually yields as its strength decreases, and the column rapidly contracts at some point between 20 and 30 minutes after exposure to fire. At this stage, the concrete-filling starts carrying more and more of the load. The strength of the concrete decreases with time, and ultimately, when the column can no longer support the load, failure occurs either through buckling or compression. The elapsed time that it takes for the column to fail is the measure of its fire resistance. The behavior of the column, after steel yields, is dependent on the type of concrete-filling. Both FC and RC-filled HSS columns have higher fire resistance than PC-filled HSS columns.

It can be seen in Figure 2 that the deformation behaviour of the FC-filled steel column is similar, during the later stages of fire exposure, to that of the RC-filled steel column. The initial higher deformations in the FC-filled HSS column might be due to higher thermal expansion of steel fiber-reinforced concrete. The fire resistance of the RC-filled HSS column is slightly higher than that of the FC-filled HSS column, which in turn is significantly higher than the PC-filled HSS column. Results from the fire resistance tests conducted at NRCC indicated that plain concrete filled HSS columns can be relied upon to provide 1-2 hours of fire resistance, steel fibre reinforced columns can provide 2-3 hours of fire resistance, and bar reinforced columns can provide in excess to 3 hours of fire resistance under standard ASTM E-119 (ASTM 2001) fire exposure.

*Components of Performance-Based Design*

The above fire tests have been conducted primarily under standard fire conditions, recently however, there has been an increased impetus on moving toward a performance-based approach for fire safety design (Meacham and Custer 1992, and Kodur 1999). This is mainly due to the fact that the current prescriptive-based approach has serious limitations restricting the use of alternate, cost effective solutions for providing fire resistance. There are two basic methods by which performance-based fire safety design can be accomplished: fire tests can be performed on a representative structure, or, numerical/computational simulations can be carried out by exposing the simulated structure to realistic fire, loading, and restraint conditions. Due to the high cost, time, and effort associated with full-scale fire testing, the first option is mostly used to validate numerical models. The second option of utilizing numerical modeling allows the consideration of most significant factors that influence fire resistance. The most important factors to be considered in performance-based fire safety design are fire scenario, load conditions, member interactions, and failure limit states (Parkinson and Kodur 2007). These main components are discussed here:

*Fire Scenario*

The current practice of evaluating fire resistance of CFHSS columns is based on standard fire tests or computer models, in which the column is exposed to a standard fire as specified in standards such as ASTM E-119 (ASTM 2001) or ISO 834 (ISO 975). While standard fire resistance tests are useful benchmarks to establish the relative performance of different CFHSS columns under standard fire condition, they should not be relied upon to determine the survival time of CFHSS columns under realistic fire scenarios. Nor does the standard heating (fire) condition bare any resemblance to the often less severe heating environments encountered in real fires.

Figure 2 illustrates various time-temperature curves for standard and some realistic (design) fire scenarios (Magnusson and Thelandersson 1970). In the standard fires (ASTM E-119 fire and hydrocarbon fire) (Lie and Chabot 1992, and FCSA 1989), the fire size is the same (irrespective of compartment characteristics), temperature increases with time throughout the fire duration, and there is no decay phase. However, in real fires, the fire size is a function of compartment characteristics, such as ventilation, fuel load, and lining materials, and there is a decay phase as clearly shown in Figure 2 (design fires FV02 - FV12) (Magnusson and Thelandersson 1970). In the decay phase of the realistic fire scenarios, the cross section of the column enters the cooling phase, in which the concrete and/or steel recovers part of its strength and stiffness, and thus, the fire resistance of the column increases.

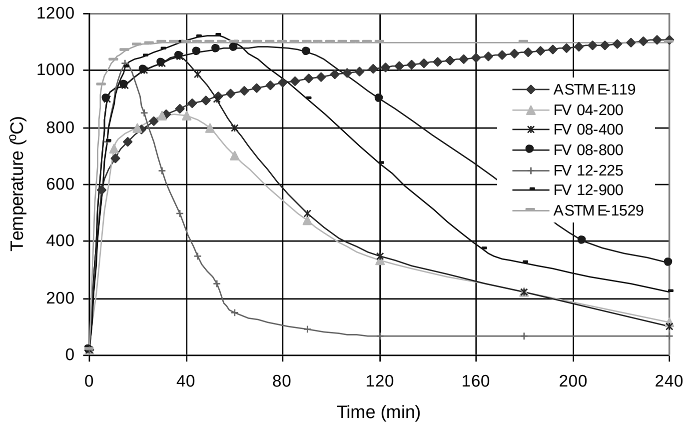


Figure 2: Time-temperature relationships for various fire scenarios

*Load Level*

The current codes of practice for evaluating fire resistance through standard fire tests are generally based on a load ratio of about 50%. Load ratio is defined as the ratio of the applied load on the column under fire conditions to the strength capacity of the column at room temperature. Load ratio depends on many factors including the type of occupancy, the dead load to live load ratio, the safety factors (load and capacity factors) used for design under both room temperature and fire conditions. The loads that are to be applied on CFHSS columns, in the event of fire, can be estimated based on the guidance given in ASCE-07 standard (ASCE 2005) (1.2 dead load + 0.5 live load) or through actual calculations based on different load combinations. Based on ASCE-07 (ASCE 2005) and AISC LRFD Manual (AISC 2005), and for typical dead to live load ratios (in the range of 2 to 3), the load ratio for CFHSS columns ranges between 30% and 50%.

Further, the load ratio might influence the fire resistance of CFHSS columns calculated based on realistic failure criteria. Thus, for innovative, realistic, and cost effective performance-based fire safety design, it is important to evaluate the fire resistance of CFHSS columns based on actual load levels.

*Member Interactions*

Fire resistance tests and numerical simulations involve a high degree of complexity in order to account for the combined thermo mechanical loading on the member. As a means of reducing testing costs, and overall complexity, the majority of fire resistance tests/models only consider a single structural member exposed to fire, with simulated end conditions. This practice fails to consider the beneficial effect of member interactions as a structural member is exposed to extreme temperatures. Member interactions facilitate redistribution of loads to other cooler parts of the structure, limit temperatures in critical areas, and provide stability to a failing member (Fike 2010). In order to accurately predict the fire resistance of structural members and achieve a truly performance based environment; consideration must be given to the structural interactions affecting a member during fire exposure.

*Failure Criterion*

Generally in fire resistance tests and numerical analysis of structural members, a temperature limiting criterion is applied to define failure. It is commonly assumed that once the steel section reaches a critical temperature of 538°C, approximately 50% of the room temperature strength is lost (Eurocode 2005), and failure is eminent. While sufficient for traditionally protected steel sections, the effect of the concrete core is not captured by this thermal failure criterion. To correct this, stability of the column under fire conditions needs to be considered. Columns can fail globally either by buckling or crushing depending on slenderness. Depending on factors such as end conditions, concrete stiffness, buckling or crushing could occur well after the limiting temperature of 538°C is reached. If a column is adequately restrained on both ends via a fixed-fixed connection, fire resistance can be enhanced appreciably due to redistribution of moments between critical sections. Additionally, local stability should be accounted for. CFHSS columns can fail locally without collapse due to crushing of the concrete on the inside, or local buckling of the steel wall (Kodur and Lie 1997, Kodur 2005, and Kodur and Lie 1996).

*Limitations of Current Provisions*

The reported experimental and numerical work has offered considerable insight into the response of CFHSS columns under fire exposure. However, the fire resistance tests were carried out under standard test conditions, and fail to account for realistic fire scenarios, loading, member interactions, and failure criterion. Thus, the current state of the art has resulted in a prescriptive approach wherein CFHSS columns cannot be relied upon to provide practical fire resistance levels required in most buildings without the application of external fire protection.

**Numerical Studies**

To overcome the above limitations, a set of numerical studies were carried out to evaluate the fire performance of CFHSS columns by employing a system level approach rather than treating CFHSS columns as single elements. The analysis was carried out using a finite element based computer program SAFIR, and by exposing different types of CFHSS columns to various fire and loading scenarios. Some of the details associated with the analysis are discussed here, for a full description of the analysis, the interested reader is directed to Reference (Fike 2010)

*Computer Program*

To evaluate the feasibility of using unprotected CFHSS columns to resist fire when they are part of an overall structural frame (building), SAFIR is employed to simulate an eight story steel framed building under fire exposure. Utilizing results from the numerical simulations, the feasibility of achieving unprotected steel though the use of CFHSS columns is evaluated.

The computer program SAFIR is capable of modeling: multiple materials in a cross section, cooling phase of a fire, large displacements, effects of thermal strains, non-linear material properties according to Eurocode 3, and residual stresses (Franssen 2005). Additionally, SAFIR allows the user to input any time-temperature relationship to facilitate the use of design fire scenarios.

In SAFIR, the thermal and structural analysis are performed independently. The thermal model consists of 2D solid elements where the fire exposed sides and the exposure types are specified by the user. The thermal model in SAFIR neglects heat transfer in the longitudinal direction and the effect of hydraulic migration in concrete.

For structural analysis, SAFIR uses a fiber-based approach wherein each of the solid elements in the thermal model is considered to be a fiber in the structural model. A stress and temperature dependent stiffness matrix is established that incorporates each of the fibers. Due to increasing temperature in the column, the stiffness decreases to a point where the composite column can no longer support the applied load, and failure occurs. Through the use of beam elements to simulate columns, both crushing and buckling failure of the columns can be captured. Limitations of the structural model include the assumption that there is no slip between the steel and the concrete.

*Building Description*

The building selected for the fire resistance evaluation was an eight story steel framed building constructed in 2008, and represents typical office and health care facilities built according to current non-seismic construction practices in the United States. Though the building represents typical construction for hospital and office buildings, the methods developed in this research are only sufficient to satisfy fire resistance requirements as they pertain to office buildings. This is because typical sprinklered office buildings generally require less fire resistance (1-2 hours) than health care facilities (3-4 hours), henceforth, for this reason, the building will be referred to as an office building.

The structural framing details of this building were acquired from the design firm responsible for the design and construction of the structure. A partial structural framing plan for the 5$^{th}$ story is shown as Figure 3. The secondary beams used are W16x26 and the primary beams spanning on line C and D in the middle of the structure are W21x57 sections, the edge beams are W30X124. In plan, the building is 56.7 m by 29.3 m, each of the bays is 10 m by 10 m. Emergency evacuation stairs are located in two of the corners with a third set of stairs located in the center of the building as seen in Figure 3.

Connections used in the structural frame consist of both shear and moment connections. Moment connections are provided in the exterior frames to provide resistance to lateral (wind) force. All connections on the interior of the structure, including those on the ends of secondary beams, are shear connections.

The floor slab in the building is comprised of a composite deck, shear studs, and shrinkage reinforcement. The deck in each story of the structure was poured in one continuous pour without shoring to the floor beams.

The first four stories of the structure are larger in plan to accommodate other service functionalities. As such, the fifth floor was selected for fire resistance analysis to eliminate the interaction of multiple functions on the same floor, and to reduce the size of the structure being modeled.

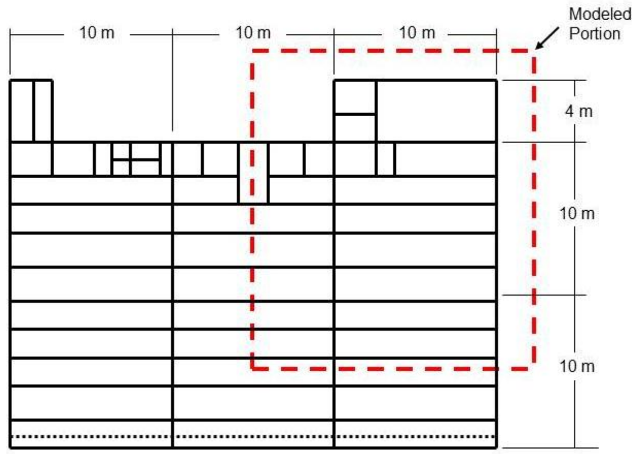


Figure 3: Framing plan for the steel framed building used to illustrate application of the developed methodologies

*Numerical Model*

Having selected a typical building to be used in the fire resistance analysis, it was necessary to construct a numerical model of the building for fire resistance analysis. While a model of the entire building would be most desirable in terms of simulation results, this is impractical due to the numerous simulations (case scenarios) desired. It was therefore decided that two levels of analysis would be conducted. As such, for fire resistance analysis a one story model was created to assess the feasibility of CFHSS columns resisting fire when the entire floor is under fire, and a three story model was constructed to assess the ability of CFHSS columns to withstand a multi-story (three) fully developed fire including a cooling phase. As described in the following section, great care was taken in constructing the model to accurately capture the system level behavior of the structure, and to fully capture the contribution of composite construction to the fire resistance of the steel framed structure.

*Discretization*

Due to the complexities that are associated with modeling a ribbed composite floor slab, it was decided to use a slab represented by a shell element of uniform thickness as has been done in previous studies (Cashell et al. 2008, and Zhang and Li 2008). The thickness chosen for the slab was that of the thickest part of the ribbed flooring system, 130 mm. This slab was modeled using four nodded shell elements with six degrees of freedom at each of the nodes. The beams supporting the slab and the columns were modeled using three nodded beam elements with seven degrees of freedom at the end nodes, and one degree of freedom at the center node. To simulate the composite action between the steel beam and the floor slab, the “SAMEALL” command was used for the nodes where the beam and shell elements coincide. This caused all of the translations and rotations at these points to be the same for the beam and slab, thus simulating the fully composite condition.

Due to the large floor plan for the entire structure, and the computational time associated with such a large model, only a portion of the building, indicated in Figure 3, was modeled taking advantage of the building symmetry. This section was modeled as a single story ($5^{th}$ story) as shown in Figure 4 and a three story model ($5^{th}$, $6^{th}$, $7^{th}$ stories) as shown in Figure 5. Modeling only the portion of the structure as illustrated in Figure 3, requires that particular attention be given to the boundary conditions used in the simulation. It was also assumed that the portion of the structure that was not modeled, being significantly larger than the modeled potion, was essentially rigid compared to the modeled portion. As such, the horizontal translation on the continuous edges was fully restrained perpendicular to the modeled edge. Due to the continuity of the slab over these points, the rotation about the length of the edge was assumed to be restrained, thus simulating the realistic support offered by the portion of the structure which was not modeled.

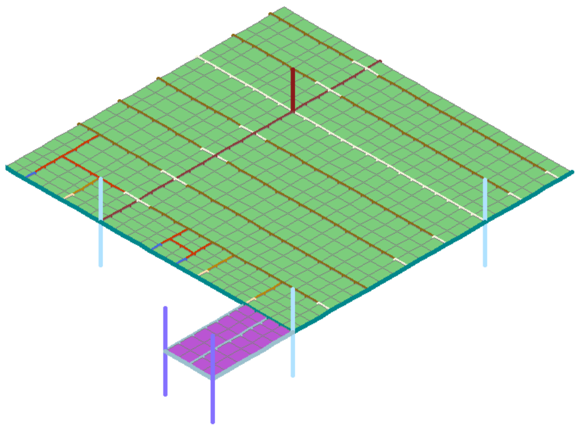

Figure 4: One story numerical model of selected building

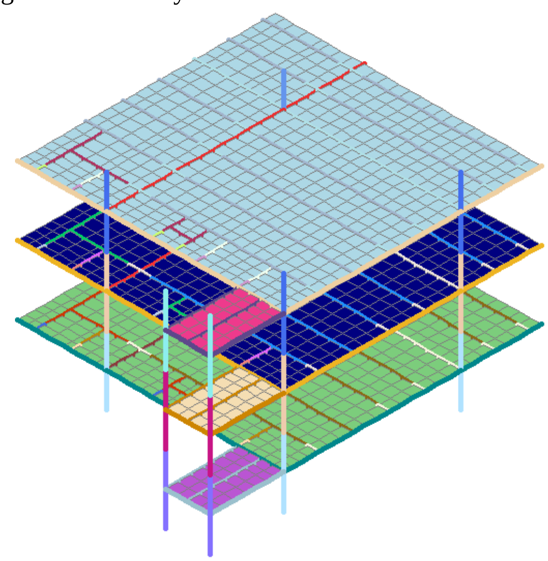

Figure 5: Three story numerical model of selected building

It was assumed in the models that the entire floor area with the exception of the stair area was exposed to fire. The stair area was designated in the plans as a fire escape rout, thus, the walls separating the stairs from the rest of the structure provided a 3-hour fire resistance rating. Loading on the structure was taken to be 4.5 kN/m$^2$ based on the reduced loads present during fire exposure, and the loads used in the Cardington test (British Steel 1998). This is a typical load for office buildings and is based on 1.2DL + 0.5LL. It was assumed that the fire exposure was the same to all parts of the fire exposed member. That is to say that the ends of beam that would be protected from direct fire exposure in a real structure, were modeled as being fully exposed to the same severity of fire as experienced by the center of the beam span. Beams and

W-Shape columns on the perimeter of the structure were assumed to be supplied with fire insulation. It was assumed that secondary beams which attach to protected structural members were also protected with the fire insulation for 150 mm past the connection, thus allowing the connection to remain cool enough that its structural integrity was not compromised. This assumption was invoked to eliminate the need to consider the complex behavior of the connection at elevated temperatures.

Four design fire scenarios and ASTM E-119 fire exposure were considered in the analyses, the design fires were developed to represent fires that could occur in a typical steel framed office building. In the development of these fires, structural geometry and typical fuel loads are taken into consideration (Fike 2010). The developed fire exposures are shown in Figure 6. All one story simulations were run for 3 hours, or until the structure became unstable and the simulation indicated failure of the structure. For the three story models, it was assumed that it took one hour from ignition of the first floor until the second floor was exposed to fire, and subsequently it took another hour for the third floor to be exposed to fire. As such, in all three story cases, the first floor was experiencing a fully developed fire when the second floor was ignited, when the third floor ignited, the second floor was fully developed and the first floor was in the cooling phase. The simulations were continued until the third floor had been exposed to three hours of fire exposure (for total fire duration of 5 hours) or until SAFIR indicated that the structure was unstable and failure of the simulation occurred.

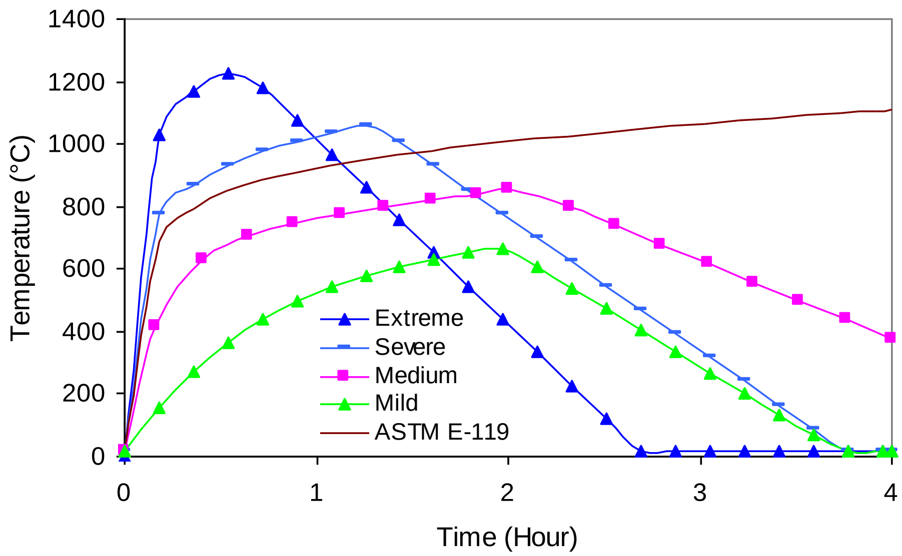


Figure 6: Possible fire scenarios used in the fire resistance analysis

The fire resistance analysis was carried out for a total of 11 cases which correspond to different fire scenarios, column configurations, and floor slab types. The analysis matrix corresponding to these cases is presented in Table 1. The analysis with Case 1 was conducted to assess the performance of the structural frame without any fire protection on the columns (W-shape) or secondary beams to determine the "weak link" in the structural system under fire conditions. Case 2 analysis was conducted to assess the beneficial effects of composite construction on the fire performance of the structure if the wide flange columns in Case 1 were replaced with equivalent CFHSS sections designed as per AISC provisions (AISC 2005). For the remaining cases (Case 3 through Case 11), the columns and floor system (concrete slab) were replaced with CFHSS columns and SFRC floor systems (respectively) designed according AISC provisions. One and three story models of this modified structural system were exposed to all of the design fires shown in Figure 6 to assess response of the structure to fire exposure. Failure times and location of failure for each case are presented in Table 1. Due to space limitations, only the results from these analyses pertaining to the use of CFHSS columns will be discussed, for a more complete discussion on the full results from these analyses, the interested reader is directed to the literature (Fike 2010).

Table 1: Various structural configurations and fire scenarios simulated in the building

| | Fire exposure | # of stories under fire | Column configuration | Floor slab Configuration* | Fire resistance (min) | Failure zone/member |
|---|---|---|---|---|---|---|
| Case 1 | ASTM E-119 | 1 | W-Shape | PC | 16.5 | W-column |
| Case 2 | ASTM E-119 | 1 | CFHSS | PC | 58 | Floor slab |
| Case 3 | ASTM E-119 | 1 | CFHSS | SFRC | 118 | Floor slab |
| Case 4 | Extreme | 1 | CFHSS | SFRC | 12.5 | Floor slab |
| Case 5 | Extreme | 3 | CFHSS | SFRC | 13 | Floor slab |
| Case 6 | Severe | 1 | CFHSS | SFRC | 37 | Floor slab |
| Case 7 | Severe | 3 | CFHSS | SFRC | 39 | Floor slab |
| Case 8 | Medium | 1 | CFHSS | SFRC | No failure | None |
| Case 9 | Mild | 1 | CFHSS | SFRC | No failure | None |
| Case 10 | Medium | 3 | CFHSS | SFRC | No failure | None |
| Case 11 | Mild | 3 | CFHSS | SFRC | No failure | None |

*PC = Plain Concrete, SFRC = Steel Fiber Reinforced Concrete

*Model Validation*

For validation of the numerical model, due to the lack of information on the response of the considered structure to fire exposure, it is necessary to consider the structural response of the building at ambient temperatures. To accomplish this, structural analysis on the 8-story building under ambient temperature was conducted utilizing SAFIR with a design load of 9.8 $kN/m^2$ to represent a typical ambient temperature design load on the structure. Deflection predictions from SAFIR were then compared with deflection limits specified in codes and standards. A maximum deflection of 19.9 mm was observed in the center of the secondary W16x26 beams. By considering the deflection limit to be L/480 for a structural member supporting a structural element that could be damage by deflections, the allowable deflection of the beam is 20.8 mm, thus, the deflection limit is satisfied. Additionally, the deflected shape of the structure returned from the simulation, as shown in Figure 7, is intuitively correct based on basic structural principals.

Given that the deflection limit is satisfied by predictions from the numerical model, and that the deflected shape of the structure is intuitively correct, it is concluded that the structural model of the eight story steel framed building is realistically constructed in SAFIR. While full validation cannot be completed for the structure due to a lack of fire response data, it should be noted that the same model will be used in all of the simulations. As such, any error in model construction would be applied to all of the models. Thus, any error resulting from the numerical idealization of the structure will exist in all cases, and be self-canceling for comparison purposes.

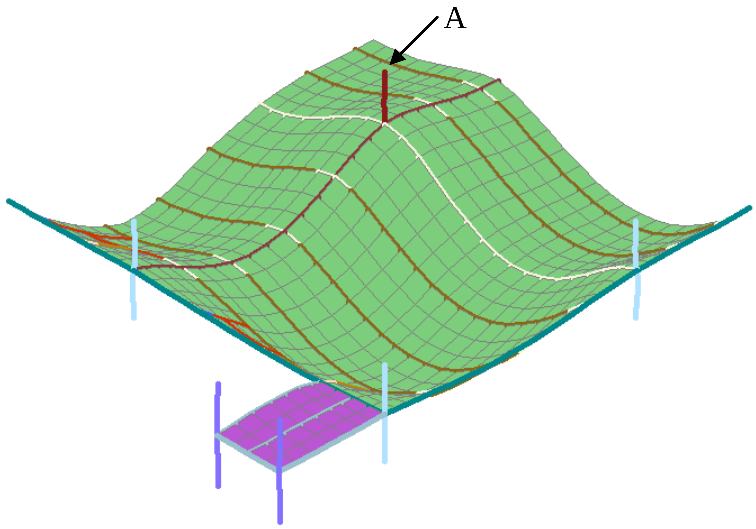


Figure 7: Deflected shape under service loads and ambient temperature (scale 1:125)

**Parametric Study**

Using the models described and validated above, the simulations corresponding to the 11 cases shown in Table 1 were conducted. Cases with particular implications to the contribution of CFHSS columns to fire resistance are presented below.

*Case 1*

To form a baseline for the study, the structural frame was simulated under ASTM E-119 fire exposure assuming that the primary and exterior beams, and the exterior W-shape columns had 2-hour fire protection, while the interior columns and secondary beams had no protection. Failure occurred in this simulation at 16.5 minutes due to failure of the central unprotected W14x61 column, which is marked with an "A" in Figure 7. Failure occurred early into the fire exposure due to the high temperatures reached in the unprotected steel section. Axial deformation at the top of the column is plotted as a function of fire exposure time in Figure 8. From Figure 8, it can be seen that the column failed suddenly at 16.5 minutes. Results from this simulation indicate that failure of the column, which was loaded to approximately 40% of its design strength, occurred at an average section temperature of 625 °C due to global buckling of the section.

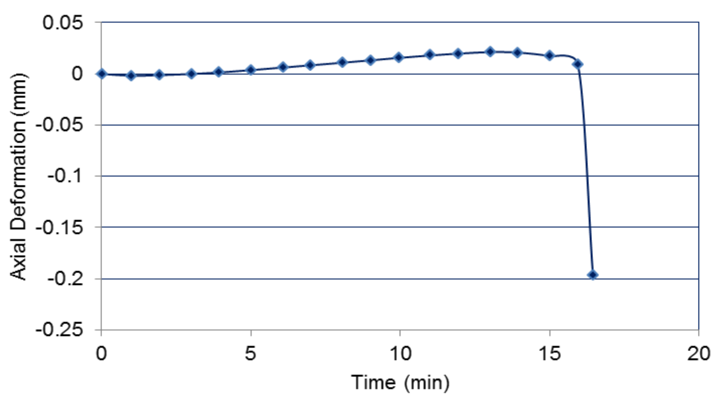


Figure 8: Variation of central W-shape column axial deformation with time corresponding to for Case 1 analysis

*Cases 2 and 3*

Results from Case 1 indicate that the weak link in the simulated building was the W-shape column. To explore the possibility of improving fire resistance through the use of CFHSS columns, an equivalent CFHSS column was selected to replace the W14x61 section used in Case 1. Design of the column was conducted according to AISC (AISC 2005) ambient temperature strength design. It was found that a 254 mm square HSS column with 12.5 mm thick walls and filled with carbonate aggregate concrete with a compressive strength of 35 MPa will provide equivalent load capacity at ambient temperatures, and a two hour fire resistance rating.

The fire resistance analysis of the entire frame, with CFHSS columns, was carried out using SAFIR. Results from the simulation indicate that the structure fails at approximately 58 minutes due to instability in the plain concrete floor system. To overcome the limitation of the floor assembly, based on previous work by the authors, (Fike and Kodur 2011) Case 3 utilized a steel fiber reinforced concrete floor slab that was postulated to have a higher fire resistance than the existing plain concrete floor assembly. With this change implemented, results from the SAFIR analysis indicated that the structure was able to withstand the effects of fire for 118 minutes under ASTM E-119 fire exposure. Again however, failure of the structure was due to weakness in the floor assembly.

Despite the premature failure of the simulation, the results can be utilized to assess the response of the CFHSS column under fire exposure. Figure 9 illustrates the axial deformation of the end of the column as a function of fire exposure time. From Figure 9 it can be seen that the column initially expanded followed by rapid contraction at approximately 36 minutes. This indicates that the HSS section was unable to support the applied load beyond 36 minutes, and the load transferred to the cooler concrete core which then supported the structure for the duration of the simulation. Though the CFHSS column did not fail (since the simulation terminated due to floor failure), by comparison with results from other simulations (Kodur and Fike 2009, and Fike and Kodur 2009), it is postulated that while the column would have achieved a two hour fire resistance, it would not have achieved much more than a two hour fire resistance rating. To get a better picture of the fire resistance that can be achieved through the use of CFHSS columns in place of W-shape sections, the remaining simulations were carried out considering the various design fire scenarios illustrated in Figure 6.

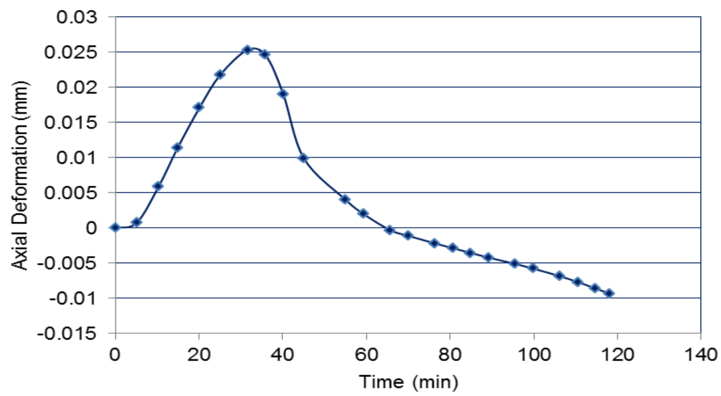


Figure 9: Axial deformation of the CFHSS column as a function of fire exposure time corresponding to Case 3 analysis

*Cases 4 thru 7*
When the fire resistance of the assembly was considered under the extreme and severe fire exposures shown in Figure 6, it was observed that the floor assembly failed prematurely; not allowing the response of the CFHSS column to be fully evaluated. Simulations were carried out for both the one and three story models, with results from the two simulations for each fire scenario closely coinciding. The similarity of the results from the one and three story models offers little insight into the relative behavior of the two models, but was advantageous as it reinforced that the models were similarly constructed within SAFIR. Since however, fires of this severity would be highly unlikely in typical office buildings, the low fire resistance demonstrated by the complete floor assembly is not unreasonable. For further discussion on these fire resistance simulations, the interested reader is directed to the Reference (Fike 2010).

*Case 8*
Under the medium fire exposure shown in Figure 6, a much higher fire resistance was achieved. Fire resistance was enhanced to such an extent that the structure was able to survive complete burnout under the design fire scenario. As was the case under standard fire exposure, the steel section reached temperatures under the medium fire exposure sufficient to cause the majority of the load to transfer from the HSS section to the concrete core, as indicated by the contraction shown in Figure 10 at approximately 57 minutes. Due however to the lower maximum temperature, and the presence of a decay phase in the fire medium fire exposure, at the end of the three hour simulation period, the column was just beginning to achieve a negative overall deformation. Based on these results, it can be concluded that the CFHSS column utilized in this simulation is capable of withstanding the medium fire exposure and achieve compartment burnout, the highest possible fire resistance under a design fire exposure.

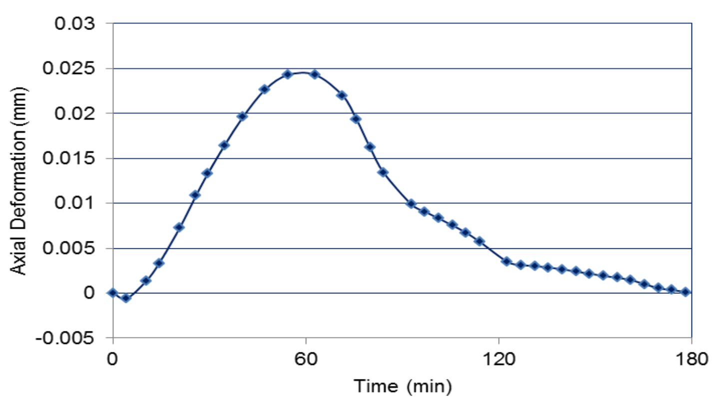


Figure 10: Axial deformation of the CFHSS column as a function of fire exposure time corresponding to Case 8 analysis

*Case 9*
To further illustrate the ability of CFHSS columns to enhance fire resistance through the development of composite action, an additional case study was conducted using SAFIR under the mild fire exposure shown in Figure 6. Unlike the ASTM and Medium fire exposures discussed previously, full load transfer from the steel HSS section to the concrete core was not observed under the mild fire exposure. This is illustrated in Figure 11 (which shows axial deformation of the CFHSS column as a function of fire exposure time) by the absence of a period in which the column rapidly contracts due to gross yielding of the steel section. Rather, at

approximately 110 minutes, the column slowly begins to contract due to weakening of the steel section, this continues until approximately 120 minutes at which point the fire enters the decay phase. As the fire temperatures decrease, contraction of the column slows as the steel begins to slowly cool down, and regain strength and stiffness. With the HSS section again contributing to the strength of the composite column, the column slowly contracts due to thermal shrinkage as the fire temperatures decrease. Though not shown in Figure 11 for the sake of clarity, it should be noted that the column returned to within 3mm of its original loaded length after cooling down to ambient temperatures. Based on these results, it can be concluded that the CFHSS column utilized in these simulations is capable of withstanding the mild design fire exposure with little residual damage or deformation after complete compartment burnout.

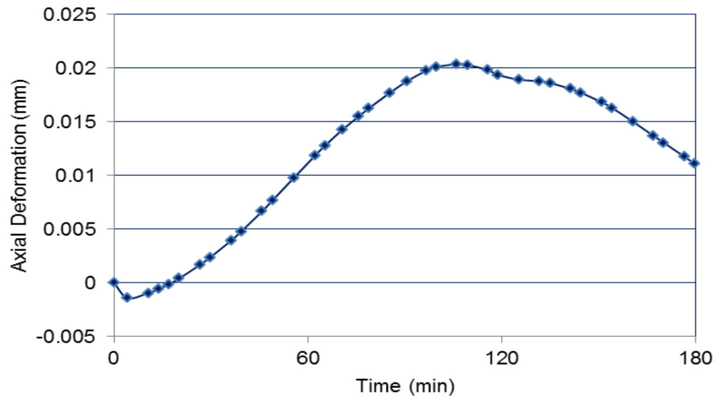


Figure 11: Axial deformation of the CFHSS column as a function of fire exposure time corresponding to Case 9 analysis

*Cases 10 and 11*

To illustrate the full capability of composite construction and CFHSS columns to enhance structural fire resistance without the need for applied fire protection on columns, the three-story case with CFHSS columns and SFRC floor systems was simulated under the medium and mild fire exposures of Figure 6. In both cases, a one hour delay in ignition from one story to the next was assumed. The fire on the second story started one hour after the fire on the first story, and the fire on the third story started one hour after the start of the fire on the second story and two hours after the start of the fire on the first story. The result is a 5 hour fire exposure time in place of the 3 hour fire exposures considered in the one story models. Due to similarities between results from both the medium and mild fire simulations, only the mild fire (representative of office occupancies) will be used to illustrate the behavior of the structure, though the discussion applies to both fire exposures.

Results from the SAFIR simulations indicate that the structure was able to survive burnout of three consecutive stories under both of the design fire exposures. Figure 12 shows the axial deformation as recorded at the top of each story during the five hour mild fire exposure simulation. It should be noted at this point that in the original building design the same W-Shape column was utilizes at all three stories, as such, the same CFHSS column was utilized on all three stories in the simulation. The result is that the bottom level CFHSS column was at a higher load ratio than the middle and top columns.

Results from the simulation, as shown in Figure 12, indicate that all three of the columns initially expanded as they were exposed to fire, with the top column realizing the greatest deformation due to compounding effects. It is also observed from Figure 12, that after the initial thermal expansion of the lower column, there appears to be rapid contraction between 110 and 120 minutes due to load transfer occurring between the steel and concrete in the column cross section as discussed previously. Due however to the design fire entering the cooling phase, this rapid contraction rate is slowed at approximately 125 minutes, from which point the column slowly contracts due to a slow reduction in the column cross section temperatures. This same load transfer does not appear to be present for either of the upper stories, this is attributed to the lower load ratio on the upper story columns as compared to the lower story column. The lower load ratio on the columns of the upper stories enables the columns to withstand the maximum fire temperature, and reach the decay phase without transferring appreciable load to the concrete core.

In addition to the different loadings on the columns supporting different stories, it should be noted that the flashover portion of the medium and mild fire exposures, shown in Figure 6, are greater than the one hour delay assumed between ignition of subsequent stories. As such, the CFHSS column is actually exposed to a fully developed fire for two adjacent spans simultaneously. Fire exposure over multiple spans increases the demand on the column as compared to fire exposure over a single span. Thus, it is critical that a system level approach be taken to accurately predict the response of a CFHSS column under realistic fire exposure conditions. Results from these system-level simulations clearly illustrated that through the use of CFHSS columns, in place of typical W-shape columns, it is possible to realize aesthetically pleasing exposed structural steel with minimal limitations on structural fire resistance.

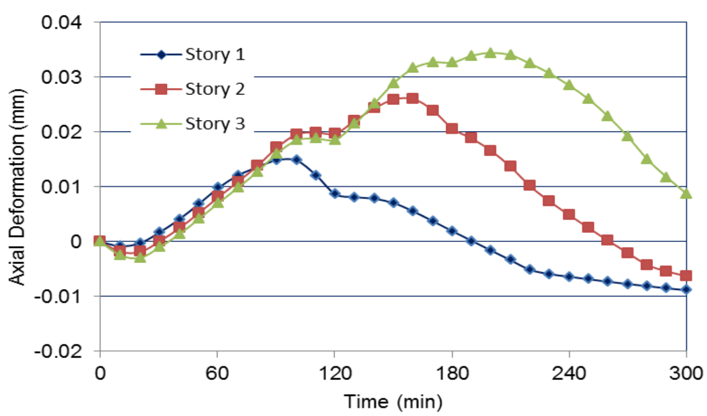


Figure 12: Axial deformation of the CFHSS columns as a function of fire exposure time corresponding to Case 11 analysis

## Conclusions

The numerical simulations presented here highlight the ability of CFHSS columns as part of a larger structure to resist the effect of fire, specific conclusions from this research are:

- Composite construction, when properly implemented, can significantly enhance fire resistance of structural steel framing.
- Fire resistance of CFHSS columns can be significantly improved by considering realistic loading, fire exposure, failure criterion, and member interactions through system level analysis.

- In steel framed office buildings, it is possible to eliminate external fire protection to columns in a performance based environment by taking into consideration the beneficial effects of composite construction through the use of CFHSS columns.
- Concrete filled steel columns can withstand two hours of standard fire exposure, or complete burnout of medium or less severe design fires without any fire protection.


## Acknowledgments

The research, presented in this paper, is primarily supported by the American Institute of Steel Construction through AISC Faculty Fellowship Award to Prof. Kodur.


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