# Response of Concrete-Filled HSS Columns in Real Fires

VENKATESH K.R. KODUR and RUSTIN FIKE


## ABSTRACT

The use of concrete filling offers a practical alternative for achieving the required fire resistance in steel hollow structural section (HSS) columns. However, the current prescriptive-based approach has a number of constraints that in many applications restrict the utilization of concrete filling for achieving the required fire resistance. To overcome such constraints, a performance-based methodology for fire resistance design is presented in this paper. A set of numerical simulations were carried out to investigate the effect of realistic fire scenarios, loading and stability-based failure criterion on the fire resistance of concrete-filled HSS columns with lengths ranging from 3.8 m (12.5 ft) to 10 m (32.8 ft). It is demonstrated that by adopting a performance-based approach, it is possible to achieve the required fire resistance in CFHSS columns in most practical situations.


**Keywords:** column stability, hollow structural sections, standard fire, design fire, concrete-filled steel tubes, fire resistance, fire and temperature effects, performance-based design.

## BACKGROUND

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 framed structures. Fire safety is one of the primary considerations in the design of high-rise buildings, and hence, building codes specify fire protection requirements 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 proofing (adhesion 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 their stiffness, torsional rigidity and load-bearing capacity. The two components of the composite column complement 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, a higher fire resistance is obtained without using external fire protection for the steel, thus increasing the usable space in the building and removing the need for application and maintenance of the external fire protection. Properly designed concrete-filled columns can lead economically to the realization of architectural and structural design with visible steel, without any restrictions on fire safety (Kodur and Lie, 1995a; Klingsch and Wuerker, 1985; Twilt et. al., 1996).

Design guidelines for achieving fire resistance through concrete filling have been incorporated into codes and standards (ASCE, 1999; AISC, 2005; NBC, 2005). However, the current fire guidelines are limited in scope and restrictive in application since they were developed based on the standard fire test from ASTM E-119, *Standard Methods for Fire Tests of Building Construction and Materials* (ASTM, 2007). Hence, they are valid only for the standard fire exposure conditions and for column lengths dictated by the capacity of available testing furnaces. In many applications, such as atriums, schools and airports, where long spans of exposed steel are highly desired, the current prescriptive provisions cannot be applied due to limitations on column size. Thus, designers cannot take advantage of the intrinsic fire resistance present in concrete-filled steel columns.

This paper presents a performance-based methodology for fire safety design of concrete-filled HSS (CFHSS) columns. A review of the fire performance of CFHSS columns is presented, and the drawbacks and limitations of the current fire resistance evaluation approaches are discussed. Results from a numerical study on a set of CFHSS columns exposed to various fire and loading scenarios are presented. The analysis was carried out using finite element–based computational

Venkatesh K.R. Kodur, Professor, Department of Civil and Environmental Engineering, 3546 Engineering Building, Michi-gan State University, East Lansing, MI, 48824 (corresponding author). E-mail: kodur@egr.msu.edu

Rustin Fike, Graduate Student, Department of Civil and Environmental Engineering, 3546 Engineering Building, Michigan State University, East Lansing, MI, 48824. E-mail: fikerust@msu.edu

model SAFIR, wherein the material and geometric nonlinearity, and stability-based failure criterion are considered. Results from the analysis are used to present a framework for a performance-based fire engineering methodology. It is demonstrated that fire resistance in HSS columns in the full practical range can be achieved through concrete filling.

## PERFORMANCE-BASED DESIGN

Recently there has been an impetus to move toward a performance-based approach for fire safety design (Meacham and Custer, 1992; Kodur, 1999). This is mainly due to the fact that the current prescriptive-based approach has serious limitations and does not provide alternate, cost-effective and innovative solutions. There are two basic methods by which performance-based fire safety design can be accomplished: tests can be performed wherein the structural performance of the system to be built is evaluated, or numerical/computational simulations can be used to evaluate the system to be built. Due to the cost, time and effort associated with full-scale fire testing, the first option is mostly used to validate numerical models. The validated models can then be used to undertake performance-based fire safety design. The most important factors to be considered in any performance-based fire safety design are fire scenario, load level, failure criterion and geometric conditions (Parkinson and Kodur, 2007). These main components are discussed in the following sections.

### Fire Scenario

The current practice of evaluating fire resistance of CFHSS columns is based on standard fire tests or models, in which the column is exposed to a standard fire as specified in standards such as ASTM E-119 or ISO 834 (ISO, 1975). While standard fire resistance tests are useful benchmarks to establish the relative performance of different CFHSS columns, they should not be relied upon to determine the survival time of CFHSS columns under realistic fire scenarios. Additionally, the standard heating condition has little resemblance to the often less severe heating environments encountered in real fires.

Unlike standard fires, design fires are derived based on different combinations of fuel load and ventilation scenarios in compartments. In the standard fires (ASTM E-119 fire and hydrocarbon fire) (ASTM, 2007; ASTM, 2001), the fire severity is the same (irrespective of compartment characteristics), and temperature increases with time throughout the fire duration with no decay phase. However, in real fires, the fire severity is a function of compartment characteristics, such as ventilation, fuel load and lining materials, and there is a decay phase following the initial growth phase as shown in Figure 1 (Magnusson and Thelandersson, 1970). Often, in the decay phase, the cross section of the column enters the cooling phase, in which the steel recovers part of its strength and stiffness, leading to an enhanced fire resistance that allows the column to withstand compartment burnout.

Figure 1 shows time-temperature curves for two standard fire exposures (ASTM E-119 and ASTM E-1529) (ASTM, 2007; ASTM, 2001) and three levels of design fires: severe, medium and mild. To simulate a "mild" fire, a ventilation factor of 0.04 and a fuel load of 200 MJ/m$^2$ were chosen (FV04-200), while for a "medium" fire, a ventilation factor of 0.12 was chosen with a fuel load of 900 MJ/m$^2$ (FV12-900). A "severe" fire was simulated using a ventilation factor of 0.08, and a fuel load of 800 MJ/m$^2$ (FV08-800). Specific time-temperature relationships for a given compartment can be developed through simple spreadsheet calculations or retrieved from published tables (Magnusson and Thelandersson, 1970) as was done in this study.

### Load Ratio

The current provisions in codes of practice for evaluating fire resistance 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, and the safety factors (load and capacity factors) used for design under both room temperature and fire conditions. The loads on the column under fire exposure scenarios can be estimated based on the guidance given in ASCE 7 standard (ASCE, 2005) (1.2 dead load + 0.5 live load or through actual calculations based on different load combinations). Based on specifications in ASCE 7 (ASCE, 2005) and the AISC *Manual* (AISC, 2005), for typical dead to live load ratios (in the range of two to three), the loads on a CFHSS column under fire scenario can range between 30% and 50% of its ultimate capacity. The low load ratio effective

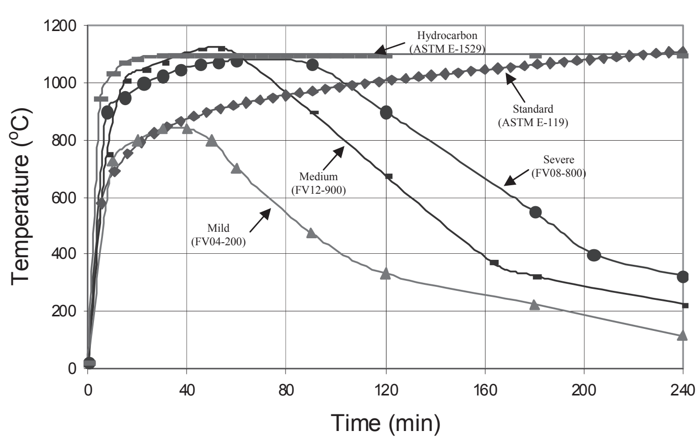


*Fig. 1. Time-temperature relationships for various fire scenarios.*

under fire conditions is mainly due to the fact that fire is a rare event so only service loads (with reduced live loads) are considered in the analysis.

Load ratio could presumably 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.

### Failure Criterion

For evaluating fire resistance of steel structures, a limiting temperature criterion is often used to define failure. It is commonly assumed that once the steel section reaches a critical temperature of 538 °C (1000 °F), approximately 50% of the room temperature strength is lost (Eurocode, 2005a) and failure is eminent. While sufficient for traditionally protected steel sections, the effect of the concrete core is not properly captured by the limiting temperature criterion. To overcome this drawback, strength and stability limit states of the column under fire conditions need to be considered. Depending on factors such as the support conditions and overall column length, buckling or crushing could occur well after the limiting (critical) temperature of 538 °C (1000 °F) is reached. In addition, CFHSS columns can fail locally (without collapse) due to local crushing of the concrete on the inside, or local buckling of the steel wall (Kodur, 2005; Kodur and Lie, 1996; Kodur and Lie, 1997). Thus, local stability should also be accounted for in the analysis.

## STATE OF THE ART

Alternate approaches for achieving fire resistance in HSS columns have been studied in the past three decades. Methods such as filling the HSS columns with liquid (water) and concrete are among the most popular approaches studied by researchers (Kodur and Lie, 1995a; Bond, 1975; Klingsch and Wittnecker, 1988). However, the use of concrete filling is the most attractive and feasible proposition developed thus far.

### Experimental and Numerical Studies

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, 1995b; Lie and Chabot, 1992; Lie and Caron, 1988; Lie and Irwin, 1991). Both square and circular HSS columns were tested, and the influence of various factors including type of concrete filling, concrete strength, intensity of loading, and column size were investigated under the ASTM E-119 standard fire exposure. The tests reported by European and Chinese studies (Klingsch and Wittbecker, 1998; Grandjean et. al., 1981; FCSA, 1989; Han and Lin, 2004) are similar to NRCC tests, but the fire exposure was that of the ISO 834 (ISO, 1975) standard fire; which has a time-temperature curve similar to that of ASTM E-119.

The numerical studies, primarily carried out at NRCC, consisted of developing mathematical models for predicting the fire behavior of circular and square CFHSS columns (Kodur, 1997; Lie and Chabot, 1991; Lie and Kodur, 1996). In these models, the fire resistance is evaluated in various time steps, consisting of the calculation of the fire temperature, the temperatures in the cross-section of the column, its deformations, its strength during exposure to fire, and finally, its fire resistance. Full details on the development and validation of these models are given in Kodur and Lie (1997), Lie and Chabot (1992) and Kodur (1997).

Data reported from NRCC tests and numerical studies can be used to illustrate the behaviour of concrete-filled HSS columns under standard fire conditions. Figure 2 shows the axial deformation as a function of time for three typical HSS columns, each filled with a different type of concrete filling, namely, plain concrete, steel-fiber-reinforced concrete, or bar reinforced concrete (Kodur and Lie, 1995b). The three columns were of similar size and were subjected to similar load levels, hence the results can be used to illustrate the comparative behavior of the three types of concrete filling.

In CFHSS columns prior to fire exposure, the applied load on the composite column is carried by both the concrete and the steel. When the column is exposed to fire, steel carries most of the load during the early stages because the steel section expands more rapidly than the concrete core. As time progresses and temperatures increase, the steel section gradually yields due to loss of strength in steel at elevated temperatures, and the expansion phase gives way to a contraction

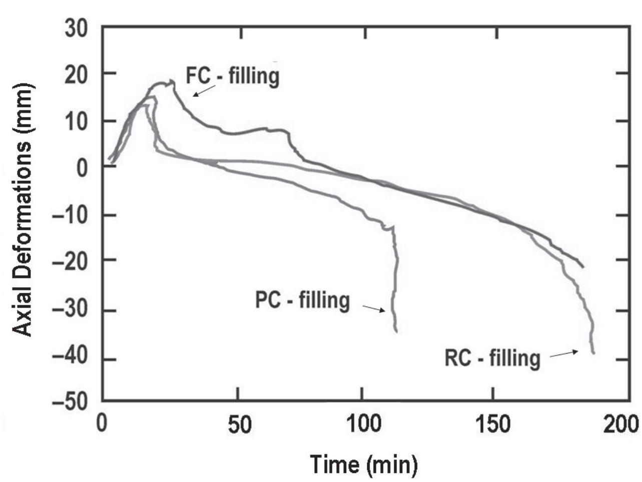


*Fig. 2. Axial deformation in CFHSS columns as a function of time.*

phase after about 20 minutes of fire exposure. At this stage, the concrete starts to carry an increased portion of the load. As fire progresses, the strength of concrete also deteriorates to a level where the column strength is less than the applied load, and failure occurs in the column. The elapsed time from ignition to failure is the fire resistance of the column.

It can be seen in Figure 2 that the deformation behavior of HSS columns filled with bar and steel-fiber-reinforced concrete is similar in the expansion zone to that of columns filled with plain concrete. The initial slightly higher deformation in the steel-fiber-reinforced concrete-filled HSS column could be attributed to higher thermal expansion of steel-fiber-reinforced concrete. In the contraction phase, the behavior of HSS columns filled with bar- and steel-fiber-reinforced concrete is significantly different than columns filled with plain concrete due to the contraction in these columns being slower. This can be attributed to slower loss of strength and stiffness in the concrete due to the presence of steel reinforcement. Consequently, this results in higher fire resistance in HSS columns filled with steel-fiber- and bar-reinforced concrete as compared to that of columns filled with plain concrete.

## Design Equation and Method for Evaluating Fire Resistance

Based on the experimental and numerical studies reported in literature (Kodur and Lie, 1996; Kodur, 1997; Lie and Stringer, 1994), it was found that the parameters with the most influence on fire resistance of CFHSS columns are type of concrete filling (plain, bar-reinforced, and fiber-reinforced), outside diameter or width of the column, load on the column, effective length of the column, concrete strength, and type of aggregate. Using this as the basis, the following unified design equation has been developed to calculate the fire resistance of circular and square HSS columns filled with any of the three types of concrete (Lie and Stringer, 1994; Kodur, 1999; Kodur and McKinnon, 2000):

$$R = f\,\frac{(f_c' + 20)}{(KL - 1000)}\,D^2\sqrt{\frac{D}{C}} \qquad (1)$$

where $R$ = fire resistance in minutes; $f_c'$ = specified 28-day concrete strength in MPa; $D$ = outside diameter or width of the column in mm; $C$ = applied load in kN; $K$ = effective length factor; $L$ = unsupported length of the column in mm; $f$ = a parameter to account for the type of concrete filling (plain, steel-fiber-reinforced, or bar-reinforced concrete), the type of aggregate used (carbonate or siliceous) in concrete, the percentage of reinforcement, the thickness of concrete cover, and the cross-sectional shape of the HSS column (circular or square), values of which can be found in Lie and Kodur (1996).

Equation 1 has been validated by comparing the predictions from the equation with data from fire tests conducted at various laboratories (Twilt et. al., 1996; Kodur, 1997; Lie and Kodur, 1996). The fire resistances obtained using the equation are conservative (about 10–15%) compared to those obtained from the tests, particularly for fire resistances over three hours.

Eurocode 4 (Eurocode, 2005b) also presents a method by which the fire resistance of CFHSS columns exposed to standard fire can be determined. The analysis is composed of two steps: determination of the temperature distribution and calculation of the field buckling load at the temperature previously determined. For further details on the calculation method presented in Eurocode 4, the reader is directed to that reference (Eurocode, 2005b).

## Limitations

Both the design equation and the Eurocode method just presented, though providing a convenient way of evaluating fire resistance of CFHSS columns, have a number of limitations—the greatest of which is that they are only applicable for the prescriptive-based standard fire scenario, in which no consideration is given to a decay phase. This is a serious limitation since the high fire performance that can be achieved using CFHSS columns under most design fire scenarios cannot be realized. As an illustration, plain concrete-filled HSS columns can only provide fire resistance of an hour or less in some situations. While recognizing that columns filled with steel- and bar-reinforced concrete perform better than columns filled with plain concrete, the fire rating assigned to these columns is still markedly less than the fire resistance which can be obtained in design fire scenarios. In addition, the application of Equation 1 is limited to CFHSS columns up to only 4.5 m (14.75 ft) in length, since the equation was derived based on test data which ranged in length from 2.5 to 4.5 m (8.2 to 14.75 ft) (restricted by furnace construction). In applications such as airports, schools, atriums and

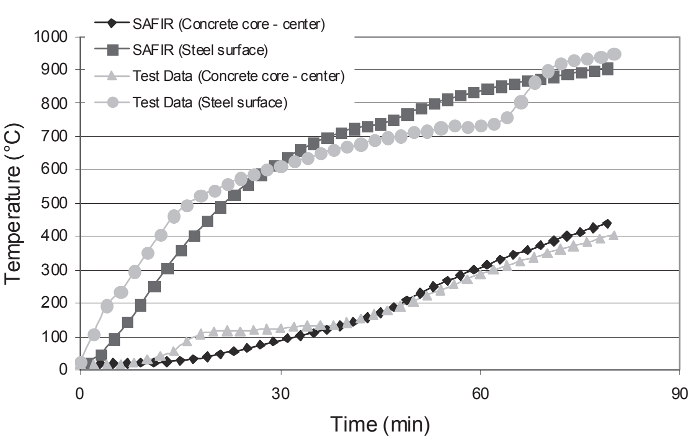


*Fig. 3. Comparison of temperatures from SAFIR with test data for column SP-178.*

| Table 1. Current Design Limitations and Range of Parameters Encountered in Practical Applications | | |
|---|---|---|
| **Parameter** | **Current Limitation** | **Practical Applications** |
| Column shape | square and circular | square, circular and rectangular |
| Column size | 400 mm | 600 mm and beyond |
| Column length | 4.5 m | 6–10 m |
| Concrete strength | 55 MPa | 100 MPa |
| Fire scenarios | ASTM E-119 | design fires and hydrocarbon fires |
| Load level | strength of concrete core | service loads |
| Eccentric loads | not allowed | always present |

hotels, columns are generally longer than 4.5 m (14.75 ft), and hence designers cannot capitalize on the intrinsic fire resistance present in CFHSS columns.

The various limitations on the applicability of design Equation 1 are presented in Table 1, along with the range of variables encountered in most practical applications. It can be seen that the current design equation, in addition to the limitations on length and fire exposure, is valid only over a narrow range of column parameters such as diameters up to 400 mm (15.75 in.), concrete strengths up to 55 MPa (8.0 ksi), and concentric loads. Therefore, design Equation 1, and relevant solutions, cannot practically be used under the recently introduced performance-based codes, which provide rational, cost-effective and innovative fire safety solutions. Only in rare circumstances (such as the Orange County Performing Arts Center in California) has performance-based fire design incorporating design fires been undertaken for utilizing CFHSS columns (Chen and Gemeny, 2004).

The previously stated limitations restrict architects and designers from taking advantage of high fire resistance ratings (and other advantages) that can be achieved through CFHSS columns. Also, the limitations of the preceeding research for only ASTM E-119 fire exposure hinders the use of HSS columns in offshore structures and oil platform applications. Thus, the full potential for the use of CFHSS columns has not been realized.

## NUMERICAL STUDIES

To overcome some of the previously stated limitations, a numerical study was carried out to develop an approach for performance-based fire resistant design of CFHSS columns. The analysis was carried out using finite element-based computer program SAFIR, and the fire response of CFHSS columns was simulated under various fire and loading scenarios. Some of the details associated with the analysis are presented here.

### Computer Program

The computer program SAFIR, developed at the University of Liege in Belgium, is capable of accounting for multiple materials in a cross section, both heating and cooling phase of fire exposure, large displacements, different strain components, nonlinear material properties according to Eurocode 3 (Eurocode, 2005), and residual stresses. Additionally, SAFIR allows the user to input any time-temperature relationship to facilitate the use of design fire scenarios. SAFIR has undergone extensive validation and predictions from SAFIR have been shown to closely match test data and predictions from other numerical models (Franssen, 2005; Gilvary, 1997; Talamona, 2005).

In SAFIR, the thermal and structural analyses are performed independently. For thermal analysis, 2D solid elements are used 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 assumes that every cross-section has the same temperature profile unless otherwise specified. The energy consumed for evaporation of water present in the concrete is included, but that associated with hydraulic migration within the cross-section is neglected. Additionally, due to the lack of a material model for steel-fiber-reinforced concrete within SAFIR, the thermal properties of steel-fiber-reinforced concrete were assumed to be the same as those for normal concrete (Lie and Kodur, 1996).

For structural analysis, SAFIR uses a fiber-based approach wherein each of the solid elements in the thermal model is considered as a fiber in the structural model. These fibers can have different material properties, allowing the modeling of the steel shell and steel bars in bar-reinforced concrete-filled CFHSS columns. A stress and temperature dependent stiffness matrix is established that incorporates all of the fibers. Due to the increasing temperature in the

| Table 2. Summary of Test Parameters and Fire Resistance Values for CFHSS Columns | | | | | | |
|---|---|---|---|---|---|---|
| Column Designation | Diameter or Width (mm) | Length (mm) | AISC Factored Load (kN) | Load Ratio | Fire Resistance (min.) | |
| | | | | | Test | SAFIR |
| RP-168 | 168.3 | 3810 | 1197 | 0.13 | 81 | 82 |
| RP-273 | 273.1 | 3810 | 3508 | 0.15 | 143 | 128 |
| RP-355 | 355.6 | 3810 | 5120 | 0.18 | 170 | 164 |
| SP-152 | 152.4 | 3810 | 1409 | 0.20 | 86 | 74 |
| SP-178 | 177.8 | 3810 | 1976 | 0.28 | 80 | 71 |
| RF-324 | 323.9 | 3810 | 4573 | 0.35 | 199 | 200 |
| RF-356 | 355.6 | 3810 | 6616 | 0.23 | 227 | 238 |
| SF-203 | 203.2 | 3810 | 3506 | 0.26 | 128 | 121 |
| SF-219 | 219.1 | 3810 | 3793 | 0.16 | 174 | 185 |
| RB-273a | 273.1 | 3810 | 3323 | 0.32 | 188 | 158 |
| RB-273b | 273.1 | 3810 | 3333 | 0.57 | 96 | 98 |
| SB-203 | 203.2 | 3810 | 2345 | 0.21 | 150 | 130 |
| SB-254a | 254 | 3810 | 3405 | 0.42 | 113 | 110 |
| SB-254b | 254 | 3810 | 3405 | 0.65 | 70 | 69 |

column, the stiffness decreases to a point where the 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. The main limitation of the structural model is the assumption that there is no slip between the steel and the concrete. While the assumption of no slip will initially place higher tension on the concrete core due to the differential thermal expansion of the steel and concrete, the steel rapidly looses strength (within 20 minutes) such that during the critical portion of fire exposure this assumption does not place unrealistic loads on the concrete core. As was the case in the thermal model, SAFIR does not include a mechanical model for steel-fiber-reinforced concrete. As such, the same constitutive model was used for steel-fiber-reinforced concrete as normal concrete, with the exception that the tensile strength of the concrete was increased to account for the steel fibers. This is believed to be a conservative assumption due to the superior stiffness retention demonstrated in steel-fiber-reinforced concrete (Kodur and Harmathy, 2002; Lie and Kodur, 1996).

## Test Columns

Fourteen CFHSS columns tested at NRCC under the standard fire scenario were selected for numerical studies (Kodur and Lie, 1996; Lie and Kodur, 1996; Lie and Chabot, 1992; Lie and Irwin, 1995; Chabot and Lie, 1992). All pertinent information for the CFHSS columns is provided in Table 2. In the designation of columns (for example, RP-355), the first letter (R) represents section shape (round or square), the second letter (P, F, B) denotes filling type (plain, steel-fiber-reinforced concrete, and bar-reinforced concrete, respectively), and the number (355) denotes the diameter (for circular) or width (for square) of the HSS column section in mm.

## Model Validation

For the validation of SAFIR, fourteen columns were analyzed under ASTM E-119 fire exposure. The thermal and structural response and ultimate failure times generated by SAFIR were compared with the measured test data for all of the columns. All of the columns for which test data were available underwent the same level of scrutiny as the single column used to illustrate the following validation. However, results from the validation of all columns could not be presented due to space constraints.

A comparison of the temperatures predicted by SAFIR and those measured in a fire test at the steel surface (HSS section) and at the center of the concrete core is shown for column SP-178. Good agreement between predicted and measured temperatures is observed for the concrete core after 100 °C (212 °F), at which point free water is driven off. The steel temperatures initially deviate such that temperatures observed in tests are hotter than those predicted by SAFIR. This can be attributed to the assumption in SAFIR that there is perfect thermal contact between the steel and the concrete. However, when the steel approaches the critical phase transformation temperature of 700 to 750 °C (1300 to 1400 °F), at which point the bonded water in the concrete is also driven off, SAFIR begins to overpredict the temperature in the steel. This is mainly due to the assumption in SAFIR that hydraulic migration in concrete can be neglected. Toward the end of the test, the temperatures compare reasonably well such that there is only a 30 °C (90 °F) temperature difference between the test data and the SAFIR simulation. Overall, the temperature predictions from SAFIR are in reasonable agreement with data measured from tests.

The structural response predicted by SAFIR was validated by comparing the axial deformations for column SP-178 with those measured during tests (Figure 4). The column initially expands as a result of the quick rise in steel temperatures. This increased temperature leads to eventual loss of strength and yielding of steel, at which point concrete starts to take over most of the load bearing function, causing the column to contract slightly. A peak deflection (expansion) of 18.3 mm (0.72 in.) was observed in the tests, while the corresponding peak deflection from SAFIR was 17.9 mm (0.705 in.). These two maximum deflections show good agreement occurring at 20 and 23 minutes, respectively. The differences in deflection between SAFIR data and test data are a result of the temperature discrepancies seen in Figure 3. The initially higher temperatures in the steel shell produces higher expansion of the column in the fire test than in SAFIR, as seen in Figure 4. After peak deflections are reached, the temperatures in the steel shell as predicted by SAFIR are hotter than those observed in tests. This results in the steel retaining slightly higher strength in tests than in the SAFIR simulation. Thus, there is a slightly larger contraction of the column in the SAFIR simulation than observed in the test. Overall, the predicted deformations compare well with those measured during the fire test.

The predicted and measured values of fire resistance for the 14 CFHSS columns are tabulated in Table 2. All of the columns were simulated with the loads applied centrally, with a fixed connection at the base and a connection only allowing vertical translation (under loading) at the top. The failure times in fire tests shown in Table 2 correspond to the point at which the column could not sustain the applied load. In SAFIR, failure is likewise taken as the point where the column can no longer support the applied load. The time to reach that point is defined as the fire resistance of the column. A comparison of the fire resistance values indicates that the SAFIR predictions are in good agreement with measured fire resistance values. Due to the inherent variability of laboratory testing, and assumptions made within a computational model, the results presented here are in reasonable agreement, and the models, (both the thermal and structural) constructed here are deemed acceptably conservative to continue with the parametric studies.

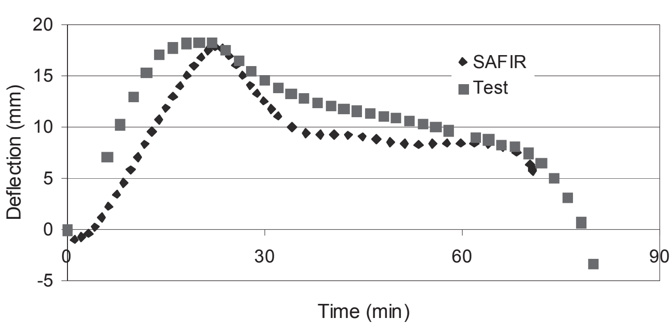


*Fig. 4. Comparison of axial deformations from SAFIR with test data for column SP-178.*

## PARAMETRIC STUDIES

The validated computer program SAFIR was used to carry out a set of parametric studies to quantify the effect of critical parameters on the fire resistance of CFHSS columns. For the parametric studies, 20 CFHSS columns were selected. Fourteen of these columns were used in previous fire tests (listed in Table 2), while the remaining six are columns specifically selected to cover a wider range of column variables. In order to fully investigate the effect of length on fire resistance, columns 3.81, 5, 6, 7, 8, 9 and 10 m (12.5, 16.4, 19.7, 23.0, 26.3, 29.5 and 32.8 ft) long were modeled for each cross-section. Fire exposure was assumed on all four sides with the bottom and top 5% of the column unexposed to fire. Fire loading other than four-side exposure should be considered as a topic of future research. Loads were modified to maintain a constant load ratio for all of the lengths modeled for a specific cross-section according to the AISC analysis procedure (AISC, 2005). The applied loads on the columns take into account the effect that the type of concrete filling has on fire resistance: steel-fiber- and bar-reinforced concrete-filled HSS columns can take higher loads. Each of these cross section-length combinations were modeled under seven fire scenarios, five of which are shown in Figure 1. The remaining two fire scenarios were also design fires. This parametric study generated a total of 980 numerical simulations covering a wide range of loads, geometry, concrete types and fire scenarios. The trends (results) discussed later were observed for all of the filling types (plain, bar-reinforced, and steel-fiber-reinforced) considered unless otherwise noted. However, the results and discussion are mainly illustrated using plain-concrete-filled HSS columns. Another point to be noted is that the analysis was continued until the column attained failure or 240 minutes of exposure to fire. As such, if a fire resistance of 240 minutes is reached, it is indicative of the column withstanding compartment burnout, not that the column failed at 240 minutes unless otherwise noted.

### Effect of Fire Exposure

The effect of fire severity on fire resistance is illustrated in Figure 5 by plotting the fire resistance as a function of length for column RP-273 under different fire scenarios.

Intuitively, as the fire severity decreases, the fire resistance of the column increases. It can be seen in Figure 5 that a fire resistance of four hours or more can be obtained for columns up to 10 m (32.8 ft) long under mild fire conditions. However, for other fire exposures fire resistance decreases with an increase in length. A closer look at Figure 5 indicates that fire resistance of a 5 m (16.4 ft) long CFHSS column ranges from 240 minutes for medium and mild fire exposure to 68 minutes under severe exposure, with ASTM exposure yielding 100 minutes. The reason for this decreased fire resistance with increased fire severity can be attributed to the higher internal temperatures attained under severe fire exposure. Consequently, the column loses its strength and stiffness at a faster rate leading to early failure. Figures 6 and 7 show the difference in internal temperatures observed in the steel shell and at the center of the concrete core for the fire exposures shown in Figure 1.

It can be seen in Figure 6 that two of the three design fires produce higher initial temperatures in steel than the ASTM E-119 fire. This effect is seen in concrete temperatures also (see Figure 7), though to a lesser extent. The presence of the decay phase in the design fires causes the temperatures in all locations of the column to be less than those for the ASTM E-119 fire at the end of the simulation period. Column stability is maintained under design fires, despite the more severe initial temperatures, due to the decay phase of the fire allowing cooling of the steel before significant loss of strength in the concrete core occurs.

To illustrate the effect of fire scenario on the structural response, Figure 8 displays the axial deformation of column RP-273 resulting from ASTM E-119, severe, medium and mild fire exposure. In all simulations, the column initially expanded due to the increasing steel temperature. After this initial expansion, the response of the column is significantly influenced by the type of fire exposure. In the case of severe and medium fire scenarios, significant contraction occurs before failure of the column. This could be attributed to the

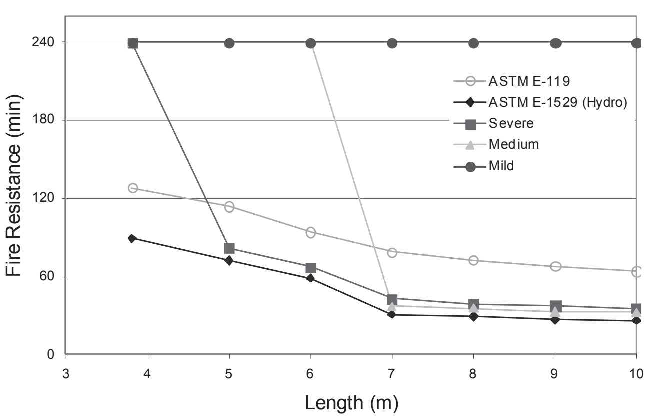


*Fig. 5. Fire resistance as a function of length for column RP-273 under different fire scenarios.*

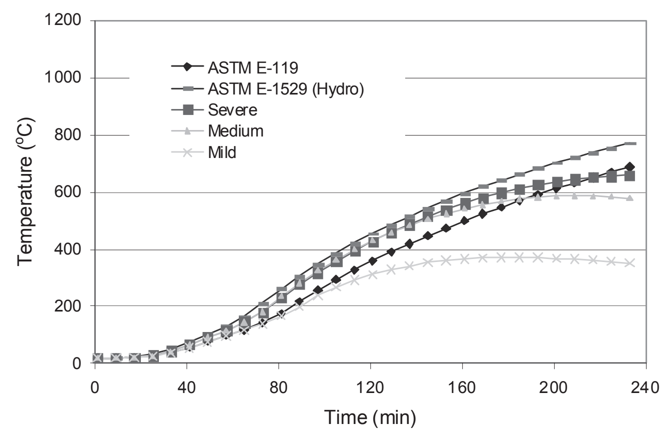


*Fig. 7. Temperatures at the center of concrete core for column RP-273 exposed to different fire scenarios.*

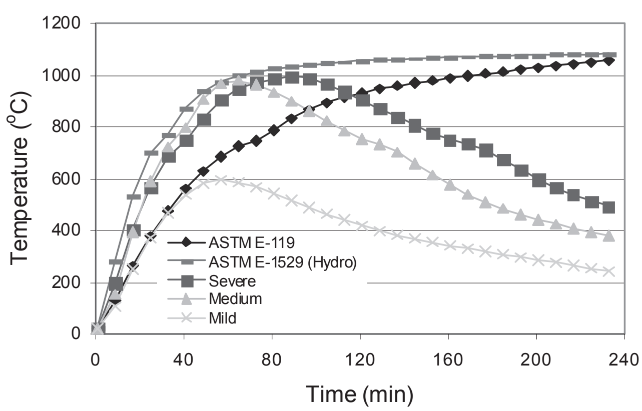


*Fig. 6. Temperatures on the steel surface for column RP-273 exposed to different fire scenarios.*

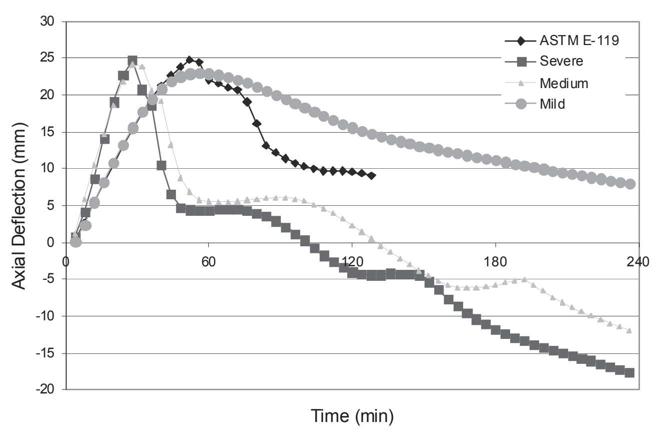


*Fig. 8. Axial deformation as a function of time for column RP-273 under different fire scenarios.*

presence of a cooling phase in design fire scenarios. However, under ASTM E-119 fire exposure, the column failed in 32 minutes without too much contraction.

### Effect of Length

The results presented in Figure 5 can be used to demonstrate the effect of length on the fire resistance of CFHSS columns. In the analysis, the load on the column was reduced as the length was increased such that the load ratio on a single column was kept constant through all of the simulations. As would be expected, fire resistance for a given fire exposure decreases with an increase in length of the column. This is due to the increase in slenderness that accompanies the increase in length. Fire resistance is drastically reduced when the failure mode switches from crushing to buckling with increased length. This is most pronounced for the "medium" and "severe" fire exposure as can be seen in Figure 5. Fire resistance under the medium fire drops from 240 to 45 minutes when length is increased from 5 to 7 m (16.4 to 23 ft). Under severe fire exposure, the fire resistance decreases from 240 to 75 minutes for an increase in length from 3.81 to 5 m (12.5 to 16.4 ft). However, under mild fire exposure, fire resistance remains high for all cases, and the length does not have any influence. This study clearly illustrates the fact that length has a significant influence on the resulting fire resistance, specifically under severe fire exposures.

### Effect of Concrete Filling

The effect of the type of concrete filling on fire resistance is illustrated by analyzing HSS columns with different concrete filling types under ASTM E-119 fire exposure. As was the case with the effect of length, the applied load was modified according to AISC analysis procedures (AISC, 2005) to account for the different types of concrete filling for this analysis. The fire resistance is plotted as a function of length for three similar CFHSS columns (Figure 9), each with a different type (plain, steel-fiber-reinforced, bar-reinforced) of concrete filling, columns selected for this comparison are RP-355, RF-356 and RB-406, respectively. The fire resistance decreases with an increase in length for all of the filling types. However, columns filled with bar or steel-fiber-reinforced concrete demonstrate higher fire resistance than plain concrete-filled columns at all lengths. This can be attributed to the increased load carrying capacity of the concrete core provided by the inclusion of reinforcement, and is also due to the slower loss of strength in columns filled with bar- and steel-fiber-reinforced concrete. These results indicate that it is possible to significantly enhance the fire resistance of CFHSS columns by changing the type of concrete filling. The required fire resistance in most situations can be obtained for columns up to 10 m (32.8 ft) in length by simply altering the type of concrete filling.

### Effect of Load Ratio

The effect of load ratio on fire resistance was investigated by analyzing three columns under ASTM E-119 fire with load ratios ranging from 10% to 100% (0.1 to 1.0). The analysis was carried out for three types of concrete filling, namely, plain (RP-273), steel-fiber-reinforced (SF-219), and bar-reinforced (RB-273) filling. It can be seen in Figure 10 that only the bar-reinforced concrete-filled HSS column withstood the ASTM E-119 fire for 240 minutes (actual failure time) with a load ratio of 10% (0.1). Columns SF-219 and RP-273 lasted for 234 and 170 minutes, respectively. The fire resistance decreases rapidly with an increase in load ratio up to 0.4, after which the rate of decrease in resistance is slower. This can be attributed to the fact that concrete filling generally provides a load bearing contribution of about 30% to 40% of the overall composite column capacity. In a fire scenario, the steel shell looses its strength very quickly, and concrete carries most of the load. Thus, for load ratios higher

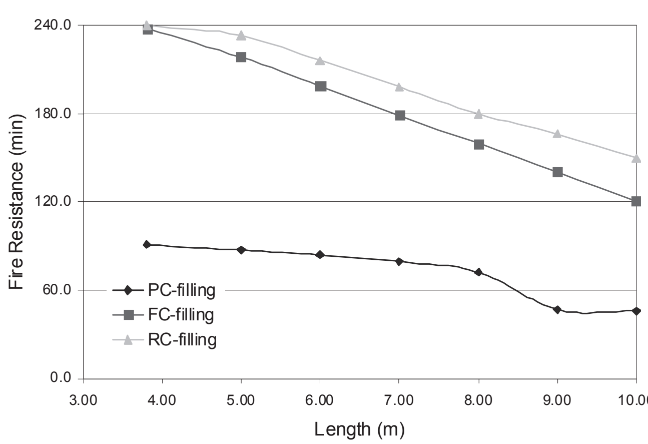


*Fig. 9. Effect of concrete filling on fire resistance of CFHSS columns.*

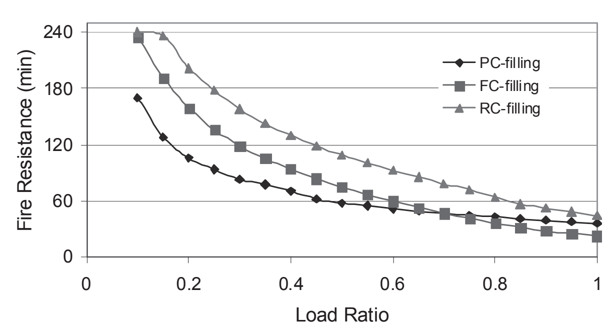


*Fig. 10. Effect of load ratio on fire resistance of CFHSS columns.*

than 40%, the concrete filling has to be strengthened either through the use of bar reinforcement, or through the use of steel fibers to achieve higher fire resistance as can be seen in Figure 5. In design fires, the same trend is observed as in the ASTM E-119 fire, with the exception that the times to reach failure are increased depending on the type of design fire considered. Design fires allow the use of load ratios in the range of 40% to 50% while still achieving the required fire resistance. It should be noted that only concentric loads were considered in the analysis presented here. Other loading types such as eccentric or combined axial and bending loads can influence the fire resistance of CFHSS columns and require further research.

### Effect of Cross-Section Size

Results from the analysis indicate that the fire resistance of CFHSS columns increases with an increase in cross-sectional size. This is due to the increased contribution of the concrete core to the column strength. When cross-section size is increased, the concrete core comprises a larger percentage of the load-bearing capacity of the section. When a CFHSS column is exposed to fire, the steel section loses its strength quickly and transfers the load to the concrete core. The larger core is capable of providing longer fire resistance times. The strength loss in the concrete core is slower than in the HSS section, allowing the column to achieve enhanced fire resistance. The increased cross-sectional size also enhances stiffness, and thus resistance to buckling of the column, allowing higher fire resistances to be achieved in longer columns. Columns filled with plain concrete, however, realize no additional advantage from cross-section increases beyond 400 mm (15.75 in.). This is due to high-temperature instability of the concrete core causing premature failure for larger cross-sectional sizes.

### Effect of Aggregate Type

Results from the analysis indicate that the type of aggregate has a moderate influence on the fire resistance of CFHSS columns. The two common types of aggregate used in CFHSS columns are carbonate aggregate (primarily consisting of limestone) and siliceous aggregate (primarily consisting of quartz). Carbonate aggregate concrete typically demonstrate higher fire resistance than siliceous aggregate concrete (Kodur and Lie, 1996; Kodur and Lie, 1997). This is due to an endothermic reaction occurring in carbonate aggregate at 600 to 800 °C (1100 to 1500 °F), in which the dolomite within the aggregate dissociates. Consequently, the heat capacity of carbonate aggregate increases significantly and is approximately 10 times higher than siliceous aggregate in the same temperature range. As a result, there is a slower increase in temperature in the carbonate aggregate concrete, and thus a slower loss of strength. Therefore, the fire resistance of CFHSS columns filled with carbonate aggregate concrete is about 10% higher than siliceous aggregate concrete-filled HSS columns.

### Effect of Failure Criterion

The two limit states considered for defining failure of steel columns under fire conditions are limiting temperature and stability retention. ASTM E 119 defines fire resistance as the time it takes to reach a maximum average section temperature of 538 °C (1000 °F), or a maximum point temperature of 649 °C (1200 °F). On the contrary, stability-based failure criteria are based on the duration of time during which a column maintains structural stability (strength) during fire exposure. In the case of CFHSS columns, using the limiting temperature criterion does not reflect realistic fire resistance ratings. As an illustration, column RP-273 achieved fire resistance of 143 minutes in testing and 128 minutes in the SAFIR simulation. Thermal failure criterion, however, would only yield a resistance of 38 minutes for this column. Clearly, thermal failure criteria do not reflect the contribution of the concrete filling to the fire resistance of CFHSS columns. As such, it is necessary to employ stability-based failure criterion in the evaluation of the fire resistance of CFHSS columns.

## PERFORMANCE-BASED DESIGN METHODOLOGY

In recent years, the performance-based approach to fire safety design is becoming popular since cost-effective and rational fire safety solutions can be developed using this approach (Kodur, 1999). One of the key aspects in any performance-based design is the fire-resistant design of structural members. Through the application of a performance-based approach, the full benefits of CFHSS columns under fire condition can be realized. For evaluating fire resistance, numerical models that can simulate the response of CFHSS columns under realistic fire, loading and restraint scenarios can be used. The main steps involved in undertaking a rational approach for performance-based design are:

1. Identifying proper design (realistic) fire scenarios and realistic loading levels on the CFHSS columns under consideration.

2. Carrying out detailed thermal and structural analysis by exposing the CFHSS column to fire conditions.

3. Developing relevant practical solutions, such as the use of different types of concrete filling, to achieve required fire resistance.

### Development of Fire Scenario and Loading

The design fire scenarios for any given situation can be established either through the use of parametric fires (time-temperature curves) specified in Eurocode 1 (Eurocode, 2002) or through design tables (Magnusson and Thelandersson,

1970) based on ventilation, fuel load and surface lining characteristics. To use the design tables, the ventilation factor, $F_v$, has to be established using the relationship:

$$F_v = \frac{A_v}{A_t}\sqrt{H_v} \tag{2}$$

where $A_v$ is the area of the window opening (m$^2$), $A_t$ is the total internal area of the bounding surface (m$^2$), and $H_v$ is the height of the window opening (m) (Buchanan, 2005). The fuel load in the compartment is determined by considering the total bounding surface (not just the floor surface area). Typical fuel loads for common compartment types are available (Parkinson and Kodur, 2007; Buchanan, 2005). Figure 1 shows typical design fire curves that can be generated using performance-based fire safety design. The presence of sprinklers can also be accounted for in the development of design fire scenarios.

The loads that are to be considered on concrete-filled HSS columns under fire conditions should be estimated based on the guidance given in ASCE 7 (1.2 dead load + 0.5 live load) (ASCE, 2005) or through actual calculations based on different load combinations.

### Structural Analysis Under Fire Exposure

Once the fire scenarios and load level are established, the next step is to select a computer program for the analysis of CFHSS columns exposed to fire. The computer program should be able to trace the response of the CFHSS column in the entire range of loading up to collapse under fire. Computer programs such as SAFIR (as demonstrated in this paper), ANSYS, or ABAQUS can be adopted for the analysis. Using the computer program, a coupled thermal-structural analysis shall be carried out at various time steps. In each time step, the fire behavior of a CFHSS column is estimated using a complex, coupled heat transfer/strain equilibrium analysis, based on theoretical heat transfer and mechanics principles. The analysis should be performed in three steps: calculation of fire temperatures to which the column is exposed; calculation of temperatures in the column; and calculation of resulting deflections and strength, including an analysis of the stress and strain distribution.

The computer program used in the analysis should be capable of accounting for nonlinear high-temperature material properties, complete structural (column) behavior, various fire scenarios, high-temperature creep, different concrete types (concrete with and without steel fibers), and failure criteria. In the analysis, geometric nonlinearity, an important factor for the slender columns that are used in many practical applications, should be taken into consideration. Thus, the fire response of the column may be traced in the entire range of behavior, from a linear elastic stage to the collapse stage under any given fire and loading scenario. Through this coupled thermal-structural analysis, various critical output parameters such as temperatures, stresses, strains, deflections and strengths have to be generated at each time step.

The temperatures in the concrete and reinforcement, strength capacities and computed deflections of the column shall be used to evaluate failure of the column at each time step. At every time step, the failure of the column shall be checked against a predetermined set of failure criteria. The time increments continue until a certain point at which the strength failure criterion has been reached, or the axial deformations reach their limiting state. At this point, the column becomes unstable and will be assumed to have failed. The time to reach this failure point is the fire resistance of the column.

### Development of Practical Alternatives

Results from the analysis can be utilized to develop practical solutions for achieving the required fire resistance in CFHSS columns. The most feasible solution is through changing the type of concrete filling (plain, bar-reinforced, or steel-fiber-reinforced concrete). Other factors, such as the type of aggregate in the concrete, reinforcement in the column or load level can be varied to achieve the required fire resistance in HSS columns. As an example, while plain concrete filling can provide one-hour fire resistance in HSS columns, by switching to steel-fiber-reinforced concrete filling, up to three hours of fire resistance can be obtained even under severe design fire scenarios. As is currently the case with CFHSS columns, it is required that steam vent holes be provided at the top and the bottom of compartment elevations for any fire resistance to be achieved.

## DESIGN IMPLICATIONS

The approach presented here is capable of tracing the behavior of CFHSS columns from the initial pre-fire stage to the failure of the column under realistic fire, loading and failure criterion. The proposed approach can be used to overcome many of the current limitations in achieving unprotected steel in HSS columns in most practical applications. Thus, the use of this approach will lead to designs that are not only economical, but that are based on rational design principles. Further, the approach can be applied to conduct parametric studies, which can then be used to develop rational fire safety design guidelines for incorporation into codes and standards.

Through implementation of the design process outlined here, structural safety and integrity will be improved, construction time and cost will be reduced, and exposed structural steel can be achieved. Applications for these CFHSS columns could include airports, schools, detention facilities and high-rise buildings, where structural stability in fire is paramount and relatively high fire resistance is required.

## CONCLUSIONS

Based on the results of this study, the following conclusions can be drawn:

- The current fire resistance provisions, developed based on limited standard fire tests under "standard fire scenarios," are prescriptive and simplistic in application and, thus, cannot be applied for rational fire safety design of CFHSS columns under performance-based codes.
- Type of fire exposure has a significant influence on the fire resistance of CFHSS columns. The fire resistance of CFHSS columns under most design fire scenarios is higher than that under ASTM E-119 standard fire exposure.
- Apart from fire exposure, the other significant factors that affect the fire resistance of CFHSS columns are length, type of concrete filling, load ratio and failure criteria.
- It is possible to obtain unprotected CFHSS columns up to 10 m (32.8 ft) in length capable of withstanding complete compartment burnout through the use of different types of concrete filling.
- Through the use of a performance-based design approach, it is possible to significantly enhance the fire resistance of CFHSS columns by varying parameters such as the type of concrete filling, type of aggregate in the concrete, and the fire scenario.
- The limiting criterion, used for determining failure, has a significant influence on the fire resistance of CFHSS columns. The conventional failure criteria—for example, limiting steel temperature—cannot be applied to CFHSS columns. Strength and deformation failure criteria should be considered for evaluating fire resistance of CFHSS columns.
- Further research is needed to develop simplified design equations to evaluate the fire resistance of CFHSS columns under design fire exposure without the need for advanced numerical models.

## ACKNOWLEDGMENT

The research presented in this paper was primarily supported by the American Institute of Steel Construction through an AISC Faculty Fellowship Award to Venkatesh Kodur.

## REFERENCES

AISC (2005), *Steel Construction Manual*, 13th ed., American Institute of Steel Construction, Chicago, IL.

ASCE 7-05 (2005), *Minimum Design Loads for Buildings and Other Structures*, American Society of Civil Engineers, Reston, VA.

ASCE/SFPE 29 (1999), *Standard Calculation Method for Structural Fire Protection*, American Society of Civil Engineers, Reston, VA.

ASTM (2001), *Standard Methods for Fire Test of Building Construction and Materials*, Test Method E1529-93, American Society for Testing and Materials, West Conshohocken, PA.

ASTM (2007), *Standard Methods of Fire Test of Building Construction and Materials*, Test Method E119-00, American Society for Testing and Materials, West Conshohocken, PA.

Bond, G.V.L. (1975), *Fire and Steel Construction, Water cooled Hollow Columns*, Constrado, Croydon.

Buchanan, A.H. (2005), *Structural Design for Fire Safety*, John Wiley and Sons Ltd., New York.

Chabot, M. and Lie, T.T., (1992), "Experimental Studies on the Fire Resistance of Hollow Steel Columns Filled with Bar-Reinforced Concrete," IRC Internal Report No. 628, National Research Council of Canada.

Chen, F.F. and Gemeny, D.F. (2004), "Case Study Using SAFIR to Predict Fire Resistance of a Column in a Performance-Based Environment," *SFPE Fire Protection Magazine,* No. 23, Summer 2004, pp. 30–35.

Eurocode 1 (2002), ENV 1991 1-2: "General Actions—Actions on Structures Exposed to Fire," European Committee for Standardization.

Eurocode 3 (2005a), ENV 1993 1-2: "General Rules—Structural Fire Design," European Committee for Standardization.

Eurocode 4 (2005b), ENV 1994 1-2: "Design of Composite Steel and Concrete Structures," European Committee for Standardization.

FCSA (1989), *Fire Technical Design Manual for Composite Columns with Concrete Filled Hollow Steel Sections*, Finnish Constructional Steelwork Association, Helsinki, Finland, pp. 66.

FEMA (2002), "World Trade Center Building Performance Study: Data Collection, Preliminary Observations, and Recommendations," Federal Emergency Management Agency, Washington, DC.

Franssen, J.M. (2005), "SAFIR: A Thermal/Structural Program for Modeling Structures Under Fire," *Engineering Journal,* Vol. 42, No. 3, pp. 143–155.

Gilvary, K. and Dexter, R. (1997), "Evaluation of Alternative Methods for Fire Rating Structural Elements," NIST Building and Fire Research Laboratory, Gaithersburg, MD.

Grandjean, G., Grimault, J.P. and Petit, L. (1981), "Determination de la durée au feu des profils creux remplis de béton," Rapport final, Commission des Communautés Européennes, Recherche Technique Acier, Luxembourg.

Han, L. and Lin, X. (2004), "Test on Cyclic Behavior of Concrete Filled Hollow Structural Steel Columns after Exposure to the ISO-834 Standard Fire," *Journal of Structural Engineering,* Vol. 130, No. 11, pp. 1807–1819.

ISO 834 (1975), "Fire Resistance Tests—Elements of Building Construction," International Organization for Standardization, Geneva, Switzerland.

Klingsch, W. and Wittbecker, F.W. (1988), "Fire Resistance of Hollow Section Composite Columns of Small Cross Sections," Bergische Universität, Wuppertal, Germany, pp. 103.

Klingsch, W. and Wuerker, K. (1985), "New Developments in Fire Resistance of Hollow Section Structures," *Proceedings of the Symposium on Hollow Structural Sections in Building Construction,* Chicago, IL.

Kodur, V.K.R. (2005), "Achieving Fire Resistance Through Steel Concrete Composite Construction," *Proceedings of the 2005 ASCE Structures Congress*, New York.

Kodur, V.K.R. (1999), "Performance-based Fire Resistance Design of Concrete-filled Steel Columns," *Journal of Constructional Steel Research*, Vol. 51, pp. 21–36.

Kodur, V.K.R. (1997), "Design Equations for Evaluating Fire Resistance of SFRC-filled Steel Columns," *Journal of Structural Engineering*, Vol. 124, No. 6, pp. 671–678.

Kodur, V.K.R. and Harmathy, T.Z. (2002), "Properties of Building Materials," *SFPE Handbook of Fire Protection Engineering*, National Fire Protection Association, Quincy, MA.

Kodur, V.K.R. and Lie, T.T. (1995a), "Experimental Studies on the Fire Resistance of Circular Hollow Steel Columns Filled With Steel Fibre Reinforced Concrete," IRC Internal Report No. 691, National Research Council of Canada.

Kodur, V.K.R. and Lie, T.T. (1995b), "Fire Performance of Concrete-filled Hollow Steel Columns," *Journal of Fire Protection Engineering*, Vol. 7, No. 3, pp. 89–98.

Kodur, V.K.R. and Lie, T.T. (1996), "Fire Resistance of Circular Steel Columns Filled With Fiber-reinforced Concrete," *Journal of Structural Engineering*, Vol. 122, No. 7, pp. 776–782.

Kodur, V.K.R. and Lie, T.T. (1997), "Evaluation of Fire Resistance of Rectangular Steel Columns Filled With Fiber-Reinforce Concrete," *Canadian Journal of Civil Engineering*, Vol. 24, pp. 339–349.

Kodur, V.K.R. and MacKinnon, D.H. (2000), "Fire Endurance of Concrete-filled Hollow Structural Steel Columns," *Engineering Journal*, Vol. 37, No. 1, pp. 13–24.

Lie, T.T. and Caron, S.E. (1988), "Fire Resistance of Circular Hollow Steel Columns Filled with Carbonate Aggregate Concrete: Test Results," IRC Internal Report No. 573, National Research Council of Canada.

Lie, T.T. and Chabot, M. (1990), "A Method to Predict the Fire Resistance of Circular Concrete Filled Hollow Steel Columns," *Journal of Fire Protection Engineering*, Vol. 2, No. 4, 111–126.

Lie, T.T. and Chabot, M. (1992), "Experimental Studies on the Fire Resistance of Hollow Steel Columns Filled With Plain Concrete," IRC Internal Report No. 611, National Research Council of Canada.

Lie, T.T. and Irwin, R.J. (1995), "Fire Resistance of Rectangular Columns Filled with Bar-Reinforced Concrete," *Journal of Structural Engineering*, Vol. 121, No. 5, pp. 797–805.

Lie, T.T., Irwin, R.J., and Chabot, M (1991), "Factors Affecting the Fire Resistance of Circular Hollow Steel Columns Filled With Plain Concrete," IRC Internal Report No. 612, National Research Council of Canada.

Lie, T.T. and Kodur, V.K.R. (1996), "Fire Resistance of Steel Columns Filled With Bar-reinforced Concrete," *Journal Structural Engineering*, Vol. 122, No. 1, pp. 30–36.

Lie, T.T. and Kodur, V.K.R. (1996), "Thermal and Mechanical Properties of Steel Fiber Reinforced Concrete at Elevated Temperatures," *Canadian Journal of Civil Engineering*, Vol. 23, No. 2, pp. 511–517.

Lie, T.T. and Stringer, D.C. (1994), "Calculation of Fire Resistance of Steel Hollow Structural Steel Columns Filled With Plain Concrete," *Canadian Journal of Civil Engineering*, Vol. 21, No. 3, pp. 382–385.

Magnusson, S.E. and Thelandersson, S. (1970), "Temperature-Time Curves of Complete Process of Fire Development; Theoretical Study of Wood Fuel Fires in Enclosed Spaces," *Civil Engineering and Building* Series 65, Acta, Polytechnica, Scandinavia.

Meacham, B.J. and Custer, R.P.L. (1992), "Performance-based Fire Safety Engineering: An Introduction of Basic Concepts," *Journal of Constructional Steel Research*, Vol. 7, pp. 35–54.

NBC (2005), *National Building Code of Canada,* National Research Council of Canada, Canada.

NIST (2005), "Final Report of the National Construction Safety Team on the Collapse of World Trade Center Towers," NCSTAR1, National Institute of Standards and Technology, Gaithersburg, MD.

Parkinson, D.L. and Kodur, V.K.R. (2007), "Performance-Based Design of Structural Steel for Fire Conditions—A Calculation Methodology," *International Journal of Steel Structures,* Vol. 7, No. 3, pp. 219–226.

Talamona, D. (2005), "A Quadralateral Shell Finite Element for Concrete and Steel Structures Subject to Fire," *Journal of Fire Protection Engineering,* Vol. 15, No. 4, pp. 237–264.

Twilt, L. Hass, R. Klingsch, W. Edwards, M. and Dutta, D. (1996), *Design Guide 4 for Structural Hollow Section Columns Exposed to Fire*, first edition, CIDECT.