An Approach for Evaluating the Fire Resistance of CFHSS Columns under Design Fire Scenarios

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ABSTRACT: The use of concrete filling offers a practical alternative for achieving the required fire resistance in steel hollow structural section columns. However, the current prescriptive-based approach which evaluates fire resistance based on standard fire exposure does not account for realistic fire scenarios in the design of concrete-filled hollow structural section (CFHSS) columns. This article presents a methodology for evaluating the fire resistance of CFHSS columns under design fire scenarios without the need for costly computational models. The proposed approach is a derivative of the equal area concept, and evaluates the equivalent fire resistance of the column by comparing the time temperature curve of the standard fire exposure with that of the design fire exposure. The method has been validated against the results generated from finite element analysis (coupled heat transfer and strength analysis) on numerous CFHSS columns under a large number of design fires. The applicability of the approach in design situations is illustrated through a numerical example, and it is concluded that the proposed approach offers an attractive alternative for deriving equivalent fire resistance of CFHSS columns exposed to design fire scenarios.

KEY WORDS: concrete filling, equal area, concrete-filled hollow structural steel, design fire, ASTM E-119, HSS column, equivalent fire resistance.

INTRODUCTION

STEEL HOLLOW STRUCTURAL sections (HSS) are very efficient in resisting compression, torsion, and seismic loads, and are widely used as compression members in the construction of steel-framed structures. Structural steel, like most construction materials, loses strength and stiffness and expands as temperatures increases, thus making stability under fire

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exposure one of the primary considerations in the design of high-rise buildings. Building codes address this problem in a prescriptive manner by acquiring 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, reduced esthetics, increased weight of the structure, and decreased 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 [1,2].

Often these HSS columns are filled with concrete to enhance the stiffness, torsional rigidity, and load-bearing capacity. The two components of the composite column complement each other ideally. The steel casing confines the concrete allowing it to act as in tri-axial compression and develop its optimum compressive strength, while the concrete, in turn, enhances resistance to local buckling of the steel wall and global buckling of the column. Concrete filling also significantly enhances fire resistance and thus, external fire protection for the steel can be completely eliminated in many situations. 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 [3–5].

Design guidelines for achieving fire resistance through concrete filling have been incorporated into codes and standards [6–8]. However, the current fire guidelines are limited in scope and restrictive in application since they were developed based on the standard [9] fire test, and are valid only for standard fire exposure conditions. In many applications, such as atriums, schools, and airports, where exposed steel is highly desired, the current prescriptive provisions preclude the use of concrete-filled hollow structural section (CFHSS) columns due to a number of limitations including the use of a prescriptive fire exposure. Thus, designers cannot take advantage of concrete-filling options for achieving fire resistance in HSS columns.

As an alternative to the current prescriptive methods, performance-based fire safety design is gaining momentum the world over. Performance-based fire safety takes into account several parameters such as fire scenario, loading, restraint, and failure criterion. The consideration of these factors makes performance-based design desirable, but at the same time complicates the design process. Of the different factors, fire exposure is key since it has a significant influence on fire resistance. The use of probabilistic design fires that take into account the occupancy type, geometry, and layout of the compartment in which the column is located gives a realistic estimation of the fire resistance. However, the infinite number of possible fire scenarios would restrict the use of testing options for evaluating fire resistance. Although detailed finite element analysis, involving heat transfer and strength (deflection) calculations, can be carried out to evaluate fire resistance, it is quite complex and time consuming. As an alternative to conducting detailed finite element analysis for numerous fire-column combinations, a simplified approach is presented in this article for evaluating the equivalent fire resistance of CFHSS columns exposed to design fires. The proposed approach is a derivative of the equivalent area concept, and establishes fire performance of a column by comparing the area under the standard time temperature curve to that under the design fire curve. Full details on the development of the approach including validation and applicability in design situations are discussed.

PERFORMANCE-BASED DESIGN

Recently there has been an increased impetus in moving toward a performance-based approach for fire safety design [10,11]. 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 achieving fire safety. Fire resistance of structural members is a critical component of overall fire safety. There are two basic methods by which performance-based fire resistance design can be accomplished: through full-scale fire resistance tests on structural members, or through numerical/computational simulations. Due to the high cost, time, and effort associated with full-scale fire testing, the first option is mostly used to validate numerical models. Numerical models allow consideration of most of the significant factors that influence fire resistance. The most important factors to be considered in performance-based fire design of structural systems are fire scenario, load conditions, and failure criterion [12].

Fire Scenario

The current practice for evaluating fire resistance of CFHSS columns is based on fire tests or empirical methods, in which the column is exposed to a standard fire as specified in standards such as ISO 834, ASTM E-119, or ASTM E-1529 [13,9,14]. While standard fire resistance tests are useful benchmarks to establish the relative performance of different CFHSS columns, they bear little resemblance to the often less-severe heating environments encountered in real fires in which a decay (cooling) phase is present. Figure 1 illustrates various time–temperature curves for standard and some realistic (design) fire scenarios [15]. In the standard fires (ASTM E-119 and E-1529) [9,14], the fire duration and intensity 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 (FV04 to

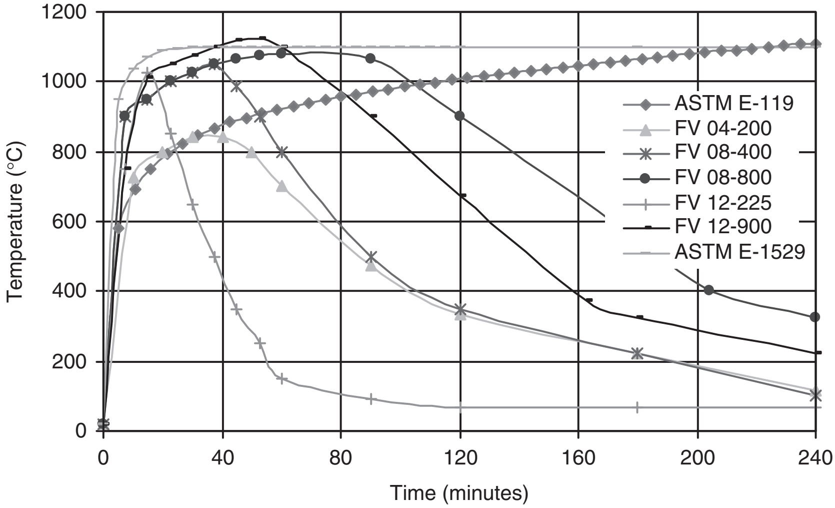

Figure 1. *Time–temperature relationships for various fire scenarios.*

FV12), the fire severity is a function of compartment characteristics, such as ventilation, fuel load, and lining materials, and there is a well-defined decay phase as clearly shown in Figure 1 [15]. In the decay phase of the realistic fire scenarios, the cross-section of the column enters the cooling phase, in which the steel recovers part of its strength and stiffness, and thus, a column reaching this point will have sufficient fire resistance to withstand complete compartment burnout.

Load Ratio

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 that are to be applied on CFHSS columns, in the event of fire, can be estimated based on the guidance given in the ASCE-07 standard [16] (1.2 dead load + 0.5 live load) or through actual calculations based on different load combinations. Based on ASCE-07 [16] and the AISC LRFD Manual [7], and based on typical dead to live load ratios (in the range of 2–3), the load ratio for CFHSS columns under fire conditions ranges between 30% and 50%. It is necessary to consider realistic load levels under fire exposure because the strength of the column decreases with fire exposure time due to the degradation of the steel (and concrete) properties, and failure occurs when the strength of the column goes below the applied (external) loads Thus, higher applied loads shorten the time required to reach failure. Therefore, it is necessary to evaluate the fire resistance of structural members with realistic load levels.

Failure Criterion

Many tests and numerical analysis procedures use a limiting steel temperature to define failure. The limiting temperature, often referred to as critical temperature, is defined as the temperature at which the steel has about 50% of its room temperature strength. For structural steel, the limiting temperature is 538°C [17] and it is commonly assumed that once the steel section reaches this temperature, failure is imminent. While this approach may be sufficient for traditionally protected steel sections, the effect of the concrete core in CFHSS columns is not fully captured by this thermal failure criterion. To ascertain realistic fire resistance predictions, the stability of the CFHSS column under fire conditions needs to be considered. Depending on factors such as the end conditions, 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 connection with adjoining members, fire resistance can be enhanced appreciably due to redistribution of moments between critical sections of the column. 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 [18–20]. 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 last three decades. Methods such as filling the HSS columns with liquid (water) and concrete are among the popular approaches studied by researchers [3,21,22]. However, the use of concrete-filling is the most attractive and feasible proposition developed thus far. Three types of concrete filling, namely plain concrete (PC), fiber-reinforced concrete (FC), and bar-reinforced concrete (RC), were proposed by researchers to achieve fire resistance (up to 3 hours) in CFHSS columns.

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 [5,22–27]. The experimental program at NRCC consisted of fire tests on about 80 full-scale CFHSS columns [3,23–26]. Both square and circular CFHSS 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 [9] standard fire exposure. The tests reported by other European and Chinese studies [22,27,28] are similar to NRCC tests, but the fire exposure was mainly that of the ISO 834 [13] standard fire, whose time–temperature curve is similar to that of ASTM E-119 [9].

The numerical studies, primarily carried out NRCC, led to the development of a mathematical model for predicting the fire behavior of circular and square CFHSS columns [29–31] under ASTM E-119 fire exposure. In this model, the fire resistance is evaluated in various time steps, consisting of the calculation of the temperature of the fire, to which the column is exposed, the temperatures in the column, their deformations and strength during exposure to fire, and finally, their fire resistance. Full details on the development and validation of these models are given in references [20,24,29].

Design Equation for Evaluating Fire Resistance

Using the results from computer-simulated parametric studies, as well as fire resistance tests, a unified design equation has been developed for calculating the fire resistance of circular and square HSS columns filled with any of the three types of concrete [32–34]. Equation (1) expresses the fire resistance of a CFHSS column, under ASTM E-119 exposure, as a function of influencing parameters:

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

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 (PC, RC, and FC), the type of aggregate used (carbonate or siliceous), 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 reference [31].

Equation (1) though offering a convenient way of evaluating fire resistance, is limited to standard fire exposure only and thus may not lead to cost effective and realistic fire resistance design. The type of fire exposure has a significant influence on fire resistance of CFHSS columns and should be accounted for in order to estimate the realistic fire performance of CFHSS columns.

Fire Resistance Evaluation Methods

The infinite number of realistic (design) fires to which a structural member could be exposed makes it impossible to analyze all of the possible member-fire combinations. A feasible approach for evaluating the fire resistance (duration of column stability) of a structural member under realistic (design) fires is to establish equivalency between the performance of a member under design and standard fire exposures. A review of the literature indicates that several equivalent fire severity techniques have been developed over the past several years for steel structural members. These techniques attempt to correlate the fire resistance under standard fire exposure to that under design fire exposure. Some of the methods widely used to do this are discussed below.

EQUAL AREA CONCEPT

The equal area concept, as the name implies, establishes the fire resistance equivalency using the principle of equivalent fire exposure wherein the area under the standard (ASTM E-119 or ISO 834) time–temperature curve is compared to the area under the design fire time–temperature curve. In this method, the area under the standard fire time–temperature curve at the time the structural member fails is determined. Then the area under the possible design fire curve is estimated. The time at which the area under the design fire curve is equal to that under the standard fire curve (at the time of structural member failure) is the equivalent fire resistance of the member in the design fire. This approach has been validated for steel structural members and reasonable agreement between predicted and measured values has been observed. However, the main limitation of the method is that it is not applicable for design fires that are either very hot and short, or cold and long as compared to the standard fire. Thus, the method does not accurately correlate the fire resistance times due to the reliance on the meaningless unit of area. Additionally, the equal area concept has not been validated for use on composite structural members including CFHSS columns.

MAXIMUM TEMPERATURE CONCEPT

This method was developed based on the assumption that steel structural members fail when steel reaches a critical temperature. For steel columns, the critical temperature is often taken as 538°C. To use this method, the maximum steel temperature achieved under a given design fire is determined first. The steel temperature under standard (ASTM E-119 or ISO 834) fire exposure is then determined. The time under standard fire exposure that it takes to reach the maximum steel temperature reached in the design fire is taken as the equivalent fire exposure. The limitation to this method is that when the temperatures reached in the design fire are well above or below those that would cause failure in the section, the method is not very accurate. Also, this method cannot be directly applied to composite members, (columns) since a limiting temperature in steel does not necessarily indicate failure of the member due to the composite action between the steel and the concrete.

MINIMUM LOAD CAPACITY CONCEPT

This approach utilizes a strength failure limit state to establish equivalency between a standard and a design fire exposure. Similar to the maximum temperature concept, the minimum load capacity concept compares the time that it takes to reach a certain load capacity of the section under the two fires. The minimum load capacity of the section under the design fire is determined. The load capacity under the standard fire exposure is then determined. The point where the load capacity of the column under the standard fire is the same as that under design fire exposure is the equivalent fire resistance. The limitation to this method is the assumption that load level is about 50% of the room temperature capacity of the member, and it does not take into account the actual load level on the structural member. Further, the method cannot fully account for the effect of concrete-filling in CFHSS columns.

TIME EQUIVALENT FORMULA

In addition to the above approaches, several empirical relationships such as the CIB and Law formulae have been proposed to establish equivalency between standard and design fire exposures [35]. Globally, these methods were developed for concrete or protected steel sections. As such, the empirical formulae available in literature are incapable of capturing the effect of concrete-filling on the fire resistance of CFHSS columns.

Limitations of Fire Resistance Evaluation Methods

The fire test data, numerical models, and resulting design equation available in literature represent a considerable contribution to the understanding of the performance of CFHSS columns under fire exposure. While forming a good basis for comparison of column fire resistance, the exclusive use of standard fire exposure in the fire resistance assessment precludes determination of the actual failure time of a CFHSS column exposed to real fires. Standard fire exposure does not take into account any of the room characteristics such as ventilation or fuel supply, and presents a monotonically increasing time–temperature relationship in which no consideration is given to the decay phase, thus representing an unrealistic and overly conservative fire condition. The further inability of any of the equivalent fire severity methods to estimate the equivalent fire resistance of composite structural members has precluded the use of CFHSS columns in many applications to which they would be well suited in a performance-based environment.

Evaluating fire resistance under realistic conditions, including fire exposure, requires detailed finite element analysis, which is complex, costly and time consuming. In lieu of detailed finite element analysis, simple methods can be applied for evaluating fire resistance, provided an equivalency is established between fire performance of CFHSS columns under standard and design fire scenarios.

NUMERICAL STUDIES

The development of an approach for evaluating equivalent fire resistance of CFHSS columns requires a large data set to capture the fire response of columns exposed to standard and design fire scenarios. To generate such data, a set of numerical studies was carried out to evaluate the effect of fire exposure on the fire resistance of CFHSS columns. The numerical simulations were carried out using a finite element based computer program, SAFIR, to expose CFHSS columns to various fire and loading scenarios. The details associated with the analysis are discussed in the following section.

Computer Program

The computer program SAFIR, developed at the University of Liege in Belgium, can model the behavior of structural systems under any fire exposure. The program is capable of accounting for multiple materials in a cross-section, both heating and cooling phase of fire exposure, large displacements, different strain components, non-linear material properties according to Eurocode 3, 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 results from SAFIR have been shown to closely match those from other numerical models and test data. Details of the validations can be seen in the literature [36–38].

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.

For structural analysis, SAFIR uses a fiber-based beam element 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 takes into account the contribution of each of the fibers. Due to the increasing temperature in the column, the strength and stiffness decrease 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. Limitations of the structural model include the assumption that there is no slip between the steel and the concrete, and that concrete once cracked in tension retains all of its compressive strength.

Test Columns

Fourteen CFHSS columns tested at NRCC under the standard fire scenario were selected for validation of the SAFIR computer model [19,31,39–41]. All pertinent information for the CFHSS columns is presented in Table 1. In the designation of columns (e.g. RP-355), the first letter (R) represents section shape (round or square), the second letter (P, F, B) denotes concrete-filling type (plain, steel fiber, and bar-reinforced concrete), and the number (355) denotes the diameter (for circular) or width (for square) of the HSS.

Model Validation

For validation of SAFIR, the above 14 columns were analyzed by exposing them to the ASTM E-119 fire scenario. The thermal and structural response, and ultimate failure times, generated by SAFIR were compared with the measured test data for all of the columns. Full details of the analysis using SAFIR including discretization and model validation is given by Kodur and Fike [42]. The fire resistance times for all 14 columns are presented in Table 1, and detailed results for column SP-178 are discussed below. The fire resistance presented in the Table 1 refers to the time to reach failure based on the strength limit state. All of the columns for which test data were available underwent the same scrutiny as the single column (SP-178) used to illustrate the validation, below. All of the results, however, could not be presented due to space constraints.

Figure 2 shows the comparison of temperatures predicted by SAFIR at the steel surface (HSS section), and at the center of the concrete core, with the measured temperatures in a fire test for column SP-178. It can be seen that steel and concrete temperatures increase with fire exposure time.

Table 1. Test parameters and fire resistance values for CFHSS columns.

					Fire resistance		
Column designation	Dia. or width (mm)	Length (mm)	AISC factored load (KN)	Load ratio	Test (minutes)	SAFIR (minutes)	Ratio (SAFIR/test)
RP-168	168.3	3810	1197	0.13	81	82	1.01
RP-273	273.1	3810	3508	0.15	143	128	0.90
RP-355	355.6	3810	5120	0.18	170	164	0.96
SP-152	152.4	3810	1409	0.20	86	74	0.86
SP-178	177.8	3810	1976	0.28	80	71	0.89
RF-324	323.9	3810	4573	0.35	199	200	1.01
RF-356	355.6	3810	6616	0.23	227	238	1.05
SF-203	203.2	3810	3506	0.26	128	121	0.95
SF-219	219.1	3810	3793	0.16	174	185	1.06
RB-273a	273.1	3810	3323	0.32	188	158	0.84
RB-273b	273.1	3810	3333	0.57	96	98	1.02
SB-203	203.2	3810	2345	0.21	150	130	0.87
SB-254a	254	3810	3405	0.42	113	110	0.97
SB-254b	254	3810	3405	0.65	70	69	0.99

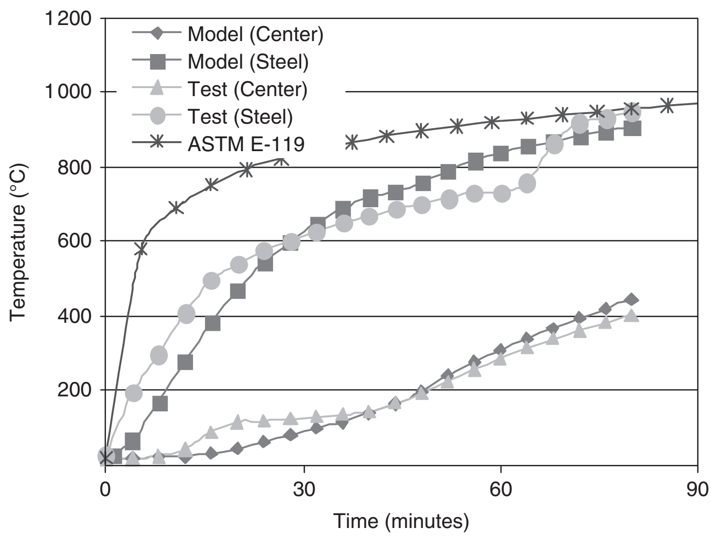

Figure 2. *Comparison of predicted and measured temperatures for column SP-178.*

However, the steel temperatures increase at a faster rate than the temperatures at the mid-depth of the concrete core, and this is due to the fact that the steel is directly exposed to the fire. Good agreement between predicted and measured temperatures is observed for the concrete core after about 100°C, at which point the free water is driven off. The steel temperatures initially deviate such that temperatures in tests are higher 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. When however, the steel approaches the critical phase transformation temperature of 700–750°C, at which point the chemically bonded water in the concrete is also driven off, SAFIR begins to over-predict the temperature in the steel. This is mainly due to the assumption in SAFIR that hydraulic migration in concrete can be neglected, thus less heat is removed from the steel by the concrete causing higher steel temperatures. Toward the end of the test, the temperatures compare reasonably well such that there is only a 30°C 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 fire tests (Figure 3). 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 gradually take over the load-bearing function, leading to contraction in the CFHSS column. A peak deformation (expansion) of 18.3 mm was observed in the tests, while the corresponding peak deformation from SAFIR was 17.9 mm. These two maximum axial deformations show good agreement occurring at 20 and 23 minutes respectively. The slight differences in deformations between

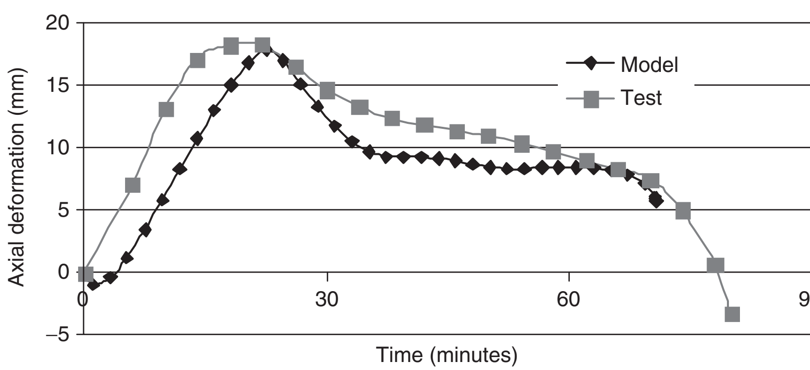

Figure 3. *Comparison of measured and predicted axial deformation for column SP-178.*

SAFIR predictions and test data are a result of the minor temperature discrepancies seen in Figure 2. The initially higher temperatures in the steel shell produce higher expansion of the column in the fire test than in SAFIR, as seen in Figure 3. After peak deformations are reached, the temperatures in the steel shell as predicted by SAFIR are higher than those observed in tests. This results in the steel retaining slightly higher stiffness 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 1. 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.

Parametric Studies

The validated SAFIR computer model was used to carry out a set of parametric studies to quantify the effect of design fires on the fire resistance of CFHSS columns. For this study, the 14 columns presented in Table 1 were analyzed by exposing them to 75 design fires. The result was 1050 fire-column combinations being simulated, thus providing the wide range of data necessary to establish an approach for determining fire resistance equivalency. The time–temperature curves for each of these design fires were selected by assessing different compartment characteristics. The selected fire scenarios ranged from a severe fire, representing a library room or records storage area, to a mild fire scenario, representing an average office room in a typical building. Five of these fires, including the most severe fire and the mildest fire are shown in Figure 1 along with 2 standard fires (ASTM E-119 and ASTM E-1529). All of the design fires used in this study were taken from Magnusson and Thelandersson [15].

Load ratios on the columns were maintained at the same levels noted in Table 1. This allowed the influence of other inherent properties of the column to be investigated without the added ambiguity of varying loads. All of the loads were applied as a concentric load on the top of the column (including the self-weight). The point loads were applied at the neutral axis of the section such that no eccentricity resulted from the load application. This was done to simulate the conditions believed to be present during testing. In order, however, to simulate the constructional imperfections present in any structure, the columns were assumed to have an out of straightness of $H/500$ at the center of the column, where H is the height of the column.

Failure is said to occur when the column can no longer support the applied load. In SAFIR, this point generally corresponds to the time at which the stiffness matrix is no longer positive. Failure of the column can occur due to either local failure within the beam elements used for the column cross-section, or it can be due to global buckling of the column. Due to space limitations, the large volume of results cannot all be presented here, though a representative set of data is shown in Table 2.

DEVELOPMENT OF NEW METHODOLOGY

As pointed out in previous sections, many of the available methods for equivalent fire severity were developed for traditionally protected steel members, failure of which is based on a limited temperature rise in steel. When these methods (equivalent area, maximum temperature, and minimum load) discussed above were applied to the CFHSS columns presented in Table 1, it was quickly determined that none of the methods can accurately predict the failure time of CFHSS columns based on the ASTM E-119 fire resistance. The main reason for this is that CFHSS columns derive their fire resistance from the strength contribution of the concrete core, and thus determining the fire resistance based on critical steel temperature does not indicate true failure.

In the process of checking each of the equivalent fire severity concepts for CFHSS columns, it was observed that CFHSS columns either fail in less time in the design fire than the standard fire, or they survive complete compartment burnout. Further scrutiny revealed that a modification to the equal area concept produced a method by which failure of a column in a design fire can be predicted with a high degree of reliability. Rather than using the equal area concept to determine an equivalent time of failure in the design fire based on standard (ASTM E-119 or ISO 834) fire resistance, the equal area concept was used to compare initial fire severity and to evaluate the survival of a CFHSS column in a compartment burnout. Since a CFHSS column will either fail in less time in a design fire than in the standard fire or will survive complete compartment burnout, only the severity of the fire up to the time of column failure under standard fire exposure needs to be compared. It should however be pointed out that determining the time of failure for the column in a design fire is not the objective, it is rather to determine if the column will fail in the design fire or withstand complete compartment burnout.

Table 2. Comparison of fire resistance equivalency as predicted by SAFIR and the proposed approach.

Type of fire exposure	RP 168		SP 152		RF 324		SF 203		RB 273b		SB 203	
	SAFIR failure	Predicted failure	SAFIR failure	Predicted failure	SAFIR failure	Predicted failure	SAFIR failure	Predicted failure	SAFIR failure	Predicted failure	SAFIR failure	Predicted failure
ASTM	82	NA	74	NA	204	NA	121	NA	98	NA	130	NA
FIRE 1	-	N	-	N	-	N	-	N	-	N	-	N
FIRE 2	-	N	-	N	-	N	-	N	-	N	-	N
FIRE 3	-	N	-	N	-	N	-	N	-	N	-	N
FIRE 4	-	N	-	N	-	N	-	N	-	N	-	N
FIRE 5	-	Y	70	Y	-	N	-	N	-	Y	-	N
FIRE 6	77	Y	70	Y	-	N	123	Y	94	Y	-	Y
FIRE 7	-	Y	47	Y	-	N	-	N	-	Y	-	N
FIRE 8	49	Y	47	Y	-	N	85	Y	67	Y	-	Y
FIRE 9	49	Y	47	Y	-	Y	83	Y	67	Y	88	Y
FIRE 10	42	Y	40	Y	-	N	-	N	-	Y	-	N
FIRE 11	42	Y	40	Y	-	N	74	Y	58	Y	-	Y
FIRE 12	42	Y	40	Y	-	Y	73	Y	58	Y	77	Y
FIRE 13	-	N	-	N	-	N	-	N	-	N	-	N
FIRE 14	-	N	-	N	-	N	-	N	-	N	-	N
FIRE 15	-	N	-	N	-	N	-	N	-	N	-	N
FIRE 16	-	N	-	N	-	N	-	N	-	N	-	N
FIRE 17	-	N	-	N	-	N	-	N	-	N	-	N
FIRE 18	-	Y	-	Y	-	N	-	N	-	N	-	N
FIRE 19	54	Y	52	Y	-	N	94	Y	72	Y	-	Y
FIRE 20	54	Y	52	Y	-	Y	89	Y	72	Y	94	Y
FIRE 21	45	Y	43	Y	-	N	-	N	-	Y	-	N
FIRE 22	45	Y	43	Y	-	N	77	Y	62	Y	-	Y

(Continued)

Table 2. Continued.

Type of fire exposure	RP 168		SP 152		RF 324		SF 203		RB 273b		SB 203	
	SAFIR failure	Predicted failure	SAFIR failure	Predicted failure	SAFIR failure	Predicted failure	SAFIR failure	Predicted failure	SAFIR failure	Predicted failure	SAFIR failure	Predicted failure
FIRE 23	45	Y	43	Y	-	Y	77	Y	62	Y	80	Y
FIRE 24	-	N	-	N	-	N	-	N	-	N	-	N
FIRE 25	-	N	-	N	-	N	-	N	-	N	-	N
FIRE 26	-	N	-	N	-	N	-	N	139	N	-	N
FIRE 27	-	Y	-	Y	-	N	-	N	-	N	-	N
FIRE 28	59	Y	58	Y	-	N	-	Y	82	Y	-	Y
FIRE 29	59	Y	58	Y	-	N	99	Y	81	Y	-	Y
FIRE 30	44	Y	42	Y	-	N	-	N	-	Y	-	N
FIRE 31	44	Y	42	Y	-	N	76	Y	61	Y	-	Y
FIRE 32	44	Y	42	Y	-	Y	76	Y	61	Y	80	Y
FIRE 33	39	Y	37	Y	-	N	-	N	67	Y	-	N
FIRE 34	39	Y	37	Y	-	N	69	Y	54	Y	72	Y
FIRE 35	39	Y	37	Y	-	Y	69	Y	54	Y	72	Y
FIRE 36	-	N	-	N	-	N	-	N	-	N	-	N
FIRE 37	-	N	-	N	-	N	-	N	-	N	-	N
FIRE 38	178	N	178	N	-	N	-	N	146	N	-	N
FIRE 39	-	Y	-	Y	-	N	-	N	-	N	-	N
FIRE 40	63	Y	60	Y	-	N	-	Y	88	Y	-	N
FIRE 41	63	Y	60	Y	-	N	110	Y	85	Y	-	Y
FIRE 42	45	Y	43	Y	-	N	-	N	-	Y	-	N

Proposed Approach

The proposed approach can be applied to establish equivalency (survival of a column) between a standard and design fire. As a first step, the fire resistance of CFHSS columns under standard fire is evaluated using Equation [1]. To determine the survivability of the column under a design fire exposure, the equivalency can be established by comparing the area under the standard time–temperature curve to that under the design fire time–temperature curve.

Contrary to the traditional equal area concept, the goal of this method is not to determine the failure time of the column in a given design fire. The objective of this approach is to determine if the column will fail in a design fire. To accomplish this, the area under both the standard and the design fire time–temperature curve is determined at the time the column fails in the standard fire. If the area under the standard fire curve is greater than the area under the design fire curve, the column will not fail in the design fire, that is to say the column will survive complete compartment burnout. If however, the area under the standard fire curve is less than that under the design fire curve, the column will fail in the design fire. Most notably, in the latter case, the column will fail in less time in the design fire than in the standard fire.

The applicability of this approach is illustrated in Figure 4 which shows the time–temperature curves corresponding to a standard (ASTM E-119) and a design fire to which a CFHSS column was exposed in simulations.

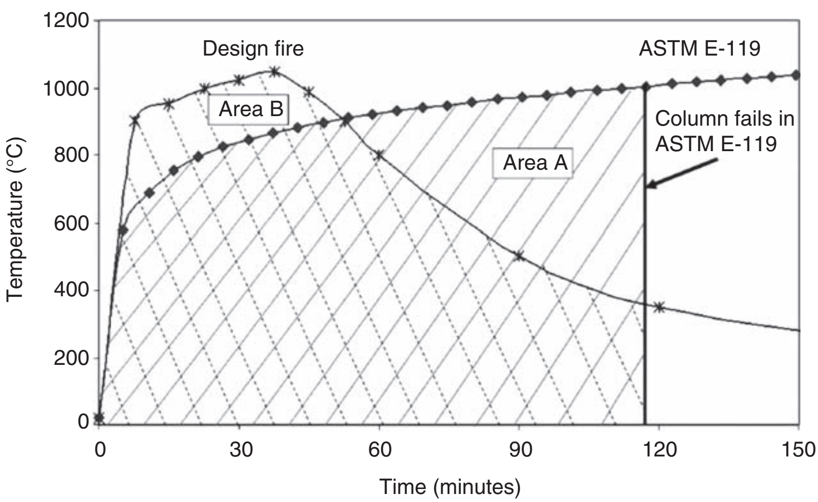

Figure 4. *Illustration of proposed approach for a CFHSS column.*

Based on the room characteristics such as fuel load and ventilation, the design fire can be established (this is elaborated upon later) as shown in Figure 4. Knowing the basic details of the column, the fire resistance of the column under standard fire exposure can be estimated by applying Equation (1). Once both the design fire and the failure time of the column under standard fire exposure are known, the area under the standard fire time–temperature curve (area A in Figure 4) and the area under the design fire time–temperature curve (area B in Figure 4) can be determined at the time of column failure under standard fire exposure. The areas A and B are now compared: if the area under the standard fire curve (area A) is less than the area under the design fire curve (area B) the column is said to have failed in the design fire, and most notably in less time than in the standard fire. If however, the area under the standard fire curve (area A) is greater than that under the design fire curve (area B), the column is said to not have failed in the design fire, that is, the column will survive complete compartment burnout.

Basis for Proposed Approach

The approach proposed here is predicated on the concept that design fires have a similar initial rate of temperature increase and almost all design fires achieve higher initial temperatures than the standard fire exposure, as seen in Figure 1. It should be noted that the proposed method is not applicable for design fires that do not have a similar initial temperature rise as that of the ASTM E-119 fire exposure. Since the failure time of the column under standard (ASTM E-119 or ISO 834) fire exposure is known (using Equation (1)), only the fire severity up to this point needs to be compared with that of the design fire. Given the previous observation that the initial temperatures in most design fires are higher than in the standard fire, the only way for the design fire to be less severe than the standard fire over the same time period is if the design fire enters the cooling phase (in which the column regains strength) prior to the end of the time period (defined as column fire resistance under standard fire exposure). The area under the respective time–temperature relationships serves as a ready means for comparing the relative fire severity. Since only the initial portion of the fires is being compared, the problem of drastically different fire exposures is avoided and an accurate comparison is possible. Hence the conclusion that if the area under the standard fire curve (area A in Figure 4) is greater than the area under design fire curve (area B in Figure 4), the column will not fail in the design fire. On the contrary, if the area under the standard fire curve (area A in Figure 4) is less than that under the design fire curve (area B in Figure 4), the column will fail in the design fire.

Validation of the Proposed Approach

The proposed approach was applied to the 1050 column-fire combinations analyzed using SAFIR to determine if the columns fail or survive complete compartment burnout. In 761 out of the 1050 column-fire combinations analyzed, both SAFIR and the proposed approach indicated that no failure occurs in the column. This high percentage of survivability can be attributed to the fact that the decay phase of the fire allows the column to regain part of its strength, resulting in a fire exposure that is less severe than the 'standard' fire exposure. The agreement in failure predictions between SAFIR and the proposed approach for a large number of columns illustrates the strength of the proposed approach.

Of the 272 remaining column-fire combinations, both SAFIR and the proposed approach predicted failure in 182 of them. The failure of CHFSS column can be attributed to the high severity of fires encountered in these cases. Again, the agreement between the actual (SAFIR) failure and that predicted by the proposed approach indicates the validity of the proposed approach. Further, for another 90 column-fire combinations while failure was not observed in SAFIR, the proposed approach predicted failure. This can be attributed to fires that are similar to ASTM E-119 for the considered time period entering the decay phase possibly only minutes before the column failed under ASTM E-119 fire exposure. This can however be taken as conservative for design purposes. For the remaining 17 column-fire combinations, SAFIR predicted that the columns would fail while the proposed approach did not. This can be attributed to fires with a low ventilation coefficient and a comparatively large fuel load. The resulting long-duration cool fires impart more heat to the columns than short-duration fires that are hotter causing them to fail much later in the simulation.

To illustrate the effectiveness of the proposed approach, a statistical representation of the 1050 numerical simulations is presented in Figure 5. In 72.5% (761) of the column-fire combinations, both the SAFIR and proposed approach indicated that the column would not fail. In 17.3% (182) of the cases, both SAFIR and the proposed approach indicate that the column-fire combination will result in failure of the column. SAFIR predicts that failure will not occur while the proposed approach indicated that failure will occur in 8.6% (90) of the column-fire combinations, thus making it a conservative prediction. In the remaining 1.6% (17) of the column-fire combinations, SAFIR indicates the column will fail while the proposed method indicated that the column will not. This low (1.6%) probability of an unconservative estimation of column failure for the selected range of fire exposures serves to further reinforce the strength of the proposed method.

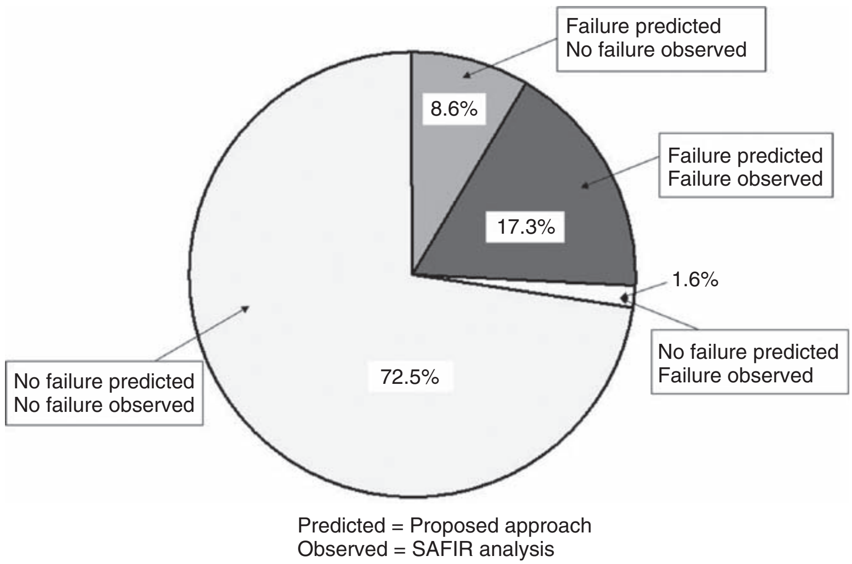

Figure 5. *Graphical illustration of the effectiveness of the proposed approach.*

A subset of the data is presented in Table 2 to illustrate the results from analysis. For each of the column-fire combinations, Table 2 presents the failure time as determined by SAFIR where applicable, and whether or not the column is determined to fail using the proposed method. Boxes occupied by a "-" in the 'SAFIR failure' column indicate that no failure was observed in SAFIR analysis under the design fire exposure. Of the 252 column-fire combinations in Table 2, both SAFIR and the proposed approach indicate that the column will not fail in 131 of them. Of the 121 remaining column-fire combinations, both SAFIR and the proposed approach predicted failure in 86 of them. For another 31 column-fire combinations, while failure was not observed in SAFIR, the proposed approach predicted failure. In the remaining four column-fire combinations, SAFIR predicted that the columns would fail while the proposed approach did not. Reasons for the observed results are the same for this subset as for the entire data set, as such; the reasons given above will not be reiterated here, though this data subset serves to further reinforce the validity of the proposed approach.

As an example from Table 2, SAFIR analysis of CFHSS column RP 168 under standard fire exposure yielded a failure time of 82 minutes, while no failure was observed for 'FIRE 1' through 'FIRE 5' with failure occurring in 'FIRE 6'. The proposed approach also predicts no failure for 'FIRE 1' through 'FIRE 4', predicting failure for 'FIRE 5' and 'FIRE 6' though failure is not observed in 'FIRE 5'.

It should be reiterated that this method does not determine the fire resistance of CFHSS columns in design fires in a quantified manner; rather, it predicts whether or not the column will survive the design fire exposure. If it is determined that the column does not survive the fire exposure, that is all the information available and the duration of survival is not known from this method. As such, there is a built-in factor of safety in that all columns designed according to this method will have sufficient fire resistance to withstand complete burnout of the probabilistic fires to which the column would be exposed, rather than satisfy specific fire resistance requirements. It might be possible to achieve more deterministic results using more sophisticated methods, but that would increase the complexity of the calculations and make the approach less practical for use in design.

Limitations of Proposed Approach

The proposed method offers a practical approach for evaluating the fire resistance of CFHSS columns exposed to design (real) fires. However, the applicability of this method is limited to the range of parameters that were considered in the numerical study which formed the basis for developing this approach. The explicit limitations for the proposed methods are as follows:

- The initial temperature rise (in the first 5–10 minutes) in the design fire must be similar to the ASTM E-119 temperature rise.
- The maximum fire temperature reached must be greater than that of ASTM E-119 during the first 20 minutes. Fire FV04-200 shown in Figure 1 can be taken as a lower bound for the applicable design fires.
- This method is not applicable for the ASTM E-1529 hydrocarbon fire exposure due to the absence of a decay phase.
- The approach is only applicable for unprotected CFHSS columns.
- Since the approach makes use of Equation (1), all of the limitations that apply to Equation (1) (length, concrete compressive strength, end conditions, and cross-section) also apply to this method.
- This method is applicable for bar, steel fiber, and plain, concrete-filled HSS columns.

It should be noted that the above limitations are specified due to a lack of validation for these conditions. Through further validation, it should be possible to extend this method to overcome some of these limitations.

PERFORMANCE–BASED DESIGN

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 such an approach [11]. One of the key aspects in any performance-based fire safety design is the fire-resistant design of structural members. Currently, to evaluate the fire resistance of structural members, numerical models that can simulate the response of CFHSS columns under realistic fire, loading, and restraint scenarios must be used. The main steps needed to undertake a rational approach for performance-based design of CFHSS columns are:

(a) identifying proper design (realistic) fire scenarios and realistic loading levels on HSS columns under consideration;
(b) carrying out detailed thermal and structural analysis by exposing the CFHSS column to specified fire conditions; and
(c) developing relevant practical solutions, such as the use of different types of concrete-filling to achieve the required fire resistance.

Undertaking detailed thermal and structural analysis for every column and fire combination, through providing accurate and reliable results requires considerable time, effort, and expertise. In lieu of detailed analysis, the equivalent area concept presented in this article can be used to evaluate the failure or survivability of a CFHSS column under design fire exposure. Available test data (from literature) or Equation (1) could be utilized to determine the fire resistance of CFHSS columns under standard fire exposure which in turn can be used to determine if the column will fail in any of the probabilistic design fires. The steps required to undertake such a performance-based design using the proposed method are detailed below.

Development of Fire Scenario and Loading

The design fire scenarios for any given situation should be established either through the use of parametric fires (time–temperature curves) specified in Eurocode [17] or through design tables [15] based on ventilation, fuel load, and surface lining characteristics. The ventilation factor (F_v) can 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) [40]. Next, the fuel load in MJ should be determined per m^2 of the total bounding surface (not just the floor area). Typical fuel loads for common compartment uses are readily available, [12,40] and can be used for this relatively simplistic calculation. Figure 1 shows typical standard and real fire exposure curves that can be generated for performance-based fire safety design. The presence of sprinklers can be accounted for in the development of the fire scenarios to be modeled.

Determination of Fire Resistance

The second step in the proposed approach involves evaluating fire resistance of the CFHSS columns under standard fire exposure. Knowing the basic features of the column, Equation (1) can be applied to determine the fire resistance of the column within the limits of applicability for the equation. Otherwise, testing data available from a variety of sources can be consulted [24–26] to determine the fire resistance of the CFHSS column if it were to be exposed to standard (ASTM E-119 or ISO 834) fire. Once having obtained this failure time, the area under the standard time temperature curve at that time should be calculated. Likewise, the area under the design fire time–temperature curves, determined in Step 1, should be calculated at the time of failure in the standard fire exposure. The area under the design fire curve should then be compared to the area under the standard curve; if the calculated area under the design fire is less than that calculated under the standard fire, the column will survive compartment burnout. If the area calculated under the design fire curve is more than that calculated under the standard curve, the column will not survive the design fire, and practical alternatives can be applied to enhance the fire resistance of the column.

Development of Practical Alternatives

The most feasible method to achieve higher fire resistance is through changing the type of concrete-filling. Other factors, such as the type of aggregate in the concrete, reinforcement in the column, or load level can be varied to achieve higher fire resistance in HSS columns. As an example, while plain concrete filling can provide 2 hours fire resistance in HSS columns, by switching to steel fiber-reinforced concrete filling, up to 3 hours of fire resistance can be obtained under standard fire exposure. Having obtained longer fire resistance under standard fire exposure (determined through the use of Equation (1) or from available test data) by altering column parameters, the area under the fire curves should again be calculated and compared as above to determine if the column can withstand complete compartment burnout. This iterative process should be continued until the column has sufficient fire resistance to withstand complete compartment burnout under a specified design fire.

Design Example

Application of the proposed equivalent area approach for a design situation is illustrated through a numerical example. Detailed calculations for this example are presented in the Appendix. A CFHSS column located in a room 3 m high with a 6 m by 4 m floor that has a fuel load of 550 MJ/m^2 (of floor area) is to be designed. The room has one window opening 3 m wide by 2 m high. In this compartment, the architect has proposed a 273 mm diameter circular HSS column filled with plain concrete made with siliceous aggregate. The height of the column needs to be 3.81 m (to accommodate the drop ceiling and utilities) with fixed connections on both ends. The building code requires the column to have a 2 hour fire resistance rating. Now it is desired to know if this column will satisfy the code requirements by withstanding complete burnout of the fire that would occur within the compartment.

The first step is to determine the design fire that is most likely to occur within the compartment. The fire scenario can be determined using the Eurocode [17] method. The detailed calculations associated with this example are presented in the Appendix, along with further details on the compartment characteristics. The ventilation factor for this example is 0.079 $m^{1/2}$ and the burning duration is 0.79 hour. The time–temperature curve determined using this method and the standard (ASTM E-119) time–temperature curve are shown in Figure 6.

The second step is to determine the fire resistance of the column under standard fire exposure (ASTM E-119 or ISO 834). Applying Equation (1), the fire resistance of the column is determined to be 101 minutes. The area under the standard time–temperature curve at 101 minutes is 1462 minutes $\times$ °C. The corresponding area under the design fire curve at 101 minutes

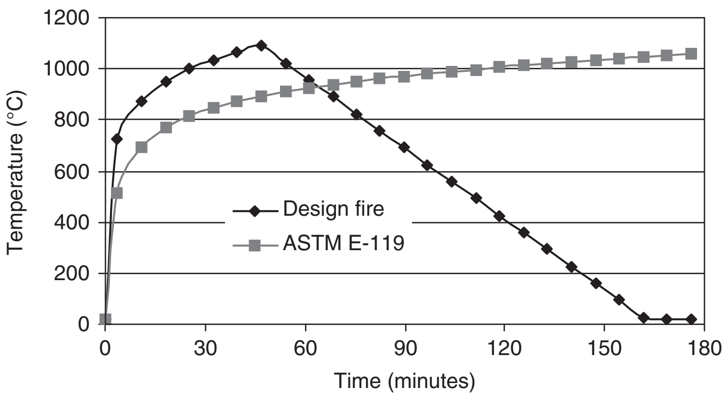

Figure 6. *Comparison of standard and design fire exposure.*

is 1503 minutes $\times$ °C. Since the area under the design fire time–temperature curve is greater than that under the standard time–temperature curve, the column will fail in less time in the design fire than when exposed to standard fire; as such, practical alternatives need to be developed.

The most practical alternative is to simply change the type of aggregate used in the concrete. By switching to carbonate aggregate, according to Equation (1) the fire resistance of the column increases from 101 to 116 minutes under standard fire exposure. At 116 minutes, the area under the standard (ASTM E-119) time–temperature curve is 1701 minutes $\times$ °C, while that under the design fire time temperature curve is 1626 minutes $\times$ °C. As such, since the area under the design fire curve is less than that under the standard fire curve, the column will survive complete compartment burnout; thus, the column is safe for use in this building because it has the highest fire resistance rating possible, surviving complete compartment burnout when carbonate aggregate is used in the concrete. Thus, the proposed approach facilitates the use of CFHSS columns in buildings where traditional prescriptive design would not use these columns.

DESIGN IMPLICATIONS

The approach presented here is capable of determining whether a CFHSS column will fail in a given fire exposure based on the fire resistance demonstrated by that column when exposed to a standard fire. The proposed approach, which was built on the large amount of data available for CFHSS columns exposed to fire, can be used to overcome many of the current limitations in achieving unprotected HSS columns in most practical applications. Thus, the use of this approach will lead to designs that are not only economical, but are based on rational design principles.

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 imperative, 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 design of CFHSS columns under performance-based codes.
- Type of fire exposure has a significant influence on the fire resistance of CFHSS columns. Unprotected CFHSS columns have fire resistance sufficient to withstand complete compartment burnout under most design fire scenarios.
- An approach based on the equivalent area concept is proposed for evaluating the survivability of a CFHSS column exposed to design fires.
- The proposed approach is capable of predicting survival of CFHSS columns under design fire exposure with a high degree of accuracy.
- The proposed approach is quite simple and easy to apply, thus facilitating its use in lieu of detailed finite element analysis which requires the use of costly and time-consuming computer models.

ACKNOWLEDGEMENT

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APPENDIX

DESIGN EXAMPLE

Room properties

Room dimensions: 6 m wide by 4 m deep by 3 m high
Ventilation: one window, 3 m wide by 2 m high
Fire load: 550 MJ/m^2
Lining material: concrete with the following properties
Thermal conductivity: $k = 1.6$ W/mK
Density: $\rho = 2300$ kg/m^3
Specific heat $c_p = 980$ J/kg K

Column properties

Length: 3.81 m
Effective length factor: 0.65 [7]
Concrete compressive strength: 27.4 MPa
Diameter: 273.1 mm
Load: 750 KN
$F = 0.07$ (siliceous aggregate) 0.08 (carbonate aggregate) see ref [32–34]

Calculation of design fire as per Eurocode

Thermal inertia of concrete:

$$b = \sqrt{kpc_p} = \sqrt{1.6 * 2300 - 980} = \frac{1900\,Ws^{0.5}}{\mathrm{m^2K}}$$

Floor area:

$$A_f = 6 * 4 = 24\,\mathrm{m}^2$$

Area of internal surface:

$$A_t = 6 * 4 * 2 + 3 * 6 * 2 + 3 * 4 * 2 = 108\,\mathrm{m}^2$$

Ventilation factor:

$$F_v = \frac{A_v\sqrt{H_v}}{A_t} = \frac{(3 * 2)\sqrt{2}}{108} = 0.079\,\mathrm{m}^{0.5}$$

Fuel load energy density:

$$e_f = \frac{550\,\mathrm{MJ}}{\mathrm{m}^2}$$

Total fuel load:

$$E = e_f A_f = 550 * 24 = 13{,}200\,\mathrm{MJ}$$

Duration of fire:

$$t_d = \frac{0.00013E}{\left(A_v\sqrt{H_v}\right)} = \frac{0.00013 * 13200}{\left(6 * \sqrt{2}\right)} = 0.202\,\text{hours}$$

Imaginary time used in Eurocode time–temperature relationship:

$$t^* = \frac{t_d(F_v/0.04)^2}{(b/1900)^2} = \frac{0.202(0.079/0.04)^2}{(1900/1900)^2} = 0.789\,\text{hours}$$

Time–temperature relationship:

$$T = 1325(1 - 0324e^{-0.2t^*} - 0.204e^{-1.7t^*} - 0.472e^{-19t^*})$$

Fire decay rate:

Interpolate between a decay rate of 625°C/hr for fire lasting ½ hour or less and 250°C for fires lasting more than 2 hours = 553°C/hour after 0.789 hour of combustion, see Figure 6 for graphical representation of time–temperature relationship for the design and standard fire

CALCULATION OF ASTM FIRE RESISTANCE BASED ON EQUATION (1)

1st Iteration:

The fire resistance (R) of a CFHSS column under standard fire exposure can be computed using Equation (1)

$$\begin{aligned} R &= f\frac{(f'_c + 20)}{(KL - 1000)}D^2\sqrt{\frac{D}{C}} \\ &= 0.07\frac{(27.4 + 20)}{(3810 * .65 - 1000)}(273.1)^2\sqrt{\frac{273.1}{750}} = 101 \text{ minutes} \end{aligned}$$

Area under ASTM E-119 time-temperature curve at 101 minutes: 1462 minutes°C

Area under the design fire time-temperature curve at 101 minutes: 1503 minutes°C

Since the area under the standard fire curve is less than that under the design fire the column will fail in the design fire. An alternative must be selected since this does not satisfy the 2 hour fire rating required for the column.

2nd Iteration:

By changing the aggregate type used in the concrete from siliceous to carbonate, the fire resistance of the column can be enhanced.

$$\begin{aligned} R &= f\frac{(f'_c + 20)}{(KL - 1000)}D^2\sqrt{\frac{D}{C}} \\ &= 0.08\frac{(27.4 + 20)}{(3810 * .65 - 1000)}(273.1)^2\sqrt{\frac{273.1}{750}} = 116 \text{ minutes} \end{aligned}$$

Area under ASTM E-119 time–temperature curve at 116 minutes: 1701 minutes°C

Area under the design fire time–temperature curve at 116 minutes: 1626 minutes°C

Since the area under the standard fire curve is greater than that under the design fire the column will not fail in the design fire; thus the CFHSS column can be used in this application.

A detailed finite element analysis was carried out on this CFHSS column using the computer program SAFIR. Results from the analysis indicate that the column does not fail under the design fire but rather survives compartment burnout when carbonate aggregates are used. This is in agreement with the predictions from the proposed equivalent area approach that requires significantly less time and effort.

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