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Alexandra Thewis

Stephen Pessiki

Michael McGinnis

ATLSS Report No. 07-13

May 2007

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ABSTRACT

One way to increase the performance of a steel column in a fire may be to simply increase the actual weight of the column. A small increase in column weight may provide enhanced strength of that column in the event of a fire, without increasing construction costs significantly. This research probes the supposition that small increases in column weight (which do not necessarily lead to large increases in associated construction costs) can lead to substantial increases in fire duration of the given column. Nonlinear heat transfer analyses were performed on SFRM insulated steel columns exposed to a standard E119 fire time-temperature curve for varying amounts of time. Subsequent structural analyses, accounting for the degradation of steel material properties at the resulting elevated temperatures, yielded the ultimate strengths of the columns. The research investigated three series of columns, a Light series (W360x134 to W360x162), a Medium series (W360x347 to W360x421), and a Heavy series (W360x744 to W360x900), representing columns that may be used at the upper, middle and lower stories of a high rise building, respectively. The behavior of these column series was determined at fire durations of 1, 2, 3 and 4 hours. Because an increase in column size results in a new design SFRM thickness for a given fire duration rating, increased column sizes were investigated with original (i.e. thicker) SFRM thickness, as well as with the reduced SFRM thicknesses required for the larger column size.

The major findings of the research were that: (1) for all columns considered, increasing the column weight 10% and providing design SFRM thickness on the new column did provide some benefit (approximately 5-20% increase in capacity depending on the column and fire duration considered); (2) the benefit of a 10% increase in weight was greatest for the Light and Medium column groups and least for the Heavy column group; (3) Increasing the column weight 10% and maintaining SFRM thickness from the base column on the new column provided a substantial benefit in all cases (approximately 10-30% increase in capacity depending on the column and fire duration considered); (4) Increasing the column weight 20% provided substantial benefit for all columns considered, regardless of SFRM thickness or fire duration (approximately 25-40% increase in capacity); and (5) the columns with a 20% weight increase with SFRM thickness maintained from the original (lighter) column outperformed those with design SFRM significantly due to the increased insulation provided.

CHAPTER 1

INTRODUCTION

1.1 Introduction

Columns play a key role in the overall stability of building frames. During a fire, the axial load capacity of a steel column may be significantly diminished. This is due to changes of the material properties of steel at the elevated temperatures present during a fire. The temperature of a steel column rises during the duration of a fire which in turn causes the loss of steel strength and stiffness. This can ultimately lead to failure of a column which in turn can lead to failure of a building frame.

Steel columns are often protected from the full effects of the fire by various forms of insulation. One common insulation system, sprayed fire resistive material (SFRM), involves coating steel columns with a cementitious material that acts as a thermal insulator. The thickness of SFRM insulation is determined by specifying a given fire rating, and depends upon the column weight and perimeter exposed to fire. As the weight (per length) of a steel column rises, the amount of SFRM required to meet a given fire duration rating decreases.

One potential way to increase the performance of a steel column in a fire is to simply increase the actual weight of the column. For example, if a column size is determined, upon considering all other design constraints and loading conditions, to be a W360x347, a W360x382 could be used instead to potentially provide additional capacity in the event of a fire. A small increase in column weight may provide enhanced strength of that column in the event of a fire, without increasing construction costs significantly.

Factors that impact the cost of steel construction include steel weight, fabrication, transportation, and erection. The fabrication, transportation and erection costs for a steel column will most likely not increase if the column weight is increased by a small amount. Increasing the weight of a steel column is therefore not directly related to the cost. This report describes research performed to evaluate the strength gained in fires by increasing the area of steel columns. The results should provide insight into an approach that may be taken by designers to increase the fire safety of steel buildings without an excessive escalation in cost.

1.2 Statement of Research Objectives

This research probes the supposition that small increases in column weight (which do not necessarily lead to large increases in associated construction costs) can lead to substantial increases in fire duration of the given column.

1.4 Summary of Approach

Nonlinear heat transfer analyses were performed on SFRM insulated steel columns exposed to a standard E119 fire time-temperature curve for varying amounts of time. Subsequent structural analyses, accounting for the degradation of steel material properties at the resulting elevated temperatures, yielded the ultimate strengths of the columns. The research investigated three series of columns, a Light series (W360x134 to W360x162), a Medium series (W360x347 to W360x421), and a Heavy series (W360x744 to W360x900), representing columns that may be used at the upper middle and lower stories of a high rise building, respectively. The behavior of these column series was determined at fire durations of 1, 2, 3 and 4 hours. Because an increase in column size results in a new design SFRM thickness for a given fire duration rating, increased column sizes were investigated with original (i.e. thicker) SFRM thickness, as well as with the thinner SFRM thicknesses required for the larger column size. This research calculated the ultimate load of stub columns, and therefore buckling behavior of columns in fire was not assessed.

1.4 Notation

Notation used in this report is as follows:

R = Fire resistance in hours
H = Thickness of SFRM
D = Heated perimeter of steel column in inches
W = Weight of steel column in lb/linear ft.
P_u = Ultimate load capacity of column
ΔFD = Change in fire duration (see Figure 7.4)

1.5 Unit Conversions

Table 1.1 summarizes the metric notation for the columns treated in this research and their standard imperial (U.S.) equivalents. In the metric notation, a W360x134 designation corresponds to a column that is approximately 360 mm deep (and weighs approximately 134 newtons per meter. In standard imperial units, a W14x90 designation corresponds to a column that is approximately 14 inches deep and weighs approximately 90 pounds per foot.

Table 1.1: Metric to imperial column designation conversion information

Metric Designation	Imperial (U.S.) Designation
W360x134	W14x90
W360x147	W14x99
W360x162	W14x109
W360x347	W14x233
W360x382	W14x257
W360x421	W14x283
W360x744	W14x500
W360x818	W14x550
W360x900	W14x605

CHAPTER 2

PROTOTYPE COLUMNS

2.1 Introduction

In order to ensure the relevance of the research to issues facing current practice, columns were chosen for analysis from an industry designed office building. The prototype office building is located in a major city on the east coast of the U.S. and is approximately 50 stories. The complete column schedule and a typical floor plan for the prototype building were made available by the structural Engineer of Record. All of the columns in the building consist of A572 (ASTM 2007a) Grade 50 steel, are in the W360 family, and have SFRM applied to achieve a 3 hour fire duration rating. A typical column extending the entire height of the building was chosen to obtain column sizes at various floor heights. Example column sizes from the upper, middle, and lower floors were chosen in order to obtain a wide range of W360 column sizes. An example of the loads and locations of some of the columns in the building as determined by the Engineer of Record are given in Table 2.1.

The P/P_u values listed in Table 2.1 are a measure of the degree to which a column is loaded. The columns being examined are part of the external frame, which is designed to sustain large lateral loads due to loading scenarios such as wind and earthquake. When only gravity loads are taken into account, external columns appear to be somewhat over-designed, which explains the low P/Pu values. In tall buildings, internal gravity columns are loaded to a much higher degree than external columns, with P/Pu values approximately equal to 0.6.

2.2 Column Selection

In order to determine the specific column sizes used, a study of various column sizes was performed by comparing the relative weight of a W360 column with the next lightest column. Figure 2.1 shows the relative weight increase of each column in the W360 family in relation to the next lowest weight column. A series of three columns in the light, medium, and heavy range were chosen for analysis. Each series column was selected so that an approximately equal percentage weight increase from one column to the next within a series was obtained. In order to obtain a relatively constant percentage weight increase, column sizes at the beginning of a plateau were chosen. Horizontal lines marking the plateaus are shown in Figure 2.1. The base columns, the column sizes from which each successive column size increase would be made, were chosen to be the W360x134, W360x347, and W360x744. As shown in the figure, the columns are referred to in groups as the Light (base column W360x134), Medium (base column W360x347) and Heavy (base column W360x744) columns.

Table 2.1: Column sizes, locations and applied loads

	Floor	Column Size	Service Dead Load (kN)	Service Live Load (kN)	Total Load, P (kN)	P_u (kN)	P/P_u
Upper	46	W360x134	1348	587	1935	5894	0.328
	45	W360x134	1521	649	2171	5894	0.368
Middle	30	W360x347	4426	2344	6770	15235	0.444
	27	W360x347	5044	2642	7686	15235	0.505
Lower	4	W360x500	10355	5200	15555	32694	0.476
	3	W360x500	11103	5631	16734	32694	0.512

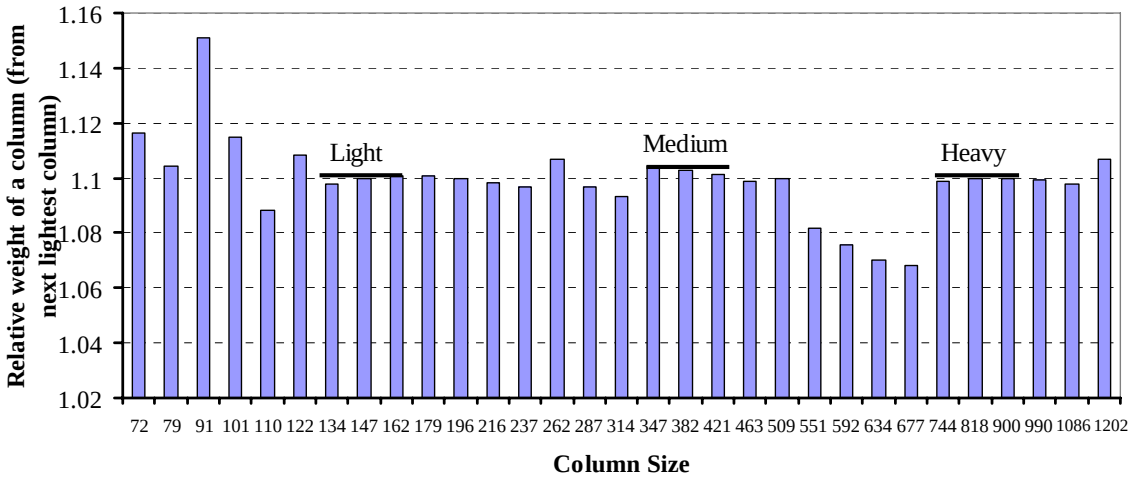


Figure 2.1: Relative weight increase of all columns in W360 family

CHAPTER 3

ANALYSIS CASES

3.1 Introduction

To investigate the effect of increasing a column's size on axial strength gain in the event of a fire, three series of three columns each were chosen. From the base columns described in Section 2.3, the column size was increased twice while maintaining a constant SFRM thickness. Analysis cases were then developed on the effect of reducing the SFRM thickness as the column size was increased.

3.2 SFRM Thickness

Sprayed fire resistive material (SFRM) was used to insulate the columns. The insulation thicknesses for all columns were as specified from a design table provided by the manufacturer (Isolatek International 2006) for a 3-hour fire rating. The equation used to populate this table is given in Equation (3-1).

$$h = \frac{R}{1.01(W/D) + 0.66} \qquad \text{Equation (3-1)}$$

where R is the fire resistance (in hours), h is the thickness of SFRM (in inches), D is the heated perimeter of steel column (in inches), and W is the weight of steel column (in lb/linear ft) (Underwriters Laboratories 2006). After an SFRM thickness has been calculated in inches, it may be converted into millimeters by multiplying by 25.4 (mm/inch). The required SFRM thickness (in millimeters) for each column examined in this study is shown in Table 3.1. As column sizes increase the design SFRM thickness decreases, as depicted in Table 3.1.

It is understood that in practice it is impossible to specify SFRM thicknesses to the nearest 0.1 mm. However in this research, the exact thickness was used (to the nearest 0.1 mm), so that effects of changing SFRM thicknesses were not masked by round-off concerns.

3.2 Column Designations

To illustrate the designation of the various analysis cases, a description of the cases for the Light (base column W360x134) columns is provided. Analysis case designations for the Medium and Heavy column groups are similar. Three analysis cases were created by maintaining a constant SFRM as the column size was increased. Starting with the base column W360x134 and applying its calculated SFRM thickness (42.9 mm), the first analysis is designated as 134 (SFRM134). In the next analysis, the column size is

increased to the next available size of W360x147, and the SFRM thickness for the W360x134 is maintained. Thus the second analysis case is designated as 147 (SFRM 134). For the third analysis, the column size is increased once more to W360x162 and the SFRM thickness for the W360x134 is again maintained. Thus the third analysis is designated as 162 (SFRM 134).

Two additional analysis cases were developed by reducing the SFRM thickness as the column size was increased. In the first, the calculated SFRM thickness for the W360x147 (41.3 mm) is applied to a W360x147 column and designated as 147 (SFRM 147). In the second, the analysis for the W360x162 was performed with the SFRM thickness for a W360x162 (39.7 mm), and designated as 162(SFRM 162). All of the analysis cases and their respective designations are presented in Table 3.2.

Table 3.1 Calculated insulation thickness for 3 hour rating

Column Designation	Insulation Thickness (mm)
W360x134	42.9
W360x147	41.3
W360x162	39.7
W360x347	23.8
W360x382	22.2
W360x421	20.6
W360x744	14.3
W360x818	12.7
W360x900	12.7

Table 3.2 Analyses matrix

Column Designation	Insulation Thickness (mm)	Analysis Case Designation
W360x134	42.9	134(SFRM134)
W360x147	42.9	147(SFRM134)
W360x162	42.9	162(SFRM134)
W360x147	41.3	147(SFRM147)
W360x162	39.7	162(SFRM162)
W360x347	23.8	347(SFRM347)
W360x382	23.8	382(SFRM347)
W360x421	23.8	421(SFRM347)
W360x382	22.2	382(SFRM382)
W360x421	20.6	421(SFRM421)
W360x744	14.3	744(SFRM744)
W360x818	14.3	818(SFRM744)
W360x900	14.3	900(SFRM744)
W360x818	12.7	818(SFRM818)
W360x900	12.7	900(SFRM900)

CHAPTER 4

ANALYTICAL APPROACH

4.1 Introduction

A finite element model was created for each of the analysis cases outlined in Table 3.2. A heat transfer analysis and subsequent structural analysis were performed on each model. The heat transfer analysis was conducted to determine the temperatures within the column when exposed to fire. These temperatures were then applied to the structural analysis model in order to examine the structural behavior of the column at elevated temperatures. Both the heat transfer and structural analyses account for the fact that many of the material properties of SFRM and steel are highly temperature dependent. All heat transfer and structural analysis were performed using ABAQUS, a commercially available nonlinear finite element analysis program.

4.2 Material Properties

4.2.1 Steel

As specified in the column schedule for the prototype building, all columns consist of A572 Grade 50 (345 MPa) steel. The density of structural steel has a standard value of 7850 kg/m^3, which is constant with respect to temperature. The thermal properties of steel include the thermal conductivity, specific heat, and coefficient of thermal expansion. Eurocode 3 (BSI 2001), hereafter referred to as EC3, provides relationships for these properties with respect to temperature. These relationships are shown in Figures 4.1 through 4.3. The thermal conductivity, shown in Figure 4.1, decreases as the temperature of the steel increases and remains constant after 800˚C. The specific heat relationship given in EC3 was altered slightly for this research in order to avoid numerical stability problems with the finite element model. This modification was exactly as performed in Lee et al. (2006). Figure 4.2 displays the modified specific heat curve where it can be seen that the specific heat remains roughly the same with increasing temperature, except for an upward spike at approximately 730˚C due to the presence of a phase change in the steel. Figure 4.3 illustrates how the coefficient of thermal expansion remains roughly constant as the temperature increases.

The mechanical properties of steel include the stress-strain relationship at elevated temperatures, and Poisson's ratio. An idealized stress-strain relationship for Grade 50 steel at elevated temperatures can be found in EC3, and is shown in Figure 4.4. From Figure 4.4 it is apparent that at temperatures below 400°C, as the temperature increases the yield strength of the steel will decrease, but the initial modulus and the ultimate strength remain unchanged. At temperatures above 400°C, the initial modulus, the yield

strength and the ultimate strength of the steel all decrease with increasing temperature. The Poisson's ratio is assumed not to vary with temperature and a value of 0.3 is used.

4.2.2 Sprayed Fire Resistive Material (SFRM)

The sprayed fire resistive material simulated in this study is a Portland cement based material designed to insulate structural steel and concrete. The density of this product is 240 kg/m^3. The thermal conductivity and specific heat with respect to temperature of the product has been determined by other researchers (NIST 2004). Figures 4.5 and 4.6 show plots of these quantities versus temperature. The mechanical properties of SFRM such as strength and stiffness are significantly less than those of steel. Consequently the structural effects of SFRM are negligible, and the SFRM was removed for the structural analysis.

4.3 Finite Element Model

The finite element model for each column did not represent the actual height of the column as given in the prototype building column schedule. When the actual column lengths are considered, global and local buckling may govern failure. For this research, global buckling, local buckling and other length effects (i.e. P-Δ effects) were not considered and thus only the axial strength of the cross-section was considered. Ultimate loads in the structural analysis were calculated considering only reductions in stiffness and strength due to temperature effects. Thus only stub column behavior was captured. This allows comparisons to be made between columns based on simplified behavior. Future work should consider these length issues. Figure 4.7 depicts a column schematic used to determine the height of each column model. The height of each model varied slightly based on the thickness (x) noted in the figure, which changes for each analysis case.

The element types and boundary conditions required for each analysis type (heat transfer or structural) are different, and are discussed in Sections 4.4 and 4.5 respectively.

Due to the considerable difference in steel and SFRM thickness between the Light, Medium and Heavy column groups, a varied number of elements through the thickness were used in each model. In order to obtain the most accurate results the mesh was refined depending on the thickness of the steel or SFRM based on convergence studies by Lee et al. (2006). Figure 4.8a is a section view of an example of a Light column which requires a substantial amount of insulation. In the figure the dark grey elements represent the steel and the light grey elements represent the SFRM. Due to the thickness of SFRM four elements were used through its thickness and only two through the thickness of the steel. Figure 4.8b is a section view of an example of a Medium column with three elements through the steel and four though the thickness of the SFRM. Figure 4.8c is a section view of a Heavy column which requires only a thin amount of SFRM. Two elements were used though the SFRM, six through the steel web, and seven through the steel flanges. In the axial direction fifteen elements were used for each analysis case.

The only difference in the mesh between the heat transfer and structural analysis is that the FEM employed in the structural analysis consisted of bare steel with no SFRM. As noted the axial resistance provided by the SFRM is insignificant compared to that of the steel and can be ignored. An example structural analysis FEM is shown in Figure 4.9.

The symmetric nature of the column and SFRM could allow for a 2-D heat transfer analysis. However, a 3-D model was created in order to more easily apply the nodal temperature results obtained from the heat transfer analysis to the 3-D structural analysis.

4.4 FEM Heat Transfer Analysis

The nonlinear heat transfer analysis performed considered the heat transfer mechanisms of conduction, convection and radiation. The surfaces of the SFRM exposed to fire were given boundary conditions to specify convection and radiation. The boundary conditions in the current study were applied in the same manner as those in Lee et al. (2006). A brief description of the boundary conditions that govern these two phenomena follows.

Convection occurs when heat is transferred by the motion of a fluid, and may arise due to temperature differences within the fluid, or between the fluid and its boundaries. In this case convection will result in heat transfer between the structural element and the surrounding gasses that have been heated within a fire. Convection is governed by a standard equation which is governed by a convection heat transfer coefficient and by the difference in temperature between a surface and the surrounding fluid. The convection heat transfer coefficient for the current work was taken as 6.5 W/m^2.

Radiation is defined as the transfer of heat through electromagnetic waves in the heat spectrum. In this case standard greybody terminology has been used and thus the resultant emissivity between the structural components and the surrounding heated air was defined. A value of 0.7 was used as the effective emisivity between the fire gasses and the exposed SFRM surface, consistent with the work of Kwon and Pessiki (2006) and with the recommendations found in Wang (2002).

The heat transfer mesh element type specified in the heat transfer analysis accounts for the conduction of heat through the SFRM and steel. Three-dimensional eight-node linear heat transfer elements (ABAQUS DC3D8) were assigned to the steel and SFRM.

Figure 4.10 displays the ASTM E119 (ASTM 2007) temperature vs. time curve used to model the thermal loading on the column. The ASTM E119 curve is for design purposes and is not intended to be representative of an actual fire. An initial temperature of 20°C was used before the thermal loading was applied.

Figure 4.11 shows an example of the 347 (SFRM 347) heat transfer FEM. Nonlinear heat transfer analyses were performed for each column considered for fire durations of 1, 2 , 3 and 4 hours.

4.5 FEM Structural Analysis

The structural analysis consisted of two major modules. In the first module, the nodal temperatures from the heat transfer analysis are applied to the structural model as an initial thermal loading. The application of these temperatures caused strains within the member in relation to the amount of heating. The temperatures were applied in increments so that if the temperature profile within the steel caused strains in excess of the yield strain, this nonlinear behavior would be captured. The second module was

performed subsequent to the first and consisted of the application of the structural loads representing compression within the column.

To perform the nonlinear structural analysis three-dimensional eight-node continuum elements (ABAQUS C3D8I) were assigned to the steel column. The structural analysis was performed by imposing a prescribed displacement on the plane of nodes on one end the column until the ultimate load was reached. As the displacement was applied the reaction at the other end of the column was used to determine the applied load. Due to the symmetry of the column and SFRM, and the elimination of buckling due to the short column height, geometrical nonlinearity was not considered.

Figure 4.12 shows the boundary conditions imposed on the column, which simulate symmetry about two axes. A vertical plane of nodes down the center of the cross section was constrained in the 1 direction and a horizontal plane of nodes through the center of the web was constrained in the 2 direction (to comply with the features noted in Figure 4.12a). The nodes of the bottom face were constrained in the 3 direction only (to comply with Figure 4.12b). These boundary conditions allowed the steel to deform symmetrically as the displacement was imposed.

Ultimate loads in this report are reported as the maximum load reached for a given analysis. In each analysis case (see for example Figures 6.2-6.16), the axial displacements were imposed on the column until a clear maximum in load was reached and the cross section had fully plastified.

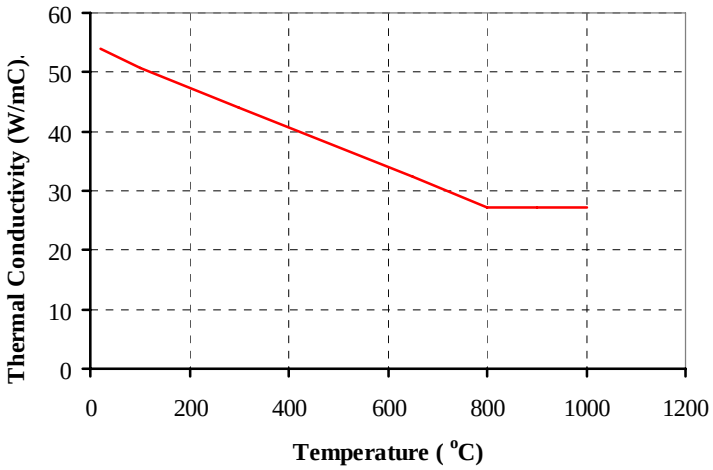


Figure 4.1: Temperature vs. thermal conductivity for steel as defined by EC3 (BSI 2001)

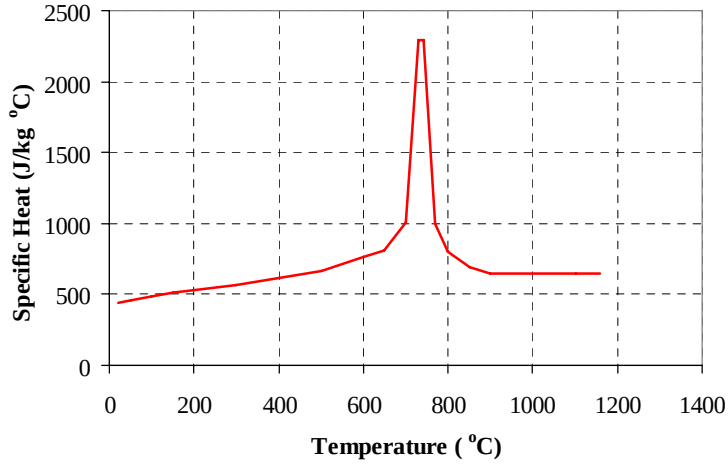


Figure 4.2: Temperature vs. specific heat for steel as adapted from EC3 (BSI 2001)

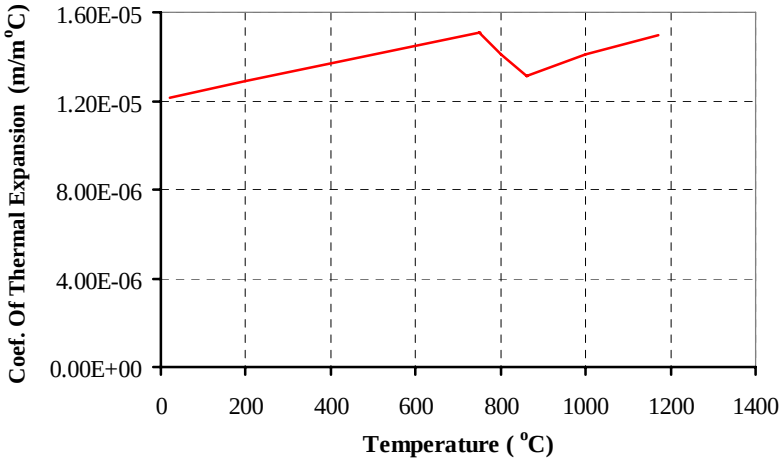


Figure 4.3: Temperature vs. coefficient of thermal expansion for steel as defined by EC3 (BSI 2001)

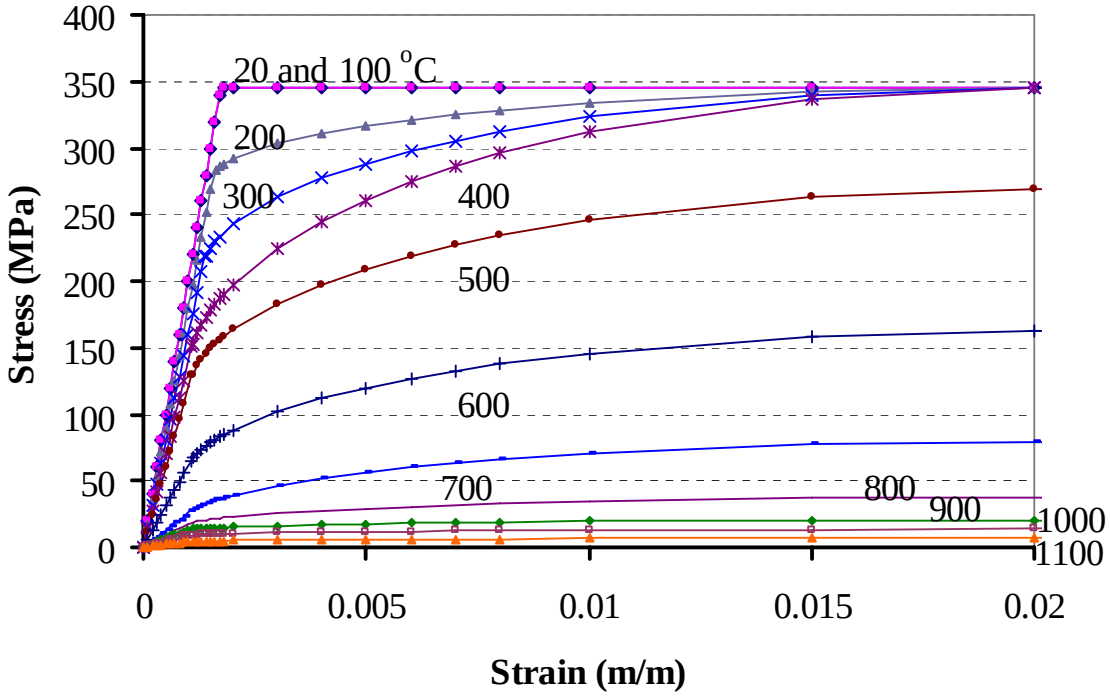


Figure 4.4: Stress-strain curve for ASTM 572 Grade 50 steel at elevated temperatures as defined by EC3 (BSI 2001)

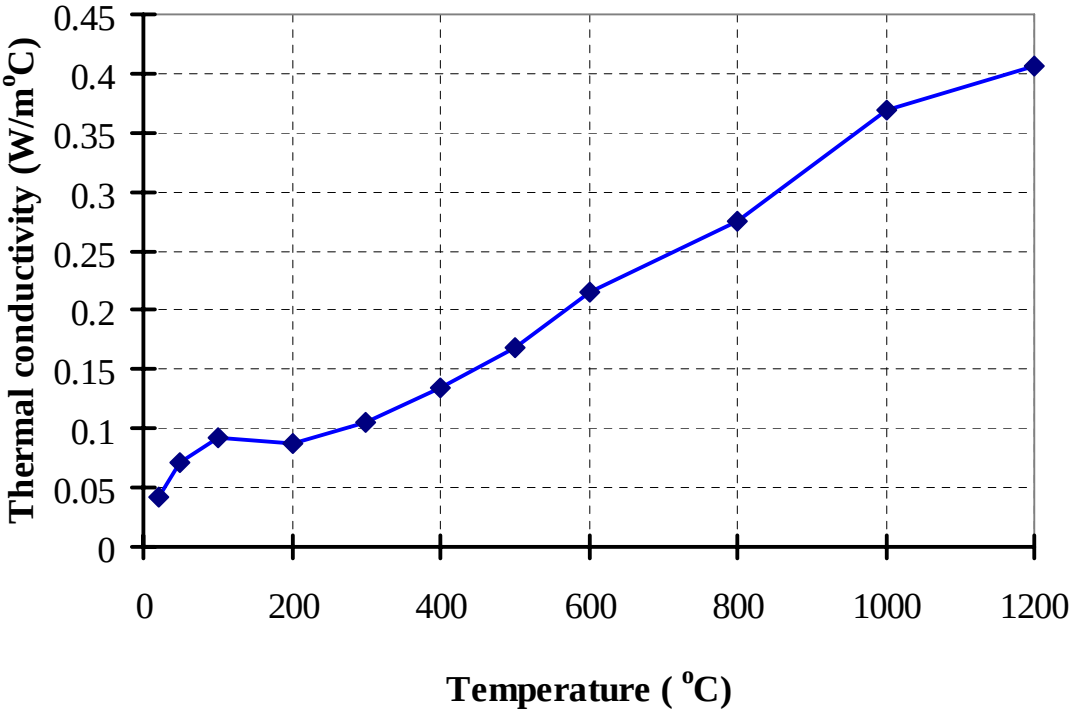


Figure 4.5: Thermal conductivity vs. temperature for SFRM (NIST 2004)

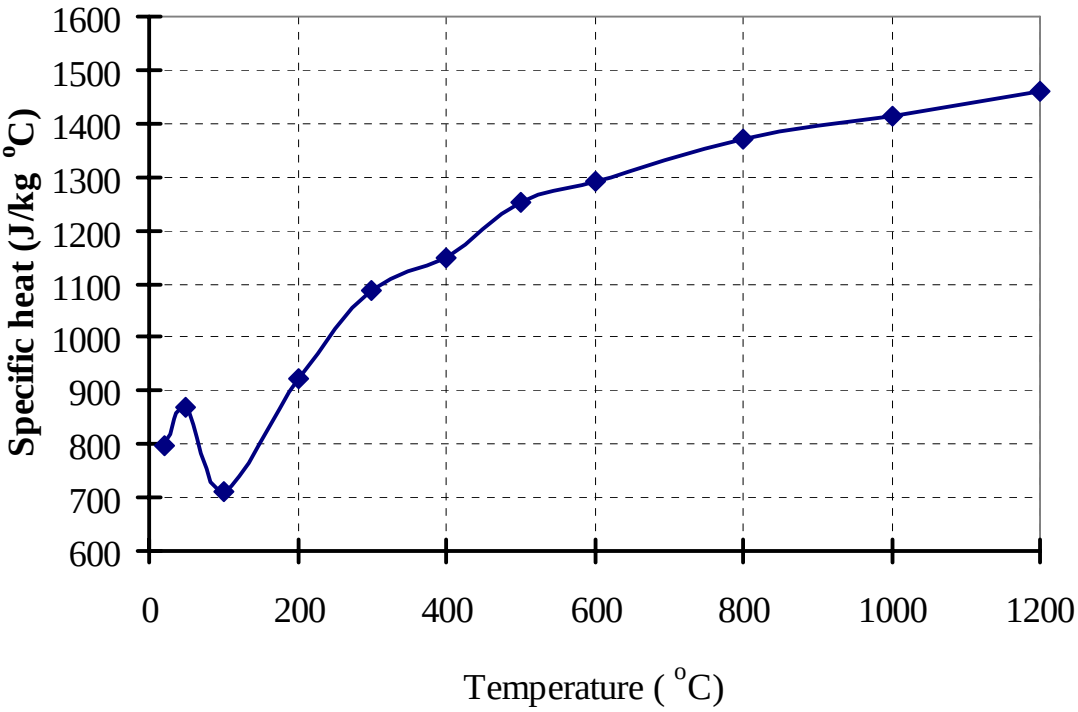


Figure 4.6: Specific heat vs. temperature for SFRM (NIST 2004)

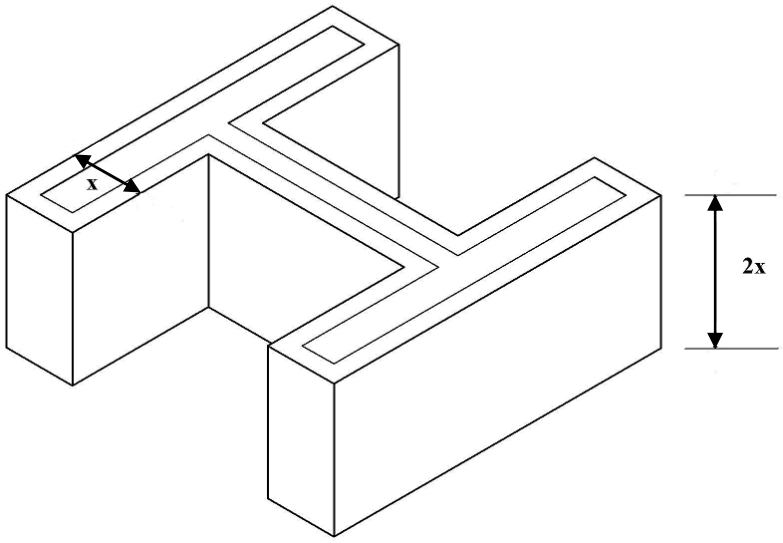
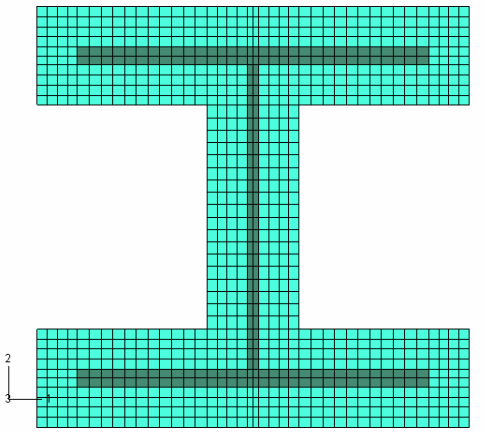
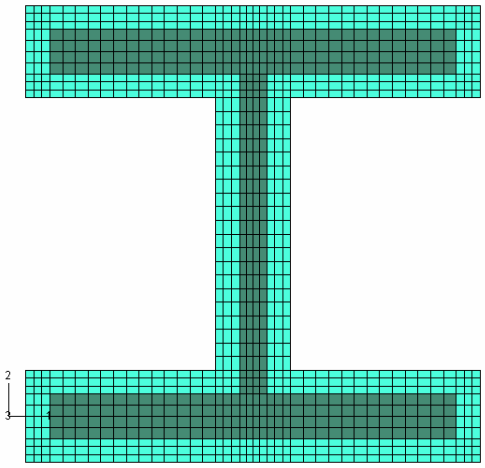


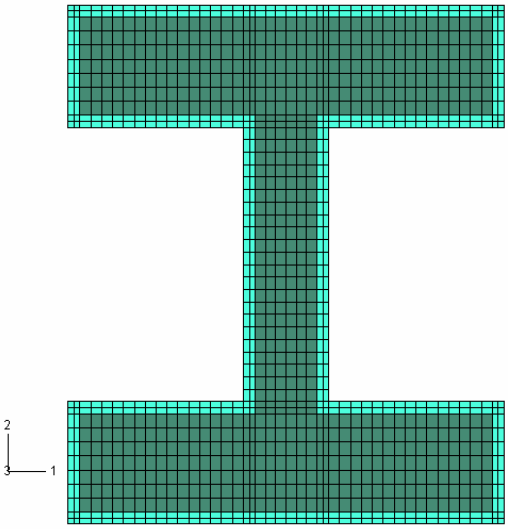
Figure 4.7: Schematic of steel column and SFRM model



(a)



(b)



(c)

Figure 4.8: Example FEM mesh for (a) Light (b) Medium and (c) Heavy columns

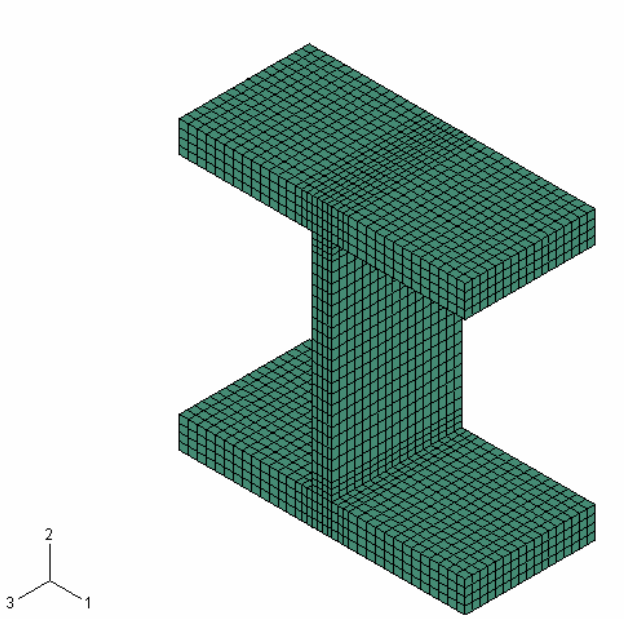


Figure 4.9: W360x347 structural analysis FEM

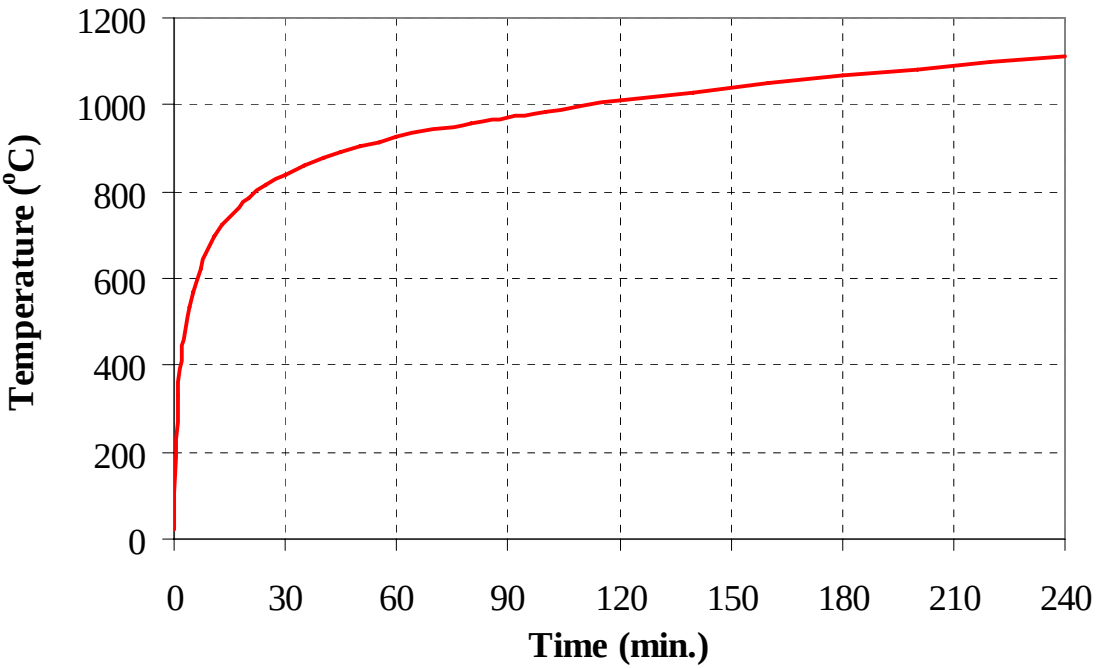


Figure 4.10: ASTM E-119 time temperature curve

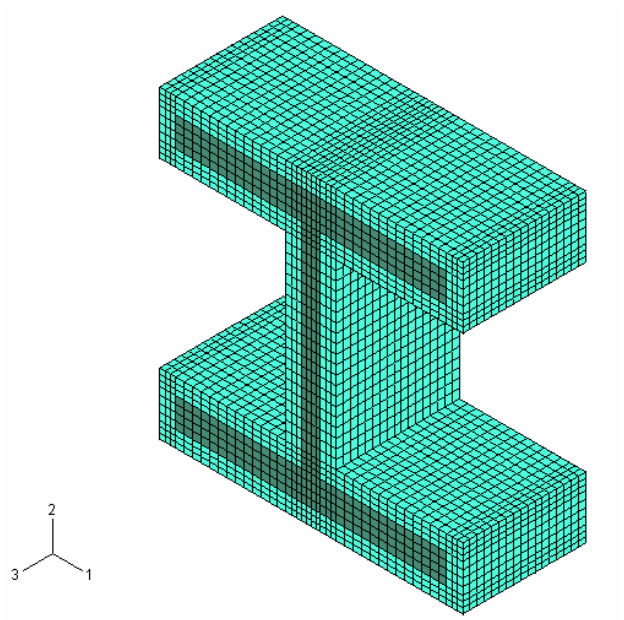
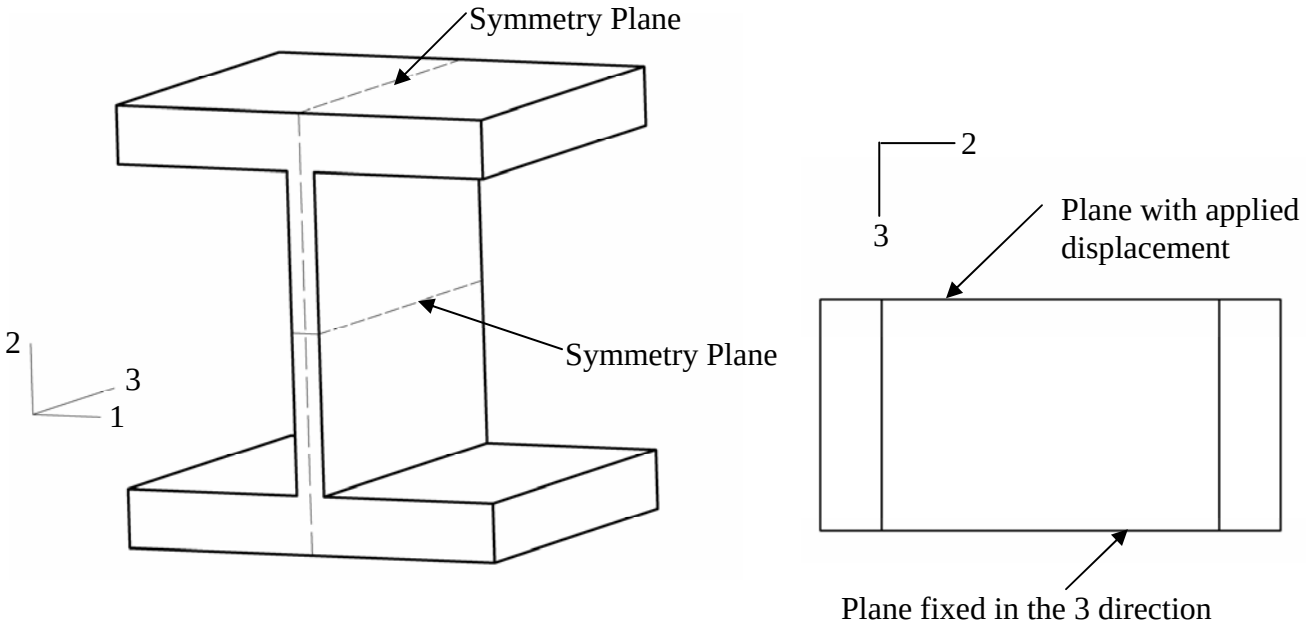


Figure 4.11: 347(SFRM347) heat transfer FEM



(a) Isometric view

(b) Elevation view

Figure 4.12: Boundary conditions imposed on column for structural analysis

CHAPTER 5

HEAT TRANSFER ANALYSIS RESULTS

5.1 Introduction

Heat transfer analyses were performed for 1, 2, 3 and 4 hour fires for the analysis cases depicted in Table 3.2. From the heat transfer analysis the temperature distribution across the cross section of the column in both the SFRM and the steel was obtained. Due to the symmetry of the column and SFRM, the temperature gradient through the cross section is also symmetric.

5.2 Function of SFRM

Figure 5.1 shows the temperature gradient for a 3 hour fire through a cross section of an example of a Light, Medium and Heavy column surrounded by SFRM. The temperature differential through the thickness of the SFRM is significantly greater than the differential through the steel. The temperature in the steel is somewhat uniform and significantly cooler when compared to the temperature of the SFRM.

Figure 5.2 displays the heat transfer results of a W360x162 column exposed to a 3 hour fire with two different SFRM thicknesses. The insulation has been removed in the figure to focus on the temperature gradient through the steel. The maximum temperature in both sections is reached at the center of the web. The 162(SFRM134) column is insulated with 42.9 mm of SFRM and the 162(SFRM162) column is insulated with 39.7 mm of SFRM. Although the basic temperature patterns in these two analysis cases are similar, at the steel flange tip the temperature varies by approximately 30˚C. For this case 3.2 mm of insulation provides a 30˚C difference in the steel. The thickness of SFRM applied to a column plays a role in the resultant temperature of the steel, as for example, with these two cases where a relatively small difference in SFRM thickness (3.2 mm) results in a 30˚C difference in the steel temperatures. In cases where the temperatures are near 400˚C (the temperature at which the steel strength and stiffness begins to dramatically decrease, see Figure 4.4), a 30˚C temperature difference could cause significant differences in structural behavior. The following section explores in more detail the temperatures in the steel compared to column size and SFRM thickness.

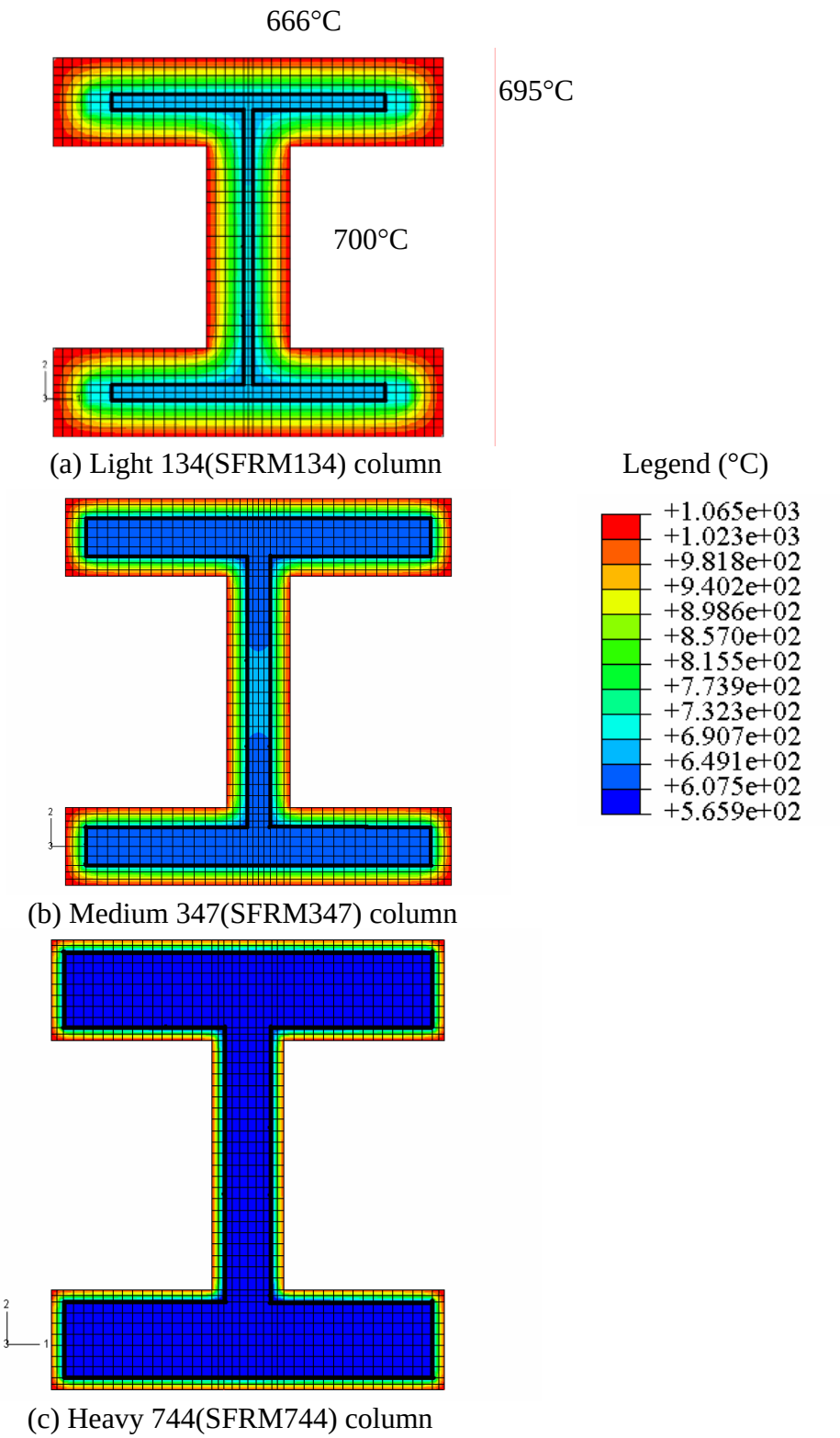
5.3 Steel Temperatures

The temperature in the steel varies throughout the cross section. In order to explore the degree of variation, the temperatures as related to location in the steel were obtained and plotted. Figure 5.3 illustrates the two axes along which the temperatures in the flange and web are plotted. The axes are aligned along the centerlines of the flange and web.

Figure 5.4 displays plots of temperature vs. distance along the axis in the center of the flange for the Light, Medium and Heavy column groups after a 1 hour fire. The general shape of the curves indicates the increase in steel temperature approaching the flange tips. Similarly, Figure 5.5 displays plots of temperature vs. distance along the axis in the center of the web for the Light, Medium and Heavy column groups after a 1 hour fire. The general shape of the curves indicates the decrease in temperature approaching the flanges. Similar plots for 2, 3 and 4 hour fires are displayed in Figures 5.6 through 5.11. The maximum temperatures in the section occur at the flange tips and the center of the web. The maximum temperature across the section for Light and Medium columns is located at the center of the web. The maximum temperature experienced by Heavy columns is located at the flange tips. In a particular column there is only a slight temperature difference between the maximum in the center of the web and the flange tips.

It might be expected that a series of columns with design SFRM thickness (i.e. 134(SFRM134), 147(SFRM147), and 162(SFRM162)) would have approximately the same temperatures for a given fire duration, since the design SFRM thickness for each column in the series differs and is specified so that the column will reach a certain fire duration rating. This behavior can be seen in Figure 5.8b and 5.9b for the Medium columns, where the temperatures reached at 3 hours for the 347(SFRM347), 382(SFRM 382), and 421(SFRM421) are approximately the same. For the Light and Heavy column groups, this relationship doesn't appear to hold as strongly however. Within these groups, as the columns get heavier, the temperature rise drops slightly.

At 1 hour the temperature ranges experienced by Light, Medium and Heavy columns are approximately equal. As the fire duration increases, the temperatures reached in Light columns are significantly higher than those reached in Heavy columns.

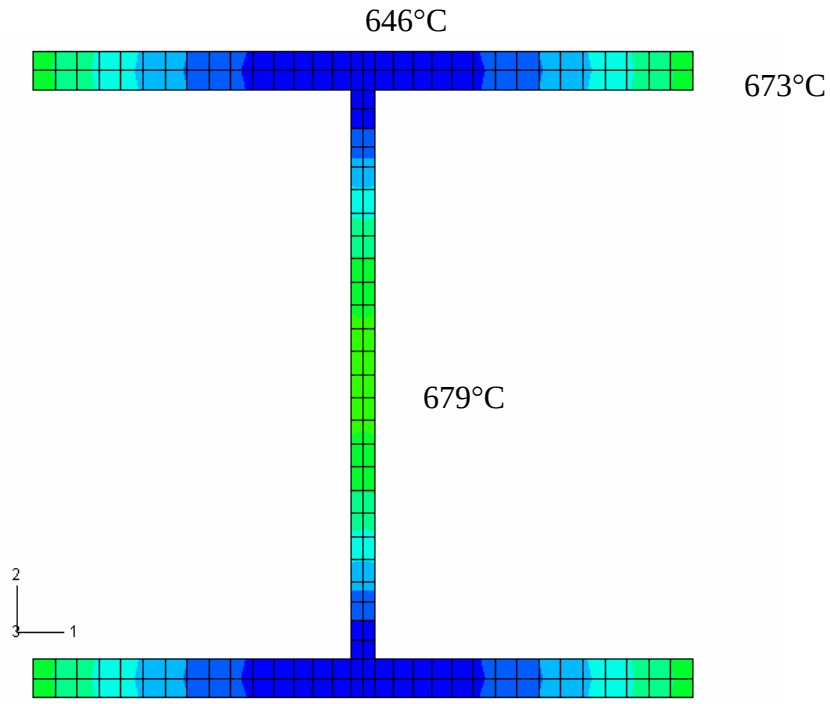


(a) Light 134(SFRM134) column

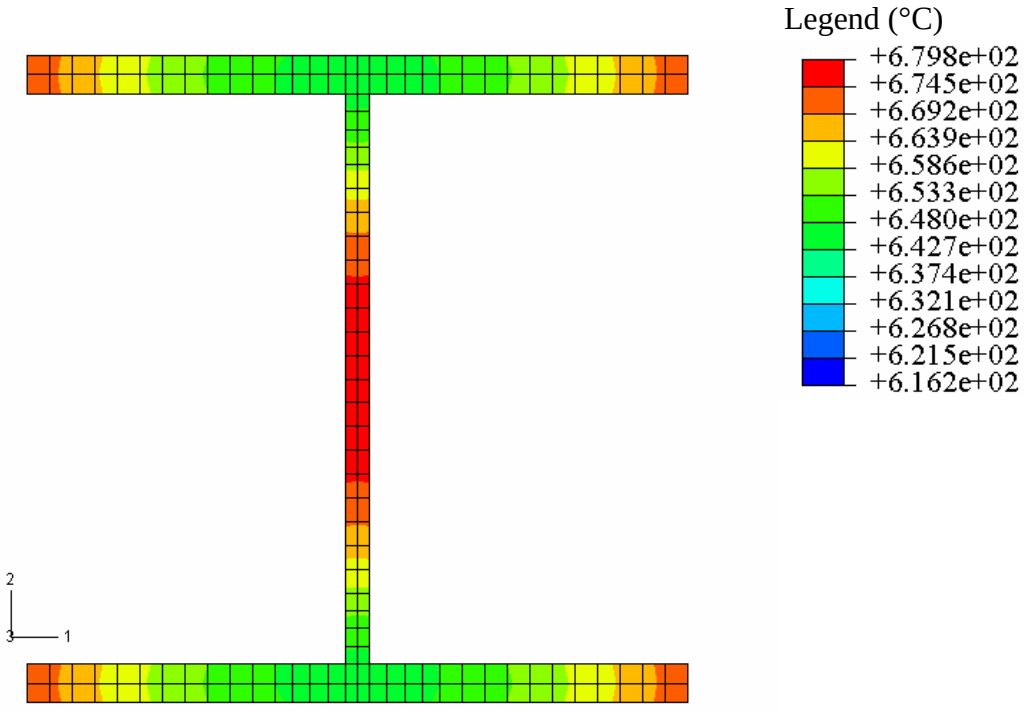
(b) Medium 347(SFRM347) column

(c) Heavy 744(SFRM744) column

Figure 5.1: Temperature gradient in base columns and SFRM at a fire duration of 3 hours: (a) Light; (b) Medium and (c) Heavy



(a) Temperature gradient through steel only 162(SFRM134) after 3 hour fire



(b) Temperature gradient through steel only 162(SFRM162) after 3 hour fire

Figure 5.2: Temperature gradients in W360x162 columns after a 3 hour fire: (a) 162(SFRM134); (b) 162(SFRM162) (SFRM not shown in figure)

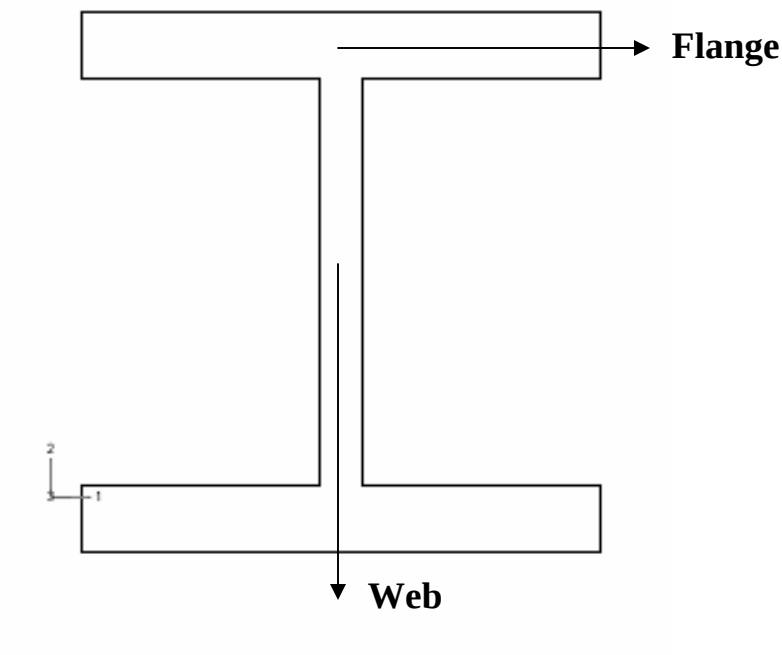
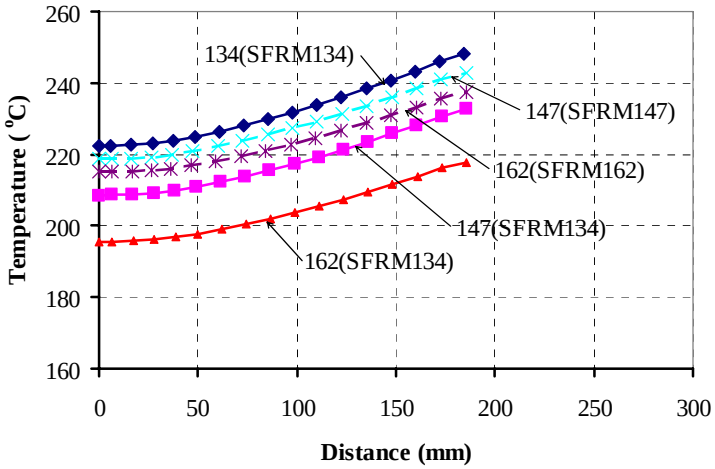


Figure 5.3: Axes along which temperatures are plotted



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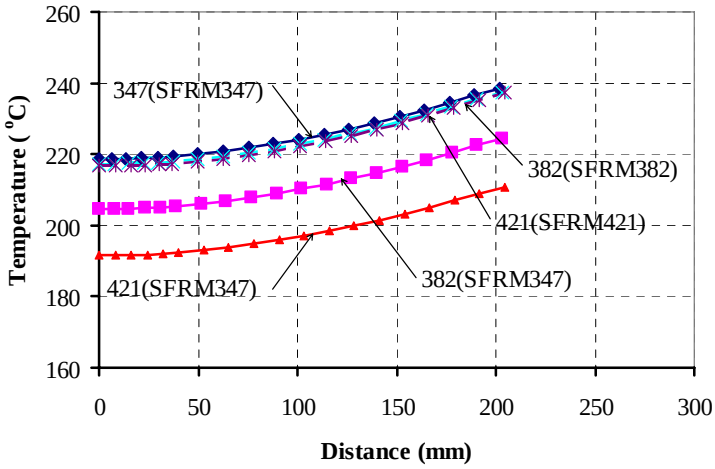

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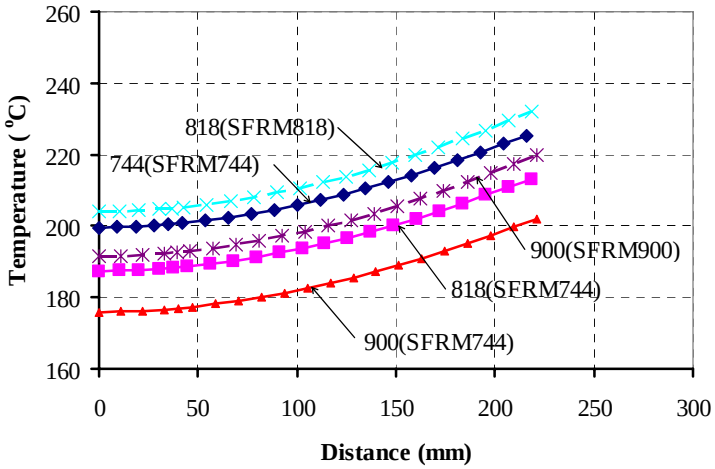
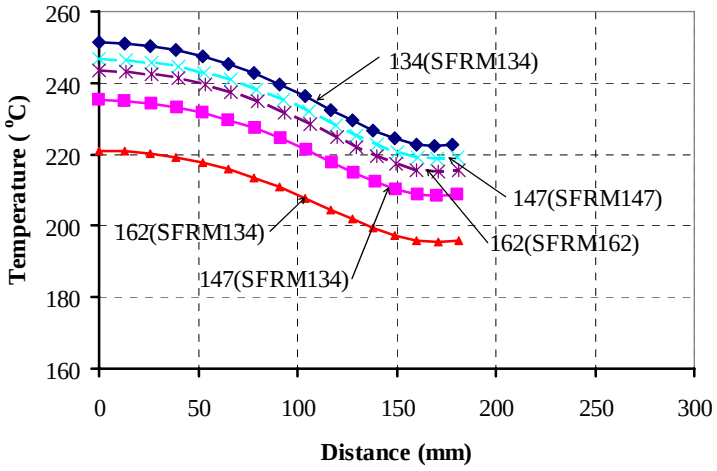
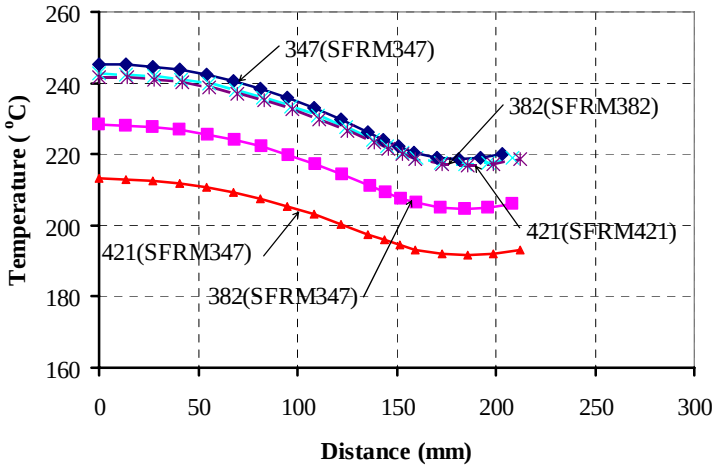


(c)

Figure 5.4: Distance vs. temperature in **flange** at **1 hour** for (a) Light (b) Medium and (c) Heavy columns



(a)



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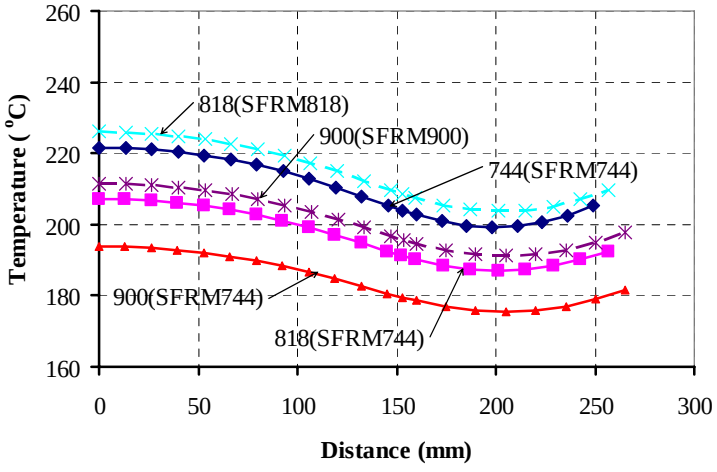
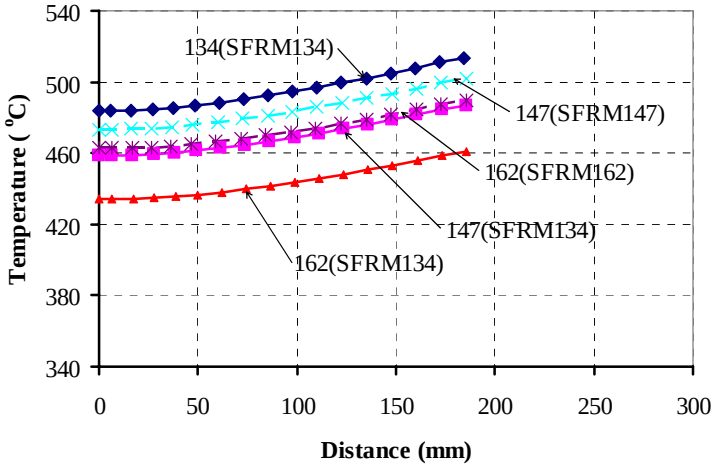
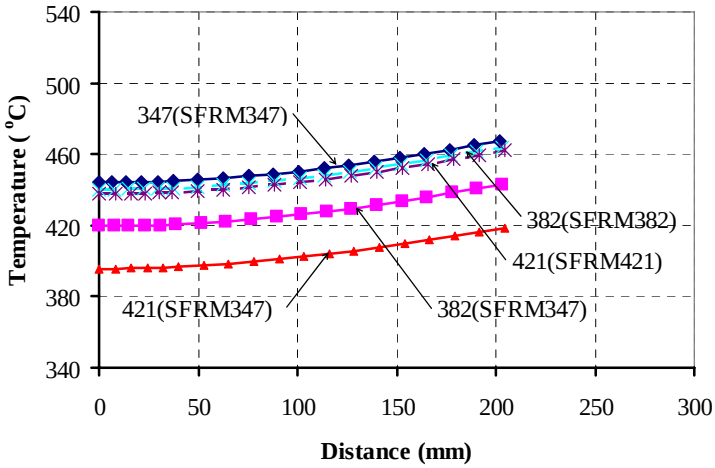


(c)

Figure 5.5: Distance vs. temperature in **web** at **1 hour** for (a) Light (b) Medium and (c) Heavy columns



(a)



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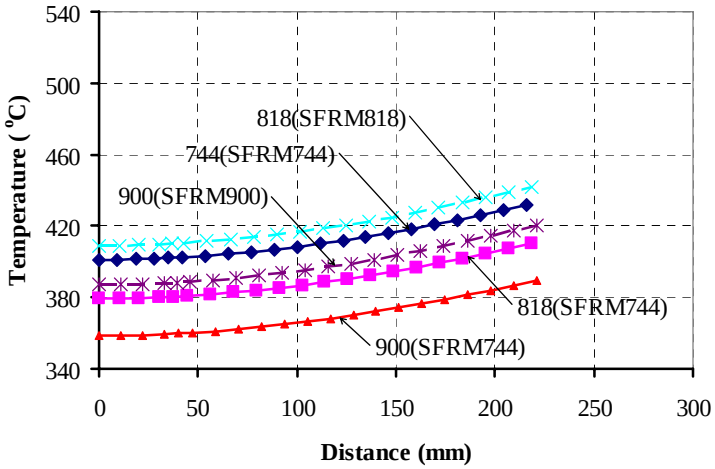
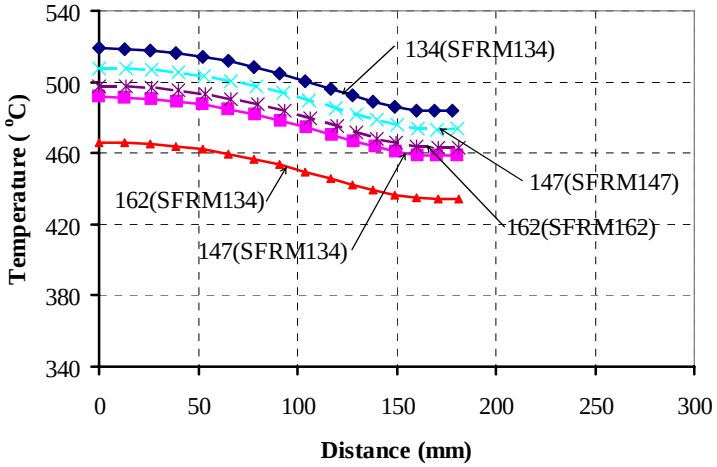
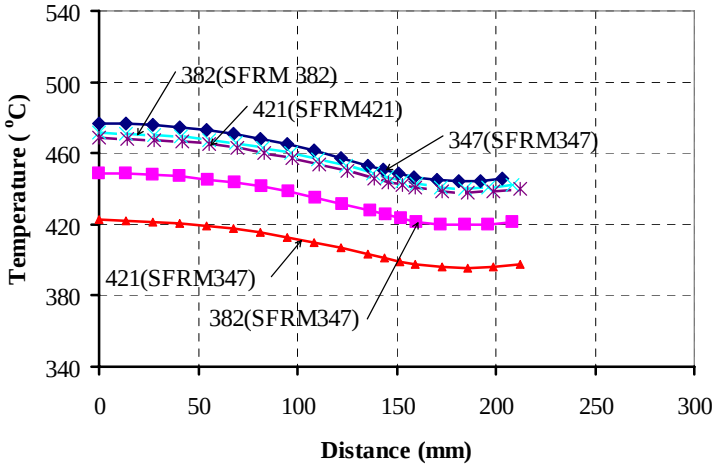


(c)

Figure 5.6: Distance vs. temperature in **flange** at **2 hours** for (a) Light (b) Medium and (c) Heavy columns



(a)



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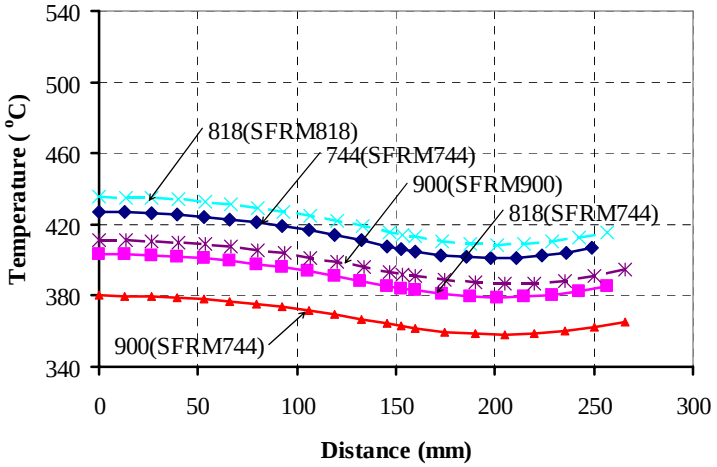
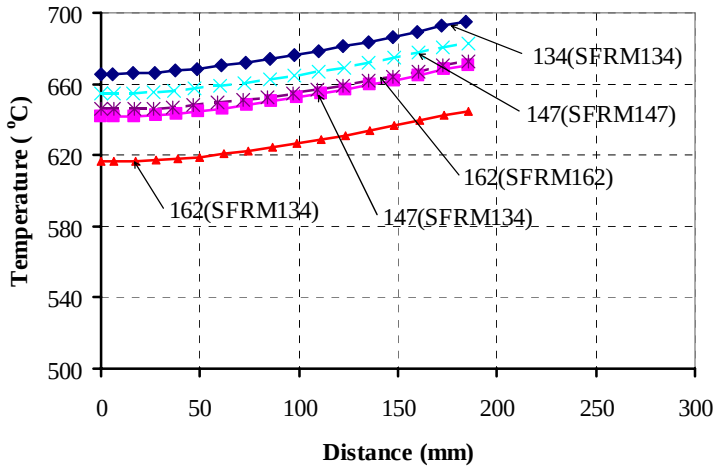
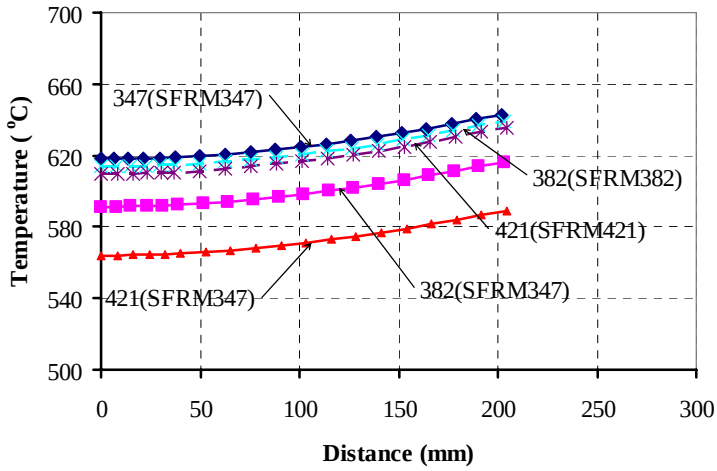


(c)

Figure 5.7: Distance vs. temperature in **web** at **2 hours** for (a) Light (b) Medium and (c) Heavy columns



(a)



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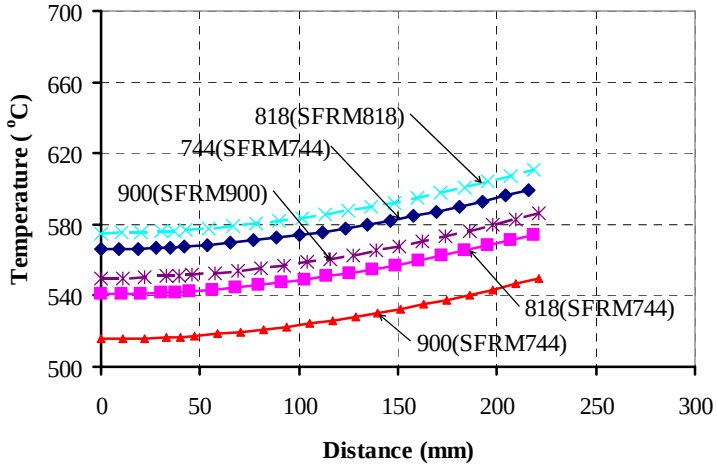
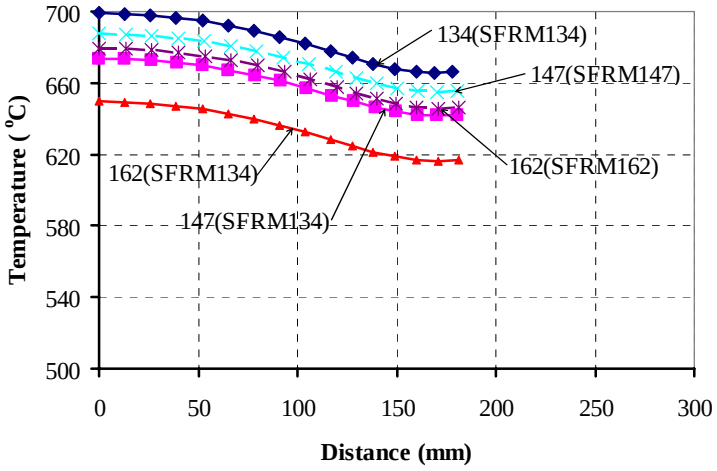
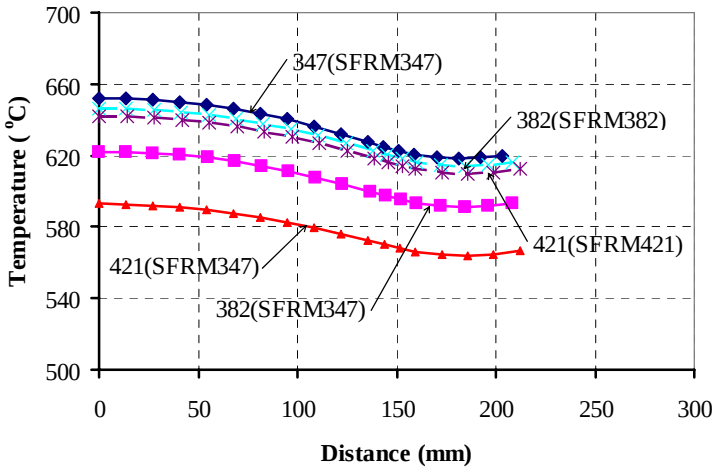


(c)

Figure 5.8: Distance vs. temperature in **flange** at **3 hours** for (a) Light (b) Medium and (c) Heavy columns



(a)



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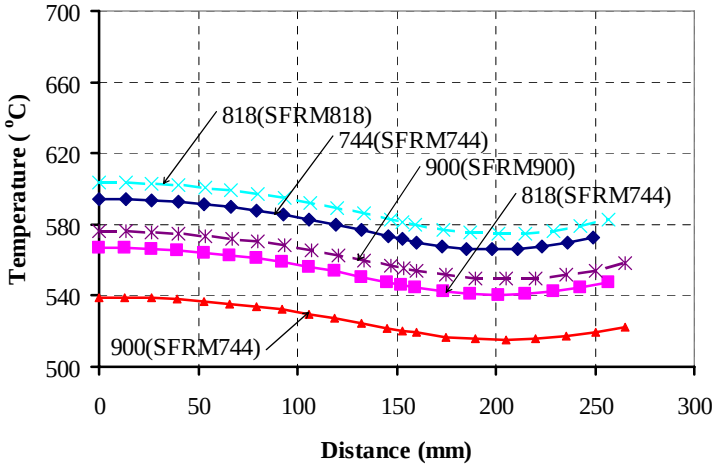
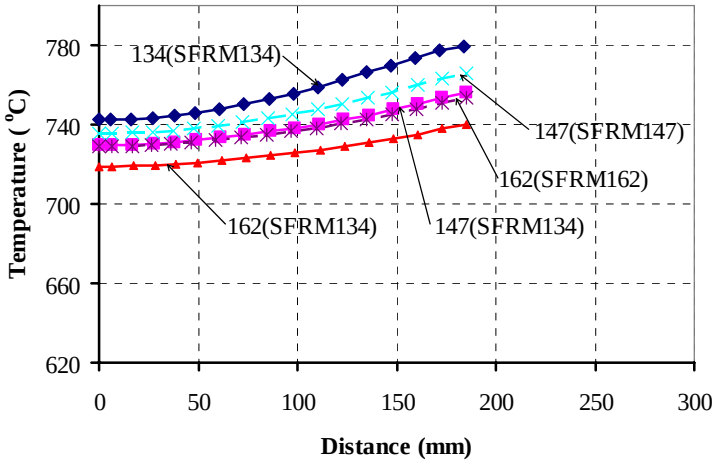
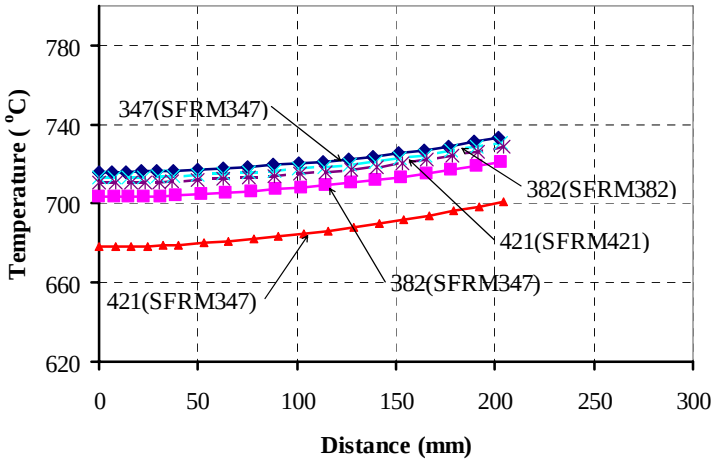


(c)

Figure 5.9: Distance vs. temperature in **web** at **3 hours** for (a) Light (b) Medium and (c) Heavy columns



(a)



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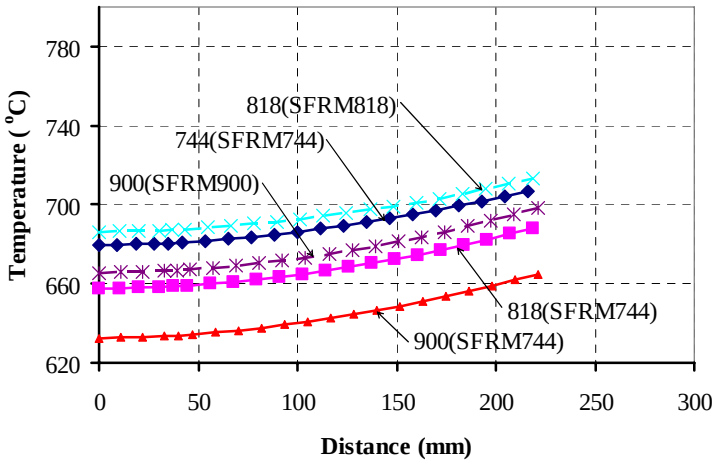
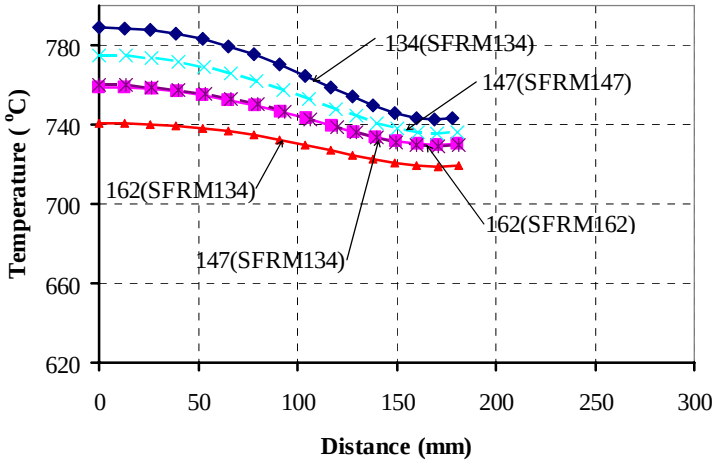
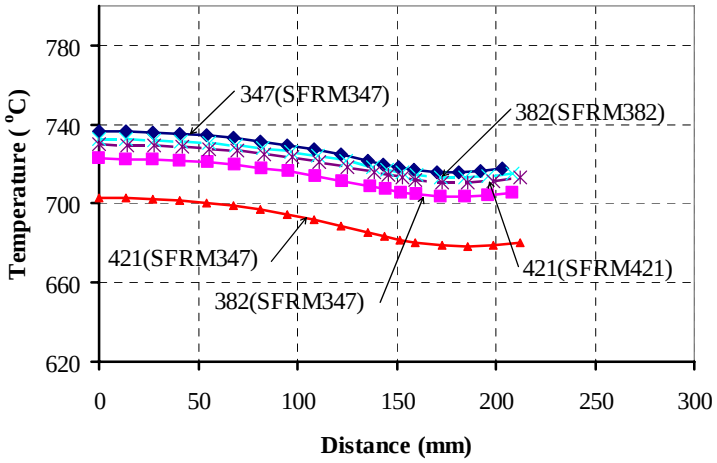


(c)

Figure 5.10: Distance vs. temperature in **flange** at **4 hours** for (a) Light (b) Medium and (c) Heavy columns



(a)



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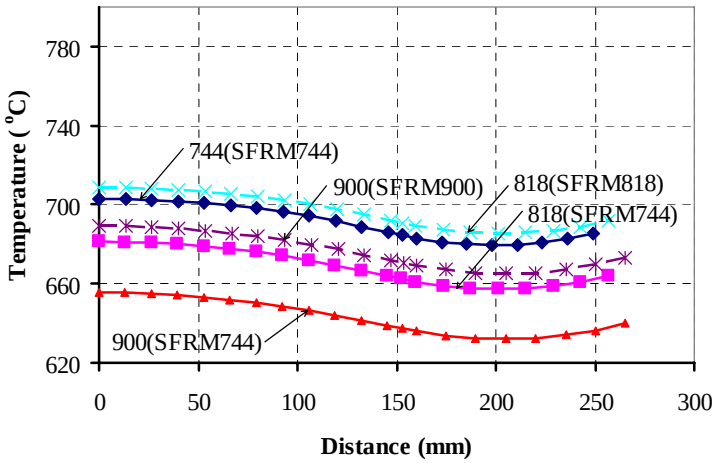


(c)

Figure 5.11: Distance vs. temperature in **web** at **4 hours** for (a) Light (b) Medium and (c) Heavy columns

CHAPTER 6

STRENGTH ANALYSIS RESULTS

6.1 Introduction

Following the heat transfer analyses, structural analyses were performed for all cases outlined in Table 3.2. The compressive axial displacement in each column was increased until a strain of approximately 0.075 in each column was achieved. As discussed, the structural analyses were performed with the temperatures determined in the heat transfer analyses as inputs, and thus structural analyses were performed for each analysis case considered at 1, 2, 3, and 4 hour fire duration. In addition, the ultimate load was also found for the case where no fire was present to establish an upper bound on strength.

6.2 Column Deformation

Due to the boundary conditions described in Section 4.5 and the symmetry of the column cross section, all columns deformed symmetrically about the centroid in the 1 and 2 direction. Figure 6.1a shows the deformed shape of an example column. This symmetry was expected. The column cross-section deforms equally along its length as it deforms and all edges remain plane (i.e. no warping of the cross-section occurs). Figure 6.1b illustrates this fact with an elevation view of the deformed column.

6.3 Determination of Maximum Load

Figure 6.2 though 6.16 show the axial strain vs. axial load curves for all analyses performed. In all columns, the axial load increases as the strain increases and eventually the load reaches a plateau. The load at which the plateau occurs is identified as the ultimate axial load capacity for a given column at a given fire time. Table 6.1 summarizes all of the axial load capacities determined from the curves. The curves demonstrate that as fire duration increases column behavior becomes more nonlinear in nature. The higher steel temperatures reached at longer fire durations mean that the steel has a lower yield point at longer fire durations (see Figure 4.4), and thus the columns begin to yield at lower loads at the longer fire durations. The fire duration also has an effect on the ultimate load capacity of the column. In a 1 hour fire, the columns all exhibited nonlinear behavior at a lower axial load than the corresponding room temperature column, however they all reached the same ultimate load. This is because the maximum temperatures reached at 1 hour in all of the columns are below 400˚C, and the ultimate strength of steel (see Figure 4.4) remains constant below 400˚C, although its stiffness does change. At longer fire durations, both the onset of reduction of axial stiffness in the columns and the ultimate load of the columns were reduced compared to the room temperature cases.

Table 6.1: Summary of structural steel analyses ultimate load results

	Ultimate Load (kN) Time (hour)				
Designation	**0**	**1**	**2**	**3**	**4**
134(SFRM134)	5811	5811	4547	1628	918
147(SFRM134)	6429	6429	5417	2177	1161
162(SFRM134)	7054	7054	6334	2820	1386
147(SFRM147)	6429	6429	5201	1977	1097
162(SFRM162)	7054	7054	5882	2328	1283
347(SFRM347)	15216	15216	13360	6051	3064
382(SFRM347)	16764	16764	15634	7779	3626
421(SFRM347)	18428	18428	18137	10048	4745
382(SFRM382)	16764	16764	14858	6834	3428
421(SFRM421)	18428	18428	16423	7682	3824
744(SFRM744)	32604	32604	31541	17248	8146
818(SFRM744)	35875	35875	35761	21800	10742
900(SFRM744)	39473	39473	39473	27065	14121
818(SFRM818)	35875	35875	34005	17890	8494
900(SFRM900)	39473	39473	39017	22715	10984

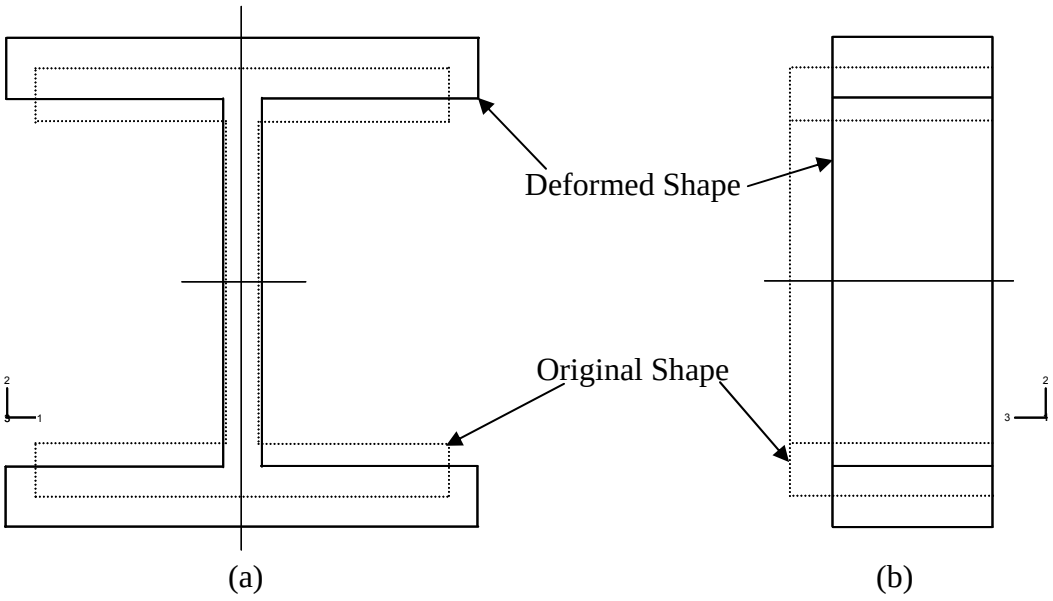


Figure 6.1: Deformed column shape (a) section view (b) elevation view

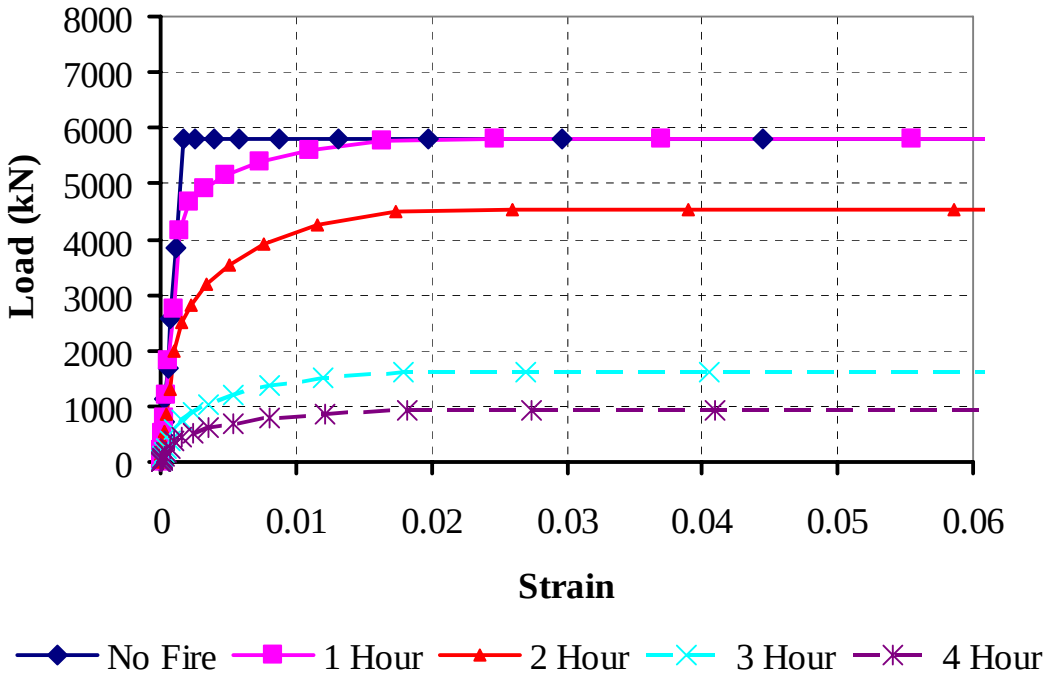


Figure 6.2: Axial load vs. axial strain plot for 134(SFRM134)

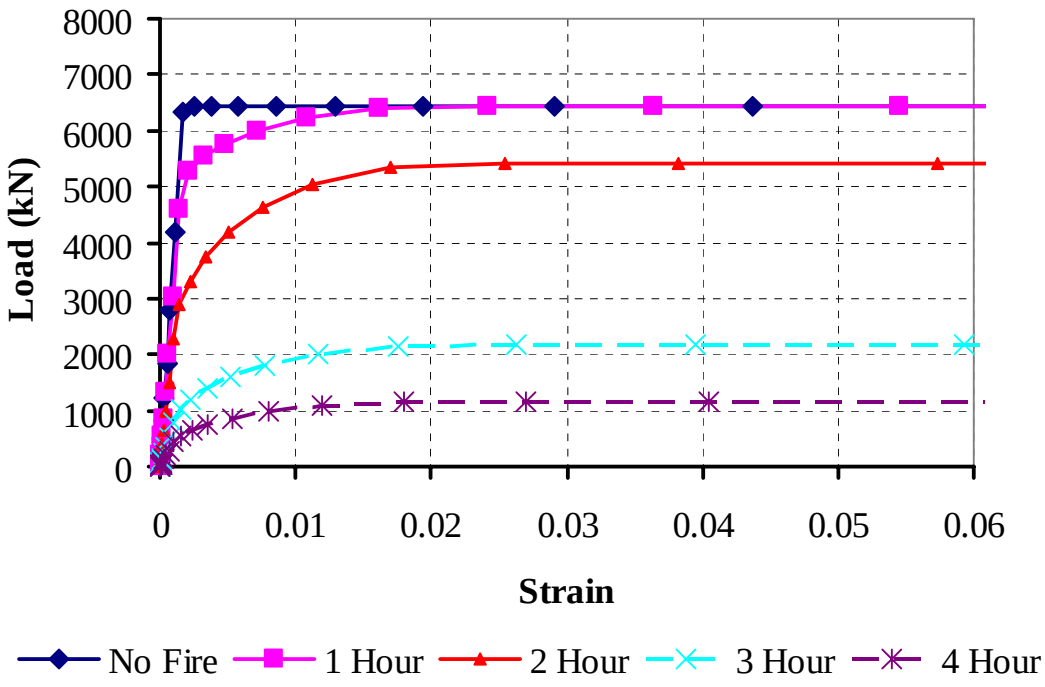


Figure 6.3: Axial load vs. axial strain plot for 147(SFRM134)

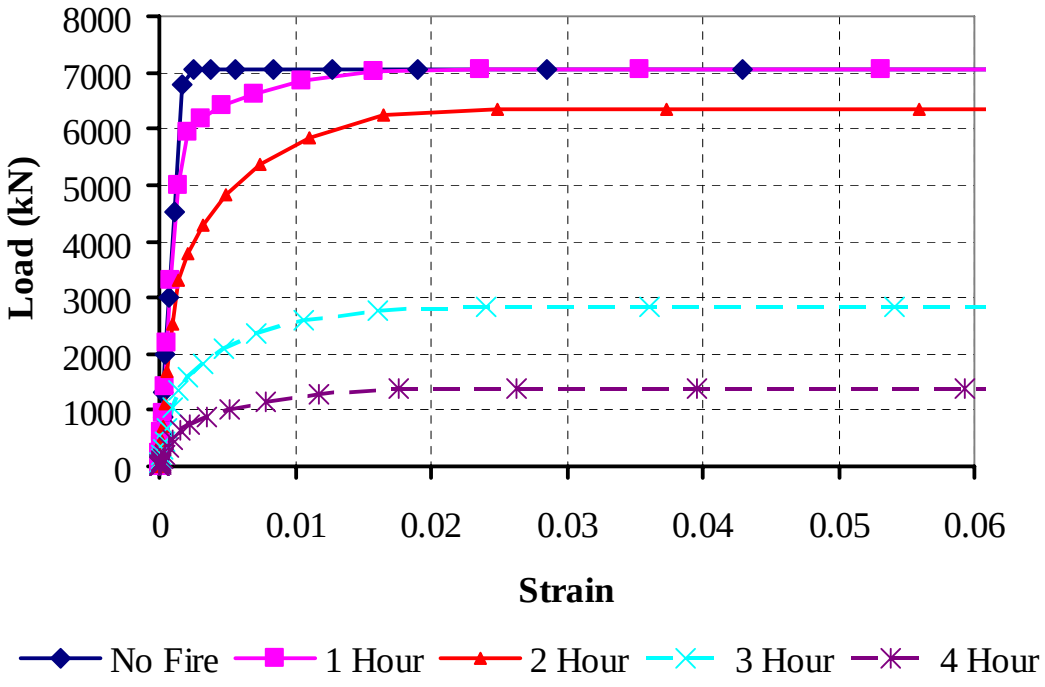


Figure 6.4: Axial load vs. axial strain plot for 162(SFRM134)

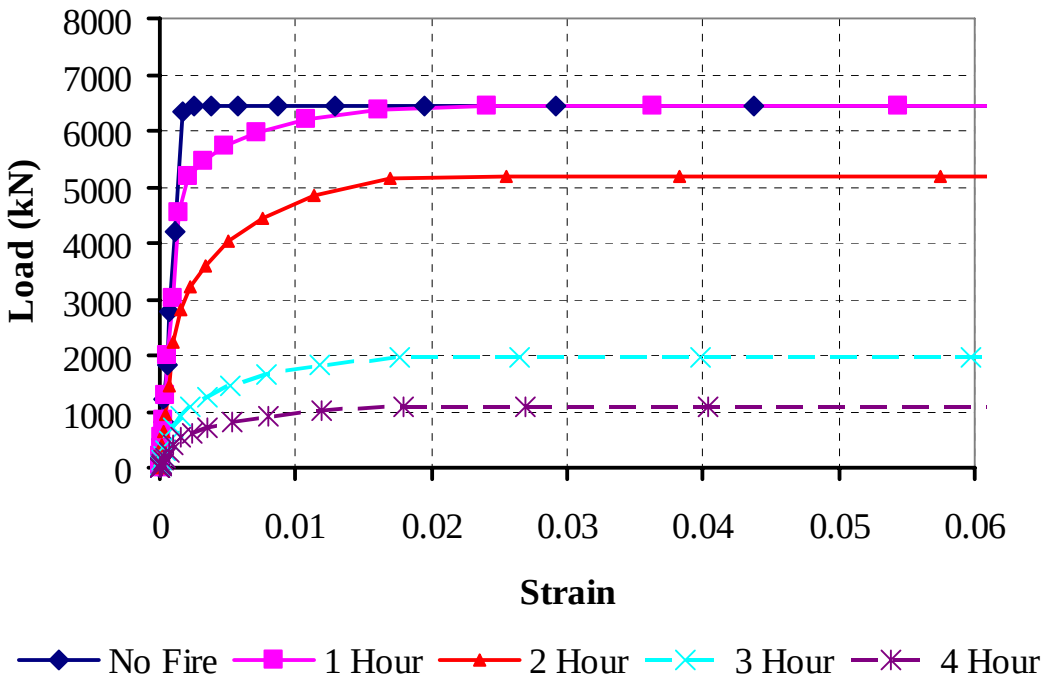


Figure 6.5: Axial load vs. axial strain plot for 147(SFRM147)

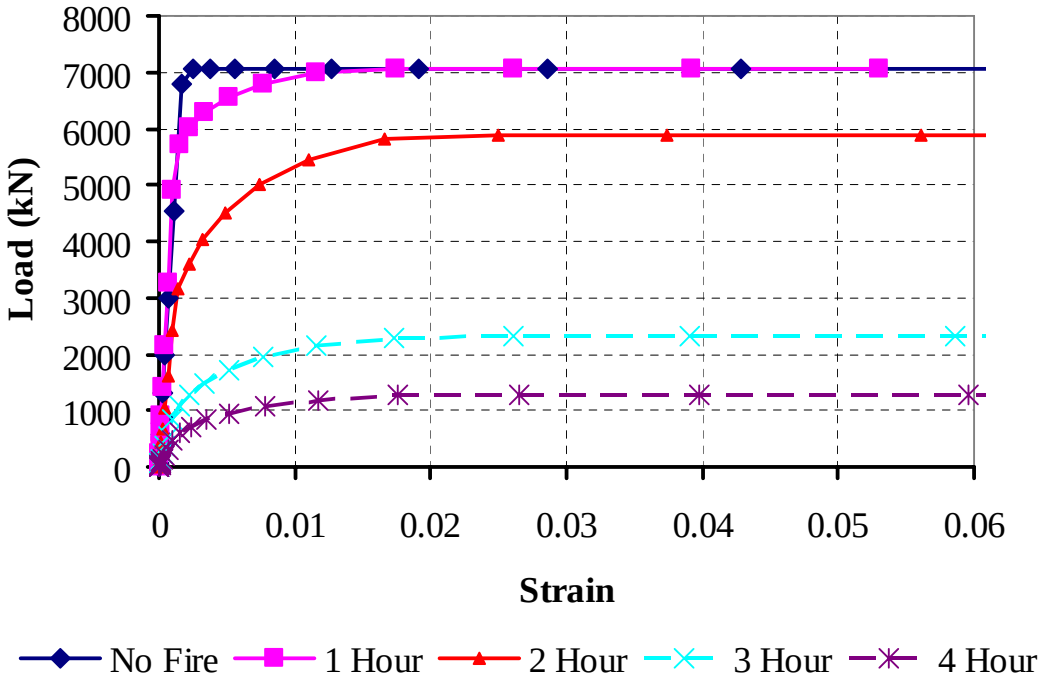


Figure 6.6: Axial load vs. axial strain plot for 162(SFRM162)

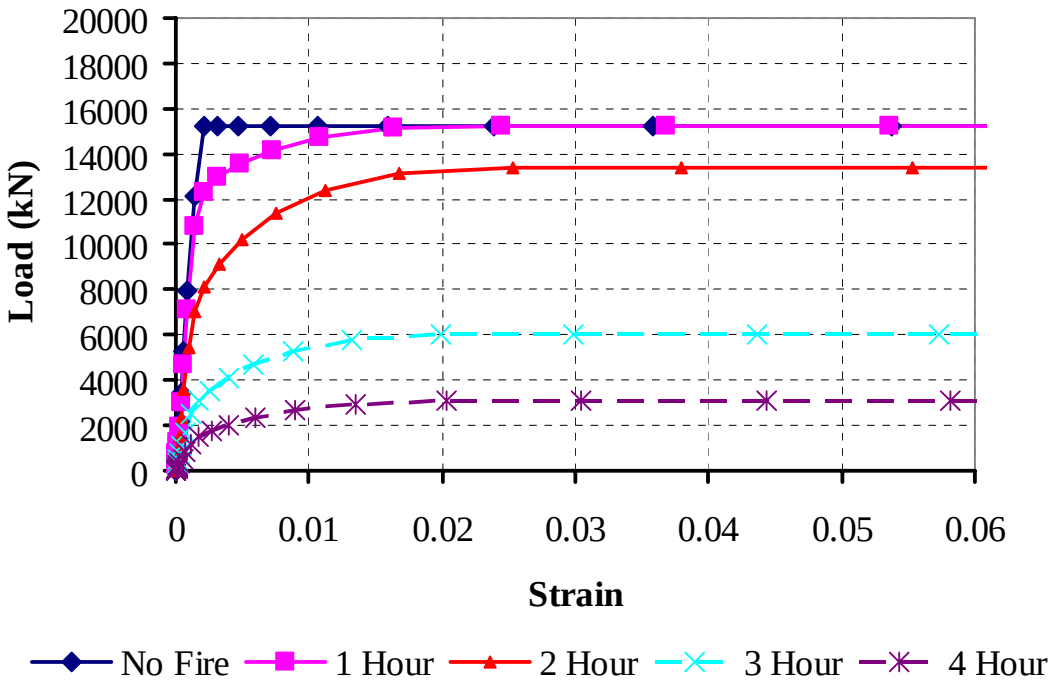


Figure 6.7: Axial load vs. axial strain plot for 347(SFRM347)

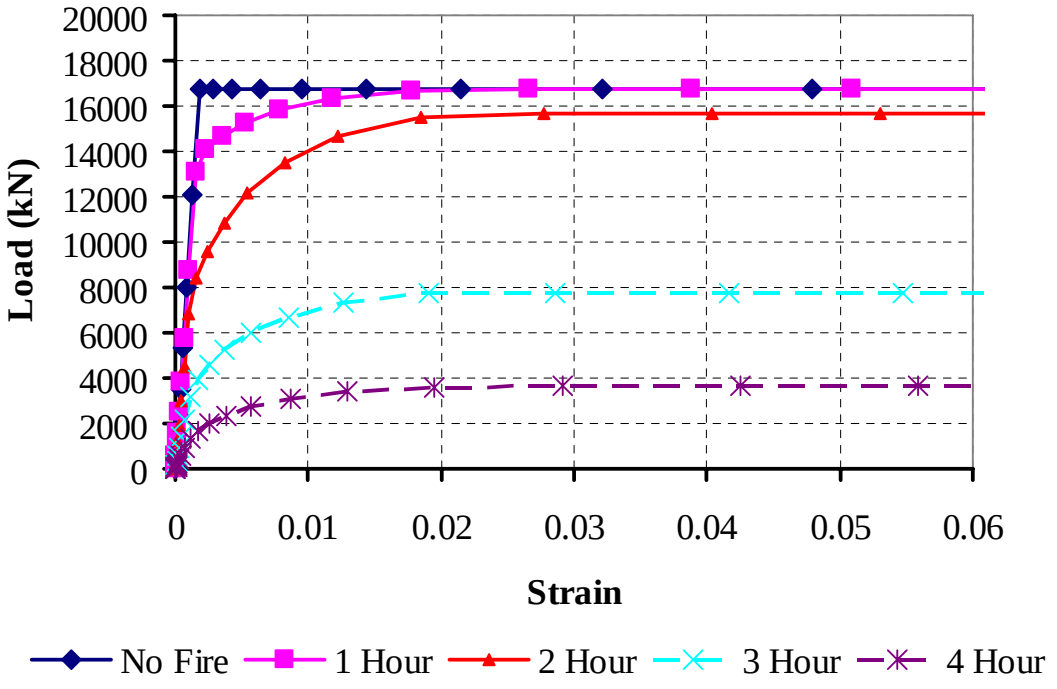


Figure 6.8: Axial load vs. axial strain plot for 382(SFRM347)

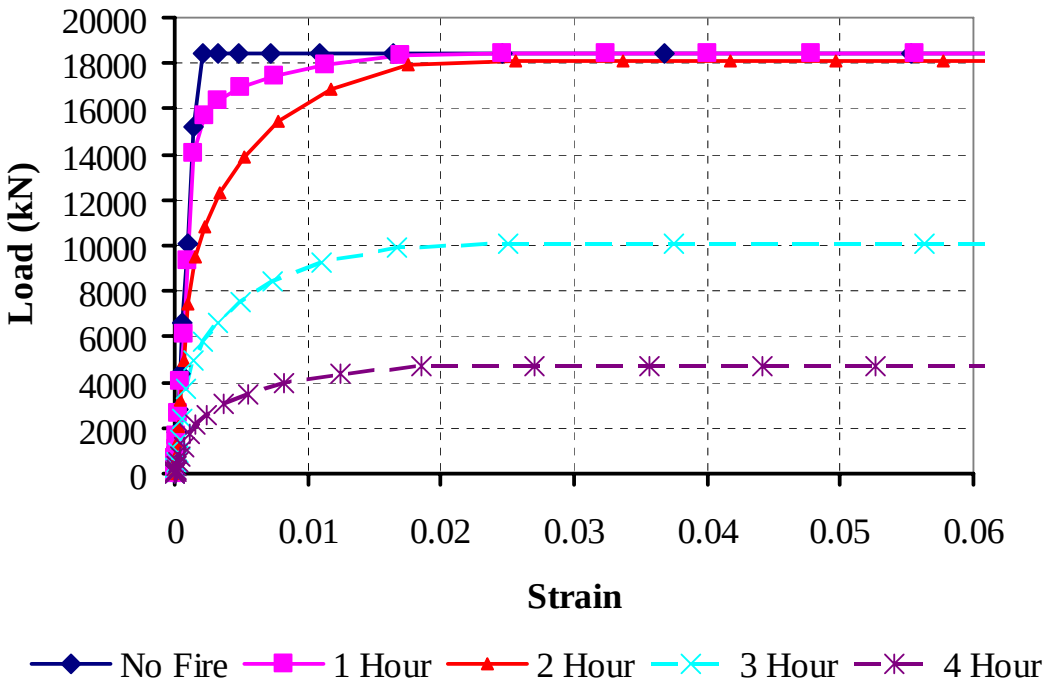


Figure 6.9: Axial load vs. axial strain plot for 421(SFRM347)

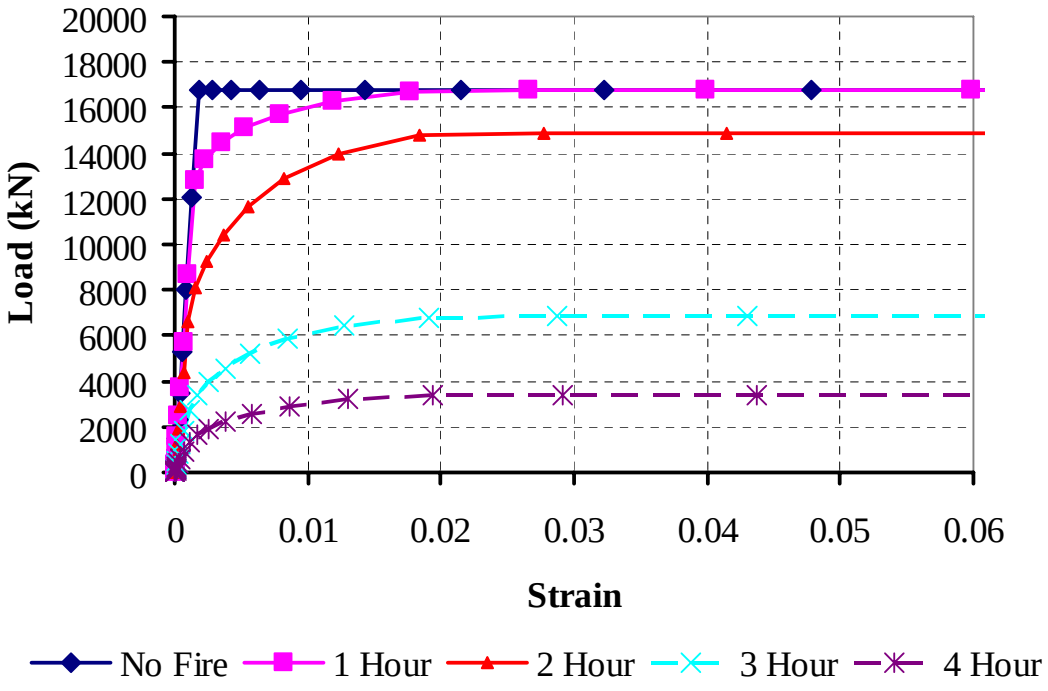


Figure 6.10: Axial load vs. axial strain plot for 382(SFRM382)

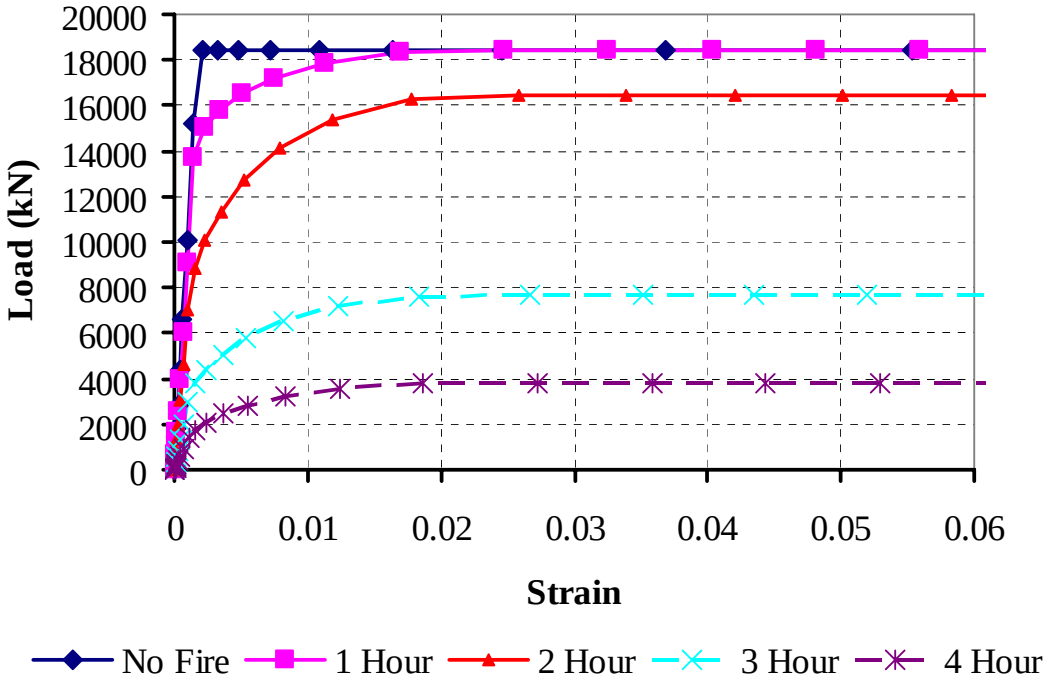


Figure 6.11: Axial load vs. axial strain plot for 421(SFRM421)

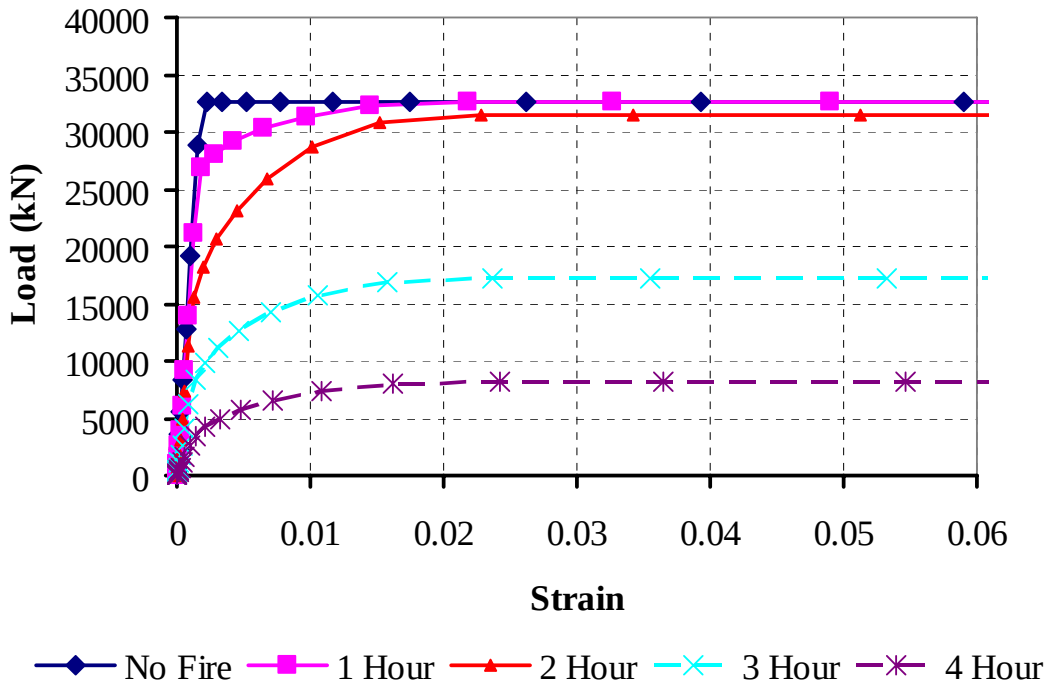


Figure 6.12: Axial load vs. axial strain plot for 744(SFRM744)

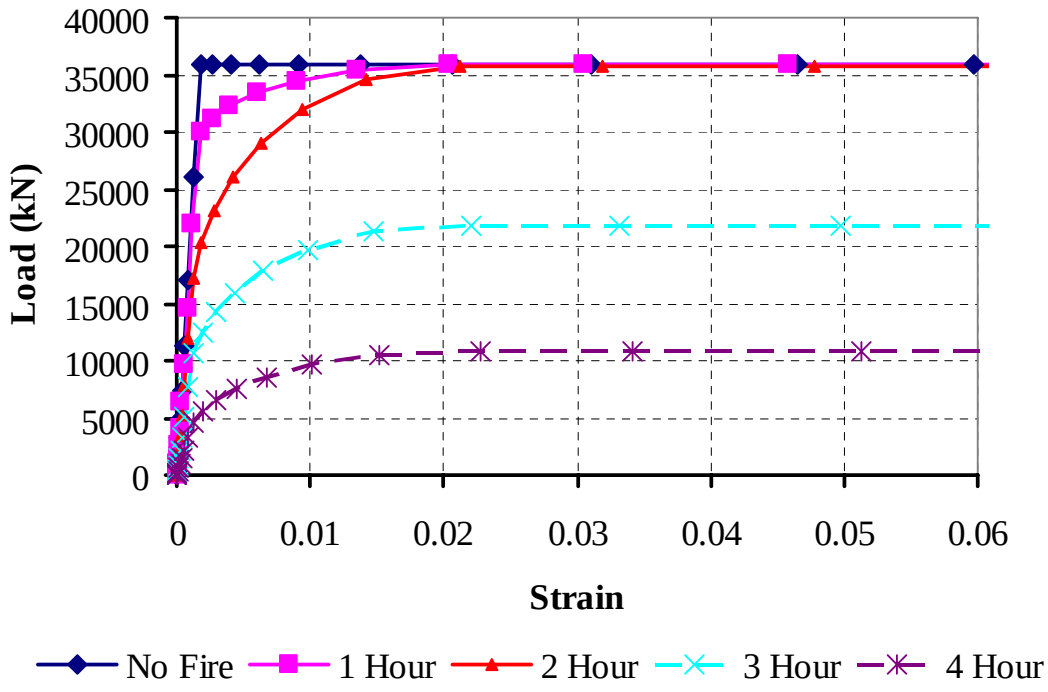


Figure 6.13: Axial load vs. axial strain plot for 818(SFRM744)

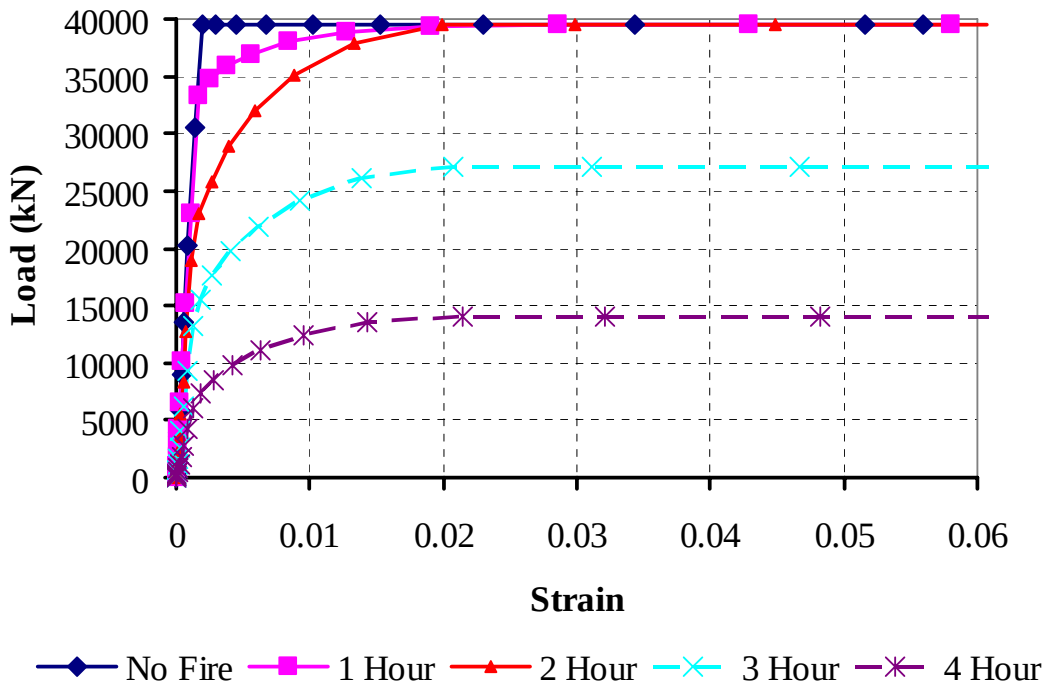


Figure 6.14: Axial load vs. axial strain plot for 900(SFRM744)

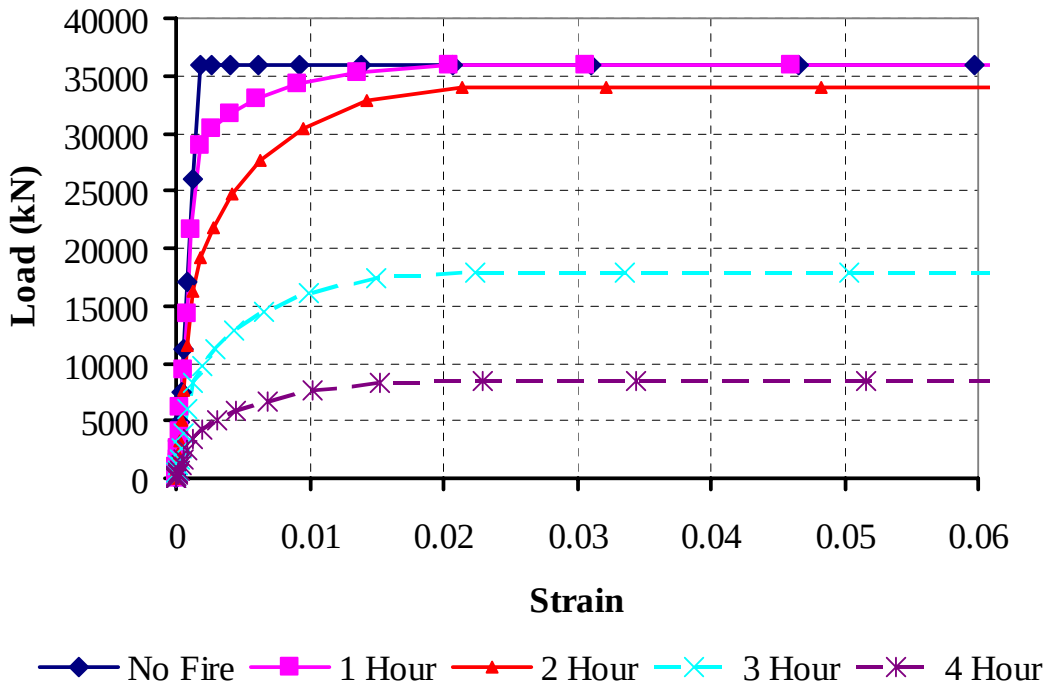


Figure 6.15: Axial load vs. axial strain plot for 818(SFRM818)

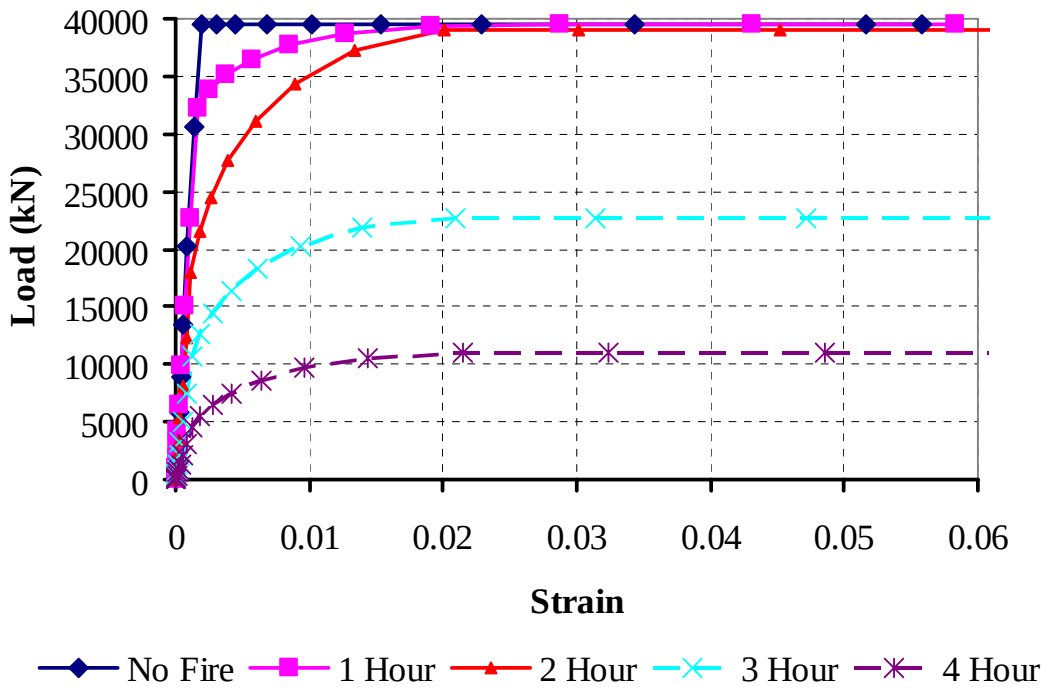


Figure 6.16: Axial load vs. axial strain plot for 900(SFRM900)

CHAPTER 7

DISCUSSION

7.1 Introduction

This section discusses the results from the thermal and structural analyses that were described in Chapter 5 and Chapter 6.

7.2 Fire Duration vs. Normalized Load Capacity

Figures 7.1 – 7.3 show the relationship between the normalized axial load capacity and the time duration of the fire for the Light, Medium and Heavy column groups. The ultimate loads of the columns were normalized with respect to the ultimate load capacity of the base column (defined in Section 2.3).

Figure 7.1 is the plot of normalized axial load capacity with time for the Light column group. The solid lines represent the three successive column sizes where the SFRM thickness was maintained and the dashed lines represent the two column sizes where the SFRM thickness was reduced to its design value for the given section.

The P/Pu value with no fire for the 147 and 162 columns represents the increase in percent area as compared to the 134 (SFRM 134), i.e. when moving from the 134 to the 147 there is an increase in area of approximately 10%, and from 134 to 162 there is a 21% increase. This is due to the axial load capacity being based solely on the yield strength of the steel and the cross sectional area (i.e. buckling and other length effects not considered).

It is apparent that as the column size is increased strength is gained. The magnitude of this gain varies depending on fire duration and is dependent upon SFRM thickness and the percent weight increase of the column.

Between 0 and 1 hour the SFRM thickness has no effect on load capacity. After 1 hour all of the plots begin to slope downward indicating a loss in strength. The negative effect of decreased SFRM thickness can be seen clearly by the increased downward slope of the 147(SFRM147) and 162(SFRM162) (as compared to the 147(SFRM134) and 162(SFRM134) respectively). The SFRM thickness has the largest impact on strength at 3 hours, while at 4 hours the impact is less. At 4 hours, the temperatures in the Light columns are generally above 740˚C, and little strength remains in the steel. This causes the ultimate load in all of the Light columns to begin to converge near 4 hours.

Figure 7.2 and 7.3 display similar plots for the Medium and Heavy column groups, respectively. The behavior of the Medium columns from 0 to 1 hour is comparable to that of Light columns, where the ultimate capacity is not affected by the SFRM thickness, and the capacities of the columns have not reduced. Heavy columns with maintained SFRM (e.g. 818(SFRM744, 900(SFRM744)) maintain roughly the same

load capacity up to a 2 hour fire duration before they begin to slope downward at longer fire times. This is because the steel temperatures in these Heavy columns at 2 hours have not yet reached 400˚C, at which point they would begin to yield. In contrast, Heavy columns with design SFRM (e.g. 818(SFRM818), 900(SFRM900)) begin to lose capacity even at the two hour mark. In all cases (Heavy, Medium or Light), increased column sizes with design SFRM under-perform those with maintained SFRM, especially at the mid-range fire durations (i.e. 2 – 3 hours).

The plot of the Medium column group (Figure 7.2), shows that at 3 hours the 382(SFRM347) has the same capacity as that of the 421(SFRM421). This demonstrates the advantage of maintaining SFRM thickness as column size is increased. The 382(SFRM347) is clearly a smaller column than the 421(SFRM421), but because the thicker SFRM was applied to the smaller column its strength in a 3 hour fire was increased to that of the larger column.

7.3 Normalized Load Capacity vs. ΔFD

An alternate way to interpret the results of these analyses is to compare the added fire duration (ΔFD) that a given column can resist at a given axial load level (P/Pu). Utilizing Figures 7.1 – 7.3, the increase in fire duration for a given column at a range of P/P_u values was determined. Figure 7.4 illustrates how this time increase was measured with respect to the base column. In the figure, the change in fire duration (ΔFD) was calculated at a P/P_u of 0.8

Figures 7.5 – 7.7 display the results of the ΔFD versus P/P_u curves for light, Medium and Heavy columns, respectively. The usefulness of these plots is best explained through example. If a designer has an original design for a 134(SFRM134) column loaded at a particular P/P_u and wishes to increase the column size or thickness of SFRM, this chart will tell the designer how much increased fire duration that the increase in column size will provide during a standard fire. For example, if the 134(SFRM134) column is loaded at a P/P_u equal to 0.4, increasing that column to a 162(SFRM134) will increase the fire duration of that column for almost 35 minutes.

It is useful to explain the fire duration gains in ranges of time due to the variability of each curve. For example, according to Figure 7.6 replacing a 347(SFRM347) with a 382(SFRM382) will gain about 7 to 14 minutes during a fire depending on the level of axial load imposed on the considered column. If the same 347(SFRM347) is replaced with a 421(SFRM347) anywhere from 33 to 45 minutes can be gained in a standard fire.

In general for all groups, increasing the column size 10% and maintaining the SFRM thickness will obtain about the same amount of increase in fire duration as increasing the column size 20% and changing the SFRM thickness to the design value, a gain of 15 to 25 minutes. This is helpful in design when cost is always a concern. For the 744(SFRM744), increasing to a 900(SFRM744) will gain 31 to 45 minutes of fire duration while increasing to a 900(SFRM900) will only gain 17 to 26 minutes of fire duration in a standard fire. The 1.6mm of extra SFRM on the 900(SFRM744) will gain approximately 15 to 20 minutes of fire duration in a standard fire.

7.4 Peak Load versus Column Weight

In order to further quantify the relationship between increasing column size and the ultimate load capacity, plots of normalized peak load capacity versus normalized column weight were created. The percent increase in both capacity and weight is normalized with respect to the capacity and weight of the base column. Figures 7.8a-d display this plot when the design value of SFRM is applied for 1, 2, 3 and 4 hour fire durations, respectively. Figure 7.8 indicates that for columns with design SFRM, a slight increase in column size is most effective for Light columns (except at 1 hour where the temperatures reached in all of the columns are below 400 °C regardless of size and thus all columns behave similarly). It is also interesting to note the slopes of the trend lines noted in the figure. The trend lines are based on the data for all three column groups, and have slopes that vary from approximately 1.0 to 1.7, depending on the time of fire. If it is supposed that SFRM thickness guidelines are designed so that columns will achieve a given fire duration rating, it might be expected that the slopes of the trend lines in the figure would all be approximately 1.0. This is only true for the 1 hour fire time. At all other fire durations, the trend line slopes are greater than 1. This shows that there is an additional thermal benefit to increasing the size of a column, one that likely relates to the concept of thermal inertia. In a fire it is simply more difficult to change the temperature of a larger column than a smaller one because the larger column has greater thermal mass. This means that in a fire, a column with increased weight (relative to a smaller one) will out-perform the smaller column to a more significant degree than an estimate based only on the weight difference between the two would indicate.

Figure 7.9 shows a similar set of plots for columns in which the SFRM thickness for the base column was maintained for the columns with increased weights. The figure shows that for columns with maintained SFRM, a slight increase in column size is most effective for Light columns until the 4 hour fire time. At 4 hours the increase is most effective for Heavy columns. Examining the trend lines in the figure also provides an interesting insight. At the longer fire times the slopes of the trend lines are significantly greater than 1. In these columns two beneficial effects combine. In addition to the thermal mass benefit (discussed above), these columns also benefit from SFRM that is thicker than would be specified by design for a three hour standard fire. Thus these columns stay cooler than their counterparts with SFRM designed for a three hour standard fire, and increasing column size slightly is particularly effective here.

It is apparent that it is possible to gain an increase in capacity by increasing the weight of a column, or increasing the SFRM thickness of a column, or both. An advantage to increasing a column's weight is that there is a corresponding inherent strength increase at room temperature that provides reserve capacity for potential overloads from other loading scenarios.

7.5 Capacity Increase versus Fire Duration

Figure 7.10 shows the capacity increase versus fire duration time for each of the column analyses. The solid lines in the figure denote columns with design SFRM thickness, and the dashed lines denote columns with maintained SFRM thickness. The percentage increase in capacity in calculated by taking the difference in the capacity of the column considered and the base column at a particular temperature and dividing this difference by the capacity of the base column at that temperature (and multiplying by

100). The figure indicates that increasing column weight does provide some load capacity benefit for all fire durations and columns considered. The general trend is that a 20% increase in weight with maintained SFRM provides the most benefit (approximately 30-70% capacity increase at 2-4 hours), and a 10% increase in weight with design SFRM provides the least benefit (approximately 5-20% capacity increase at 2-4 hours). As previously noted, a 20% increase in weight with design SFRM performs similarly to a 10% increase in weight with maintained SFRM thickness, and both provide approximately a 15-40% capacity increase at 2-4 hours. The figure also shows that a 10% increase in column weight with design SFRM is most effective for the Light column group (approximately 10-20% benefit), and least effective for the Heavy column group (approximately 5-10% benefit). When the SFRM thickness is maintained, the Heavy group performs much better. This seems to indicate that a 10% weight increase in weight may be an appropriate design consideration for light and medium weight columns, but not for Heavy columns, unless SFRM thickness in maintained. For Heavy columns, it may simply be more effective to increase SFRM thickness rather than weight, although more research is needed on this topic.

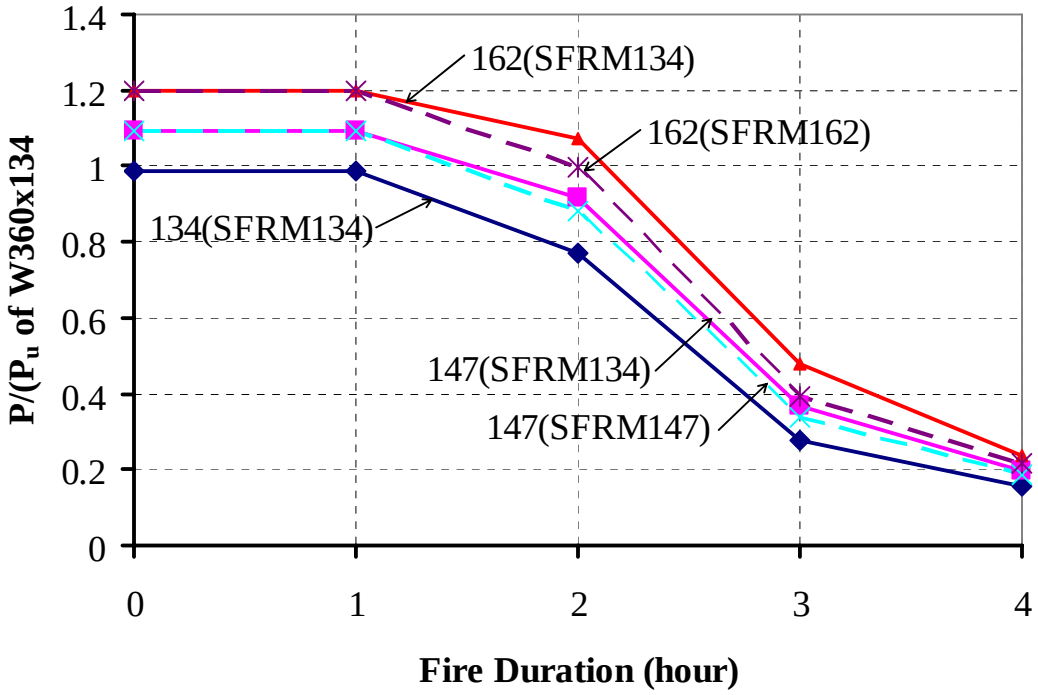


Figure 7.1: Normalized load vs. time for the **Light** column group

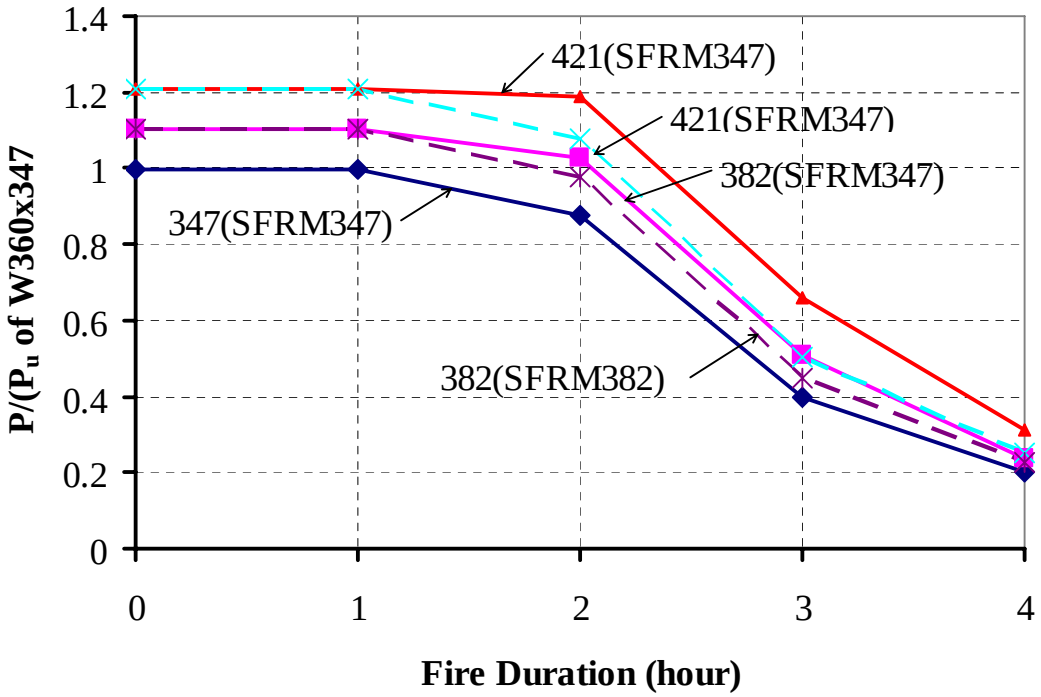


Figure 7.2: Normalized load vs. time for the **Medium** column group

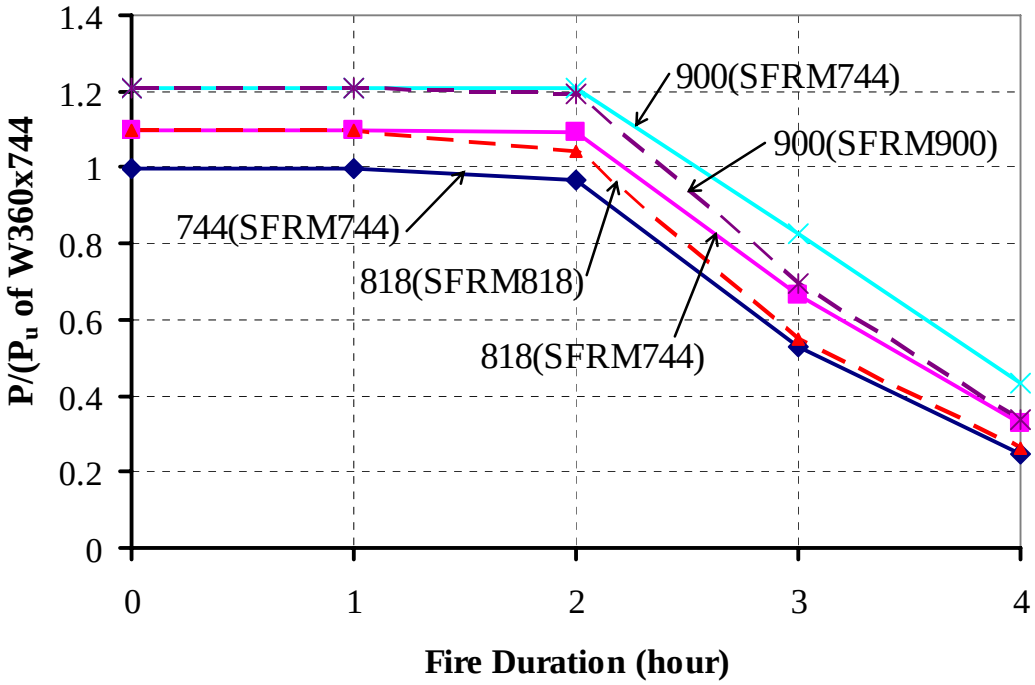


Figure 7.3: Normalized load vs. time for the **Heavy** column group

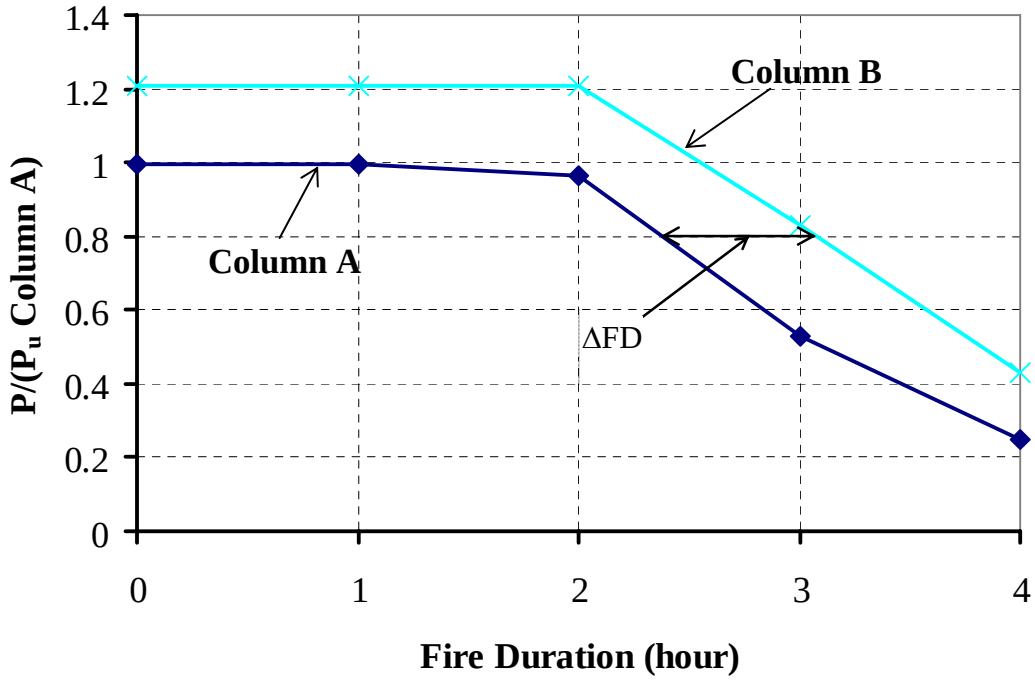


Figure 7.4: Obtaining ΔFD from normalized load vs. time curve

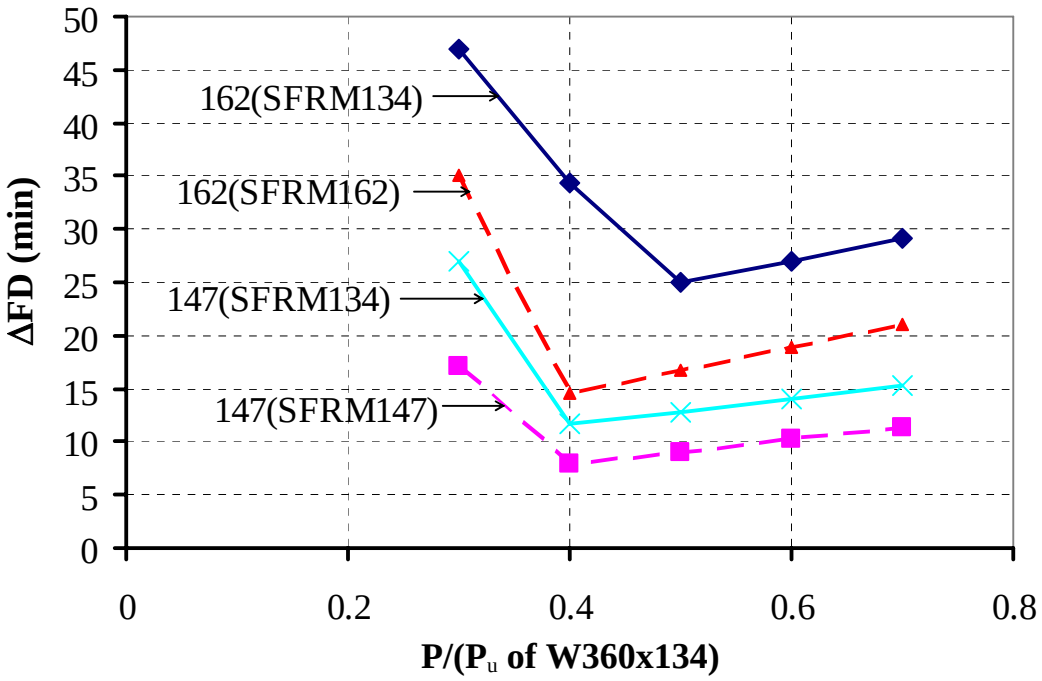


Figure 7.5: ΔFD vs. P/P_u for the **Light** column group

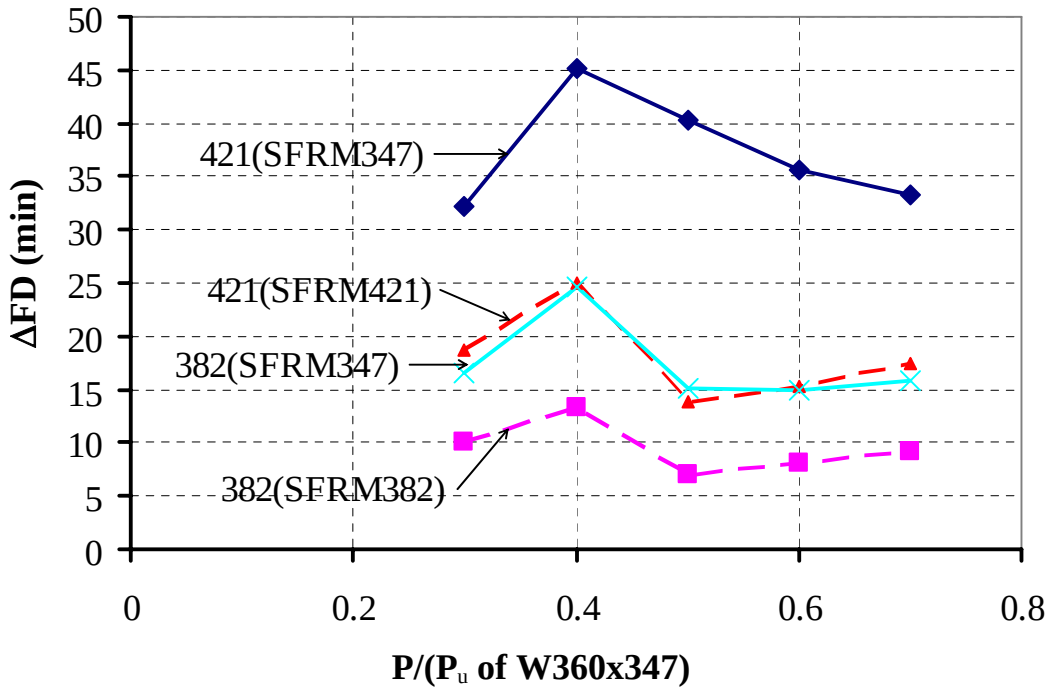


Figure 7.6: ΔFD vs. P/P_u for the **Medium** column group

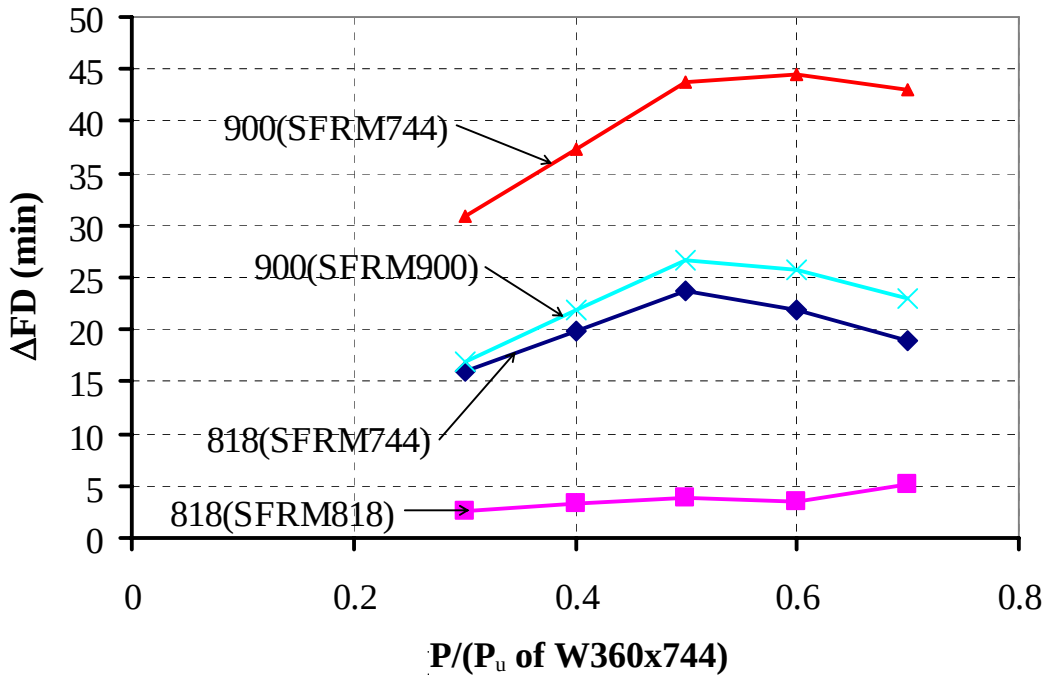
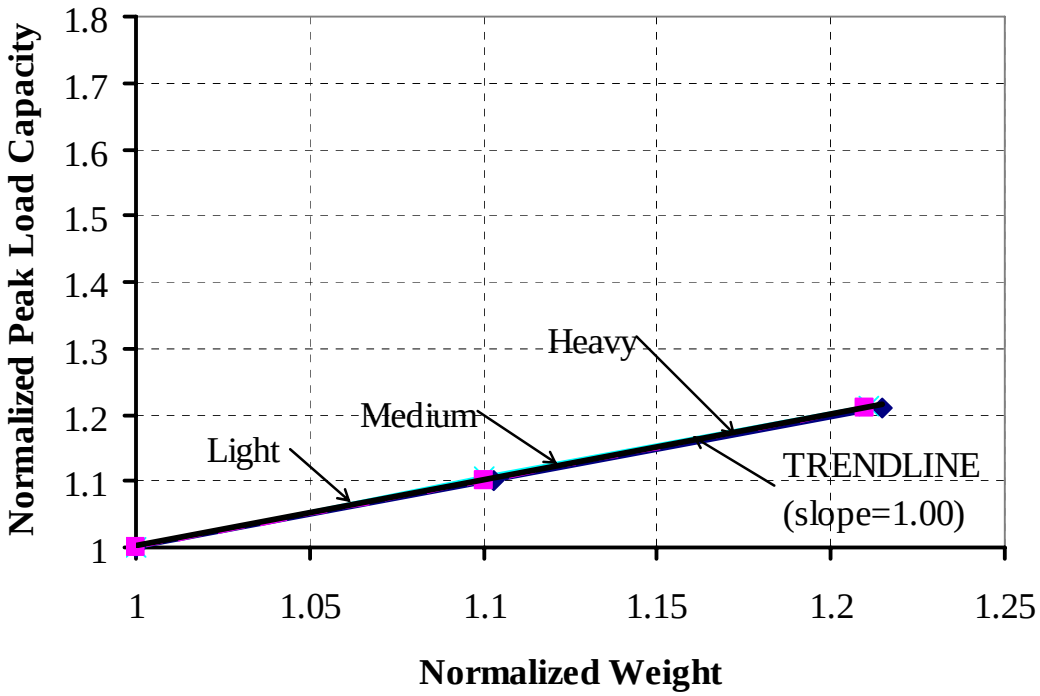
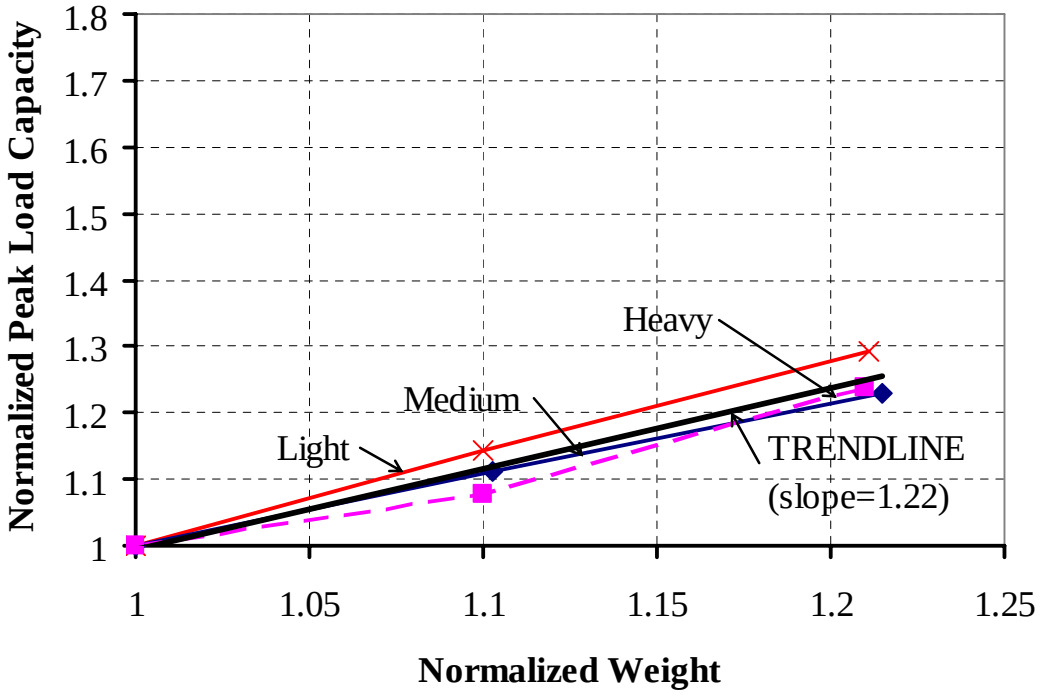


Figure 7.7: ΔFD vs. P/P_u for the **Heavy** column group

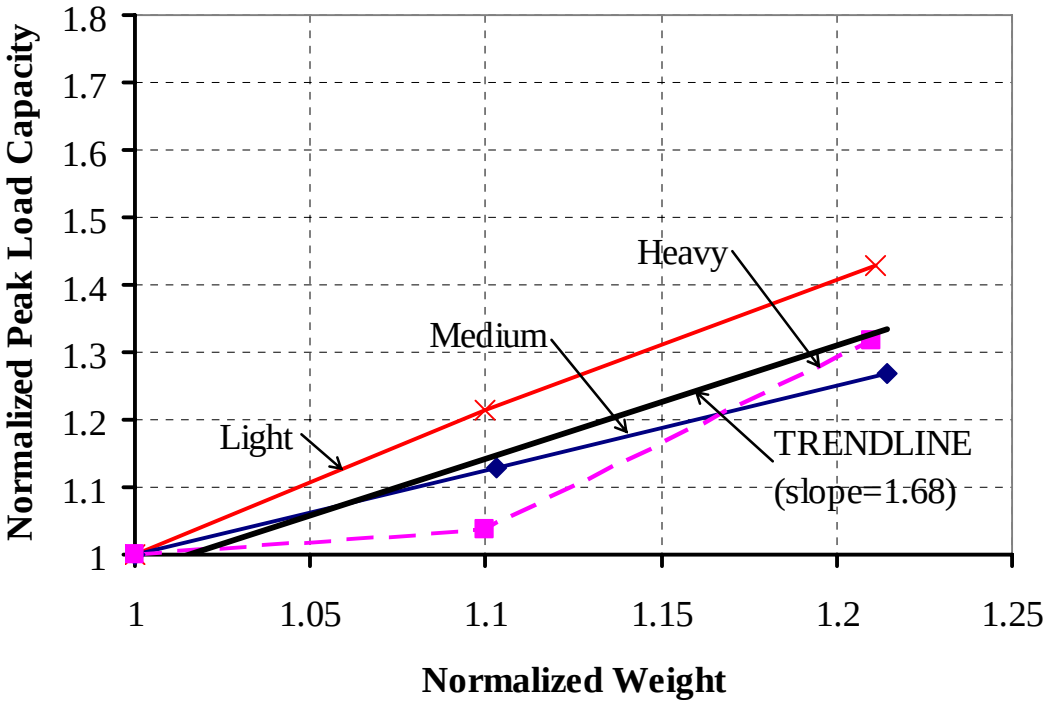


(a)

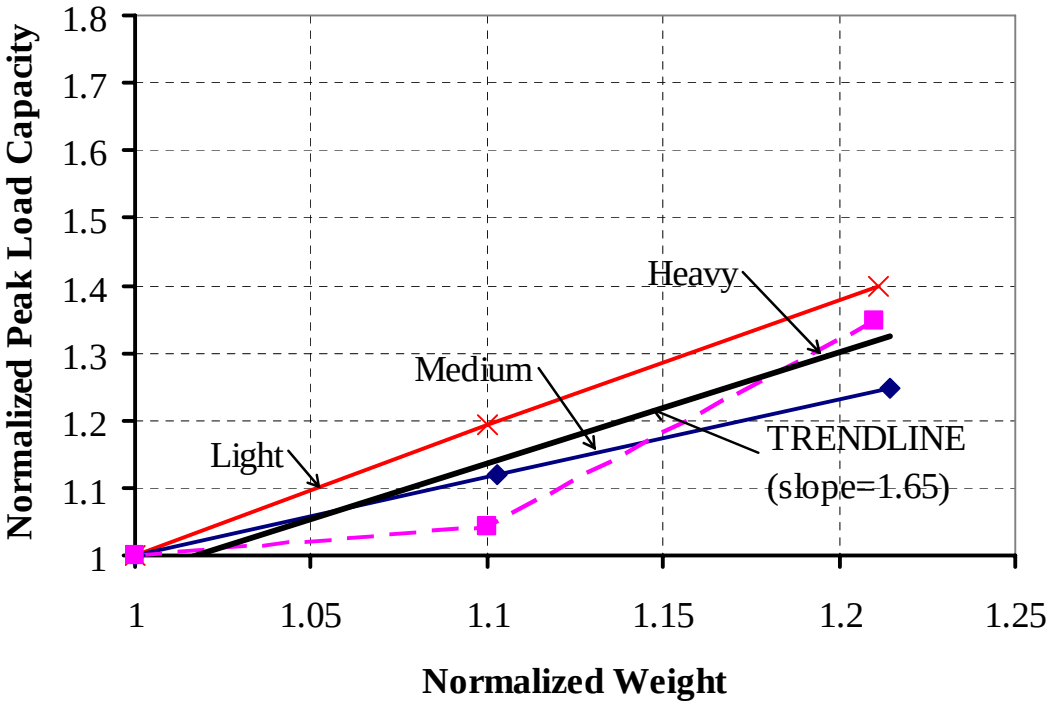


(b)

Figure 7.8: Normalized peak load capacity vs. normalized weight for columns with design SFRM thickness at: (a) 1 hour; (b) 2 hours; [continued]

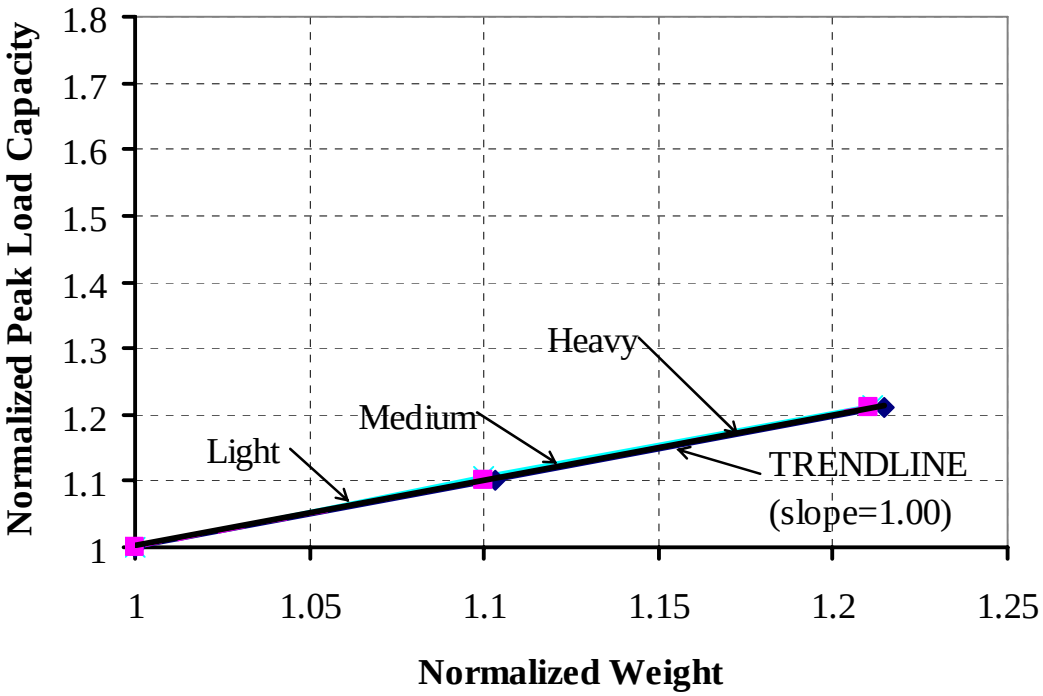


(c)

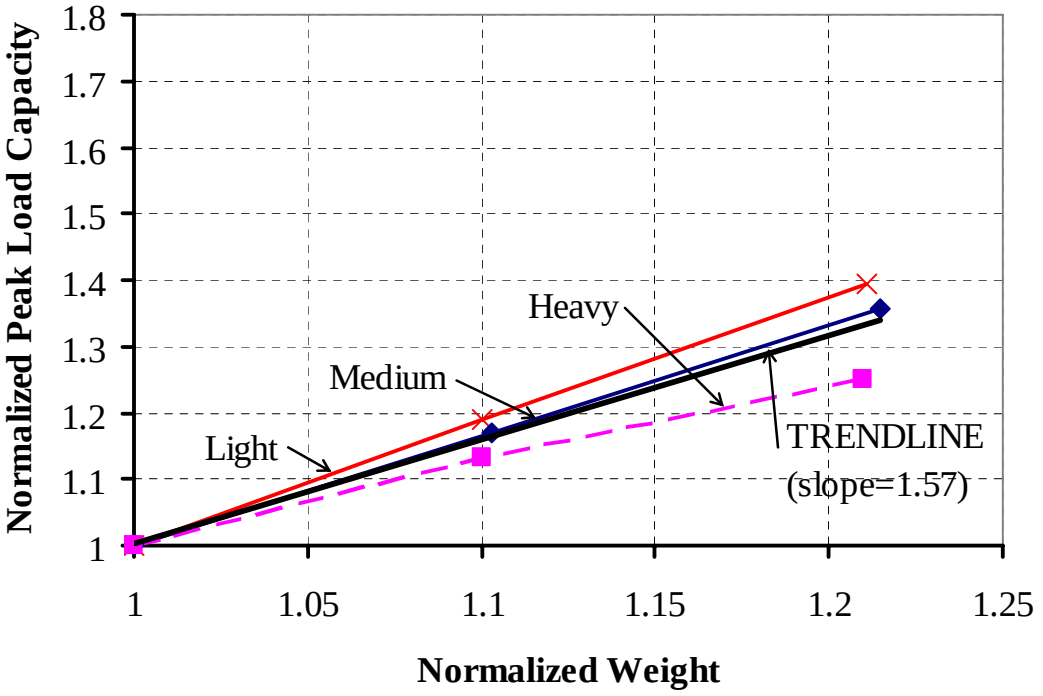


(d)

Figure 7.8: [continued] Normalized peak load capacity vs. normalized weight for columns with design SFRM thickness at: (c) 3 hours; and (d) 4 hours

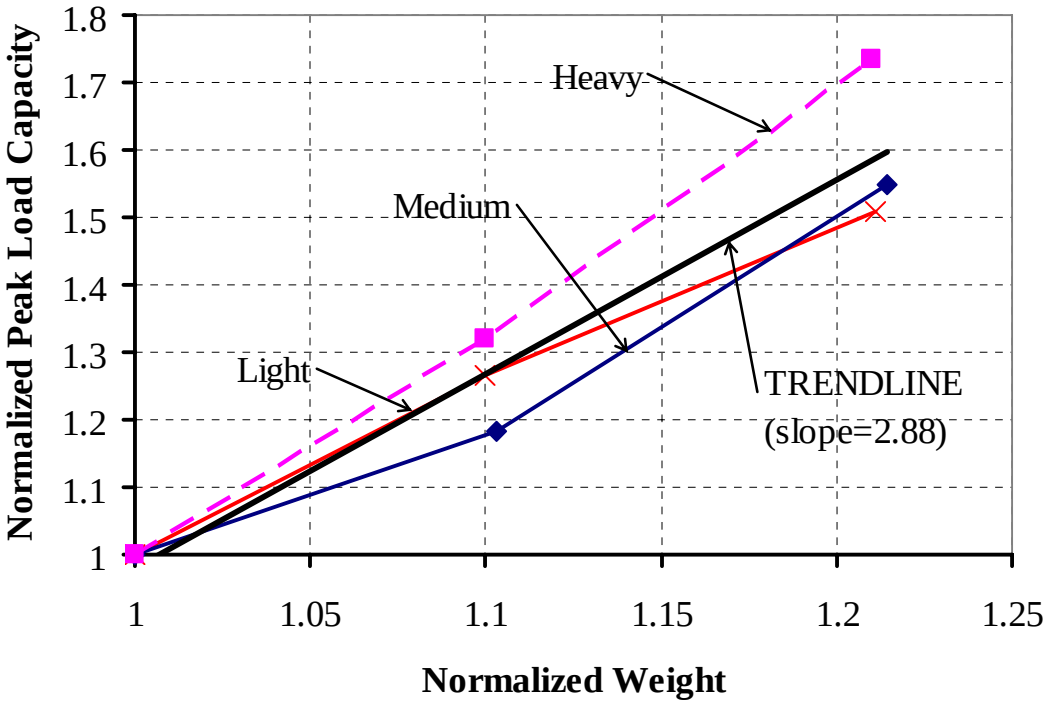


(a)

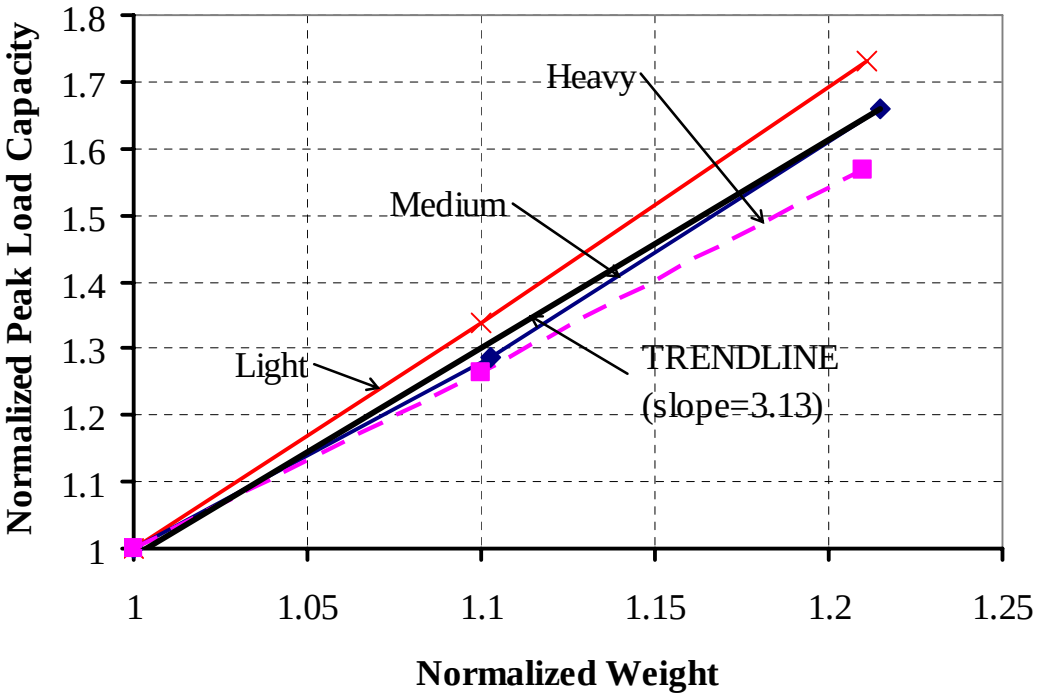


(b)

Figure 7.9: Normalized peak load capacity vs. normalized weight for columns with maintained SFRM thickness at: (a) 1 hour; (b) 2 hours; [continued]



(c)



(d)

Figure 7.9: [continued] Normalized peak load capacity vs. normalized weight for columns with maintained SFRM thickness at: (c) 3 hours; and (d) 4 hours

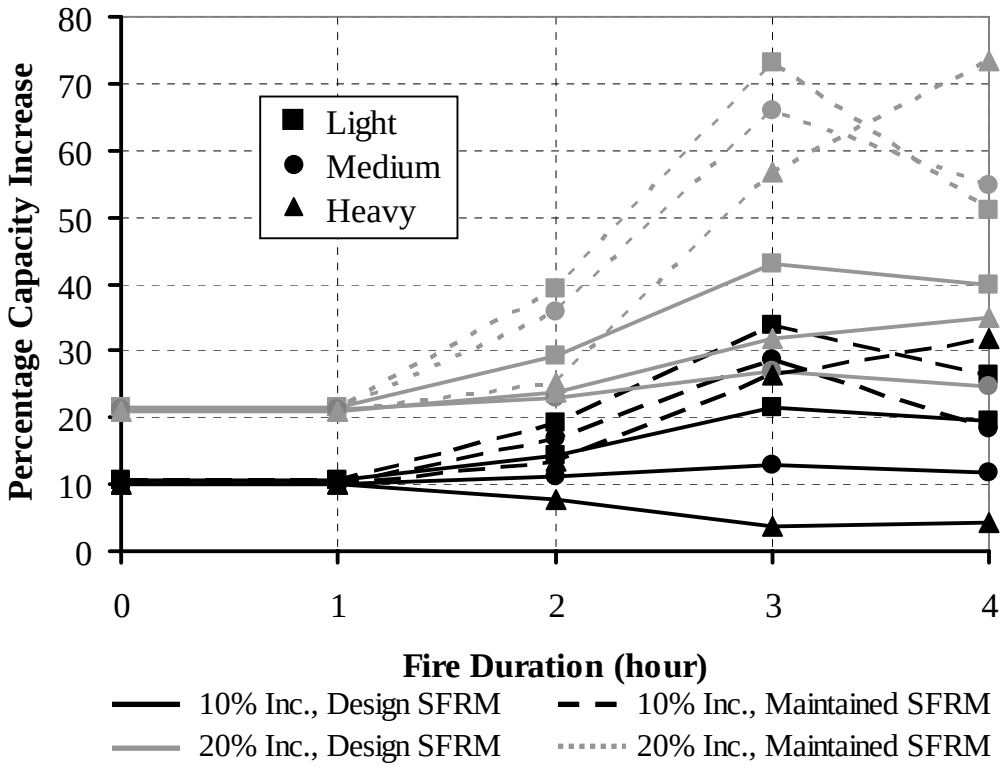


Figure 7.10: Increase in capacity (as measured from the base column analysis at a given fire duration time) versus fire duration time

CHAPTER 8

CONCLUSIONS

8.1 Introduction

The objective of the research described in this report was to probe the supposition that small increases in column weight (which do not necessarily lead to large increases in associated construction costs) can lead to substantial increases in fire duration of the given column. The supposition was probed by performing sequential nonlinear heat transfer and structural analyses of a series of columns to determine their capacities when exposed for varying times to the temperatures associated with ASTM E119 standard fire.

8.2 Heat Transfer Conclusions

Conclusions from the heat transfer portion of the research are as follows:

1) As expected, thickness of SFRM plays an important role in the evolution of temperature within the structural steel. As an example, for the W360x162 analyses, there was a 30°C temperature difference in the 3 hour fire between the analysis case (162(SFRM162))with the design SFRM thickness for the column (39.7 mm), and the analysis case (162(SFRM 134)) with the SFRM thickness (42.9 mm) maintained from the base column.

2) In any particular column, there is only a slight difference in temperature between the temperature at the center of the web or at the flange tips. For the Light and Medium column series, the maximum temperatures were at the center of the web; for the Heavy series the maximum temperatures were at the flange tips.

3) The Medium series columns with design SFRM thickness (i.e. 347(SFRM347), 382(SFRM382) and 421(SFRM421)), have approximately the same temperatures for a given fire duration. Intuitively it would seem that SFRM design thickness variations are specified so as to control the temperatures reached with particular columns and thus the same behavior was expected for the Heavy and Light series columns also. However, for those series, as the columns get heavier the temperatures drop slightly for a given fire duration.

4) At the 1 hour fire duration, the temperatures of all the columns considered in this research remained below 400°C. Furthermore, the temperatures reached in all the columns were essentially similar at that duration. At longer fire durations,

the temperatures in the Light series of columns were higher than those reached in the Medium and Heavy series.

5) As the fire duration increases beyond one hour, the temperatures reached in Light columns are significantly higher than those reached in Heavy columns.

8.3 Structural Analysis Conclusions

Conclusions from the structural analysis portion of the research are as follows:

1) For all columns considered at all fire durations considered, increasing the column weight 10% and providing design SFRM on the new column did provide some benefit (approximately 5-20% increased capacity over the appropriate base column depending on the column and fire duration considered). The benefit was greatest for the Light and Medium column groups and small for the Heavy column group.

2) For all columns considered at all fire durations considered, increasing the column weight 10% and maintaining SFRM on the new column provided a substantial benefit in all cases (approximately 10-30% increased capacity over the appropriate base column depending on the column and fire duration considered).

3) Increasing the column weight 20% provided substantial benefit for all columns considered, regardless of SFRM thickness or fire duration (approximately 25-40% increase in capacity over the appropriate base column depending on the column and fire duration considered). The 20% weight increased columns with maintained SFRM out-performed those with design SFRM due to the additional insulation provided. These columns thus exhibited particularly large (approximately 25-75% increase in capacity) performance gains since they benefited from both extra insulation and increased weight.

4) At 1 hour of fire exposure all of the columns considered in this research reached the full room temperature plastic load capacity of the section. This follows directly from the conclusion noted above that temperatures in all of the columns at this fire duration were below 400°C.

5) At 1 hour fire exposure, the amount of variability in SFRM thickness that was explored herein exhibited no effect on the capacity of the columns considered.

6) For all groups, increasing column size 10% and maintaining SFRM thickness will obtain about the same increase in fire duration as increasing the column size 20% and changing the SFRM thickness to the design value, a gain of 15 to 25 minutes.

7) An increase in capacity in fire can be obtained by increasing column weight, increasing SFRM thickness, or both. An added advantage to increasing column

weight is that the inherent additional strength at room temperature is available for resisting any overload in the column due to other loading conditions.

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