NOTE TO USERS

This reproduction is the best copy available.

UMI®

Reduction in Fire Resistance of Steel Beams Caused by Partial Fire Protection Loss

Submitted by

Yu Kang, B. Eng

South China University of Technology, P.R. China

A thesis submitted to the Faculty of Graduate Studies and
Research in partial fulfillment of the requirements for the
degree of

Master of Applied Science

Ottawa-Carleton Institute for Civil Engineering

Department of Civil and Environmental Engineering

Carleton University

Ottawa, Ontario, Canada

December, 2005

Copyright© Yu Kang, 2005

Library and
Archives Canada

Bibliothèque et
Archives Canada

Published Heritage
Branch

Direction du
Patrimoine de l'édition

395 Wellington Street
Ottawa ON K1A 0N4
Canada

395, rue Wellington
Ottawa ON K1A 0N4
Canada

Your file   Votre référence

ISBN: 0-494-13441-0

Our file   Notre référence

ISBN: 0-494-13441-0

NOTICE:

The author has granted a non-exclusive license allowing Library and Archives Canada to reproduce, publish, archive, preserve, conserve, communicate to the public by telecommunication or on the Internet, loan, distribute and sell theses worldwide, for commercial or non-commercial purposes, in microform, paper, electronic and/or any other formats.

The author retains copyright ownership and moral rights in this thesis. Neither the thesis nor substantial extracts from it may be printed or otherwise reproduced without the author's permission.

AVIS:

L'auteur a accordé une licence non exclusive permettant à la Bibliothèque et Archives Canada de reproduire, publier, archiver, sauvegarder, conserver, transmettre au public par télécommunication ou par l'Internet, prêter, distribuer et vendre des thèses partout dans le monde, à des fins commerciales ou autres, sur support microforme, papier, électronique et/ou autres formats.

L'auteur conserve la propriété du droit d'auteur et des droits moraux qui protègent cette thèse. Ni la thèse ni des extraits substantiels de celle-ci ne doivent être imprimés ou autrement reproduits sans son autorisation.

In compliance with the Canadian Privacy Act some supporting forms may have been removed from this thesis.

Conformément à la loi canadienne sur la protection de la vie privée, quelques formulaires secondaires ont été enlevés de cette thèse.

While these forms may be included in the document page count, their removal does not represent any loss of content from the thesis.

Bien que ces formulaires aient inclus dans la pagination, il n'y aura aucun contenu manquant.

Canada

ABSTRACT

Spray-on fire protection is used widely to improve the fire resistance of steel beams. As this protective coating may be damaged during the service life of the beams, it is essential that the effect of this damage on the fire resistance of a steel beam be understood. A numerical study has been conducted to investigate the reduction in the cross-section moment capacity of protected I-shape steel beams exposed to the ISO 834 standard fire curve due to the partial loss of spray-on protection. Hot-rolled I-shape steel beams according to CAN/CSA-G40.20/G40.21-98 specifications were used in this study.

The results of the study indicate that the cross-section moment capacity reduction due to partial protection loss strongly depends on the area of the protection damage. In addition, the reduction is also affected by other factors such as damage shape, damage location, depth of protection loss and weight of steel section.

ACKNOWLEDGEMENTS

I would like to thank Dr. Heng Aik Khoo and Dr. George V. Hadjisophocleous, supervisors of this thesis, for their guidance, advice and support.

Thanks should also go to Zhiqi Wen and Ruogang Zhao, who gave valuable advice during the research.

This research project is funded by the Natural Sciences and Engineering Research Council of Canada (NSERC).

Last but not the least I would like to thank my parents and my sister for their understanding and encouragement during the course of the study.

Table of Contents

Chapter	Page
Chapter 1 Introduction	1
1.1 Objective of the Thesis and Methodology Used in the Research	1
1.2 Organization of the Thesis	3
Chapter 2 Literature Review.....	4
2.1 Introduction to the Fire Resistance of Steel Structures.....	4
2.1.1 Fire Severity and Standard Fire.....	4
2.1.2 Structural Fire Resistance.....	5
2.1.2.1 Fire-Resistance Rating	5
2.1.2.2 Fire-Resistance Test	6
2.1.2.3 Fire Resistance by Mathematical Analysis	7
2.1.3 Fire Protection Systems for Steel Structures	7
2.2 Heat Transfer Analysis for Steel Structures Exposed to Fire.....	8
2.2.1 An Overview on Heat Transfer Analysis of Steel Structures Exposed to Fire.....	8
2.2.2 Studies on Heat Transfer Analysis of Steel Members Exposed to fire.....	10
2.3 Mechanical Properties of Structural Steel at Elevated Temperatures	14
2.3.1 Stress-Strain Relationship	15
2.3.2 Strain Components	16

2.3.3	Test Methods	16
2.4	Structural Analysis of Steel Beams Exposed to Fire.....	17
2.4.1	Structural Behavior of Steel Beams Exposed to Fire	17
2.4.2	Studies of the Structural Behavior of Steel Beams Exposed to Fire .	19
2.4.3	Simple Methods for Calculating the Moment Resistance of Steel Sections	23
Charter 3	Heat Transfer and Structural Analysis.....	26
3.1	Steel Beam Assemblies	27
3.1.1	Steel Sections.....	27
3.1.2	Fire Protection	28
3.1.3	Studied Steel Beams	29
3.2	Properties of Materials.....	30
3.2.1	Thermal Properties of Materials.....	30
3.2.1.1	Thermal Properties of Steel.....	31
3.2.1.2	Thermal Properties of the Fire Protection Material	32
3.2.2	Mechanical Properties of Steel at Elevated Temperatures	33
3.3	Numerical Heat Transfer Analysis	35
3.3.1	Basic Characteristics of Standard Fire Test Modelling	35
3.3.1.1	Time-Temperature Curve of the Fire.....	35
3.3.1.2	Heat Transfer within the Specimen	36
3.3.1.3	Boundary Conditions	36

3.3.2	Numerical Heat Transfer Models	38
3.3.2.1	Two-Dimensional Finite Element Model.....	38
3.3.2.2	Three-Dimensional Finite Element Model.....	40
3.3.3	Model Validation.....	43
3.4	Structural Analysis by Analytical Calculation	45
3.4.1	Calculation Method	45
3.4.1.1	Analytical Formulas for Cross-Section Moment Capacity Calculation	46
3.4.1.2	Classification of Cross-sections	50
3.4.2	Implementation of the Calculation Method.....	51
3.5	Sequentially Coupled Thermal-Stress Analysis.....	52
3.5.1	Description of the Numerical Model	53
3.5.2	Mesh Study	54
Charter 4	Results and Discussions	85
4.1	Moment Capacities by Analytical Calculation	85
4.1.1	Moment Capacity at the Mid-Span	85
4.1.1.1	W410×74 Beams.....	87
4.1.1.2	Beams of Different Sections	97
4.1.2	Cross-Section Moment Capacity along the Length	102
4.1.3	Temperature-Dependent Thermal Conductivity of the Fire Protection Material	103

4.2	Maximum Mid-Span Moment by Numerical Modelling	105
Charter 5	Summary, Conclusions, and Recommendations	142
5.1	Summary	142
5.2	Conclusions.....	144
5.3	Recommendations.....	146
References	.....	149
Appendix A: Finite Element Buckling Analysis of Columns and Plates.....		155

List of Tables

Table	Page
3.1 Geometrical properties of steel sections	56
3.2 Thickness of fire protection	56
3.3 Beams of W410×74 section.....	57
3.4 Beams of W410×60 section.....	59
3.5 Beams of W310×60 section.....	59
3.6 Beams of W310×39 section.....	59
3.7 Beams of W250×24 section.....	60
3.8 Thermal properties of spray-on mineral fiber material, from ECCS (1995) .	60
3.9 Thermal properties of spray-on fire protection, from Ryder et al. (2002).....	60
3.10 Reduction factors for stress-strain relationship of steel at elevated temperatures, from EC3 (1995)	61
3.11 Section classification for Grade 350 steel at normal temperatures.....	62
3.12 Values of the constant c for Grade 350 steel.....	62
3.13 Section classes of the studied steel sections at normal temperatures	62
4.1 Relative cross-section moment capacities and relative fire resistances for W410×74 beams	108
4.2 Relative cross-section moment capacities and relative fire resistances for W410×60 beams	109
4.3 Relative cross-section moment capacities and relative fire resistances for	

	W310×60 beams	109
4.4	Relative cross-section moment capacities and relative fire resistances for	
	W310×39 beams	110
4.5	Relative cross-section moment and relative fire resistances for W250×24	
	beams	110
A.1	Geometrical properties and buckling strength of columns	157
A.2	Geometrical properties and buckling strength of plates	157

List of Figures

Figure	Page
2.1 Flow chart for calculating the load carrying capacity of a structure exposed to fire (modified from Buchanan 2001).....	25
2.2 Stress-strain relationships for mild structural steel ASTM A36 (modified from Malhotra 1982).....	26
3.1 Steel section	63
3.2 Steel section uniformly protected with spray-on material	63
3.3 Symmetric protection loss on the flange (1BF1-5, 1BF8-11, 2BF1-5, 2BF8-11, 1TF1-5, 1TF8-11, 2TF1-5, 2TF8-11)	64
3.4 Asymmetric protection loss on the flange (1BF6, 2BF6, 1TF6, 2TF6)	64
3.5 Asymmetric protection loss on the flange (1BF7, 2BF7, 1TF7, 2TF7)	65
3.6 Protection loss on the web, symmetric to mid-height (1W1, 2W1)	65
3.7 Protection loss on the web, below mid-height (1W2, 2W2).....	66
3.8 A simply supported beam with protection loss at the mid-span	66
3.9 Specific heat of steel as a function of temperature (generated from EC3 1995).....	67
3.10 Thermal conductivity of steel as a function of temperature (generated from EC3 1995).....	67
3.11 Thermal conductivity of a spray-on fire protection material as a function of temperature (generated from Ryder et al. 2002).....	68

3.12	Stress-strain relationship for steel at elevated temperatures (modified from EC3 1995).....	68
3.13	Reduction factors for the stress-strain relationship of steel at elevated temperatures (generated from EC3 1995).....	69
3.14	Yield strength of structural steel S355 at elevated temperatures (modified from Outinen et al. 2001).....	69
3.15	Modulus of elasticity of structural steel at elevated temperatures (modified from Outinen et al. 2001).....	70
3.16	Time-temperature curve of ISO 834 standard fire.....	70
3.17	Structure and boundary conditions of a two-dimensional heat transfer Model.....	71
3.18	Four meshes for the two-dimensional heat transfer models	72
3.19	Comparison of predicted temperature distributions on the lower surface of the bottom flange for two-dimensional models with different meshes	73
3.20	Three-dimensional finite element heat transfer model of a beam with the bottom flange protection loss.....	73
3.21	Longitudinal spacing of three meshes for the three-dimensional heat transfer Model.....	74
3.22	Selected locations in W410×74 section for the longitudinal mesh study	75
3.23	Comparison of predicted longitudinal temperature distributions of selected locations between different meshes	76

3.24	Predicted longitudinal temperature distributions after 2 hours standard fire exposure for a W410×74 beam subjected to partial protection loss	77
3.25	Comparison of predicted longitudinal temperature distributions of selected locations between different model lengths	78
3.26	Average temperature of a W10×49 steel column with 19 mm of protection.	79
3.27	Average temperature of an H-300×300×10×15 steel column with 22.3 mm of protection.....	79
3.28	Temperatures of the bottom flange (BF), top flange (BF) and web for an H-400×200×8×13 steel beam with 11.3 mm of protection.....	80
3.29	Strain and stress distribution in a steel beam cross-section	80
3.30	Flow chart for calculating the moment capacity of steel sections	81
3.31	Boundary and loading conditions of a structural model	82
3.32	Predicted moment versus displacement relationship at the mid-span of a W310×39 steel beam after 40 minutes standard fire exposure	82
3.33	Longitudinal spacing of three meshes for the sequentially coupled thermal-stress models	83
3.34	Predicted moment versus displacement relationships at the mid-span for different meshes.....	84
4.1	Predicted relative cross-section moment capacities for unprotected and protected W410×74 beams	111
4.2	Predicted relative cross-section moment capacities for 1-hour fire	

	resistance rating W410×74 beams with full protection or with symmetric bottom flange protection loss.....	111
4.3	Predicted relative cross-section moment capacities for 2-hour fire resistance rating W410×74 beams with full protection or with symmetric bottom flange protection loss.....	112
4.4	Temperature distribution contour of W410×74 beam 1BF3 after 1 hour of standard fire exposure.....	113
4.5	Predicted relative cross-section moment capacities for W410×74 beams 2BF4 and 2BF8.....	114
4.6	Predicted relative cross-section moment capacities for W410×74 beams 1BF4 and 1BF8.....	114
4.7	Relative fire resistances of 1-hour fire resistance rating W410×74 beams with full protection or with symmetric bottom flange protection loss	115
4.8	Relative fire resistances of 2-hour fire resistance rating W410×74 beams with full protection or with symmetric bottom flange protection loss	115
4.9	Predicted relative cross-section moment capacities for 2-hour fire resistance rating W410×74 beams with symmetric or asymmetric bottom flange protection loss.....	116
4.10	Predicted relative cross-section moment capacities for 1-hour fire resistance rating W410×74 beams with symmetric or asymmetric bottom flange protection loss.....	116

4.11	Predicted temperature profiles along the bottom flange for beams 2BF2 and 2BF6 after 40 minutes of standard fire exposure.....	117
4.12	Predicted temperature profiles along the web for beams 2BF2 and 2BF6 after 80 minutes of standard fire exposure.....	117
4.13	Relative fire resistances of 1-hour fire resistance rating W410×74 beams with full protection or with symmetric or asymmetric bottom flange protection loss.....	118
4.14	Relative fire resistances of 2-hour fire resistance rating W410×74 beams with full protection or with symmetric or asymmetric bottom flange protection loss.....	118
4.15	Fire resistance reductions of 1-hour fire resistance rating W410×74 beams with the bottom flange protection loss.....	119
4.16	Fire resistance reductions of 2-hour fire resistance rating W410×74 beams with the bottom flange protection loss.....	120
4.17	Predicted relative cross-section moment capacities for 1-hour fire resistance rating W410×74 beams with protection loss on the bottom flange or on the web.....	121
4.18	Predicted relative cross-section moment capacities for 2-hour fire resistance rating W410×74 beams with protection loss on the bottom flange or on the web.....	121
4.19	Relative fire resistances of 1-hour fire resistance rating W410×74 beams	

	with the bottom flange or web protection loss.....	122
4.20	Relative fire resistances of 2-hour fire resistance rating W410×74 beams with the bottom flange or web protection loss.....	122
4.21	Predicted relative cross-section moment capacities for 1-hour fire resistance rating W410×74 beams with various depths of the bottom flange protection loss	123
4.22	Predicted relative cross-section moment capacities for 2-hour fire resistance rating W410×74 beams with various depths of the bottom flange protection loss.....	123
4.23	Predicted relative cross-section moment capacities for 1-hour fire resistance rating W410×74 beams with an 87.5% penetrated bottom flange protection loss.....	124
4.24	Predicted relative cross-section moment capacities for 2-hour fire resistance rating W410×74 beams with an 87.5% penetrated bottom flange protection loss.....	124
4.25	Relative fire resistances of 1-hour fire resistance rating W410×74 beams with various depths of the bottom flange protection loss	125
4.26	Relative fire resistances of 2-hour fire resistance rating W410×74 beams with various depths of the bottom flange protection loss	125
4.27	Relative fire resistances of 1-hour fire resistance rating W410×74 beams with an 87.5% penetrated bottom flange protection loss.....	126

4.28	Relative fire resistances of 2-hour fire resistance rating W410×74 beams with an 87.5% penetrated bottom flange protection loss.....	126
4.29	Predicted relative cross-section moment capacities for 1-hour fire resistance rating fully protected beams of different sections.....	127
4.30	Predicted relative cross-section moment capacities for 2-hour fire resistance rating fully protected beams of different sections.....	127
4.31	Predicted relative cross-section moment capacities for 1-hour fire resistance rating W310×39 beams with protection loss on the bottom or top flange	128
4.32	Predicted relative cross-section moment capacities for 2-hour fire resistance rating W310×39 beams with protection loss on the bottom or top flange	128
4.33	Predicted relative cross-section moment capacities for 1-hour fire resistance rating W310×60 beams with protection loss on the bottom or top flange	129
4.34	Predicted relative cross-section moment capacities for 2-hour fire resistance rating W310×60 beams with protection loss on the bottom or top flange	129
4.35	Predicted relative cross-section moment capacities for 1-hour fire resistance rating W410×60 beams with protection loss on the bottom or top flange	130

4.36	Predicted relative cross-section moment capacities for 2-hour fire resistance rating W410×60 beams with protection loss on the bottom or top flange	130
4.37	Predicted relative cross-section moment capacities for 1-hour fire resistance rating W250×24 beams with protection loss on the bottom or top flange	131
4.38	Predicted relative cross-section moment capacities for 2-hour fire resistance rating W250×24 beams with protection loss on the bottom or top flange	131
4.39	Predicted modified relative cross-section moment capacities for 1-hour fire resistance rating W410×60 and W310×60 beams with protection loss on the bottom flange	132
4.40	Predicted modified relative cross-section moment capacities for 2-hour fire resistance rating W410×60 and W310×60 beams with protection loss on the bottom flange	132
4.41	Predicted modified relative cross-section moment capacities for 1-hour fire resistance rating W410×74 and W410×60 beams with protection loss on the bottom flange	133
4.42	Predicted modified relative cross-section moment capacities for 2-hour fire resistance rating W410×74 and W410×60 beams with protection loss on the bottom flange	133

4.43	Predicted modified relative cross-section moment capacities for 1-hour fire resistance rating W310×60 and W310×39 beams with protection loss on the bottom flange	134
4.44	Predicted modified relative cross-section moment capacities for 2-hour fire resistance rating W310×60 and W310×39 beams with protection loss on the bottom flange	134
4.45	Predicted modified relative cross-section moment capacities after 1 hour of standard fire exposure for 1-hour fire resistance rating W410×74 beams with symmetric bottom flange protection loss.....	135
4.46	Predicted modified relative cross-section moment capacities after 2 hours of standard fire exposure for 2-hour fire resistance rating W410×74 specimens with symmetric bottom flange protection loss.....	135
4.47	Predicted relative cross-section moment capacities for 1-hour fire resistance rating fully protected W410×74 beams with constant and varied thermal conductivities of the protective material	136
4.48	Predicted relative cross-section moment capacities for 2-hour fire resistance rating fully protected W410×74 beams with constant and varied thermal conductivities of the protective material	136
4.49	Predicted relative cross-section moment capacities for 1-hour fire resistance rating W410×74 beams with symmetric bottom flange protection loss and with constant and varied protective material thermal	

	conductivities.....	137
4.50	Predicted relative cross-section moment capacities for 2-hour fire resistance rating W410×74 beams with symmetric bottom flange protection loss and with constant and varied protective material thermal conductivities.....	137
4.51	Relative fire resistances of 1-hour rating W410×74 beams with symmetric bottom flange protection loss at the mid-span and with constant and varied protective material thermal conductivities.....	138
4.52	Relative fire resistances of 2-hour rating W410×74 beams with symmetric bottom flange protection loss at the mid-span and with constant and varied protective material thermal conductivities.....	138
4.53	Predicted relative cross-section moment capacities for a 1-hour fire resistance rating fully protected W310×39 beam by analytical calculation and numerical modelling	139
4.54	Predicted relative cross-section moment capacities for 1-hour fire resistance rating W310×39 beams with symmetric bottom or top flange protection loss by analytical method and numerical modelling	139
4.55	Deformed shape of a failed fully protected W310×39 beam at 40 minutes of standard fire exposure.....	140
4.56	Deformed shape of a failed W310×39 beam with symmetric bottom flange protection loss at 40 minute of standard fire exposure	140

4.57	Deformed shape of a failed W310×39 beam with symmetric top flange protection loss at 40 minute of standard fire exposure	141
A.1	Dimensions, loading condition and boundary conditions of columns and plates	158
A.2	Predicted maximum average stress for columns with various slenderness ratios.....	158
A.3	Deformed shape and normal stress components of column C100-50	159
A.4	Predicted average cross-section stress versus displacement relationship for columns C50-50 and C100-50	160
A.5	Deformed shape of a plate with one unloaded edge free and the other one pinned (Deformation scale factor: 2.5).....	160
A.6	Deformed shape of a plate with one unloaded edge free and the other one fixed (Deformation scale factor: 10.0).....	161
A.7	Predicted average cross-section stress versus displacement relationships for plates with one unloaded edge free and the other one pinned.....	161
A.8	Predicted average cross-section stress versus displacement relationships for plates with one unloaded edge free and the other one fixed	162

List of Symbols

a	- distance between the web and the flange tip of a steel section
a_l	- length of the protection loss area
A	- cross-section area
A_i	- area of element i
A_L	- area of the protection loss
b	- flange width of a steel section
b_e	- effective width of a plate
b_l	- width of the protection loss area
c	- specific heat, or a constant for the effective width calculation
c_i	- specific heat of protection material
c_s	- specific heat of steel
C_1, C_2	- material constants for a specific fire protection material
d	- depth of a steel section
d_i	- thickness of the fire protection
D	- heated perimeter per unit length
E	- modulus of elasticity of steel at normal temperature
$E_{a,T}$	- slope of the linear elastic range
$f_{p,T}$	- proportional limit of steel at elevated temperatures
f_y	- yield strength of steel at normal temperature
$f_{y,T}$	- effective yield strength of steel at elevated temperatures

F_r	- fire resistance reduction
h_c	- convection coefficient
k	- buckling coefficient at normal temperature
k_T	- buckling coefficient at elevated temperatures
$k_{E,T}$	- reduction factor of the elastic modulus for steel at elevated temperatures
$k_{E,T,0}$	- reduction factor for the elastic modulus of the furthest steel fiber
$k_{E,T,i}$	- reduction factor of the elastic modulus for steel at temperature T_i
$k_{y,T,0}$	- reduction factor for the steel yield stress of the furthest steel fiber
$k_{y,T,i}$	- reduction factor of the steel yield stress for steel at temperature T_i
M	- cross-section bending moment
M_0	- cross-section moment capacity at normal temperature
$M_{cr,f}$	- moment capacity governed by elastic local buckling for a cross-section with a non-uniform temperature distribution
M_f	- cross-section moment capacity at elevated temperatures
$M_{f,F}$	- cross-section moment capacity of a fully protected beam at elevated temperatures
$M_{p,f}$	- plastic moment of a cross-section with a non-uniform temperature distribution
M_r	- relative moment capacity
M_y	- yield moment at normal temperature

$M_{y,f}$	- yield moment of a cross-section with a non-uniform temperature distribution
M'_r	- modified relative moment capacity
n	- number of elements in a cross-section
q''	- heat flux on the fire-exposed surface
q_c''	- heat flux due to convection
q_r''	- heat flux due to radiation
$\dot{q}''$	- internally generated heat flux
R	- fire resistance
R_m	- cross-section moment capacity reduction
R_r	- relative fire resistance
t	- time, or the flange thickness
T	- temperature
T_0	- normal temperature
T_f	- fire temperature
T_s	- steel temperature
w	- web thickness
W	- weight of a steel section per unit length
y	- distance to the neutral axis
y_i	- distance from the neutral axis to the centroid of the element i
Y_0	- distance from the neutral axis to the yielded fiber or to the buckled

flange

ϵ	- strain
$\epsilon_{cr,T,0}$	- buckling strain of the flange at elevated temperatures
ϵ_f	- flame emissivity
$\epsilon_{p,T}$	- strain at the proportional limit of steel at elevated temperatures
ϵ_{res}	- resultant emissivity
ϵ_s	- surface emissivity
$\epsilon_{t,T}$	- limiting strain for yield strength of steel at elevated temperatures
$\epsilon_{u,T}$	- ultimate strain of steel at elevated temperatures
ϵ_y	- yield strain of steel at normal temperature
$\epsilon_{y,T}$	- yield strain of steel at elevated temperatures
$\epsilon_{y,T,0}$	- yield strain of the furthest steel fiber
Φ	- view factor
λ	- thermal conductivity
λ_i	- thermal conductivity of the fire protection material
λ_s	- thermal conductivity of steel
$\lambda_x, \lambda_y, \lambda_z$	- thermal conductivities in x, y, z directions
ρ	- density
ρ_i	- density of the fire protection material
ρ_s	- density of steel
σ	- stress, or the Stefan-Boltzmann constant

- σ_{cr} -   elastic buckling stress of a steel plate at normal temperature
- $\sigma_{cr,T}$ -   elastic buckling stress of a steel plate at elevated temperatures
- $\sigma_{cr,T,0}$ -   buckling stress of the flange at elevated temperatures
- σ_i -   average stress of element i

Chapter 1 Introduction

The strength and stiffness of steel decrease significantly at elevated temperatures. In a fire exposure, the temperature of unprotected steel members increases rapidly due to the high thermal conductivity of steel. As a result, unprotected steel structures do not have desirable resistance against fire.

Spray-on insulation coatings are frequently used to protect steel structures against fire. Spray-on fire protection materials have thermal properties that reduce heat transfer from the hot gas to the steel and hence lower the rate of steel temperature increase. However, spray-on materials are often rather soft, and the protective coatings can be easily damaged during service, resulting in a possible reduction of the fire resistance of the steel member that is being protected.

The fire resistance of a protected steel member can be assessed by fire tests or calculation methods. A few numerical studies have been carried out to investigate the effect of spray-on protection loss on the fire resistance of steel columns. These studies focused on local steel temperature rise at the damaged region, using a temperature limit as the criterion of fire resistance. However, the fire resistance of steel beams subjected to partial fire protection damage has not yet been investigated. Moreover, in order to obtain a more realistic assessment of the fire resistance of steel members, the investigation should look into the actual load-carrying capacity and deflection of the member. This study only looks at the reduction in load-carrying capacity.

1.1 Objective of the Thesis and Methodology Used in the Research

The objective of this study is to investigate the reduction of the fire resistance of protected steel beams caused by partial loss of spray-on fire protection. The study is carried out using computer-based analyses. Each analysis consists of two stages: the heat transfer analysis and the structural analysis. The heat transfer analysis simulates the temperature rise of steel beams subjected to the standard fire exposure, and the structural analysis calculates the load-carrying capacity of steel beams at the resulting temperature.

Beams studied are hot-rolled I-shape steel beams protected with spray-on mineral fiber fire protection. To model fire protection loss, a small piece of fire protection is removed from a selected location of a beam. A series of heat transfer analyses are conducted to simulate the temperature rise of beams exposed to the ISO 834 standard fire. Models for heat transfer analyses are developed and carried out using the finite element software ABAQUS. Thermal properties of steel and the fire protection are applied to the models, and a user-defined subroutine is developed to calculate the heat flux on the specimen boundary. The heat transfer models are validated with available test data. Moment capacities of steel sections are computed using theoretical flexural formulas based on the temperature profiles obtained from the heat transfer analyses and the reduced material properties of steel at elevated temperatures. These calculations are implemented using Visual Basic programs. In addition, a few numerical sequentially coupled thermal-stress analyses are conducted to study the fire performance of simply

supported steel beams with partial protection loss.

1.2 Organization of the Thesis

Chapter 2 consists of a literature review of fire resistance studies of steel structures, including an introduction on the fire safety of steel structures, heat transfer studies of steel structures, high temperature properties of structural steels and studies on the behaviour of steel beams exposed to fire.

Chapter 3 presents the methodology of the heat transfer and structural analyses. This chapter discusses the beams, and the thermal and mechanical properties of materials used in this study. The numerical heat transfer analysis and the procedure to calculate cross-section moment capacities are described in detail. The numerical coupled thermal-stress analysis of a simply supported beam is also presented.

Chapter 4 discusses cross-section moment capacities of beams exposed to the standard fire. The moment capacity reduction caused by partial protection loss is discussed. In addition, results of the analytical calculation are compared to that of the coupled thermal-stress analysis for a fully braced simply supported beam.

Chapter 5 consists of a summary of the study, conclusions and recommendations.

Chapter 2 Literature Review

Fire poses a severe threat to steel structures. It may cause significant structural damage and endanger human lives. Therefore, an important topic in structural engineering is to evaluate the fire resisting capacity of a steel structure. In evaluating the fire resisting capacity of steel structures, the evolution of steel temperatures in steel structures exposed to fire, the mechanical properties of structural steel at elevated temperatures, and the behaviour of steel structures at elevated temperatures are essential pieces of information that need to be considered.

2.1 Introduction to the Fire Resistance of Steel Structures

When a steel structure is exposed to fire, the steel temperature increases, and the load carrying capacity of the structure decreases. Whether a steel structure is able to survive a fire depends on the fire resistance of the structure and the fire severity to which the structure is exposed. Some basic concepts about the fire resistance of steel structures are introduced in the following sections.

2.1.1 Fire Severity and Standard Fires

Fire severity is a measure of the destructive potential of a fire. Fire severity is usually defined in terms of the period of exposure to the standard test fire. Standard fires are time-temperature curves used in standard fire resistance tests. There are a number of test standards used in different countries, such as international standard

ISO 834 (1975), American standard ASTM E119 (2000), and Canadian standard CAN/ULC-S101 (2004). Lie (1992) compared a number of standard fires and pointed out that the time-temperature curves of most standard fires are similar, especially in the first two hours. On the other hand, the imposed thermal impact in a test environment not only depends on the standard fire curve but is also affected by some other parameters such as the temperature measuring method, the size of furnace, the type of fuel and the furnace lining material (Buchanan, 2001).

2.1.2 Fire Resistance

Fire resistance is a measure of the ability of a building element to resist a fire. It is defined as the structural fire resistance for a structural member. The structural fire resistance can be quantified in the time domain, temperature domain, or strength domain. In other words, it can be quantified as the time that a structural member can withstand a standard fire exposure, the temperature of a structural member after a standard fire exposure, or the load capacity of a structural member after a standard fire exposure. The fire resistance of structural members can be assessed either by fire-resistance tests or by mathematical analyses.

2.1.2.1 Fire-Resistance Rating

Fire-resistance rating represents the time to failure for a building element tested in a standard fire resistance furnace. According to Buchanan (2001), fire-resistance ratings

are listed in three main categories: generic ratings, proprietary ratings, and calculated ratings. Generic and proprietary ratings are obtained from full-scale fire-resistance tests. Generic ratings assign a time of fire resistance to materials with no reference to individual manufacturers or detailed specifications. Proprietary ratings are listings that assign fire resistance to proprietary products made by specific manufactures. Calculated ratings are listings of fire resistance obtained from approved calculation methods.

2.1.2.2 Fire-Resistance Test

In a fire-resistance test, the fire resistance of a structural member is directly measured using a standard test furnace. Test facilities and test methods for a fire-resistance testing are specified by test standards such as ISO 834, ASTM E119 and ULC-S101. The fire resistance of structural members is usually measured by full-scale load-bearing tests. Failure criteria in the tests are defined in terms of deflection limit or deflection rate limit. As an exception, ASTM E119 also allows non-loaded tests on steel columns and steel beams using limiting temperatures as failure criteria. Although the full-scale fire test is the most reliable way for assessing the fire resistance of structural members, its drawbacks are also obvious. Firstly, full-scale fire tests are very expensive and time consuming. Secondly, there are always differences between structural members in real buildings and those being tested. Thirdly, the test fire may be very different from a real fire.

2.1.2.3 Fire Resistance by Mathematical Analysis

Buchanan (2001) published a flow chart for calculating the load carrying capacity of a structure exposed to a complete burnout of a fire compartment, as shown in Figure 2.1. According to this flow chart, the process of calculating the structural fire resistance has three essential components: a fire model, a heat transfer model and a structural model. The fire model describes the scenario of a fire, the heat transfer model predicts the temperature evolution of a structural member under a specific fire exposure, and the structural model calculates the performance of a structural member at elevated temperatures. Predicting fire resistance by mathematical analyses is an economical and flexible complement to fire-resistance tests. For the reliability of the calculated fire resistance, analysis methods should be verified with the results of full-scale fire resistance tests.

2.1.3 Fire Protection Systems for Steel Structures

Passive fire protection systems are widely used to improve the fire resistance of steel structures. They protect the structure by reducing the rate of temperature evolution in steel members exposed to fire. Passive fire protection systems include spray-on systems, board systems, intumescent paint, concrete encasement, concrete filling, water filling, timber protection, and flame shields. Spray-on protection is usually the cheapest among these protection systems. It is often made of cement-based materials reinforced by glass or cellulosic fibres. Spray-on protection can be applied to beams, columns, and

even members with complicated geometries such as bolted connections or steel brackets.

2.2 Heat Transfer Analysis for Steel Structures Exposed to Fire

This section presents an overview on the heat transfer analysis of steel structures exposed to fire. Beside, several heat transfer studies of fire-exposed steel members are discussed.

2.2.1 An Overview on the Heat Transfer Analysis of Steel Structures Exposed to Fire

The objective of the heat transfer analysis is to predict the evolution of steel temperatures during a fire exposure. For simple structural elements, results of the heat transfer analysis can be directly used to assess the element's fire resistance. For more complex structural members or assemblies, these results are used as input data for the subsequent structural analysis.

According to Purkiss (1996), heat transfer problems for fire exposed steel structures consist of heat transfer within the element and heat exchange at the element surface. Heat transfer within the element is basically a process of heat conduction. Thermal properties of materials, such as thermal conductivity, specific heat, density, and latent heat (if it exists) are essential information required in solving this problem. Thermal properties of steel can be found in EC3 (1995) and Lie (1992), and the thermal properties of fire protection materials can be found in ECCS (1995), Lie (1992), and Malhotra (1982). On the other hand, heat exchange at the element boundary is a process

of heat convection and radiation. The heat flux passing through the element boundary is determined by temperature evolution of the fire, radiative characteristics of the fire and the element surface, and convective characteristics of the fire. The convective and radiative characteristics of the ISO 834 standard fire are specified in EC1 (2002), and the radiative characteristics of common construction materials can be found in Purkiss (1996).

Simple procedures are available to calculate the temperature rise of steel members in the standard fire exposure. For example, EC3 (1995) provides lumped mass step-by-step methods to calculate the temperature of protected and unprotected steel members exposed to the ISO 834 standard fire. In these methods, the steel member is assumed to be a lumped mass with uniform temperature over the cross-section. The fire-exposure period is divided into small time steps, and the steel temperature at each step time is calculated using simplified formulas. However, computer-based numerical methods are able to calculate a more realistic non-uniform temperature distribution within the member for any desired fire exposure. Numerical methods for the heat transfer analysis of structural members include finite difference methods and finite element methods. Lie (1992) predicted the temperature distribution over the cross-section of a structural member using a two-dimensional finite difference method. Several finite element programs are available to solve two- and three-dimensional heat transfer problems for fire-exposed steel structures, including ABAQUS, ANSYS, FIRES-T3, SAFIR, etc.

2.2.2 Heat Transfer Studies of Steel Members Exposed to Fire

There have been a few heat transfer studies of steel members exposed to fire. In these studies, two- and three-dimensional heat transfer analyses were carried out mainly using finite difference or finite element methods.

Lie and Stanzak (1973) carried out a study on the fire resistance of protected steel columns exposed to the ISO 834 and ASTM E119 standard fires. The study was confined to steel columns protected by low-density materials. Average steel temperature rises of columns were calculated using three simplified analytical methods and a finite difference method, and the calculated results were compared with the ASTM E119 fire test data. The three simplified analytical methods assumed one-dimensional heat transfer through the protective coating, but with different assumptions of the protective material heat capacity value. The comparison showed that the finite difference method is the most realistic approach in predicting steel temperatures, and the simplified analytical methods are able to provide desirable results with proper insulation heat capacity values. The influence of the heat capacity of the protective material on values of fire resistance was studied. From the study, a semi-empirical formula for fire resistance prediction of protected steel columns was derived in the form of

$$R = \left(20 \frac{W}{D\rho} + C \right) d_i, \quad (2.1)$$

where R (hour) is fire resistance in time domain, W (lb/ft) is the weight of the steel section per unit length, D (inch) is the heating perimeter of the steel section, ρ (lb/ft³) is

the density of the protective material, d_i (inch) is the thickness of the protection, and C is a constant calculated from relevant fire resistance test data. Equation (2.1) is now widely used in fire resistance design of steel members.

Bardell (1983) conducted an investigation on spray-on protective coatings for steel columns. The thermal conductivity and specific heat of two types of spray-on coatings, cementitious mixture and sprayed fibre, were measured by tests. Two-dimensional finite difference heat transfer analyses based on the ASTM E119 fire curve were conducted on hollow structural steel columns protected by spray-on coatings using measured thermal properties of the protective materials. Temperature profiles of the steel sections and protective coatings were simulated. A parametric study was performed to investigate the influence of thermal properties and thickness of the protection on steel temperatures. The study indicated that thermal properties of the protective materials are essential pieces of information in predicting the steel temperature. Moreover, steel temperatures are very sensitive to the thermal conductivity of the protection material. The predicted steel temperatures were compared to data from full-scale fire resistance tests. Predicted results were found to be lower than the experimental results. These discrepancies were attributed to two causes. One was the moisture content in the spray-on material. The other one was due to the technique used in measuring the thermal conductivity. Because measurements could only be made up to 600°C, thermal conductivity values of the protective material at higher temperatures were derived from extrapolation of lower temperature results.

Tomecek and Milke (1993) carried out a numerical study on the reduction in fire resistance of steel columns caused by partial protection loss. The fire resistance of a column in this study was defined as the time to reach the failure temperature in an ASTM E119 standard fire exposure. Three standard column sections were used in this study, and the columns were covered with a spray-on cementitious mixture, which provided one-, two-, or three-hour fire resistance for the fully protected members. The thicknesses of fire protection were estimated using a simple semi-empirical formula. Eighty-seven two-dimensional heat transfer analyses were conducted using the finite element program FIRES-T3. Temperature dependent thermal properties of steel and protective material, as well as convective and radiative boundary conditions were applied to the heat transfer models. The loss of protection was modeled by removing the corresponding elements. Good agreement was found between the modeled average steel temperatures and the data from ASTM E119 standard fire tests.

Comparisons of the average steel temperature rise were made with respect to the protection loss area, protection loss position, initial fire rating, and column size. The study produced the following findings.

- 1) The fire resistance of steel columns decreases with the increase in the protection loss area.
- 2) A column with a higher initial fire resistance rating will experience a greater reduction in the fire resistance caused by the protection loss than that with a lower initial fire resistance rating.

- 3) The reduction of fire resistance is more severe for flange protection loss than for web protection loss due to the shadowing effect of the flange on the web.
- 4) The effect of protection loss decreases with the column size.

Ryder et al. (2002) also carried out an investigation on the fire resistance reduction of steel columns caused by protection damage. The research was conducted using a similar methodology as that of Tomecek and Milke (1993). Two standard column sections were considered, and each column was protected with a low-density spray-on coating that provided 1- or 2-hour fire resistance for the fully protected column. The fire protection was removed as a strip from the flange or from the web. Three-dimensional heat transfer analyses were conducted using the finite element program FIRES-T3. Thermal properties of the materials and the thermal boundary conditions were applied to the models. The predicted average steel temperatures using the finite element models showed a good agreement with the data from ASTM E119 standard fire tests conducted at Underwriter's Laboratories. The temperature rise at the exposed surfaces was compared with respect to variables such as the area of protection loss, protection thickness, and column size. The comparisons show that the temperature rise in the column is very sensitive to the area of protection loss but is barely affected by the original fire resistance rating, and the influence of the column size was found to be insignificant until late in the test.

Kamikawa et al. (2004) carried out a study on predicting the thermal response of a metal column exposed to a localized fire using the finite element method. Experiments

on a steel column exposed to fire were conducted to examine the finite element heat transfer model developed using the finite element software ANSYS. From the study, thermal balance parameters for heat transfer modelling were speculated, and the finite element model for predicting the temperature distribution of a metal member exposed to a localized fire was established.

Franssen (2004) carried out an investigation on three-dimensional temperature fields in steel joints exposed to fire using the finite element software SAFIR. Three fire scenarios were considered, including the ISO 834 standard fire, hydrocarbon fire, and a natural fire. The investigation indicates that net-section failure at fastener holes need to be considered because the temperature distribution of a joint becomes more uniform after a long duration of fire exposure. The investigation also shows that the modeled temperature in the joint components is higher than that assessed using the lumped mass method in EC3 (1995).

2.3 Mechanical Properties of Structural Steel at Elevated Temperatures

Mechanical properties of steel at elevated temperatures are vital pieces of information for studying the behaviour of steel structures exposed to fire. The stress-strain relationships at elevated temperatures as well as the test methods for obtaining these properties will be discussed in the following sections.

2.3.1 Stress-Strain Relationship

Figure 2.2 illustrates typical stress-strain relationships for mild structural steel at elevated temperatures. It can be seen that both yield strength and the modulus of elasticity decrease as steel temperature increases. However, the ultimate strength does not follow the same trend. Instead, due to the strain-age hardening phenomenon, the ultimate strength increases slightly with the rise in temperature before dropping lower at higher temperatures. The yield plateau, which may be well defined at normal temperatures, vanishes at high temperatures. According to ASTM E8 (2004), there are two methods for defining the yield strength of metallic materials when there is no well-defined yield plateau. One is the offset method. The proof stress corresponding to 0.2% strain offset is often used as the yield strength of structural steel at elevated temperatures. The other one is the extension-under-load method, which is adopted by EC3 (1995). EC3 (1995) uses the stress corresponding to 2% strain as the effective yield strength of structural steel in fire designs.

The reduction factors for the mechanical properties of structural steel at elevated temperature are available in various publications, such as ECCS (1983), EC3 (1995), Lie (1992), and Outinen et al. (2001). EC3 (1995) provides a comprehensive set of equations that describes the stress-strain relationships for structural steel at temperatures from 20 to 1200°C. However, there are differences in elasticity modulus and yield strength at elevated temperatures given by different publications. These differences may be due to reasons such as test method, test facility, and the definition of yield strength.

2.3.2 Strain Components

The deformation of steel at elevated temperatures can be described by three strain components, given as

$$\Delta\epsilon = \epsilon_{th}(T) + \epsilon_{\sigma}(\sigma, T) + \epsilon_{cr}(\sigma, T, t), \quad (2.2)$$

where $\Delta\epsilon$ is the change in strain, T is the steel temperature, σ is the applied stress, t is time, ϵ_{th} is the thermal strain, ϵ_{σ} is the stress-related strain, and ϵ_{cr} is the creep strain. Buchanan (2001) pointed out that both the thermal and creep strains have little effect on a simple structural member such as a simply supported beam. However, for a complex structural system in which structural members are restrained by other parts of the structure, thermal and creep strains may induce significant forces and displacements, and they must be considered in the structural analysis.

2.3.3 Test Methods

There are two types of small-scale tension test methods used in obtaining the mechanical properties of steel at elevated temperatures. They are transient-state and steady-state test methods. In transient-state tests, specimens are heated under a constant load. On the other hand, in steady-state tests, specimens are loaded at a constant steel temperature. It has been found that different test methods usually produce different mechanical properties for steel at elevated temperatures. However in most publications, these properties are provided without specifying the test method.

Efforts have been made to couple results of the two test methods. Kirby and

Preston (1988) compared the proof stresses derived from transient-state tests with that from steady-state tests. It was observed that when the proof stresses of steel are taken as the stress corresponding to 1.0% proof strain, the differences between the results from each test method are negligible. Kirby and Preston therefore suggested that the stress at 1.0% proof strain be used as the effective yield strength for fire structural design. On the other hand, Anderberg (1988) recommended a procedure for coupling these two test methods by controlling either the strain rate or the stress rate in the tests.

2.4 Structural Analyses of Steel Beams Exposed to Fire

The structural behaviour of steel beams exposed to fire is the key to evaluate the fire resistance of steel beams. The behaviour of steel beams exposed to fire is more complex than that at ambient environment and is affected by a number of factors. However, there is a lack of extensive fire tests mainly due to the high expense of testing. As an economical supplement to fire tests, computer based numerical methods are frequently used to investigate the fire performance of steel beams. There are also simple calculation methods available to evaluate the load carrying capacity of a steel beam in fire engineering designs.

2.4.1 Structural Behaviour of Steel Beams Exposed to Fire

The behaviour of fire-exposed simply supported steel beams is mainly governed by the reduction of steel strength and stiffness. For a simply supported steel beam

exposed to fire, as the steel temperatures increase, the mid-span deflection increases due to the reduction of steel stiffness. Meanwhile, the load carrying capacity of the beam decreases due to the reduction of steel strength. Failure of the beam occurs if the load carrying capacity of the beam drops below the imposed load. Thermal expansion does not induce any internal force since the beam is free to expand.

On the other hand, the fire behaviour of steel beams with rotational or axial end restraints is more complex. Thermal deformations play an important role in the fire performance of steel beams with end restraints. Usmani et al. (2001) studied effects of thermal expansion and thermal bowing (bending caused by the temperature gradient along the depth of the beam) on fire-exposed steel and composite beams with various end restraints. For steel beams with axial restraints, restrained thermal expansion will induce an axial compressive force that results in extra deflection and sagging moment due to second order ($P-\delta$) effect. Restricting the thermal bowing due to rotational restraints will induce hogging moment, and restricting the thermal bowing due to axial restraints will induce axial tensile force. In practice, fire-exposed beams are very likely to experience the combination effect of thermal expansion and thermal bowing because structural members are often constructed with some degree of axial and rotational restraints.

Due to the fact that temperatures at the end regions are usually much lower than at the main body of the beam, end restraints actually enhance the fire resistance of steel beams. Moreover, for a beam with axial end restrains, catenary action may take place

and hold the member from collapse as the bending strength is unable to resist the load.

2.4.2 Studies on the Structural Behaviour of Steel Beams Exposed to Fire

A number of studies on the structural behaviour of steel or composite beams exposed to fire have been carried out. A series of factors that affect the fire performance of steel and composite beams were investigated. Some experimental and computer-based studies are reviewed.

Burgess et al. (1991) carried out a series of studies on the structural behaviour of steel and composite beams exposed to fire using a moment-curvature method named the secant stiffness method. Beams studied were made of standard steel sections. Temperature profiles of the beams were either obtained from fire tests or from finite element heat transfer analyses. A limiting mid-span deflection of $1/20$ span was taken as the failure criterion.

The effects of several parameters, such as sides of a beam being heated, asymmetry of a section, support conditions, continuity of the beam, temperature difference over the cross-section, span and depth of the beam, and design stress, were investigated. The study confirmed that the shielding effect of concrete slabs plays an important role in improving the fire resistance of the beams. It was observed that the deflection of the beams is governed by thermal bowing when temperatures on exposed flanges are less than 400°C . As the temperature continues to increase, material degradation gradually becomes the governing factor for deflection. Support conditions

were shown to have a significant effect on the survival of beams. A greater fixity of the supports results in an enhanced fire resistance of the beam. Beams with identical span/depth ratio seem to have similar failure temperatures, which increase as the span/depth ratio decreases. The benefit of partial heating of a continuous beam was shown to be less important. The study also indicated the possibility of improving the fire behaviour of steel beams by using asymmetric cross-sections or by reducing design loads.

Liu et al. (2002) carried out an experimental investigation of the behaviour of axially restrained steel beams exposed to fire. The investigation was carried out on steel frames that consisted of a partially protected steel beam and two fully protected steel columns. The upper flange of the beam was wrapped with insulation material in order to simulate the heat-sink effect caused by the concrete slab. Two types of beam-column connections, web-cleats and flush end-plates, were used in this study. The former were designed to provide only axial restraint, and the latter were designed to provide both axial and rotational restraints. The specimens were tested in ISO 834 standard fire under different load ratios, and their structural behaviour was recorded. The study produced the following findings.

- 5) Connections, which are usually much cooler than the main part of the beam, can reduce the mid-span moment during fire exposure and thus improve the fire resistance of a beam.
- 6) The catenary action is very pronounced in beams with flush end-plate connections;

however, it does not take place in beams with web-cleat connections.

- 7) The catenary action is more significant in beams with a low load ratio and a high axial restraint.

Becker (2002) conducted a study on the structural behaviour of simple steel structures with longitudinal non-uniform temperature distributions along the members in the ISO 834 standard fire exposure. The study was carried out on both single beams and frames. The temperature profiles of the members were obtained by heat transfer analyses in a previous study. Temperatures at a joint and the vicinity of a joint were much cooler than that along the main part of the member. Structural analyses in this study were carried out using the program SAFIR. The study demonstrated that the longitudinal non-uniform temperature distribution is accompanied by the redistribution of internal forces, causing the location of plastic hinges in beams to shift away from the joints. This phenomenon delays the formation of plastic hinges and increases the survival period against structural collapse.

Moss et al. (2004) also carried out an investigation on the effect of support conditions on behaviours of steel and composite beams exposed to fire using SAFIR. Numerical studies were carried out on unprotected steel beams and steel-concrete composite beams, respectively. Two heating conditions (uniformly increasing temperatures and the ISO 834 fire exposure) and four support conditions (pin-roller, pin-pin, fixed-slide, and fixed-fixed supports) were considered in this study. Findings from this study are listed below.

- 1) The behaviour of steel and composite beams at elevated temperatures is far more complex than that at normal temperatures.
- 2) The behaviour is very different for the four support conditions, but similar for the two heating conditions.
- 3) The behaviour is very sensitive to the level of current stress with respect to the temperature-dependent proportional limit and yield stress. Deflections and internal forces often undergo sudden changes as stresses at the critical cross-section exceed the proportional limit or yield stress.
- 4) A bare steel beam and a steel-concrete composite beam of similar size have similar overall behaviours in a fire.

Local buckling is an important behaviour of thin-wall steel members subjected to bending or compression. Lee and Mahendran (2005) carried out a series of fire tests and finite element analyses to investigate the local buckling behaviour of cold-formed steel members. Based on the test and analysis results, the authors suggested that the buckling stress of steel plates at elevated temperatures can be calculated using the theoretical elastic local buckling stress equation with a reduced elasticity modulus, and the buckling coefficients are constant when the reduced elasticity modulus was used. The study also showed that at elevated temperatures, a high strength steel section experiences a greater effective width reduction than a low strength steel section.

Fontana and Knobloch (2004) conducted a study on the local buckling behaviour of steel plates in bending and compression at elevated temperatures using finite element

analyses. From the study, the authors proposed that due to the nonlinear stress-strain relationship of steel at elevated temperatures, a strain-based formulation for the effective width calculation is more appropriate for calculating the load-carrying capacity of steel plates exposed to fire.

Research on the lateral torsional buckling of I-shape steel beams exposed to fire has also been carried out by Piloto & Vila Real (2000) and Vila Real & Franssen (2000) by experiments and finite element modelling. Their studies showed that the EC3 (1995) simple method for calculating the lateral torsional buckling resistance at elevated temperatures is unsafe for beams of intermediate slenderness.

2.4.3 Simple Methods for Calculating the Moment Resistance of Steel Sections

In fire engineering designs, the load carrying capacity of a fire-exposed steel beam is usually calculated using simple methods. EC3 (1995) provides simple formulas to calculate the cross-section moment resistance of a steel beam exposed to fire. The moment resistance is calculated based on the same elastic and plastic design concepts employed for a beam at normal temperatures. For a beam with a uniform cross-section temperature, the moment resistance is determined by multiplying the moment resistance of the beam at normal temperatures by the yield strength reduction factor. For a cross-section with a non-uniform temperature distribution, the moment resistance can be determined by dividing the cross-section into small pieces and summing the moment contribution of each of these pieces. Alternatively, the moment resistance can also be

calculated using the formula for a uniform cross-section temperature with the maximum temperature being considered as the average temperature of the cross-section. If the beam is statically indeterminate, EC3 allows the calculated resistance at the supports to be increased by 12% with the recognition that steel temperatures are lower around the supports than within the span of the beam. Besides, for a member subjected to both bending and compression in a fire exposure, EC3 (1995) suggests that the cross-section resistance should be calculated using the normal temperature design equations combined with the reduced yield strength of steel at high temperatures.

Figure 2.1 Flow chart for calculating the load carrying capacity of a structure exposed to fire (modified from Buchanan 2001)

Figure 2.2 Stress-strain relationships for mild structural steel ASTM A36 (modified from Malhotra 1982)

Charter 3   Heat Transfer and Structural Analysis

The objective of the heat transfer and structural analysis study is to calculate the sectional moment capacity of steel beams exposed to the standard test fire. The heat transfer and structural analysis consists of two stages: heat transfer analysis stage and structural analysis stage. In this study, heat transfer analyses were carried out by numerical modelling, and structural analyses were carried out mainly by analytical calculations and by a few numerical structural analyses. Steel beam assemblies, material properties and the methodology used in the analysis program are discussed in this chapter.

3.1   Steel Beam Assemblies

Steel beam assemblies that consisted of a steel section and spray-on fire protection were studied. The steel section, the fire protection material and the beam assembly are described in the following.

3.1.1   Steel Sections

Steel sections used in the study were CAN/CSA-G40.20/G40.21-98 (1998) Grade 350W standard hot-rolled steel I-beams listed in CISC (2004). Five I-shape sections, W410×74, W410×60, W310×60, W310×39 and W250×24, were used. At normal temperatures, these sections have a density ρ_s of 7850 kg/m³, a yield strength f_y of 350 MPa, a modulus of elasticity E of 200,000 MPa, and a Poisson's ratio ν of 0.3. The

geometrical properties of these sections are shown in Figure 3.1 and Table 3.1. It should be pointed out that all corners in the cross-section are taken as right angles for the convenience of numerical modelling.

3.1.2 Fire Protection

Spray-on mineral fiber was used as the fire protection of the steel beams. According to Harper (2004), the mineral fiber mixture combines the fibers, binding agents, air, and water, and it is directly sprayed on the steel surfaces with specially designed equipment. The study only deals with the ability of the protection to delay the increase of steel temperature in a fire exposure. The fire protection was applied uniformly along the perimeter of the beam cross-section as shown in Figure 3.2. The fire protection material was assumed to maintain its chemical stability, integrity, and adhesion to the steel surface during fire exposure. In addition, the fire protection material was not considered to have any load-bearing ability.

Two protection thicknesses, which represent 1- and 2-hour fire resistance for a fully protected steel member, were considered for each steel section. According to Lie and Stanzak (1973), simple equations based on one-dimensional heat transfer analyses can be used for estimating the necessary fire protection thickness. Lie (1992) published a series of simple equations for protection thickness estimation, given in the following form:

$$d_i = \frac{R}{\left(C_1 \frac{W}{D} + C_2 \right)} \text{ with} \quad (3.1)$$

$$D = 4b + 2d - 2w, \quad (3.2)$$

where d_i (inch) is the thickness of protection, R (minute) is fire resistance, W (lb/ft) is the weight of the steel section per unit length, D (inch) is the heated perimeter, b (inch) is flange width, d (inch) is section depth, w (inch) is web thickness, C_1 and C_2 are material constants for a specific fire protection material. The thickness of mineral fiber material is given by

$$d_i = \frac{R}{\left(60 \frac{W}{D} + 42 \right)}, \quad (3.3)$$

with C_1 and C_2 in (3.1) taken as 60 and 42. It is noted that (3.2) and (3.3) were originally developed to estimate the protection thickness required to design the fire resistance rating of a steel column subjected to ASTM E119 standard fire exposure. Equation (3.3) was adopted in this study because the beams were assumed to be exposed to fires on four sides, and the time-temperature curve of the ISO 834 standard fire is very similar to that of the ASTM E119 standard fire, especially in the first two hours. The calculated protection thicknesses for each steel section are given in Table 3.2.

3.1.3 Studied Steel Beams

This study focused on protected steel beams with partial fire protection loss. For comparison, a few fully protected and unprotected steel beams were also studied.

Beams that have been studied are listed in Tables 3.3 to 3.7. To simulate protection loss, small pieces of fire protection, of various sizes and shapes were removed from the flanges or the web of the beams. The fire protection loss configurations are illustrated in Figures 3.3 to 3.7. All beams were assumed to be simply supported. For a beam subjected to a partial protection loss, its mid-span cross-section was assumed to pass through the center of the damage area. Figure 3.8 shows a typical simply supported beam with protection loss at the mid-span.

In total, 95 beam models were studied, 71 of which were associated with the W410×74 section. All beams were studied using the uncoupled numerical heat transfer analysis and the analytical structural analysis. In addition, a few beams were also studied using the numerical sequentially coupled thermal-stress analysis.

3.2 Properties of Materials

Material properties used in this study are described in this section. These properties include thermal properties of steel and the fire protection material, and mechanical properties of steel at elevated temperatures.

3.2.1 Thermal Properties of Materials

To model the heat transfer process within a protected steel member, thermal properties of both steel and the fire protection material are needed. Thermal properties involved in this study are density, specific heat, thermal conductivity, and moisture

content (for the fire protection material only).

3.2.1.1 Thermal Properties of Steel

1) Density

The density of steel at normal temperatures is 7850 kg/m^3 . Because the volume change of steel at high temperature is small, the density of steel is considered to remain constant with temperature.

2) Specific heat

According to EC3 (1995), the specific heat of steel, c_s (J/kgK), is determined by the following equations:

for $20^\circ\text{C} \leq T_s < 600^\circ\text{C}$,

$$c_s = 425 + 7.73 \times 10^{-1} T_s - 1.69 \times 10^{-3} T_s^2 + 2.22 \times 10^{-6} T_s^3, \quad (3.4a)$$

for $600^\circ\text{C} \leq T_s < 735^\circ\text{C}$,

$$c_s = 666 + \frac{13002}{738 - T_s}, \quad (3.4b)$$

for $735^\circ\text{C} \leq T_s < 900^\circ\text{C}$,

$$c_s = 545 + \frac{17820}{T_s - 731}, \quad (3.4c)$$

for $900^\circ\text{C} \leq T_s < 1200^\circ\text{C}$,

$$c_s = 650, \quad (3.4d)$$

where T_s ($^\circ\text{C}$) is the steel temperature. The specific heat versus temperature curve is plotted in Figure 3.9. The peak at 735°C results from a metallurgical phase change.

3) Thermal conductivity

According to EC3 (1995), the thermal conductivity of steel, λ_s (W/mK), is determined as follow.

For $20^\circ\text{C} \leq T_s < 800^\circ\text{C}$,

$$\lambda_s = 54 - 3.33 \times 10^{-2} T_s, \quad (3.5a)$$

for $800^\circ\text{C} \leq T_s < 1200^\circ\text{C}$,

$$\lambda_s = 27.3. \quad (3.5b)$$

The thermal conductivity versus temperature curve is plotted in Figure 3.10.

3.2.1.2 Thermal Properties of the Fire Protection Material

According to Bardell (1984), the thermal conductivity and specific heat of spray-on mineral fiber are temperature dependent. However, most publications only provide constant thermal properties for fire protection materials. Lack of reliable temperature dependant data on the thermal properties, heat transfer analyses in this study were carried out mainly based on the constant thermal properties provided by ECCS (1995). These properties are presented in Table 3.8. In addition, a few analyses were conducted using temperature dependent thermal properties of the fire protection material. These thermal properties, as shown in Table 3.9, were obtained from Ryder et al. (2002). The temperature dependent thermal conductivity of the fire protection material is plotted against temperature in Figure 3.11.

3.2.2 Mechanical Properties of Steel at Elevated Temperatures

In this study, the mechanical properties of steel at elevated temperatures are referred specifically to the stress-strain relationship of hot-rolled structural steels. Thermal strains and creep strains were not included because these strains do not induce significant internal forces for the studied beams. EC3 (1995) provides a comprehensive material model for hot-rolled structural steels at elevated temperatures, and this model was adopted to describe the mechanical properties in the numerical study.

According to the EC3 model, the stress-strain relationship of steel at elevated temperatures is described by the following equations.

For $\varepsilon \leq \varepsilon_{p,T}$,

$$\sigma = \varepsilon E_{a,T}, \quad (3.6a)$$

for $\varepsilon_{p,T} < \varepsilon \leq \varepsilon_{y,T}$,

$$\sigma = f_{p,T} - c + \left(\frac{b}{a} \right) \left[a^2 - (\varepsilon_{y,T} - \varepsilon)^2 \right]^{\frac{1}{2}}, \quad (3.6b)$$

for $\varepsilon_{y,T} < \varepsilon \leq \varepsilon_{t,T}$,

$$\sigma = f_{y,T}, \quad (3.6c)$$

for $\varepsilon_{t,T} < \varepsilon \leq \varepsilon_{u,T}$,

$$\sigma = f_{y,T} \frac{1 - (\varepsilon - \varepsilon_{t,T})}{(\varepsilon_{u,T} - \varepsilon_{t,T})}, \quad (3.6d)$$

for $\varepsilon = \varepsilon_{u,T}$,

$$\sigma = 0, \quad (3.6e)$$

where

$$\varepsilon_{p,T} = f_{p,T} / E_{a,T},$$

$$\varepsilon_{y,T} = 0.02,$$

$$\varepsilon_{t,T} = 0.15,$$

$$\varepsilon_{u,T} = 0.2,$$

$$a^2 = (\varepsilon_{y,T} - \varepsilon_{p,T})(\varepsilon_{y,T} - \varepsilon_{p,T} + c / E_{a,T}),$$

$$b^2 = c(\varepsilon_{y,T} - \varepsilon_{p,T})E_{a,T} + c^2,$$

$$c = \frac{(f_{y,T} - f_{p,T})^2}{(\varepsilon_{y,T} - \varepsilon_{p,T})E_{a,T} - 2(f_{y,T} - f_{p,T})}.$$

In (3.6), $f_{y,T}$ is the effective yield strength, $f_{p,T}$ is the proportional limit, $E_{a,T}$ is the slope of the linear elastic range, $\varepsilon_{p,T}$ is the strain at the proportional limit, $\varepsilon_{y,T}$ is the yield strain, $\varepsilon_{t,T}$ is the limiting strain for yield strength and $\varepsilon_{u,T}$ is the ultimate strain. The stress-strain relationship in (3.6) is shown in Figure 3.12. It should be noted that strain hardening is not included in this model. The reduction factors for the stress-strain relationship of steel at elevated temperatures, including the reduction factor for effective yield strength, elastic modulus and proportional limit, are given in Table 3.10. The variation of these reduction factors with temperature is plotted in Figure 3.13.

Outinen et al. (2001) carried out a series of tests to study the mechanical properties of structural steels at elevated temperatures and compared the test results with that from the EC3 model. The comparisons of yield strength and the modulus of elasticity between the EC3 model and test results for hot-rolled S355 structural steel are shown in Figures 3.14 and 3.15. It can be seen that there is a good agreement between

the yield strength and the modulus of elasticity from the EC3 model and test results.

3.3 Numerical Heat Transfer Analyses

The objective of the numerical heat transfer analysis was to predict the temperature distributions of steel beams in ISO 834 standard fire resistance tests. The analysis was carried out based on three basic characteristics of the ISO 834 standard fire resistance test. Finite element heat transfer models of the beams were developed, and the heating process of the standard fire resistance test was simulated. Before completing the heat transfer analyses, the heat transfer model was first verified against standard fire test data. The analysis was carried out using the commercial finite element program ABAQUS (2004).

3.3.1 Basic Characteristics of Standard Fire Test Modelling

The basic characteristics of standard fire test modelling include the time-temperature curve of the fire environment, heat transfer within the test specimen, and the boundary conditions of the specimen.

3.3.1.1 Time-Temperature Curve of the Fire

In a standard fire resistance test, the fire temperature in the test furnace follows a standard time-temperature curve. In this study, the ISO 834 standard fire curve, as shown in Figure 3.16, was adopted for the fire resistance test modelling. The fire

temperature T ($^{\circ}\text{C}$) at time t is given by

$$T = 345 \log_{10}(8t + 1) + T_0, \quad (3.7)$$

where T_0 is the ambient temperature, taken as 20°C .

3.3.1.2 Heat Transfer within the Specimen

Heat transfer within a specimen is transient heat conduction in a solid body. The governing differential equation of the conduction problem is given by

$$\frac{\partial}{\partial x} \left(\lambda_x \frac{\partial T}{\partial x} \right) + \frac{\partial}{\partial y} \left(\lambda_y \frac{\partial T}{\partial y} \right) + \frac{\partial}{\partial z} \left(\lambda_z \frac{\partial T}{\partial z} \right) + \dot{q}'' = \rho c \frac{\partial T}{\partial t}, \quad (3.8)$$

where T is temperature, x , y , z are spatial coordinates, λ_x , λ_y , λ_z are thermal conductivities in x , y , z directions, $\dot{q}''$ is internally heat generation, ρ is density, c is specific heat, and t is time.

Two assumptions were made in this study when modelling the heat conduction process within a specimen:

- 1) The material is isotropic, which means that λ_x , λ_y , and λ_z of the material can be replaced by a single value λ .
- 2) Thermal resistance on the contact surface between the steel and the fire protection material is negligible.

3.3.1.3 Boundary Conditions

When a structural member is exposed to a fire, the exposed surface of the member

receives heat from the fire by convection and radiation. Heat flux on the exposed surface, q'' (W/m^2), is given by

$$q'' = q_c'' + q_r'', \quad (3.9)$$

where q_c'' and q_r'' are heat flux due to convection and radiation respectively. The convective heat flux is given as

$$q_c'' = h_c (T_f - T_s), \quad (3.10)$$

where h_c ($\text{W/m}^2\text{K}$) is the convection coefficient, T_f (K) is the fire temperature, and T_s (K) is the surface temperature. EC1 (2002) recommends a convection coefficient h_c of 25 $\text{W/m}^2\text{K}$ for the standard fire. The radiative heat flux is given as

$$q_r'' = \Phi \epsilon_{\text{res}} \sigma (T_f^4 - T_s^4) \quad \text{with} \quad (3.11)$$

$$\epsilon_{\text{res}} = \frac{\epsilon_f \epsilon_s}{\epsilon_f + \epsilon_s - \epsilon_f \epsilon_s}, \quad (3.12)$$

where Φ is the view factor, σ is the Stefan-Boltzmann constant, ϵ_{res} is the resultant emissivity, ϵ_f is the flame emissivity, and ϵ_s is the surface emissivity. The view factor Φ is a function of the incidence angle between the emitting source and the receiving surface. Purkiss (1996) suggests that Φ can be conservatively taken as 1.0 in the standard fire test. The Stefan-Boltzmann constant σ is $5.67 \times 10^{-8} \text{ W/m}^2\text{K}^4$. According to Kay et al. (1996) and Purkiss (1996), the flame emissivity ϵ_f should be taken as 0.8. The surface emissivity ϵ_s varies with materials. Purkiss (1996) recommends a value of 0.625 as the surface emissivity of steel. Tomecek and Milke (1993) and Ryder et al. (2002) used a value of 0.9 as the surface emissivity of the

spray-on protective material, and this value was adopted in the study. Substituting (3.10) and (3.11) into (3.9), gives

$$q'' = h_c(T_f - T_s) + \Phi \epsilon_{\text{res}} \sigma (T_f^4 - T_s^4). \quad (3.13)$$

3.3.2 Numerical Heat Transfer Models

Two- and three-dimensional finite element heat transfer models were developed using ABAQUS/Standard input files. Transient heat transfer analyses that simulated the standard fire resistance tests were conducted on the models. Mesh studies were conducted to determine the proper element size.

3.3.2.1 Two-Dimensional Finite Element Model

- Description of the Model

Two-dimensional finite element models were developed to model the cross-section of unprotected and fully protected steel beams. The four-node solid element DC2D4 was employed in modelling the cross-sections. DC2D4 elements are linear two-dimensional solid elements for diffusive heat transfer or mass diffusion analysis. Two element sets were used to form the cross-section of a protected beam: one represents the steel section, and the other one represents the fire protection material. Figure 3.17 shows the mesh of a two-dimensional heat transfer model. Thermal properties of steel and the protective material were respectively applied to the two element sets. Because the thermal resistance between the steel and the protection was

assumed to be zero, nodes on the steel-protection interface were shared by both steel elements and fire protection elements. The initial temperature of the cross-section was 20°C.

A transient heat transfer analysis that simulated the ISO 834 standard fire test was conducted on each model. Heat flux at the model boundaries was calculated based on the ISO 834 standard time-temperature curve and the convective and radiative boundaries described by (3.13). A user subroutine written in FORTRAN code was developed and used in the analysis to apply heat flux for the heating process. As illustrated in Figure 3.17, heat flux was applied on the outer boundaries of each model because the beams were assumed to be exposed to fire on all sides. The heating period for each protected steel beam was equal to its fire resistance rating, and for the unprotected beams it was 2 hours. The output of the heat transfer analysis was the temperature at each node of the steel elements during heating, at a time interval of 5 minutes.

- Mesh Study

The spray-on protective material has a very low thermal conductivity. Therefore, when a protected beam is under transient heating conditions, large temperature gradients across the protection thickness will appear. The purpose of the mesh study was to find out the proper mesh across the protection thickness. The mesh study was carried out on a W410x74 section covered with 32 mm fire protection. Four different meshes, which divided the protection into 1, 2, 3 and 4 layers respectively, as shown in Figure 3.18, were

considered. Temperature distributions on the bottom flange of the steel section after 120 minutes standard fire exposure are plotted in Figure 3.19. It can be observed that the temperature difference between the finest and the coarsest mesh is about 16°C. Such a temperature difference does not have a significant effect on the structural behaviour because there is little difference in mechanical properties of steel within this temperature range. Although 1-layer is sufficient, for the ease of modelling protection loss and to ensure the accuracy of three-dimensional heat transfer analyses, the 4-layer mesh was adopted. Moreover, a combined 4- and 8-layer mesh was used to model the 87.5% damage penetration.

3.3.2.2 Three-Dimensional Finite Element Model

- Description of the Model

Three-dimensional finite element models were developed to model the protected steel beams subjected to partial protection loss. They were symmetric models with the symmetry plane passing through the mid-span of the beam. These models were constructed using the solid brick element DC3D8, an 8-node linear heat transfer element. The transverse mesh of a three-dimensional model was identical to that of a corresponding two-dimensional model that has the same steel section and fire protection thickness. Thermal properties of the steel section and the fire protection were identical to that in the two-dimensional models. The loss of fire protection was modeled by removing elements at the designed damage area. Figure 3.20 presents a

three-dimensional model of a protected steel beam with partial protection loss on the bottom flange.

Transient heat transfer analyses were conducted on the models to simulate standard fire resistance tests. The boundary conditions of the models were calculated using the same user subroutine as in the two-dimensional heat transfer analysis, except that the ends of the models were assumed to be fully insulated. The heating period for each analysis was equal to the fire resistance rating of a corresponding fully protected beam. The output of the heat transfer analysis was the nodal temperature of the steel elements every 5 minutes of the heating.

- Mesh Study

A very fine transverse mesh has been used for the three-dimensional models, and a relatively coarse longitudinal mesh was used to reduce the number of elements. It is known that if a protected steel beam with partial protection loss is exposed to fire, a high temperature gradient will appear at the damage region. Therefore, the mesh at the damage region should be fine enough to predict the temperature distribution. A mesh study was carried out to determine the appropriate longitudinal mesh at the damage region. The study was conducted using a W410x74 steel beam with 32 mm fire protection. The span of the beam was 4.10 m. A piece of fire protection with a size of 10 cm x 10 cm was removed from the bottom flange of the beam in such a manner that the damage area and the beam have the same line of symmetry at the mid-span.

Figure 3.21 shows three symmetric models with different longitudinal meshes at the damage region considered in the mesh study. The longitudinal temperature distributions of two selected positions in the model cross-section (shown in Figure 3.22) after 120 minutes standard fire exposure were plotted in Figure 3.23. It should be noted that in Figure 3.23, the abscissa represents the distance from the damage center. Although the comparison shows that the difference in the temperature distribution is insignificant between different meshes, it can be seen that a finer mesh is better to capture small temperature changes. Therefore, Mesh C and slightly modified meshes based on Mesh C were adopted for the three-dimensional heat transfer analyses.

- Length of the Models

Figure 3.24 shows the temperature distributions from the mid-span to the support after 120 minutes standard fire exposure for the W410x74 beam, on which the mesh study was carried out. It can be seen that the effect of the fire protection loss on steel temperatures is negligible at cross-sections further than 0.7 m away from the damage center. This result indicates that the length of the three-dimensional models can be shortened to improve the computing efficiency without the loss of accuracy. The investigation on the model length was conducted on the same beam as in the three-dimensional mesh study. A comparison was made between the full-length model and a shortened model with a length of 1.25 m. The longitudinal temperature distributions of the two selected positions in Figure 3.22 after 120 minutes standard fire

exposure are plotted in Figure 3.25. It can be seen that there is very little difference between the results of these two models. Therefore, the length of 1.25 m was adopted for the three-dimensional models used in the analyses.

The heat transfer analyses were conducted using the finite element software ABAQUS 6.4.1 on an Opteron 2.4 GHz computer with the Linux operation system. It takes approximately 6 hours to run a typical 1-hour fire exposure analysis and 12 hours to run a typical 2-hour fire exposure analysis..

3.3.3 Model Validation

It is necessary to verify the finite element models with test data before moving forward with the numerical study. Since there was no experimental work included in this study, the test data were obtained from existing published literatures. The data of three different fire resistance tests were used for the model validation.

Ryder et al. (2002) used the data of an ASTM standard fire resistance test done in Underwriter's Laboratories to validate the numerical heat transfer models. The test measured the average steel temperature of a W250×73 (W10×49 in imperial designation) steel column covered with 19 mm of spray-on fire protection in ASTM E119 standard fire exposure. The same test data were used in this study for model validation. A finite element model of the column cross-section was developed using solid element DC2D4. Thermal properties of the protective material provided by Ryder et al. (2002) are shown in Table 3.9, and the thermal properties of steel were taken from EC3 (1995). Two

two-dimensional heat transfer analyses were carried out to simulate the fire resistance test; one was based on the ASTM E119 standard fire curve and the other one was based on the ISO 834 standard fire curve. The measured average steel temperature and the results of the heat transfer analyses are compared in Figure 3.26. The comparison shows that the modeled steel temperatures based on these two standard fire curves are similar, and they agree with the measured steel temperature very well.

Another two groups of test data were obtained from Ding et al. (2003). A protected steel column and a protected steel beam were tested in the Japanese JIS A 1994 standard fire. The column was an H-300×300×10×15 section covered with a 22.3 mm thick mineral wool. During the test, the column was exposed to fire on all sides, and the average steel temperatures rise in the steel section was recorded. The beam was an H-400×200×8×13 section protected with an 11.3 mm thick mineral wool. During the test, the beam was exposed to fire on three sides, leaving the top unexposed and covered with concrete panels. The temperature rise of the bottom flange, top flange and the web was recorded. Again, the column cross-section and the beam cross-section were modeled using solid element DC2D4. The author provided only the thermal conductivity of the protective material, 0.13 W/(mK), so the other thermal properties of the protective material were taken from Table 3.6, and thermal properties of steel were taken from EC3 (1995). Heat transfer analyses were conducted to simulate the heating process of these two fire resistance tests. The analyses were conducted using the ISO 834 standard fire curve because the time-temperature curve of these two standard fires

are quite similar for the time period of the test. Figure 3.27 demonstrates the comparison of steel temperatures between the fire resistance test and the heat transfer analysis for the column. It can be seen that the results of heat transfer analysis are in good agreement with the test data. The predicted temperatures of the beam are compared with the test results in Figure 3.28. The discrepancy may be attributed to two reasons. One reason is that a constant thermal conductivity of the protective material was used in the analysis. In fact, the thermal conductivity varies with temperature. The other reason is that in the numerical analysis, the unexposed surface of the beam was assumed to be completely insulated. However, during the heating, the concrete panels covering the beam act as heat sinks and kept absorbing heat from the beam, causing continuous heat loss from the outer surfaces of these panels. For these reasons even though there is discrepancy between predicted and experimental temperatures, it can be concluded that the numerical heat transfer models are valid.

3.4 Structural Analysis by Analytical Calculation

The structural analysis was carried out to calculate the cross-section moment capacity of steel beams at elevated temperatures. The calculation method was developed based on classical pure bending and plate buckling theories. Reduction factors of mechanical properties for steel at elevated temperatures were employed in the formulas developed for the structural analysis. Temperature profiles of steel beam cross-sections were obtained from the numerical heat transfer analysis. The structural

analysis was implemented using computer programs written in the Visual Basic programming language.

3.4.1 Calculation Method

Formulas for calculating the moment capacity of steel sections with a non-uniform temperature profile are discussed in this section. The classification of steel sections at high temperatures due to fire exposure is also discussed.

3.4.1.1 Analytical Formulas for Cross-Section Moment Capacity Calculation

When a beam is subjected to pure bending, the plane section initially perpendicular to the neutral axis of the beam is assumed to remain plane and perpendicular to the neutral axis. The bending moment in a cross-section of the beam is given by

$$M = \int_A \sigma y dA, \quad (3.14)$$

where M is the bending moment of the cross-section, A is the area of the cross-section, y is distance from the neutral axis to a finite area dA , and σ is stress at the finite area dA .

In practice, a discrete expression is more frequently used, given as

$$M = \sum_{i=1}^n \sigma_i A_i y_i, \quad (3.15)$$

where n is the number of elements in the cross-section, A_i is the area of element i , y_i is the distance from the neutral axis to the centroid of the element i , and σ_i is the average stress

of element i , taken as positive for tension and negative for compression. Figure 3.29 illustrates the strain and stress distributions in a steel beam cross-section.

For a steel beam cross-section with no residual stress and with an increasing curvature, the fiber furthest from the neutral axis will yield first. The stress of each element is given by

$$\sigma_i = \frac{\epsilon_y E y_i}{Y_0}, \quad (3.16)$$

where ϵ_y is the yield strain of the furthest fiber, Y_0 is the distance between the yielded fiber and the neutral axis, taken as positive value. Combining (3.15) and (3.16), the yield moment M_y can be calculated by

$$M_y = \sum_{i=1}^n \epsilon_y \frac{E A_i y_i^2}{Y_0}. \quad (3.17)$$

For a cross-section with a non-uniform temperature distribution, the yield moment $M_{y,f}$ can be calculated by substituting the modified elasticity modulus of each element into (3.17), thus gives

$$M_{y,f} = \sum_{i=1}^n \epsilon_{y,T,0} k_{E,T,i} \frac{E A_i y_i^2}{Y_0} \quad \text{with} \quad (3.18)$$

$$\epsilon_{y,T,0} = \frac{k_{y,T,0} f_y}{k_{E,T,0} E}, \quad (3.19)$$

where $k_{E,T,i}$ is the reduction factor of the elastic modulus at steel temperature T_i , $\epsilon_{y,T,0}$ is the yield strain of the furthest fiber, $k_{y,T,0}$ is the reduction factor for the yield stress of the furthest fiber, and $k_{E,T,0}$ is the reduction factor for the elastic modulus of the furthest fiber.

If there is a non-uniform temperature distribution along the flange, a conservative

simplification is adopted by taking $\epsilon_{y,T,0}$ as the minimum yield strain on the flange. The neutral axis of the cross-section is determined by

$$\sum_{i=1}^n \epsilon_{y,T,0} k_{E,T,i} E A_i y_i = 0. \quad (3.20)$$

If the cross-section is able to undergo large rotation without any local buckling or distortional buckling, the plastic moment capacity may be reached. Therefore, the plastic moment $M_{p,f}$ can be calculated by

$$M_{p,f} = \sum_{i=1}^n k_{y,T,i} f_y A_i y_i, \quad (3.21)$$

where $k_{y,T,i}$ is the reduction factor of the yield stress at steel temperature T_i , taken as positive for tension and negative for compression. The neutral axis of the cross-section is determined by

$$\sum_{i=1}^n k_{y,T,i} f_y A_i = 0. \quad (3.22)$$

There is also a possibility that local buckling may occur on the compression elements of a steel beam before yield moment is reached. In this circumstance, the moment capacity of the critical cross-section is governed by local buckling. At normal temperatures, the critical stress σ_{cr} for a steel plate due to elastic buckling is given by

$$\sigma_{cr} = \frac{k\pi^2 E}{12(1-\nu^2) \left(\frac{b}{t}\right)^2}, \quad (3.23)$$

where k is the buckling coefficient, ν is the Poisson's ratio, and b/t is the width to thickness ratio of the element. Buckling coefficient for long flat plates under

compression and bending are given by SSRC (1988). Lee and Mahendran (2005) gave an equation to calculate the elastic buckling stress of a steel plate at elevated temperatures by substituting the modified elasticity modulus into (3.23). The equation is given as

$$\sigma_{cr,T} = \frac{k_T \pi^2 k_{E,T} E}{12(1-\nu^2) \left(\frac{b}{t}\right)^2}, \quad (3.24)$$

where $\sigma_{cr,T}$ is the elastic buckling stress at temperature T , and k_T is the buckling coefficient at temperature T . The paper also presents results of experimental and finite element studies that show that buckling coefficients vary very little with temperature. In this study, the buckling coefficient k_T is taken as 0.42 for flange buckling and 7.8 for web buckling. Equation (3.24) was adopted in this study to calculate the buckling stress of a steel plate with a non-uniform temperature profile. Conservatively, the buckling stress was calculated based on the maximum steel temperature of the plate.

For an I-shape steel beam governed by the elastic local buckling, the moment capacity of the critical cross-section $M_{cr,f}$ is determined by

$$M_{cr,f} = \sum_{i=1}^n \epsilon_{cr,T,0} k_{E,T,i} \frac{EA_i y_i^2}{Y_0} \quad \text{with} \quad (3.25)$$

$$\epsilon_{cr,T,0} = \frac{\sigma_{cr,T,0}}{k_{E,T,0} E}. \quad (3.26)$$

If local buckling occurs on the compression flange, $\epsilon_{cr,T,0}$ is the buckling strain of the flange, $\sigma_{cr,T,0}$ is the buckling stress of the flange, $k_{E,T,0}$ is the reduction factor for the elastic modulus of the flange, and Y_0 is the distance from the neutral axis to the buckled flange. If local buckling occurs in the web, then $\epsilon_{cr,T,0}$ is the strain of the furthest

compression fiber of the web, $\sigma_{cr,T,0}$ is the buckling stress of the web, $k_{E,T,0}$ is the reduction factor for the elastic modulus of the furthest compression fiber of the web, and Y_0 is the distance between the neutral axial and the furthest compression fiber of the web. The neutral axis of the cross-section is determined by

$$\sum_{i=1}^n \varepsilon_{cr,T,0} k_{E,T,i} E A_i y_i = 0. \quad (3.27)$$

3.4.1.2 Classification of Cross-Sections

In steel structure design codes, steel sections are usually classified into several classes, depending on the width-to-thickness ratio of their elements subjected to compression. In CAN/CSA-S16-01 (2001), steel sections are classified as Class 1, 2, 3, and 4. Class 1 and Class 2 sections are able to undergo large rotation and achieve plastic moment M_p before any local buckling occurs. A Class 3 section is able to reach the yield moment M_y , while a Class 4 cross-section has a lower rotational ability with its moment capacity M_{cr} governed by elastic local buckling. The classification of steel sections is based on the width to thickness ratio limit of the plate, which can be expressed as

$$\frac{b_e}{t} \leq c \sqrt{\frac{E}{f_y}}, \quad (3.28)$$

where b_e is the effective width of a plate, t is the plate thickness, and c is a constant. For an I-shape steel beam, b_e is taken as a half of the width of the flanges when calculating the effective width-to-thickness ratio of the flanges, and it is taken as the

depth of the cross-section when calculating the effective width-to-thickness ratio of the web. Table 3.11 gives CAN/CSA-S16-01 section classification for Grade 350 steel at normal temperatures. Values of the constant c for Grade 350 steel are listed in Table 3.12. These values are calculated by combining (3.28) and the CAN/CSA-S16-01 section classification. Section classes of the studied steel sections at normal temperatures are presented in Table 3.13.

When a steel plate is heated, its yield strength and elastic modulus decrease at different rates. As a result, the effective width of the plate changes with temperature. In fire engineering designs, EC3 (1995) suggests using the reduced yield strength and elasticity modulus to classify steel sections at elevated temperatures. Thus, a factor $(k_{E,T} / k_{y,T})^{1/2}$ is added into the right hand side of (3.26), gives

$$\frac{b_e}{t} = c \sqrt{\frac{k_{E,T} E}{k_{y,T} f_y}}. \quad (3.29)$$

For a cross-section of which the web or flanges have a non-uniform temperature distribution, the classification is based on the minimum value of $\sqrt{(k_{E,T} E) / (k_{y,T} f_y)}$ along the flange or the web.

3.4.2 Implementation of the Structural Analysis

The analytical structural analysis was implemented using Visual Basic programs in the Microsoft Excel application. Figure 3.30 shows a flow chart for calculating the moment capacity of a beam cross-section. Commentaries of the flow chart are as

follow.

- 1) The geometrical properties of a cross-section were generated in a Visual Basic program, and the cross-section was divided into small elements using the same mesh as that in the corresponding heat transfer model.
- 2) The input temperature profiles consist of nodal temperatures in a given steel cross-section at each time step of the heat transfer analysis.
- 3) The time step in the moment capacity calculation is 5 minutes, which is equal to the time step of the heat transfer analysis.
- 4) The mechanical properties include the yield strength and the modulus of elasticity for steel at an elevated temperature.
- 5) The initial depth of the neutral axial of a cross-section is taken as a half of the depth of the cross-section. During the calculation, the position of the neutral axis is adjusted using (3.20), (3.22) or (3.27), depending on the cross-section class. For Class 1 and 2 sections, the neutral axis can be determined by direct calculation. For Class 3 and 4 sections, the neutral axis is determined using trial-and-error until the magnitude of the summation of all axial forces over the cross-section is less than 10^{-4} N.

3.5 Sequentially Coupled Thermal-Stress Analysis

The numerical sequentially thermal-stress analysis was carried out to verify results of the uncoupled heat transfer and structural analysis, and to study the structural

behaviour of simply supported steel beams exposed to fire. A sequentially coupled thermal-stress analysis consists of two runs: a heat transfer analysis and a sequential stress analysis. In this study, a heat transfer analysis provided the temperature profiles of a steel beam during heating, and a sequential stress analysis analyzed the structural response of the beam at a particular time step of the heating. The sequentially couple thermal-stress analysis was also carried out using finite element program ABAQUS (2004).

3.5.1 Description of the Numerical Model

Symmetric heat transfer models of the beams were developed using the solid heat transfer element DC3D8. These models were symmetric about the mid-span. Making use of symmetry, only half the beam length was modeled. A transient heat transfer analysis was conducted on each model in the same way as the uncoupled heat transfer analysis described in Section 3.3. Nodal temperatures of a model during heating were stored in a result file and imported into the sequential stress analyses.

The structural models were constructed using solid stress/displacement element C3D8I. The mesh and node numbers of a structural model were identical to that of a corresponding heat transfer model in order to extract the temperature profile from a result file of a heat transfer analysis. The EC3 material model for hot-rolled structural steels at an elevated temperature, which has been presented in Section 3.2.2, was used in the analyses. Boundary and loading conditions of a structural model are illustrated in

Figure 3.31. Vertical constraints were applied on one end, and the other end was defined as the line of symmetry. In order to prevent lateral torsional buckling, lateral supports were applied along the joint between the compression flange and the web. A two-step static stress analysis was conducted on each structural model. At the first step, boundary conditions were applied, elements that represent the fire protection were removed, and the temperature profile at a given exposure time was imported. At the second step, a uniform distributed load was added on the top flange of the beam. The Riks method, a modified arc length method according to Riks (1979), was used at the second step to predict the collapse of the beam.

A moment versus displacement curve was plotted after each stress-displacement analysis to show the change in the mid-span moment during the loading process. For example, Figure 3.32 shows the moment-displacement curve for a W310×39 beam with 1-hour fire resistance rating after 40 minutes standard fire exposure. The peak value of the moment-displacement curve is the cross-section moment capacity of the beam.

3.5.2 Mesh Study

The mesh study was carried out on a W310×39 steel beam using the static stress analysis at a normal temperature of 20°C. The study focused on the layout of the mesh in the longitudinal direction. Three different meshes, as shown in Figure 3.33, were investigated. Mesh A has a uniform longitudinal spacing of 50 mm. For Mesh B and Mesh C, the longitudinal spacing close to the mid-span region was respectively changed

to 25 mm and 12.5 mm. Only the mesh near the mid-span was changed because flexural failure was expected to occur at mid-span. Moment-displacement curves for the three meshes are shown in Figure 3.34. It can be observed that all three curves follow the same path before they reach their peak values, and the peak value differences are very small. But in the sequentially coupled thermal-stress analysis, the mesh of a structural model has to be identical to that of a corresponding heat transfer model. Thus, the mesh used should not be coarser than that of the three-dimensional heat transfer models which have been described in Section 3.3.3.2. Both Mesh B and Mesh C met this requirement. Mesh C was adopted in the sequentially coupled thermal-stress analysis since the difference in computing time between Mesh B and Mesh C was negligible in the analysis.

Table 3.1 Geometrical properties of steel sections

Designation	Depth	Flange Width	Flange Thickness	Web Thickness	Distance	Section Area	Surface Area per meter length
	d	b	t	w	a		
	mm	mm	mm	mm	mm	mm ²	m ²
W410x74	413	180	16.0	9.7	85	9456	1.53
W410x60	407	178	12.8	7.7	85	7494	1.51
W310x60	303	203	13.1	7.5	98	7395	1.40
W310x39	310	165	9.7	5.8	80	4886	1.27
W250x24	253	145	6.4	5.0	70	3057	1.08

Table 3.2 Thickness of fire protection

Designation	Weight	Surface (heated) Perimeter	Massivity Factor	Thickness of Protection	
	W	D	W/D	d _i	
	kg/m	m	kg/m ²	mm	
				1 hour	2 hour
W410x74	74.9	1.53	49.0	16.0	32.0
W410x60	59.5	1.33	44.7	17.0	34.0
W310x60	59.6	1.40	42.6	17.5	35.0
W310x39	38.4	1.27	30.2	20.5	41.0
W250x24	24.0	1.08	22.3	23.0	46.0

Table 3.3 Beams of W410×74 section

Label of model		Location of protection loss	a_l cm	b_l cm	$a_l \times b_l$ cm ²	Penetration of the damage %
Unp		Unprotected				
1	2	Fully protected				
1BF1	2BF1	Bottom Flange	10	10.0	100.0	100.0
1BF1b	2BF1b		10	10.0	100.0	50.0
1BF1c	2BF1c		10	10.0	100.0	75.0
1BF1d	2BF1d		10	10.0	100.0	87.5
1BF2	2BF2		10	5.0	50.0	100.0
1BF3	2BF3		20	10.0	200.0	100.0
1BF3d	2BF3d		20	10.0	200.0	87.5
1BF4	2BF4		10	18.0	180.0	100.0
1BF5	2BF5		5	18.0	90.0	100.0
1BF6	2BF6		10	5.0	50.0	100.0
1BF7	2BF7		20	1.6	32.0	100.0
1BF8	2BF8		18	10.0	180.0	100.0
1BF9	2BF9		20	18.0	360.0	100.0
1BF10	2BF10		30	18.0	540.0	100.0
1BF10d	2BF10d		30	18.0	540.0	87.5
1BF11	2BF11		L	18.0	L x 18.0	100.0
1TF1	2TF1	Top Flange	10	10.0	100.0	100.0
1TF1b	2TF1b		10	10.0	100.0	50.0
1TF1c	2TF1c		10	10.0	100.0	75.0
1TF1d	2TF1d		10	10.0	100.0	87.5
1TF2	2TF2		10	5.0	50.0	100.0
1TF3	2TF3		20	10.0	200.0	100.0
1TF3d	2TF3d		20	10.0	200.0	87.5
1TF4	2TF4		10	18.0	180.0	101.0
1TF5	2TF5		5	18.0	90.0	100.0
1TF6	2TF6		10	5.0	50.0	100.0
1TF7	2TF7		20	1.0	19.4	100.0
1TF8	2TF8		18	10.0	180.0	100.0
1TF9	2TF9		20	18.0	360.0	100.0

Table 3.3 (Continued)

Label of model		Location of protection loss	a_l cm	b_l cm	$a_l \times b_l$ cm^2	Penetration of the damage %
1TF10	2TF10	Top Flange	30	18.0	540.0	100.0
1TF10d	2TF10d		30	18.0	540.0	87.5
1TF11	2TF11		L	18.0	$L \times 18.0$	100.0
1W1	2W1	Web	10	10.0	100.0	100.0
1W2	2W2		10	10.0	100.0	100.0

Naming convention:

(Fire resistance rating) (Location of protection loss) (Case Number)

Initial fire resistance rating: 1 hour (1) and 2 hour (2)

Location of protection loss: bottom flange (BF), top flange (TF) and web (W)

L is the length of the beam.

Table 3.4 Beams of W410×60 section

Label of specimen		Location of protection loss	a_l cm	b_l cm	$a_l \times b_l$ cm ²	Penetration of the damage %
1	2	Fully protected				
1BF1	2BF1	Bottom flange	10	10	100	100
1TF1	2TF1	Top flange	10	10	100	100

Table 3.5 Beams of W310×60 section

Label of specimen		Location of protection loss	a_l cm	b_l cm	$a_l \times b_l$ cm ²	Penetration of the damage %
1	2	Fully protected				
1BF1	2BF1	Bottom flange	10	10	100	100
1TF1	2TF1	Top flange	10	10	100	100

Table 3.6 Beams of W390×39 section

Label of specimen		Location of protection loss	a_l cm	b_l cm	$a_l \times b_l$ cm ²	Penetration of the damage %
1	2	Fully protected				
1BF1	2BF1	Bottom flange	10	10	100	100
1TF1	2TF1	Top flange	10	10	100	100

Table 3.7 Beams of W250×24 section

Label of specimen		Location of protection loss	a_l cm	b_l cm	$a_l \times b_l$ cm ²	Penetration of the damage %
1	2	Fully protected				
1BF1	2BF1	Bottom flange	10	10	100	100
1TF1	2TF1	Top flange	10	10	100	100

Table 3.8 Thermal properties of spray-on mineral fiber material, from ECCS (1995)

Density ρ_i (kg/m ³)	Thermal conductivity λ_i (W/mK)	Specific heat c_i (J/kgK)	Equilibrium moisture content %
300	0.12	1200	1

Table 3.9 Thermal properties of spray-on fire protection, from Ryder et al. (2002)

Temperature T °C	Density ρ_i (kg/m ³)	Thermal conductivity k_i (W/mK)	Specific heat c_i (J/kgK)
20	240	0.0598	1090
205		0.0598	1090
400		0.1160	1270
1090		0.2890	1460

Table 3.10 Reduction factors for stress-strain relationship of steel at elevated temperatures, from EC3 (1995)

Steel temperature	Reduction factor for effective yield strength	Reduction factor for proportional limit	Reduction factor for the modulus of elasticity
T (°C)	$k_{y,T}=f_{y,T}/f_y$	$k_{p,T}=f_{p,T}/f_p$	$k_{E,T}=E_T/E$
20	1.0000	1.0000	1.0000
100	1.0000	1.0000	1.0000
200	1.0000	0.8070	0.9000
300	1.0000	0.6130	0.8000
400	1.0000	0.4200	0.7000
500	0.7800	0.3600	0.6000
600	0.4700	0.1800	0.3100
700	0.2300	0.0750	0.1300
800	0.1100	0.0500	0.0900
900	0.0600	0.0375	0.0675
1000	0.0400	0.0250	0.0450
1100	0.0200	0.0125	0.0225
1200	0.0000	0.0000	0.0000

Table 3.11 Section classification for Grade 350 steel at normal temperatures

Section element	Section classification		
	Class 1	Class 2	Class 3
Flange	$\frac{b}{t} \leq \frac{145}{\sqrt{F_y}} = 7.8$	$\frac{b}{t} \leq \frac{170}{\sqrt{F_y}} = 9.1$	$\frac{b}{t} \leq \frac{200}{\sqrt{F_y}} = 10.7$
Web	$\frac{h}{w} \leq \frac{1100}{\sqrt{F_y}} = 58.8$	$\frac{h}{w} \leq \frac{1700}{\sqrt{F_y}} = 90.9$	$\frac{h}{w} \leq \frac{1900}{\sqrt{F_y}} = 101.6$

Table 3.12 Values of the constant c for Grade 350 steel

Section element	c		
	Class 1	Class 2	Class 3
Flange	0.32	0.38	0.45
Web	2.46	3.80	4.25

Table 3.13 Section classes of the studied steel sections at normal temperatures

Steel section	Section element	Width to thickness ratio of the section element			Section class
		Class 1	Class 2	Class 3	
W410x74	Flange	5.6			1
	Web	42.6			
W410x60	Flange	7.0			1
	Web	52.9			
W310x60	Flange	7.7			1
	Web	40.4			
W310x39	Flange		8.5		2
	Web	53.4			
W250x24	Flange			11.3	3
	Web	50.6			

Figure 3.1 Steel section

Figure 3.2 Steel section uniformly protected with spray-on material

Figure 3.3 Symmetric protection loss on the flange (1BF1-5, 1BF8-11, 2BF1-5, 2BF8-11, 1TF1-5, 1TF8-11, 2TF1-5, 2TF8-11)

Figure 3.4 Asymmetric protection loss on the flange (1BF6, 2BF6, 1TF6, 2TF6)

Figure 3.5 Asymmetric protection loss on the flange (1BF7, 2BF7, 1TF7, 2TF7)

Figure 3.6 Protection loss on the web, symmetric to mid-height (1W1, 2W1)

Figure 3.7 Protection loss on the web, below mid-height (1W2, 2W2)

Figure 3.8 A simply supported beam with protection loss at the mid-span

Figure 3.9 Specific heat of steel as a function of temperature (generated from EC3 1995)

Figure 3.10 Thermal conductivity of steel as a function of temperature (generated from EC3 1995)

Figure 3.11 Thermal conductivity of a spray-on fire protection material as a function of temperature (generated from Ryder et al. 2002)

Figure 3.12 Stress-strain relationship for steel at elevated temperatures (modified from EC3 1995)

Figure 3.13 Reduction factors for the stress-strain relationship of steel at elevated temperatures (generated from EC3 1995)

Figure 3.14 Yield strength of structural steel S355 at elevated temperatures (modified from Outinen et al. 2001)

Figure 3.15 Modulus of elasticity of structural steel at elevated temperatures (modified from Outinen et al. 2001)

Figure 3.16 Time-temperature curve of the ISO 834 standard fire

Figure 3.17 Structure and boundary conditions of a two-dimensional heat transfer model

Figure 3.18 Four meshes for the two-dimensional heat transfer models

Figure 3.19 Comparison of predicted temperature distributions on the lower surface of the bottom flange between different meshes

Figure 3.20 Three-dimensional finite element heat transfer model of a beam with the bottom flange protection loss

a) Mesh A

b) Mesh B

c) Mesh C

Figure 3.21 Longitudinal spacing of three meshes for the three-dimensional heat transfer model

Figure 3.22 Selected locations in W410×74 section for the longitudinal mesh study

a) Position 1

b) Position 2

Figure 3.23 Comparison of predicted longitudinal temperature distributions of selected locations between different meshes

Figure 3.24 Predicted longitudinal temperature distributions after 2 hours standard fire exposure for a W410×74 beam subjected to partial protection loss

a) Position 1

b) Position 2

Figure 3.25 Comparison of predicted longitudinal temperature distributions of selected locations between different model lengths

Figure 3.26 Average temperature of a W10x49 steel column with 19 mm of protection

Figure 3.27 Average temperature of an H-300x300x10x15 steel column with 22.3 mm of protection

Figure 3.28 Temperatures of the bottom flange (BF), top flange (BF) and web (W) for an H-400x200x8x13 steel beam with 11.3 mm of protection

Figure 3.29 Strain and stress distributions in a steel beam cross-section

Figure 3.30 Flow chart for calculating the moment capacity of steel sections

Figure 3.31 Boundary and loading conditions of a structural model

Figure 3.32 Predicted moment versus displacement relationship at the mid-span of a W310×39 steel beam after 40 minutes standard fire exposure

a) Mesh A

b) Mesh B

c) Mesh C

Figure 3.33 Longitudinal spacing of three meshes for the sequentially coupled thermal-stress models

Figure 3.34 Predicted moment versus displacement relationships at the mid-span for different meshes

Chapter 4 Results and Discussions

In this chapter, the effect of partial protection loss on the moment capacity reduction for steel beams exposed to the ISO 834 standard fire is discussed. Comparisons of different factors, such as damage area, damage shape, damage location, depth of protection loss and weight of steel section, are made based on cross-section moment capacities obtained by analytical calculations. In addition, a few results obtained by finite element structural analyses are also discussed.

4.1 Cross-Section Moment Capacity by Analytical Calculation

When a protected steel member with partial fire protection loss is exposed to a fire, cross-sections within or in the vicinity of the damage may experience significant reduction in moment capacity due to the increase in steel temperature. This section specifically discusses the reduction of moment capacity at the mid-span cross-section of a beam. The reduction of cross-section moment capacities along the length of a beam is also discussed. In addition, results associated with constant and varied thermal properties of the fire protection material are compared. The cross-section moment capacities were obtained using numerical heat transfer analyses and analytical structural analyses.

4.1.1 Moment Capacity at the Mid-Span

Reductions of cross-section moment capacity at the mid-span of beams during the

standard fire exposure are discussed in this section. As mentioned in Chapter 3, the damaged protection area in the model is symmetric about the mid-span of the beam. Therefore, when the beam is exposed to fire, the mid-span is expected to have the highest temperature profile and experience the largest reduction of moment capacity. The cross-section moment capacity reduction R_m is defined as

$$R_m = 1 - M_r \quad \text{with} \quad (4.1)$$

$$M_r = \frac{M_f}{M_0}, \quad (4.2)$$

where M_r is the relative moment capacity, M_f is the cross-section moment capacity at elevated temperatures and M_0 is the cross-section moment capacity at normal temperatures.

To compare the impact of protection loss on fire resistance, the cross-section moment capacity corresponding to an exposure time equal to the fire resistance rating of the fully protected beam, as shown in Figure 4.1, is used as reference. The reduction of fire resistance F_r is defined as

$$F_r = 1 - R_r \quad \text{with} \quad (4.3)$$

$$R_r = \frac{R_f}{R_{f,0}} \quad (4.4)$$

where R_r is the relative fire resistance, R_f is the exposure time for the moment capacity of a beam with protection loss to drop to the reference value and $R_{f,0}$ is the fire resistance rating of the fully protection beam. The relative cross-section moment capacities for all studied beams at 1 or 2 hours of standard fire exposure and the relative fire resistances of

these beams are listed in Tables 4.1 to 4.5.

4.1.1.1 W410x74 Beams

The effect of partial protection loss on the reduction of the mid-span cross-section moment capacity for W410×74 beams during the ISO 834 standard fire exposure is discussed in the following sections. As shown in Table 3.13, the W410×74 section is a Class 1 section at normal temperatures.

- Unprotected and Fully Protected Beams

Figure 4.1 presents the relative cross-section capacities for unprotected and fully protected W410×74 beams during the ISO 834 standard fire exposure. It can be seen that a fully protected steel beam exhibits a good performance throughout the duration of its designed fire resistance rating time. The cross-section moment capacity of the 1-hour rating beam reduces by only about 15% after 1 hour of fire exposure, and that for the 2-hour rating beam reduces by only about 20% after 2 hours of fire exposure. These values are used as the reference values for comparing the fire resistance rating of beams with protection loss. The unprotected beam, however, has a very low fire resistance. The cross-section moment capacity of the unprotected specimen drops to about 45% of its initial value after 15 minutes of standard fire exposure and to below 20% after 25 minutes.

- Beams with Symmetric Protection Loss on the Bottom Flange

The relative cross-section moment capacities for fully protected W410×74 beams and W410×74 beams with symmetric bottom flange protection loss are plotted against fire-exposure time in Figures 4.2 and 4.3, respectively for 1- and 2-hour original fire resistance ratings. The original fire resistance rating is defined as the fire resistance rating of a protected beam before its fire protection coating is damaged. The results clearly show that the cross-section moment capacity of a beam with partial protection loss reduces faster than that of its fully protected counterpart subjected to the standard fire exposure. The temperature contour of a 1-hour original rating beam with a 20 cm × 10 cm size symmetric bottom flange protection loss (1BF3) after 1 hour of standard fire exposure is shown in Figure 4.4. The temperature profile at the damage region is shown to be much higher than that located away. Obviously, it is the high local temperature profile at the mid-span cross-section that causes a greater reduction in the moment capacity.

Figures 4.2 and 4.3 also show that the reduction of cross-section moment capacity strongly depends on the area of protection damage. Generally, a larger damage area results in a greater reduction of the moment capacity. For example, the cross-section moment capacity of beam 1BF2 (50 cm² protection loss) remains 65% of its initial value after 1 hour of standard fire exposure, while that of beam 1BF3 (200 cm² protection loss) drops to 39% of its initial value after the same exposure period.

The reduction of the cross-section moment capacity is not only affected by the

area of protection loss but also by the shape of the damage. The influence of damage shape can be found by comparing the results between beams 2BF4 and 2BF8 in Figure 4.5. According to Table 3.3, both beams 2BF4 and 2BF8 have a protection loss area of 180 cm^2 , while the width to length ratio of the damage is 1.8 for beam 2BF4 and 0.56 for beam 2BF8. It can be seen that the cross-section moment capacity of beam 2BF4 decreases faster than that of beam 2BF8 during the first 100 minutes, but decrease at nearly the same rate thereafter. A similar characteristic can be found by comparing the results between beams 1BF4 and 1BF8 in Figure 4.6, although the fire-exposure time is not long enough for the moment capacity reduction of beam 1BF8 to catch up with that of beam 1BF4. The reason is related to the diffusion of heat entering from the damage area. At the early stage of the fire exposure, the fire temperature rises very fast, and the temperature of the bottom flange rises rapidly before the heat entering from the damage area diffuses away. Therefore, in the mid-span cross-section, a wider damage leads to a faster increasing rate of the bottom flange temperature due to a wider opening in this cross-section, resulting in a faster reduction rate of moment capacity. After a certain time of fire exposure, however, the fire temperature rises very slowly, and the temperature profile of the mid-span cross-section becomes more uniform. Thus, the influence of damage shape on the moment capacity reduction diminishes. These phenomena indicate that a larger width to length ratio of protection damage results in a greater reduction in the moment capacity during early stages of standard fire exposure, but this effect becomes smaller for longer fire-exposure time. Overall, the shape of the

damage has much less influence than the area of the damage.

The reduction of the cross-section moment capacity is also affected by protection thickness. Comparing results between beams with different protection thicknesses, for example, 1BF2 and 2BF2 in Figures 4.2 and 4.3, it is found that the beam with a thicker fire protection has a smaller reduction of the cross-section moment capacity than the one with a thinner fire protection at the same fire-exposure time. However, it should be noted that the former has a greater final cross-section moment capacity reduction than the latter at their originally designed fire resistance rating times of 1 hour and 2 hours respectively.

Relative fire resistances of fully protected W410×74 beams and W410×74 beams with symmetric bottom flange protection loss at the mid-span are plotted in Figures 4.7 and 4.8. Similar to the reduction of moment capacity, the fire resistance is strongly dependent on the area of protection loss and to a lesser extent on the effect of damage shape.

- **Beams with Asymmetric Protection Loss on the Bottom Flange**

A comparison is made on the effect of damage location for W410×74 beams with the bottom flange protection loss. Reductions of the cross-section moment capacity associated with symmetric and asymmetric bottom flange protection loss are compared in Figures 4.9 and 4.10. The comparison indicates that the damage location on the bottom flange does affect the reduction of the cross-section moment capacity. Taking beams

2BF2 and 2BF6 for instance, the protection losses of beams 2BF2 and 2BF6 are identical in size and shape but vary in location. As illustrated in Figure 4.9, the cross-section moment capacity of beam 2BF6 starts to drop 10 minutes prior to that of beam 2BF2. However, the moment capacity reduction of beam 2BF2 starts to exceed that of beam 2BF6 after 65 minutes of fire exposure, and the relative moment capacity of beam 2BF2 ends up 8% lower than that of beam 2BF6 at the end of the fire exposure. A similar characteristic can be observed when comparing the results between beams 1BF2 and 1BF6 in Figure 10. It should be noted that the effect of damage location on the moment capacity reduction is not significant for beams with bottom flange protection loss within 2 hours of standard fire exposure.

This phenomenon can be explained by the temperature profiles of the beams. Figure 4.11 compares the temperature profile along the bottom flange for beams 2BF2 and 2BF6 after 40 minutes of fire exposure. It can be observed that beam 2BF6 has a much higher local temperature at the damage region, and its local temperature exceeds 400°C (the temperature at which the yield strength of steel starts to drop) earlier than that of beam 2BF2. Since the cross-section moment capacity of W410×74 beams is calculated based on the yield strength of each element in the cross-section, a higher local temperature profile results in an earlier drop of the moment capacity. Compared to the symmetric protection loss, protection loss on the edge of the flange causes a faster rise of the local temperature because the damage is located away from the web so that the heat sink effect is smaller. On the other hand, a symmetric protection loss causes a higher

overall temperature profile in the cross-section than the asymmetric protection loss because the heat sink effect is greater. Figure 4.12 compares the temperature profile along the web for beams 2BF2 and 2BF6 after 80 minutes of fire exposure. The temperature profile of beam 2BF2 is higher than that of beam 2BF6. This comparison suggests that beam 2BF2 has a higher overall temperature profile in the cross-section. If the fire-exposure time is long enough, cross-sections with a higher overall temperature profile will also have a greater overall yield strength reduction, and will result in a greater reduction of the moment capacity.

Among beams with different locations of bottom flange protection loss, it can be seen that beams 1BF7 and 2BF7 have the lowest reduction of moment capacity because these two beams have the smallest area of protection loss. Relative fire resistances of W410×74 beams with symmetric and asymmetric bottom flange protection loss are plotted in Figures 4.13 and 4.14. The influence of the protection loss location on the fire resistance can be observed, but it is not significant.

- Fire Resistance Reduction with Respect to the Area of Protection Loss

For W410×74 beams with the bottom flange protection loss, since the fire resistance reduction strongly depends on the area of protection loss, and effects of the damage shape and damage location are not significant, it is possible to characterize the fire resistance reduction using simple equations with respect to the area of protection loss. Two equations are proposed using the analytical results to describe the fire resistance

reduction of W410×74 beams with the bottom flange protection loss with respect to the area of protection loss respectively for 1- and 2-hour original fire resistance ratings. For 1-hour original fire resistance rating, the fire resistance reduction is given by

$$F_r = 0.635 - \frac{508}{(800 - A_L^{1.4734})}, \quad (4.5)$$

where A_L is the area of the protection loss. For 2-hour original fire resistance rating, the fire resistance reduction is given by

$$F_r = 0.79 - \frac{316}{(400 - A_L^{1.4379})}. \quad (4.6)$$

Figures 4.15 and 4.16 show the fire resistance reductions from (4.5) and (4.6) along with the results of analyses. For W410×74 beams with the bottom flange protection loss at the mid-span, the fire resistance reduces rapidly with the area of the damage when the area is less than about 500 cm². The reduction rate reduces with a greater damage area. The fire resistance reduction approaches a limit when the protective coating at the bottom is completely removed. This limit is about 0.635 for 1-hour original fire resistance rating W410×74 beams and about 0.79 for 2-hour original fire resistance rating W410×74 beams. It should be noted that (4.5) and (4.6) only apply to W410x74 beams with the flange protection loss.

- Beams with Protection Loss on the Top Flange

For W410×74 beams, protection loss on the top flange is found to have identical effect on the reduction of the cross-section moment capacity as protection loss on the

bottom flange. The reason is that the cross-section of a W410×74 section is able to attain the plastic moment capacity at elevated steel temperatures regardless of whether the protection loss is on the bottom or top flange. According to the moment capacity calculation method discussed in Section 3.4.1, protection losses on the top flange and on the bottom flange are expected to have different effects on the reduction of the cross-section moment capacity under two circumstances. One is that section class changes occur (from Class 1, 2 to Class 3 or from Class 3 to Class 4) during the fire exposure. Because the section class limit is related to the mechanical properties of the compression (top) flange, a top flange protection loss will cause an earlier class change than a bottom flange protection loss, resulting in an earlier drop of the moment capacity reduction curve. The other is that the beam is a Class 4 section at normal temperatures, and its moment resistance is governed by local buckling of the compression (top) flange. In that case, a top flange protection loss will cause more severe moment capacity reduction than a bottom flange protection loss.

- **Beams with Protection Loss on the Web**

Figures 4.17 and 4.18 compare the reduction of the cross-section moment capacity for W410×74 beams with protection loss on the web and those with protection loss on the bottom flange. The comparison shows that the web protection damage has a smaller effect on the reduction of the cross-section moment capacity than the flange protection damage. Moreover, among beams with the web protection loss, a protection

loss at the mid-height of the web (1W1 and 2W1) has a smaller effect on the moment capacity reduction than a protection loss close to the flange (1W2 and 2W2). This is to be expected since the difference in the moment capacity is related to the contribution of different regions of a cross-section to the cross-section moment capacity. It is clear that the flange has a greater contribution to the cross-section moment capacity than the web. Thus, the weakening of the flange results in a greater reduction in moment capacity than the weakening of the web. Similarly, beams with the web protection loss closer to the flange have a greater effect on the moment capacity reduction than that located away.

Relative fire resistances of W410×74 beams with the web protection loss and those with the bottom flange protection loss are compared in Figures 4.19 and 4.20. It can be clearly observed that the bottom flange protection loss causes more reduction in fire resistance than the web protection loss.

- Effect of the Damage Penetration

An investigation was conducted on the effect of the penetration of the protection damage on the reduction of the cross-section moment capacity. The investigation was carried out on W410×74 beams with a symmetric protection loss of 10 cm × 10 cm size on the bottom flange. The damage penetration is respectively equal to 100%, 87.5%, 75%, or 50% of the designed protection thickness of a beam. Reductions of the cross-section moment capacity are compared in Figures 4.21 and 4.22. It can be seen that a deeper damage penetration results in a larger reduction of the cross-section moment

capacity due to a standard fire exposure.

An interesting result is observed that the effect of protection loss on the reduction of the cross-section moment capacity is relatively small if the damage does not fully penetrate the protective coating, even when the remaining protection at the damage area is very thin. Taking beam 1BF1d (with a damage of 87.5% of the protective coating) as an example, as shown in Figure 4.21, the relative cross-section moment capacity after 1 hour of standard fire exposure is only about 15% lower than that of the fully protected beam. The reason may partly be attributed to the great difference between the conductivity of steel and the protective material, with the conductivity of steel being about 400 times higher. Therefore, the heat can diffuse in the steel section at a much faster rate than crossing through the remaining protective coating at the damage area. As a result, a relatively low temperature profile in the steel cross-section can still be maintained with a thin remaining protection compared to 100% damage penetration.

To further investigate the moment capacity reduction for beams with a thin remaining protection at the damage area, more analyses were carried out on W410×74 beams with different protection loss areas. Figures 4.23 and 4.24 compare the moment capacity reduction for fully protected W410×74 beams and W410×74 beams with different protection loss areas and with an 87.5% penetrated protection damage on the bottom flange. It can be seen that the moment capacity reduction caused by protection loss becomes significant when the protection loss area is large enough, even though there is still a thin remaining protective coating at the damage area. For example, as shown in

Figure 4.23, the relative cross-section moment capacity of beam 1BF10d remains only 46% after 1 hour of standard fire exposure, and it is 39% lower than that of the fully protected beam.

Relative fire resistances of W410×74 beams with a 10 cm × 10 cm size symmetric protection loss on the bottom flange and with different damage penetrations are presented in Figures 4.25 and 4.26, and relative fire resistances of W410×74 beams with an 87.5% penetrated protection loss on the bottom flange are presented in Figures 4.27 and 4.28. These results suggest that beams with a thin remaining protective coating at the damage area are able to retain much of the fire resistance comparable to a fully protected beam if the damage area is small, but the reduction of fire resistance becomes significant when the damage area is large enough.

4.1.1.2 Beams of Different Sections

In order to compare the effect of protection loss among beams of different steel sections, reductions of the cross-section moment capacity of beams associated with W410×60, W310×60, W310×39 and W250×24 sections were calculated. At normal temperatures, as shown in Table 3.13, W410×60 and W310×60 are Class 1 sections, W310×39 is a Class 2 section, and W250×24 is a Class 3 section. For these four steel sections, only the fully protected beams and beams with symmetric bottom or top flange protection loss were studied.

- Beams with Full Protection

Figures 4.29 and 4.30 show reductions of the cross-section moment capacity for fully protected W410×60, W310×60, W310×39 and W250×24 beams with 1- and 2-hour fire resistance ratings. A sudden drop of the curve is observed for W310×39 and W310×60 beams in both Figures 4.29 and 4.30. This drop is due to the class change of the steel section at elevated temperatures. At normal temperatures, W310×39 is a Class 2 section, and W310×60 is a Class 1 section. At elevated temperatures, since the elastic modulus of steel reduces faster than the yield strength, according to (3.29), the limit for the Class 2 section decreases. As a result, W310×39 and W310×60 drop to Class 3 sections at certain steel temperatures. Besides, it can be observed that reductions of the cross-section moment capacity are different between different beams with the same fire resistance rating. This phenomenon may be caused by two reasons: one is the accuracy of (3.3), which determines the thickness of fire protection, and the other is the section class change during the fire exposure.

- Beams with Symmetric Protection Loss on the Flange

The relative cross-section moment capacities for beams with symmetric protection loss on the bottom or top flange are plotted against fire-exposure time in Figures 4.31 to 4.38 respectively for steel beams of different sections and with different original fire resistance ratings. Cross-section class changes are again observed for W310×39 and W310×60 beams. Taking W310×39 beams for example, as shown in

Figures 4.31 and 4.32, a W310×39 beam with top flange protection loss experience earlier class change than that with bottom flange protection loss because the compression (top) flange heats up faster for beams with top flange protection loss. However, once W310×39 beams with either bottom or top flange protection loss has changed in class, reductions of the moment capacity become identical. A similar characteristic can be observed on W310×60 beams in Figures 4.33 and 4.34. On the other hand, the other beams show identical cross-section moment capacity reduction regardless whether the protection loss is on the top flange or on the bottom flange, and no sudden drop of the curve is observed. This indicates that during the entire heating period, W410×60 beams can still attain the plastic moment capacity, and W250×24 beams can still attain the yield moment capacity.

- Effect of the Bottom Flange Protection Loss on the Moment Capacity Reduction for Beams of Different Sections

The effect of the bottom flange protection loss on the moment capacity reduction was compared among beams of different sections. Comparisons were made with respect to two parameters: the section depth d , and the weight per unit length W . Values of d and W for each section are presented in Tables 3.1 and 3.2. Since reductions of the cross-section moment capacity are different between fully protected beams of different sections but with the same fire resistance rating, a modified relative moment capacity is used in the comparison in order to make the effect of the protection loss comparable.

The modified relative moment capacity M'_r is taken as the ratio of the cross-section moment capacity of a beam with protection loss to that of its fully protected counterpart, given as

$$M'_r = \frac{M_f}{M_{f,F}}, \quad (4.7)$$

where M_f is the cross-section moment capacity of a beam at elevated temperatures and $M_{f,F}$ is the cross-section moment capacity of its fully protected counterpart at elevated temperatures. Since W310×60 and W310×39 sections change in class but W410×60 and W250×24 sections do not, class changes of the steel section during the fire exposure are ignored in order to make the moment capacity comparable.

Figures 4.39 and 4.40 compare reductions of the cross-section moment capacity between W410×60 and W310×60 beams with symmetric protection loss of 10 cm × 10 cm size on the bottom flange. W410×60 and W310×60 beams have similar weights per unit length, but the depth of a W410×60 beam is 34% higher than that of a W310×60 beam. It can be seen that the moment capacity reductions of W310×60 beams are initially smaller, but they become greater than that of W410×60 beams in longer fire-exposure time, and the difference of the moment capacity reduction between W410×60 and W310×60 beams increases with time thereafter. Overall, the difference of the moment capacity reduction is more significant for 2-hour original rating beams than for 1-hour original rating beams.

Figures 4.41 and 4.42 compare reductions of the cross-section moment capacity

between W410×74 and W410×60 beams with symmetric protection loss of 10 cm × 10 cm size on the bottom flange. W410×74 and W410×60 beams have similar depths, but the weight per unit length of a W410×74 beam is 26% higher than that of a W410×60 beam. It can be seen that the moment capacity reductions of W410×60 beams are greater than that of W410×74 beams. Similarly, in Figures 4.43 and 4.44, the moment capacity reductions of W310×39 beams are also found to be greater than that of W310×60 beams. These comparisons indicate that a beam of a lighter section has a greater reduction of the moment capacity in a fire than that of a heavier section if they both have an identical protection loss. This phenomenon is due to the heat capacity difference between a lighter section and a heavier section. Another phenomenon is that the difference of the moment capacity reduction between a lighter beam and a heavier beam tends to decrease in longer fire-exposure time. Taking beams W410×60-2BF1 and W410×60-2BF1 for example, the difference of the modified relative moment capacity between these two beams is about 11% at 70 minutes of fire exposure, but it reduces to only about 4% at 120 minutes. It can be observed in Figures 4.41 to 4.44 that the difference of the moment capacity reduction between a lighter beam and a heavier beam is more significant for 1-hour original rating beams than for 2-hour original rating beams. In summary, the above comparisons show that parameters W and d do affect the reduction of the cross-section moment capacity for beams with the flange protection loss. This means that (4.5) and (4.6) have to be modified for different beam sections.

4.1.2 Cross-Section Moment Capacity along the Length

The influence of protection loss along the length of beams was investigated. Figures 4.45 and 4.46 present the modified cross-section moment capacities along the length respectively for 1-hour original fire resistance rating W410×74 beams with symmetric bottom flange protection loss after 1 hour of standard fire exposure and for 2-hour original fire resistance rating W410×74 beams with the same level of protection loss after 2 hours of standard fire exposure. It is noted that the origin of the abscissa in Figures 4.45 and 4.46 represents the centerline of the damage, and its value represents the longitudinal distance from the damage center along the length.

The results confirm that the largest reduction of the moment capacity occurs on the mid-span of a beam. It can be seen that the effect of partial protection loss on the reduction of the moment capacity is very significant at cross-sections within the damage area. However, the effect of protection loss decreases rapidly with distance from the damage area. Taking beam 1BF4 for example, as shown in Figure 4.45, the modified relative cross-section moment capacity is only 0.46 at the edge of the damage area (50 mm from the damage center), and it increases to 0.89 at 200 mm from the damage center.

The results also show that the aspect ratio of the damage area has some effect on the cross-section moment capacity reduction along the length. For example, beams 1BF4 and 1BF8 have an identical protection loss area of 180 cm², yet the length to width ratio of the damage is 0.56 for beam 1BF4 and 1.8 for beam 1BF8. As shown in Figure 4.45, the protection damage with a larger length to width ratio (1BF8) causes more

reduction in the cross-section moment capacity at the vicinity of its damage area than the one with a lower length to width ratio (1BF4). The difference of the cross-section moment capacity between beams 1BF4 and 1BF8 at the vicinity of their damage areas is greater than that at the damage center. A similar characteristic can be found by comparing the results between beams 2BF4 and 2BF8 in Figure 4.46. The cause of this phenomenon can be explained by looking at the distribution of the modified relative moment capacity with respect to the shape of the protection loss. It can be seen that the effect of protection loss on the moment capacity reduction is very significant at cross-sections within the damage area but decreases rapidly with the distance. Since the damage area of beams 1BF8/2BF8 is 8 cm longer than that of beams 1BF4/2BF4 in the length direction, at distances between the edges of the damage areas of beams 1BF4/2BF4 and 1BF8/2BF8, the moment capacity of beams 1BF4/2BF4 increases rapidly while there is little change for beams 1BF8/2BF8. As a result, the cross-section moment capacity of beams 1BF4/2BF4 is obviously higher than that of beam 1BF8/2BF8 at the vicinity of their damage areas.

4.1.3 Temperature-Dependent Thermal Conductivity of the Fire Protection Material

Constant thermal properties of the spray-on fire protection material were used in this study to predict the temperature profiles of beams exposed to fire. However in practice, the thermal conductivity and the specific heat of spray-on fire protection materials are temperature-dependent. Hence, an attempt was made to correlate the

results of constant and varied thermal properties of the fire protection material. The temperature-dependent thermal conductivity listed in Table 3.9 was used in this investigation, and a constant specific heat of the fire protection material recommended by ECCS (1995) was used since the heat capacity of a low density protective material does not significantly affect the temperature rise of the steel section.

The investigation started with fully protected W410×74 beams. Figures 4.47 and 4.48 present the relative cross-section moment capacities for fully protected W410×74 beams with a varied protective material thermal conductivity. Two constant thermal conductivities, 0.16 (W/mK) and 0.18 (W/mK), were adopted to represent the varied conductivity respectively for beams with 1- and 2-hour fire resistance ratings. These constant values are determined by making the final moment capacity reductions identical for both with a constant and with a varied protective material thermal conductivity, as shown in Figures 4.47 and 4.48.

Figures 4.49 and 4.50 compare the reductions of the cross-section moment capacity associated with constant and varied protective material thermal conductivity for beams with symmetric protection loss on the bottom flange. It can be seen that final reductions of moment capacity are identical between a beam associated with a varied protective material thermal conductivity and its counterpart with a constant protective material thermal conductivity. Besides, the moment capacity associated with the constant thermal conductivity is always conservative.

The fire resistances of W410×74 beams with symmetric bottom flange protection

loss and associated with constant and temperature-dependent protective material thermal conductivity are presented in Figures 4.51 and 4.52. The fire resistance associated with a constant protective material thermal conductivity is slightly more conservative than that associated a temperature-dependent protective material thermal conductivity.

4.2 Maximum Mid-Span Moment by Numerical Modelling

Results of numerical sequentially coupled thermal-stress analyses are discussed in this section. The relative maximum mid-span moment for a 1-hour rating simply supported W310×39 beam is plotted against fire-exposure time in Figure 4.53 along with the relative mid-span cross-section moment capacity of the beam. The former is the result of finite element structural modelling, and the latter is calculated analytically. It should be noted that the value of unity along the ordinate in Figure 4.36 represents the cross-section moment capacity calculated analytically at normal temperatures. Similarly, the numerical and analytical moment capacities for a 1-hour original rating simply supported W310×39 beam with flange protection loss at the mid-span are compared in Figure 4.54.

Figures 4.53 and 4.54 show that the predicted numerical maximum moments are consistent with the predicted analytical moment capacities during the whole period of fire exposure if the cross-section class change is disregarded. Since the material model used in the finite element structural analyses does not include strain-hardening of steel, the compression flange is expected to buckle when the yield stress is reached. Thus, the

predicted maximum moment from the finite element analysis is not expected to exceed the cross-section yield moment at the mid-span since the elastic modulus is zero at yielding when there is no strain hardening. However, it can be seen in both Figures 4.53 and 4.54 that the predicted maximum mid-span moments exceed the yield moments and even reach the plastic moments. Unlike a column buckling, local buckling of the flange involves biaxial stress that enables the flange to maintain a load carrying capacity close to its yield strength over a longer range of strain. This provides the required rotational capacity to develop the plastic moment. This can be illustrated with a series of finite element buckling analyses of columns and plates. Details of the buckling study are presented in Appendix A. Results of finite element analyses show that the calculated moment capacity using the analytical method in Section 3.4 is generally conservative. The similarity between numerical and analytical results also supports the validity of the analytical procedure.

Two different beam failure modes under standard fire exposure were observed. Figure 4.55 shows the deformed shape of a failed 1-hour rating fully protected W310×39 beam at 40 minutes of standard fire exposure. The distorted top flange at the mid-span indicates that the failure is due to local buckling of the top flange. Figure 4.56 presents the deformed shape of a failed protected W310×39 beam with bottom flange protection loss at the mid-span at 40 minutes of standard fire exposure. It clearly shows that failure of the beam is caused by the yielding of the bottom flange at the mid-span since the yield strength of steel at this region has been severely reduced due to high

temperatures. The deformed shape of a failed protected W310×39 with top flange protection damage at the mid-span at 40 minutes of standard fire exposure is shown in Figure 4.57. The top flange close to the mid-span is distorted, which indicates that the failure is caused by local buckling of the top flange.

Table 4.1 Relative cross-section moment capacities and relative fire resistances for W410×74 beams

Label of model	Relative cross-section moment capacity at 1 hour	Relative fire resistance	Label of model	Relative cross-section moment capacity at 2 hours	Relative fire resistance
Unp	0.052	0.2		0.031	
1	0.845	1.0	2	0.793	1.0
1BF1	0.490	0.7	2BF1	0.267	0.5
1BF1b	0.827	1.0	2BF1b	0.771	1.0
1BF1c	0.788	0.9	2BF1c	0.724	0.9
1BF1d	0.719	0.9	2BF1d	0.645	0.9
1BF2	0.648	0.8	2BF2	0.387	0.7
1BF3	0.391	0.6	2BF3	0.200	0.4
1BF3d	0.635	0.8	2BF3d	0.555	0.8
1BF4	0.386	0.5	2BF4	0.213	0.4
1BF5	0.479	0.7	2BF5	0.271	0.5
1BF6	0.675	0.8	2BF6	0.461	0.7
1BF7	0.744	0.9	2BF7	0.530	0.8
1BF8	0.400	0.6	2BF8	0.207	0.4
1BF9	0.300	0.4	2BF9	0.154	0.3
1BF10	0.270	0.4	2BF10	0.131	0.2
1BF10d	0.455	0.6	2BF10d	0.374	0.6
1BF11	0.242	0.4	2BF11	0.108	0.2
1TF1	0.490	0.7	2TF1	0.267	0.5
1TF1b	0.827	1.0	2TF1b	0.771	1.0
1TF1c	0.788	0.9	2TF1c	0.724	0.9
1TF1d	0.719	0.9	2TF1d	0.645	0.9
1TF2	0.648	0.8	2TF2	0.387	0.7
1TF3	0.391	0.6	2TF3	0.200	0.4
1TF3d	0.635	0.8	2TF3d	0.555	0.8
1TF4	0.386	0.5	2TF4	0.213	0.4
1TF5	0.479	0.7	2TF5	0.271	0.5
1TF6	0.675	0.8	2TF6	0.461	0.7
1TF7	0.744	0.9	2TF7	0.530	0.8
1TF8	0.400	0.6	2TF8	0.207	0.4
1TF9	0.300	0.4	2TF9	0.154	0.3
1TF10	0.270	0.4	2TF10	0.131	0.2

Table 4.1 (Continued)

Label of model	Relative cross-section moment capacity at 1 hour	Relative fire resistance	Label of model	Relative cross-section moment capacity at 2 hours	Relative fire resistance
1TF10d	0.455	0.6	2TF10d	0.374	0.6
1TF11	0.242	22.2	2TF11	0.108	25.9
1W1	0.758	54.0	2W1	0.581	97.3
1W2	0.670	49.1	2W2	0.419	82.5

Table 4.2 Relative cross-section moment capacities and relative fire resistances for W410×60 beams

Label of model	Relative cross-section moment capacity at 1 hour	Relative fire resistance	Label of model	Relative cross-section moment capacity at 2 hours	Relative fire resistance
1	0.764	1.0	2	0.708	1.0
1BF1	0.381	0.7	2BF1	0.209	0.5
1TF1	0.381	0.7	2TF1	0.209	0.5

Table 4.3 Relative cross-section moment capacities and relative fire resistances for W310×60 beams

Label of model	Relative cross-section moment capacity at 1 hour	Relative fire resistance	Label of model	Relative cross-section moment capacity at 2 hours	Relative fire resistance
1	0.735	1.0	2	0.705	1.0
1BF1	0.266	0.7	2BF1	0.084	0.5
1TF1	0.266	0.6	2TF1	0.084	0.4

Table 4.4   Relative cross-section moment capacities and relative fire resistances for W310×39 beams

Label of model	Relative cross-section moment capacity at 1 hour	Relative fire resistance	Label of model	Relative cross-section moment capacity at 2 hours	Relative fire resistance
1	0.735	1.0	2	0.705	1.0
1BF1	0.266	0.7	2BF1	0.084	0.5
1TF1	0.266	0.6	2TF1	0.084	0.4

Table 4.5   Relative cross-section moment capacities and relative fire resistances for W250×24 beams

Label of model	Relative cross-section moment capacity at 1 hour	Relative fire resistance	Label of model	Relative cross-section moment capacity at 2 hours	Relative fire resistance
1	0.671	1.0	2	0.760	1.0
1BF1	0.122	0.4	2BF1	0.068	0.3
1TF1	0.122	0.4	2TF1	0.068	0.3

Figure 4.1 Predicted relative cross-section moment capacities for unprotected and protected W410x74 beams

Figure 4.2 Predicted relative cross-section moment capacities for 1-hour fire resistance rating W410x74 beams with full protection or with symmetric bottom flange protection loss

Figure 4.3 Predicted relative cross-section moment capacities for 2-hour fire resistance rating W410×74 beams with full protection or with symmetric bottom flange protection loss

Figure 4.4 Temperature distribution contour of W410×74 beam 1BF3 after 1 hour of standard fire exposure

Figure 4.5 Predicted relative cross-section moment capacities for W410×74 beams 2BF4 and 2BF8

Figure 4.6 Predicted relative cross-section moment capacities for W410×74 beams 1BF4 and 1BF8

Figure 4.7 Relative fire resistances of 1-hour fire resistance rating W410x74 beams with full protection or with symmetric bottom flange protection loss

Figure 4.8 Relative fire resistances of 2-hour fire resistance rating W410x74 beams with full protection or with symmetric bottom flange protection loss

Figure 4.9 Predicted relative cross-section moment capacities for 2-hour fire resistance rating W410×74 beams with symmetric or asymmetric bottom flange protection loss

Figure 4.10 Predicted relative cross-section moment capacities for 1-hour fire resistance rating W410×74 beams with symmetric or asymmetric bottom flange protection loss

Figure 4.11 Predicted temperature profiles along the bottom flange for beams 2BF2 and 2BF6 after 40 minutes of standard fire exposure

Figure 4.12 Predicted temperature profiles along the web for beams 2BF2 and 2BF6 after 80 minutes of standard fire exposure

Figure 4.13 Relative fire resistances of 1-hour fire resistance rating W410×74 beams with full protection or with symmetric or asymmetric bottom flange protection loss

Figure 4.14 Relative fire resistances of 2-hour fire resistance rating W410×74 beams with full protection or with symmetric or asymmetric bottom flange protection loss

Figure 4.15 Fire resistance reductions of 1-hour fire resistance rating W410×74 beams with the bottom flange protection loss

Figure 4.16 Fire resistance reductions of 2-hour fire resistance rating W410x74 beams with the bottom flange protection loss

Figure 4.17 Predicted relative cross-section moment capacities for 1-hour fire resistance rating W410×74 beams with protection loss on the bottom flange or on the web

Figure 4.18 Predicted relative cross-section moment capacities for 2-hour fire resistance rating W410×74 beams with protection loss on the bottom flange or on the web

Figure 4.19 Relative fire resistances of 1-hour rating W410×74 beams with the bottom flange or web protection loss

Figure 4.20 Relative fire resistances of 2-hour rating W410×74 beams with the bottom flange or web protection loss

Figure 4.21 Predicted relative cross-section moment capacities for 1-hour fire resistance rating W410×74 beams with various depths of the bottom flange protection loss

Figure 4.22 Predicted relative cross-section moment capacities for 2-hour fire resistance rating W410×74 beams with various depths of the bottom flange protection loss

Figure 4.23 predicted relative cross-section moment capacities for 1-hour fire resistance rating W410x74 beams with an 87.5% penetrated bottom flange protection loss

Figure 4.24 Predicted relative cross-section moment capacities for 2-hour fire resistance rating W410x74 beams with an 87.5% penetrated bottom flange protection loss

Figure 4.25 Relative fire resistances of 1-hour fire resistance rating W410×74 beams with various depths of the bottom flange protection loss

Figure 4.26 Relative fire resistances of 2-hour fire resistance rating W410×74 beams with various depths of the bottom flange protection loss

Figure 4.27 Relative fire resistances of 1-hour fire resistance rating W410x74 beams with an 87.5% penetrated bottom flange protection loss

Figure 4.28 Relative fire resistances of 2-hour fire resistance rating W410x74 beams with an 87.5% penetrated bottom flange protection loss

Figure 4.29 Predicted relative cross-section moment capacities for 1-hour fire resistance rating fully protected beams of different sections

Figure 4.30 Predicted relative cross-section moment capacities for 2-hour fire resistance rating fully protected beams of different sections

Figure 4.31 Predicted relative cross-section moment capacities for 1-hour fire resistance rating W310x39 beams with protection loss on the bottom or top flange

Figure 4.32 Predicted Relative cross-section moment capacities for 2-hour fire resistance rating W310x39 beams with protection loss on the bottom or top flange

Figure 4.33 Predicted relative cross-section moment capacities for 1-hour fire resistance rating W310×60 beams with protection loss on the bottom or top flange

Figure 4.34 Predicted relative cross-section moment capacities for 2-hour fire resistance rating W310×60 beams with protection loss on the bottom or top flange

Figure 4.35 Predicted relative cross-section moment capacities for 1-hour fire resistance rating W410×60 beams with protection loss on the bottom or top flange

Figure 4.36 Predicted relative cross-section moment capacities for 2-hour fire resistance rating W410×60 beams with protection loss on the bottom or top flange

Figure 4.37 Predicted relative cross-section moment capacities for 1-hour fire resistance rating W250×24 beams with protection loss on the bottom or top flange

Figure 4.38 Predicted relative cross-section moment capacities for 2-hour fire resistance rating W250×24 beams with protection loss on the bottom or top flange

Figure 4.39 Predicted modified relative cross-section moment capacities for 1-hour fire resistance rating W410×60 and W310×60 beams with protection loss on the bottom flange

Figure 4.40 Predicted modified relative cross-section moment capacities for 2-hour fire resistance rating W410×60 and W310×60 beams with protection loss on the bottom flange

Figure 4.41 Predicted modified relative cross-section moment capacities for 1-hour fire resistance rating W410×74 and W410×60 beams with protection loss on the bottom flange

Figure 4.42 Predicted modified relative cross-section moment capacities for 2-hour fire resistance rating W410×74 and W410×60 beams with protection loss on the bottom flange

Figure 4.43 Predicted modified relative cross-section moment capacities for 1-hour fire resistance rating W310×60 and W310×39 beams with protection loss on the bottom flange

Figure 4.44 Predicted modified relative cross-section moment capacities for 2-hour fire resistance rating W310×60 and W310×39 beams with protection loss on the bottom flange

Figure 4.45 Predicted modified relative cross-section moment capacities after 1 hour of standard fire exposure for 1-hour fire resistance rating W410×74 beams with symmetric bottom flange protection loss

Figure 4.46 Predicted modified relative cross-section moment capacities after 2 hours of standard fire exposure for 2-hour fire resistance rating W410×74 specimens with symmetric bottom flange protection loss

Figure 4.47 Predicted relative cross-section moment capacities for 1-hour fire resistance rating fully protected W410×74 beams with constant and varied thermal conductivities of the protective material

Figure 4.48 Predicted relative cross-section moment capacities for 2-hour fire resistance rating fully protected W410×74 beams with constant and varied thermal conductivities of the protective material

Figure 4.49 Predicted relative cross-section moment capacities for 1-hour fire resistance rating W410×74 beams with symmetric bottom flange protection loss and with constant and varied protective material thermal conductivities

Figure 4.50 Predicted relative cross-section moment capacities for 2-hour fire resistance rating W410×74 beams with symmetric bottom flange protection loss and with constant and varied protective material thermal conductivities

Figure 4.51 Relative fire resistances of 1-hour rating W410×74 beams with symmetric bottom flange protection loss at the mid-span and with constant or varied protective material thermal conductivities

Figure 4.52 Relative fire resistances of 2-hour rating W410×74 beams with symmetric bottom flange protection loss at the mid-span and with constant and varied protective material thermal conductivities

Figure 4.53 Predicted relative cross-section moment capacities for a 1-hour fire resistance rating fully protected W310×39 beam by analytical calculation and numerical modeling

Figure 4.54 Predicted relative cross-section moment capacities for 1-hour fire resistance rating W310×39 beams with symmetric bottom or top flange protection loss by analytical method and numerical modelling

Figure 4.55 Deformed shape of a failed fully protected W310×39 beam at 40 minutes of standard fire exposure

Figure 4.56 Deformed shape of a failed W310×39 beam with symmetric bottom flange protection loss at 40 minute of standard fire exposure

Figure 4.57 Deformed shape of a failed W310×39 beam with symmetric top flange protection loss at 40 minute of standard fire exposure

Chapter 5   Summary, Conclusions, and Recommendations

5.1   Summary

Heat transfer studies on fire-exposed steel members, mechanical properties of structural steels at elevated temperatures, and studies on the fire behaviour of steel beams were reviewed in Chapter 2. The heat transfer studies focused on the temperature evolution of steel members exposed to the standard fire. The studies on the fire behaviour of steel beams investigated specific structural behaviours of steel beams at elevated temperatures as well as factors affecting these structural behaviours.

This study focused on the cross-section moment capacity of fire-exposed steel beams with partial fire protection loss. The study was carried out mainly by means of numerical heat transfer analyses and theoretical flexural calculations. In addition, a few numerical structural analyses were also conducted.

Five steel sections, W410×74, W410×60, W310×60, W310×39 and W250×24, were used in this study. The studied beams were simply supported beams covered with a spray-on fire protection which provides 1- or 2-hour fire resistance rating for a fully protected member. A number of finite element heat transfer analyses were carried out to predict the temperature distribution of steel beams with partial fire protection loss under the ISO 834 standard fire exposure. These analyses were carried out using the finite element software ABAQUS. Finite element models in heat transfer analyses were constructed with solid heat transfer elements. Thermal properties of steel and the fire protection material were applied to the models. The heat flux at the model boundaries

was applied using a user-defined subroutine. Mesh studies were carried out to determine the proper meshes for the heat transfer models. These models were validated with data of standard fire tests. Visual Basic programs were developed to compute the cross-section moment capacity of steel beams exposed to fire. The calculation method was derived based on theoretical flexural formulas. In the calculation of the cross-section moment capacity, the temperature profiles of steel sections were obtained from results of the heat transfer analyses. Mechanical properties of steel at elevated temperatures were adopted from EC3 (1995).

The effect of partial protection loss on the reduction of the mid-span cross-section moment capacity was studied and discussed. Several factors, such as protection damage area, damage shape, damage location, damage penetration, protection thickness and the size of steel sections, were investigated in the study. The influence of protection loss on the cross-section moment capacity along the longitudinal direction was also investigated. Results from analyses using constant thermal properties of the protective material were compared to those using varied thermal properties.

A few numerical sequentially coupled thermal-stress analyses were also carried out to study the fire performance of simply supported steel beams with partial protection loss at the mid-span. Results of these analyses were compared to those obtained from the analytical structural calculations.

5.2 Conclusions

A number of significant conclusions have been drawn from results of this study.

- 1) Overall, the study shows that partial loss of spray-on fire protection will cause a reduction of the cross-section moment capacity for protected steel beams exposed to the standard fire. The effect of fire protection loss on the cross-section moment capacity reduction strongly depends on the area of protection loss, and it is also affected by other factors, such as damage shape, damage location, damage penetration and section weight.
- 2) The reduction of the cross-section moment capacity strongly depends on the area of protection loss; the larger the area of protection loss, the greater the reduction in moment capacity. For W410×74 beams with the bottom flange protection loss at the mid-span, the fire resistance reduces rapidly with the damage area when the damage area is less than about 500 cm². The reduction rate reduces with a greater damage area. The fire resistance reduction approaches a limit when the protective coating at the bottom is completely removed. This limit is about 0.635 for 1-hour original fire resistance rating W410×74 beams and about 0.79 for 2-hour original fire resistance rating W410×74 beams.
- 3) The reduction of the cross-section moment capacity is affected by the shape of the protection damage. A larger width to length ratio of the damage results in a larger reduction of moment capacity during early stages of standard fire exposure, but this effect diminishes with for a longer fire-exposure time.

- 4) The effect of protection loss on the reduction in the cross-section moment capacity is affected by the protection thickness. A beam with a thicker fire protection has a greater final cross-section moment capacity reduction than the one with the same type of fire protection loss but with a thinner fire protection at their original fire resistance rating times.
- 5) The reduction of the cross-section moment capacity also depends on the location of protection loss. The flange protection loss has a greater effect on the moment capacity reduction than the web protection loss. For beams with web protection loss, the damage closer to the flange has a greater impact on the moment capacity reduction than that located away. Protection losses on top and bottom flanges have the same effect on the moment capacity reduction if the class designation of the steel section for local buckling does not change during the fire exposure. If the class designation change does occur, a higher moment capacity reduction is expected with the top flange protection loss if the top flange being in compression due to bending.
- 6) For beams with a small area of protection loss, the depth of damage penetration has relatively minor effect on the cross-section moment capacity unless the damage has fully penetrated the protective coating. For W410×74 beams with a 10 cm × 10 cm size protection loss on the bottom flange, loss of protective coating up to 87.5% was found to have little impact on the cross-section moment reduction compared to a fully protected beam. However, even if the damage has

not fully penetrated the protective coating, the impact on the cross-section moment reduction becomes significant when the damage area becomes large.

- 7) For beams with an identical flange protection loss, a lighter section has a greater cross-section moment capacity reduction in fire exposure than a heavier section, but the influence of the section weight is more significant for 1-hour original rating beams than for 2-hour original rating beams. On the other hand, a beam with a lower depth has a greater cross-section moment capacity reduction, but the influence of the depth is more significant for 2-hour original rating beams than for 1-hour original rating beams.
- 8) The effect of partial protection loss on the moment capacity reduction is very significant at cross-sections within the damage area, but this effect decreases rapidly with distance from the damage area. The affected distance is about 40 cm from the edge of the damage area for 1 hour of standard fire exposure and 60 cm for 2 hours of standard fire exposure.
- 9) Under the standard fire exposure, a lateral restrained simply supported I-shape steel beam with the bottom flange protection loss at the mid-span may fail by yielding of the bottom flange. On the other hand, the beam with the top flange protection loss at the mid-span may fail by local buckling of the top flange.

5.3 Recommendations

- 1) More analyses should be carried out on beams of hot-rolled steel sections with

different weights and sizes so that a more general fire resistance reduction equation for the protection loss can be developed.

- 2) Only 1- and 2-hour standard fire exposures are considered in this study. Analyses should be carried out for longer standard fire exposures.
- 3) The present study is based on numerical and analytical analyses. It is recommended that a few full-scale fire tests be conducted to verify results of this study.
- 4) The present study only considers protection loss at the mid-span. The effect of protection loss at different location along the length should be investigated.
- 5) The current study assumes that steel beams are subjected to four sided fire exposure. Studies should also be carried out on steel beams under three sided fire exposure.
- 6) The present study only deals with hot-rolled steel beams. Studies should also be carried out on cold-formed steel beams with partial fire protection loss.
- 7) Further investigation should be carried out on the local buckling behaviour of steel beams exposed to fire using finite element methods with a more sophisticated material model.
- 8) Additional studies should also be carried out on steel beams with different end restraints. In steel structures, beams may have axial and rotational end restraints.
- 9) The present study considers a fire exposure to the standard time-temperature

curve. Studies should also be conducted with realistic time-temperature curves.

- 10) Studies that look into the effect of protection loss at connections should also be conducted.

References

1. ABAQUS Version 6.4.1 (2004), *ABAQUS/ Standard User's manual*, Hibbitt, Karlsson & Sorensen Inc.
2. Anderberg, Y. (1988), "Modelling Steel Behaviour", *Fire Safety Journal*, volume 13, pp. 17-26.
3. ASTM E8 (2004), *Standard Methods of Tension Testing of Metallic Materials*, American Society of Testing and Materials, Philadelphia, USA.
4. ASTM E119 (2000), *Standard Test Methods for Fire Tests of Building Construction and Materials*, American Society of Testing and Materials, Philadelphia, USA.
5. Bardell, K. (1983), "Spray-Applied Fire Resistive Coatings for Steel Building Columns," *Fire Resistive Coatings: The Need for Standards*, ASTM STP 826, American Society for Testing and Materials, pp. 40-55.
6. Becker, R. (2002), "Structural behavior of simple steel structures with non-uniform longitudinal temperature distributions under fire conditions," *Fire Safety Journal*, Vol. 37, pp. 495-515.
7. Buchanan, A.H. (2001), *Structural Design for Fire Safety*, John Wiley & Sons, Chichester, England.
8. Burgess, I.W., Rimawi, J. and Plank, R.J. (1991), "Studies of the Behavior of Steel Beams in Fire," *Journal of Constructional Steel Research*, Vol. 19, pp. 285-312.

9. CAN/CSA-G40.20/G40.21-98 (1998), *General Requirement for Rolled or Welded Structural Quality Steel/Structural Quality Steel*, Canadian Standards Association, Rexdale, Ontario, Canada.
10. CAN/CSA-S16-01 (2001), *Limit States Design of Steel Structures*, Canadian Standards Association, Rexdale, Ontario, Canada.
11. CAN/ULC-S101-04 (2004), *Standard Methods of Fire Endurance Tests of Building Construction and Materials*, Underwriter's Laboratories of Canada, Scarborough, Canada.
12. CISC (2004), *Handbook of Steel Construction*, 8th Edition, Canadian Institute of Steel Construction, Willowdale, Canada.
13. Ding, J., Li G. and Sakumoto, Y. (2004), "Parametric Studies on Fire Resistance of Fire-Resistant Steel Members," *Journal of Constructional Steel Research*, Vol. 60, pp1007-1027.
14. ECCS (1983), *European Recommendations for the Fire Safety of Steel Structures: Calculation of the Fire Resistance of Load Bearing Elements and Structural Assemblies Exposed to the Standard Fire*, European Convention for Constructional Steelwork, Technical Committee 3, Brussels, Belgium.
15. ECCS (1995), *Fire Resistance of Steel Structures*, *ECCS Technical Note No. 89*, European Convention for Constructional Steelwork, Technical Committee 3, Brussels, Belgium.
16. EC1 (2002), *Eurocode 1: Actions on structures - Part 1-2: General actions -*

Actions on structures exposed to fire, European Committee for Standardization, Brussels, Belgium.

17. EC3 (1995), *Eurocode 3: Design of Steel Structures - Part 1-2: General Rules - Structural Fire Design*, European Committee for Standardization, Brussels, Belgium.
18. Fontana, M. and Knobloch, M. (2004), "Local Buckling Behaviour and Strain-Based Effective Widths of Steel Structures Subjected to Fire," *Structures in Fire, Proceedings of the Third International Workshop*, pp. 71-88.
19. Franssen, Jean-Marc (2004), "Numerical Determination of 3D Temperature Fields in Steel Joints," *Fire and Materials*, Vol. 28, pp. 63-82.
20. Galambos, T.V. (1988), *Guide to Stability Design Criteria for Metal Structures*, 4th Edition, John Wiley & Sons, New York, USA.
21. Harper, C.A. (2004), *Handbook of Building Materials for Fire Protection*, Mc-Graw-Hill, New York.
22. ISO 834 (1975), *Fire Resistance Tests-Elements of Building Construction*, International Organization for Standardization, Switzerland.
23. Kaitila, O. (2002), *Finite Element Modeling of Cold-Formed Steel Members at High Temperatures*, Laboratory of Steel Structures publications: TKK-TER-24, Finland.
24. Kamikawa, D., Hasemi, Y., Wakamatsu, T. and Kagiya, K. (2004), "Prediction of the Thermal Response of a Metal Column Exposed to a Localized Fire,"

Structures in Fire, Proceedings of the Third International Workshop, pp. 25-34.

25. Kirby, B.R. and Preston, R.R. (1988), "High Temperature Properties of Hot-Rolled, Structural Steels for Use in Fire Engineering Design Studies," *Fire Safety Journal*, Vol. 13, pp. 27-37.
26. Lee, J.H. and Mahendran, M. (2005), "Effects of Fire Conditions on the Interactive Local Buckling Behavior of Cold-Formed Steel Lipped Channel Members," *Developments in Mechanics of Structures and Materials*, Taylor & Francis Group plc, London, UK, Vol. 2, pp. 277-582.
27. Lie, T.T. (1992), *Structural Fire Protection*, American Society of Civil Engineers, New York, USA.
28. Lie, T.T. and Stanzak, W.W. (1973), "Fire Resistance of Protected Steel Columns," *Engineering Journal/American Institute of Steel Construction*, Vol. 10, pp. 82-94.
29. Liu, T.C.H., Fahad, M.K. and Davies, J. M. (2002), "Experimental Investigation of Behaviour of Axially Restrained Steel Beams in Fire," *Journal of Constructional Steel Research*, Vol. 58, pp. 1211-1230.
30. Malhotra, H.L. (1982), *Design of Fire-Resisting Structures*, Surrey University Press, New York, USA.
31. Moss, P.J., Buchanan, A.H., Seputro, J., Wastney, C. and Welsh, R. (2004), "Effect of Support Conditions on the Fire Behavior of Steel and Composite Beams," *Fire and Materials*, Vol. 28, pp. 159-175.

32. Outinen, J., Kaitila, O. and Makelainen, P. (2001), *High-Temperature Testing of Structural Steel and Modeling of Structures at Fire Temperatures*, Laboratory of Steel Structures publications: TKK-TER-23, Finland.
33. Outinen, J. and Makelainen, P. (2004), "Mechanical Properties of Structural Steel at Elevated Temperatures and after Cooling Down," *Fire and Materials*, Vol. 28, pp. 237-251.
34. Piloto, P.A.G. and Vila Real, P.M.M. (2000), "Lateral Torsional Buckling of Steel I-Beam in Case of Fire: Experimental Evaluation," *Structures in Fire, Proceedings of the First International Workshop*, pp. 95-105.
35. Purkiss, J.A. (1996), *Fire Safety Engineering Design of Structures*, Butterworth-Heinemann, Oxford, UK.
36. Riks, E. (1979), "An Incremental Approach to the Solution of Snapping and Buckling Problems", *Int. J. Solids Structures*, Vol. 15, pp. 529-551.
37. Ryder, N.L., Wolin, S.D. and Milke, J.A. (2002), "An Investigation of the Reduction in Fire Resistance of Steel Columns Caused by Loss of Spray-Applied Fire protection," *Journal of Fire Protection Engineering*, Vol. 12, pp. 31-44.
38. Tomecek, D.V. and Milke, J.A. (1993), "A Study of the Effect of Partial Loss of Protection on the Fire Resistance of Steel Columns," *Fire Technology*, Vol. 29, pp. 4-21.
39. Usmani, A.S., Rotter, J.M., Lamont, S., Sanad, A.M. and Gillie, M. (2001), "Fundamental Principles of Structural Behavior under Thermal Effects," *Fire*

Safety Journal, Vol. 36, pp. 721-774.

40. Vila Real, P.M.M. and Franssen, J.-M. (2000), "Lateral Torsional Buckling of Steel I-Beam in Case of Fire: Numerical Modeling," *Structures in Fire, Proceedings of the First International Workshop*, pp. 71-93.

Appendix A: Finite Element Buckling Analysis of Columns and Plates

A series of finite element buckling analyses were carried out to investigate the effect of biaxial stress on column and plate buckling. Geometrical properties and buckling strength of each column and plate are listed in Tables A.1 to A.3. Dimensions, loading condition and boundary conditions of the columns and plates are illustrated in Figure A.1. The difference between a column and a plate is that a column has no restraint on both unloaded edges while a plate has one of the unloaded edges pinned or fixed. The material is assumed to be elastic-perfectly plastic, with the yield strength 350 MPa and the elastic modulus 200,000 MPa. The finite element analyses were carried out using the software ABAQUS (2004), and the bi-linear shell element S4 was used in the finite element models. A geometrical imperfection was introduced to each model by offsetting two element strips at the middle of its length by 2%.

Numerically modeled maximum average stresses of the columns are plotted against the slenderness ratio in Figure A.2 along with the theoretical stress versus slenderness ratio curve. It is shown that the numerical results are very close to the theoretical stress versus slenderness ratio relationship. It can also be observed that the maximum average cross-section stress of column C100-50 exceeds the theoretical value while others are below the theoretical curve. As shown in Figure A.3, column C100-50 bends in both Z and Y directions, and the biaxial bending deformation induces biaxial normal stress on the deformed surfaces. Figure A.4 shows the average cross-section

stress versus displacement relationships for columns C50-50 and C100-50. Compared to column C50-50, column C100-50 is under the effect of a greater biaxial stress. Therefore, column C100-50 is able to maintain a cross-section stress close to the yield strength over a longer range of displacement after buckling occurs, and the stress even reaches a level higher than the yield strength before dropping down. It is noted that due to the increased cross-section area of the column under compression, the observed average cross-section stress is slightly higher than the yield strength. Actually, the average true cross-section stress of column C100-50 is close to but does not exceed the yield strength. Similarly, the biaxial bending deformation of the plates (Figures A.5 and A.6) induces biaxial stress on their deformed surfaces. The biaxial stress enables the plates to maintain a stress level close to and exceed the yield strength over a longer range of displacement, as shown in Figures A.7 and A.8.

Table A.1 Geometrical properties and buckling strength of columns

Label of column	Length	Width	Thickness	Cross-section Area	Axis Y-Y		Buckling stress
	L	b	t	A	Moment of inertia I_y	ratio of gyration r_y	σ
	mm	mm	mm	mm ²	mm ⁴	mm	MPa
C50-10	400	50	10	500	4166.7	2.9	102.8
C50-12	400	50	12	600	7200.0	3.5	147.9
C50-14	400	50	14	700	11433.3	4.0	200.7
C50-16	400	50	16	800	17066.7	4.6	259.5
C50-18	400	50	18	900	24300.0	5.2	313.9
C50-20	400	50	20	1000	33333.3	5.8	334.5
C50-22	400	50	22	1100	44366.7	6.4	339.8
C50-24	400	50	24	1200	57600.0	6.9	349.1
C50-30	400	50	30	1500	112500.0	8.7	345.1
C50-50	400	50	50	2500	520833.3	14.4	348.8
C100-50	400	100	50	5000	1041666.7	14.4	353.3

Table A.2 Geometrical properties and buckling strength of plates

Label of plate	Unloaded edges		Length	Width	Thickness	Buckling stress
			a	b	t	σ
			mm	mm	mm	MPa
P8-p	Free	Pinned	400	100	8	326.1
P10-p	Free	Pinned	400	100	10	351.5
P12-p	Free	Pinned	400	100	12	353.0
P14-p	Free	Pinned	400	100	14	355.0
P8-f	Free	Fixed	400	100	8	351.9
P10-f	Free	Fixed	400	100	10	353.8
P12-f	Free	Fixed	400	100	12	356.5
P14-f	Free	Fixed	400	100	14	361.0

Figure A.1 Dimensions, loading condition and boundary conditions of columns and plates

Figure A.2 Predicted maximum cross-section stress for columns with various slenderness ratios

a) Deformed shape (Deformation scale factor: 5.0)

b) Normal stress components at the marked areas

Figure A.3 Deformed shape and normal stress components of column C100-50

Figure A.4 Predicted average cross-section stress versus displacement relationships for columns C50-50 and C100-50

Figure A.5 Deformed shape of a plate with one unloaded edge free and the other one pinned (Deformation scale factor: 2.5)

Figure A.6 Deformed shape of a plate with one unloaded edge free and the other one fixed (Deformation scale factor: 10.0)

Figure A.7 Predicted average cross-section stress versus displacement relationships for plates with one unloaded edge free and the other one pinned

Figure A.8 Predicted average cross-section stress versus displacement relationships for plates with one unloaded edge free and the other one fixed