UNIVERSITY OF TORONTO

ELLIPTICAL HOLLOW SECTION T AND X CONNECTIONS

by

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ABSTRACT

Elliptical hollow sections (EHS) are the newest steel shape to emerge in the industry, but appropriate design guidance is lacking, being completely absent from Canadian codes and guidelines. Geometric property and compressive resistance tables were established to be potentially added to the Canadian guides. The equivalent RHS method, originally proposed by Zhao and Packer in 2009, was simplified and modified to validate its use for the design of EHS columns and beams. An experimental programme was developed to investigate the behaviour of EHS-to-EHS welded connections. Twelve T and X connection tests were performed to study the effect of connection angle, orientation type and loading. Two methods were developed to predict connection capacities and failure modes: the equivalent CHS and the equivalent RHS approaches. Both methods proved to be conservative on average, but the equivalent RHS approach proved to be more successful at capturing the actual failure mode of EHS-to-EHS connections.

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TABLE OF CONTENTS

Abstract	ii
Acknowledgements	iii
Table of Contents	iv
List of Figures	vii
List of Tables	xi
List of Notations	xiv
 1.0 Introduction	 1
1.1 EHS Defined.....	1
1.2 Common Applications.....	3
1.3 Advantages of EHS	5
 2.0 Literature Review	 6
2.1 EHS Properties	6
2.1.1 Geometric Properties	6
2.1.2 Mechanical Properties	8
2.2 EHS in Axial Compression	8
2.2.1 Historical Developments	9
2.2.2 Buckling of EHS	12
2.2.3 Equivalent CHS Approaches.....	15
2.2.4 Elastic Buckling Stress Transition from CHS to Plate	18
2.2.5 Experimental Tests on EHS Long Columns.....	24
2.2.6 Equivalent RHS Approach	25
2.3 Bending, Shear and Combined Loading.....	26
2.3.1 Bending Resistance	26
2.3.2 Shear Resistance.....	28
2.3.3 Interaction Curves	29
2.4 Concrete Filled EHS.....	30
2.5 Stainless Steel OHS.....	35
2.5.1 Unfilled.....	35
2.5.2 Filled.....	37
2.6 EHS Connections	38
2.6.1 K Connections	40

2.6.2 X Connections	40
2.6.3 Gusset Plate End Connections.....	43
2.6.4 Branch and Through Plate Connections	45
3.0 Preliminary Work	47
3.1 EHS Dimension and Gross Property Table	47
3.1.1 Warping, Shear and Torsional Constants	47
3.1.2 Properties About the Axes	49
3.2 EHS Compressive Resistance Tables	50
3.3 Equivalent RHS Approach	51
3.2.1 Columns	51
3.2.2 Beams	56
3.2.3 Advantages and Summary.....	59
4.0 Experimental Programme	64
4.1 Material Property Tests	64
4.1.1 Tensile Coupon Tests	64
4.1.2 Stub Column Test.....	65
4.2 Test Specimens.....	66
4.2.1 Design Considerations.....	67
4.3 Test Setup and Instrumentation	69
4.3.1 Strain Gauges	70
4.3.2 LVDTs.....	72
4.3.3 LEDs.....	74
4.3.4 MTS Load Frame	74
4.3.5 Lateral Supports	74
5.0 Results and Analysis.....	77
5.1 Material Properties	77
5.1.1 Tensile Coupon Tests	77
5.1.2 Stub Column Test.....	78
5.1.3 Previous Material Property Tests	79
5.2 Specimen Dimensions	80
5.3 LED and LVDT Validation	81
5.4 Load-Displacement Curves and Failure Modes	82
5.4.1 Observations.....	91
5.5 Chord Deformation Profiles	96

5.5.1 90° Connections	96
5.5.2 45° Connections	100
5.5.3 Cross-sections.....	102
5.6 Brace Stresses.....	103
5.6.1 Type 1 Connections.....	108
5.6.2 Type 2 Connections.....	109
5.6.3 Type 3 Connections.....	110
6.0 Capacity Predictions	111
6.1 X and T Connections from the University of Toronto	111
6.1.1 Equivalent CHS Approach	111
6.1.2 Equivalent RHS Approach	113
6.1.3 Comparison	117
6.2 X and T Connections from the National University of Singapore	117
6.2.1 Equivalent CHS Approach	118
6.2.2 Equivalent RHS Approach	119
6.2.3 Comparison	120
6.3 Summary	121
7.0 Conclusions and Recommendations	122
References	124

Appendices

Appendix 3A – EHS Dimension and Gross Property Tables	129
Appendix 3B – EHS Compressive Resistance Tables.....	132
Appendix 4A – Tensile Coupon Data Sheets	139
Appendix 4B – Fabrication Drawings.....	143
Appendix 4C – Walters Inc. Fabrication Drawings	156
Appendix 4D – LVDT Instrumentation	167
Appendix 4E – LED Locations	174
Appendix 4F – T Connection End Frame.....	179
Appendix 5A – Specimen Measurements	181
Appendix 5B – Weld Measurements.....	187
Appendix 5C – Connection Displacement Measurement.....	192
Appendix 5D – Experimental Summaries	198

LIST OF FIGURES

Figure 1.1: EHS basic dimensions.....	2
Figure 1.2: Honda exhibit (Corus, 2005).....	4
Figure 2.1: Local buckling of EHS according to Kempner (adapted from Chan and Gardner, 2009).....	10
Figure 2.2: Three-dimensional visualization of local buckling (Chan and Gardner, 2007).....	10
Figure 2.3: Maximum deformations at ends and mid-length (adapted from Zhu and Wilkinson, 2007)....	13
Figure 2.4: Buckling wavelengths (reproduced from Bradford and Roufeginejad, 2008).....	13
Figure 2.5: Buckling modes (Silvestre, 2007).....	15
Figure 2.6: CHS to plate transition (reproduced from Ruiz-Teran and Gardner, 2008).....	18
Figure 2.7: Longitudinal strips and transverse rings of CHS (Ruiz-Teran and Gardner, 2008).....	20
Figure 2.8: CHS buckling non-axi-symmetrically (Ruiz-Teran and Gardner, 2008).....	20
Figure 2.9: Longitudinal and transverse strip of a plate (Ruiz-Teran and Gardner, 2008)	21
Figure 2.10: Equivalent RHS for an EHS	25
Figure 2.11: Interaction surface for EHS with $a/b = 2.13$ (Nowzartash and Mohareb, 2009).....	29
Figure 2.12: Loading conditions on concrete filled EHS (Brienza, 2008).....	33
Figure 2.13: Components of EHS-to-EHS connection.....	39
Figure 2.14: AXA truss connection (Bortolotti et al., 2003)	40
Figure 2.15: EHS X connection (Pietrapertosa and Jaspart, 2003)	40
Figure 2.16: EHS connection orientation types (reproduced from Choo et al., 2003)	42
Figure 2.17: Gusset plate-to-EHS end connection (reproduced from Martinez-Saucedo et al., 2008)	44
Figure 2.18: Branch and through plate-to-EHS connections (Willibald et al., 2006b)	45
Figure 3.1: International vs. Canadian convention for sectional axes.....	49
Figure 3.2: Equivalent RHS using the simple and modified equivalent RHS approach	54
Figure 3.3: EHS column design procedure using the equivalent RHS method	62
Figure 3.4: EHS beam design procedure using the equivalent RHS method	63
Figure 4.1: Tensile coupon locations	64
Figure 4.2: Stub column relevant dimensions and strain gauge locations	65
Figure 4.3: Experimental programme orientation types	67
Figure 4.4: Force flow at 2:1 ratio	68
Figure 4.5: Strain gauge locations for 90° specimens	70

Figure 4.6: Strain gauge locations for 45° specimens	71
Figure 4.7: LVDT instrumentation for T90-1C	73
Figure 4.8: LVDT-CC setting to measure connection displacement	73
Figure 4.9: Typical experimental test setup for X connections (X90-2T).....	75
Figure 4.10: Typical test setup for T connections (T90-1C)	75
Figure 4.11: Lateral support for compression-loaded X connections (X45-2C)	76
Figure 5.1: Tensile coupon engineering stress-strain curves.....	77
Figure 5.2: Stub column test results	78
Figure 5.3: EHS220 x 110 x 6 stub column failure	79
Figure 5.4: X90-1T chord and brace dimensions	80
Figure 5.5: X45-3C chord and brace dimensions	81
Figure 5.6: LVDT and LED time synchronization for X90-2T.....	82
Figure 5.7: Load-connection displacement graph of a) T90-1C	84
Figure 5.8: Load-connection displacement graph of b) X45-1C	84
Figure 5.9: Load-connection displacement graph of c) X90-1C	85
Figure 5.10: Load-connection displacement graph of d) X90-1T	85
Figure 5.11: Load-connection displacement graph of e) T90-2C	86
Figure 5.12: Load-connection displacement graph of f) X45-2C	86
Figure 5.13: Load-connection displacement graph of g) X90-2C	87
Figure 5.14: Load-connection displacement graph of h) X90-2T	87
Figure 5.15: Load-connection displacement graph of i) T90-3C	88
Figure 5.16: Load-connection displacement graph of j) X45-3C	88
Figure 5.17: Load-connection displacement graph of k) X90-3C	89
Figure 5.18: Load-connection displacement graph of l) X90-3T	89
Figure 5.19: Ultimate failure modes	90
Figure 5.20: Type 1 connections in compression	92
Figure 5.21: Type 2 connections in compression	93
Figure 5.22: X90-2C chord sidewall failure	93
Figure 5.23: X90-2T brace failure and chord side wall failure	94
Figure 5.24: Type 3 connections in compression	95
Figure 5.25: Chord deformation profiles of 90° X connections in tension	97
Figure 5.26: Chord deformation profiles of 90° X connections in compression.....	98
Figure 5.27: Chord deformation profiles of 90° T connections in compression	99

Figure 5.28: Chord deformation profile of X45-1C	101
Figure 5.29: Chord deformation profile of X45-2C	101
Figure 5.30: Chord deformation profile of X45-3C	102
Figure 5.31: Cut-out sections of T connections.....	103
Figure 5.32: Strain gauge designations to their strain gauge location number.....	104
Figure 5.33: Brace stress profiles for Type 1 connections	105
Figure 5.34: Brace stress profiles for Type 2 connections	106
Figure 5.35: Brace stress profiles for Type 3 connections	107
 Figure 6.1: Equivalent RHS approach for EHS connections (all dimensions in mm).....	115
 Figure 4B.1: Fabrication drawing for T90-1C.....	144
Figure 4B.2: Fabrication drawing for T90-2C.....	145
Figure 4B.3: Fabrication drawing for T90-3C.....	146
Figure 4B.4: Fabrication drawing for X90-1C	147
Figure 4B.5: Fabrication drawing for X90-2C	148
Figure 4B.6: Fabrication drawing for X90-3C	149
Figure 4B.7: Fabrication drawing for X90-1T	150
Figure 4B.8: Fabrication drawing for X90-2T	151
Figure 4B.9: Fabrication drawing for X90-3T	152
Figure 4B.10: Fabrication drawing for X45-1C	153
Figure 4B.11: Fabrication drawing for X45-2C	154
Figure 4B.12: Fabrication drawing for X45-3C	155
Figure 4C.1: Walters Inc. fabrication drawing for T90-1C	157
Figure 4C.2: Walters Inc. fabrication drawing for T90-2C	158
Figure 4C.3: Walters Inc. fabrication drawing for T90-3C	159
Figure 4C.4: Walters Inc. fabrication drawing for X90-1T and X90-1C	160
Figure 4C.5: Walters Inc. fabrication drawing for X45-1C	161
Figure 4C.6: Walters Inc. fabrication drawing for X90-2T and X90-2C	162
Figure 4C.7: Walters Inc. fabrication drawing for X90-3T and X90-3C	163
Figure 4C.8: Walters Inc. fabrication drawing for X45-2C	164
Figure 4C.9: Walters Inc. fabrication drawing for X45-3C	165
Figure 4C.10: Walters Inc. weld detail.....	166
Figure 4D.1: LVDT locations for T90-1C	168

Figure 4D.2: LVDT locations for T90-2C	169
Figure 4D.3: LVDT locations for T90-3C	170
Figure 4D.4: LVDT locations for X90-1C and X90-1T.....	171
Figure 4D.5: LVDT locations for X90-2C and X90-2T.....	172
Figure 4D.6: LVDT locations for X90-3C and X90-3T.....	173
Figure 4E.1: LED locations for X90-2T.....	174
Figure 4E.2: LED locations for X90-2C	175
Figure 4E.3: LED locations for X90-3C	175
Figure 4E.4: LED locations for X45-1C	176
Figure 4E.5: LED locations for X45-2C	176
Figure 4E.6: LED locations for X45-3C	177
Figure 4E.7: LED locations for T90-1C.....	177
Figure 4E.8: LED locations for T90-2C.....	178
Figure 4E.9: LED locations for T90-3C.....	178
Figure 4F.1: T connection end frame	180
Figure 5A.1: Specimen measurements of X90-1T	181
Figure 5A.2: Specimen measurements of X90-2T	181
Figure 5A.3: Specimen measurements of X90-3T	182
Figure 5A.4: Specimen measurements of X90-1C.....	182
Figure 5A.5: Specimen measurements of X90-2C.....	183
Figure 5A.6: Specimen measurements of X90-3C.....	183
Figure 5A.7: Specimen measurements of X45-1C.....	184
Figure 5A.8: Specimen measurements of X45-2C.....	184
Figure 5A.9: Specimen measurements of X45-3C.....	185
Figure 5A.10: Specimen measurements of T90-1C	185
Figure 5A.11: Specimen measurements of T90-2C	186
Figure 5A.12: Specimen measurements of T90-3C	186
Figure 5B.1: Location of weld measurements.....	188
Figure 5C.1: Connection displacement measurements for tension-tested X connections at 90°	193
Figure 5C.2: Connection displacement measurements for compression-tested X connections at 90°	194
Figure 5C.3: Connection displacement measurements for compression-tested X connections at 45°	195
Figure 5C.4: Connection displacement measurements for compression-tested T connections at 90°	196
Figure 5C.5: Additional rotation of X90-2C	197

LIST OF TABLES

Table 2.1: Dimension and sectional property equations	6
Table 2.2: EHS tolerances	8
Table 2.3: EHS mechanical properties	8
Table 3.1: Predicted capacity of columns buckling about the major axis using the equivalent RHS approach - Method 1 (experimental results taken from Chan and Gardner (2009))	52
Table 3.2: Equivalent RHS approaches - Method 1 vs. Method 2 and (a) vs. (b).....	54
Table 3.3: Predicted capacity of columns buckling about the minor axis using the equivalent RHS approach - Methods 1 and 2 (experimental results taken from Chan and Gardner (2009))	55
Table 3.4: Equivalent RHS properties	57
Table 3.5: Predicted capacity of beams bending about the major axis using the equivalent RHS approach - Methods 2a and 2b (experimental results taken from Chan and Gardner (2008))	58
Table 3.6: Predicting capacity of beams bending about the minor axis using the equivalent RHS approach - Methods 2a and 2b (experimental results taken from Chan and Gardner (2008))	58
Table 3.7: RHS Class limits for bending, where $c = H - 2t$ or $B - 2t$ (and H or B can be equivalent dimensions)	60
Table 3.8: CHS Class limits (and D can be the equivalent diameter)	60
Table 4.1: Stub column measurements.....	65
Table 4.2: Test specimens	66
Table 5.1: Tensile coupon test results	78
Table 5.2: Current vs. previously determined material properties	79
Table 5.3: Measured brace and chord lengths	80
Table 5.4: Two methods of specimen categorization	83
Table 5.5: Summary of experiments and results	91
Table 5.6: Load-displacement graph and failure mode observations based on orientation type groups.....	95
Table 6.1: Relevant CIDECT CHS connection design equations	112
Table 6.2: Connection capacity predictions using the equivalent CHS approach.....	113
Table 6.3: Relevant CIDECT RHS connection design equations	114

Table 6.4: Connection capacity predictions using equivalent RHS approach	116
Table 6.5: Equivalent CHS approach vs. equivalent RHS approach	117
Table 6.6: Experimental programme from NUS (Packer et al., 2011)	118
Table 6.7: Equivalent CHS approach to predict NUS experiments.....	119
Table 6.8: Equivalent RHS approach to predict NUS experiments	120
Table 6.9: Equivalent CHS approach vs. equivalent RHS approach for NUS tests.....	120
Table 6.10: Summary of University of Toronto and NUS predictions.....	121
Table 3A.1: EHS dimension and gross property table	130
Table 3B.1: EHS compressive resistance table	133
Table 4A.1: Tensile coupon 1 data sheet.....	140
Table 4A.2: Tensile coupon 2 data sheet.....	141
Table 4A.3: Tensile coupon 3 data sheet.....	142
Table 5B.1: Weld measurements for X90-1T	188
Table 5B.2: Weld measurements for X90-2T	188
Table 5B.3: Weld measurements for X90-3T	189
Table 5B.4: Weld measurements for X90-1C	189
Table 5B.5: Weld measurements for X90-2C	189
Table 5B.6: Weld measurements for X90-3C	190
Table 5B.7: Weld measurements for X45-1C	190
Table 5B.8: Weld measurements for X45-2C	190
Table 5B.9: Weld measurements for X45-3C	191
Table 5B.10: Weld measurements for T90-1C.....	191
Table 5B.11: Weld measurements for T90-2C.....	191
Table 5B.12: Weld measurements for T90-2C.....	191
Table 5D.1: Experimental summary of X90-1T.....	199
Table 5D.2: Experimental summary of X90-2T.....	200
Table 5D.3: Experimental summary of X90-3T.....	201
Table 5D.4: Experimental summary of X90-1C	202
Table 5D.5: Experimental summary of X90-2C	203
Table 5D.6: Experimental summary of X90-3C	204
Table 5D.7: Experimental summary of X45-1C	205
Table 5D.8: Experimental summary of X45-2C	206

Table 5D.9: Experimental summary of X45-3C	207
Table 5D.10: Experimental summary of T90-1C.....	208
Table 5D.11: Experimental summary of T90-2C.....	209
Table 5D.12: Experimental summary of T90-3C.....	210

LIST OF NOTATIONS

ACRONYMS

3%DL	3% Deformation Limit
CDP	Chord Deformation Profile
CF	Concrete Filled
CHS	Circular Hollow Section
CIDECT	Comité International pour le Développement et l'Etude de la Construction Tubulaire
CISC	Canadian Institute of Steel Construction
COV	Coefficient of Variation
CS	Carbon Steel
CSM	Continuous Strength Method
EC3	Eurocode 3
EHS	Elliptical Hollow Section
FE	Finite Element
HSS	Hollow Structural Section
LED	Light Emitting Diode
LVDT	Linear Variable Differential Transformer
NUS	National University of Singapore
OHS	Oval Hollow Section
RHS	Rectangular Hollow Section
SG	Strain Gauge
SHS	Square Hollow Section
SS	Stainless Steel
St.Dev.	Standard Deviation
TC	Tensile Coupon
UL	Ultimate Limit
UT	University of Toronto

VARIABLES

0	As a subscript, refers to the chord
1	As a subscript, refers to the brace
a	Half of the larger dimension of elliptical hollow section (mm)
a _m	Parameter to calculate mean perimeter (mm)

A	Cross-sectional area (mm^2)
A_c	Area of concrete (mm^2)
A_{eff}	Effective area (mm^2)
A_{EHS}	Area of an elliptical hollow section (mm^2)
A_{gv}	Gross shear area (mm^2)
A_h	Enclosed area of a hollow structural section (mm^2)
A_n	Net area (mm^2)
A_{nt}	Net area in tension (mm^2)
A_s	Area of steel (mm^2)
A_v	Shear area (mm^2)
b	Half of the smaller dimension of elliptical hollow section (mm); Width (mm)
b_0	Chord width (mm)
$b_{0,\text{eq}}$	Equivalent chord width (mm)
b_1	Brace width (mm)
b_m	Parameter to calculate mean perimeter (mm)
B	Smaller dimension of elliptical hollow section (mm)
B_{eq}	Smaller dimension of equivalent rectangular hollow section (mm)
c	Element length for cross-sectional classification, equal to $H - 2t$ or $B - 2t$ (mm)
C_r	Compressive resistance (kN)
C_{rt}	Shear constant
C_t	Torsional modulus constant
C_w	Warping constant
C_x	Coefficient for elastic buckling stress that is dependent on the relative length of a section
$C_{x,\text{CHS}}$	Coefficient for elastic buckling stress of a circular hollow section
$C_{x,\text{EHS}}$	Coefficient for elastic buckling stress of an elliptical hollow section
d_0	Chord diameter (mm)
$d_{0,\text{eq}}$	Equivalent chord diameter (mm)
d_1	Brace diameter (mm)
$d_{1,\text{eq}}$	Equivalent brace diameter (mm)
D	Diameter (mm)
D_e	Equivalent diameter (mm)
$D_{e,\text{new}}$	New equivalent diameter (mm)
$D_{e,\text{RHS}}$	Equivalent rectangular hollow section depth (mm)

E	Young's modulus (MPa)
E_{avg}	Average Young's modulus (MPa)
f	Parameter for new equivalent diameter equation
f'_c	Compressive strength of concrete (MPa)
f_k	Term in connection design equations to account for limiting material strength
f_u	Ultimate stress (MPa)
f_y	Yield stress (MPa)
$f_{y,0}$	Yield stress of chord (MPa)
$f_{y,avg}$	Average yield stress (MPa)
$f_{y,eff}$	Effective yield stress (MPa)
g	Gravitational constant = 9.81N/kg
G	Shear modulus (MPa)
h	Height (mm)
h_0	Chord height (mm)
$h_{0,eq}$	Equivalent chord height (mm)
h_1	Brace height (mm)
h_c	Effective height (mm)
h_m	Parameter to calculate mean perimeter (mm)
H	Larger dimension of elliptical hollow section (mm)
H_{eq}	Larger dimension of equivalent rectangular hollow section
I	Moment of inertia
I_x	Moment of inertia about the major axis (mm ⁴)
I_y	Moment of inertia about the minor axis (mm ⁴)
J	Torsional inertia constant or St. Venant's torsional constant
k	Buckling coefficient
k^*	Alternative buckling coefficient
L	Length (mm)
L_c	Chord length (mm)
L_w	Wavelength (mm); Length of connection (mm)
M	Moment (kN.m); Mass (kg)
M_0	Moment in chord (kN.m)
M_{el}	Elastic moment capacity (kN.m)
M_{pl}	Plastic moment capacity (kN.m)

$M_{pl,0}$	Plastic moment capacity of the chord (kN.m)
M_{pred}	Predicted moment capacity (kN.m)
M_r	Moment resistance (kN.m)
M_u	Ultimate moment (kN.m)
M_x	Moment applied about the major axis (kN.m)
M_y	Moment applied about the minor axis (kN.m)
n	Number of half longitudinal waves; Value used in compressive resistance equation; Stress ratio in chord
N	Axial load (kN)
N_l	Brace load or Connection load (kN)
N_l^*	Connection resistance (kN)
$N_{l(3\%)}$	Brace load at the 3% deformation limit (kN)
N_{lu}	Brace load at the ultimate limit (kN)
P	Perimeter (mm); Load (kN)
$P_{pl,0}$	Load to cause chord to reach plastic moment capacity (kN)
P_m	Mean perimeter (mm)
P_u	Ultimate load (kN)
P_y	Yield load (kN)
Q	Statical moment of area (mm ³)
Q_f	Factor to account for chord normal stresses
r	Radius (mm); Radius of curvature (mm); Radius of gyration (mm)
r_{cr}	Critical radius (mm)
r_e	Equivalent radius (mm)
r_i	Inner radius (mm)
r_{max}	Maximum radius of curvature (mm)
r_{min}	Minimum radius of curvature (mm)
r_o	Outer radius (mm)
r_p	Radius of a circle with perimeter P (mm)
r_x	Radius of gyration about the major axis (mm)
r_y	Radius of gyration about the minor axis (mm)
s	Point along the circumference
S	Elastic section modulus (mm ³)
S_{eff}	Effective elastic section modulus (mm ³)

S_x	Elastic section modulus about the major axis (mm^3)
S_y	Elastic section modulus about the minor axis (mm^3)
t	Thickness (mm)
T	Applied torque (kN.m)
V	Applied shear force (kN)
w	Lateral deformation local buckle (mm); Plate width (mm); Unconnected material length (mm)
X	Position along x-axis on Cartesian co-ordinate system
Y	Position along y-axis on Cartesian co-ordinate system
Z	Plastic section modulus (mm^3)
Z_x	Plastic section modulus about the major axis (mm^3)
Z_y	Plastic section modulus about the minor axis (mm^3)
α	Fraction for inward buckling length; Imperfection factor for buckling curves
β	Localization parameter; Brace width-to-chord width ratio
Δ_1	Connection displacement (mm)
$\Delta_{1(3\%)}$	Connection displacement at 3% deformation limit (mm)
Δ_{1u}	Connection displacement at ultimate limit (mm)
ε	Class limit parameter
ε_u	Elongation at fracture (%)
η	Brace height-to-chord width ratio
θ	Angle measured counter-clockwise from y-axis; Torsional twist
θ_1	Brace angle to chord (degrees)
λ	Member slenderness
$\bar{\lambda}$	Normalized slenderness
λ_0	Limiting slenderness
λ_1	Euler's slenderness
ξ	Eccentricity of the section
ρ	Density (kg/m^3)
σ_c	Axial compressive stress (MPa)
σ_{cr}^C	Corus-based critical buckling stress (MPa)
σ_{cr}^K	Kempner-based critical buckling stress (MPa)
$\sigma_{cr,CHS}$	Critical or elastic buckling stress of a circular hollow section (MPa)
$\sigma_{cr,EHS}$	Critical or elastic buckling stress of a elliptical hollow section (MPa)
$\sigma_{cr,PLATE}$	Critical or elastic buckling stress of a plate (MPa)

τ'	Parameter to increase concrete strength due to confinement
τ_0	St. Venant's shear stress at most external surface (MPa)
$\tau_{\max}$	Maximum shear stress (MPa)
ν	Poisson's ratio
ϕ	Resistance factor
ϕ_{SD}	Parameter for buckling coefficient of an elliptical hollow section
χ	Strength reduction factor for columns

1.0 INTRODUCTION

Elliptical hollow sections (EHS) are the latest steel shape to emerge in construction. EHS have been implemented into various structures found worldwide for their aesthetic appeal and some structural advantages, but this implementation has been done without appropriate design guidelines or equations. Currently, EHS are absent from Canadian codes and guidelines, but some international guides, such as that published by the Steel Construction Institute and British Constructional Steelwork Association (SCI/BSCA, 2008), have recently adopted conservative design equations. Despite being adopted in a variety of applications, structural design guidance is required in order for EHS to be more widely and more efficiently used (Chan and Gardner, 2007). More specifically, non-corporate publication of mechanical and geometric properties will increase their utilization (Packer, 2008). The motivation for research on EHS is to establish both safe and economical design guidelines and equations. As EHS popularity has been growing for truss-based systems, the need to establish these design guidelines and equations for EHS welded connections becomes crucial.

The objectives of this thesis are: 1) to provide a comprehensive overview of EHS research to date including the latest research on EHS welded connections; 2) to introduce EHS to Canada by developing basic geometric property and compressive resistance tables to be potentially added to a future edition of the Canadian Institute of Steel Construction Handbook of Steel Construction; 3) to establish a base for finite element modelling and parametric analyses of EHS connections by studying the behaviour of various EHS-to-EHS T and X connections and the effects of various parameters on the connection; and finally, 4) to develop preliminary design guidelines for EHS T and X connections.

In this thesis, Chapter 1 gives a background to EHS including motivation for research, key terms, applications and advantages. Chapter 2 gives a literature review of EHS related work from EHS buckling modes to EHS connections. Chapter 3 gives the author's contributions to EHS research that is not related to EHS connections, including the establishment of tables to implement into Canadian guides and examining methods to design for EHS columns and beams. Chapter 4 gives the experimental programme and setup for the EHS T and X connection tests, the focus of the research and thesis. Chapter 5 gives the results and analysis of the experiments work. Chapter 6 examines methods to predict T and X connection capacities. Finally, Chapter 7 gives concluding remarks and recommendations for future research.

1.1 EHS DEFINED

EHS are a type of Hollow Structural Section (HSS) that are a relatively new shape to the steel construction world. An ellipse is a specific oval shape which has two different axes of symmetry. EHS

are defined as having one large dimension and one small dimension. The ratio of the large dimension ($H = 2a$) to the small dimension ($B = 2b$) is referred to as the aspect ratio. The aspect ratio of all currently manufactured EHS = 2 (Packer, 2008). The general equation for an ellipse is (Ruiz-Teran and Gardner, 2008):

$$\frac{x^2}{a^2} + \frac{y^2}{b^2} = 1 \quad (1.1)$$

where x and y are the Cartesian co-ordinates, and a and b are the half of the large and small outer dimensions of the EHS, respectively (see Figure 1.1).

In general, HSS are tubular sections that can be either cold-formed or hot-formed (or hot-finished). EHS, however, are manufactured only by the hot-finishing process, and as such, they meet G40.20-04/G40.21-04 (CSA, 2004) Class H or A501 (ASTM, 2007) standards in North America (Packer, 2008). In general, HSS can be manufactured by a seamless process or a welding process. Seamless manufacturing involves an extrusion-type process that pierces solid material to form the tube shape. Weld manufacturing involves bending flat-rolled steel into a tubular shape and then seam welding the edges (CSA, 2003). EHS are currently manufactured by electric resistance welding from a plate and then hot-finishing to the final shape (Corus, 2005).

Figure 1.1: EHS basic dimensions

The behaviour of EHS is a mixture between that of Circular Hollow Sections (CHS) and Rectangular Hollow Sections (RHS). EHS are like CHS in terms of many general properties and behaviour; however, they are different since EHS has a changing radius of curvature whereas CHS does not. EHS are like

RHS in that both have one major axis and one minor axis of symmetry, but they are different because RHS has stiffened corners and flat faces.

The range of products includes: 150 x 75 x 4.0mm up to 500 x 250 x 16mm (H x B x t or 2a x 2b x t, where t is the thickness). The minimum radius of curvature, $r_{\min} = b^2/a$, occurs at the end of the minor y-y axis and is the stiffest part of the EHS cross-section; it can be referred to as the “corner” of the EHS. The maximum radius of curvature, $r_{\max} = a^2/b$, occurs at the end of the major x-x axis and is the least stiff part of the cross-section; it can be referred to as the “flat” portion of the EHS (Chan and Gardner, 2007). The radius of curvature at any point on the section can be found using Equation (1.2) (Theofanous et al., 2009a) where θ is shown in Figure 1.1.

$$r = \frac{b^2}{a} \left(\sin^2(\theta + \pi/2) + \frac{a^2}{b^2} \cos^2(\theta + \pi/2) \right)^{3/2} \quad (1.2)$$

1.2 COMMON APPLICATIONS

EHS have been produced since 1994 in France by Tubeurop, now owned by Condesa (Packer, 2008). Currently, the other world producers include Corus in the United Kingdom, that manufacture the line Celsius® 355 Ovals, and Ancofer Stahlhandel GmbH in Germany (Packer et al., 2009a).

The first structural design which attempted to include EHS dates back to 1845 for the Britannia Bridge; EHS were originally considered for the compression flange of the main box girder (Ruiz-Teran and Gardner, 2008). In 1859, the Royal Albert Bridge designed by Brunel fabricated elliptical sections out of wrought-iron and used them for the primary compression arches (Ruiz-Teran and Gardner, 2008).

More recently, EHS have become popular for glazing systems (glass façades and glass roofs) because they provide good resistance to bending when the strong axis is oriented towards the imposed loads, that is, the wind loads. In addition, they provide an elegant visual appeal and give a sense of being light weight (Packer et al., 2009a). Examples include the Cœur Défense atrium in Paris, France by architect J.P. Viguier (Packer et al., 2009a) and a truss-girder glass system in the AXA building in Paris (Bortolotti et al., 2003).

EHS are also used as columns, as seen in the Swords office at the Airside Business Park in Dublin Airport, Ireland by architects RKD Architects and engineers Thomas Garland and Partners (Packer et al., 2009a). Another structural application of EHS is in the Jarold Department Store in Norwich, United Kingdom (Corus, 2005).

Figure 1.2: Honda exhibit (Corus, 2005)

EHS can be found in some of the newest steel sculptures, as seen at the Honda Exhibit at the Festival of Sound 2005 in Goodwood, Sussex, England by architect Gerry Judah and engineer NRM Bobrowski (see Figure 1.2). The main structure is composed of three 45-metre CHS arches, 457 mm in diameter that support six, 55 metre EHS arms that swing up and down creating a sense of kinetic wonder for the audiences. Each arm supports one of the featured cars and is supported further by small EHS 400x200 to stiffen the arms and enhance appearance (Corus, 2005).

Canadian examples of EHS being used in buildings are the Legends Centre in Oshawa, Ontario and the Electronic Arts stairwell in Vancouver. The latter was the first building in Canada to have used EHS. The Legends Centre in Oshawa is a multipurpose recreational centre where the aquatic centre of the building incorporates EHS columns and was the first in Ontario to have used EHS. This building was awarded the Canadian Institute of Steel Construction (CISC) 2006 Ontario Steel Design Award in the Architectural Category.

EHS are also found in modern airports such as the coach station in Terminal 3 and main building in Terminal 5 of Heathrow Airport in London, as well as the main building in Terminal 4 of Barajas Airport in Madrid (Ruiz-Teran and Gardner, 2008).

EHS are also used in electricity transmission line pylons by EDF, France, pedestrian bridges in the UK, wind turbine masts, urban furniture (such as bus shelters), and handrails (Packer, 2008).

Despite being used in a variety of applications, structural design guidance is required in order that EHS are more widely and more efficiently used (Chan and Gardner, 2007). More specifically, non-corporate publication of mechanical and geometric properties will increase their utilization (Packer, 2008).

1.3 ADVANTAGES OF EHS

Even though minimal design guidance is available, EHS are increasingly being used for certain advantages listed here. An architectural reason for the use of EHS is their modern look (Packer et al., 2009a), plus they are interesting and unusual in appearance (Ruiz-Teran and Gardner, 2008). In glazing systems, they provide an elegant visual appeal and give a sense of being light weight. In addition to their aesthetic appeal, hot-formed EHS have superior mechanical properties compared to North American manufactured HSS (Packer et al., 2009a). This includes their fine grained structure, full weldability, negligible residual stress, and ideal nature for hot-dip galvanizing (Corus, 2005). Because it is a hot-finished product, it has superior resistance to overall flexural buckling when used in compression (Bortolotti et al., 2003).

Since EHS have two different principal axes, there are different flexural rigidities about each of these axes. This allows the section to be oriented to most efficiently resist the applied load (Ruiz-Teran and Gardner, 2008), more specifically wind load for glazing systems (Bortolotti et al., 2003). EHS sections also have high torsional stiffness since they are closed sections. Compared to a CHS with the same area or weight, an EHS has greater bending capacity because of the different principal axes while maintaining a smooth, closed shape (Packer, 2008). In addition, the buckling failure mode for thin CHS may be sudden whereas it is not for thin EHS (Bradford and Roufeginejad, 2008).

2.0 LITERATURE REVIEW

This chapter provides a comprehensive up-to-date literature review of EHS research.

2.1 EHS PROPERTIES

EHS are completely absent from Canadian codes and guidelines. Even the basic EHS sectional property equations and EHS mechanical properties are absent. These properties have been published in the European EN10210 (CEN, 2006a and 2006b), but they have yet to be incorporated into the Canadian Handbook of Steel Construction (CISC, 2010).

2.1.1 GEOMETRICAL PROPERTIES

The EHS sectional properties and dimensions equations that have been published in EN10210 (CEN, 2006a; CEN, 2006b) are shown in (Table 2.1).

Table 2.1: Dimension and sectional property equations

Sectional Property	Formula	Units
Superficial (Surface) Area	$A_s = \frac{P}{10^3}$	(m ² /m)
Cross Sectional Area	$A = \frac{\pi[HB - (H - 2t)(B - 2t)]}{4}$	(mm ²)
Mass per unit length	$M = 0.00785A$	(kg/m)
Moment of Inertia		
Major Axis	$I_x = \frac{\pi}{64} [BH^3 - (B - 2t)(H - 2t)^3]$	(mm ⁴)
Minor Axis	$I_y = \frac{\pi}{64} [HB^3 - (H - 2t)(B - 2t)^3]$	(mm ⁴)
Radius of Gyration		
Major Axis	$r_x = \sqrt{\frac{I_x}{A}}$	(mm)
Minor Axis	$r_y = \sqrt{\frac{I_y}{A}}$	(mm)
Elastic Section Modulus		
Major Axis	$S_x = \frac{2I_x}{H}$	(mm ³)
Minor Axis	$S_y = \frac{2I_y}{B}$	(mm ³)
Plastic Section Modulus		
Major Axis	$Z_x = \frac{[H^2B - (H - 2t)^2(B - 2t)]}{6}$	(mm ³)

Sectional Property	Formula	Units
Minor Axis	$Z_y = \frac{[B^2H - (B - 2t)^2(H - 2t)]}{6}$	(mm ³)
Torsional Inertia Constant	$J = \frac{4A_m^2 t}{U} + \frac{Ut^3}{3}$	(mm ⁴)
Torsional Modulus Constant	$C_t = \frac{J}{t + \frac{2A_m}{U}}$	(mm ³)
	$A_m = \frac{\pi(H - t)(B - t)}{4}$	(mm ²)
	$P = \frac{\pi}{2}(H + B) \left(1 + 0.25 \left(\frac{H - B}{H + B} \right)^2 \right)$	(mm)
	$U = \frac{\pi}{2}(H + B - 2t) \left(1 + 0.25 \left(\frac{H - B}{H + B - 2t} \right)^2 \right)$	(mm)

Note: $H = 2a$ and $B = 2b$

Some of the equations found in Table 2.1 have been scrutinized for their over-conservatism. One of these over-conservative CEN equations is the cross-sectional area formula (Chan and Gardner, 2007), which is repeated as Equation (2.1):

$$A = \frac{\pi}{4} [2a \times 2b - (2a - 2t)(2b - 2t)] \quad (2.1)$$

Chan and Gardner (2007) instead proposed that a more appropriate cross-sectional area (A) would be a product of the mean perimeter (P_m) and thickness (t) of the cross-section, see Equation (2.2). P_m would be calculated based on the mean perimeter formulation developed by Ramanujan (Chan and Gardner, 2007), see Equation (2.3):

$$A = P_m \times t \quad (2.2)$$

$$P_m = \pi(a_m + b_m) \left(1 + \frac{3h_m}{10 + \sqrt{4 - 3h_m}} \right) \quad (2.3)$$

where $a_m = (2a - t)/2$, $b_m = (2b - t)/2$ and $h_m = (a_m - b_m)^2/(a_m + b_m)^2$.

In addition to the cross-sectional area, the elastic section modulus (S) and plastic section modulus (Z) of an EHS shown in Table 2.1 are too conservative as well, according to Chan and Gardner (2008). They proposed the following equations for the x-axis (major axis) and y-axis (minor axis), where a_m and b_m are the same as before, I_x and I_y are the moments of inertia about the major and minor axes, respectively, and θ is measured counter-clockwise from the y-axis:

$$S_x = \frac{I_x}{a} = \frac{4a_m^3 t}{a} \int_0^{\frac{\pi}{2}} \cos^2 \theta \sqrt{\sin^2 \theta + \frac{b_m^2}{a_m^2} \cos^2 \theta} d\theta \quad (2.4)$$

$$S_y = \frac{I_y}{b} = \frac{4a_m b_m^2 t}{b} \int_0^{\frac{\pi}{2}} \sin^2 \theta \sqrt{\sin^2 \theta + \frac{b_m^2}{a_m^2} \cos^2 \theta} d\theta \quad (2.5)$$

$$Z_x = 4a_m^3 t \int_0^{\frac{\pi}{2}} \cos\theta \sqrt{\sin^2\theta + \frac{b_m^2}{a_m^2} \cos^2\theta} d\theta \quad (2.6)$$

$$Z_y = 4a_m b_m t \int_0^{\frac{\pi}{2}} \sin\theta \sqrt{\sin^2\theta + \frac{b_m^2}{a_m^2} \cos^2\theta} d\theta \quad (2.7)$$

2.1.2 MECHANICAL PROPERTIES

EHS are currently produced as hot-finished hollow sections at normalizing temperatures according to EN10210 with the common grade being S355J2H: the yield strength $f_y \geq 355\text{MPa}$, with a Charpy impact resistance of 27 J at -20°C (Packer, 2008 and 2009a). The tolerances on the shapes are as follows (Table 2.2 and Table 2.3) according to EN 10210 (CEN, 2006a; CEN, 2006b):

Table 2.2: EHS tolerances

Characteristic	Tolerance
Outside Dimension	+/- 1% (doubled if $H < 250$ mm) with minimum +/- 0.5mm
Thickness	-10%
Twist	2 mm plus 0.5 mm/m length (both values doubled if $H < 250$ mm)
Straightness	0.2% (doubled if $H < 250$ mm) of total length and 3 mm over any 1 m length
Mass	+/- 6% on individual delivered lengths (+8/-6% for seamless hollow sections)

Table 2.3: EHS mechanical properties

Specified Thickness (mm)	Minimum Yield Strength (MPa)		Specified Thickness (mm)	Tensile Strength (MPa)		Specified Thickness (mm)	Minimum Elongation (%)
$t \leq 16$	355		$t \leq 3$	510-680		$t \leq 40$	22
$16 \leq t \leq 40$	345		$3 \leq t \leq 100$	470-630		$40 \leq t \leq 63$	21
$40 \leq t \leq 63$	335		$100 \leq t \leq 120$	450-600		$63 \leq t \leq 100$	20
$63 \leq t \leq 80$	325					$100 \leq t \leq 120$	18
$80 \leq t \leq 100$	315						
$100 \leq t \leq 120$	295						

2.2 EHS IN AXIAL COMPRESSION

The development of the compressive resistance of EHS and the study of EHS behaviour undergoing axial compression have been investigated for over fifty years. This section will describe the history behind EHS compressive resistance development, plus the experimental tests and finite elemental models to derive cross-sectional classification and design guidelines for EHS under axial compression.

2.2.1 HISTORICAL DEVELOPMENT

The history of EHS research begins with the study of CHS. Independent work by Lorenz in 1908, Timoshenko in 1910, and Southwell in 1914 determined the same elastic buckling stress formula for a CHS (Ruiz-Teran and Gardner, 2008). The elastic buckling stress of a CHS under pure axial compression, $\sigma_{cr,CHS}$, is given in Equation (2.8) where E is Young's modulus, t is the thickness, D is the diameter of the circle and ν is Poisson's ratio (Zhu and Wilkinson, 2007).

$$\sigma_{cr,CHS} = \frac{2Et}{D\sqrt{3(1-\nu^2)}} \quad (2.8)$$

In 1951, the first study on non-circular hollow sections was conducted by Marguerre (Ruiz-Teran and Gardner, 2008). He focused on cylindrical shells of varying curvature, otherwise known as Oval Hollow Sections (OHS). An OHS could be defined by the Fourier polynomial terms shown in Equation (2.9), which is a function of the radius of curvature (r) at a point along the circumference of the section (s), the eccentricity of the section (ξ), the perimeter (P) and the radius of a circle with the perimeter P (r_p).

$$\frac{1}{r} = \frac{1}{r_p} \left[1 - \xi \cos \left(4\pi \frac{s}{P} \right) \right] \quad (2.9)$$

Marguerre showed that the OHS defined by Equation (2.9) was comparable to an ellipse if $0 \leq \xi \leq 1$. When the eccentricity $\xi = 0$ the equation represented a CHS, and when $\xi = 1$ the maximum radius of curvature (r_{max}) was infinity and the aspect ratio became equal to 2.06 (recall the aspect ratio = H/B and is equal to 2 for all currently manufactured EHS). Marguerre assumed a deflection function that i) located the maximum deflection at any given cross-section which was close to, but not at, the point of the r_{max} , and ii) set the deflection at the point of r_{max} equal to zero (Ruiz-Teran and Gardner, 2008). A note to the reader: Marguerre's assumptions later proved to be erroneous (Bradford and Roufeginejad, 2008).

In 1962, Kempner expanded on the ideas proposed by Marguerre by examining OHS, but he decided to assume a different deflection function (Ruiz-Teran and Gardner, 2008). Unlike Marguerre, the deflection function he assumed placed the maximum deflection at the point of r_{max} (see Figure 2.1). The deflection occurred in a localized region of length = $2\beta a$, where β is a localization parameter and $2a$ is the largest EHS outer dimension. Three-dimensional visualization of this localized buckling can be seen in Figure 2.2. Kempner's conclusion was that the elastic buckling stress of a CHS could accurately predict the lower bound solution of the elastic buckling stress of an OHS if the diameter of a CHS was replaced with an equivalent diameter of the OHS (D_e). The equivalent radius of the OHS (r_e) would be equal to r_{max} of the OHS, so the equivalent diameter would be twice the equivalent radius, see Equation (2.10).

$$r_e = r_{max} = \frac{a^2}{b} \text{ and } D_e = 2 \frac{a^2}{b} \quad (2.10)$$

Equation (2.8) thus becomes Equation (2.11):

$$\sigma_{cr,EHS} = \frac{Et}{a^2/b\sqrt{3(1-\nu^2)}} \quad (2.11)$$

Since all currently produced EHS have an aspect ratio of 2, the equivalent diameter can be simplified to $D_e = 2H = 4a$. The development of the buckling stress of an OHS by Kempner in 1962 proved to be more accurate and has been used as a basis for many future papers (Bradford and Roufeginejad, 2008). In summary, to determine the elastic buckling stress of an EHS under pure axial compression ($\sigma_{cr,EHS}$) (Zhu and Wilkinson, 2007), one can use Equation (2.11).

Figure 2.1: Local buckling of EHS according to Kempner (adapted from Chan and Gardner, 2009)

Figure 2.2: Three-dimensional visualization of local buckling (Chan and Gardner, 2007)

In 1964, Kempner and Chen studied the post-buckling behaviour of OHS (Ruiz-Teran and Gardner, 2008). They found that as the aspect ratio approached a value of 1, the post-buckling behaviour became unstable. Note that an aspect ratio $a/b = 1$ corresponds to a CHS. However, as the aspect ratio increased

and approached a plate-like form, the post-buckling behaviour became more stable. They further noted that as the ratio of radius to thickness (r/t) decreased, the post-buckling stability also decreased (Ruiz-Teran and Gardner, 2008).

In 1966, Kempner and Chen showed that OHS with high aspect ratios could attain load carrying capacities above the bifurcation load. The reason for this phenomenon was believed to be due to the redistribution of stresses to the stiffer regions of the section, which are located at the $r_{\min}$ regions (Ruiz-Teran and Gardner, 2008), and not necessarily because of quick strain-hardening (Bradford and Roufeginejad, 2008).

In 1968, Hutchinson was the first person to study the initial and post-buckling behaviour of EHS (note: EHS not OHS¹). He showed that Kempner's 1962 proposal for the elastic buckling stress of an OHS could be applied to an EHS if the section was sufficiently thin. His post-buckling behaviour tests on EHS, however, conflicted with the findings of Kempner and Chen from 1964. Hutchinson found that the post-buckling behaviour of EHS was instead unstable due to high imperfection sensitivity. His reason for the conflicting test results was not because of the difference in oval and elliptical geometry, but rather because of the assumed deflection function that was chosen by Kempner and Chen. Their assumed deflection function did not appear to be suitable to examine initial post-buckling response (Ruiz-Teran and Gardner, 2008).

Kempner and Chen, later in 1968, expanded on their 1964 work by concluding that OHS with small eccentricities (ξ approaching 0), see Equation (2.9), indeed had high imperfection sensitivity that affected post-buckling behaviour. OHS that had large eccentricities (ξ approaching 1), however, had lower imperfection sensitivity. The post-buckling behaviour was thus stable, and loads above the bifurcation load could be attained (Ruiz-Teran and Gardner, 2008).

Both Hutchinson's 1968 and Kempner & Chen's 1968 studies were confirmed by Tennyson, Booton and Caswell in 1971 when they studied the buckling behaviour of EHS with aspect ratios ranging from 1 to 2. Also in 1971, Feinstein, Chen, and Kempner studied the effect of length on the buckling behaviour of OHS. They found that the elastic buckling stress changed as the length approached infinity (Ruiz-Teran and Gardner, 2008).

In 1976, Tvergaard studied the buckling of elastic-plastic OHS under axial compression. He showed that load carrying capacities above the bifurcation load were not possible when the elastic-plastic material behaviour was considered since the $r_{\max}$ regions would prematurely yield. This conflicted with the

¹ An oval is a generic term, while an ellipse is a precise term: all ellipses are ovals, but not all ovals are ellipses. An oval refers to any "squashed" circle; an ellipse is defined as the conic section produced by the intersection of a circular based cone and a plane without the plane intersecting the vertex. Ellipses must have two foci, must have two axes of symmetry and are defined by a mathematical formula (Equation 1.1). Example: the shape of an egg is an oval, but it is not an ellipse.

proposition by Kempner and Chen in 1968, and the studies of Tennyson et al. in 1971. Tvergaard also found that elastic-plastic OHS with high aspect ratios were significantly imperfection sensitive, and the sensitivity increased as the aspect ratio decreased, again in contrast to previous works (Ruiz-Teran and Gardner, 2008). A note to the reader: in the following sections, Kempner and Chen's 1968 finding will be shown to be more accurate.

Over two decades passed before the EHS topic was breached again. The following sections will describe the advances in EHS research regarding compression-loaded EHS in relatively recent years.

2.2.2 BUCKLING OF EHS

The history of EHS has shown that attempts have been made to base EHS compressive resistances on CHS equations. To continue with this, Zhu and Wilkinson (2007) performed finite element (FE) analyses using the FE program, ABAQUS, to simulate the local buckling behaviour of CHS and EHS stub columns.

Typically, for their CHS models, mesh densities were kept the same, but for EHS, higher mesh densities were used around the $r_{\min}$ regions. Three steps of analyses were performed. The first step was to run an eigenvalue buckling analysis on a "perfect" structure to determine probable collapse modes. The second step was to introduce imperfections to the geometry of the "perfect" structure based on the first step. The third step was to perform a nonlinear load-displacement analysis of the structure containing the imperfection from step two using Riks method. Riks method would further perform post-buckling analyses of "stiff" structures that show linear behaviour before buckling (Zhu and Wilkinson, 2007).

The first step was to test for pure elastic buckling. The studies were performed on EHS with aspect ratios (a/b) ranging from 1 to 3 and with slenderness values (H/t) ranging from 20 to 120. The objective was to determine the transition into a buckling state. The FE results were compared to the Kempner proposed equation, see Equation (2.11) (Zhu and Wilkinson, 2007).

For $a/b = 1$, which is equivalent to a CHS, the results from the FE analyses and Kempner's equation approximately matched. Increasing a/b , however, increased the discrepancies between the two values. For larger a/b , it was also found that decreasing the slenderness (H/t) also increased the discrepancies. These discrepancies possibly exist because Kempner's equation was derived based on a CHS elastic buckling formula (Zhu and Wilkinson, 2007). For practical applications, the effect of varying aspect ratios is irrelevant as the products are only currently manufactured with an aspect ratio of 2. As well, the effect of stocky sections not fitting the models is almost irrelevant since the stockiest section currently manufactured is hardly considered stocky. Thus, Zhu and Wilkinson (2007) still support the use of Kempner's approximate equation to safely determine the elastic buckling stresses of EHS.

The FE results also showed large deformations at the $r_{\max}$ regions and little to no deformation at the $r_{\min}$ regions. This is due to the higher stiffnesses at the $r_{\min}$ regions. Near the top and bottom boundaries, but not at the boundaries themselves, the EHS would outwardly deform, and at the mid-length of the model, the EHS would inwardly deform, see Figure 2.3.

Figure 2.3: Maximum deformations at ends and mid-length (adapted from Zhu and Wilkinson, 2007)

Figure 2.4: Buckling wavelengths (reproduced from Bradford and Roufeginejad, 2008)

Figure 2.3 is discussed further by Bradford and Roufeginejad (2008). They assumed that the behaviour shown in Figure 2.3 would occur continuously along the length of an empty EHS if a constant uniaxial strain was applied in the z-direction until a critical strain value was reached. At this critical strain value, elastic buckling would occur. The continuous behaviour is demonstrated in Figure 2.4 where L_w is the wavelength, w is the lateral deformation and α is a value from 0 to 1, such that αL_w is the length of the section deforming inwards (Zhu and Wilkinson, 2007).

The second step in the FE analyses (Zhu and Wilkinson, 2007) was to introduce imperfections based on the eigenvalue buckling analyses from step one. When introducing the imperfections, it was found that larger imperfections resulted in lower buckling stresses. The slenderness of a section appeared to have no effect. It was through these analyses that it was shown that CHS had a greater sensitivity to imperfections compared to EHS; that is, the elastic buckling stress of CHS decreased more than EHS for equivalent areas. This is a reason why EHS are sometimes preferred over CHS. This finding supports the research done by Hutchinson in 1968 and Kempner and Chen in 1968.

The third and final step was to determine the effect of inelastic buckling by introducing plastic material properties to the models (Zhu and Wilkinson, 2007). The models showed that stockier sections would reach their yield point before buckling, while the slender ones would buckle before yielding (i.e. Class 4 behaviour). It was found that the deformation capacity of EHS and their equivalent CHS counterparts, in terms of area, for both stocky and slender sections were very similar. This finding supports the use of the equivalent diameter formula proposed by Kempner, recall Equation (2.11). The results also showed that EHS were generally more ductile than their CHS counterparts; this is another reason why EHS maybe be preferred over CHS.

Zhu and Wilkinson (2007) reported on the University of Toronto compression tests on EHS stub columns and attempted to replicate these tests with FE models. It was found that FE analyses tended to underestimate the real results. The reason for this discrepancy could have been that the FE model may have had a different cross-sectional area when modelled, and the stress-strain behaviour determined from tensile coupon tests may have been inaccurate due to methodology. These tests showed, however, that stocky sections were less sensitive to imperfections. They suggested that additional tests be performed.

Further studies were conducted by Silvestre (2008) to determine the various buckling modes of EHS under compression. Silvestre (2008) formulated the deformations of an EHS by using generalized beam theory and FE analyses to determine the effects of length on the buckling mode.

Using EHS150 x 100 x 6, he determined many buckling modes, including higher order buckling modes. These higher order modes would not likely occur and include asymmetrical behaviour with waves propagating about the EHS perimeter. The predominant or lowest buckling modes, which are likely to occur, are summarized here. For any member length (L) < 720mm, the critical buckling mode was a local-

shell mode similar to Kempner's deformation function (see Figure 2.5: mode 1ls). The buckling pattern would repeat along the length of the member as longitudinal waves. He found that the relationship between L and the number of half-longitudinal wavelengths (n) was $n = L/60$. For $720\text{mm} < L < 2000\text{mm}$, the critical buckling mode was a distortional mode (see Figure 2.5: mode 5), and $2 < n < 4$. The exception occurred when $1200\text{mm} < L < 1300\text{mm}$. For this range, the bifurcation load for mode 1ls was less than that of mode 5, so buckling reverted back to mode 1ls. For $L > 2000\text{mm}$, the critical buckling mode was global buckling (Figure 2.5: mode 2) (Silvestre, 2008).

Figure 2.5: Buckling modes (Silvestre, 2008)

Parametric analyses were conducted to determine how changing t and a/b would affect the buckling mode length ranges (Silvestre, 2008). Through the investigation, it was determined that Equation (2.11) provided a lower bound to the critical buckling stress of an EHS for local shell buckling (mode 1ls), while providing an upper bound to the critical buckling stress of an EHS for distortional buckling (mode 5). Overall, Equation (2.11) became more accurate as t decreased. It was also shown that increasing t decreased the length range for which the local shell mode (mode 1ls) and distortional mode (mode 5) occurred. Thickness had negligible effects on global buckling (mode 2). It was also shown that changing a/b had no effect on the length ranges of buckling modes. The only conclusion that was drawn was that as a/b approached 1, the section would behave like a CHS, and when a/b approached infinity, it behaved like a plate.

2.2.3 EQUIVALENT CHS APPROACHES

This section explores equivalent CHS approaches for the design of EHS compressive members.

2.2.3.1 Elastic Buckling Stress

Chan and Gardner (2007) performed tests on 25 stub columns to study deformation and load-carrying capacities. They restricted their experimental tests to existing manufactured products, that is, EHS

ranging from EHS150 x 75 x 4 to EHS500 x 250 x 16 with an aspect ratio of 2. They found that the stiffness at any given point along the section would vary depending on the radius of curvature and that stiffer parts generally attract more load. Thus, the overall compressive response of an EHS could be given by Equation (2.12), where N is the axial load, σ_c is the axial compressive stress, t is the thickness and P_m is the mean perimeter based on Ramanujan's formula, see Equation (2.3). This equation shows how stockier EHS offer greater load carrying capacities versus their CHS counterparts (in terms of area) since the stiffer regions of the EHS would strain-harden and develop strength.

$$N = \int_0^{P_m} (\sigma_c t) dP_m \quad (2.12)$$

EHS stub columns were tested for the ultimate load (P_u) and the results normalized against the yield load (P_y). For moderately stocky sections, the sections reached and maintained their yield load before failing by inelastic buckling. However, for very stocky sections, the boundary conditions became strain-hardening regions and allowed for ultimate loads greater than the yield load to be achieved, i.e. $P_u/P_y > 1.0$. Chan and Gardner (2007) thus stated that the elastic buckling stress of an EHS ($\sigma_{cr,EHS}$) in compression may be approximated with Equation (2.13). A note to the reader: Equation (2.13) is Kempner's equation, recall Equation (2.11), with the additional term C_x . C_x is the coefficient dependent on the relative length of the section. Based on design guidelines for CHS, for short lengths, $C_x > 1.0$; for medium-length tubes $C_x = 1.0$; and for long tubes $C_x < 1.0$, and could be determined from Equation (2.14) (Chan and Gardner, 2007). Currently, the elastic buckling stress formula found in Eurocode 3 (EC3) (CEN, 2005) uses Equation (2.13) and Equation (2.14).

$$\sigma_{cr,EHS} = \frac{E}{\frac{D_e}{2t} \sqrt{3(1-\nu^2)}} C_x \quad (2.13)$$

$$C_x = 1 + \frac{0.2}{6} \left(1 - 4\sqrt{2} \frac{L}{2a} \sqrt{\frac{t}{2a}} \right) \leq 1.0 \quad (2.14)$$

EC3 may have adopted these equations, but as Ruiz-Teran and Gardner (2008) later show, the effect of length on the elastic buckling stress diminishes as the aspect ratio increases, and the effects of shear deformations contribute less as the length increases. While C_x does account for length and relative slenderness of the section ($2a/t$), the aspect ratio should also be included.

2.2.3.2 Preliminary Cross-Sectional Classification

When EHS undergo pure compression, one of the primary concerns is whether the section will locally buckle in the elastic range; that is, will it exhibit Class 4 behaviour (Chan and Gardner, 2007). For most sections, the Class is determined by comparing a slenderness parameter, which is either the width-to-

thickness ratio or diameter-to-thickness ratio, with Class limits; however, EHS do not have flanges or webs to determine width-to-thickness ratios, nor do they have a constant diameter to determine diameter-to-thickness ratios. As it was found that the elastic buckling stress formula of a CHS could be used for EHS if an equivalent diameter was found, it was important to determine whether the same equivalent diameter could be used in conjunction with CHS Class limits. If not, it would imply that new limits for EHS would have to be derived or new methods to classify the EHS cross-section would have to be developed.

Gardner and Chan (2007) studied the cross-sectional classification system and section classification limits for EHS in compression and made the first propositions of slenderness parameter and limits for EHS. According to EC3 (CEN, 2005), the slenderness parameter for a CHS is defined by Equation (2.15), where D is the diameter of the circle, t is the thickness and f_y is the yield stress. For any value of this CHS slenderness parameter ≥ 90 , the section is considered Class 4 (EN1993).

$$\frac{D}{t\epsilon^2} \quad \text{where} \quad \epsilon^2 = \frac{235}{f_y} \quad (2.15)$$

It was suggested (Gardner and Chan, 2007) that the cross-sectional classifications for circular hollow sections could be adopted for EHS except by using the D_e proposed by Kempner from 1962, recall Equation (2.10). It was thus proposed that the slenderness parameter of an EHS could be written as Equation (2.16). Equation (2.16) can alternatively be expressed as $2H / t\epsilon^2$ for currently manufactured EHS with an aspect ratio = 2.

$$\frac{D}{t\epsilon^2} = 2 \frac{a^2/b}{t\epsilon^2} \quad (2.16)$$

FE models on stub columns were verified against the experimental stub column tests performed by Chan and Gardner (2007). Using verified models, the objective was to perform parametric studies to determine the influence of slenderness and aspect ratios on the ultimate carrying-capacity of EHS, and the ratio of P_u to P_y . Through the numerical analyses they were able to observe stiffness, ultimate loads and general load-end shortening response and failure patterns.

Chan and Gardner (2007) found that as the above slenderness parameter, i.e. Equation (2.16), increased, P_u/P_y decreased. They compared their EHS results to both hot- and cold-formed CHS and showed that stocky EHS exhibited greater load carrying-capacity than their equivalent CHS counterparts (in terms of area). Their belief was that the stiffer regions found at the point of $r_{\min}$ of an EHS experienced strain hardening resulting in the higher load-carrying capacity. For slender sections, however, local buckling occurred before strain hardening occurred, and CHS were more efficient in compression. Thus, the proposition to use the cross-sectional classification system for CHS, per EC3 (CEN, 2005), could safely be adopted for EHS; a slenderness parameter, Equation (2.16), < 90 would be

considered Class 3 (Chan and Gardner, 2007). They further stated that the semi-compact slenderness limit given by the British BS5950-1, the non-compact limit given by the American AISC360-05 and the yield slenderness limit given by Australian AS4100 = 94 were also valid. As such, the equivalent value from the Canadian CSA-S16-09 = 97.9 (for 350MPa yield strength steel) would also appear to be conservative and valid.

By using the Kempner-based equivalent diameter with CHS classification limits, many EHS products currently manufactured will be considered as Class 4 (Packer, 2009a) and subject to over-conservative design (Ruiz-Teran and Gardener, 2008). Even though conservative, the Steel Construction Institute and the British Constructional Steelwork Association have adopted Equation (2.16) (SCI/BCSA, 2008).

2.2.4 ELASTIC BUCKLING STRESS TRANSITION FROM CHS TO PLATE

Figure 2.6: CHS to plate transition (reproduced from Ruiz-Teran and Gardner, 2008)

The research thus far has described approaches to establish design rules for EHS based on an equivalency to CHS. The research, however, acknowledged that EHS behaved differently: EHS with aspect ratios greater than 1.0 were less sensitive to imperfections than their CHS counterparts. Ruiz-Teran and Gardner (2008) performed a study to determine the elastic buckling behaviour of EHS, such that they could explore the transition of EHS from CHS-like behaviour to plate-like behaviour (Ruiz-Teran and Gardner, 2008). When $a/b = 1$, or $a = b$, the section is a CHS, and when a/b approaches infinity, b approaches 0 and the section is similar to two adjacent plates, see Figure 2.6. Ruiz-Teran and Gardner

(2008) investigated both the equivalent radius proposed by Kempner, recall Equation (2.10), as well as the equivalent radius of an EHS as proposed by Corus, a producer of EHS, see Equation (2.17).

$$r_e = a\sqrt{\frac{a}{b}} \quad (2.17)$$

In the work done by Ruiz-Teran and Gardner (2008), the equivalent radii proposed by both Kempner and Corus have been substituted into the CHS elastic buckling stress equation and then used to compare with their results. The Kempner-based elastic buckling stress equation for an EHS ($\sigma_{cr,EHS}^K$) is shown below and is equivalent to Equation (2.11):

$$\sigma_{cr,EHS}^K = \frac{E}{\sqrt{3(1-\nu^2)}} \frac{t}{a^2/b}$$

The Corus-based elastic buckling stress equation for an EHS ($\sigma_{cr,EHS}^C$) is shown in Equation (2.18).

$$\sigma_{cr,EHS}^C = \frac{E}{\sqrt{3(1-\nu^2)}} \frac{t}{a\sqrt{a/b}} \quad (2.18)$$

Ruiz-Teran and Gardner (2008) also compared their results to the elastic buckling stress of a compressed flat plate ($\sigma_{cr,PLATE}$), which was determined by Bryan in 1891 and is given by Equation (2.19). The term w is the width of the plate, and k is the buckling coefficient dependent on the aspect ratio and boundary conditions. E and ν are Young's modulus and Poisson's ratio, respectively.

$$\sigma_{cr,PLATE} = \frac{k\pi^2 E}{12(1-\nu^2)} \left(\frac{t}{w}\right)^2 \quad (2.19)$$

From these three elastic buckling stress equations, Ruiz-Teran and Gardner (2008) found that for a/b approaching a value of 1, both $\sigma_{cr,EHS}^K$ and $\sigma_{cr,EHS}^C$ equations were suitable. However, as a/b increased, the elastic buckling stress of the EHS became similar to a flat plate and neither the $\sigma_{cr,EHS}^K$ or $\sigma_{cr,EHS}^C$ equations predicted it accurately. Ruiz-Teran and Gardner (2008) concluded that the equivalent diameter (or radius) must therefore be dependent on both the aspect ratio (a/b) and the relative thickness ($t/2a = t/H$).

Typically, when CHS undergo uniform axial compression, there are two mechanisms at work that try to resist the buckling effect: a flexural response by the longitudinal strips and an axial response by the transverse rings. The rings are in tension with outward deformation and in compression with inward deformation, see Figure 2.7. The strips and rings have different stiffnesses and require different loads to be forced to buckle. The stiffness of the rings is a function of thickness (t) and radius (r). If the rings are stiff enough, the compression rings can adequately resist the inward deflection. The buckling response, whether axi-symmetric or non-axi-symmetric, is therefore dependent on the ratio r/t . Figure 2.7 demonstrates an example of axi-symmetrical buckling and Figure 2.8 demonstrates a “checker-board” effect or non-axi-symmetrical buckling. It was found that for small r/t or stockier sections, the rings were

stiff enough and the CHS would buckle axi-symmetrically; however, for larger r/t or slender sections, the rings were not stiff enough and CHS would buckle non-axi-symmetrically. Further, the larger the r/t ratio, the more waves per cross-section would result.

Figure 2.7: Longitudinal strips and transverse rings of CHS (Ruiz-Teran and Gardner, 2008)

Figure 2.8: CHS buckling non-axi-symmetrically (Ruiz-Teran and Gardner, 2008)

Similarly, when plates undergo uniform axial compression there are also two mechanisms that resist buckling. These mechanisms, however, are different than those of CHS. The mechanisms for plate buckling are: a flexural response by the longitudinal strips and a flexural response by the transverse strips, see Figure 2.9. The longitudinal strips of a plate behave like the longitudinal strips of a CHS, but the transverse strips are quite stiff at their ends providing high rotational restraint, which is similar to a fixed edge. Thus axi-symmetrical deformations would occur away from these fixed edges only.

The two models that have been just described have been used to model an EHS response to uniform axial compression. EHS will have similar longitudinal strips to both models, but the transverse rings of an EHS will have combined effects from the transverse rings of a CHS and the transverse strips of a plate. Ruiz-Teran and Gardner (2008) showed that the effects of the transverse rings were equal to the effects of the longitudinal strips regardless of aspect ratio (a/b) or relative thickness ($t/2a$); thus, the elastic buckling stress of an EHS was approximated to be twice the Euler buckling stress of longitudinal strips.

Figure 2.9: Longitudinal and transverse strip of a plate (Ruiz-Teran and Gardner, 2008)

Ruiz-Teran and Gardner (2008) created FE models to test a variety of aspect ratios (a/b) ranging from 1 to infinity, and relative slenderness ($2a/t$) ranging from 10 to 4000. Using the idea of the models, elements were placed in both the circumferential and longitudinal directions. The results showed that for very thin tubes (though more slender than currently produced), Kempner's equivalent radius, Equation (2.10), fitted, while for stocky tubes (though more stocky than currently produced), Corus' equivalent radius, Equation (2.17), fitted. The σ_{cr}^K formula showed to be consistently conservative; the σ_{cr}^C formula resulted in non-conservative predictions that are more economical for design purposes.

Their work also showed that a/b and $2a/t$ contributed to the stiffness of the section, and the stiffness in the longitudinal direction versus the transverse direction affected the buckling mode. It affected whether the buckling mode was more CHS-like or more plate-like. Generally, they found CHS-like behaviour involved buckling non-axi-symmetrically and plate-like behaviour involved buckling axi-symmetrically. It was found that for smaller values of $2a/t$, a smaller a/b was required to change buckling modes from CHS-like to plate-like. Based on the currently available sections, the stiffness of the EHS tended to favour CHS.

Ruiz-Teran and Gardner (2008) further investigated the transition of EHS behaviour from CHS-like to plate-like behaviour. They said that for any a/b and $2a/t$, EHS may buckle like a CHS or a plate depending on whether or not higher buckling modes appeared or not. Generally, the lowest buckling mode for EHS with high a/b is the plate-like mode, but the lowest buckling mode for EHS with low a/b is the CHS-like mode. For the practical range of relative thicknesses, a value of $a/b = 1.2$ appeared to be the transition

point. Based on the extensive work they performed on EHS, Ruiz-Teran and Gardner (2008) have proposed the following equations:

$$\sigma_{cr,EHS} = \frac{E}{\sqrt{3(1-\nu^2)}} \frac{2t}{D_e} C_x > \sigma_{cr,PLATE} \quad (2.20)$$

$$\sigma_{cr,PLATE} = \frac{k^* \pi^2 E}{12(1-\nu^2)} \left(\frac{t}{2a}\right)^2 \varphi_{SD} \quad (2.21)$$

$$k^* = \begin{cases} 6.97 + 0.03(5 - L/2a)^3, & 2 \leq L/2a \leq 5 \\ 6.97, & 5 \leq L/2a \end{cases} \quad (2.22)$$

$$\varphi_{SD} = 1 - 16 \left(\frac{t}{2a}\right)^2 \quad (2.23)$$

$$C_{x,EHS} = \begin{cases} 1 - (1 - C_{x,CHS}) \left(\frac{2.5 - a/b}{1.5}\right)^2, & a/b < 2.5 \\ 1.0, & a/b \geq 2.5 \end{cases} \quad (2.24)$$

With the effects of shear deformation, Ruiz-Teran and Gardner (2008) proposed a new and less conservative equivalent diameter ($D_{e,new}$) to be substituted as D_e above, see Equations (2.25) and (2.26), where f is a parameter to account for the relative slenderness of the section.

$$D_{e,new} = H \left[1 + f \left(\frac{H}{B} - 1 \right) \right] \quad (2.25)$$

$$f = 1 - 2.3 \left(\frac{t}{H} \right)^{0.6} \quad (2.26)$$

The cross-sectional slenderness limit would then be:

$$\frac{D_{e,new}}{2a} = 1 + f \left(\frac{a}{b} - 1 \right) < \frac{8\sqrt{3(1-\nu^2)}}{k^* \pi^2} \frac{2a}{t} \frac{1}{\varphi_{SD}} \quad (2.27)$$

This new equivalent diameter will be used in Chapter 3 when developing the compressive resistance tables for a future edition of the Canadian Handbook of Steel Construction, except the Canadian CSA-S16 CHS Class limits will be used.

To support the work by Ruiz-Teran and Gardner (2008), Silvestre and Gardner (2011) also investigated the transitional behaviour of EHS from CHS to plates. They understood that there was a transitional behaviour, but looked more closely at the elastic post-buckling behaviour of EHS. They performed parametric FE analyses on six EHS with varying aspect ratios ($a/b = 1.1, 1.25, 1.5, 2, 3$, and 5), all with the same cross-sectional area. With regard to imperfection sensitivity, they confirmed the transitional behaviour: low a/b led to imperfection sensitivity and shell-type buckling, while higher a/b ratios led to imperfection insensitivity and plate-type buckling.

They showed that regardless of the aspect ratio, all curves on stress vs. axial shortening graphs followed the same linear path until the EHS column reached its peak stress. The peak stress was a value less than the theoretical critical stress. After this peak stress, all EHS showed local minima in the graphs. After this, for the EHS with $a/b \leq 1.5$, the capacity of the column never reached the peak stress again, but for $a/b \geq 2$, the capacities increased to above the peak stress. As the aspect ratios increased, the slope of the curves post-peak increased as well.

Silvestre and Gardner (2011), in addition, looked at the stress profiles about the perimeter of the EHS (with varying aspect ratios) at the column's mid-height. Apparent in all stress profiles, regardless of aspect ratio was i) an accumulation of compressive stresses at the regions of $r_{\min}$, and ii) low, but relatively uniform compressive stresses at the regions of $r_{\max}$. With this result, they proposed that an approach based on the effective width concept that is often used for the strength analysis of flat plates be adapted for the design of EHS tubes with moderate to high aspect ratios ($a/b > 1.5$).

2.2.4.1 Class 4 Sections

After using the less conservative equivalent diameter formulas, Equations (2.25) and (2.26) with CHS Class limits to determine cross-sectional classification, methods to treat the Class 4 sections were investigated by Chan and Gardner (2007). There are a few methods currently available that can be carried forth to determine the design strength of a Class 4 section. One method to handle Class 4 sections is to use an effective area. Chan and Gardner (2007) proposed the following effective area (A_{eff}) formula based on the effective area for a CHS from BS5959-1:

$$A_{\text{eff}} = A \left[\frac{90}{D_{e,\text{new}}/t} \frac{235}{f_y} \right]^{0.5} \quad (2.28)$$

Silvestre and Gardner (2010) made additional proposals about the design of EHS Class 4 members in axial compression based on adaptations of CHS Class 4 design rules found in EC3. According to EC3 (CEN, 2005), the slenderness of a column (λ) is the square-root of the yield stress (f_y) to critical stress (f_{cr}) ratio, and the ultimate strength of a CHS tube is $f_u = \chi f_y$ where χ is the strength reduction factor. If $\lambda < 0.2$, then $\chi = 1.0$ and $f_u = f_y$. If $\lambda > \text{plastic limit}$, then a fully elastic buckling collapse occurs. Then $\chi = \text{function of } \lambda$ and an elastic imperfection reduction factor (the elastic imperfection reduction factor is a function of the imperfection sensitivity). If $0.2 < \lambda < \text{plastic limit}$, then an elastic-plastic failure occurs. Then $\chi = \text{function of } \lambda$ and the plastic limit, where the plastic limit is also a function of an elastic imperfection reduction factor. Accounting for an equivalent diameter of an EHS, they proposed the lower limit could change from 0.2 to 0.3, and since EHS are less imperfection sensitive than CHS, the plastic limit could

adjust as well; they determined the plastic limit to be 1.5. They proposed Equation (2.29) to be the strength reduction factor for EHS.

$$\chi = \begin{cases} 1 & \text{if } \lambda \leq 0.3 \\ 1.15 - 0.5\lambda & \text{if } 0.3 < \lambda < 1.5 \\ \frac{0.9}{\lambda^2} & \text{if } \lambda \geq 1.5 \end{cases} \quad (2.29)$$

Silvestre and Gardner (2010) compared their strength curves with the strength curve generated if Chan and Gardner's (2007) effective area formula, Equation (2.28), was used, i.e. $\chi = A_{\text{eff}} / A$. Silvestre and Gardner (2010) found that the Chan and Gardner's (2007) curve was much more conservative than the curve they developed. The improvement came from accounting for the higher post-critical strength of EHS, as opposed to CHS.

2.2.5 EXPERIMENTAL TESTS ON EHS LONG COLUMNS

Thus far, experimental studies focused on sectional behaviour. Chan and Gardner (2009) studied the behaviour of pin-ended EHS long columns under flexural buckling. The studies included experimental tests on EHS150 x 75 sections with thicknesses of 4, 5 and 6.3 mm, and column lengths ranging from 0.7 to 3.1 metres. Experiments were conducted for the column-buckling responses for both major axis buckling and minor axis buckling.

For the load-vertical deflection response, it was found that for stocky sections that were fully effective in compression (e.g. EHS 150 x 75 x 6.3), columns that buckled about their major axis or minor axis reached similar peak loads. During unloading, however, the columns exhibited different behaviour: the load capacity of columns that buckled about the minor axis dropped quicker in comparison to columns buckling about the major axis. For these minor axis buckling columns, the maximum compressive stress occurred at the r_{max} parts of the section, which is the most susceptible to local buckling. When unloading these columns, the effects of inelastic local buckling and hinge formation were allowed to factor into the behavioural response. This led to a quick drop in load carrying capacity. For the columns buckling about the major axis, the maximum compressive stress occurred at the r_{min} parts of the section, which is the stiffest portion of the section. The local buckling effects that had affected the minor axis buckling columns were not as influential; thus, for the columns that buckled about the major axis, there were more gradual drops in load carrying capacity. For the load-lateral deflection response, the tests showed good correlation with the theoretical second-order elastic and plastic models. Loading members exhibited second-order elastic response and unloading members exhibited second-order plastic response (Chan and Gardner, 2009).

Chan and Gardner (2009) followed their experiments with FE modelling in order to replicate the experimental results, validate the numerical models and to perform parametric studies. The objective was to investigate the influence of the cross-sectional slenderness, aspect ratio and member slenderness on the load-carrying capacity. The results were compared against three codes: the European EC3, the North American AISC360 and the Australian AS4100. It was determined that the EHS150 x 75 sections with thicknesses of 5 or 6.3 mm were fully effective members (Class 1 to 3), but the EHS150 x 75 x 4 section was a slender member (Class 4). The results for buckling about both axes followed a similar trend to the CHS counterparts. The buckling curves for the above three standards were shown to be very close to each other, and it was found that the compressive resistance curves for CHS could be adopted for the buckling of EHS about either axis.

2.2.6 EQUIVALENT RHS APPROACH

While many studies are establishing an equivalency between an EHS and a CHS, so that the slenderness limits of a CHS can be used, Zhao and Packer (2009) suggested that determining an equivalent RHS shape would be more worthwhile. As discussed, the local buckling failure mode for currently manufactured EHS stub columns in axial compression exhibits behaviour similar to plate buckling (or the buckling mode of a RHS) rather than the circumferential shell buckling mode of a CHS. For this reason, Zhao and Packer (2009) proposed to use the equivalent RHS depth ($D_{e,RHS}$) equal to $2a = H$, with an effective width determined by maintaining the same cross-sectional area and thickness, see Figure 2.10.

Figure 2.10: Equivalent RHS for an EHS

This approach is different to other proposed equivalency methods. Where other D_e equations have been established for classification purposes, $D_{e,RHS}$ attempts to create a section actually equivalent in terms of cross-sectional area and thickness. The equivalent RHS is a rational conversion method for the design of compression and tension members since it maintains the same cross-sectional area as the original EHS. Zhao and Packer (2009) showed that using this equivalent dimension, the slenderness limit of a RHS can safely be adopted for all EHS using the EC3 (CEN, 2005) or S16 (CSA, 2009) RHS Class limits, see Equation (2.30). Note: the corner radii of RHS were taken as $1.5t$ for EC3 (CEN, 2005) and $2t$ for S16 (CSA, 2009).

$$\frac{D_{e,RHS} - 3t}{t} \leq 42 \sqrt{\frac{235}{f_y}} \quad \text{or} \quad \frac{D_{e,RHS} - 4t}{t} \leq \frac{670}{\sqrt{f_y}} \quad (2.30)$$

In Chapter 3, the reader will be shown the additional work performed by this author to determine the validity of this equivalent RHS approach in predicting capacities of long columns and beams.

2.3 BENDING, SHEAR AND COMBINATION LOADING

Other than the behaviour of EHS under axial compression, studies have been conducted to determine EHS moment resistance, shear resistance and the resistance under combined loadings.

2.3.1 BENDING RESISTANCE

Bending tests on EHS about the major axis and minor axis were performed by Chan and Gardner (2008). They performed 18 bending tests on EHS400 x 200 with thicknesses of 8, 10, 12, 14 and 16mm, and on EHS500 x 250 x 8. Seven beams were tested about the major axis and 11 beams about the minor axis. They also performed both 3-point and 4-point bending tests. Chan and Gardner (2008) used their experimental data with FE analyses to derive less conservative elastic and plastic section moduli for design purposes, recall Equations (2.4) to (2.7). From their experiments, they captured the moment-rotation behaviour and established its relationship to the cross-sectional slenderness. Overall, they found that the moment capacities of 3-point bending tests were higher than 4-point bending tests.

Law and Gardner (2010) studied EHS in bending to investigate member instability, i.e. lateral torsional buckling. Since EHS are closed sections, they have high torsional stiffness; thus, it was expected that lateral torsional buckling was not an inherent dominant failure mode. Law and Gardner (2010) performed eight 3-point bending beam tests on EHS150 x 75 x 5, loaded about the major axis. They investigated beam lengths of 4, 6, 8 and 10 metres to obtain member non-dimensionalized slenderness ratios ranging from 0.29 to 0.48. They found that all tests largely failed by in-plane bending. The only

apparent influence of lateral torsional buckling occurred when the member approached its peak moment. At this point, the beam laterally deflected and twisted slightly. As expected, increasing the member length (i.e. member slenderness) increased the effect of lateral torsional buckling. Shorter beams actually reached an ultimate moment capacity higher than the plastic moment capacity because of the strain hardening phenomena that would have occurred in the stiffer portions of the EHS.

These experimental results were validated against FE models. With parametric analyses, they investigated the influence of aspect ratios. As the aspect ratio increased (transitioning from a CHS to plate), instability increased. They conservatively recommend that, based on the buckling curves found in EC3 (CEN, 2005), buckling curve “b” is good for $a/b \leq 2$, buckling curve “c” for $2 < a/b \leq 3.1$ and buckling curve “d” for $a/b > 3.1$. They also investigated the influence of the member length. They found that by increasing the member length they increased member instability; for an EHS with an aspect ratio = 2 a member length of 5.66 metres or greater resulted in lateral torsional buckling.

2.3.1.1 Cross-Sectional Classification

The cross-sectional classification of EHS in axial compression used D_e , $D_{e,new}$ or $D_{e,RHS}$ to determine a slenderness ratio. When EHS undergo pure compression, one of the primary concerns is whether the section will locally buckle in the elastic range; that is, will it exhibit Class 4 behaviour (Chan and Gardner, 2007). But when EHS undergo flexural compression, the section can be considered Class 1, 2, 3 or 4. According to EC3 (CEN, 2005) or S16 (CSA, 2009), Class 1 sections are able to reach and maintain full plastic moment capacity in bending and thus can be used in plastic design; Class 2 sections also reach the full plastic moment in bending, but with somewhat less deformation capacity; Class 3 sections cannot reach the full plastic moment due to inelastic local buckling and this limits the bending moment resistance to the elastic (yield) moment; finally Class 4 sections locally buckle in the elastic range and the bending resistance is typically determined based on an effective cross section.

Recall that for classification purposes, Ruiz-Teran and Gardner (2008) developed the less conservative $D_{e,new}$ equation for EHS under axial compression, but the more conservative D_e equation that was originally proposed by Kempner, recall Equation (2.10), was developed for pure axial compression assuming that local buckling initiated at the point of r_{max} . It was determined for EHS bending about the minor axis that local buckling also initiated at the point of r_{max} . Thus, Equation (2.10) was found suitable for EHS bending about the minor axis. However, for major axis bending, Gardner and Chan (2007) stated that buckling was not initiated at the point of r_{max} or r_{min} , but at a critical radius (r_{cr}) defined by Equation (2.31). Through FE modelling with parametric analyses to determine the influence of aspect ratios on the

location of buckling initiation, Chan and Gardner (2008) proposed the following slenderness parameters for major axis bending, see Equation (2.32).

$$r_{cr} = 0.65 \frac{a^2}{b} \quad (2.31)$$

$$\frac{D_e}{t\epsilon^2} = \begin{cases} 0.8 \frac{(a^2/b)}{t\epsilon^2} & \frac{a}{b} > 1.357 \\ 2 \frac{(b^2/a)}{t\epsilon^2} & \frac{a}{b} \leq 1.357 \end{cases} \quad (2.32)$$

In Equation (2.32), the 1.357 limit physically means that for lower aspect ratios, local buckling will occur at the region of maximum compressive stress; this coincides with the point of r_{min} , which is the stiffest region of the EHS. For higher aspect ratios, local buckling will occur away from the maximum compressive stress region, but closer to the neutral axis where the less stiff region exists (Theofanous et al., 2009b). Recall, a/b for all currently manufactured EHS = $2 > 1.357$. Overall, they found that the EC3 Class limits for CHS could be safely adopted for EHS. Gardner and Chan (2007) proposed, however, that the slenderness limits be relaxed for Class 3, for both CHS and EHS, to $140\epsilon^2$ instead of $90\epsilon^2$.

2.3.1.2 Class 4 Sections

For Class 4 sections in bending, S16 (CSA, 2009) Clause 13.5c stipulates the use of either an effective yield stress or an effective section modulus. Chan and Gardner (2008) have developed an effective elastic section modulus (S_{eff}), as shown in Equation (2.33), where the elastic section modulus (S) is determined based on Equation (2.4) or Equation (2.5), D_e is based on Kempner's equation, Equation (2.10), and assuming the EC3 Class 3 limit can be relaxed to $140\epsilon^2$ compared to $90\epsilon^2$.

$$S_{eff} = S \left[\frac{140}{D_e} \frac{235}{f_y} \right]^{0.25} \quad (2.33)$$

In Chapter 3, the author will show how the equivalent RHS approach can be used to predict moment capacities and how to deal with Class 4 sections.

2.3.2 SHEAR RESISTANCE

Experimental studies by Gardner et al. (2008) were conducted on EHS150 x 75 with thicknesses of 4, 5 and 6.3 mm to determine the cross-sectional shear resistance of EHS. A three-point bending set-up was used with varying span-to-depth ratios to develop a bending moment gradient with uniform shear zones. The span-to-depth ratios ranged from 1 to 4 for major axis bending and 2 to 8 for minor axis bending. Overall, it was found that the moment capacity decreased with increasing shear.

FE models were developed to replicate the experiments, validate numerical models and perform parametric studies. The goal was to investigate the influence of the cross-sectional slenderness, aspect ratio and span on the ultimate moment capacity and shear capacity. Gardner et al. (2008) proposed equations to design for the plastic shear strength of an EHS. They found that the shear area (A_v) of an EHS can be found by using Equation (2.34). They also deduced that if the design shear strength is $< 50\%$ of the plastic shear resistance, the shear effect on bending is negligible; however, if the design shear strength is $> 50\%$, the shear would cause the moment resistance to decrease.

$$A_v = \begin{cases} 2t(2a - t) & \text{major axis bending} \\ 2t(2b - t) & \text{minor axis bending} \end{cases} \quad (2.34)$$

2.3.3 INTERACTION CURVES

Figure 2.11: Interaction surface for EHS with $a/b = 2.13$ (Nowzartash and Mohareb, 2009)

Nowzartash and Mohareb (2009) derived potential plastic interaction relationships for EHS when applying axial forces, bi-axial bending moments, and torsion. They used a lower-bound theorem of plasticity since it would underestimate the exact value, thus being suitable for design. For derivation purposes, they assumed that the EHS cross-section remained undistorted, which is acceptable for EHS with low D/t ratios where cross-sectional ovalization or distortion is negligible. This led to the assumption that there was no distinction between true stress and engineering stress. They also assumed that the entire section would attain full plastic resistance before locally buckling (i.e. Class 2 according to EC3 (CEN, 2005) and S16 (CSA, 2009) limits for CHS). They idealized the stress-strain behaviour of steel to be bilinear; this was done to neglect strain hardening effects leading to the lower bound approximation. Normal stresses that were induced by warping, shear stresses through the thickness that

were normal to the mid-surface stresses, and circumferential stresses were all considered negligible and neglected. The effects of residual stresses and failure via global buckling were neglected as well.

Through their derivations (Nowzartash and Mohareb, 2009), the interaction surfaces for EHS were developed. An example is shown in Figure 2.11 for $a/b = 2.13$, which is approximately the aspect ratio of existing EHS product. The relations they developed can be used for design purposes to determine whether a section can withstand combined forces. The solution was verified using FE software, which they were able to use to generate a single, simplified design equation.

Experimental tests on EHS under compression and biaxial bending were performed by Chan et al. (2010). They performed tests on EHS150 x 75 x 5 and EHS150 x 75 x 6.3 with column lengths shorter than 300mm. They were thus looking at the interaction of compression and bending on a sectional level only. An axial load was applied to the columns to obtain a compressive force, and the load was applied eccentrically to obtain bending moment about either axis. The maximum moment was taken at mid-height and = applied load x (eccentricity + mid-height deflection). As expected, increasing the load eccentricities decreased capacities. Chan et al. (2010) further performed FE analyses to first determine buckling mode shapes, which could then be incorporated into geometric and non-linear analyses. Using equivalent CHS diameter approaches to determine cross-sectional classification, they found that all sections were Class 1-3 for axial compression, Class 1 for major axis bending and Class 3 for minor axis bending. Using the less conservative equations for the elastic and plastic moduli of an EHS to determine elastic and plastic moment capacity, recall Equations (2.4) to (2.7), and the single, simplified interaction equation proposed by Nowzartash and Mohareb (2009), they proposed one equation for Class 1 or 2 sections, and a separate equation for Class 3 sections. Both these proposed equations would provide safe-side predictions.

As Chan et al. (2010) were investigating at a sectional level, Law and Gardner (2010) looked at the member level and investigated lengths of 1, 2 and 3 metres. They performed 33 tests: 6 pure compression tests (3 buckling about the major axis and 3 about the minor axis), 18 eccentric compression tests (9 inducing major axis bending, and 9 inducing minor axis bending), and 9 compression plus induced bending about both axes. Using a maximum moment = applied load x (eccentricity + mid-height deflection), they found that the tests generally followed the interaction curve shown by Nowzartash and Mohareb (2009); however, Law and Gardner (2010) also noted that some data points fell below the curves. This still requires further research.

2.4 CONCRETE FILLED EHS

Before tests were conducted on concrete filled (CF) EHS, they were first conducted on other types of HSS filled with either plain or fibre-reinforced concrete (Campione et al., 2000). In these earlier tests, it

was shown that the concrete prevented or reduced the risk of buckling, both local and global, and significantly increased the column capacities. It was determined that, similar to spirally reinforced concrete columns which provide confinement to a concrete core through the use of spiral steel reinforcement, HSS provided confinement to the concrete as well. For CF CHS, steel failure occurred when the steel longitudinal stress went to zero and the transversal ring (or hoop) stress went to rupture, and the effects of local buckling could be neglected if $D/t < 90\epsilon^2$ (Campione et al., 2000).

More tests were done to distinguish the effects of CF Square Hollow Sections (SHS), RHS and CHS (Tao et al., 2005). Like Campione et al. (2000), they showed that the steel tube provided confinement and the concrete core prevented inward buckling of the steel tube. Additionally, they showed that SHS and RHS CF columns were not as efficient as CF CHS because the confining pressures were not as good. As a result, local buckling was more likely to occur.

Experiments were further conducted to determine the effects of different loading conditions. CF CHS stub columns were loaded uni-axially; the specimens were loaded on i) the entire section, ii) only on the steel tube, and iii) only on the concrete core (Johansson and Gylltoft, 2002). Specimens loaded over the entire section experienced confinement effects, as shown by Campione et al. (2000), but were additionally affected by the difference in the Poisson's ratios of steel and concrete. The Poisson ratio of steel is greater than that of concrete, thus the lateral expansion of the steel tube was higher than the concrete core. This initial condition did not allow the steel tube to confine the concrete core. However, once the concrete began to plasticize, the lateral expansion of the concrete caught up with the steel and the steel began to undergo tensile hoop stress confining the concrete. Specimens loaded only on the steel behaved similarly to empty tubes. The load caused the steel to expand outwards, and the steel-concrete bond thus broke. This bond breakage prevented any load from redistributing into the concrete. Specimens loaded only on the concrete core showed the greatest axial capacity and confinement. The concrete would ultimately fail by crushing and expanded most at the mid-height of the column.

With increasing research on EHS, CF EHS research began. Like other HSS, studies showed that CF EHS columns provide greater axial compressive capacity than their unfilled EHS counterparts. Like other HSS, the infill prevented the tube from buckling inward thus increasing the local buckling capacity of the section (Bradford and Roufeginejad, 2008).

The effects of infill inside an EHS were studied by using an energy-based technique to determine the elastic local buckling stress for thin-walled EHS when concentric axial compression was applied (Roufeginejad, 2007). Empty EHS were considered to have zero elastic stiffness and filled EHS were considered to be infinitely stiff. Recall Figure 2.4, which showed the penetration of inward buckling length as a function of the wavelength (L_w) and a parameter α , which has a value between 0 and 1. For an empty EHS with zero stiffness, α was equal to 0.5; for a filled EHS with an infinitely stiff infill, α was

equal to 0 (Bradford and Roufeginejad, 2008). Depending on the stiffness of the infill, the buckling mode would vary in terms of the regions buckling outward and the regions that would penetrate the infill. The energy formulation for EHS involved one-degree of freedom and included the strain energy stored due to bending, membrane strain energy due to stretching, work done by compressive forces and strain energy stored in the elastic infill when the buckled tube penetrated into it. The formulation focused on the change in potential energy from a pre-buckled state to a buckled state and it was found, by minimizing the change in potential energy, that an eigenvalue representation of the buckling load could be achieved. The results showed that the rigid infill tube was square root of 3 times stronger than the empty tube (Roufeginejad, 2007). This eigenvalue representation is called the buckling co-efficient, and is fundamentally a function of the localization parameter β (Figure 2.1) and the buckling wavelength L_w (Figure 2.4) (Bradford and Roufeginejad, 2008). The results also demonstrated that the replacement of CHS by EHS provided little to no difference in terms of the economy of design against local buckling (Roufegarindjad, 2007).

In 2007, tests on EHS filled with self-compacting concrete were performed (Zhao et al., 2007). Zhao et al. (2007) tested unfilled and filled EHS150 x 75 with thicknesses of 4, 5 and 6.3 mm and EHS200 x 100 with thicknesses of 5, 6.3, 8 and 10 mm with an average concrete strength of 69.2MPa. They found that slender unfilled EHS locally buckled, and stocky unfilled EHS buckled in the inelastic range. The classification limits of AS4100 were shown to be suitable for these tests. Filled EHS delayed or almost eliminated the local buckling effects, similar to plain-concrete filled tubes. Similarly, the self-compacting concrete increased the load carrying capacity, ductility and energy absorption. Predicting the ultimate load using $P_u = A_s f_y + A_c f'_c$ overestimated the load carrying capacity of the specimen by 3.2% while $P_u = A_s f_y + 0.85 A_c f'_c$ underestimated it by 3.2%. In these equations, A_s is the area of steel, A_c is the area of concrete, f_y is the yield strength of steel, and f'_c is the compressive strength of concrete.

Tests similar to those performed by Johansson and Gulltoft (2002) were carried out at the University of Toronto except on CF EHS (Brienza, 2008). The tests involved five EHS stub columns with various loading conditions, see Figure 2.12. The results from the steel-only loaded specimen showed similar behaviour to that of an empty EHS specimen. The ultimate strength could be predicted by $P_u = A_s f_y$. Post-peak behaviour varied, however, as CF EHS could sustain higher loads at these higher deformations in comparison to their empty EHS counterparts. This was because the concrete prevented inward local buckling of the steel tube. Steel and concrete loaded specimens were under-predicted by approximately 3% by using $P_u = A_s f_y + A_c f'_c$; thus, the confinement effects were not as much as previously thought. As such, this topic requires further research. Concrete-only loaded specimens showed considerably more strength due to confinement compared with the other tests, thus showing that “passive confinement” is better for strength gain than “active confinement”.

Figure 2.12: Loading conditions on concrete filled EHS (Brienza, 2008)

Yang et al. (2008) expanded on the experiments done by Zhao et al. (2007) by additionally performing 21 tests: 15 tests were compositely-loaded, of which 6 were greased before the concrete was poured inside, and another 6 tests were core-loaded. The results of these tests showed that the compressive response of CF EHS were dependent on both the steel tube thickness and the concrete strength. Increasing the tube thickness increased the load-carrying capacity and ductility. Increasing the concrete strength increased load-carrying capacity, but decreased ductility. Greasing the specimens simulated the behaviour of concrete shrinkage. It was found that there was less than 5% variation in the ultimate load of greased vs. non-greased sections (i.e. shrinkage effects are minimal); however, further research was recommended. Similar to previous findings, core-loaded specimens showed greater load-carrying capacity and ductility than compositely loaded specimens. When comparing design equations from various design guides, codes, and studies on CF EHS columns, the conclusion was that CF EHS showed an intermediate behaviour between CF RHS and CF CHS, as previously determined.

Zhao and Packer (2009) have since proposed design methods for CF EHS in axial compression based on the end-loading conditions. For compositely loaded sections, they looked at the simple superposition approach, such that $P = A_s f_y + A_c f'_c$, and found that the mean ratio of P_u/P was 1.014 with a COV of 0.062. They looked at a CF CHS approach, which used equations prescribed in CSA S16-01 and Eurocode 4 for CF CHS. To use the CHS equations, they determined D_e for an EHS. They found that the predicted capacities matched the experimental values within 2 to 5%, with a COV of approximately 0.05. They finally looked at a CF RHS approach, which used the equations prescribed in CSA S16-01 and

Eurocode 4 for CF RHS. To use the RHS equations, they used an equivalent RHS, recall Section 2.2.6. They that found the expressions simplified back into the simple superposition approach. Overall, the predicted ultimate capacities were considered good.

For loading through the concrete alone, the same three approaches were analyzed except the steel terms were eliminated. The simple superposition and CF CHS approaches significantly underestimated the load carrying capacity. The simple superposition approach simply ignored the confinement effects, explaining the underestimation. For the CHS approach, the S16 expressions were analyzed further. In S16 (CSA, 2009), τ' is a term which represents the increased concrete strength due to confinement. By using the equivalent diameter of an EHS, it resulted in a diameter much larger than the actual largest dimension of the EHS. The term D_e/t became too large and L/D_e became too small. As such, τ' was low and the load carrying capacity was underestimated. It was found, however, that by using an equivalent RHS approach with an equivalent RHS depth, the D/t and L/D values were more appropriate and the predicted capacities were much closer to the experimental results. The results show a mean ratio of 1.053 with a COV of 0.064. This result supports the Zhao and Packer (2009) proposal to use an equivalent RHS dimension to determine the cross-sectional slenderness of an EHS.

Finally, for loading through the steel alone, the same approaches were analyzed except the concrete terms were eliminated, and the confinement terms were set equal to 1.0. The only difference between the filled and unfilled stub columns was that filled columns changed the typical buckling mode to only outward deflection. The predicted capacities matched the experimental results with a mean ratio of 0.999 and COV of 0.014.

Thus far, the studies described above all deal with short columns. Jamaluddin et al. (2010) performed 10 tests on CF EHS with column heights of 300mm (stub column), 1500mm and 2500mm. They also varied the concrete strength using both normal and high strength concrete infill. They first confirmed their predecessors' observations through their stub column tests. They also showed that the capacities of CF EHS stub columns dropped after reaching the peak load, but became stable before complete failure. Conversely, the CF EHS longer columns did not achieve the same stability after peak load; capacities dropped suddenly due to global buckling. As expected, increasing the column length decreased column capacity. By increasing the global slenderness ratio, they found that the confinement effect reduced and elastic buckling was more prominent than plastic failure.

Additionally, Jamaluddin et al. (2010) observed that the strength and stiffness of the CF EHS was proportional to concrete strength; however, by using the higher strength concrete, shear failure in the concrete occurred. Shear failure in the concrete meant a more sudden failure. They also noted that as the concrete strength increased, the inner core had a higher stiffness. A higher stiffness meant less lateral

expansion of the concrete core. With less expansion, the core did not engage with the exterior EHS, so there was less confinement pressure.

2.5 STAINLESS STEEL OHS

As discussed, currently existing EHS products are made from hot-rolled carbon steel and a large number of studies have focused on these existing products; however, a smaller market does exist for cold-formed stainless steel OHS. While EHS have an aspect ratio = 2, SS-OHS have an aspect ratio = 1.5. Where carbon steel (CS) has a definitive yield stress, the stainless steel (SS) stress-strain curve is rounded; therefore, a 0.2% proof (or strain offset) stress must be determined in lieu of a yield stress for structural resistance calculations, and a secant modulus must be determined for serviceability calculations. While the stainless steel curve demonstrates the strain-hardening phenomena, design codes typically do not incorporate it. The austenitic stainless steel grade investigated was 1.4401 (316), which has a minimum specified yield stress (0.2% proof stress) of 240MPa (Theofanous et al., 2009a and 2009b).

Stainless steel is initially more expensive than carbon steel, but in comparison to carbon steel it has higher strength and stiffness, approximately twice the ductility, excellent corrosion resistance, and better retention of strength and stiffness at elevated temperatures. Eurocode 4 specifies design guidelines for stainless steel in Part 1.4 (vs. Part 1.1 for carbon steel) (Theofanous et al., 2009b). Studies have been conducted on empty SS-OHS and concrete filled SS-OHS (CF SS-OHS) and will be presented in this section.

2.5.1 UNFILLED

2.5.1.1 Columns

Theofanous et al. (2009b) studied the structural response of cold-formed SS-OHS. They performed laboratory tests, which included tensile material tests, compressive stub column tests and flexural buckling tests. They then performed FE analyses and validated their initial models against their experimental results. They were then able to perform parametric analyses on the FE models.

The tensile material tests were performed to determine the basic stress-strain response. The compressive stub column tests were performed to determine a suitable Class 3 limit for SS-OHS in compression. Tests were conducted on: OHS121 x 76 x 2, OHS121 x 76 x 3 and OHS86 x 58 x 3. Like CS-EHS, ultimate failure was typically due to local buckling of the cross-section. The flexural buckling tests were performed about both major and minor axes in order to determine ultimate load carrying capacity data to get suitable buckling curves for SS-OHS. Tests were performed on OHS86 x 58 x 3 pin-

ended columns with lengths ranging from 700mm to 3100mm, resulting in global non-dimensional slenderness values ranging from 0.34 to 2.07. All columns failed by global buckling.

FE models were created, and a linear-elastic buckling analysis was used to obtain buckling mode shapes. The Modified Riks method was used for non-linear analysis to obtain post-ultimate load response. Two sets of numerical analyses were performed: one using the tensile material properties and the second with the stub column material properties. When using stub column material properties, the results were more consistent and better mirrored the experimental overall lateral deflection response than compared to when tensile material properties were used.

From FE modelling, it was found that the lowest buckling mode determined from eigenvalue buckling analyses was similar to that of CS-EHS. Theofanous et al. (2009b) suggested using a similar slenderness parameter as CS-EHS, recall Equation (2.16), with a Class 3 limit of 90. The additions they proposed were that the proof stress of stainless steel and a secant elastic modulus be used. They further proposed using $D_{e,new}$ proposed by Ruiz-Teran and Gardner (2008), recall Equations (2.25) and (2.26), since it provided a better representation of the behaviour of SS-OHS over a range of aspect ratios.

FE models for flexural buckling were also created and validated against experimental results (Theofanous et al., 2009b). It was noted that buckling curves for CS products were derived based on the product's loss of stiffness after yielding, resulting in instability. Since SS stress-strain curves are rounded, strain hardening would occur instead. Nonetheless, the same provisions would still be suitable for SS with the addition of a strength reduction factor (χ), which is a function of the imperfection factor (α), limiting slenderness (λ_0) and member slenderness. (The EC3 column buckling resistance is given by $\chi A f_y$). It was assumed that $\alpha = 0.49$ and $\lambda_0 = 0.4$.

Comparing the codified buckling curves for SS-SHS and -RHS with experimental CHS and OHS, it was found that most points were under the buckling curve. This meant that the curves were unsafe for stainless steel CHS and OHS and need revision. This unsafe provision can be attributed to SHS and RHS having corners. The cold-worked corners have higher strength and offset the point of buckling. Theofanous et al. (2009b) suggested a derived buckling curve, which uses $\alpha = 0.49$ and $\lambda_0 = 0.49$. This curve was better as it both agreed with the experimental results and was safe.

2.5.1.2 Beams

Theofanous et al. (2009a) further investigated the flexural behaviour of SS-OHS and compared the results with existing results on CS-EHS and SS-CHS. Their experimental study included six 3-point bending tests on OHS121 x 76 x 2, OHS121 x 76 x 3 and OHS86 x 58 x 3 in major and minor axis bending. All specimens underwent ovalization. All specimens failed by inelastic local buckling in the

compression portion of the section at the region of maximum moment. There was a difference, however, between the failure of specimens under minor and major axis bending. For minor axis bending, local buckling occurred at the point of maximum compressive stress, which coincided with the point of $r_{\max}$ and the least stiff region. For major axis bending, local buckling occurred in the compression region of the section, but initiated closer to the neutral axis. The reason was because even though the compressive stress decreased towards the neutral axis, the stiffness also reduced.

Since for minor axis bending local buckling occurred at the $r_{\max}$ region, just like EHS stub columns under axial compression, it was proposed that the slenderness parameter for EHS in axial compression be adopted for minor axis bending, recall Equation (2.16). Since major axis bending initiated local buckling away from both the most extreme compression fibre and the neutral axis, the slenderness parameters proposed for CS-EHS can also be used for SS-OHS, recall Equation (2.32) (Theofanous et al., 2009a). FE models were created and validated against experimental results such that parametric analyses could be performed for a wider range of cross-sectional slenderness (ranging between 40 and 320 for both major and minor axis) and aspect ratios (equal to 1.5 or 2). If M_u is the ultimate moment capacity, and M_{el} is the elastic moment capacity, the FE models showed that by increasing cross-sectional slenderness, the M_u/M_{el} ratio decreased and, for a given slenderness, increasing the aspect ratio increased the M_u/M_{el} ratio. Overall, SS-OHS behaved similarly to CS-EHS (Theofanous et al., 2009a).

Theofanous et al. (2009a) proposed that the Class 3 limit originally proposed by Chan and Gardner (2008) for CS-CHS and EHS major and minor-axis bending, namely $140\epsilon^2$, can be used for SS-OHS undergoing bending. They also proposed that the Class 1 and 2 limits, $50\epsilon^2$ and $70\epsilon^2$ respectively, can be used as well. As for the actual bending capacity of the SS-OHS, it was suggested to use the continuous strength method (CSM) to account for strain hardening. The CSM is explained by Gardner (2008). The CSM led to an increase in efficiency of 15% and a decrease in the prediction scatter by 50%.

2.5.2 FILLED

Studies on CF SS-OHS have been conducted by Lam et al. (2008). As mentioned previously, SS has certain advantages over CS, and CF SS-OHS has the same advantages over CF CS-EHS. Lam et al. (2010) investigated the behaviour of axially loaded CF SS-OHS with normal and high strength concrete. All experimental results were based on stub column tests; therefore, the study was limited to comparing CF SS-OHS cross-sectional capacity against existing design guidance in Eurocode 4 for CF SS-SHS, CF SS-CHS and CF CS-EHS. In addition, the CSM was employed to determine the strength of the stainless steel as it more accurately and consistently predicted the axial capacity. The CSM does this because it more precisely predicts the stainless steel tube contribution to the composite resistance.

The specimens tested were: OHS121 x 76 x 2, OHS121 x 76 x 3 and OHS86 x 58 x 3; each section was tested three times, once unfilled, once with normal strength concrete and once with high strength concrete. Unfilled OHS typically failed by inward local buckling, while filled OHS typically failed by outward local buckling and the crushing of concrete. Overall, it was found that increasing the tube thickness increased the load-carrying capacity and ductility, and increasing the concrete strength increased load-carrying capacity, but lowered ductility. Since CF OHS are not currently covered by existing design codes, experimental results were compared against CF CHS equations from Eurocode 4, and Lam et al. (2008) propose design equations for using the CSM.

FE analyses were performed by Lam and Dai (2010) by validating the FE models with experimental results. Through parametric analyses, they found similar trends to CS CF EHS. They defined the following terms: “strength enhancement index”, which is the ratio of the maximum load to the summation of the strengths of the individual components, which measures the confinement effect; “ductility index”, which is the ratio of the column’s end shortening when the load is 85% of the maximum load to the end shortening at maximum load; “constraining factor”, which is the ratio of the yield stress of the stainless steel to the cracking strength of the unconfined concrete; “section strength factor”, which is the ratio of the average outer diameter to wall thickness; “section slenderness factor”, which is the ratio of the major to minor outer radius (i.e. the aspect ratio). Their observations were made related to these terms: increasing the concrete strength led to higher capacities, but it reduced both the ductility and strength enhancement indices. Increasing the section wall thickness led to higher capacities. Increasing the section constraining factor increased the ductility and strength enhancement indices. Increasing the section strength factor increased the ductility and strength enhancement indices as well. Finally, increasing the aspect ratio (transitioning from a CHS to an EHS) lowered the strength enhancement index, but resulted in no change in the ductility index.

2.6 EHS CONNECTIONS

Generally, many limit states design requirements and failure models are applicable to EHS connections, but EHS connections can be problematic. This is because there is an overall lack of data and knowledge about these types of connections and EHS welded connections are often complex due to the profiling required to make the connections (Packer, 2009a).

CIDECT (Comité International pour le Développement et l’Étude de la Construction Tubulaire) has published nine design guides for HSS connections in a series entitled: Construction with Hollow Steel Sections. These books contain the internationally recognized design equations for HSS. EHS have not been included in these design guidelines. Design Guide 1: for circular hollow section (CHS) joints under

predominantly static loading (Wardenier et al., 2008), and Design Guide 3: for rectangular hollow section (RHS) joints under predominantly static loading (Packer et al., 2009b), have recently been revised into second editions. These two design guides are relevant to EHS since, as discussed, an EHS behaves in-between that of a CHS and RHS.

In Section 1.3 of Design Guides 1 and 3 (Wardenier et al., 2008 and Packer et al., 2009b), it states: “The term ‘joint’ is used to represent the zone where two or more members are interconnected, whereas, ‘connection’ is used to represent the location at which two or more elements meet. The ‘through member’ of a joint is termed the ‘chord’ and attached members are termed ‘braces’ (although the latter are also often termed bracings, branch members or web members). Such terminology for joints, connections and braces follows Eurocode 3.”

Figure 2.13: Components of EHS-to-EHS connection

The definitions of joint and connection are sometimes interchanged: the term *joint* according to EC3 is referred to as *connection* in North America. In this thesis, the term connection is used. Figure 2.13 shows the typical make-up of an EHS-to-EHS connection, which includes a chord member and one or more braces. The values b , h and t are the width, height and thickness of the member, respectively, and the subscripts 0 and 1 represent the chord and the brace, respectively. The included angle between the brace and the chord is θ_1 and the brace load is N_1 . When $\theta_1 < 90^\circ$, the joint at the acute (small) angle is called the heel, and the joint at the obtuse (large) angle is called the toe.

2.6.1 K CONNECTIONS

EHS may be used for trusses, and this is what drives research to determine design requirements for truss connections. One example of EHS already being used in a truss-girder system can be found in the façade of the AXA building in Paris, which was analyzed by Bortolotti et al. (2003). One connection of this façade is a K connection, defined by one EHS480 x 240 x 12 chord, one EHS320 x 160 x 8 brace, and one CHS60.3 x 4 diagonal member in tension, see Figure 2.14. A K connection is defined as a connection in which one brace force is equilibrated (within 20%) by a brace on the same side of the chord (Packer et al., 2009b).

Figure 2.14: AXA Truss Connection
(Bortolotti et al., 2003)

Figure 2.15: EHS X Connection
(Pietrapertosa and Jaspart, 2003)

With limited information regarding the behaviour of the above connection, rules from EC3 were extrapolated, numerical simulations were conducted and experimental testing was performed. When extrapolating rules from EC3, the strength of equivalent RHS and CHS connections was determined and the lowest resistance calculated was used for the analyses. A note to the reader: these equivalent RHS and CHS approaches are different to those presented in this thesis. The case study conducted by Bortolotti et al. (2003) was inconclusive due to lack of experimental data.

2.6.2 X CONNECTIONS

Studies were conducted on X connections by Pietrapertosa and Jaspart (2003) to investigate behaviour and to develop a base to numerically model these types of connections (Bortolotti et al., 2003). An X connection is defined as a connection in which one brace force is equilibrated by another brace on the opposite side of the chord (Packer et al., 2009b). The schematics of their tests are roughly shown in

Figure 2.15 where two EHS120 x 60 x 4 braces were welded orthogonally to either an EHS320 x 160 x 8 chord or an EHS220 x 110 x 10 chord. The braces were subjected to either tension or compression.

In tension, Pietrapertosa and Jaspart (2003) found that yield lines developed on the chord and appeared to form a mechanism; however, on the load-displacement curves, there was no yield plateau to explain the mechanism. The chord did not even fail by yielding. What occurred was that as the rigidity of the connection decreased, the connection experienced second-order $P-\delta$ effects and membrane effects appeared. The membrane effects caused membrane stresses to appear parallel to the plane of the chord, which increased the resistance of the connection. The connection failed when the brace failed. In compression, the failure mode known as a "snap through" occurred where the brace penetrated into the chord.

The FE analyses showed that second-order effects had a major influence on tension-loaded connections and that membrane effects increased the resistance of the connection. In compression, however, the instability of the connection led to connection failure before the yield strength could be attained. The results from FE analyses showed that as the length of the chord increased, the stiffness and resistance of the connection also increased, up to a certain limit. The results also showed that when the ratio of brace width-to-chord width was low, punching shear would be the failure mode, but if that ratio was high, the connection would fail in tension by the brace yielding or in compression by the brace buckling.

Pietrapertosa and Jaspart (2003) proposed using the CHS design equations found in Packer and Henderson (1997) for the various failure mechanisms. They proposed that the strength of a connection for the chord yielding failure mode be multiplied by a factor to account for the second order effects that led to increased plastic strength in tension and decreased strength in compression. For tension, the terms should be multiplied by a factor greater than 1.0, and for compression, the term should be multiplied by a factor less than 1.0.

Further developments on these types of connections were conducted (Bortolotti et al., 2003) to develop analytical formulations of the connection's stiffness and resistance properties. The connection studied consisted of either an EHS320 x 160 x 8 chord or EHS220 x 110 x 8 mm with EHS120 x 60 x 4 braces welded 90° to the flatter sides (regions of r_{max}) of the EHS, see Figure 2.15. Two specimens of each type of assembly were tested by loading the braces in tension, then in compression. In tension, it was found that plastic hinges developed in the chord and reduced the connection stiffness. Following this, there was a progressive development of strain-hardening and "ovalization" of the chord. Finally, the connection failed via excessive plasticity in tension. In compression, plasticity formed and the connection stiffness reduced until the brace locally buckled or there was a "snap through" phenomenon. After that point, the resistance dropped, but increased once again due to membrane effects in the chord. Both tension and

compression scenarios showed excellent connection ductility and numerical models simulated the mode of failure and level of resistance in these simple brace-to-chord X connections.

Figure 2.16: EHS connection orientation types (reproduced from Choo et al., 2003)

Further FE analyses were conducted by Choo et al. (2003) to study the behaviour of EHS X connections in four different orientation types subjected to brace axial compression. These results were then compared with CHS X connections with equivalent connection areas. The four orientation types can be found in Figure 2.16. Type 1 connections have the minor axis of the chord parallel to the ground, and the braces are oriented such that their minor axes are parallel to the length of the chord. Type 2

connections have the minor axis of the chord parallel to the ground (as in Type 1), but the braces are oriented such that their minor axes are perpendicular to the length of the chord. Type 3 connections have the minor axis of the chord perpendicular to the ground, and the braces are oriented such that their minor axes are parallel to the length of the chord (as in Type 1). Type 4 connections have the minor axis of the chord perpendicular to the ground (as in Type 3), but the braces are oriented such that their minor axes are perpendicular to the length of the chord (as in Type 2). Each configuration behaves differently due to the varying effects of relative membrane action and plate bending action of the chord walls.

FE models of CHS connections were developed by Choo et al. (2003) for all four EHS orientations. They maintained the chord's larger dimension at 508mm and chord thicknesses were varied from 10, 12, 16 to 20mm. The β ratios tested were 0.25, 0.43, 0.64, 0.80 and 0.98, where β is the ratio of brace width (b_1) to chord width (b_0).

The results showed that Type 4 connections were highest in strength followed by 3, 2, and then 1. The strength of a comparable CHS X connection was in-between the strength of Type 2 and Type 3. There was also a direct correlation between connection stiffness and connection strength. Type 3 and Type 4 connections showed a peak load on the load-displacement plot, which was taken as the ultimate strength of the connection. However, Type 3 and Type 4 connections sometimes prematurely failed in the brace if β was small and the chord walls were thick.

Type 1 and Type 2 connections did not show this phenomenon, nor did either show a peak load in the load-displacement graph. Choo et al. (2003) used a deformation limit proposed by Lu and colleagues in 1994, otherwise known as the 3% deformation limit for these connections. The proposition was to take the ultimate strength of the connection when the connection deformation was equal to 3% of the chord width, and this 3% deformation limit governed the strengths of these two connections types.

To explain the difference in strengths, the varying effects of relative membrane action and plate bending action were investigated (Choo et al., 2003). Generally, membrane action can sustain relatively higher loads than plate bending action. In the EHS, relative membrane and plate bending actions were dependent on β , η (the ratio of brace height (h_1)-to-chord width (b_0)), and the orientation type. Type 3 and 4 connections demonstrated higher membrane action and this action was integral to the transfer of load from the brace to the chord. Types 1 and 2 demonstrated higher plate bending action which became the dominant action for load transfer at the crown and saddle points of the connection.

2.6.3 GUSSET PLATE END CONNECTIONS

Since a potential application for EHS is within truss systems or as bracing, one of the easier connection methods is an end connection using a gusset plate (Willibald et al., 2006a; Martinez-Saucedo

et al., 2008). Gusset plate-to-EHS end connections were compared with gusset plate-to-CHS end connections by studying the shear lag induced tensile fracture of the hollow section when the connection was loaded in full tension. The CHS connections were tested in three ways: a) sliding a gusset plate into a slotted CHS with no return weld, b) sliding a gusset plate into a slotted CHS with a return weld (such that the net area becomes equal to the gross area), and c) sliding a CHS into a slotted gusset plate. The EHS connections were also tested in three ways: a) sliding a gusset plate into a slotted EHS on the flatter side (Figure 2.17), b) sliding an EHS into a slotted gusset plate along the flatter side, and c) sliding a gusset plate into a slotted EHS along the small radii of curvatures.

In 1994, studies by Korol (Willibald et al., 2006a) determined two modes of failure for gusset plate-to-HSS end connections: 1) circumferential tensile fracture, and 2) block shear failure or tear-out. If the length of the connection (L_w) was larger than the distance between the welds (w), the connection was not affected by shear lag and the connection strength was equal to the tensile capacity of the section. L_w and w are shown in Figure 2.17. If L_w was less than $0.6w$, block shear failure became the critical failure mode; fracture started at the toe of the weld and carried to the end of the tube. For $0.6w \leq L_w \leq w$, the connection was influenced by shear lag and either failure mode, even a mixture of both failure modes, was possible. The relative thickness (D/t) was also influential in some regards, as increasing this value had a negative effect on the connection.

Figure 2.17: Gusset plate-to-EHS end connection (reproduced from Martinez-Saucedo et al., 2008)

The results from the EHS end connection tests (Willibald et al., 2006a; Martinez-Saucedo et al., 2008) showed that specimens generally supported Korol's findings: failure modes were circumferential fracture or block shear tear out. They also found that all failure was accompanied by ovalization, which is an extreme deformation prior to failure, and even the bowing of the gusset plate. Specimens with slotted

gusset plates ovalized more due to the less stiff connection profile, resulting in decreased the connection strength. They also showed that L_w was the most influential factor to determine connection strength, or alternately the eccentricity of the connection. The only exception to the failure modes described was when the connection was loaded in compression, and then local buckling occurred near the connection end.

Overall, the American Institute of Steel Construction (AISC) and the Canadian Standards Association (CSA) codes did a conservative job defining connection capacity (Willibald et al., 2006b). Martinez-Saucedo et al. (2008) developed more appropriate ultimate limit state capacity formulae for these connections types.

2.6.4 BRANCH AND THROUGH PLATE CONNECTIONS

Studies were conducted to determine if current design guidelines from CIDECT for branch and through plate connections to RHS, and for branch connections to CHS, could be applied to branch and through plate connections to EHS (Willibald et al., 2006b). Plate connections are often favoured since they are easily fabricated and handled, but they are more often used with W-sections where the force can easily transmit to the web of the W-section. The same force transmission does not occur for HSS. Generally, branch plate connections to the face of an HSS have low resistance since the connection has a high flexibility. Slotting the HSS to make a through plate connection is typically stronger.

Figure 2.18: Branch and through plate-to-EHS connections (Willibald et al., 2006b)

Both longitudinal plate orientation and transverse plate orientation were tested. Transverse branch plate-to-RHS connections showed four basic failure modes: chord face plastification, punching shear, chord side wall failure, and branch yielding (Willibald et al., 2006b). Longitudinal plate-to-RHS connections fail by chord face plastification, and branch plates to CHS fail by punching shear or plastification failure modes (Wardenier et al., 2008; Packer et al., 2009b).

Experiments were conducted on 6 EHS220 x 110 x 6.3 T connections consisting of branch and through plates that were welded orthogonally to the section and loaded in axial tension. Figure 2.18 shows the six plate-to-EHS orientations that were investigated. These six EHS connection tests were

inconclusive with regard to attempts to apply equivalent CHS or equivalent RHS techniques to establish the EHS connection capacity.

3.0 PRELIMINARY WORK

Since EHS have not been mentioned anywhere in Canadian codes or guides, in addition to studying the behaviour of EHS-to-EHS welded connections, preliminary work has been performed to address this concern. This preliminary work includes the development of EHS dimension and geometric property tables and compressive resistance tables for a future edition of the CISC Handbook of Steel Construction. Additionally, the establishment of design rules for EHS members based on an equivalent CHS diameter approach has been explored in the literature, but the design of EHS members based on an equivalent RHS approach has only been investigated minimally. This section will examine the validity of an equivalent RHS approach to design for EHS members in axial compression or bending.

3.1 EHS DIMENSION AND GROSS PROPERTY TABLE

The EHS property equations that have been defined by EN10210 (CEN, 2006a and 2006b) and found in Table 2.1 of this thesis have been used to create a section property table in accordance with the CISC Handbook of Steel Construction format. This newly created table can be found in Appendix 3A.

The table includes designation, design wall thickness, mass, dead load, area, torsional inertia constant, torsional modulus constant and surface area. The first moment of inertia, elastic section modulus, radius of gyration, and plastic section modulus about both primary axes have also been calculated. These properties are later used to create compression resistance tables.

3.1.1 WARPING, SHEAR, AND TORSIONAL CONSTANTS

Existing dimensional and geometric properties currently listed in the Canadian Handbook (CISC, 2010) include three constants: warping constant (C_w), shear constant (C_{τ}) and torsional inertia constant (J). Both Corus (2005) and EN10210 (CEN 2006a and 2006b), however, mention a different constant called the torsional modulus constant (C_t).

The warping constant, C_w , measures the resistance of members to non-uniform or warping torsion. It is used to calculate the buckling moment resistance of laterally unsupported beams and torsional-flexural buckling of compression members. For open sections, C_w is listed; however, for closed sections, such as HSS, the warping deformation is very small and negligible, thus C_w can safely be assumed to be zero (CISC, 2002). Since EHS are closed sections, C_w is assumed to be negligible and equal to zero.

The shear constant, C_{τ} is used to determine the maximum shear stress in a cross-section due to the applied load (CISC, 2002). For HSS, the maximum shear stress ($\tau_{\max}$) is shown in Equation (3.1), where

V is the applied shear force, Q is the statical moment of area, t is the wall thickness and I is the moment of inertia. The shear constant is simply the ratio of the applied shear force to the maximum permissible shear stress (CISC, 2002), as shown in Equation (3.2). The European EN10210 does not list this constant for EHS (CEN, 2006b). For the time being, the tables created herein for the CISC Handbook do not include this constant either.

$$\tau_{max} = \frac{VQ}{2tI} \quad (3.1)$$

$$C_{rt} = \frac{V}{\tau_{max}} = \frac{2tI}{Q} \quad (3.2)$$

The torsional inertia constant, J, is otherwise known as the St. Venant Torsional Constant. This constant measures the resistance of the structural member to pure or uniform torsion. It is also used to calculate the buckling moment resistance of laterally unsupported beams and torsional flexural buckling of compression members (CISC, 2002). When torque is applied to a member, and both ends are restrained from warping and member buckling is avoided, the member experiences a torsional twist (θ). The constant J can be calculated according to Equation (3.3), where T is the applied torque, L is the member length, and G is the shear modulus. The torsional modulus constant (C_t) is the ratio of applied torque (T) to the St. Venant shear stress at the external surface (τ_0) (Ridley-Ellis et al., 2002). As such, J and C_t can be seen to be related.

$$J = \frac{TL}{G\theta} \quad (3.3)$$

$$C_t = \frac{T}{\tau_0} \quad (3.4)$$

Both the torsional inertia constant (J) and the torsional modulus constant (C_t) can be defined in two different ways according to: (i) the thin-walled theory formulated by Bredt in 1896, and (ii) the thick-walled theory formulated by Marshall in 1971 (Ridley-Ellis et al., 2002).

The Thin-Walled Theory is good for tubes with very slender walls because it assumes that shear stress is uniform through its thickness (Ridley-Ellis et al., 2002). Equations (3.5) and (3.6) define J and C_t respectively, for a RHS according to this theory. The term h_c is the effective height, b is the width, h is the height, t is the thickness, r_o is the outer corner radius, r_i is the inner corner radius, and A_h is the enclosed area of the section. For RHS, the CISC uses Bredt's thin-wall theory equation for J with $r_o = 2t$ and $r_i = t$.

$$J_{Bredt} = \frac{4tA_h^2}{h_c} \quad (3.5)$$

$$C_{t,Bredt} = 2tA_h \quad (3.6)$$

$$h_c = 2[(b - t) + (h - t)] - (4 - \pi)(r_o + r_i) \quad (3.7)$$

$$A_h = (b - t)(h - t) - \frac{(4 - \pi)(r_o + r_i)^2}{4} \quad (3.8)$$

The Thick-Walled Theory accounts for the variation of the shear stress through the thickness; that is, the torsional shear stress decreases through the wall thickness. It ignores the corner radii as a simplification. Equations (3.9) and (3.10) define J and C_t respectively, for a RHS according to this theory (Ridley-Ellis et al., 2002), where h_c and A_h are the same as above, see Equations (3.7) and (3.8).

$$J_{Marshall} = \frac{4tA_h^2}{h_c} + \frac{t^3h_c}{3} \quad (3.9)$$

$$C_{t,Marshall} = \frac{h_c J}{th_c + 2A_h} \quad (3.10)$$

Marshall's thick-wall theory equations for J and C_t appear to have been used to derive the EHS constants stated by Corus (2005) and EN10210 (CEN, 2006b). It appears that the CISC (CISC, 2010) uses the thin-walled theory equations and the European design standards use the thick-walled theory equations (CEN, 2006b). Herein, the European-approved approach, namely Equations (3.9) and (3.10), is used to generate the properties in Appendix 3A. Ridley-Ellis et al. (2002) have written that the thick-walled theory is a better representation of the torsional behaviour of RHS.

3.1.2 PROPERTIES ABOUT THE AXES

Figure 3.1: International vs. Canadian convention for sectional axes

Apparent in literature and international standards is the convention to use the y-axis and z-axis as the principal axes of a shape's cross-section, so that the x-direction may be used to define the length of a beam in bending. This modern approach is suited to using the x-y axes for planar frame layout in a computer frame analysis program. The Canadian convention, however, uses the x-axis and y-axis as the principal axes of a shape's cross-section, such that the z-direction defines the length of a member. See

Figure 3.1 to see this difference in conventions. This thesis, including the newly developed tables, adopts the Canadian convention.

3.2 COMPRESSION RESISTANCE TABLES

The Steel Construction Institute and the British Constructional Steelwork Association became the first to adopt EHS design rules using an equivalent diameter approach (SCI/BSCA, 2008). Within BS5950-1, the slenderness parameter of an EHS was determined by finding an equivalent CHS diameter using the more conservative method of $D_e = 2a^2/b$, whereby design rules for a CHS would then be used.

In an attempt to establish preliminary compressive resistance design tables for Section 4 of the CISC Handbook of Steel Construction, an equivalent CHS diameter approach was also used, but using the less conservative equivalent diameter $D_e = 2a[1 + f(a/b - 1)] = D_{e,new}$ (Ruiz-Teran and Gardner, 2008). Once the equivalent diameter was established, the Canadian slenderness parameter (D_e/t) was checked with the CHS cross-sectional slenderness limit, Table 1 from S16 (CSA, 2009). If the slenderness parameter $> 23,000/f_y$, where f_y is taken as 350MPa, then the section was Class 4; otherwise, the section was not considered Class 4.

The compressive resistance tables consist of two parts: 1) properties and design data and 2) compressive resistances for varying lengths. For the properties and design data part of the tables, the moment resistance (M_r) about each axis was determined using Clause 13.5 of S16 (CSA, 2009). The D_e/t slenderness parameter was checked with CHS cross-sectional slenderness limits for elements in flexural compression, Table 2 from S16 (CSA, 2009). If the section was Class 1 or 2, $M_r = \phi Z f_y$, and if the section was Class 3, $\phi S f_y$, where $\phi = 0.9$. None of the sections were considered to be Class 4 for flexural compression.

For the compressive resistances part of the table, the compressive resistance (C_r) of a column was computed using Clause 13.3 of S16 (CSA, 2009), which is reproduced as Equation (3.11). The resistance factor $\phi = 0.9$, A is the area and can be taken from the geometric properties table developed in Section 3.1, f_y is the yield stress = 350MPa, and n is taken as 2.24 for hot-formed sections. The non-dimensional slenderness ratio (λ) from Clause 13.3 of S16 (CSA, 2009), is reproduced as Equation (3.12). The effective length factor $k = 1.0$, L is the length of the member, r is the minimum EHS radius of gyration of the section taken from the geometric properties table developed in Section 3.1, and $E = 200,000\text{MPa}$.

$$C_r = \phi A f_y (1 + \lambda^{2n})^{-1/n} \quad (3.11)$$

$$\lambda = \frac{kL}{r} \sqrt{\frac{f_y}{\pi^2 E}} \quad (3.12)$$

For non-Class 4 sections, all the values can be substituted into the above equations in a straightforward manner, but if the section was Class 4, the $f_{y,eff}$ was computed and substituted into the same equations. The computed C_r values were put into the tables providing the section and length passed the global slenderness check given by Clause 10.4.2.1 of S16 (CSA, 2009); that is, if $kL/r \leq 200$. The compressive resistance tables can be found in Appendix 3B.

3.3 EQUIVALENT RHS APPROACH

Various equivalent CHS approaches have dominated literature thus far and have therefore predominantly been used to establish design rules, but there are some shortcomings to these methods. In 2009, Zhao and Packer proposed converting EHS into equivalent RHS in an attempt to combat some of these shortcomings. Their method, herein called the equivalent RHS approach, is investigated here against existing experimental results on columns and beams.

3.3.1 COLUMNS

The equivalent RHS approach, as originally proposed by Zhao and Packer (2009) is simplified for this thesis. Zhao and Packer (2009) had accounted for the corner radii of the RHS, but real or assumed corner radii vary depending on the national or regional design specifications and manufacturing practices. This thesis simplifies the approach by neglecting the RHS corner radii and treating the RHS as a “box section”. In this chapter, this simplified equivalent RHS method is called **Method 1**. By maintaining the same cross-sectional area (A_{EHS}) and thickness (t) as the original EHS, the equivalent large and small dimensions of the equivalent RHS (H_{eq} and B_{eq} , respectively) using Method 1 can be calculated according to Equation (3.13), where H is the large dimension of the EHS.

$$\begin{cases} H_{eq} = H \\ B_{eq} = \frac{A_{EHS} - 2Ht + 4t^2}{2t} \end{cases} \quad (3.13)$$

To determine the validity of this equivalent RHS approach for columns, predictions using Method 1 are compared against the experimental results presented by Chan and Gardner (2009). They performed 24 EHS column tests; 12 columns buckled about the major axis and 12 columns buckled about the minor axis. The column capacities have been predicted using the equivalent RHS approach in conjunction with the column resistance equations from S16 (CSA, 2009), shown in Equations (3.11) and (3.12), with $\phi = 1.0$ and $k = 1.0$. Experimentally determined column lengths (L) and material properties ($f_{y,avg}$ and E_{avg}) are used as well.

3.3.1.1 Major axis buckling

Looking initially at columns buckling about the major axis, the first step is to convert the EHS into an equivalent RHS with Equation (3.13). The RHS cross-sectional classification limits for elements in axial compression, found in S16 (CSA, 2009), are used to determine if the section is or is not Class 4. The RHS area is calculated from the EHS area using the area equation found in Table 2.1 of this thesis, using measured dimensions. However, if the section is Class 4, either an effective area could be determined by cutting out ineffective portions using the Class limit $= 670/\sqrt{f_y}$, or an effective yield stress could be calculated. Using an effective yield stress is more conservative than cutting out ineffective portions since an effective yield stress would sacrifice the strength of the entire section; as such, the effective area approach is recommended. A note to the reader: none of the experimentally tested sections are found to be Class 4; therefore, a description of how to deal with Class 4 sections is for theoretical discussion only.

Table 3.1: Predicted capacity of columns buckling about the major axis using the equivalent RHS approach - Method 1 (experimental results taken from Chan and Gardner (2009))

Column							Class		r_x		Actual Predicted	
	P_u	L	$f_{y,avg}$	E_{avg}	H_{eq}	B_{eq}	H_{eq}	B_{eq}	(a)	(b)	(a)	(b)
	kN	mm	MPa	MPa	mm	mm			mm	mm		
150x75x4-c2	573	700	376.5	217,550	150.54	28.73	1,2,3	1,2,3	47.1	48.0	1.051	1.051
150x75x5-c2	677	700	369.0	217,050	150.21	29.37	1,2,3	1,2,3	46.7	47.6	1.060	1.060
150x75x6.3-c2	866	700	390.5	216,450	150.28	29.80	1,2,3	1,2,3	46.3	47.1	1.044	1.044
150x75x4-c4	538	1500	376.5	217,550	150.05	28.93	1,2,3	1,2,3	46.9	47.8	0.993	0.992
150x75x5-c4	680	1500	369.0	217,050	150.10	29.73	1,2,3	1,2,3	46.7	47.6	1.075	1.074
150x75x6.3-c4	836	1500	390.5	216,450	148.47	30.47	1,2,3	1,2,3	45.8	46.7	1.029	1.028
150x75x4-c6	489	2300	376.5	217,550	150.34	28.79	1,2,3	1,2,3	47.0	47.9	0.968	0.963
150x75x5-c6	611	2300	369.0	217,050	150.05	29.31	1,2,3	1,2,3	46.6	47.5	1.020	1.016
150x75x6.3-c6	814	2300	390.5	216,450	148.77	30.26	1,2,3	1,2,3	45.9	46.8	1.082	1.076
150x75x4-c8	429	3100	376.5	217,550	150.46	28.75	1,2,3	1,2,3	47.1	48.0	0.968	0.956
150x75x5-c8	509	3100	369.0	217,050	150.03	29.44	1,2,3	1,2,3	46.6	47.5	0.964	0.952
150x75x6.3-c8	648	3100	390.5	216,450	148.60	30.40	1,2,3	1,2,3	45.9	46.8	1.007	0.992
(a) Method 1a: using original EHS properties									Mean	1.022	1.017	
(b) Method 1b: using equivalent RHS properties									St.Dev.	0.042	0.045	
									COV	0.041	0.045	

The second step is to calculate the non-dimensional slenderness parameter (λ). The gross radius of gyration (r) is used regardless of whether the section is, or is not, Class 4. The question becomes whether to use the r of the original EHS, herein called Method 1a, or the r of the equivalent RHS, herein called Method 1b. In **Method 1a**, the radius of gyration is calculated using the equations found in Table 2.1. Using the original EHS properties may be more intuitive at first, but if the section is Class 4, then the procedure would use an effective RHS area with the gross EHS radius of gyration. The procedure would seem inconsistent. In **Method 1b**, the moment of inertia (I) of the equivalent RHS is calculated, and the

radius of gyration is calculated by: $r = \sqrt{I/A}$. Using the equivalent RHS property means that if the section was Class 4, both area and radius of gyration would be based on the equivalent RHS. This is more consistent.

The third step is to predict the column capacity. The prediction results for columns buckling about the major axis are shown in Table 3.1. The table includes the experimental ultimate load (P_u), length of column (L), average material properties ($f_{y,avg}$ and E_{avg}), equivalent RHS dimensions, Class of equivalent RHS elements, calculated radius of gyration about the major axis (r_x), and the actual-to-predicted capacity ratios. On average, the equivalent RHS approach predicts column capacities quite accurately. There is a very small difference between using EHS properties or the equivalent RHS properties. The mean and COV of the actual-to-predicted capacity ratios are 1.022 and 0.041, respectively, when the r_x of the original EHS are used; the mean and COV of the actual-to-predicted capacity ratios are 1.017 and 0.045, respectively, using the r_x of the equivalent RHS. Based on the available data, either method would be appropriate to use for design purposes. To reiterate: none of the sections are considered Class 4. It is recommended for future work that more slender cross-sections be tested in order to determine the validity of the equivalent RHS approach with Class 4 sections.

3.3.1.2 Minor Axis Buckling

For columns buckling about the minor axis, the procedure outlined above is repeated. As before, the procedures are called Method 1a and Method 1b. Supplementary to this, a modification to the equivalent RHS approach is proposed and is called **Method 2**. In Method 1, the large dimension of the EHS is maintained regardless of buckling direction. Method 2 accounts for buckling about the minor axis. For minor axis buckling, the small dimension of the EHS is set equal to one equivalent RHS dimension (B_{eq}), and the second equivalent RHS dimension (H_{eq}) is calculated by assuming the area and thickness of the original EHS remains constant, as shown in Equation (3.14).

$$\begin{cases} B_{eq} = B \\ H_{eq} = \frac{A_{EHS} - 2Bt + 4t^2}{2t} \end{cases} \quad (3.14)$$

Like Method 1, Method 2 can be broken down into Method 2a and Method 2b. **Method 2a** uses the original EHS properties and **Method 2b** uses the equivalent RHS properties. Table 3.2 summarizes the equivalent RHS methods described in this chapter, and Figure 3.2 shows the EHS to equivalent RHS conversions. For major axis buckling, Method 1 and Method 2 are equivalent procedures; thus, the predictions using the equivalent RHS approach do not change. A note to the reader: by using the modified equivalent RHS Method 2, none of the sections tested were found to be Class 4.

Figure 3.2: Equivalent RHS using the simple and modified equivalent RHS approach

Table 3.2: Equivalent RHS approaches - Method 1 vs. Method 2 and (a) vs. (b)

Method 1	Large EHS dimension always maintained, neglecting corner radii. See Equation (3.13) and Figure 3.2 (left) (a) Using original EHS properties (b) Using equivalent RHS properties
Method 2	Accounting for buckling about the minor axis For major axis buckling, see Equation (3.13) and Figure 3.2 (left) For minor axis buckling, see Equation (3.14) and Figure 3.2 (right) (a) Using original EHS properties (b) Using equivalent RHS properties

Table 3.3: Predicted capacity of columns buckling about the minor axis using the equivalent RHS approach - Methods 1 and 2 (experimental results taken from Chan and Gardner (2009))

Column			Method 1		Method 2		r _y			Actual Predicted		
	P _u	L	H _{eq}	B _{eq}	H _{eq}	B _{eq}	1a/2a	1b	2b	1a/2a	1b	2b
	kN	mm	mm	mm	mm	mm	mm	mm	mm			
150x75x4-c1	495	700	150.56	28.75	103.83	75.48	27.3	11.8	30.4	0.932	1.059	0.930
150x75x5-c1	614	700	150.08	29.68	103.76	76.00	27.2	11.8	30.2	0.960	1.086	0.959
150x75x6.3-c1	820	700	150.37	29.52	104.64	75.25	26.5	11.3	29.5	1.005	1.184	1.003
150x75x4-c3	507	1500	150.44	28.84	103.75	75.53	27.3	11.8	30.3	1.032	2.773	0.998
150x75x5-c3	647	1500	150.31	29.25	104.08	75.48	27.0	11.6	30.0	1.101	3.006	1.064
150x75x6.3-c3	789	1500	148.36	30.26	103.00	75.62	26.6	11.6	29.5	1.084	3.071	1.042
150x75x4-c5	365	2300	150.26	28.78	103.64	75.40	27.3	11.8	30.3	1.038	4.545	0.923
150x75x5-c5	393	2300	150.11	29.20	103.91	75.40	27.0	11.6	29.9	0.952	4.208	0.845
150x75x6.3-c5	452	2300	148.82	30.40	103.30	75.92	26.7	11.6	29.7	0.895	3.954	0.788
150x75x4-c7	234	3100	150.50	28.77	103.82	75.45	27.3	11.8	30.3	1.042	5.266	0.876
150x75x5-c7	242	3100	149.93	29.53	103.67	75.79	27.1	11.7	30.1	0.917	4.580	0.771
150x75x6.3-c7	292	3100	148.77	30.34	103.26	75.85	26.7	11.6	29.6	0.931	4.657	0.778
1a/2a: Using original EHS properties									Mean	0.991	3.282	0.915
1b: Using equivalent RHS properties with Method 1									St.Dev.	0.069	1.506	0.103
2b: Using equivalent RHS properties with Method 2									COV	0.069	0.459	0.113

The prediction results using both Methods 1 and 2 for columns buckling about the minor axis are shown in Table 3.3. The table includes the experimental ultimate load (P_u), measured effective length (L), equivalent RHS dimensions, calculated radius of gyration about the minor axis (r_y), and the actual-to-predicted capacity ratios. Material properties are the same as found in Table 3.1 for each of the respective sections. Since none of the sections are considered Class 4, Method 1a and Method 2a become equivalent procedures because both methods use the r_y of the original EHS.

Using the r_y of the EHS (Method 1a or 2a) results in a mean and COV of 0.991 and 0.069, respectively. Using Method 1b results in a mean and COV of 3.282 and 0.459, respectively. For short columns ($L = 700\text{mm}$), the predictions are good, but as the length of the column increases, the predictions become much too conservative. This conservatism occurs because the r_y of the equivalent RHS using Method 1 is significantly lower than the r_y of the original EHS. The global slenderness parameter therefore plays a larger detrimental effect in the predictions. This conservatism is solved by using Method 2; the r_y is much closer to the actual EHS radius of gyration. Using Method 2b results in a mean and COV of 0.915 and 0.113, respectively. While on average, the predictions are good for Method 2b, the predictions are unsafe for longer columns. The actual-to-predicted capacity ratios reached as low as 0.771 for one of the 3100mm long columns.

Based on the available data, it is shown that Method 2a can be used to predict column capacities for both major and minor axis buckling. Method 2a would be preferred over Method 2b because Method 2b over-predicts the capacities of longer columns buckling about the minor axis. Additional experimental

data is required on more slender cross-sections to determine the actual validity of these prediction methods when dealing with Class 4 sections.

3.3.2 BEAMS

Zhao and Packer (2009) have shown that the classification limits for RHS undergoing axial compression can be used for EHS columns when the EHS is converted into an equivalent RHS. Similarly, a paper to which this author contributed (Zhao et al., 2010), showed that the classification limits for RHS undergoing flexural compression can be used for EHS beams when a simple and modified equivalent RHS approach is used that accounts for minor axis bending, see Method 2 above. In this subsection, the author's contribution to that article is shown. The contribution included the prediction of flexural capacity of experimentally tested beams performed by Gardner and Chan (2008) using the equivalent RHS approach, Method 2. Gardner and Chan's (2008) experiments included beams bending about both the major axis and minor axis. Flexural resistance design equations from S16 (CSA, 2009) are used with $\phi = 1.0$. Experimentally determined material properties ($f_{y,avg}$ and E_{avg}) are used as well.

The first step is to convert the EHS beam cross-section into an equivalent RHS. Equation (3.13) is used for beams bending about the major axis and Equation (3.14) is used for beams bending about the minor axis. See Figure 3.2 for an illustration of the two schemes. It was found that the over-conservatism of Method 1 for columns buckling about the minor axis was also apparent for beams bending about the minor axis; thus, only Method 2 is shown for beams. Using the S16 RHS Class limits for the equivalent RHS, the Class of the flanges and webs was determined. (The Class limits are reproduced in Table 3.7).

The second step is to determine the elastic or plastic section modulus (S or Z , respectively). If the section is Class 1 or Class 2, Z is used and the predicted moment capacity and $M_{pred} = Zf_y$. If the section is Class 3, S is used and $M_{pred} = Sf_y$. If the section is Class 4, an effective elastic section modulus (S_{eff}) or effective yield stress ($f_{y,eff}$) needs to be calculated, so $M_{pred} = S_{eff}f_y$ or $= Sf_{y,eff}$. Like the columns, the question becomes whether to use the original EHS properties using the equations found in Table 2.1 (Method 2a), or to use the equivalent RHS properties (Method 2b). For Method 2a, Class 4 sections can be dealt with in one of two ways: i) use S_{eff} of the equivalent RHS with f_y or ii) use an $f_{y,eff}$ with the S of the EHS. For Method 2b, Class 4 sections are dealt with by finding the S_{eff} of the equivalent RHS with f_y . The equivalent RHS properties are listed in Table 3.4 for both major and minor axis bending. B_{eff} and H_{eff} are the effective flange width and web depth, respectively, if ineffective portions are cut-out for Class 4 sections. Unlike the column data, some experimental beam sections are considered Class 4; the viability of the equivalent RHS method on Class 4 beam sections can therefore be investigated.

Table 3.4: Equivalent RHS properties

Property	Major Axis Bending	Minor Axis Bending
Flange Width	$B_{eq} = \frac{A_{EHS} - 2Ht + 4t^2}{2t}$	$H_{eq} = \frac{A_{EHS} - 2Bt + 4t^2}{2t}$
Web Depth	$H_{eq} = H$	$B_{eq} = B$
Moment of Inertia: I (mm ⁴)	$= \frac{1}{12}(B_{eq}H^3 - (B_{eq} - 2t)(H - 2t)^3)$	$= \frac{1}{12}(H_{eq}B^3 - (H_{eq} - 2t)(B - 2t)^3)$
Plastic Section Modulus: Z (mm ³)	$= \frac{1}{4}(B_{eq}H^2 - (B_{eq} - 2t)(H - 2t)^2)$	$= \frac{1}{4}(H_{eq}B^2 - (H_{eq} - 2t)(B - 2t)^2)$
Elastic Section Modulus: S (mm ³)	$= \frac{2I}{H}$	$= \frac{2I}{B}$
Effective Moment of Inertia: I _{eff} (mm ⁴)	$= I - \frac{1}{6}(B_{eq} - 2t - B_{eff})t^3$ $- \frac{1}{2}t(B_{eq} - 2t - B_{eff})(H - t)^2$ $- \frac{1}{6}t(H - 2t - H_{eff})^3$	$= I - \frac{1}{6}(H_{eq} - 2t - H_{eff})t^3$ $- \frac{1}{2}t(H_{eq} - 2t - H_{eff})(B - t)^2$ $- \frac{1}{6}t(B - 2t - B_{eff})^3$
Effective Elastic Section Modulus: S _{eff} (mm ³)	$= \frac{2I_{eff}}{H}$	$= \frac{2I_{eff}}{B}$

The major axis bending predictions are shown in Table 3.5. Sections are considered to be Class 1 or 2 according to equivalent RHS Method 2 with the S16 (CSA, 2009) RHS Class limits; therefore, only the plastic section modulus is required. The table includes the EHS beam section, whether the beam was tested in 3- or 4-point bending, the experimental ultimate moment capacity (M_u), the EHS large dimension (H) and small dimension (B), the average thickness (t), average yield stress ($f_{y,avg}$), the flange and web Class using the RHS Class limits, the predicted moment capacities (M_{pred}), and the actual-to-predicted capacity ratio. There is little difference between using either the original EHS properties or the equivalent RHS properties; the actual-to-predicted capacity ratios result in a mean and COV of 1.281 and 0.156, respectively, using EHS properties, and 1.262 and 0.158, respectively, using the equivalent RHS properties. By using either Method 2a or 2b, predictions are better for 4-point bending in comparison to

3-point bending. Overall, the prediction methods for EHS undergoing major axis bending are slightly conservative regardless of whether original EHS or equivalent RHS properties are used.

Table 3.5: Predicted capacity of beams bending about the major axis using the equivalent RHS approach - Methods 2a and 2b (experimental results taken from Chan and Gardner (2008))

							Class		M _{pred}		<u>Actual</u> Predicted	
Beam	#- Point Bend- ing	M _u	H	B	t	f _{y,avg}	Flange	Web	(a)	(b)	(a)	(b)
		kNm	mm	mm	mm	MPa			kNm	kNm		
400x200x12.5	4	548	401.34	200.77	12.13	395.0	1	1	477	484	1.150	1.133
400x200x14.0	4	659	399.90	201.06	14.48	397.5	1	1	561	570	1.174	1.157
500x250x8.0	4	497	491.74	260.92	7.78	413.0	2	1	513	524	0.968	0.948
400x200x10.0	4	485	399.02	202.06	9.57	401.0	1	1	386	392	1.257	1.236
400x200x12.5	3	681	403.08	198.53	11.94	395.0	1	1	470	477	1.448	1.429
400x200x14.0	3	808	400.90	197.82	14.34	397.5	1	1	554	561	1.459	1.440
400x200x16.0	3	862	403.53	200.72	15.33	378.5	1	1	570	578	1.511	1.491
(a) Using original EHS properties										Mean	1.281	1.262
(b) Using equivalent RHS properties										St. Dev.	0.200	0.199
										COV	0.156	0.158

Table 3.6: Predicted capacity of beams bending about the minor axis using the equivalent RHS approach - Methods 2a and 2b (experimental results taken from Chan and Gardner (2008))

							Class		M _{pred}		<u>Actual</u> Predicted	
Beam	#- Point Bend- ing	M _u	H	B	t	f _{y,avg}	Flange	Web	(a)	(b)	(a)	(b)
		kNm	mm	mm	mm	MPa			kNm	kNm		
400x200x8.0	4	186	396.09	207.63	7.75	429.0	4	1	167	202	1.116 0.921*	0.921
400x200x10.0	4	232	396.06	207.54	9.65	401.0	2	1	248	276	0.936	0.841
400x200x12.5	4	288	401.54	201.01	12.13	395.0	1	1	292	326	0.986	0.884
400x200x14.0	4	343	400.32	200.04	14.48	397.5	1	1	341	381	1.005	0.901
400x200x16.0a	4	331	403.16	201.08	15.63	378.5	1	1	352	393	0.941	0.843
400x200x16.0b	4	346	403.28	202.15	15.63	378.5	1	1	354	395	0.977	0.875
500x200x8.0	4	291	495.34	255.85	7.78	413.0	4	1	165	261	1.764 1.116*	1.116
400x200x10.0	3	245	400.52	199.40	9.61	401.0	3	1	187	230	1.313	1.066
400x200x12.5	3	330	402.24	200.10	11.98	395.0	2	1	288	321	1.147	1.028
400x200x14.0	3	388	400.50	199.21	14.34	397.5	1	1	337	376	1.153	1.032
400x200x 16.0	3	401	403.35	199.49	15.27	378.5	1	1	341	381	1.175	1.052
(a) Using original EHS properties and effective yield stress;										Mean	1.138	0.960
*using effective section modulus of equivalent RHS										St. Dev.	0.239	0.100
(b) Using equivalent RHS properties										COV	0.210	0.104
										Mean*	1.061*	
										St. Dev.*	0.127*	
										COV*	0.120*	

The minor axis bending results are shown in Table 3.6. Most sections are either Class 1 or 2, one section was Class 3, and two sections were considered Class 4. The table includes the same headings as Table 3.5. Using the original EHS properties with an effective yield stress for Class 4 sections, the mean and COV are 1.138 and 0.210, respectively. Using the original EHS properties with an effective elastic section modulus of an equivalent RHS for Class 4 sections, the mean and COV are 1.061 and 0.120, respectively. By using the equivalent RHS properties, the mean and COV are 0.960 and 0.104, respectively. Overall, the prediction methods are good for minor axis bending.

In addition to comparing the actual-to-predicted moment capacity ratios when using the original EHS properties versus using the equivalent RHS properties, the difference between using an effective yield stress and the equivalent RHS effective elastic section modulus was compared. On average, the use of the equivalent RHS S_{eff} instead of $f_{y,eff}$ improves predictions. EHS400 x 200 x 8.0 under 4-point bending was considered a Class 4 section. By using the S_{eff} of an equivalent RHS instead of $f_{y,eff}$, the actual-to-predicted capacity ratio went from within 12% to within 8%. Likewise, EHS500 x 250 x 8.0 under 4-point bending was considered a Class 4 section. In that case, the actual-to-predicted capacity ratio went from a 77% disparity to within 12% by using the equivalent RHS S_{eff} . For sections not Class 4 that underwent 4-point bending, predictions using EHS properties were within 8% of maximum load, versus 16% using the equivalent RHS properties. However, for sections that underwent 3-point bending, predictions using equivalent RHS properties were within 7%, versus 32% using the original EHS properties.

Based on the available data, Method 2a can be used to predict moment capacities for both major and minor axis bending. To deal with Class 4 sections, the S_{eff} of an equivalent RHS can be used. Method 2a would be preferred over Method 2b because Method 2b over-predicts the capacities of 4-point tested beams while Method 2a is generally safer. Additional experimental data is required on more slender cross-sections to determine the actual validity of these prediction methods when dealing with Class 4 sections.

3.3.3 ADVANTAGES AND SUMMARY

In comparison to an equivalent CHS approach, the advantages of an equivalent RHS approach are reported here. Recall that Zhao and Packer (2009) developed this approach because the local buckling failure mode of an axially-loaded EHS stub column (with an aspect ratio of 2:1) was closer to that of plate buckling than of shell buckling.

Advantage 1: the equivalent RHS approach is a potentially useful method to design for axially-loaded members since it maintains the same cross-sectional area as the original EHS, unlike the equivalent CHS approach (Zhao and Packer, 2009). It is also rational in terms of buckling and torsional buckling since the

equivalent RHS has a major and minor axis like the original EHS. This will play a crucial role when developing design rules for EHS under biaxial bending (Law and Gardner, 2010).

Advantage 2: the cross-sectional classification of the equivalent RHS approach employs RHS class limits for compression or bending, which are fairly consistent regardless of which international code is used. In particular, the EC3 Class 3 limit for CHS (CEN, 2005) is very conservative for bending as it is treated the same as for axial compression. Table 3.7 and Table 3.8 show the comparison of the S16 (CSA, 2009) limits with the EC3 limits.

Table 3.7: RHS Class limits for bending, where $c = H - 2t$ or $B - 2t$ (and H or B can be equivalent dimensions)

	Code	Class 1	Class 2	Class 3
Elements subject to compression only	CSA-S16-09	$\frac{c}{t} \leq \frac{420}{\sqrt{f_y}}$	$\frac{c}{t} \leq \frac{525}{\sqrt{f_y}}$	$\frac{c}{t} \leq \frac{670}{\sqrt{f_y}}$
	EN1993:2005	$\frac{c}{t} \leq \frac{506}{\sqrt{f_y}}$	$\frac{c}{t} \leq \frac{583}{\sqrt{f_y}}$	$\frac{c}{t} \leq \frac{644}{\sqrt{f_y}}$
Elements subject to bending	CSA-S16-09	$\frac{c}{t} \leq \frac{1100}{\sqrt{f_y}}$	$\frac{c}{t} \leq \frac{1700}{\sqrt{f_y}}$	$\frac{c}{t} \leq \frac{1900}{\sqrt{f_y}}$
	EN1993:2005	$\frac{c}{t} \leq \frac{1104}{\sqrt{f_y}}$	$\frac{c}{t} \leq \frac{1272}{\sqrt{f_y}}$	$\frac{c}{t} \leq \frac{1901}{\sqrt{f_y}}$

Table 3.8: CHS Class limits (and D can be the equivalent diameter)

CHS	Code	Class 1	Class 2	Class 3
Section in axial compression	CSA-S16-09			$\frac{D}{t} \leq \frac{23000}{f_y}$
	EN1993:2005			$\frac{D}{t} \leq \frac{21150}{f_y}$
Section in bending	CSA-S16-09	$\frac{D}{t} \leq \frac{13000}{f_y}$	$\frac{D}{t} \leq \frac{18000}{f_y}$	$\frac{D}{t} \leq \frac{66000}{f_y}$
	EN1993:2005	$\frac{D}{t} \leq \frac{11750}{f_y}$	$\frac{D}{t} \leq \frac{16450}{f_y}$	$\frac{D}{t} \leq \frac{21150}{f_y}$

Advantage 3: in the equivalent RHS approach, if EHS properties are not used, all the equivalent RHS properties are determined based on geometry alone. Thus to deal with Class 4 sections, an effective area can be simply determined by using the Class 3 limits to “cut out” ineffective portions. This procedure is consistent regardless of which code is used. If ineffective portions are not cut, an effective yield stress needs to be calculated. An effective yield stress will diminish the strength of the entire section leading to

conservative results. The literature consists of a few methods for dealing with Class 4 sections. Chan and Gardner (2007) determined an effective area equation for columns, and Gardner and Chan (2008) have determined an effective elastic section modulus applicable for beams; however, these equations do not provide a single consistent procedure to be used for all EHS design.

Advantage 4: the fundamental basis of the equivalent RHS approach does not change regardless of the EHS member or loading. To be seen in Chapter 6 is the use of the equivalent RHS method in conjunction with existing RHS connection resistance equations to predict connection capacity. It is anticipated that EHS connection design, along with EHS column and beam design, can all benefit from a uniform equivalent RHS approach.

In summary, the equivalent RHS approach can be used to predict both EHS column and EHS beam capacities. It was shown that Method 2a was the preferred method for both columns and beams. The summaries of the design procedures are shown in Figure 3.3 for columns and Figure 3.4 for beams.

Figure 3.3: EHS column design procedure using the equivalent RHS method

Figure 3.4: EHS beam design procedure using the equivalent RHS method

4.0 EXPERIMENTAL PROGRAMME

An experimental programme was developed to test various EHS-to-EHS T and X welded connections at the University of Toronto. The objective of this study was to investigate the behaviour of these connections under quasi-static loading. The investigation included determining the load-carrying capacities, failure modes, stress distributions around the brace near the connection, and chord deformation profiles. The experimental programme includes material property tests and twelve large-scale experiments with varying EHS member orientations. All EHS had nominal dimensions of 220 x 110 x 6mm, and were hot-formed, S355J2H sections conforming to EN10210 (CEN 2006a and 2006b). All plates were 300W grade steel with thicknesses of 32mm.

4.1 MATERIAL PROPERTY TESTS

To determine material properties of the EHS, which includes the yield stress (f_y), ultimate stress (f_u) and Young's modulus (E), three tensile coupons (TC) and one stub column were fabricated from the same EHS stock that was used to fabricate the connections.

4.1.1 TENSILE COUPON TESTS

Three tensile coupons were created from the same cross-section of the EHS. Two coupons were created from the maximum radius of curvature ($r_{\max}$) regions, and one coupon was created from the minimum radius of curvature ($r_{\min}$) region. Figure 4.1 shows the locations from which the coupons were created and Appendix 4A shows the TC measurements.

Figure 4.1: Tensile coupon locations

4.1.2 STUB COLUMN TEST

In addition to the tensile coupon tests, a stub column test was performed to determine EHS properties under compression loading. The stub column test was performed according to the procedures outlined in Appendix B.3 Technical Memorandum No. 3: Stub-Column Test Procedure of the Structural Stability Research Council Guide (Ziemian, 2010). The stub column measurements are shown in Table 4.1. The cross-sectional area (A_{EHS}) from Table 4.1 was determined by dividing the mass (M) by the nominal density (ρ) and length (L) of the stub column: $A_{EHS} = M / (\rho L)$.

Figure 4.2: Stub column relevant dimensions and strain gauge locations

Table 4.1: Stub column measurements

Variable	Measurement
Height (h) – mm	611
Height of Cross-Section (H) – mm	220
Width of Cross-Section (B) – mm	110
Thickness of Cross-Section: at r_{min} – mm	6.03, 6.04
Thickness of Cross-Section: at r_{max} – mm	5.85, 5.83
Average thickness (t) – mm	5.94
Mass (M) – kg	14.6
Nominal Density (ρ) – kg/m ³	7850
Cross-sectional area (A_{EHS}) – mm ²	3044

Two LVDTs were placed to measure the end-shortening of the stub column and designated as LVDT-north and LVDT-south. Four strain gauges were placed around the EHS perimeter at mid-length too, identified by direction (N: north, S: south, W: west, E: east). Figure 4.2 shows the relevant measurement locations and location of strain gauges. A note to the reader: Ziemian (2010) states that the stub column should have a length between $MIN(2D + 250\text{mm}; 3D)$ and $MAX(20r_y, 5D)$, where D is the diameter of the CHS. For EHS, D is taken as the largest dimension = H . Therefore, the stub column length should be

$660 \leq L \leq 1100$. The length of the stub column does not lie in this range because not enough material remained to make a stub column of appropriate length.

4.2 TEST SPECIMENS

The focus of this research is to study the behaviour of various EHS-to-EHS welded connections. This experimental programme included twelve large-scale specimens tested in a 2700kN-capacity MTS universal testing machine. The twelve specimens vary in terms of connection type, angle, brace loading and orientation type. The details of the twelve tests are shown in Table 4.2.

Table 4.2: Test specimens

Test No.	Connection Type	θ_1 (°)	Orientation Type	Brace Loading	β	η	Chord Length (mm)	Brace Length (mm)	Experimental Designation
1	X	90	1	Tension	0.5	1.0	2200	1100	X90-1T
2	X	90	2	Tension	1.0	0.5	1000	1100	X90-2T
3	X	90	3	Tension	1.0	2.0	1020	1100	X90-3T
4	X	90	1	Compression	0.5	1.0	2200	1100	X90-1C
5	X	90	2	Compression	1.0	0.5	1000	1100	X90-2C
6	X	90	3	Compression	1.0	2.0	1020	1100	X90-3C
7	X	45	1	Compression	0.5	1.0	1066	1100	X45-1C
8	X	45	2	Compression	1.0	0.5	1332	1100	X45-2C
9	X	45	3	Compression	1.0	2.0	2200	1100	X45-3C
10	T	90	1	Compression	0.5	1.0	1100	1100	T90-1C
11	T	90	2	Compression	1.0	0.5	1100	1100	T90-2C
12	T	90	3	Compression	1.0	2.0	1100	1100	T90-3C

The first character in the experimental designation is an X or a T, representing either an X connection or a T connection. Tests 1 through 9 were tests of EHS X connections where two brace members were welded onto opposing sides of the chord member. An X connection is classified as a connection in which the force in one of the braces is equilibrated with the brace on the opposite side of the chord. Tests 10 through 12 were tests of EHS T connections where a single brace member was welded onto the chord. A T connection is classified as a connection in which the brace force is equilibrated by shear forces in the chord member (Packer et al., 2009). The first number in the experimental designation is a 90 or 45, representing the angle of the connection in degrees (θ_1). Tests 1 to 6 and 10 to 12 had the brace(s) welded to the chord at 90° angles. Tests 7 to 9 had the braces welded to the chord at 45° angles. The second number in the experimental designation is a 1, 2 or 3, representing the orientation type. Recall from Section 2.6 that there are four possible connection orientations. Since the cross-sections of the braces and chord are the same, connection orientation Type 4 is not physically possible. As such, only the other three

types were investigated; those types are shown in Figure 4.3. The last character in the experimental designation is a T or a C, representing whether the braces were loaded in quasi-static tension or compression. All T connection tests and all 45° connection tests were subjected to brace axial compression; the 90° X connections were tested both in brace compression and tension. Also listed in Table 4.2 is β , which is the ratio of the brace width (b_1) to chord width (b_0) and η , which is the ratio of the brace height (h_1) to chord width (b_0). Since all members are made from the same cross-section, each orientation type results in a specific value of β and η . The fabrication drawings for the test specimens can be found in Appendix 4B and Appendix 4C.

Figure 4.3: Experimental programme orientation types

4.2.1 DESIGN CONSIDERATIONS

When designing the test specimens, the geometric and mechanical properties found in Martinez-Saucedo (2007) were used to determine design loads, as that project used the same EHS material. The limitations of the MTS-2700kN capacity load frame were used to determine maximum specimen lengths and plate thicknesses.

4.2.1.1 Brace Lengths

Except for the connection under consideration, all other joints in each test specimen were designed to be non-critical. Using the values from Martinez-Saucedo (2007), who had performed material property tests using the same stock of EHS, an upper bound on the capacity was established as $A_{\text{EHS}}f_u = (3000\text{mm}^2)(530\text{MPa}) = 1590\text{kN}$. The lengths of the chords and braces were selected to provide sufficient

length to minimize the effects from boundary conditions on the connection in question. Each brace was made to have a length of approximately 1100mm, resulting in a total specimen length of approximately 2.2m for the X connections.

4.2.1.2 End Connections

The MTS load frame requires plates of certain dimensions to clamp onto in order to load the specimens. These are denoted as the end connections, and are located at the far ends of the braces (i.e. the top and bottom). The end connections were not designed as a gusset-plate end connection, as described earlier. To avoid the effects of shear lag, the connection length would have to be at least equal to the distance between the welds (Martinez-Saucedo et al., 2008), which is approximately 200mm. Making this connection length too long would then cause the boundary conditions to affect the connection in question. A simpler alternative without the shear lag issues was to weld the EHS to a plate at the remote end of a brace, and then weld a second plate orthogonally to this first plate oriented along the long-axis of the EHS. Plate thickness and weld sizes were oversized to make them non-critical. Plate thicknesses were also checked to ensure that appropriate force flow from the machine apparatus into the EHS was achieved. It was assumed that a dispersion of stresses would occur at a 2:1 slope, see Figure 4.4.

Figure 4.4: Force flow at 2:1 ratio

4.2.1.3 Connection under consideration

Originally, the weld joint that was to be under investigation was designed as an all-around, complete joint penetration groove weld (CJPG). This was chosen because this would allow the connection to definitely develop the full strength of the member. However, the CJPG weld was found to be practically

unsuitable for this type of connection. To complete a CJPJG weld from one side, a backing-bar is normally required to provide sufficient support. Since the specimen is a HSS connection with a very unique connection profile, providing backing-bars would be very difficult; moreover, the effect of leaving the backing-bars on the interior of the connection would be unknown. For these reasons, a partial joint penetration groove weld (PJPG) combined with a fillet weld was selected.

According to W59 (CSA, 2003) Clause 4.3.4 and Table 4.3, a 60° groove angle with a wall thickness less than 12mm requires a minimum groove depth equal to 5mm. The EHS wall thickness was 6mm, which meant that only 1mm of wall would remain. Although acceptable by standards, this small thickness would not typically be done in practice. The profiling requirements of these connections are quite complex as well. The crown of a 90° connection can require a simple fillet weld, while the saddle of the connection can require extensive bevelling. Due to these factors, the weld joint design was left to the steel fabricator, Walters Inc., with the following note: "EHS-to-EHS welds to be PJPG + Fillet, with Effective Throat = 8mm minimum."

An effective throat of 8mm was determined as follows. To fully obtain the wall strength of the brace, an effective throat of $1.1t$ would be required (Packer and Henderson, 1997), where t is the wall thickness and is equal to 6mm. However, the yield strength of the EHS was 421MPa, thus $1.1t$ must be multiplied by the ratio of the measured EHS yield strength to the nominal steel yield strength (421/350). The effective throat then required is $1.1(6\text{mm})(421\text{MPa} / 350\text{MPa}) = 7.94\text{mm}$. Rounding up this value gives an 8mm effective throat.

Further discussion with the steel fabricators revealed that 8mm may be too large for practical purposes. An effective throat of 6.9mm was deemed sufficient by the fabricator, based on $t = 6.3\text{mm}$, and $1.1t = 1.1(6.3\text{mm}) = 6.9\text{mm}$. The fabrication drawings originally delivered to Walters Inc. were hence modified for fabrication feasibility and appropriate welding specifications. The final fabrication drawings created by Walters Inc. can be found in Appendix 4C. These drawings also include the welding details.

4.3 TEST SETUP AND INSTRUMENTATION

To investigate stress distributions around the brace near the connection, and to obtain load-connection displacement curves and chord deformation profiles, the following instrumentation was applied to the test specimens.

4.3.1 STRAIN GAUGES

Figure 4.5: Strain gauge locations for 90° specimens

(a) Type 1 or 3 connections

(b) Type 2 connections

(c) Side view

 Figure 4.6: Strain gauge locations for 45° specimens

To investigate the stress distributions around the brace near the connection, 7 strain gauges were attached to the top brace member, equidistant around half the EHS perimeter. The strain gauges were placed only on one brace because it was assumed that the top and bottom braces would behave symmetrically. Likewise, the strain gauges were placed only around half the perimeter because it was assumed that opposite halves of the brace would behave symmetrically. All strain gauges were placed parallel to the loading direction and located at a perpendicular distance of 35mm from the top of the chord. This distance was assumed to be close enough to the connection to determine stresses near the connecting EHS face, but far enough such that the heat affected zone caused by welding was avoided. Strain gauge locations for the 90° specimens and 45° specimens are shown in Figure 4.5 and Figure 4.6, respectively. The strain gauge designations from left to right were: SG-LC, -L2, -L1, -CC, -R1, -R2 and -RC.

4.3.2 LVDTs

To measure connection displacements and to establish deformation profiles along the length of the chord, LVDTs (linear variable differential transformers) were placed at various locations along each of the specimens. For HSS connections, the readily adopted convention is to measure displacements perpendicular to the top of the chord. For the 90° connections, the direction of loading was perpendicular to the top of the chord; as such, for these connections, a mount to support the LVDTs was attached to the top brace at a distance of 115 mm from, and parallel to, the top of the chord. LVDTs were placed along the mount to measure deformations normal to the chord. An example of the LVDT locations is shown in Figure 4.7; the remaining LVDT instrumentation diagrams can be found in Appendix 4D.

From left to right, the LVDT designations were: LVDT-LE, -L3, -LM, -L1, -RM, and -RE. Since the chord lengths of X90-1C and X90-1T were longer in comparison to the other specimen chord lengths, an additional LVDT was added to the left side of these connections such that the LVDT designations, in order from left to right, were LVDT-LE, -LM, -L3, -L2, -L1, -RM and -RE. It was expected that due to the symmetry of the specimen, LVDT-RM and LVDT-LM should have similar measurements. As well, LVDT-LE should have similar measurements to LVDT-RE.

Connection displacement is the localized deformation of the connection where the brace and chord centrelines meet. It is measured from the chord centreline to a reference point on the brace. To make this measurement, a plate was welded onto the chord centreline and an LVDT was set atop the plate, as shown in Figure 4.8. This LVDT was designated as LVDT-CC. LVDT-CC was also attached to the same LVDT mount.

For the compression-tested X connections, an additional LVDT was placed with the designation: LVDT-HOR. This LVDT was used to measure the horizontal displacement of the specimens to determine if the specimen was experiencing global out-of-plane buckling. For the T connections, LVDT-HOR was used to measure the horizontal displacement of the T connection end frame to determine if there were any end rotations.

4.3.3 LEDs

For the 45° connections, however, LVDT mounting was arduous, not only due to the complex geometry of the EHS connections but due to the changing geometry between different specimens. As such, a secondary measurement system was employed. To determine connection displacements and chord deformation profiles, a K610 optical camera in conjunction with strobing LEDs (light emitting diodes) was used. The LEDs were affixed to the specimen and the optical camera would track the 3-dimensional coordinates of each LED with time. For LED locations and numeration, see Appendix 4E.

4.3.4 MTS LOAD FRAME

All connections were placed into the MTS 2700kN-capacity universal testing frame and initially loaded at a rate of 0.25mm/second until non-linear behaviour was apparent, at which point the loading rate was increased. For the X connections, the bottom brace end plate was inserted into the MTS base, and the top brace end plate was inserted into the MTS moving head (see Figure 4.9 for typical test setup of X connections). For the T connections, the top brace end plate was inserted into the MTS moving head, but an additional T connection end frame was required to provide restraints for the chord. The end frames were designed for shear and moment and were oversized to make them non-critical. The fabrication drawing of the end frames can be found in Appendix 4F. The chord ends were bolted to these end frames, which were then bolted onto a double-web I-beam with stiffeners, available from the Structural Testing Facility the University of Toronto (see Figure 4.10 for typical test setup of T connections).

4.3.5 LATERAL SUPPORT

Following the test of one compression loaded X connection (X90-2C), global out-of-plane buckling was found to be a serious problem. The connection region yielded much earlier than the remaining specimen leading to the development of a pin. This pin formation caused the global buckling problem. To remedy this, a lateral support system was constructed around the test specimen, which supported the bottom brace just below the connection. The lateral support system comprised of: 1) four pillars that were post-tensioned to the base of the MTS load frame, 2) cross-beams that stabilized the structure and 3) bracing that extended from the cross-beam with a wooden mount to hold onto the specimen, just below the connection region. The post-tensioned structure that braced specimen X45-2C is shown in Figure 4.11. LVDT-HOR was mounted onto the MTS load-frame and measured the lateral displacement of the wooden mount. This post-tensioned lateral restraint system was used for X90-3C, X45-1C, X45-2C, and X45-3C.

Figure 4.9: Typical experimental test setup for X connections (X90-2T)

Figure 4.10: Typical test setup for T connections (T90-1C)

Figure 4.11: Lateral support for compression-loaded X connections (X45-2C)

5.0 RESULTS AND ANALYSIS

In the previous section, the experimental programme was outlined. In this section, the results and analysis from the experimental tests are reported.

5.1 MATERIAL PROPERTIES

The material properties of the EHS (yield stress, f_y , ultimate stress, f_u , and Young's modulus, E), found by tensile coupon and stub column tests, are reported here. Since the same stock of EHS was also used in previous experiments by Martinez-Saucedo (2007), the results from these material property tests will also be compared with the material properties determined by Martinez-Saucedo (2007).

5.1.1 TENSILE COUPON TESTS

Figure 5.1: Tensile coupon engineering stress-strain curves

The results of the tensile coupon (TC) tests are presented here. Figure 5.1 shows the engineering stress-strain curves of the three coupons. Recall that TC-1 and TC-2 were taken from the $r_{\max}$ regions and TC-3 was taken from the $r_{\min}$ region. All TCs exhibit an initial linear-elastic behaviour, the slope of which is taken as E . While both TC-1 and TC-2 exhibit a peak upper yield point, TC-3 does not. TC-3 exhibits a rounded stress-strain behaviour, indicating that the $r_{\min}$ regions of the EHS may not have been entirely

stress-relieved. While TC-2 and TC-3 exhibit a fairly constant yield plateau, TC-1 exhibits a variable yield plateau. The average value through the yield plateau has been taken as f_y . After strain-hardening, the maximum value obtained is taken as f_u . The elongation at failure (ϵ_u) is the strain at which fracture occurs, obtained by measurement after failure by joining the broken coupon pieces together. Table 5.1 lists the summary of the TC test results.

Table 5.1: Tensile coupon test results

Coupon	E (GPa)	f_y (MPa)	f_u (MPa)	ϵ_u (%)
TC-1	211.4	408.0	522.6	36.0%
TC-2	201.5	399.5	516.0	34.0%
TC-3	217.3	397.1	511.6	36.1%
AVERAGE	210.1	401.5	516.7	35.4%

5.1.2 STUB COLUMN TEST

(a) Stress-strain curves

(b) Load-end shortening curve

Figure 5.2: Stub column test results

One stub column test was performed. Figure 5.2a) shows the resulting stress-strain curve; each line represents readings from one strain gauge (SG). All SGs read similar values until the stub column reached its ultimate stress of 430 MPa. The onset of the load-drop occurred when the stub column failed by inelastic local buckling near its top, see Figure 5.3. The local buckling occurred in the r_{max} regions. The SGs located at the r_{max} regions (SG-W and SG-E) read smaller compressive strains while the SGs located at the r_{min} regions (SG-N and SG-S) continued to carry compressive strains. Figure 5.2b) shows the load-end shortening curve. The end-shortening was measured by LVDT-north and LVDT-south. The

measurements from both LVDTs were very close, so an average value was taken and plotted. The stub column reached a peak compressive load of 1380kN, and the post-peak behaviour showed no increase in capacity. From these graphs, it was determined that $E = 208.1\text{GPa}$, only 0.95% less than the tensile coupon average readings ($E = 210.1\text{GPa}$).

(a) Side view

(b) Top view

Figure 5.3: EHS220 x 110 x6 stub column failure

5.1.3 PREVIOUS MATERIAL PROPERTY TESTS

Table 5.2: Current vs. previously determined material properties

Material Property	Current Value	Previous Value*	Current/Previous
Tensile Coupon Tests			
E (GPa)	210.1	216	0.973
f_y (MPa)	401.5	421	0.954
f_u (MPa)	516.7	530	0.975
Stub Column Test			
P_u (kN)	1380	1396	0.989
f_u (MPa)	430	457	0.941
A_{EHS} (mm ²)	3044	3053	0.997

* Previous values from Martinez-Saucedo (2007)

Martinez-Saucedo (2007) performed four tensile coupon tests and one stub column test on the same EHS stock that was used for these experiments. Currently determined material properties are compared with the material properties determined by Martinez-Saucedo (2007) in Table 5.2. The table compares E , f_y and f_u of the tensile coupon tests, and P_u , f_u and A_{EHS} of the stub column test. The A_{EHS} measurements nearly match, which makes the two sets of results very comparable. All currently determined values are lower than those reported previously by Martinez-Saucedo (2007), but are likely within the margins of experimental error and variability.

5.2 SPECIMEN DIMENSIONS

Chord, brace and weld dimensions were measured, with all values reported in mm. First, the chord and brace lengths were measured. The chord length does not include the plates. The brace length is either the top or bottom of the brace, not including the plate, to the top or bottom of the chord at the crown. For the 45° connections, two brace lengths are given since the distances from each of the crowns to the other end of the brace were different. Table 5.3 shows the chord and brace lengths, as well as the chord length (L_c) to chord width (b_0) ratio. Figure 5.4 shows an example of an X connection at 90°, and Figure 5.5 shows an example of an X connection at 45°. The complete set of measurement diagrams are found in Appendix 5A. Second, the welds were measured. All welds were found to be larger than specified weld sizes. The complete set of weld measurements is found in Appendix 5B.

Table 5.3: Measured brace and chord lengths

Test #	Designation	Chord Length - L_c (mm)	L_c / b_0	Brace Length (mm)
1	X90-1T	2198	9.99	1089
2	X90-2T	994	4.52	1045
3	X90-3T	1016	9.24	989
4	X90-1C	2193	9.97	1089
5	X90-2C	997	4.53	1043
6	X90-3C	1016	9.24	989
7	X45-1C	2264	10.29	890/1110*
8	X45-2C	963	4.38	987/1097*
9	X45-3C	1326	12.05	879/1099*
10	T90-1C	1098	4.99	1089
11	T90-2C	1098	4.99	1042
12	T90-3C	1100	10.00	1032

*Indicates two brace lengths for 45° connections

Figure 5.4: X90-1T chord and brace dimensions

Figure 5.5: X45-3C chord and brace dimensions

5.3 LED AND LVDT VALIDATION

As described in Chapter 4, both LVDTs and LEDs with the K610-camera were used to measure displacements. To ensure that both measurement systems provided the same results, they were validated against each other using the X90-2T specimen at two locations. X90-2T was one of the first specimens instrumented with both LVDTs and LEDs and, since it was loaded under quasi-static tension, the likelihood of out-of-plane displacement or rotations was minimized.

LVDT-L1 displacements were compared against the vertical displacements between LED 17 and LED 7, and LVDT-LE displacements were compared against the vertical displacements between LED 16 and LED 4. As shown in Figure 5.6, the displacements from either measurement system nearly match. In the earlier performed tests (X90-2T, X90-2C, X45-2C and T90-1C), the MTS load and displacement signals were not connected with the K610 camera. Other than time, there was no method to link the systems to one another. In order for the LED displacements of X90-2T to synchronize with the LVDT displacements, the LED displacements had to be shifted by 55 seconds. Similarly, the time shift required for X90-2C was 500 seconds, for X45-2C was 240 seconds and for T90-1C was 14 seconds. The time shift is arbitrary; it depended only on the time difference between starting the K610 camera system and the start of the test. For the later performed tests (X90-3C, X45-1C, X45-3C, T90-2C and T90-3C), the MTS load and MTS displacement signals were connected with the K610 camera, hence measurements from either system were directly relatable.

a) LVDT-L1 and LED(17-7)

b) LVDT-LE and LED(16-4)

Figure 5.6: LVDT and LED time synchronization for X90-2T

5.4 LOAD-DISPLACEMENT CURVES AND FAILURE MODES

The load-displacement curves and failure modes of each of the 12 specimens were determined. To recall, two methods were used to measure the connection displacement (Δ_1): LVDTs and LEDs. The first method used LVDT-CC. The contraction or expansion of the LVDT-CC gauge distance is referred to as Δ_1 . The second method used LEDs positioned on the brace, approximately 115mm from the top of the chord, and at the chord centreline. The relative displacement between these LEDs is also Δ_1 . Typically, if no connection rotations were present, the LVDT measurement was used for the load-displacement graphs, but if there was rotation, or for the 45° connections, the LED measurements were used. Since the K610-

camera x, y and z axes did not necessarily correspond to the specimen alignment, geometric transformations were required. For the 45° connections, the displacement component perpendicular to the length of the chord is referred to as Δ_1 . Appendix 5C shows the schematics of how Δ_1 was measured.

When analyzing the results of the experiments, certain specimens behaved similarly. It was observed that the 12 specimens could be categorized in two ways. The first way categorizes the specimens according to their orientation type. There are four specimens per orientation type and these are indicated by the columns in Table 5.4. The second way categorizes the specimens according to their joint type, angle and loading protocol (Joint.Angle-#Load). There are three specimens per Joint.Angle-#Load group and these are indicated by the rows in Table 5.4.

Table 5.4: Two methods of specimen categorization

		Orientation Type		
		Type 1 $\beta = 0.5, \eta = 1.0$	Type 2 $\beta = 1.0, \eta = 0.5$	Type 3 $\beta = 1.0, \eta = 2.0$
Joint.Angle-#Load	T90-#C	a) T90-1C	e) T90-2C	i) T90-3C
	X45-#C	b) X45-1C	f) X45-2C	j) X45-3C
	X90-#C	c) X90-1C	g) X90-2C	k) X90-3C
	X90-#T	d) X90-1T	h) X90-2T	l) X90-3T

Figures 5.7 to 5.18 show the load-displacement curves of each specimen. The curves are presented in the alphabetic order shown in Table 5.4. For compression-loaded specimens, compressive load is positive and connection contraction is positive. Conversely, for tension-loaded specimens, tensile load is positive and connection expansion is positive. Two important limits are shown on the load-displacement graphs: the ultimate load (UL) and the 3% deformation limit load (3%DL). The UL is a load limit and is shown with the green lines. It occurs when the connection reaches its maximum load capacity. The load that occurs at the UL is denoted as N_{1u} , and the associated connection displacement is denoted as Δ_{1u} . The 3%DL is a displacement limit, proposed and readily adopted since Lu and colleagues first proposed it in 1994. It is shown with the blue lines. The 3%DL limits the capacity of the connection to when the connection displacement = 3% of the chord width (b_0). For the Type 1 and 2 connections, 3% of b_0 is = 6.6mm, and for the Type 3 connections, 3% of b_0 is = 3.3mm. The displacement corresponding to the 3%DL is denoted as $\Delta_{1(3\%)}$, and its associated load is denoted as $N_{1(3\%)}$. Figure 5.19 shows the ultimate failure mode of each specimen, presented in the same order that is shown in Table 5.4. For scale, the key dimensions to recall are the large dimension of the EHS, $H = 220\text{mm}$, and the small dimension of the EHS, $B = 110\text{mm}$. The results are summarized in Table 5.5. For one-page experimental summaries, see Appendix 5D.

Figure 5.7: Load-connection displacement graph of a) T90-1C

Figure 5.8: Load-connection displacement graph of b) X45-1C

Figure 5.9: Load-connection displacement graph of c) X90-1C

Figure 5.10: Load-connection displacement graph of d) X90-1T

Figure 5.11: Load-connection displacement graph of e) T90-2C

Figure 5.12: Load-connection displacement graph of f) X45-2C

Figure 5.13: Load-connection displacement graph of g) X90-2C

Figure 5.14: Load-connection displacement graph of h) X90-2T

Figure 5.15: Load-connection displacement graph of i) T90-3C

Figure 5.16: Load-connection displacement graph of j) X45-3C

Figure 5.17: Load-connection displacement graph of k) X90-3C

Figure 5.18: Load-connection displacement graph of l) X90-3T

Figure 5.19: Ultimate failure modes

Table 5.5: Summary of experiments and results

Test #	Specimen	Chord		Brace		Parameters			Load*		Ultimate Failure Mode**
		b_0	h_0	b_1	h_1	θ_1	$\beta =$	$\eta =$	UL	3%DL	
		(mm)	(mm)	(mm)	(mm)	°	b_1 / b_0	h_1 / b_0	(kN)	(kN)	
10	a) T90-1C	220	110	110	220	90	0.5	1.0	216.5	211.4	CP
7	b) X45-1C	220	110	110	220	45	0.5	1.0	350.3	258.5	CP
4	c) X90-1C	220	110	110	220	90	0.5	1.0	202.4	150.5	CP
1	d) X90-1T	220	110	110	220	90	0.5	1.0	339.1	187.9	CP; CT
11	e) T90-2C	220	110	220	110	90	1.0	0.5	353.0	338.2	SW, CP
8	f) X45-2C	220	110	220	110	45	1.0	0.5	627.8	531.0	SW
5	g) X90-2C	220	110	220	110	90	1.0	0.5	539.7	537.8	GB, SW
2	h) X90-2T	220	110	220	110	90	1.0	0.5	596.8	574.5	SW
12	i) T90-3C	110	220	110	220	90	1.0	2.0	593.8	550.6	SW, CP
9	j) X45-3C	110	220	110	220	45	1.0	2.0	701.0	656.9	SW
6	k) X90-3C	110	220	110	220	90	1.0	2.0	555.1	459.8	SW
3	l) X90-3T	110	220	110	220	90	1.0	2.0	1557.0	1188.8	BF

*Highlighted values = governing capacity. Note the UL may govern over the 3%DL, even if the failure load is higher, if the ultimate load precedes the (lower) 3% displacement limit load.

**Ultimate Failure Mode heading: BF = brace failure, CP = chord plastification, CT = chord tearout, GB = global (overall) buckling, SW = chord sidewall failure.

5.4.1 OBSERVATIONS

Observations regarding the load-displacement graphs and failure modes are made here. Some common behaviours can be found based on the categories shown in Table 5.4. The observations made below are summarized below in Table 5.6 according to those categories.

5.4.1.1 Type 1 Connections (see Figure 2.16)

For the Type 1 connections, a) T90-1C, b) X45-1C, c) X90-1C and d) X90-1T with $\beta = 0.5$ and $\eta = 1.0$, chord plastification was the dominant failure mode. For Type 1 connections in compression, the chord increasing flattened as the MTS displacement increased. The connections were fairly deformable and their capacities were governed by the 3%DL. While technically for T90-1C, the UL occurred first and therefore should govern, the UL and the 3%DL nearly occurred at the same connection displacement of 6.6mm. As such, it can also be said that T90-1C was governed by the 3%DL. For the Type 1 connections in compression, the load-connection displacement curves eventually reached a local maxima or a plateau. The local maxima or plateau signified the yielding of the chord at its centreline. Afterwards, the connection capacity increased again. This change in slope is attributed to second order effects. As the connection increasingly deformed, tension stiffening would have occurred at the top and bottom of the chord (only the top of the chord for the T connection) leading to increased capacity. For X45-1C and X90-1C, a second slope increase occurred at the end of the curves. This is attributed to the two braces

making contact with each other through the complete flattening of the chord. See Figure 5.20 for a comparison of Type 1 connections in compression.

For the Type 1 connection in tension (X90-1T), the failure mode was also chord plastification, but instead of the chord flattening, it circularized. In addition, the bottom brace eventually caused tearout of the chord leading to the drop in capacity. The governing capacities of Type 1 connections were the lowest in comparison to their Type 2 and Type 3 counterparts.

Figure 5.20: Type 1 connections in compression

5.4.1.2 Type 2 Connections (see Figure 2.16)

For the Type 2 connections, e) T90-2C, f) X45-2C, g) X90-2C and h) X90-2T with $\beta = 1.0$ and $\eta = 0.5$, chord sidewall failure was the dominant failure mode. With the chord width equalling the brace width, it is reasonable to expect that the sidewall of the chord would be the critical region. It was observed that the capacities of Type 2 connections in compression were governed by the UL. For T90-2C and X45-2C, the UL is marked by a local maxima. After this, the connection capacity increased; this can be attributed to second order tension stiffening effects that would have occurred at the top of the chord for T90-2C and the top and bottom of the chord for X45-2C. This behaviour was not seen in X90-2C, but it is hypothesized that it would have occurred had it been laterally restrained. As X90-2C was not restrained from lateral displacement as the other compression-loaded specimens were, it underwent overall global buckling. See Figure 5.21 for a comparison of Type 2 connections in compression.

Even though X90-2C globally buckled, on the local scale the braces eventually pinched then punctured the sidewall of the chord ultimately leading to chord sidewall failure, see Figure 5.22. This pinching action can also be seen in X45-2C (Figure 5.19f).

Figure 5.21: Type 2 connections in compression

(a) Chord sidewall pinching

(b) Brace puncturing the chord sidewall

Figure 5.22: X90-2C chord sidewall failure

The first sign of failure with X90-2T occurred when the top brace fractured just above the weld. This fracture did not lead to a significant drop in load, however. It is surmised that it was only a surface crack

and did not go through the entire brace thickness; thus, this brace crack is not considered as an ultimate limit state. X90-2T ultimately failed via chord sidewall failure by the formation of another crack. This crack did go through the entire chord thickness and did result in a drop in capacity. Once this crack formed, both the brace crack (see Figure 5.23a) and the chord sidewall crack (see Figure 5.23b) propagated as the MTS displacement was increased. The governing capacities of Type 2 connections were in between the governing capacities of their Type 1 and Type 3 counterparts.

Figure 5.23: X90-2T brace failure and chord side wall failure

5.4.1.3 Type 3 Connections (see Figure 2.16)

For the Type 3 connections, i) T90-3C, j) X45-3C, k) X90-3C and l) X90-3T with $\beta = 1.0$ and $\eta = 2.0$, chord sidewall failure was the dominant failure mode. Again, with the chord width equalling the brace width, it is reasonable to expect that the sidewall of the chord would be the critical region. The exception is X90-3T. This tension tested specimen failed by the yielding and eventual rupturing of the top brace. This phenomenon gave a means to approximately calculate f_y and f_u of the EHS and to compare with the tensile coupon values (recall Table 5.1). From the load-connection displacement curve (Figure 5.18), the ultimate load = 1557.0kN and the load just before strain hardening is approximately = 1200kN. Using $A_{EHS} = 3044\text{mm}^2$, $1557\text{kN}/3044\text{mm}^2 = 512\text{MPa} \sim f_u = 517\text{MPa}$, and $1200\text{kN}/3044\text{mm}^2 = 396\text{MPa} \sim f_y = 402\text{MPa}$. The X90-3T results corroborate the tensile coupon results.

For Type 3 connections in compression, small deformations were apparent; thus, the UL governed the connection capacities. Unlike the Type 2 connections, after reaching the UL, the chord sidewalls locally buckled, tension stiffening did not occur and connection capacities did not increase. See Figure 5.24 for a comparison of Type 3 connections in compression. The governing capacities of Type 3 connections are the highest in comparison to the governing capacities of their Type 1 and Type 2 counterparts.

Figure 5.24: Type 3 connections in compression

Table 5.6: Load-displacement graph and failure mode observations based on orientation type groups

	Type 1	Type 2	Type 3
Failure Mode	Chord plastification (X90-1T also experienced chord tear out)	Chord sidewall failure (X90-2C also globally buckled; X90-2T also experienced brace failure)	Chord sidewall failure (Exception: X90-3T underwent brace failure)
Compression loaded connection capacities governed by:	3%DL (Technically UL for T90-1C, but UL and 3%DL nearly coincide)	UL	UL
Compression loaded specimens load- displacement curve	After initial drop in capacity, the chord top and/or bottom may have experienced tension stiffening resulting in the connection increasing in capacity again.		Chord sidewall buckling led to a continual decrease in connection capacity after the UL
Governing connection capacities	Weakest	Between Types 1 and 3	Strongest

5.4.1.4 Joint, Angle and Loading

Some common behaviours can be found based on the second categorical method (Joint.Angle-#Load). Typically, 45° connections had higher connection capacities than their counterparts. The only exception is X90-3T which had a higher capacity than X45-3C, but X90-3T was considered a special case since it was

a tension test that failed in the brace. It was also observed that capacities of T connections were governed by their UL, and that the capacities of tension-loaded connections were governed by their 3%DL. It should also be noted that the T connections would have experienced bending and as such, compressive normal stresses would have developed at the connecting face. It is recommended for future T connection tests that strain gauges be placed on the chord to determine stresses in the chord at the connection.

5.5 CHORD DEFORMATION PROFILES

To develop chord deformation profiles (CDPs), LVDTs and LEDs were placed along the top of the chord to measure relative chord displacements at various load stages. For the 90° connections, CDPs were established using the LVDTs, and for the 45° connections, CDPs were established using LEDs.

5.5.1 90° CONNECTIONS

For the 90° connections (except X90-2C, for which global buckling occurred), the chords remained perpendicular to the direction of loading until, at least, the ultimate limit; therefore, LVDTs could simply be used to generate the CDPs. In developing the CDPs, LVDT-LE (the LVDT located at the left-end) and LVDT-RE (the LVDT located at the right-end), were used as reference points. The LVDTs located on the left side of the braces used LVDT-LE as their reference point, and the LVDTs located on the right side of the braces used LVDT-RE as their reference point. Figures 5.25 to 5.27 show the CDPs for the 90° connections. Figure 5.25 shows the CDPs for the tension-tested X connections (a) X90-1T, b) X90-2T and c) X90-3T), Figure 5.26 shows the CDPs for the compression-tested X connections (a) X90-1C, b) X90-2C and c) X90-3C), and Figure 5.27 shows the CDPs for the compression-tested T connections (a) T90-1C, b) T90-2C and c) T90-3C). In Figures 5.25 to 5.27, the points represent the measured LVDT displacements relative to the end LVDTs, the solid lines indicate the linear interpolation between known displacements, and the dashed lines show the assumed displacements based on symmetry of the specimen and brace locations. The different colours signify the CDPs at various load stages. Red represents the load stage at the UL, blue represents the load stage at 50% of the UL, green represents the load stage at the 3%DL, and orange represents the load stage at 50% of the 3%DL. For the specimens that had the UL govern the load capacity of the connection, only the red and blue lines are shown. For the specimens that had the 3%DL govern the load capacity of the connection, the red, green and orange lines are shown.

Figure 5.25: Chord deformation profiles of 90° X connections in tension

Figure 5.26: Chord deformation profiles of 90° X connections in compression

Figure 5.27: Chord deformation profiles of 90° T connections in compression

The CDPs show that the maximum deformations were localized around the connection region. Generally, the Type 1 connections were the most deformable and the Type 3 connections were the least deformable. This was most obvious when comparing X90-1C (Figure 5.26a) and X90-3C (Figure 5.26c). Figure 5.25a) and Figure 5.26a) show the CDPs of X90-1T and X90-1C, respectively. Due to the expected deformability of these connections, chord lengths were extended to minimize the effects of end

conditions. As seen with both X90-1T and X90-1C, there were negligible deformations from the ends to the next closest LVDT, but high deformations around the connection region. An oddity with X90-1C was observed: there was an upward displacement at the second node from the left despite the load being applied downward. Due to some second order effects, it appears that some circularization occurred at this region.

At the location of the brace, the assumed CDP is shown to be a horizontal dashed line. Generally, the connection displacement (Δ_1) exceeds the value of this horizontal dashed line. This supports the argument that the maximum deformations occurred at the connection region. However, this is not true for T90-2C (Figure 5.27b) and T90-3C (Figure 5.27c). In these tests, Δ_1 was less than the indicated horizontal line. This suggests that the bending action experienced by the chords of T connections contributes to the chord deformations more than the localized indentation at the connection region.

Figure 5.26b shows the CDP of X90-2C, which is unlike the other CDPs. The reason is that X90-2C experienced global buckling leading to non-symmetrical behaviour. The left side compressed the LVDTs while the right side allowed the LVDTs to expand. No dashed lines were drawn in this case since the braces did not remain perpendicular to the chord.

5.5.2 45° CONNECTIONS

For the 45° connections, developing CDPs was complex since the chord did not simply displace in the direction of loading, but the chord itself rotated in the plane of the specimen. For this reason, the planar positions of the LEDs were used to track the chord displacements, which are shown in Figures 5.28 to 5.30. In these figures, the LED positions were shifted to make the origin correspond with the connection centre. The black lines show the original shape and red lines show the chord shape after discernable deformation (at the chord top, mid-height and bottom). Figure 5.28 shows X45-1C originally and at the UL. There was a slight rotation counter-clockwise, but there was prominent indentation at the connection region. Figure 5.29 shows X45-2C originally and when $\Delta_1 = 5.2\Delta_{1u}$. There was a slight rotation counter-clockwise, but the rotation came from the connection region; the chord ends remained relatively undeformed. Figure 5.30 shows X45-3C originally and when $\Delta_1 = 7.2\Delta_{1u}$. Even at 7.2 times the displacement at the UL, this connection showed very little in-plane displacement, just like its 90° counterpart, X90-3C.

Figure 5.28: Chord deformation profile of X45-1C

Figure 5.29: Chord deformation profile of X45-2C

Figure 5.30: Chord deformation profile of X45-3C

5.5.3 CROSS-SECTIONS

Post-experimental cross-sections were cut out of the T connections to further investigate the deformations at the connection region. Images of these cut-out cross-sections are shown in Figure 5.31. Note for scale: section cuts were made through the short dimension of the brace = 110mm. In Figure 5.31a), a cut-out of T90-1C through the chord cross-section is shown. The brace remained relatively unchanged, but the chord showed excessive deformation. The top and bottom of the chord have squashed such that the elliptical form had essentially turned into two plates. The only portion to remain in the elliptical form was at the top of the chord between the welds. Similarly, in Figure 5.31c), the cut-out of T90-3C through the chord cross-section is shown. The brace again remained relatively unchanged, but the chord showed excessive deformation. Apparent in this figure are the chord sidewalls buckling. Like T90-1C, the only portion to remain in the elliptical form was at the top of the chord between the welds. Figure 5.31b) shows the cut-out section of T90-2C along the length of the chord. The chord top remained straight between the welds. An oddity was observed at the bottom of the chord just below the brace. Since the T connections experienced bending, one would expect the chord to sag down; however, at this localized region, the bottom of the chord is not sagging but rather hogging. It appears that as the connection displacement increased, the squashing of the chord resulted in out-of-plane displacements. When this occurred, the sides of the chord pulled the bottom of the chord upward resulting in this hogging effect.

Figure 5.31: Cut-out sections of T connections

5.6 BRACE STRESSES

To establish brace stress profiles, strain gauges (SG) were placed equidistant around half the perimeter of the EHS brace. Figure 5.32 shows the SG designations during the test and their associated SG location number. The front of the specimen was considered to be the side facing the K610-camera. With this directionality, all SG-L#s are on the left side of the specimen and all SG-R#s are on the right side. For the X45-1C and X45-2C specimens, the front views of the specimens had the high end of the chord on the left side. For the X45-3C specimen, however, the front view of the specimen had the high end of the chord on the right side. In order to be able to compare stress profiles better, the plots produced for X45-3C are the mirror images of the actual results such that the high end of the chord (i.e. the heel) is always oriented to the left side. The axis of symmetry for the 90° connections lies through SG-4, such that the measurements from SG-1 should correspond to SG-7, and likewise, SG-2 with SG-6 and SG-3 with SG-5.

Figure 5.32: Strain gauge designations to their strain gauge location number

Twelve brace stress profile graphs were developed for each of the 12 connection tests. The 12 graphs are found below separated into 3 figures according to their orientation type (Type 1, 2 or 3). Figure 5.33, Figure 5.34 and Figure 5.35 show the brace stress profiles for Type 1, 2 and 3 connections, respectively. The graphs plot stress on the vertical axis and SG location numbers on the horizontal axis. For tests that underwent quasi-static compression (specimen designations ending with “C”), positive stress indicates compression, and those tests that underwent quasi-static tension (specimen designations ending with “T”), positive stress indicates tension. On the graphs, the points up to yield show the strain measurements multiplied by the E determined from the tensile coupon tests ($E = 210,040\text{MPa}$), and the lines connecting the points show a linear interpolation between these measured values. Beyond f_y , the stresses were determined from the tensile coupon stress-strain curves. This came into effect in Figure 5.34b: X45-2C, Figure 5.34c: X90-2C, and Figure 5.35d: X90-3C. For the strain measurements which exceeded the yield point, it was found that the strains lay on the yield plateau, thus the stresses were taken as $f_y = 402\text{MPa}$.

(a) T90-1C: compressive stress positive

(b) X45-1C compressive stress positive

(c) X90-1C: compressive stress positive

(d) X90-1T: tensile stress positive

(e) Type 1 orientation and strain gauge locations

Figure 5.33: Brace stress profiles for Type 1 connections

— $N_{1u} = 353.0\text{kN}$; — $50\% N_{1u} = 179.6\text{kN}$

(a) T90-2C: compressive stress positive

— $N_{1u} = 627.8\text{kN}$; — $50\% N_{1u} = 317.6\text{kN}$

(b) X45-2C: compressive stress positive

— $N_{1u} = 539.7\text{kN}$; — $50\% N_{1u} = 270.4\text{kN}$

(c) X90-2C: compressive stress positive

— $N_{1u} = 596.8\text{kN}$;
- - - $N_{1(3\%)} = 574.5\text{kN}$; - . - $50\% N_{1(3\%)} = 288.9\text{kN}$

(d) X90-2T: tensile stress positive

(e) Type 2 orientation and strain gauge locations

Figure 5.34: Brace stress profiles for Type 2 connections

— $N_{1u} = 539.7\text{kN}$; — $50\% N_{1u} = 296.4\text{kN}$

(a) T90-3C: compressive stress positive

— $N_{1u} = 701.0\text{kN}$; — $50\% N_{1u} = 351.8\text{kN}$

(b) X45-3C: compressive stress positive

— $N_{1u} = 555.1\text{kN}$; — $50\% N_{1u} = 277.5\text{kN}$

(c) X90-3C: compressive stress positive

- - $N_{1(3\%)} = 1188.8\text{kN}$; - - $50\% N_{1(3\%)} = 594.8\text{kN}$

(d) X90-3T: tensile stress positive, capped at $f_y = 402\text{MPa}$

(e) Type 3 orientation and strain gauge locations

Figure 5.35: Brace stress profiles for Type 3 connections

In the cases where the connection capacity was governed by the UL, the brace stress profiles at the ultimate load (solid black) and at 50% of the ultimate load (solid gray) are plotted. In the cases where the connection capacity was governed by the 3%DL, the brace stress profiles at the ultimate load (solid black), at the 3%DL load (dashed black), and at 50% of the 3%DL load (dashed gray) are plotted. An exception occurs with Figure 5.35d). The stress profile of X90-3T at the UL load stage is not shown. The brace stresses at the 3%DL load stage were mostly at f_y . At the UL stage, the stresses were all f_y , hence they are not plotted. This is in agreement with the failure mode of this specimen: brace failure. (Note: even though the ultimate failure was the rupturing of the brace, the strain gauge readings corresponded to f_y and not f_u).

5.6.1 TYPE 1 CONNECTIONS

Additional observations regarding the brace stress profiles for Type 1 connections are given here. T90-1C and X90-1C showed similar behaviours, while X45-1C and X90-1T showed similar behaviours.

5.6.1.1 T90-1C and X90-1C

For T90-1C, the connection capacity was technically governed by the UL, but this coincided with the 3%DL. Therefore, it was generally observed that the capacities of Type 1 connections were governed by the 3%DL. At 50% of the UL or 3%DL load stage, the brace stress profile is shown in Figure 5.33a). The compressive stresses were highest at the “corners” of the EHS brace and lowest at the “flat” region. The shapes of the stress profiles are “U” shaped symmetric about SG-4. As T90-1C reached its capacity (load stage at the UL or 3%DL), the compressive stresses were still highest at the “corners”, but the compressive stress at the “flat” region (corresponding to SG-4) increased; the shape of the stress profile changed into a “W”. It is to be expected that the highest stresses would occur at the “corners” because these are the stiffest parts of the EHS.

The same stress profile shapes can also be seen in X90-1C (Figure 5.33c). At 50% of the 3%DL load stage, X90-1C had the highest compressive stresses at the “corners” and lowest stresses (in fact small tensile stresses) at the “flat” region. The shapes of the stress profiles are “U” shaped symmetric about SG-4, which is similar to T90-1C. As X90-1C reached its capacity (load stage at the 3%DL), the compressive stresses were still highest at the “corners”, but the compressive stress at the “flat” region (corresponding to SG-4) had increased; the shape of the stress profile is like a “W”, which is similar to T90-1C. Again, it is expected that the highest stresses occurred at the “corners” because these are the stiffest parts of the EHS.

X90-1C is different to T90-1C at the UL load stage. At the UL load stage for X90-1C, the lowest stresses occurred in the “corners” and the highest stresses occurred at the “flat” region (corresponding to SG-4). The shape of the stress profile is like an inverted “U”. At the UL, the top and bottom brace would have just made contact with each other. Based on the stress profile, it is assumed that the “flat” regions made contact first. Shortly after the braces made contact, the test was stopped. It is suspected that if the test was to have continued, stresses across the section would attempt to become more uniform.

5.6.1.2 X45-1C and X90-1T

The stress profiles for X45-1C (Figure 5.33b) are asymmetric, which is understandable due to the physical asymmetry of the specimen. On all three stress profiles, the stresses at SG-1 and SG-2 (the left “corner” or heel) are nearly equivalent. They are also the values closest to zero, though still in compression. At both 50% of 3%DL and 3%DL load stages, at stresses at SG-7 (the right “corner” or toe) are in tension. The shapes of the stress profiles from SG-2 to SG-7 are like inverted “U”s peaking at SG-4, the middle of the brace. Thus, as X45-1C reached its capacity (load stage at the 3%DL), both the heel and toe of the brace have minimal loading in compression. At the UL load stage, the top and bottom brace would have made contact with each other and it can be seen that the stresses across the section were attempting to become uniform.

The brace stress profiles of X90-1T (Figure 5.33d) are somewhat similar to X45-1C except X90-1T was loaded in tension and displayed more symmetric behaviour. At both 50% of the 3%DL and 3%DL load stages, the shapes of the stress profiles are like inverted “U”s peaking at SG-4, which is similar to X45-1C. Even though the connection was loaded in tension, the “corners” (corresponding to SG-1 and SG-7) experienced compressive stresses. It is hypothesized that since the chord “circularized” with increasing loads, the chord at the saddle regions (corresponding to the “flat” regions of the brace) would have deformed more than the chord at the crown regions (corresponding to the “corners” of the brace). As “circularization” occurred and the chord yielded in this region, it would have pulled at the “flat” regions of the braces into tension, but pulled brace “corners” into compression. Thus as X90-1T reached its capacity (load stage at the 3%DL), the tension loaded specimen experienced compressive stresses at the brace “corners”. Finally, at the UL load stage when the bottom brace tears the chord at the base of the weld, the stresses across the section attempted to become uniform and return to zero.

5.6.2 TYPE 2 CONNECTIONS

The stress profiles of Type 2 connections are shown in Figure 5.34. The stresses at the “corners” of the brace (corresponding to SG-4) were always the peak values, which is reasonable considering the width

ratio $\beta = 1.0$, and the force flows principally from the top brace to the bottom brace of such connections via the chord sides. Thus, in general, the shapes of the stress profiles are like inverted “V”s, symmetric about SG-4.

For T90-2C (Figure 5.34a) and X90-2C (Figure 5.34c), even though the specimens were loaded in brace compression, at their capacities (load stages at the UL), the braces experienced tensile stresses at the “flat” portions (corresponding to SG-7 for T90-2C and SG-1 for X90-2C. Note: SG-7 readings were not available for X90-2C). Similarly, for X45-2C (Figure 5.34b), even though the specimen was loaded in brace compression, at its capacity (load stage at the UL), the brace experience tensile stresses at the heel (corresponding to SG-6 and SG-7). The difference is that X45-2C did not experience symmetric behaviour about SG-4, which is expected due to the geometric asymmetry. For X90-2T (Figure 5.34d), the “flat” portions of the brace (corresponding to SG-1 and SG-7) experienced compressive stresses at 50% of 3%DL load stage. As X90-2T reached its capacity (load stage at the 3%DL), all stresses became tensile. Finally, when X90-2T reached the UL load stage, the tensile stresses across the section attempted to become more uniform.

5.6.3 TYPE 3 CONNECTIONS

The stress profiles of Type 3 connections are shown in Figure 5.35. Generally, the peak stresses occurred at SG-4, corresponding to the side of the brace flush with the chord. With the chord width equalling the brace width, the force flow path from the top brace to the bottom brace of X connections and from the top brace to the chord of T connections would occur through the SG-4 line. Another general observation is that the shapes of the stress profiles at the UL load stage are shaped like inverted “U”s.

The stress profiles for X45-3C (Figure 5.35b) are asymmetric with stresses being higher at the brace heel and lower at the toe. This is similar to the stress profile trend of X45-1C (Figure 5.33b). Stress profiles of X90-3C (Figure 5.35c) resemble the stress profile of X90-1C at the UL load stage (Figure 5.33c). The difference is that the stresses at the “corners” of X90-3C (SG-1 and SG-7) stayed relatively close to zero. As mentioned previously, X90-3T (Figure 5.35d) ultimately failed by brace failure and most of the stresses at capacity (load stage at the 3%DL) equalled f_y . The stresses at the UL load stage were also equal to f_y , and thus not shown.

6.0 CAPACITY PREDICTIONS

In Chapter 5, the test specimen connection capacities and ultimate failure modes were determined. In this chapter, attempts to devise preliminary methods to predict connection capacities and failure modes are made. A pragmatic approach in recent years has been to establish some equivalency between an EHS and a secondary shape for which design limits or equations already exist. Recall, two conversion methods have surfaced to determine the cross-sectional classification of an EHS. The first method determines an equivalent CHS diameter. In this chapter, the conversion of an EHS connection into an equivalent CHS connection is called the “equivalent CHS approach”. The second method determines an equivalent RHS by maintaining the cross-sectional area and thickness. In this chapter, the conversion of an EHS connection into an equivalent RHS connection is called the “equivalent RHS approach”. While both methods were derived for classification purposes, they have been adapted here and made applicable to predict connection capacities.

6.1 X AND T CONNECTIONS FROM THE UNIVERSITY OF TORONTO

The following describes the two methods used to predict capacities of the EHS T and X connections that were tested at the University of Toronto and described in Chapters 4 and 5.

6.1.1 EQUIVALENT CHS APPROACH

The method termed the “equivalent CHS approach” is the first prediction method established here. The approach is based on the equations developed by Ruiz-Teran and Gardner (2008). The approach converts the EHS chord into an equivalent CHS using their equations, see Equations (6.1) and (6.2), and the brace diameter is determined by ensuring that β (brace diameter to chord diameter) = d_1/d_0 remains constant, see Equation (6.3). The variables $d_{0,eq}$ and $d_{1,eq}$ are the equivalent chord and brace diameters, respectively.

$$f = 1 - 2.3 \left(\frac{t}{H} \right)^{0.6} \quad (6.1)$$

$$d_{0,eq} = D_{e,new} = H \left[1 + f \left(\frac{H}{B} - 1 \right) \right] \quad (6.2)$$

$$d_{1,eq} = \beta d_{0,eq} \quad (6.3)$$

The equivalent diameters are calculated by using the measured properties: $H = 220\text{mm}$, $B = 110\text{mm}$, and $t = 5.94\text{mm}$ (Table 4.1). Completing the calculations, for all connections $d_{0,eq} = 382.1\text{mm}$. For Type 1 connections, $d_{1,eq} = 191.1\text{mm}$, and for Type 2 and 3 connections $d_1 = 382.1\text{mm}$. These values are then inputted as d_0 and d_1 into CHS connection design equations. The design equations found in Table 4.1 of

CIDECT Design Guide 1 for Circular Hollow Section (CHS) Joints (Wardenier et al., 2008) are used with experimentally determined material properties ($f_y = 402\text{MPa}$ and $E = 210,040\text{MPa}$) to make connection capacity predictions. The relevant CHS connection design equations are reproduced in Table 6.1 where N_1^* is the connection strength and taken as the predicted connection capacity. The predictions are summarized in Table 6.2. The values with asterisks indicate the governing predictions, and the “Actual/Predicted” heading is the ratio of the actual capacity to the governing prediction. These predicted connection capacities are all less than the yield strength of the brace ($A_{\text{EHS}}f_y$). For the T connections, these predicted connection capacities are also lower than the load that would have caused the shear capacity or the plastic moment capacity of the chord to be reached. The shear capacity of the chord is calculated by taking the shear area of the EHS chord (A_v), Equation (2.34), multiplied by the chord shear yield strength of $0.58f_y$.

Table 6.1: Relevant CIDECT CHS connection design equations

Criterion	Equations	
Chord plastification	$N_1^* = Q_u Q_f \frac{f_{y0} t_0^2}{\sin \theta_1}$	
	X Connection	T Connection
	$Q_u = 2.6 \left(\frac{1 + \beta}{1 - 0.7\beta} \right) \gamma^{0.15}$ $Q_f = 1.0$	$Q_u = 2.6(1 + 6.8\beta^2)\gamma^{0.2}$ $Q_f = (1 - n)^{c_1}$ $n = \frac{N_0}{N_{pl,0}} + \frac{M_0}{M_{pl,0}}$ Compressive stress at connection: $c_1 = 0.45 - 0.25\beta$ Tensile stress at connection: $c_1 = 0.2$
Chord punching shear (only for $d_1 \leq d_0 - 2t_0$)	$N_1^* = 0.58f_{y0}\pi d_1 t_0 \frac{k_a}{\sin \theta_1} \quad ; \quad k_a = \frac{1 + \sin \theta_1}{2 \sin \theta_1}$	

For the T connections, moments in the chord were introduced causing normal compressive stresses in the chord at the connection. This is accounted for through the factor Q_f . The plastic moment capacity of the chord ($M_{pl,0}$) is determined by finding the plastic section modulus (Z) of the original EHS chord using measured dimensions and the equations found in Table 2.1 and multiplying it with the experimentally determined yield stress. For Type 1 and 2 connections, Z_y is determined. For Type 3 connections, Z_x is determined. The end rotations were found to be negligible, so fixed-fixed end conditions are assumed. Thus the load which would cause $M_{pl,0}$ is denoted as $P_{pl,0}$ and is calculated using Equation (6.4), where L_c is the length of the chord and h_1 is the height of the original EHS brace. The value n is the stress ratio of the chord; since there were no axial forces, n is taken as the ratio of the governing experimental load to $P_{pl,0}$ ($= M_0/M_{pl,0}$). This value of n is used to determine Q_f (see Table 6.1).

$$P_{pl,0} = \frac{8M_{pl,0}}{L_c - h_1} \quad (6.4)$$

Table 6.2: Connection capacity predictions using the equivalent CHS approach

Test	Parameters			Actual Capacity (kN)	Experimental Failure Mode	Predicted Capacity (kN)		<u>Actual</u> Predicted
	θ_1 (°)	β	η			Chord Plastification	Punching Shear	
X90-1T	90	0.5	1.0	187.9	CP; CT	143.2*	831.2	1.312
X90-2T	90	1.0	0.5	574.5	BF; SW	413.8*	n/a	1.388
X90-3T	90	1.0	2.0	1188.8	BF	413.8*	n/a	2.873
X90-1C	90	0.5	1.0	150.5	CP	143.2*	831.2	1.051
X90-2C	90	1.0	0.5	539.7	GB, SW	413.8*	n/a	1.304
X90-3C	90	1.0	2.0	555.1	SW	413.8*	n/a	1.342
X45-1C	45	0.5	1.0	258.5	CP	202.6*	1418.9	1.205
X45-2C	45	1.0	0.5	627.8	SW	585.2*	n/a	1.073
X45-3C	45	1.0	2.0	701.0	SW	585.2*	n/a	1.198
T90-1C	90	0.5	1.0	216.5	CP	155.0*	831.2	1.397
T90-2C	90	1.0	0.5	353.0	SW	236.2*	n/a	1.495
T90-3C	90	1.0	2.0	593.8	SW	359.2*	n/a	1.653
BF: brace failure; CP: chord plastification; CT: chord tearout; GB: global (overall) buckling; SW: chord sidewall failure *Governing prediction							Mean	1.441
							COV	0.334

Using the equivalent CHS approach, the mean and COV of the actual-to-predicted capacity ratios are 1.441 and 0.334, respectively. The predictions are conservative and result in a high scatter. The conservatism can be attributed to the inability of the CHS design equations to capture the actual failure mode. There are only two CHS connection design criteria, chord plastification and punching shear; the CHS connection design equations consistently predict that chord plastification will govern even though there are more types of failures apparent through experimentation. Overall, the CHS design equations generally cannot capture the actual failure modes.

6.1.2 EQUIVALENT RHS METHOD

The method termed the “equivalent RHS approach” is the second prediction method established here. It is based on the proposal by Zhao and Packer (2009). The approach converts the EHS chord into an equivalent RHS that has the same area and thickness as the original cross-section. (Recall in Chapter 3 that the simplified equivalent RHS, which neglected corner radii, was used to predict column and beam capacities). Like the equivalent CHS approach, to maintain the physical parameter $\beta = b_1/b_0$, the chord width (b_0) is kept constant, regardless of connection orientation type.

Table 6.3: Relevant CIDECT RHS connection design equations

Limit state	Equations		
Chord face plastification (for $\beta \leq 0.85$)	$N_1^* = Q_u Q_f \frac{f_{y0} t_0^2}{\sin \theta_1}$ $Q_u = \frac{2\eta}{(1 - \beta) \sin \theta_1} + \frac{4}{\sqrt{1 - \beta}}$		
	X Connection	T Connection	
	$Q_f = 1.0$	$Q_f = (1 - n)^{C_1}$ $n = \frac{N_0}{N_{pl,0}} + \frac{M_0}{M_{pl,0}}$ Compressive stress at connection: $C_1 = 0.6 - 0.5\beta$ Tensile stress at connection: $C_1 = 0.1$	
Local yielding of brace	$N_1^* = f_{y1} t_1 l_{b,eff}$ $l_{b,eff} = 2h_1 + 2b_e - 4t_1 \quad ; \quad b_e = \left(\frac{10}{b_0/t_0} \right) \left(\frac{f_{y0} t_0}{f_{y1} t_1} \right) b_1 \leq b_1$		
Chord punching shear (for $b_1 \leq b_0 - 2t_0$)	$N_1^* = \frac{0.58 f_{y0} t_0}{\sin \theta_1} l_{p,eff}$ $l_{p,eff} = \frac{2h_1}{\sin \theta_1} + 2b_{e,p} \quad ; \quad b_{e,p} = \left(\frac{10}{b_0/t_0} \right) b_1 \leq b_1$		
Chord shear (for X connections if $\cos \theta_1 > h_1/h_0$)	$N_1^* = \frac{0.58 f_{y0} A_v}{\sin \theta_1}$ $A_v = 2h_0 t_0$		
Chord sidewall failure (for $\beta = 1.0$)	$N_1^* = \frac{f_k t_0}{\sin \theta_1} b_w Q_f$ $b_w = \frac{2h_1}{\sin \theta_1} + 10t_0$		
	Brace tension	Brace compression	
	$f_k = f_{y0}$	X Connection	T Connection
		$f_k = 0.85 \chi f_{y0} \sin \theta_1$	$f_k = \chi f_{y0}$
	X Connection	T Connection	
$Q_f = 1.0$	$Q_f = (1 - n)^{C_1}$ $n = \frac{N_0}{N_{pl,0}} + \frac{M_0}{M_{pl,0}}$ Compressive stress at connection: $C_1 = 0.6 - 0.5\beta$		

By setting the EHS chord width, area and thickness the same in the equivalent RHS and neglecting corner radii, an equivalent RHS chord height ($h_{0,eq}$) is calculated. The measured cross-sectional area $A_{EHS} = 3044 \text{ mm}^2$, thickness $t = 5.94 \text{ mm}$, large dimension $H = 220.0 \text{ mm}$ and small dimension $B = 110.0 \text{ mm}$ are inputted into Equation (6.5) to determine the equivalent RHS chord width ($b_{0,eq}$) and chord height ($h_{0,eq}$).

$$\begin{cases} b_{0,eq} = b_0 \\ h_{0,eq} = \frac{A_{EHS} - 2b_0t + 4t^2}{2t} \end{cases} \quad (6.5)$$

For Type 1 and 2 orientations, the width of the chord is $H = 220\text{mm}$, so $h_{0,eq} = 48.1\text{mm}$. For Type 3 orientations, the width of the chord is $B = 110\text{mm}$, so $h_{0,eq} = 158.1\text{mm}$, see Figure 6.1. These values are then inputted as b_0 and h_0 into RHS connection design equations. The design equations found in Table 4.1 of CIDECT Design Guide 3 for Rectangular Hollow Section (RHS) Joints (Packer et al., 2009b) are used with experimentally determined material properties ($f_y = 402\text{MPa}$ and $E = 210,040\text{MPa}$) to make connection capacity predictions. The relevant RHS design equations are reproduced in Table 6.3, where N_1^* is the connection strength or the predicted connection capacity. The predictions are summarized in Table 6.4. The values with asterisks indicate the governing predictions. The “Actual/Predicted” heading is the ratio of the actual capacity to the governing prediction.

These predicted connection capacities are all less than the yield strength of the brace. For the T connections, these predicted connection capacities are also lower than the load that would have caused the plastic moment capacity of the chord to be reached (the procedure to calculate the plastic moment capacity was described in the previous section); however, the predicted connection capacities for T90-2C were not lower than the load that would have caused the shear capacity of the chord to be reached (the procedure to calculate the shear capacity was also described in the previous section). Even though chord shear failure was not observed, the shear capacity of the chord may govern member design; as such, it is included as one of the predicted capacities. For the Type 1 and 2 T connections, A_v is 1236.2mm^2 , and for the Type 3 T connection, A_v is 2543.0mm^2 . Note that the chord member shear failure mode is conceptually different to the connection chord shear failure mode in Table 6.3. The connection chord shear design criterion is not applicable to any of the X-connections in Table 6.4.

Figure 6.1: Equivalent RHS approach for EHS connections (all dimensions in mm)

Table 6.4: Connection capacity predictions using equivalent RHS approach

Test	Parameters			Actual Capacity (kN)	Experimental Failure Mode	Predicted Capacity (kN)					<u>Actual</u> Predicted
	θ_1 (°)	β	η			Chord Face Plastic.	Punching Shear	Brace Failure	Chord Side Wall Failure	EHS Chord Shear†	
X90-1T	90	0.5	1.0	187.9	CP,CT	137.0*	691.7	1135.8	n/a	n/a	1.372
X90-2T	90	1.0	0.5	574.5	BF; SW	n/a	n/a	752.3	667.2*	n/a	0.861
X90-3T	90	1.0	2.0	1188.8	BF	n/a	n/a	1277.6	1192.5*	n/a	0.997
X90-1C	90	0.5	1.0	150.5	CP	137.0*	691.7	1135.8	n/a	n/a	1.099
X90-2C	90	1.0	0.5	539.7	GB,SW	n/a	n/a	752.3	532.8*	n/a	1.013
X90-3C	90	1.0	2.0	555.1	SW	n/a	n/a	1277.6	571.7*	n/a	0.971
X45-1C	45	0.5	1.0	258.5	CP	226.9*	1335.1	1135.8	n/a	n/a	1.139
X45-2C	45	1.0	0.5	627.8	SW	n/a	n/a	752.3	705.0*	n/a	0.891
X45-3C	45	1.0	2.0	701.0	SW	n/a	n/a	1277.6	600.1*	n/a	1.168
T90-1C	90	0.5	1.0	216.5	CP	104.5*	691.7	1135.8	n/a	288.2	2.072
T90-2C	90	1.0	0.5	353.0	SW	n/a	n/a	752.3	426.5	288.2*	1.225
T90-3C	90	1.0	2.0	593.8	SW	n/a	n/a	1277.6	564.4*	592.9	1.052
BF: brace failure; CP: chord plastification; CT: chord tearout; GB: global (overall) buckling; SW: chord sidewall failure										Mean =	1.155
*Governing prediction †Chord member shear failure, not chord shear failure of connection										COV =	0.279

For the T connections, the Q_f factor is calculated in the same way as for the equivalent CHS approach. In order to calculate the chord side-wall failure limit state, the limiting stress of the material has an effect on the f_k term. For tension, $f_k = f_{y0} = 402\text{MPa}$, but for compression, χ = a reduction factor for column buckling, becomes relevant. The following describes the procedure used to calculate χ . The slenderness λ is calculated using the equation given in Table 4.1 of CIDECT Design Guide 3 (Packer et al., 2009b):

$$\lambda = 3.46 \left(\frac{h_0}{t_0} - 2 \right) \sqrt{\frac{1}{\sin \theta_1}} \quad (6.6)$$

The normalized slenderness $\bar{\lambda}$ is calculated by the following equations:

$$\bar{\lambda} = \frac{\lambda}{\lambda_1} \quad (6.7)$$

$$\lambda_1 = \pi \sqrt{\frac{E}{f_y}} \quad (6.8)$$

The normalized slenderness is entered into the Canadian S16-09 (CSA, 2009) column flexural buckling formula which uses Equation (6.9), where $n = 2.24$, the value used for hot-formed hollow sections. The use of the appropriate Eurocode 3 buckling curve should generate similar results.

$$\chi = (1 + \bar{\lambda}^{2n})^{-1/n} \quad (6.9)$$

The mean and COV of the actual-to-predicted capacity ratios using the equivalent RHS approach are 1.155 and 0.279, respectively. The equivalent RHS approach is, on average, slightly conservative with a high scatter. Looking at the individual predictions, the RHS approach does a very good job of capturing the actual failure modes.

6.1.3 COMPARISON

A comparison of the equivalent CHS and RHS approaches is shown in Table 6.5. Overall, the equivalent RHS approach does a better job of predicting load capacities in comparison to the equivalent CHS method. While the scatter of both approaches is large, the equivalent RHS approach predicts, on average, within 16% while the equivalent CHS approach predicts, on average, within 45%. The equivalent RHS approach also does a much better job of capturing the actual failure mode.

Table 6.5: Equivalent CHS approach vs. equivalent RHS approach

Test	Parameters		Failure Mode at Ultimate	Equivalent CHS Approach		Equivalent RHS Approach	
	θ_1 (°)	β		<u>Actual</u> Predicted	Predicted Failure	<u>Actual</u> Predicted	Predicted Failure
X90-1T	90	0.5	CP;CT	1.312	CP	1.372	CP
X90-2T	90	1.0	BF; SW	1.388	CP	0.861	SW
X90-3T	90	1.0	BF	2.873	CP	0.997	SW
X90-1C	90	0.5	CP	1.051	CP	1.099	CP
X90-2C	90	1.0	GB,SW	1.304	CP	1.013	SW
X90-3C	90	1.0	SW	1.342	CP	0.971	SW
X45-1C	45	0.5	CP	1.205	CP	1.139	CP
X45-2C	45	1.0	SW	1.073	CP	0.891	SW
X45-3C	45	1.0	SW	1.198	CP	1.168	SW
T90-1C	90	0.5	CP	1.397	CP	2.072	CP
T90-2C	90	1.0	SW	1.495	CP	1.225	CS
T90-3C	90	1.0	SW	1.653	CP	1.052	SW
Mean				1.441	Mean	1.155	
COV				0.334	COV	0.279	

BF: brace failure; CP: chord plastification; CS: EHS chord shear; CT: chord tearout; GB: global (overall) buckling; SW: chord sidewall failure

6.2 X AND T CONNECTIONS FROM THE NATIONAL UNIVERSITY OF SINGAPORE

A sister experimental programme was carried out at the National University of Singapore (NUS) (Packer et al., 2011). Their connections were comprised of EHS250 x 125 x 8 chords and EHS120 x 60 x 5 braces. A summary of the NUS experiments is given in Table 6.6. In total, the NUS performed 16 tests: four 90° X connections loaded in tension (Types 1, 2, 3 and 4), four 90° X connections loaded in compression (Types 1, 2, 3, and 4), two 45° X connections loaded in tension (Types 2 and 4), two 45° X connections loaded in compression, three 90° T connections loaded in compression (Types 2, 3, and 4)

and one 90° T connection loaded in tension (Type 4). Since NUS used a smaller brace than the chord, the joint orientation Type 4 became possible. In the NUS programme, Type 1 through 4 connections had $\beta = 0.24, 0.48, 0.48$ and 0.96 , respectively. NUS determined the f_y of the chord and brace to be 360MPa and 355MPa, respectively.

Table 6.6: Experimental programme from NUS (Packer et al., 2011)

Test	Chord		Brace		Parameters			Experimental Capacity		Failure Mode
	b_0	h_0	b_1	h_1	θ_1	$\beta =$	$\eta =$	UL	3%DL	
	(mm)	(mm)	(mm)	(mm)	(°)	b_1 / b_0	h_1 / b_0	(kN)	(kN)	
1. X90-1T	250	125	60	120	90	0.24	0.48	243.0	163.0*	CP,CT
2. X90-2T	250	125	120	60	90	0.48	0.24	277.0	185.0*	CP,CT
3. X90-3T	125	250	60	120	90	0.48	0.96	517.0	450.0*	CP,BF
4. X90-4T†	125	250	120	60	90	0.96	0.48	517.0	468.0*	BF
5. X90-1C	250	125	60	120	90	0.24	0.48	206.0	145.0*	CP
6. X90-2C	250	125	120	60	90	0.48	0.24	246.0	163.0*	CP
7. X90-3C	125	250	60	120	90	0.48	0.96	363.0*	362.0	CP,GB
8. X90-4C†	125	250	120	60	90	0.96	0.48	471.0*	-	BF,GB
9. X45-2C	250	125	120	60	90	0.48	0.24	230.0*	203.0	CP
10. X45-4C†	125	250	120	60	90	0.96	0.48	485.0*	-	BF,GB
11. X45-2T	250	125	120	60	45	0.48	0.24	375.0	329.0*	CT,BF
12. X45-4T†	125	250	120	60	45	0.96	0.48	556.0	468.0*	BF
13. T90-2C	250	125	120	60	90	0.48	0.24	162.0*	151.0	CP
14. T90-3C†	125	250	60	120	90	0.48	0.96	331.0*	330.0	CP,CB
15. T90-4C†	125	250	120	60	90	0.96	0.48	390.0*	389.0	CB
16. T90-4T†	125	250	120	60	90	0.96	0.48	455.0	408.0*	CB

BF: brace failure, CB: chord bending, CP: chord plastification, CT: chord tear out, GB: global (overall) buckling

*Governing capacity

†Excluded for connection capacity predictions

Out of the sixteen tests performed by NUS, seven experienced member failure before connection failure: X90-4T, X90-4C, X45-4C and X45-4T yielded in the brace, and T90-3C, T90-4C and T90-4T failed in bending. Since these tests would not help evaluate connection ultimate failure modes, they have been excluded from the connection capacity predictions.

6.2.1 EQUIVALENT CHS APPROACH

Applying the equivalent CHS approach described previously, the equivalent chord diameter $d_0 = 426.5\text{mm}$ and the equivalent brace diameters for Types 1 through 4 respectively are $d_1 = 102.4\text{mm}$, 204.7mm , 204.7mm and 409.9mm . According to NUS, the chord end supports of their T connections acted more as pinned-pinned, rather than fixed-fixed. As such, $P_{pl,0}$ for the NUS tests was determined by Equation (6.10), where $L_c = 1500\text{mm}$. A summary of the predictions using the equivalent CHS approach

is shown in Table 6.7. The values with the asterisk are the governing predictions. The “Actual/Predicted” column gives the ratio of the actual capacity to the governing prediction.

$$P_{pl,0} = \frac{4M_{pl,0}}{L_c - h_1} \quad (6.10)$$

After excluding the seven specimens that experienced member failure, the remaining predicted connection capacities are all less than the yield strength of the brace. For the T connections, these predicted connection capacities are also lower than the load that would have caused the shear capacity or the plastic moment capacity of the chord to be reached.

Table 6.7: Equivalent CHS approach to predict NUS experiments

Test	Parameters			Actual Capacity (kN)	Predicted Capacity (kN)		<u>Actual</u> Predicted
	θ_1 (°)	β	η		Chord Plastification	Punching Shear	
X90-1T	90	0.24	0.48	163.0	149.5*	543.9	1.090
X90-2T	90	0.48	0.24	185.0	223.6*	1087.9	0.827
X90-3T	90	0.48	0.96	450.0	223.6*	1087.9	2.013
X90-1C	90	0.24	0.48	145.0	149.5*	543.9	0.970
X90-2C	90	0.48	0.24	163.0	223.6*	1087.9	0.729
X90-3C	90	0.48	0.96	363.0	223.6*	1087.9	1.624
X45-2C	45	0.24	0.48	230.0	316.2*	1857.1	0.727
X45-2T	45	0.24	0.48	329.0	316.2*	1857.1	1.041
T90-2C	90	0.24	0.48	162.0	173.7*	1087.9	0.933
*Governing prediction						Mean	1.106
						COV	0.392

The equivalent CHS approach applied to the NUS experiments results in a mean and COV of actual-to-predicted capacity ratios of 1.106 and 0.392, respectively.

6.2.2 EQUIVALENT RHS APPROACH

Applying the equivalent RHS approach described previously, the equivalent chord dimensions for Type 1 and 2 connections were $b_{0,eq} = 250\text{mm}$ and $h_{0,eq} = 47.8\text{mm}$ and for Type 3 and 4 connections were $b_{0,eq} = 125\text{mm}$ and $h_{0,eq} = 172.8\text{mm}$. As before with the equivalent CHS approach applied to the NUS tests, the T connections were assumed to be pinned-pinned, so Equation (6.10) was used. The equivalent RHS approach predictions are shown in Table 6.8. The asterisks indicate the governing predictions. After excluding the seven specimens that experienced member failure, the remaining predicted capacities are all less than the yield strength of the brace. For the T connections, these predicted connection capacities are also lower than the load that would have caused the shear capacity or plastic moment capacity of the chord to be reached.

Table 6.8: Equivalent RHS approach to predict NUS experiments

Test	Parameters			Actual Capacity (kN)	Predicted Capacity (kN)			<u>Actual Predicted</u>
	θ_1 (°)	β	η		Chord Face Plastification	Punching Shear	Brace Failure	
X90-1T	90	0.24	0.48	163.0	138.2*	497.3	471.7	1.179
X90-2T	90	0.48	0.24	185.0	152.8*	399.1	334.5	1.211
X90-3T	90	0.48	0.96	450.0	218.2*	609.1	537.4	2.062
X90-1C	90	0.24	0.48	145.0	138.2*	497.3	471.7	1.049
X90-2C	90	0.48	0.24	163.0	152.8*	399.1	334.5	1.067
X90-3C	90	0.48	0.96	363.0	218.2*	609.1	537.4	1.663
X45-2C	45	0.24	0.48	230.0	228.9*	399.1	591.9	1.005
X45-2T	45	0.24	0.48	329.0	228.9*	399.1	591.9	1.437
T90-2C	80	0.24	0.48	162.0	83.2*	399.1	334.5	1.946
*Governing prediction							Mean	1.402
							COV	0.286

The equivalent RHS approach applied to the NUS experiments results in a mean and COV of actual-to-predicted capacity ratios of 1.402 and 0.286, respectively.

6.2.3 COMPARISON

The equivalent CHS and equivalent RHS approaches applied to the NUS tests are compared with each other in Table 6.9. While on average, the equivalent CHS approach predicts connection capacity better than the equivalent RHS approach (actual-to-predicted capacity ratio mean of 1.106 versus 1.402), the equivalent CHS approach also results in a larger scatter (COV of 0.392 versus 0.286). Both approaches capture the chord plastification failure mode, which is generally the experimental failure mode.

Table 6.9: Equivalent CHS approach vs. equivalent RHS approach for NUS tests

Test	Parameters		Failure Mode at Ultimate	Equivalent CHS Approach		Equivalent RHS Approach	
	θ_1 (°)	β		<u>Actual Predicted</u>	Predicted Failure	<u>Actual Predicted</u>	Predicted Failure
X90-1T	90	0.24	CP,CT	1.090	CP	1.179	CP
X90-2T	90	0.48	CP,CT	0.827	CP	1.211	CP
X90-3T	90	0.48	CP,BF	2.013	CP	2.062	CP
X90-1C	90	0.24	CP	0.970	CP	1.049	CP
X90-2C	90	0.48	CP	0.729	CP	1.067	CP
X90-3C	90	0.48	CP,GB	1.624	CP	1.663	CP
X45-2C	45	0.24	CP	0.727	CP	1.005	CP
X45-2T	45	0.24	CT, BF	1.041	CP	1.437	CP
T90-2C	80	0.24	CP	0.933	CP	1.946	CP
Mean				1.106	Mean	1.402	
COV				0.392	COV	0.286	

BF: brace failure; CP: chord plastification, CT: chord tear out, GB: global (overall) buckling

6.3 SUMMARY

Combining the 12 University of Toronto (UT) tests with the 9 NUS tests that underwent connection failure, Table 6.10 shows the mean and COV of the actual-to-predicted capacity ratios using the equivalent CHS and equivalent RHS approaches. The mean and COV are 1.296 and 0.481, respectively, for the equivalent CHS approach, and 1.261 and 0.370 for the equivalent RHS approach. Both approaches are conservative and result in high scatter. Table 6.10 also shows the number of correctly predicted failure modes: the equivalent RHS approach can capture the actual failure mode much better than the equivalent CHS approach.

Table 6.10: Summary of University of Toronto and NUS predictions

	Mean		COV		Correctly Predicted Failure Mode	
Equivalent CHS Approach	UT	NUS	UT	NUS	UT	NUS
	1.441	1.106	0.334	0.392	4 of 12 (33%)	9 of 9 (100%)
	Combined					
	1.296		0.481		13 of 21 (62%)	
Equivalent RHS Approach	UT	NUS	UT	NUS	UT	NUS
	1.155	1.402	0.279	0.286	10 of 12 (83%)	9 of 9 (100%)
	Combined					
	1.261		0.370		19 of 21 (90%)	

Based on these results, it is suggested that the equivalent RHS approach be investigated further for EHS connection design purposes. It would be ideal that additional EHS connection tests be performed which ultimately fail with different failure modes.

7.0 CONCLUSIONS AND RECOMMENDATIONS

EHS are the newest steel shape to have emerged in the industry. They have been incorporated in a variety of structures around the world without structural design guidelines and EHS are completely absent from Canadian codes and guides. A possible application of EHS is within truss-systems and, as such, a research programme was developed to study the behaviour of EHS-to-EHS welded T and X connections.

A comprehensive literature review has been presented herein to summarize the main topics of EHS research to date. EHS geometric property tables and compressive resistance tables were developed using the less conservative equivalent diameter ($D_{e,new}$) proposed by Ruiz-Teran and Gardner (2008), see Equations (2.25) and (2.26). These tables were developed to be potentially added to the next edition of the CISC Handbook of Steel Construction.

The equivalent RHS approach was investigated to determine its validity for the design of EHS columns and beams. The approach converts an EHS into an equivalent RHS by maintaining the same cross-sectional area and thickness. The equivalent RHS approach that was first proposed by Zhao and Packer (2009) was simplified for this thesis to neglect corner radii, and was modified to account for columns buckling and beams bending about the minor axis. Equation (3.13) is used to convert an EHS into an equivalent RHS for major axis buckling or bending, Equation (3.14) is used to convert an EHS into an equivalent RHS for minor axis buckling or bending, and Figure 3.2 shows these conversion methods. This simplified, modified equivalent RHS approach was found to work relatively well. The design procedure for EHS columns is summarized in Figure 3.3, and the design procedure for EHS beams is summarized in Figure 3.4. The equivalent RHS approach worked best when EHS properties (A , r_x , r_y , S and Z) were used, rather than using the equivalent RHS properties. Additional tests are required on more slender sections to determine the validity of the equivalent RHS approach for the design of Class 4 members.

An experimental programme was developed to study the behaviour of EHS T and X connections. Twelve connections were tested, varying in terms of connection type (T or X), connection angle (45° or 90°), orientation type (1, 2, or 3, see Figure 4.3) and brace loading (axial tension or compression). Load capacities, failure modes, chord deformation profiles and brace stress profiles were established. In this experimental programme it was found that Type 1 connections were the most deformable, had the lowest capacities in comparison to their Type 2 and 3 counterparts, and generally failed via chord plastification. Type 2 connections generally failed via chord sidewall failure. Type 3 connections were the least deformable, had the highest capacities in comparison to their Type 1 and 2 counterparts, and generally failed via chord sidewall failure. The 45° X connections had higher capacities in comparison to their 90°

counterparts. It is recommended for future study that strain gauges be placed on the chord top and bottom surfaces near the connection in order to measure stresses in the chord at the connection.

Two connection capacity prediction methods were implemented here based on existing connection design equations: the equivalent CHS approach (Equations 6.1 to 6.3 with equations found in Table 6.1) and the equivalent RHS approach (Equation 6.5 with equations found in Table 6.3). These approaches were used to predict the connection capacities of 12 EHS connection tests performed by this author as well as 9 tests performed at the National University of Singapore. Combining both sets of experimental data, the mean and COV of the equivalent CHS approach was 1.296 and 0.481, respectively, and the mean and COV of the equivalent RHS approach was 1.261 and 0.370, respectively. Both methods are conservative and result in a high scatter. However, based on the 21 tests performed, the equivalent CHS approach correctly predicted 13 failure modes, but the equivalent RHS approach correctly predicted 19 failure modes. Thus, the equivalent RHS approach can capture the actual failure mode much better than the equivalent CHS approach. It is therefore suggested that the equivalent RHS method be investigated further as a preliminary design method for all types of EHS connections.

For future work, it is recommended that finite element models be created and validated with these experimental results. It is suggested that these finite element models be used to perform extensive parametric analyses to further validate the use of the equivalent RHS approach, or to develop suitable design equations. With the many parameters under investigation, it is recommended that additional experiments also be performed to increase the database of EHS connection tests.

REFERENCES

- ASTM, 2007. Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing, ASTM A501-07, West Conshohocken, Pennsylvania, USA.
- Bortolotti, E., Jaspart, J. P., Pietrapertosa, C., Nicaud, G., Petitjean, P., and Grimault, J. P., 2003. Testing and modelling of welded joints between elliptical hollow sections. 10th International Symposium on Tubular Structures, Madrid, Spain, Proceedings, pp. 259-264.
- Bradford, M. A. and Roufeginejad, A., 2008. Elastic local buckling of thin-walled elliptical tubes containing elastic infill material. *Interaction and Multiscale Mechanics*, Volume 1, Number 1, pp. 143-156.
- Brienza, V. L., 2008. Concrete-filled elliptical hollow section stub columns in axial compression. M.Eng. thesis, University of Toronto, Toronto, Canada.
- Campione, G., Mindess, S., Scibilia, N., and Zingone, G., 2000. Strength of hollow circular steel sections filled with fibre-reinforced concrete. *Canadian Journal of Civil Engineering*, Volume 27, pp. 364-372.
- CEN, 2005. Eurocode 3 – Design of steel structures, EN1993: 2005, European Committee for Standardization, Brussels, Belgium.
- CEN, 2006a. Hot finished structural hollow sections of non-alloy and fine grain steels - Part 1: Tolerances, dimensions and sectional properties, EN10210-1, European Committee for Standardization, Brussels, Belgium.
- CEN, 2006b. Hot finished structural hollow sections of non-alloy and fine grain steels - Part 2: Tolerances, dimensions and sectional properties, EN10210-2, European Committee for Standardization, Brussels, Belgium.
- Chan, T. M. and Gardner, L., 2007. Compressive resistance of hot-rolled elliptical hollow sections. *Engineering Structures*, Volume 30, Number 2, pp. 522-532.
- Chan, T.M. and Gardner, L., 2008. Bending strength of hot-rolled elliptical hollow sections. *Journal of Constructional Steel Research*, Volume 64, Number 9, pp. 971-978.
- Chan, T. M. and Gardner, L., 2009. Flexural buckling of elliptical hollow section columns. *Journal of Structural Engineering*, Volume 135, Number 5, pp. 546-557.

- Chan, T.M., Abela, J.M. and Gardner, L., 2010. Biaxial bending and compression of elliptical hollow sections. 13th International Symposium on Tubular Structures, Hong Kong, China, Proceedings, pp. 303-311.
- Choo, Y.S., Liang, J.X., and Lim, L.V., 2003. Static strength of elliptical hollow section X-joint under brace compression. 10th International Symposium on Tubular Structures, Madrid, Spain, Proceedings, pp. 253-258.
- CISC, 2002. Torsional Section Properties of Steel Shapes. Canadian Institute of Steel Construction, Online. Available: <<http://www.cisc-icca.ca/files/technical/techdocs/updates/torsionprop.pdf>>
- CISC, 2010. Handbook of Steel Construction. 10th ed., Canadian Institution of Steel Construction, Toronto, Canada.
- Corus, 2005. Celsius Oval HSS sizes and design strengths.
- CSA, 2003. Welded Steel Construction (Metal Arc Welding). CAN/CSA-W59-03, Canadian Standards Association, Toronto, Canada.
- CSA, 2004. General Requirements for Rolled or Welded Structural Quality Steel/Structural Quality Steel, CAN/CSA G40.20-04/G40.21-04, Canadian Standards Association, Toronto, Canada.
- CSA, 2009. Design of Steel Structures, CSA-S16-09, Canadian Standards Association, Toronto, Canada.
- Gardner, L. 2008. The continuous strength method. Proceedings of the Institution of Civil Engineers, Structures and Buildings, Volume 161, Number SB3, pp. 127-133.
- Gardner, L. and Chan, T. M., 2007. Cross-section classification of elliptical hollow sections. Steel and Composite Structures, Volume 7, Number 3, pp. 185-200.
- Gardner, L., Chan, T.M., and Wadee, M. A., 2008. Shear response of elliptical hollow sections. Proceedings of the Institution of Civil Engineers, Structures and Buildings, Volume 161, Number SB6, pp. 301-309.
- Jamaluddin, N., Lam, D. And Ye, J., 2010. Experimental studies of elliptical concrete-filled tube columns. 13th International Symposium on Tubular Structures, Hong Kong, China, Proceedings, pp. 333-340.
- Johansson, M. and Gylltoft, K., 2002. Mechanical behavior of circular steel-concrete composite stub columns. Journal of Structural Engineering, Volume 130, Number 1, pp. 152-153.

- Lam, D. and Dai, X., 2010. Numerical modelling of concrete-filled stainless steel elliptical hollow sections. 13th International Symposium on Tubular Structures, Hong Kong, China, Proceedings, pp. 323-331.
- Lam, D., Gardner, L., and Burdett, M., 2008. Behaviour of axially loaded concrete filled stainless steel elliptical hollow sections, 2nd International Symposium on Innovative Design of Steel Structures, Hong Kong, Proceedings, pp. 493-500.
- Law, K.H. and Gardner, L., 2010. Instabilities in elliptical hollow section members. 13th International Symposium on Tubular Structures, Hong Kong, China, Proceedings, pp. 313-321.
- Martinez-Saucedo, G., 2007. Slotted end connections to hollow sections. Ph.D. thesis, University of Toronto, Toronto, Canada.
- Martinez-Saucedo, G., Packer J. A., and Zhao, X L., 2008. Static design of elliptical hollow section end connections. Proceedings of the Institution of Civil Engineers, Structures and Buildings, Volume 161, Number SB2, pp. 103-113.
- Nowzartash, F. and Mohareb, M. 2009. "Plastic interaction relations for elliptical hollow sections", Thin-Walled Structures, Volume 47, Numbers 6/7, pp. 681-691.
- Packer, J.A., 2008. Going elliptical. Modern Steel Construction, American Institute of Steel Construction, March 2008.
- Packer, J. A. and Henderson, J.E., 1997. Hollow Structural Section Connection and Trusses – A Design Guide. 2nd ed., Canadian Institute of Steel Construction, Toronto, Canada.
- Packer, J.A., Wardenier, J., Choo, Y.S. and Chiew, S.P., 2009a. Elliptical Steel Tubes. Steel News and Notes, Singapore Structural Steel Society, 25th Anniversary Issue, pp. 86-90.
- Packer, J.A., Wardenier, J., Zhao, X.L., van der Vegte, G.J., and Kurobane, Y., 2009b. Design Guide for Rectangular Hollow Section (RHS) Joints Under Predominantly Static Loading. 2nd ed., CIDECT, Geneva, Switzerland.
- Packer, J.A., Haque, T., Choo, Y.S., Fang, Y., van der Vegte, G.J., Wardenier, J., and Mustard, T., 2011. Axially Loaded T and X Joint of Elliptical Hollow Sections. CIDECT 3rd Interim Report, Report 5BW-6/11.

- Pietrapertosa, C. and Jaspart, J. P., 2003. Study of the behaviour of welded joints composed of elliptical hollow sections. 10th International Symposium on Tubular Structures, Madrid, Spain, Proceedings, pp. 601-608.
- Read Jones Christoffersen (RJC), 2008. What we do: structural engineering, Main event: Legends Centre.
- Ridley-Ellis, D.J., Owen, J.S., and Davies, G., 2002. Theoretical and measured torsional behaviour of RHS. 12th International Offshore and Polar Engineering Conference, Kitakyushu, Japan, Proceedings, pp. 41-47.
- Roufegarinejad, A., 2007. Chapter 8: Elastic buckling of thin-walled elliptical tubes containing an elastic infill. Stability of Thin-Walled Metal Tubes With Elastic Uni-Lateral Internal Restraint, Ph.D. thesis, University of New South Wales, Sydney, Australia.
- Ruiz-Teran, A. M. and Gardner, L., 2008. Elastic buckling of elliptical tubes. Thin Walled Structures, Volume 46, Number 11, pp. 1308-1318.
- SCI/BSCA, 2008. Steel building design: design data in accordance with Eurocodes and the UK national annexes. Steel Construction Institute and British Constructional Steelwork Association, Ascot/London, UK, SCI publication P363.
- Silvestre, N., 2008. Buckling behaviour of elliptical cylindrical shells and tubes under compression. International Journal of Solids and Structures, Volume 45, Number 16, pp. 4427-4447.
- Silvestre, N. and Gardner, L., 2010. Numerical study on the ultimate strength of elliptical stub columns. 13th International Symposium on Tubular Structures, Hong Kong, China, Proceedings, pp. 283-291.
- Silvestre, N. and Gardner, L., 2011. Elastic local post-buckling of elliptical tubes. Journal of Constructional Steel Research, Volume 67, Number 3, pp. 281-292.
- Tao, Z., Han, L., Wang, Z., 2005. Experimental behaviour of stiffened concrete-filled thin-walled hollow steel structural (HSS) stub columns. Journal of Constructional Steel Research, Volume 61, Number 7, pp. 962-983.
- Theofanous, M., Chan T.M., and Gardner, L., 2009a. Flexural behaviour of stainless steel oval hollow sections, Thin Walled Structures, Volume 47, pp. 776-787.
- Theofanous, M., Chan T.M., and Gardner, L., 2009b. Structural response of stainless steel oval hollow section compression members, Engineering Structures, Volume 31, Number 4, pp. 922-934.

- Wardenier, J., Kurobane, Y., Packer, J.A., van der Vegte, G.J., and Zhao, X.L., 2008. Design Guide for Circular Hollow Section (CHS) Joints Under Predominantly Static Loading. 2nd ed., CIDECT, Geneva, Switzerland.
- Wardenier, J., Packer, J.A., Haque, T., Choo, Y.S., Fang, Y., Shen, W., van der Vegte, G.J., and Mustard, T., 2010. Axially Loaded T and X Joints of Elliptical Hollow Sections. CIDECT 2nd Interim Report, Report 5BW-6/10.
- Willibald, S., Packer, J. A., and Martinez-Saucedo, G., 2006a. Behaviour of gusset plate connections to ends of round and elliptical hollow structural section members. Canadian Journal of Civil Engineering, Volume 33, Number 4, pp. 373-384.
- Willibald, S., Packer, J. A., Voth, A., and Zhao, X., 2006b. Through-plate joints to elliptical and circular hollow sections. 11th International Symposium on Tubular Structures, Quebec City, Canada, Proceedings, pp. 221-228.
- Yang, H., Lam, D., and Gardner, L., 2008. Testing and analysis of concrete-filled elliptical hollow sections. Engineering Structures, Volume 30, Number 12, pp. 3771-3781.
- Zhao, X.L., Lu, H., and Galteri, S., 2007. Tests of elliptical hollow sections filled with SCC (Self-Compacting Concrete). 5th International Conference on Advances in Steel Structures, Singapore, Proceedings, pp. 950-955.
- Zhao, X.L., and Packer, J.A., 2009. Tests and design of concrete-filled elliptical hollow section stub columns. Thin-Walled Structures, Volume 47, Numbers 6/7, pp. 617-628.
- Zhao, X.L., Kernot, A., Packer, J.A., and Haque, T., 2010. Slenderness limits for EHS and OHS subject to bending using the RHS approach. 13th International Symposium of Steel Structures, Hong Kong, China, Proceedings, pp. 293-301.
- Zhu, Y. and Wilkinson, T., 2007. Finite element analysis of structural steel elliptical hollow sections in compression, University of Sydney, Australia.
- Ziemian, R.D., editor, 2010. Guide to stability design criteria for metal structures, 6th ed., Appendix B3, Wiley, New York, USA.

Appendix 3A – EHS Dimension and Gross Property Table

Shown here is the dimension and gross-property table of all currently manufactured EHS. The EHS property equations that have been defined by EN10210 (CEN, 2006b) and found in Table 2.1 of this thesis have been used to generate the table in accordance with the CISC Handbook of Steel Construction format.

Table 3A.1: EHS dimension and gross property table

HOLLOW STRUCTURAL SECTIONS CSA G40.20 Elliptical

PROPERTIES AND DIMENSIONS

Designation*	Wall Thickness	Mass	Dead Load	Area	Axis X-X				Axis Y-Y				Torsion Inertia Constant	Torsion Modulus Constant	Surface Area
					I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y			
mm x mm x mm	mm	kg/m	kN/m	mm ²	10 ⁶ mm ⁴	10 ³ mm ³	mm	10 ³ mm ³	10 ⁶ mm ⁴	10 ³ mm ³	mm	10 ³ mm ³	10 ³ mm ⁴	10 ³ mm ³	m ² /m
EHS 500x250															
x16	16.00	142	1.39	18 000	437	1 750	156	2 460	145	1 160	89.8	1 500	437 000	2 590	1.21
x12.5	12.50	112	1.10	14 200	350	1 400	157	1 960	118	943	91.0	1 200	353 000	2 110	1.21
x10	10.00	90.0	0.883	11 500	285	1 140	158	1 580	96.8	775	91.9	976	290 000	1 740	1.21
EHS 480x240															
x14	14.00	119	1.17	15 200	342	1 420	150	2 000	114	953	86.7	1 220	343 000	2 120	1.16
x12	12.00	103	1.01	13 100	298	1 240	151	1 730	100	835	87.4	1 060	300 000	1 870	1.16
x10	10.00	86.3	0.847	11 000	252	1 050	151	1 460	85.3	711	88.1	897	255 000	1 590	1.16
EHS 400x200															
x14	14.00	98.7	0.969	12 600	194	968	124	1 370	64.2	642	71.4	832	193 000	1 420	0.969
x12.5	12.50	88.6	0.869	11 300	175	877	125	1 230	58.4	584	71.9	753	176 000	1 300	0.969
x12	12.00	85.2	0.836	10 900	169	845	125	1 190	56.5	565	72.1	726	170 000	1 260	0.969
x10	10.00	71.5	0.702	9 110	143	717	125	1 000	48.3	483	72.8	615	145 000	1 080	0.969
x8	8.00	57.6	0.565	7 340	117	584	126	811	39.7	397	73.5	500	119 000	890	0.969
EHS 320x160															
x14	14.00	78.0	0.765	9 940	96.0	600	98.3	855	31.4	392	56.2	517	94 800	863	0.776
x12	12.00	67.5	0.662	8 600	84.2	526	99.0	745	27.8	347	56.9	453	83 800	769	0.776
x10	10.00	56.7	0.556	7 230	71.8	449	99.7	631	23.9	299	57.5	385	71 900	665	0.776
x8	8.00	45.8	0.449	5 830	58.8	367	100	513	19.8	247	58.2	315	59 300	553	0.776
EHS 300x150															
x16	16.00	82.5	0.809	10 500	87.3	582	91.2	837	28.1	374	51.7	503	85 300	818	0.727
x12.5	12.50	65.5	0.643	8 340	71.2	475	92.4	674	23.3	311	52.9	409	70 500	686	0.727
x10	10.00	53.0	0.520	6 750	58.7	391	93.2	551	19.5	260	53.7	336	58 700	577	0.727
x8	8.00	42.8	0.420	5 450	48.1	321	93.9	449	16.2	215	54.4	275	48 500	481	0.727
EHS 250x125															
x12.5	12.50	53.9	0.529	6 870	40.0	320	76.3	458	12.9	207	43.4	276	39 200	453	0.606
x12	12.00	51.9	0.510	6 620	38.6	309	76.4	442	12.5	201	43.5	267	38 000	440	0.606
x10	10.00	43.8	0.429	5 580	33.2	265	77.1	376	10.9	174	44.2	228	32 900	385	0.606
x8	8.00	35.4	0.347	4 510	27.3	219	77.8	307	9.09	145	44.9	188	27 300	323	0.606
x6.3	6.30	28.2	0.276	3 590	22.1	176	78.4	246	7.42	119	45.5	151	22 200	265	0.606
x6	6.00	26.9	0.263	3 420	21.1	169	78.5	235	7.11	114	45.6	145	21 300	255	0.606
EHS 220x110															
x10	10.00	38.2	0.375	4 870	22.2	201	67.4	287	7.22	131	38.5	174	21 800	288	0.533
x8	8.00	31.0	0.304	3 950	18.3	167	68.1	235	6.06	110	39.2	143	18 200	244	0.533
x6	6.00	23.5	0.231	3 000	14.2	129	68.8	181	4.76	86.6	39.9	111	14 300	193	0.533
EHS 200x100															
x12.5	12.50	42.4	0.416	5 400	19.5	195	60.2	284	6.19	124	33.9	169	18 900	269	0.485
x10	10.00	34.5	0.339	4 400	16.4	164	61.0	235	5.29	106	34.7	141	16 000	232	0.485
x8	8.00	28.0	0.275	3 570	13.6	136	61.7	193	4.46	89.3	35.4	117	13 500	197	0.485
x6.3	6.30	22.3	0.219	2 840	11.0	110	62.3	155	3.68	73.5	36.0	94.7	11 000	163	0.485

* Depth x Width x Thickness

Table 3A.1: EHS dimension and gross property table

HOLLOW STRUCTURAL SECTIONS CSA G40.20 Elliptical

PROPERTIES AND DIMENSIONS

Designation*	Wall Thickness	Mass	Dead Load	Area	Axis X-X				Axis Y-Y				Torsion Inertia Constant	Torsion Modulus Constant	Surface Area
					I_x	S_x	r_x	Z_x	I_y	S_y	r_y	Z_y	J	C_t	A_s
mm x mm x mm	mm	kg/m	kN/m	mm ²	10 ⁶ mm ⁴	10 ³ mm ³	mm	10 ³ mm ³	10 ⁶ mm ⁴	10 ³ mm ³	mm	10 ³ mm ³	10 ³ mm ⁴	10 ³ mm ³	m ² /m
EHS 180x90															
x10	10.00	30.8	0.302	3 930	11.7	130	54.6	187	3.75	83.3	30.9	112	11 400	182	0.436
x8	8.00	25.1	0.246	3 190	9.74	108	55.2	154	3.18	70.6	31.6	93.3	9 610	155	0.436
x6	6.00	19.1	0.187	2 430	7.61	84.6	55.9	119	2.53	56.2	32.2	72.6	7 600	125	0.436
EHS 150x75															
x10	10.00	25.3	0.248	3 220	6.49	86.6	44.9	126	2.04	54.5	25.2	75.1	6 250	118	0.364
x8	8.00	20.6	0.202	2 630	5.46	72.8	45.6	105	1.76	46.8	25.9	62.9	5 330	102	0.364
x6.3	6.30	16.5	0.162	2 100	4.48	59.7	46.2	84.9	1.47	39.1	26.4	51.5	4 430	86.3	0.364
x6	6.00	15.8	0.155	2 010	4.30	57.3	46.3	81.3	1.41	37.7	26.5	49.3	4 260	83.2	0.364
x5	5.00	13.3	0.130	1 690	3.67	48.9	46.6	68.9	1.22	32.5	26.9	42.0	3 670	72.2	0.364
x4	4.00	10.7	0.105	1 360	3.01	40.1	47.0	56.1	1.01	26.9	27.2	34.4	3 030	60.1	0.364
EHS 120x60															
x8	8.00	16.2	0.159	2 060	2.66	44.3	35.9	64.7	0.837	27.9	20.2	38.4	2 560	60.4	0.291
x6	6.00	12.4	0.122	1 580	2.12	35.4	36.6	50.7	0.686	22.9	20.8	30.5	2 080	50.1	0.291
x5	5.00	10.5	0.103	1 340	1.82	30.4	36.9	43.2	0.597	19.9	21.2	26.2	1 800	43.9	0.291
x4	4.00	8.48	0.083	1 080	1.50	25.1	37.3	35.3	0.499	16.6	21.5	21.5	1 500	36.9	0.291
x3.2	3.20	6.85	0.067	873	1.23	20.5	37.6	28.7	0.414	13.8	21.8	17.6	1 240	30.8	0.291

* Depth x Width x Thickness

Appendix 3B – EHS Compressive Resistance Table

The following table gives the factored axial compressive resistance, C_r , calculated in accordance with Clause 13.3 of CSA S16-09 (CSA, 2009) for hot-formed hollow sections using $n = 2.24$. Axial compressive resistances are determined for all EHS sizes listed in Appendix 3A, for various compression member effective lengths, and presented in a format compatible with the CISC Handbook of Steel Construction Section 4. For a description of the “Properties and Design Data” contained at the bottom of each page in this table, please refer to Section 3.2 of this thesis.

Table 3B.1: EHS compressive resistance table

G40.21 350W
CLASS H
 $\phi = 0.90$

ELLIPTICAL HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)

Designation (mm x mm x mm)		EHS 500 x 250			EHS 480 x 240		
		16.0	12.5**	10.0**	14.0	12.0**	10.0**
Mass (kg/m)		142	112	90.0	119	103	86.3
Effective length (KL) in millimetres with respect to the least radius of gyration	0	5 670	4 203	2 675	4 788	3 877	2 673
	400	5 670	4 203	2 675	4 788	3 877	2 673
	800	5 670	4 203	2 675	4 788	3 877	2 673
	1 200	5 669	4 202	2 674	4 787	3 876	2 673
	1 600	5 666	4 200	2 674	4 784	3 874	2 672
	2 000	5 659	4 196	2 672	4 777	3 870	2 670
	2 400	5 645	4 188	2 669	4 764	3 861	2 666
	2 800	5 621	4 173	2 664	4 740	3 844	2 659
	3 200	5 583	4 149	2 656	4 702	3 818	2 648
	3 600	5 524	4 113	2 643	4 645	3 779	2 631
	4 000	5 442	4 063	2 624	4 565	3 724	2 606
	4 400	5 332	3 994	2 598	4 459	3 650	2 572
	4 800	5 192	3 905	2 564	4 326	3 556	2 528
	5 200	5 023	3 796	2 520	4 167	3 441	2 473
	5 600	4 826	3 668	2 467	3 985	3 308	2 407
	6 000	4 607	3 522	2 403	3 785	3 159	2 330
	6 400	4 371	3 362	2 330	3 573	3 000	2 243
	6 800	4 126	3 193	2 249	3 357	2 833	2 148
	7 200	3 879	3 019	2 161	3 142	2 665	2 048
	7 600	3 634	2 843	2 067	2 932	2 498	1 944
	8 000	3 397	2 670	1 971	2 731	2 336	1 840
	8 400	3 171	2 503	1 873	2 542	2 181	1 736
	8 800	2 958	2 344	1 775	2 365	2 036	1 635
	9 200	2 759	2 193	1 679	2 200	1 899	1 538
	9 600	2 574	2 051	1 586	2 049	1 772	1 445
	10 000	2 403	1 919	1 497	1 910	1 655	1 358
	10 400	2 246	1 797	1 412	1 782	1 546	1 276
	10 800	2 101	1 684	1 332	1 665	1 447	1 199
	11 200	1 968	1 580	1 256	1 558	1 355	1 128
	11 600	1 846	1 483	1 185	1 460	1 271	1 061
	12 000	1 733	1 395	1 119	1 371	1 194	1 000
PROPERTIES AND DESIGN DATA							
Area (mm ²)		18 000	14 200	11 500	15 200	13 100	11 000
Z_x (10 ³ mm ³)		2 460	1 960	1 580	2 000	1 730	1 460
S_x (10 ³ mm ³)		1 750	1 400	1 140	1 420	1 240	1 050
r_x (mm)		156	157	158	150	151	151
Z_y (10 ³ mm ³)		1 500	1 200	976	1 220	1 060	897
S_y (10 ³ mm ³)		1 160	943	775	953	835	711
r_y (mm)		89.8	91.0	91.9	86.7	87.4	88.1
r_x / r_y		1.74	1.73	1.72	1.73	1.73	1.71
M_{rx} (kN·m)		551	441	359	447	391	331
M_{ry} (kN·m)		365	297	244	300	263	224
IMPERIAL SIZE AND MASS							
Mass (lb./ft.)		95.4	75.3	60.5	80.0	69.2	58.0
Thickness (in.)		0.630	0.492	0.394	0.551	0.472	0.394
Size (in.)		20 x 10			19 x 9		

 ** Class 4: C_r calculated according to S16-01 Clause 13.3.3(b)

Table 3B.1: EHS compressive resistance table

ELLIPTICAL HOLLOW SECTIONS
**Factored Axial Compressive
Resistances, C_r (kN)**

G40.21 350W
CLASS H
 $\phi = 0.90$

Designation (mm x mm x mm)		EHS 400 x 200					EHS 320 x 160				
		14.0	12.5	12.0	10.0**	8.0**		14.0	12.0	10.0	8.0**
Mass (kg/m)		98.7	88.6	85.2	71.5	57.6		78.0	67.5	56.7	45.8
Effective length (KL) in millimetres with respect to the least radius of gyration	0	3 969	3 560	3 434	2 696	1 707		3 131	2 709	2 277	1 725
	400	3 969	3 559	3 433	2 696	1 707		3 131	2 709	2 277	1 725
	800	3 969	3 559	3 433	2 696	1 707		3 130	2 708	2 277	1 725
	1 200	3 967	3 558	3 432	2 695	1 707		3 126	2 705	2 274	1 723
	1 600	3 961	3 553	3 427	2 692	1 706		3 113	2 694	2 266	1 718
	2 000	3 948	3 541	3 416	2 685	1 703		3 083	2 670	2 246	1 706
	2 400	3 922	3 518	3 394	2 670	1 698		3 026	2 622	2 208	1 682
	2 800	3 876	3 479	3 357	2 646	1 689		2 931	2 545	2 145	1 641
	3 200	3 805	3 417	3 297	2 606	1 675		2 795	2 432	2 053	1 582
	3 600	3 703	3 328	3 213	2 549	1 653		2 620	2 285	1 934	1 502
	4 000	3 568	3 210	3 100	2 472	1 623		2 417	2 113	1 793	1 405
	4 400	3 402	3 063	2 960	2 375	1 583		2 200	1 929	1 640	1 297
	4 800	3 210	2 894	2 797	2 260	1 534		1 984	1 743	1 485	1 184
	5 200	3 000	2 708	2 619	2 130	1 475		1 779	1 567	1 337	1 074
	5 600	2 783	2 515	2 433	1 993	1 408		1 592	1 404	1 200	969
	6 000	2 567	2 322	2 247	1 852	1 334		1 425	1 258	1 076	873
	6 400	2 358	2 135	2 067	1 713	1 257		1 277	1 129	966	787
	6 800	2 162	1 959	1 898	1 580	1 179		1 148	1 015	869	710
	7 200	1 981	1 796	1 740	1 454	1 102		1 035	916	785	643
	7 600	1 815	1 646	1 595	1 338	1 027		936	829	711	583
	8 000	1 665	1 511	1 464	1 231	955		850	753	646	530
	8 400	1 529	1 388	1 346	1 134	888		775	686	589	484
	8 800	1 407	1 278	1 239	1 046	825		709	628	539	443
	9 200	1 298	1 179	1 143	967	767		650	576	494	407
	9 600	1 200	1 090	1 057	895	714		599	530	455	375
	10 000	1 111	1 010	979	830	665		553	490	420	346
	10 400	1 032	938	909	771	620		512	454	389	321
	10 800	960	872	846	718	579		475	421	361	298
	11 200	895	814	789	670	542		442	392	336	278
	11 600	836	760	737	626	508					259
	12 000	783	712	690	587	476					
PROPERTIES AND DESIGN DATA											
Area (mm ²)		12 600	11 300	10 900	9 110	7 340		9 940	8 600	7 230	5 830
Z_x (10 ³ mm ³)		1 370	1 230	1 190	1 000	811		855	745	631	513
S_x (10 ³ mm ³)		968	877	845	717	584		600	526	449	367
r_x (mm)		124	125	125	125	126		98.3	99.0	99.7	100
Z_y (10 ³ mm ³)		832	753	726	615	500		517	453	385	315
S_y (10 ³ mm ³)		642	584	565	483	397		392	347	299	247
r_y (mm)		71.4	71.9	72.1	72.8	73.5		56.2	56.9	57.5	58.2
r_x / r_y		1.74	1.74	1.73	1.72	1.71		1.75	1.74	1.73	1.72
M_{rx} (kN·m)		432	276	266	226	184		269	235	141	116
M_{ry} (kN·m)		262	184	178	152	125		163	143	94.2	77.8
IMPERIAL SIZE AND MASS											
Mass (lb./ft.)		66.3	59.5	57.3	48.0	38.7		52.4	45.4	38.1	30.8
Thickness (in.)		0.551	0.492	0.472	0.394	0.315		0.551	0.472	0.394	0.315
Size (in.)		16 x 8						13 x 6			

 ** Class 4: C_r calculated according to S16-01 Clause 13.3.3(b)

Table 3B.1: EHS compressive resistance table

G40.21 350W
CLASS H
 $\phi = 0.90$

ELLIPTICAL HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)

Designation (mm x mm x mm)		EHS 300 x 150				EHS 250 x 125					
		16.0	12.5	10.0	8.0	12.5	12.0	10.0	8.0	6.3**	6.0**
Mass (kg/m)		82.5	65.5	53.0	42.8	53.9	51.9	43.8	35.4	28.2	26.9
Effective length (KL) in millimetres with respect to the least radius of gyration	0	3 308	2 627	2 126	1 717	2 164	2 085	1 758	1 421	1 072	968
	400	3 307	2 627	2 126	1 717	2 164	2 085	1 758	1 421	1 072	968
	800	3 306	2 626	2 126	1 716	2 162	2 084	1 756	1 420	1 071	968
	1 200	3 300	2 622	2 122	1 714	2 153	2 075	1 750	1 414	1 068	965
	1 600	3 280	2 607	2 111	1 705	2 125	2 048	1 729	1 399	1 058	957
	2 000	3 235	2 575	2 086	1 686	2 064	1 990	1 682	1 364	1 035	939
	2 400	3 149	2 513	2 040	1 650	1 956	1 887	1 600	1 301	995	906
	2 800	3 014	2 414	1 964	1 592	1 802	1 739	1 481	1 209	933	856
	3 200	2 828	2 276	1 857	1 509	1 617	1 562	1 336	1 095	855	789
	3 600	2 601	2 105	1 723	1 405	1 422	1 374	1 181	972	766	713
	4 000	2 354	1 915	1 573	1 286	1 237	1 195	1 031	851	677	634
	4 400	2 105	1 721	1 418	1 163	1 071	1 036	895	741	594	559
	4 800	1 870	1 535	1 268	1 042	928	898	777	645	519	491
	5 200	1 657	1 364	1 130	930	807	781	677	563	455	431
	5 600	1 469	1 212	1 005	829	706	683	593	493	400	380
	6 000	1 305	1 079	896	740	621	601	522	434	353	336
	6 400	1 163	963	801	661	549	532	462	385	313	298
	6 800	1 041	863	718	593	489	473	412	343	279	266
	7 200	936	776	646	534	438	424	369	307	250	239
	7 600	845	701	584	483	394	381	332	276	225	215
	8 000	766	636	529	438	356	345	300	250	204	195
	8 400	697	579	482	399	324	313	273	227	185	177
	8 800	637	529	441	365			249	207	169	162
	9 200	584	485	404	335						
	9 600	537	446	372	308						
	10 000	495	412	343	284						
	10 400		381	318	263						
	10 800				244						
	11 200										
	11 600										
	12 000										
PROPERTIES AND DESIGN DATA											
Area (mm ²)		10 500	8 340	6 750	5 450	6 870	6 620	5 580	4 510	3 590	3 420
Z _x (10 ³ mm ³)		837	674	551	449	458	442	376	307	246	235
S _x (10 ³ mm ³)		582	475	391	321	320	309	265	219	176	169
r _x (mm)		91.2	92.4	93.2	93.9	76.3	76.4	77.1	77.8	78.4	78.5
Z _y (10 ³ mm ³)		503	409	336	275	276	267	228	188	151	145
S _y (10 ³ mm ³)		374	311	260	215	207	201	174	145	119	114
r _y (mm)		51.7	52.9	53.7	54.4	43.4	43.5	44.2	44.9	45.5	45.6
r _x / r _y		1.76	1.75	1.74	1.73	1.76	1.76	1.74	1.73	1.72	1.72
M _{rx} (kN·m)		264	212	174	101	144	139	118	69.0	55.4	53.2
M _{ry} (kN·m)		158	129	106	67.7	86.9	84.1	71.8	45.7	37.5	35.9
IMPERIAL SIZE AND MASS											
Mass (lb./ft.)		55.4	44.0	35.6	28.8	36.2	34.9	29.4	23.8	18.9	18.1
Thickness (in.)		0.630	0.492	0.394	0.315	0.492	0.472	0.394	0.315	0.248	0.236
Size (in.)		12 x 6				10 x 5					

 ** Class 4: C_r calculated according to S16-01 Clause 13.3.3(b)

Table 3B.1: EHS compressive resistance table

ELLIPTICAL HOLLOW SECTIONS
**Factored Axial Compressive
Resistances, C_r (kN)**

G40.21 350W
CLASS H
 $\phi = 0.90$

Designation (mm x mm x mm)		EHS 220 x 110			EHS 200 x 100			
		10.0	8.0	6.0	12.5	10.0	8.0	6.3
Mass (kg/m)		38.2	31.0	23.5	42.4	34.5	28.0	22.3
Effective length (KL) in millimetres with respect to the least radius of gyration	0	1 534	1 244	945	1 701	1 386	1 125	895
	400	1 534	1 244	945	1 701	1 386	1 124	895
	800	1 532	1 243	944	1 697	1 383	1 122	893
	1 200	1 521	1 234	938	1 676	1 367	1 111	884
	1 600	1 488	1 210	921	1 614	1 322	1 076	859
	2 000	1 418	1 157	883	1 493	1 230	1 007	807
	2 400	1 306	1 071	821	1 319	1 096	904	728
	2 800	1 161	957	738	1 124	941	781	633
	3 200	1 006	834	646	939	791	660	538
	3 600	859	715	556	781	661	554	452
	4 000	730	609	476	652	553	465	381
	4 400	622	520	407	549	467	393	322
	4 800	533	446	350	467	397	335	275
	5 200	460	386	303	401	341	288	236
	5 600	400	336	264	347	296	250	205
	6 000	350	294	231	303	259	218	179
	6 400	309	260	204	267	228	192	158
	6 800	275	231	181		202	171	140
	7 200	246	206	162				
	7 600	221	186	146				
	8 000							
	8 400							
	8 800							
	9 200							
	9 600							
	10 000							
	10 400							
	10 800							
	11 200							
	11 600							
	12 000							
PROPERTIES AND DESIGN DATA								
Area (mm ²)		4 870	3 950	3 000	5 400	4 400	3 570	2 840
Z_x (10 ³ mm ³)		287	235	181	284	235	193	155
S_x (10 ³ mm ³)		201	167	129	195	164	136	110
r_x (mm)		67.4	68.1	68.8	60.2	61.0	61.7	62.3
Z_y (10 ³ mm ³)		174	143	111	169	141	117	94.7
S_y (10 ³ mm ³)		131	110	866	124	106	89.3	73.5
r_y (mm)		38.5	39.2	39.9	33.9	34.7	35.4	36.0
r_x / r_y		1.75	1.74	1.72	1.78	1.76	1.74	1.73
M_{rx} (kN·m)		90.4	74.0	40.6	89.5	74.0	60.8	34.7
M_{ry} (kN·m)		54.8	45.0	272.8	53.2	44.4	36.9	23.2
IMPERIAL SIZE AND MASS								
Mass (lb./ft.)		25.7	20.8	15.8	28.5	23.2	18.8	15.0
Thickness (in.)		0.394	0.315	0.236	0.492	0.394	0.315	0.248
Size (in.)		9 x 4			8 x 4			

 ** Class 4: C_r calculated according to S16-01 Clause 13.3.3(b)

Table 3B.1: EHS compressive resistance table

G40.21 350W
CLASS H
 $\phi = 0.90$

ELLIPTICAL HOLLOW SECTIONS
Factored Axial Compressive Resistances, C_r (kN)

Designation (mm x mm x mm)		EHS 180 x 90			EHS 150 x 75					
		10.0	8.0	6.0	10.0	8.0	6.3	6.0**	5.0	4.0
Mass (kg/m)		30.8	25.1	19.1	25.3	20.6	16.5	15.8	13.3	10.7
Effective length (KL) in millimetres with respect to the least radius of gyration	0	1 238	1 005	765	1 014	828	662	633	532	428
	400	1 238	1 005	765	1 014	828	661	633	532	428
	800	1 233	1 001	763	1 005	822	656	628	529	426
	1 200	1 210	984	751	960	789	633	606	511	412
	1 600	1 146	936	717	854	709	572	549	465	377
	2 000	1 029	847	653	702	591	481	462	395	321
	2 400	877	729	566	553	470	385	371	318	260
	2 800	725	607	474	432	369	304	293	252	207
	3 200	592	498	391	341	292	242	233	201	165
	3 600	485	410	322	274	235	195	188	162	133
	4 000	401	339	268	224	193	159	154	133	109
	4 400	336	284	224	186	160	133	128	111	91
	4 800	284	241	190	157	135	112	108	93	77
	5 200	244	207	163			96	92	80	66
	5 600	211	179	141						
	6 000	184	156	123						
	6 400			109						
	6 800									
	7 200									
	7 600									
	8 000									
	8 400									
	8 800									
	9 200									
	9 600									
	10 000									
	10 400									
	10 800									
	11 200									
	11 600									
	12 000									
PROPERTIES AND DESIGN DATA										
Area (mm ²)		3 930	3 190	2 430	3 220	2 630	2 100	2 010	1 690	1 360
Z_x (10 ³ mm ³)		187	154	119	126	105	84.9	81.3	68.9	56.1
S_x (10 ³ mm ³)		130	108	84.6	86.6	72.8	59.7	57.3	48.9	40.1
r_x (mm)		54.6	55.2	55.9	44.9	45.6	46.2	46.3	46.6	47.0
Z_y (10 ³ mm ³)		112	93.3	72.6	75.1	62.9	51.5	49.3	42.0	34.4
S_y (10 ³ mm ³)		83.3	70.6	56.2	54.5	46.8	39.1	37.7	32.5	26.9
r_y (mm)		30.9	31.6	32.2	25.2	25.9	26.4	26.5	26.9	27.2
r_x / r_y		1.77	1.75	1.74	1.78	1.76	1.75	1.75	1.73	1.73
M_{rx} (kN·m)		58.9	48.5	37.5	39.7	33.1	26.7	25.6	21.7	12.6
M_{ry} (kN·m)		35.3	29.4	22.9	23.7	19.8	16.2	15.5	13.2	8.47
IMPERIAL SIZE AND MASS										
Mass (lb./ft.)		20.7	16.9	12.8	17.0	13.8	11.1	10.6	8.9	7.2
Thickness (in.)		0.394	0.315	0.236	0.394	0.315	0.248	0.236	0.197	0.157
Size (in.)		7 x 4			6 x 3					

 ** Class 4: C_r calculated according to S16-01 Clause 13.3.3(b)

Table 3B.1: EHS compressive resistance table

ELLIPTICAL HOLLOW SECTIONS
**Factored Axial Compressive
Resistances, C_r (kN)**

G40.21 350W
CLASS H
 $\phi = 0.90$

Designation (mm x mm x mm)		EHS 120 x 60					
Mass (kg/m)			8.0	6.0	5.0	4.0	3.2
Effective length (KL) in millimetres with respect to the least radius of gyration	0		649	498	422	340	275
	400		648	497	422	340	275
	800		633	487	414	334	270
	1 200		568	442	378	306	249
	1 600		450	356	308	252	206
	2 000		333	267	233	192	158
	2 400		246	198	174	144	119
	2 800		185	150	132	109	90
	3 200		144	117	103	85	70
	3 600		114	93	82	68	56
	4 000		93	75	66	55	46
	4 400						
	4 800						
	5 200						
	5 600						
	6 000						
	6 400						
	6 800						
	7 200						
	7 600						
	8 000						
	8 400						
	8 800						
	9 200						
	9 600						
	10 000						
	10 400						
	10 800						
	11 200						
	11 600						
	12 000						
PROPERTIES AND DESIGN DATA							
Area (mm ²)			2 060	1 580	1 340	1 080	873
Z_x (10 ³ mm ³)			64.7	50.7	43.2	35.3	28.7
S_x (10 ³ mm ³)			44.3	35.4	30.4	25.1	20.5
r_x (mm)			35.9	36.6	36.9	37.3	37.6
Z_y (10 ³ mm ³)			38.4	30.5	26.2	21.5	17.6
S_y (10 ³ mm ³)			27.9	22.9	19.9	16.6	13.8
r_y (mm)			20.2	20.8	21.2	21.5	21.8
r_x / r_y			1.78	1.76	1.74	1.73	1.72
M_{rx} (kN·m)			20	16	14	11	6
M_{ry} (kN·m)			12	10	8	7	4
IMPERIAL SIZE AND MASS							
Mass (lb./ft.)			10.89	8.33	7.06	5.70	4.60
Thickness (in.)			0.315	0.236	0.197	0.157	0.126
Size (in.)			5 x 2				

Appendix 4A – Tensile Coupon Data Sheets

The following are the tensile coupon data sheets.

Table 4A.1: Tensile coupon 1 data sheet

UNIVERSITY OF TORONTO

 DEPARTMENT OF CIVIL ENGINEERING
STRUCTURES LABORATORY

TENSILE COUPON TEST DATA

PROJECT : ELLIPTICAL HOLLOW SECTION CONNECTIONS	
OPERATOR :	DATE: MARCH 8, 2010
SUPERVISOR: J.A. PACKER	TIME: 4:00

SPECIMEN NO :	1			
GAUGE LENGTH :	49.97 mm			
" AFTER RUPTURE:	mm			
MACHINED WIDTH :	12.56 mm			
	12.56 mm			
	12.59 mm			
AVERAGE/MIN :	12.57 mm			
THICKNESS :	5.86 mm			
	5.84 mm			
	5.87 mm			
AVERAGE/MIN :	5.86 mm	LENGTH L :	460 mm	REMARKS: TC closest to FHS weld "flat side"
SECTION AREA :	73.66 mm ²	GRIP P :	184 & 185 mm	
		WIDTH W :	27.97 mm	

TEST DATA :	1	
YIELD LOAD Ty :	kN	
STATIC YIELD LOAD Tst1 :	28.41676 kN	
	Tst2 : 28.26543 kN	
ULTIMATE LOAD Tu :	38.49365 kN	
MAX. STRAIN AT RUPTURE :	0.183398 με	
YOUNGS MODULUS :	211350 MPa	
STRAIN HARDENING MODULUS:	MPa	
YIELD STRESS σy :	408.9 MPa	
STATIC YIELD STRESS σst:	MPa	
ULTIMATE STRESS σu :	522.6 MPa	

EQUIPMENT	MTS 1000 kN
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CONTROL:	RANGE 1	STRAIN/STROKE/LOAD	RATE: 0.004 mm/s
	2	STRAIN/STROKE/LOAD	RATE: up to 0.02 mm/s

SUPPLEMENTARY DATA:

Table 4A.2: Tensile coupon 2 data sheet

UNIVERSITY OF TORONTO
DEPARTMENT OF CIVIL ENGINEERING
STRUCTURES LABORATORY

TENSILE COUPON TEST DATA

PROJECT : ELLIPTICAL HOLLOW SECTION CONNECTIONS	
OPERATOR :	DATE: MARCH 9, 2010
SUPERVISOR: J.A. PACKER	TIME: 1:30 PM

SPECIMEN NO : 2 GAUGE LENGTH : 50.02 mm " AFTER RUPTURE: 69.87 mm MACHINED WIDTH : 12.83 mm 12.87 mm 12.90 mm AVERAGE/MIN : 12.87 mm THICKNESS : 5.86 mm 5.82 mm 5.92 mm AVERAGE/MIN : 5.87 mm SECTION AREA : 75.55 mm ²	
LENGTH L : 460 mm GRIP P : 182 & 185 mm WIDTH W : 26.14 mm	REMARKS: farthest from weld TC "flat side"

TEST DATA : 2 YIELD LOAD T_y : kN STATIC YIELD LOAD T_{st1} : 27.93644 kN T_{st2} : 27.28385 kN ULTIMATE LOAD T_u : 38.98483 kN MAX. STRAIN AT RUPTURE : 19.55108 $\mu\epsilon$ YOUNGS MODULUS : 201200 MPa STRAIN HARDENING MODULUS: MPa YIELD STRESS σ_y : 399.5 MPa STATIC YIELD STRESS σ_{st} : MPa ULTIMATE STRESS σ_u : 516.01 MPa	
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EQUIPMENT MTS-1000KN

CONTROL: RANGE 1	STRAIN/STROKE/LOAD	RATE: 0.004 mm/s
2	STRAIN/STROKE/LOAD	RATE: up to 0.02 mm/s

SUPPLEMENTARY DATA:

Table 4A.3: Tensile coupon 3 data sheet

UNIVERSITY OF TORONTO
DEPARTMENT OF CIVIL ENGINEERING
STRUCTURES LABORATORY

TENSILE COUPON TEST DATA

PROJECT : ELLIPTICAL HOLLOW SECTION CONNECTIONS.	
OPERATOR :	DATE: MARCH 9, 2010
SUPERVISOR: J.A. PACKER	TIME: 2:30 PM

SPECIMEN NO : 3 GAUGE LENGTH : 49.80 mm " AFTER RUPTURE: mm MACHINED WIDTH : 12.85 mm 12.80 mm 12.83 mm AVERAGE/MIN : 12.83 mm THICKNESS : 6.01 mm 5.97 mm 6.33 mm AVERAGE/MIN : 6.10 mm SECTION AREA : 78.26 mm ²	
LENGTH L : 460 mm GRIP P : 181 & 182 mm WIDTH W : 27.08 mm	REMARKS: "corner" TC

TEST DATA : 3 YIELD LOAD T_y : kN STATIC YIELD LOAD T_{st1} : kN T_{st2} : kN ULTIMATE LOAD T_u : kN MAX. STRAIN AT RUPTURE : $\mu\epsilon$ YOUNGS MODULUS : 217260 MPa STRAIN HARDENING MODULUS: MPa YIELD STRESS σ_y : 239.9 MPa STATIC YIELD STRESS σ_{st} : MPa ULTIMATE STRESS σ_u : 511.5 MPa	
--	--

EQUIPMENT

CONTROL: RANGE 1	STRAIN/STROKE/LOAD	RATE:
2	STRAIN/STROKE/LOAD	RATE:

SUPPLEMENTARY DATA:

Appendix 4B – Fabrication Drawings

The following are the EHS-to-EHS connection fabrication drawings provided to Walters Inc.

Figure 4A.1: Fabrication drawing for T90-1C

Figure 4A.2: Fabrication drawing for T90-2C

NOTES:

- 1. EHS-to-EHS welds to be PJP + Fillet with Effective Throat = 8 mm minimum.
- 2. Use E490 electrodes.
- 3. Use ASTM 1" A325 bolts.
- 4. All dimensions in mm.

UNIVERSITY OF TORONTO	
DWN BY: Tarana Haque	
APPROVED BY: J. A. Packer	
DATE: May 6, 2009	REV:
DWG NAME: EHS T-CONNECTION TYPE 3 - COMPRESSION	

Figure 4A.3: Fabrication drawing for T90-3C

Figure 4B.4: Fabrication drawing for X90-1C

NOTES:

1. EHS-to-EHS welds to be PJPG + Fillet with Effective Throat = 8 mm minimum.
2. Use E490 electrodes.
3. Use ASTM 1" A325 bolts.
4. All dimensions in mm.

UNIVERSITY OF TORONTO	
DWN BY: Tarana Haque	
APPROVED BY: J. A. Packer	
DATE: May 6, 2009	REV:
DWG NAME:	
EHS X-CONNECTION (90°)	
TYPE 3 - COMPRESSION	

Figure 4B.6: Fabrication drawing for X90-3C

Figure 4B.7: Fabrication drawing for X90-1T

Figure 4B.8: Fabrication Drawing for X90-2T

1. EHS-to-EHS welds to be PJPJG + Fillet with Effective Throat = 8 mm minimum.
2. Use E490 electrodes.
3. Use ASTM 1" A325 bolts.
4. All dimensions in mm.

Figure 4B.9: Fabrication drawing for X90-3T

NOTES:

1. EHS-to-EHS welds to be PJPG + Fillet with Effective Throat = 8 mm minimum.
2. Use E490 electrodes.
3. Use ASTM 1" A325 bolts.
4. All dimensions in mm.

UNIVERSITY OF TORONTO	
DWN BY: Tarana Haque	
APPROVED BY: J. A. Packer	
DATE: May 6, 2009	REV:
DWG NAME:	
EHS X-CONNECTION (45°)	
TYPE 1 - COMPRESSION	

Figure 4B.10: Fabrication drawing for X45-1C

NOTES:

1. EHS-to-EHS welds to be PJP + Fillet with Effective Throat = 8 mm minimum.
2. Use E490 electrodes.
3. Use ASTM 1" A325 bolts.
4. All dimensions in mm.

UNIVERSITY OF TORONTO	
DWN BY: Tarana Haque	
APPROVED BY: J. A. Packer	
DATE: May 6, 2009	REV:
DWG NAME:	
EHS X-CONNECTION (45°)	
TYPE 2 - COMPRESSION	

Figure 4B.11: Fabrication drawing for X45-2C

Figure 4B.12: Fabrication drawing for X45-3C

Appendix 4C – Walters Inc. Fabrication Drawings

The following are the modified fabrication drawings made by Walters Inc., including welding details.

Figure 4C.2: Walters Inc. fabrication drawing for T90-2C

Figure 4C.3: Walters Inc. fabrication drawing for T90-3C

Figure 4C.4: Walters Inc. fabrication drawing for x90-1T and x90-1C

Figure 4C.5: Walters Inc. fabrication drawing for X45-1C

Figure 4C.7: Walters Inc. fabrication drawing for X90-3T and X90-3C

Figure 4C.8: Walters Inc. fabrication drawing for X45-2C

Figure 4C.9: Walters Inc. fabrication drawing for X45-3C

Appendix 4D – LVDT Instrumentation

The following show the LVDT placements for the 90° connections.

Figure 4D.1: LVDT locations for T90-1C

UNIVERSITY OF TORONTO	
DRAWN BY: Tarana Haque	
DATE: June 8, 2009	REV:
DWG NAME: Instrumentation - T90 2C	

Figure 4D.2: LVDT locations for T90-2C

Figure 4D.3: LVDT locations for T90-3C

UNIVERSITY OF TORONTO	
DWN BY: Tarana Haque	
DATE: June 8, 2009	REV:
DWG NAME: Instrumentation - X90-1C/T	

Figure 4D.4: LVDT locations for X90-1C and X90-1T

Figure 4D.5: LVDT locations for X90-2C and X90-2T

Appendix 4E – LED Locations

The following are the LED locations and numeration for each of the EHS tests. LEDs were not used for X90-1T, X90-3T or X90-1C. For the remaining tests, the camera's x,y,z coordinates were taken and then geometrically transformed to correspond with the axes of the specimen, an x', y', z' coordinate axis system. Typically, the y-axis of the camera was approximately horizontal and the x-axis of the camera was approximately vertical. The y' -axis of the specimen is in the direction of loading and the x' -axis is in the direction of the chord length for the 90° connections. The LEDs were first rotated such that the x-axis of the camera and the y' -axis of the specimen corresponded. The z-coordinates of the LEDs that should be on the x', y' plane were then compared and the appropriate transformations about the x' -axis and y' -axis were made.

Figure 4E.1: LED locations for X90-2T

LED 16, 17 and 35 were placed on the LVDT mount, LED 19 was placed on the LVDT-CC plate, and the remaining LEDs were placed on the EHS. The y' -axis of the specimen and the x-axis of the camera varied by 3.2°. The coordinates were rotated about the y' -axis by 0.76° (by checking the z-coordinates of LED 8 with 34) and then about the x' -axis by 2.7° (by checking the z' -coordinates of LED 18 and 20 with 22).

Figure 4E.2: LED locations for X90-2C

LED 7, 15 and 27 were placed on the LVDT mount, LED 21 was placed on the LVDT-CC plate, and the remaining LEDs were placed on the EHS. The y' -axis of the specimen and the x -axis of the camera varied by 2.5° . The coordinates were rotated about the y' -axis by 4.0° (by checking the z -coordinates of LED 8 with 26) and then about the x' -axis by 0.83° (by checking the z' -coordinates of LED 20 and 22 with 19).

Figure 4E.3: LED locations for X90-3C

LED 24, 4, 5 and 6 were placed on the LVDT mount, LED 10 was placed on the LVDT-CC plate, and the remaining LEDs were placed on the EHS. The y' -axis of the specimen and the x -axis of the camera varied by 0.70° . The coordinates were rotated about the y' -axis by 7.5° (by checking the z -coordinates of LED 29 with 18) and then about the x' -axis by 0.83° (by checking the z' -coordinates of LED 34 and 13 with 12).

Figure 4E.4: LED locations for X45-1C

The y' -axis of the specimen and the x -axis of the camera varied by 0.97° . The coordinates were rotated about the x' -axis by 6.9° (by checking the z -coordinates of LED 16 with 19). Additional rotation angles were negligible.

Figure 4E.5: LED locations for X45-2C

The y' -axis of the specimen and the x -axis of the camera varied by 3.76° . The coordinates were rotated about the x' -axis by 4.1° (by checking the z -coordinates of LED 18 with 21). Additional rotations angles were negligible.

Figure 4E.6: LED locations for X45-3C

The y' -axis of the specimen and the x -axis of the camera varied by 1.03° . The coordinates were rotated about the x' -axis by 7.4° (by checking the z -coordinates of LED 17 with 22). Additional rotations angles were negligible.

Figure 4E.7: LED locations for T90-1C

LED 34, 33, 13, 14 and 15 were placed on the LVDT mount, LED 32, 31, 11 and 12 were placed on the end-frame, and the remaining LEDs were placed on the EHS. The y' -axis of the specimen and the x -axis corresponded. The coordinates were rotated about the y' -axis by 4.0° (by checking the z -coordinates of LED 30 with 9) and then about the x' -axis by 0.70° (by checking the z' -coordinates of LED 29 with 30 and 10 with 9).

Figure 4E.8: LED locations for T90-2C

LED 33, 10, and 13 were placed on the LVDT mount, LED 15 was placed on the end-frame, and the remaining LEDs were placed on the EHS. The y' -axis of the specimen and the x -axis of the camera varied by 1.0° . The coordinates were rotated about the x' -axis by 2.5° (by checking the z -coordinates of LED 14 with 5) and then about the y' -axis by 0.70° (by checking the z' -coordinates of LED 21 with 5).

Figure 4E.9: LED locations for T90-3C

LED 10, 31 and 30 were placed on the LVDT mount, LED 13 and 14 were placed on the end-frame, and the remaining LEDs were placed on the EHS. The y' -axis of the specimen and the x -axis of the camera varied by 1.0° . The coordinates were rotated about the y' -axis by 4.0° (by checking the z -coordinates of LED 1 with 21) and then about the y' -axis by 0.50° (by checking the z' -coordinates of LED 1).

Appendix 4F – T Connection End Frame

The following is the fabrication drawing for the end frame that supports the chord ends of the T connections. The chord ends are bolted to the T connection end frame and the end frame is then bolted to a double-web I-beam.

Figure 4F.1: T connection end frame

Appendix 5A – Specimen Measurements

The measured chord and brace dimensions of the 12 EHS-to-EHS welded connection specimens are given here. All dimensions are in mm and all dimensions were measured to the closest mm.

Figure 5A.1: Specimen measurements of X90-1T

Figure 5A.2: Specimen measurements of X90-2T

Figure 5A.3: Specimen measurements of X90-3T

Figure 5A.4: Specimen measurements of X90-1C

Figure 5A.5: Specimen measurements of X90-2C
 Note: The chord member was not entirely perpendicular to the braces.
 The chord is off the orthogonal by 2.54°

Figure 5A.6: Specimen measurements of X90-3C

Figure 5A.7: Specimen measurements of X45-1C

Figure 5A.8: Specimen measurements of X45-2C

Figure 5A.9: Specimen measurements of X45-3C

Figure 5A.10: Specimen measurements of T90-1C

Figure 5A.11: Specimen measurements of T90-2C

Figure 5A.12: Specimen measurements of T90-3C

Appendix 5B – Weld Measurements

Weld measurements of the 12 EHS-to-EHS connections are given here. All weld measurements are reported in mm. Due to the unique geometry of the EHS connections, the conventional method of reporting weld sizes at various locations around the connection could not be done. As a substitute, the following weld measurements were taken. Figure 1 below shows the schematics of the weld measurement locations. Figure 1(a) shows the weld measurement locations for Type 1 connections and Figure 1(b) shows the weld measurement locations for Type 2 or 3 connections. For every brace, there are two crown locations. At the crowns, the size of the weld parallel to the direction of loading was taken and was denoted as w_1 . There were seven strain gauges placed around half the brace perimeter; the size of the weld perpendicular to the direction of loading at each of these strain gauge locations was taken and was denoted as w_2 . The last measurement taken was the surface length of the weld at the seven strain gauge locations and was denoted as L . Note: the surface length measurement did not account for the convexity or concavity of the weld.

Regarding the applicable measurements for 90° connections: 1) the w_1 measurement can only be taken at the crowns (i.e. 2 per brace); 2) the w_2 measurement for type 2 and 3 connections can only be taken at the crowns (i.e. 2 per brace); and 3) not all measurements were taken since there are redundancies due to symmetry, both about the brace centreline (left vs. right) and about the chord centreline (top vs. bottom). Regarding the applicable measurements for 45° connections: i) the w_1 measurement is never applicable; ii) the w_2 measurement is only applicable for the Type 1 connection; and iii) not all measurements were taken since there are redundancies due to symmetry.

The left and right directionality was assigned once the specimen was placed in the MTS load frame; the front is the side facing east (side facing the K610 camera). The top and bottom directionality was assigned, as well, once the specimen was placed in the MTS load frame. Measurements were taken at locations corresponding to the strain gauges locations. The following 12 tables show the measurements taken from each of the 12 specimens.

(a) Type 1 connections

(b) Type 2 and 3 connections

Figure 5B.1: Location of weld measurements

Table 5B.1: Weld measurements for X90-1T

Top-Left				Top-Centre			Top-Right			
Location	w1	w2	L	Location	w2	L	Location	w1	w2	L
SG-LC	11	11		SG-CC	12	18	SG-R1		13	
SG-L2		13	16				SG-R2		14	
SG-L1		12.5	18				LG-RC	11	14	
Bottom-Left				Bottom-Centre			Bottom-Right			
Location	w1	w2	L	Location	w2	L	Location	w1	w2	L
SG-LC	10	13		SG-CC	11		SG-R1		12	
SG-L2		15					SG-R2		12	
SG-L1		12.5					LG-RC	9	13	

Table 5B.2: Weld measurements for X90-2T

Top-Left				Top-Centre			Top-Right			
Location	w1	w2	L	Location	w2	L	Location	w1	w2	L
SG-LC	10	10		SG-CC			SG-R1			
SG-L2							SG-R2			
SG-L1							LG-RC	10.5	10	
Bottom-Left				Bottom-Centre			Bottom-Right			
Location	w1	w2	L	Location	w2	L	Location	w1	w2	L
SG-LC	10	10.5	14.5	SG-CC		14	SG-R1			15.5
SG-L2			14.5				SG-R2			15
SG-L1			14.5				LG-RC	10	11	15.5

Table 5B.3: Weld measurements for X90-3T

Top-Left				Top-Centre			Top-Right			
Location	w1	w2	L	Location	w2	L	Location	w1	w2	L
SG-LC	10	10		SG-CC			SG-R1			
SG-L2							SG-R2			
SG-L1							LG-RC	10	10	
Bottom-Left				Bottom-Centre			Bottom-Right			
Location	w1	w2	L	Location	w2	L	Location	w1	w2	L
SG-LC	10	10	15	SG-CC		19.5	SG-R1			18.5
SG-L2			17.5				SG-R2			18
SG-L1			18.5				LG-RC	10	10	15

Table 5B.4: Weld measurements for X90-1C

Top-Left				Top-Centre			Top-Right			
Location	w1	w2	L	Location	w2	L	Location	w1	w2	L
SG-LC	10	13		SG-CC	11		SG-R1		11	
SG-L2		12					SG-R2		13	
SG-L1		12					LG-RC	10	13	
Bottom-Left				Bottom-Centre			Bottom-Right			
Location	w1	w2	L	Location	w2	L	Location	w1	w2	L
SG-LC	10	12		SG-CC	12	18	SG-R1		13	18.5
SG-L2		12					SG-R2		13	18
SG-L1		12					LG-RC	10	14	18

Table 5B.5: Weld measurements for X90-2C

Top-Left				Top-Centre			Top-Right			
Location	w1	w2	L	Location	w2	L	Location	w1	w2	L
SG-LC	11	11.5	16.5	SG-CC		17	SG-R1			
SG-L2			16				SG-R2			
SG-L1			15.5				LG-RC			
Bottom-Left				Bottom-Centre			Bottom-Right			
Location	w1	w2	L	Location	w2	L	Location	w1	w2	L
SG-LC	10.5	12.5	17	SG-CC		16	SG-R1			
SG-L2			17				SG-R2			
SG-L1			17.5				LG-RC			

Table 5B.6: Weld measurements for X90-3C

Top-Left				Top-Centre			Top-Right			
Location	w1	w2	L	Location	w2	L	Location	w1	w2	L
SG-LC	10	10	14.5	SG-CC		14	SG-R1			13
SG-L2			12.5				SG-R2			11
SG-L1			13				LG-RC	10	10	12.5
Bottom-Left				Bottom-Centre			Bottom-Right			
Location	w1	w2	L	Location	w2	L	Location	w1	w2	L
SG-LC	10	10	14	SG-CC		18.5	SG-R1			
SG-L2							SG-R2			
SG-L1							LG-RC	9	9.5	15

Table 5B.7: Weld measurements for X45-1C

Top-Left				Top-Centre			Top-Right			
Location	w1	w2	L	Location	w2	L	Location	w1	w2	L
SG-LC				SG-CC			SG-R1			
SG-L2							SG-R2			
SG-L1							LG-RC			
Bottom-Left				Bottom-Centre			Bottom-Right (narrow)			
Location	w1	w2	L	Location	w2	L	Location	w1	w2	L
SG-LC		8	21.5	SG-CC	10	17	SG-R1		10	15.5
SG-L2		9	19				SG-R2		10	15
SG-L1		10	18				LG-RC		-	15

Table 5B.8: Weld measurements for X45-2C

Top-Left				Top-Centre			Top-Right			
Location	w1	w2	L	Location	w2	L	Location	w1	w2	L
SG-LC				SG-CC			SG-R1			
SG-L2							SG-R2			
SG-L1							LG-RC			
Bottom-Left (narrow)				Bottom-Centre			Bottom-Right			
Location	w1	w2	L	Location	w2	L	Location	w1	w2	L
SG-LC			11.5	SG-CC		19	SG-R1			13.5
SG-L2			11.5				SG-R2			12.5
SG-L1			12				LG-RC			12

Table 5B.9: Weld measurements for X45-3C

Top-Left				Top-Centre			Top-Right (narrow)			
Location	w1	w2	L	Location	w2	L	Location	w1	w2	L
SG-LC			15.5	SG-CC		20	SG-R1			18
SG-L2			19				SG-R2			16
SG-L1			19				LG-RC			12
Bottom-Left				Bottom-Centre			Bottom-Right			
Location	w1	w2	L	Location	w2	L	Location	w1	w2	L
SG-LC				SG-CC			SG-R1			
SG-L2							SG-R2			
SG-L1							LG-RC			

Table 5B.10: Weld measurements for T90-1C

Top-Left				Top-Centre			Top-Right			
Location	w1	w2	L	Location	w2	L	Location	w1	w2	L
SG-LC	10	15		SG-CC	14.5		SG-R1		14	
SG-L2		15					SG-R2		14	
SG-L1		14.5					LG-RC	10	14	

Table 5B.11: Weld measurements for T90-2C

Top-Left				Top-Centre			Top-Right			
Location	w1	w2	L	Location	w2	L	Location	w1	w2	L
SG-LC	11	12	16.5	SG-CC		17	SG-R1			17
SG-L2			17				SG-R2			17.5
SG-L1			17				LG-RC	12	13	17.5

Table 5B.12: Weld measurements for T90-3C

Top-Left				Top-Centre			Top-Right			
Location	w1	w2	L	Location	w2	L	Location	w1	w2	L
SG-LC	9	9	13.5	SG-CC		19	SG-R1			18
SG-L2			15				SG-R2			16
SG-L1			17				LG-RC	9	9	15

Appendix 5C – Connection Displacement Measurement

The schematics of how connection displacements (Δ_1) were measured for each of the 12 tests are given here. For the 90° connections, “LVDT Δ_1 Measurement” indicates Δ_1 was measured by LVDT displacement, and “LED Δ_1 Measurement” indicates Δ_1 was measured by the relative displacement between LEDs. For the 45° connections, “LED Displacement Measurement” is the relative displacement measured between LEDs, but is not Δ_1 . This measurement is multiplied by the cosine(angle) to determine Δ_1 , a displacement perpendicular to the chord. This direction is indicated by “ Δ_1 Measurement”.

b) X90-2T

a) X90-1T

c) X90-3T

Figure 5C.1: Connection displacement measurements for tension-tested X connections at 90°

4. X90-1C

5. X90-2C

Due to global buckling, additional steps were taken to get the connection displacement perpendicular to the chord, see below.

6. X90-3C

Figure 5C.2: Connection displacement measurements for compression-tested X connections at 90°

8. X45-2C

7. X45-1C

9. X45-3C

Figure 5C.3: Connection displacement measurements for compression-tested X connections at 45°

10. T90-1C

12. T90-3C

11. T90-2C

Figure 5C.4: Connection displacement measurements for compression-tested T connections at 90°

X90-2C experienced global buckling. Typically for the 90° connections, the LVDT displacement or the vertical displacement between LEDs can be taken. Since X90-2C globally buckled, one cannot simply take the relative vertical displacement.

After geometric transformations of the LED positions such that the one axis corresponded to the brace vertical axis and the other axis corresponded to the chord horizontal axis, the average LED20 and LED 22 was used relative to LED15 to determine connection displacement. The relative vertical displacement between LED20&22 average to LED15 is denoted as dx . The relative horizontal displacement between LED20&22 average to LED15 is denoted as dy . The total relative displacement (R) is:

$$R = \sqrt{dx^2 + dy^2}$$

As noted in Appendix 5A, the chord of X90-2C was off orthogonal by 2.64°. The figure below shows the the direction of the relative displacement (R) and the direction of the connection displacement (Δ_1) and the direction of the chord centreline.

Figure 5C.5: Additional rotation of X90-2C

Based on the above figure: $\alpha = 2.54 - r$ and $r = \tan^{-1} \frac{dy}{dx}$

Therefore: $\Delta_1 = R \cos \alpha$

Appendix 5D – Experimental Summaries

The following are the experimental summaries of the 12 EHS-to-EHS welded connection tests. The experimental summaries include the following:

- Chord and brace cross-sectional dimensions
- Chord and brace lengths
- Material properties
- Geometrical parameters: β , η , γ , τ , and θ_1
- Schematic of connection displacement (Δ_1) measurement
- Connection load-displacement curve
- Ultimate load, load at the 3% deformation limit, and brace yield load
- Picture of specimen at ultimate load
- General notes about experiment

Table 5D.1: Experimental summary of X90-1T

Chord nominal dimensions: 220 x 110 x 6						Steel grade: EN10210 S355J2H		Test no. X90-1T			
Brace(s) nominal dimensions: 220 x 110 x 6											
Chord (measured geometrical dimensions)						Connection configuration (all dimensions in mm)
b_0 (mm)	h_0 (mm)	t_0 (mm)	A_0 (mm ²)	f_{y0} (MPa)	f_{u0} (MPa)						
220.0	110.0	5.94	3044	402	517						
Brace(s) (measured geometrical dimensions)						Measurements (all dimensions in mm)
b_1 (mm)	h_1 (mm)	t_1 (mm)	A_1 (mm ²)	f_{y1} (MPa)	f_{u1} (MPa)						
110.0	220.0	5.94	3044	402	517						
Geometrical parameters											
$\beta = b_1/b_0$	$\eta = h_1/b_0$	$\gamma = b_0/2t_0$	$\tau = t_1/t_0$	θ_1 (deg)							
0.5	1.0	18.52	1.0	90							
Connection capacity (kN)			Brace yield (kN)								
$N_{1u} = 339.1$		$N_{1(3\%)} = 187.9$		$N_{1,yield} = 1223.7$							
Connection load-displacement curve:

Failure mode: Chord plastification until chord tear-out						Remarks: Initial loading rate was 0.25mm/min and was increased only when yielding was apparent. Chord began to “circularize” shortly after load vs. connection deflection curve became nonlinear. Significant yielding occurred at chord centreline, and load-carrying capacity increased until brace tore the chord at the base of the weld. 3% deformation limit governs capacity of the connection.

Table 5D.2: Experimental summary of X90-2T

Chord nominal dimensions: 220 x 110 x 6						Steel grade: EN10210 S355J2H		Test no. X90-2T			
Brace(s) nominal dimensions: 220 x 110 x 6											
Chord (measured geometrical dimensions)						Connection configuration (all dimensions in mm)					
b_0 (mm)	h_0 (mm)	t_0 (mm)	A_0 (mm ²)	f_{y0} (MPa)	f_{u0} (MPa)
220.0	110.0	5.94	3044	402	517						
Brace(s) (measured geometrical dimensions)											
b_1 (mm)	h_1 (mm)	t_1 (mm)	A_1 (mm ²)	f_{y1} (MPa)	f_{u1} (MPa)						
220.0	110.0	5.94	3044	402	517						
Geometrical parameters											
$\beta = b_1/b_0$	$\eta = h_1/b_0$	$\gamma = b_0/2t_0$	$\tau = t_1/t_0$	θ_1 (deg)							
1.0	0.5	18.52	1.0	90							
Connection capacity (kN)			Brace yield (kN)								
$N_{1u} = 596.8$			$N_{1(3\%)} = 574.5$			$N_{1,yield} = 1223.7$					
Connection load-displacement curve:						Measurements (all dimensions in mm)

Failure mode: Brace cracked above weld on opposite side. Side wall failure caused load drop.						Remarks: Initial loading rate was 0.25mm/min and was increased only when yielding was apparent. Chord began to “circularize” shortly after load vs. connection deflection curve became nonlinear. First failure mode was a crack formed in the top brace above the weld. The second failure mode was the tearing of the chord at its side centreline causing a sudden drop in load-carrying capacity. First test performed with both LVDTs and optical scanner. Confirmed both systems produced similar results. 3% deformation limit governs capacity of the connection.					

Table 5D.3: Experimental summary of X90-3T

Chord nominal dimensions: 220 x 110 x 6						Steel grade: EN10210 S355J2H		Test no. X90-3T			
Brace(s) nominal dimensions: 220 x 110 x 6											
Chord (measured geometrical dimensions)						Connection configuration (all dimensions in mm)					
b_0 (mm)	h_0 (mm)	t_0 (mm)	A_0 (mm ²)	f_{y0} (MPa)	f_{u0} (MPa)
110.0	220.0	5.94	3044	402	517						
Brace(s) (measured geometrical dimensions)											
b_1 (mm)	h_1 (mm)	t_1 (mm)	A_1 (mm ²)	f_{y1} (MPa)	f_{u1} (MPa)						
110.0	220.0	5.94	3044	402	517						
Geometrical parameters											
$\beta = b_1/b_0$	$\eta = h_1/b_0$	$\gamma = b_0/2t_0$	$\tau = t_1/t_0$	θ_1 (deg)							
1.0	2.0	9.26	1.0	90							
Connection capacity (kN)			Brace yield (kN)								
$N_{1u} = 1557.0$			$N_{1(3\%)} = 1188.8$			$N_{1,yield} = 1223.7$					
Connection load-displacement curve:						Measurements (all dimensions in mm)

Failure mode: Top branch rupture						Remarks:
Initial loading rate was 0.25mm/min and was increased only when yielding was apparent.					
						No significant chord deformation and/or rotation, so LVDT measurements were used for the connection load-displacement.					
						3% deformation limit governs capacity of the connection.					

Table 5D.4: Experimental summary of X90-1C

Chord nominal dimensions: 220 x 110 x 6						Steel grade: EN10210 S355J2H		Test no. X90-1C			
Brace(s) nominal dimensions: 220 x 110 x 6											
Chord (measured geometrical dimensions)						Connection configuration (all dimensions in mm)					
b_0 (mm)	h_0 (mm)	t_0 (mm)	A_0 (mm ²)	f_{y0} (MPa)	f_{u0} (MPa)
220.0	110.0	5.94	3044	402	517						
Brace(s) (measured geometrical dimensions)						Measurements (all dimensions in mm)					
b_1 (mm)	h_1 (mm)	t_1 (mm)	A_1 (mm ²)	f_{y1} (MPa)	f_{u1} (MPa)
110.0	220.0	5.94	3044	402	517						
Geometrical parameters											
$\beta = b_1/b_0$	$\eta = h_1/b_0$	$\gamma = b_0/2t_0$	$\tau = t_1/t_0$	θ_1 (deg)							
0.5	1.0	18.52	1.0	90							
Connection capacity (kN)			Brace yield (kN)								
$N_{1u} = 202.4$			$N_{1(3\%)} = 150.5$			$N_{1,yield} = 1223.7$					
Connection load-displacement curve:

Failure mode: Chord plastification						Remarks:
Initial loading rate was 0.25mm/min and was increased only when yielding was apparent.					
						Chord began to increasingly flatten shortly after connection load-deflection curve became nonlinear.					
						Load-carrying capacity increases after plateau probably due to tension-stiffening in the top and bottom of the chord, and increases again once branches contact each other.					
						3% deformation limit governs capacity of the connection.					

Table 5D.5: Experimental summary of X90-2C

Chord nominal dimensions: 220 x 110 x 6						Steel grade: EN10210 S355J2H		Test no. X90-2C			
Brace(s) nominal dimensions: 220 x 110 x 6											
Chord (measured geometrical dimensions)						Connection configuration (all dimensions in mm)
b_0 (mm)	h_0 (mm)	t_0 (mm)	A_0 (mm ²)	f_{y0} (MPa)	f_{u0} (MPa)						
220.0	110.0	5.94	3044	402	517						
Brace(s) (measured geometrical dimensions)											
b_1 (mm)	h_1 (mm)	t_1 (mm)	A_1 (mm ²)	f_{y1} (MPa)	f_{u1} (MPa)						
220.0	110.0	5.94	3044	402	517						
Geometrical parameters						Measurements (all dimensions in mm)
$\beta = b_1/b_0$	$\eta = h_1/b_0$	$\gamma = b_0/2t_0$	$\tau = t_1/t_0$	θ_1 (deg)							
1.0	0.5	18.52	1.0	90							
Connection capacity (kN)			Brace yield (kN)								
$N_{1u} = 539.7$ $N_{1(3\%)} = 535.8$			$N_{1,yield} = 1223.7$								
Connection load-displacement curve:

Failure mode: Global buckling with chord sidewall failure

Remarks: Initial loading rate was 0.25mm/min and increased only when yielding was apparent. Global (overall) buckling occurred in the plane of the test specimen. Higher load-carrying capacities are therefore possible. Eventually the top branch punctured through chord side-wall via a pinching action. Ultimate load governs capacity of the connection.											

Table 5D.6: Experimental summary of X90-3C

Chord nominal dimensions: 220 x 110 x 6						Steel grade: EN10210 S355J2H		Test no. X90-3C			
Brace(s) nominal dimensions: 220 x 110 x 6											
Chord (measured geometrical dimensions)						Connection configuration (all dimensions in mm)					
b_0 (mm)	h_0 (mm)	t_0 (mm)	A_0 (mm ²)	f_{y0} (MPa)	f_{u0} (MPa)
110.0	220.0	5.94	3044	402	517						
Brace(s) (measured geometrical dimensions)											
b_1 (mm)	h_1 (mm)	t_1 (mm)	A_1 (mm ²)	f_{y1} (MPa)	f_{u1} (MPa)						
110.0	220.0	5.94	3044	402	517						
Geometrical parameters											
$\beta = b_1/b_0$	$\eta = h_1/b_0$	$\gamma = b_0/2t_0$	$\tau = t_1/t_0$	θ_1 (deg)							
1.0	2.0	9.26	1.0	90							
Connection capacity (kN)			Brace yield (kN)								
$N_{1u} = 555.1$			$N_{1(3\%)} = 459.8$								
Connection load-displacement curve:						Measurements (all dimensions in mm)

Failure mode: Chord side-wall failure						Remarks:
Initial loading rate was 0.25mm/min and was increased only when yielding was apparent.					
						Failure occurred as the chord side-walls began to buckle causing an out-of-plane chord rotation.					
						Ultimate load governs capacity of the connection.					

Table 5D.7: Experimental summary of X45-1C

Chord nominal dimensions: 220 x 110 x 6						Steel grade: EN10210 S355J2H		Test no. X45-1C			
Brace(s) nominal dimensions: 220 x 110 x 6											
Chord (measured geometrical dimensions)						Connection configuration (all dimensions in mm)					
b_0 (mm)	h_0 (mm)	t_0 (mm)	A_0 (mm ²)	f_{y0} (MPa)	f_{u0} (MPa)
220.0	110.0	5.94	3044	402	517						
Brace(s) (measured geometrical dimensions)											
b_1 (mm)	h_1 (mm)	t_1 (mm)	A_1 (mm ²)	f_{y1} (MPa)	f_{u1} (MPa)						
110.0	220.0	5.94	3044	402	517						
Geometrical parameters											
$\beta = b_1/b_0$	$\eta = h_1/b_0$	$\gamma = b_0/2t_0$	$\tau = t_1/t_0$	θ_1 (deg)							
0.5	1.0	18.52	1.0	45							
Connection capacity (kN)			Brace yield (kN)								
$N_{1u} = 350.3$			$N_{1(3\%)} = 258.5$			$N_{1,yield} = 1223.7$					
Connection load-displacement curve:

Failure mode: Chord plastification						Remarks:
Initial loading rate was 0.25mm/min and was increased only when yielding was apparent.					
						Chord began to increasingly flatten shortly after load vs. connection deflection curve became nonlinear.					
						Load-carrying capacity increases after plateau probably due to tension-stiffening in the top and bottom of the chord, and increases again once the branches contact each other.					
						3% deformation limit governs capacity of the connection.					

Table 5D.8: Experimental summary of X45-2C

Chord nominal dimensions: 220 x 110 x 6						Steel grade: EN10210 S355J2H		Test no. X45-2C			
Brace(s) nominal dimensions: 220 x 110 x 6											
Chord (measured geometrical dimensions)						Connection configuration (all dimensions in mm)					
b_0 (mm)	h_0 (mm)	t_0 (mm)	A_0 (mm ²)	f_{y0} (MPa)	f_{u0} (MPa)
220.0	110.0	5.94	3044	402	517						
Brace(s) (measured geometrical dimensions)											
b_1 (mm)	h_1 (mm)	t_1 (mm)	A_1 (mm ²)	f_{y1} (MPa)	f_{u1} (MPa)						
220.0	110.0	5.94	3044	402	517						
Geometrical parameters											
$\beta = b_1/b_0$	$\eta = h_1/b_0$	$\gamma = b_0/2t_0$	$\tau = t_1/t_0$	θ_1 (deg)							
1.0	0.5	18.52	1.0	45							
Connection capacity (kN)			Brace yield (kN)								
$N_{1u} = 627.8$			$N_{1(3\%)} = 531.0$			$N_{1,yield} = 1223.7$					
Connection load-displacement curve:						Measurements (all dimensions in mm)

Failure mode: Chord sidewall failure						Remarks:
Initial loading rate was 0.25mm/min and was increased only when yielding was apparent.					
						Failure mode was chord plastification. The increase in load-carrying capacity again may be attributed to the tension-stiffening in the chord top and bottom. The pinching action of the two branches caused the chord to crack at its side-wall centreline.					
						Ultimate load governs capacity of the connection.					

Table 5D.9: Experimental summary of X45-3C

Chord nominal dimensions: 220 x 110 x 6						Steel grade: EN10210 S355J2H		Test no. X45-3C			
Brace(s) nominal dimensions: 220 x 110 x 6											
Chord (measured geometrical dimensions)						Connection configuration (all dimensions in mm)					
b_o (mm)	h_o (mm)	t_o (mm)	A_o (mm ²)	f_{y0} (MPa)	f_{u0} (MPa)
110.0	220.0	5.94	3044	402	517						
Brace(s) (measured geometrical dimensions)											
b_l (mm)	h_l (mm)	t_l (mm)	A_l (mm ²)	f_{yl} (MPa)	f_{ul} (MPa)						
110.0	220.0	5.94	3044	402	517						
Geometrical parameters											
$\beta = b_l/b_o$	$\eta = h_l/b_o$	$\gamma = b_o/2t_o$	$\tau = t_l/t_o$	θ_l (deg)							
1.0	2.0	9.26	1.0	45							
Connection capacity (kN)			Brace yield (kN)								
$N_{1u} = 701.0$			$N_{1(3\%)} = 656.9$			$N_{1,yield} = 1223.7$					
Connection load-displacement curve:

Failure mode: Chord sidewall failure						Remarks:
Initial loading rate was 0.25mm/min and was increased only when yielding was apparent.					
						Failure occurred as the chord side-walls began to buckle, causing an out-of-plane chord rotation.					
						Ultimate load governs capacity of the connection.					

Table 5D.10: Experimental summary of T90-1C

Chord nominal dimensions: 220 x 110 x 6						Steel grade: EN10210 S355J2H		Test no. T90-1C			
Brace(s) nominal dimensions: 220 x 110 x 6											
Chord (measured geometrical dimensions)						Connection configuration (all dimensions in mm)					
b_0 (mm)	h_0 (mm)	t_0 (mm)	A_0 (mm ²)	f_{y0} (MPa)	f_{u0} (MPa)
220.0	110.0	5.94	3044	402	517						
Brace(s) (measured geometrical dimensions)											
b_1 (mm)	h_1 (mm)	t_1 (mm)	A_1 (mm ²)	f_{y1} (MPa)	f_{u1} (MPa)						
110.0	220.0	5.94	3044	402	517						
Geometrical parameters											
$\beta = b_1/b_0$	$\eta = h_1/b_0$	$\gamma = b_0/2t_0$	$\tau = t_1/t_0$	θ_1 (deg)							
0.5	1.0	18.52	1.0	90							
Connection capacity (kN)			Brace yield (kN)								
$N_{1u} = 216.5$			$N_{1(3\%)} = 211.4$			$N_{1,yield} = 1223.7$					
Connection load-displacement curve:

Failure mode: Chord plastification						Remarks:
Initial loading rate was 0.25mm/min and was increased only when yielding was apparent.					
						Indentation occurred at top of chord only. Load-carrying capacity increased again due to bending action of the chord.					
						Ultimate load governs capacity of the connection.					

Table 5D.11: Experimental summary of T90-2C

Chord nominal dimensions: 220 x 110 x 6						Steel grade: EN10210 S355J2H		Test no. T90-2C			
Brace(s) nominal dimensions: 220 x 110 x 6											
Chord (measured geometrical dimensions)						Connection configuration (all dimensions in mm)					
b_0 (mm)	h_0 (mm)	t_0 (mm)	A_0 (mm ²)	f_{y0} (MPa)	f_{u0} (MPa)
220.0	110.0	5.94	3044	402	517						
Brace(s) (measured geometrical dimensions)											
b_1 (mm)	h_1 (mm)	t_1 (mm)	A_1 (mm ²)	f_{y1} (MPa)	f_{u1} (MPa)						
220.0	110.0	5.94	3044	402	517						
Geometrical parameters											
$\beta = b_1/b_0$	$\eta = h_1/b_0$	$\gamma = b_0/2t_0$	$\tau = t_1/t_0$	θ_1 (deg)							
1.0	0.5	18.52	1.0	90							
Connection capacity (kN)			Brace yield (kN)								
$N_{1u} = 353.0$			$N_{1(3\%)} = 338.2$			$N_{1,yield} = 1223.7$					
Connection load-displacement curve:

Failure mode: Chord sidewall failure						Remarks:
Initial loading rate was 0.25mm/min and was increased only when yielding was apparent.					
						Indentation occurred at top of chord only. The chord sidewalls bulged out. Load-carrying capacity tends to increase again due to bending action of the chord.					
						Ultimate load governs capacity of the connection.					

Table 5D.12: Experimental summary of T90-3C

Chord nominal dimensions: 220 x 110 x 6						Steel grade: EN10210 S355J2H		Test no. T90-3C			
Brace(s) nominal dimensions: 220 x 110 x 6											
Chord (measured geometrical dimensions)						Connection configuration (all dimensions in mm)					
b_0 (mm)	h_0 (mm)	t_0 (mm)	A_0 (mm ²)	f_{y0} (MPa)	f_{u0} (MPa)
110.0	220.0	5.94	3044	402	517						
Brace(s) (measured geometrical dimensions)											
b_1 (mm)	h_1 (mm)	t_1 (mm)	A_1 (mm ²)	f_{y1} (MPa)	f_{u1} (MPa)						
110.0	220.0	5.94	3044	402	517						
Geometrical parameters											
$\beta = b_1/b_0$	$\eta = h_1/b_0$	$\gamma = b_0/2t_0$	$\tau = t_1/t_0$	θ_1 (deg)							
1.0	2.0	9.26	1.0	90							
Connection capacity (kN)			Brace yield (kN)								
$N_{1u} = 593.8$			$N_{1(3\%)} = 550.6$			$N_{1,yield} = 1223.7$					
Connection load-displacement curve:						Measurements (all dimensions in mm)

Failure mode: Chord sidewall failure						Remarks:
Initial loading rate was 0.25mm/min and was increased only when yielding was apparent.					
						Indentation occurred at top of chord only. The chord sidewalls bulged out.					
						Ultimate load governs capacity of the connection.					