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ANALYSIS, DESIGN, AND DETAILING OF FLEXIBLE AND EXTENDED END PLATE CONNECTIONS

By
Min Bao

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

MASTER OF ENGINEERING

*The Civil Engineering Program is a joint program with University of Ottawa
administered by the Ottawa-Carleton Institute for Civil Engineering.

**Department of Civil and Environmental Engineering
Carleton University
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**Analysis, Design, and Detailing of Flexible
and Extended End Plate Connections
by
Min Bao (Civil)**

**in Partial Fulfilment of the Requirements
for the Degree of Master of Engineering**

Thesis Supervisor

Chairman, Department of Civil Engineering

**Department of Civil and Environmental Engineering
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March, 1996**

ABSTRACT

A computer program entitled, "End Plate Connection Design" (EPCD) was developed to analyze and design both flexible and extended end plate connections according to the Canadian Standard for Limit State Design of Steel Structures, CAN/CSA-S16.1-M89, and current fabricating practices. It includes recent research in determining internal force distribution and resistance determinations. In conjunction a graphic user interface was developed for the connection design and review process that simplifies data input, allows for fabricators preferences in material usage, allows fabricators to input confidential unit cost information for evaluating the connection cost, allows a complete review of the resistances for all probable failure modes and presents a scaled three dimensional rendition of the design connection. Moment rotation curves are calculated and displayed. Design examples for orthogonal beam-to-column flange or web, and beam-to-beam joints are given. Comparisons are made with a commercially available end plate connection design program and published design examples by the American Institute of Steel Construction (AISC) and Canadian Institute of Steel Construction (CISC) to verify the design algorithms. The program EPCD provides an effective design and learning tool for the design of end plate connections for students, engineers and fabricators.

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List of Symbols

a	a distance from the centerline of the bolt hole towards the plate edge or distance from bolt line to edge of tee flange
A_b	cross-sectional area of a bolt based on its nominal diameter
A_m	area of fusion face
A_n	critical net area
A_{ne}	effective net area
A_w	shear area, effective throat area of weld
B_i	tension force in the bolt level of interest
B_r	factored bearing resistance of a member
b	distance from bolt line (gauge line) to face of tee stem (mm) or the width of stiffener
b'	$b - d/2$
c_1	coefficient used to determine slip resistance
C	compression force in connecting member
C_b	unit cost for bolts (\$/bolt)
C_f	compression force in connecting member under factored load
C_{fi}	unit cost of fitter (\$/h)
C_g	unit cost of general labour (\$/h)
C_l	total cost of labour (\$)
C_m	total cost of material (\$)
C_{ma}	unit cost of machinist (\$/h)
C_p	unit cost of plate (\$/kg)
C_s	the compression force resisted by the stiffener provided opposite the compression flange of connected beams
C_t	total cost of a designed connection (\$)
C_w	unit cost of welder (\$/hour)
C_{w1}	unit cost of weld (\$/mm)

C_{ws}	unit cost of stiffener (\$/kg)
C_1	curve-fitting constants
C_2	curve-fitting constants
C_3	curve-fitting constants
d	diameter of bolt
d_h	nominal hole diameter (mm)
d_h	diameter of the bolt hole
d_p	length of the end plate
ds	the length of stiffener along web of the column
D	the size of weld, the clearance for the impact wrenches, or density of steel (kg/m ³)
e	end distance or edge distance for plate
E_p	modulus of elasticity of end plate
F_y	specified minimum yield strength
F_s	ultimate shear stress
F_u	specified minimum tensile strength
F_{up}	specified minimum tensile strength for the plate
F_{uw}	ultimate tensile strength of the bolt
F_{yp}	specified minimum yield strength for end plate
g	gauge distance
H_i	the horizontal reaction force acting at bolt level of interest
H_{max}	is the maximum horizontal shear force that can be developed
H_1	shear resistance of bolts
H_2	shear resistance of the net section of the end plate
H_3	bearing resistance of the end plate for $e \leq 3d$
H_4	bearing resistance of the end plate for $e > 3d$
I_p	moment of inertia of end plate
k	distance from outer face of flange to web-toe of fillet of l-shaped sections
k_1	distance from the center of the web to toe of the fillet on the flange

l	the distance between the inside edge of the nearest bolt hole and the toe of the fillet weld of the undeformed plate
l_p	plate length (mm)
l_{ps}	stiffener length (mm)
l_w	weld length for the beam (mm)
l_{ws}	weld length for the stiffener (mm)
L	two times the end plate length
L_n	net length (gross length less design allowance for holes within the length)
M	the maximum moment acting in clear span of end plate
M_f	factored moment applied to the extended end plate connection
M_{pp}	the plastic moment capacity for a unit width of end plate based on F_{up} across the section
M_{rc}	flexural resistance of coped section assuming no shear force is present
M_e	the eccentric moment
M_{up}	the plastic moment capacity for a unit width of end plate at the stress level of F_{up}
m	the shear planes in the bolted joint
n	the number of bolts, or the number of pair of bolts, or number of flange bolts in tension
p	length of flange tributary to each bolt, or bolt pitch (mm)
P	applied unfactored tensile load per bolt (kN)
P_f	applied factored tensile load per bolt (kN)
P_{yp}	the yield tensile capacity of a unit width of plate
Q	prying force per bolt at factored load (kN), $Q = T_f - P_f$
t	the thickness of the end plate, or stiffeners, or flange
t_p	the thickness of the end plate
t_{ps}	the thickness of stiffener (mm)
t_w	beam web thickness

T	tensile strength in connecting member
T_f	tensile force in a member under factored load or load per bolt including prying action at factored load (kN)
T_{fi}	time estimate for fitter (hour)
T_g	time estimate for general labour (hour)
T_m	time estimate for machinist (hour)
T_w	time estimate for welder (hour)
T_r	factored tensile resistance of the bolts
T_{rs}	the tensile resistance of stiffener
T_s	the tensile force resisted by the stiffener provided opposite the tension flange of connected beams
V_c	maximum shear resistance in the beam web
V_f	shear force in member under factored load
V_{ft}	total factored shear force at connection
V_i	factored shear resistance of connection component in the tension region at the level of the i th row of bolts, the i th level of end plate, the i th level of weld group, and the beam web segment with length $l_p/50$
V_r	factored shear resistance of a member
V_{rb}	shear resistance for the bolt group
V_{rc}	shear resistance of the beam web at the coped section assuming no flexural stresses are present
V_{rs}	the shear resistance of stiffener
V_p	factored shear resistance of end plate in tension region
V_{ru}	factored shear resistance of filled weld group in tension region
V_s	slip resistance of a bolt joint or shear force in a stiffener
V_T	transverse shear force
V_U	ultimate shear strength of a bolt through the threads
w_p	width of the end plate

w_p	width of the end plate (mm)
w_{ps}	width of stiffener (mm)
W_n	net width (gross length less design allowance for holes within the width)
X_u	ultimate strength as rated by the electrode classification number
α	ratio of sagging moment at bolt line to hogging moment at stem of tee
β	interaction factor derived from tests relating shear and tension forces in bolt
Δ	the plate deformation
Δ_e	maximum elastic end plate deformation
Δ_p	maximum tensile deformation of end plate occurring when H_{max} is first reached
θ	relative rotation of the end beam to the column
ϕ	resistance factor, taken as 0.90
ϕ_b	bolts resistance factor, taken as 0.67 for bolt
ϕ_w	weld resistance factor, taken as 0.67 for weld
ϕ_p	plate resistance factor, taken as 0.67 for plate
σ_c	compressive stress at interface of end plate and beam web
δ	ratio of net area (at bolt line) to gross area at face of tee stem

List of Abbreviations

AISC	American Institute of Steel Construction
CDSSSS	connection design for simply supported steel shapes
CISC	Canadian Institute of Steel Construction
CONXPRT	computer program of the design of connection in steel framed buildings
EEPC	extended end plate connection
EEPCD	extended end plate connection design
EPCD	end plate connection design including both flexible and extended end plate connection design
FEPC	flexible end plate connection
FEPCD	flexible end plate connection design
LRFD	load and resistance factor design
SCADD	steel connection analysis, design, and detailing

Chapter 1

Introduction

1.1 Connection Design and Detailing

The analysis and design of connections is an important step in the design, fabrication and erection of structural steel building frames, as there are usually a large number of them and the economics of the project can be greatly affected by the fabrication and erection costs associated with poor connection design. The designer should strive for simplicity and duplication, and should avoid complicated and costly fabrication. Most fabricators in Canada use standard connections types; such as end plates, which are generally shop welded and field bolted. This makes it easier for erection and achieves maximum economy.

To minimize the time required to design and detail standard connections, various computer programs and a "shop drawing standard", which contains a design summary of preferred geometric designs according to the Handbook of Steel Construction (CISC 1989), have been developed by fabricators. These connection designs are standardized, have simple details, and conform to current fabrication processes in Canada.

Steel connection analysis, design, and detailing (SCADD), Devitt (1992) is a stand alone and an integrated program developed at Carleton University, for the design of standard connections. It accounts for fabricators preferences which reflect their plant operation, analyses the moment rotation characteristics of the connection, and evaluates the cost of the connection according to confidential cost data supplied by the fabricator. Four simple, well defined interactive screens were designed for the second edition of the program to provide the user (the word user implies that the person(s) using this software program are structural engineers, students, professors, or fabricators) with the ability to input data without guidance. A number of minor pop-up screens are provided where required. A 3D rendition of the completed design is illustrated so that it can be viewed from any direction. An on-screen engineering report, materials report, moment and rotation curves for the design connection, and a connection cost table are produced simultaneously from the same program execution. A shop drawing tool option that transcribes these images into Auto-CAD drawings for fabrication is envisaged but has not yet been incorporated in the program code. This thesis focuses on the incorporation of flexible and extended end plate connection designs into the program SCADD.

Connection design programs that are related to the topic and presented in this thesis, include a computer program for the design of connections in steel framed buildings (CONXPRT) and a noncommercial program flexible end plate connection design (CDSSSS).

The software CONXPRT-LRFD: Module I (1991), is a load and resistance factor design (LRFD) knowledge based system for the design of connections in steel framed buildings that runs on a PC platform and is supported by the American Institute of Steel Construction. Three types of connections are designed: framing angles connection, single plate connection, and shear end plate connection. Possible framing configurations include: beam-to-column flange, beam-to-column web, and beam-to-girder web. The user can select any combination of connection type and framing.

Single or double copes for the beam, bolt stagger, and offset beam elevations are considered as part of the geometric design. The connection is designed or the shear capacity is calculated depending whether or not a specified shear value or a maximum number of bolt rows for shear end plate connection is specified. Figures 1.1 and 1.2 illustrate examples of typical output for shear end plate connection designs.

The program CDSSSS designs standardized flexible end plate connections according to given the beam size, span, and uniform loading. The geometric and material properties for the design connection plate, bolts and the fillet welds are specified by the user. The cost of the connection is calculated from user specified cost factors for the three basic connection components; the costs of the plate (\$/kg), the fillet weld (\$/mm), and the bolts (\$/bolt). Moment rotation characteristics are evaluated for the designed connection. An example output gives a complete record of the geometric and material properties, the connection geometry and the resistances are illustrated in Figures 1.3, 1.4, and 1.5.

1.2 Objective and Scope

1.2.1 Objective

The objective of this study is to develop a computer program, written in C language, for the analysis, design and evaluation of the standard flexible end plate connections (FEPC) and the extended end plate connections (EEPC), for consulting engineers, architects, fabricators/erectors and students. As part of this development, is the creation of the graphic user interface that simulates the design process which minimizes on screen instructions for input and display. 3D renditions of structural images, tables, and graphs to simplify the review and verification processes of the design connection.

1.2.2 Scope

Flexible end plate connections can be designed for a set of forces consistent with the transfer of shear through the end plate to the supporting member or for those forces determined from the analysis of the connection using the inelastic model developed by Kriviak and Kennedy (1984). In either case the failure modes are explicitly recognized and the components are designed according to CAN/CSA-S16.1-M89 or to rational design procedures referenced by the standard. Only beam-to-column flange or web or beam-to-beam orthogonal joints are considered. Extended end plate connection designs are based on a lower bound theorem approach and are only applicable for beam-to-column flange orthogonal joints. Stiffeners and the corresponding welds are designed if required. Coping details are determined and incorporated in the strength design considerations for both types of end plate connections.

The appropriate moment and rotation curves for each designed connection are calculated and drawn using the Kriviak and Kennedy inelastic model (1984), the Frye-Morris polynomial model (1976), and the Ang-Morris power model (1984).

The connection cost table provides a summary of the labour time spent fabricating the connection and the cost of the labour and material used. Customized estimates for costs of each task can be supplied by the individual fabricators for their operation.

The results of the design are represented by an engineering report which gives a summary of resistances for each possible failure mode and a materials report. The moment rotation curves for both flexible and extended end plate connections, and the connection cost table are also displayed. Design loads, cost of labour and material, rotation limits for the inelastic analysis, fabrication preferences and all of the geometric data for the connection can be selected and enter by the user through the existing SCADD graphic interface. Because the program performing the design executes quickly, the design for a given connection can be efficiently repeated with

varied design parameters within a short period. It is possible for a fabricator to optimize the design of the connection with respect to their operation.

1.3 Thesis Outline

The thesis presents the analysis and the design procedures for flexible end plate connection (FEPC) and the extended end plate connection (EEPC) and briefly describes related design software.

Chapter 2 describes the behaviour interface distributions, moment rotation characteristics and resistance formulations for the design of the flexible and extended end plate connections. Chapter 3 gives a description of the design philosophy, design algorithms, program architecture and functions (strength and geometric design rules), and the functionality of the graphic user interface. Chapter 4 illustrates design examples for both connections and gives comparisons with documented designs. Appropriate discussions are included. Conclusions, recommendations, and future work for incorporating other connection types and functions are given in the last chapter.

Figure 1.1: CONXPRT Example, Graphic Input/Output Screen

Connection Description:

shear and plate, beam to column flange connection.

Beam: W18x35 A36 Steel $F_y = 36 \text{ ksi}$ $F_u = 58 \text{ ksi}$
 $d = 17.7 \text{ in.}$ $t_w = 0.3 \text{ in.}$

Column: W21x122 A36 Steel $F_y = 36 \text{ ksi}$ $F_u = 58 \text{ ksi}$
 $t_f = 0.96 \text{ in.}$

Factored beam end reaction, V_u : 80 Kips
Connection strength, ϕV_n : 91.5 Kips

Strength Limit State:

Beam gross shear strength (AISC LRFD Sect. F2):
 $\phi V_n = (0.9) \times (0.6 F_y A_w) = 103.2 \text{ Kips} > 80 \text{ Kips}$ OK

Maximum beam web strength at plate $= 0.8 \times (0.7 F_y) (L_p - 2x_{\text{t weld}}) \times t_w$
 $= 91.5 \text{ Kips} > 80 \text{ Kips}$ OK

Beam web connection:

Use two 3/16 in. - E70 XX fillet welds
Total weld shear strength $= 126.3 \text{ Kips} > 80 \text{ Kips}$ OK

Column flange connection:

Use ten (10) 3/4 in. $\phi \times 2\text{-}1/2"$ w / 5/32" thick washers
A325-N STD bolts in single shear
(user may need to adjust bolt length for other conditions.)
Total bolt shear design strength:
 $\phi R_b = 155.1 \text{ Kips} > 80 \text{ Kips}$ OK (AISC LRFD J3.3)

Column flange bearing / tear out strength (AISC LRFD Sect. J3.6):
 $\phi P_b = 0.75 P_b = 751.7 \text{ Kips} > 80 \text{ Kips}$ OK

Connection Plate:

PL 1/4 x 6 x 15-1/2"
A 36 Steel plate $F_y = 36 \text{ ksi}$ $F_u = 58 \text{ ksi}$

Plate gross shear strength (AISC LRFD Equ. J5-3):
 $\phi V_n = 0.8 V_n = 156.2 \text{ Kips} > 80 \text{ Kips}$ OK

Plate net shear strength (AISC LRFD Sect. J4):
 $\phi V_n = 0.75 V_n = 149.3 \text{ Kips} > 80 \text{ Kips}$ OK

Plate block shear strength (AISC LRFD Sect. J5.2.c):
 $\phi V_b = 0.75 V_b = 142.1 \text{ Kips} > 80 \text{ Kips}$ OK

Plate bearing / tear out strength (AISC LRFD Sect. J3.6, J3.9 & J3.10):
 $\phi P_b = 0.75 P_b = 195.7 \text{ Kips} > 80 \text{ Kips}$ OK

Figure 1.2: CONXPRT Example, Engineering Report

WELDED: Available in Canada And U.S. & normally used As A Beam

CONNECTION #1 PER-200-21-140 75-25-25 75-611 81-825-15-0 W-480-61
 Supports Flanges of W480x61 Columns Material Properties as per Supported Member
 Supports Assumed to be rigid
 Supported member is uncased

#	Beam Length (mm)	Length COST	Plate Length (mm)	# of Ribs Area	Conn COST (S)	Column Length	FACT SHEAR (kN)	$\frac{V_{DES}}{V_u}$	$\frac{V_{DES}}{V_u}$	$\frac{V_{DES}}{V_u}$	$\frac{V_{DES}}{V_u}$	FACT MOM (kNm)
1	1409	3 0	891	891	891	891	892 8	0 94	1 11	1 02	1 09	891
2	3614	6 0	445 0	12	222 17	891	892 8	1 10	1 27	1 15	1 21	20
3	5427	9 0	295 0	8	147 52	891	461 7	1 10	1 26	1 14	1 22	14
4	7236	12 0	229 0	6	110 81	891	345 0	1 10	1 26	1 13	1 22	8
5	9045	15 0	235 0	6	147 72	891	277 0	1 63	2 10	1 92	2 02	15
6	10854	18 0	279 0	10	184 95	891	220 8	2 74	2 17	2 46	3 48	25
7	12663	21 0	279 0	10	184 95	891	197 9	2 20	2 70	2 35	3 48	26
8	14472	24 0	445 0	12	222 17	891	172 1	4 28	5 10	4 54	4 73	61
9	16281	27 0	445 0	12	222 17	891	152 9	4 94	5 74	5 15	5 30	43
10	18090	30 0	445 0	12	222 17	891	134 5	5 49	6 38	5 73	5 86	45
#	Max PL Depth (mm)	Max PL Depth (mm)	Length (mm)	Length (mm)	Beam P. (kN)	Beam P. (kN)	Dist Top Conn (mm)	Dist Top Beam (mm)	$\frac{M}{V}$	$\frac{M}{V}$	$\frac{M}{V}$	
1	891	891	891	891	891	891	891	891	891	891	891	
2	5 3	4 2	891	891	0 0246	0 0206	1 7	1 8	1 31	0 64		
3	12 2	9 7	444	592	0 0070	0 0257	1 4	1 6	1 24	0 02		
4	22 0	17 6	229	412	0 0077	0 0077	1 4	1 4	1 27	0 01		
5	32 9	26 9	257	226	0 0 9	0 0234	2 1	2 4	1 26	0 02		
6	47 8	37 3	227	244	0 0 39	0 0110	4 5	4 0	1 30	0 02		
7	65 0	51 5	195	246	0 0 12	0 0128	5 3	4 7	1 39	0 02		
8	82 4	65 1	195	222	0 0174	0 0140	7 0	6 1	1 34	0 06		
9	103 8	82 1	195	195	0 0199	0 0157	7 8	6 4	1 34	0 06		
10	127 6	107 0	142	195	0 0220	0 0174	8 6	7 5	1 35	0 06		

CONNECTION #2 PER-200-21-140 75-25-25 75-611 81-825-15-0 W-480-61
 Supports Flanges of W480x61 Columns Material Properties as per Supported Member
 Supports Assumed to be rigid
 Supported member is uncased

#	Beam Length (mm)	Length Depth (mm)	Plate Length (mm)	# of Ribs Area	Conn. Depth (mm)	Column Depth (mm)	Fact. Shear (kN)	$\frac{V_{DES}}{V_u}$	$\frac{V_{DES}}{V_u}$	$\frac{V_{DES}}{V_u}$	$\frac{V_{DES}}{V_u}$	Fact. Moment (kNm)
1	1409	3 0	510 0	14	264 52	891	892 8	1 02	1 11	1 02	1 08	48
2	3614	6 0	445 0	8	226 87	891	492 5	1 14	1 22	1 02	1 18	50
3	5427	9 0	295 0	6	150 84	891	461 7	1 14	1 23	1 02	1 19	23
4	7236	12 0	229 0	6	112 83	891	345 0	1 14	1 22	1 01	1 20	18
5	9045	15 0	220 0	6	121 84	891	277 0	1 42	1 52	1 29	1 50	14
6	10854	18 0	295 0	8	150 84	891	220 8	2 28	2 46	2 04	2 35	25
7	12663	21 0	295 0	8	150 84	891	197 9	2 64	2 87	2 34	2 72	26
8	14472	24 0	445 0	12	188 80	891	172 1	4 82	4 10	3 02	3 81	42
9	16281	27 0	445 0	12	188 80	891	152 9	4 28	4 64	3 71	4 31	42
10	18090	30 0	445 0	12	220 54	891	134 5	5 70	6 21	5 52	5 86	64

#	Max. PL Depth (mm)	Max. PL Depth (mm)	Length (mm)	Length (mm)	Beam P. (kN)	Beam P. (kN)	Dist. Top Conn. (mm)	Dist. Top Beam (mm)	$\frac{M}{V}$	$\frac{M}{V}$	
1	0 8	0 6	22 9	2825	0 0212	0 0212	0 5	0 4	2 18	0 06	
2	5 1	4 0	4 5	8 2	0 0214	0 0221	1 5	1 3	1 28	0 06	
3	12 0	9 5	412	570	0 0212	0 0221	1 4	1 3	1 30	0 02	
4	21 8	17 2	222	418	0 0214	0 0221	1 4	1 3	1 28	0 02	
5	31 6	25 0	226	221	0 0 1	0 0221	2 2	2 0	1 28	0 02	
6	47 8	37 3	226	221	0 0 23	0 0172	2 4	2 0	1 28	0 02	
7	65 0	51 5	195	221	0 0 21	0 0172	3 4	2 9	1 30	0 02	
8	82 4	65 1	195	222	0 0 73	0 0171	5 4	4 8	1 34	0 05	
9	104 0	81 8	195	199	0 0220	0 0155	6 2	5 4	1 34	0 05	
10	127 6	95 8	148	199	0 0208	0 0152	7 8	6 6	1 39	0 04	

Figure 1.4: CDSSSS, Output for End Plate Thickness of 6 and 8 mm

W16S191 : Available in Canada and U.S.A. normally used as a Beam

CONNECTION #3 PER 300-21-140- 75-35-35- 75-10-L-1 B- 825-10-0-1 W16S1- 61
 SUPPORTS: Flanges of W36S70
 SUPPORTS assumed to be RIGID Columns Material Properties as per Supported Member
 SUPPORTED MEMBER IS UNCOLORED

#	Beam Length (mm)	Length Depth	Plate Length (mm)	# of Ribs	Conn Cost (15)	Govern Cond:1	FACT SHEAR (mm)	Yield (1)	Yield (1)	Yield (1)	Yield (1)	Yield (1)	FACT HANG (mm)
1	1809	3 0	520 0	14	269 85	8011	802 6	1 01	1 06	1 71	1 06	1 06	64
2	2612	6 0	445 0	12	230 85	8011	802 6	1 06	1 12	1 01	1 14	1 14	70
3	5427	9 0	295 0	8	152 55	8011	461 7	1 08	1 13	1 00	1 15	1 15	33
4	7236	12 0	220 0	6	114 85	Weld	246 3	1 06	1 07	1 89	1 16	1 16	10
5	9045	15 0	220 0	4	114 85	Weld	277 0	1 35	1 34	2 36	1 44	1 44	30
6	10854	18 0	145 0	4	76 15	Weld	230 6	1 09	1 09	1 87	1 18	1 18	8
7	12663	21 0	220 0	6	114 85	Weld	187 9	1 89	1 86	3 39	1 99	1 99	21
8	14472	24 0	220 0	8	152 55	Weld	173 1	2 85	2 85	5 04	2 83	2 83	37
9	16281	27 0	370 0	10	192 25	Weld	152 9	3 89	3 99	7 14	3 96	3 96	58
10	18090	30 0	370 0	10	192 25	Weld	138 5	4 42	4 43	7 83	4 42	4 42	99

#	Max FL (mm)	Max DL (mm)	Length FL (mm)	Length DL (mm)	Conn FL (mm)	Conn DL (mm)	Disp Top Conn FL (mm)	Disp Top Beam DL (mm)	M-M (mm)	M-M (mm)		
1	0 7	0 6	2491	3132	0 0012	0 0010	0 4	0 3	2 30	0 00		
2	4 8	3 8	747	983	0 0041	0 0032	1 2	1 1	1 41	0 00		
3	11 8	9 3	461	544	0 0056	0 0054	1 4	1 3	1 32	0 04		
4	21 5	17 1	336	424	0 0094	0 0075	1 5	1 4	1 29	0 02		
5	32 6	26 6	269	339	0 0116	0 0082	1 9	1 8	1 28	0 03		
6	49 4	39 4	220	276	0 0165	0 0115	1 6	1 7	1 27	0 01		
7	65 8	52 1	183	242	0 0164	0 0130	2 7	2 6	1 28	0 03		
8	82 1	65 4	174	221	0 0160	0 0141	4 0	3 7	1 32	0 05		
9	100 4	76 4	162	208	0 0191	0 0148	5 4	4 8	1 38	0 08		
10	122 5	95 5	146	187	0 0212	0 0154	6 1	5 2	1 34	0 08		

CONNECTION #4 PER 300-21-140- 75-35-35- 75-12-L-1 B- 825-10-0-1 W16S1- 61
 SUPPORTS: Flanges of W36S70
 SUPPORTS assumed to be RIGID Columns Material Properties as per Supported Member
 SUPPORTED MEMBER IS UNCOLORED

#	Beam Length (mm)	Length Depth	Plate Length (mm)	# of Ribs Ribs	Conn Cost (15)	Conn Cond:	FACT SHEAR (mm)	Yield (1)	Yield (1)	Yield (1)	Yield (1)	Yield (1)	Yield (1)
1	1809	3 0	8011	8011	8011	8011	802 6	0 92	0 95	2 05	1 05	8011	8011
2	2612	6 0	Weld	Weld	Weld	Weld	802 6	1 06	0 99	2 68	1 28	Weld	Weld
3	5427	9 0	445 0	12	235 34	Weld	461 7	1 35	1 25	3 44	1 59	Weld	Weld
4	7236	12 0	295 0	8	156 46	Weld	246 3	1 21	1 16	3 04	1 43	Weld	Weld
5	9045	15 0	295 0	8	156 46	Weld	277 0	1 51	1 43	3 40	1 79	Weld	Weld
6	10854	18 0	220 0	6	117 02	Weld	230 6	1 25	1 08	3 40	1 61	Weld	Weld
7	12663	21 0	220 0	6	117 02	Weld	197 9	1 45	1 27	3 86	1 87	Weld	Weld
8	14472	24 0	220 0	6	117 02	Weld	173 1	1 66	1 45	4 53	2 11	Weld	Weld
9	16281	27 0	220 0	6	156 46	Weld	152 9	2 37	2 02	6 43	2 96	Weld	Weld
10	18090	30 0	220 0	8	156 46	Weld	138 5	2 62	2 75	7 58	3 30	Weld	Weld

#	Max FL (mm)	Max DL (mm)	Length FL (mm)	Length DL (mm)	Conn FL (mm)	Conn DL (mm)	Disp Top Conn FL (mm)	Disp Top Beam DL (mm)	M-M (mm)	M-M (mm)		
1	8011	8011	8011	8011	8011	8011	8011	8011	8011	8011		
2	Weld	Weld	Weld	Weld	Weld	Weld	Weld	Weld	Weld	Weld		
3	12 3	7 9	578	681	0 0057	0 0042	1 6	1 4	1 47	0 12		
4	20 5	16 0	254	451	0 0088	0 0068	1 7	1 6	1 35	0 06		
5	31 4	25 0	214	342	0 0100	0 0066	2 1	2 0	1 35	0 06		
6	47 7	37 7	227	248	0 0138	0 0108	2 1	2 0	1 31	0 03		
7	66 9	51 3	195	247	0 0162	0 0128	2 5	2 4	1 37	0 03		
8	84 7	66 9	171	216	0 0185	0 0145	2 8	2 6	1 31	0 04		
9	102 5	80 3	159	203	0 0196	0 0132	4 1	3 8	1 74	0 06		
10	128 8	99 0	142	183	0 0216	0 0148	4 6	4 2	1 36	0 06		

Figure 1.5: CDSSSS, Output for End Plate Thickness of 10 and 12 mm

Chapter 2

End Plate Connection Behaviour and Design

2.1 Behaviour of Flexible End Plate Connections

A flexible end plate connection consists of an end plate which is welded to the beam web and bolted to the supporting member may extend to the full depth of the web or only part of it as shown in Figure 2.1. Flexible end plate connections are considered to transmit shear only as the flexural capacity is small.

The behaviour of the flexible end plate connections can be characterized by the load deformation characteristics (moment rotation curves) and by the internal force distributions. Two different internal force distributions can be used for the design, one is based on shear only (assuming zero rotation) where the shear is carried by the bolts equally and the moment caused by the shear acting at an eccentricity of the end plate thickness is transmitted by the bolts to the supporting member assuming a linear distribution of equivalent forces. The other is based on an inelastic model,

Kriviak and Kennedy (1984) where the force distribution and rotation are calculated for the factored shear and are determined as a function of the connection flexibility. Sizing the connection for these forces according to the resistance formulations and detailing rules given in CAN/CSAS16.1M89 ensures that the lower bound theorem is satisfied and that the design is satisfactory.

Figure 2.1: Flexible End Plate Connection

Kriviak and Kennedy (1984) developed an inelastic model which gives the internal force distribution and moment and rotation characteristics of flexible end plate connections. The model is based on research by Sommer (1969) and Hafez (1982) and the moment rotation curves compared to test data by Hafez (1982). The general deformation and internal forces of the end plate in the tension region and the end rotation of the beam are illustrated in Figures 2.2 and 2.3, respectively.

The maximum end rotation for beams that meet the serviceability deflection requirements range from 0.01 to 0.03 radians. The inelastic model determines the moment for 40 values of rotation at equal intervals up to the maximum rotation specified. The corresponding shear resistance of the connection is determined subsequently by evaluating the remaining capacity of the connection that is not utilized in carrying

the moment, secondary horizontal shear forces and prying forces in the bolts. For each rotation the location of the neutral axis is assumed and the corresponding internal forces are calculated at each segment (50 equal segments in total) along the depth of the end plate. The internal force distribution are illustrated in Figures 2.2, and 2.3 and are calculated from the deformations that are consistent with the assumed rotation and the bolt deformation curves for the end plate. The location of the neutral axis is adjusted until equilibrium is satisfied. The process is repeated for all forty values of rotation. A description of the equations which give the internal tension and compression forces perpendicular to the end plate follow. The sum of these forces multiplied by the perpendicular distances about the neutral axis is equal to the moment.

Tension Region

The tensile force that is developed in the beam web for segment where $\Delta \leq \Delta_e$ is given by,

$$T = \frac{24E_p I_p \Delta}{l^3} \quad (2.1)$$

where,

E_p = modulus of elasticity of end plate

I_p = moment of inertia of end plate

Δ = the plate deformation

l = the distance between the toe of a fillet weld and the inside edge of the nearest bolt hole

Δ_e = maximum elastic end plate deformation given by,

$$\Delta_e = \frac{4H_{max} M_{pp} l^2}{24E_p I_p} \quad (2.2)$$

and where, H_{max} is the maximum horizontal shear force that can be developed and is the lesser of H_1 , H_2 , H_3 , and H_4 ,

$$\begin{aligned} H_1 &= 0.42\phi_b m n A_b F_u && \text{(threads intercepted)} \\ \text{or } H_1 &= 0.60\phi_b m n A_b F_u && \text{(threads excluded)} \end{aligned} \quad (2.3)$$

where,

H_1 = shear resistance of bolts

ϕ_b = resistance factor of the bolts

m = the shear planes in the bolted joint

n = the number of the bolts

A_b = cross-sectional area of a bolt based on its nominal diameter

F_u = specified minimum tensile strength for the bolts

$$H_2 = (p - d_h)tF_{up} \quad (2.4)$$

where,

H_2 = shear resistance of the net section of the end plate

d_h = diameter of the bolt hole

F_{up} = specified minimum tensile strength for the plate

$$H_3 = etF_{up} \quad (2.5)$$

where,

H_3 = bearing resistance of the end plate for $e \leq 3d$

e = edge distance for plate

$$H_4 = 3dtF_{up} \quad (2.6)$$

where,

H_4 = bearing resistance of the end plate for $e > 3d$

d = diameter of the bolts

t = thickness of the end plate

The plastic moment capacity for a unit width of end plate based on F_{up} across the section

$$M_{pp} = \frac{t^2}{4} F_{up} \quad (2.7)$$

The tensile force that is developed in the beam web for segment where $\Delta_c < \Delta < \Delta_p$ is given by,

$$T = \frac{P_{yp}^2 \Delta^2}{4M_{up}l} + \frac{4M_{up}l}{(l^2 + \Delta^2)} \quad (2.8)$$

where, P_{yp} = the yield tensile capacity of a unit width of plate

$$P_{yp} = t_p F_{yp} \quad (2.9)$$

and, M_{up} = the plastic moment capacity for a unit width of end plate at the stress level of F_{up}

$$M_{up} = \frac{t_p^2}{4} F_{up} \quad (2.10)$$

and, Δ_p = maximum tensile deformation of end plate occurring when H_{max} is first reached,

$$\Delta_p = \frac{4H_{max}M_{up}}{P_{yp}^2 p} \quad (2.11)$$

The tensile force that is developed in the beam web for segment where $\Delta > \Delta_p$ is given by,

$$T = \frac{2H_{max}}{pl} \Delta + \frac{4M_{up}l}{(l^2 + \Delta^2)} - \frac{4M_{up}H_{max}^2}{p^2 l P_{yp}^2} \quad (2.12)$$

where, p = bolt pitch

Figure 2.2: Internal Force Distribution, Kriviak and Kennedy (1984)

Compression Region

The compressive force at each segment is calculated by multiplying the compressive stress σ_c , which is evaluated at mid-depth of the segment by the segment length and beam web width. The compressive stress is given as

$$\sigma_c = 534 \left(\frac{F_{uw}}{442} \right) \Delta \quad (2.13)$$

$$\text{for} \quad 0 \leq F_c \leq 166 \left(\frac{F_{uw}}{442} \right)$$

$$\sigma_c = \left(\frac{F_{uw}}{442} \right) (-146 + 12.3\Delta - 330\Delta^{1/2} + 727\Delta^{1/3}) \quad (2.14)$$

$$\text{for} \quad 166 \left(\frac{F_{uw}}{442} \right) < \sigma_c$$

where, F_{uw} is ultimate tensile strength of the beam web.

(a) deformation of the beam web in the compression region

(b) internal force distribution in the beam web resulting from moment resistance of the connection

Figure 2.3: Deformation and Internal Force Distribution for the Beam Web

Frye and Morris (1976) have developed a polynomial model that gives the moment rotation curves for single web angle connections, double web angle connections, header end plate connections, top and seat angle connections, end-plate connections without column stiffeners, end-plate connections with column stiffeners, and T-stub connections, (Chen, 1991) that are empirically based. The general form of the moment rotation polynomial model is given as,

$$\theta_r = C_1(KM)^1 + C_2(KM)^3 + C_3(KM)^5 \quad (2.15)$$

where, K is a standardization parameter dependent upon the connection type and geometry; and C_1 , C_2 , and C_3 are curve-fitting constants. The value of the C_1 , C_2 , C_3 , and K are defined for flexible end plate connections as:

$$C_1 = 5.1 \times 10^{-5}$$

$$C_2 = 6.2 \times 10^{-10}$$

$$C_3 = 2.4 \times 10^{-13}$$

$$K = d_p^{-2.3} t_p^{-1.6} g^{1.6} t_w^{-0.5}$$

The geometric parameters illustrated in Figure 2.4 required for the calculation of K must be expressed in inches and are defined as,

d_p = length of the end plate

g = gauge distance

t_p = end plate thickness

t_w = beam web thickness

Figure 2.4: Geometric Parameters Required for Moment Rotation Curves for FEPC

Ang and Morris (1984) used a power model curve fit to the same experimental data, Chen (1991), for the moment rotation curves, which has the general form given by Equation 2.16.

$$\frac{\theta}{(\theta_r)_0} = \frac{KM}{(KM)_0} \left[1 + \left(\frac{KM}{(KM)_0} \right)^{n-1} \right] \quad (2.16)$$

The values for $(\theta_r)_0$, $(KM)_0$, n and K are:

$$(\theta_r)_0 = 7.04 \times 10^{-3}$$

$$(KM)_0 = 186.77$$

$$n = 4.32$$

$$K = d_p^{-2.41} t_p^{-1.54} g^{2.12} t_w^{-0.45}$$

2.2 Behaviour of Extended End Plate Connections

Extended end plate connections are considered to be rigid and transfer moment and shear from a beam to a column flange through a bolt group. A typical connection which is not subject to load reversal is shown in Figure 2.5. Generally, these connections are designed using a lower bound theorem approach in which the designer

first assumes a logical distribution of internal forces that is in equilibrium with the external loads. The component is then designed to provide adequate resistance and ductility.

The shear is assumed to be carried by all bolts equally. The moment is expressed as a force couple at the flanges. The tensile force is transmitted by two rows of bolts about the tension flange of the beam to the column flange. These bolts are subjected to tension, prying action and shear. The compression force is transmitted in bearing through the column flange to the web. Stiffeners are designed, if required, for any force in excess of the column's ability to carry the tensile or compression force resulting from the beam flange.

Figure 2.5: Extended End Plate Connection

The load deformation characteristics can be described using the Frye-Morris polynomial model (1976) in which the moment rotation curves are given by Equation 2.15, and where the coefficients for joints for columns without stiffeners are,

$$C_1 = 1.83 \times 10^{-3}$$

$$C_2 = -1.04 \times 10^{-4}$$

$$C_3 = 6.38 \times 10^{-6}$$

$$K = d_g^{-2.4} t_p^{-0.4} d_b^{-1.5}$$

where, C_1, C_2, C_3 and K are curve fitting coefficients, and for column with column

stiffeners are,

$$C_1 = 1.79 \times 10^{-3}$$

$$C_2 = 1.76 \times 10^{-4}$$

$$C_3 = 2.04 \times 10^{-4}$$

$$K = d_g^{-2.4} t_p^{-0.6}$$

The geometric parameters illustrated in Figure 2.6 required for the calculation of K must be expressed in inches and are defined as,

d_b = diameter of the bolts

d_g = the distance between the upper and lowest row of bolts

t_p = end plate thickness

Figure 2.6: Geometric Parameters Required for Moment Rotation Curves for EEP

2.3 Design Criteria, Assumptions and Resistances

2.3.1 Design Criteria and Assumptions

The Canadian standard for Limit State Design of Steel Structures, CAN/CSA-S16.1-M89 is used in conjunction with rational analysis and design procedures and equations by Kriviak and Kennedy (1984) (flexible end plates), Lesik and Kennedy (1988) and Jiang (1995), (eccentrically loaded welds), and Kulak, Fisher and Struik (1987), (prying action of high strength bolts) and internal force distributions consistent with the lower bound theorem to size both flexible and extended end plate connections. Designing the connection with CAN/CSA-S16.1-M89 ensures that the connection has sufficient resistance and ductility. The failure modes for which the connections are designed for are listed in Table 2.1. The section numbers shown give the section in the thesis where a description of the resistance calculation corresponding to that failure mode is made. Flexible end plate connections are designed either for shear or as a flexible inelastic connection (Kriviak and Kennedy (1984) model) if a beam end rotation is specified. The first case assumes zero rotation, equal shear in all bolts and a distribution of tension in the bolts due to the moment caused by the shear acting with an eccentricity of the end plate thickness that is linear and has the neutral axis located at midlength of the plate. The extended end plate connections are designed for the load distributions described in section 2.2.

Table 2.1: Failure Modes

Flexible End Plate Connections	Extended End Plate Connections
beam web <ul style="list-style-type: none"> • gross shear (including copes) (3.3.1.5) • net shear about the end plate (3.3.1.5) • shear tension capacity (Kriviak and Kennedy (1984)) (3.3.1.7) 	beam web • gross shear (3.3.1.15)
end plate <ul style="list-style-type: none"> • gross shear (3.3.1.5) • block shear (3.3.1.9) • net shear (Kriviak and Kennedy (1984)) (3.3.1.7) • bearing (3.3.1.5, 3.3.1.7) 	end plate <ul style="list-style-type: none"> • block shear (3.3.1.9) • bearing (3.3.1.15)
bolts <ul style="list-style-type: none"> • shear (3.3.1.5) • shear tension prying action (3.3.1.7, 3.3.1.12) 	bolts <ul style="list-style-type: none"> • shear (3.3.1.15) • shear tension prying action for the bolt group about the tension flange (3.3.1.14)
welds <ul style="list-style-type: none"> • shear (3.3.1.5) • eccentrically loaded weld group (3.3.1.11) • interaction equation (Kriviak and Kennedy (1984)) (3.3.1.7) 	welds <ul style="list-style-type: none"> • shear (3.3.1.15) • eccentrically loaded weld group (web) (3.3.1.11)
supporting member • bearing (3.3.1.5)	supporting member <ul style="list-style-type: none"> • web crippling (3.3.1.14) • flange (4 - hinge mechanism) (3.3.1.14) (stiffeners design)

The following should be noted:

- 1) For flexible end plate connections that are analysed and designed using Kriviak and Kennedy's inelastic model which accounts for the deformation of the connection and calculates the end rotation of the beam. A limit is placed on the location of the neutral axis which This guards against excessively flexible connections with large rotations and is specified section 3.3.1.6.
- 2) Stiffeners are designed if required. Refer to section 3.3.1.14.

The geometric design portion of the program makes use of detailing rules within the standard and data files for required clearances for impact wrenches, standard gauge distances, bolt data, section data, plate and weld sizes to ensure that the connection is detailed for Canadian fabrication and erection practices. Specific coping details and data related to locating the end plate are given in sections 3.3.1.2 and 3.3.1.5, respectively.

2.3.2 General Resistances: CAN/CSA-S16.1-M89

A list of general resistances as specified in CAN/CSA-S16.1-M89 that are used throughout the program End Plate Connection Design are given for each component in the sections which follow;

- **Beam**

Beam shear is calculated for the gross web area (accounting for coping details if required), and for the net area defined by the length of the end plate according to § 13.4.1.1, where,

$$V_r = \phi A_w F_s \quad (2.17)$$

where, A_w is shear area as described, and F_s is given as,

$$F_s = 0.66 F_y \quad \text{for} \quad \frac{h}{w} \leq 439 \sqrt{\frac{k_v}{F_y}} \quad (2.18)$$

$$F_s = 290 \frac{\sqrt{F_y k_v}}{(h/w)} \quad \text{for} \quad 439 \sqrt{\frac{k_v}{F_y}} < \frac{h}{w} \leq 502 \sqrt{\frac{k_v}{F_y}} \quad (2.19)$$

$$F_s = 290 \frac{\sqrt{F_y k_v}}{(h/w)} \quad \text{for} \quad 502 \sqrt{\frac{k_v}{F_y}} < \frac{h}{w} \leq 621 \sqrt{\frac{k_v}{F_y}} \quad (2.20)$$

$$F_s = \frac{180000 k_v}{(h/w)^2} \quad \text{for} \quad 621 \sqrt{\frac{k_v}{F_y}} < \frac{h}{w} \quad (2.21)$$

where, $k_v = 5.34$ as a/h is taken as infinity.

- **Supporting Member (Column, Girder)**

The bearing resistance of the column flange or web, or girder web is given by, § 13.10.1(c) as,

$$B_r = 3\phi t d n F_u \quad (2.22)$$

The resistance of the column web and flange for extended end plate connections is calculated from § 21.3 (a) and from equations which give the appropriate plastic mechanism capacity of the flange, and is detailed in section 3.3.1.14.

- **End Plate**

The general resistances for flexible and extended end plate calculations include,

1. bearing according to § 13.10.1(c) where the lesser of Equations 2.23 and 2.24 governs

$$B_r = 3\phi t d n F_u \quad (2.23)$$

$$B_r = \phi t n e F_u \quad (2.24)$$

2. shear through the gross section according to § 13.4.4

$$V_r = 0.5\phi L t F_u \quad (2.25)$$

where, L = two times the end plate length.

3. block shear according to § 13.2 and § 12.3.1

$$V_r = 0.85\phi A_{ne} F_u \quad (2.26)$$

where, the net area for segment normal to the shear is given by,

$$A_n = W_n t \quad (2.27)$$

and for segment parallel to the shear by,

$$A_n = 0.6 L_n t \quad (2.28)$$

and the total effective net area is

$$A_{ne} = \sum A_n \quad (2.29)$$

- **Bolts**

The resistance of bolts for bearing type connections includes;

1. shear resistance where the threads are not intercepted by the shear plane, according to § 13.11.2 (b)

$$V_r = 0.60\phi_b n m A_b F_u \quad (2.30)$$

which is modified by a factor of 0.7 if the threads are intercepted,

2. tensile resistance according to § 13.11.3

$$T_r = 0.75\phi_b n A_b F_u \quad (2.31)$$

3. interaction equation for shear and tension resistance according to § 13.11.4

$$\frac{V_f^2}{m^2} + \beta T_f^2 \leq 0.56\phi^2\beta(A_b F_u)^2 \quad (2.32)$$

where β is an interaction factor derived from test results

- $\beta = 0.69$ for A325 bolts, shear plane through shank
- $= 0.41$ for A325 bolts, shear plane through threads
- $= 0.56$ for A490 bolts, shear plane through shank
- $= 0.30$ for A490 bolts, shear plane through threads

The resistance of slip-critical (friction) type connections includes,

1. shear resistance according to § 13.12.2

$$V_s = 0.53c_1 k_s m n A_b F_u \quad (2.33)$$

2. interaction equation for shear and tension resistance according to § 13.12.3

$$\frac{V}{V_s} + 1.9 \frac{T}{n A_b F_u} \leq 1.0 \quad (2.34)$$

where, V_s = slip resistance of a bolted joint

• Welds

The basic resistances of fillet weld is calculated according to § 13.13.1, which refer to Table 3(a), which gives the shear resistance of the fillet weld as, the

lesser of

(a) base metal

$$V_r = 0.67\phi A_m F_y \quad (2.35)$$

or (b) weld metal

$$V_r = 0.67\phi_w A_w X_u \quad (2.36)$$

where,

$A_w = \frac{D}{\sqrt{2}}$, D = fillet weld leg size

A_m = effective size times length of weld

2.3.3 Resistance: Kriviak and Kennedy (1984)

The resistance for the bolt group, end plate, fillet welds and beam web in shear account for the inelastic behaviour of the connection and the corresponding internal forces including secondary forces (horizontal shear, prying action) that develop in the end plate. The appropriate ultimate strength formulations for each component follow:

- Bolt Group

The shear resistance for the bolt group is equal to the sum of all the resistances of each bolt row and is given as,

$$V_{rb} = \sum_{i=1}^{i=n} V_i \quad (2.37)$$

where, the shear that can be carried by the i th row of bolts in the tension region is given by,

$$V_i = 2\sqrt{[(0.75\phi A_b F_{up})^2 - B_i^2]\beta - H_i^2} \quad (2.38)$$

where,

$$B_i = H \tan \alpha \left(1 + \frac{d_h}{2a}\right) + \frac{M}{a} \left(1 + \frac{d_h}{l} + \frac{2a}{l}\right) \quad (2.39)$$

where, a = a distance of 1.25 ($i + D$)

$$M = \left(1 - \frac{H^2}{\cos \alpha P_{yp}^2} \right) M_{up} \quad (2.40)$$

$$H = H_{max} \quad (2.41)$$

and the shear that can be carried by the i th row of bolts in the compression region is given by Equation 2.30 and in both cases cannot exceed the bearing capacity of the end plate given as,

$$V_i = 2\phi t_p e F_{up} \leq 6\phi t_p d_h F_{up} \quad (2.42)$$

If four lines of bolts are required, the outside lines carry shear only and have a resistance equal to that given by Equation 2.30. Note, Equation 2.38 is a simple rearrangement of the tension shear interaction equation 2.32 that includes the effects of prying action and horizontal shear.

• End Plate

The shear resistance of the end plate is equal to the sum of all the resistances of the plate sections about each row of bolts (with length equal to the pitch) and is given as;

$$V_{rp} = \sum_{i=1}^{i=n} V_i \quad (2.43)$$

The shear that can be carried by the i th segment of plate in the tension region is given by,

$$V_i = 2t_p(p - d_h) \sqrt{\frac{1}{3} \left(\phi^2 F_{up}^2 - \frac{H_i^2}{(p - d_h)^2 t_p^2} \right)} \quad (2.44)$$

The shear that can be carried by the i th segment of plate in the compression region is given as,

$$V_i = 2\phi p t_p 0.55 F_{yp} \quad (2.45)$$

• Fillet Welds

The shear resistance of the fillet weld group is equal to the sum of all the

resistances of the weld segments for each plate section with length p (centred about each bolt row) and is given as;

$$V_{rw} = \sum_{i=1}^{i=n} V_i \quad (2.46)$$

where,

$$V_i = 1.34\phi p \frac{D}{\sqrt{2}} X_u + T_i - \frac{1.2T_i^2}{0.67\phi A_w X_u} \quad (2.47)$$

The throat area of two fillet welds over a length equal to the pitch is,

$$A_w = \frac{2}{\sqrt{2}} Dp \quad (2.48)$$

This is derived from the parabolic interaction developed by Kriviak and Kennedy (1985), for fillet welds subjected to longitudinal and transverse shear and is expressed as,

$$1.2 \left(\frac{V_T}{V_U} \right)^2 - \left(\frac{V_T}{V_U} \right) + \left(\frac{V_L}{V_U} \right) = 1.0 \quad (2.49)$$

where,

V_L = longitudinal shear force

V_T = transverse shear force

and V_U is governing shear resistance of the fillet weld where failure occurs through the throat of the weld metal and is given by Equation 2.35. The term V_L is V_i and V_T is T_i in Equation 2.47.

● Beam Web

The shear resistance of the beam web is the sum of the shear resistances of web segments with length $l_p/50$ and is given as;

$$V_{rw} = \sum V_i + V_{ic} + V_c \quad (2.50)$$

where the shear resistance in the tension region is given by,

$$V_i = w \frac{l_p}{50} \sqrt{\frac{1}{3} \left[(1.14\phi F_{yw})^2 - \left(\frac{T_i}{w \frac{l_p}{50}} \right)^2 \right]} \quad (2.51)$$

The shear resistance of the web segments in the compression region is given by,

$$V_{i_c} = 0.66\phi \frac{l_p}{50} w F_{yw} \quad (2.52)$$

The shear resistance of the portion of the web above the end plate is given by,

$$V_c = 0.66\phi d_t w F_{yw} \quad (2.53)$$

If the top flange is coped then V_c is equal to zero.

Chapter 3

Design Philosophy, Program Architecture and Functions, Graphic Interface

The program architecture and functions, algorithm rules for redesign for the program end plate connection design (EPCD), and the graphic interface for data input and design review are discussed in this chapter. The program is written to function as a submodule within the computer program SCADD. The necessary design data and fabrication constraints (preference for plate, angle, bolt size and strength) can be selected by the user from the program graphic interface according to the user's design requirements. The program EPCD designs both flexible and extended end plate connections and displays the results of the designed connection in terms of an engineering report, materials report, moment and rotation curves, connection cost table and a three dimensional rendition that can viewed from any orientation. The program only considers orthogonal beam-to-beam and beam-to-column connections. Descriptions for the design procedure of each connection follows.

3.1 End Plate Connection Design Philosophy and Program Architecture

3.1.1 Flexible End Plate Connection Design

Figure 3.1 illustrates the flow chart for the flexible end plate connection design process. The end plate connection design can be designed with or without consideration of secondary forces which are calculated in accordance with the stiffness model developed by Kriviak and Kennedy (1984) as described in section 2.4.1. The geometric design and limits to connection size are based on the orientation of the connecting members, fabrication practices in Canada and erection tolerances. Given the internal forces, the bolts, plate, welds and beam web are analyzed, and the end plate sized, the size and number of bolts and welds selected to provide adequate resistance. A detailed discussion of the changes in number, size, and strength of the bolts, plate, and welds for redesign follows:

• Bolts

The shear strength and the combined shear, tension and the prying action capacity of bolts where the shear plane passes through the shank or threads are calculated in the program for the appropriate bolt size. If the resistance of the bolts is less than the factored load, then:

1. The length of end plate is increased to accommodate an additional row of bolts. The connection is reanalyzed and compared with the factored load.
2. If the end plate reaches its maximum size and the resistance of the bolts is less than the factored load, then the connection is checked to see if it can accommodate 4 lines of bolts. The gauge distance for this extra set of bolt lines is set to the values specified as the usual gauges on page 6-158 of the

Handbook of Steel Construction (CISC, 1989). If 4 lines are possible, then the number of rows is increased until the resistance is satisfactory. If 4 lines are not possible or the required resistance cannot be achieved, then the bolt diameter is increased for the entire group.

3. If the maximum diameter and number is reached and the resistance is still not satisfactory, then the grade is changed from A325 to A490 and the diameter is reinitialized at the smallest value. The number of the bolts within the bolt group remains same. The diameter is increased until satisfactory resistance is achieved.
4. If the resistance is still not satisfactory then a message is displayed which instructs the user to select a different connection. This is not likely to occur for realistic connections.

• End Plate

The shear strength, bearing strength and block shear capacity of the end plate are calculated. If one of these resistances is less than the factored shear, then the end plate is resized as specified below and the resistance recalculated.

1. Increase the length of the plate to accommodate additional rows of bolts.
2. If the plate length reaches the maximum possible value and the resistance is less than the factored shear, then the thickness of the plate is increased.
3. If the maximum thickness of the plate is reached (25 mm for flexible end plate connections, 40 mm for extended end plate connections), and the resistance is still less than the factored shear, then a message is displayed instructing the user to select a different connection.

• Welds

The shear resistances of the welds connecting the end plate to the beam and the

stiffeners for the supporting member are calculated and checked according to the CAN/CSA-S16.1-M89. The eccentric load capacity for the weld group is calculated using the strength formulations from Lesic and Kennedy (1984). If the secondary forces are considered, the weld group capacity is also calculated using Kriviak and Kennedy (1984) interaction equation. If the resistance of the welds is less than the factored loads, the length of the connection is increased to accommodate an additional row of bolts (pair). The resistance is re-evaluated. If the maximum length of the connection is reached and the resistance is still not satisfactory, then the size of the fillet welds is increased incrementally and the resistance re-evaluated up to a maximum weld size of 30mm. If the resistance is still not satisfactory then a message is displayed instructing the user to select a different connection.

- **Beam Web**

The shear resistance of the beam web is calculated for both coped (if required) and un-coped beams. If the shear resistance of the beam web is less than the factored shear, then the length of the connection is increased to accommodate an additional row of bolts and the resistance recalculated. If the maximum length of the connection is reached, and the resistance is still less than the factored shear, then a message is displayed instructing the user to select a section with a larger beam web.

Figure 3.1: Flow Chart of Flexible End Plate Connection Design

3.1.2 Extended End Plate Connection Design

The design process of the extended end plate connection is illustrated in Figure 3.2 and is designed according to S16.1-M89 (CSA, 1989) and the recommendations given in the CISC Connections Design manual (CISC, 1990). The geometric design and limits to the connection size are based on the orientation of the connecting elements, fabrication practices in Canada and erection tolerances. The number, size, and length of bolts, the size of the end plate and welds are revised according to section 3.1.1 if the resistance is not adequate. Stiffeners are designed if required for the web of the supporting column to resist excessive deformation of column flange and tearing of the weld between the end plate and beam flange; and to give sufficient compressive resistance to the web to prevent local yielding and buckling that may result from the compressive force from the beam flange. The stiffeners are sized to give the same class of behaviour as the beam flange. The welds transferring force to the stiffeners from the flange and to the web are designed appropriately.

3.2 Graphic Interface and Main Menu Functions

The main menu of SCADD consists of twenty buttons at the top of screen that can be selected by the mouse during the operation of the program and is shown in Figure 3.3. These are divided into two groups, one for system operation, system information and development of graphics for new screens; and one for modular or integrated connection design.

Figure 3.2: Flow Chart of Extended End Plate Connection Design

3.2.1 System Operation and Information Buttons

The functions of the system operation and information buttons, Srna (1995) are listed below.

System info Displays various information about the HOOPS, Wiegard et al (1992), database. The "Picture" is the HOOPS top segment and represents the whole application window. The segment tree can be displayed starting from any segment in the database hierarchy. The whole path of a segment drawn on the screen is displayed by "show segment".

Files Lists or views files.

Multiple screen Stored segments or metafiles can be displayed simultaneously, viewed or checked if they exist.

Selective photo Creates PostScript file of a selected segment, with portrait or landscape orientation, and is assigned a user-specified name. Similarly, it creates a metafile (a picture capture redisplayable by HOOPS), or a file in HPGL format (plot file). A segment can be stored in a temporary database of HOOPS and displayed by "Multiple screen".

Snapshot Creates a PostScript file of a predefined name in landscape orientation.

Add text Provides the capacity to edit a note on the display screen.

Help Initially provides a label marking the position within the execution of the system. This label is used to call a viewer of an associated help file. Cross referencing depends on the type of the help files.

Exit Stops the execution of program SCADD.

Quit Exits the current or active screen, and has not been utilized in this thesis.

Continue Activating the continue button prompts sub-screens within a particular task. This button has not been utilized in this thesis.

3.2.2 Modular and Integrated Connection Design Buttons

A flow chart which illustrates the interactive data input/output for connection design is shown in Figure 3.4. The buttons shown in Figure 3.3 that are related to integrated connection design, multiple connections, integrated design and print results are currently not active. A description of the functions of the remaining buttons follows:

Job manager Activates the existing data files. The user must select the “default” data file. The return button will return the main menu screen.

Constraints Activates the fabricators constraints screen, shown in Figure 3.3, where preferred material properties bolt sizes, angles, plate thicknesses and welds can be selected and used as the initial default values for design.

Connection types Activates the connection types screen, shown in Figure 3.5, and allows the user to select the appropriate connection type. Beam-to-beam, beam-to-column flange or beam-to-column web may be selected for flexible end plate connection design. Beam-to-column flange is the only applicable connection for an extended end plate connection (moment resisting connection).

Connection details Activates the connection details screen shown in Figure 3.5, which enables the user to select the connecting members, the type of the connectors, the type of the connection (friction versus bearing), surface preparation and relative positioning of one member with respect to another.

Design of a single connection Activates the design of a single connection screen, shown in Figure 3.5, and enables the user to specify the loads, the type of analysis

and to review the design which includes an engineering report, materials report, moment rotation curves, the cost of the fabricated connection and a 3D rendering of the designed connection. The design and review is controlled by the following eight buttons, Loads, Rotation, Design, Engineering Report, Material Report, Analysis, Evaluation and Drawings. The drawing button is currently inactive but is reserved to convert the design connection data into an Auto-CAD format to illustrate the appropriate shop drawing. The remaining buttons in the lower right hand sub-window are used to manipulate the structural image which can be rotated, panned and enlarged.

Loads Activates the information sub-window, left hand side of the screen, where the user is prompted to enter appropriate un-factored and factored loads, and whether or not load reversal is going to occur. Refer to the example window shown in Figure 3.5. These loads can be accepted or redefined. Once accepted the design can proceed.

Rotation Inputs the maximum end rotation of the beam (in radians) for flexible end plate connections where secondary forces are being considered and a non-linear analysis is specified. Note; (1) the maximum end rotation for most beams is not likely to exceed 0.03 radians, (2) the maximum end rotation for this program has been set to 0.08 radians, and (3) if the end rotation is excessive then the connection length is increased. The rotation can be accepted or redefined and once accepted the design can proceed.

Designs Simply executes the appropriate analysis and design algorithms based on the previously defined connection. The designed connection (3D rendering) and the engineering report are illustrated automatically once the design procedures is complete.

Engineering report The engineering report shown in the information window gives all the resistances and ratios of factored loads to resistances for each probable failure mode. An examination of this table will identity the critical element in

the design of the connection and may lead to better design algorithms (dynamic resizing of the end plate or renumbering of connectors) in future versions of the program. Currently, the redesign rules are predetermined.

Materials report The materials report lists by category the sizes and material strength for all plates, bolts, welds and stiffeners (if required). This data are used subsequently in determining the cost of the fabricated connection.

Analysis Executes a function which calculates and illustrates moment rotation curves for both flexible and extended end plate connections using the Frye-Morris' polynomial model, the Ang-Morris' power model and Kriviak-Kennedy model, Kriviak-Kennedy (1984) where applicable. This information will become relevant for the integrated analysis design program where advance structural analysis programs are incorporating connection load determination responses.

Evaluation Displays a confidential basic unit cost table where the fabricator can enter their current costs. Accepting these unit and below costs automatically displays in the same window the cost of the designed connection. The time estimates for various tasks, fitting, welding, machining and general labour are give by Wells (1991) as a function of connection type, beam size and experience. These time estimates can be defined by the user by selecting the corresponding windows and entering the appropriate number in the prompt space but will be redefined to Well's estimate if the value is unrealistic. The total button must be selected to re-evaluate the cost if any changes are made to the time estimates.

EXIT	HELP	JOB MANAGER	MULTIPLE CONNECTIONS	INTEGRATED DESIGN	PRINT RESULTS	SELECTING MATHS	SNAPSHOT	QUIT	
DRAWING INFO		CONSTRAINTS		CONNECTION TYPES		DESIGN OF A SINGLE CONN.		MULTIPLE DESIGN	
Run Step 10 with New 40 40						ADD TEXT		CONTINUE	

Preferred Bolt

Hole Type

☐ Punched

☐ Drilled

Type

☐ A502

☐ A508M

☐ A490M

Size

M16	M20	M22
M24	M27	M30
M36		

pitch is 80 mm CHANGE

gauge is 80 mm CHANGE

gauge offset is 80 mm CHANGE

Thread interruption? YES CHANGE

Preferred Angle

Shape

Available Thicknesses	30	25	20	15	10
L200x250					
L200x180					
L200x150					
L150x150					
L150x100					
L125x125					
L125x100					

Grade

230	280	300	360	400
-----	-----	-----	-----	-----

Preferred Weld

Type

☐ FILLET

☐ BACK

☐ BEVEL

☐ FLARE-BEVEL

☐ FLARE-J

☐ SQUARE

Size

5	6	8
10	13	16
20	25	30

Preferred Plate

Edge Treatment

☐ FLAME-CUT

☐ SHEARED

Grade

230	280	300
360	400	450
700		

Thickness

5	6	8
10	12	14
16	18	20
22	25	28
30	35	40

Current File : default Messages :

EXIT	HELP	JOB MANAGER	MULTIPLE CONNECTIONS	INTEGRATED DESIGN	PRINT RESULTS	SELECTING MATHS	SNAPSHOT	QUIT	
DRAWING INFO		CONSTRAINTS		CONNECTION TYPES		DESIGN OF A SINGLE CONN.		MULTIPLE DESIGN	
Run Step 10 with New 40 40						ADD TEXT		CONTINUE	

<p style="text-align: center;">Beam to Column Web</p> <p style="text-align: center;">Supporting Member</p>	<p style="text-align: center;">Beam to Column Flange</p> <p style="text-align: center;">Supporting Member</p>	<p style="text-align: center;">Beam to Beam</p> <p style="text-align: center;">Supporting Member</p>			

Current File : default Messages :

Figure 3.3: Constraint and Connection Type Screens

Figure 3.4: Flow Chart of Interactive Data Input/Output for EPCD

Date: Jan 15, 1995 Time: 12:00

Connection Details

Connecting Member	W810X219	Bolts	Welded
Supporting Member	W810X241	Bolts	Welded

SHEAR		MOMENT	
SINGLE ANGLE	DOUBLE ANGLE		
SINGLE PLATE	DOUBLE PLATE		
END PLATE	YES		
UNSAFE SEA	SAFE SEA		

Bolt Design		Surface Preparation		step is 30 mm		CHANGE
Friction		Class A		gap is 20 mm		CHANGE
Bearing		Class C		Orientation		CHANGE

Member Selection

Shape	Available Weights					
W810	241	217	185	174	145	140
W810	121	111	101	91	84	82
W810	72					
W810	241	185	174	145	140	138
W810	121	101	101	91	82	72
W810	72	72	68			
W810	177	151	144	121	113	108

Grade:

Current File : default Messages :

Date: Jan 15, 1995 Time: 12:00

Current Design Loads

Shear Unfactored = 350.0 kN Factored = 450.0 kN
Moment Unfactored = 250.0 kNm Factored = 300.0 kNm
Reversal Reversal is no

Engineers Report

Analysis

Drawings

Design

Materials Report

Evaluation

Rotation

Loads

1	2	3	4	5	6	7	8	9	10
11	12	13	14	15	16	17	18	19	20
21	22	23	24	25	26	27	28	29	30

Current File : default Messages :

Figure 3.5: Connection Details and Design Screens

3.3 Program Functions

The file structure for SCADD with an exploded view of the program functions for EPCD and related functions for developing the 3D scaled rendition of the connection, the moment rotation curves, and the connection cost table is shown in Figure 3.6. The main program (SCADD) links the SCADD user interface module with other existing connection design modules (simple, base plate), EPCD, "End Plate Connection Design" module, and to functions related to generating curves, tables, or graphic images. Other modules for the design of moment connections, tension connections, cover plate connections, gusset plate and splice plate connections; and for the design of any arbitrary connection can be added to SCADD as they are developed. The letter group is listed for completeness.

3.3.1 EPCD - End Plate Connection Design

The program function calls three functions, input, analysis_design and data_3Dimage, which retrieve and send data to SCADD and to the functions that create graphic images, curves and tables; and designs the appropriate flexible or extended end plate connection as prescribed.

3.3.1.1 Input

This function reads the data entered through the SCADD interface windows and data files for the section geometric data, clearance, impact wrench geometry, bolt data, available plate sizes, and material properties for welds and different grades of steel.

Details of the input data files userpref and default.in from the SCADD interface

Figure 3.6: File Structure

windows are presented in the following pages.

- **userpref**

The data for **userpref** are entered from the fabricator's constraint screen and are used as default values in the initial design. The corresponding file structure is listed for completeness.

1. bolt size, type, method of making bolt hole, pitch, gauge
2. weld size, type
3. angle size, grade
4. plate thickness (limited to the range of 5 to 25 mm for FEPC and 5 to 40 mm for EEPC), grade, edge preparation

- **default.in**

The data for **default.in** is entered from the connection type, connection details and the single connection design screens, and has the following general format. Specific details are identical for both the flexible and extended end plate connections with the acronyms FEPC and EEPC, respectively.

1. connecting member shape, mass, grade, alignment, and orientation
2. connecting member half depth, half depth (repeated), left flange width, right flange width, flange thickness, web thickness, k, k_1 , member length, and the line number of the section within the CISC section geometry data file
3. supporting member shape, mass, grade, alignment, and orientation
4. supporting member half depth, half depth (repeated), left flange width, right flange width, flange thickness, web thickness, k, k_1 , member length, and the line number of the section within the CISC section geometry data file

5. connection geometry (beam-to-column flange (FEPC, EEPC), beam-to-column web (FEPC), beam-to-beam (FEPC)), connection fixity (shear (FEPC), moment (EEPC)), connection type (end plate), connection gap (end plate thickness), connection step
6. bolt design (friction, bearing), bolt surface
7. connector for connecting member (welded)
8. connector for supporting member (bolted)
9. service and factored shear loads (FEPC, EEPC)
10. service and factored moments (EEPC)
11. load reversal (yes, no, applies to EEPC only)
12. unit cost for fitters, welders, machinists, general labour, bolts, welds, and plates
13. fabrication time for fitters, welders, machinists, and general labour (based on estimates provided by Wells (1991))
14. rotation (0.0 to 0.8 radians (EEPC))

3.3.1.2 Analysis design

This function calls functions which perform the geometric design (calculation of clearances and fit, maximum and minimum end plate size, location of end plate, copes where required), strength design (determination of the number of bolts, length and size of welds, and end plate size), calculation of resistances and redesign if the resistances are not satisfactory, the calculation of the ratio of the appropriate loads to resistances. In addition the data for engineering and materials reports are written to a default file, for both flexible and extended end plate connections. Contained within this function are:

1. the calculations for coping details which are illustrated in Figure 3.7 and are done prior to calls to the functions listed on Figure 3.6.
2. the calculations for the shear tension interaction equations for bearing and slip resistance bolted connections given by clauses 13.11.4 and 13.12.3. These values are returned to the function report.

3.3.1.3 Unit_strength

This function calculates the unit strength for beam web shear assuming it yields (kN/mm, § 13.4.1.1), end plate shear (kN/mm, § 13.4.4), bearing on the column and end plate (kN/mm, § 13.10.1(c)), shear or shear tension resistance of bolts for bearing and friction type connections (kN/bolt, § 13.11.2, § 13.11.3, § 13.11.4, § 13.12.2, § 13.12.3) and for fillet welds (kN/mm, § 13.13.3) are calculated in accordance with CAN/CSA-S16.1-M89, CSA (1989). The unit strength and the appropriate clause are referenced in brackets as indicated.

3.3.1.4 Web_shear

This function calculates the ultimate shear stress of the beam web for $a/h = \infty$ (without stiffeners) according to the § 13.4.1.1.

a) top cope of the beam

b) bottom cope of the beam

c) top and bottom
copes of the beam

d) left end right flange copes on
both top and bottom of the
beam

Figure 3.7: Copes for Beam-to-Beam Flexible End Plate Connection

3.3.1.5 Plate_size_and_shear

This function is applicable to flexible end plate connections only and calculates:

- the maximum and minimum end plate width and length based on the section sizes specified, a minimum 2 rows of bolts, standard gauge distances, clearances for the impact wrench and end distances are illustrated in Figure 3.10.
- the required end plate length for each applicable failure mode (bolt shear, weld shear, shear on the net section of the beam web) and adjusts the plate length to accommodate a standard bolt pattern. If the required length as calculated for bolt shear, weld shear, or net section web shear is greater than the maximum, then increase the number of lines to 4, the bolt diameter and strength as per section 3.1.1, the fillet weld size increased appropriately or a message send to increase the web size for the beam, respectively.
- the mid-depth of the beam and centres the end plate about that point.
- the resistance and ratio of the factored shear to resistance for each failure mode listed for the preliminary end plate configuration.

3.3.1.6 Con_moment

This function calculates 40 pairs of points which describe the moment rotation curve for flexible end plate connection according to the nonlinear model developed by Kriviak and Kennedy (1984). Refer to section 2.1 for details. Based on this analysis the geometry of the end plate may be modified as follows:

- If the horizontal distance between the centre of the beam web and the adjacent line of bolts is less than that required for clearance of an impact wrench, the

horizontal distance is set equal to that value and the plate width adjusted accordingly.

- If the bottom flange of the beam bears on the supporting member as it rotates then the plate length is increased incrementally by one row of bolts to accommodate a standard bolt pattern.
- If the end plate interferes with bottom flange as a result of increasing its length then the plate is shifted to the upper most position on the beam as shown in Figure 3.8.
- If the neutral axis is located above $4/5$ ths of the beam depth from the bottom flange the user is instructed to select a different connection type.

3.3.1.7 Shear_strength

This function calculates the shear strength of the bolt group (including prying action and secondary forces due to horizontal membrane forces that develop in the end plate as the connection rotates and deforms), shear strength of the end plate through the two bolt lines adjacent to the web, shear strength of the fillet welds and the shear strength of the net beam web (about the end plate) according to the nonlinear model developed by Kriviak and Kennedy (1984). If the resistance for any one of these failure modes is insufficient then the connection is modified according to the design philosophy outlined in section 3.1.1. Depending on the modifications the program returns to `unit_strength` or `plate_size_and_shear` and reanalyses (possibly redesigns) the connection. If no further modifications are necessary then the program continues to the next function `bolt_coordinates`.

a) end plate is located in the center of the beam

b) end plate is located in the top of the beam

Figure 3.8: Location of End Plate for FEPC

3.3.1.8 Bolt_coordinates_fepc

This function first checks and repositions the end plate according to the guidelines given in the Figures 3.8 and 3.9. The pitch is also checked to ensure that a minimum distance of three bolt diameters is maintained. The plate is resized if necessary. Subsequently, this function calculates the x, y coordinates of each bolt relative to the x and y axis which are located along the top flange of the beam and one edge of the end plate, respectively. The positive direction is indicated in Figure 3.9. These data are required for calculations for the function `block_shear`, `tension_prying`, and for `data_3Dimages` (development of the 3D graphic image).

3.3.1.9 Block_shear

This function calculates block shear resistance for both flexible and extended end plate connections for the only probable failure paths, as shown in Figure 3.10, according to clauses 12.3 and 13.2 of CAN/CSA-S16.1-M89.

3.3.1.10 Weld_data

This function retrieves all data with respect to weld geometry, weld metal strength, load location and line of action, and formats these data to be read by the function `weldecc`. The data has the following format;

1. number of weld groups which is equal to one. The eccentric load capacity which is calculated by `weldecc` is subsequently multiplied by two as two equal size fillet welds exist, one on each side of the beam web.
2. number of points required to define the weld group. This is always equal to two.

Figure 3.9: Location of Bolt Group and End Plate Dimensions for FEPC

Figure 3.10: Block Shear Failure Path Consideration

3. the y and z coordinates of these two points, where the y, z plane lies in the same plane as the beam web.
4. a number equal to one which indicates that the weld group is doubly symmetric.
5. a number equal to one which indicates that the load is perpendicular to the z -axis.
6. two sets of y, z coordinates which define the eccentricity of the load with respect to the weld group.
7. The strength of electrodes X_u (MPa) and the fillet weld size (mm).
8. The yield strength of the base metal F_y (MPa) as specified within the constraints screen.

3.3.1.11 Weldecc

The resistance of the eccentrically loaded weld group (connecting the end plate and beam web) is determined from an analytical model, Jiang (1995), which is based on the load deformation characteristic for fillet welds loaded at any angle, Miazga and Kennedy (1986) and Lesik and Kennedy (1988) and the following assumptions;

1. the connection remains rigid and rotates about an instantaneous centre under eccentric load, and
2. the ultimate strength of the weld group is reached when the ultimate shear strength and deformation of any weld segment is reached, and
3. the ultimate capacity of a fillet weld subjected to a tension-induced shearing force is the same as for a similar weld loaded in compression-induced shear.

Generally the eccentricity is very small (end plate thickness) and has little effect on the resistance of the weld. If the resistance is less than the factored load then the fillet weld size is increased as per section 3.1.1. Note, the grade of the weld is not changed if the resistance is unsatisfactory.

3.3.1.12 Tension_Prying

This function calculates the factored tensile force including the effects of prying action according to Kulak, Fisher, and Struik (1987), for any bolt along the effects of lines of bolts adjacent to the beam web for flexible end plate connections only.

The range of applicable end plate thickness is given by equation 3.1

$$t = \left(\frac{K P_f}{(1 + \delta \alpha)} \right)^{1/2} \quad \begin{array}{ll} \text{when } \alpha = 1.0, & t = t_{min} \\ \text{when } \alpha = 0.0, & t = t_{max} \end{array} \quad (3.1)$$

where,

$$K = 4b'10^3 / \phi p F_y \quad (3.2)$$

$$\delta = 1 - d'/p \quad (3.3)$$

$$\alpha = \left(\frac{K P_f}{t^2} - 1 \right) 1/\delta \quad (3.4)$$

The factored tensile force including prying action is given as,

$$T_f = P_f \left[1 + \left(\frac{b'}{a'} \frac{\delta \alpha}{1 + \delta \alpha} \right) \right] \leq T_r \quad (3.5)$$

where

P_f = applied factored tensile load per bolt (kN)

T_f = load per bolt including prying action at factored load(kN)

Q = prying force per bolt at factored load (kN), $Q = T_f - P_f$

T_r = factored tensile resistance of bolts (kN) = $0.67(0.75 A_b F_u)$

F_y = yield strength of flange material (MP_a)

- p = length of flange tributary to each bolt, or bolt pitch (mm)
- t = thickness of flange (mm)
- b = distance from bolt line (gauge line) to face of tee stem (mm)
- a = distance from bolt line to edge of tee flange, not more than $1.25b$ (mm)
- d = bolt diameter (mm)
- d' = nominal hole diameter (mm)
- $a' = a + d/2$
- $b' = b - d/2$
- n = number of bolts in tension
- δ = ratio of net area (at bolt line) to gross area at face of tee stem
- α = ratio of sagging moment at bolt line to hogging moment at stem of tee

If $T_f \leq T_r$, the end plate design is acceptable. Otherwise, the diameter of the bolts is increased the resistance recalculated. If the maximum diameter is reached, then the bolt diameter is set to the smallest specified value and the type to ASTM A490. The diameter is increased until the resistance is satisfactory. If the resistance is still unsatisfactory then a message is displayed instructing the user select a different connection type. Although it is not currently included in this function, the user may specify, if required, a thicker end plate to minimize the effects of prying action.

3.3.1.13 Bolt_length

This function calculates the required bolt lengths for both flexible and extended end plate connections. The grip length, equal to the thickness of the end plate plus the column flange or web thickness, or beam web thickness, is calculated and the appropriate bolt length selection from the table given on page 6-150 of the Handbook of Steel Construction (CISC, 1989).

3.3.1.14 Extended design

This function designs extended end plate connections (moment resisting type connection) and stiffeners for the supporting column web if required. The design steps are;

1. The service and factored tension forces in the flange of the beam, $T = M/(d_b - t_b)$,
 $T_f = M_f/(d_b - t_b)$ are calculated.
2. The applied bolt force corresponding to 2 or 4 lines of bolts for both service and factored loads are calculated, $P = T/4$ or $P = T/8$, $P_f = T_f/4$ or $P_f = T_f/8$, respectively.
3. Based on the specified bolt diameter, beam flange thickness, and wrench clearance, the standard bolt arrangement is determined (p , e , b' , d'). Note, the minimum gauge distance is three times the bolt diameter. The number of rows of bolts is set to either 3 or 4 depending on whether or not load reversal is specified refer to Figure 3.11.
4. The minimum end plate thickness is calculated according to the equations for prying action and the minimum width of the end plate is set to the beam flange width plus one end plate thickness or ten times the bolt diameter, whichever is greater.
5. Based on geometry the length of the end plate is calculated.
6. The tensile force in the critical bolts, including prying action, is calculated and the bolt is selected such as that the resistance is greater than T_f , according to section 3.3.1.12.
7. The shear tension (including prying action) interaction equation for both slip resistant and bearing type connections is checked (where applicable) and the

bolt size revised if required.

8. Stiffeners are designed for the column to carry or transmit loads that are in excess of the column webs capacity in compression or the column flange capacity in tension. The design loads, resistances and stiffener geometry are determined as follows:

Stiffeners opposite the tension flange

- (i) the tension force transmitted to the column flange is equal to tension force transmitted by the bolt group (including prying action).
- (ii) the flange capacity of the column is determined from the plastic mechanism model used to develop the prying action equation,

$$T = \frac{(1 + \delta\alpha)Mn}{b} \quad (3.6)$$

where,

$$M = \frac{1}{4}pt^2F_y \quad (3.7)$$

p = pitch, distance between bolts along the length of the column

n = number of bolts, (4 or 8)

b = distance from the centre line of the column web to the inside of the bolt hole for the adjacent line of bolts

- (iii) the force that must be transmitted by the pair of stiffeners from the column flange to the column web is

$$T_s = T_f - T \quad (3.8)$$

Stiffeners opposite the compression flange

- (i) the compression force is,

$$C_f = M_f / (d - t_b) \quad (3.9)$$

- (ii) the column web capacity is given as a modified version of clause 21.3 (a) as,

$$B_r = \phi w_c(t_b + 2t_p + 5k_c)F_{yc} \quad (3.10)$$

for class 1 and 2 webs, or as,

$$B_r = \phi \frac{640000}{(h_c/w_c)^2} w_c (t_b + 2t_p + 5k_c) \quad (3.11)$$

for class 3 and 4 webs.

(iii) the force that must be carried by the pair of stiffeners is

$$C_s = C_f - B_r \quad (3.12)$$

Design

(i) the design force for the pair of stiffeners is equal to the larger value of T_s and C_s , hence the required area is the larger of,

$$A_{required} = \frac{T_s}{\phi F_y} \quad \text{or} \quad A_{required} = \frac{C_s}{\phi F_y} \quad (3.13)$$

(ii) the stiffener is proportioned to give class 3 behaviour

$$\frac{b}{t} = \frac{200}{\sqrt{F_y}} \quad (3.14)$$

(iii) the corresponding required values for b and t are

$$b_{required} = \sqrt{\frac{100 A_{required}}{\sqrt{F_y}}} \quad (3.15)$$

$$t_{required} = \sqrt{\frac{A_{required} \sqrt{F_y}}{400}} \quad (3.16)$$

(iv) the maximum width of stiffener is equal $(b_c - w_c)/2$. If $b_{required}$ is greater than b_{max} then increase the thickness to achieve the required area.

(v) the standard plate thickness is selected such that $t > t_{required}$ and an appropriate value of b is selected (rounded up to the nearest 5 mm) so that $A > A_{required}$, where

$$A = 2bt \quad (3.17)$$

9. diagonal stiffeners for web panel shear are not designed. The unit force that the weld must transfer from the column flange to the stiffener is

$$V_f = T_s/2(b_c - 2k_{1c}) \quad (3.18)$$

Select the appropriate fillet weld so that V_r , kN/mm according to § 13.3.3 which is greater than V_f . The same weld size is used to connect the stiffener to the column web and is welded along its full length. Use the same fillet weld size for the compression stiffener, welded all around.

3.3.1.15 Extended_shear

This function is applicable to EEPC only and calculates:

- the shear resistance and slip resistance of the bolt group and increases the diameter, if required, to provide sufficient resistance.
- the resistance of the specified fillet weld size and resizes the weld, if required, to provide sufficient resistance. The resistance of the weld group, considered as an eccentrically loaded connection, is determined by the function `weldecc`.
- the shear resistance and bearing resistance of the beam web. A message is set to the user instructing them to select a new beam if either of resistances is unsatisfactory.
- the bearing resistance of the end plate and increases the thickness, if required, to provide sufficient resistance.

Note, the block shear capacity is determined by the function `block_shear`.

3.3.1.16 Bolt_coordinates_eepe

This function calculates the coordinates of the bolt group for extended end plate connection and creates the corresponding data file to be used for calculating the

Figure 3.11: Standard Bolt Templets for EEPC

block shear capacity and for drawing the 3D rendition of the connection. The axes for the coordinate system used to define the location of the bolts with respect to the end plate are shown in Figure 3.11.

3.3.1.17 Report

This function creates the data files called `default.report` and `default.report2` which contain the engineering report and materials report, respectively, for both flexible and extended end plate connection design. An example is illustrated in Figure 3.12.

3. cope and partial cope dimensions for the bottom flange of the connecting member (9 values)
4. connection type (end plate), connection fixity (shear (FEPC), moment (EEPC)), connector for connecting member (welded), connector for supporting member (bolted)
5. grade of steel for the angle, long leg length, short leg length, angle section, length, position, direction (for end plate connections this line is a series of zeros since angles are not used).
6. grade of steel for end plate, width, thickness, length, position and a value 0 or 1 which indicates whether or not stiffeners are required for the column in EEPC.
7. bolt diameter, grade, number of washers, grip, pitch, gauge, pitch offset, gauge offset, rows, number of lines of bolts for connecting member (this line appears a series of zeros as welds are used to connect the end plate to the beam)
8. bolt diameter, grade, number of washers, grip, pitch, gauge, pitch offset, gauge offset, rows, number of lines of bolts for supporting member
9. design message, which is either OK or specific instructions related to connection redesign for obscure cases not explicitly handled by the program

3.3.2 Generation of 3D Graphic Image

Member sizes, positions, orientation and coping details are used by the function `create_conn_members` which draws the supporting and connecting members in the appropriate 3D image window as illustrated in Figure 3.12. The gap of the connection is set equal to the thickness of the end plate. A brief description of the functions

required to complete the drawing of end plate connections is given in the following sections.

3.3.2.1 Create_conn_pieces

This function calls `insert_end_plate` and `insert_stiffeners` (if required for EEPC) and maps (position, orientation) of these 3D images onto specified common points within the global coordinate system.

3.3.2.2 Insert_end_plate

This function creates the local x, y, z coordinates and that defines the 3D rendering of the end plate for flexible and extended end plate connections, specifies the common point and draws the image.

3.3.2.3 Insert_stiffeners

This function creates the local x, y, z coordinates defining the 3D rendering of the stiffeners required for extended end plate connections, specifies the common points and draws the image.

3.3.2.4 Create_conn_connectors_ep

This function calls `insert_weld` and `insert_bolt`, locates and draws the appropriate connectors related to the coordinate system for the end plate. The 3D rendering for weld group is not drawn for clarity.

3.3.3 Determination of the Moment Rotation Curves

3.3.3.1 Show_m_r_curve

This function determines the appropriate scales and intervals and plots them for the moment rotation curves as illustrated in Figure 3.13. Also drawn, are the axis labels, legend, and the appropriate equations with calculated coefficients for the Frye-Morris polynomial model (1976), Ang-Morris power model (1984). The function calls `insert_m_r_curves` and `insert_sub_curves` to calculate and plot the applicable curves for the designed connection.

3.3.3.2 Insert_m_r_curves

This function retrieves the 40 x, y coordinates (pairs of points) from a data file created by `con_moment` which describes the moment rotation curve for flexible end plate connection with secondary force as determined by the Kriviak and Kennedy model (1984) and plots them.

3.3.3.3 Insert_sub_curves

This function calculates 40 values to describe the moment rotation curves as given by the Frye-Morris polynomial model (1976) and the Ang-Morris power model (1984) for flexible end plate connections and by the Frye-Morris polynomial model (1976) for extended end plate connections. The appropriate curves are then plotted.

Figure 3.13: Moment Rotation Curves Example

3.3.4 Evaluation-Connection Cost Tables

3.3.4.1 Show_conn_cost

This function draws the connection cost table for the designed connection (an additional category for stiffeners is shown for EEPC) as illustrated in Figure 3.14. Each labour task including the time required to complete the task, the materials required and the corresponding unit costs are listed. The fabricator may modify the default times given to complete specified tasks if they differ from their time estimates for that connection by activating the corresponding button. The default values are based on time estimates provided by CANRON, Wells (1991) as shown in Figure 3.15, 3.16, 3.17, and 3.18.

3.3.4.2 Unit_cost

This function allows the fabricator to input confidential unit cost data (tailored to the operation) for labour, and materials. To change any value, simply activate the redefine button followed by either the labour or materials buttons. Prompts will be displayed automatically requiring the user to enter their unit costs. Once accepted these become the default values and should only be redefined if required.

3.3.4.3 Time_fitter

This function is activated by selecting the time unit button associated with the fitter category which displays a window that prompts the fabricator for the total time estimate for this category of work. This value will be used to calculate the corresponding cost.

3.3.4.4 Time_welder

This function is activated by selecting the welder button which displays a window that prompts the fabricator for the total time estimate for this category of work. This value will be used to calculate the corresponding cost.

3.3.4.5 Time_machinist

This function is activated by selecting the machinist button which displays a window that prompts the fabricator for the total time estimate for this category of work. This value will be used to calculate the corresponding cost.

3.3.4.6 Time_general_labour

This function is activated by selecting the general labour button which displays a window that prompts the fabricator for the total time estimate for this category of work. This value will be used to calculate the corresponding cost.

3.3.4.7 Fab_estimate

This function determines the time required for all tasks associated with the fabrication of flexible and extended end plate connections, based on estimates provided by CANRON, Wells (1991), and categorizes them into one of the following groups, fitter, welder, machinist, and general labour.

Figure 3.14: Connection Cost Table Example

Figure 3.15: Labour Estimates for Welding and Fitting Headers (Wells, 1991)

Figure 3.16: Labour Estimates for Drilling (Wells, 1991)

2 of /de 2

PM-1 3½"x4" PHOTOGRAPHIC MICROCOPY TARGET
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Figure 3.17: Labour Estimates for Coping, Cutting, and Punching (Wells, 1991)

Figure 3.18: Labour Estimates for Bolting, Cleaning, and Marking (Wells, 1991)

3.3.4.8 Total cost

This function calculates the total cost of the designed connection and reports that value in the connection cost table.

The total cost of the labour to fabricate the connection is.

$$C_l = T_{fi}C_{fi} + T_wC_w + T_mC_{ma} + T_gC_g \quad (3.19)$$

where,

C_{fi} = unit cost of fitter (\$/h)

C_g = unit cost of general labour (\$/h)

C_l = total cost of labour (\$)

C_{ma} = unit cost of machinist (\$/h)

T_{fi} = time estimate for fitter (hour)

T_g = time estimate for general labour (hour)

T_m = time estimate for machinist (hour)

T_w = time estimate for welder (hour)

The total cost of the material required for flexible end and extended end plate connections with and without load reversal is,

$$C_m = nC_b + 2l_wC_{w1} + l_pw_pt_pDC_p + 8l_{ws}C_{ws} + l_{ps}w_{ps}t_{ps}DC_p \quad (3.20)$$

where,

C_b = unit cost for bolts (\$/bolt)

C_m = total cost of material (\$)

C_p = unit cost of plate (\$/kg)

C_w = unit cost of welder (\$/hour)

C_{w1} = unit cost of weld (\$/mm²)

C_{ws} = unit cost of stiffener (\$/kg)

D = density of steel (kg/m³)

l_p = plate length (mm)

l_{ps} = stiffener length (mm)

l_w = weld length for the beam (mm)

l_{ws} = weld length for the stiffener (mm)

n = number of the bolts

t_{ps} = stiffener thickness (mm)

w_p = width of the end plate (mm)

w_{ps} = width of stiffener (mm)

The total cost of the designed connection is,

$$C_t = C_l + C_m \quad (3.21)$$

where,

C_t = total cost of a designed connection (\$)

Chapter 4

The Designed Connection: Illustrations and Verification

The design logic for a variety of flexible end plate and extended end plate connection designs are detailed in sections 4.1 and 4.2, respectively. Complete traces of the program logic, calculations and output (engineering and materials reports, moment rotation curves, fabrication cost and a 3D rendering of the designed connection) are given. Calculated values were retrieved by tracing through the program by use of the XUPS debugger tool. The examples listed in the Table 4.1 illustrate the versatility of the program. Section 4.3 gives six comparisons with connection designs produced by CONXPRT, a design program supported by the American Institute of Steel Construction (AISC) and with design examples given in the Manual of Steel Construction Volume II: Connections (AISC, 1992) and with an example given in connection design manual entitled, "Connection for Steel Structures" (CISC, 1989).

Table 4.1: Example End Plate Connection Designs

Flexible End Plate Connection Designs	
Section	Description
4. 1. 1	Beam-to-Column Flange: Two Lines of Bolts
4. 1. 2	Beam-to-Column Web with Coped Beam Flanges; Bearing Type Connection
4. 1. 3	Beam-to-Beam with Copes; Friction Type Connection
4. 1. 4	Beam-to-Column Flange: Four Lines of Bolts
Extended End Plate Connection Designs (Beam-to-Column Flange Only)	
4. 2. 1	Bearing Connection
4. 2. 2	Bearing Connection with Load Reversal as per 4.2.1
4. 2. 3	Column Web Stiffeners Not Required
4. 2. 4	Large Moments: Four Lines of Bolts

4.1 Flexible End Plate Connection Designs

The following four examples illustrate the versatility of the program in designing flexible end plate connections for beam-to-column web, beam-to-column flange, and beam-to-beam connections that include all probable combinations of coping, bolt arrangements and performance type (bearing or friction).

4.1.1 Beam to Column Flange; Two Lines of Bolts

The engineering report in Figure 4.1 gives the strengths and ratios of factored load effects to strengths for each probable failure which includes bearing, block shear, beam shear, bolt shear-tension and prying action, and weld shear (direct and eccentrically loaded). A comparison of the end plate designs based on Kriviak and Kennedy (1984) which includes secondary forces and a better description of internal forces

in the bolt group (inelastic model) with that based on the CISC recommendations, CISC (1979), yields the following conclusions:

1. The designs are identical for this particular case, and
2. The ultimate shear capacity of the connection is governed by the bolts if the secondary forces are properly accounted for at the maximum probable rotation. Refer to the moment rotation curve illustrated in Figure 4.2 where the moment corresponding to 0.02 radians is approximately 34 kNm. The secondary forces are not considered in the CISC solution. It indicates considerable reserve capacity in the critical bolt within the group when evaluating the interaction equation for shear tension and prying action, and
3. The analysis of the eccentric weld capacity is the same for both and gives a ratio of factored shear to resistance of 0.72. This weld capacity governs the design in the CISC case.

The materials report in Figure 4.1 lists the sizes, lengths and numbers of plates, bolts, washers and welds where applicable, and as required for the connection. Figure 4.2 illustrates the moment rotation curves for the empirical power models of Frye and Morris (1976), Ang and Morris (1984) and the rational deformation model of Kriviak and Kennedy (1984). Currently, there isn't a rational explanation as to the reason for the differences in the moment rotation curves by Frye and Morris (1976) and Ang and Morris (1984), and those of Kriviak and Kennedy (1984). It can only be stated that the predictable moment rotation curves by Kriviak and Kennedy (1984) closely match the test results of Hafiz (1982) for points up to that where the bottom flange of the supported beam bears against the supporting column. The mean value and coefficient of variation for the test to predicted values for the corresponding moment are 1.06 and 0.10, respectively. Figure 4.2 also gives the basic unit fabrication cost for labour and material (which can be modified by the fabricator) and the

resulting connection cost.

A complete trace of the program logic and calculations for example 1, (defined below) follows.

Connecting Member: W530X150, 300W
Supporting Member: W610X195, 300W

Service load: $V_s = 250$ kN

Factored load: $V_f = 300$ kN

Connection type: bearing

Plate: $t = 10$ mm, 300W
edge: sheared

Bolt type: A325M, M20

pitch: $p = 80$ mm

gauge: $g = 90$ mm

Fillet weld type: $t_w = 6$ mm

Rotation: 0.02 radians

Thread: intercepted

End plate Connection

W610X195

W530X150

$d_c = 622$ mm

$d_b = 543$ mm

$b_c = 327$ mm

$b_b = 312$ mm

$t_c = 24.4$ mm

$t_b = 20.3$ mm

$w_c = 15.4$ mm

$w_b = 12.7$ mm

$k_c = 43$ mm

$k_b = 38$ mm

$k_{ic} = 25$ mm

$k_{ib} = 23$ mm

• Calculation of unit strengths: function unit_strength

Bolts: $V_{rbolt} = 0.42\phi_b m A_b F_u = 73445$ N/bolt

$T_{rbolt} = 0.75\phi_b A_b F_u = 131153$ N/bolt

Welds: $V_{rweld} = 0.67\phi_w / \sqrt{2} X_u = 914$ N/mm

Beam Web: $V_{rbeam} = 0.66\phi_w b F_y = 2281$ N/mm

Plate: $V_{rplate} = 0.50\phi_t F_u = 2025$ N/mm

$V_{bplate} = \phi_t e F_u = 137700$ N/bolt

Column: $V_{bsup} = 3\phi_t c d F_u = 443205$ N/bolt

- Calculation of shear capacity: function plate_shear_design

the number of bolts required: $rnbolt = V_f/V_{bplate} = 4$

$$l_p = 2e + (rnbolt/2 - 1)p = 228 \text{ mm}$$

$$top = (d_b - l_p)/2 = 157.5 \text{ mm}$$

$$dtop = (d_b - l_p)/2 + e = 191.5 \text{ mm}$$

$$vbolt = 2((l_p - 2e)/p + 1)V_{rbolt} = 440672 \text{ N} > V_f = 300 \text{ kN} \quad \text{ok}$$

$$vbear = 2((l_p - 2e)/p + 1)V_{bplate} = 826200 \text{ N} > V_f = 300 \text{ kN} \quad \text{ok}$$

$$vweld = 2l_p V_{rweld} = 416862 \text{ N} > V_f = 300 \text{ kN} \quad \text{ok}$$

$$vplate = 2l_p V_{rplate} = 923400 \text{ N} > V_f = 300 \text{ kN} \quad \text{ok}$$

$$\text{net } vbeam = l_p V_{rbeam} = 520059 \text{ N} > V_f = 300 \text{ kN} \quad \text{ok}$$

- Calculation of parameters for moment rotation curves: function con_moment

the clearance for the impact wrench, $D = 54 \text{ mm}$

$$l_{check} = 2D - d_h - 2t_w - w_b = 30.6 \text{ mm}$$

$$l_{clear} = (g - d_h - 2t_w - w_b)/2 = 21.6 \text{ mm} < l_{check} = 30.6 \text{ mm}$$

- therefore, set $l_{clear} = 30.6 \text{ mm}$

- The capacities for a 1 mm width of plate

$$M_{yp} = t^2 F_y / 6 = 5000 \text{ Nmm} \quad M_{pp} = t^2 F_y / 4 = 7500 \text{ Nmm}$$

$$M_{up} = t^2 F_u / 4 = 11250 \text{ Nmm} \quad I_p = t^3 / 12 = 83.33 \text{ mm}^4$$

- the maximum elastic deformation of the end plate is

$$aelas = 4M_{pp} l_{clear}^2 / (24EI_p) = 0.07$$

$$P_{yp} = tF_y = 3000 \text{ N/mm} \quad P_{up} = tF_u = 4500 \text{ N/mm}$$

$$h_{max} = V_{rbolt} = 73445 \text{ N}$$

- the maximum plastic deformation of the end plate is

$$aelasp = 4h_{max} M_{up} / (P_{yp}^2 p) = 4.59$$

$$l_i = l_p / 50 = 4.56 \text{ mm}$$

$$l_t = ((50 - 13) / 50) l_p = 168.72 \text{ mm}$$

- the maximum tensile deformation of the end plate:

$$def_{ten_{12}} = \tan \theta l_t = 3.37 \text{ mm}$$

$$\Delta = defcom_{12} = \tan\theta(l_p - l_t)/25.4 = 0.04 \text{ in}$$

$$\text{because } \Delta > 0.012(F_u/442)$$

$$\sigma_x = (-21.2 + 45.4\Delta - 241.0\Delta^{1/2} + 310.0\Delta^{1/3})(6.895)(F_u/442) = 284.89 \text{ MPa}$$

• the compressive deformation in the web of the beam is

$$dc = ((12 - 11)/12)defcom_{12} = 0.004 \text{ in} < 0.012(F_u/442)$$

$$\sigma_c = 2000(dc)(6.895)(F_u/442) = 54.62 \text{ MPa}$$

• determination of the location of the neutral axis

The end plate is subdivided into 50 equal length segments. The location of the neutral axis is determined interactively by assuming its location, calculating the corresponding compression and tension forces on each segment and comparing the totals to determine if equilibrium is satisfied. The first guess is at $l_p/50$ from the bottom of the plate and is incremented by one segment each successive iteration. Initially the total compression force is small. The process is terminated when this force exceeds the total tension force, which implies that the neutral axis lies within the last segment assumed for its location. The total force in the compression region is $c_{total} = \sum_{i=0}^{i=12} C_i = 249.8 \text{ kN}$

$$\text{The total force in the tension region is } t_{total} = \sum_{i=13}^{i=49} T_i = 247.6 \text{ kN}$$

$$c_{total} > t_{total}, \quad \text{ok}$$

where, the compression and tensile forces in each segment are calculated from the expressions given below:

$$c_i = 0.5l_i\sigma_c w_b$$

$$t_i = (tens_{i-1} + tens_i)l_i/2 \quad tens_i = (P_{yp}^2 dt^2)/(4M_{up}l_{clear} + 4M_{up}l_{clear}/(l_{clear}^2 + dt^2))$$

• the maximum deformation in tension region is $dt_{max} = 3.37 \text{ mm}$

• the position of the neutral axis is $r_{neutr} = 168.72 \text{ mm}$

• shear capacity on the beam web

$$v_{beam} = \sum_{i=13}^{i=49} v_i = w_b l_i \sqrt{\frac{1}{3} \left[(1.14\phi F_y)^2 - \left(\frac{T_u}{w_b l_i} \right)^2 \right]} = 439.8 \text{ kN}$$

- end moment calculation:

$$m_{total} = \sum_{i=13}^{i=49} M_i = T_i(50 - 13 - 0.5)l_i = 33.13 \text{ kNm}$$

- Calculation of secondary forces: function shear_strength

- the number of pairs of bolts in the tension region of the connection at the design load:

$$n_{t\text{bolt}} = (l_t - e)/p + 1 = 2$$

- determination of connection deformation

- the location of a row of bolts relative to the neutral axis is

$$l_0 = l_t - d - p(n_{t\text{bolt}} - 1) = 134.72 \text{ mm}$$

- the tensile deformation of the end plate is $del\text{bol}_0 = \tan\theta l_0 = 2.69 \text{ mm}$

$$l_1 = l_0 + 2p = 54.72 \text{ mm} \quad del\text{bol}_1 = \tan(\theta)l_1 = 1.09 \text{ mm}$$

$$P_{yp} = t_p F_{yp}/4 = 240000 \quad r_{mup} = t^2 p F_u/4 = 900000 \text{ N mm}$$

$$\alpha_0 = \text{atan}(del\text{bol}_0/l_{clear}) = 0.09 \quad \alpha_1 = \text{atan}(del\text{bol}_1/l_{clear}) = 0.04 \quad \Delta_{t0} =$$

$$del\text{bol}_0 = 2.69 \text{ mm} \quad elong_0 = \sqrt{\Delta_{t0}^2 + l_{clear}^2} - l_{clear} = 0.12 \text{ mm}$$

$$\Delta_{t1} = del\text{bol}_1 = 1.09 \text{ mm} \quad elong_1 = \sqrt{\Delta_{t1}^2 + l_{clear}^2} - l_{clear} = 0.02 \text{ mm}$$

$$t_{ava} = t_p/3 + w_t/3 = 19.60 \text{ mm}$$

- the maximum shear deformation per bolt,

$$dbol_{tm} = k_1 k_v k_s A_b F_u / (d_b t_{ava}) = 4.28 \text{ mm}$$

- the maximum tensile deformation is $d_{netm} = d_h/4 = 5.5 \text{ mm}$

- the maximum bearing deformation is $dbearm = 0.4d_h = 8.8 \text{ mm}$

- the shear deformation of a bolt associated with a given force is given as

$$boldef_0 = k_2 \text{forcem}_0 / (d_b t_{ava}) = 0.09 \text{ mm}$$

$$boldef_1 = k_2 \text{forcem}_1 / (d_b t_{ava}) = 0.04 \text{ mm}$$

$$\text{forcem} = V_{rbolt}/150 = 489 \text{ N}$$

$$\sigma = \text{forcem} / (t_p(p - d_h)) = 68.4 \text{ MPa} < F_{yp} = 300 \text{ MPa} \quad \text{ok}$$

$$netdef = \sigma p / (2e_p) = 0.014 \text{ mm}$$

$$berdef = \text{forcem} / F_{yp} t_p d = 0.34 \text{ mm}$$

- the total horizontal deformation of the bolts in shear is

$$totdef_0 = boldef + netdef + berdef = 1.15 \text{ mm}$$

$$totdef_1 = boldef + netdef + berdef = 1.15 \text{ mm}$$

- Calculation of secondary forces

- Tension region:

$$h_0 = forcem_0 = 2448 \text{ N}$$

$$h_1 = forcem_1 = 979 \text{ N}$$

- the membrane force present in an end plate at the i th level of bolts for a given connection rotation is $pplate_0 = h_0 / \cos(\alpha_0) = 2458 \text{ N}$

- the moment along the hinge lines of the end plate for a given rotation is

$$mplate_0 = (1 - pplate_0^2 / P_{yp}^2) = 899906 \text{ N mm}$$

$$pplate_1 = h_1 / \cos(\alpha_1) = 980 \text{ N}$$

$$mplate_1 = (1 - pplate_1^2 / P_{yp}^2) = 899985 \text{ N mm}$$

- the prying force resultant is located at

$$consta = 1.25(l_{clear} + t_w) = 45.81 \text{ mm} > e = 34.0 \text{ mm}$$

- therefore, set $consta = 34 \text{ mm}$

- the tension force in a bolt at the i th levels are

$$tbolt_0 = h_0 \tan \alpha_0 (1 + d_h / 2(consta)) + (mplate_0 / consta) (1 + d_h / l_{clear} + 2(consta) \cos^2 \alpha_0 / l_{clear}) = 104472 \text{ N}$$

$$tbolt_1 = h_1 \tan \alpha_1 (1 + d_h / 2(consta)) + (mplate_1 / consta) (1 + d_h / l_{clear} + 2(consta) \cos^2 \alpha_1 / l_{clear}) = 104243 \text{ N}$$

- the prying force in a bolt at the i th levels are

$$pbolt_0 = (mplate_0 + mplate_0 d_h / l_{clear} + h_0 d_h / 2 \tan \alpha_0) / consta = 45535.5 \text{ N}$$

$$pbolt_1 = (mplate_1 + mplate_1 d_h / l_{clear} + h_1 d_h / 2 \tan \alpha_1) / consta = 45481 \text{ N}$$

- the tension force in the beam web at the i th levels are,

$$tweb_0 = 2h_0 \tan \alpha_0 + 4mplate_0 \cos^2 \alpha_0 / l_{clear} = 116972 \text{ N}$$

$$tweb_1 = 2h_1 \tan \alpha_1 + 4mplate_1 \cos^2 \alpha_1 / l_{clear} = 117374 \text{ N}$$

- the connection shear capacity at the i th levels of bolts are

$$tvbolt_0 = 2\sqrt{0.56\phi_b^2\beta(A_bF_u)^2 - \beta tbolt_0^2 - h_0^2} = 100800 \text{ N}$$

$$tvb_{olt_1} = 2\sqrt{0.56\phi_b^2\beta(A_bF_u)^2 - \beta tb_{olt_1}^2 - h_1^2} = 101287 \text{ N}$$

because $tvb_{olt} > V_{rb_{olt}}$, therefore, $tvb_{olt} = 2V_{rb_{olt}} = 23014 \text{ N}$

• the bearing strength at the i th level is

$$tvb_{ear} = 2V_{bsup} = 1217964 \text{ N}$$

because $2V_{bplat} < tvb_{ear}$, therefore, $tvb_{ear} = 2V_{bplat} = 492480 \text{ N}$

• the shear capacity at the i th levels of plate are

$$tvplate_0 = 2t_p(p - d_h)\sqrt{\frac{1}{3}\left(\phi^2\sigma_{up}^2 - \frac{h_p^2}{(p-d_h)^2t_p^2}\right)} = 237600 \text{ N}$$

$$tvplate_1 = 2t_p(p - d_h)\sqrt{\frac{1}{3}\left(\phi^2\sigma_{up}^2 - \frac{h_p^2}{(p-d_h)^2t_p^2}\right)} = 271236 \text{ N}$$

$$vcheck = 2\phi_p pt 0.55F_y = 237600 \text{ N}$$

because $tvplate_1 > vcheck$, therefore, $tvplate_1 = 237600 \text{ N}$

because $tvplate > vcheck$, therefore, $tvplate = 373248 \text{ N}$

• the shear capacity at the i th levels of welds are

$$twweld_0 = 1.34\phi pf/\sqrt{2}X_u + T_0 - 1.2\sqrt{2}T_0^2/(0.67\phi 2pt_w X_u) = 150986 \text{ N}$$

$$twweld_1 = 1.34\phi pf/\sqrt{2}X_u + T_1 - 1.2\sqrt{2}T_1^2/(0.67\phi 2pt_w X_u) = 150615 \text{ N}$$

• the shear capacities

$$vb_{olt_0} = tvb_{olt_0} = 100800 \text{ N}$$

$$vb_{olt_1} = vb_{olt_0} + tvb_{olt_1} = 100800 + 101287 = 202087 \text{ N}$$

$$vb_{ear_0} = tvb_{ear_0} = 882792 \text{ N}$$

$$vb_{ear_1} = vb_{ear_0} + tvb_{ear_1} = 882792 + 882792 = 1765584 \text{ N}$$

$$vb_{earl_0} = tvb_{earl_0} = 275400 \text{ N}$$

$$vb_{earl_1} = vb_{earl_0} + tvb_{earl_1} = 275400 + 275400 = 550800 \text{ N}$$

$$vw_{eld_0} = tww_{eld_0} = 150986 \text{ N}$$

$$vw_{eld_1} = vw_{eld_0} + tww_{eld_1} = 150986 + 150615 = 301601 \text{ N}$$

$$vplate_0 = tvplate_0 = 237600 \text{ N}$$

$$vplate_1 = vplate_0 + tvplate_1 = 237600 + 237600 = 475200 \text{ N}$$

• Compression region:

$$exb_{olt} = (((l_p - 2e)/p + 1) - nt_{bolt})2V_{rb_{olt}} = 146891 \text{ N}$$

$$tb_{olt_2} = exb_{olt} = 146891 \text{ N}$$

$$vbolt_2 = vbolt_1 + tbolt_2 = 202087 + 146891 = 348978 \text{ N}$$

$$toplen = e - p/2 = -6.0 \text{ mm}$$

$$overla = bolloc - p/2.0 = 14.72 \text{ mm}$$

$$botlen = l_p - l_t = 59.28 \text{ mm}$$

$$explat = (toplen/p)tvplate = -17820 \text{ N}$$

$$explat = explat + 2(botlen + overla)V_{rplate} = 281880 \text{ N}$$

$$vplate_2 = vplate_1 + explat = 475200 + 281880 = 757080 \text{ N}$$

$$exweld = (toplen/p)tvweld = -11296 \text{ N}$$

$$exweld = exweld + 2overla(V_{rweld}) = 15617 \text{ N}$$

$$exweld = exweld + 1.34\phi_w t_w X_u botlen / \sqrt{2} = 124001 \text{ N}$$

$$vweld_2 = vweld_1 + exweld = 301601 + 124001 = 425602 \text{ N}$$

- Calculation of the strength of the beam web above the end plate

$$column1 = w_b t_w \phi F_y = 20574 \text{ N} \quad column2 = w_b (d_{top} - e) 0.66 F_y \phi = 356445 \text{ N}$$

since $column2 > column1$, then,

$$vbeam = vbeam + column1 = 448890 + 20574 = 469464 \text{ N}$$

- Calculation of block shear for the end plate: function block-shear

$$V_r = 0.85\phi_p A_{net} F_{up} = 334611 \text{ N} \quad \text{where } A_{net} = t_p 0.6 l_{pn}$$

$$V_{block} = 2V_r = 669222 \text{ N} > V_f = 300 \text{ kN} \quad \text{ok}$$

- Calculation of eccentrically loaded weld capacity: function weldecc

$$load = 415.17 \text{ kN} > V_f = 300 \text{ kN} \quad \text{ok}$$

- Calculation of the required bolt length: function bolt-length

$$l_b = t_{cf} + t_p + \frac{5}{4}d = 59.4 \text{ mm}$$

- select a bolt length of 60 mm and 1 washer for each bolt

- Function report

This function writes and formats the engineers and materials report to the appropriate files.

[illegible]

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Figure 4.2: Moment Rotation Curves, Cost Design Table, for Section 4.1.1

4.1.2 Beam-to-Column Web with Coped Beam Flanges; Bearing Connection

This example illustrates partial flange copes for both the top and bottom flanges and the shear capacity of bolts in which the threads are not intercepted by the shear plane. The input data and design parameters are listed below. The results for the designed connection are given in terms of an engineering and materials report in Figure 4.3; and moment rotation curves and a connection cost table in Figure 4.4. The 3D renderings show the designed connection with the partial flange copes made appropriately, illustrating the correct geometric calculations for fit. One flange of the column is not shown to make the connection visible. A comparison of the designs resulting from the Kriviak and Kennedy (1984) model versus the CISC model show that the shear capacity of the bolts govern as a result of the secondary forces and forces the connection to be constructed with eight bolts instead of six. Other variations are attributed to differences in end plate dimensions. Even for very small rotations, approximately 1/40th of that at ultimate (well within the service range), as shown in Figure 4.4, the moment transmitted to the supporting member is not insignificant. Any design assumptions regarding these support conditions being modelled as a hinge would be in error. It is apparent that the connection behaviour for semi-rigid connections should be included in the analytical model for analysing the structure.

Connecting Member	W530 X 196, 300W
Supporting Member	W360 X 101, 300W
Service, Factored Loads	350 kN, 400 kN
Beam Rotation:	0.02 radians
Plate:	8 mm, 300W, edge: sheared
Bolts:	A325M M20, pitch: 90 mm gauge: 100 mm
Welds:	8 mm, fillet weld
Threads:	not intercepted by the shear plane

Figure 4.4: Moment Rotation Curves, Cost Design Table, Example for Section 4.1.2

4.1.3 Beam-to-Beam with Copes; Friction Connection

This example illustrates the calculation and provision of copes for both top and bottom flanges of the beam and design of a connection classified as a friction type connection. The threads are considered to be intercepted. The input data and design parameters are listed below. The results for the designed connection are given in term of an engineering and materials report in Figure 4.5; and moment rotation curves and a connection cost table in Figure 4.6.

The connection is designed as a bearing type connection and then checked as a friction type. If the connection proves unsatisfactory for the latter case then the design is revised. The engineering report shows both resistances and the respective ratios with service and factored loads for the bolts. As indicated by the ratios of the interaction equation for friction and the total shear resistance predicted by Kriviak and Kennedy (1984), the friction requirements would given the bolt group design. A comparison with the results for a connection analysed without considering rotation shows the same. For this case, the connection design is governed by the weld capacity as predicted by Kriviak and Kennedy (1984), parabolic interaction equation for the force distribution that is based on the deformed configuration of the connection. For comparison with the force distribution on the undeformed configuration and the corresponding eccentric weld analysis, Lesik and Kennedy (1988) predict the weld group capacity to be 2.1 times the former. The eccentric weld analysis assumes that the connection remains rigid which it is clearly not and may not be applicable for this case even though it takes into account the eccentricity of the shear force.

Connecting Member	W610 X 101, 300W
Supporting Member	W460 X 177, 300W
Service, Factored Loads	180 kN, 200 kN
Beam Rotation:	0.02 radians

Step and Gap:	10 mm, 20 mm
Plate:	12 mm, 300W, edge: sheared
Bolts:	A325M M24, pitch: 80 mm, gauge: 80 mm
Welds:	6 mm, fillet weld
Threads:	intercepted (i.e. in the shear plane)

The results for the designed connection are illustrated in terms of an engineering and materials report in Figure 4.5 and moment rotation curves and cost design table in Figure 4.6.

Figure 4.5: Engineering and Materials Reports, Example for Section 4.1.3

Figure 4.6: Moment Rotation Curves, Cost Design Table, Example for Section 4.1.3

4.1.4 Beam-to-Column Flange; Four Lines of Bolts

This example illustrates the rare case where the number of bolts required to carry the factored shear cannot fit in two lines within the depth of the member and the bolts are then rearranged into 4 lines. For this case it is assumed that the two outside lines of bolts carry shear only and the two inside lines carry both shear and tension and are subjected to prying action as in the previous connections. The input data and design parameters are listed below.

Connecting Member	W610 X 241, 300W
Supporting Member	W920 X 342, 300W
Service, Factored Loads	1000 kN, 1100 kN
Connection Type:	bearing
Beam Rotation:	0.02 radians
Plate Thickness:	8 mm, 300W, edge: sheared
Bolts:	A325M M20, pitch: 80 mm gauge: 80 mm
Welds:	13 mm, fillet weld
Threads:	intercepted

The results for the designed connection are given in terms of an engineering and materials report in Figure 4.7; and the moment rotation curves and connection cost table in Figure 4.8. Due to the large shears, the shear capacity of the web of the connecting member governs. All other parts of the connection are designed for that load and are satisfactory.

4.2 Extended End Plate Connection Designs

The following four examples illustrate the versatility of the program in designing extended end plate connections for beam-to-column flange connections that include bearing and friction type connections, and conditions that generate connections without stiffeners in the column and four lines of bolts (large moments).

4.2.1 Beam-to-Column Flange: Bearing Connection

This example illustrates a typical beam-to-column flange bearing type extended end plate connection. The input data and design parameters are listed below. The results for the designed connection are given in terms of an engineering report (computed strengths for all probable failure modes) and materials report in Figure 4.9; and moment rotation curves and a connection cost table in Figure 4.10.

For this connection the shear tension prying action capacity of the series of bolts about the tension flange of the beam govern the design. The most effective method to improve the design is to reduce the prying action, which amounts to 18.4% of the tension in the critical bolts, by thickening the end plate. Placing additional bolts between the flanges to reduce the shear component of the interaction equation will have a very small effect in increasing the load capacity and is considered to be an uneconomical alternative for improving the connection design.

If the connection is designed for load reversal, then two additional bolts are added about the bottom flange (a mirror image of the top flange). As a result the bolt size becomes M30.

Connecting Member	W530 X 219, 300W
Supporting Member	W530 X 165, 300W

Service, Factored Loads	300 kN, 450 kN
Service, Factored Moment	450 kNm, 500 kNm
Plate:	20 mm, 300W. edge: sheared
Bolts:	A325M, M27, pitch: 150 mm, gauge: 120 mm
Welds:	8 mm, fillet weld
Load Reversal:	no
Threads:	not intercepted

A complete trace of the program logic and calculations for this example follow,

• Calculation of unit strengths: function `unit_strength`

Bolts:	$V_r = 0.60\phi_b m A_b F_u = 339690 \text{ N/bolt}$
	$T_r = 0.75\phi_b A_b F_u = 424612 \text{ N/bolt}$
Welds:	$V_r = 0.67\phi_t w F_y = 1447 \text{ N/mm}$
Beam Web:	$V_r = 0.66\phi A F_y = 3261 \text{ N/mm}$
Plate:	$V_r = 0.50\phi_p t F_u = 4050 \text{ N/mm}$
	$V_b = \phi t e F_u = 421200 \text{ N/bolt}$
Column:	$V_b = 3\phi t d F_u = 379890 \text{ N/bolt}$

• Calculation of tension and prying: function `extended_design`

$$T = M_f / (d - t f_c) = 941.97 \text{ kN}$$

$$P_f = T/4 = 235.49 \text{ kN}$$

• Extended End Plate

$$w_p = g + 2e_p = 313.00 \text{ mm}; b = 60.40 \text{ mm}; b' = 42.40 \text{ mm}; e = 90.00 \text{ mm}$$

$$a = 75.50 \text{ mm}; a' = 93.50 \text{ mm}; \delta = 1 - (d_h)/p = 0.68$$

$$K = 4b'10^3/(\phi p F_y) = 5.23; t_{min} = K P_f^{1/2} = 27 \text{ mm}$$

$$t_{max} = (K P_f / (1 + \delta \alpha))^{1/2} = 35 \text{ mm} \bullet \text{ select } 28 \text{ mm}$$

$$\text{plate: } V_r = 0.50\phi_p t F_u = 5670 \text{ N/mm}$$

$$V_b = \phi t e F_u = 589680 \text{ N/bolt}$$

$$\alpha = \left(\frac{K P_f}{I^2} - 1 \right) 1/\delta = 0.84$$

$$T_f = P_f \left[1 + \left(\frac{b'}{a'} \frac{\delta \alpha}{1 + \delta \alpha} \right) \right] = 274.37 \text{ kN} < T_r = 424.61 \text{ kN}$$

$$l_p = 2e + d_b + 2b = 738.40 \text{ mm}$$

• select size 320x28x740 mm³ for the end plate

$$Q\% = 100 \left(\frac{b'}{a'} \frac{\delta \alpha}{1 + \delta \alpha} \right) = 16.507\%$$

• **Column Flange**

$$b = 50.85 \text{ mm}; b' = 32.85 \text{ mm}; a = 96.5 \text{ mm}; a' = 114.5 \text{ mm}$$

$$\delta = 1 - d_h/p = 0.76 \quad K = 4b'10^3/(\phi p F_y) = 4.06; \alpha = 2.84$$

• for $\alpha > 1.0$, set α equal to 1.0

$$T_{rf} = 587.34 < T = 941.97 \text{ kN}$$

$$T_s = T - T_{rf} = 354.64 \text{ kN}$$

$$h/w = 35.83 < (670/\sqrt{F_y} = 38.7),$$

• select Class 1 web $t = 85.20 \text{ mm}$;

$$C_c = 1078.06 \text{ kN} > T = 941.97 \text{ kN} \quad T_s = T - C_c = -136.03 \text{ kN}$$

• **Design web stiffeners opposite of the compression flange**

$$T_s = 354.63 \text{ kN}$$

$$A_{sr} = T_s/\phi F_y = 1313.47 \text{ mm}^2; b = \sqrt{100 A_{sr}/\sqrt{F_y}} = 83.00 \text{ mm};$$

$$t = \sqrt{A_{sr} \sqrt{F_y}/400} = 7.54 \text{ mm}$$

• select $t = 8 \text{ mm}$,

$$A_s = 2bt = 1393.32 \text{ mm}$$

$$T_{rs} = C_{rs} = \phi A_s F_y = 188 \text{ kN}$$

$$d = d_c - 2tf_c = 501.60 \text{ mm}$$

• select size 83x8x501 mm³ for the stiffener

$$V_{rs} = 0.50 \phi d t F_y = 541.73 \text{ kN} < T = 941.97 \text{ kN}$$

$$V_{wr} = T_{rs}/(2b + w_b - 2k_{1b}) = 1.50 \text{ kN/mm}$$

• select a 5 mm, E410XX for the stiffeners

$$vweld = 4(b - w)V_{wr} = 479.69 \text{ kN}$$

- Calculation of shear capacity: function `extended_shear`

$$v_{bolt} = nV_r = 2038.14 \text{ kN}$$

$$v_{weld} = 2(d_b - 2k_b)V_{wr} = 1212.93 \text{ kN}$$

$$v_{plate} = 2(d_b - 2k_b)V_{wr} = 3538.08 \text{ kN}$$

$$v_{sup} = 2(d_b - 2k_b)V_{wr} = 2431.30 \text{ kN}$$

$$v_{beam} = (d_b - 2t_f)V_r = 1635.75 \text{ kN}$$

- Calculation of coordinates of the bolts: function `bolt_coordination1`

$$g_1 = 2.5d_b = 90.0 \text{ mm} \quad g_2 = (w_p - g)/2 = 96.5 \text{ mm}$$

$$x_1 = g_2 = 96.5 \text{ mm} \quad y_1 = g_1 = 90.0 \text{ mm}$$

$$x_2 = g_2 + g = 216.5 \text{ mm} \quad y_2 = g_1 + p = 240.0 \text{ mm}$$

$$y_3 = l_p - t_p - t_f - D = 626.8 \text{ mm}$$

$$nlines = 2, \quad rows = 3$$

- Calculation of block-shear for the end plate: function `block-shear`

$$V_r = 0.85\phi A_{net}F_{up} = 2315.16 \text{ kN}$$

$$T = 2V_r = 4630.31 \text{ kN} > V_f = 450 \text{ kN} \quad \text{ok}$$

- Calculation of the eccentric load capacity for the weld connecting the end plate to the beam: function `weldecc`

$$load = 1132.66 \text{ kN} > V_f = 450 \text{ kN} \quad \text{ok}$$

- Calculation of shear and tension capacity including prying action for the bolts

$$V_r = V_{rbolt}/1000 = 235.97 \text{ kN} \quad T_r = T_{rbolt}/1000 = 294.97 \text{ kN}$$

$$V_{part} = V_f^2 / (0.56\phi^2\beta(A_b F_u)^2) m^2 = 0.094 \quad T_{part} = T_f^2 / (0.56\phi^2\beta(A_b F_u)^2) = 0.852$$

$$V_{part} + T_{part} = 0.946 < 1.0 \quad \text{ok}$$

- Calculation of the bolt length: function `bolt.Length`

- select a bolt length of 90 mm and 1 washer for each bolt

Figure 4.10: Moment Rotation Curve, Design Cost Table, Example for Section 4.2.1

4.2.2 Beam-to-Column Flange with the Load Reversal: Friction Type Connection

This example illustrates the variation in design, which results by specifying the connection as a friction type. All the input data design parameters are as in example 4.2.1. The results for the designed connection are given in terms of an engineering and materials report in Figure 4.11, and moment rotation curves and a connection cost table in Figure 4.12.

For this case the design for friction and bearing results in similar values for the respective interaction equations (both govern the design). For friction, the shear portion of the interaction equation dominates and for bearing, the tension portion dominates, as would be expected.

4.2.3 Beam-to-Column Flange: Column Web Stiffeners Not Required

This example illustrates a beam-to-column flange extended end plate connection where the moments are relatively small and stiffeners are not required for the column web. All input data and design parameters are listed below. The results for the designed connection are given in terms of an engineering and materials report in Figure 4.13; and moment rotation curve and a connection cost table in Figure 4.14. For this case the shear tension prying action capacity of the bolt groups about the tension flange governs.

Connecting Member	W530 X 219, 300W
Supporting Member	W610 X 241, 300W
Service, Factored Loads	350 kN, 450 kN
Service, Factored Moment	250 kNm, 300 kNm
Connection Type:	bearing
Bolts:	A325M M20, pitch: 150 mm, gauge: 120 mm
Welds:	8 mm, fillet weld
Plate:	20 mm, 300W, edge: sheared
Load Reversal:	no
Threads:	intercepted

[illegible]

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Figure 4.14: Moment Rotation Curve, Design Cost Table, Example for Section 4.2.3

4.2.4 Beam-to-Column Flange With Large Moment; Four Lines of Bolts

This example illustrates the effect of large moments on the design of an extended end plate connection which results in an increase in the number of bolts from four to eight about the tension flange. The input data and design parameters are listed below. The results for the designed connection are given in terms of an engineering and materials report in Figure 4.15; and moment rotation curve and a connection cost table in Figure 4.16. The design is governed by the shear tension prying capacity of the bolt group about the top flange with the shear portion of the interaction equation being relatively small. The end plate connection design module explicitly recognizes that the most effective way to increase the connection capacity in a redesign procedure is to increase the bolt diameter, the number of bolts from 4 to 8 (these are the only possible numbers of bolts that can generate a symmetric connection), and lastly the strength.

Connecting Member	W530 X 138, 300W
Supporting Member	W760 X 284, 300W
Service, Factored Loads	800 kN, 900 kN
Service, Factored Moment	760 kNm, 800 kNm
Connection Type:	bearing
Bolts:	A325M, M20, pitch: 120 mm, gauge: 90 mm
Welds:	13 mm, fillet weld
Plate:	14 mm, 300W, edge: sheared
Load Reversal:	no
Threads:	intercepted

Figure 4.15: Engineering and Materials Reports, Example for Section 4.2.4

Figure 4.16: Moment Rotation Curve, Design Cost Table, Example for Section 4.2.4

4.3 Design Comparisons

4.3.1 General

Comparison designs for flexible and extended end plate connections have been made between SCADD, existing commercial software and published connection designs, listed in Table 4.2 to verify the accuracy and to illustrate the program usage and versatility. Based on this limited sample it could be stated that the program will generate practical designs for most situations.

Some fundamental differences between steel grades bolt sizes, plate sizes and strength equations for Canadian and American produced products which are manifested in the designed connections include:

1. Specified steel grade A36, $F_y = 250$ MPa, and $F_u = 400$ MPa. The closest Canadian match is a grade 260W steel with $F_y = 260$ MPa, and $F_u = 410$ MPa.
2. Bolt diameters, A325N: 3/4" (19.05 mm) and A325 M20 (20.0 mm) for example.
3. Plate thicknesses, 3/4" and 20 mm.
4. Minor differences in resistance and load factors for the strength equations for fasteners.

Other differences in the designed connections may result from specified different values required by one or both programs. For example, the default value for the width of an extended end plate in EPCD is given as $b + t_p$, where b is the beam flange width and t_p is the end plate thickness. These kinds of differences are to be expected and may lead to what is classified as minor variations in design.

Table 4.2: Design Comparison: Example Types

Example	Design Software	Connection Description
1	CONXPRT	Beam-to-Girder, Flexible End Plate
2	CONXPRT	Beam-to-Column Flange, Flexible End Plate
3	CONXPRT	Beam-to-Column Web, Flexible End Plate
4	AISC EXAMPLE	Beam-to-Girder, Flexible End Plate
5	AISC EXAMPLE	Extended End Plate
6	CISC EXAMPLE	Extended End Plate

4.3.2 CONXPRT Design Comparisons (Flexible End Plate Connection Design)

CONXPRT is a PC platform knowledge-based program for the load and resistance factor design (LRFD) of selected steel connections for steel buildings and is supported by the American Institute of steel Construction (AISC). As a result some minor variations in the designed connection are to be expected. CONXPRT is limited to flexible end plate connection design and prompts the user for all required input data including dimensions for copes. Note, CONXPRT evaluates the connection for shear only unlike EPCD where the eccentricity resulting from the end plate thickness and secondary forces are considered. The bolt pitch must be specified by the user for both programs. To illustrate the versatility in calculating the shear capacity for the beam with various coping details three different configurations were investigated, beam-to-girder, beam-to-column flange, and beam-to-column web.

Example 1. Flexible End Plate, Beam-to-Girder, Bearing Type Connection

Connection Data

- unfactored shear 300 kN, factored shear 356 kN
- beam step = 0.0 mm
- bolt gauge = 89 mm, bolt pitch = 76 mm
- members and specified weld and bolt sizes and strengths

	EPCD	CONXPRT
Beam	W460X52 (260W)	W18X35 (A36 steel)
Column	W530X182 (260W)	W21X122 (A36 steel)
Weld	E410XX	E70XX
Bolts	A325M, M20	A325-N, 3/4"

The output for the designed connections for EPCD and CONXPRT for this example are illustrated in Figures 4.17 and 4.18, respectively. If summary of the end plate dimensions, number and size of bolts and weld size (extends the full length of the end plate on both sides for both programs) is given in Table 4.3. The connection design for EPCD is governed by shear on the net section of the beam, which leads to a connection length of 350 mm and 4 rows of the bolts. The same failure mode controls for CONXPRT but leads to 5 rows of bolts as the bolt diameter is slightly less and it just happens for this case to fit in the same length. CONXPRT gives a simple image of the connection with dimensions. EPCD gives a 3D rendering and has the capacity to give a dimensioned shop drawing, as well as additional information regarding the moment-rotation characteristics and fabrication costs.

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Figure 4.18: CONXPRT Design Example 1

Table 4.3: Comparison with CONXPRT; Beam to Girder, Flexible End Plate

	EPCD (mm,unit)	CONXPRT	
		(in,unit)	(mm,unit)
Plate	10x160x350 (260W)	3/8x6x14-1/2 (A36 Steel)	10x152x368 (A36 Steel)
Bolts	8, A325M, M20, 55 long	10, 2-1/4 long	10, 57 long
Welds	5	3/16	5

Example 2. Flexible End Plate, Beam-to-Column Flange, Bearing Type Connection

Connection data

- unfactored shear 300 kN, factored shear 356 kN
- bolt gauge = 89 mm, bolt pitch = 76 mm, threads intercepted
- plate edge: flame-cut
- members and specified weld and bolt sizes and strengths

	EPCD	CONXPRT
Beam	W460X52 (260W)	W18X35 (A36 steel)
Column	W530X182 (260W)	W21X122 (A36 steel)
Weld	E410XX	E70XX
Bolts	A325M, M20	A325-N, 3/4

The output for the designed connections for EPCD and CONXPRT for this example are illustrated in Figures 4.19 and 4.20, respectively. A summary of the end plate dimensions, number and size of bolts, and weld size are given in Table 4.4. The designs are essentially the same. The same connection, redesigned with an end plate thickness of 8 mm, resulted in a reduction in the number of bolts required to eight.

Figure 4.20: CONXPRT Design Example 2

Table 4.4: Comparison with CONXPRT; Beam-to-Column Flange, Flexible End Plate

	EPCD (mm,unit)	CONXPRT	
		(in,unit)	(mm,unit)
Plate	6x160x350 (260W)	1/4X6X15-1/2 (A36 Steel)	6X152X394 (A36 Steel)
Bolts	10, A325M, M20, 60 long	10, 2-1/4 long	10, 57 long
We'ds	5	3/16	5

Example 3. Flexible End Plate, Beam-to-Column Web, Bearing Type Connection

Connection Data

- unfactored shear 400 kN, factored shear 445 kN
- bolt gauge = 89 mm, bolt pitch = 76 mm, threads: intercepted
- plate edge: sheared
- members and specified weld and bolt sizes and strengths

	EPCD	CONXPRT
Beam	W530X109 (260W)	W21X73 (A36 steel)
Column	W530X92 (260W)	W21X62 (A36 steel)
Weld	E410XX	E70XX
Bolts	A325M, M20	A325-N, 3/4

The output for the designed connections for EPCD and CONXPRT for this example are illustrated in Figures 4.21 and 4.22, respectively. A summary of the end plate dimensions, number and size of bolts and weld size are given in Table 4.5. The designs are the same for both EPCD and CONXPRT, verifying the design process and program. EPCD provides additional information in the form of a materials report, moment rotation curves for the designed connection and the fabrication cost. EPCD has the capacity to conduct a plastic analysis which includes secondary forces, Kriviak and Kennedy (1984), and includes the effects of eccentricity on both the welds. These provide a more realistic force distribution within the connection and a realistic description of the connection behaviour. As a result the failure modes are predicted more accurately. Although all three examples yielded the same designs, it may be possible that differences will occur if the loads or the geometric requirements are abnormal. EPCD is able to design these connection.

Note:

1. EPCD designs both slip resistant and bearing type connections whereas CONXPRT can only design bearing type connections.
2. CONXPRT illustrates the dimensions of the connection on its drawing. These values are not shown on the 3D rendition in EPCD but will be shown on the shop drawing once drawing model is developed.

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Figure 4.22: CONXPRT Design Example 3

Table 4.5: Comparison with CONXPRT; Beam-to-Column Web, Flexible End Plate

	EPCD (mm,unit)	CONXPRT	
		(in,unit)	(mm,unit)
Plate	6X160X300 (260W)	1/4X6X11-1/2 (A36 Steel)	6X152X292 (A36 Steel)
Bolts	8, A325M, M20, 50 long	8	8
Welds	6	1/4	6

4.3.3 AISC Connection Manual Design Comparisons

Example 4. Flexible End Plate, Beam-to-Girder, Bearing Type Connection

Connection Data

- unfactored shear 200 kN, factored shear 294 kN
- beam step = 30 mm
- bolt gauge = 140 mm, bolt pitch = 76 mm
- members and specified weld and bolt sizes and strengths

	EPCD	AISC
Beam	W530X109 (260W)	W14X68 (A36 steel)
Column	W530X182 (260W)	W27X84 (A36 steel)
Weld	E410XX	E70XX
Bolts	A325M, M20	A325-N, 3/4

The output for the designed connection for EPCD and the summary of the end plate dimensions, number and size of bolts and weld size for both are given in Figure 4.23 and Table 4.6, respectively. The connection designs are the same as indicated by the values given in the Table 4.6.

Table 4.6: Comparison with AISC Example, Flexible End Plate

	EPCD (mm,unit)	AISC Volumn II connection, example,page3-98	
		(in,unit)	(mm,unit)
Plate	6X210X220 (260W)	1/4X8X8-1/2 (A36 Steel)	6X203X216 (A36 Steel)
Bolts	6	6	6
Welds	6	1/4	6

Example 5. Extended End Plate, Bearing Type Connection

Connection Data

- unfactored shear 178 kN, factored shear 267 kN
- unfactored moment 217 kNm, factored moment 325 kNm
- bolt pitch = 89 mm, bolt gauge = 110 mm, threads: intercepted
- plate edge: gas cut
- members and specified weld and bolt sizes and strengths

	EPCD	AISC
Beam	W460X74 (260W)	W18X50 (A36 steel)
Column	W360X64 (260W)	W14 (A36 steel)
Weld	E410XX	E70XX
Bolts	A325M, M24	A325-N, 1"

The output for the designed connections for EPCD and the summary of the end plate dimensions number and size of bolts and weld size for EPCD and AISC are given in Figure 4.24 and Table 4.7, respectively. The only variation is the bolt diameter which results from discrete bolt sizes in British or metric units. EPCD gives additional information on stiffener design (if required), moment rotations curves, and the fabrication cost.

Table 4.7: Comparison with AISC, Extended End Plate

	EPCD (mm,unit)	AISC Volumn II connection, example,page4-26	
		(in,unit)	(mm,unit)
Plate	25X210X590 (260W)	1-1/8X8-1/2, the length is not calculated. (A36 Steel)	20X216, the length is not calculated (A36 Steel)
Bolts	6, A325M, M27	6, 1	6, 25.4
Welds	8, E410XX	5/16, E70XX	8, E70XX
Stiffeners	8X90X320 (260W)	not calculated	-----

4.3.4 CISC Connection Manual Design Comparison

Example 6. Extended End Plate, Bearing Type Connection

Connection Data

- unfactored shear 400 kN, factored shear 468 kN
- unfactored moment 280 kNm, factored moment 310 kNm
- bolt gauge = 125 mm, bolt pitch = 100 mm
- members and specified weld and bolt sizes and strengths

	EPCD	CISC
Beam	W410X67 (300W)	W410X67 (300W)
Column	W310X86 (300W)	W310X86 (300W)
Plate	25 mm	25 mm
Bolts	A325M, M24	A325, M25

Note, the factored shear was not specified but was assumed to be in the same proportion to the shear resistance as the ratio of factored moment to moment resistance.

The output for the designed connections for EPCD and the summary of the end plate dimensions number and size of bolts and weld size for EPCD and CISC are given in Figure 4.25 and Table 4.8. The example given in the CISC Connection Design Manual 1990 is partially complete as indicated by comments listed in Table 4.8.

Again the only variation is the bolt diameter which results from discrete bolt sizes in British or metric units. Note, the program EPCD provides a complete design, reports on strengths for all failure modes, a materials report, a estimate of the connection cost using fabricators confidential pricing and a 3D rendering for the engineer or fabricator to review. Stiffeners are designed for the column (if required) and illustrated on the rendering.

Table 4.8: Comparison with CISC, Extended End Plate

	EPCD (mm,unit)	CISC (mm,in,unit)
Plate	25X210X560 (300W)	25X250 (300W) plate length is not calculated
Bolts	6, A325M, M27, 80 long	1'
Welds	410XX, 13	E480XX, >12mm
Stiffeners	6X65X277 (300W)	not calculated

Chapter 5

Summary, Conclusions, Recommendations and Future Work

5.1 Summary and Conclusions

A program End Plate Connection Design (EPCD) was developed to analyse and design both flexible and extended end plate connections according to the Canadian Standard for Limit States Design of Steel Structures, CAN/CSA-S16.1-M89, and current fabricating practices; and includes recent research in determining internal force distributions and resistance determinations. In conjunction a graphic user interface was developed for the connection design and review process that simplifies data input, allows for fabricators preferences in material usage, allows fabricators to input confidential unit cost information for evaluating the connection cost during the design process, allows a complete review of the resistance for all probable failure modes and a presents scaled 3D rendition of the designed connection. EPCD includes

the calculation and display of the moment rotation characteristics of the designed connection using the appropriate models from the following set, the Kriviak and Kennedy inelastic model (1984), Frye-Morris polynomial model (1976), and the Ang-Morris power model (1984).

All possible connection design permutations are given with detailed calculations illustrating the design process. Comparisons were made with a commercially available end plate connection design program and published design examples by the American Institute of Steel Construction and the Canadian Institute of Steel Construction to verify the designs produced by EPCD.

The following conclusions can be made:

1. Flexible end plate connection designs based on the Kriviak and Kennedy inelastic model (1984) and those based on CISC recommendations result in similar end plate sizes, number of bolts and weld size for most designs investigated. This is a consequence of using a discrete number of fasteners (arranged in pairs or in rare cases groups of four for symmetry) and detailing practices, and not as a consequence of the internal force distributions nor the resistance determination controlling the design, which are often very different. In some cases, additional bolts were required for the designs based on the Kriviak and Kennedy model as it includes secondary forces (horizontal shear) due to membrane forces that develop in the end plate as the connection deforms under load.
2. Flexible end plate connections develop moments even for small beam end rotations. Any design and analysis assumptions regarding these support conditions as being modelled as a hinge would be in error. The connection behaviour should be included in the analytical model for analysing the structure and the resulting moment included in the design of the beam column (supporting member).

3. The program EPCD designs slip critical end plate connections, provides moment rotation curves and a connection cost table and illustrates a 3D rendering of the designed connection. These features are not included in CONXPRT.
4. The designs produced by EPCD are compatible to those generated by CONXPRT and to those given by the AISC and the CISC.
5. The program EPCD provides an effective design and learning tool for the design of the end plate connections for students, engineers and fabricators.

5.2 Recommendations and Future Work

1. Redesign the program architecture for the connection design module to include recent work on a generalized structural steel class library written in C++.
2. Rewrite all modules in C++ to group common functions and to increase the efficiency of the program in the terms of maintenance and updating. Redesign the graphic user interface to include recent developments. This would include the integration of this module with the analysis design programs.
3. The continued development of other connection design algorithms, for example, moment resistant connections, gusset plate connections, stiffened seat angles, and splices.
4. The addition of an algorithm to develop the appropriate shop drawings in Auto-CAD.
5. The existing program could be modified to include joints where the members are not orthogonal.

REFERENCES

American Institute of Steel Construction. 1991. CONXPRT-LRFD MODULE I, Version 1.02. American Institute of Steel Construction Inc.

American Institute of Steel Construction, Inc. 1992. Manual of Steel Construction Volume II Connections. ASD 9th edition / LRFD, 1th edition. American Institute of Steel Construction, Inc. One East Wacker Drive, Chicago.

Canadian Institute of Steel Construction (CISC), Gilmor, M. I., editor. 1991. Handbook of Steel Construction, 5th edition. Canadian Institute of Steel Construction, Willowdale, Ontario.

Canadian Institute of Steel Construction (CISC), 1990. Connections For Steel Structures, 1st edition. Canadian Institute of Steel Construction, Willowdale, Ontario.

Canadian Institute of Steel Construction (CISC), Henderson, J. E., editor. 1993. Standardized Connections. Canadian Institute of Steel Construction.

Chen, W. F., 1985. Connection Flexibility and Steel Frames. New York, American S. of Civil Engineers. ref. Ang-Morris, Frye-Morris.

Chen, W. F. and Lui, E. M., 1991. Stability Design of Steel Frames. New York, Boca Raton, Fla., CRC Press.

Devitt, B. A., 1992. The Design, Analysis and Detailing of Bolted and Welded Simple Steel Connections. Master of Engineering Thesis . Carleton University, Ottawa.

Disque, R. O. P. E., 1971. Applied Plastic Design in Steel. New York, Van Nostrand Reinhold Company.

Dowling, P. J., Marding, J. E. and Bjorhovde R., 1992. Constructional Steel Design.

London, Elsevier.

Jiang, Y., 1995. Interactive Design of Continuous Flexural Steel Members. Master of Engineering Thesis. Carleton University, Ottawa.

Kennedy, D. J. L. and Kriviak, G. J., 1984. The strength of fillet welds under longitudinal and transverse shear: a paradox. Department of Civil Engineering, University of Alberta, Edmonton. Alta., Canada

Kriviak, G. J. and Kennedy, D. J. L., 1984. Standardized Flexible End Plate Connections For Steel Beams. Structural Engineering Report No. 121, University of Alberta, Canada.

Kulak, G. L., Fisher J. W., and Struik J. H. A., 1987. Guide to Design Criteria for Bolted and Riveted Joints, 2nd edition. New York, John Wiley & Sons.

Miazga, G. S. and Kennedy, D.J. L., 1986. Behaviour of Fillet Welds as a Function of the Angle of Loading. Structural Engineering Report 133, University of Alberta. March 1986, 145p.

Srna, P. 1995. Development of an Interactive Design and Analysis Program for Steel Beam-Columns. Master of Engineering Thesis. Carleton University, Ottawa.

Wells, P., 1991. Private Communication.

Wiegand, G., Covey, R., and Couch P., 1992. Hoops 3.2 Reference Manual. Ithaca Software, Alameda.

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