STEEL CONNECT

Structural Steelwork Connections Guide : Design Procedures

SCNZ 14·1:2007

Structural Steelwork Connections Guide

DESIGN PROCEDURES

SCNZ Report 14.1: 2007

February 2008

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Amendments

Date	Description
July 2004	Amendment to MEP and MEPS connections
November 2005	Addition of BBS connections, amendment to BCS and BTS connections
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a	Bolt offset from flange
a_c	End gap
a_{cb}	Bottom cope depth
a_e , $a_{e1\text{–}e8}$	Bolt edge distances
a_{eli} , a_{elwi} , $a_{el\,\mathrm{sup}i}$	Bolt edge distances in cleats along beam longitudinal axis
a_{elb}	Bolt edge distances in beam along beam longitudinal axis in beam
a_{eti}	Bolt edge distances in cleats transverse to beam longitudinal axis
a_{etb}	Bolt edge distances in beam transverse to beam longitudinal axis
a_f	Top of flange to bolt hole centre above flange; MEPS-F plate edge end distance to flange
a_m	Base plate edge distance variable
a_p	Top of flange to bolt hole centre below flange
a_1 , a_2 , a_3	Base plate edge distance variables
A_c	HD bolt core area
A_e	Effective rigid bearing area
A_{ef}	Effective flange area
A_{ew}	Effective web area
A_g	Gross cross section area
A_{gf}	Gross flange area
A_{gfi}	Gross outer flange cleat area
A_{gi2}	Gross doubler flange cleats area
A_{gs}	Block gross shear area
A_{gsi}	Cleat block gross shear area
A_{gt}	Block gross tension area
A_{gw}	Gross web area
A_{gwi}	Single web plate gross area
A_H	Equivalent H shaped bearing area under base plate
A_{gi}	Gross cleat area
A_{gsi}	Cleat block gross shear area
A_{gti}	Cleat block gross tension area
A_{nf}	Net flange area
A_{nfi}	Net outer flange cleat area
A_{ni}	Cleat net area
A_{ni2}	Doubler cleats net area
A_{ns}	Block net shear area
A_{nsi}	Cleat block net shear area
$A_{n\,\mathrm{sup}i}$	Support cleat leg net shear area
A_{nt}	Block net tension area
A_{nti}	Cleat block net tension area
A_{nwi}	Single angle cleat net web leg area
A_o	Area enclosed by CHS perimeter on base plate
A_{rf}	Flange rigid bearing area
A_{ro}	CHS rigid bearing area
A_{rw}	Web rigid bearing area
A_w	Web shear area

A_1	Steel base plate area
A_2	Concrete bearing area in accordance with NZS3101:1995, Section 8.3.5
AC	Angle cleat connection
$ACx - y$	Angle cleat connection with x bolt columns in web and y angle cleats
$ACE - y$	Extended Angle cleat connection with y angle cleats
b	Section width
b_{ef}	Effective flange width
b_{ei}	Effective end plate width
b_f	Flange width
b_i	Cleat width
b_{if}	Flange cleat width
b_{iw}	Web cleat width
b_{i2}	Doubler flange plate width
b_{ww}	Web weld horizontal width
BBS	Bolted beam splice
$BBS1$	Bolted beam splice, with single side flange plates
$BBS2$	Bolted beam splice, with double side flange plates
BCS	Bolted column compression splice
$BCS1$	Bolted column compression splice, with single side flange plates
$BCS2$	Bolted column compression splice, with double side flange plates
BPP	Base plate pinned connection
$BTS1$	Bolted column tension splice, with single side flange plates
$BTS2$	Bolted column tension splice, double side flange plates & $n_g = 2$
$BWBS$	Bolted and welded beam splice
c	BWBS end gap; BP P rigid plate cantilever extension
d	Section depth
d_{bc}	Coped section depth
d_{ew}	Effective web depth
d_f	Fastener diameter
d_h	Hole diameter
d_i	Cleat length
d_{if}	Flange cleat length
d_{iw}	Web cleat length
d_o	Outer diameter of CHS section
d_{rx}	Lever arm of bolt row x from centre of bottom flange
$d_{\sup}$	Depth of supporting member
d_w	Depth of coped web excluding flange
DWC	Double web coped section
e	Reaction eccentricity; End plate edge distance bolt
$e_{\sup}$	Eccentricity of support bolt group
e_v	Cope eccentricity
e_w	Weld group eccentricity
E_{drift}	Seismic drift induced design shear for beam $\frac{L_n}{d} \geq 20$
$f_c^{'}$	28 day concrete compressive strength
$f_m^{'}$	28 day mortar compressive strength
f_{ub}	Section tensile strength
f_{uf}	HD Bolt ultimate tensile strength
f_{ui}	Cleat tensile strength
f_{us}	Support tensile strength

f_{uw}	Weld tensile strength
f_y	Section yield stress
f_{yfi}	Flange cleat yield stress
f_{yi}	Cleat yield stress
f_{yw}	Web yield stress
f_{ywi}	Web cleat yield stress
FE	Flexible end plate connection
G	Design dead load
h	Bolt washer thickness
i_{bp}	Bolt group polar moment of inertia
$i_{bp\,\mathrm{sup}}$	Support bolt group polar moment of inertia
i_{bpw}	Web bolt group polar moment of inertia
i_{cope}	Moment of inertia of coped section
i_{wp}	Weld group polar moment of inertia
k_f	Section form factor for compression
l_{bolt}	Overall bolt length
l_{ex}	Equivalent length of yield line in equivalent T-stub for yielding pattern x
l_{erx}	Critical equivalent length of yield line in equivalent T-stub for bolt row x
$l_{gn\,\max}$	Maximum net bolt and washer assembly grip length
$l_{gn\,\min}$	Minimum net bolt and washer assembly grip length
l_s	Bolt unthreaded shank length
k_h	Factor for different hole types
L_{w3}	Total length of 3-sided fillet weld
L_w	Total length of fillet weld
L_c	Cope length
L_n	Beam clear span length
m	Governing endplate bolt to plate weld edge distance
$m_{n\,max}$	Maximum nut thickness
m_1	Endplate bolt to web weld edge distance
m_2	Endplate bolt to flange weld edge distance
m_3	Endplate bolt to vertical gusset weld edge distance
m_4	Endplate bolt to flange weld edge distance
M^*	Design bending moment
M^*_{drift}	Seismic drift induced design bending moment
M^*_{oms}	Seismic induced over-strength design bending moment
$MEP-x$	Moment end plate, beam / column connection with x bolts
$MEP-Gx$	Gusseted Moment end plate, beam / column connection with x bolts
$MEPS-Fx$	Moment end plate , flush, beam splice connection with x bolts
$MEPS-Ex$	Moment end plate , extended, beam splice connection with x bolts
$MEPS-Gx$	Gusseted moment end plate , beam splice connection with x bolts
n	Effective endplate bolt to plate edge distance
n_a	Number of angle cleats
n_b	Number of bolts in base plate; total number of bolts in beam for AC
n_c	Number of bolt columns in beam web
n_{bb}	Number of bolts at bottom flange bolt group
n_{ei}	Number of effective interfaces
n_g	Number of flange gauge rows
n_p	Number of bolt rows
n_{pf}	Number of bolt rows in flange

$n_{p\,\mathrm{sup}}$	Number of bolt rows in support
n_{pw}	Number of bolt rows in web
n_{sup}	Total number of bolts in support
n_{tb}	Number of bolts at top flange
N_{ptf}	Minimum bolt tension at installation
N_{tf}	Bolt tension nominal capacity
N_c^*	Section design axial compression load
N_{fbt}^*	Flange bolt group design tension
N_{fmt}^*	Flange moment induced design tension
N_{ft}^*	Flange design tension
N_{fti}^*	End plate design tension
N_{gfi}^*	End plate pull-out flexure yield tension
N_{gti}^*	Cleat gross tension yield force
N_p^*	Bolt prying force
N_{splice}^*	Minimum compression action NZS3404:1997 cl.9.1.4.1.b.v
N_t^*	Section design axial tension load
N_{ww}^*	Web weld design tension
NC	No copes to section
p	Bolt thread pitch; BPP bolt set-out pitch
p_f	Top of flange to bolt hole centre below flange
Q_u	Design long term ultimate live load to NZS4203:1992
r	Section root radius
s_g	Bolt gauge distance
s_{g1}	Angle cleat corner to bolt column in support
s_{g2}	Inner bolt gauge of BCS and BTS web splice; Angle cleat corner to first bolt column in web
s_{g3}	Angle cleat gauge distance between first and second bolt columns in web
s_p	Bolt pitch
s_{pg}	AC double bolt column pitch and gauge factor
s_{p1}	Inner bolt pitch in flange splice of BCS and BCS
$s_{p\,\mathrm{sup}}$	AC bolt pitch in support
s_{pw}	Bolt pitch in web
S_{cope}	Coped section plastic modulus.
$STP-8$	Steltech portal, moment end plate, 8 bolt, rafter /col. connection
SWC	Single web coped section
t_i	Cleat thickness
t_{if}	Flange cleat thickness
t_{ig}	Gusset plate thickness
t_{iw}	Web cleat thickness
t_f	Flange thickness
t_s	Support thickness
t_w	Web thickness
t_{wi}	Cleat fillet weld leg length
t_{wf}	Flange fillet weld leg length
t_{wg}	Gusset fillet weld leg length
t_{ww}	Web fillet weld leg length
v_{wwres}^*	Resultant design shear per mm at toe

v_x^*	Design x-direction shear per mm at toe
v_y^*	Design y-direction shear per mm at toe
V_{fn}	Bolt threads-included nominal shear capacity
V_{fx}	Bolt threads-excluded nominal shear capacity
V^*	Design shear force
V_{drift}^*	Beam / column seismic drift and gravity design shear force
V_{ls}^*	Longitudinal shear / bolt
$V_{\max}^*$	Maximum design shear satisfying connection governing criteria
V_{res}^*	Resultant maximum bolt shear
V_{ts}^*	Transverse shear / bolt
WM	Welded moment beam / column connection
WP	Web side plate connection
x_{ct}	x-distance from weld centroid to toe
y_c	Elastic neutral axis from underside of flange
y_s	Plastic neutral axis from underside of flange
Z_b	Bolt interaction factor for single line of bolts $n_p \neq 1$
$Z_{b\sup}$	Bolt interaction factor for single line of bolts in support
Z_{bw}	Bolt interaction factor for bolts in beam web
Z_{cope}	Coped section elastic modulus.
Z_e	Bolt group flexure factor, single line of bolts, $n_p \neq 1$
Z_{ec}	Coped section effective modulus
$Z_{el\sup}$	AC support bolt tearing factor orthogonal to beam longitudinal axis
Z_{elw}	AC bolt tearing factor in web along beam longitudinal axis
Z_{etw}	AC bolt tearing factor in web transverse to beam longitudinal axis
Z_{ex}	Effective section modulus, NZS 3404:1997
Z_{nex}	Net section effective flexural modulus
Z_1	Sub-factor for Z_{bw}
α	CHS rigid area concentration factor
μ_s	Slip factor
ϕ_b	Bolt capacity reduction factor
ϕ_c	Concrete bearing capacity reduction factor, NZS3101:1995, cl. 3.4.2.2
ϕ_{cv}	Concrete shear capacity reduction factor, NZS3101:1995, cl. 3.4.2.2
ϕ_s	Steel capacity reduction factor
ϕ_{slip}	Slip strength reduction factor
ϕ_w	Weld capacity reduction factor
λ	End plate edge distance ratio
λ_{ef}	Flange slenderness ratio
λ_{ew}	Web slenderness ratio
λ_{ey}	Plate element slenderness ratio yield limit
ϕ_{om}	Supplier material variation factor
ϕM_b	Bolt group design moment capacity
ϕM_c	Connection compression flange moment capacity
ϕM_{con}	Connection design moment capacity
ϕM_{rcon}	Connection moment capacity with axial load
ϕM_s	Section design moment capacity
ϕM_t	Connection tension flange moment capacity
ϕN_1	Mode 1 bolt row design capacity

ϕN_2	Mode 2 bolt row design capacity
ϕN_3	Mode 3 bolt row design capacity
ϕN_b	Flange bolt group shear design tension capacity
ϕN_{bf}	Flange bolt hole 1st bearing design tension capacity
ϕN_{bi}	Cleat bolt hole 1st bearing design tension capacity
ϕN_{bp}	Steel base plate bearing design capacity
ϕN_{bweb}	Web splice bolts direct compression design capacity
ϕN_{cf}	Flange splice design compression capacity
ϕN_{con}	Connection design compression capacity
ϕN_{cw}	Web splice design compression capacity
ϕN_f	Flange design tension capacity
ϕN_{ft}	Flange splice design tension capacity
ϕN_{gfi}	End plate flexure yield design tension capacity
ϕN_{gsi}	End plate pull-out shear / flexure design tension capacity
ϕN_{gtf}	Flange gross yield tension design capacity
ϕN_{gti}	Cleat gross yield tension design capacity
ϕN_{gtw}	Web gross tension yield design capacity
ϕN_{gtwi}	Web plates gross tension yield design capacity
ϕN_{ntw}	Web net tension ultimate design capacity
ϕN_{ntwi}	Web plates net tension ultimate design capacity
ϕN_i	End plate design tension capacity
ϕN_s	Section design compression capacity
ϕN_{ltf}	Flange bolt hole 1st longitudinal tearing design tension capacity
ϕN_{lti}	Cleat bolt hole 1st longitudinal tearing design tension capacity
ϕN_m	Concrete bearing design capacity
ϕN_{r1}	Bolt row 1 design capacity
ϕN_{r2}	Bolt row 2 design capacity
ϕN_{r3}	Bolt row 3 design capacity
ϕN_{rx}	Bolt row x design capacity
ϕN_{ntf}	Flange net ultimate tension design capacity
ϕN_{nti}	Cleat net ultimate tension design capacity
ϕN_{ntw}	Web net tension ultimate design capacity
ϕN_{splice}	Splice fittings compression design capacity
ϕN_{tb}	Bolt group tension design capacity
ϕN_{tf}	Bolt tension design capacity
ϕN_{ti}	Cleat design tension capacity
ϕN_{twi}	Web cleat design tension capacity
ϕN_{wf}	Flange fillet weld tension design capacity
ϕN_{wt}	Weld group tension ultimate design capacity
ϕN_{ww}	Web fillet weld tension design capacity
ϕv_{ww}	Web weld design capacity per mm
ϕV_b	Bolt group design shear capacity at eccentricity e
ϕV_{bi}	Cleat bolt hole 1st resultant bearing design shear capacity
ϕV_{bsi}	Block shear design capacity of end plate
$\phi V_{b\,\mathrm{sup}}$	Support bearing design shear capacity
$\phi V_{b\,\mathrm{sup}i}$	Support cleat bearing design shear capacity
$\phi V_{bs\,\mathrm{sup}i}$	Support cleat design block shear capacity
ϕV_{bsw}	Web block shear design capacity

ϕV_{bw}	Web bolt hole 1st resultant bearing design shear capacity
ϕV_{con}	Connection design shear capacity
ϕV_{f}	Bolt design shear capacity
$\phi V_{fb\,\mathrm{sup}}$	Support bolt resultant shear and bearing design capacity
ϕV_{fbw}	Web bolt resultant shear and bearing design capacity
ϕV_{fn}	Bolt design shear capacity with threads included in shear plane
ϕV_{fw}	Web bolt design shear capacity with consideration of shear planes 1 and 2
ϕV_{fx}	Bolt design shear capacity with threads excluded in shear plane
ϕV_{f1}	Web bolt design shear capacity on shear plane 1
ϕV_{f2}	Web bolt design shear capacity on shear plane 2
ϕV_{gfi}	Cleat gross flexure yield design shear capacity
$\phi V_{gf\,\mathrm{sup}i}$	Support angle cleat leg gross flexure yield design shear capacity
ϕV_{gfw}	Web gross flexure yield design shear capacity
ϕV_{gsb}	Web gross shear design capacity
ϕV_{gsi}	Cleat gross shear yield design capacity & Cleat gross shear / flexure yield interaction design capacity
$\phi V_{gs\,\mathrm{sup}i}$	Support angle cleat leg gross shear yield design capacity & Support cleat leg gross shear / flexure yield interaction design capacity
ϕV_{gsw}	Web gross shear yield design capacity & Web gross shear / flexure yield design capacity
ϕV_{gwi}	Angle cleat web leg gross interaction shear design capacity
ϕV_{i}	Cleat design shear capacity
ϕV_{lti}	Cleat bolt hole 1st longitudinal tearing design shear capacity
ϕV_{lw}	Web and cleat bolt longitudinal tearing design shear capacity
$\phi V_{ls\,\mathrm{sup}}$	Support bolt hole 1st longitudinal tearing design shear capacity
ϕV_{ltw}	Web bolt hole 1st longitudinal tearing design shear capacity
ϕV_{nfi}	Cleat net flexure ultimate design shear capacity
$\phi V_{nf\,\mathrm{sup}i}$	Support angle cleat leg net flexure ultimate design shear capacity
$\phi V_{ns\,\mathrm{sup}i}$	Support angle cleat leg net shear ultimate design capacity & Net shear ultimate interaction design capacity
ϕV_{nfw}	Web net flexure ultimate design shear capacity
ϕV_{nsw}	Web net ultimate design shear capacity
ϕV_{nsi}	Cleat net ultimate design shear capacity
ϕV_{nwi}	Angle cleat web leg net interaction shear design capacity
ϕV_{rcon}	Connection design shear capacity with axial load
ϕV_{sup}	Support shear design shear capacity
$\phi V_{ls\,\mathrm{sup}}$	Support bolt hole 1st transverse tearing design shear capacity
$\phi V_{t\,\mathrm{sup}}$	Support tearing design shear capacity
ϕV_{tti}	Cleat bolt hole 1st transverse tearing design shear capacity
$\phi V_{tt\,\mathrm{sup}i}$	Support cleat leg bolt hole 1st transverse tearing design shear capacity
ϕV_{tw}	Web and cleat bolt transverse tearing design shear capacity
ϕV_{ttw}	Web bolt hole 1st transverse tearing design shear capacity
ϕV_{us}	Design shear capacity adjacent to concrete free edge
ϕV_{wb}	Web design shear capacity
ϕV_{web}	AC web connection design shear capacity
ϕV_{ws}	Weld design shear capacity
ϕV_{ww}	Web weld group design shear capacity
ψ_{rx}	Bolt row x capacity reduction factor for plastic distribution limits

σ_b	Concrete bearing capacity under a base plate
$\sigma_{y2.5\%}$	Supplier statistical 2.5 percentile yield stress for a steel grade
$\sigma_{y2.5\%}$	Supplier statistical 97.5 percentile yield stress for a steel grade
Admt 7/04	Amendment to MEP and MEPS connections July 2004
Admt 11/05	Amendment to BCS and BTS and inclusion of BBS connections November 2005

III. Introduction

The purpose of the *Structural Steelwork Connections Guide* is to provide structural engineers with a rapid and cost-effective way to specify the majority of structural steelwork connections, in accordance with accepted fabrication industry norms. Specification of these connections also facilitates the development of reliable cost estimates by designers, fabricators, consulting quantity surveyors and constructors.

This third edition of the *Structural Steelwork Connections Guide* now short titled *Steel Connect* is technically identical to the second edition which incorporated updated design methods to some of the connections used in the 1999 version as well as the new Angle Cleat connection range. The 1999 version was therefore superseded by the second edition. The second edition therefore remains technically valid, however future amendments and developments will only be published for Steel Connect leaving the second edition obsolete.

The *Steel Connect* is published in two parts. Part 1 includes detailed design procedures for the connections. Part 2 includes tables of pre-engineered connections developed using the procedures described in Part 1.

In Part 1 the technical background for each of the connections includes a summary of the key design aims, and the steps and calculations required to achieve them. The maximum design actions and design capacities listed in Part 2 are based on the limit states method. Connection design capacities are calculated in accordance with the *New Zealand Steel Structures Standard, NZS3404:1997* . Factored limit state design actions on connections are calculated in accordance with the *New Zealand Loadings Standard, AS/NZS 1170*.

The *Structural Steelwork Connections Guide* has two companion documents that have been developed by the Steel Structures Analysis Service to assist with the preparation of design documentation and budget estimates for steel structures. These are the *Online Structural Steelwork Estimating Guide, SCNZ-21,* and the *Code of Practice for Structural Steelwork Documentation, SCNZ-12 :2006 .* Both these documents utilise the connection designation system used in *Steel Connect.*

... ," he has made the first one obsolete; and what is obsolete and aging will soon disappear."

Hebrews 8:13

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IV. Isometric Drawings of Connection Types

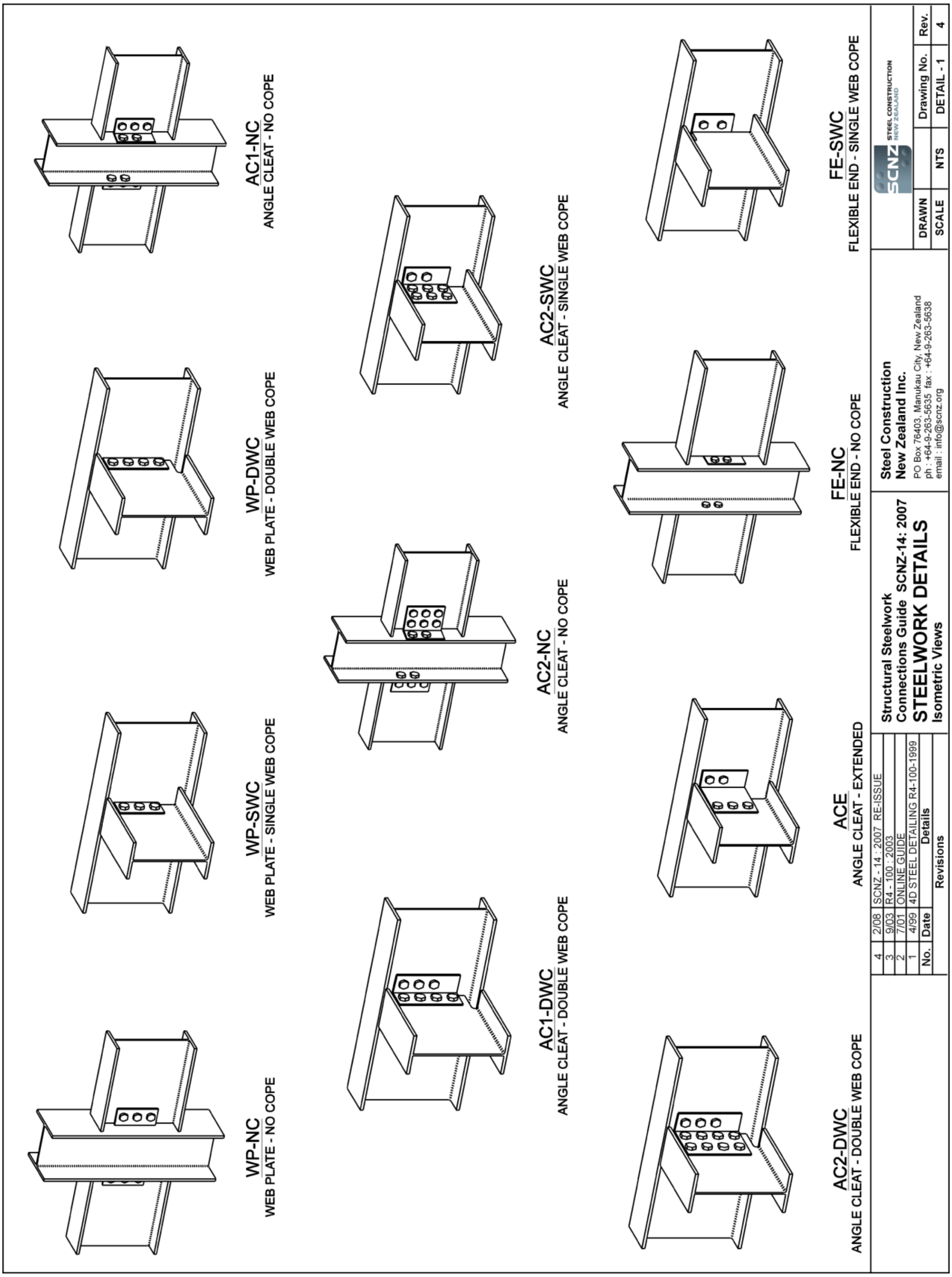


Figure 1 Details-1 drawing

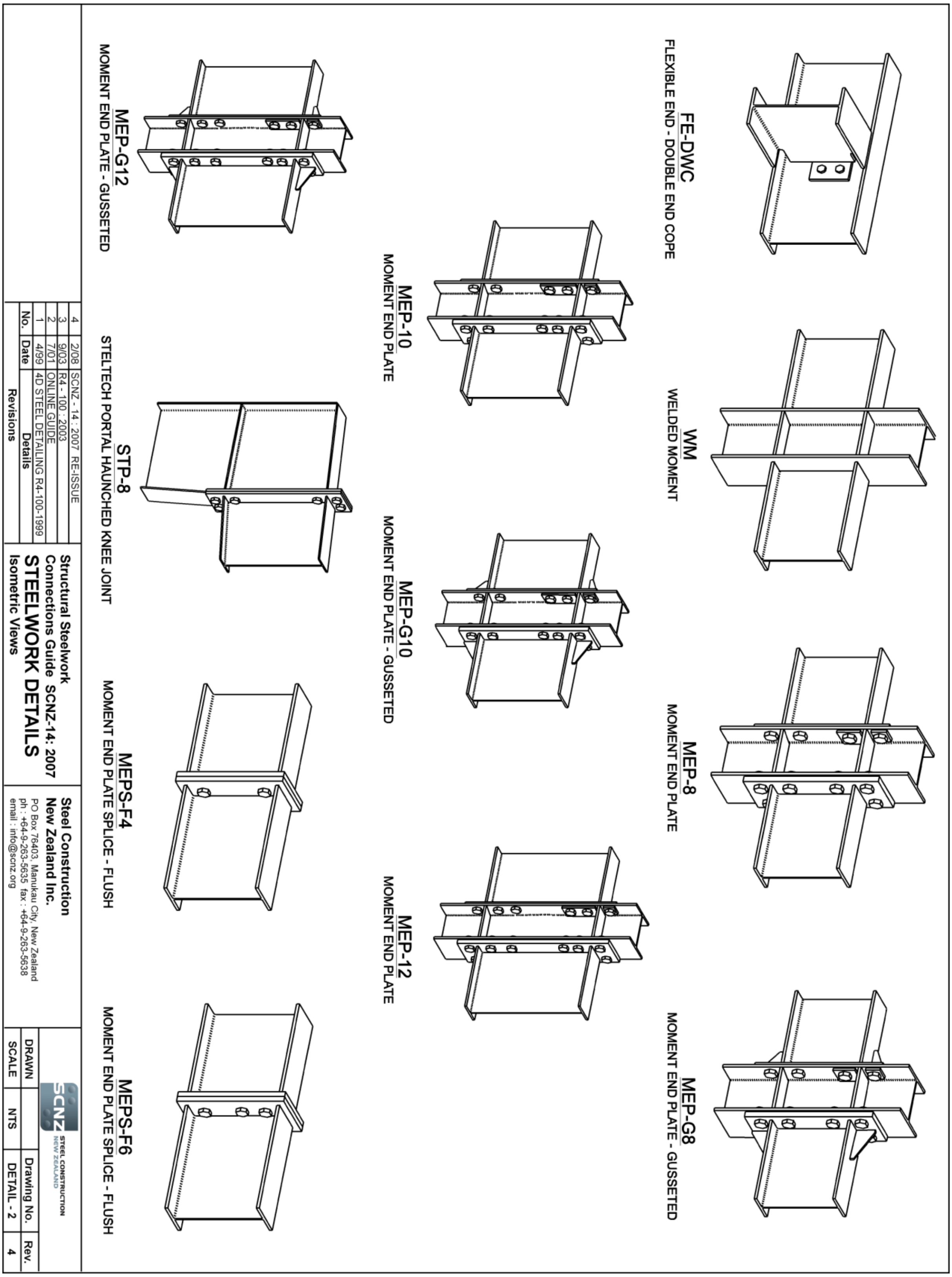


Figure 2 Details-2 drawing

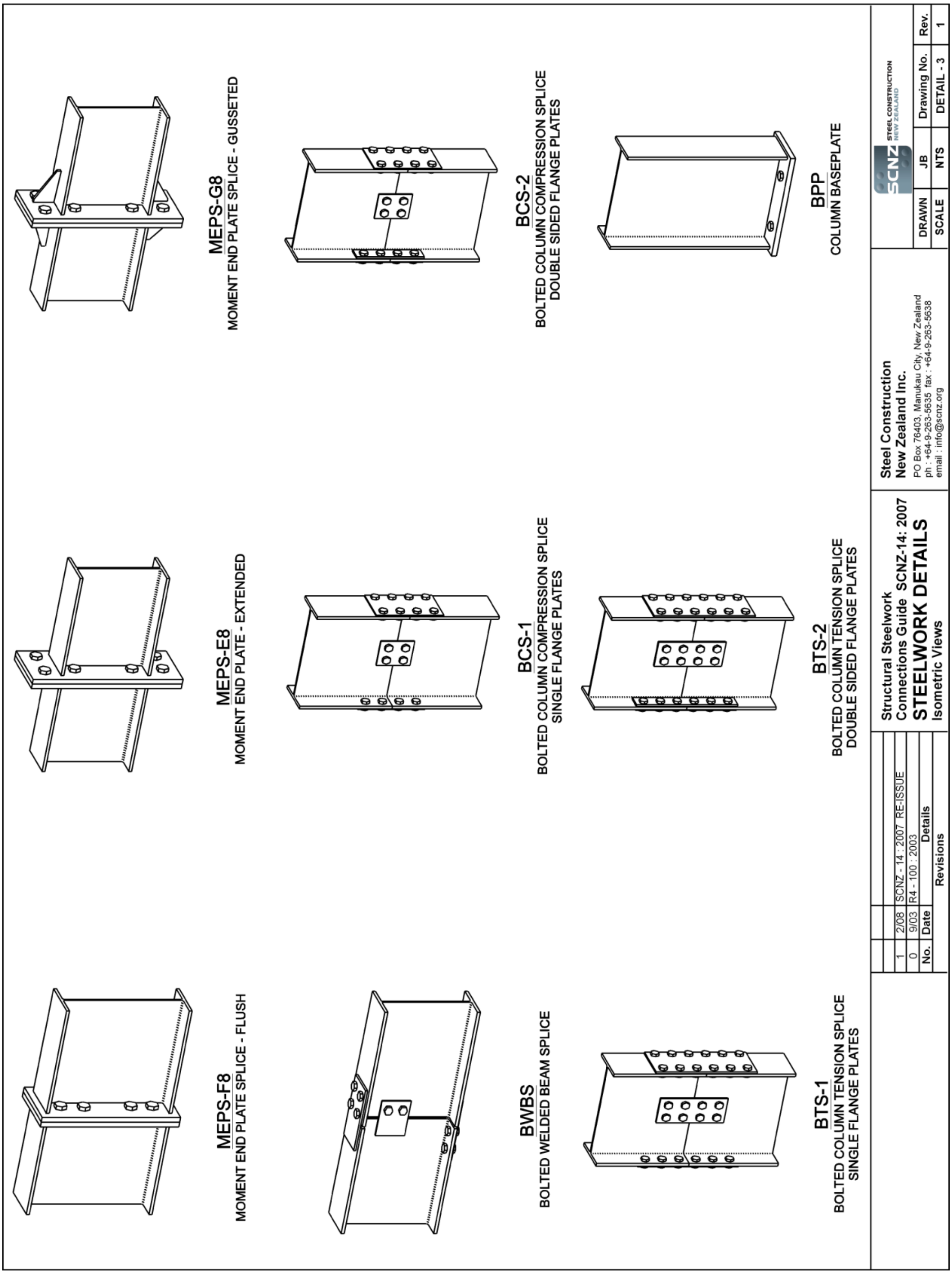


Figure 3 Details-3 drawing

V. Using Connections Guide Part 1: Design Procedures

A. General

The remaining sections of this Guide describe the method and give the necessary information for selecting and specifying steelwork connections.

Typical connection design detail drawings are included in the last section of the Guide. Designers are encouraged to use these details as the basis for preparing design documentation consistent with industry conventions.

Each connection category begins with a discussion of the design objectives and method used to design the connections tabulated. A design procedure is listed, accompanied with notation drawings, designated with the prefix, ENG-. The notation drawings help the designer identify the variables used in the design procedure.

Tables of pre-engineered connections are included in Part 2: Connection Tables. They are designated by configuration and minimum load rating as a percentage of relevant section capacity. Each connection within the tables is also shown with a calculated design capacity not less than that corresponding to its percentage loading designation. In many cases the design capacity of the connections will be greater than the percentage loading designation.

Also in Part 2: Connection Tables are detailed drawings, that when read in conjunction with the connection tables and detailing constants, have sufficient information to allow workshop detailers to prepare fabrication drawings of the connections.

B. Specific Design of Connections

The design procedures have been used in preparing the pre-engineered connections in Part 2: Connection Tables. They are particularly suited to incorporation into spreadsheet and other software that may developed for in-house use by structural engineering designers. Refer to the Copyright Notice for limitations on commercial software development using Part 1: Design Procedures and Part 2: Connection Tables.

The procedures allow a structural engineering designer to readily develop connections that fall outside the loadings and sections range of those tabulated in Part 2: Connection Tables.   It is recommended that the Detailing Constants set out in Part 2: Connection Tables, be used wherever possible to maintain fabrication consistency and economy.

C. Notes on Definition of Terms

All actions on the bolt group, plate or beam are referenced in terms of the section longitudinal and transverse axes. For example "Bolt hole 1st transverse tearing", refers to tearing in a direction transverse to or across the section. Transverse direction may also be referred to as y-direction. "Bolt hole 1st longitudinal tearing" refers to tearing in a direction parallel with the longitudinal axis of the beam or column section. Longitudinal direction may also be referred to as x-direction.

Where the term "resultant" is used it describes the resultant action derived from the vector addition of the transverse and longitudinal components of the relevant action.
Actions on or dimensions of plate elements are typically described with a subscript *i*. Actions on or dimensions of a beam or column section are described with a subscript *b*.

Subscripts are provided to material strength reduction factors as appropriate as follows:

ϕ_b = Bolt strength reduction factor.

ϕ_c = Concrete strength reduction factor.

ϕ_s = Steel strength reduction factor.

ϕ_w = Weld strength reduction factor.

Specific definitions related to each connection are noted within each relevant section.

Refer to the relevant connection drawings for description of other terms.

D. Connection Ductility Demands

Beam to column face connections are required to possess design capacity sufficient to satisfy the ductility demands of primary members in frames of varying levels of seismic ductility. Under design seismic events the connection components will be subjected to over-strength actions developed in the members. The over-strength actions are related to frame ductility category, material variability and strain hardening. In the Connections Guide the three frame ductility categories are referenced. These include:

Ductile Frames:	$3 < \mu \leq 6.0$
Limited Ductile Frames:	$1.25 < \mu \leq 3.0$
Elastic Frames:	$\mu = 1.0$

E. Steel Section and Plate Properties

Capacity design approaches are used for many of the moment resisting connections in the Structural Steelwork Connections Guide. These assume the use of AS/NZS 3679.1 G300 sections and NZ Steel 300MOD welded sections with a supplier's material variation factor,

$\phi_{om} = \dfrac{\sigma_{y97.5\%}}{\sigma_{y2.5\%}} \leq 1.20$. For steel sections of other grades or sourced from suppliers with higher material variation factors, greater over-strength factors need to be used. Refer to Table 12.2.8.1(1) NZS3404:1997 for the appropriate over-strength factors and ensure weld sizes and other components which are required to resist section over-strength actions are adjusted as necessary.

F. Weld Design Information

Welds are specified as SP E48XX in accordance with AS1554.1 unless noted otherwise, with $\phi_w = 0.8$ and $f_{uw} = 480$ MPa.

Fillet welds are used in preference to butt welds for most connections. However where fillet welds with leg lengths greater than 12mm would be required by calculation, complete penetration butt welds are specified. This is not for any technical reason but reflects better welding economy for the commonly used structural welding processes. Where larger leg fillet welds are being considered, seek the advice of a local fabricator as to recommended practice.

G. Bolt Design Information

All bolts are specified as Property Class 8.8 high strength bolts in accordance with AS/NZS 1252:1996, unless noted otherwise. The pinned base plate connections BPP use Property Class 4.6 /S holding down bolts.

Bolts are designed using N/-, i.e. Design capacities calculated for threads included in shear plane, in all connections except for WP, Web Plate, connections.

In Web Plate connections, M20 8.8/S bolts are designed using X/S, or threads excluded from the shear plane design capacities, where the cleat plate and the web of the section are each not greater than 9.0mm thick. For other web plate cleat or web thickness, 8.8/S bolts are designed for N/S, or threads included, design capacities.

H.Section Properties of SWC I-Sections

The formulae for determining properties of SWC I-sections, used in the connection tables are as follows. The contribution of the section root radius has been ignored in all cases. A for more detailed derivation using section root radius is possible.[1]

1. Elastic Section Modulus

$$Z_{cope} = \left[\frac{i_{cope}}{(d_w + t_f - y_c)}; \frac{i_{cope}}{y_c} \right]_{\min}$$ Elastic section modulus

$$i_{cope} = \frac{b_f t_f^3}{12} + b_f t_f (y_c - 0.5t_f)^2 + \frac{t_w d_w^3}{12} + t_w d_w (t_f - y_c + 0.5d_w)^2$$ Moment of inertia

$$y_c = \frac{0.5 b_f t_f^2 + d_w t_w \left(t_f + 0.5 d_w\right)}{b_f t_f + d_w t_w}$$ Elastic neutral axis from underside of flange

$$d_w = d_{bc} - t_f$$ Depth of coped web

2. Plastic Section Modulus

a) Plastic Neutral Axis in Web

$$S_{cope} = b_f t_f \left(y_s - \frac{t_f}{2} \right) + \frac{t_w}{2} \left(\left(y_s - t_f \right)^2 + \left(d_w + t_f - y_s \right)^2 \right)$$ Plastic section modulus

$$y_s = \frac{d_w t_w + 2 t_f t_w - b_f t_f}{2 t_w} \geq t_f$$ Plastic neutral axis from underside of flange

b) Plastic Neutral Axis in Flange

$$S_{cope} = d_w t_w \left(t_f - y_s + \frac{d_w}{2} \right) + \frac{b_f}{2} \left(y_s^2 + \left(t_f - y_s \right)^2 \right)$$ Plastic section modulus

$$y_s = \frac{d_w t_w + b_f t_f}{2 b_f} \leq t_f$$ Plastic neutral axis from underside of flange

[1] Hogan, Thomas, Syam, "Design of Structural Connections, 4th Edition", AISC, North Sydney, 1994, p.321

VI. WP: Web Side Plates

A. Design Objectives

Possess design capacity to satisfy gravity ultimate limit state loads

Provide twist restraint to the supported and supporting beams about their respective longitudinal axes, consistent with the restraint provisions of NZ Steel Structures Standard NZS3404:1997 and HERA report R4-92.

Have sufficient rotation ductility to accommodate gravity load and seismic drift induced rotations of 0.030 radians, without collapse.

Have sufficient ductility to accommodate thermal strains induced by extreme fire events without collapse.

B. Design Features

Typical limiting conditions are : Shear or flexural yield of the cleat or web; Tearing yield of cleat or beam web adjacent to an extreme bolt; Block shear / tension yield of the beam web; bolt shear.

The web plate depths are typically greater than half the supported beam depth.

To prevent the possibility of bolt shear under extreme seismic drift induced rotations and fire conditions, the cleat plate thickness is limited to half the bolt diameter and the bolt group capacity is greater than the lowest limiting flexural condition of the cleat or beam web.

M20 8.8/S bolts are designed for X/S, or threads excluded, design capacities, where the cleat plate and the web of the section are each not greater than 9.0mm thick. This is because bolts specified in accordance with AS/NZS 1252:1996, have a minimum unthreaded shank length of 10mm. Therefore threads will always be excluded from the shear plane, no matter what side of the connection the head of the bolt is located on. A minimum tolerance for shimming of 1.0mm is also available without affecting the bolt design capacity. For other plate or web thickness, M20 8.8/S bolts are designed for N/S, or threads included, design capacities.

Welds of cleats to the supports have design tensile capacity greater than the design tension yield capacity of the cleat plate. This improves connection performance under high thermal strain conditions during fire.

The calculation of net flexure ultimate design shear capacity of the web plates, ϕV_{nfi} , uses a similar approach used for the gross flexure yield design shear capacity, ϕV_{gfi} . However $0.85f_{ui}$ substitutes for f_{yi} , and the plastic section modulus of the net section is based on the gross section plastic modulus adjusted by the ratio of net area to gross area of the plate. A similar approach is used for calculating the design shear capacity of DWC webs.

C. Design Procedure

1. Governing Criteria

The minimum bolt group, web plate and section web design strength limits shall exceed the applied design action.

For beam to column joints there are two additional seismic design requirements. The first is that the bolt group design flexural capacity shall exceed the minimum design flexural strength limit of the plate or section web. The second is that adequate end gap between the support and the beam is available to accommodate 0.030 radians of relative rotation.

The design tension ultimate capacity of the welds between the cleat and the support shall be not less than the design gross tension yield capacity of the cleat.

The plate or web thickness shall not exceed half the bolt diameter.

The web plate depth shall not be much less than half the supported beam depth.

Bolt edge distances.

2. Design Actions

Factored shear force.

Beam to column connections, seismic drift induced shear, based on $\frac{L_n}{d} \geq 20$.

Plate gross tension yield force

3. Connection Strength Limits

Connection design shear capacity

4. Bolt Group Strength Limits.

Shear at eccentricity e.
Flexure at zero transverse shear.
Bolt capacity related to plate thickness.

5. Web Plate Design Strength Limits.

Bolt hole 1^{st} resultant bearing.
Bolt hole 1^{st} transverse tearing.
Bolt hole 1^{st} longitudinal tearing.
Cleat gross flexure yield
Cleat net flexure ultimate
Cleat gross shear yield
Cleat gross shear / flexure yield
Cleat net shear ultimate
Cleat net shear / flexure ultimate

6. Section Web Design Strength Limits.

Bolt hole 1^{st} resultant bearing.
Bolt hole 1^{st} transverse tearing.
Bolt hole 1^{st} longitudinal tearing.

a) No Cope

Gross shear yield: HR & Welded sections

b) Single Web Cope

Gross flexure yield
Gross shear yield
Gross shear / flexure yield
Web shear area SWC HR & Welded sections
Block shear

c) Double Web Cope

Gross flexure yield
Net flexure ultimate
Gross shear yield
Gross shear / flexure yield
Net shear ultimate
Net shear / flexure ultimate
Block shear

7. Weld Design Strength Limits

Weld tension ultimate.

D. Design Formulae

1. Governing Criteria.

$V^* \le \phi V_{con}$	General condition.
$V_{drift}{}^* \le \left[\phi V_b ; \phi V_i ; \phi V_{wb}\right]_{\min}$	Beam / column seismic shear
$M^*_{drift} \le \phi M_b$	Beam / column seismic moment
$\dfrac{a_c}{s_{g1} - a_{e1}} \le 33$	Beam / column seismic end gap
$\dfrac{a_c}{s_{g1} - a_{e1}} \le 50$	Beam / beam end gap
$N^*_{gti} \le \phi N_{ww}$	Weld
$t_i \le \dfrac{d_f}{2}$	Plate / bolt thickness
$d_i \ge 0.45d$	Plate depth.
$\left[a_{e4}; a_{e5}; a_{e6}; a_{e7}\right] \ge 1.75 d_f$	Edge distances: Manual flame cut or crop

2. Design Actions

V^*	Factored shear force
$V_{drift}{}^* = G + Q_u + E_{drift}$	Beam / column seismic drift
$E_{drift} = \dfrac{M^*_{drift}}{10d}$	Seismic shear for beam $\dfrac{L_n}{d} \ge 20$
$M^*_{drift} = e\left[\phi V_{tti} ; \phi V_{gfi} ; \phi V_{nfi}\right]_{\min}$	Seismic moment for NC beam / column joint
$M^*_{drift} = e\left[\phi V_{tti} ; \phi V_{gfi} ; \phi V_{nfi} ; \phi V_{gfw} ; \phi V_{nfw}\right]_{\min}$	Seismic moment for SWC or DWC beam /column
$N^*_{gti} = \phi_s d_i t_i f_{yi}$	Plate gross tension yield force

3. Connection Strength Limits

$\phi V_{con} = \left[\phi V_b ; \phi V_i ; \phi V_{wb}\right]_{\min}$	Connection design shear capacity

4. Bolt Group Strength Limits

$\phi V_b = Z_b \phi V_f$	Shear at eccentricity e
$\phi M_b = n_p Z_e \phi V_f e$	Flexure
$\phi V_f = \phi_b V_{fx}$ if $\left[t_i ; t_w\right]_{\max} \le 9.0$ mm	Threads excluded strength
$\phi V_f = \phi_b V_{fn}$ if $\left[t_i ; t_w\right]_{\max} > 9.0$ mm	Threads included strength

5. Web Plate Strength Limits.

$$\phi V_i = \left[\phi V_{bi} ; \phi V_{tti} ; \phi V_{lti} ; \phi V_{gfi} ; \phi V_{nfi} ; \phi V_{gsi} ; \phi V_{nsi}\right]_{\min}$$

$\phi V_{bi} = \phi_s Z_b 3.2 t_i d_f f_{ui}$	Bolt hole 1st resultant bearing
$\phi V_{tti} = n_p \phi_s a_{eyi} t_i f_{ui}$	Bolt hole 1st transverse tearing

$\phi V_{lti} = n_p \phi_s Z_e a_{e7} t_i f_{ui}$ Bolt hole 1st longitudinal tearing

$\phi V_{gfi} = \phi_s \dfrac{t_i d_i^{\ 2}}{4e} f_{yi}$ Cleat gross flexure yield

$\phi V_{nfi} = \phi_s \left(1 - \dfrac{n_p d_h}{d_i}\right) \dfrac{t_i d_i^2}{4e} 0.85 f_{ui}$ Cleat net flexure ultimate

$\phi V_{gsi} = \phi_s 0.5 t_i d_i f_{yi}$ if $V^* \le 0.75 \phi V_{gfi}$ Cleat gross shear yield

$\phi V_{gsi} = \phi_s 0.5 t_i d_i f_{yi} \left[2.2 - \left(\dfrac{16V^*}{\phi V_{gfi}} \right) \right]$ if $V^* > 0.75 \phi V_{gfi}$

Cleat gross shear / flexure yield

$\phi V_{nsi} = \phi_s 0.6 f_{ui} A_{ni}$ if $V^* \le 0.75 \phi V_{nfi}$ Cleat net shear ultimate

$\phi V_{nsi} = \phi_s 0.6 f_{ui} A_{ni} \left[2.2 - \left(\dfrac{16V^*}{\phi V_{nfi}} \right) \right]$ if $V^* > 0.75 \phi V_{nfi}$

Cleat net shear / flexure ultimate

6. Section Web Strength Limits.

a) General

$\phi V_{wb} = \left[\phi V_{bw} ; \phi V_{ttw} ; \phi V_{ltw} ; \phi V_{gsw} \right]_{\min}$ No Copes

$\phi V_{wb} = \left[\phi V_{bw} ; \phi V_{ttw} ; \phi V_{ltw} ; \phi V_{gfw} ; \phi V_{nfw} ; \phi V_{gsw} ; \phi V_{nsw} ; \phi V_{bsw} \right]_{\min}$

SWC & DWC

$\phi V_{bw} = \phi_s Z_b 3.2 t_w d_f f_{ub}$ Bolt hole 1st resultant bearing

$\phi V_{ttw} = n_p \phi_s a_{eyb} t_w f_{ub}$ Bolt hole 1st transverse tearing.

$\phi V_{ltw} = n_p \phi_s Z_e a_{e1} t_w f_{ub}$ Bolt hole 1st longitudinal tearing.

b) No Cope

$\phi V_{gsw} = \phi_s 0.6 t_w d f_{yw}$ Gross shear yield: HR sections

$\phi V_{gsw} = \phi_s 0.6 t_w (d - 2t_f) f_{yw}$ Gross shear yield: Welded sections

c) Single Web Cope

$\phi V_{gfw} = \phi_s \dfrac{Z_{ec}}{e_v} f_{yw}$ Gross flexure yield

$\phi V_{gsw} = \phi_s 0.6 A_w f_{yw}$ if $V^* \le 0.75 \phi V_{gfw}$ Gross shear yield

$\phi V_{gsw} = \phi_s 0.6 A_w f_{yw} \left[2.2 - \left(\dfrac{16V^*}{\phi V_{gfw}} \right) \right]$ if $V^* > 0.75 \phi V_{gfw}$

$A_w = d_{bc} t_w$ Web shear area HR sections

$A_w = (d_{bc} - t_f) t_w$ Web shear area Welded sections

$\phi V_{bsw} = \phi_s \begin{bmatrix} 0.6 A_{gs} f_{yw} + A_{nt} f_{ub} ; \\ 0.6 A_{ns} f_{ub} + A_{gt} f_{yw} \end{bmatrix}_{\max}$ Block shear

d) Double Web Cope

$\phi V_{gfw} = \phi_s \dfrac{t_w d_{bc}^2}{4e_v} f_{yw}$ Gross flexure yield

$$\phi V_{nfw} = \phi_s \left(1 - \frac{n_p d_h}{d_{bc}}\right) \frac{t_w d_{bc}^2}{4e} 0.85 f_{ub}$$ Net flexure ultimate

$$\phi V_{gsw} = \phi_s 0.5 t_w d_{bc} f_{yw} \quad \text{if } V^* \le 0.75 \phi V_{gfw}$$ Gross shear yield

$$\phi V_{gsw} = \phi_s 0.5 t_w d_{bc} f_{yw} \left[2.2 - \left(\frac{16V^*}{\phi V_{gfw}}\right)\right] \quad \text{if } V^* > 0.75 \phi V_{gfw}$$

Gross shear / flexure yield

$$\phi V_{nsw} = \phi_s 0.6 f_{ub} \left(d_{bc} - n_p d_h\right) t_w \quad \text{if} \quad V^* \le 0.75 \phi V_{nfw}$$

Net shear ultimate

$$\phi V_{nsw} = \phi_s 0.6 f_{ub} \left(d_{bc} - n_p d_h\right) t_w \left[2.2 - \left(\frac{16V^*}{\phi V_{nfw}}\right)\right] \text{ if } V^* > 0.75 \phi V_{nfw}$$

Net shear / flexure ultimate

$$\phi V_{bsw} = \phi_s \begin{bmatrix} 0.6 A_{gs} f_{yw} + A_{nt} f_{ub}; \\ 0.6 A_{ns} f_{ub} + A_{gt} f_{yw} \end{bmatrix}_{\max}$$ Block shear

7. Welds

$$\phi N_{ww} = 2 \phi_w 0.6 f_{uw} \frac{t_{ww}}{\sqrt{2}} d_i$$ Weld tension ultimate.

$$\phi_w = 0.8$$ SP fillet weld

8. Definitions of Terms

$a_c = d - a + a_{e6} - d_i$	End gap
$a_{eyi} = \left[a_{e3}, a_{e6}\right]_{\min}$	
$a_{eyb} = a_{e3}$	NC edge distances
$a_{eyb} = \left[a_{e3}; a_{e4}\right]_{\min}$	SWC edge distances
$a_{eyb} = \left[a_{e3}; a_{e4}; a_{e5}\right]_{\min}$	DWC edge distances
$a_{e3} = s_p - \frac{d_h}{2}$	Inter-bolt edge distance
a_{e4}	Top bolt to top coped edge y-distance
$a_{e5} = d_{\sup} - a - \left(n_p - 1\right) s_p - a_{cb}$	Bottom bolt to coped edge y-distance
$a_{e6} = \frac{\left(d_i - \left(n_p - 1\right) s_p\right)}{2}$	End bolt edge y-distance
$a_{e7} = b_i - s_{g1}$	Cleat bolt side edge x-distance
$A_{ni} = \left(d_i - n_p d_h\right) t_i$	Plate net area
$A_{gt} = a_{e1} t_w$	Block gross tension area
$A_{nt} = \left(a_{e1} - d_h / 2\right) t_w$	Block net tension area
$A_{gs} = \left[a_{e4} + \left(n_p - 1\right) s_p\right] t_w$	Block gross shear area
$A_{ns} = A_{gs} - \left[\left(n_p - 0.5\right) d_h\right] t_w$	Block net shear area
$d_{bc} = d - a + a_{e4}$	SWC coped section depth
$d_{bc} = d - a + a_{e4} - a_{cb}$ if $d \le d_{\sup}$	DWC coped section depth
$d_{bc} = d_{\sup} - a + a_{e4} - a_{cb}$ if $d > d_{\sup}$	DWC coped section depth
$d_h = d_f + 2$ for $d_f \le 24$	Hole diameter
$e = s_{g1}$	Reaction eccentricity
$e_v = L_c - a_{e1} + s_{g1}$	Cope eccentricity

S_{cope}	SWC Coped section plastic modulus
$Z_b = \dfrac{n_p}{\sqrt{1+\left[\dfrac{6e}{s_p\left(n_p+1\right)}\right]^2}}$	Bolt interaction factor for single line of bolts $n_p \neq 1$
Z_{cope}	SWC Coped section elastic modulus
$Z_e = \dfrac{s_p\left(n_p+1\right)}{6e}$	Bolt group flexure factor, single line of bolts, $n_p \neq 1$
$Z_{ec} = \left[S_{cope};15Z_{cope}\right]_{\min}$	SWC Cope effective section modulus

Note: Refer to Section V for formulas for S_{cope} and Z_{cope} .

E. WP Web Plate Drawings

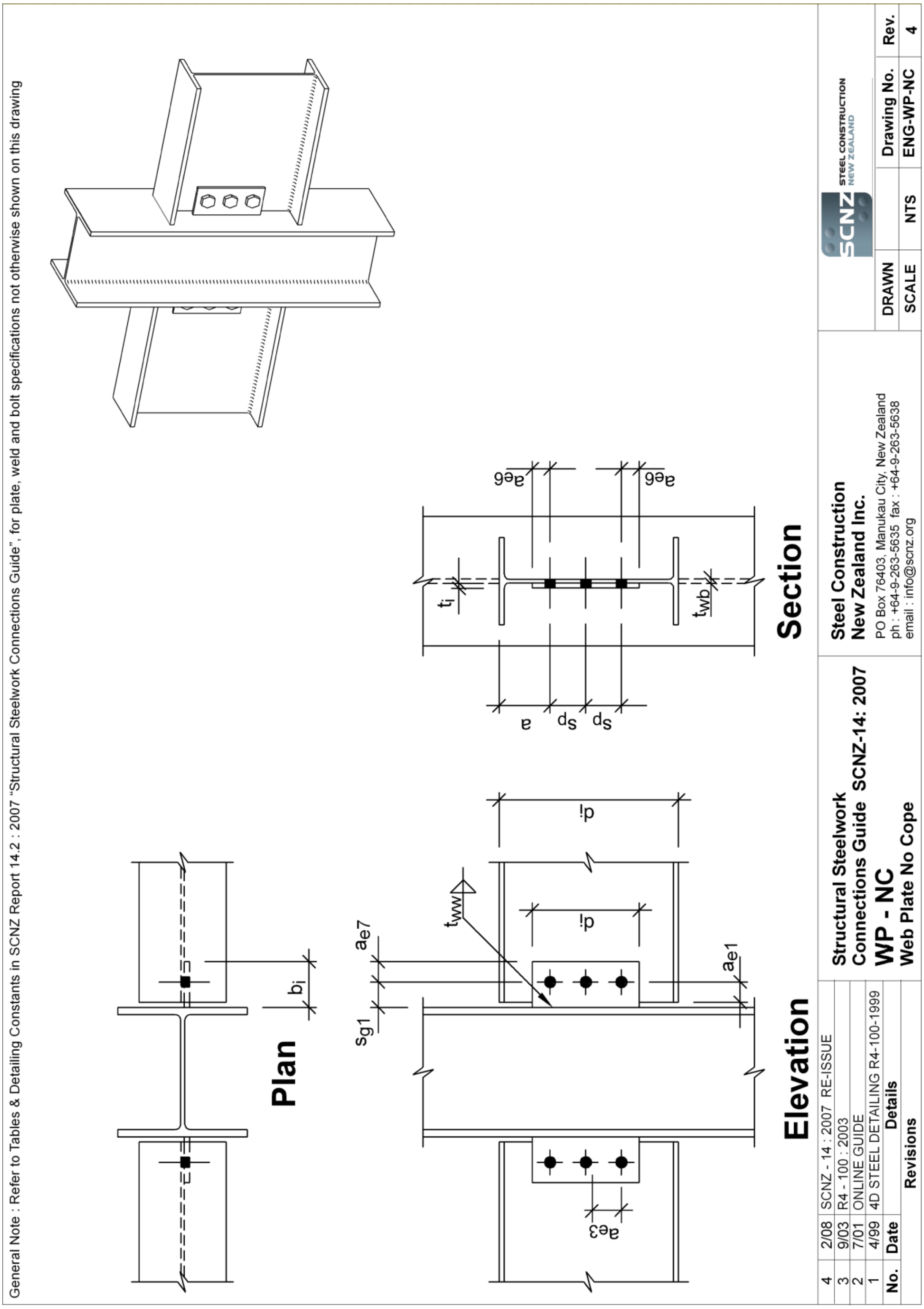


Figure 4 WP NC drawing

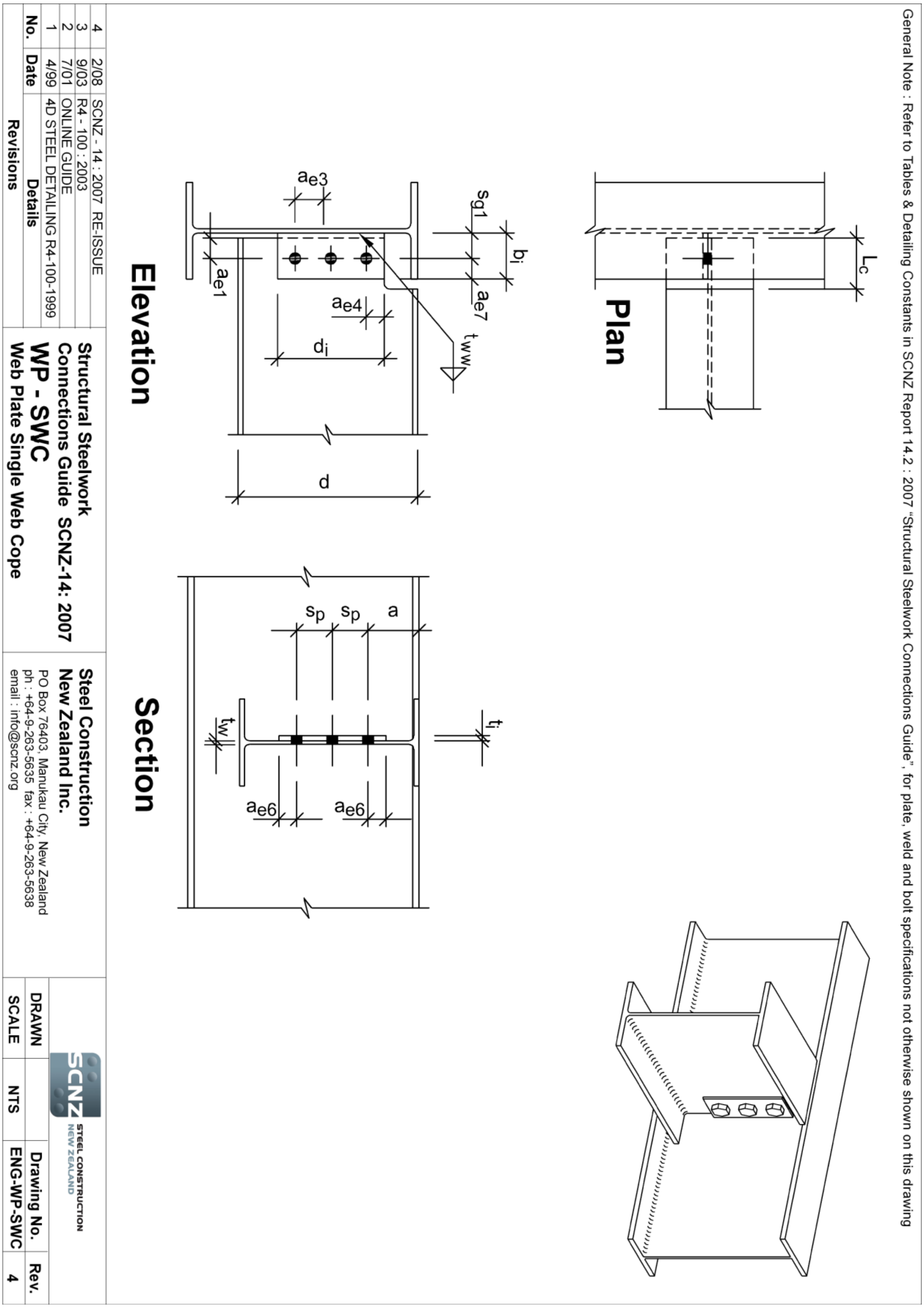


Figure 5 WP-SWC drawing

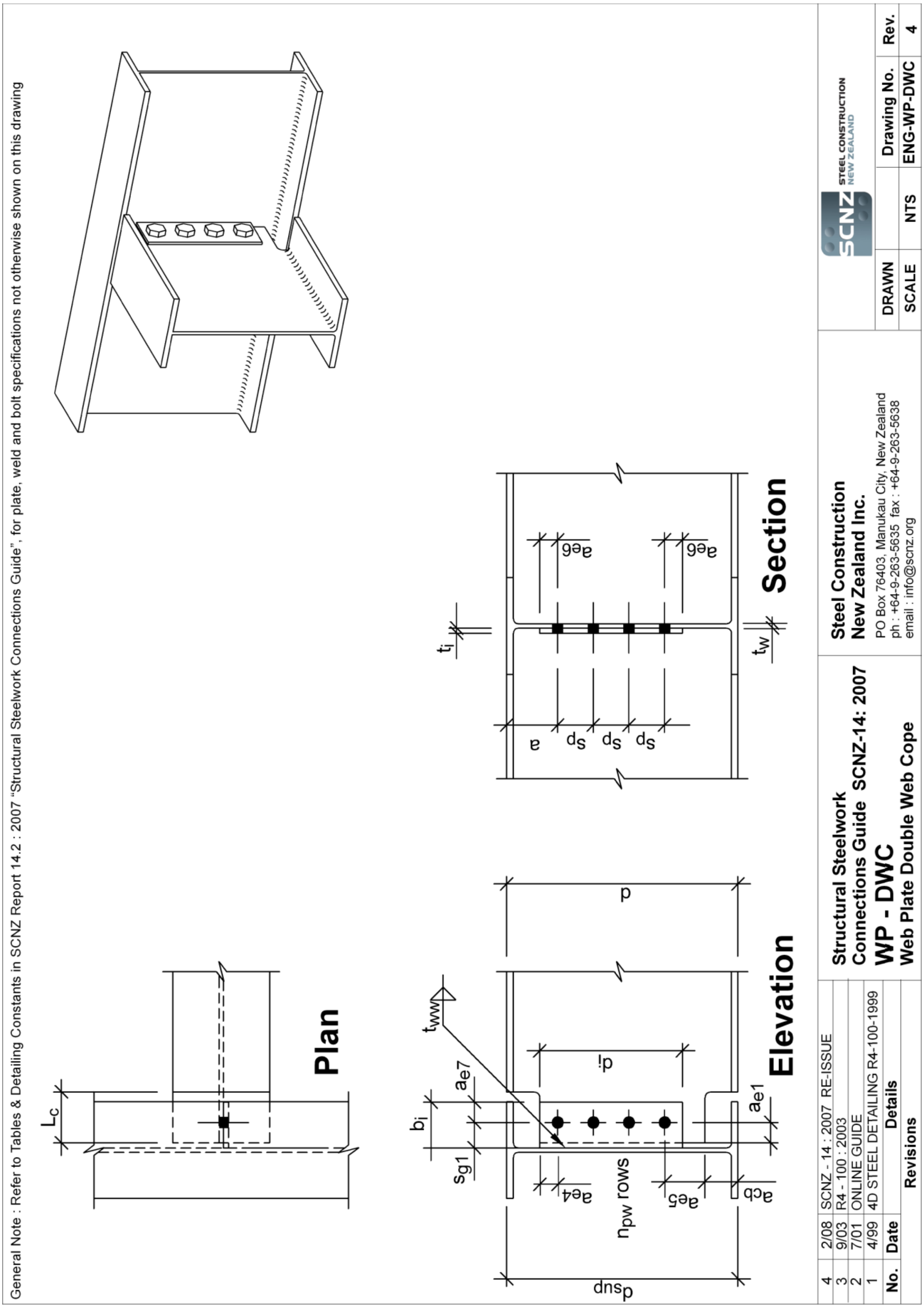


Figure 6 WP-DWC drawing

VII. AC: Angle Cleat

A. Design Objectives

Possess design capacity to satisfy gravity ultimate limit state loads

Provide twist restraint to the supported and supporting beams about their respective longitudinal axes, consistent with the restraint provisions of NZ Steel Structures Standard NZS3404:1997 and HERA report R4-92.

Have sufficient rotation ductility to accommodate gravity load and seismic drift induced rotations of 0.030 radians, without collapse.

Have sufficient ductility to accommodate thermal strains induced by extreme fire events without collapse.

B. Design Features

Typical limiting conditions are : Shear of the cleat or bolts; Block shear around bolt groups in support leg and beam web; Shear / flexural yield of coped section.
The angle cleat depths are typically greater than half the supported beam depth.

To prevent the possibility of bolt shear under extreme seismic drift induced rotations and fire conditions, the angle cleat leg thickness is limited to half the bolt diameter and the bolt group capacity is greater than the lowest limiting flexural condition of the cleat or beam web.

M20 8.8/S web bolts are designed for threads excluded, X/S, design capacities, through one shear plane, where the cleat angle and the web of the section are each not greater than 9.0 mm thick. This is because bolts specified in accordance with AS/NZS 1252:1996, have a minimum unthreaded shank length of 10 mm. Therefore threads will always be excluded from one shear plane, no matter what side of the connection the head of the bolt is located on. A minimum tolerance for shimming of 1.0 mm is also available without affecting the bolt design capacity. Where double angle cleats are used, the second shear plane is assumed to coincide with the threaded portion of the bolt shaft and the total bolt shear capacity is calculated as the sum of the capacity on each shear plane. For greater angle cleat or web thickness and all bolting to supporting members, M20 8.8/S bolts are designed for N/S, or threads included, design capacities.

Where beams of different section size and loading are supported either side of the same plate element of a section, ie. the web of a beam or column, care should be taken to ensure that both angle cleats are compatible. Sufficient holes or clear space needs to be available to allow the connecting bolts to be installed. Where necessary provide additional holes to one of the angle cleats to match the other and allow opposing beam support bolts through.

Bolt spacings are limited by a cleat buckling criteria.

C. Design Procedure: Support Connection

1. Governing Criteria

Cleat leg compactness

2. Support Connection Design Strength Limits

Support connection capacity
Bolt shear and bearing
Transverse tearing
Longitudinal tearing

3. Support Bolt Design Strength Limit

Single shear / bolt

4. Support Cleat Leg Limits

Block shear of support leg
Bolt hole 1st resultant bearing / bolt
Bolt hole 1st transverse tearing / bolt
Bolt hole 1st longitudinal tearing / bolt
Gross flexure yield
Net flexure ultimate
Gross shear yield.
Gross shear / flexure interaction
Net shear ultimate
Net shear / flexure ultimate interaction

5. Support Design Strength Limits

Bolt hole 1st resultant bearing / bolt
Bolt hole 1st longitudinal tearing / bolt

D. Design Procedure: Web Connection

1. Governing Criteria

Beam / column seismic shear
Beam / column seismic end gap
Beam / beam end gap
Angle cleat and web thickness / bolt diameter
Angle cleat compactness
Edge distance: manual flame cut or crop
Edge distance: HR or automatic flame cut
Minimum cleat length

2. Web Design Actions

Beam / column seismic drift
Seismic shear for beam $\frac{L_n}{d} \geq 20$
Seismic moment for beam / column joint

3. Web Connection Design Strength Limits

Web connection shear
Resultant bolt shear and bearing
Transverse bolt hole tearing
Longitudinal bolt hole tearing

4. Web Bolt Group Design Strength Limit

Web bolt group capacity
Shear / bolt

5. Web Leg Design Strength Limits

Web plates gross interaction shear
Web plates net interaction shear
Web plates gross flexure yield at N_t^*

Web plates net flexure ultimate at N_t^*
Web plates gross shear yield.
Web plates gross shear / flexure yield interaction
Web plates net shear ultimate
Web plates net shear / flexure ultimate interaction
Bolt hole 1st resultant bearing.
Bolt hole 1st transverse tearing.
Bolt hole 1st longitudinal tearing.

6. Section Web Design Strength Limits

a) General

Uncoped section shear
Single and double coped section shear
Bolt hole 1st resultant bearing
Bolt hole 1st transverse tearing
Bolt hole 1st longitudinal tearing

b) No Web Cope

Gross shear yield: HR sections
Gross shear yield: Welded sections

c) Single Web Cope

Gross flexure yield at cope notch
Gross shear yield at cope notch
Shear / flexure interaction at cope notch
Web shear area HR sections
Web shear area Welded sections
Block shear

d) Double Web Cope

Gross flexure yield at cope notch
Gross shear yield at cope notch
Shear / flexure interaction at cope notch
Block shear

E. Design Formulae: Combined Connection

1. Governing Criteria

$V^* \leq \phi V_{con}$

$\phi V_{con} = \lfloor \phi V_{\sup}; \phi V_{web} \rfloor_{\min}$ Connection design capacity

F. Design Formulae: Support Connection

1. Governing Criteria

$\dfrac{s_{p\,sup}}{t_i} \leq 17.5$ Cleat leg compactness

2. Support Connection Design Strength Limits

$$\phi V_{\sup} = \lfloor \phi V_{gf\,\sup i}; \phi V_{nf\,\sup i}; \phi V_{bs\,\sup i}; \phi V_{gs\,\sup i}; \phi V_{ns\,\sup i}; \phi V_{fb\,\sup}; \phi V_{ts\,\sup}; \phi V_{ls\,\sup} \rfloor_{\min}$$

Support connection capacity

$\phi V_{fb\sup} = Z_{b\sup}\left[\phi V_{f\sup};\phi V_{b\sup i};\phi V_{b\sup}\right]_{min}$ Bolt shear and bearing, $n_a = 1$

$\phi V_{fb\sup} = 2n_{p\sup}\left[\phi V_{f\sup};\phi V_{b\sup i};\phi V_{b\sup}\right]_{min}$ Bolt shear and bearing, $n_a = 2$

$\phi V_{ts\sup} = n_a n_{p\sup}\phi V_{tt\sup i}$ Transverse tearing

$\phi V_{ls\sup} = n_{p\sup} Z_{el\sup}\left[\phi V_{lt\sup i};\phi V_{lt\sup}\right]_{min}$ Longitudinal tearing, $n_a = 1$

3. Support Bolt Design Strength Limit

$\phi V_{f\sup} = \phi_b V_{fn}$ Single shear / bolt

4. Support Cleat Leg Limits

$$\phi V_{bs\sup i} = n_a \phi_s \begin{bmatrix} 0.6A_{gsi}f_{yi} + A_{nti}f_{ui}; \\ 0.6A_{nsi}f_{ui} + A_{gti}f_{yi} \end{bmatrix}_{max}$$ Block shear of support leg

$\phi V_{b\sup i} = n_a \phi_s 3.2 f_{ui} d_f t_i$ Bolt hole 1st resultant bearing / bolt

$\phi V_{tt\sup i} = n_a \phi_s a_{eti} t_i f_{ui}$ Bolt hole 1st transverse tearing / bolt

For $n_a = 1$

$\phi V_{lt\sup i} = \phi_s a_{el\sup i} t_i f_{ui}$ Bolt hole 1st longitudinal tearing / bolt

$$\phi V_{gf\sup i} = \phi_s \frac{t_i d_i^{\,2}}{4e_{\sup}} f_{yi}$$ Gross flexure yield

$$\phi V_{nf\sup i} = \phi_s \left(\frac{A_{n\sup i}}{A_{gi}}\right)\frac{t_i d_i^2}{4e_{\sup}} 0.85 f_{ui}$$ Net flexure ultimate

$\phi V_{gs\sup i} = \phi_s 0.5 t_i d_i f_{yi}$ if $V^* \le 0.75\phi V_{gf\sup i}$ Gross shear yield.

$$\phi V_{gs\sup i} = \phi_s 0.5 t_i d_i f_{yi}\left[2.2 - \left(\frac{1.6V^*}{\phi V_{gf\sup i}}\right)\right] \text{ if } V^* > 0.75\phi V_{gf\sup i}$$

Gross shear / flexure interaction

$\phi V_{ns\sup i} = \phi_s 0.6 f_{ui} A_{n\sup i}$ if $V^* \le 0.75\phi V_{nf\sup i}$ Net shear ultimate

$$\phi V_{ns\sup i} = \phi_s 0.6 f_{ui} A_{n\sup i}\left[2.2 - \left(\frac{1.6V^*}{\phi V_{nf\sup i}}\right)\right] \text{ if } V^* > 0.75\phi V_{nf\sup i}$$

Net shear / flexure ultimate interaction

For $n_a = 2$

$\phi V_{gs\sup i} = 2\phi_s 0.5 t_i d_i f_{yi}$ Gross shear yield.

$\phi V_{ns\sup i} = 2\phi_s 0.6 f_{ui} A_{n\sup i}$ Net shear ultimate

5. Support Design Strength Limits

$\phi V_{b\sup} = \phi_s 3.2 f_{u\sup} d_f t_{\sup}$ Bolt hole 1st resultant bearing / bolt

For $n_a = 1$

$\phi V_{lt\sup} = \phi_s a_{el\sup} t_{\sup} f_{us}$ Bolt hole 1st longitudinal tearing / bolt

G. Design Formulae: Web Connection

1. Governing Criteria

$V_{drift}{}^* \le \phi V_{web}$	Beam / column seismic shear
$\dfrac{a_c}{s_{g2} - a_{e4}} \le 33$	Beam / column seismic end gap
$\dfrac{a_c}{s_{g2} - a_{e4}} \le 50$	Beam / beam end gap
$d_f \ge [2t_{iw}, 2t_w]_{\min}$	Thickness / bolt diameter, $n_a = 1$
$d_f \ge [2t_{iw}, t_w]_{\min}$	Thickness / bolt diameter, $n_a = 2$
$\dfrac{\lfloor s_{g2}; s_{g3}; s_{pw} \rfloor_{min}}{t_i} \le 17.5$	Cleat angle leg compactness
$[a_{e4}; a_{e5}; a_{e6}]_{\min} \ge 1.75 d_f$	Edge distance: manual flame cut or crop
$[a_{e4}; a_{e5}; a_{e6}]_{\min} \ge 1.5 d_f$	Edge distance: HR or automatic flame cut
$d_i \ge 0.45d$	Minimum cleat length

2. Web Design Actions

$V_{drift}{}^* = G + Q_u + E_{drift}$	Beam / column seismic drift
$E_{drift} = \dfrac{M^*_{drift}}{10d}$	Seismic shear for beam $\dfrac{L_n}{d} \ge 20$
$M^*_{drift} = e\phi V_{web}$	Seismic moment for beam / column joint

3. Web Connection Design Strength Limits

$\phi V_{web} = \lfloor \phi V_b; \phi V_{gwi}; \phi V_{nwi}; \phi V_{fbw}; \phi V_{tsw}; \phi V_{lsw}; \phi V_{wb} \rfloor_{\min}$	Web connection shear
$\phi V_{fbw} = Z_{bw}[\phi V_{fw}; \phi V_{bwi}; \phi V_{bw}]_{\min}$	Resultant bolt shear and bearing
$\phi V_{tw} = n_{pw}[\phi V_{ttwi}; \phi V_{ttw}]_{\min}$	Transverse bolt hole tearing, $n_c = 1$
$\phi V_{tw} = n_c n_{pw} Z_{etw}[\phi V_{ttwi}; \phi V_{ttw}]_{\min}$	Transverse bolt hole tearing, $n_c = 2$
$\phi V_{lw} = n_c n_{pw} Z_{elw}[\phi V_{ltwi}; \phi V_{ltw}]_{\min}$	Longitudinal bolt hole tearing

4. Web Bolt Group Design Strength Limit

$\phi V_b = Z_{bw} \phi V_{fw}$	Web bolt group capacity
$\phi V_{fw} = \phi V_{f1} + (n_a - 1)\phi V_{f2}$	Shear / bolt
$\phi V_{f1} = \phi_b V_{fx}$ if $[t_i; t_w]_{\max} \le 9.0$ mm	Threads excluded shear plane 1
$\phi V_{f1} = \phi_b V_{fn}$ if $[t_i; t_w]_{\max} > 9.0$ mm	Threads included shear plane 1
$\phi V_{f2} = \phi_b V_{fn}$	Threads included shear plane 2

5. Web Leg Design Strength Limits

$\phi V_{gwi} = \lfloor \phi V_{gfwi}; \phi V_{gswi} \rfloor_{\min}$	Angle cleats gross interaction shear
$\phi V_{nwi} = [\phi V_{nfwi}; \phi V_{nswi}]_{\min}$	Angle cleats net interaction shear

$\phi V_{gfwi} = n_a \phi_s \dfrac{t_i d_i^{\,2}}{4e} f_{yi}$ — Angle cleats gross flexure yield at N_t^*

$\phi V_{nfwi} = n_a \phi_s \dfrac{A_{nwi}}{A_{gi}} \dfrac{t_i d_i^2}{4e} 0.85 f_{ui}$ — Angle cleats net flexure ultimate at N_t^*

$\phi V_{gswi} = n_a \phi_s 0.5 t_i d_i f_{yi}$ if $V^* \le 0.75 \phi V_{gfwi}$ — Angle cleats gross shear yield.

$\phi V_{gswi} = n_a \phi_s 0.5 t_i d_i f_{yi} \left[2.2 - \left(\dfrac{1.6V^*}{\phi V_{gfwi}} \right) \right]$ if $V^* > 0.75 \phi V_{gfwi}$

Angle cleats gross shear / flexure yield interaction

$\phi V_{nswi} = n_a \phi_s 0.6 f_{ui} A_{nwi}$ if $V^* \le 0.75 \phi V_{nfwi}$ — Angle cleats net shear ultimate

$\phi V_{nswi} = n_a \phi_s 0.6 f_{ui} A_{nwi} \left[2.2 - \left(\dfrac{1.6V^*}{\phi V_{nfwi}} \right) \right]$ if $V^* > 0.75 \phi V_{nfwi}$

Angle cleats net shear / flexure ultimate interaction

$\phi V_{bwi} = n_a \phi_s 3.2 f_{ui} d_f t_i$ — Bolt hole 1st resultant bearing.

$\phi V_{ttwi} = n_a \phi_s a_{eti} t_i f_{ui}$ — Bolt hole 1st transverse tearing.

$\phi V_{ltwi} = n_a \phi_s a_{elwi} t_i f_{ui}$ — Bolt hole 1st longitudinal tearing.

6. Section Web Design Strength Limits

a) General

$\phi V_{wb} = \phi V_{gsw}$ — Uncoped section shear

$\phi V_{wb} = \lfloor \phi V_{gfw} ; \phi V_{gsw} ; \phi V_{bsw} \rfloor_{\min}$ — Single and double coped section shear

$\phi V_{bw} = \phi_s 3.2 f_{ub} d_f t_w$ — Bolt hole 1st resultant bearing

$\phi V_{ttw} = \phi_s a_{e7} t_w f_{ub}$ — Bolt hole 1st transverse tearing

$\phi V_{ltw} = \phi_s a_{elw} t_w f_{ub}$ — Bolt hole 1st longitudinal tearing

b) No Web Cope

$\phi V_{gsw} = \phi_s 0.6 d t_w f_{yw}$ — Gross shear yield: HR sections

$\phi V_{gsw} = \phi_s 0.6 (d - 2t_f) t_w f_{yw}$ — Gross shear yield: Welded sections

c) Single Web Cope

$\phi V_{gfw} = \phi_s \dfrac{Z_{ec}}{e_v} f_{yw}$ — Gross flexure yield at cope notch

$\phi V_{gsw} = \phi_s 0.6 A_w f_{yw}$ if $V^* \le 0.75 \phi V_{gfw}$ — Gross shear yield at cope notch

$\phi V_{gsw} = \phi_s 0.6 A_w f_{yw} \left[2.2 - \left(\dfrac{16V^*}{\phi V_{gfw}} \right) \right]$ if $V^* > 0.75 \phi V_{gfw}$

Shear / flexure interaction at cope notch

$\phi V_{bsw} = \phi_s \begin{bmatrix} 0.6 A_{gs} f_{yw} + A_{nt} f_{ub} ; \\ 0.6 A_{ns} f_{ub} + A_{gt} f_{yw} \end{bmatrix}_{\max}$ — Block shear

d) Double Web Cope

$$\phi V_{gfw} = \phi_s \frac{t_w d_{bc}{}^2}{4e_v} f_{yw}$$ Gross flexure yield at cope notch

$$\phi V_{gsw} = \phi_s 0.5 t_w d_{bc} f_{yw} \quad \text{if } V^* \le 0.75\phi V_{gfw}$$ Gross shear yield at cope notch

$$\phi V_{gsw} = \phi_s 0.5 t_w d_{bc} f_{yw} \left[2.2 - \left(\frac{16V^*}{\phi V_{gfw}} \right) \right] \quad \text{if } V^* > 0.75\phi V_{gfw}$$

Shear / flexure interaction at cope notch

$$\phi V_{bsw} = \phi_s \begin{bmatrix} 0.6 A_{gs} f_{yw} + A_{nt} f_{ub}; \\ 0.6 A_{ns} f_{ub} + A_{gt} f_{yw} \end{bmatrix}_{\max}$$ Block shear

H. Definition of Terms

$a_c = d - a + a_{e6} - d_i$	Edge distance to underside of cleat
$a_{elwi} = [a_{e5}; a_{e8}]_{\min}$ $\quad a_{el\sup i} = a_{e3}$ $\quad a_{elw} = [a_{e4}; a_{e8}]_{\min}$	Longitudinal beam axis edge distance
$a_{eti} = [a_{e6}; a_{e7}]_{\min}$	Transverse beam axis edge distance
$a_{e3} = b_{i\sup} - s_{g1}$	Bolt side edge distance
a_{e4}	Web edge x-distance
$a_{e5} = b_{iw} - s_{g2} - s_{g3}$	Web cleat edge x-distance
$a_{e6} = \dfrac{d_i - (n_{pw} - 1)s_{pw}}{2}$	Top bolt cope and cleat edge y-distance
$a_{e7} = s_{pw} - \dfrac{d_h}{2}$	Inter-bolt edge y-distance
$a_{e8} = s_{g3} - \dfrac{d_h}{2}$	Web inter-bolt edge x-distance
$a_{e9} = s_{g4} - \dfrac{d_h}{2}$	Support bolt edge x-distance, $n_a = 1$
$A_{gs} = [a_{e6} + (n_{pw} - 1)s_{pw}]t_w$	Web block gross shear area
$A_{gsi} = [a_{e6} + (n_{p\sup} - 1)s_{p\sup}]t_i$	Support leg block gross shear area
$A_{gt} = [a_{e3} + (n_c - 1)s_{g3}]t_w$	Web block gross tension area
$A_{gti} = a_{e3} t_i$	Support leg block gross tension area
$A_{gi} = d_i t_i$	Single leg gross area
$A_{ns} = A_{gs} - [(n_{pw} - 0.5)d_h]t_w$	Web block net shear area
$A_{nsi} = A_{gsi} - [(n_{p\sup} - 0.5)d_h]t_i$	Support leg block net shear area
$A_{nt} = [a_{e4} + s_{g3} - (n_c - 0.5)d_h]t_w$	Web block net tension area
$A_{nti} = (a_{e3} - d_h/2)t_i$	Support leg block net tension area
$A_{n\sup i} = (d_i - n_{p\sup} d_h)t_i$	Single support leg net area
$A_{nwi} = (d_i - n_{pw} d_h)t_i$	Single web leg net area
$A_w = d_{bc} t_w$	Web shear area HR sections
$A_w = (d_{bc} - t_f)t_w$	Web shear area Welded sections
$d_{bc} = d - a + a_{e6}$	SWC coped section depth
$d_{bc} = d - a + a_{e6} - a_{cb}$ if $d \le d_{\sup}$	DWC coped section depth
$d_{bc} = d_{\sup} - a + a_{e6} - a_{cb}$ if $d > d_{\sup}$	DWC coped section depth
$d_h = d_f + 2$ for $d_f \le 24$	Hole diameter
$e = s_{g2}$	Eccentricity, $n_c = 1$

$e = s_{g2} + \dfrac{s_{g3}}{2}$	Eccentricity, $n_c = 2$
$e_{\sup} = s_{g1}$	Support cleat eccentricity
$e_v = L_c + s_{g2} - a_{e4}$	Cope eccentricity
$i_{bpw} = \dfrac{n_{pw} s_{pw}^2 \left(n_{pw}^2 - 1\right)}{12}$	Bolt group polar moment of inertia, $n_c = 1$
$i_{bpw} = \dfrac{n_{pw}}{6}\left[s_{pw}^2\left(n_{pw}^2 - 1\right) + 3 s_{g3}{}^2\right]$	Bolt group polar moment of inertia, $n_c = 2$
$i_{bp\sup} = \dfrac{n_{p\sup} s_{p\sup}^2 \left(n_{p\sup}^2 - 1\right)}{12}$	Bolt group polar moment of inertia
n_a	Number of angle cleats
n_c	Number of web bolt columns
$n_{p\sup}$	Number of support bolt rows
n_{pw}	Number of web bolt rows
$s_{pg} = \dfrac{s_{g3}}{\left(n_{pw} - 1\right) s_{pw}}$	Double bolt column pitch & gauge factor
$Z_{ec} = \left[S_{cope} ; 15 Z_{cope}\right]_{\min}$	SWC coped section effective modulus.
$Z_{bw} = \dfrac{n_{pw}}{\sqrt{1 + \left[\dfrac{6e}{s_{pw}\left(n_{pw} + 1\right)}\right]^2}}$	Web bolt factor, $n_c = 1$
$Z_{bw} = \dfrac{2 n_{pw}}{\sqrt{\left[1 + Z_1\right]^2 + \left[\dfrac{Z_1}{s_{pg}}\right]^2}}$	Web bolt factor, $n_c = 2$
$Z_{b\sup} = \dfrac{n_{p\sup}}{\sqrt{1 + \left[\dfrac{6 e_{\sup}}{s_{p\sup}\left(n_{p\sup} + 1\right)}\right]^2}}$	Support bolt factor, $n_a = 1$
$Z_{el\sup} = \dfrac{s_{p\sup}\left(n_{p\sup} + 1\right)}{6 e_{\sup}}$	Support bolt tearing longitudinal capacity factor, $n_a = 1$
$Z_{el\sup} = 1$	Support bolt tearing longitudinal capacity factor, $n_a = 2$
$Z_{elw} = \dfrac{s_{pw}\left(n_{pw} + 1\right)}{6e}$	Web bolt tearing longitudinal capacity factor, $n_c = 1$
$Z_{elw} = \dfrac{i_{bpw}}{e\left(n_{pw} - 1\right) n_{pw} s_{pw}}$	Web bolt tearing longitudinal capacity factor, $n_c = 2$
$Z_{etw} = \dfrac{1}{1 + \dfrac{n_{pw} e s_{g3}}{i_{bpw}}}$	Web bolt tearing transverse capacity factor, $n_c = 2$
$Z_1 = 1$	Sub-factor for Z_{bw}, $n_c = 1$
$Z_1 = \dfrac{\dfrac{2e}{s_{g3}}\left[\left(n_{pw} - 1\right)\dfrac{s_{pw}}{s_{g3}}\right]^2}{1 + \dfrac{n_{pw} + 1}{3\left(n_{pw} - 1\right)}}$	Sub-factor for Z_{bw}, $n_c = 2$

I. AC Angle Cleat Drawings

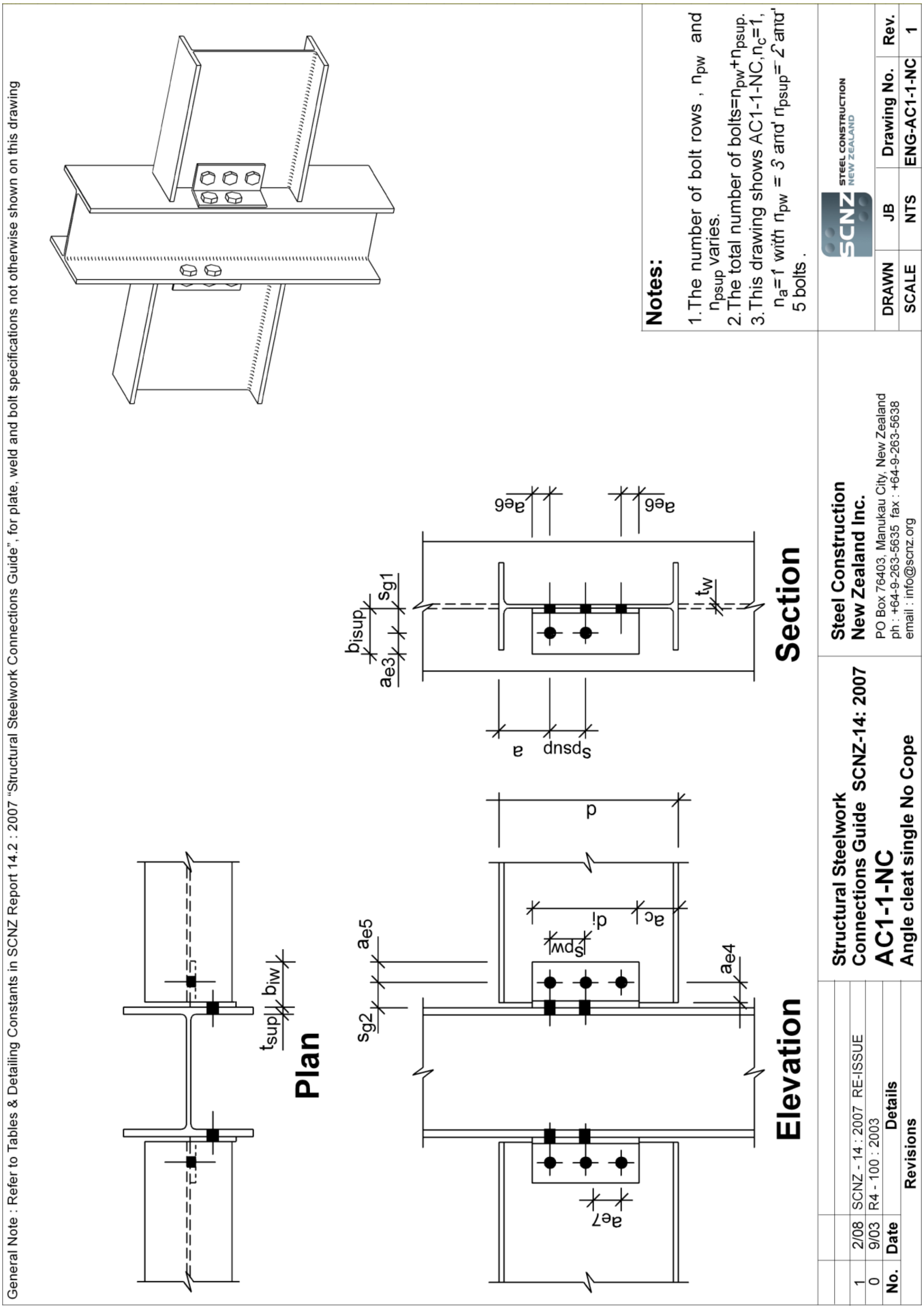


Figure 7 AC1-1-NC drawing

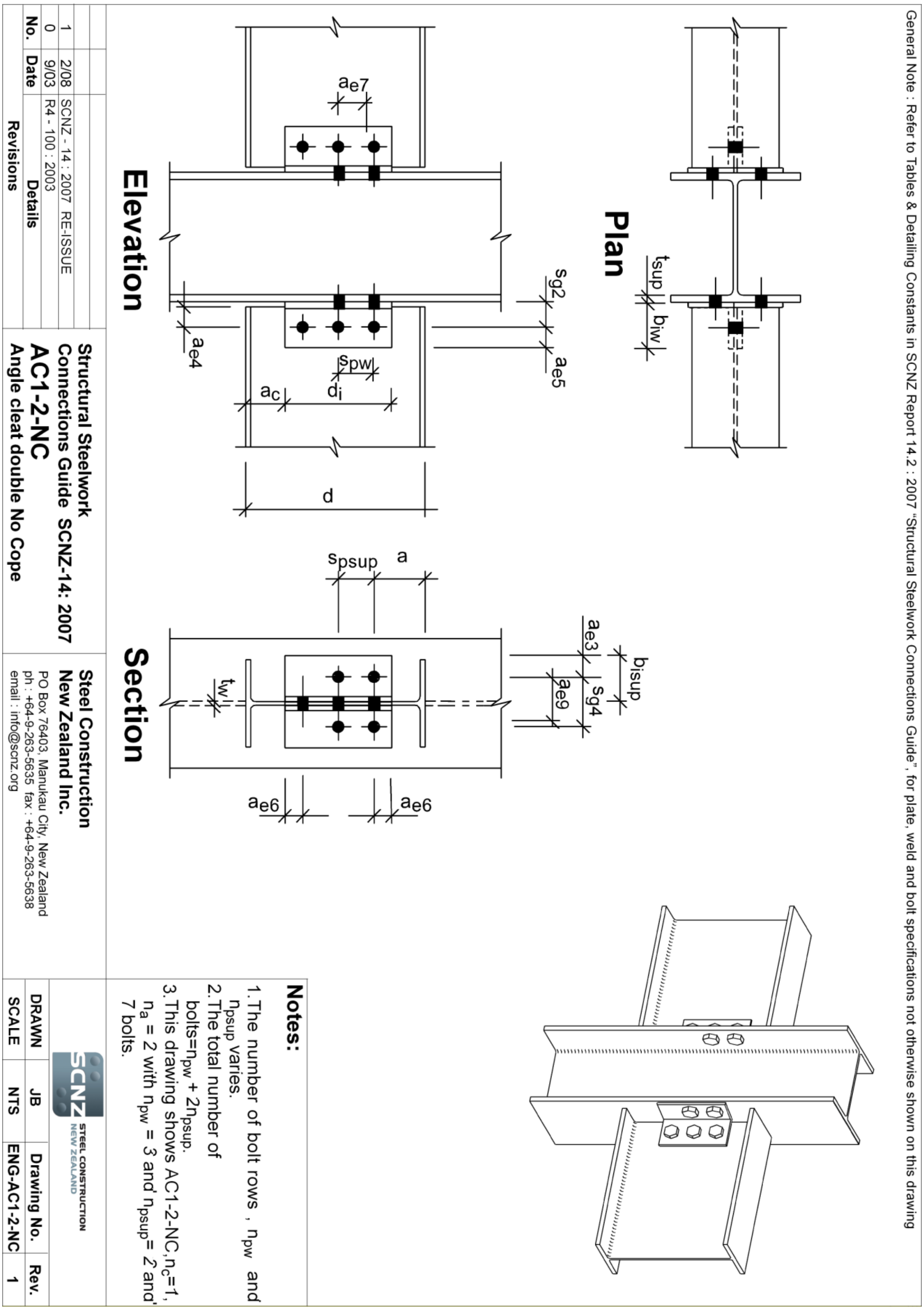


Figure 8 AC1-2-NC drawing

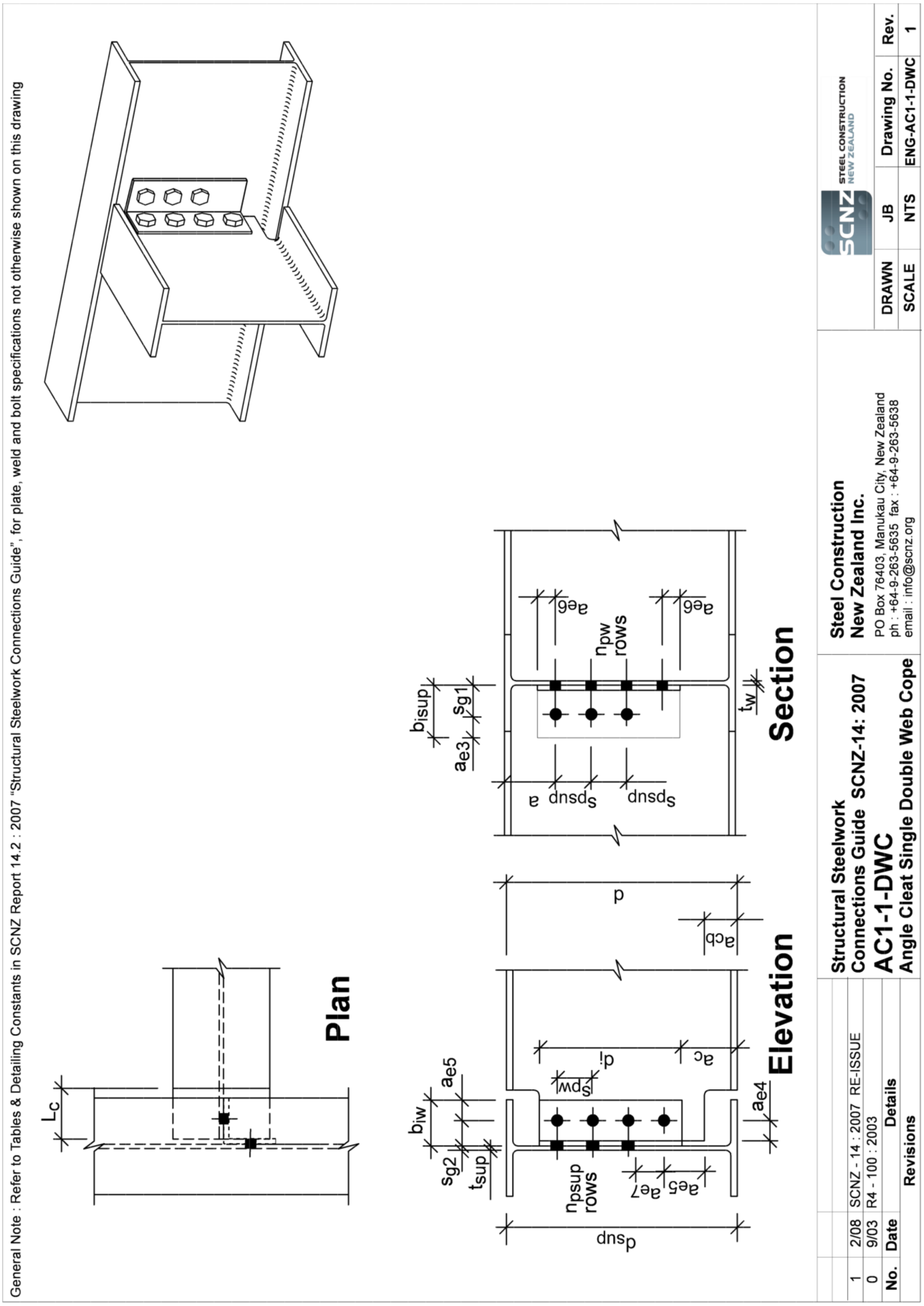


Figure 9 AC1-1-DWC drawing

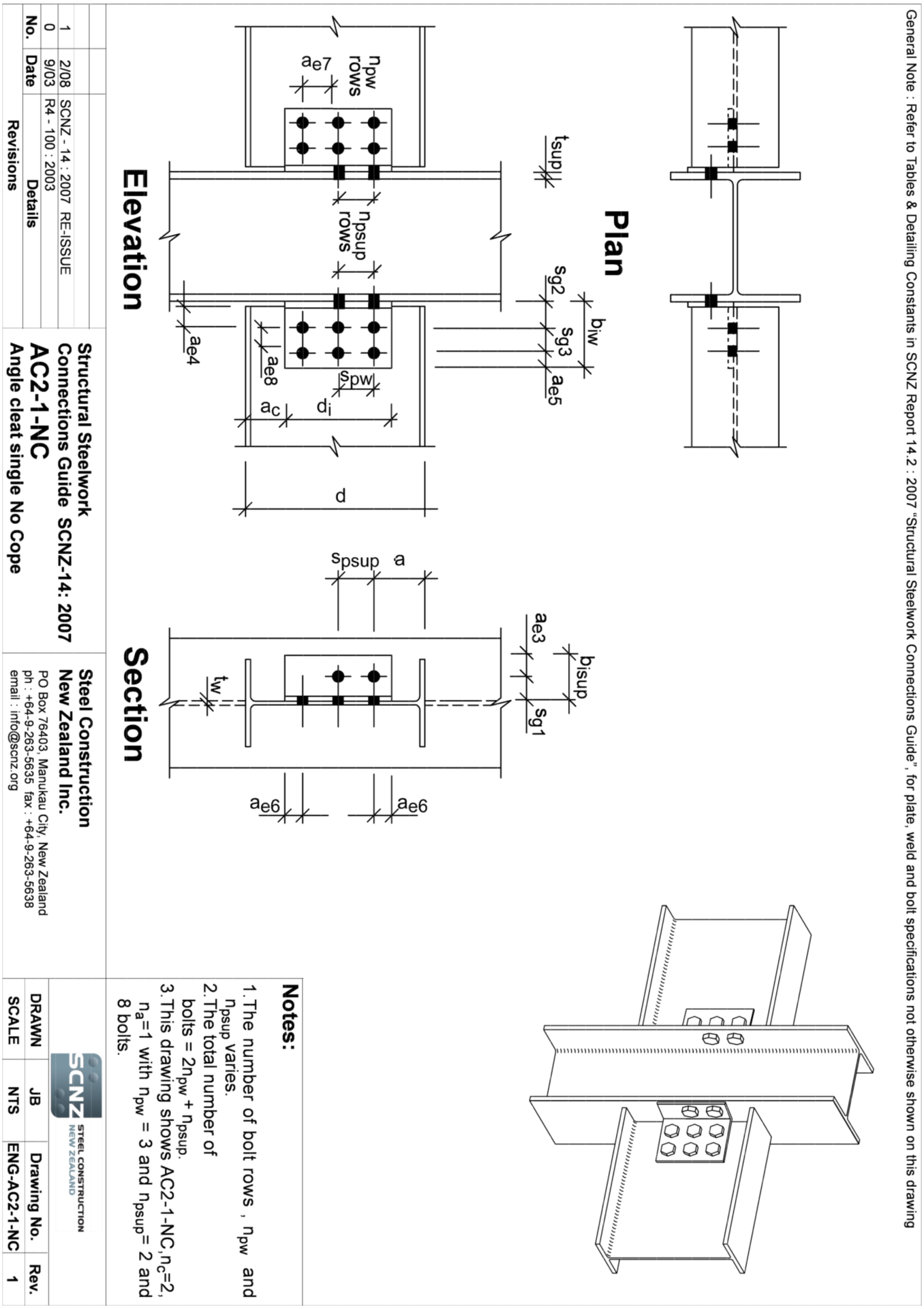


Figure 10 AC2-1-NC drawing

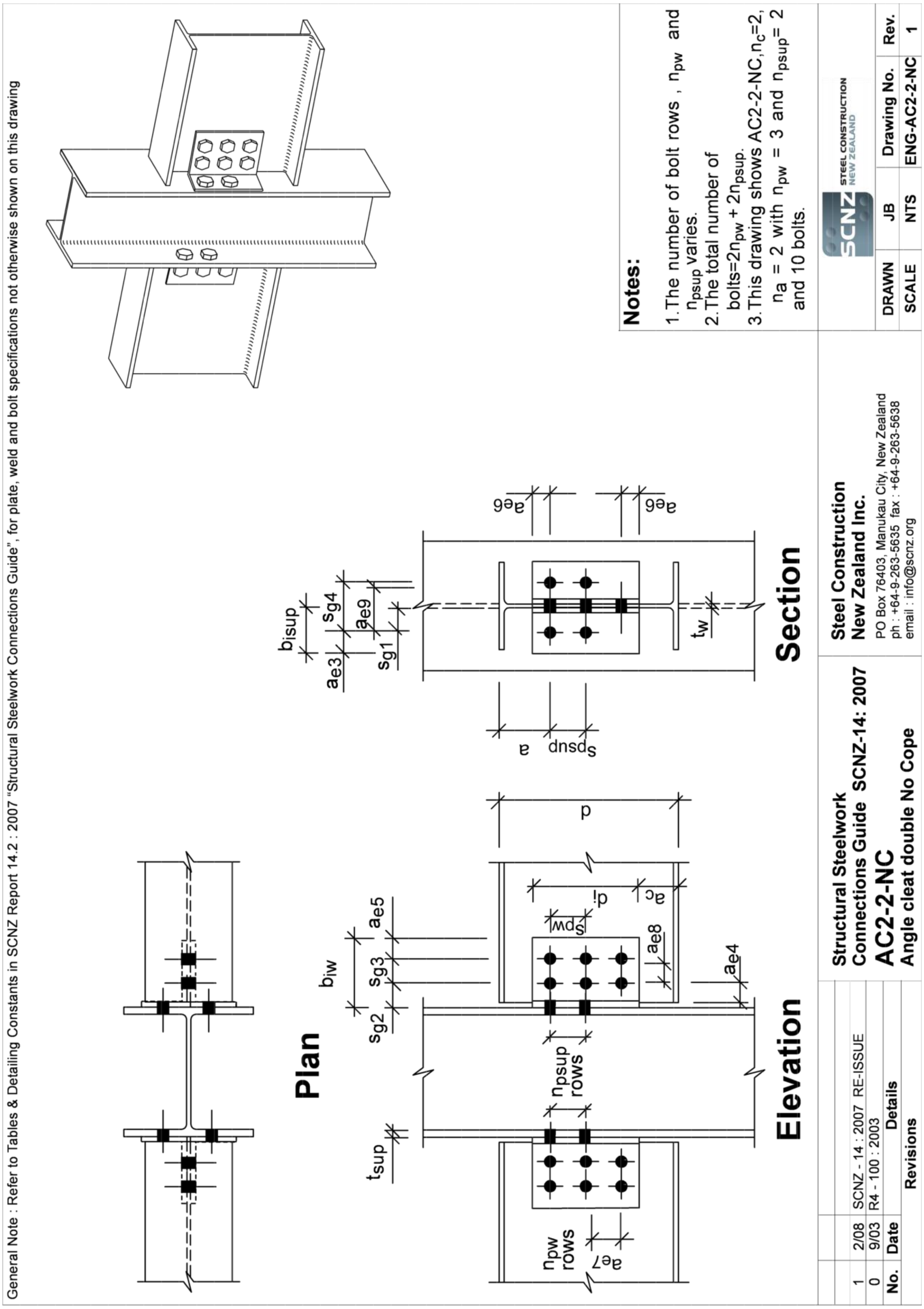


Figure 11 AC2-2-NC drawing

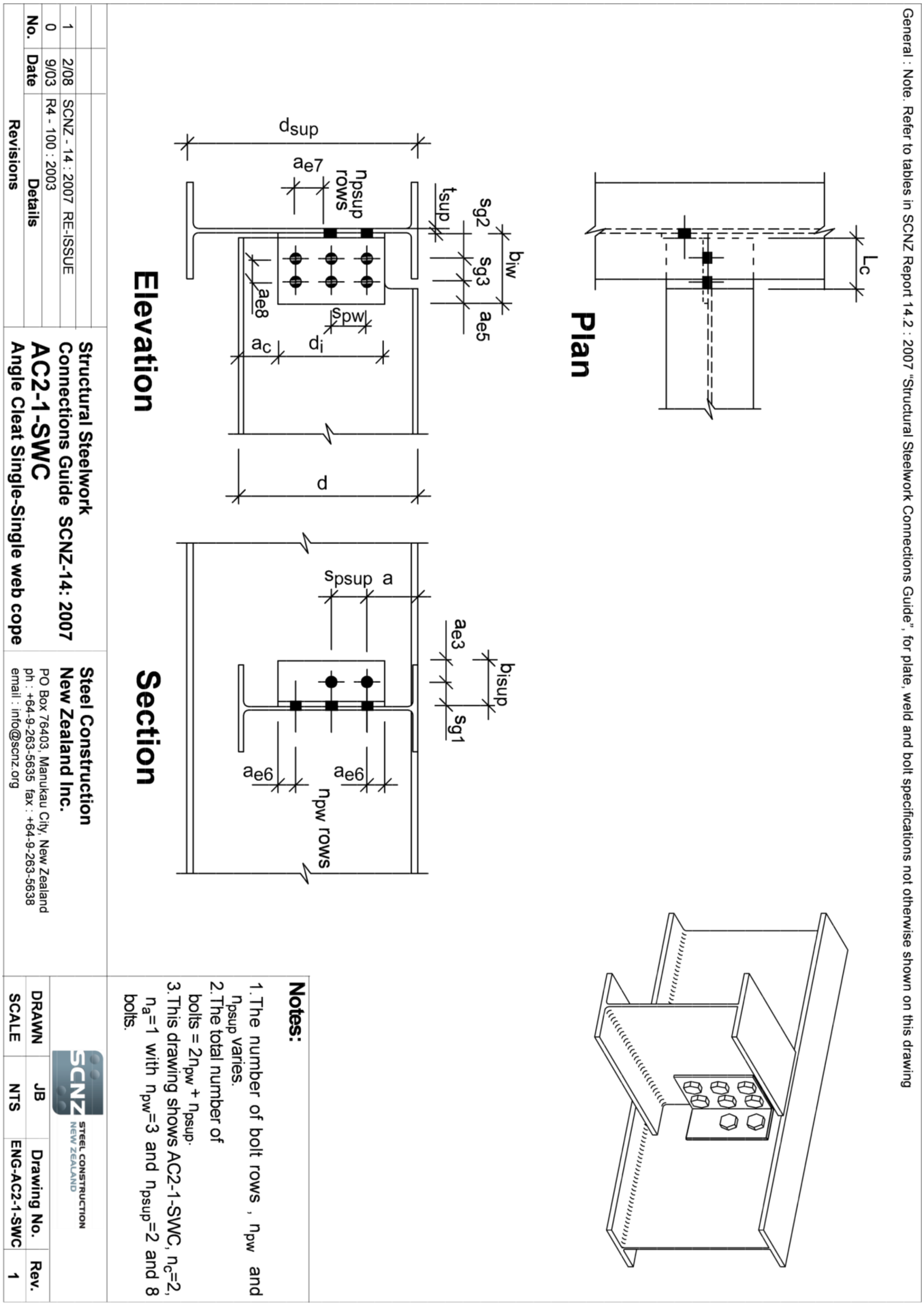


Figure 12 AC2-1-SWC drawing

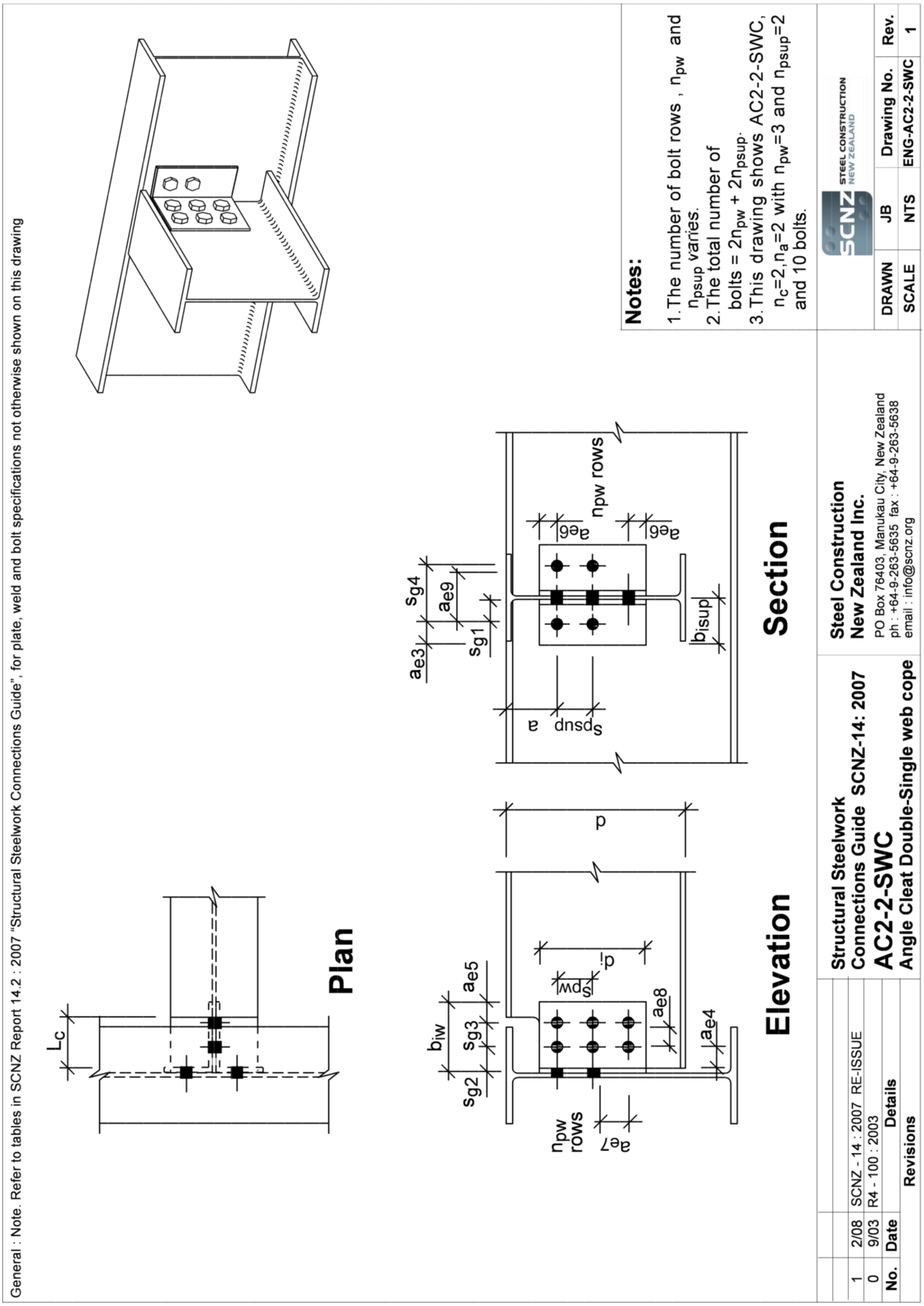


Figure 13 AC2-2-SWC drawing

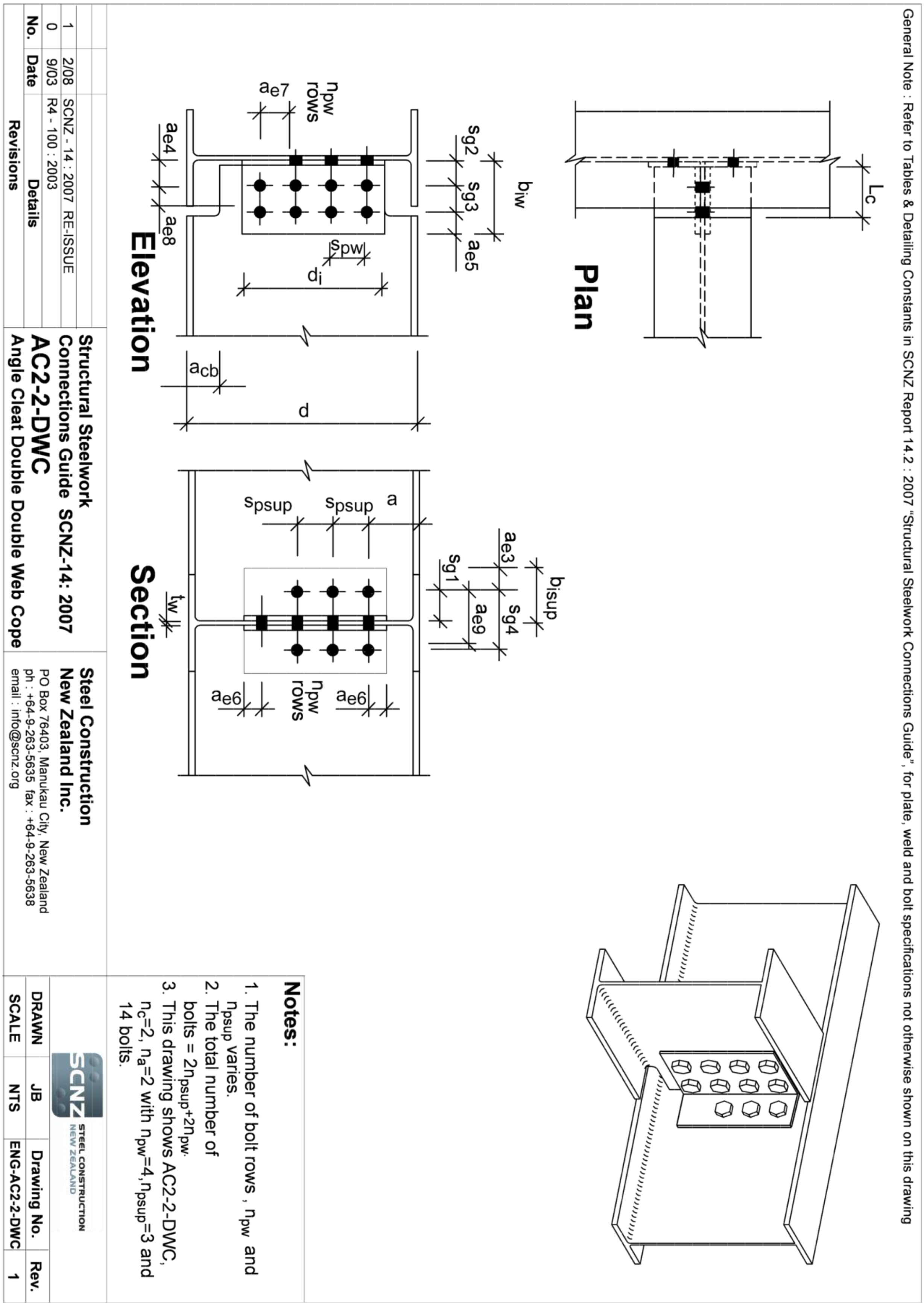


Figure 14 AC2-2-DWC drawing

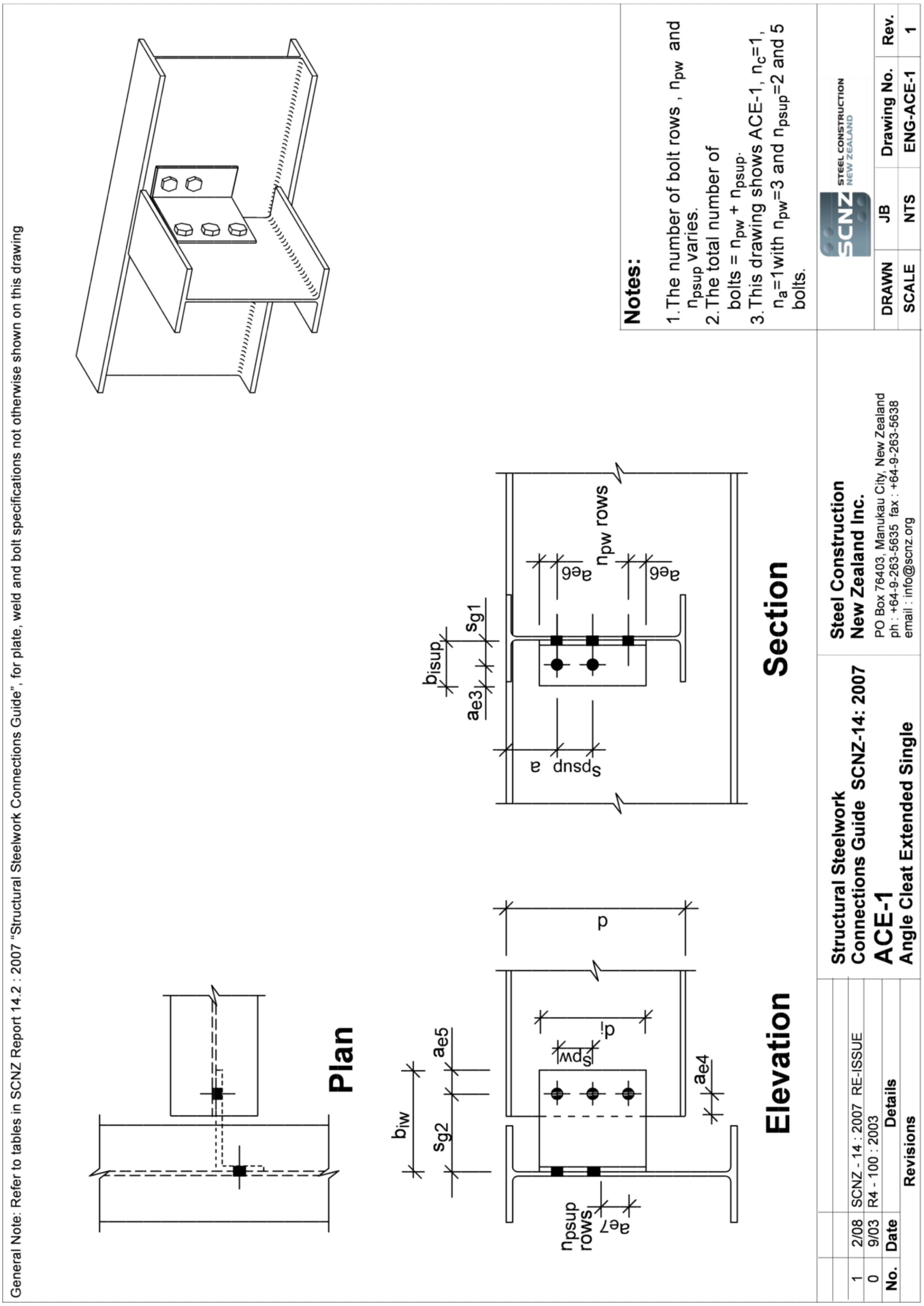


Figure 15 ACE-1 drawing

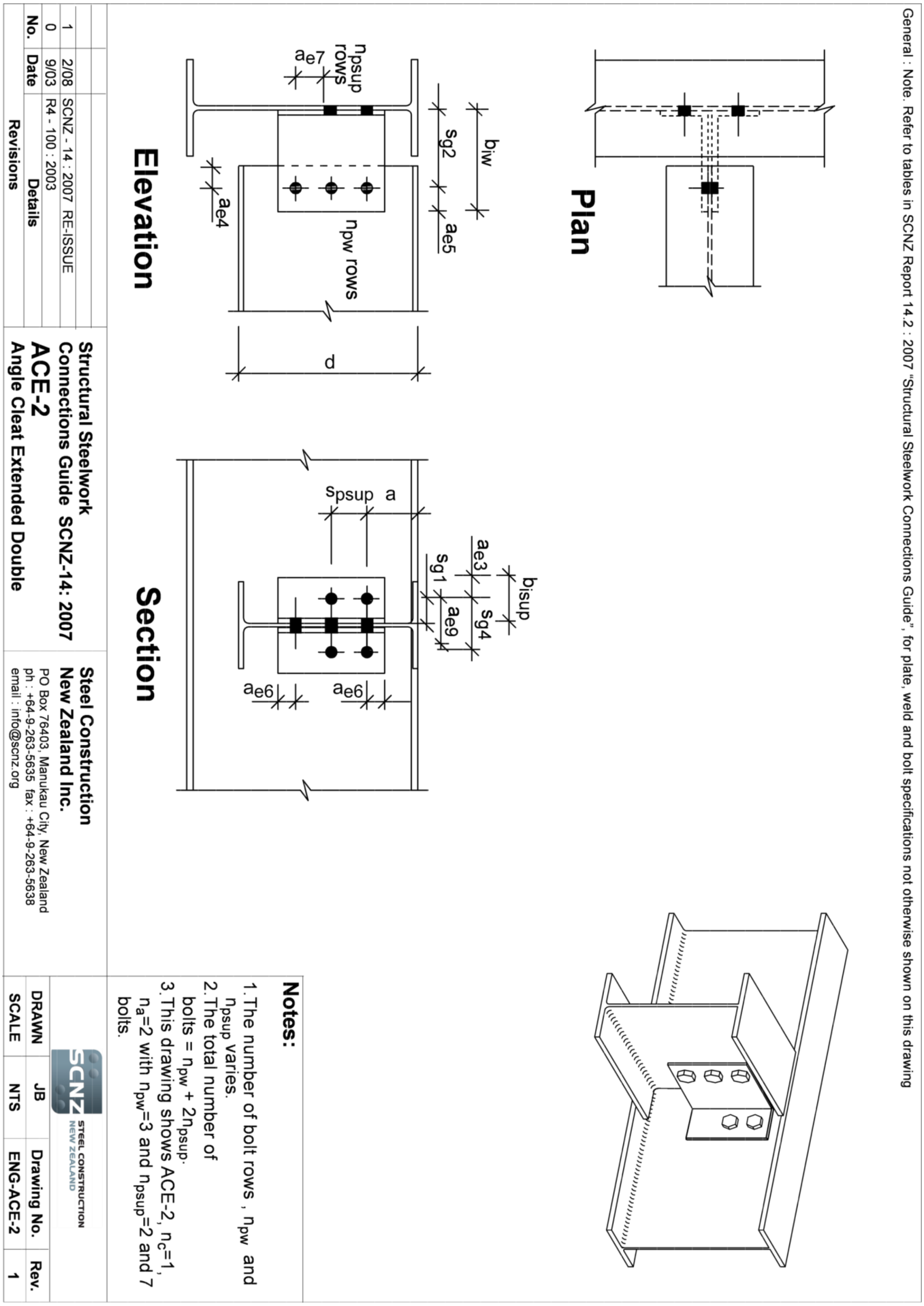


Figure 16 ACE-2 drawing

VIII. FE: Flexible End Plates

A. Design Objectives

Possess design capacity to satisfy gravity ultimate limit state loads

Provide twist restraint to the supported and supporting beams about their respective longitudinal axes, consistent with the restraint provisions of NZ Steel Structures Standard NZS3404:1997 and HERA report R4-92.

Have sufficient rotation ductility to accommodate gravity load and seismic drift induced rotations of 0.030 radians, without collapse.

Have sufficient ductility to accommodate thermal strains induced by extreme fire events without collapse.

B. Design Features

Typical limiting conditions are : Shear of the cleat or bolts; Block shear of a block extending 20mm into the web and as deep as the end plate; Shear / flexural yield of coped section.

Welds of cleats to the beams have design capacity greater than that the resultant action due to development of over-strength of 1.2 times the flexural yield of the end plate under tensile load and direct shear. This is to ensure ductile performance of the connection under seismic and fire conditions.

The beam web adjacent to the top of the cleat have sufficient capacity to resist combined shear and local longitudinal tension, using a Von Mises based stress criterion.

To prevent transverse tensile tearing in the web below the bottom edge of the end plate, block transverse shear and tensile yield capacity is assessed based upon a web block 20mm wide and as deep as the end plate. Compression yield strength is assessed to develop along the top edge of the web block shear element as long as two loacl web buckling criteria are satisfied. The first of these is that the top flange is laterally restrained at that location by a concrete slab or other means. The second criterion is that the clear web depth to web thickness ratio between the top of the cleat and the underside of the flange, is less than or equal to 17.5.

The end plate is maintained as flexible by limiting the thickness of the endplate as a function of the bolt gauge.

Where beams of different section size and loading are supported either side of the same plate element of a section, ie. the web of a beam or column, care should be taken to ensure that both flexible end plates are compatible. Sufficient holes or clear space needs to be available to allow the connecting bolts to be installed. Where necessary provide additional holes to one of the end plates to match the other and allow opposing beam support bolts through.

C. Design Procedure

1. Governing criteria

The minimum bolt group, end plate, section web and supporting section design strength limits shall exceed the applied load.

End plate flexibility shall be maintained by ensuring an adequate ratio between the end plate thickness and the supporting bolt group lateral spacing.

The weld shall be able to resist resultant shear and end plate pull-out tensile actions.

Ultimate tensile strength shall not be exceeded by gravity, seismic and fire design actions, based upon a Von Mises criterion, in the web adjacent to the weld.

Connection shear strength shall not exceed the shear and tensile yield capacity of a 20mm wide by end plate depth web shear block.

For beam to column connections, an additional seismic design requirement is that the end gap clearances must be sufficient to accommodate a 0.030 radian relative rotation.

End plate depth and bolt edge distances must be within limits.

2. Design Actions

Ultimate design shear
End plate pull-out flexure over-strength tension
Pull-out tensile stress in web

3. Connection Design Strength Limits

Connection design shear capacity

4. Bolt Group Design Strength Limits

Transverse shear

5. Flexible End Plate Design Strength Limits

Bolt hole 1st transverse bearing
Bolt hole 1st transverse tearing
Plate gross shear yield
Plate block shear

6. Support Design Strength Limits

Bolt hole 1st bearing
Bolt hole 1st tearing

7. Section Design Strength Limits

a) General

Uncoped section shear
Coped section shear
Web shear / tension adjacent to top of cleat
Block shear in web

b) Single Web Cope

Gross flexure yield at cope notch
Gross shear yield at cope notch
Shear / flexure interaction at cope notch

c) Double Web Cope

Gross flexure yield at cope notch
Gross shear yield at cope notch
Shear / flexure interaction at cope notch

8. Weld Design Strength Limits

Resultant weld shear
Weld direct shear

D. Design Formulae

1. Governing Criteria

$V^* \leq \phi V_{con}$ Shear

$0.45d \leq d_i \leq d - t_f + a_{e1} - a$ End plate depth limits

$11 \leq \frac{s_g}{t_i} \leq 14$ End plate flexibility

$\frac{a_c}{t_i} \leq 33$ End gap

$a_{e1} \geq 1.75 d_f$ Manual flame cut or crop top edge

$a_{e3} \geq 1.5 d_f$ HR or CNC flame cut side edge

$d_i - a_{e1} - (n_p - 1)s_p \geq 1.75 d_f$ Manual flame cut or crop bottom edge

2. Design Actions

V^* Design shear force

$N^*_{gfi} = \frac{2 d_i t_i^2 (1.2 f_{yi})}{s_g - d_h}$ End plate pull-out over-strength tension

$\sigma^*_{wx} = \frac{N^*_{gfi}}{d_i t_w}$ Pull-out tensile stress in web

3. Connection Design Strength Limits

$\phi V_{con} = \lfloor \phi V_b ; \phi V_i ; \phi V_{\sup} ; \phi V_{wb} ; \phi V_{weld} \rfloor_{\min}$ Connection design shear capacity

4. Bolt Group Design Strength Limits

$\phi V_b = 2 n_p \phi_b V_{fn}$ Transverse shear

5. Flexible End Plate Design Strength Limits

$$\phi V_i = \left[\phi V_{bi} ; \phi V_{tti} ; \phi V_{gsi} ; \phi V_{bsi} \right]_{\min}$$

$\phi V_{bi} = 2 n_p \phi_s 3.2 t_i d_f f_{ui}$ Bolt hole 1st transverse bearing

$\phi V_{tti} = 2 n_p \phi_s a_{eyi} t_i f_{ui}$ Bolt hole 1st transverse tearing.

$\phi V_{gsi} = \phi_s 0.5 t_i 2 d_i f_{yi}$ Gross shear yield.

$\phi V_{bsi} = 2\phi_s \begin{bmatrix} 0.6 A_{gs} f_{yi} + A_{nt} f_{ui} ; \\ 0.6 A_{ns} f_{ui} + A_{gt} f_{yi} \end{bmatrix}_{\max}$ Block shear of end plate

6. Support Design Strength Limits

$$\phi V_{\sup} = \left[\phi V_{b\sup} ; \phi V_{t\sup} \right]_{\min}$$

$\phi V_{b\sup} = 2 n_p \phi_s 3.2 t_s d_f f_{us}$ Bolt hole 1st bearing

$\phi V_{t\sup} = 2 n_p \phi_s a_{e2} t_s f_{us}$ Bolt hole 1st tearing

7. Section Design Strength Limits

a) General

$\phi V_{wb} = \lfloor \phi V_{wc}; \phi V_{gsw}; \phi V_{bsw} \rfloor_{\min}$ Uncoped section shear

$\phi V_{wb} = \lfloor \phi V_{wc}; \phi V_{gfw}; \phi V_{gsw}; \phi V_{bsw} \rfloor_{\min}$ Single coped section shear

$\phi V_{wb} = \lfloor \phi V_{wc}; \phi V_{gfw}; \phi V_{gsw} \rfloor_{\min}$ Double coped section shear

$\phi V_{wc} = d_i t_w \sqrt{\dfrac{0.8 f_u^2 - \sigma_{wx}^{*\,2}}{3}}$ Web shear / tension at top of cleat

$\phi V_{bsw} = \min \begin{bmatrix} \phi_s (0.6 t_w d_i f_{yw} + n 20 t_w f_{yw}) \\ \phi_s 0.6 t_w d_i f_u \end{bmatrix}$ Block shear in web

b) No Cope

$\phi V_{gsw} = \phi_s 0.6 A_w f_{yw}$ if $V^* \le 0.75 \phi V_{gfw}$ Gross shear yield at cope notch

c) Single Web Cope

$\phi V_{gfw} = \phi_s \dfrac{Z_{ec}}{e_v} f_{yw}$ Gross flexure yield at cope notch

$\phi V_{gsw} = \phi_s 0.6 A_w f_{yw}$ if $V^* \le 0.75 \phi V_{gfw}$ Gross shear yield at cope notch

$\phi V_{gsw} = \phi_s 0.6 A_w f_{yw} \left[2.2 - \left(\dfrac{16 V^*}{\phi V_{gfw}} \right) \right]$ if $V^* > 0.75 \phi V_{gfw}$

Shear / flexure interaction at cope notch

d) Double Web Cope

$\phi V_{gfw} = \phi_s \dfrac{t_w d_{bc}^{\,2}}{4 e_v} f_{yw}$ Gross flexure yield at cope notch

$\phi V_{gsw} = \phi_s 0.5 t_w d_{bc} f_{yw}$ if $V^* \le 0.75 \phi V_{gfw}$ Gross shear yield at cope notch

$\phi V_{gsw} = \phi_s 0.5 t_w d_{bc} f_{yw} \left[2.2 - \left(\dfrac{16 V^*}{\phi V_{gfw}} \right) \right]$ if $V^* > 0.75 \phi V_{gfw}$

Shear / flexure interaction at cope notch

8. Weld Design Strength Limits

$\phi V_{weld} = \sqrt{\phi V_{ww}^{\,2} - N_{gfi}^{*\,2}}$ Resultant weld shear

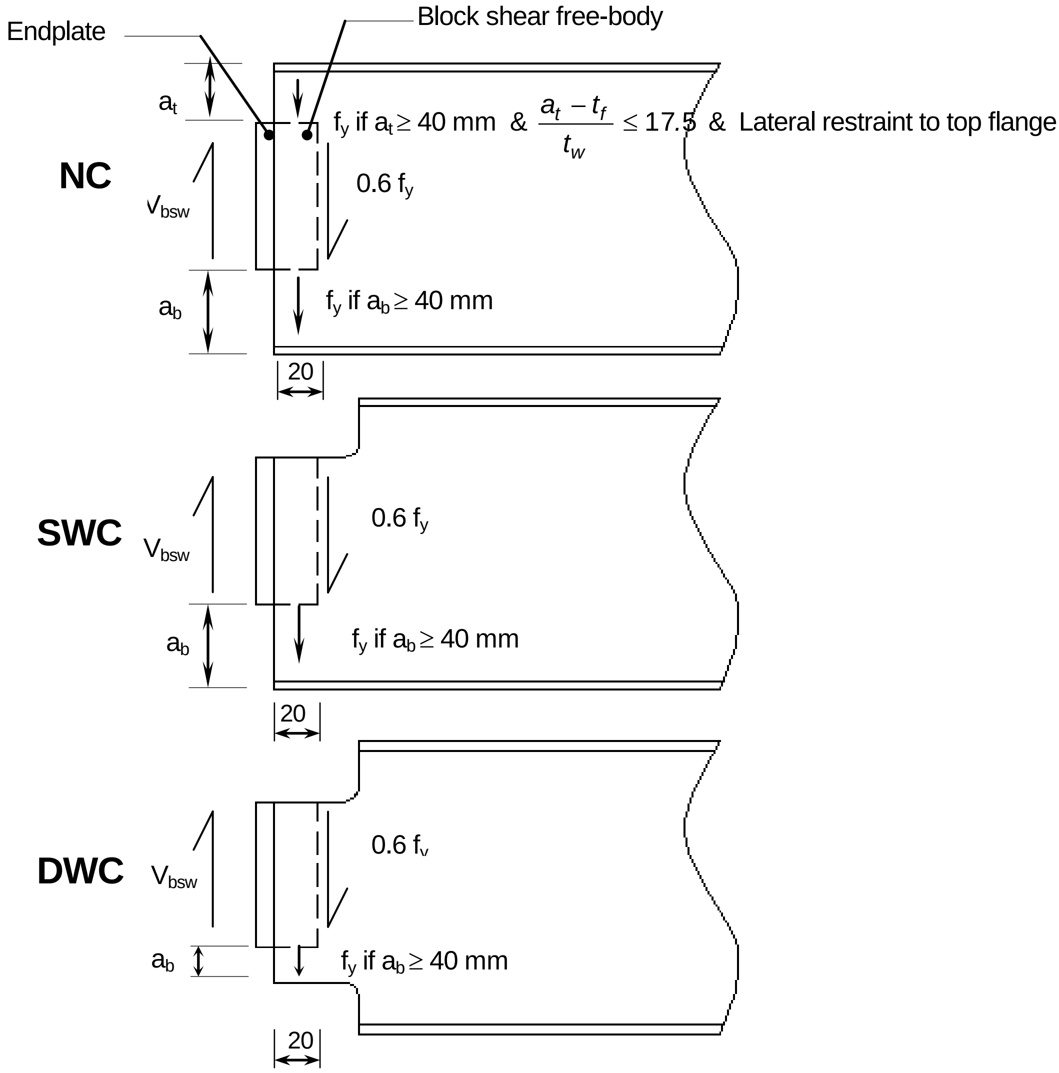
$\phi V_{ww} = \phi_w 0.6 f_{uw} \dfrac{t_{ww}}{\sqrt{2}} 2 d_i$ Weld direct shear

$\phi_w = 0.8$ SP fillet weld

9. Definition of Terms

$a_b = d - d_i - a + a_{e1}$	NC, SWC distance to underside of cleat
$a_b = d_{bc} - d_i - a_{e4} + a_{e1}$	DWC distance to underside of cleat
$a_c = d - a + a_{e1} - d_i$	End gap
$a_{eyi} = \left[a_{e1}, a_{e2}\right]_{\min}$	Edge distance
a_{e1}	Top bolt edge y-distance
$a_{e2} = s_p - \frac{d_h}{2}$	Inter-bolt edge distance
$a_{e3} = \frac{b_i - s_g}{2}$	Bolt side edge distance
a_{e4}	Top bolt cope edge y-distance
$a_t = a - a_{e1}$	NC edge distance to top of cleat
$a_t = a_{e4} - a_{e1}$	DWC,SWC edge distance to top of cleat
$A_{gs} = \left[a_{e1} + \left(n_p - 1\right)s_p\right]t_i$	End plate block gross shear area
$A_{gt} = a_{e3}t_i$	End plate block gross tension area
$A_{ns} = A_{gs} - \left[\left(n_p - 0.5\right)d_h\right]t_i$	End plate block net shear area
$A_{nt} = \left(a_{e3} - d_h/2\right)t_i$	End plate block net tension area
$A_w = d_{bc}t_w$	Web shear area HR sections
$A_w = \left(d_{bc} - t_f\right)t_w$	Web shear area Welded sections
$d_{bc} = d - a + a_{e4}$	SWC coped section depth
$d_{bc} = d - a + a_{e4} - a_{cb}$ if $d \le d_{\sup}$	DWC coped section depth
$d_{bc} = d_{\sup} - a + a_{e4} - a_{cb}$ if $d > d_{\sup}$	DWC coped section depth
$d_h = d_f + 2$ for $d_f \le 24$	Hole diameter
$e_v = L_c + t_i$	Cope eccentricity
$n = n_b + n_i$	Block shear
$n_b = 1$ if $a_b \ge 40$; $n_b = 0$ if $a_b < 40$	Block tension bottom of cleat
$n_t = 1$ if $a_t \ge 40$ and $\frac{a_t - t_f}{t_w} \le 17.5$; $n_t = 0$ if $a_t < 40$	Block compression buckling TOC
$Z_{ec} = \left[S_{cope}; 1.5Z_{cope}\right]_{\min}$	Coped section effective modulus.

10. Shear Block Mechanisms



Block shear mechanisms in beam webs of flexible end plate connections
Note: $V_{bsw} \leq 0.6\, f_u\, d_i\, t_w$

E. FE Flexible End Plate Drawings

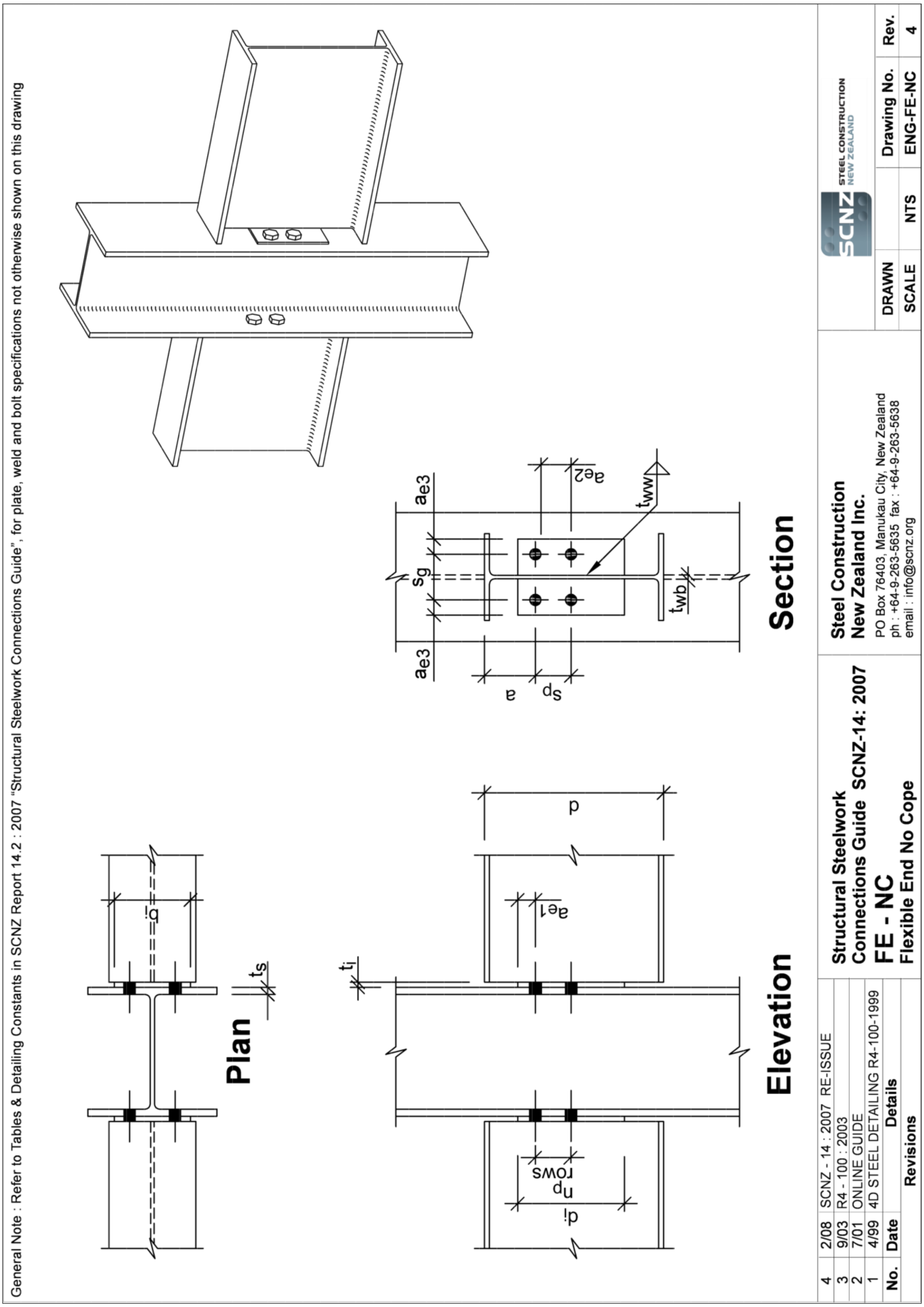


Figure 17 FE-NC drawing

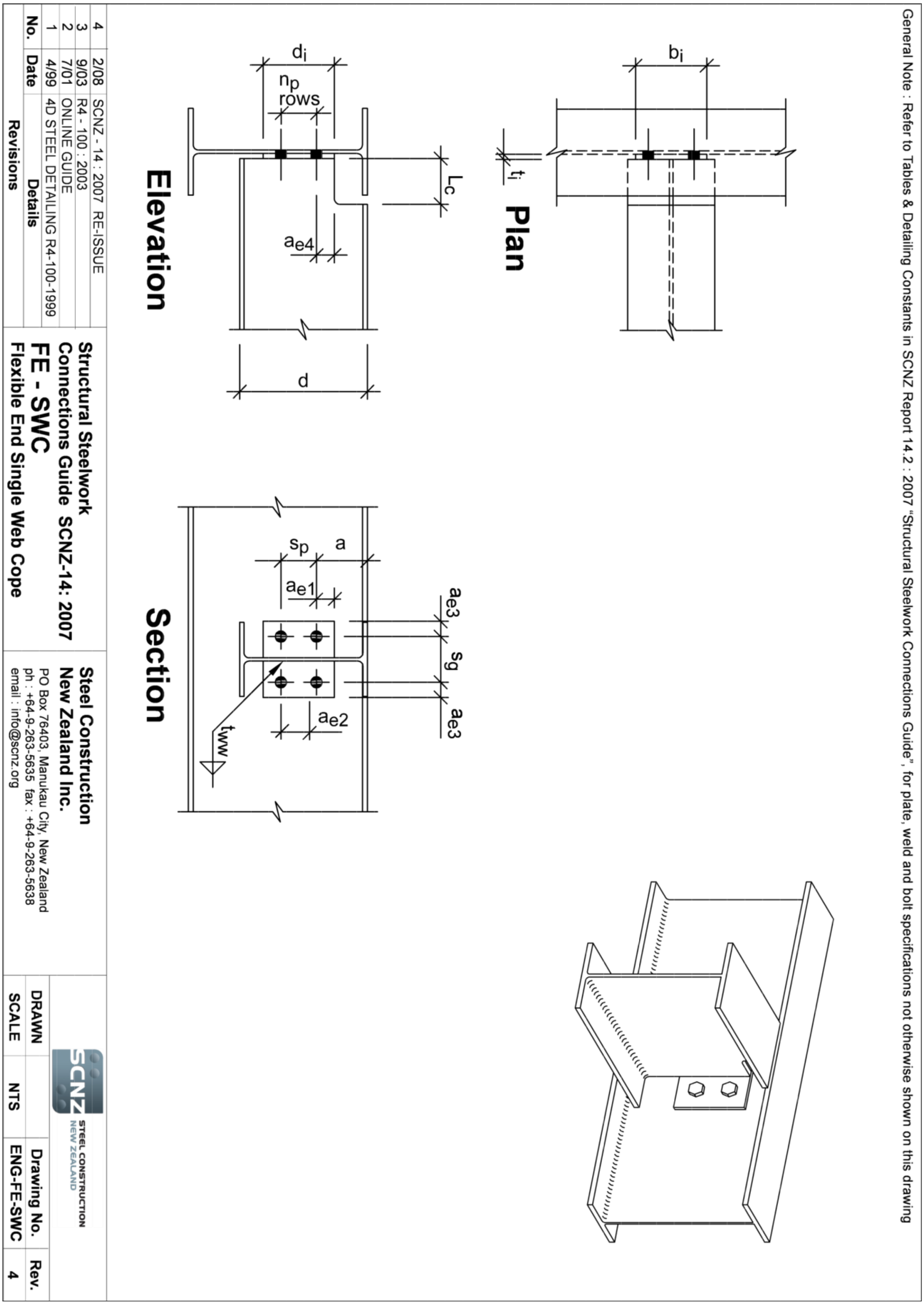


Figure 18 FE-SWC drawing

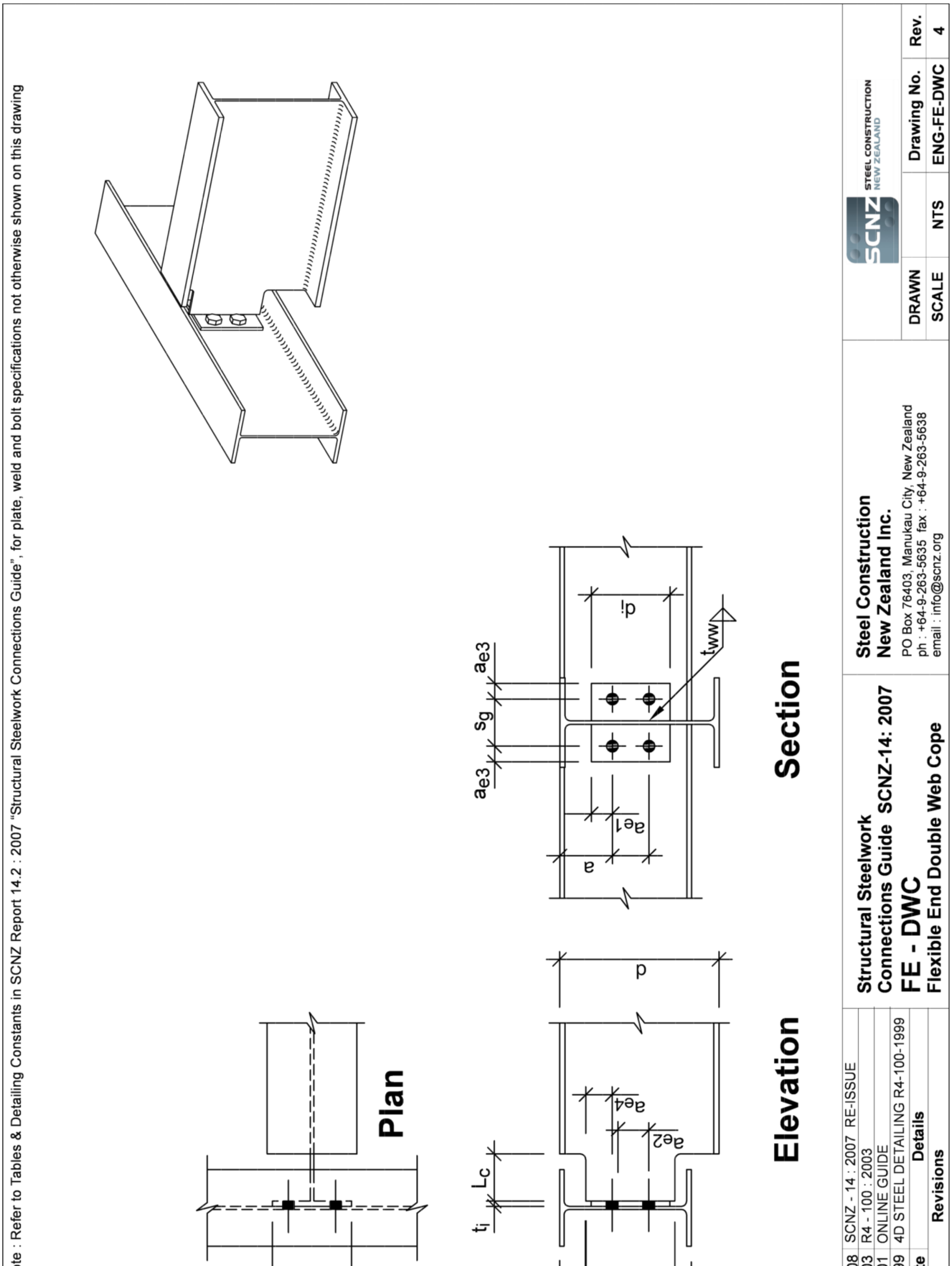


Figure 19 FE-DWC drawing

IX. WM: Welded Moment

A. Design Objectives

A beam to column face welded connection possessing design capacity sufficient to satisfy the demands of primary members in frames of varying levels of seismic ductility category,. These include ductile frames with $3 < \mu \leq 6.0$, limited ductile frames with $1.25 < \mu \leq 3.0$, and elastic frames with $\mu = 1.0$. Seismic flexural over-strength actions are to be combined with shear loads equal to 60% design shear yield capacity of the section. The connection is to maintain integrity under fire conditions.

B. Design Features

Typical limiting conditions are : Flexure or shear of the section.

The welds shall be sized to develop the design capacity of the web and flanges so as to enhance ductile behaviour under fire conditions. It is considered that the welds should be able to develop the section capacity of the section to cope with shrinkage stresses that may develop during fire cooling phases.

The welds of the flanges may be either complete penetration butt welds or symmetrical fillet welds placed either side of the flanges. Complete penetration butt welds to the flanges are suitable for all members in elastic, limited ductile and ductile frames, without specific design. However these welds must not be ground flush, but should maintain minimum butt weld reinforcement in accordance with AS / NZS 1554.1.

Where fillet welds are specified these are designed to develop the corresponding over-strength design capacity of the flange of the section acting as a primary member in a frame of the given seismic ductility demand category. Over-strength factors ϕ_{oms} replace ϕ_s for primary members of ductile and limited ductile frames, as per Table 12.2.8(1) NZS3404:1997.

In accordance with NZS 3404:1997, the flange over-strength calculation formulae listed assumes use of AS/NZS 3679.1 G300 sections and 300MOD welded sections with a supplier's material variation factor, $\phi_{om} = \dfrac{\sigma_{y97.5\%}}{\sigma_{y2.5\%}} \leq 1.20$. For steel sections of other grades or sourced from suppliers with higher material variation factors, refer to Table 12.2.8.1(1) NZS3404:1997 for the appropriate over-strength factors and adjust weld sizes as necessary.

Welds to the web are designed to develop the design tension capacity of the section web.

Compression and tension stiffeners and flange doubler plates to the supporting column are not specified but will often be required in order to develop the design reactions from the connection . Refer to section 12.9.5.3 NZS3404:1997 for design requirements.

Sections must satisfy the material and section geometry requirements of NZS3404:1997 Section 12.4 and 12.5, appropriate to the seismic ductility demand category of the frame.

C. Design Procedure

1. Governing Criteria

Connection moment and shear design capacity shall be greater than design actions.

Flange fillet welds shall develop the flange over-strength tension capacity for primary members of limited ductile and ductile frames with ductility demand of $\mu > 1.25$, and the flange design capacity for all members of elastic frames with frame ductility of $\mu \leq 1.25$.

Web welds shall develop design tension yield capacity of the section web for primary members of limited ductile and ductile frames with ductility demand of $\mu > 1.25$, and the web design capacity for all members of elastic frames with frame ductility of $\mu \leq 1.25$.

2. Design Actions

Factored design shear
Factored design moment
Flange tension yield and over-strength
Web tension yield

3. Connection Design Strength Limits

Connection design shear capacity
Connection design moment capacity

4. Web Shear Strength Limits

Shear yield with maximum flexure.

5. Flange Fillet Weld Design Strength Limits

Flange tension

6. Web Weld Design Strength Limits

Weld tension.
Weld shear.

D. Design Formulae

1. Governing Criteria

$V^* \leq \phi V_{con}$	Shear
$M^* \leq \phi M_{con}$	Moment
$N_{ft}^* \leq \phi N_{wf}$	Flange weld
$N_{ww}^* \leq \phi N_{ww}$	Web weld

2. Design Actions

V^*	Factored design shear
M^*	Factored design moment
$N_{ft}^* = 1.25 b_f t_f f_{yf}$	Flange tension over-strength: ductile
$N_{ft}^* = 1.15 b_f t_f f_{yf}$	Flange tension over-strength: limited ductile
$N_{ft}^* = 0.9 b_f t_f f_{yf}$	Flange tension yield: elastic
$N_{ww}^* = 0.9\left(d - 2t_f\right) t_w f_{yw}$	Web tension yield

3. Connection Design Strength Limits

$\phi V_{con} = \left[\phi V_{gsb}; \phi V_{ww}\right]_{\min}$	Connection design shear capacity
$\phi M_{con} = \phi_s Z_{ex} f_{yf}$	Connection design moment capacity

4. Web Design Strength Limits

$\phi V_{gsb} = 0.6\phi_s 0.6 d t_w f_{yw}$	Web shear with max. flexure: HR sections
$\phi V_{gsb} = 0.6\phi_s 0.6(d - 2t_f)t_w f_{yw}$	Web shear with max. flexure: Welded sections

5. Flange Fillet Welds Design Strength Limits

$\phi N_{wf} = 2\phi_w 0.6 f_{uw} \frac{t_{wf}}{\sqrt{2}} b_f$	Weld tension
$\phi_w = 0.8$	SP fillet weld

6. Web Fillet Welds Design Strength Limits

$\phi N_{ww} = 2\phi_w 0.6 f_{uw} \frac{t_{ww}}{\sqrt{2}}(d - 2t_f)$	Weld tension
$\phi V_{ww} = \phi N_{ww}$	Weld shear
$\phi_w = 0.8$	SP fillet weld

E. WM Welded Moment Drawings

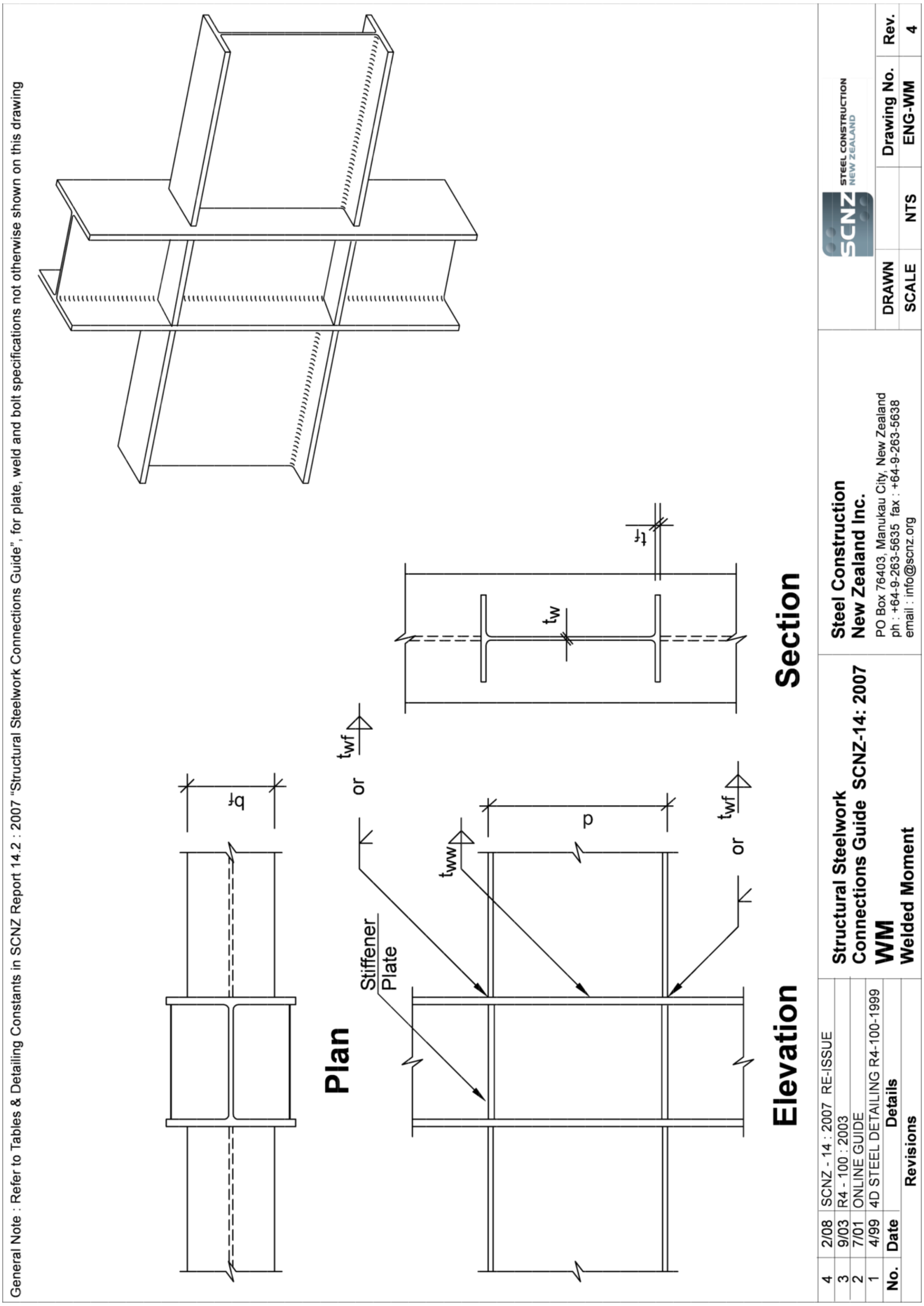


Figure 20 WM drawing

X. MEP: Moment End Plate: Extended

A. Design Objectives

The connections are designed as rigid beam-end connections for beam to column joints.

They must possess design capacity to satisfy the flexural and shear ultimate limit state loads for primary members of limited ductile, $1.25 < \mu \leq 3.0$ and elastic, $\mu = 1.0$, seismic ductility demand category frames.

Design to ensure reliable rigid connection rotational stiffness characteristics.

Design to ensure endplates, welds and bolts have sufficient design capacity to accommodate over-strength actions resulting from the development of significant plastic deformation in primary members of limited ductile frames. For connections subject to only elastic demand the endplates and bolts shall accommodate design actions. However the welds to the flanges and webs shall develop the design capacity of the web and flange to enhance ductile behaviour under fire conditions.

The column-side aspects of the beam to column joint are not covered in the guide but will need to be checked by the designer. Column-side aspects include: tension pull-out capacity of the column supporting flange; Tension and compression stiffeners; and web panel reinforcement. Guidance is given in HERA Report R4-142: 2007 “Columns in Moment Resisting Connections”.

B. Design Features

The design method is draws upon the approach set out by the BCSA & SCI in their publication “Joints in Steel Construction: Moment Connections”, P207/95. The method is modified to account for the over-strength actions expected to develop in connections located adjacent to plastic hinge zones in rigid seismic resisting frames. Typical limiting conditions are : Flexure or shear of the section, yield of the endplate or endplate support, bolt tensile capacity.

The connections are able to resist loads of reversing sign. Design moment capacities of opposing sign are tabulated for reversing moments that are limited by either the top or bottom bolt group design tension capacities.

In connections for primary members of limited ductile frames, bolts and welds ,can resist design actions resulting from the member reaching its limited ductile over-strength section moment capacity, $\phi_{oms} M_s$. In connections for primary members of elastic frames, bolts can resist design actions required from analysis but not less than 60% of section design capacity.

In accordance with NZS 3404:1997, the flange over-strength calculation formulae listed assumes use of AS/NZS 3679.1 G300 sections and 300MOD welded sections with a supplier’s material variation factor, $\phi_{om} = \dfrac{\sigma_{y97.5\%}}{\sigma_{y2.5\%}} \leq 1.20$. For steel sections of other grades or sourced from suppliers with higher material variation factors, refer to Table 12.2.8.1(1) NZS3404:1997 for the appropriate over-strength factors and adjust weld sizes as necessary.

$\phi_{oms} = 1.15$	Limited ductile G300MOD over-strength
$\phi_{oms} = 1.25$	Limited ductile G300 AS/NZS 3679.1 over-strength

The end plates for primary members in limited ductile frames, are sized for over-strength actions developed by the yielding of the section . The end plates for primary members in elastic frames are designed for design actions required from analysis. However to ensure adequate frame rigidity in connections with end plates less than 20 mm thick, the minimum plate thickness and bolt combination required to develop the required design capacity should not be limited by mode 1 behaviour. This is in conjunction with a maximum bolt offset , m , from the beam flange, web or gusset plate of 60 mm.

Bolt row design capacities and the resulting connection design moment capacity is assessed assuming that the beam end connection capacity is not limited by column-side limit states . Load transfer stiffeners in the column, aligned with the incoming beam flanges, will typically be necessary. Where column flanges are thinner than the proposed beam end plates then flange doubler plates will usually be required.

One bolt group is assumed to resist all section flexural tension forces so as to develop the required moment design capacity. Bolt prying effects are accounted for through the assessment of three potential failure modes that incorporate prying effects. The other bolt group is assumed to resist only shear actions.

The welds shall be sized to develop the design capacity of the web and flanges so as to enhance ductile behaviour under fire conditions. It is considered that the welds should be able to develop the section capacity of the section to cope with shrinkage stresses that may develop during fire cooling phases. The bolts and end plates are assumed to have more ability to stretch and deform in response to the fire conditions than the welds and so are not affected by fire design considerations.

The welds of the flanges may be either complete penetration butt welds or symmetrical fillet welds placed either side of the flanges. Complete penetration butt welds to the flanges are suitable for all members, without specific design. However these welds shall not be ground flush, but shall maintain minimum butt weld reinforcement in accordance with AS / NZS 1554.1.

Where fillet welds are specified these are designed to develop the corresponding over-strength or design capacity of the flange for the given seismic frame ductility category. Over-strength factor ϕ_{oms} replaces ϕ_s for primary members of limited ductile frames, as per Table 12.2.8(1) NZS3404:1997. For primary members of elastic frames no over-strength factor is required.

Welds to the web are designed to develop the design tension capacity of the section web.

Sections shall satisfy the material and section geometry requirements of NZS3404:1997 Section 12.4 and 12.5, for the seismic ductility classification.

End plates with thickness $t_i > 50$ mm will need to be ultrasonically tested for through thickness plate defects adjacent to the welded zone. The designer should specify ‘Through-thickness tested plate”, designated with a “Z” after the grade e.g. G350Z.

C. Design Procedure

1. Governing Criteria

Connection moment capacity shall be greater than the design moment for elastic connections. For connections in limited ductile frames, moment capacity shall be greater than the limited ductile over-strength section design moment capacity. End plate rigidity requires limits on plate thickness and bolt offsets.

Shear capacity of the compression flange bolt group shall be greater than the applied ultimate limit state shear.

Flange fillet welds shall develop the flange over-strength tension capacity for connections in primary members of limited ductile frames and have design capacity greater than the design capacity of the flange for connections in elastic frames.

Web welds shall develop design tension yield capacity of the section web for connections to all members.

Bolt edge distance and internal bolt pitch between the flanges shall satisfy criteria.

2. Design Actions

Design moment
Design shear
Web tension yield force

3. Connection Design Strength Limits

Connection design moment capacity
Connection design shear capacity

4. End Plate Transverse Design Strength Limits

Plate shear
Bolt hole 1st bearing
Bolt hole 1st transverse tearing
Gross transverse shear yield

5. End Plate Bolt Row Design Capacities

End plate pull-out tension modes 1, 2 & 3
Pull-out shear with max. flexure

6. Web Shear Strength Limits

Shear yield with maximum moment

7. Support Design Strength Limits

Bolt hole 1st bearing.
Bolt hole 1st tearing.

8. Flange Fillet Weld Design Strength Limits

Flange tension.

9. Web Weld Design Strength Limits

Weld shear.
Weld tension.

D. Design Formulae

1. Governing Criteria

$V^* \leq \phi V_{con}$	Shear
$M^* \leq \phi M_{con}$	Moment : Elastic demand
$M^*_{oms} \leq \phi M_{con}$	Moment : Limited ductile demand
$\phi N_1 \geq [\phi N_2, \phi N_3]_{\min}$ if $t_i < 20$	Rigidity with minimum t_i and d_f
$m \leq 60$	Rigidity
$N^*_{ft} \leq \phi N_{wf}$	Flange weld
$N^*_{ww} \leq \phi N_{ww}$	Web weld
$175 d_f \leq a_{e1} \leq 2.5 d_f$	Bolt edge end distance: Manual flame cut

$\frac{b_i - s_g}{2} \geq 1.75 d_f$	Bolt edge side distance: Manual flame cut
$d_i - 2(a_{e1} + a_f + p_f) \geq 70$ for $d_f \leq 20$; 90 for $d_f \geq 24$	Internal bolt pitch between flanges

2. Design Actions

M^*	Design moment : Elastic
$M^*_{oms} = \phi_{oms} Z_{ex} f_{yf}$	Over-strength moment: Limited ductile
V^*	Design shear
$N^*_{ft} = \phi_{oms} b_f t_f f_{yf}$	Flange over-strength: Limited ductile
$N^*_{ft} = 0.9 b_f t_f f_{yf}$	Flange yield capacity: Elastic
$N^*_{ww} = 0.9(d - 2t_f) t_w f_{yw}$	Web tension yield capacity

3. Connection Design Strength Limits

$\phi M_{con} = \phi N_{r1} d_{r1} + \phi N_{r2} d_{r2} + \psi_{r3} \phi N_{r3} d_{r3}$	Connection design moment capacity
$\phi V_{con} = \left[\phi V_b ; \phi V_i ; \phi V_{\sup} ; \phi V_{gsb} ; \phi V_{ww}\right]_{\min}$	Connection design shear capacity
$\phi V_b = n_{bb} \phi_b V_{fn}$	Bottom bolt group shear capacity

4. End Plate Design Transverse Strength Limits

$\phi V_i = \left[\phi V_{bi} ; \phi V_{tti} ; \phi V_{gsi}\right]_{\min}$	Plate shear
$\phi V_{bi} = n_{bb} \phi_s 3.2 f_{ui} d_f t_i$	Bolt hole 1st bearing
$\phi V_{tti} = n_{bb} \phi_s a_{e1} t_i f_{ui}$	Bolt hole 1st transverse tearing
$\phi V_{gsi} = 2\phi_s 0.5 f_{yi} d_i t_i$	Gross transverse shear yield

5. End Plate Bolt Row Design Capacities

a) General

$\phi N_{rx} = [\phi N_1, \phi N_2, \phi N_3, \phi N_v]_{min}$	Bolt row capacity
$\phi N_1 = \frac{\phi_s f_{yi} l_{erx} t_i^2}{m}$	Mode 1: 4 Plastic Hinges in T-Stub
$\phi N_2 = \frac{0.5 \phi_s f_{yi} l_{erx} t_i^2 + n 2 \phi_b N_{tf}}{m + n}$	Mode 2: 2 Plastic Hinges in T-Stub
$\phi N_3 = 2\phi_b N_{tf}$	Mode 3: Bolt only mode
$\phi N_v = 0.6 \phi_s 0.5 f_{yi} 2 l_{erx} t_i$	Moment interaction pull-out shear
$e = \frac{(b_i - s_g)}{2}$	Edge distance for stiffened end plates

$$\alpha = \begin{pmatrix} 8.13 + 4.49\lambda_1 - 3.44\lambda_2 - 16.7\lambda_1^2 + 4.66\lambda_2^2 - 6.8\lambda_1\lambda_2 + 8.75\lambda_1^3 - 1.2\lambda_2^3 \\ -1.23\lambda_1\lambda_2^2 + 8.32\lambda_1^2\lambda_2, \\ 2\pi \end{pmatrix}_{min}$$

Stiffened end plate factor where $\lambda_1 \leq 0.75$ and $\lambda_2 \geq 0.45$

$$\alpha = \begin{pmatrix} 1.25 + 39.33\lambda_1 - 3.58\lambda_2 - 55.94\lambda_1^2 + 40.54\lambda_2^2 - 55.34\lambda_1\lambda_2 + 21.05\lambda_1^3 - 33.00\lambda_2^3 \\ + 2.79\lambda_1\lambda_2^2 + 44.06\lambda_1^2\lambda_2, \\ 2\pi \end{pmatrix}_{min}$$

Stiffened end plate factor where $\lambda_1 \leq 0.75$ and $\lambda_2 < 0.45$

b) Effective T-Stub Length: Top Row of Bolts Without Gusset

$m_3 = a_f - 0.8t_{wf}$ — Bolt distance from flange weld

$m = m_3$ — Rows in endplate extension without gusset

$n = [a_{e1}, 1.25m_3]_{min}$ — Effective edge distance in endplate extension without gusset

$l_{er1} = [l_{e7}, l_{e8}, l_{e9}, l_{e10}, l_{e11}]_{min}$ — Top bolt row effective T-stub length

$l_{e7} = 0.5b_i$ — Double curvature plate extension pattern

$l_{e8} = 2m_3 + 0.625a_{e1} + 0.5s_g$ — Group end yielding plate extension pattern

$l_{e9} = 2m_3 + 0.625a_{e1} + e$ — Corner yielding plate extension pattern

$l_{e10} = 4m_3 + 1.25a_{e1}$ — Individual end yielding plate extension pattern

$l_{e11} = 2\pi m_3$ — Circular yielding plate extension pattern

c) Effective T-Stub Length: Top Row of Bolts With Gusset

$m_3 = a_f - 0.8t_{wf}$ — Bolt distance from flange weld

$m_4 = \frac{s_g}{2} - \frac{t_{ig}}{2} - 0.8t_{wg}$ — Bolt distance from gusset weld

$m = m_4$ — Rows adjacent to endplate gusset

$n = [e, 1.25m_4]_{min}$ — Effective edge distance adjacent to gusset

$\lambda_1 = \frac{m_4}{m_4 + e} \qquad \lambda_2 = \frac{m_3}{m_4 + e}$ — Edge distance ratios

$l_{er1} = [l_{e1}, (l_{e2}, l_{e3})_{max}, (l_{e5}, l_{e6})_{max}]_{min}$ — Top bolt row effective T-stub length

$l_{e1} = 2\pi m_4$ — Circular yielding pattern

$l_{e2} = 4m_4 + 1.25e$ — Side yielding pattern

$l_{e3} = \alpha m_4$ — Side yielding near flange pattern

$l_{e5} = 2m_4 + 0.625e + a_{e1}$ — Corner yielding pattern

$l_{e6} = \alpha m_4 - (2m_4 + 0.625e) + a_{e1}$ — Corner yielding near stiffener pattern

d) Effective T-Stub Length: Second Row of Bolts

$m_1 = \frac{s_g}{2} - \frac{t_w}{2} - 0.8t_{ww}$ — Bolt distance from web weld

$m_2 = p_f - t_f - 0.8t_{wf}$ — Bolt distance from flange weld

$m = m_1$ — Rows adjacent to web

$n = [e, 1.25m_1]_{min}$ — Effective edge distance adjacent to web

$\lambda_1 = \frac{m_1}{m_1 + e} \qquad \lambda_2 = \frac{m_2}{m_1 + e}$ — Edge distance ratios [2]

[2] SCI, "Joints in Steel Construction: Moment Connections", P207/95, Steel Construction Institute, UK, 1995, pp..23, 139

$l_{er2} = \lfloor (l_{e2}, l_{e3})_{max}, l_{e1} \rfloor_{min}$ Second of two rows

$l_{er2} = \lfloor (l_{e2}, l_{e3})_{max}, l_{e1}, (0.5 l_{e2}, (l_{e3} - 0.5 l_{e2}))_{max} + 0.5 s_p \rfloor_{min}$

Second of three rows

$l_{e1} = 2\pi m_1$ Circular yielding pattern[3]

$l_{e2} = 4m_1 + 1.25e$ Side yielding pattern

$l_{e3} = \alpha m_1$ Side yielding near flange pattern

e) Effective T-Stub Length: Third Row of Bolts

$m = m_1$ Rows adjacent to web

$n = [e, 1.25m_1]_{min}$ Effective edge distance adjacent to web

$l_{er3} = \lfloor l_{e1}, l_{e2}, 0.5(l_{e2} + s_p) \rfloor_{min}$ Third bolt row effective T-stub length

$l_{e1} = 2\pi m_1$ Circular yielding pattern

$l_{e2} = 4m_1 + 1.25e$ Side yielding pattern

6. Web Design Strength Limits

$\phi V_{gsb} = 0.6\phi_s 0.6 d t_w f_{yw}$ Web shear with max. moment: HR sections

$\phi V_{gsb} = 0.6\phi_s 0.6(d - 2t_f) t_w f_{yw}$ Web shear with max. moment: Welded sections

7. Support Design Strength Limits

$\phi V_{sup} = [\phi V_{b\,sup}; \phi V_{t\,sup}]_{min}$

$\phi V_{b\,sup} = n_{bb} \phi_s 3.2 f_{us} d_f t_s$ Support bearing

$\phi V_{t\,sup} = n_{bb} \phi_s a_{e2} t_s f_{us}$ Support tearing

8. Flange Fillet Welds Design Strength Limits

$\phi N_{wf} = 2\phi_w 0.6 f_{uw} \frac{t_{wf}}{\sqrt{2}} b_f$ Flange weld design tension capacity *(Admt 7/04)*

9. Web Fillet Welds Design Strength Limits

$\phi V_{ww} = \phi N_{ww}$ Weld Shear

$\phi N_{ww} = 2\phi_w 0.6 f_{uw} \frac{t_{ww}}{\sqrt{2}} (d - 2t_f)$ Weld tension

10. Definitions of Terms

a_{e1} Bolt edge end distance

$a_{e2} = [a_f + p_f, s_p]_{min} - \frac{d_h}{2}$ Inter-bolt edge distance

$d_h = d_f + 2$ for $d_f \leq 24$ Bolt hole diameter

$d_h = d_f + 3$ for $d_f > 24$ Bolt hole diameter

$d_{r1} = d - 0.5t_f + a_f$ Bolt row 1 lever arm

$d_{r2} = d_{r1} - a_f - p_f$ Bolt row 2 lever arm

$d_{r3} = d_{r2} - s_p$ Bolt row 3 lever arm

[3] ibid, p.19

$\phi_w = 0.8$	SP fillet weld
$\phi_b = 0.8$	Bolt strength reduction factor
$\psi_{r3} = \left[\dfrac{d_{r3}\phi N_{r2}}{d_{r2}\phi N_{r3}}, 1.0\right]_{min}$ for $t_i \geq \dfrac{d_f}{1.9}\sqrt{\dfrac{f_{uf}}{f_{yi}}}$	Bolt row force distribution triangular limit[4]
$\psi_{r3} = 1.0$ for $t_i < \dfrac{d_f}{1.9}\sqrt{\dfrac{f_{uf}}{f_{yi}}}$	Bolt row force plastic distribution

[4] SCI, "Joints in Steel Construction: Moment Connections", P207/95, Steel Construction Institute, UK, 1995, p.25

E. MEP Moment End Plate Drawings

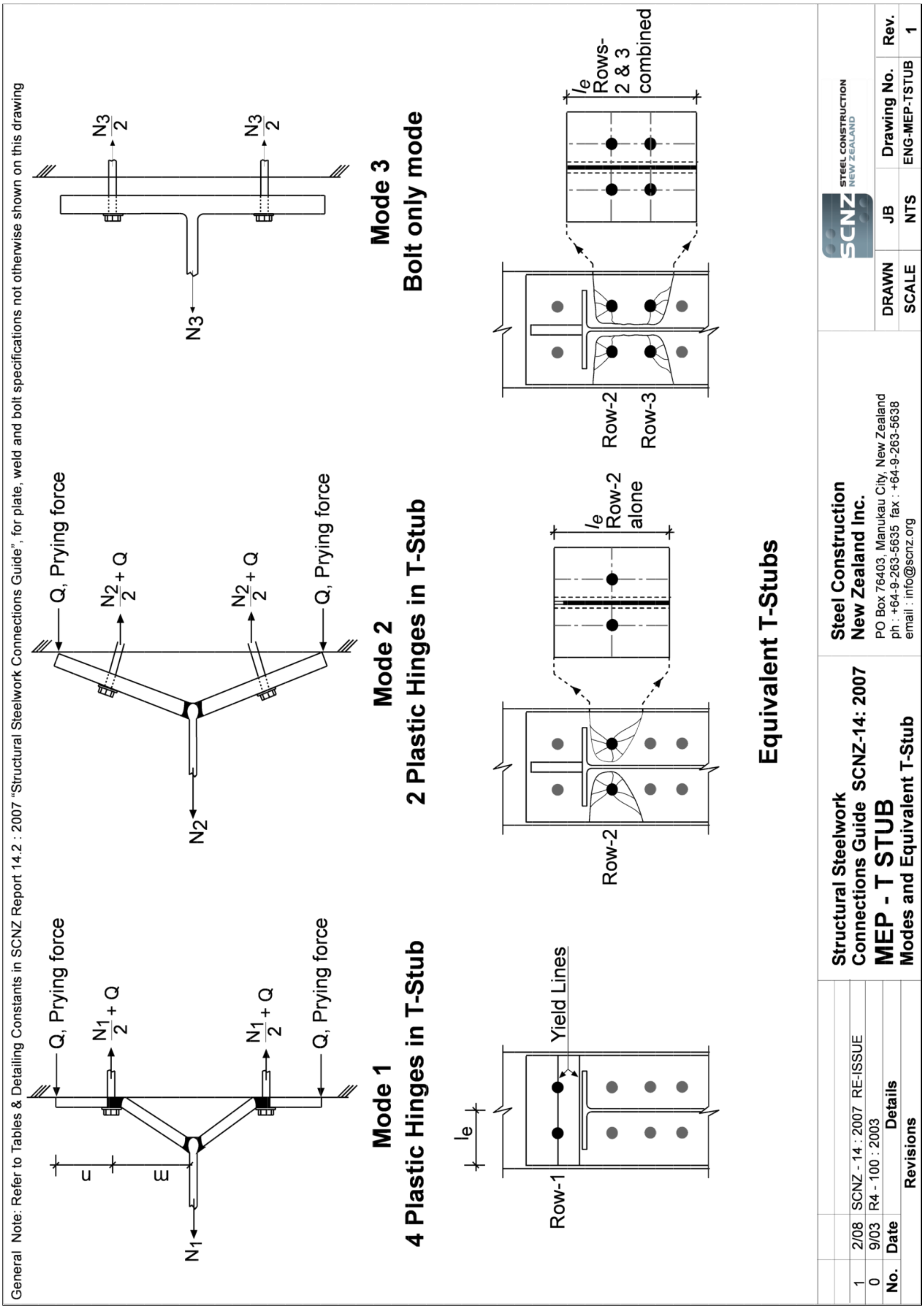


Figure 21 MEP failure modes and equivalent T-stub lengths

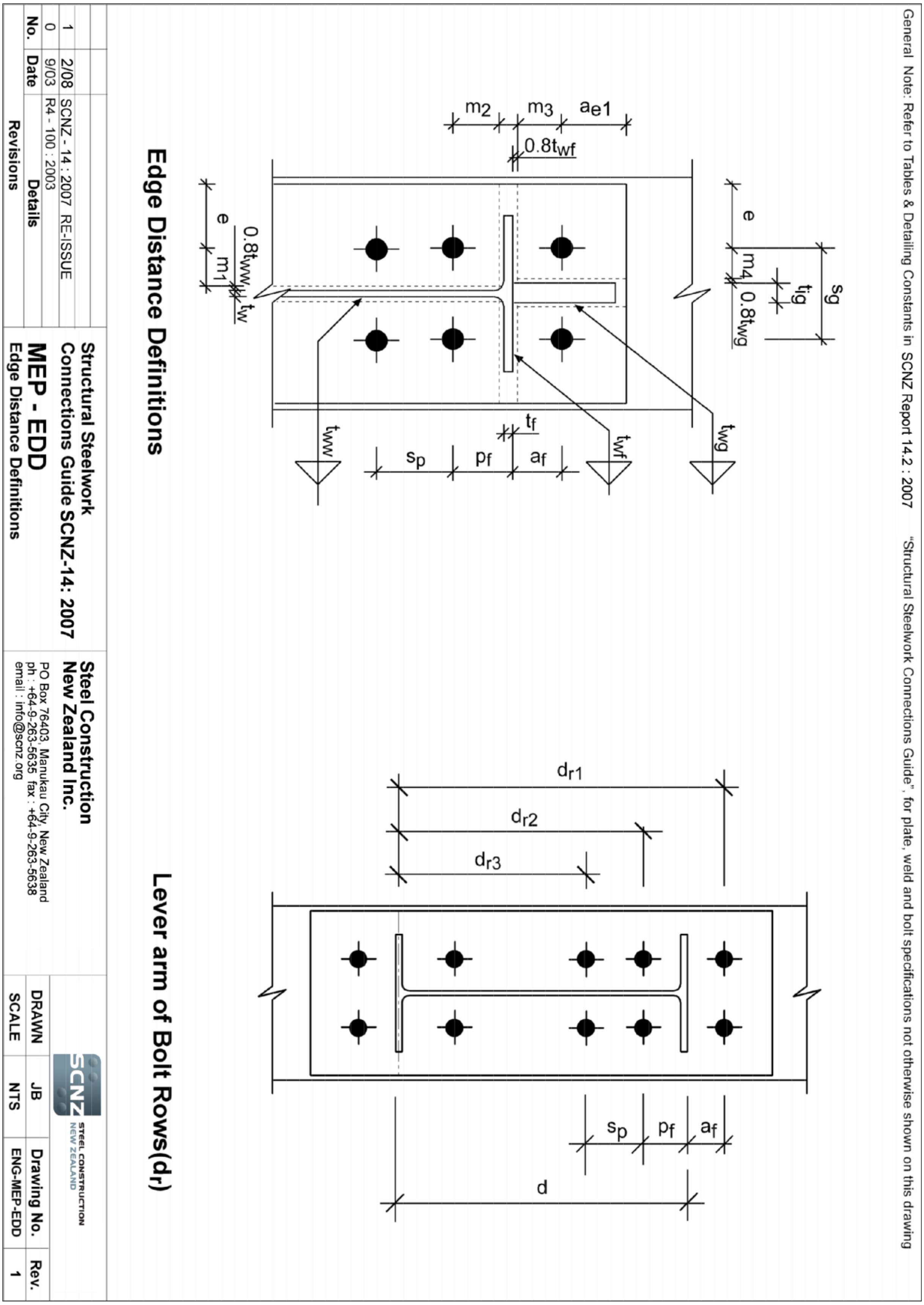


Figure 22 MEP bolt row edge distances

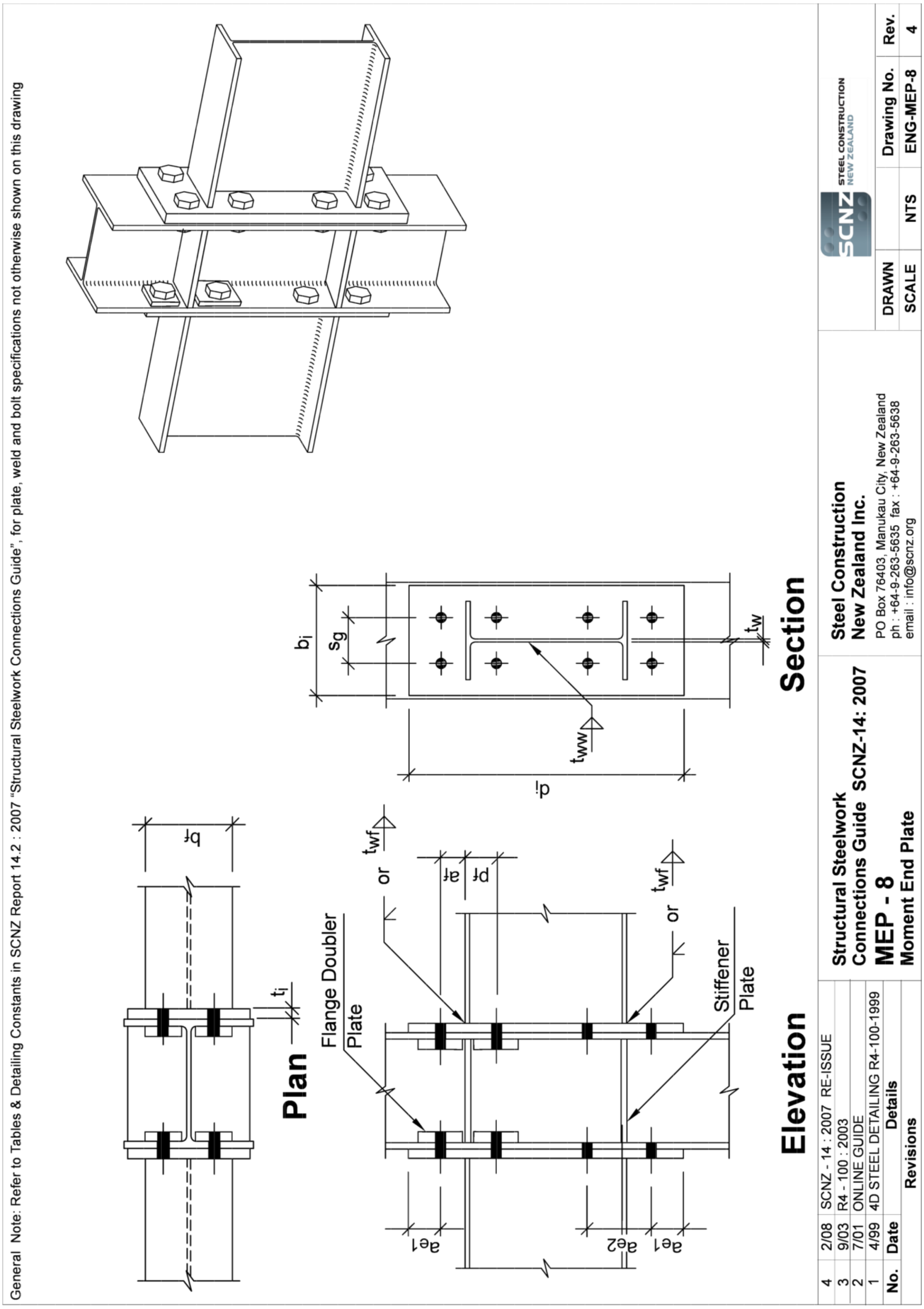


Figure 23 MEP-8 drawing

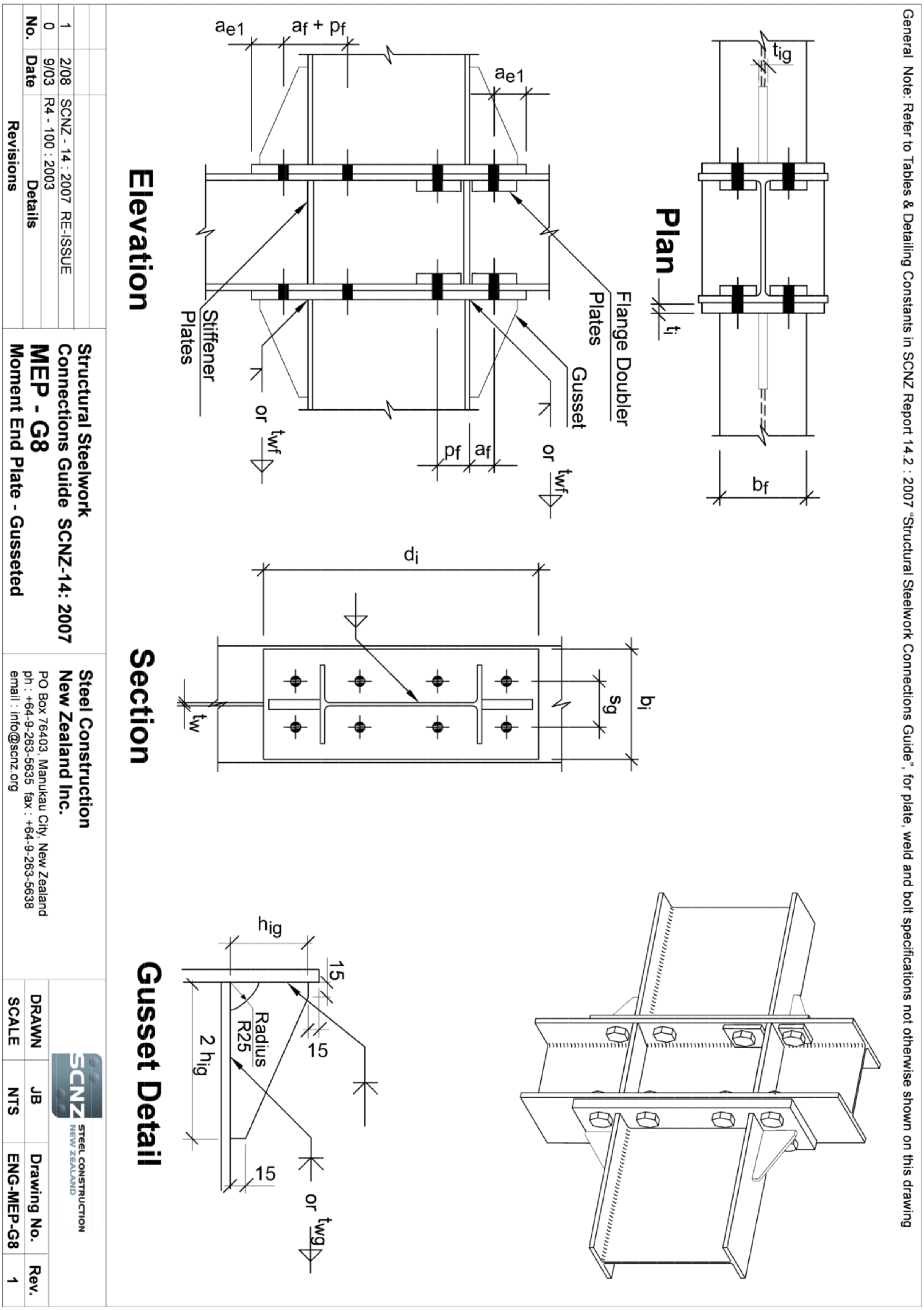


Figure 24 MEP-G8 drawing

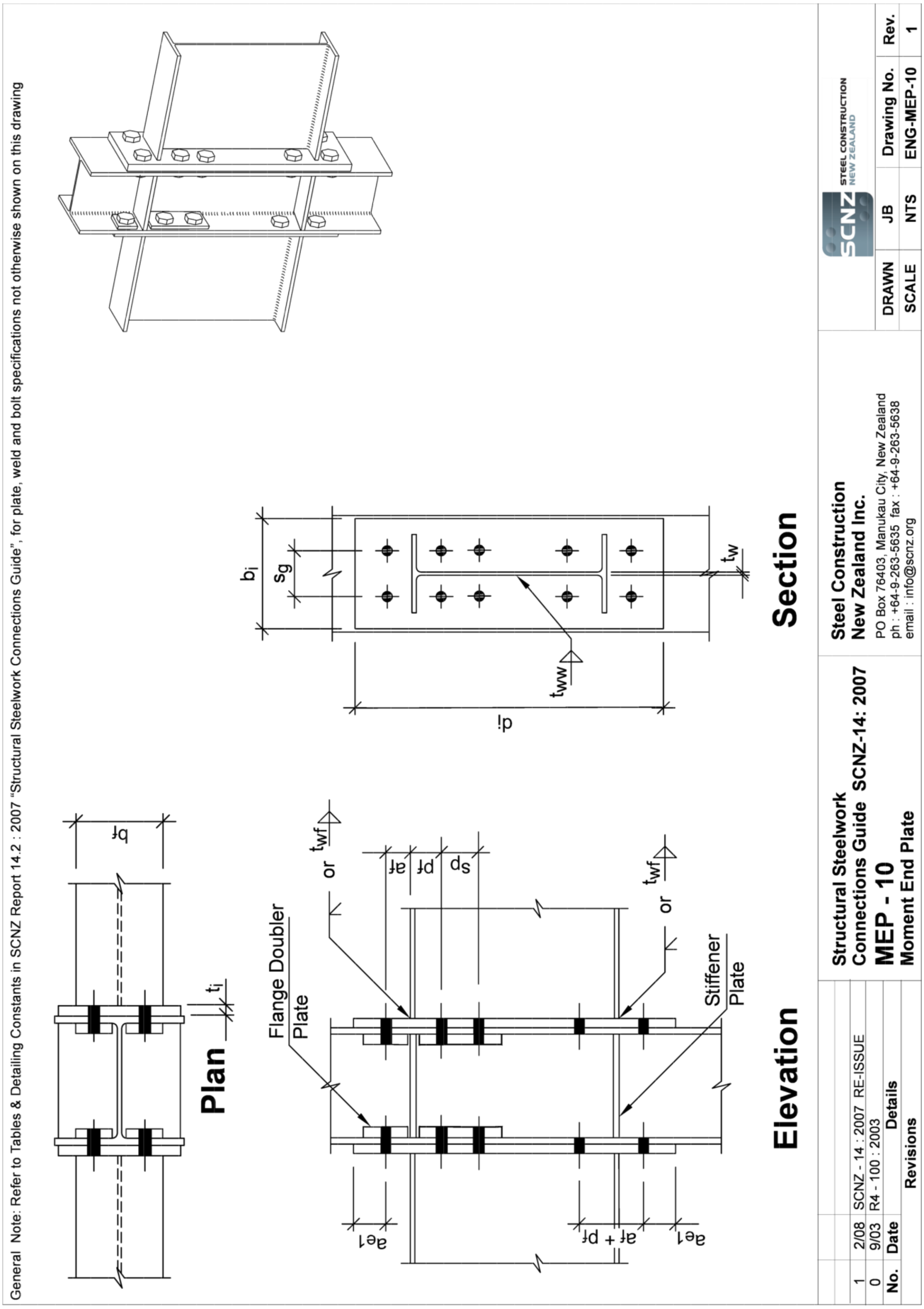


Figure 25 MEP-10 drawing

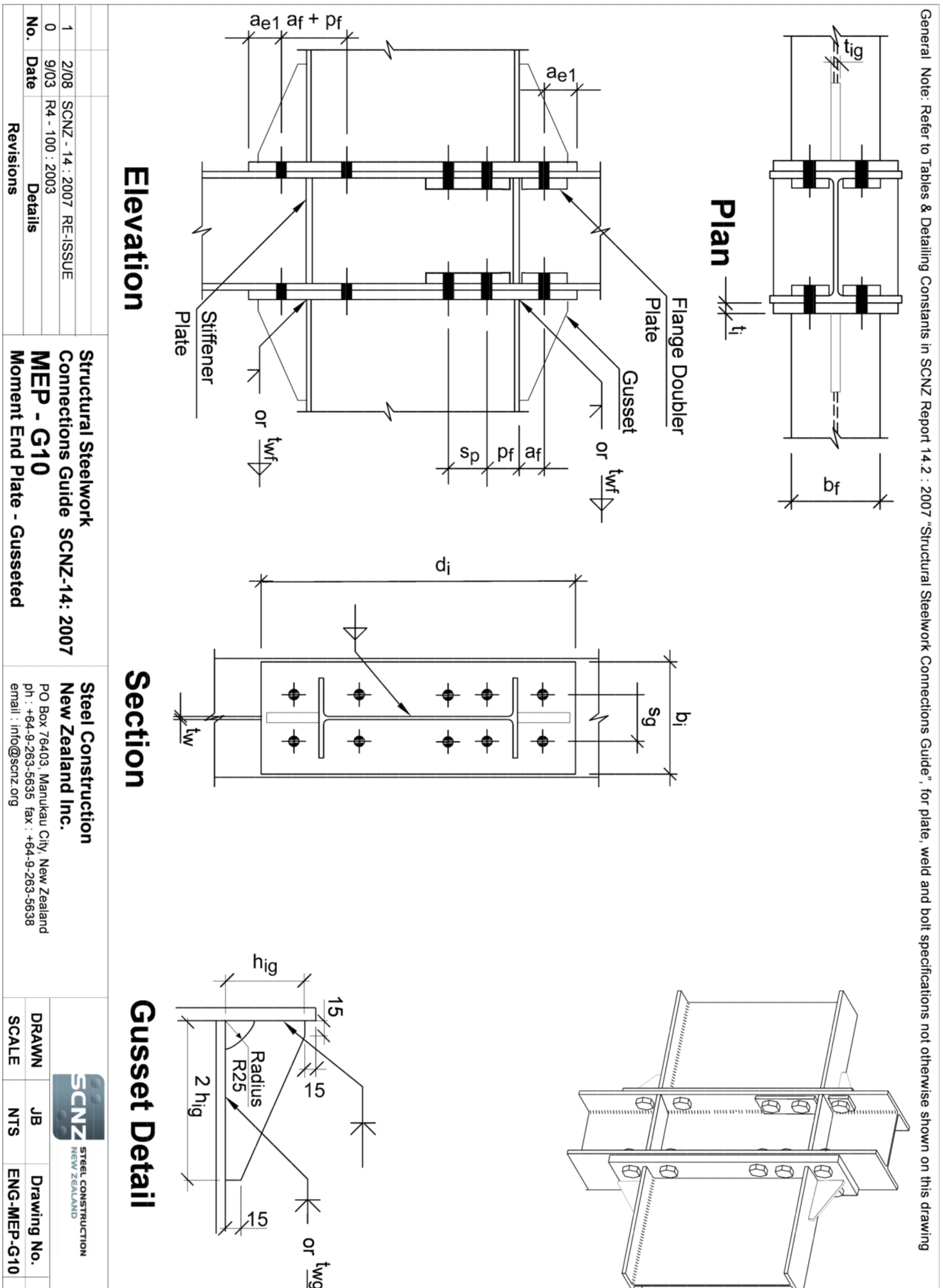


Figure 26 MEP-G10 drawing

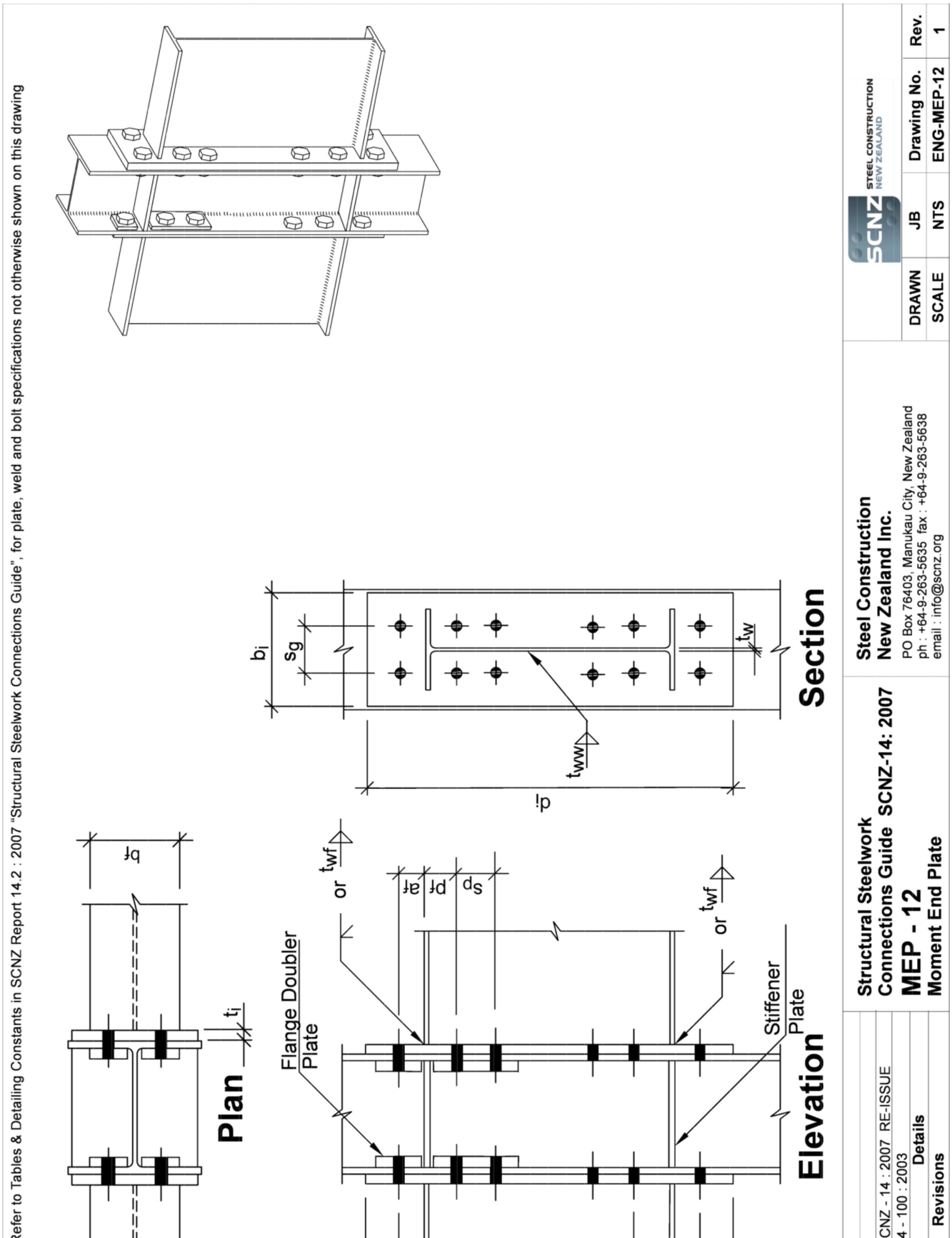


Figure 27 MEP-12 drawing

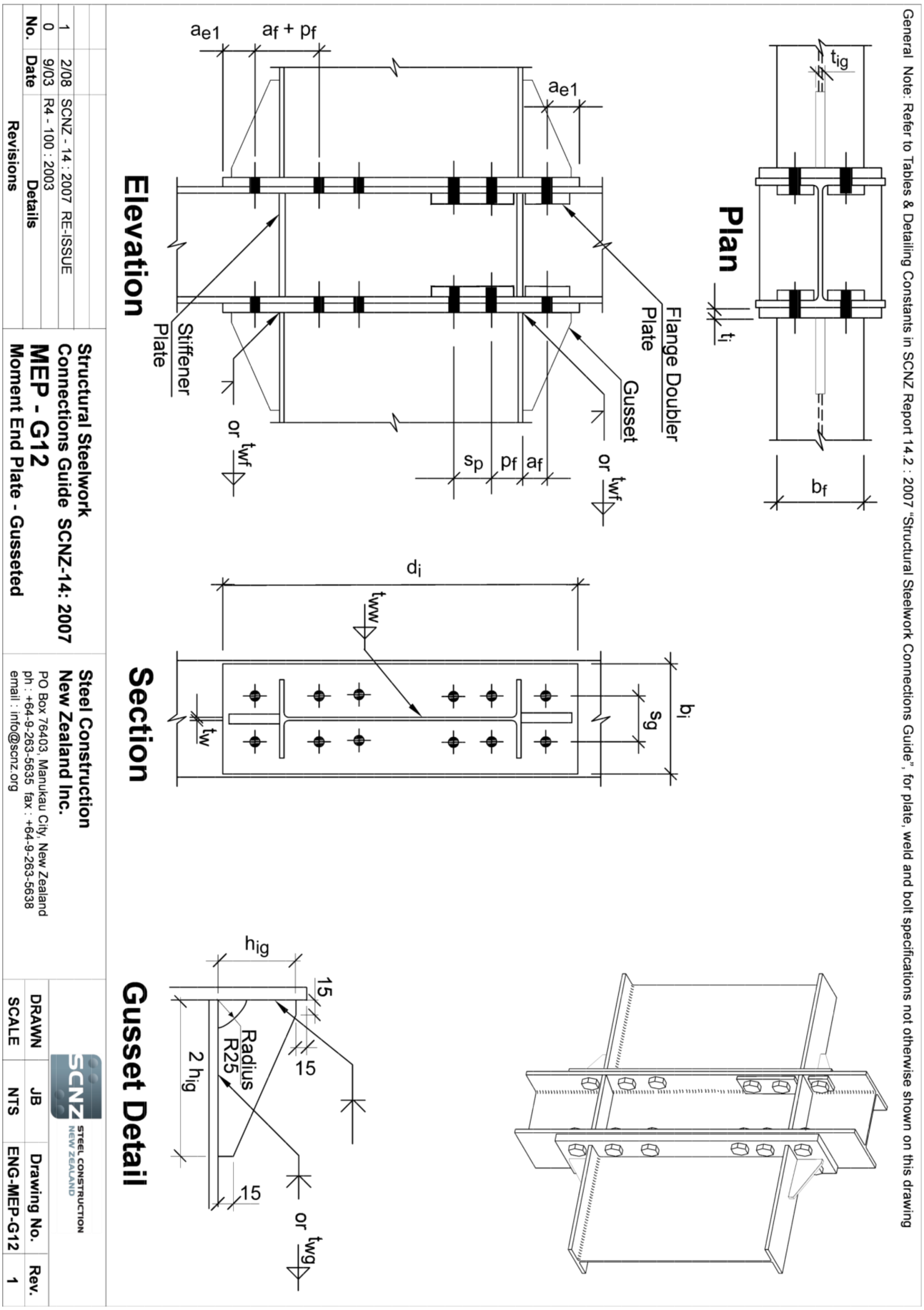


Figure 28 MEP-G12 drawing

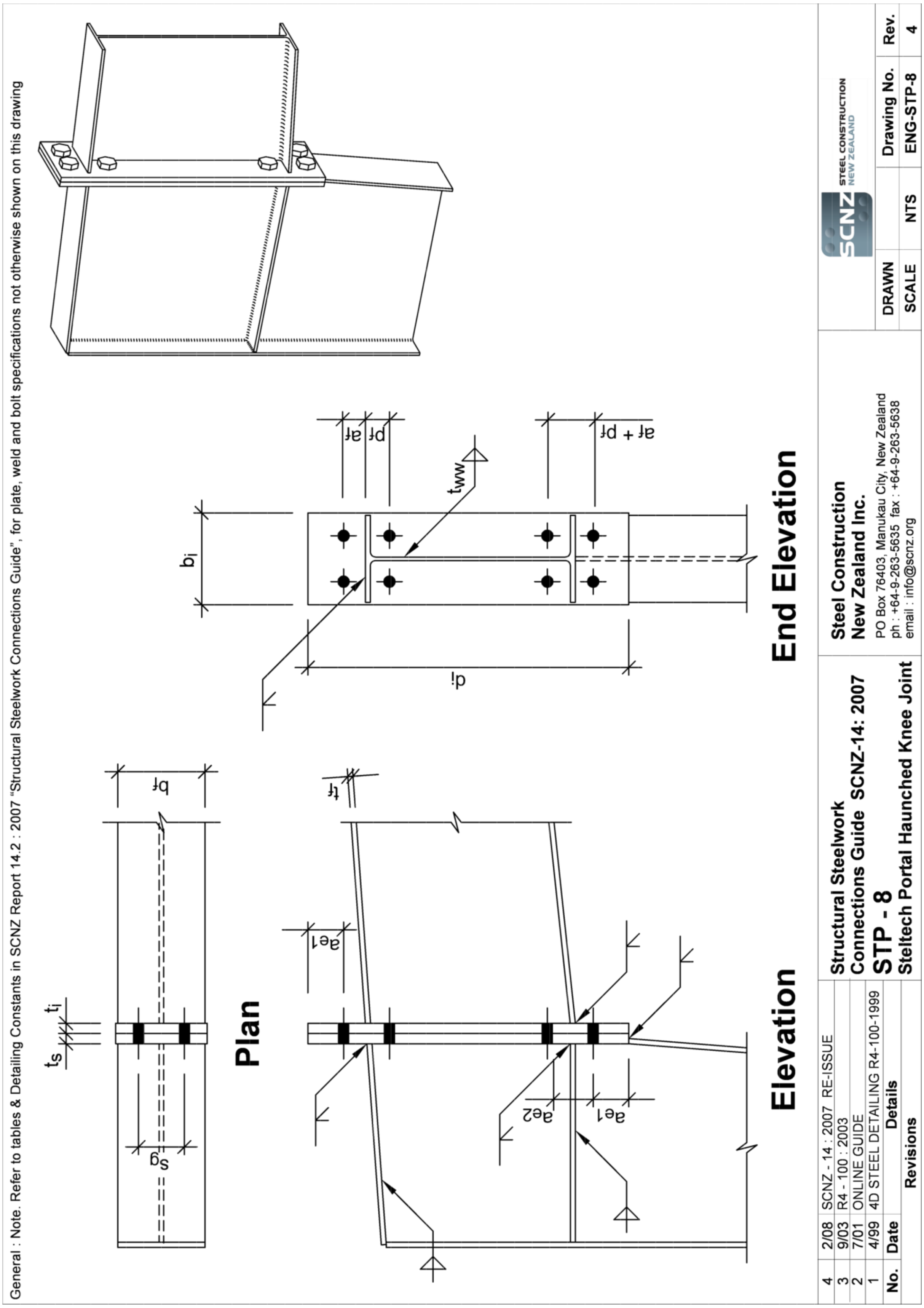


Figure 29 STP-8 drawing

XI. MEPS: Moment End Plate Splice: Flush

A. Design Objectives

Flush end-plate splices with a total four or six bolts located on the inside face of the flanges. Refer to drawing MEPS-F for typical details and set out.

Possess design capacity and ductility to resist flexural and shear ultimate limit state loads in locations away from potential yielding regions.

Respond in a ductile manner under fire conditions.

B. Design Features

Typical limiting conditions are : Flexure or shear of the section, yield of the endplate, bolt tension or shear, or weld tension or shear.

The connections are able to resist loads of reversing sign. Design moment capacities of opposing sign are tabulated for reversing moments that are limited by either the top or bottom bolt group design tension capacities.

The endplates for MEPS-F connections are sized assuming equal thickness end plates. The design method draws upon the approach set out by the BCSA & SCI in their publication "Joints in Steel Construction: Moment Connections", P207/95. Typical limiting conditions are : Flexure or shear of the section, yield of the endplate or endplate support, bolt tensile capacity.

As elastic connections that are not expected to sustain any inelastic demand, the end plates and bolts only need to resist design actions required from analysis. However to ensure adequate frame rigidity in connections with end plates less than 20 mm thick, the minimum plate thickness and bolt combination required to develop the required design capacity should not be limited by mode 1 behaviour. This is in conjunction with a maximum bolt offset , m , from the beam flange, web or gusset plate of 60 mm.

One bolt group is assumed to resist all section flexural tension forces so as to develop the required moment design capacity. Bolt prying effects are accounted for through the assessment of three potential failure modes that incorporate prying effects. The other bolt group is assumed to resist only shear actions.

The welds shall be sized to develop the design capacity of the web and flanges so as to enhance ductile behaviour under fire conditions. It is considered that the welds should be able to develop the section capacity of the section to cope with shrinkage stresses that may develop during fire cooling phases. The bolts and end plates are assumed to have more ability to stretch and deform in response to the fire conditions than the welds and so are not affected by fire design considerations.

The welds of the flanges may be either complete penetration butt welds or symmetrical fillet welds placed either side of the flanges. The flange welds shall have design capacity not less than that of the section flange. Complete penetration butt welds to the flanges are suitable for all members, without specific design. However these welds must not be ground flush, but should maintain minimum butt weld reinforcement in accordance with AS / NZS 1554.1.

Welds to the web shall be designed to develop the design tension capacity of the section web.

Sections shall satisfy the section geometry requirements of NZS3404:1997 Section 12.5, for primary members of elastic seismic frames.

End plates with thickness $t_i > 50$ mm will need to be ultrasonically tested for through thickness plate defects adjacent to the welded zone. The designer should specify 'Through-thickness tested plate", designated with a "Z" after the grade e.g. G350Z.

C. Design Procedure

1. Governing criteria

Endplate connection moment capacity shall be greater than the applied design moment. End plate rigidity requires limits on plate thickness and bolt offsets.

Shear capacity of the compression flange bolt group shall be greater than the applied ultimate limit state shear.

End plate pull-out tension capacity shall be greater than the applied design moment divided by the flange to flange lever arm.

Flange bolt group tension capacity shall be greater than the applied design moment over the flange to flange lever arm.

Flange fillet welds shall develop the flange design tension yield capacity. This recognises that load transfer from the bolts into the flange is from one side of the flange only and will induce flexural loads across the weld, requiring redistribution.

Web welds shall develop the design tension yield capacity of the section web, reduced by the ratio of the applied design moment to the section design moment capacity.

Bolt edge distance criteria shall satisfy requirements for manual flame cutting of end plates.

2. Design Actions

Design moment
Design shear
Flange bolt group tension
Flange tension yield
Web tension yield force

3. Connection Design Strength Limits

Connection design moment capacity
Connection design shear capacity

4. Bolt Group Design Strength Limits

Bolt group tension.
Bolt group shear.

5. End Plate Strength Limits *(Admt 7/04)*

Plate shear
Pull-out tension
Bolt hole 1st bearing.
Gross transverse shear

6. Web Shear Strength Limits

Shear yield with maximum flexure.

7. Flange Weld Design Strength Limits

Flange tension.

8. Web Weld Design Strength Limits

Web tension.
Web shear.

D. Design Formulae

1. Governing Criteria

$V^* \leq \phi V_{con}$	Shear
$M^* \leq \phi M_{con}$	Moment
$\phi N_1 \geq [\phi N_2, \phi N_3]_{\min}$ if $t_i < 20$	Rigidity with minimum t_i and d_f
$m \leq 60$	Rigidity
$N_{ft}^* \leq \phi N_{wf}$	Flange weld
$N_{ww}^* \leq \phi N_{ww}$	Web weld
$\dfrac{b_i - s_g}{2} \geq 175 d_f$	Bolt side edge distance
$d_i - 2(a_f + p_f) \geq$ 70 for $d_f \leq 20$; 90 for $d_f \geq 24$	Internal bolt pitch limits

2. Design Actions

M^*	Design moment
V^*	Design shear
$N_{fbt}^* = \dfrac{M^*}{d - p_f}$	Flange bolt group tension force
$N_{ft}^* = 0.9 b_f t_f f_{yf}$	Flange tension yield
$N_{ww}^* = 0.9(d - 2t_f) t_w f_{yw}$	Web tension yield

3. Connection Design Strength Limits

$\phi M_{con} = \phi N_{r1} d_{r1} + \psi_{r2} \phi N_{r2} d_{r2}$	Connection design moment capacity
$\phi V_{con} = \lfloor \phi V_b ; \phi V_i ; \phi V_{gsb} ; \phi V_{ww} \rfloor_{\min}$	Connection design shear capacity
$\phi V_b = n_{bb} \phi_b V_{fn}$	Bottom bolt group shear capacity

4. End Plate Bolt Row Design Capacities

a) General

$\phi N_{rx} = [\phi N_1, \phi N_2, \phi N_3, \phi N_v]_{\min}$	Bolt row force
$\phi N_1 = \dfrac{\phi_s f_{yi} l_{erx} t_i^2}{m}$	Mode 1: 4 Plastic Hinges in T-Stub
$\phi N_2 = \dfrac{0.5 \phi_s f_{yi} l_{erx} t_i^2 + n 2 \phi_b N_{tf}}{m + n}$	Mode 2: 2 Plastic Hinges in T-Stub
$\phi N_3 = 2 \phi_b N_{tf}$	Mode 3: Bolt only mode
$\phi N_v = 0.6 \phi_s 0.5 f_{yi} 2 l_{erx} t_i$	Moment interaction pull-out shear
$m = m_1$	Rows adjacent to web

$n = [e, 1.25m_1]_{\min}$	Effective edge distance adjacent to web
$e = \dfrac{(b_i - s_g)}{2}$	Edge distance
$l_{e1} = 2\pi m_1$	Circular yielding pattern [5]
$l_{e2} = 4m_1 + 1.25e$	Side yielding pattern
$l_{e3} = \alpha m_1$	Side yielding near flange pattern

$$\alpha = \begin{pmatrix} 8.13 + 4.49\lambda_1 - 3.44\lambda_2 - 16.7\lambda_1^2 + 4.66\lambda_2^2 - 6.8\lambda_1\lambda_2 + 8.75\lambda_1^3 - 1.2\lambda_2^3 \\ -1.23\lambda_1\lambda_2^2 + 8.32\lambda_1^2\lambda_2, \\ 2\pi \end{pmatrix}_{\min}$$

Stiffened end plate factor where $\lambda_1 \le 0.75$ and $\lambda_2 \ge 0.45$

$$\alpha = \begin{pmatrix} 1.25 + 39.33\lambda_1 - 3.58\lambda_2 - 55.94\lambda_1^2 + 40.54\lambda_2^2 - 55.34\lambda_1\lambda_2 + 21.05\lambda_1^3 - 33.00\lambda_2^3 \\ + 2.79\lambda_1\lambda_2^2 + 44.06\lambda_1^2\lambda_2, \\ 2\pi \end{pmatrix}_{\min}$$

Stiffened end plate factor where $\lambda_1 \le 0.75$ and $\lambda_2 < 0.45$

$\lambda_1 = \dfrac{m_1}{m_1 + e} \qquad \lambda_2 = \dfrac{m_2}{m_1 + e}$	Edge distance ratios [6]
$m_1 = \dfrac{s_g}{2} - \dfrac{t_w}{2} - 0.8t_{ww}$	Bolt distance from web weld
$m_2 = p_f - t_f - 0.8t_{wf}$	Bolt distance from flange weld

b) Effective T-Stub Length: Top Row of Bolts *(Admt 7/04)*

One row, MEPS – F4

$l_{er1} = [(0.5(l_{e2} + l_{e3}), l_{e2})_{\max}, l_{e1}]_{\min}$	for $s_g > 0.7b_f$ *or* $t_f < 0.8t_i$
$l_{er1} = [(l_{e2}, l_{e3})_{\max}, l_{e1}]_{\min}$	otherwise

Two rows, MEPS – F6 or MEPS – F8

$l_{er1} = [(0.5(l_{e2} + l_{e3}), l_{e2})_{\max}, l_{e1}, (0.5l_{e2}, 0.5l_{e3})_{\max} + 0.5s_p]_{\min}$	for $s_g > 0.7b_f$ *or* $t_f < 0.8t_i$
$l_{er1} = [(l_{e2}, l_{e3})_{\max}, l_{e1}, (0.5l_{e2}, (l_{e3} - 0.5l_{e2}))_{\max} + 0.5s_p]_{\min}$	otherwise

c) Effective T-Stub Length: Second Row of Bolts

$l_{er2} = [l_{e1}, l_{e2}, 0.5(l_{e2} + s_p)]_{\min}$	Second bolt row effective T-stub length

5. End Plate Design Transverse Strength Limits *(Admt 7/04)*

$\phi V_i = [\phi V_{bi}; \phi V_{tti}; \phi V_{gsi}]_{\min}$	Plate shear
$\phi V_{bi} = n_{bb}\phi_s 3.2 f_{ui} d_f t_i$	Bolt hole 1st bearing
$\phi V_{gsi} = 2\phi_s 0.5 f_{yi} d_i t_i$	Gross transverse shear yield

6. Web Design Strength Limits

$\phi V_{gsb} = 0.6\phi_s 0.6 d t_w f_{yw}$	Web shear yield with max. moment: HR

[5] SCI, “Joints in Steel Construction: Moment Connections”, P207/95, Steel Construction Institute, UK, 1995, p.19
[6] ibid, pp..23, 139

$\phi V_{gsb} = 0.6\phi_s 0.6(d - 2t_f)t_w f_{yw}$ — Web shear yield with max. moment: Welded

7. Flange Fillet Weld Design Strength Limits

$\phi N_{wf} = 2\phi_w 0.6 f_{uw} \frac{t_{wf}}{\sqrt{2}} b_f$ — Flange weld tension *(Admt 7/04)*

$\phi_w = 0.8$ — SP fillet weld

8. Web Fillet Weld Design Strength Limits

$\phi V_{ww} = \phi N_{ww}$ — Weld Shear

$\phi N_{ww} = 2\phi_w 0.6 f_{uw} \frac{t_{ww}}{\sqrt{2}}(d - 2t_f)$ — Web weld tension

$\phi_w = 0.8$ — SP fillet weld

9. Definitions of Terms

a_f — Plate edge end distance to flange

$d_h = d_f + 2$ for $d_f \leq 24$ — Bolt hole diameter

$d_h = d_f + 3$ for $d_f > 24$

$d_{r1} = d - p_f - 0.5t_f$ — Bolt row 1 lever arm

$d_{r2} = d_{r1} - s_p$ — Bolt row 2 lever arm

p_f — Inside bolt to top flange distance

$\psi_{r2} = \left[\frac{d_{r2}\phi N_{r1}}{d_{r1}\phi N_{r2}}, 1.0\right]_{\min}$ for $t_i \geq \frac{d_f}{1.9}\sqrt{\frac{f_{uf}}{f_{yi}}}$ — Bolt row force distribution triangular limit[7]

$\psi_{r2} = 1.0$ for $t_i < \frac{d_f}{1.9}\sqrt{\frac{f_{uf}}{f_{yi}}}$ — Bolt row force plastic distribution

[7] SCI, "Joints in Steel Construction: Moment Connections", P207/95, Steel Construction Institute, UK, 1995, p.25

E. MEPS Moment End Plate Splice Drawings

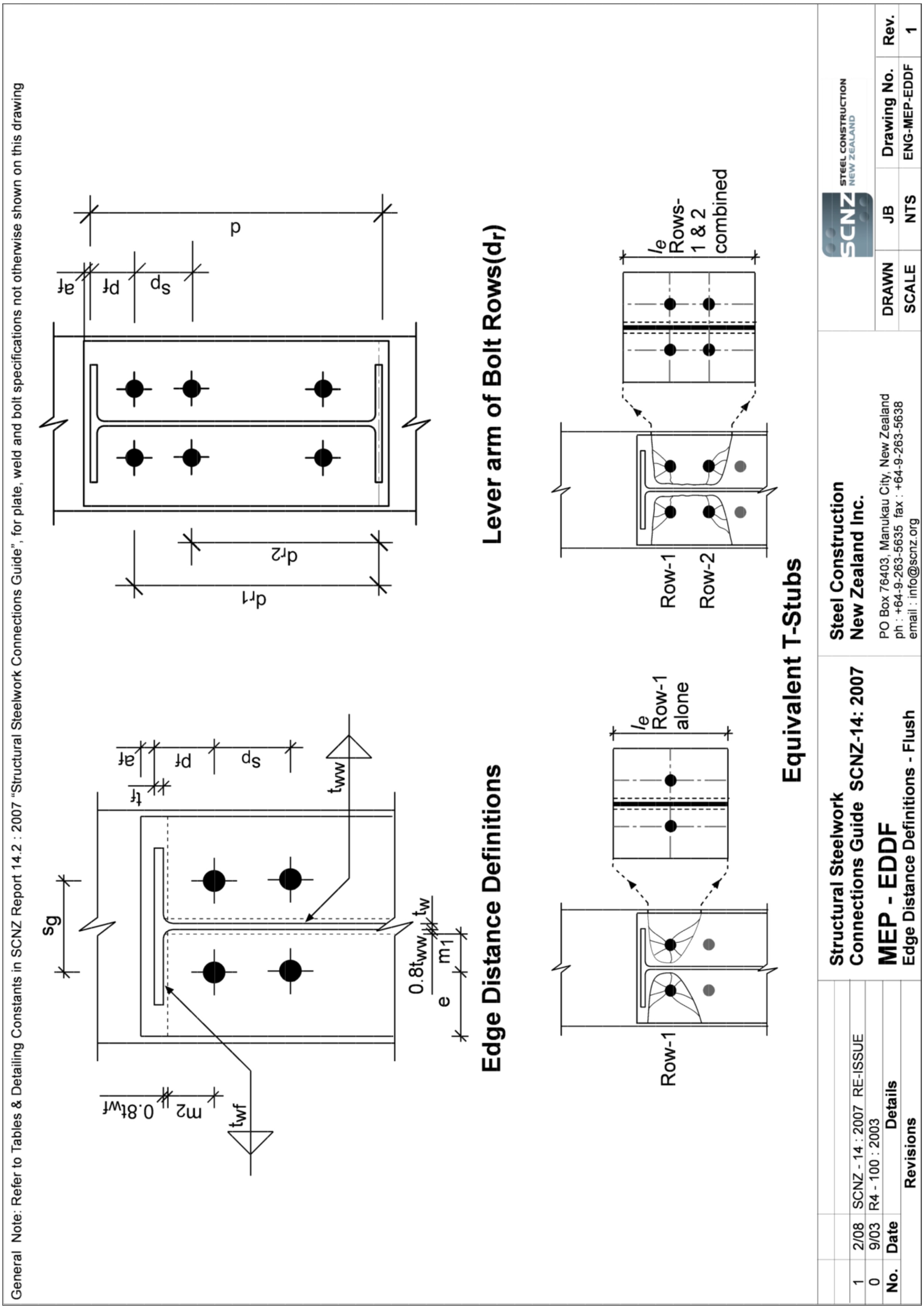


Figure 30 MEPS-F Edge distances

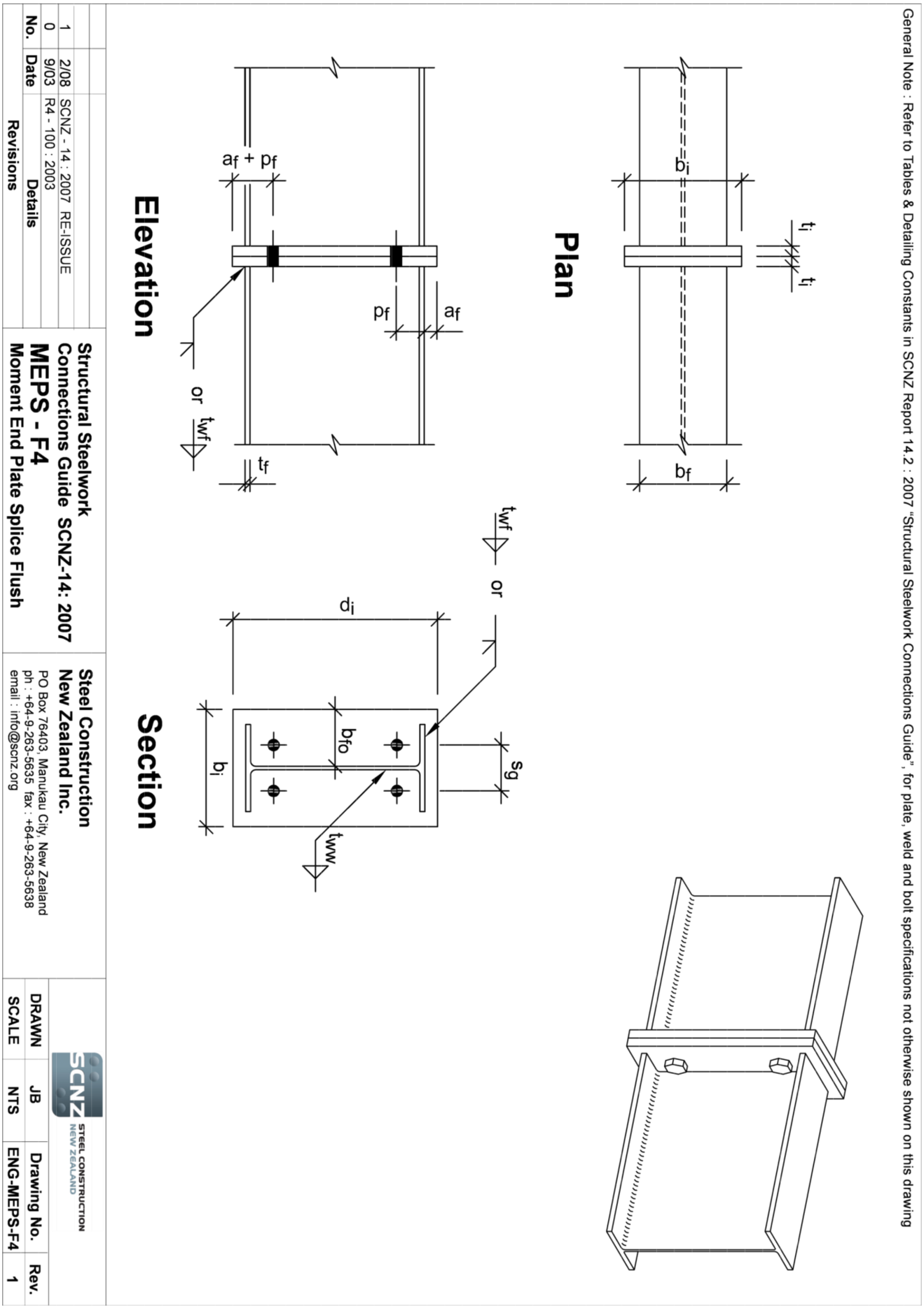


Figure 31 MEPS-F4 drawing

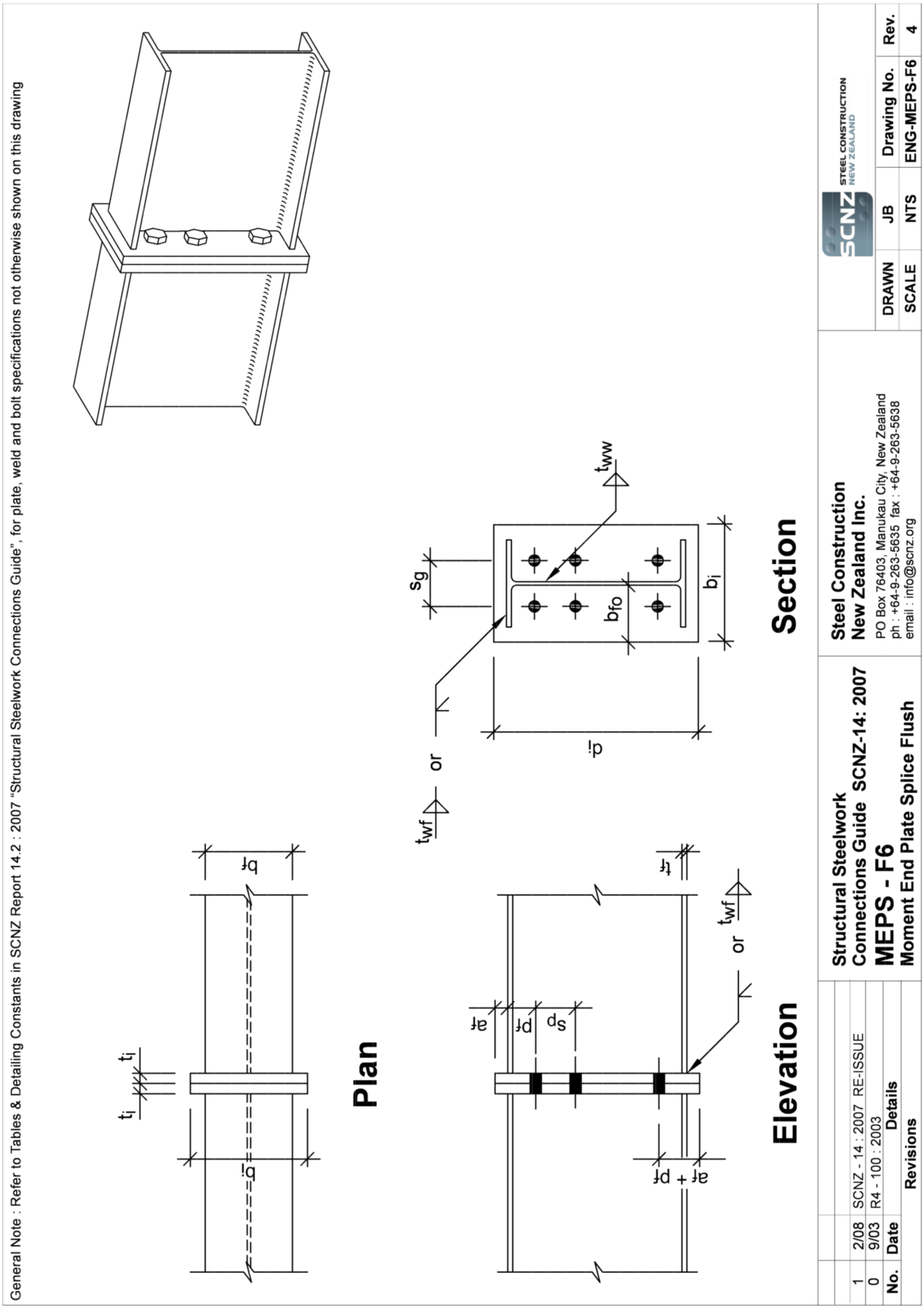


Figure 32 MEPS-F6 drawing

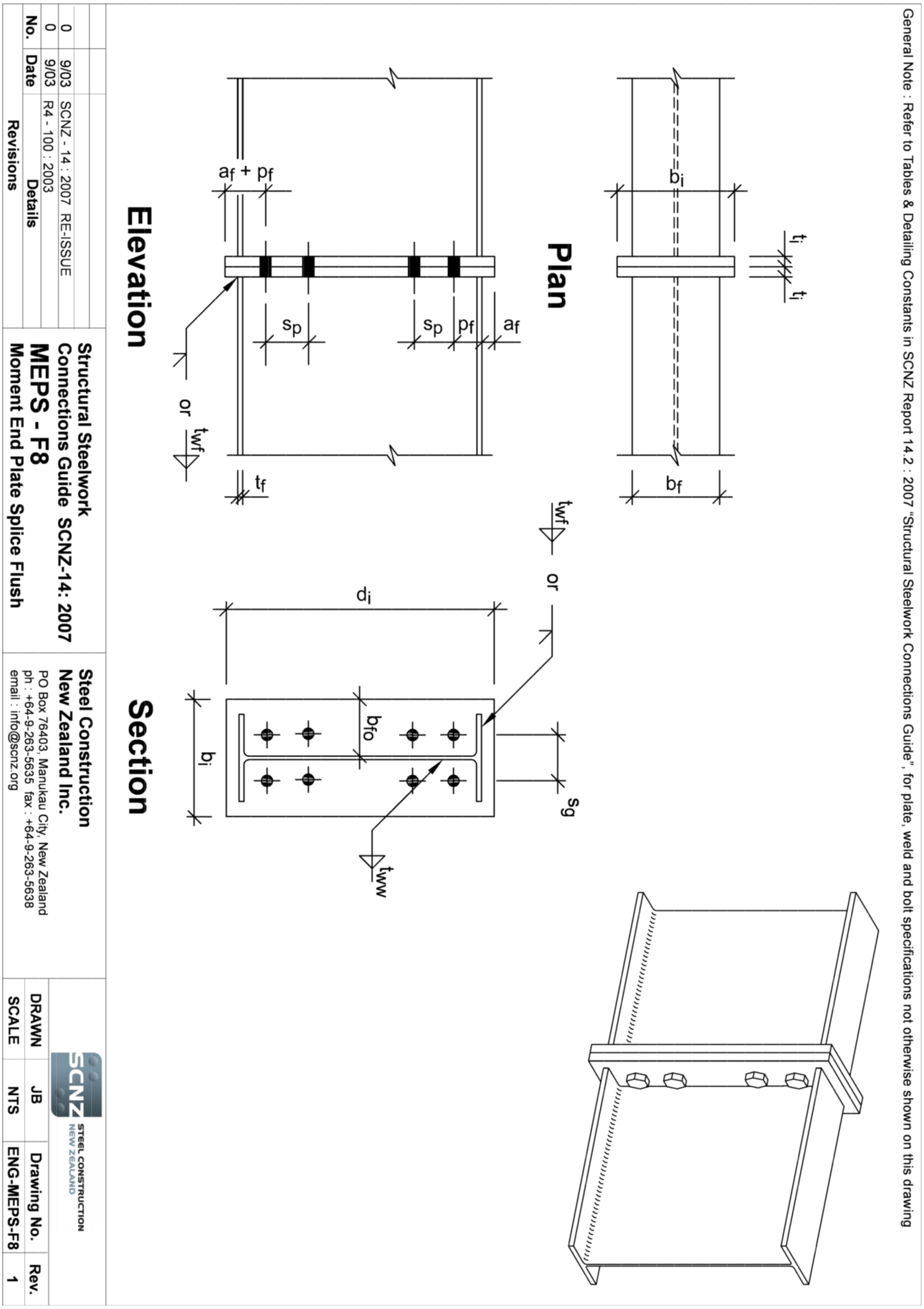


Figure 33 MEPS-F8 drawing

XII. MEPS: Moment End Plate Splice: Extended

A. Design Objectives

Extended end-plate splices with a total eight, ten or twelve bolts. Refer to drawings MEPS-E for typical details and set out.

Possess design capacity to resist flexural and shear ultimate limit state loads in locations away from potential yielding regions.

Respond in a ductile manner to fire conditions.

B. Design Features

Typical limiting conditions are : Flexure or shear of the section, yield of the endplate, bolt tension or shear, or weld tension or shear.

The connection is able to resist loads of reversing sign. Design moment capacities of opposing sign are tabulated for reversing moments that are limited by either the top or bottom bolt group design tension capacities.

Either the top or bottom bolt group is assumed to resist all design flexural tension forces. The other bolt group is assumed to resist shear actions. The endplates for MEPS-E connections are sized assuming equal thickness end plates. The design method draws upon the approach set out by the BCSA & SCI in their publication “Joints in Steel Construction: Moment Connections”, P207/95. Typical limiting conditions are : Flexure or shear of the section, yield of the endplate or endplate support, bolt tensile capacity.

As elastic connections that are not expected to sustain any inelastic demand, the end plates and bolts only need to resist design actions required from analysis. However to ensure adequate frame rigidity in connections with end plates less than 20 mm thick, the minimum plate thickness and bolt combination required to develop the required design capacity should not be limited by mode 1 behaviour. This is in conjunction with a maximum bolt offset , m , from the beam flange, web or gusset plate of 60 mm.

One bolt group may be assumed to resist all section flexural tension forces so as to develop the required moment design capacity. Bolt prying effects are accounted for through the assessment of three potential failure modes that incorporate prying effects. The other bolt group is assumed to resist only shear actions.

The welds of the flanges may be either complete penetration butt welds or symmetrical fillet welds placed either side of the flanges. Complete penetration butt welds to the flanges are suitable for all members, without specific design. However these welds should not be ground flush, but maintain minimum butt weld reinforcement in accordance with AS / NZS 1554.1.

The welds shall be sized to develop the design capacity of the web and flanges so as to enhance ductile behaviour under fire conditions. It is considered that the welds should be able to develop the section capacity of the section to cope with shrinkage stresses that may develop during fire cooling phases. The bolts and end plates are assumed to have more ability to stretch and deform in response to the fire conditions than the welds and so are not affected by fire design considerations.

Welds to the web shall be designed to develop the design tension capacity of the section web.

Sections shall satisfy the section geometry requirements of NZS3404:1997 Section 12.5, for members of elastic frames.

End plates with thickness $t_i > 50$ mm should be ultrasonically tested for through thickness plate defects adjacent to the welded zone. The designer should specify 'Through-thickness tested plate", designated with a "Z" after the grade e.g. G350Z.

C. Design Procedure

1. Governing Criteria

Endplate connection design moment capacity shall be greater than the applied design moment. End plate rigidity requires limits on plate thickness and bolt offsets.

Shear capacity of the compression flange bolt group shall be greater than the applied ultimate limit state shear.

The flange welds shall be designed to resist the lesser of the design tension capacity of the flange or the applied ultimate limit state moment divided by the flange to flange lever arm.

2. Design Actions

Design moment
Design shear
Flange moment tension
Flange tension yield
Web tension yield

3. Connection Design Strength Limits

Connection design moment capacity
Connection design shear capacity

4. Bolt Group Design Strength Limits

Bolt group tension.
Bolt group shear.

5. End Plate Strength Limits

Pull-out flexure
Pull-out shear
Transverse shear
Bolt hole 1st bearing.
Bolt hole 1st tearing.

6. Web Shear Strength Limits

Shear yield with maximum flexure.

7. Flange Weld Design Strength Limits

Flange tension.

8. Web Weld Design Strength Limits

Web weld tension.
Web weld shear.

D. Design Formulae

1. Governing Criteria

$V^* \le \phi V_{con}$	Shear
$M^* \le \phi M_{con}$	Moment
$\phi N_1 \ge [\phi N_2, \phi N_3]_{\min}$ if $t_i < 20$	Rigidity with minimum t_i and d_f
$m \le 60$	Rigidity
$N_{ft}^* \le \phi N_{wf}$	Flange weld
$N_{ww}^* \le \phi N_{ww}$	Web weld
$175 d_f \le a_{e1} \le 2.5 d_f$	Bolt edge end distance
$\dfrac{b_i - s_g}{2} \ge 1.75 d_f$	Bolt edge side distance
$d_i - 2(a_{e1} + s_p) \ge$ 70 for $d_f \le 20$; 90 for $d_f \ge 24$	Internal bolt pitch

2. Design Actions

M^*	Design moment
V^*	Design shear
$N_{ft}^* = 0.9 b_f t_f f_{yf}$	Flange tension yield
$N_{ww}^* = 0.9(d - 2t_f)t_w f_{yw}$	Web tension yield

3. Connection Design Strength Limits

$\phi M_{con} = \phi N_{r1} d_{r1} + \phi N_{r2} d_{r2} + \psi_{r3} \phi N_{r3} d_{r3}$	Connection design moment capacity
$\phi V_{con} = [\phi V_b ; \phi V_i ; \phi V_{gsb} ; \phi V_{ww}]_{\min}$	Connection design shear capacity
$\phi V_b = n_{bb} \phi_b V_{fn}$	Bolt group shear capacity

4. End Plate Bolt Row Design Capacities

a) General

$\phi N_{rx} = [\phi N_1, \phi N_2, \phi N_3, \phi N_v]_{min}$	Bolt row x capacity
$\phi N_1 = \dfrac{\phi_s f_{yi} l_{erx} t_i^2}{m}$	Mode 1: 4 Plastic Hinges in T-Stub
$\phi N_2 = \dfrac{0.5 \phi_s f_{yi} l_{erx} t_i^2 + n 2 \phi_b N_{tf}}{m + n}$	Mode 2: 2 Plastic Hinges in T-Stub
$\phi N_3 = 2 \phi_b N_{tf}$	Mode 3: Bolt only mode
$\phi N_v = 0.6 \phi_s 0.5 f_{yi} 2 l_{erx} t_i$	Moment interaction pull-out shear
$e = \dfrac{(b_i - s_g)}{2}$	Edge distance for stiffened end plates

$$\alpha = \begin{pmatrix} 8.13 + 4.49\lambda_1 - 3.44\lambda_2 - 16.7\lambda_1^2 + 4.66\lambda_2^2 - 6.8\lambda_1\lambda_2 + 8.75\lambda_1^3 - 1.2\lambda_2^3 \\ -1.23\lambda_1\lambda_2^2 + 8.32\lambda_1^2\lambda_2, \\ 2\pi \end{pmatrix}_{min}$$

Stiffened end plate factor where $\lambda_1 \leq 0.75$ and $\lambda_2 \geq 0.45$

$$\alpha = \begin{pmatrix} 1.25 + 39.33\lambda_1 - 3.58\lambda_2 - 55.94\lambda_1^2 + 40.54\lambda_2^2 - 55.34\lambda_1\lambda_2 + 21.05\lambda_1^3 - 33.00\lambda_2^3 \\ + 2.79\lambda_1\lambda_2^2 + 44.06\lambda_1^2\lambda_2, \\ 2\pi \end{pmatrix}_{\min}$$

Stiffened end plate factor where $\lambda_1 \leq 0.75$ and $\lambda_2 < 0.45$

b) Effective T-Stub Length: First Row of Bolts in Extension Without Gusset

$m_3 = a_f - 0.8t_{wf}$ Bolt distance from flange weld

$m = m_3$ Rows in endplate extension without gusset

$n = [a_{e1}, 1.25m_3]_{\min}$ Effective edge distance in endplate extension without gusset

$l_{er1} = [l_{e7}, l_{e8}, l_{e9}, l_{e10}, l_{e11}]_{\min}$ Top bolt row effective T-stub length

$l_{e7} = 0.5b_i$ Double curvature plate extension pattern

$l_{e8} = 2m_3 + 0.625a_{e1} + 0.5s_g$ Group end yielding plate extension pattern

$l_{e9} = 2m_3 + 0.625a_{e1} + e$ Corner yielding plate extension pattern

$l_{e10} = 4m_3 + 1.25a_{e1}$ Individual end yielding plate extension pattern

$l_{e11} = 2\pi m_3$ Circular yielding plate extension pattern

c) Effective T-Stub Length: First Row of Bolts in Extension With Gusset

$m_3 = a_f - 0.8t_{wf}$ Bolt distance from flange weld

$m_4 = \frac{s_g}{2} - \frac{t_{ig}}{2} - 0.8t_{wg}$ Bolt distance from gusset weld

$m = m_4$ Rows adjacent to endplate gusset

$n = [e, 1.25m_4]_{\min}$ Effective edge distance adjacent to gusset

$\lambda_1 = \frac{m_4}{m_4 + e}$ $\lambda_2 = \frac{m_3}{m_4 + e}$ Edge distance ratios

$l_{er1} = \left[l_{e1}, (l_{e2}, l_{e3})_{\max}, (l_{e5}, l_{e6})_{\max}\right]_{\min}$ Top bolt row effective T-stub length

$l_{e1} = 2\pi m_4$ Circular yielding pattern

$l_{e2} = 4m_4 + 1.25e$ Side yielding pattern

$l_{e3} = \alpha m_4$ Side yielding near flange pattern

$l_{e5} = 2m_4 + 0.625e + a_{e1}$ Corner yielding pattern

$l_{e6} = \alpha m_4 - (2m_4 + 0.625e) + a_{e1}$ Corner yielding near stiffener pattern

d) Effective T-Stub Length: Second Row of Bolts

$m_1 = \frac{s_g}{2} - \frac{t_w}{2} - 0.8t_{ww}$ Bolt distance from web weld

$m_2 = p_f - t_f - 0.8t_{wf}$ Bolt distance from flange weld

$m = m_1$ Rows adjacent to web

$n = [e, 1.25m_1]_{\min}$ Effective edge distance adjacent to web

$\lambda_1 = \dfrac{m_1}{m_1 + e} \qquad \lambda_2 = \dfrac{m_2}{m_1 + e}$ Edge distance ratios [8]

$l_{er2} = \lfloor (l_{e2}, l_{e3})_{\max}, l_{e1} \rfloor_{\min}$ Second row of two rows

$l_{er2} = \lfloor (l_{e2}, l_{e3})_{\max}, l_{e1}, (0.5 l_{e2}, (l_{e3} - 0.5 l_{e2}))_{\max} + 0.5 s_p \rfloor_{\min}$

Second row of three rows

$l_{e1} = 2\pi m_1$ Circular yielding pattern[9]

$l_{e2} = 4m_1 + 1.25e$ Side yielding pattern

$l_{e3} = \alpha m_1$ Side yielding near flange pattern

e) Effective T-Stub Length: Third Row of Bolts

$m = m_1$ Rows adjacent to web

$n = [e, 1.25m_1]_{\min}$ Effective edge distance adjacent to web

$l_{er3} = \lfloor l_{e1}, l_{e2}, 0.5(l_{e2} + s_p) \rfloor_{\min}$ Third bolt row effective T-stub length

$l_{e1} = 2\pi m_1$ Circular yielding pattern

$l_{e2} = 4m_1 + 1.25e$ Side yielding pattern

5. End Plate Design Transverse Strength Limits

$\phi V_i = \left[\phi V_{bi}; \phi V_{tti}; \phi V_{gsi}\right]_{\min}$ Plate shear

$\phi V_{bi} = n_{bb} \phi_s 3.2 f_{ui} d_f t_i$ Bolt hole 1st bearing

$\phi V_{tti} = n_{bb} \phi_s a_{e1} t_i f_{ui}$ Bolt hole 1st transverse tearing

$\phi V_{gsi} = 2\phi_s 0.5 f_{yi} d_i t_i$ Gross transverse shear yield

6. Web Design Strength Limits

$\phi V_{gsb} = 0.6\phi_s 0.6 d t_w f_{yw}$ Web shear yield with max. moment: HR

$\phi V_{gsb} = 0.6\phi_s 0.6(d - 2t_f) t_w f_{yw}$ Web shear yield with max. moment: Welded

7. Flange Fillet Weld Design Strength Limits

$\phi N_{wf} = 2\phi_w 0.6 f_{uw} \dfrac{t_{wf}}{\sqrt{2}} b_f$ Flange weld tension *(Admt 7/04)*

$\phi_w = 0.8$ SP fillet weld

8. Web Fillet Weld Design Strength Limits

$\phi N_{ww} = 2\phi_w 0.6 f_{uw} \dfrac{t_{ww}}{\sqrt{2}} (d - 2t_f)$ Web weld tension

$\phi V_{ww} = \phi N_{ww}$ Weld shear

$\phi_w = 0.8$ SP fillet weld

[8] SCI, "Joints in Steel Construction: Moment Connections", P207/95, Steel Construction Institute, UK, 1995, pp..23, 139

[9] ibid, p.19

9. Definitions of Terms

a_{e1}	Bolt edge end distance
$d_h = d_f + 2$ for $d_f \leq 24$	Bolt hole diameter
$d_h = d_f + 3$ for $d_f > 24$	Bolt hole diameter
$d_{r1} = d - 0.5t_f + a_f$	Bolt row 1 lever arm
$d_{r2} = d_{r1} - a_f - p_f$	Bolt row 2 lever arm
$d_{r3} = d_{r2} - s_p$	Bolt row 3 lever arm
$\phi_w = 0.8$	SP fillet weld
$\phi_b = 0.8$	Bolt strength reduction factor
$\psi_{r3} = \left[\dfrac{d_{r3}\phi N_{r2}}{d_{r2}\phi N_{r3}}, 1.0\right]_{min}$ for $t_i \geq \dfrac{d_f}{1.9}\sqrt{\dfrac{f_{uf}}{f_{yi}}}$	Bolt row force distribution triangular limit[10]
$\psi_{r3} = 1.0$ for $t_i < \dfrac{d_f}{1.9}\sqrt{\dfrac{f_{uf}}{f_{yi}}}$	Bolt row force plastic distribution

[10] SCI, “Joints in Steel Construction: Moment Connections”, P207/95, Steel Construction Institute, UK, 1995, p.25

A. MEPS-E Moment End Plate Extended Drawings

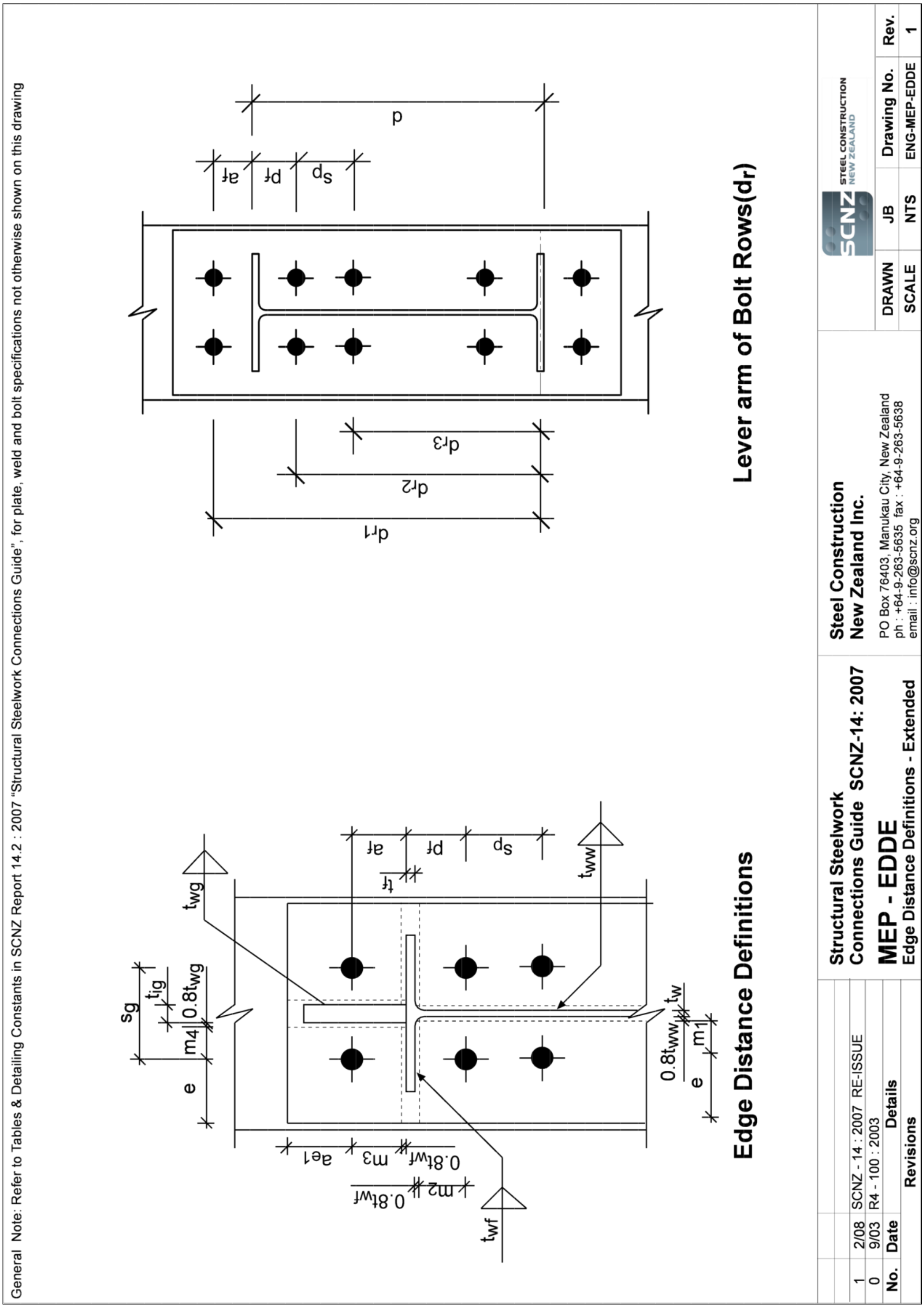


Figure 34 MEPS-E Extended moment end plate splice edge distances

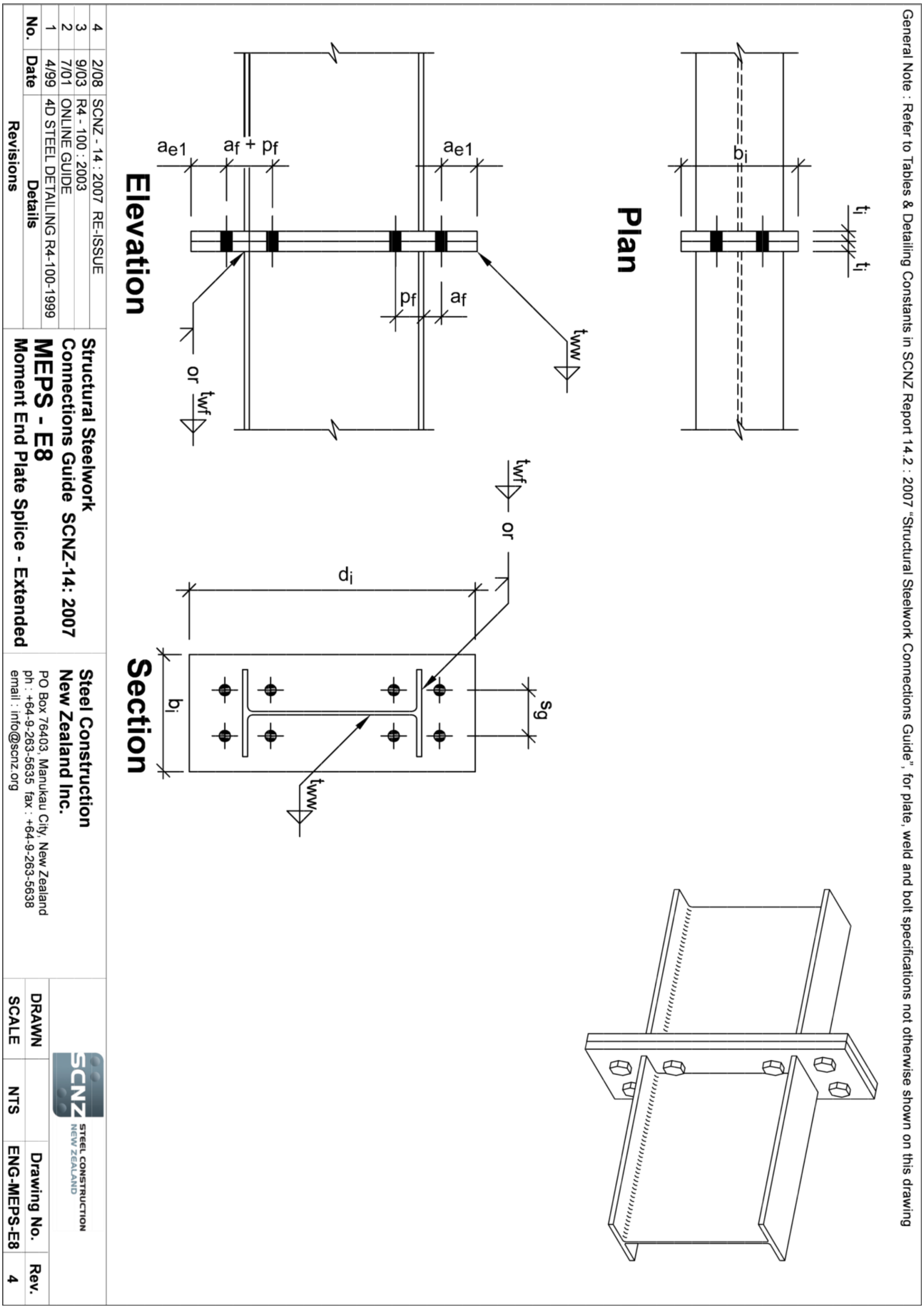


Figure 35 MEPS-E8 drawing

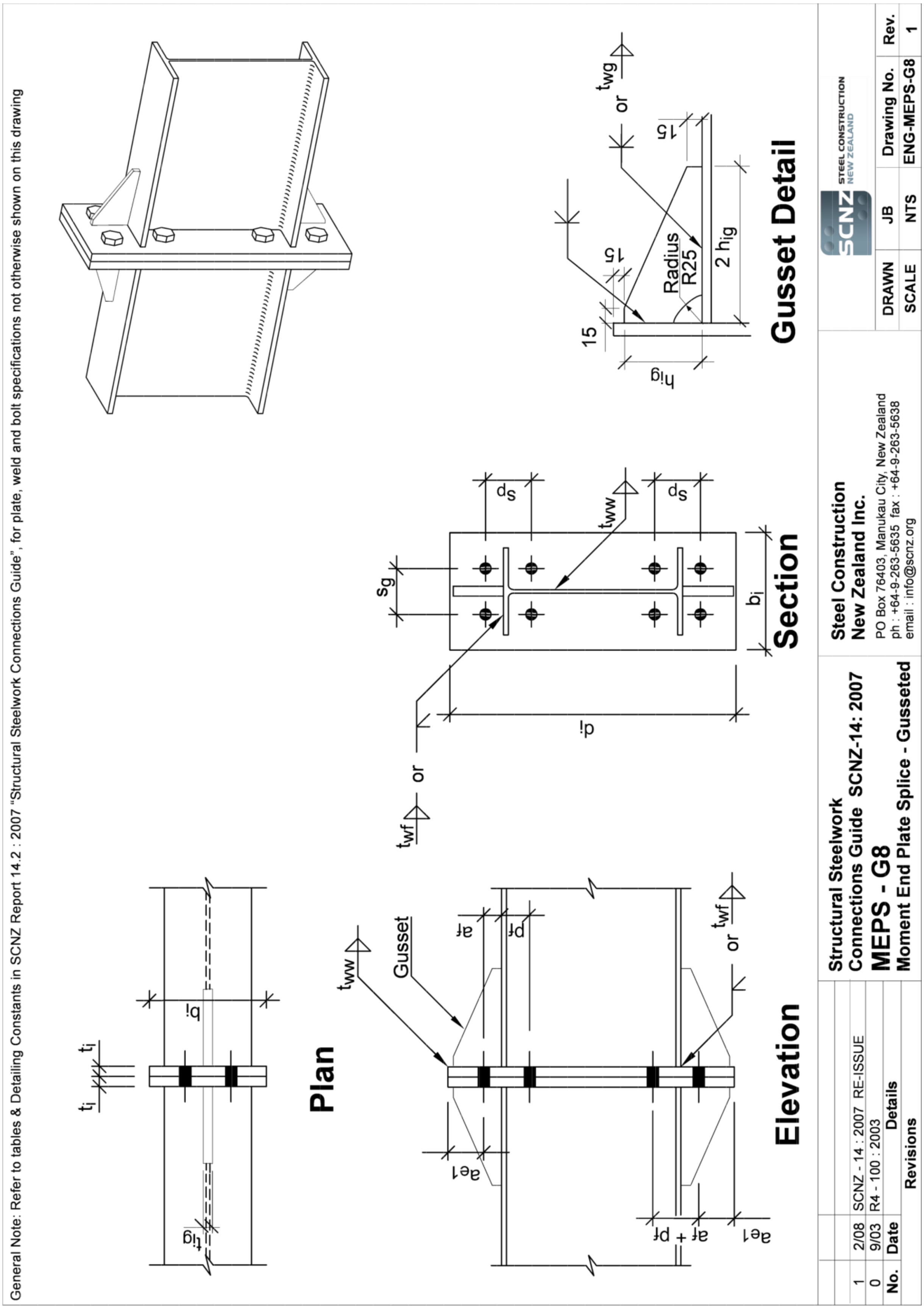


Figure 36 MEPS-G8 drawing

XIII. BWBS: Bolted Welded Beam Splice

A. Design Objectives

Possess design capacity to satisfy gravity and seismic design actions derived from relevant design or over-strength actions of primary members of seismic resisting frames.

The splices are located away from potential seismic yielding regions of the member.

Maintain ductile performance under fire restraint conditions.

B. Design Features

Typical limiting conditions are : Shear of the web cleat or bolts; Tension yield of the flange plates, shear of flange bolts or welds.

Doubly symmetrical I sections only.

All flexural actions are resisted by the flange plate couple.

All shear actions are resisted by the web cleat.

Flange and web cleats are welded to one side of the connection and bolted the other.

The eccentricity used for the design of the web bolt group, e , is less than that used for the design of the weld group, e_w . This recognises that the weld group is more rigid than the bolt group and the cleat may transfer shear across the splice in single curvature rather than the bolt group forcing double curvature, as in a splice bolted both sides.

Weld groups are designed in accordance with NZS3404:1997 cl 9.8.1.1, the "General method of analysis". Superposition of in -plane direct shears and moment resisting shear couples is used. Direct shear is applied at the weld centroid, and distributed evenly over the total weld length. The moment induced transverse and longitudinal shears are assumed to act at right angles to the x or y direction radius from that point to the weld group instantaneous centre of rotation, and are taken as proportional to that radius.

The calculation of net flexure ultimate design shear capacity of the web plates, ϕV_{nfi} , uses a similar approach used for the gross flexure yield design shear capacity, ϕV_{gfi} . However, $0.85 f_{ui}$ substitutes for f_{yi} , and the plastic section modulus of the net section is based on the gross section plastic modulus adjusted by the ratio of net area to gross area of the plate.

To maintain ductile behaviour under fire restraint and seismic overload conditions the welds to the flange plate are sized to have design capacity greater than the design tensile capacity of the plate. The flange bolt group is similarly designed to have design capacity greater than the flange or flange plate design tensile capacity. It is considered that the web cleat will be protected if the flange splice maintains integrity under seismic overload and fire events.

C. Design Procedure: Flange Splice

1. Governing criteria

The minimum bolt group, flange plate, section and weld design strength limits shall exceed the applied design load.
Flange plate widths to satisfy bolt edge distance criteria.
Flange plate compactness
Flange welds stronger than flange or flange plate
Flange bolt group stronger than flange or flange plate

2. Connection Design Strength Limits

Splice design tension capacity.

3. Bolt Group Design Strength Limits

Bolt shear resisting resultant flange tension or compression.

4. Flange Plate Design Strength Limits

Bolt hole 1^{st} bearing.
Bolt hole 1^{st} longitudinal tearing.
Plate gross and net tension.

5. Section Design Strength Limits

Bolt hole 1^{st} bearing.
Bolt hole 1^{st} longitudinal tearing.
Flange gross and net tension.

6. Weld Design Strength Limits

Weld tension

D. Design Procedure: Web Splice

1. Governing Criteria

Design shear
Weld group resultant shear per mm

2. Design Actions

Design shear
Resultant weld group design shear per mm at toe
x-direction weld group design shear per mm at toe
y-direction weld group design shear per mm at toe

3. Connection Design Strength Limits

Web splice design shear capacity

4. Bolt Group Design Strength Limits

Transverse shear at eccentricity e .

5. Web Plate Design Strength Limits

Bolt hole 1^{st} resultant bearing.
Bolt hole 1^{st} transverse tearing.
Bolt hole 1^{st} longitudinal tearing.
Plate gross and net shear.
Plate gross and net flexure.
Plate shear / flexure interaction.

6. Section Web Design Strength Limits

Bolt hole 1^{st} resultant bearing.
Bolt hole 1^{st} transverse tearing.
Bolt hole 1^{st} longitudinal tearing.

7. Weld Design Strength Limits

Weld resultant shear.

E. Design Formulae: Flange Splice

1. Governing Criteria

$N^*_{ft} \le \phi N_{ft}$	Tension
$[a_{e1}; a_{e3}] \ge 1.75 d_f$	Edge distance: manual flame cut or crop
$b_{if} \ge s_g + 3.0 d_f$	Flange plate width: HR or CNC flame cut
$\lvert b_{if} - b_f \rvert \ge 2 t_{wf}$	Flange plate width: weld width
$b_f \ge s_g + 2.5 d_f$	Hot rolled section flange width
$b_f \ge s_g + 3.0 d_f$	BHP welded section flange width
$t_{wf} \le t_{if} - 1$	Fillet weld height limit
$\dfrac{s_g}{t_{if}} \le 17.5$	Flange plate compactness
$\phi N_b \ge \lfloor \phi N_{ti}, \phi N_f \rfloor_{\min}$	Flange bolt group fire ductility
$\phi N_{wf} \ge \phi N_{ti}$	Flange weld group fire ductility

2. Design Actions

$$N^*_{ft} = \frac{M^*}{d - t_f}$$ Flange splice design tension

3. Connection Design Strength Limits

$$\phi N_{ft} = [\phi N_b; \phi N_{ti}; \phi N_f; \phi N_{wf}]_{\min}$$ Flange splice design tension capacity

4. Bolt Design Strength Limits

$$\phi N_b = 2 n_{pf} \phi_b V_{fn}$$ Flange bolt group tension

5. Flange Plate Design Strength Limits

$$\phi N_{ti} = [\phi N_{bi}; \phi N_{lti}; \phi N_{gti}; \phi N_{nti}]_{\min}$$ Flange cleat tension

$\phi N_{bi} = 2 n_{pf} \phi_s 3.2 t_{if} d_f f_{ui}$	Bolt hole 1st bearing
$\phi N_{lti} = 2 n_{pf} \phi_s a_{exi} t_{if} f_{ui}$	Bolt hole 1st longitudinal tearing.
$\phi N_{gti} = \phi_s f_{yfi} A_{gfi}$	Gross tension yield.
$\phi N_{nti} = \phi_s 0.85 f_{ui} A_{nfi}$	Net tension ultimate.

6. Section Design Strength Limits

$$\phi N_f = [\phi N_{bf}; \phi N_{ltf}; \phi N_{gtf}; \phi N_{ntf}]_{\min}$$ Flange tension

$\phi N_{bf} = 2 n_{pf} \phi_s 3.2 t_f d_f f_{ub}$	Bolt hole 1st bearing
$\phi N_{ltf} = 2 n_{pf} \phi_s a_{exb} t_f f_{ub}$	Bolt hole 1st longitudinal tearing
$\phi N_{gtf} = \phi_s f_{yf} t_f b_f$	Flange gross tension.
$\phi N_{ntf} = \phi_s 0.85 f_{ub} A_{nf}$	Flange net tension.

7. Weld Design Strength Limits

$\phi N_{wf} = \phi_w 0.6 f_{uw} \dfrac{t_{wf}}{\sqrt{2}} L_{w3}$ Weld group tension

$\phi_w = 0.8$ SP fillet weld

8. Definition of Terms

$a_{exb} = \left[a_{e2}, a_{e3}\right]_{\min}$ $a_{exi} = \left[a_{e1}, a_{e2}\right]_{\min}$ Edge distances

$a_{e2} = s_{pf} - \dfrac{d_h}{2}$ Inter-bolt edge distance

$a_{e3} = 0.6 d_{if} - a_{e1} - s_{pf}\left(n_{pf} - 1\right) - c$ Flange edge distance

$A_{nf} = t_f\left(b_f - n_g d_h\right)$ Net flange area

$A_{nfi} = t_{if}\left(b_{if} - n_g d_h\right)$ Net flange cleat area

$A_{gfi} = t_{if} b_{if}$ Gross flange cleat area

$L_{w3} = 0.8 d_{if} + \left[b_f ; b_{if}\right]_{\min}$ Flange 3-sided weld length

F. Formulae: Web Splice

1. Governing Criteria

$V^* \leq \left[\phi V_b ; \phi V_i ; \phi V_{wb}\right]_{\min}$ Shear

$v^*_{wwres} \leq \phi v_{ww}$ Weld resultant shear per mm

$t_{ww} \leq t_{iw} - 1$ Web fillet weld height

$d_{iw} \geq 0.4 d$ Web cleat depth

2. Design Actions

V^* Design shear

$v^*_{wwres} = \sqrt{{v^*_x}^2 + {v^*_y}^2}$ Resultant weld group design shear per mm at toe

$v^*_x = -\dfrac{V^* e_w d_{iw}}{2 i_{wp}}$ x-direction weld group design shear per mm at toe

$v^*_y = \dfrac{V^*}{d_{iw} + 2 b_{ww}} + \dfrac{V^* e_w x_{ct}}{i_{wp}}$ y-direction weld group design shear per mm at toe

3. Connection Design Strength Limits

$\phi V_{con} = V^*_{\max}$ Web splice design shear capacity

4. Bolt Design Strength Limits

$\phi V_b = \phi V_f Z_b$ Bolt group shear at eccentricity e

$\phi V_f = \phi_b V_{fx}$ if $\left[t_{iw} ; t_w\right]_{\max} \leq 9.0$ mm Threads excluded strength

$\phi V_f = \phi_b V_{fn}$ if $\left[t_{iw} ; t_w\right]_{\max} > 9.0$ mm Threads included strength

5. Web Plate Design Strength Limits

$$\phi V_i = \left[\phi V_{bi} ; \phi V_{tti} ; \phi V_{lti} ; \phi V_{gfi} ; \phi V_{nfi} ; \phi V_{gsi} ; \phi V_{nsi}\right]_{\min}$$

$\phi V_{bi} = \phi_s Z_b 3.2 t_{iw} d_f f_{ui}$	Bolt hole 1st resultant bearing
$\phi V_{tti} = n_{pw} \phi_s a_{eyi} t_{iw} f_{ui}$	Bolt hole 1st transverse tearing
$\phi V_{lti} = n_{pw} \phi_s Z_e a_{e5} t_{iw} f_{ui}$	Bolt hole 1st tearing along section
$\phi V_{gfi} = \phi_s \dfrac{t_{iw} d_{iw}{}^2}{4e} f_{ywi}$	Gross flexure yield
$\phi V_{nfi} = \phi_s \left(1 - \dfrac{n_{pw} d_h}{d_{iw}}\right) \dfrac{t_{iw} d_{iw}^2}{4e} 0.85 f_{ui}$	Net flexure ultimate
$\phi V_{gsi} = \phi_s 0.5 t_{iw} d_{iw} f_{ywi}$ if $V^* \le 0.75 \phi V_{gfi}$	Gross shear yield.

$$\phi V_{gsi} = \phi_s 0.5 t_{iw} d_{iw} f_{ywi} \left[2.2 - \left(\frac{16V^*}{\phi V_{gfi}}\right)\right] \text{ if } V^* \ge 0.75 \phi V_{gfi}$$

Gross shear / flexure interaction

$\phi V_{nsi} = \phi_s 0.6 f_{ui} A_{nwi}$ if $V^* \le 0.75 \phi V_{nfi}$	Net shear ultimate

$$\phi V_{nsi} = \phi_s 0.6 f_{ui} A_{nwi} \left[2.2 - \left(\frac{16V^*}{\phi V_{nfi}}\right)\right] \quad \text{if } V^* \ge 0.75 \phi V_{nfi}$$

Net shear / flexure ultimate interaction

6. Section Web Design Strength Limits

$\phi V_{wb} = \left[\phi V_{bw} ; \phi V_{ttw} ; \phi V_{ltw} ; \phi V_{gsb}\right]_{\min}$	Web shear
$\phi V_{bw} = \phi_s Z_b 3.2 t_w d_f f_{ub}$	Bolt hole 1st resultant bearing
$\phi V_{ttw} = n_{pw} \phi_s a_{eyb} t_w f_{ub}$	Bolt hole 1st transverse tearing.
$\phi V_{ltw} = n_{pw} \phi_s Z_e a_{e4} t_w f_{ub}$	Bolt hole 1st tearing along section.
$\phi V_{gsb} = \phi_s 0.6 d t_w f_{yw}$	Gross shear yield: HR sections
$\phi V_{gsb} = \phi_s 0.6 (d - 2t_f) t_w f_{yw}$	Gross shear yield: Welded sections

7. Weld Design Strength Limits

$\phi v_{ww} = \phi_w 0.6 f_{uw} \dfrac{t_{ww}}{\sqrt{2}}$	Web weld group design capacity per mm

8. Definition of Terms

$a_{eyb} = a_{e7} \qquad a_{eyi} = \left[a_{e6}, a_{e7}\right]_{\min}$	Edge distances
$a_{e6} = \dfrac{d_{iw} - (n_{pw} - 1) s_{pw}}{2}$	Bolt edge y-distance
$a_{e7} = s_{pw} - \dfrac{d_h}{2}$	Inter-bolt edge distance
$A_{nwi} = (d_{iw} - n_{pw} d_h) t_{iw}$	Single web plate net area
$b_{ww} = b_{iw} - c - a_{e4} - a_{e5}$	Web weld horizontal width
$d_h = d_f + 2$ for $d_f \le 24$	Hole diameter.
$d_{iw} = s_{pw} (n_{pw} - 1) + 2a_{e6}$	Depth of cleat

$e = a_{e4} + \frac{c}{2}$ Bolt group eccentricity

$e_w = x_{ct} + c + a_{e4}$ Weld group eccentricity

$i_{wp} = \frac{d_{iw}^2}{12}\left(6b_{ww} + d_{iw}\right) + \frac{b_{ww}^3}{3}\frac{\left(b_{ww} + 2d_{iw}\right)}{\left(2b_{ww} + d_{iw}\right)}$ Weld group polar moment of inertia

$x_{ct} = \frac{{b_{ww}}^2 + d_{iw} b_{ww}}{2b_{ww} + d_{iw}}$ x-distance from weld centroid to toe

$Z_b = \frac{n_{pw}}{\sqrt{1 + \left[\frac{6e}{s_{pw}\left(n_{pw} + 1\right)}\right]^2}}$ Bolt interaction factor for single line of bolt, $n_{pw} \neq 1$

$Z_e = \frac{s_{pw}\left(n_{pw} + 1\right)}{6e}$ Bolt group flexure factor, single line of bolts, $n_{pw} \neq 1$

G. BWBS Bolted Welded Beam Splice Drawings

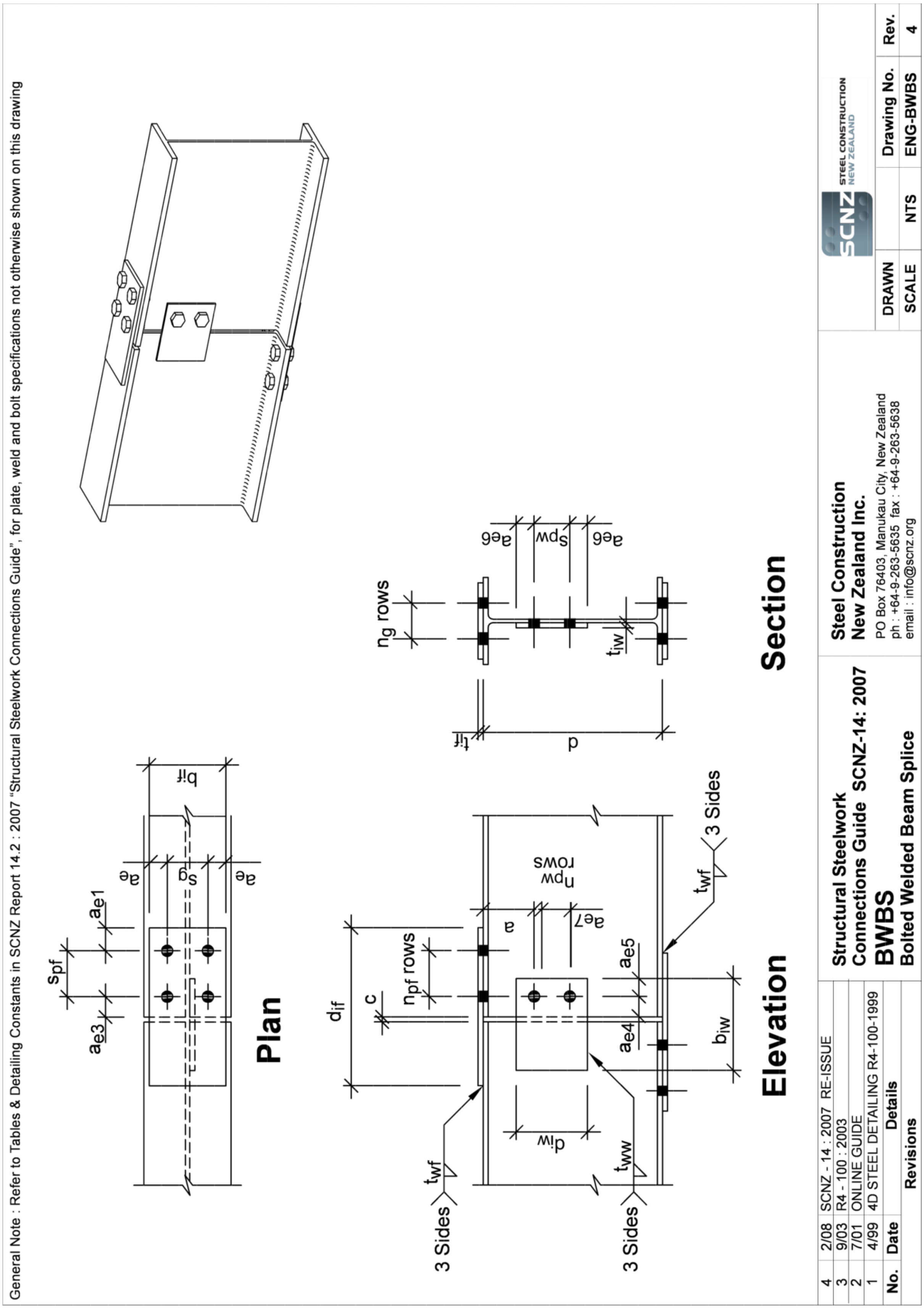


Figure 37 BWBS drawing

XIV. BBS: Bolted Beam Splice

A. Design Objectives

Possess design capacity to satisfy gravity and seismic design actions derived from relevant design or over-strength actions of primary members of seismic resisting frames.

The splices are located away from potential seismic yielding regions of the member.

Maintain ductile performance under fire restraint conditions.

B. Design Features

Typical limiting conditions are : Shear of the web cleat or bolts; Tension yield of the flange plates, shear of flange bolts.

Doubly symmetrical I sections only.

All flexural actions are resisted by the flange plate couple.

All shear actions are resisted by the web splice.

Pairs of web cleats are set out central to and either side of the beam section web of BBS1 and BBS2 connections. A single row of web bolts is used each side of the splice.

The calculation of net flexure ultimate design shear capacity of the web plates, ϕV_{nfi} , uses a similar approach used for the gross flexure yield design shear capacity, ϕV_{gfi} . However, $0.85 f_{ui}$ substitutes for f_{yi} , and the plastic section modulus of the net section is based on the gross section plastic modulus adjusted by the ratio of net area to gross area of the plate.

Bolt gauge is set out from the centre-line of the plates and flanges.

Single flange plates located on the outer face of the beam flanges are used for BBS1 connections. For BBS2 connections, flange plates of equal thickness are located on both faces of the beam flanges.

To maintain ductile behaviour under fire restraint and seismic overload conditions the flange bolt group is designed to have design capacity greater than the flange or flange plate design tensile capacity. It is considered that the web cleat will be protected if the flange splice maintains integrity under seismic overload and fire events.

To minimise slip splices are detailed with a minimum of two rows of bolts in the flanges, each side of the splice. The flange splice is designed to resist a minimum serviceability load of 15% of section moment capacity by the flange bolts in friction tension mode. A slip factor, μ_s of 0.45, in accordance with recommendations of BS4604: Part 1: 1970 is appropriate for unpainted steel without mill scale. For inorganic zinc silicate and alkyd primer a slip factor, μ_s, of 0.5 and 0.11 respectively are appropriate.

C. Design Procedure: Flange Splice

1. Governing criteria

The minimum bolt group, flange plate and section design strength limits shall exceed the applied design load.

Flange plate widths to satisfy bolt edge distance criteria.
Flange plate compactness
Flange bolt rows
Flange bolt group stronger than flange or flange plate

2. Design Actions

Design moment
Resultant flange splice tension.

3. Connection Design Strength Limits

Splice design tension capacity.

4. Bolt Group Design Strength Limits

Bolt shear resisting resultant flange tension or compression.

5. Bolt Group Design Serviceability Limits

Bolt shear resisting resultant flange tension or compression.

6. Flange Plate Design Strength Limits

a) BBS1: Single Side Flange Plates

Bolt hole 1st bearing
Bolt hole 1st longitudinal tearing
Plate gross and net tension

b) BBS2: Double Side Flange Plates

Bolt hole 1st bearing
Bolt hole 1st longitudinal tearing
Plate gross and net tension, limited by bolt shear transfer between plate and section

7. Section Design Strength Limits

Bolt hole 1st bearing.
Bolt hole 1st longitudinal tearing.
Flange gross and net tension.

D. Design Procedure: Web Splice

1. Governing Criteria

Design shear

2. Design Actions

Design shear

3. Connection Design Strength Limits

Web splice design shear capacity

4. Bolt Group Design Strength Limits

Double shear / bolt
Transverse shear at eccentricity e.

5. Bolt Group Design Serviceability Limits

Double shear / bolt
Transverse shear at eccentricity e.

6. Web Plate Design Strength Limits

Bolt hole 1st resultant bearing.
Bolt hole 1st transverse tearing.
Bolt hole 1st longitudinal tearing.
Plate gross and net shear.
Plate gross and net flexure.
Plate shear / flexure interaction.

7. Section Web Design Strength Limits

Bolt hole 1st resultant bearing.
Bolt hole 1st transverse tearing.
Bolt hole 1st longitudinal tearing.

E. Design Formulae: Flange Splice

1. Governing Criteria

$N^*_{ft} \le \phi N_{ft}$	Flange splice design tension
$N^*_{fslip} \le \phi N_{bs}$	Flange splice slip
$[a_{e1}; a_{e3}] \ge 1.75 d_f$	Edge distance: manual flame cut or crop
$b_{if} \ge s_g + 3.0 d_f$	Flange plate width: HR or CNC flame cut
$2.5 d_f \le b_{i2} \le s_g - 2r - t_w$	Flange doubler plate width: HR flat
$b_f \ge s_g + 2.5 d_f$	Hot rolled section flange width
$b_f \ge s_g + 3.0 d_f$	BHP welded section flange width
$\frac{s_g}{t_{if}} \le 17.5$	Flange plate compactness
$n_{pf} \ge 2$	Flange bolt rows
$\phi N_b \ge [\phi N_{ti}, \phi N_f]_{min}$	Flange bolt group fire ductility

2. Design Actions

$N^*_{ft} = \frac{M^*}{d - t_f}$	Flange splice design tension
$N^*_{fslip} = \frac{0.15 \phi M_s}{d - t_f}$	Flange splice slip tension

3. Connection Design Strength Limits

$\phi N_{ft} = [\phi N_b; \phi N_{ti}; \phi N_f]_{min}$	Flange splice design tension capacity

4. Bolt Design Strength Limits

a) BBS1: Single Side Flange Plates

$\phi N_b = 2 n_{pf} \phi V_{fn}$	Flange bolt group tension

b) BBS2: Double Side Flange Plates

$\phi N_b = 2n_{pf} 2\phi V_{fn}$ Flange bolt group tension

5. Bolt Design Serviceability Limits

$\phi N_{bs} = 2n_{pf} \phi V_{sf}$ Flange bolt group slip

$\phi V_{sf} = \phi_{slip} \mu_s n_{ei} N_{ptf} k_h$ Bolt slip shear capacity

6. Flange Plate Design Strength Limits

a) BBS1: Single Side Flange Plates

$\phi N_{ti} = \left[\phi N_{bi}; \phi N_{lti}; \phi N_{gti}; \phi N_{nti}\right]_{\min}$ BBS1 Flange plate tension

$\phi N_{bi} = \phi_s 2n_{pf} 3.2 t_{if} d_f f_{ui}$ Bolt hole 1st bearing

$\phi N_{lti} = \phi_s 2n_{pf} a_{exi} t_{if} f_{ui}$ Bolt hole 1st longitudinal tearing

$\phi N_{gti} = \phi_s f_{yfi} A_{gfi}$ Gross tension yield.

$\phi N_{nti} = \phi_s 0.85 f_{ui} A_{nfi}$ Net tension ultimate.

b) BBS2: Double Side Flange Plates

$\phi N_{ti} = \left[\phi N_{bi}; \phi N_{lti}; \phi N_{gnti}\right]_{\min}$ BBS2 Flange plates tension

$\phi N_{bi} = 2\phi_s 2n_{pf} 3.2 t_{if} d_f f_{ui}$ Bolt hole 1st bearing

$\phi N_{lti} = 2\phi_s 2n_{pf} a_{exi} t_{if} f_{ui}$ Bolt hole 1st longitudinal tearing

$\phi N_{gnti} = \phi N_{ti1} + \phi N_{ti2}$ Total plates gross / net tension

$\phi N_{ti1} = \left[\phi N_{gti1}; \phi N_{nti1}; \phi N_b / 2\right]_{\min}$ Outer plate gross / net tension

$\phi N_{ti2} = \left[\phi N_{gti2}; \phi N_{nti2}; \phi N_b / 2\right]_{\min}$ Doubler plates gross / net tension

$\phi N_{gti1} = \phi_s f_{yfi} A_{gfi}$ Outer plate gross tension yield

$\phi N_{nti1} = \phi_s 0.85 f_{ui} A_{nfi}$ Outer plate net tension ultimate

$\phi N_{gti2} = \phi_s f_{yfi} A_{gi2}$ Doubler plates gross tension yield

$\phi N_{nti2} = \phi_s 0.85 f_{ui} A_{ni2}$ Doubler plates net tension ultimate

7. Section Design Strength Limits

$\phi N_f = \left[\phi N_{bf}; \phi N_{ltf}; \phi N_{gtf}; \phi N_{ntf}\right]_{\min}$ Flange tension

$\phi N_{bf} = \phi_s 2n_{pf} 3.2 t_f d_f f_{ub}$ Bolt hole 1st bearing

$\phi N_{ltf} = \phi_s 2n_{pf} a_{exb} t_f f_{ub}$ Bolt hole 1st longitudinal tearing

$\phi N_{gtf} = \phi_s f_{yf} t_f b_f$ Flange gross tension.

$\phi N_{ntf} = \phi_s 0.85 f_{ub} A_{nf}$ Flange net tension.

F. Formulae: Web Splice

1. Governing Criteria

$V^* \le \phi V_{con}$ Web splice gross / net shear

$V^*_{slip} \le \phi V_{fs}$	Web splice slip
$d_{iw} \ge 0.4d$	Web cleat depth

2. Design Actions

V^*	Design shear

3. Connection Design Strength Limits

$$\phi V_{con} = \lfloor \phi V_{fb}; \phi V_{gsb}; \phi V_{gfi}; \phi V_{nfi}; \phi V_{gsi}; \phi V_{nsi}; \phi V_{ls}; \phi V_{ts} \rfloor_{\min}$$

Splice shear

$\phi V_{fb} = Z_b \left[\phi V_f ; \phi V_{bi} ; \phi V_{bw}\right]_{\min}$	Resultant bolt shear and bearing / bolt
$\phi V_{ts} = n_{pw} \left[\phi V_{tti} ; \phi V_{ttw}\right]_{min}$	Transverse tearing / bolt
$\phi V_{ls} = n_{pw} Z_{el} \left[\phi V_{lti} ; \phi V_{ltw}\right]_{min}$	Longitudinal tearing / bolt

4. Bolt Design Strength Limits

$\phi V_f = 2\phi_b V_{fn}$	Double shear / bolt

5. Web Plate Design Strength Limits

$\phi V_{gfi} = 2\phi_s \dfrac{t_{iw} d_{iw}{}^2}{4e} f_{ywi}$	Gross flexure yield
$\phi V_{nfi} = 2\phi_s \left(1 - \dfrac{n_{pw} d_h}{d_{iw}}\right) \dfrac{t_{iw} d_{iw}^2}{4e} 0.85 f_{ui}$	Net flexure ultimate
$\phi V_{gsi} = 2\phi_s 0.5 t_{iw} d_{iw} f_{ywi}$ if $V^* \le 0.75\phi V_{gfi}$	Gross shear yield.

$$\phi V_{gsi} = 2\phi_s 0.5 t_{iw} d_{iw} f_{ywi} \left[2.2 - \left(\frac{16V^*}{\phi V_{gfi}}\right)\right] \quad \text{if } V^* > 0.75\phi V_{gfi}$$

Gross shear / flexure interaction

$\phi V_{nsi} = 2\phi_s 0.6 f_{ui} A_{nwi}$ if $V^* \le 0.75\phi V_{nfi}$	Net shear ultimate

$$\phi V_{nsi} = 2\phi_s 0.6 f_{ui} A_{nwi} \left[2.2 - \left(\frac{16V^*}{\phi V_{nfi}}\right)\right] \quad \text{if } V^* > 0.75\phi V_{nfi}$$

Net shear / flexure ultimate interaction

$\phi V_{bi} = 2\phi_s 3.2 f_{ui} d_f t_{iw}$	Bolt hole 1st resultant bearing.
$\phi V_{tti} = 2\phi_s a_{ey} t_{iw} f_{ui}$	Bolt hole 1st transverse tearing.
$\phi V_{lti} = 2\phi_s a_{exwi} t_{iw} f_{ui}$	Bolt hole 1st longitudinal tearing.

6. Section Web Design Strength Limits

$\phi V_{gsb} = \phi_s 0.6 d t_w f_{yw}$	Gross shear yield: HR sections
$\phi V_{gsb} = \phi_s 0.6 (d - 2t_f) t_w f_{yw}$	Gross shear yield: Welded sections
$\phi V_{bw} = \phi_s 3.2 f_{ub} d_f t_w$	Bolt hole 1st resultant bearing.
$\phi V_{ttw} = \phi_s a_{ey} t_w f_{ub}$	Bolt hole 1st transverse tearing.
$\phi V_{ltw} = \phi_s a_{e4} t_w f_{ub}$	Bolt hole 1st longitudinal tearing.

G. Definition of Terms

$a_{exb} = \left[a_{e2}, a_{e3}\right]_{\min}$ $\quad a_{exi} = \left[a_{e1}, a_{e2}\right]_{\min}$	Edge distances
$a_{exwi} = \left[a_{e5}; a_{e8}\right]_{\min}$ $\quad a_{eyi} = \left[a_{e6}; a_{e7}\right]_{\min}$	Edge distance
$a_{e2} = s_{pf} - \frac{d_h}{2}$	Inter-bolt edge distance
$a_{e3} = \frac{s_{g1} - c}{2}$	Flange edge distance
$a_{e4} = \frac{s_{g2} - c}{2}$	Web edge distance
$a_{e5} = \frac{b_{iw} - s_{g2}}{2}$	Web cleat x-distance
$a_{e6} = \frac{d_{iw} - \left(n_{pw} - 1\right)s_{pw}}{2}$	Web cleat edge y-distance
$a_{e7} = s_{pw} - \frac{d_h}{2}$	Inter-bolt edge y-distance
$a_{e8} = s_{g2} - \frac{d_h}{2}$	Inter-bolt edge x-distance
$A_{gf} = t_f b_f$	Gross flange area
$A_{gfi} = t_{if} b_{if}$	Gross outer flange plate area
$A_{gi2} = 2t_{if} b_{i2}$	Gross doubler flange plates area
$A_{nf} = t_f \left(b_f - n_g d_h\right)$	Net flange area
$A_{nfi} = t_{if} \left(b_{if} - n_g d_h\right)$	Net outer flange plate area
$A_{ni2} = t_{if} \left(2b_{i2} - n_g d_h\right)$	BBS2 net doubler flange plates area
$A_{nwi} = \left(d_{iw} - n_{pw} d_h\right) t_{iw}$	Web single plate net area
$e = 2a_{e4} + c$	Eccentricity
$Z_b = \dfrac{n_{pw}}{\sqrt{1 + \left[\dfrac{6e}{s_{pw}\left(n_{pw} + 1\right)}\right]^2}}$	Eccentric bolt group resultant action factor
$Z_{el} = \dfrac{s_{pw}\left(n_{pw} + 1\right)}{6e}$	Eccentric bolt group long. action factor, single line of bolts, $n_{pw} \neq 1$
$\phi_{slip} = 0.7$	Slip strength reduction factor
μ_s	Slip factor
n_{ei}	Number of effective interfaces
N_{ptf}	Minimum bolt tension at installation
$k_h = 1.0$	Factor for standard bolt hole

H. BBS Bolted Beam Splice Drawings

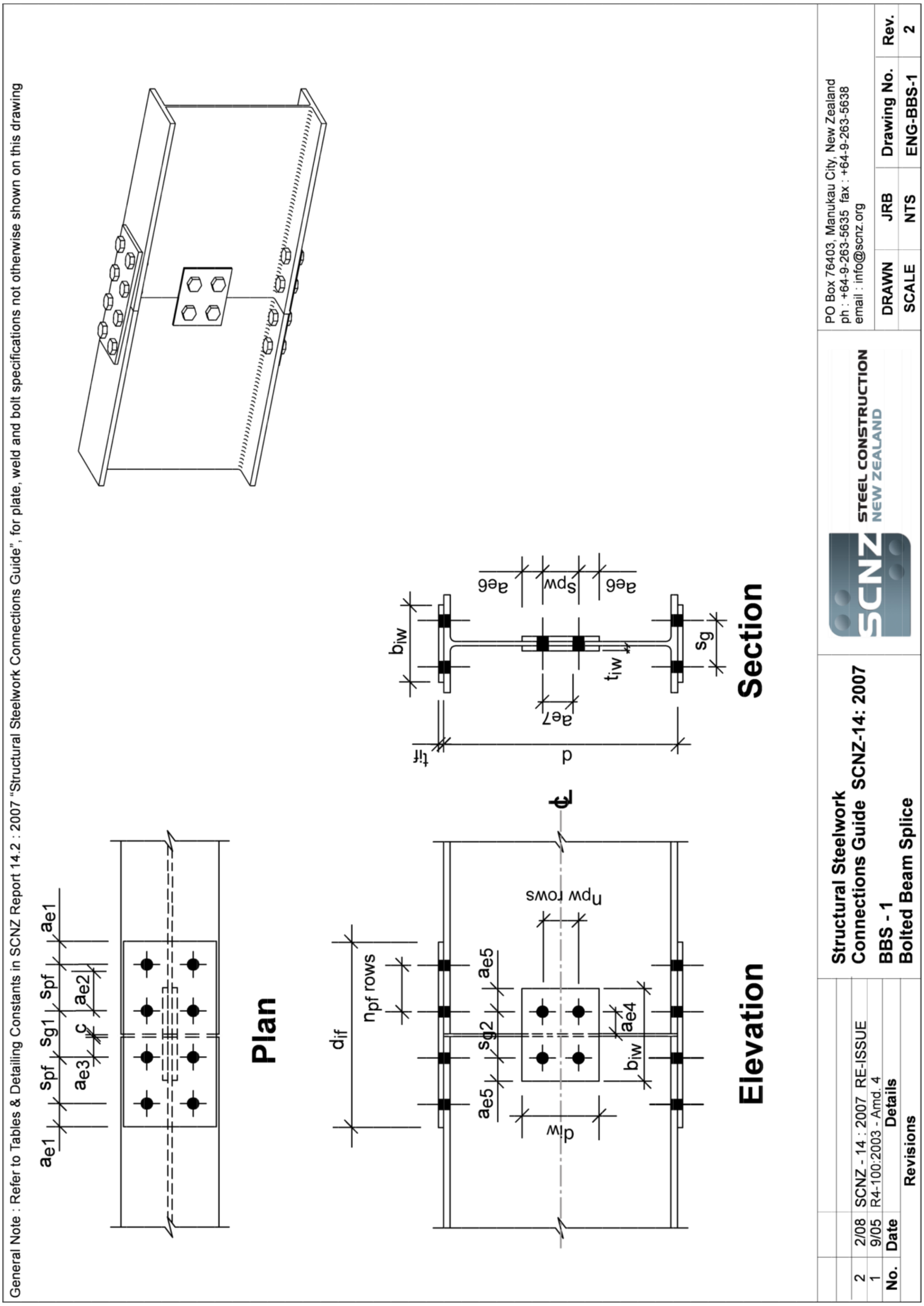


Figure 38 BBS-1 drawing

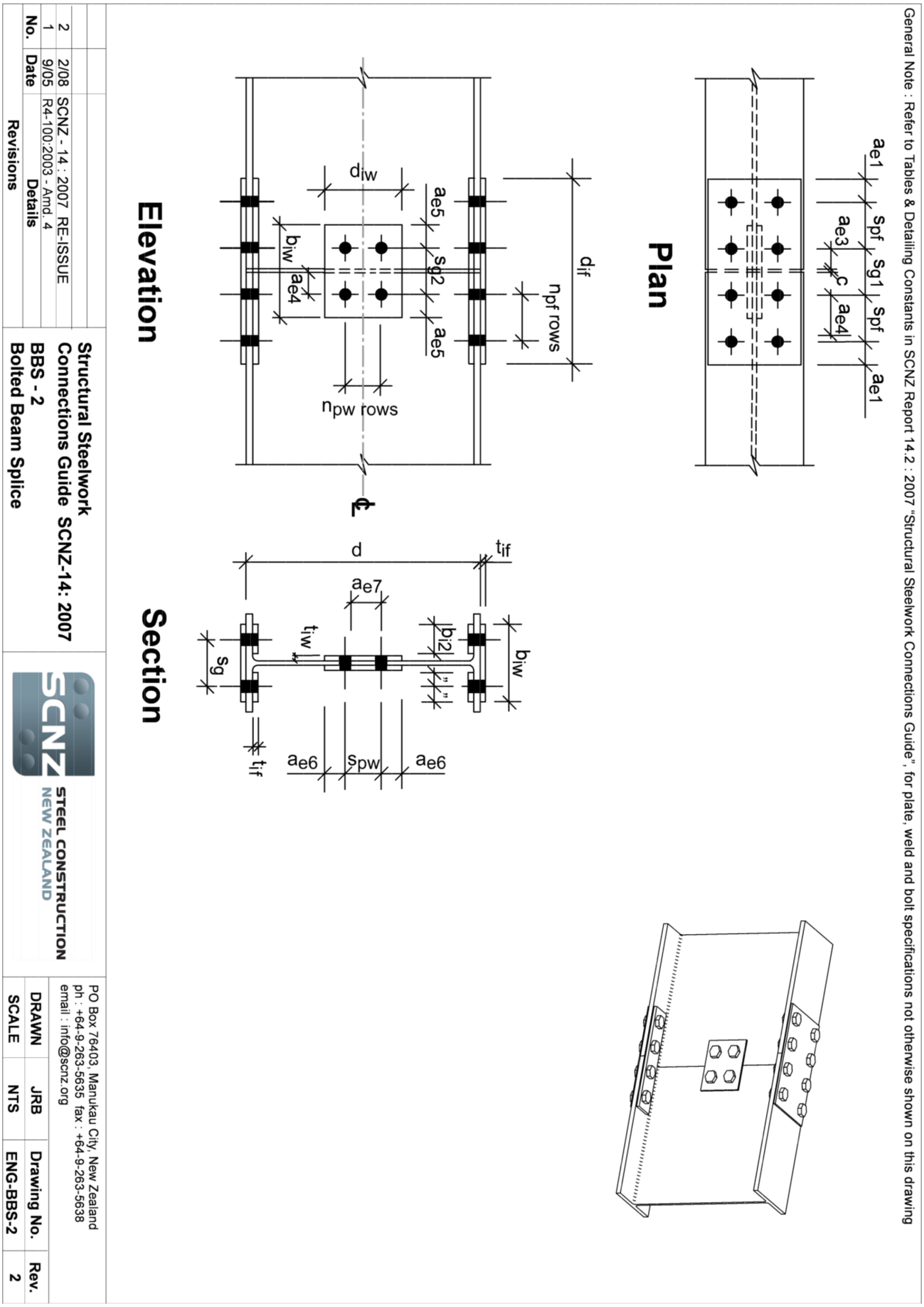


Figure 39 BBS-2 drawing

XV. BCS: Bolted Compression Splice

A. Design Objectives

Possess design capacity to satisfy gravity and seismic actions based on design or capacity design derived seismic axial compression, moment and shear ultimate limit state loads.

The splices are located away from potential seismic yielding regions of the member and are assumed to be protected by appropriate passive fire protection.

B. Design Features

Typical limiting conditions are : Shear of the web cleat or bolts; Tension yield of the flange plates, shear of flange bolts.

The splices are assumed to be located away from potential column yielding regions, at places of minimum ductility demand.

All resultant tensile forces from flexural actions are resisted by the flange plate couple.

All shear actions are resisted by the web splice.

All compression load resulting from axial and flexural actions is resisted by direct full contact end bearing of the abutting sections.

Pairs of web cleats are set out central to and either side of the column section web of BCS1 and BCS2 connections. A single row of web bolts is used each side of the splice.

The web cleat depth is not less than half the section depth.

The calculation of net flexure ultimate design shear capacity of the web plates, ϕV_{nfi} , uses a similar approach used for the gross flexure yield design shear capacity, ϕV_{gfi} . However $0.85f_{ui}$ substitutes for f_{yi} , and the plastic section modulus of the net section is based on the gross section plastic modulus adjusted by the ratio of net area to gross area of the plate.

Bolt gauge is set out from the centre-line of the plates and flanges.

Single flange plates located on the outer face of the column flanges are used for BCS1 connections. For BCS2 connections, flange plates of equal thickness are located on both faces of the column flanges.

(Admt 11/05) For stiffness compatibility of the splice with the connected sections, the combined thickness of splice plates should typically be not less than half the thickness of the web or flange spliced.

A minimum loading condition is that the fasteners, splice plates and section shall be sufficient to transmit a force equal to 15% of the section design capacity in direct compression only.

(Admt 11/05) To minimise slip during construction flange splices are detailed with a minimum of two rows of bolts in the flanges, each side of the splice. The flange splice is designed to resist a minimum serviceability load of 15% of section moment capacity by the flange bolts in friction tension mode. A slip factor, μ_s of 0.45, in accordance with recommendations of BS4604: Part 1: 1970 is appropriate for unpainted steel without mill scale. For inorganic zinc silicate and alkyd primer a slip factor, μ_s, of 0.5 and 0.11 respectively are appropriate.

C. Design Procedure: General

1. Governing Criteria

The fasteners, splice plates and section shall be sufficient to transmit a force equal to 15% of the section design capacity in direct compression only.

2. Design Actions

15% section design compression capacity

3. Connection Design Strength Limits

Splice fittings compression transfer

D. Design Procedure: Flange Splice

1. Governing Criteria

The minimum bolt group, flange plate, section design strength limits shall exceed the applied design load.

Flange splice tension
Flange splice slip
Bolt edge distance
Flange plate width
Flange doubler plate width
Section flange width
Flange plate compactness
Flange bolt rows *(Admt 11/05)*
Flange plate thickness *(Admt 11/05)*

2. Design Actions

Design moment
Design axial compression
Resultant flange splice tension.
Flange splice slip tension.

3. Connection Design Strength Limits

Connection moment capacity at design axial load
Tension flange moment limit
Maximum compression flange moment limit
Flange splice design tension capacity
Compression transfer of flange splice

4. Bolt Group Design Strength Limits

Shear strength to transfer flange resultant tension actions

5. Flange Plate Design Strength Limits

a) BCS1: Single Side Flange Plates

Bolt hole 1^{st} bearing
Bolt hole 1^{st} longitudinal tearing
Plate gross and net tension

b) BCS2: Double Side Flange Plates

Bolt hole 1st bearing
Bolt hole 1st longitudinal tearing
Plate gross and net tension, limited by bolt shear transfer between plate and section

6. Section Design Strength Limits

Bolt hole 1st bearing
Bolt hole 1st longitudinal tearing
Flange gross and net tension

E. Design Procedure: Web Splice

1. Governing Criteria

Design shear capacity shall be greater than the applied design shear force.

Design bolt shear capacity and bolt hole bearing capacity of the cleat and the section web shall be greater than the resultant bolt design shear force.

The transverse bolt hole tearing capacity of the web and the cleat shall be greater than the maximum design bolt hole transverse (i.e. across the section) shear force.

The longitudinal bolt hole tearing capacity of the web and the cleat shall be greater than the maximum design bolt hole longitudinal (i.e. along the section) shear force.

Splice plates shall satisfy compactness limits and minimum thickness limits.

Bolt edge distance requirements shall be met.

2. Design Actions

Design shear

3. Connection Design Strength Limits

Connection design shear capacity
Cleat plates shear
Resultant bolt shear and bearing
Bolt hole transverse tearing
Bolt hole longitudinal tearing
Compression transfer of web splice

4. Bolt Group Design Strength Limits

Double shear / bolt
Splice direct compression transfer

5. Web Plate Design Strength Limits

Cleat gross and net shear
Cleat gross and net flexure
Cleat shear / flexure interaction
Bolt hole 1st resultant bearing
Bolt hole 1st transverse tearing
Bolt hole 1st longitudinal tearing

Total bolt hole compression bearing
Plate compression yield

6. Section Web Design Strength Limits

Bolt hole 1st resultant bearing.
Bolt hole 1st transverse tearing.
Bolt hole 1st longitudinal tearing.
Total bolt hole compression bearing.

F. Design Formulae: General

1. Governing Criteria

$N^*_{splice} \le \phi N_{splice}$	Splice fittings compression

2. Design Actions

$N^*_{splice} = 0.15\phi_s\left(2A_{gf}f_{yf} + A_{gw}f_{yw}\right)$	NZS3404:1997 minimum cl 9.1.4.1.b.v

3. Connection Design Strength Limits

$\phi N_{splice} = 2\phi N_{cf} + \phi N_{cw}$	Splice fittings compression design capacity

G. Design Formulae: Flange Splice

1. Governing Criteria

$N^*_{ft} \le \phi N_{ft}$	Flange splice tension	
$N^*_{fslip} \le \phi N_{bs}$	Flange splice slip	*(Admt 11/05)*
$M^* \le \phi M_{rcon}$ at N^*_c	Connection moment capacity at N^*_c	
$\left[a_{e1};a_{e3}\right] \ge 175d_f$	Edge distance: manual flame cut or crop	
$b_{if} \ge s_g + 3.0d_f$	Flange plate width: HR or CNC flame cut	
$2.5d_f \le b_{i2} \le s_g - 2r - t_w$	Flange doubler plate width: HR flat	
$b_f \ge s_g + 2.5d_f$	Hot rolled section flange width	
$b_f \ge s_g + 3.0d_f$	BHP welded section flange width	
$\frac{s_g}{t_{if}} \le 17.5$	Flange plate compactness	
$n_{pf} \ge 2$	Flange bolt rows	*(Admt 11/05)*
$n_{ei}t_{if} \ge 0.5t_f$	Flange plate thickness	*(Admt 11/05)*

2. Design Actions

M^*	Design moment	
N^*_c	Design axial compression	
$N^*_{ft} = \frac{M^*}{d - t_f} - N^*_c\frac{A_{gf}}{A_g}$	Resultant flange splice tension	
$N^*_{fslip} = \frac{0.15\phi M_s}{d - t_f}$	Flange splice slip tension	*(Admt 11/05)*

3. Connection Design Strength Limits

$\phi M_{rcon} = [\phi M_t ; \phi M_c]_{\min}$	Connection moment capacity at N_c^*
$\phi M_t = \left(\phi N_{ft} + N_c^* \dfrac{A_{gf}}{A_g}\right)(d - t_f)$	Tension flange moment limit
$\phi M_c = \phi_s b_f t_f f_{yf} \left(1 - \dfrac{N_c^*}{\phi N_s}\right)(d - t_f)$	Max. compression flange moment limit
$\phi N_{ft} = [\phi N_b ; \phi N_{ti} ; \phi N_f]_{\min}$	Flange splice design tension capacity
$\phi N_{cf} = [\phi N_{bi} ; \phi N_{gti} ; \phi N_{bf} ; \phi N_b ; \phi N_{gtf}]_{\min}$	Compression transfer of flange splice

4. Bolt Group Design Strength Limits

a) BCS1: Single Side Flange Plates

$\phi N_b = 2n_{pf} \phi V_{fn}$	Flange bolt group tension

b) BCS2: Double Side Flange Plates

$\phi N_b = 2n_{pf} 2\phi V_{fn}$	Flange bolt group tension

5. Bolt Group Design Serviceability Limits *(Admt 11/05)*

$\phi N_{bs} = 2n_{pf} \phi V_{sf}$	Flange bolt group slip
$\phi V_{sf} = \phi_{slip} \mu_s n_{ei} N_{ptf} k_h$	Bolt slip shear capacity

6. Flange Plate Design Strength Limits

a) BCS1: Single Side Flange Plates

$\phi N_{ti} = [\phi N_{bi} ; \phi N_{lti} ; \phi N_{gti} ; \phi N_{nti}]_{\min}$	BCS1 Flange plates tension
$\phi N_{bi} = \phi_s 2n_{pf} 3.2 t_{if} d_f f_{ui}$	Bolt hole 1st bearing
$\phi N_{lti} = \phi_s 2n_{pf} a_{exi} t_{if} f_{ui}$	Bolt hole 1st longitudinal tearing
$\phi N_{gti} = \phi_s f_{yfi} A_{gfi}$	Gross tension yield
$\phi N_{nti} = \phi_s 0.85 f_{ui} A_{nfi}$	Net tension ultimate

b) BCS2: Double Side Flange Plates

$\phi N_{ti} = [\phi N_{bi} ; \phi N_{lti} ; \phi N_{gnti}]_{\min}$	BCS2 Flange plates tension
$\phi N_{bi} = 2\phi_s 2n_{pf} 3.2 t_{if} d_f f_{ui}$	Bolt hole 1st bearing
$\phi N_{lti} = 2\phi_s 2n_{pf} a_{exi} t_{if} f_{ui}$	Bolt hole 1st longitudinal tearing
$\phi N_{gnti} = \phi N_{ti1} + \phi N_{ti2}$	Total plates gross / net tension
$\phi N_{ti1} = [\phi N_{gti1} ; \phi N_{nti1} ; \phi N_b / 2]_{\min}$	Outer plate gross / net tension
$\phi N_{ti2} = [\phi N_{gti2} ; \phi N_{nti2} ; \phi N_b / 2]_{\min}$	Doubler plates gross / net tension
$\phi N_{gti1} = \phi_s f_{yfi} A_{gfi}$	Outer plate gross tension yield
$\phi N_{nti1} = \phi_s 0.85 f_{ui} A_{nfi}$	Outer plate net tension ultimate

$\phi N_{gti2} = \phi_s f_{yfi} A_{gi2}$ Doubler plates gross tension yield
$\phi N_{nti2} = \phi_s 0.85 f_{ui} A_{ni2}$ Doubler plates net tension ultimate

7. Section Design Strength Limits

$\phi N_f = \left[\phi N_{bf} ; \phi N_{ltf} ; \phi N_{ntf} ; \phi N_{gtf}\right]_{\min}$ Flange tension

$\phi N_{bf} = \phi_s 2 n_{pf} 3.2 t_f d_f f_{ub}$ Bolt hole 1st bearing
$\phi N_{ltf} = \phi_s 2 n_{pf} a_{exb} t_f f_{ub}$ Bolt hole 1st longitudinal tearing
$\phi N_{ntf} = \phi_s 0.85 f_{ub} A_{nf}$ Flange net tension ultimate
$\phi N_{gtf} = \phi_s t_f b_f f_{yf}$ Flange gross tension yield

8. Definition of Terms

$a_{exb} = \left[a_{e2}, a_{e3}\right]_{\min}$ $a_{exi} = \left[a_{e1}, a_{e2}\right]_{\min}$ Edge distances

$a_{e1} = \dfrac{d_{if} - 2(n_{pf} - 1)s_{pf} - s_{p1}}{2}$ End edge distance

$a_{e2} = s_{pf} - \dfrac{d_h}{2}$ Inter-bolt edge distance

$a_{e3} = \dfrac{s_{p1}}{2}$ Flange edge distance

$A_{gf} = t_f b_f$ Gross flange area
$A_{gfi} = t_{if} b_{if}$ Gross outer flange plate area.
$A_{gi2} = 2 t_{if} b_{i2}$ Gross doubler flange plates area.
$A_{nf} = t_f \left(b_f - n_g d_h\right)$ Net flange area
$A_{nfi} = t_{if} \left(b_{if} - n_g d_h\right)$ Net outer flange plate area
$A_{ni2} = t_{if} \left(2 b_{i2} - n_g d_h\right)$ BCS2 net doubler flange plates area
$\phi_{slip} = 0.7$ Slip strength reduction factor *(Admt 11/05)*
μ_s Friction coefficient *(Admt 11/05)*
n_{ei} Number of effective interfaces *(Admt 11/05)*
N_{ptf} Minimum bolt tension at installation *(Admt 11/05)*
$k_h = 1.0$ Factor for standard bolt hole *(Admt 11/05)*

H. Design Formulae: Web Splice

1. Governing Criteria

$V^* \le \phi V_{con}$ Splice plates & web gross / net shear

$\dfrac{s_{g2}}{t_{iw}} \le 17.5$ Splice plate compactness

$\left[a_{e4} ; a_{e5} ; a_{e6}\right] \ge 175 d_f$ Edge distance: manual flame cut or crop
$d_{iw} \ge 0.35 d$ Minimum cleat length
$2 t_{iw} \ge 0.5 t_w$ Web plate thickness *(Admt 11/05)*

2. Design Actions

V^* Design shear

3. Connection Design Strength Limits

$$\phi V_{con} = \lfloor \phi V_{fb}; \phi V_{gsb}; \phi V_{gfi}; \phi V_{nfi}; \phi V_{gsi}; \phi V_{nsi}; \phi V_{ls}; \phi V_{ts} \rfloor_{min}$$

Splice plates gross and net shear

$\phi V_{fb} = Z_b \left[\phi V_f ; \phi V_{bi} ; \phi V_{bw}\right]_{min}$ Resultant bolt shear and bearing / bolt

$\phi V_{ts} = n_{pw} \left[\phi V_{tti} ; \phi V_{ttw}\right]_{min}$ Transverse tearing / bolt

$\phi V_{ls} = n_{pw} Z_{el} \left[\phi V_{lti} ; \phi V_{ltw}\right]_{min}$ Longitudinal tearing / bolt

$\phi N_{cw} = \left[\phi N_{biw}; \phi N_{gciw}; \phi N_{bw}; \phi N_{bweb}\right]_{min}$ Compression transfer of web splice

4. Bolt Design Strength Limit

$\phi V_f = 2\phi_b V_{fn}$ Double shear / bolt

$\phi N_{bweb} = n_{pw} \phi V_f$ Splice bolt compression transfer

5. Web Plates Design Strength Limits

$\phi V_{gfi} = 2\phi_s \dfrac{t_{iw} d_{iw}^{\,2}}{4e} f_{ywi}$ Gross flexure yield

$\phi V_{nfi} = 2\phi_s \left(1 - \dfrac{n_{pw} d_h}{d_{iw}}\right) \dfrac{t_{iw} d_{iw}^2}{4e} 0.85 f_{ui}$ Net flexure ultimate

$\phi V_{gsi} = 2\phi_s 0.5 t_{iw} d_{iw} f_{ywi}$ if $V^* \le 0.75\phi V_{gfi}$ Gross shear yield.

$\phi V_{gsi} = 2\phi_s 0.5 t_{iw} d_{iw} f_{ywi} \left[2.2 - \left(\dfrac{16V^*}{\phi V_{gfi}}\right)\right]$ if $V^* > 0.75\phi V_{gfi}$

Gross shear / flexure interaction

$\phi V_{nsi} = 2\phi_s 0.6 f_{ui} A_{nwi}$ if $V^* \le 0.75\phi V_{nfi}$ Net shear ultimate

$\phi V_{nsi} = 2\phi_s 0.6 f_{ui} A_{nwi} \left[2.2 - \left(\dfrac{16V^*}{\phi V_{nfi}}\right)\right]$ if $V^* > 0.75\phi V_{nfi}$

Net shear / flexure ultimate interaction

$\phi V_{bi} = 2\phi_s 3.2 f_{ui} d_f t_{iw}$ Bolt hole 1st resultant bearing.

$\phi V_{tti} = 2\phi_s a_{ey} t_{iw} f_{ui}$ Bolt hole 1st transverse tearing.

$\phi V_{lti} = 2\phi_s a_{exwi} t_{iw} f_{ui}$ Bolt hole 1st longitudinal tearing.

$\phi N_{biw} = 2\phi_s n_{pw} 3.2 t_{iw} d_f f_{ui}$ Bolt hole compression bearing.

$\phi N_{gciw} = 2\phi_s t_{iw} d_{iw} f_{ywi}$ Plate compression yield.

6. Section Web Design Strength Limits

$\phi V_{gsb} = \phi_s 0.6 d t_w f_{yw}$ Gross shear yield: HR sections

$\phi V_{gsb} = \phi_s 0.6 (d - 2t_f) t_w f_{yw}$ Gross shear yield: Welded sections

$\phi V_{bw} = \phi_s 3.2 f_{ub} d_f t_w$ Bolt hole 1st resultant bearing.

$\phi V_{ttw} = \phi_s a_{e7} t_w f_{ub}$ Bolt hole 1st transverse tearing.

$\phi V_{ltw} = \phi_s a_{e4} t_w f_{ub}$ Bolt hole 1st longitudinal tearing.

$\phi N_{bw} = n_{pw} \phi V_{bw}$ Bolt hole compression bearing.

7. Definitions of Terms

$a_{exwi} = [a_{e5}; a_{e8}]_{min}$ $\quad$ $a_{ey} = [a_{e6}; a_{e7}]_{min}$ — Edge distance

$a_{e4} = \frac{s_{g2}}{2}$ — Web edge distance

$a_{e5} = \frac{b_{iw} - s_{g2}}{2}$ — Web cleat edge x-distance

$a_{e6} = \frac{d_{iw} - (n_{pw} - 1)s_{pw}}{2}$ — Web cleat edge y-distance

$a_{e7} = s_{pw} - \frac{d_h}{2}$ — Inter-bolt edge y-distance

$a_{e8} = s_{g2} - \frac{d_h}{2}$ — Inter-bolt edge x-distance

$A_{nwi} = (d_{iw} - n_{pw} d_h) t_{iw}$ — Single plate net area

$e = a_{e4}$ — Eccentricity

$$Z_b = \frac{n_{pw}}{\sqrt{1 + \left[\frac{6e}{s_{pw}(n_{pw} + 1)}\right]^2}}$$

Eccentric bolt group resultant action factor

$$Z_{el} = \frac{s_{pw}(n_{pw} + 1)}{6e}$$

Eccentric bolt group long. action factor, single line of bolts, $n_{pw} \neq 1$

I. BCS Bolted Compression Splice Drawings

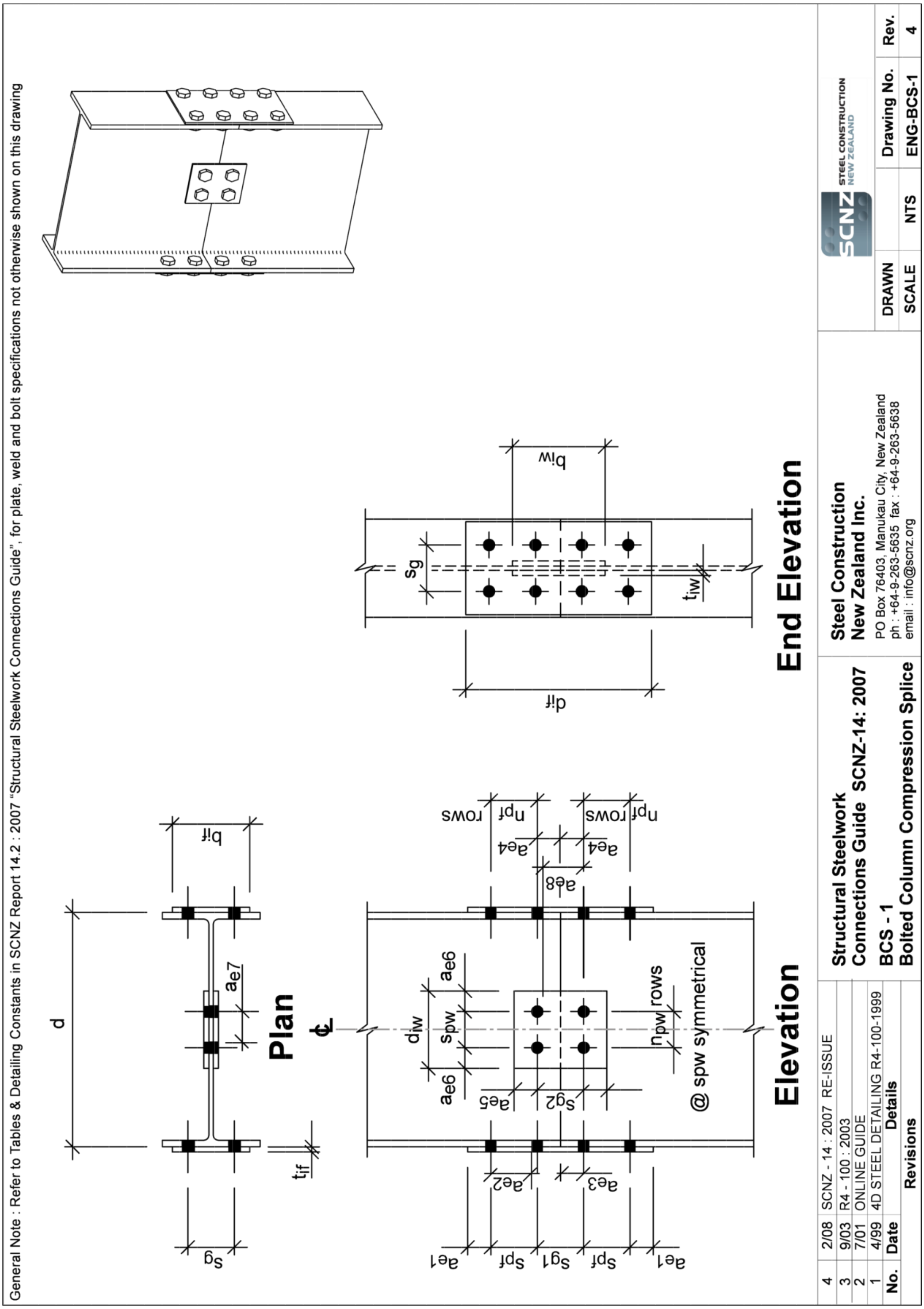


Figure 40 BCS-1 drawing

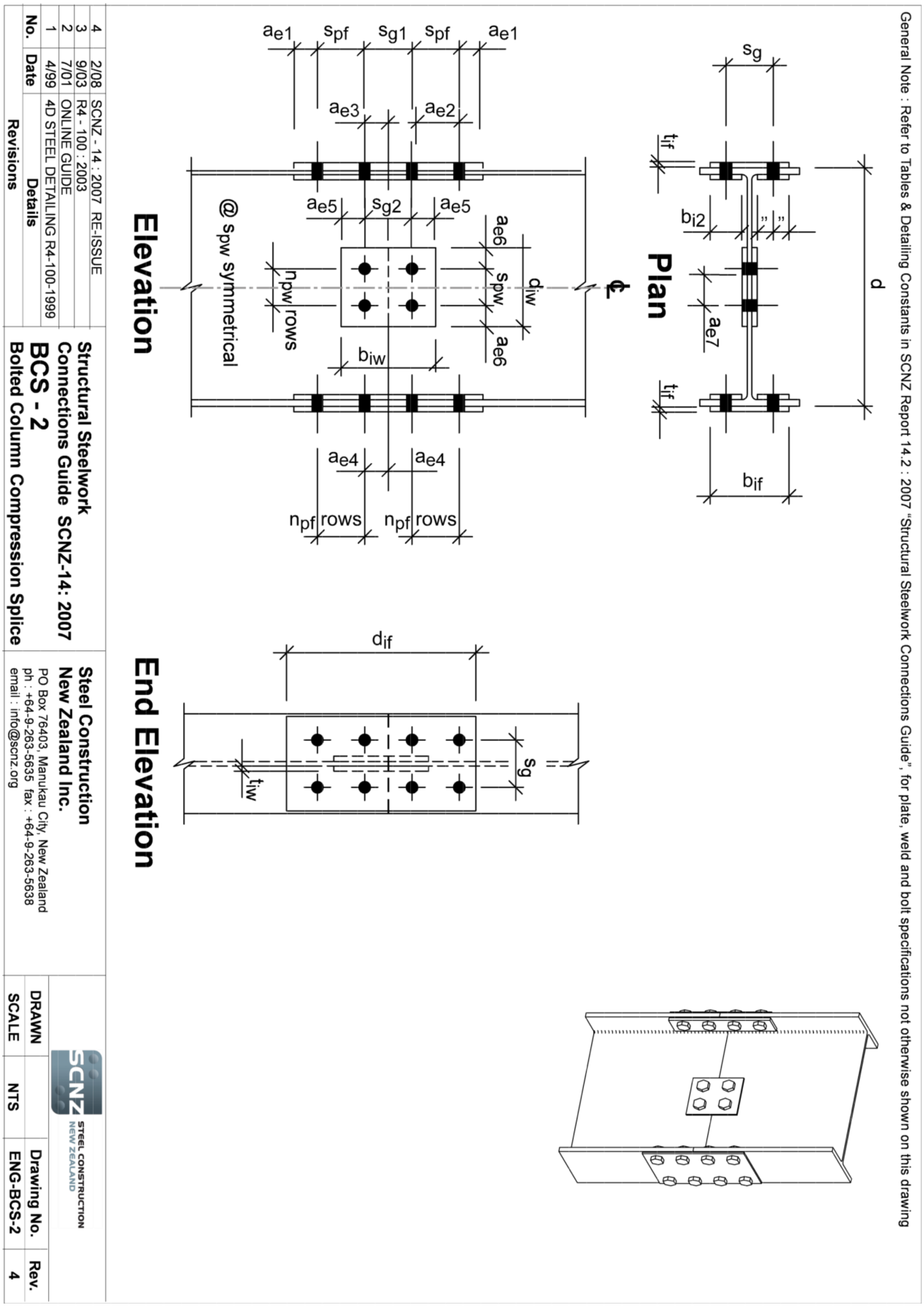


Figure 41 BCS-2 drawing

XVI. BTS: Bolted Tension Splice

A. Design Objectives

Possess design capacity to satisfy gravity and seismic actions or capacity design derived seismic axial tension, moment and shear ultimate limit state loads

The splices are located away from potential seismic yielding regions of the member and are protected by appropriate passive fire protection.

B. Design Features

Typical limiting conditions are : Shear of the web cleat or bolts; Tension yield of the flange plates, shear of flange bolts.

Located away from potential column yielding regions at places of minimum ductility demand.

All resultant tensile forces from flexural actions are resisted by the flange plate couple.

All shear actions are resisted by the web cleat.

All tension resulting from axial actions is resisted by flange and web plates in proportion to the relative area of the flanges and web. The gross flexure yield design shear capacity of the web plates ϕV_{rgfi} is calculated using a reduced plastic section modulus, due to axial load, of the web plates in accordance with equation 5.39 of [8]. The net flexure ultimate design shear capacity of the web plates ϕV_{rnfi} uses a similar approach but substitutes $0.85 f_{ui}$ for f_{yi} , and adjusts the plastic section modulus, based on gross section dimensions, by the ratio of net area to gross area of the plates. ϕV_i is the lesser of the plates gross yield design shear capacity ϕV_{gi} and the plates net area ultimate design shear capacity ϕV_{ni} . The gross shear yield design shear capacity of the web plates ϕV_{rgsi} is calculated using the axial load modified gross flexural yield design shear capacity otherwise in accordance with cl 5.12.2 NZS3404:1997.

Pairs of web cleats are set out central to and either side of the column section web for all BTS1 and BTS2 connections.

Two lines of bolts are located on either side of the web splice.

The web plate depth is not much less than half the section depth.

Single flange plates located on the outer face of the column flanges are used for BTS1 connections. For BTS2 connections, double flange plates of equal thickness are located on both faces of the column flanges.

Bolt gauge is set out from the centre-line of the plates and flanges. BTS1 and BTS2 connections have $n_g = 2$ gauge rows of bolts to the section flanges.

(Admt 11/05) For stiffness compatibility of the splice with the connected sections, the combined thickness of splice plates should typically be not less than half the thickness of the web or flange spliced.

(Admt 11/05) To minimise slip during construction flange splices are detailed with a minimum of two rows of bolts in the flanges, each side of the splice. The flange splice is designed to resist a minimum serviceability load of 15% of section moment capacity by the flange bolts in friction tension mode. A slip factor, μ_s of 0.45, in accordance with recommendations of BS4604: Part 1: 1970 is appropriate for unpainted steel without mill scale. For inorganic zinc silicate and alkyd primer a slip factor, μ_s, of 0.5 and 0.11 respectively are appropriate.

C. Design Procedure: Flange Splice

1. Governing criteria

The minimum bolt group, flange plate, section design strength limits shall exceed the applied design direct and moment induced tension.

Flange splice slip
Flange splice plate and flange widths shall satisfy bolt edge distance criteria.
Flange plate compactness
Flange bolt rows *(Admt 11/05)*
Flange plate thickness *(Admt 11/05)*

2. Design Actions

Design moment
Design axial tension
Resultant flange tension

3. Connection Design Strength Limits

Connection moment capacity with axial tension , N_t^*
Tension flange moment limit
Max. compression flange moment limit
Design tension capacity

4. Bolt Group Design Strength Limits

Shear strength to transfer flange resultant tension actions
.

5. Flange Plate Design Strength Limits

a) BTS1: Single Side Flange Plates

Bolt hole 1st bearing
Bolt hole 1st longitudinal tearing
Cleat gross and net tension

b) BTS2: Double Side Flange Plates

Bolt hole 1st bearing
Bolt hole 1st longitudinal tearing
Cleat gross and net tension, limited by bolt shear transfer between cleats and section.

6. Section Design Strength Limits

Bolt hole 1st bearing.
Bolt hole 1st longitudinal tearing.
Flange gross and net tension.

D. Design Procedure: Web Splice

1. Governing Criteria

Cleats shear
Cleats and section web tension
Resultant bolt shear and bearing
Transverse bolt hole tearing
Longitudinal bolt hole tearing
Splice plate compactness

Slice plate minimum thickness *(Admt 11/05)*

2. Design Actions

Web tension
Resultant maximum bolt shear
Maximum transverse shear / bolt
Maximum longitudinal shear / bolt

3. Bolt Group Design Strength Limits

Double shear capacity of bolt

4. Web Plate Design Strength Limits

Web plates interaction shear at N_t^*
Web plates gross interaction shear at N_t^*
Web plates net interaction shear at N_t^*
Web plates direct tension

Bolt hole 1st resultant bearing.
Bolt hole 1st transverse tearing.
Bolt hole 1st longitudinal tearing.

Web plates gross flexure yield at N_t^*
Web plates net flexure ultimate at N_t^*
Web plates gross shear yield.
Web plates gross shear / flexure yield interaction

Web plates net shear ultimate
Web plates net shear / flexure ultimate interaction

Web gross tension yield
Web net tension ultimate

5. Section Web Design Strength Limits

Bolt hole 1st resultant bearing.
Bolt hole 1st transverse tearing.
Bolt hole 1st longitudinal tearing.
Web gross and net tension.

E. Design Formulae: Flange Splice

1. Governing Criteria

$N_{ft}^* \leq \phi N_{ft}$	Tension	
$M^* \leq \phi M_{rcon}$ at N_t^*	Connection moment capacity at N_t^*	
$N_{fslip}^* \leq \phi N_{bs}$	Flange splice slip	*(Admt 11/05)*
$\left[a_{e1}; a_{e3}\right] \geq 175 d_f$	Edge distance: manual flame cut or crop	
$\dfrac{s_g}{t_{if}} \leq 17.5$	Flange plate compactness	
$n_{pf} \geq 2$	Flange bolt rows	*(Admt 11/05)*
$n_{ei} t_{if} \geq 0.5 t_f$	Flange plate thickness	*(Admt 11/05)*

For $n_g = 2$ *flange gauge rows:*

$b_{if} \geq s_g + 3.0d_f$	Flange plate width: HR or CNC flame cut
$2.5d_f \leq b_{i2} \leq s_g - 2r - t_w$	Flange doubler plate width: HR flat
$b_f \geq s_g + 2.5d_f$	Hot rolled section flange width
$b_f \geq s_g + 3.0d_f$	BHP welded section flange width

2. Design Actions

M^*	Design moment
N_t^*	Design axial tension
$N_{ft}^* = \dfrac{M^*}{d - t_f} + N_t^* \dfrac{A_{gf}}{A_g}$	Resultant flange tension
$N_{fslip}^* = \dfrac{0.15\phi M_s}{d - t_f}$	Flange splice slip tension *(Admt 11/05)*

3. Connection Design Strength Limits

$\phi M_{rcon} = \left[\phi M_t ; \phi M_c\right]_{\min}$	Connection moment capacity at N_t^*
$\phi M_t = \left(\phi N_{ft} - N_t^* \dfrac{A_{gf}}{A_g}\right)\left(d - t_f\right)$	Tension flange moment limit
$\phi M_c = \phi_s b_f t_f f_{yf} \left(1 + \dfrac{N_t^*}{\phi N_s}\right)\left(d - t_f\right)$	Max. compression flange moment limit
$\phi N_{ft} = \left[\phi N_b ; \phi N_{ti} ; \phi N_f\right]_{\min}$	Design tension capacity

4. Bolt Group Design Strength Limits

a) BTS1: Single Side Flange Plates

$\phi N_b = 2n_{pf} \phi_b V_{fn}$	Flange bolt group shear tension

b) BTS2: Double Side Flange Plates

$\phi N_b = 2n_{pf} 2\phi_b V_{fn}$	Flange bolt group shear tension

5. Bolt Group Design Serviceability Limits *(Admt 11/05)*

$\phi N_{bs} = 2n_{pf} \phi V_{sf}$	Flange bolt group slip
$\phi V_{sf} = \phi_{slip} \mu_s n_{ei} N_{ptf} k_h$	Bolt slip shear capacity

6. Flange Plate Design Strength Limits

a) BTS1: Single Side Flange Plates

$\phi N_{ti} = \left[\phi N_{bi} ; \phi N_{lti} ; \phi N_{gti} ; \phi N_{nti}\right]_{\min}$	BTS1 Flange plates tension
$\phi N_{bi} = \phi_s 2n_{pf} 3.2t_{if} d_f f_{ui}$	Bolt hole 1st bearing
$\phi N_{lti} = \phi_s 2n_{pf} a_{exi} t_{if} f_{ui}$	Bolt hole 1st longitudinal tearing
$\phi N_{gti} = \phi_s f_{yi} A_{gfi}$	Gross tension yield

$\phi N_{nti} = \phi_s 0.85 f_{ui} A_{nfi}$ Net tension ultimate

b) BTS2: Double Side Flange Plates

$\phi N_{ti} = \left[\phi N_{bi}; \phi N_{lti}; \phi N_{gnti}\right]_{\min}$	BTS2 Flange plates tension
$\phi N_{bi} = 2\phi_s 2n_{pf} 3.2 t_{if} d_f f_{ui}$	Bolt hole 1st bearing
$\phi N_{lti} = 2\phi_s 2n_{pf} a_{exi} t_{if} f_{ui}$	Bolt hole 1st longitudinal tearing
$\phi N_{gnti} = \phi N_{ti1} + \phi N_{ti2}$	Total plates gross / net tension
$\phi N_{ti1} = \left[\phi N_{gti1}; \phi N_{nti1}; \phi N_b / 2\right]_{\min}$	Outer plate gross / net tension
$\phi N_{ti2} = \left[\phi N_{gti2}; \phi N_{nti2}; \phi N_b / 2\right]_{\min}$	Doubler plates gross / net tension
$\phi N_{gti1} = \phi_s f_{yfi} A_{gfi}$	Outer plate gross tension yield
$\phi N_{nti1} = \phi_s 0.85 f_{ui} A_{nfi}$	Outer plate net tension ultimate
$\phi N_{gti2} = \phi_s f_{yfi} A_{gi2}$	Doubler plates gross tension yield
$\phi N_{nti2} = \phi_s 0.85 f_{ui} A_{ni2}$	Doubler plates net tension ultimate

7. Section Design Strength Limits

$\phi N_f = \left[\phi N_{bf}; \phi N_{ltf}; \phi N_{gtf}; \phi N_{ntf}\right]_{\min}$	Flange tension
$\phi N_{bf} = \phi_s 2n_{pf} 3.2 t_f d_f f_{ub}$	Bolt hole 1st bearing
$\phi N_{ltf} = \phi_s 2n_{pf} a_{exb} t_f f_{ub}$	Bolt hole 1st longitudinal tearing
$\phi N_{gtf} = \phi_s f_{yf} A_{gf}$	Flange gross tension yield
$\phi N_{ntf} = \phi_s 0.85 f_{ub} A_{nf}$	Flange net tension ultimate

8. Definition of Terms

$a_{exb} = \left[a_{e2}, a_{e3}\right]_{\min}$ $a_{exi} = \left[a_{e1}, a_{e2}\right]_{\min}$	Edge distances
$a_{e1} = \dfrac{d_{if} - 2(n_{pf} - 1)s_{pf} - s_{p1}}{2}$	End edge distance
$a_{e2} = s_{pf} - \dfrac{d_h}{2}$	Inter-bolt edge distance
$a_{e3} = \dfrac{s_{p1}}{2}$	Flange edge distance
$A_{gf} = t_f b_f$	Gross flange area
$A_{gfi} = t_{if} b_{if}$	Gross outer flange plate area
$A_{gi2} = 2t_{if} b_{i2}$	Gross flange doubler plates area
$A_{nf} = t_f (b_f - n_g d_h)$	Net flange area
$A_{nfi} = t_{if} (b_{if} - n_g d_h)$	Net outer flange plate area
$A_{ni2} = t_{if} (2b_{i2} - n_g d_h)$	BTS2 Net flange doubler plates area
$\phi_{slip} = 0.7$	Slip strength reduction factor *(Admt 11/05)*
μ_s	Friction coefficient *(Admt 11/05)*
n_{ei}	Number of effective interfaces *(Admt 11/05)*
N_{ptf}	Minimum bolt tension at installation *(Admt 11/05)*
$k_h = 1.0$	Factor for standard bolt hole *(Admt 11/05)*

F. Design Formulae: Web Splice

1. Governing Criteria

$V^* \le \left[\phi V_i ; \phi V_{gsb}\right]_{\min}$	Cleats & web shear at N_t^* tension
$N_w^* \le \phi N_w$	Cleats and web direct tension
$V_{res}^* \le \phi V_{res}$	Resultant bolt shear and bearing
$V_{ts}^* \le \phi V_{ts}$	Transverse bolt hole tearing
$V_{ls}^* \le \phi V_{ls}$	Longitudinal bolt hole tearing
$\dfrac{s_{g2}}{t_{iw}} \le 17.5$	Splice plate compactness
$\left[a_{e4} ; a_{e5} ; a_{e6}\right] \ge 175 d_f$	Edge distance: manual flame cut or crop
$d_{iw} \ge 0.35d$	Minimum cleat length
$2t_{iw} \ge 0.5t_w$	Web plate thickness *(Admt 11/05)*

2. Design Actions

$N_w^* = N_t^* \dfrac{A_{gw}}{A_g}$	Web tension
$V_{res}^* = \sqrt{V_{ts}^{*2} + V_{ls}^{*2}}$	Resultant maximum bolt shear
$V_{ts}^* = \dfrac{V^*}{2n_{pw}} + \dfrac{V^* e s_{g3}}{2i_{bp}}$	Maximum transverse shear / bolt
$V_{ls}^* = \dfrac{N_w^*}{2n_{pw}} + \dfrac{V^* e\left(n_{pw} - 1\right)s_{pw}}{2i_{bp}}$	Maximum longitudinal shear / bolt

3. Connection Design Strength Limits

$\phi V_{rcon} = V_{\max}^*$	Connection shear capacity at N_t^* tension
$\phi V_i = \left[\phi V_{gi} ; \phi V_{ni}\right]_{\min}$	Web plates interaction shear at N_t^*
$\phi N_w = \left[\phi N_{twi} ; \phi N_{gtw} ; \phi N_{ntw}\right]_{\min}$	Cleats and web direct tension
$\phi V_{res} = \left[\phi V_f ; \phi V_{bi} ; \phi V_{bw}\right]_{\min}$	Resultant bolt shear and bearing / bolt
$\phi V_{ts} = \left[\phi V_{tti} ; \phi V_{ttw}\right]_{\min}$	Transverse bolt hole tearing / bolt
$\phi V_{ls} = \left[\phi V_{lti} ; \phi V_{ltw}\right]_{\min}$	Longitudinal bolt hole tearing / bolt

4. Bolt Design Strength Limit

$\phi V_f = 2\phi_b V_{fn}$	Double shear / bolt

5. Web Plates Design Strength Limits

$\phi V_{gi} = \left[\phi V_{gfi} ; \phi V_{gsi}\right]_{\min}$	Web plates gross interaction shear at N_t^*
$\phi V_{ni} = \left[\phi V_{nfi} ; \phi V_{nsi}\right]_{\min}$	Web plates net interaction shear at N_t^*
$\phi N_{twi} = \left[\phi N_{gtwi} ; \phi N_{ntwi}\right]_{\min}$	Web plates direct tension

$\phi V_{bi} = 2\phi_s 3.2 f_{ui} d_f t_{iw}$ Bolt hole 1st resultant bearing.

$\phi V_{tti} = 2\phi_s a_{ey} t_{iw} f_{ui}$ Bolt hole 1st transverse tearing.

$\phi V_{lti} = 2\phi_s a_{exi} t_{iw} f_{ui}$ Bolt hole 1st longitudinal tearing.

$\phi V_{gfi} = 2\phi_s \left(1 - n^2\right) \frac{t_{iw} {d_{iw}}^2}{4e} f_{ywi}$ Web plates gross flexure yield at N_t^*

$\phi V_{nfi} = 2\phi_s \left(1 - n^2\right) \frac{A_{nwi}}{A_{gwi}} \frac{t_{iw} d_{iw}^2}{4e} 0.85 f_{ui}$ Web plates net flexure ultimate at N_t^*

$\phi V_{gsi} = 2\phi_s 0.5 t_{iw} d_{iw} f_{ywi}$ if $V^* \le 0.75\phi V_{gfi}$ Web plates gross shear yield.

$\phi V_{gsi} = 2\phi_s 0.5 t_{iw} d_{iw} f_{ywi} \left[2.2 - \left(\frac{16V^*}{\phi V_{gfi}}\right)\right]$ if $V^* > 0.75\phi V_{gfi}$

Web plates gross shear / flexure yield interaction

$\phi V_{nsi} = 2\phi_s 0.6 f_{ui} A_{nwi}$ if $V^* \le 0.75\phi V_{nfi}$ Web plates net shear ultimate

$\phi V_{nsi} = 2\phi_s 0.6 f_{ui} A_{nwi} \left[2.2 - \left(\frac{16V^*}{\phi V_{nfi}}\right)\right]$ if $V^* > 0.75\phi V_{nfi}$

Web plates net shear / flexure ultimate interaction

$\phi N_{gtwi} = 2\phi_s f_{ywi} A_{gwi}$ Web gross tension yield

$\phi N_{ntwi} = 2\phi_s 0.85 f_{ui} A_{nwi}$ Web net tension ultimate

6. Section Web Design Strength Limits

$\phi V_{gsb} = \phi_s 0.6 d t_w f_{yw}$ Gross shear yield: HR sections

$\phi V_{gsb} = \phi_s 0.6 (d - 2t_f) t_w f_{yw}$ Gross shear yield: Welded sections

$\phi V_{bw} = \phi_s 3.2 f_{ub} d_f t_w$ Bolt hole 1st resultant bearing

$\phi V_{ttw} = \phi_s a_{e7} t_w f_{ub}$ Bolt hole 1st transverse tearing

$\phi V_{ltw} = \phi_s a_{exw} t_w f_{ub}$ Bolt hole 1st longitudinal tearing

$\phi N_{gtw} = \phi_s A_{gw} f_{yw}$ Gross tension yield

$\phi N_{ntw} = \phi_s 0.85 f_{ub} A_{nw}$ Net tension ultimate

7. Definitions of Terms

$a_{exi} = \left[a_{e5} ; a_{e8}\right]_{\min}$ $a_{exw} = \left[a_{e4} ; a_{e8}\right]_{\min}$ $a_{ey} = \left[a_{e6} ; a_{e7}\right]_{\min}$

$a_{e4} = \frac{s_{g2}}{2}$ Web edge distance

$a_{e5} = \frac{b_{iw} - s_{g2} - 2s_{g3}}{2}$ Web cleat edge x-distance

$a_{e6} = \frac{d_{iw} - \left(n_{pw} - 1\right) s_{pw}}{2}$ Web cleat edge y-distance

$a_{e7} = s_{pw} - \frac{d_h}{2}$ Inter-bolt edge y-distance

$a_{e8} = s_{g3} - \frac{d_h}{2}$ Inter-bolt edge x-distance

$A_{gwi} = d_{iw} t_{iw}$ Single web plate gross area

$A_{nwi} = \left(d_{iw} - n_{pw} d_h\right) t_{iw}$ Single web plate net area

$A_{gw} = (d - 2t_f) t_w$ Web gross area

$A_{nw} = \left(d - 2t_f - n_{pw} d_h\right) t_w$	Web net area
$e = a_{e4} + \frac{s_{g3}}{2}$	Eccentricity
$i_{bp} = \frac{n_{pw}}{6}\left[s_{pw}^2\left(n_{pw}^2 - 1\right) + 3 s_{g3}{}^2\right]$	Bolt group polar moment of inertia
$n = \frac{N_w^*}{2 d_{iw} t_{iw} f_{ywi}}$	Plate axial load ratio

G. BTS Bolted Tension Splice Drawings

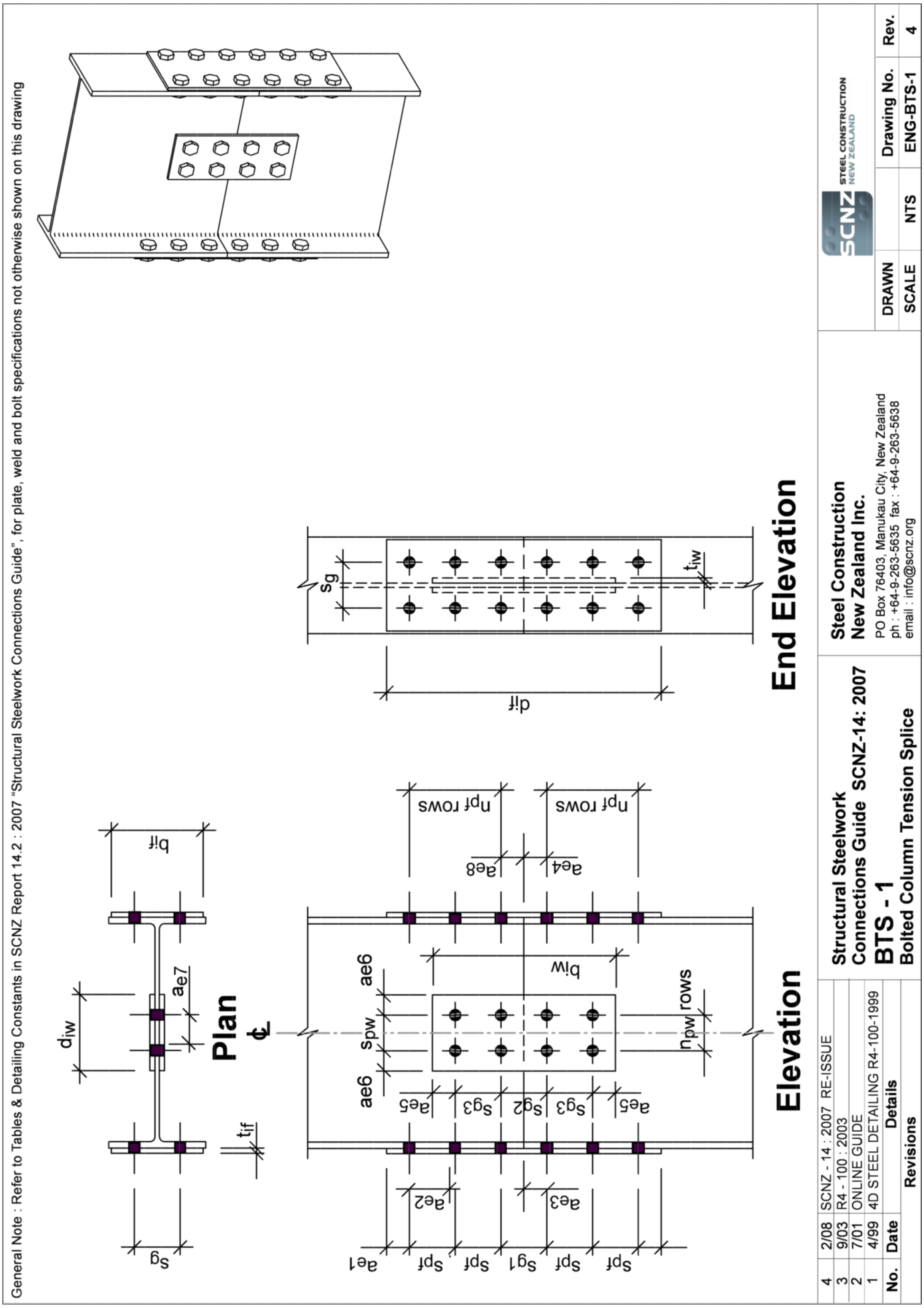


Figure 42 BTS-1 drawing

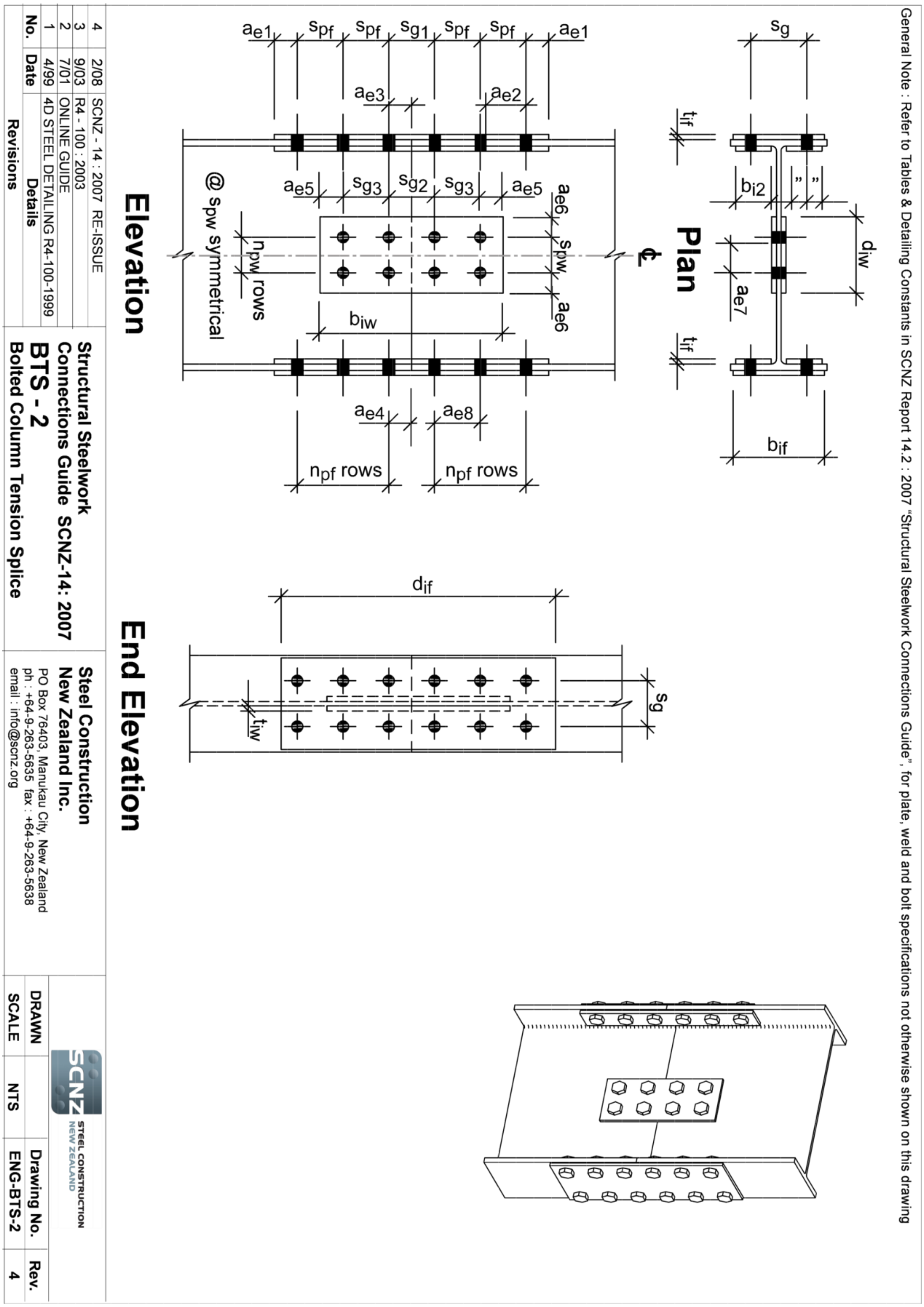


Figure 43 BTS-2 drawing

XVII. BPP: Base Plate Pinned

A. Design Objectives

Possess design axial compression and shear capacity to support ultimate limit state design loads. To have sufficient lateral shear resistance provided by the holding down bolts to provide full twist and lateral translation restraint for the axial design load.

The connection is assumed to be nominally pinned.

All columns are assumed to be prepared for full end contact bearing onto the base-plate in accordance with NZS3404:1997

B. Design Features

Typical limiting conditions are : Bearing capacity of the mortar bedding and concrete foundation; flexural /shear capacity of the steel base plate and shear capacity of the weld between the base plate and the column end and shear capacity of the holding down bolts.

All column ends are assumed prepared for full end contact bearing in accordance with NZS3404:1997. This is typically achieved by cold saw cutting.

Holding down bolts are designed have shear capacity greater than a shear force equal to 5% of the design axial compression load. Greater shear loads may require shear blocks to be welded to the underside of the base plate.

The welds of the column to the base plate are SP welds, designed to resist the prescribed design action and transfer a minimum of 15% of column section capacity.

Bearing area, A_2 , and design capacity of the concrete foundation under the base plate is derived in accordance with the provisions of NZS3101:1995 section 8.3.5.

Effective pressure distribution under the base plate is approximated to actual by using an overlapping pressure block approach. This results in the peak limiting bearing stresses occurring under the web to flange intersection points. For slender compression elements the effective stress blocks in the base plate correspond to the effective stress distributions in the elements used in the Cold Formed Steel Structures Standard, AS/NZS 4600. Therefore portions of slender elements some way from stiffening elements are not considered to contribute axial capacity to the column or develop bearing strength in the base plate. The central portion of column webs and the outer tips of open I and channel sections are typically affected.

For circular hollow sections the principle of overlapping stress blocks is approximated by assuming that the internal bearing stress annulus under the plate is fully overlapped and that the outer stress annulus isn't.

C. Design Procedure

1. General Criteria

Base plate and concrete bearing capacity must be greater than the applied compression loads.
Shear capacity of the welds and holding down bolts must be greater than the applied load.
Weld design capacity shall be not less than 15% of the section axial design capacity.
Bolt / concrete edge distance criteria shall be met.

2. Design Actions

Design compression
Design shear

3. Connection Design Strength Limits

Design compression capacity
Design shear capacity

4. Section Design Strength Limits

Design section compression capacity

5. Base Plate Strength Limits

Concrete bearing capacity
Base plate capacity

6. Bolt Group Strength Limits

Bolt shear

7. Weld Strength Limits

Ensure that weld shear capacity is greater than the design action

D. Design Formulae

1. General Criteria

$N_c^* \le \phi N_{con}$	Compression
$\phi N_{con} \ge 0.15\phi N_s$	Compression minimum
$\left[V^*; V_{res}^*\right]_{\max} \le \phi V_{con}$	Shear
$\phi V_{ws} \ge 0.15\phi N_s$	Weld minimum
$a_e > \left[d_f \sqrt{\dfrac{f_{uf}}{0.83\sqrt{f_c^{'}}}}; 12 d_f\right]_{\max}$	Bolt / concrete edge distance

2. Design Actions

N_c^*	Design compression
V^*	Design shear
$V_{res}^* = 0.05 N_c^*$	Design compression restraint shear

3. Connection Design Strength Limits

$\phi N_{con} = \left[\phi N_{bp}; \phi N_s\right]_{\min}$	Design compression capacity at N_c^*
$\phi V_{con} = \left[\phi V_{ws}; \phi V_b; \phi V_{us}\right]_{\min}$	Design shear capacity

4. Section Design Strength Limits

$\phi N_s = \phi_s k_f A_g f_y$	NZS 3404:1997 cl.6.2.1

5. Base Plate Design Strength Limits

$\sigma_b = \left[0.85 f_c^{'} \sqrt{\dfrac{A_2}{A_1}}; 1.7 f_c^{'}\right]_{\min}$	Concrete bearing stress

$\phi N_{bp} = \phi_c A_r \dfrac{\sigma_b}{2}$ Base plate bearing capacity

$\phi_c = 0.65$ Bearing, NZS3101:1995, cl. 3.4.2.2.e

6. Bolts Design Strength Limits

$\phi V_b = n_b \phi_b 0.62 f_{uf} A_c$ Shear in one direction

$\phi V_{us} = n_b \phi_{cv} a_e^2 0.32\sqrt{f_c^{'}}$ Shear adjacent to concrete free edge

$\phi_b = 0.8 \qquad \phi_{cv} = 0.75$ Shear, NZS3101:1995, cl. 3.4.2.2.d

7. Weld Design Strength Limits

$\phi V_{ws} = \phi_w 0.6 f_{uw} L_w \dfrac{t_{wi}}{\sqrt{2}}$ Shear

$\phi_w = 0.8$ SP Fillet welds

8. Definitions of Terms

a) General

$A_1 = b_i d_i$ Base plate foot print area

A_2 Design effective concrete bearing area

$b_i \geq b_f + 2(c + 0.8t_{wi})$ Minimum plate width

$c = t_i \sqrt{\dfrac{\phi_s f_{yi}}{3\phi_c \sigma_b}}$ Effective rigid cantilever outstand

$d_i \geq d + 2(c + 0.8t_{wi})$ Minimum plate depth

b) I-Section and Channel Section Columns

$A_r = 2A_{rf} + A_{rw}$ Effective rigid bearing area

$A_{rf} = (b_f + 1.6t_{wi} + 2c)(t_f + 1.6t_{wi} + 2c)$ Flange rigid plate area, $b_{ef} = b_f$

$A_{rf} = (b_{ef} + 2c)(t_f + 1.6t_{wi} + 2c)$ Flange rigid plate area, $b_{ef} < b_f$

$A_{rw} = (d_{ew} + 2t_f + 1.6t_{wi} + 2c)(t_w + 1.6t_{wi} + 2c)$
Equivalent web rigid plate area

$b_{ef} = (b_f - t_w)\left(\dfrac{\lambda_{ey}}{\lambda_{ef}}\right) + t_w \leq b_f$ Effective flange width

$d_{ew} = (d - 2t_f)\left(\dfrac{\lambda_{ey}}{\lambda_{ew}}\right) \leq (d - 2t_f)$ Effective web depth

$\lambda_{ef} = \dfrac{b_f - t_w}{2t_f}\sqrt{\dfrac{f_{yf}}{250}}$ Flange slenderness

$\lambda_{ew} = \dfrac{d - 2t_f}{t_w}\sqrt{\dfrac{f_{yw}}{250}}$ Web slenderness

$\lambda_{ey} = 16$ HR, flange yield limit

$\lambda_{ey} = 45$ HR, web yield limit

$\lambda_{ey} = 14$ HW, flange yield limit

$\lambda_{ey} = 35$ HW, web yield limit

c) RHS / SHS Columns

$A_r = 2A_{rf} + 2A_{rw}$	Effective rigid bearing area
$A_{rf} = (b_{ef} + 2t_w + 1.6t_{wi} + 2c)(t_w + 0.8t_{wi} + 2c)$	Flange rigid plate area
$A_{rw} = (d_{ew} + 2t_w + 1.6t_{wi} + 2c)(t_w + 0.8t_{wi} + 2c)$	Web rigid plate area
$b_{ef} = (b - 2t_w)\left(\dfrac{\lambda_{ey}}{\lambda_{ef}}\right) \le b - 2t_w$	Effective flange width: RHS & SHS
$d_{ew} = (d - 2t_w)\left(\dfrac{\lambda_{ey}}{\lambda_{ew}}\right) \le d - 2t_w$	Effective web depth: RHS & SHS
$\lambda_{ef} = \dfrac{b - 2t_w}{t_w}\sqrt{\dfrac{f_y}{250}}$	Flange wall slenderness
$\lambda_{ew} = \dfrac{d - 2t_w}{t_w}\sqrt{\dfrac{f_y}{250}}$	Web wall slenderness
$\lambda_{ey} = 40$	CF, wall yield limit

d) CHS Columns

$A_r = 2A_{rin} + A_{rout}$	Effective rigid bearing area
$A_{rin} = \pi\left[\left(\dfrac{d_o}{2} + 0.8t_{wi}\right)^2 - \left(\dfrac{d_o}{2} - t_w - c\right)^2\right]$	Inner rigid bearing area annulus
$A_{rout} = \pi\left[\left(\dfrac{d_o}{2} + 0.8t_{wi} + c\right)^2 - \left(\dfrac{d_o}{2} + 0.8t_{wi}\right)^2\right]$	Outer rigid bearing area annulus

E. BPP Base Plate Pinned Drawings

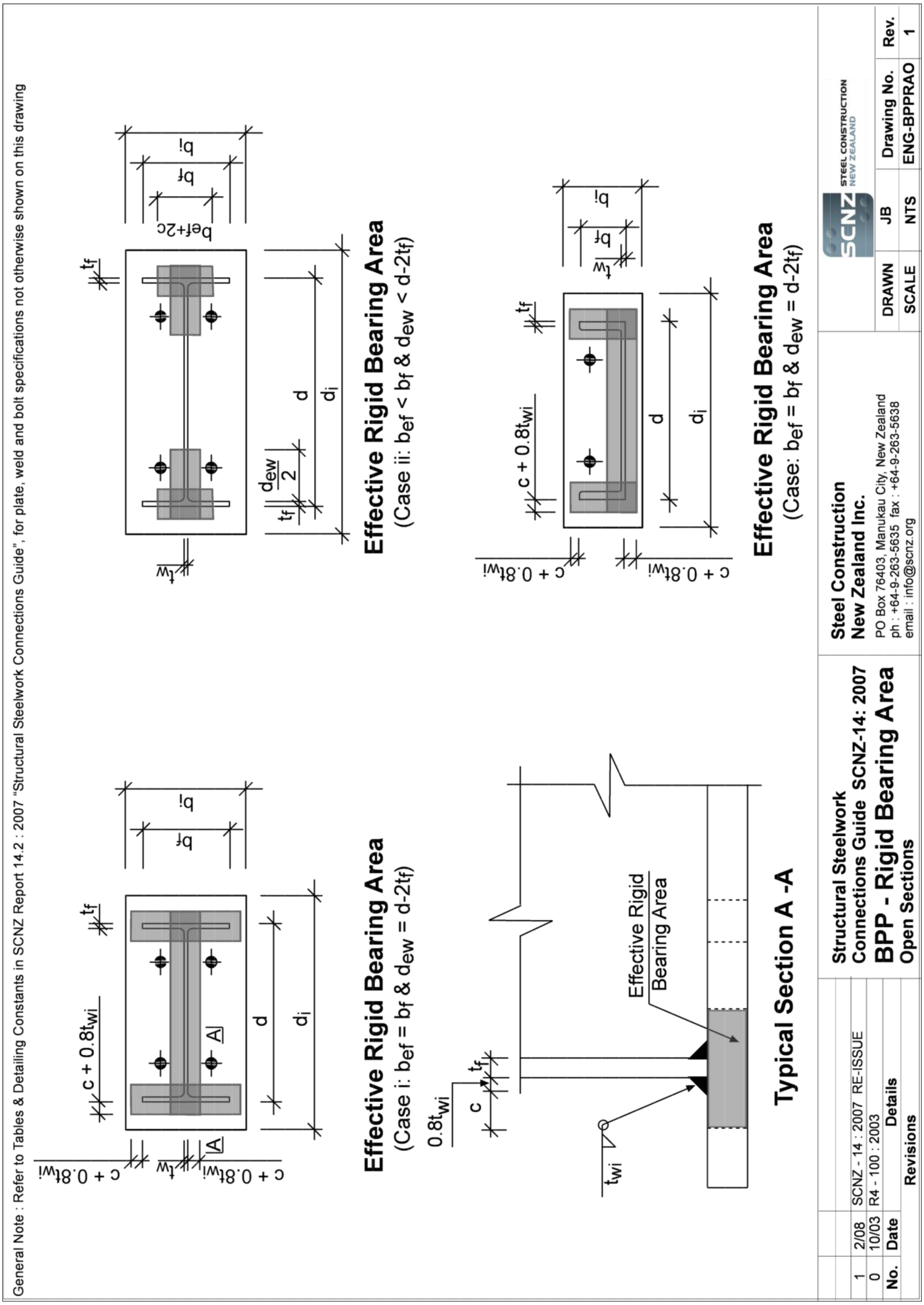


Figure 44 BPP open section rigid bearing areas

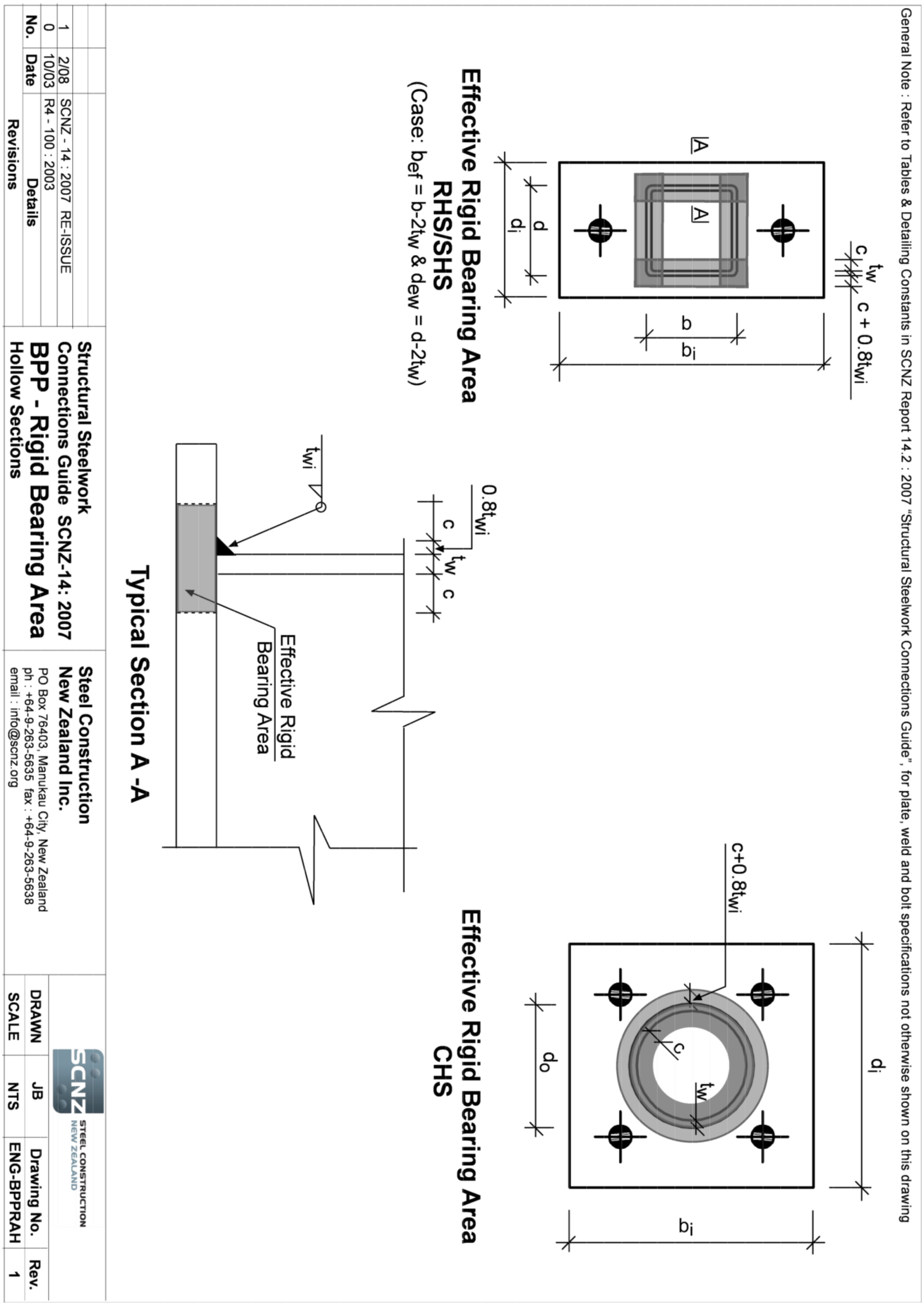


Figure 45 BPP hollow section rigid bearing areas

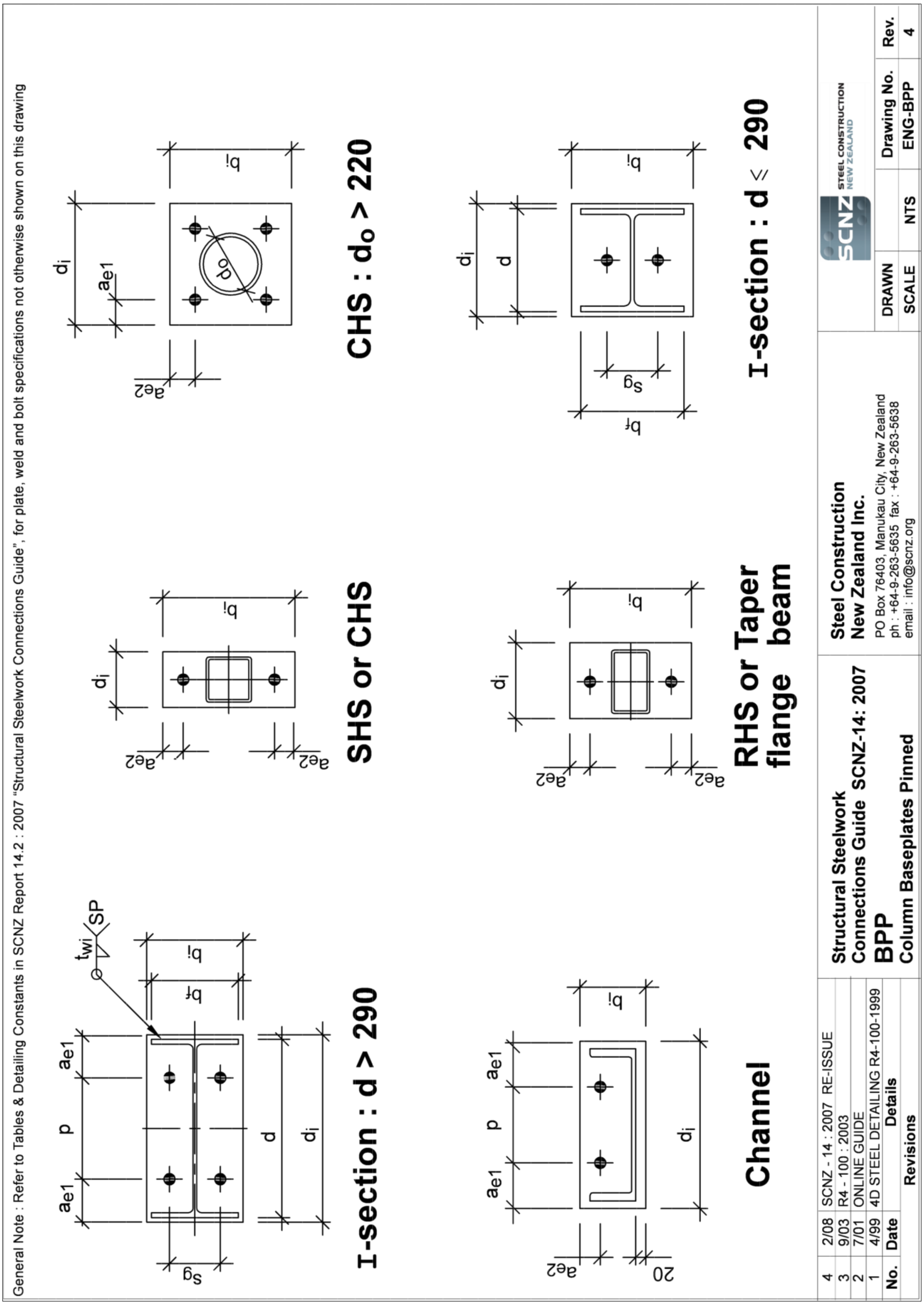


Figure 46 BPP Column Base Plates drawing