IPENZ ENGINEERS NEW ZEALAND | NEW ZEALAND INSTITUTE OF ARCHITECTS INCORPORATED | ACENZ

Building Code Clause(s) B1

PRODUCER STATEMENT – PS1 – DESIGN

(Guidance notes on the use of this form are printed on page 2)

ISSUED BY: **BVT CONSULTING LTD**
(Design Firm)

TO: Forbes and Davies
(Owner/Developer)

TO BE SUPPLIED TO: Selwyn District council
(Building Consent Authority)

IN RESPECT OF: Racking structures and Mezzanine floor
(Description of Building Work)

AT: 49 Stoneleigh Drive, Rolleston
(Address)

LOT .... **DP** .... **SO** ....

We have been engaged by the owner/developer referred to above to provide Design review
(Extent of Engagement)
services in respect of the requirements of Clause(s) B1 of the Building Code for All ☑ or Part only ☐ (as specified in the attachment to this statement), of the proposed building work.

The design carried out by us has been prepared in accordance with:

☐ Compliance Documents issued by the Ministry of Business, Innovation & Employment .... (verification method / acceptable solution) or

☑ Alternative solution as per the attached schedule BRANZ design guide, NZS 1170

The proposed building work covered by this producer statement is described on the drawings titled Forbes and Davies and numbered 16081149; together with the specification, and other documents set out in the schedule attached to this statement.

On behalf of the Design Firm, and subject to:
(i) Site verification of the following design assumptions Concrete 120 mm thick
(ii) All proprietary products meeting their performance specification requirements;

I **believe on reasonable grounds** that a) the building, if constructed in accordance with the drawings, specifications, and other documents provided or listed in the attached schedule, will comply with the relevant provisions of the Building Code and that b), the persons who have undertaken the design have the necessary competency to do so. I also recommend the following level of construction monitoring/observation:
☐CM1 ☑CM2 ☐CM3 ☐CM4 ☐CM5 (Engineering Categories) or ☐ as per agreement with owner/developer (Architectural)

I, Matt Bishop am: ☑CPEng 243276 #
(Name of Design Professional)

☐Reg Arch .... #

I am a Member of : ☑ IPENZ ☐NZIA and hold the following qualifications: BE (Hons)
The Design Firm issuing this statement holds a current policy of Professional Indemnity Insurance no less than $200,000*.
The Design Firm is a member of ACENZ: ☑

SIGNED BY Matt Bishop ON BEHALF OF BVT CONSULTING LTD
(Design Firm)

Date 11/08/16 (signature) ....

Note: This statement shall only be relied upon by the Building Consent Authority named above. Liability under this statement accrues to the Design Firm only. The total maximum amount of damages payable arising from this statement and all other statements provided to the Building Consent Authority in relation to this building work, whether in contract, tort or otherwise (including negligence), is limited to the sum of $200,000.*

This form is to accompany **Form 2 of the Building (Forms) Regulations 2004** for the application of a Building Consent.

NZBC Clause B1: Structure

Design of Steel Pallet Racking and Mezzanine Floor

Forbes and Davies, 49 Stoneleigh Drive

Prepared for: **Forbes and Davies**

By: **BVT Engineering Professional Services**

Document Revision History

Rev	Date	Revision Details	Author	Approved
A	11/08/16	For Consent	TDR	MMB

Report Completed By

Tim Riddell
Engineer, BVT Engineering Professional Services

Report Approved By

Matt Bishop, Senior Engineer BE (Hons), CPEng 243276
BVT Engineering Professional Services MIPENZ

Table of Contents

1. General

1.1 Objective

The Design Features Report (DFR) is a detailed document defining the design criteria and recording key decisions or outcomes for the design of a structure. It outlines design loading, structural modelling assumptions, material properties and design standards. The DFR also defines the calculation procedure and checking principles to be followed, providing a clear explanation of the full design.

1.2 Scope

The scope is in accordance with the Design Brief and Conditions of Engagement.

In general terms, the scope of work is as follows:

To provide engineering analysis on three racking structures and a mezzanine floor supported by racking columns to be built at Forbes and Davies' premises located at 49 Stoneleigh Drive. This also includes a global calculation of the period of structural adequacy for the unprotected steel of the mezzanine floor.

1.3 Means of Compliance

The mezzanine floor, supporting structure and separate racking structures in the warehouse, were assessed using the BRANZ Design Guide for Seismic design of high level storage racking systems with public access. The guide covers strength and deflection design using elements from the NZS 1170 suite of standards, with NZS 3404.1 and NZS 4600 as material standards. The Mezzanine timber floor was assessed using loads from NZS 1170.1: 2002 - Imposed loads, design factors from NZS 1170.0: 2002 - Structural actions and NZS 3603: 1993 - Timber structures as a materials standard.

The design of the structure is in compliance with the New Zealand Building Code (NZBC), section B1.

The following standards have been used:

- AS/NZS 1170: 2002
- NZS 3101: 2006
- NZS 3404: 1997
- NZS 4600: 2005
- NZS 3606: 1993

1.4 Alternative Solutions

The following alternative solutions have been adopted in the design of the structure:

- BRANZ Design Guide - Seismic Design of High Level Storage Racking Systems with Public Access.

2. The Structure

2.1 General

The proposed installation consists of three pallet racking structures and a mezzanine floor in a warehouse at Forbes and Davies' premises located at 49 Stoneleigh Drive.

There are four unique structure design/s, see below. Note that the mezzanine floor is supported by a racking structure and so the supporting structure has been analysed in a similar manner.

Table (1): Structure configurations

Structure Type	Levels (Above ground level)	Number of racking bays in the across aisle	Number of racking bays in the down aisle
Pallet Racking: Configuration 1	4	1	12
Pallet Racking: Configuration 2	5	1	11
Pallet Racking: Configuration 3	3	1	6
31x13 m Mezzanine floor	3-5	10/5*	10

*Note in the first half of the floor, there are back to back bays, but in the second half there is one extra wide bay.

The design life of the structures is 25 years.

The structure importance levels are all IL 1 .

2.2 Gravity Load Resisting System

Cold-formed structural steel beams span between braced frames to transfer the gravity loads to the slab, and to the surrounding soils beneath. The braced frames consist of cold-formed structural steel posts and braces.

A typical gravity system is illustrated in Figure (1) below:

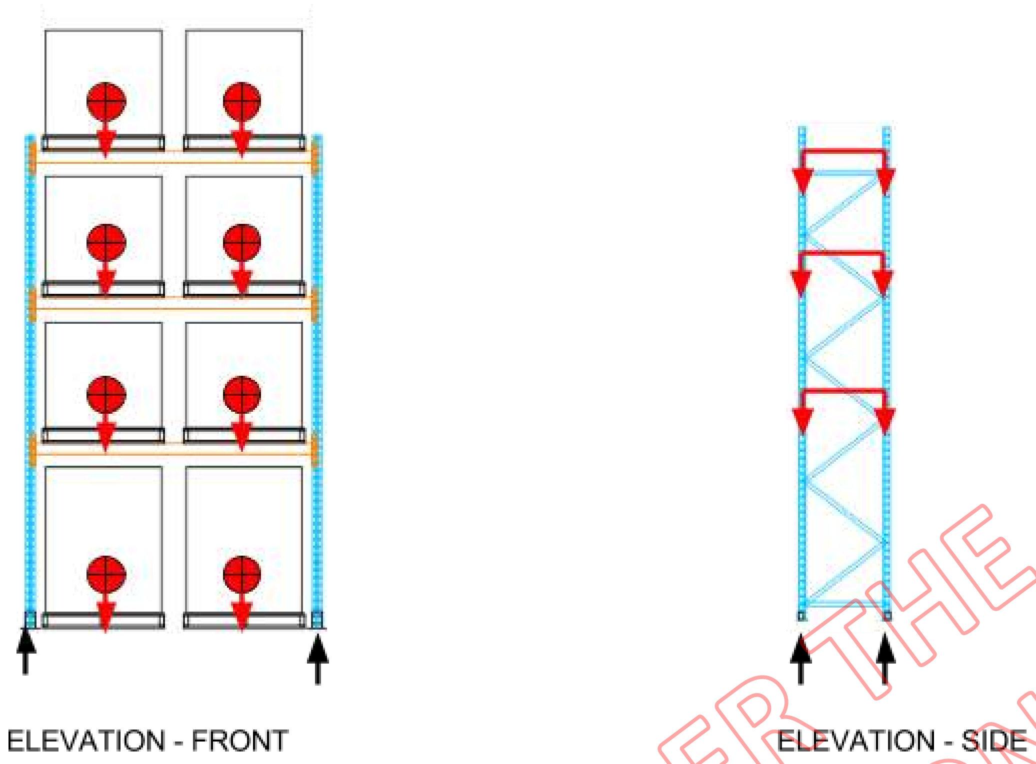


Figure (1): Elevations of typical gravity structure.

2.3 Lateral and Longitudinal Load Resisting System

The orthogonal directions for the structures are defined as across aisle and down aisle.

2.3.1 Across Aisle Lateral Load Resisting System

For this report, we define the across aisle direction as that which loads the braced frames, see Figure (2) below:

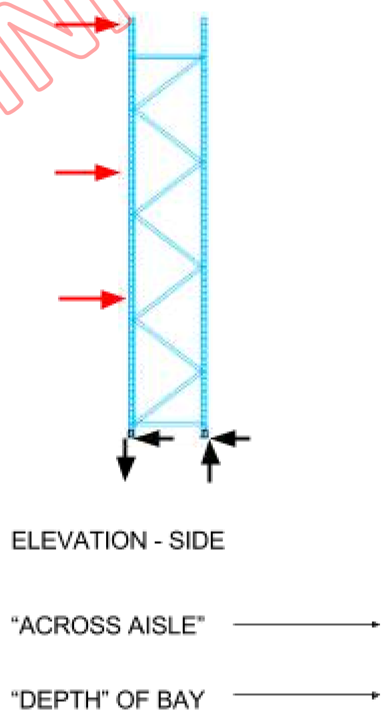


Figure (2): Elevation of across aisle lateral load resisting structure.

For the across aisle system, the lateral loads are collected by the braced frames and transferred to the slab by force-couples.

2.3.2 Down Aisle Lateral Load Resisting System

The down aisle direction is parallel to the structure "width". This direction is out-of-plane for the braced frames.

The frames are supported out-of-plane by both the slab at the base and through portal action of the bay frames.

Figure (3) below illustrates the load path described above:

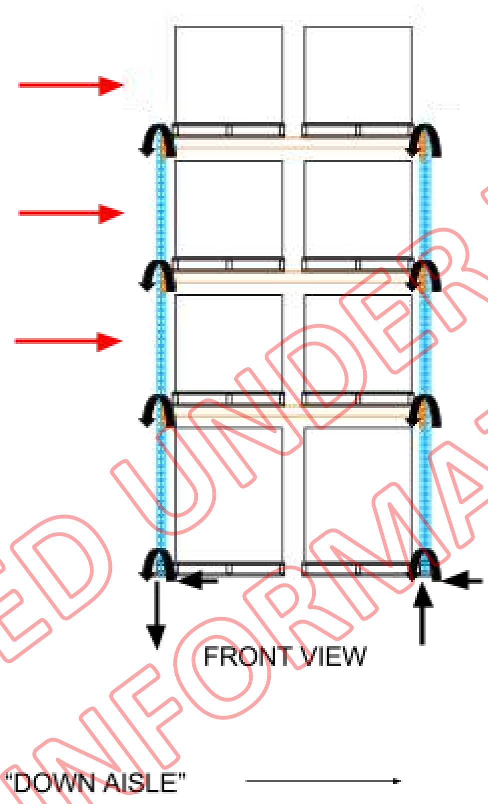


Figure (3): Plan view of down aisle lateral load resisting structure.

3. Soil Conditions

3.1 Description of Site Soil Conditions

As per section 3.3.1 of the BRANZ Design guide, soil subclass D is assumed.

4. Design Loads

4.1 General

The design loads have been determined in accordance with BRANZ Design Guide for Seismic Design of High Level Storage Racking Systems with Public Access.

4.2 Imposed Loads

4.2.1 Vertical loads

Table (2) below summarizes the vertical loads assumed for design:

Table (2): Imposed and gravity loads.

Level/Area	Use	Live Load	Superimposed Dead Load
Pallet levels	Storage	700 kg, UDL	60 kg per level
Shelf levels	Storage	400 kg UDL	60 kg per level
Mezzanine Floor	Storage	4.8 kPa	2.4 kPa

4.2.2 Design Factors

As per NZS 1170.0, the following factors have been applied for the different load cases:

Load Case	
Static	1.2G + 1.5Q
Earthquake Down aisle	G + 0.6Q + Ed
Earthquake across aisle	G + Q + Ex
Fire	G + 0.6Q

4.3 Seismic Loads

Note the below is for the racking row 1 as marked in the drawing set. A preliminary analysis indicated that is the worst case scenario. Only one set of details has been shown for clarity.

4.3.1 Analysis Methodology

The seismic analysis has been completed in accordance with the BRANZ design guide using the equivalent static analysis method.

Design Spectra are in accordance with AS/NZS 1170.5: 2004 for site subsoil class D. For the purposes of the analysis, the across aisle is considered the x direction and the down aisle direction is considered the y direction.

4.3.2 Site Parameters

Site subsoil class: D
Site hazard factor (Z): 0.30
Near-fault factor (N): 1*

*BRANZ 3.3.1 does not require the near fault factor to be considered if the period is < 1.5s, or if the location is deemed to be greater than 20km from a a major fault listed in NZS 1170.5: 2004 Table 3.6.

4.3.3 Structure Response Parameters

Structural Ductility Factor Across Aisle (μ_X): 1.25
Structural Ductility Factor Down Aisle (μ_Y): 2.0
Structural Performance Factor at ULS ($S_{P,U}$): 0.925
Inelastic Spectrum Scaling Factor (k_μ): 1.25*

*k_μ = μ as per BRANZ Design guide

4.3.4 Seismic Load Coefficient

Return period factor at ULS (R_U): 0.35

Table (3) summarises the first mode natural response period and corresponding seismic load coefficients for the ultimate limit state for the worst case (stiffest) structure.

Table (3): ULS and SLS factors for determining the seismic load coefficient.

Factor	ULS
Period, Tx	0.52s
Period, Ty	1.31 s
Ordinate of elastic site spectrum, C(T)x	3
Ordinate of elastic site spectrum, C(T)y	1.62
Design action coefficient (x), Cd(T)x	0.231
Design action coefficient (y), Cd(T)y	0.059

5. Acceptance Criteria

5.1 Drift Limits

As per the BRANZ Design Guide, drift limits are imposed to prevent the rack impacting the surrounding structure.

Displacement results at ULS are multiplied by $1.2\mu_X$ in the across aisle directions and μ_Y in the down aisle directions.

*An allowance of 0.025h has been incorporated to account for building movement.

Displacement results can be found in Appendix (A) for the mezzanine structure and Appendix (B) for the Racking, Figures (A6-7) and (B6-7).

5.2 Period of Structural Adequacy

A global check of the system to determine the Period of Structural Adequacy in the event of a fire for unprotected steel. **The PSA was calculated to be 17.5 minutes.**

6. Design Life for Durability

6.1 Design Life

Racking: 25 yrs

6.2 Durability Provisions

Durability provisions are achieved by:

Structural Steel: There is no acceptable solution available for structural steel and protection is provided through surface treatment in accordance with NZS/AS 2312:2002.

6.3 Summary of Surface Treatments

Table (4) summarises the surface treatments for the structural steel elements covered by this design features report.

Table (4): Schedule of surface treatments for structural steel elements.

Element	Design Life	Exposure Category	Surface Treatment in accordance with NZS/AS 2312	Time to first major maintenance
Indoor Racking frame steel	25	Low	Powder coated/ Galvanised	25 yrs

7. Software

Table (5) summarises the computer applications used for the analysis.

Table (5): Summary of software used for racking analysis.

Analysis type	Software used	Archive files
Loads and Section Properties	Google Spreadsheets	16081149
Structural Analysis	Axis VM	16081149 - Forbes and Davies -49 stoneleigh drive racking config A analysis.axm
		16081149 - Forbes & Davies - 49 stoneleigh drive racking - floor loading.axm

8. DRAWING AND SPECIFICATION NOTES

Refer to drawings for layout of racking systems and mezzanine floor.

8.1 Material Properties (Typical)

8.1.1 Concrete Strengths

Slab:	f'c = 32 MPa

8.1.2 Reinforcing Steel

Slab Reinforcing:	F_Y = 500 MPa

8.1.3 Structural Steel

Rolled Steel Sections:	F_Y = 350 MPa - Cold formed steel	
	F_Y = 450 MPa - Cold formed steel for columns	
Elastic modulus, typical:	E_S = 210 GPa	
Elastic modulus, braces:	$E_{S,B}$ =17.5 GPa (For vibrational analysis)	
69x80x2.2 Column Capacities:	$ØM_{S,Y}$ =	4.80 kNm
	$ØM_{S,Z}$ =	3.19 kNm
	$ØN_T$ =	187 kN
	$ØN_C$ =	56.1 kN
	Shear	60 kN
96x82x2.3 Column Capacities:	$ØM_{S,Y}$ =	5.97 kNm
	$ØM_{S,Z}$ =	2.73 kNm
	$ØN_T$ =	208 kN
	$ØN_C$ =	148 kN
	Shear	16.8 kN
90x50 Beam Capacities:	Bending Section, y	5.01 kNm
	Bending Section, z	2.66 kNm
	Axial Tension	130 kN
	Axial Compression	41.2 kN
	Shear	13.9 kN
120x50 Beam Capacities:	Bending Section, y	9.43 kNm
	Bending Section, z	4.27 kNm
	Axial Tension	240 kN
	Axial Compression	51.6 kN
	Shear	24.4 kN
C34x25x2 Brace Capacity:	Axial Tension	30.3 kN
	Axial Compression	21.2 kN
C40x25x2 Brace Capacity:	Axial Tension	34.3 kN
	Axial Compression	30.3 kN
4 Tab Joint Capacity:	Ult. Moment	2.86 kNm
	Ult. Rotation	0.0715 rad
	Stiffness	40 kNm/rad
Floor connections:	Ult. Moment	1.27 kNm - RAMSET specs
	Stiffness	100 kNm/rad
	R_Z	5.7 kN

8.1.4 Timber

Nominal Bending strength:	27.7 MPa
Elastic Modulus:	10.5 GPa
90x45 Timber bending capacity:	1.8 kNm

8.1.5 Bolts

Bolt Grades:
Grade 8.8 mild steel, M12 trubolts for Row 2, M10 elsewhere.

9. Proprietary Systems

The following proprietary elements are included in the project:

- Floor bolts – RAMSET trubolts

9.1 Manufacturer Design Requirements

Include notes here as to the design assumptions and criteria that the proprietary systems must meet. Include description of:

- Bolts must be embedded to at least 90 mm.
- Concrete has a compressive strength of at least 32 MPa and be over 120 mm thick.

Appendix A - Typical AXIS VM Result Screen Shots

(Configuration Mezzanine Floor shown)

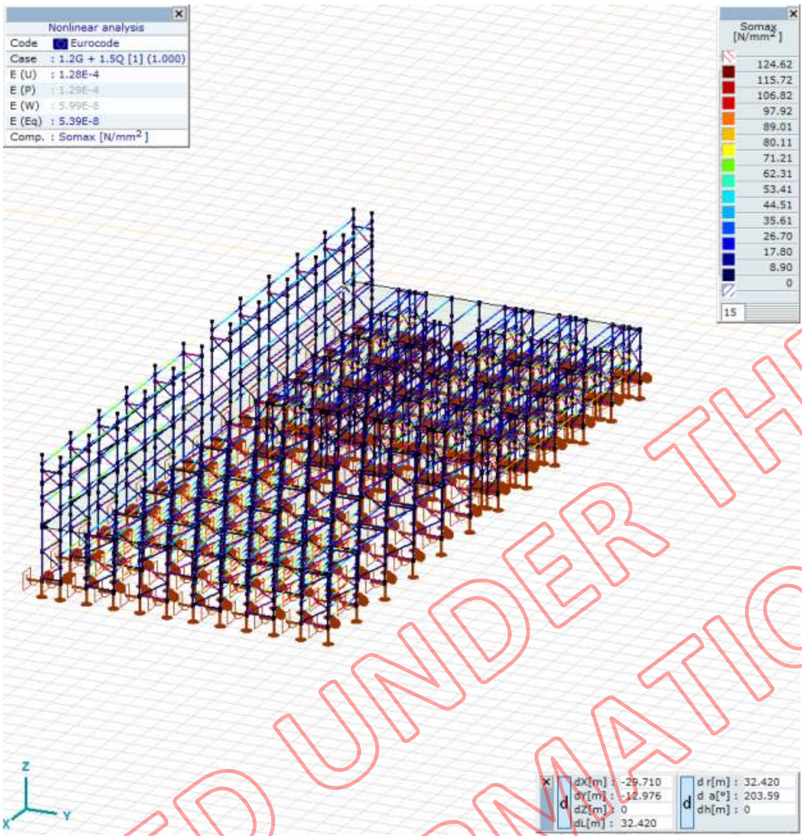


Figure (A1): Load case (1.2G + 1.5Q) stress plot.

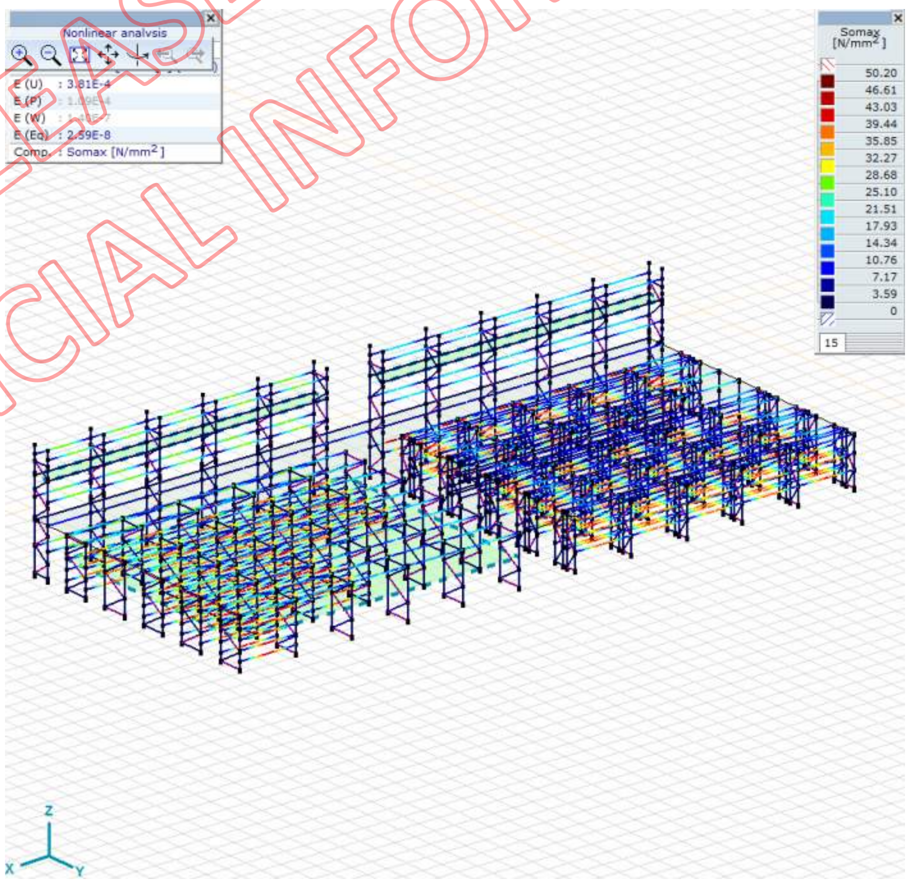


Figure (A2): Load case (G + psic_Q + Eu) down aisle stress plot.

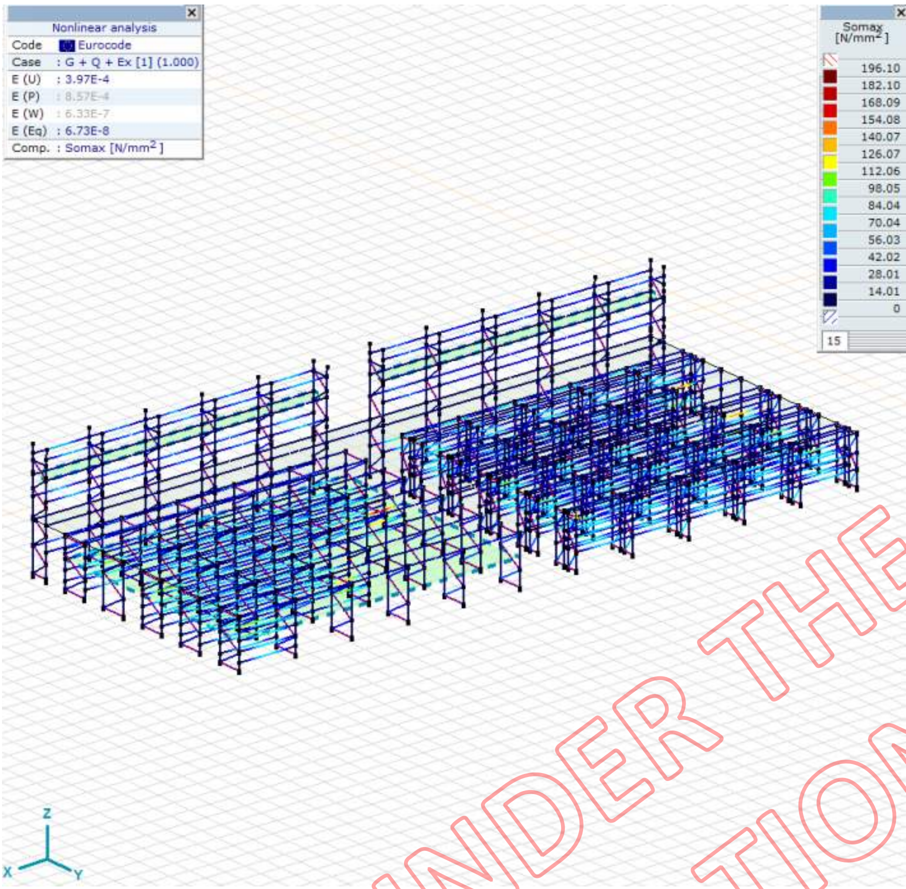


Figure (A3): Load case (G + psic_Q + Eu) across aisle stress plot.

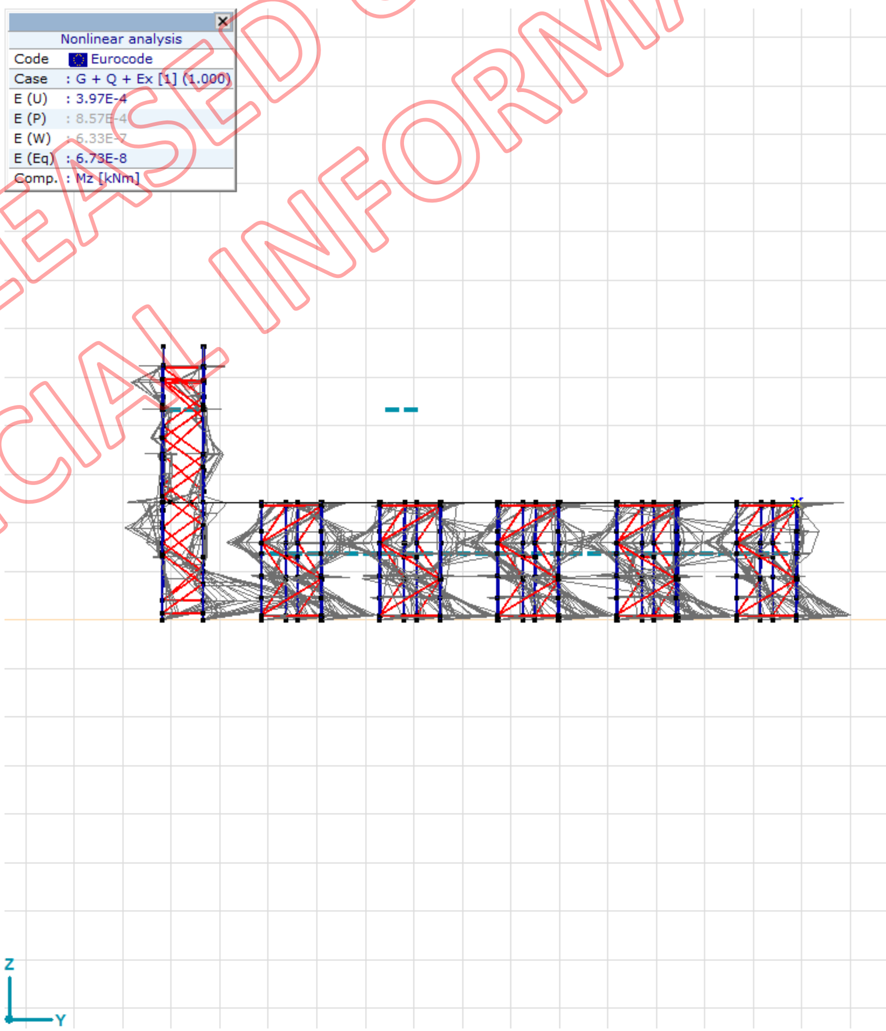


Figure (A4): Across aisle bending moment diagram.

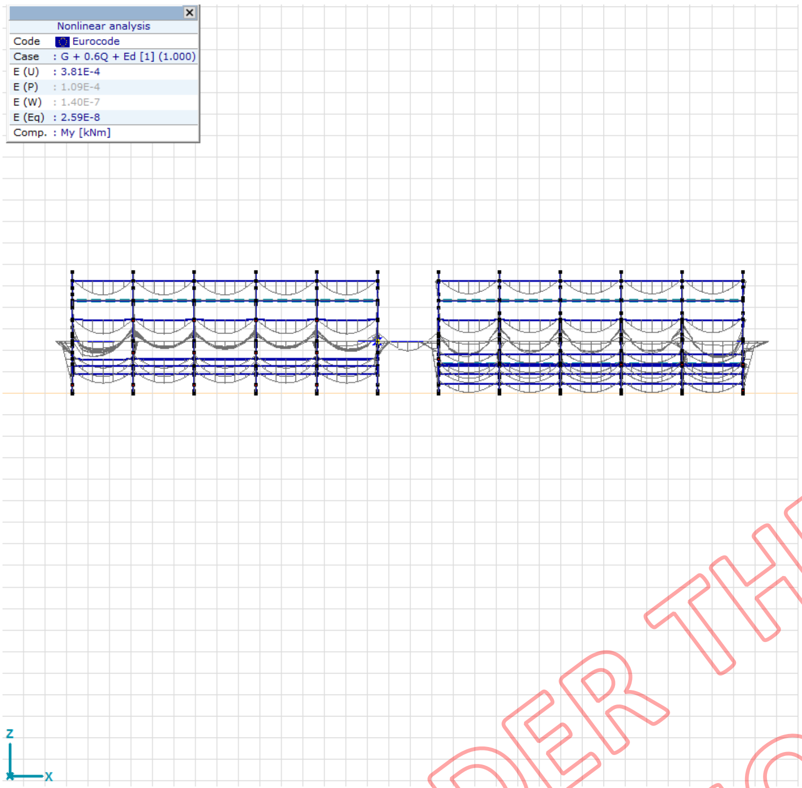


Figure (A5): Down aisle bending moment diagram.

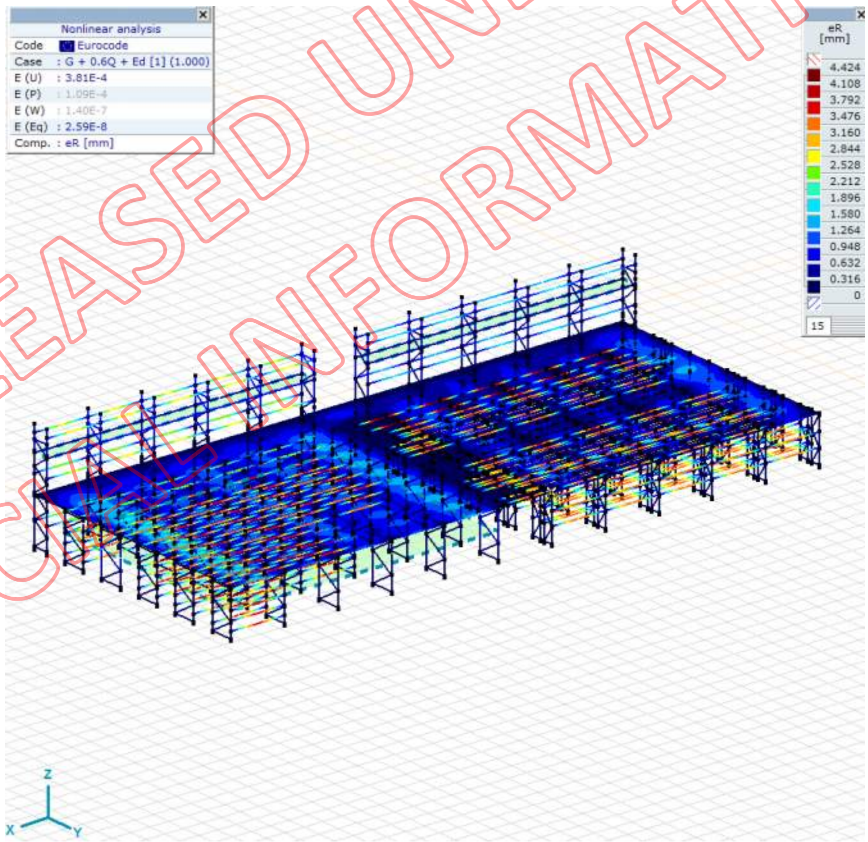


Figure (A6): Down aisle displacement.

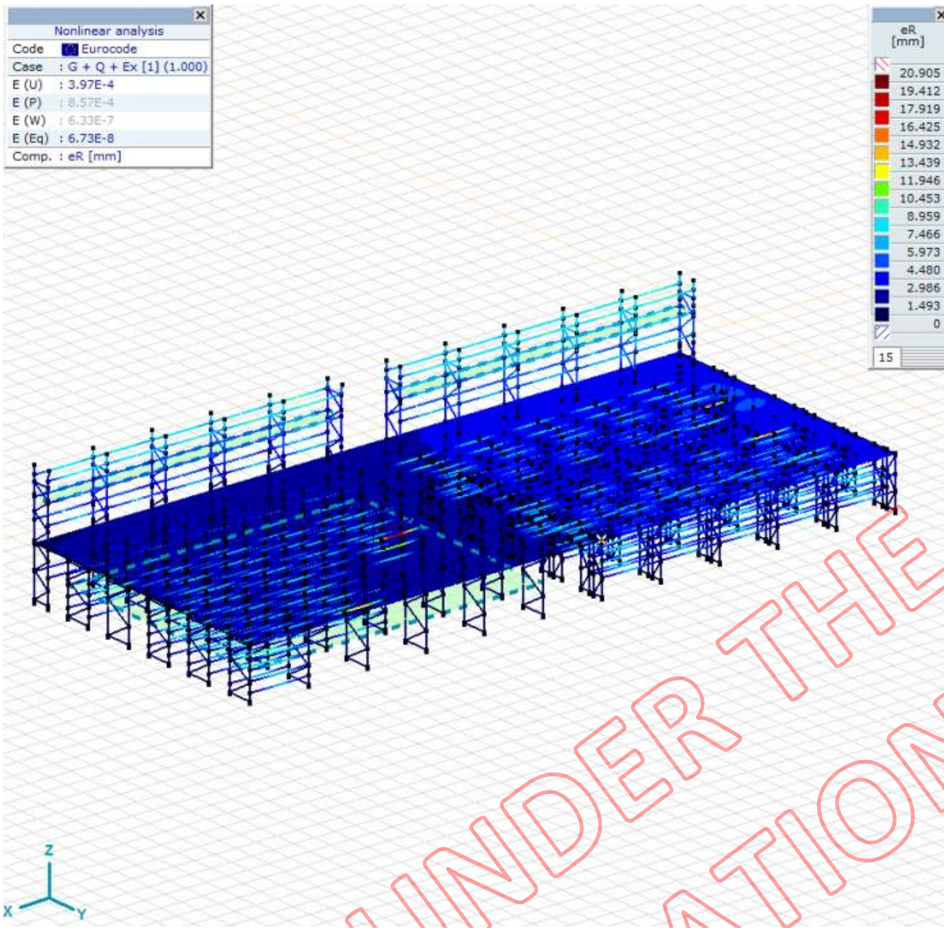


Figure (A7): Across aisle displacement.

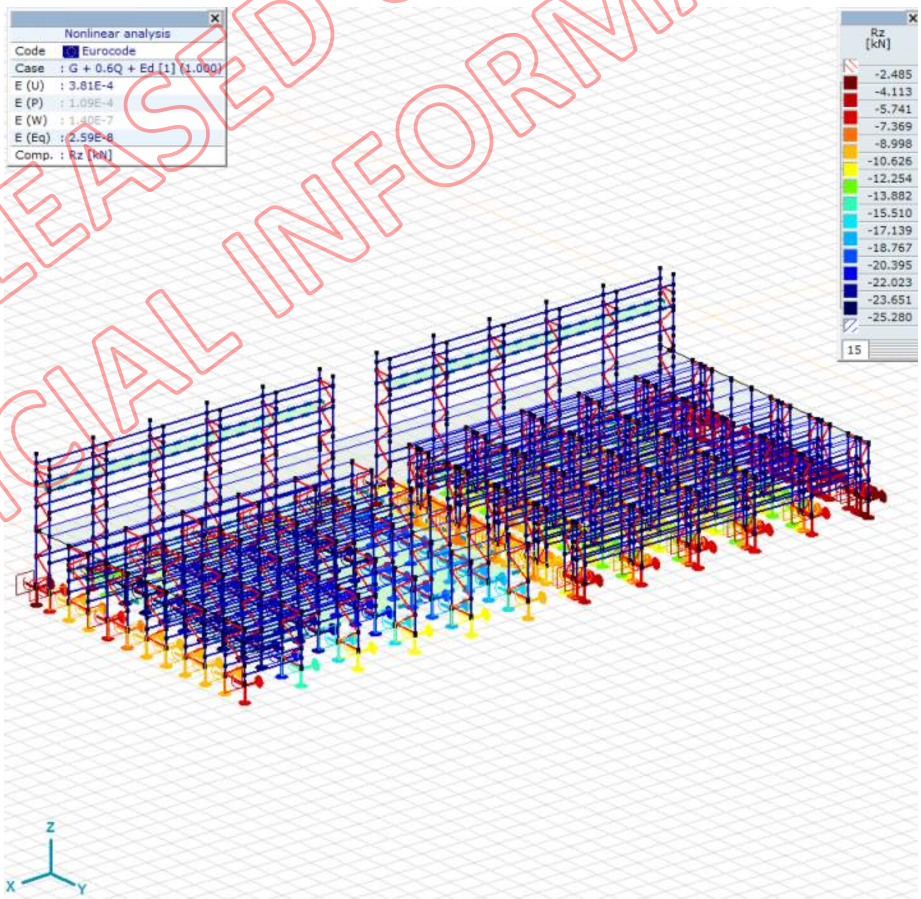


Figure (A8): Down aisle support reactions.

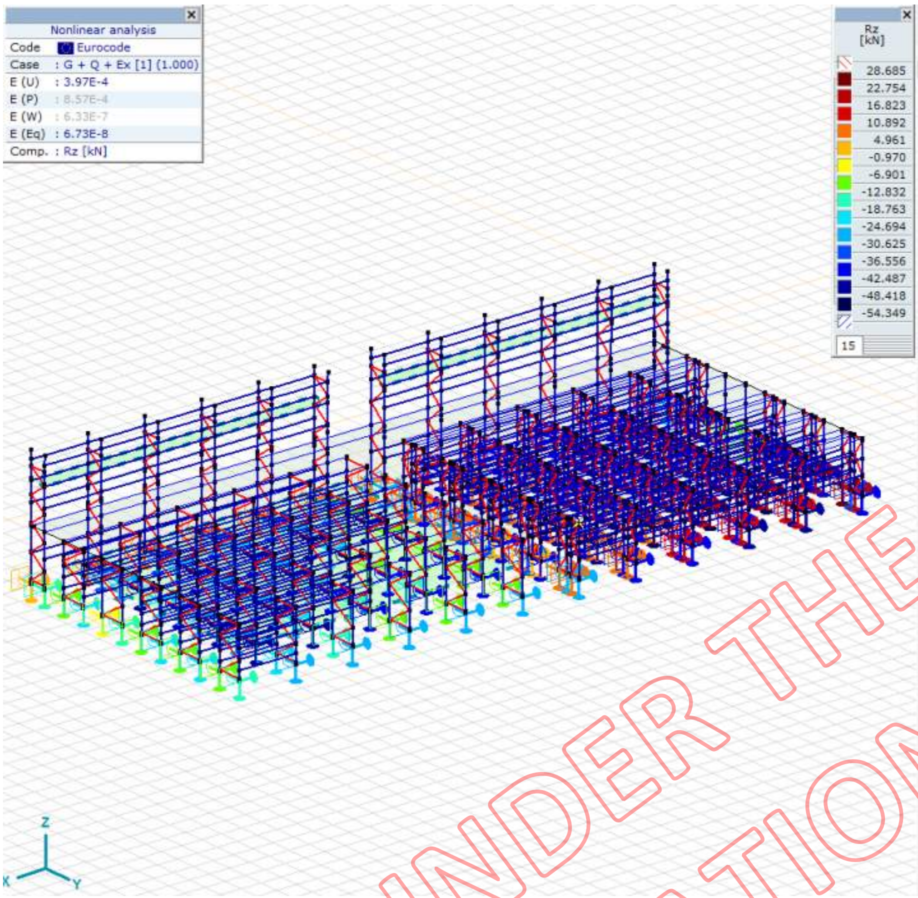


Figure (A9): Across aisle support reactions.

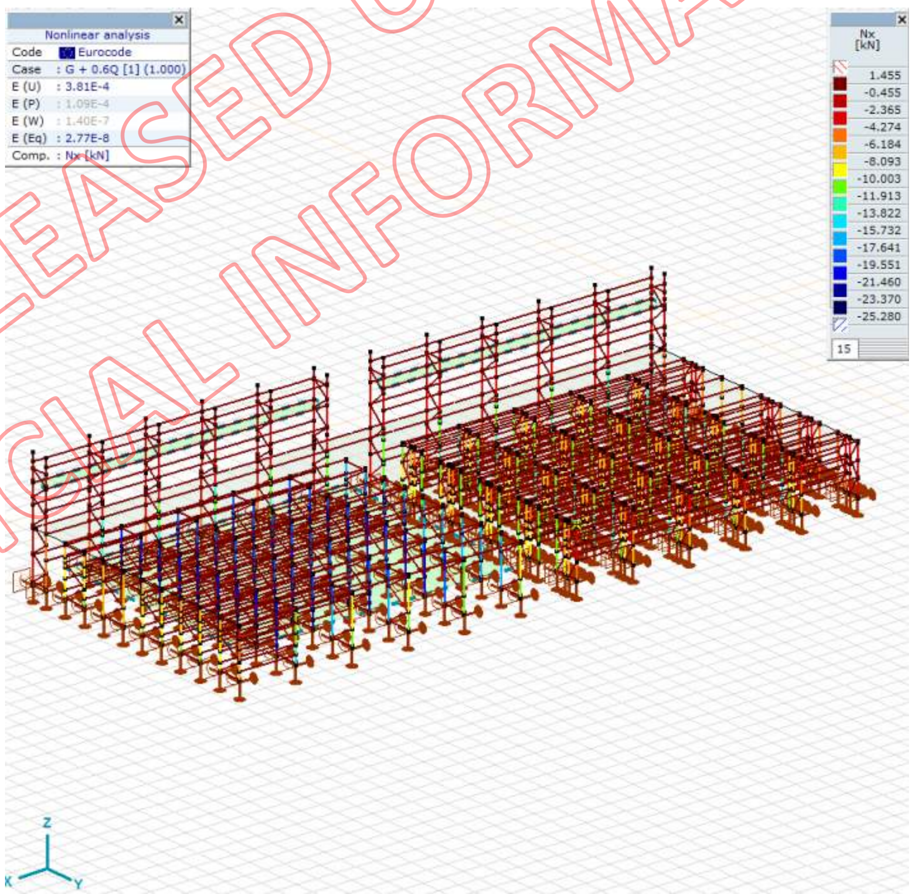


Figure (A10): G + 0.6Q Fire loading, axial loading

Appendix B - Typical AXIS VM Result Screen Shots

(Configuration A shown)

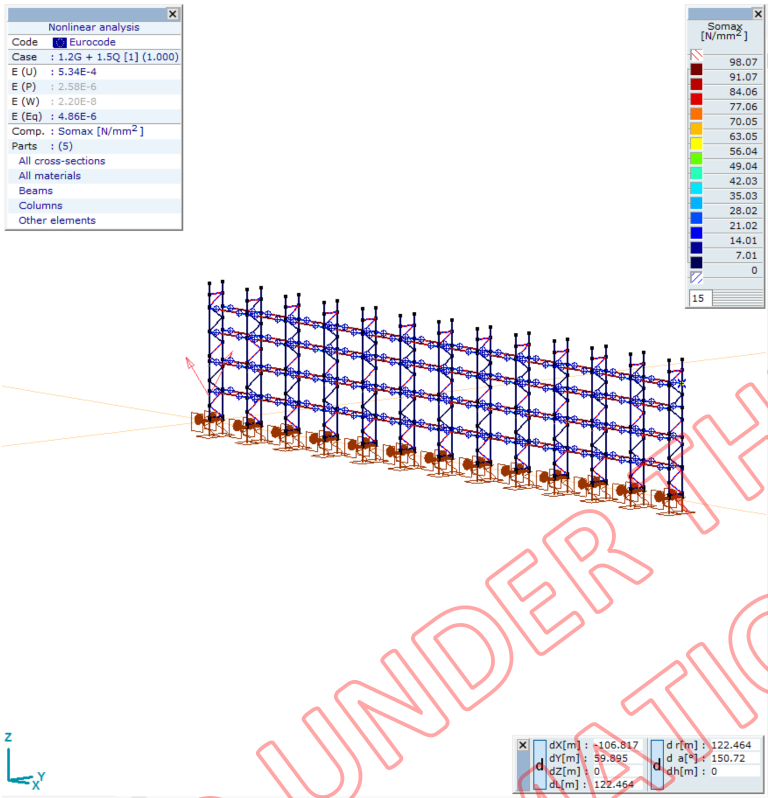


Figure (B1): Load case (1.2G + 1.5Q) stress plot.

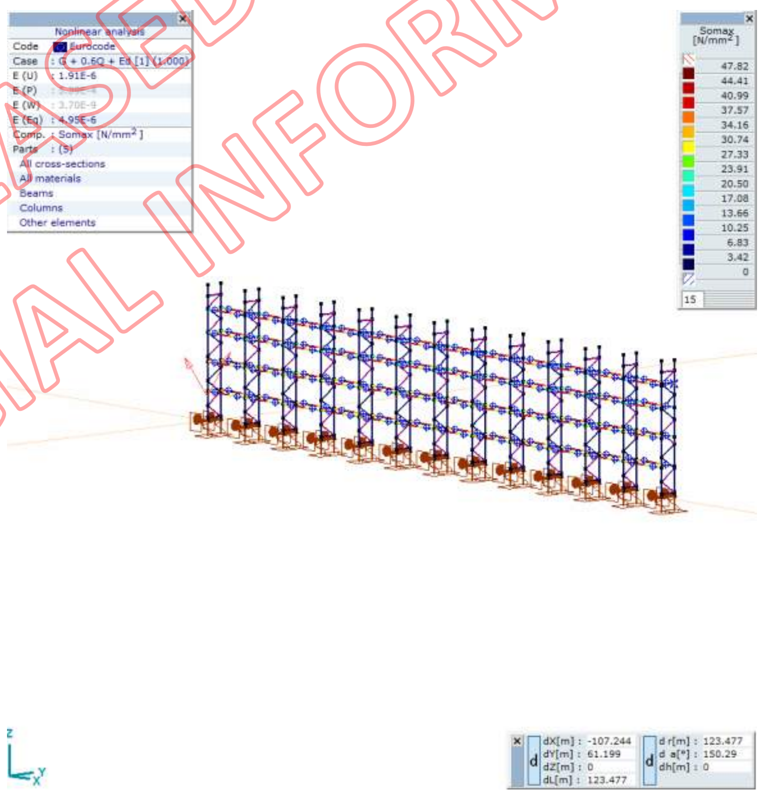


Figure (B2): Load case (G + psic_Q + Eu) down aisle stress plot.

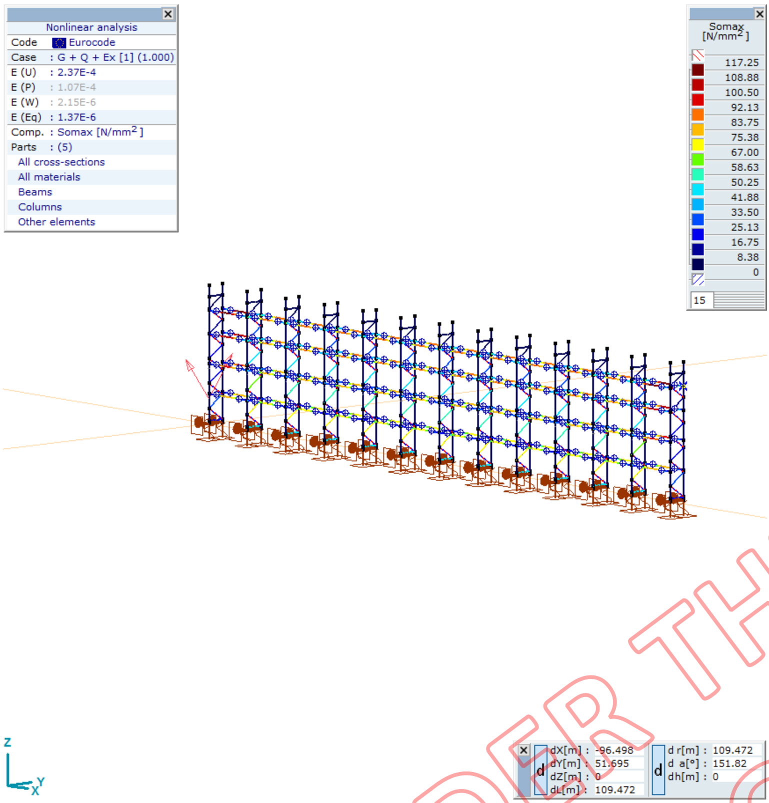


Figure (B3): Load case (G + psic_Q + Eu) across aisle stress plot.

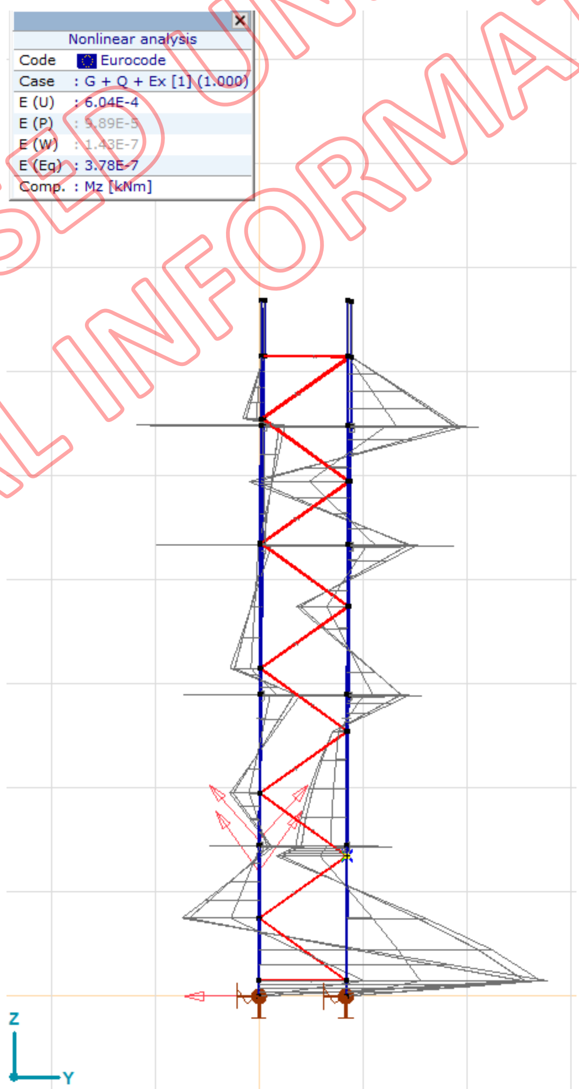


Figure (B4): Across aisle bending moment diagram.

Figure (B5): Down aisle bending moment diagram.

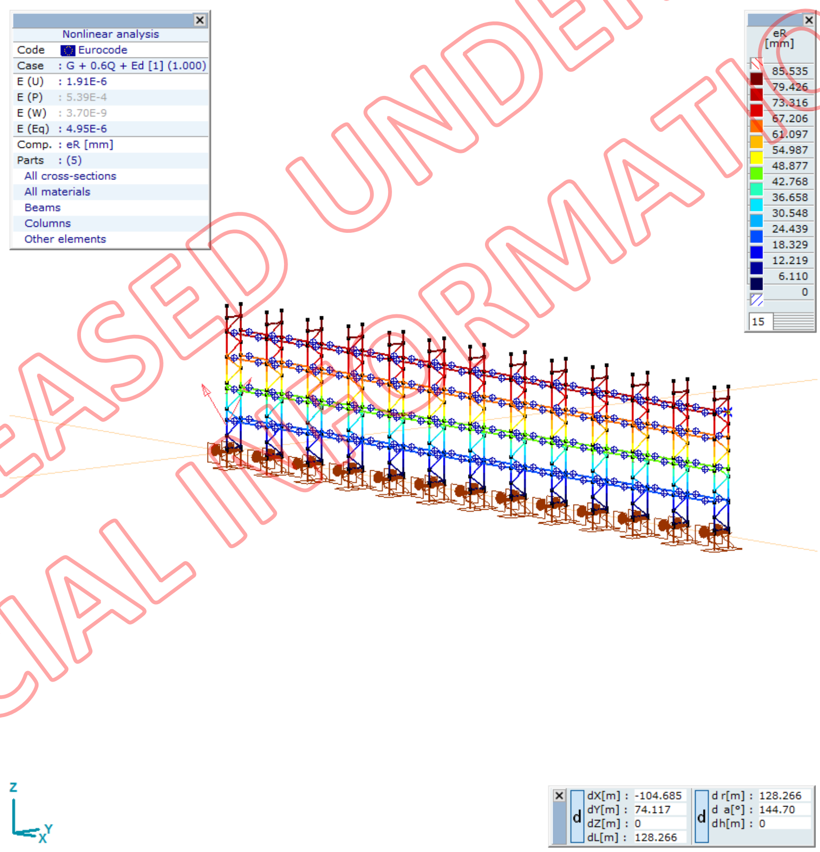


Figure (B6): Down aisle displacement.

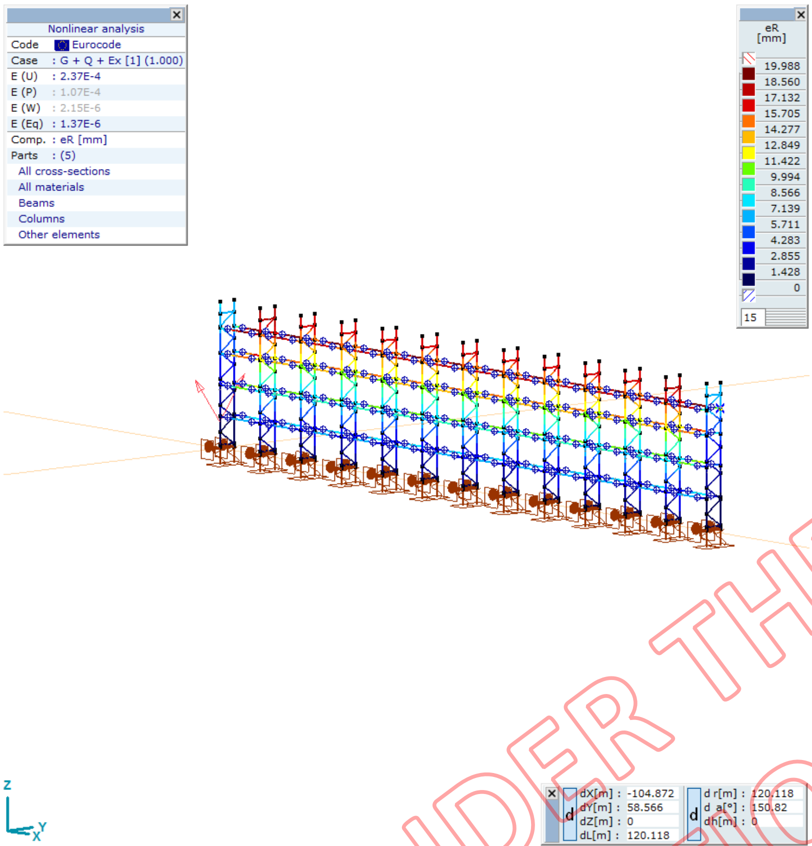


Figure (B7): Across aisle displacement.

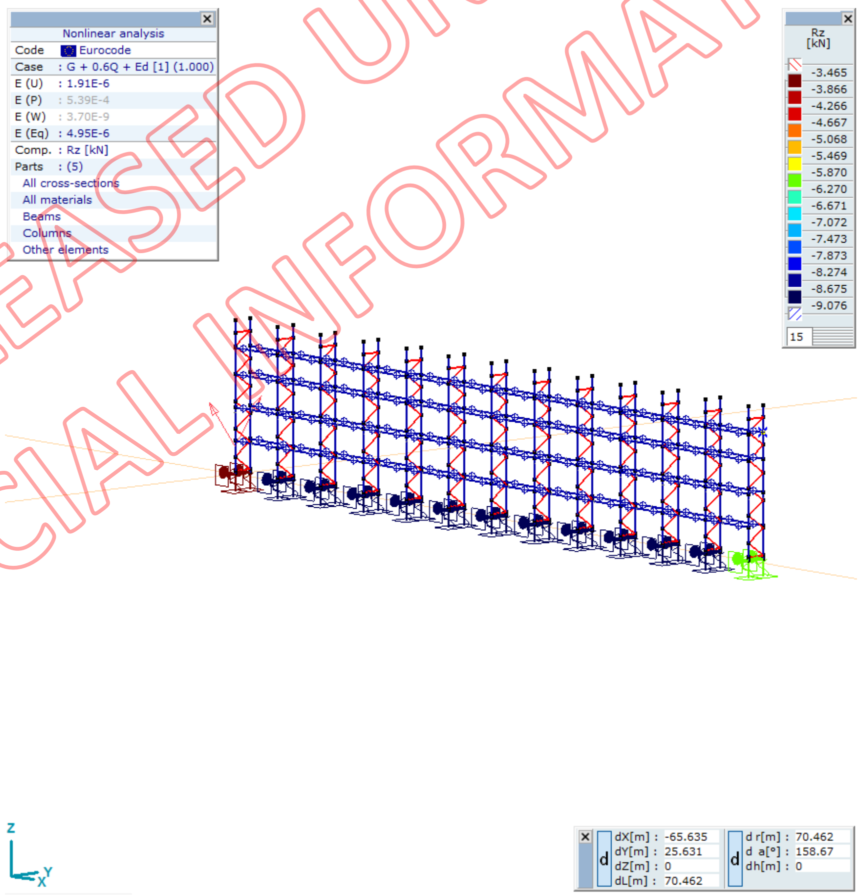


Figure (B8): Down aisle support reactions.

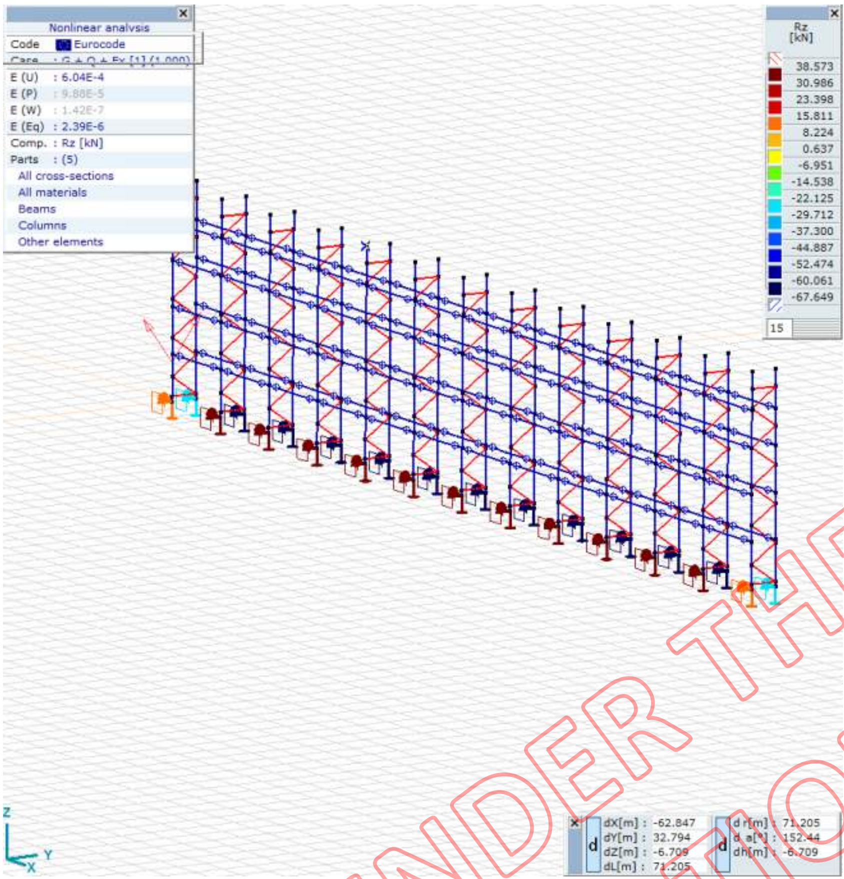
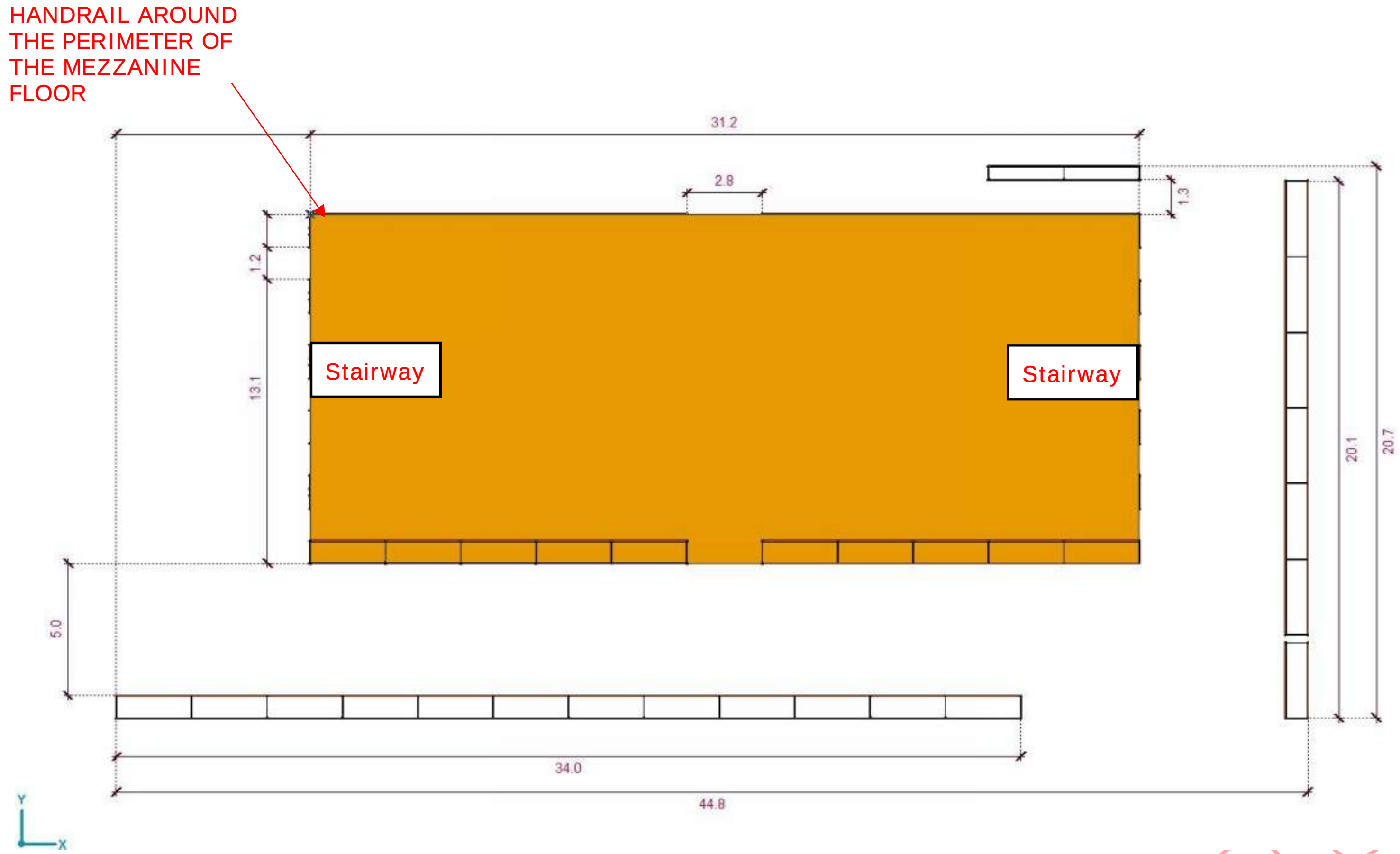
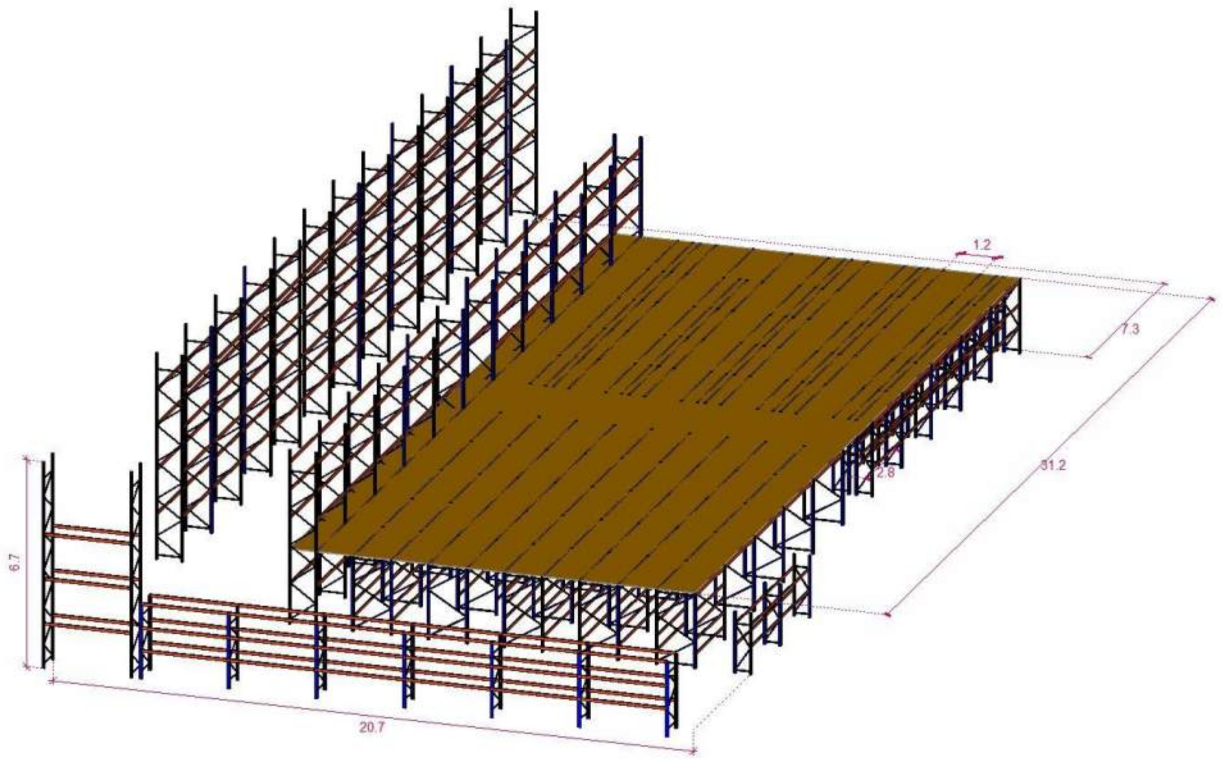
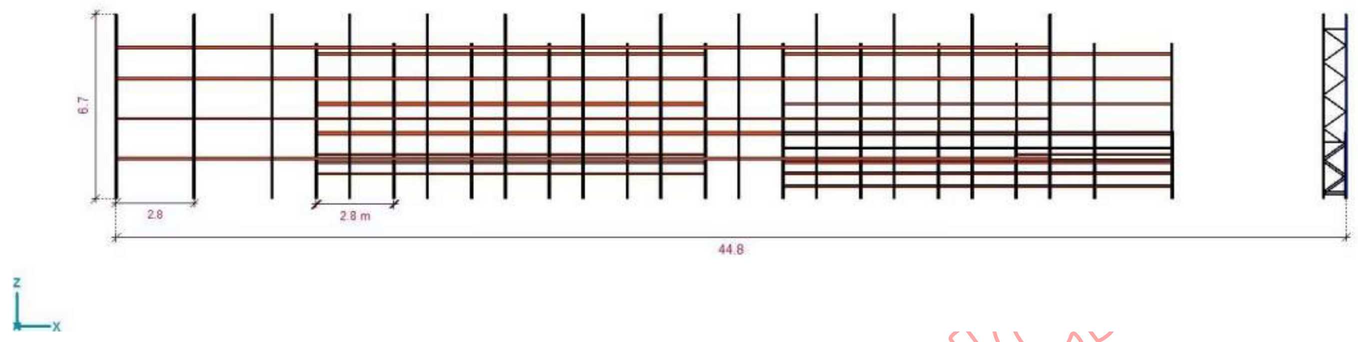


Figure (B9): Across aisle support reactions.

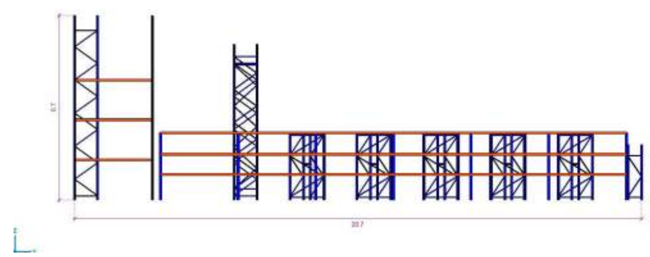


PLAN





FRONT



SIDE

NOTES:
- RACKING AND MEZZANINE FLOOR HAVE BEEN CALCULATED TO MEET THE REQUIREMENTS OF THE NZ BC - B1 FOR STRENGTH.
- LOADS HAVE BEEN CALCULATED TO NZS 1170 STANDARDS FOR MEZZANINE FLOORS AND IN ACCORDANCE WITH THE BRANZ DESIGN GUIDE FOR RACKING.
- UNDER THE DESIGN LOADS, THE STRUCTURES ARE OK IN ACCORDANCE WITH NZS 4600: 2005 - COLD FORMED STEEL STRUCTURES AND NZS 3603: 1993 - TIMBER STRUCTURES
- HANDRAIL AND STAIRWAYS MEET THE STRUCTURAL REQUIREMENTS OF AS 1657: 2013

Loads

Worst Case			Load Case
Shear V	7	kN	G+Q+Ex
Compression Load N (enter as negative)	-67	kN	G+Q+Ex
Moment M	1.1	kNm	G+0.6Q+Ed
Pullout N	38.5	kN	G+Q+Ex
Deflection Ex	20	mm	G+Q+Ex
Deflection Ed	85	mm	G+0.6Q+Ed

Revision	Description	Initial	Date
A	RELEASED FOR APPROVAL	TDR	11/08/16
B	ADDED FLOOR DETAILS	TDR	29/08/16

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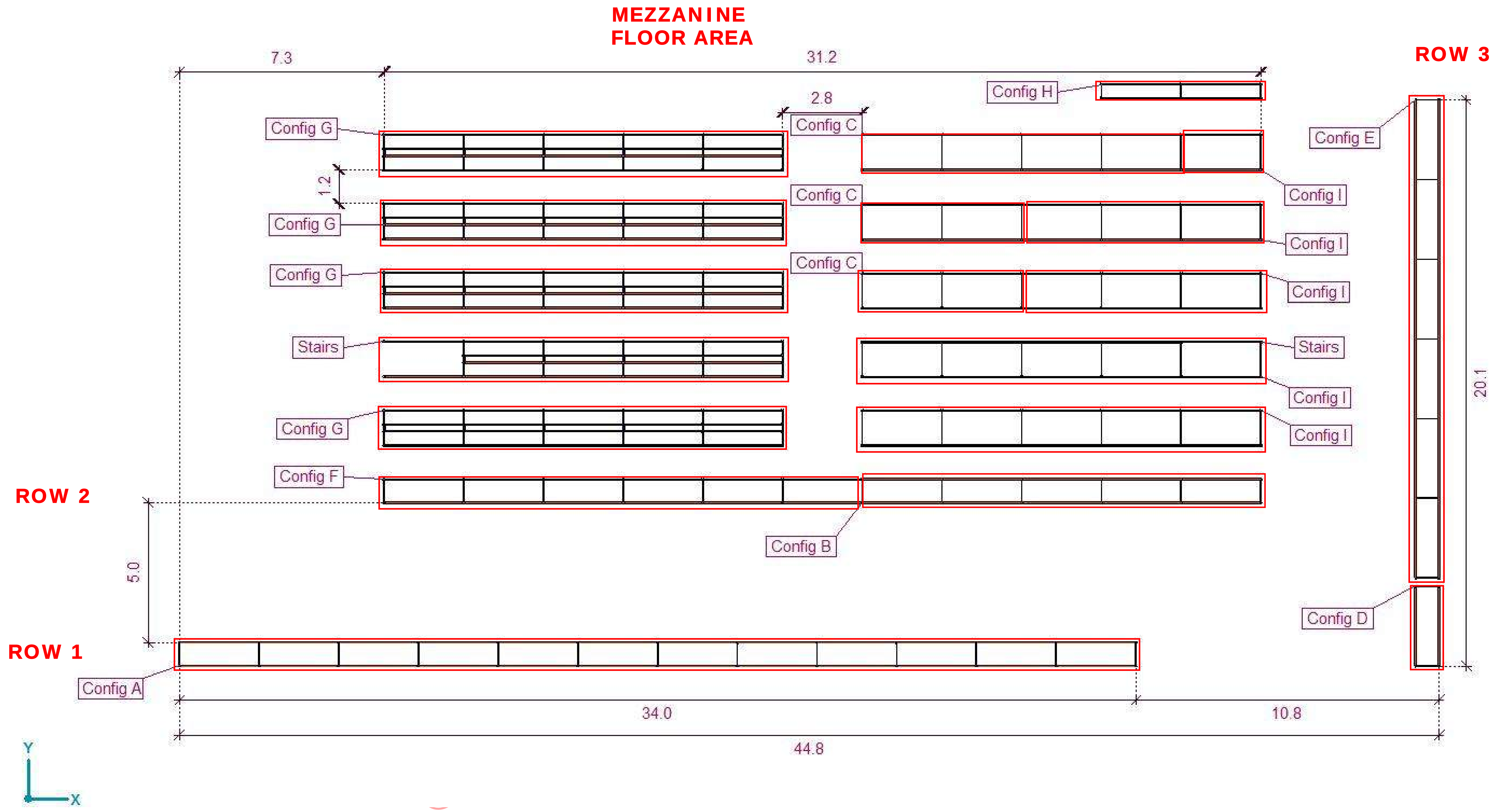
Ph: 03 371 7593
Fax: 03 371 7593

		Date
Designed	FORBES AND DAVIES	11/08/16
Drawn	T RIDDELL	11/08/16
Checked	A MERINO	11/08/16
Approved	M BISHOP	11/08/16

Tolerances (unless specified)	1-100	<1000	>1000
	± 0.5	± 1	± 2
All dim. in mm			

FORBES AND DAVIES
RACKING AT 49 STONELEIGH DRIVE

Project 16081149 | Scale Do not Scale | Sheet 1 of 7
DRG No. 16081149 - 01 | Date 11/08/16 | BVTA3



RACKING LAYOUT

NOTE:
- SEE PAGE 3 AND 4 FOR RACKING CONFIGURATION AND WORKING LOAD LIMITS.
- THE MEZZANINE FLOOR HAS BEEN FOUND TO BE COMPLIANT UNDER A 4.8 kPa FLOOR LOAD AS REQUIRED BY NZS 1170. SEE -07 FOR CROSS SECTION DETAILS

Revision	Description	Initial	Date
A	RELEASED FOR APPROVAL	TDR	11/08/16
B	ADDED FLOOR DETAILS	TDR	29/08/16

492 Moorhouse Ave
Christchurch
www.bvt.co.nz
Ph: 03 371 7593
Fax: 03 371 7593

		Date
Designed	FORBES AND DAVIES	11/08/16
Drawn	T RIDDELL	11/08/16
Checked	A MERINO	11/08/16
Approved	M BISHOP	11/08/16

Tolerances (unless specified)	1-100	<1000	>1000
	± 0.5	± 1	± 2
All dim. in mm			

FORBES AND DAVIES
RACKING AT 49 STONELEIGH DRIVE

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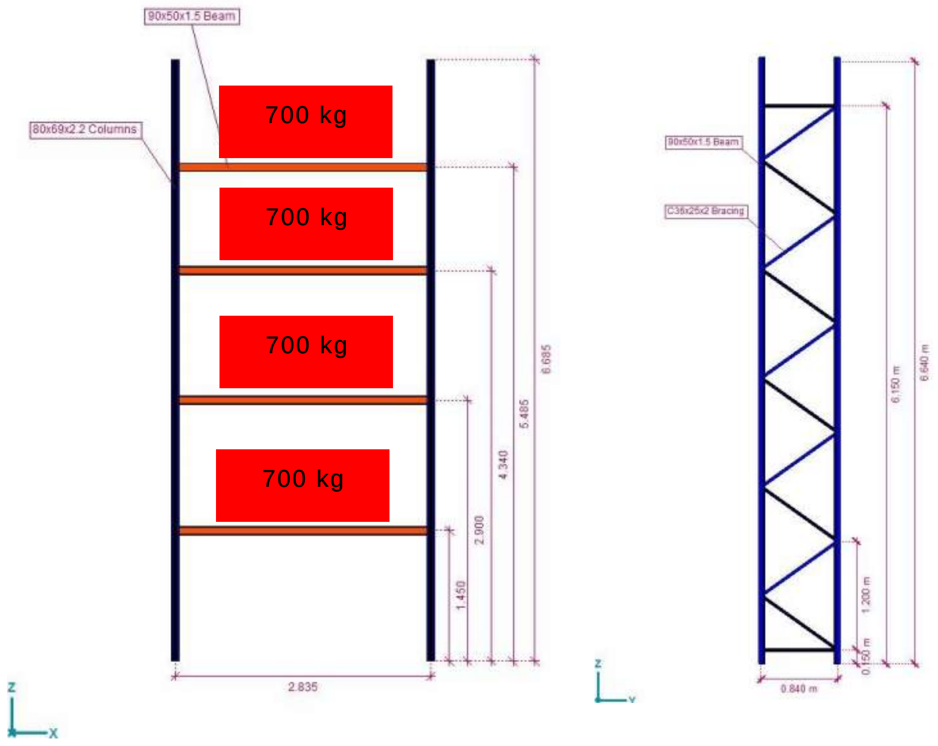
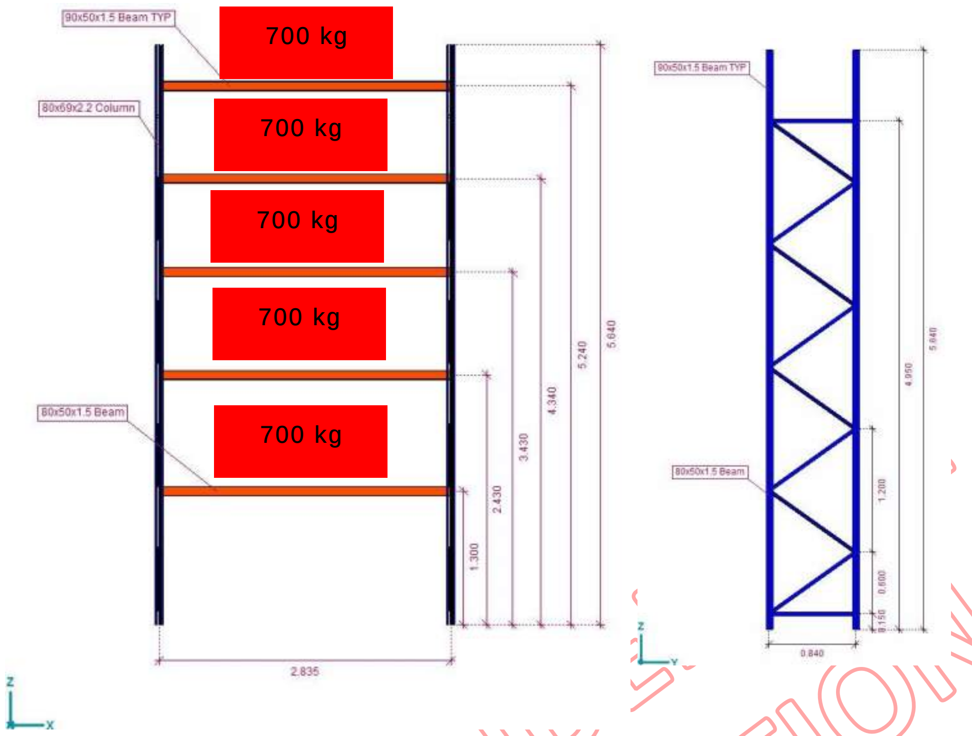
DRG No. 16081149 - 02	Date 11/08/16	BVTA3



BAY A



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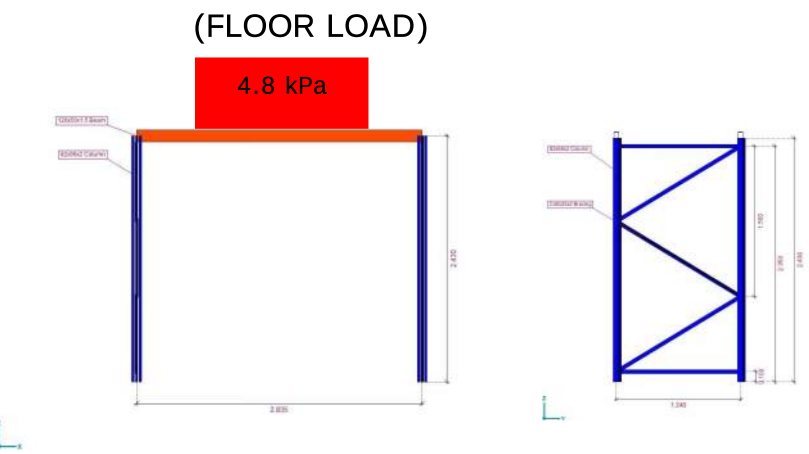
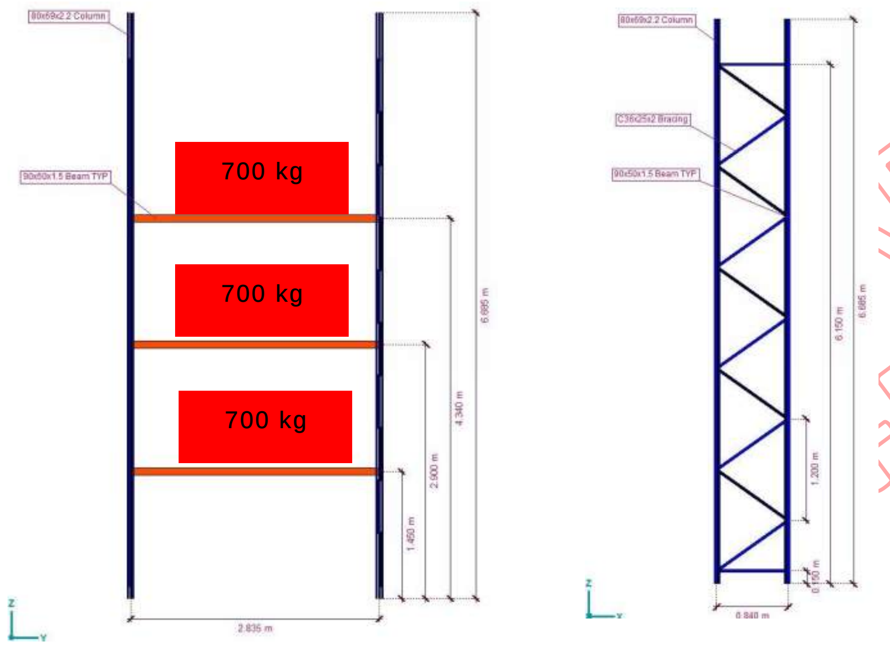
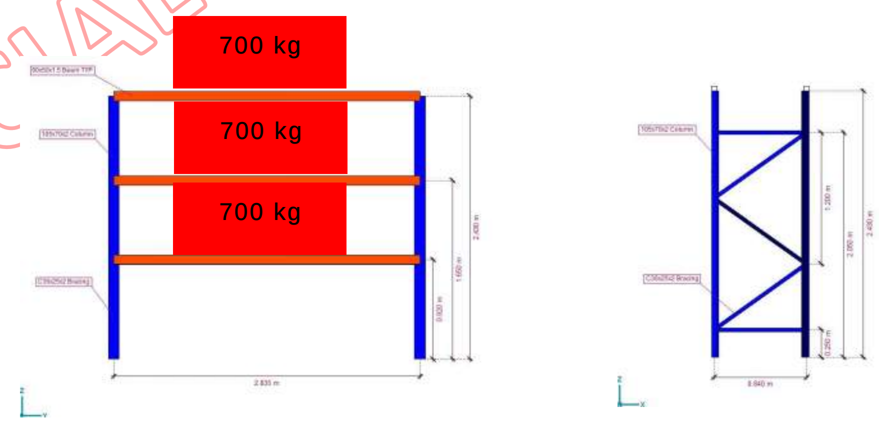


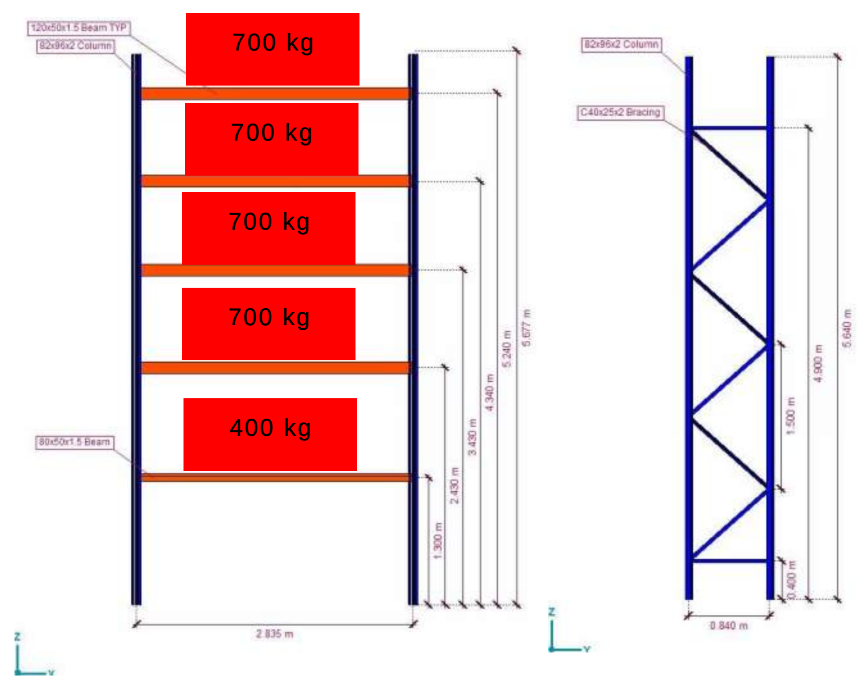
BAY C



BAY D



BAY E



BAY F

Revision	Description	Initial	Date
A	RELEASED FOR APPROVAL	TDR	11/08/16
B	ADDED FLOOR DETAILS	TDR	29/08/16

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Christchurch
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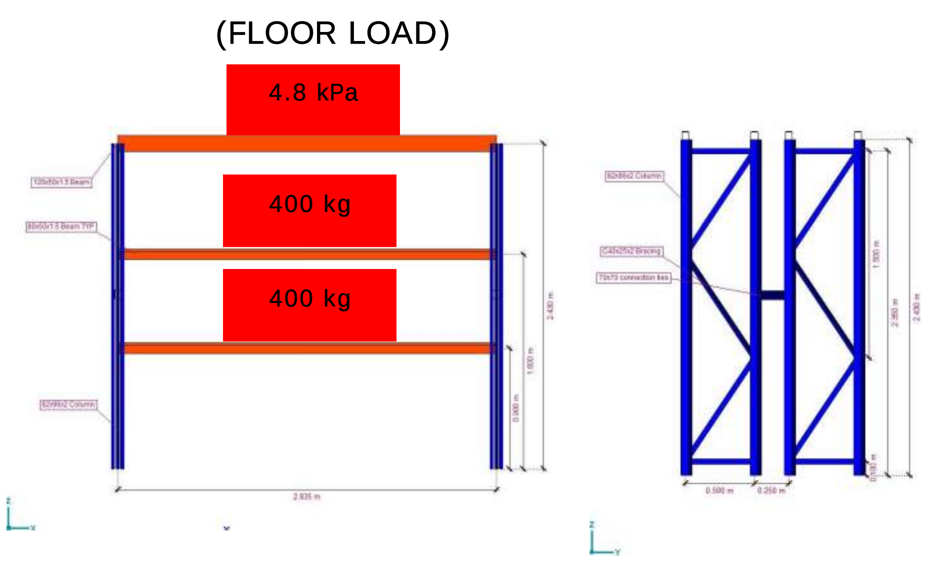
Ph: 03 371 7593
Fax: 03 371 7593

		Date
Designed	FORBES AND DAVIES	11/08/16
Drawn	T RIDDELL	11/08/16
Checked	A MERINO	11/08/16
Approved	M BISHOP	11/08/16

Tolerances (unless specified)	1-100	<1000	>1000
	± 0.5	± 1	± 2
All dim. in mm			

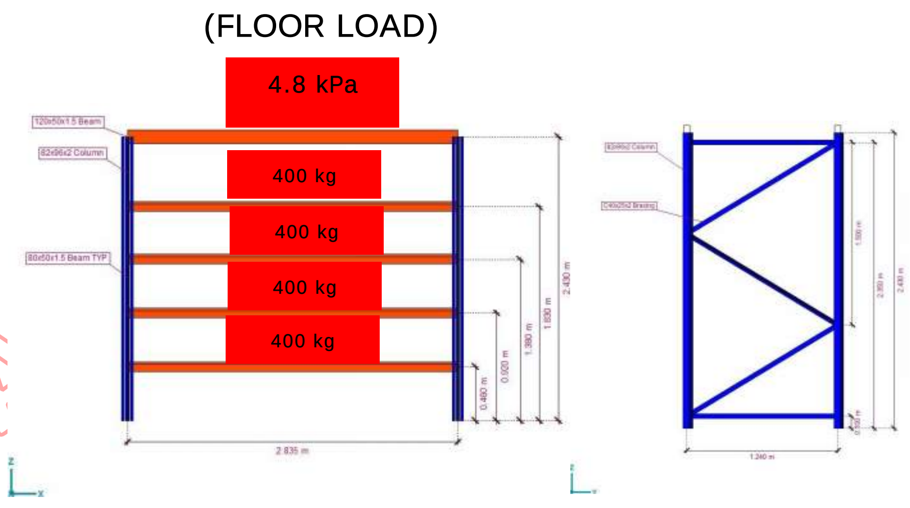
FORBES AND DAVIES
RACKING AT 49 STONELEIGH DRIVE

Project 16081149	Scale Do not Scale	Sheet 3 of 7	
DRG No. 16081149 - 03	Date 11/08/16	BVTA3	



BAY G

BAY H



BAY I

Revision	Description	Initial	Date
A	RELEASED FOR APPROVAL	TDR	11/08/16
B	ADDED FLOOR DETAILS	TDR	29/08/16

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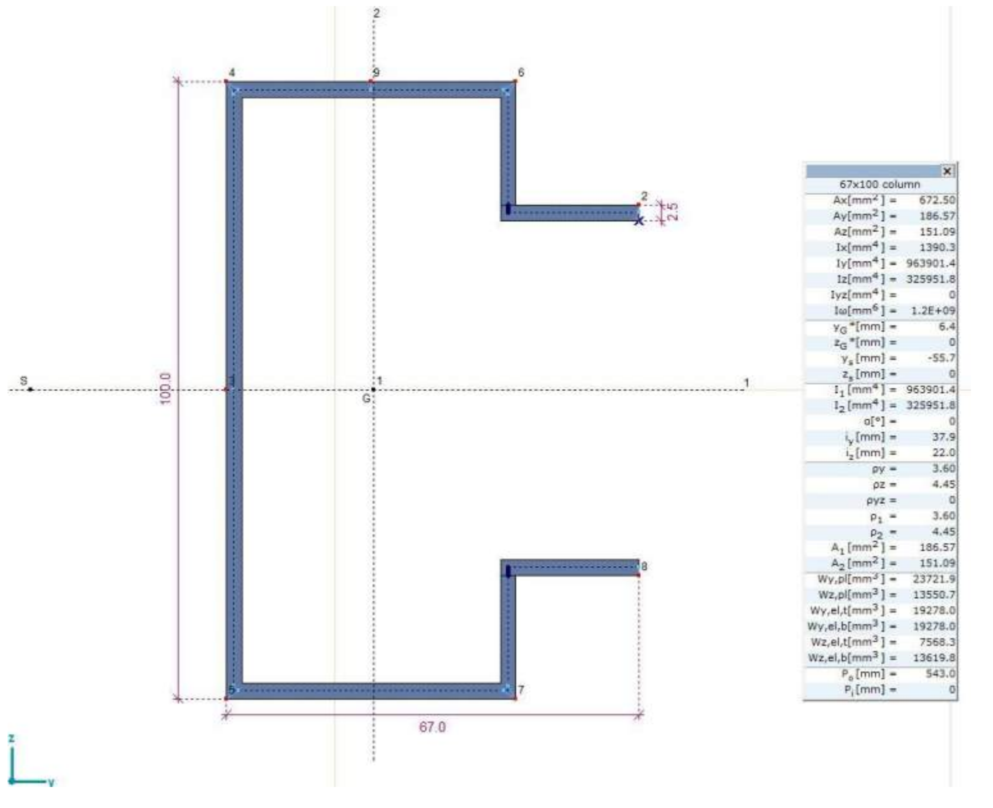
		Date
Designed	FORBES AND DAVIES	11/08/16
Drawn	T RIDDELL	11/08/16
Checked	A MERINO	11/08/16
Approved	M BISHOP	11/08/16

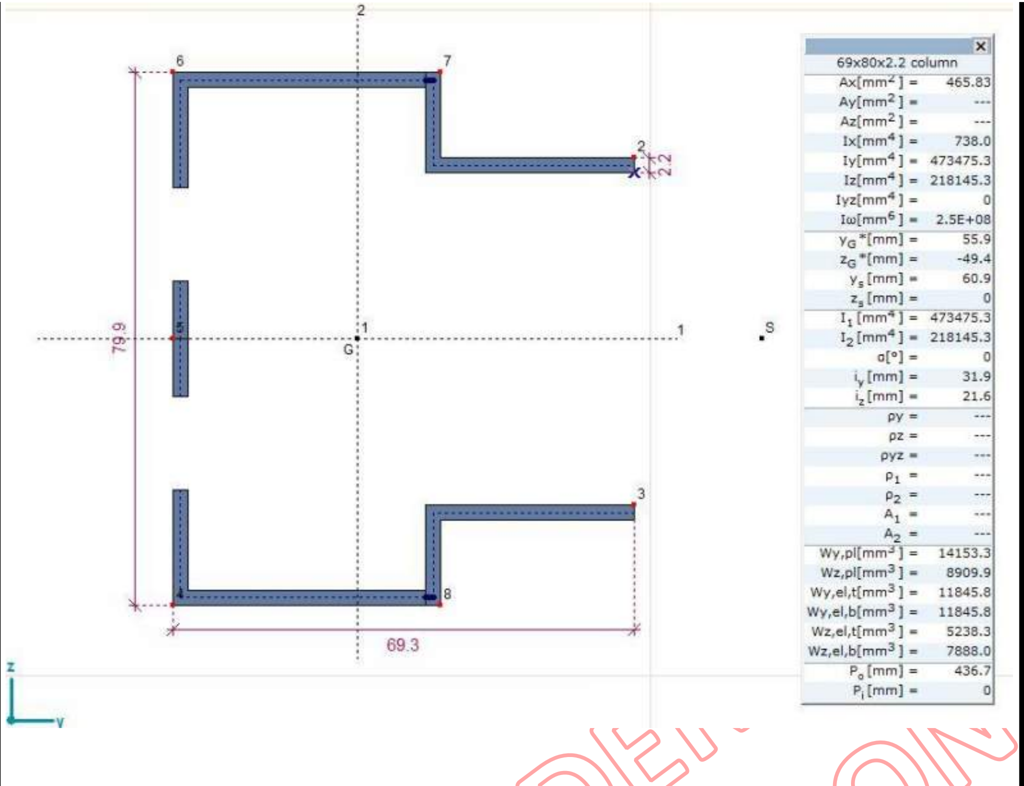
Tolerances (unless specified)	1-100	<1000	>1000
	± 0.5	± 1	± 2
All dim. in mm			

FORBES AND DAVIES
RACKING AT 49 STONELEIGH DRIVE

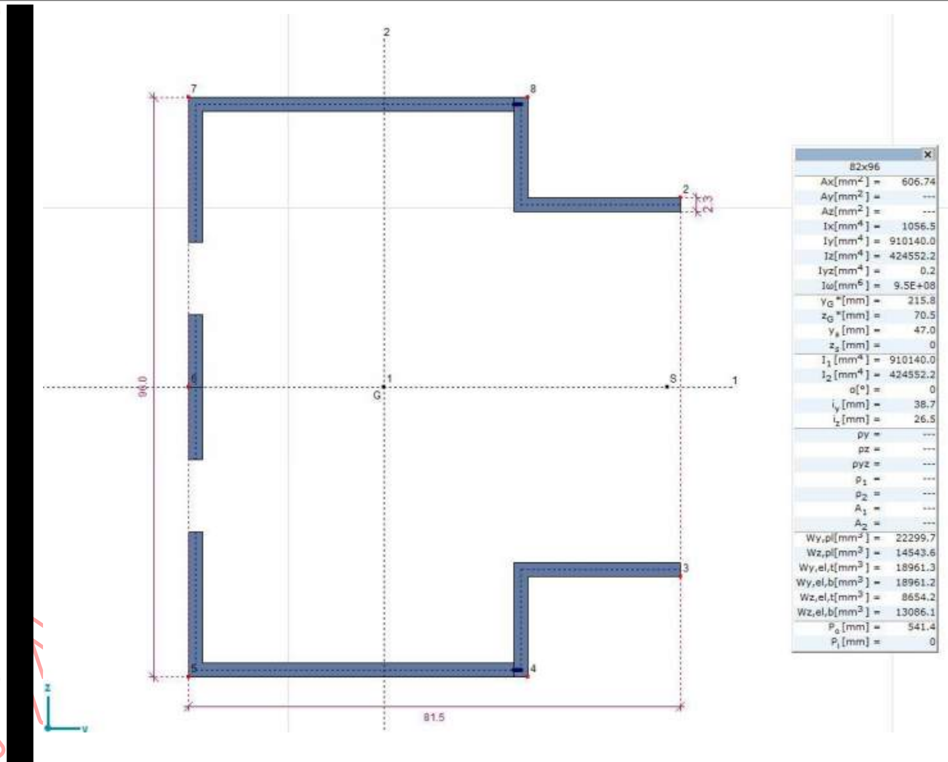
Project 16081149	Scale Do not Scale	Sheet 4 of 7	
DRG No. 16081149 - 04	Date 11/08/16	BVTA3	



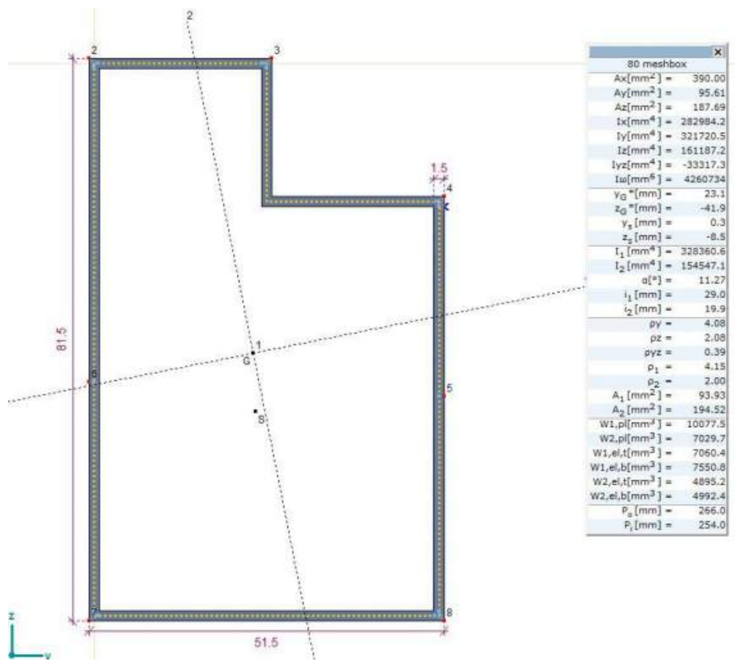
67x100x2
COLUMN



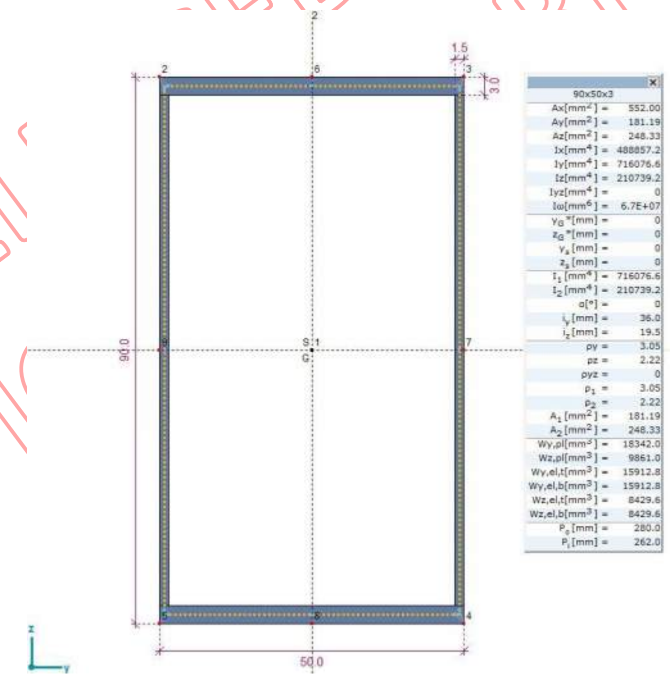
69x80x2.2
COLUMN



82x96x2.2
COLUMN



80x50x1.5
BEAM



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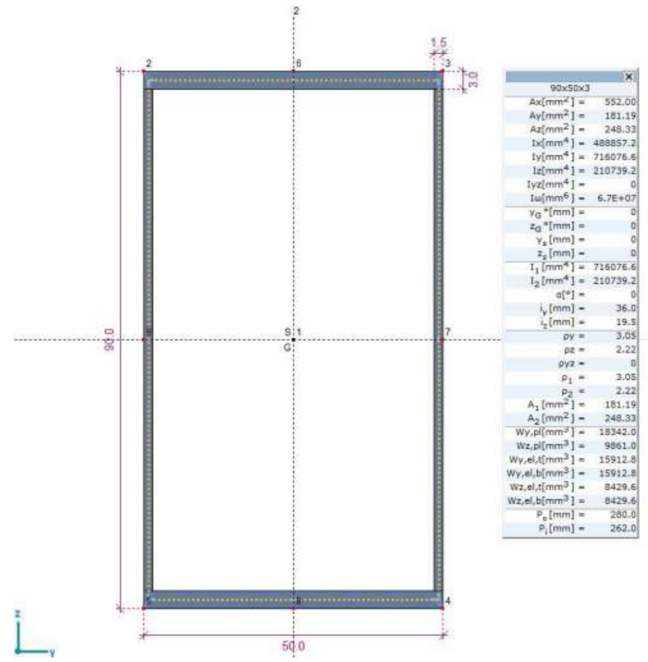
BEAM



90x50x1.5
BEAM

Revision	Description	Initial	Date
A	RELEASED FOR APPROVAL	TDR	11/08/16

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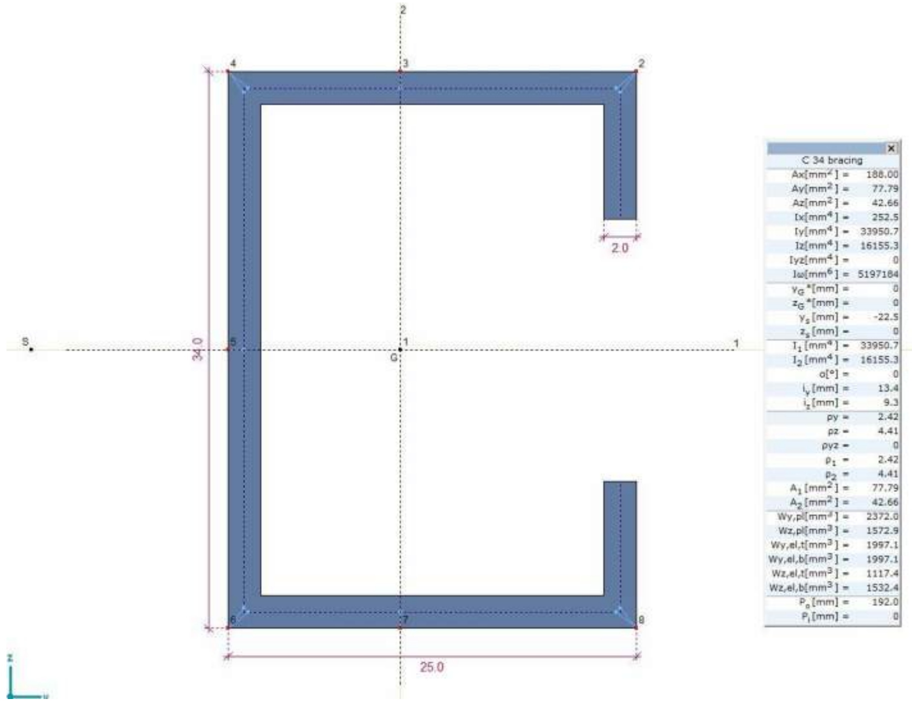
Drawn	T RIDDELL	11/08/16
Checked	A MERINO	11/08/16
Approved	M BISHOP	11/08/16

Tolerances (unless specified)	1-100	<1000	>1000
	± 0.5	± 1	± 2

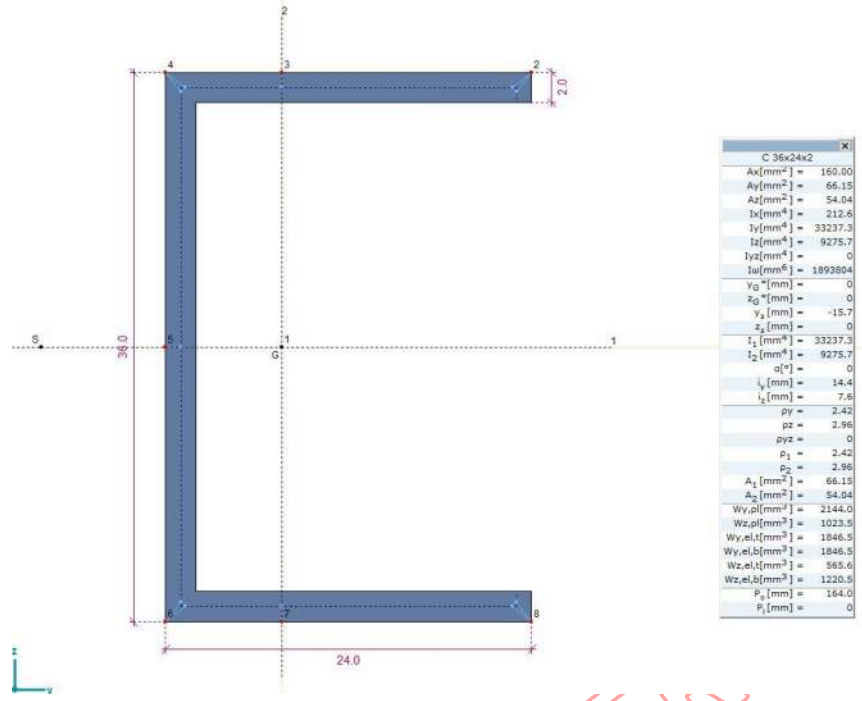
All dim. in mm

FORBES AND DAVIES
RACKING AT 49 STONELEIGH DRIVE

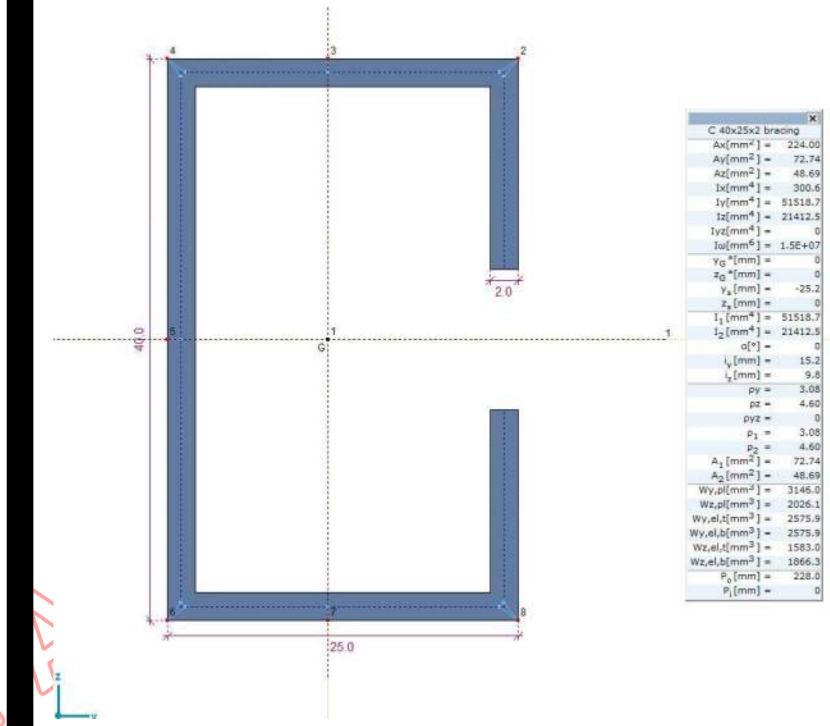
Project 16081149	Scale Do not Scale	Sheet 5 of 7	
DRG No. 16081149 - 05	Date 11/08/16	BVTA3	



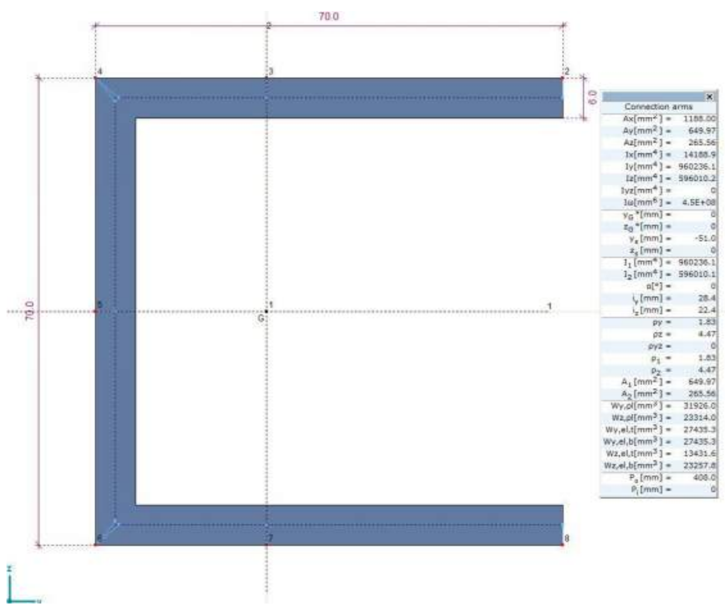
C34x25x2
BRACING



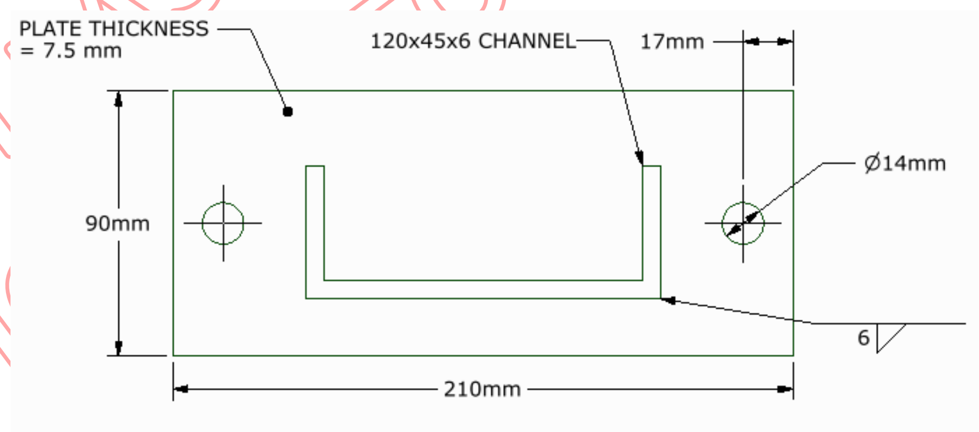
C36x24x2
BRACING



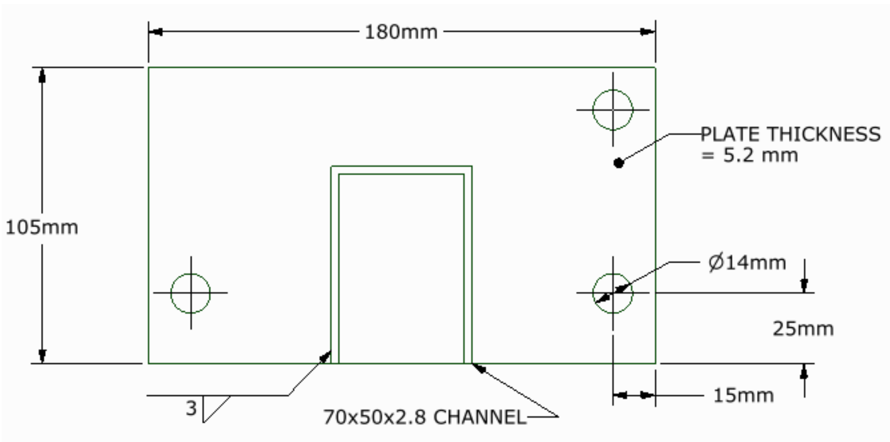
C40x25x2
BRACING



70x70
CONNECTION
ARMS



BASEPLATE 1



BASEPLATE 2

Revision	Description	Initial	Date
A	RELEASED FOR APPROVAL	TDR	11/08/16

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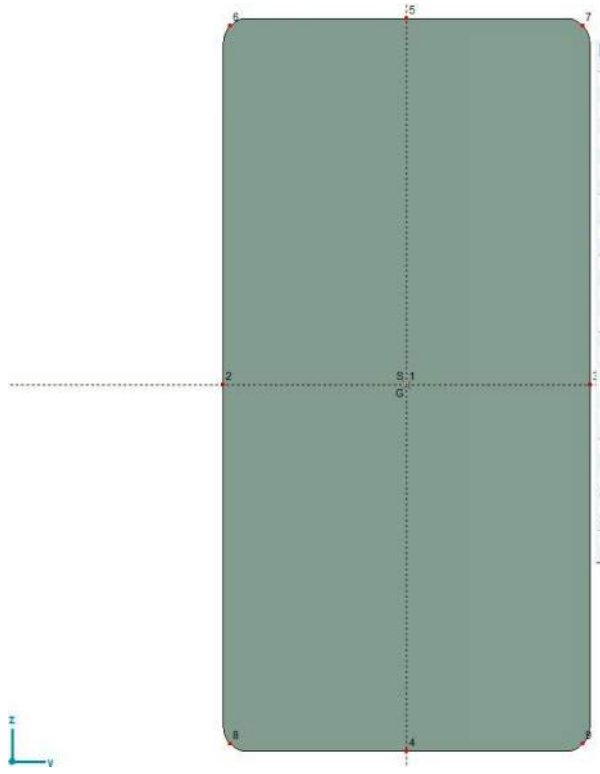
		Date
Designed	FORBES AND DAVIES	11/08/16
Drawn	T RIDDELL	11/08/16
Checked	A MERINO	11/08/16
Approved	M BISHOP	11/08/16

Tolerances (unless specified)	1-100	<1000	>1000
	± 0.5	± 1	± 2
All dim. in mm			

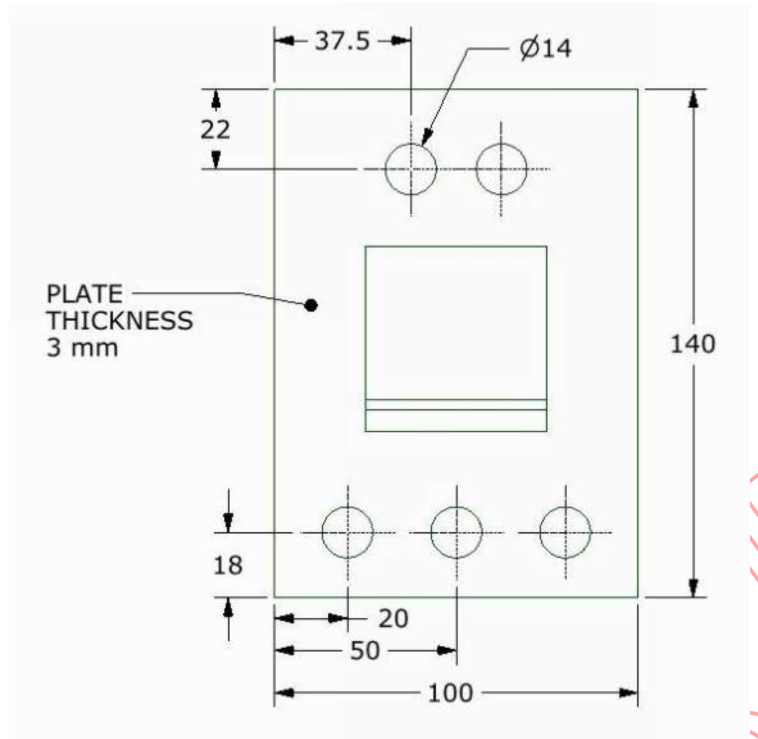
FORBES AND DAVIES
RACKING AT 49 STONELEIGH DRIVE

Project 16081149	Scale Do not Scale	Sheet 6 of 7
DRG No. 16081149 - 06	Date 11/08/16	BVTA3



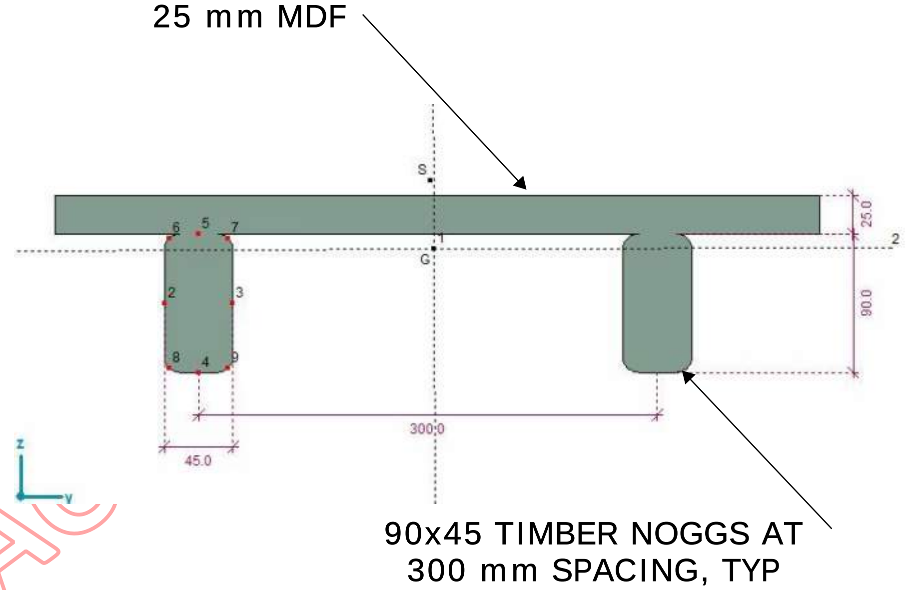
45x90	
Ax[mm²] =	4042.27
Ay[mm²] =	3369.66
Az[mm²] =	3369.18
Ix[mm⁴] =	1875135.0
Iy[mm⁴] =	2718555.0
Iz[mm⁴] =	679751.0
Iyz[mm⁴] =	0
Iω[mm⁶] =	1.7E+08
y_G*[mm] =	0
z_G*[mm] =	0
y_s[mm] =	0
z_s[mm] =	0
I_1[mm⁴] =	2718555.0
I_2[mm⁴] =	679751.0
α[°] =	0
i_y[mm] =	25.9
i_z[mm] =	13.0
ρy =	1.20
ρz =	1.20
ρyz =	0
ρ_1 =	1.20
ρ_2 =	1.20
A_1[mm²] =	3369.66
A_2[mm²] =	3369.18
Wy,pl[mm³] =	90782.3
Wz,pl[mm³] =	45393.7
Wy,el,t[mm³] =	60412.3
Wy,el,b[mm³] =	60412.3
Wz,el,t[mm³] =	30211.2
Wz,el,b[mm³] =	30211.2
P_o[mm] =	264.8
P_i[mm] =	0

90x45 Timber



SMALL BASEPLATE

NOTE BOLT MUST BE AT LEAST 90mm M10 TRUBOLT



MEZZANINE FLOOR CROSS SECTION

Revision	Description	Initial	Date
A	RELEASED FOR APPROVAL	TDR	11/08/16
B	ADDED FLOOR DETAILS	TDR	29/08/16

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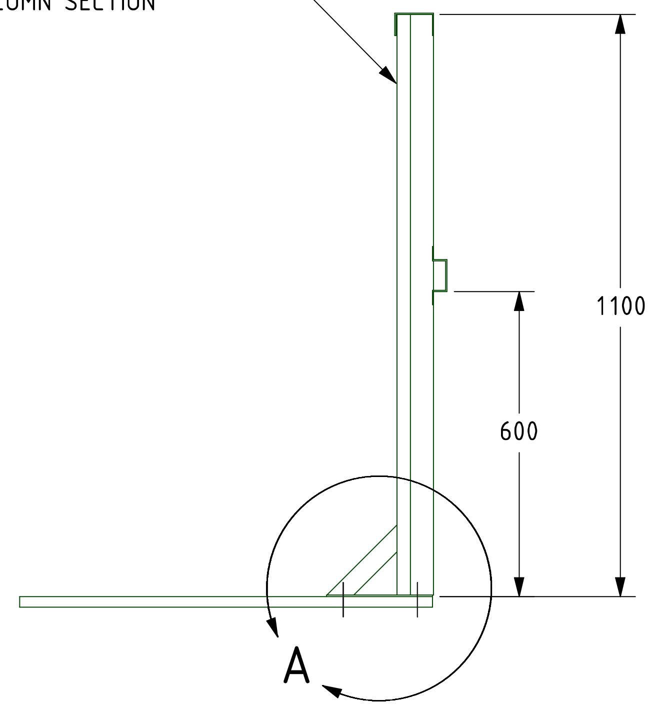
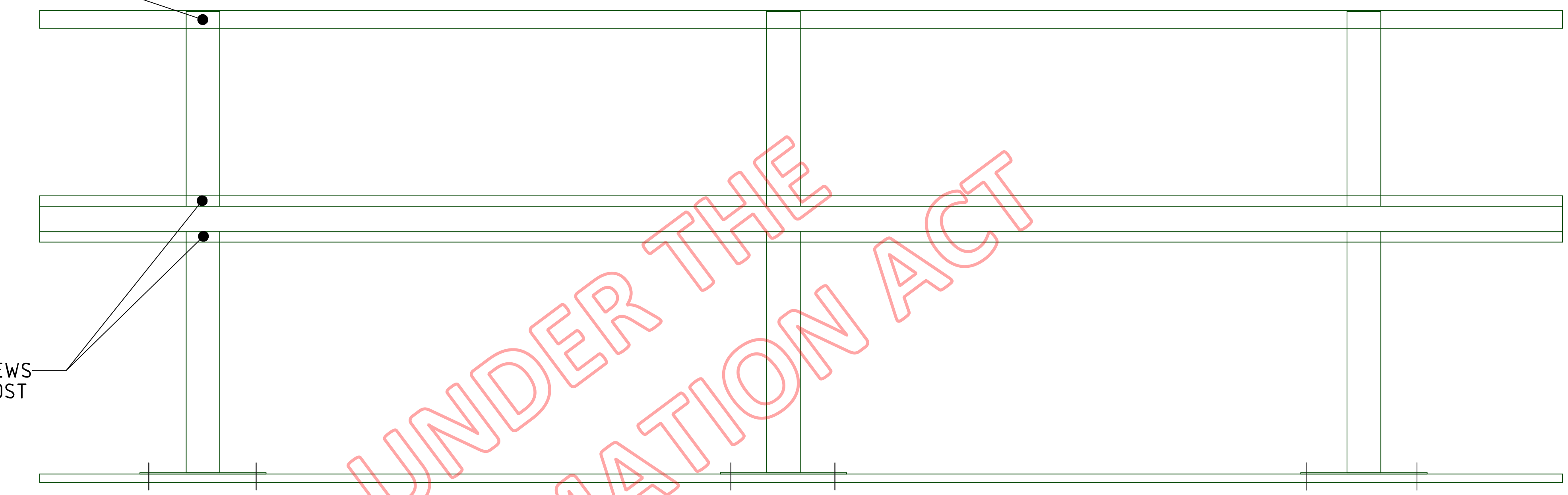
		Date
Designed	FORBES AND DAVIES	11/08/16
Drawn	T RIDDELL	11/08/16
Checked	A MERINO	11/08/16
Approved	M BISHOP	11/08/16

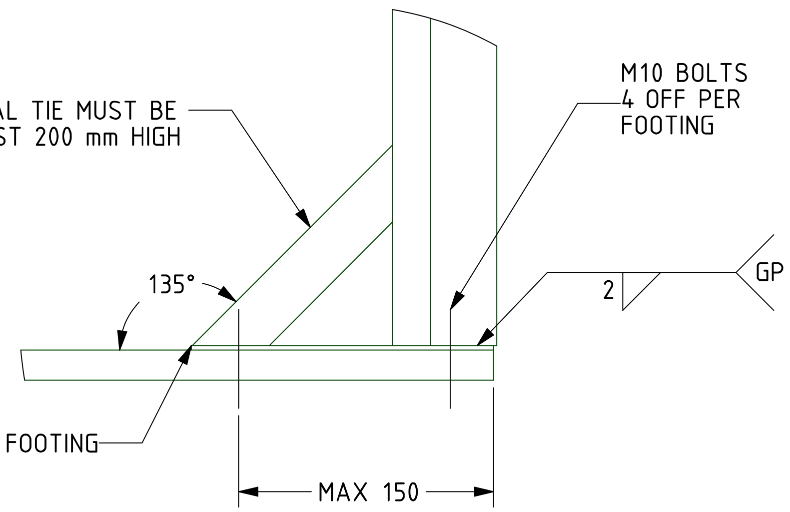
Tolerances (unless specified)	1-100	<1000	>1000
	± 0.5	± 1	± 2
All dim. in mm			

FORBES AND DAVIES
RACKING AT 49 STONELEIGH DRIVE

Project 16081149	Scale Do not Scale	Sheet 6 of 7	
DRG No. 16081149 - 07	Date 11/08/16	BVTA3	







FOOTING DETAIL

NOTES:
1. GUARDRAIL ASSEMBLY COMPLIANT TO STRENGTH
REQUIREMENTS OF AS 1657:2013

Revision	Description	Initial	Date
A	RELEASED FOR APPROVAL	LJB	29/8/16
B	ADDED DIAGONAL TIES	TDR	30/08/16

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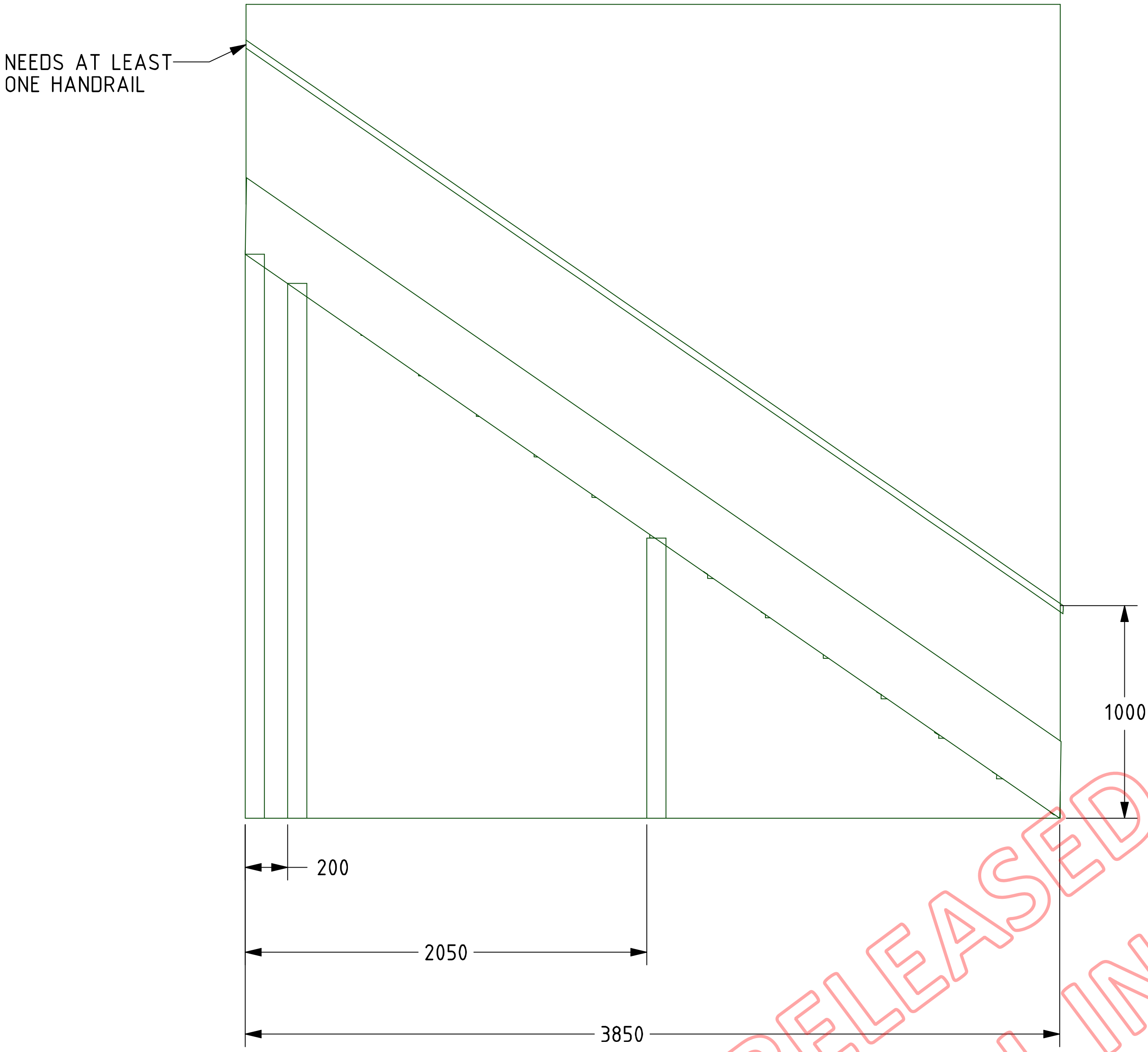
		Date
Designed	FORBES AND DAVIES	
Drawn	L BUTLER	29/8/16
Checked	T RIDDELL	30/8/16
Approved	-	

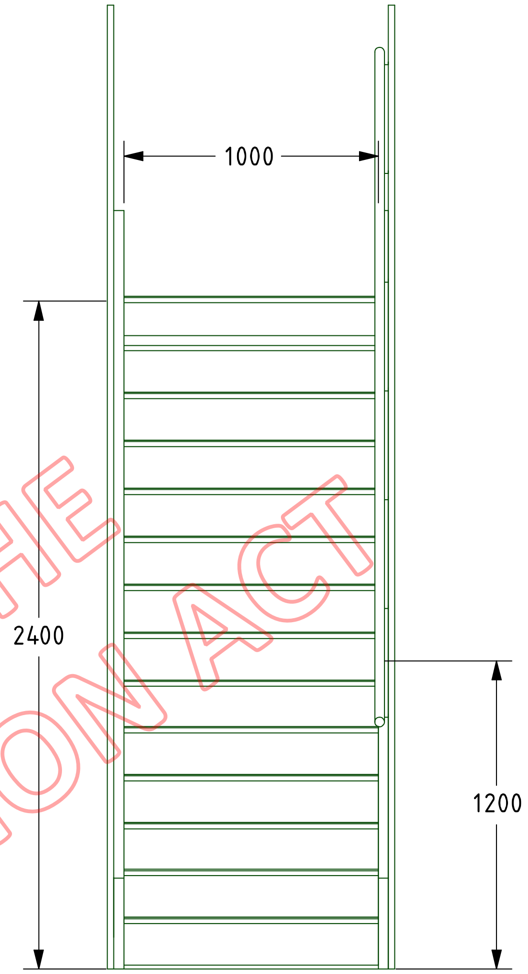
Tolerances (unless specified)	1-100	<1000	>1000
	± 0.5	± 1	± 2
All dim. in mm			

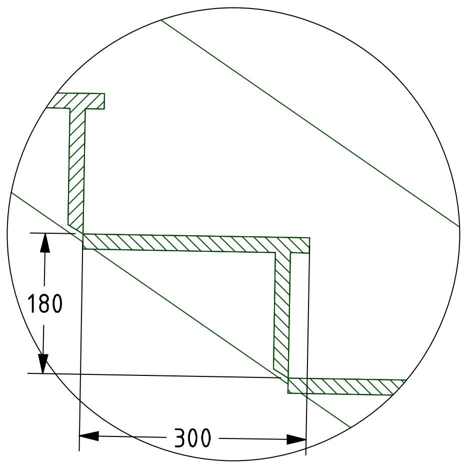
FORBES AND DAVIES

RACKING AT 49 STONELEIGH DRIVE

Project 16081149	Scale Do not Scale	Sheet 2 OF 2	
DRG No. 16081149-08	Date 29/8/16	BVTA3	







TREAD DETAIL

NOTES:
1. SUPPORT COLUMNS CONSTRUCTED OF 90 x 45 TIMBER STUDS
2. STAIRWAY ASSEMBLY COMPLIANT TO STRENGTH
REQUIREMENTS OF AS 1657:2013

Revision	Description	Initial	Date
A	RELEASED FOR APPROVAL	LJB	29/8/16
B	ADDED HANDRAIL	TDR	30/8/16

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		Date
Designed	FORBES AND DAVIES	
Drawn	L BUTLER	29/8/16
Checked	T RIDDELL	30/8/16
Approved	-	

Tolerances (unless specified)	1-100	<1000	>1000
	± 0.5	± 1	± 2
All dim. in mm			

FORBES AND DAVIES

RACKING AT 49 STONELEIGH DRIVE

Project 16081149	Scale Do not Scale	Sheet 1 OF 2
DRG No. 16081149-09	Date 29/8/16	BVTA3

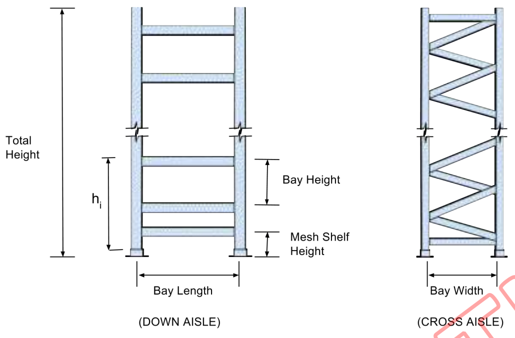
16081149 - Forbes and Davies Ltd - racking config A - Loading Summary Page

BVT Engineering Professional Services

Calc sheet No.		OF	
ENGINEER:	Tim Riddell	JOB NUMBER	16081149
CLIENT:	Forbes and Davies	DATE	12/08/2016
RACK CONFIGURATI	A		

BVT Racking Load Calculation Summary

Inputs	(green)
Outputs	(yellow)
AXIS VM Inputs	(blue)



STANDARD RACKING CONFIGURATION

Racking Design Data

Location	Christchurch
Importance Level	1
Design Working Life	25 years
Design Method - B1/VM1, BRANZ D.G	ULS

Configuration and Loading

Racking Type	Standard	
Base Footing Plate Type	Heavy-duty	
Bay Length Down Aisle	2.835	m
Bay Length Cross Aisle	0.84	m
Effective Column Length Down Isle	0.986	m
Effective Column Length Cross Isle	1.2	m
Pallet Weight	350	kg
Pallets per Bay	2	
Mesh Shelf Weight (if applicable)	0	kg/bay

Cross Section Details

Beam	N/A
Beam 2 (if present)	N/A
Column	90x67x2
Bracing	C36X24X2

Level	Level Mass [kg/bay]	hi [m]		Q Surface Load (kPa)
8	0	0	Standard Bay	0
7	0	0	Standard Bay	0
6	0	0	Standard Bay	0
5	0	0	Standard Bay	0
4	700	6.2	Standard Bay	2.883597884
3	700	5.04	Standard Bay	2.883597884
2	700	3.6	Standard Bay	2.883597884
1	700	2.175	Standard Bay	2.883597884
0 - Ground	0	0		0

Vibrational Analysis

Loading conditions to determine natural frequency

Frequency Anaylsis of:	G	Q
down aisle (loading)	1	0.536
cross aisle (loading)	1	0.67

Period/Frequency	Period (determi	Frequency(Hz)
down aisle	1.314	0.7610350076
cross aisle	0.519	1.926782274

Earthquake Loading

NZS1170.5 Seismic Design Parameters

Hazard Factor [Z]	0.3
Distance to Nearest Fault [D] (km)	20
Site Subclass Soil	D
Spectral Shape Factor [Ch(T) d]	1.62
Spectral Shape Factor [Ch(T) x]	2.97
Elastic Spectra Hazard Factor [C(T) d]	0.17
Elastic Spectra Hazard Factor [C(T) x]	0.31
Structural Ductility [mu x]	1.25
Structural Ductility [mu d]	2
Structural Performance Factor [Sp x]	0.925
Structural Performance Factor [Sp d]	0.7
Inelastic Spectrum Scaling Factor [kmu x]	1.25
Inelastic Spectrum Scaling Factor [kmu d]	2
Horizontal Design Action Coefficient [Cd(T)d]	0.059
Horizontal Design Action Coefficient [Cd(T)x]	0.231

ANSYS Loads

Level	Surface loads kPa Eu Down Isle	Eu Cross Isle	Q (live load)
8	0	0	0
7	0	0	0
6	0	0	0
5	0	0	0
4	0.1411256204	0.7098388781	2.883597884
3	0.1160326828	0.5821190895	2.883597884
2	0.08283031385	0.4156046261	2.883597884
1	0.05155898395	0.2569767358	2.883597884
Ground - 0	0	0	0

Load Case	Load Combinations
static	1.2G + 1.5Q
down	G + 0.6*Q + Eu(down)
cross	G + 1.0*Q + Eu(across)

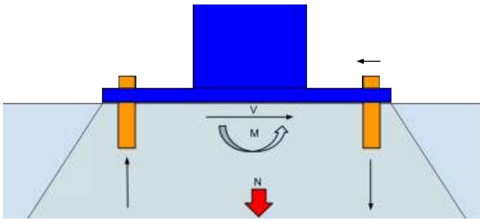
BVT
Engineering Professional Services

Calc sheet No. x OF x

ENGINEER: Tim Riddell **JOB NUMBER** 16081149
CLIENT: Forbes and Davies **DATE** 12/08/2016
RACK CONFIGURATION A

Foundation connections

Loads



	Inputs
	Outputs
	AXIS VM Inputs

Worst Case			**Load Case**
Shear V	3.1	kN	G+Q+Ex
Compression Load N (enter as negative)	-40	kN	G+Q+Ex
Moment M	1.1	kNm	G+0.6Q+Ed
Pullout N	10.2	kN	G+Q+Ex
Deflection Ex	11	mm	G+Q+Ex
Deflection Ed	85	mm	G+0.6Q+Ed

Deflection Check

Height of racking	6000	mm	
Deflection Limit (Ed)	300	mm	
Factored defelction Ed	119		
FOS	2.52		PASS
Distance between racks	300	mm	
Deflection limit (Ex)	150	mm	
Factored deflection Ex	15.26		
FOS	9.83		PASS

Bolt Specifications

Bolt Type	Trubolt	
Bolt Size	M10	
Ultimate Strength	470	MPa

Tensile Load

Resisting the moment using Dynabolt specs

Distance between bolts	197	mm	
moment arm	197	mm	
Equivalent tensile load per bolt	5.584	kN/bolt	
Limit for fixture	9.9	kN	*Manufacturer Specs*
FOS	1.8		N.A.
Check column axial pullout load			
Total pullout load	10.2	kN	
Pullout load/bolt	5.1	kN	
FOS	1.9		PASS
Total pullout load	5.583756345	kN	
FOS	1.8		PASS

Tensile load from prying *applies only when pullout is eccentric wrt bolts*

Prying distance	50	mm	
Eccentricity distance	80	mm	
Total pullout load	10.2	kN	
Prying tensile load/ bolt	13.26	kN/bolt	*assumes 2 bolts*
FOS	0.75		N.A.
Total pullout load	19	kN/bolt	*for max. pullout and moment*
FOS	0.53		N.A.

Punching shear

V*	40	kN	
Interior, edge or corner column	Interior		
Strength reduction factor	0.75		*NZS 3101 2.3.2.2*
Factored V*	53.33333333	kN	
Foot plate properties			
Short length side of plate	100	mm	
Long length side of plate	233	mm	
bo, Perimeter of plate	666	mm	
d, depth of concrete slab	120	mm	
kds	1		
Steel shear strength contribution			
Vs, (assume no stirup present)	0	kN	
Concrete shear strength contribution			
f'c, concrete strength	32	MPa	
Betac	2.3		
Alphas	20.0		
vc	1.8	MPa	*Eq 12-6*
	4.3	MPa	*Eq 12-7*

Shear

Check Bolt Capacity

Strength reduction facto	0.8		
shear planes in thread n	1		
Shear planes without th	1		
Shear area A	57.9896	mm²	
Shear force limit	27.037	kN	
FOS	8.7		PASS
Check bolt capacity in concrete			
Concrete capacity	6.8	kN	
FOS	4.4		PASS

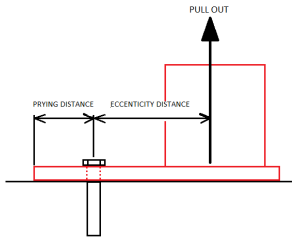
16.3 Bearing strength

16.3.1 *General*

Design bearing strength of concrete shall not exceed $\phi\,(0.85\,f'_c\,A_1)$, except when the supporting surface is wider on all sides than the loaded area, then the design bearing strength of the loaded area may be multiplied by $\sqrt{A_2/A_1}$ but by not more than two.

Bearing Strength

N*	40	kN	
Strength reduction facto	0.65		*NZS 3101 2.3.2.2*
Limit			
Concrete strength	32	MPa	
A1	23300	mm²	
A2	413540	mm²	
Fc	824	kN	
FOS	20.6		PASS



12.7.3.1 *Nominal shear strength for punching shear*
The nominal shear strength for any portion of the critical perimeter, V_n is given by:

$V_n = V_s + V_c$ ..... (Eq. 12–4)

Where $V_c = v_c b_o d$ and V_s is given by 12.7.4

and

$\frac{V^*}{\phi} \le V_n$ ..... (Eq. 12–5)

12.7.3.2 *Nominal shear stress resisted by the concrete*
For non-prestressed slabs subject to punching shear the shear stress resisted by the concrete, v_c, shall be the smallest of:

(a) $v_c = \frac{1}{6} k_{ds} \left(1 + \frac{2}{\beta_c}\right) \sqrt{f'_c}$ ..... (Eq. 12–6)

where β_c is the ratio of the long side to the short side of the concentrated load or reaction area; or

(b) $v_c = \frac{1}{6} k_{ds} \left(\frac{\alpha_s d}{b_o} + 1\right) \sqrt{f'_c}$ ..... (Eq. 12–7)

where α_s = 20 for interior columns, 15 for edge columns, 10 for corner columns; or

(c) $v_c = \frac{1}{3} k_{ds} \sqrt{f'_c}$ ..... (Eq. 12–8)

where k_{ds} allows for the influence of size on v_c and it is given by $k_{ds} = \sqrt{\frac{200}{d}}$ with the limits of $1.0 \le k_{ds} \le 0.5$, where d is the average effective depth round the critical perimeter.

12.7.4.2 *Area of shear reinforcement*
Shear reinforcement required on any side to resist V_s given by Equation 12–10, shall be calculated from appropriate expression below:
(a) Where the shear reinforcement is provided by stirrups, with a yield stress f_{yt}, placed at a spacing s, measured on the perimeter b_o, for a length b_c:

$A_v f_{yt} \frac{d}{s} \ge V_s$ and $s \le \frac{d}{2}$ ..... (Eq. 12–11)

	1.9	MPa	*Eq 12-8*
vc	1.8	MPa	
Vc	140.0	kN	
Nominal shear strength			
Vn	140.0	kN	
FOS	2.6		**PASS**

BVT Engineering Professional Services

Calc sheet No.		1	OF	5
ENGINEER:	Tim Riddell	JOB NUMBER	16081149	
CLIENT:	Forbes and Dav	DATE	42594	
RACK CONFIGURATION	A			
PROCEDURE	N/A	LOCKED BY		
VERSION	2.0	DATE	29/01/2016	

Bay loading				
Level	Level Mass [kg]	hi [m]	Q Surface Load	
8	0	0	0	kPa
7	0	0	0	kPa
6	0	0	0	kPa
5	0	0	0	kPa
4	700	6.2	2.883597884	kPa
3	700	5.04	2.883597884	kPa
2	700	3.6	2.883597884	kPa
1	700	2.175	2.883597884	kPa
0 - Ground	0	0	0	

Bay Dimensions			
	width / depth		units
Down		2.835	m
Across		0.84	m
Element	**Section**	**Area**	
Beam	N/A		m²
Beam 2 (if present)	N/A		
Column	90x67x2	0.000512	m²
Bracing	C36X24X2	0.00017885	m²
Steel density		7850	kgm⁻³
Gravity		9.81	ms⁻²

Legend
Inputs
Outputs
AXIS VM Inputs

Racking Type	Standard	
Contain Mesh Shelves?	Yes	
Psi reduction factors	**Down Aisle**	**Across Aisle**
Area reduction factor, Psi E	0.8	1
Rigid Mass factor, Psi M	0.67	0.67
Total factor	0.536	0.67

Loading conditions to determine natural frequency			
Frequency Anaylsis of:	**G**	**Q**	
down aisle (loading)	1	0.536	fraction
cross aisle (loading)	1	0.67	fraction

STEP 3.

Determine the equivalent period as:

$$T = 2\pi\left(\frac{W_s}{gK}\right)^{1/2}$$

where

W_s = the effective seismic weight contribution to one cross-aisle frame (see Section 3.1.4).

g = acceleration of gravity.

Seismic Weight, Wi [kN]					
	Down Aisle			**Across Aisle**	
	0	kN		0	kN
	0	kN		0	kN
	0	kN		0	kN
	0	kN		0	kN
	3.863659553	kN		5.007396309	kN
	3.907819308	kN		5.051556063	kN
	3.905453606	kN		5.049190362	kN
	4.023738662	kN		5.167475418	kN
	0	kN		0	kN
Total, Wt	15.70067113	kN		20.27561815	kN
Frequency	0.7610350076	Hz	**Frequency**	1.926782274	Hz
Period	1.31	s	**Period**	0.52	s

3.1.2 Load combinations

From AS/NZS 1170.0, the applicable loading combinations are:

Under **ultimate limit** state loads:

$1.2G + 1.5Q$, and

$G + \psi_c Q + E_u$,

and under **service limit** state loads:

$G + \psi_l Q + E_s$ (long-term service live load)

where

G = the dead weight of the rack structure

Q = the superimposed live load (the contents of the rack)

E_u = the ultimate limit state earthquake load

E_s = the serviceability limit state earthquake load

Ψ_c = the load combination factor

Ψ_l = the long-term load factor

The seismic weight at level i shall be calculated from:

$$W_i = G_i + \psi_E \psi_M Q_i$$

where

G_i = contributing dead weight of the rack at level i, including added components for securing contents (e.g. screens)

ψ_E = the area reduction factor
= 1.0 for cross-aisle direction
= 0.8 for down-aisle direction

ψ_M = the rigid mass factor
= 0.67

Q_i = *the maximum design stock load on the rack at level i (for one bay for the cross-aisle calculation and the full length of the rack for the down-aisle calculation) couldn't indent here or throughout the document where this occurs*

BVT Engineering Professional Services	Calc sheet No.	2	OF	5
	ENGINEER:	Tim Riddell	JOB NUMBER	16081149
	CLIENT:	Forbes and Dav	DATE	42594
	RACK CONFIGURATION	A		
	PROCEDURE	N/A	LOCKED BY	
	VERSION	2.0	DATE	29/01/2016

Elastic site spectra

(green)	Inputs
(yellow)	Outputs
(blue)	AXIS VM Inputs

NZS 1170.5 Table 3.3

#	Location	Z	D [km]
102	Christchurch	0.3	20

$$C(T_1) = C_h(T_1)ZRN(T_1, D)$$

Soil Class	D	Assume Soil D unless proven otherwise

	ULS	NZS 1170.5
Design Working Life	25 years	
IL	1	
AEP	1/50	
R	0.35	Table 3.5
N(T,D)	1.00	Clause 3.1.6
Ch(T)d	1.62	Table 3.1
Ch(T)x	2.97	
C(T)d	0.17	
C(T)x	0.31	

TABLE 3.5
RETURN PERIOD FACTOR

Required annual probability of exceedance	R_s or R_u
1/2500	1.8
1/2000	1.7
1/1000	1.3
1/500	1.0
1/250	0.75
1/100	0.5
1/50	0.35
1/25	0.25
1/20	0.20

TABLE 3.3
ANNUAL PROBABILITY OF EXCEEDANCE

Design working life	Importance level	Annual probability of exceedance for ultimate limit states			Annual probability of exceedance for serviceability limit states	
		Wind	Snow	Earthquake	SLS1	SLS2 Importance level 4 only
Construction equipment, e.g., props, scaffolding, braces and similar	2	1/100	1/50	1/100	1/25	
Less than 6 months	1	1/25	1/25	1/25	—	
	2	1/100	1/50	1/100	1/25	
	3	1/250	1/100	1/250	1/25	
	4	1/1000	1/250	1/1000	1/25	
5 years	1	1/25	1/25	1/25	—	—
	2	1/250	1/50	1/250	1/25	—
	3	1/500	1/100	1/500	1/25	—
	4	1/1000	1/250	1/1000	1/25	1/250
25 years	1	1/50	1/25	1/50	—	—
	2	1/250	1/50	1/250	1/25	—
	3	1/500	1/100	1/500	1/25	—
	4	1/1000	1/250	1/1000	1/25	1/250
50 years	1	1/100	1/50	1/100	—	—
	2	1/500	1/150	1/500	1/25	—
	3	1/1000	1/250	1/1000	1/25	—
	4	1/2500	1/500	1/2500	1/25	1/500
100 years or more	1	1/250	1/150	1/250	—	—
	2	1/1000	1/250	1/1000	1/25	—
	3	1/2500	1/500	1/2500	1/25	—
	4	*	*	*	1/25	*

* For importance level 4 structures with design working life of 100 years or more, the design events are determined by a hazard analysis but need to have probabilities less than or equal to those for importance level 3.

Design events for importance level 5 structures should be determined on a case by case basis.

3.1.6 Near-fault factor

The near-fault factor, $N(T,D)$, shall be determined from Equations 3.1(2) and 3.1(3) for locations at shortest distance, D, of less than 20 km from the nearest major fault listed in Table 3.6. The locations of these faults are shown in Figure 3.5.

3.1.6.1 *Annual probability of exceedance* $\geq {}^1/_{250}$

$$N(T,D) = 1.0 \qquad \text{. . . 3.1(2)}$$

3.1.6.2 *Annual probability of exceedance* $< {}^1/_{250}$

$$N(T,D) = N_{max}(T) \qquad D \leq 2 \text{ km}$$

$$= 1 + (N_{max}(T) - 1)\frac{20 - D}{18} \qquad 2 \text{ km} < D \leq 20 \text{ km}$$

$$= 1.0 \qquad D > 20 \text{ km} \qquad \text{. . . 3.1(3)}$$

where

D = the shortest distance (in kilometres) from the site to the nearest fault listed in Table 3.6

$N_{max}(T)$ = the maximum near-fault factor and is linearly interpolated for period T from Table 3.7

TABLE 3.3
Z-VALUES AND SHORTEST MAJOR FAULT DISTANCES *D* FOR NEW ZEALAND LOCATIONS (North to South)

#	Location	Z	D(km)[1]
1	Kaitaia	0.13	-
2	Paihia/Russell	0.13	-
3	Kaikohe	0.13	-
4	Whangarei	0.13	-
5	Dargaville	0.13	-
6	Warkworth	0.13	-
7	Auckland	0.13	-
8	Manakau City	0.13	-
9	Waiuku	0.13	-
10	Pukekohe	0.13	-
11	Thames	0.16	-
12	Paeroa	0.18	-
13	Waihi	0.18	-
14	Huntly	0.15	-
15	Ngaruawahia	0.15	-
16	Morrinsville	0.18	-
17	Te Aroha	0.18	-
18	Tauranga	0.20	-
19	Mount Maunganui	0.20	-
20	Hamilton	0.16	-
21	Cambridge	0.18	-
22	Te Awamutu	0.17	-
23	Matamata	0.19	-
24	Te Puke	0.22	-
25	Putaruru	0.21	-
26	Tokoroa	0.21	-
27	Otorohanga	0.17	-
28	Te Kuiti	0.18	-
29	Mangakino	0.21	-
30	Rotorua	0.24	-
31	Kawerau	0.29	-
32	Whakatane	0.30	-
33	Opotiki	0.30	-
34	Ruatoria	0.33	-
35	Murupara	0.30	-
36	Taupo	0.28	-
37	Taumarunui	0.21	-
38	Turangi	0.27	-
39	Gisborne	0.36	-
40	Wairoa	0.37	-
41	Waitara	0.18	-
42	New Plymouth	0.18	-
43	Inglewood	0.18	-
44	Stratford	0.18	-
45	Opunake	0.18	-
46	Hawera	0.18	-
47	Patea	0.19	-
48	Raetihi	0.26	-
49	Ohakune	0.27	-
50	Waiouru	0.29	-
51	Napier	0.38	-
52	Hastings	0.39	-
53	Wanganui	0.25	-
54	Waipawa	0.41	-
55	Waipukurau	0.41	-
56	Taihape	0.33	-
57	Marton	0.30	-
58	Bulls	0.31	-
59	Feilding	0.37	-
60	Palmerston North	0.38	8 – 16
61	Dannevirke	0.42	10
62	Woodville	0.41	≤ 2
63	Pahiatua	0.42	8
64	Foxton/Foxton Beach	0.36	-
65	Levin	0.40	-
66	Otaki	0.40	-
67	Waikanae	0.40	15 – 20
68	Paraparaumu	0.40	14 – 20
69	Masterton	0.42	6 – 10
70	Porirua	0.40	8 – 12
71	Wellington CBD (north of Basin Reserve)	0.40	≤ 2
72	Wellington	0.40	0 – 8
73	Hutt Valley–south of Taita Gorge	0.40	0 – 4
74	Upper Hutt	0.42	≤ 2
75	Eastbourne–Point Howard	0.40	4 - 8
76	Wainuiomata	0.40	5 – 8
77	Takaka	0.23	-
78	Motueka	0.26	-
79	Nelson	0.27	-
80	Picton	0.30	16
81	Blenheim	0.33	0 – 5
82	St Arnaud	0.36	≤ 2
83	Westport	0.30	-
84	Reefton	0.37	-
85	Murchison	0.34	-
86	Springs Junction	0.45	3
87	Hanmer Springs	0.55	2 – 6
88	Seddon	0.40	6
89	Ward	0.40	4
90	Cheviot	0.40	-
91	Greymouth	0.37	-
92	Kaikoura	0.42	12
93	Harihari	0.46	4
94	Hokitika	0.45	-
95	Fox Glacier	0.44	≤ 2
96	Franz Josef	0.44	≤ 2
97	Otira	0.60	3
98	Arthurs Pass	0.60	12
99	Rangiora	0.33	-
100	Darfield	0.30	-
101	Akaroa	0.16	-
102	Christchurch	0.22	-
103	Geraldine	0.19	-
104	Ashburton	0.20	-
105	Fairlie	0.24	-
106	Temuka	0.17	-
107	Timaru	0.15	-
108	Mt Cook	0.38	-
109	Twizel	0.27	-
110	Waimate	0.14	-
111	Cromwell	0.24	-
112	Wanaka	0.30	-
113	Arrowtown	0.30	-
114	Alexandra	0.21	-
115	Queenstown	0.32	-
116	Milford Sound	0.54	-
117	Palmerston	0.13	-
118	Oamaru	0.13	-
119	Dunedin	0.13	-
120	Mosgiel	0.13	-
121	Riverton	0.20	-
122	Te Anau	0.36	-
123	Gore	0.18	-
124	Winton	0.20	-
125	Balclutha	0.13	-
126	Mataura	0.17	-
127	Bluff	0.15	-
128	Invercargill	0.17	-
129	Oban	0.14	-

TABLE 3.1
SPECTRAL SHAPE FACTOR, $C_h(T)$

Period, T (seconds)	Spectral shape factor, $C_h(T)$ (g) — Site subsoil class			
	A Strong rock and B rock	C Shallow soil	D Deep or soft soil	E Very soft soil
0.0	1.89 (1.00)[1]	2.36 (1.33)[1]	3.00 (1.12)[1]	
0.1	1.89 (2.35)[1]	2.36 (2.93)[1]	3.00	
0.2	1.89 (2.35)[1]	2.36 (2.93)[1]	3.00	
0.3	1.89 (2.35)[1]	2.36 (2.93)[1]	3.00	
0.4	1.89	2.36	3.00	
0.5	1.60	2.00	3.00	
0.6	1.40	1.74	2.84	3.00
0.7	1.24	1.55	2.53	3.00
0.8	1.12	1.41	2.29	3.00
0.9	1.03	1.29	2.09	3.00
1.0	0.95	1.19	1.93	3.00
1.5	0.70	0.88	1.43	2.21
2.0	0.53	0.66	1.07	1.66
2.5	0.42	0.53	0.86	1.33
3.0	0.35	0.44	0.71	1.11
3.5	0.26	0.32	0.52	0.81
4.0	0.20	0.25	0.40	0.62

	Calc sheet No.	**2**	OF	**5**
	ENGINEER: Tim Riddell		**JOB NUMBER**	16081149

16081149 - Forbes & Davies - 49 stoneleigh drive racking

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Report

Materials

	Name	*Type*	*National design code*	*Material code*	*Model*	*E_x [N/mm^2]*	*E_y [N/mm^2]*
1	S 355	Steel	Eurocode	10025-2	Linear	210000	210000
2	Bracing steel	Steel	Eurocode	10025-2	Linear	210000	210000

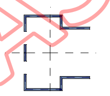
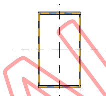
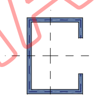
	Name	*ν*	*α_T [1/°C]*	*ρ [kg/m^3]*	*Material color*	*Contour color*	*Texture*	*P_1*
1	S 355	0.30	1.2E-5	7850			Steel	f_y[N/mm^2] = 355.00
2	Bracing steel	0.30	1.2E-5	7850			Steel	f_y[N/mm^2] = 355.00

	Name	*P_2*	*P_3*	*P_4*	*P_5*	*P_6*	*P_7*	*P_8*	*P_9*	*P_{10}*
1	S 355	f_u[N/mm^2] = 510.00	f_y^*[N/mm^2] = 335.00	f_u^*[N/mm^2] = 470.00						
2	Bracing steel	f_u[N/mm^2] = 510.00	f_y^*[N/mm^2] = 335.00	f_u^*[N/mm^2] = 470.00						

	Name	*P_{11}*	*P_{12}*	*P_{13}*	*P_{14}*
1	S 355				
2	Bracing steel				

Name: Material name; **Type:** Type of material; **Model:** Material model; **E_x:** Young's modulus of elasticity in local x direction; **E_y:** Young's modulus of elasticity in local y direction; **ν:** Poisson's ratio; **α_T:** Thermal expansion coefficient; **ρ:** Density; **Contour color:** Material outline color; **P_1, P_2, P_3, P_4, P_5, P_6, P_7, P_8, P_9, P_{10}, P_{11}, P_{12}, P_{13}, P_{14}:** Design parameter;

Cross-sections

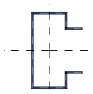
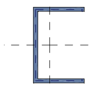
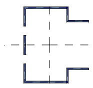
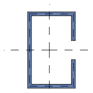
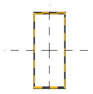
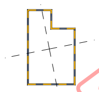
	Name	*Drawing*	*Process*	*Shape*	*h [mm]*	*b [mm]*	*tw [mm]*	*tf [mm]*	*r_1 [mm]*	*r_2 [mm]*	*r_3 [mm]*
1	69x80x2.2 column		Other	Custom	79.9	69.3	2.2	2.2	0	0	0
2	90x50x3		Welded	Box	90.0	50.0	1.5	3.0	0	0	0
3	C 34 bracing		Cold f.	C	34.0	25.0	2.0	2.0	0	0	0

	Name	*Ax [mm^2]*	*Ay [mm^2]*	*Az [mm^2]*	*Ix [mm^4]*	*Iy [mm^4]*	*Iz [mm^4]*	*Iyz [mm^4]*
1	69x80x2.2 column	465.83	0	0	738.0	473475.3	218145.3	0
2	90x50x3	552.00	181.19	248.33	488857.2	716076.6	210739.2	0
3	C 34 bracing	188.00	77.79	42.66	252.5	33950.7	16155.3	0

	Name	*I_1 [mm^4]*	*I_2 [mm^4]*	*α [°]*	*Iω [mm^6]*	*$W_{1,el,t}$ [mm^3]*	*$W_{1,el,b}$ [mm^3]*	*$W_{2,el,t}$ [mm^3]*	*$W_{2,el,b}$ [mm^3]*
1	69x80x2.2 column	473475.3	218145.3	0	2.5E+08	11845.8	11845.8	5238.3	7888.0
2	90x50x3	716076.6	210739.2	0	6.7E+07	15912.8	15912.8	8429.6	8429.6
3	C 34 bracing	33950.7	16155.3	0	5197184	1997.1	1997.1	1117.4	1532.4

	Name	*$W_{1,pl}$ [mm^3]*	*$W_{2,pl}$ [mm^3]*	*i_y [mm]*	*i_z [mm]*	*Hy [mm]*	*Hz [mm]*	*y_G [mm]*	*z_G [mm]*	*y_s [mm]*	*z_s [mm]*	*S.p.*
1	69x80x2.2 column	14153.3	8909.9	31.9	21.6	69.3	79.9	27.7	40.0	60.9	0	9
2	90x50x3	18342.0	9861.0	36.0	19.5	50.0	90.0	25.0	45.0	0	0	9
3	C 34 bracing	2372.0	1572.9	13.4	9.3	25.0	34.0	10.5	17.0	–22.5	0	8

Cross-sections

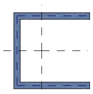
	Name	*Drawing*	*Process*	*Shape*	*h [mm]*	*b [mm]*	*tw [mm]*	*tf [mm]*	*r_1 [mm]*	*r_2 [mm]*	*r_3 [mm]*
4	67x100 column		Other	Custom	100.0	67.0	2.5	2.5	0	0	0
5	C 36x24x2		Cold f.	C	36.0	24.0	2.0	2.0	0	0	0
6	82x96		Other	Custom	96.0	81.5	2.3	2.3	0	0	0
7	C 40x25x2 bracing		Cold f.	C	40.0	25.0	2.0	2.0	0	0	0
8	50x120		Other	Custom	120.0	50.0	1.5	1.5	0	0	0
9	80 meshbox		Other	Custom	81.5	51.5	0	0	0	0	0

	Name	*Ax [mm^2]*	*Ay [mm^2]*	*Az [mm^2]*	*Ix [mm^4]*	*Iy [mm^4]*	*Iz [mm^4]*	*Iyz [mm^4]*
4	67x100 column	672.50	186.57	151.09	1390.3	963901.4	325951.8	0
5	C 36x24x2	160.00	66.15	54.04	212.6	33237.3	9275.7	0
6	82x96	606.74	0	0	1056.5	910140.0	424552.2	0.2
7	C 40x25x2 bracing	224.00	72.74	48.69	300.6	51518.7	21412.5	0
8	50x120	642.00	146.67	333.49	686587.3	1397284.0	263680.9	0
9	80 meshbox	390.00	95.61	187.69	282984.2	321720.5	161187.2	−33317.3

	Name	*I_1 [mm^4]*	*I_2 [mm^4]*	*α [°]*	*Iω [mm^6]*	*$W_{1,el,t}$ [mm^3]*	*$W_{1,el,b}$ [mm^3]*	*$W_{2,el,t}$ [mm^3]*	*$W_{2,el,b}$ [mm^3]*
4	67x100 column	963901.4	325951.8	0	1.2E+09	19278.0	19278.0	7568.3	13619.8
5	C 36x24x2	33237.3	9275.7	0	1893804	1846.5	1846.5	565.6	1220.5
6	82x96	910140.0	424552.2	0	9.5E+08	18961.3	18961.2	8654.2	13086.1
7	C 40x25x2 bracing	51518.7	21412.5	0	1.5E+07	2575.9	2575.9	1583.0	1866.3
8	50x120	1397284.0	263680.9	0	1.9E+08	23288.1	23288.1	10547.2	10547.2
9	80 meshbox	328360.6	154547.1	11.27	4260734	7060.4	7550.8	4895.2	4992.4

	Name	*$W_{1,pl}$ [mm^3]*	*$W_{2,pl}$ [mm^3]*	*i_y [mm]*	*i_z [mm]*	*Hy [mm]*	*Hz [mm]*	*y_G [mm]*	*z_G [mm]*	*y_s [mm]*	*z_s [mm]*	*S.p.*
4	67x100 column	23721.9	13550.7	37.9	22.0	67.0	100.0	23.9	50.0	−55.7	0	9
5	C 36x24x2	2144.0	1023.5	14.4	7.6	24.0	36.0	7.6	18.0	−15.7	0	8
6	82x96	22299.7	14543.6	38.7	26.5	81.5	96.0	32.4	48.0	47.0	0	9
7	C 40x25x2 bracing	3146.0	2026.1	15.2	9.8	25.0	40.0	11.5	20.0	−25.2	0	8
8	50x120	27297.0	12043.5	46.7	20.3	50.0	120.0	25.0	60.0	0	0	9
9	80 meshbox	10077.5	7029.7	28.7	20.3	51.5	81.5	23.8	38.8	0.3	−8.5	8

Cross-sections

	Name	Drawing	Process	Shape	h [mm]	b [mm]	tw [mm]	tf [mm]	r_1 [mm]	r_2 [mm]	r_3 [mm]
10	Connection arms		Cold f.	U	70.0	70.0	6.0	6.0	0	0	0

	Name	Ax [mm^2]	Ay [mm^2]	Az [mm^2]	Ix [mm^4]	Iy [mm^4]	Iz [mm^4]	Iyz [mm^4]
10	Connection arms	1188.00	649.97	265.56	14188.9	960236.1	596010.2	0

	Name	I_1 [mm^4]	I_2 [mm^4]	α [°]	Iω [mm^6]	$W_{1,el,t}$ [mm^3]	$W_{1,el,b}$ [mm^3]	$W_{2,el,t}$ [mm^3]	$W_{2,el,b}$ [mm^3]
10	Connection arms	960236.1	596010.1	0	4.5E+08	27435.3	27435.3	13431.6	23257.8

	Name	$W_{1,pl}$ [mm^3]	$W_{2,pl}$ [mm^3]	i_y [mm]	i_z [mm]	Hy [mm]	Hz [mm]	y_G [mm]	z_G [mm]	y_s [mm]	z_s [mm]	S.p.
10	Connection arms	31926.0	23314.0	28.4	22.4	70.0	70.0	25.6	35.0	–51.0	0	8

Name: Cross-section name; **Process:** Manufacturing process; **h:** Cross-section height; **b:** Cross-section width; **tw:** Web thickness; **tf:** Flange thickness; r_1, r_2, r_3: Rounding radius; **Ax:** Cross-section area; **Ay, Az:** Shear area; **Ix:** Torsional inertia; **Iy, Iz:** Flexural inertia; **Iyz:** Centrifugal inertia; I_1, I_2: Principal flexural inertia; **α:** Principal directions; **Iω:** Warping constant; $W_{1,el,t}$, $W_{1,el,b}$, $W_{2,el,t}$, $W_{2,el,b}$: Elastic modulus; $W_{1,pl}$, $W_{2,pl}$: Plastic modulus; i_y, i_z: Radius of inertia; **Hy:** Dimension in local y direction; **Hz:** Dimension in local z direction; y_G: y coordinate of the center of gravity; z_G: z coordinate of the center of gravity; y_s: y coordinate of the shear (torsion) center relative to the center of gravity; z_s: z coordinate of the shear (torsion) center relative to the center of gravity; **S.p.:** Stress calculation points;

Q: Surface loads on beams and ribs

	Direction	Type	Comp.	Value [kN/m^2]	X_{ref} [m]	Y_{ref} [m]	Z_{ref} [m]	X [m]	Y [m]	Z [m]
	Global	Constant	pX =	0				0	0	5.485
			pY =	0				0	0.840	5.485
			pZ =	–2.88				34.020	0.840	5.485
								34.020	0	5.485
	Global	Constant	pX =	0				0	0	4.340
			pY =	0				0	0.840	4.340
			pZ =	–2.88				34.020	0.840	4.340
								34.020	0	4.340
	Global	Constant	pX =	0				0	0	2.900
			pY =	0				0	0.840	2.900
			pZ =	–2.88				34.020	0.840	2.900
								34.020	0	2.900
	Global	Constant	pX =	0				0	0	1.450
			pY =	0				0	0.840	1.450
			pZ =	–2.88				34.020	0.840	1.450
								34.020	0	1.450

Comp.: Component; **Value:** Load component value; X_{ref}: X coordinate of the load value reference point; Y_{ref}: Y coordinate of the load value reference point; Z_{ref}: Z coordinate of the load value reference point; **X:** X coordinate of the load polygon vertices; **Y:** Y coordinate of the load polygon vertices; **Z:** Z coordinate of the load polygon vertices;

Ex: Surface loads on beams and ribs

	Direction	Type	Comp.	Value [kN/m^2]	X_{ref} [m]	Y_{ref} [m]	Z_{ref} [m]	X [m]	Y [m]	Z [m]
	Global	Constant	pX =	0				34.020	0	1.450
			pY =	0.26				34.020	0.840	1.450
			pZ =	0				0	0.840	1.450
								0	0	1.450
	Global	Constant	pX =	0				0	0	2.900
			pY =	0.42				0	0.840	2.900
			pZ =	0				34.020	0.840	2.900
								34.020	0	2.900
	Global	Constant	pX =	0				34.020	0.840	4.340
			pY =	0.58				34.020	0	4.340

Ex: Surface loads on beams and ribs

	Direction	Type	Comp.	Value [kN/m²]	X_{ref} [m]	Y_{ref} [m]	Z_{ref} [m]	X [m]	Y [m]	Z [m]
			pZ =	0				0	0	4.340
								0	0.840	4.340
	Global	Constant	pX =	0				0	0	5.485
			pY =	0.71				0	0.840	5.485
			pZ =	0				34.020	0.840	5.485
								34.020	0	5.485

Comp.: Component; **Value:** Load component value; **X_{ref}:** X coordinate of the load value reference point; **Y_{ref}:** Y coordinate of the load value reference point; **Z_{ref}:** Z coordinate of the load value reference point; **X:** X coordinate of the load polygon vertices; **Y:** Y coordinate of the load polygon vertices; **Z:** Z coordinate of the load polygon vertices;

Ed: Surface loads on beams and ribs

	Direction	Type	Comp.	Value [kN/m²]	X_{ref} [m]	Y_{ref} [m]	Z_{ref} [m]	X [m]	Y [m]	Z [m]
	Global	Constant	pX =	0.30				0	0	5.485
			pY =	0				0	0.840	5.485
			pZ =	0				34.020	0.840	5.485
								34.020	0	5.485
	Global	Constant	pX =	0.25				0	0	4.340
			pY =	0				0	0.840	4.340
			pZ =	0				34.020	0.840	4.340
								34.020	0	4.340
	Global	Constant	pX =	0.18				34.020	0.840	2.900
			pY =	0				34.020	0	2.900
			pZ =	0				0	0	2.900
								0	0.840	2.900
	Global	Constant	pX =	0.11				0	0	1.450
			pY =	0				0	0.840	1.450
			pZ =	0				34.020	0.840	1.450
								34.020	0	1.450

Comp.: Component; **Value:** Load component value; **X_{ref}:** X coordinate of the load value reference point; **Y_{ref}:** Y coordinate of the load value reference point; **Z_{ref}:** Z coordinate of the load value reference point; **X:** X coordinate of the load polygon vertices; **Y:** Y coordinate of the load polygon vertices; **Z:** Z coordinate of the load polygon vertices;

Custom load combinations by load cases

	Name	Type	Q	G	Ex	Ed	Comment
1	Vib D	ULS	0.54	1.00	0	0	
2	Vib X	ULS	0.67	1.00	0	0	
3	1.2G + 1.5Q	ULS	1.50	1.20	0	0	
4	G + 0.6Q + Ed	ULS	0.60	1.00	0	1.00	
5	G + Q + Ex	ULS	1.00	1.00	1.00	0	

Name: Load combination name; **Type:** Load combination type; **Q:** Q Factor; **G:** G Factor; **Ex:** Ex Factor; **Ed:** Ed Factor;

Weights per material [Filtered]

	Material name	ρ [kg/m³]	Σ V [m³]	Σ G [kg]
1	S 355	7850	0.150	1179.325
2	Bracing steel	7850	0	0
	Total		**0.150**	**1179.325**

ρ: Density; **Σ V:** Total volume; **Σ G:** Total mass;

Truss forces [Nonlin., Envelope (Load combinations)]

	Sh.	Cross-section name	Length [m]	min. max.	Case	Nx [kN]
Ext.						
2	3	C 34 bracing	1.032	Nx min	G + Q + Ex [1] (1.000)	**–8.000**
134	3	C 34 bracing	1.032	Nx min	G + Q + Ex [1] (1.000)	**–8.000**
3	3	C 34 bracing	1.032	Nx max	G + Q + Ex [1] (1.000)	**6.499**

Truss forces [Nonlin., Envelope (Load combinations)]

	Sh.	*Cross-section name*	*Length [m]*	*min. max.*	*Case*	*Nx [kN]*
135	3	C 34 bracing	1.032	Nx max	G + Q + Ex [1] (1.000)	**6.499**

Sh.: Cross-section; **Length:** Truss length; **min. max.:** Extreme type; **Case:** Load case of extreme; **Nx:** Axial force;

C34x25x2 Brace Capacity: Axial Tension 30.3 kN
Axial Compression 32.1 kN

Therefore, ok

Beam internal forces [Nonlin., Envelope (All ULS), Filtered]

Ext.	*Sh.*	*Cross-section name*	*C*	*min. max.*	*Case*	*Loc. [m]*	*Node*	*Nx [kN]*	*Vy [kN]*	*Vz [kN]*
20	1	69x80x2.2 column	Nx	min	G + Q + Ex [1] (1.000)	0	(14)	**–39.340**	–2.130	0.008
122	1	69x80x2.2 column		min	G + Q + Ex [1] (1.000)	0	(287)	**–39.340**	–2.130	–0.008
19	1	69x80x2.2 column		max	G + Q + Ex [1] (1.000)	0.750	(23)	**10.308**	–0.994	–0.011
121	1	69x80x2.2 column		max	G + Q + Ex [1] (1.000)	0.750	(288)	**10.308**	–0.994	0.011
20	1	69x80x2.2 column	Vy	min	G + Q + Ex [1] (1.000)	0	(14)	–39.340	**–2.130**	0.008
122	1	69x80x2.2 column		min	G + Q + Ex [1] (1.000)	0	(287)	–39.340	**–2.130**	–0.008
19	1	69x80x2.2 column		max	G + Q + Ex [1] (1.000)	0.113	(445)	10.287	**3.120**	–0.009
121	1	69x80x2.2 column		max	G + Q + Ex [1] (1.000)	0.113	(1442)	10.287	**3.120**	0.009
19	1	69x80x2.2 column	Vz	min	G + 0.6Q + Ed [1] (1.000)	0.925	(451)	–9.002	0.007	**–1.088**
29	1	69x80x2.2 column		min	G + 0.6Q + Ed [1] (1.000)	0.925	(544)	–8.971	0.007	**–1.087**
39	1	69x80x2.2 column		min	G + 0.6Q + Ed [1] (1.000)	0.925	(637)	–8.970	0.007	**–1.087**
49	1	69x80x2.2 column		min	G + 0.6Q + Ed [1] (1.000)	0.925	(730)	–8.970	0.007	**–1.087**
59	1	69x80x2.2 column		min	G + 0.6Q + Ed [1] (1.000)	0.925	(823)	–8.970	0.007	**–1.087**
20	1	69x80x2.2 column		max	G + 0.6Q + Ed [1] (1.000)	0.750	(483)	–9.008	–0.005	**1.087**
30	1	69x80x2.2 column		max	G + 0.6Q + Ed [1] (1.000)	0.750	(576)	–8.971	–0.005	**1.087**
40	1	69x80x2.2 column		max	G + 0.6Q + Ed [1] (1.000)	0.750	(669)	–8.972	–0.005	**1.087**
50	1	69x80x2.2 column		max	G + 0.6Q + Ed [1] (1.000)	0.750	(762)	–8.972	–0.005	**1.087**
60	1	69x80x2.2 column		max	G + 0.6Q + Ed [1] (1.000)	0.750	(855)	–8.972	–0.005	**1.086**
120	1	69x80x2.2 column	Tx	min	G + Q + Ex [1] (1.000)	5.084	(1435)	–1.918	–0.235	0.309
10	1	69x80x2.2 column		max	G + Q + Ex [1] (1.000)	5.084	(412)	–1.918	–0.235	–0.309
20	1	69x80x2.2 column	My	min	G + 0.6Q + Ed [1] (1.000)	0	(14)	–9.065	0.034	1.040
30	1	69x80x2.2 column		min	G + 0.6Q + Ed [1] (1.000)	0	(62)	–9.026	0.034	1.040

Ext.	*Sh.*	*Cross-section name*	*C*	*min. max.*	*Case*	*Loc. [m]*	*Node*	*Tx [kNm]*	*My [kNm]*	*Mz [kNm]*
20	1	69x80x2.2 column	Nx	min	G + Q + Ex [1] (1.000)	0	(14)	0	–0.006	0
122	1	69x80x2.2 column		min	G + Q + Ex [1] (1.000)	0	(287)	0	0.006	0
19	1	69x80x2.2 column		max	G + Q + Ex [1] (1.000)	0.750	(23)	0	–0.002	0.132
121	1	69x80x2.2 column		max	G + Q + Ex [1] (1.000)	0.750	(288)	0	0.002	0.132
20	1	69x80x2.2 column	Vy	min	G + Q + Ex [1] (1.000)	0	(14)	0	–0.006	0
122	1	69x80x2.2 column		min	G + Q + Ex [1] (1.000)	0	(287)	0	0.006	0
19	1	69x80x2.2 column		max	G + Q + Ex [1] (1.000)	0.113	(445)	0	0.005	–0.351
121	1	69x80x2.2 column		max	G + Q + Ex [1] (1.000)	0.113	(1442)	0	–0.005	–0.351
19	1	69x80x2.2 column	Vz	min	G + 0.6Q + Ed [1] (1.000)	0.925	(451)	0	0.098	0.006
29	1	69x80x2.2 column		min	G + 0.6Q + Ed [1] (1.000)	0.925	(544)	0	0.098	0.006
39	1	69x80x2.2 column		min	G + 0.6Q + Ed [1] (1.000)	0.925	(637)	0	0.098	0.006
49	1	69x80x2.2 column		min	G + 0.6Q + Ed [1] (1.000)	0.925	(730)	0	0.097	0.006
59	1	69x80x2.2 column		min	G + 0.6Q + Ed [1] (1.000)	0.925	(823)	0	0.097	0.006
20	1	69x80x2.2 column		max	G + 0.6Q + Ed [1] (1.000)	0.750	(483)	0	–0.288	–0.002
30	1	69x80x2.2 column		max	G + 0.6Q + Ed [1] (1.000)	0.750	(576)	0	–0.288	–0.002
40	1	69x80x2.2 column		max	G + 0.6Q + Ed [1] (1.000)	0.750	(669)	0	–0.288	–0.002
50	1	69x80x2.2 column		max	G + 0.6Q + Ed [1] (1.000)	0.750	(762)	0	–0.288	–0.002
60	1	69x80x2.2 column		max	G + 0.6Q + Ed [1] (1.000)	0.750	(855)	0	–0.287	–0.002
120	1	69x80x2.2 column	Tx	min	G + Q + Ex [1] (1.000)	5.084	(1435)	**0**	0.093	–0.031
10	1	69x80x2.2 column		max	G + Q + Ex [1] (1.000)	5.084	(412)	**0**	–0.093	–0.031
20	1	69x80x2.2 column	My	min	G + 0.6Q + Ed [1] (1.000)	0	(14)	0	**–1.087**	0.001
30	1	69x80x2.2 column		min	G + 0.6Q + Ed [1] (1.000)	0	(62)	0	**–1.087**	0.001

Beam internal forces [Nonlin., Envelope (All ULS), Filtered]

	Sh.	*Cross-section name*	*C*	*min. max.*	*Case*	*Loc. [m]*	*Node*	*Nx [kN]*	*Vy [kN]*	*Vz [kN]*
40	1	69x80x2.2 column		min	G + 0.6Q + Ed [1] (1.000)	0	(87)	–9.028	0.034	1.040
50	1	69x80x2.2 column		min	G + 0.6Q + Ed [1] (1.000)	0	(112)	–9.028	0.034	1.040
60	1	69x80x2.2 column		min	G + 0.6Q + Ed [1] (1.000)	0	(137)	–9.028	0.034	1.040
19	1	69x80x2.2 column		max	G + 0.6Q + Ed [1] (1.000)	0	(3)	–9.060	0.031	–1.040
29	1	69x80x2.2 column		max	G + 0.6Q + Ed [1] (1.000)	0	(61)	–9.031	0.030	–1.040
39	1	69x80x2.2 column		max	G + 0.6Q + Ed [1] (1.000)	0	(86)	–9.029	0.031	–1.040
49	1	69x80x2.2 column		max	G + 0.6Q + Ed [1] (1.000)	0	(111)	–9.029	0.031	–1.040
59	1	69x80x2.2 column		max	G + 0.6Q + Ed [1] (1.000)	0	(136)	–9.029	0.031	–1.040
19	1	69x80x2.2 column	Mz	min	G + Q + Ex [1] (1.000)	0.150	(22)	10.289	3.120	–0.009
121	1	69x80x2.2 column		min	G + Q + Ex [1] (1.000)	0.150	(285)	10.289	3.120	0.009
20	1	69x80x2.2 column		max	G + Q + Ex [1] (1.000)	0.150	(21)	–39.336	–2.114	0.008
122	1	69x80x2.2 column		max	G + Q + Ex [1] (1.000)	0.150	(284)	–39.336	–2.114	–0.008

	Sh.	*Cross-section name*	*C*	*min. max.*	*Case*	*Loc. [m]*	*Node*	*Tx [kNm]*	*My [kNm]*	*Mz [kNm]*
40	1	69x80x2.2 column		min	G + 0.6Q + Ed [1] (1.000)	0	(87)	0	**–1.087**	0.001
50	1	69x80x2.2 column		min	G + 0.6Q + Ed [1] (1.000)	0	(112)	0	**–1.087**	0.001
60	1	69x80x2.2 column		min	G + 0.6Q + Ed [1] (1.000)	0	(137)	0	**–1.086**	0.001
19	1	69x80x2.2 column		max	G + 0.6Q + Ed [1] (1.000)	0	(3)	0	**1.087**	–0.001
29	1	69x80x2.2 column		max	G + 0.6Q + Ed [1] (1.000)	0	(61)	0	**1.087**	–0.002
39	1	69x80x2.2 column		max	G + 0.6Q + Ed [1] (1.000)	0	(86)	0	**1.087**	–0.001
49	1	69x80x2.2 column		max	G + 0.6Q + Ed [1] (1.000)	0	(111)	0	**1.087**	–0.001
59	1	69x80x2.2 column		max	G + 0.6Q + Ed [1] (1.000)	0	(136)	0	**1.086**	–0.001
19	1	69x80x2.2 column	Mz	min	G + Q + Ex [1] (1.000)	0.150	(22)	0	0.005	**–0.468**
121	1	69x80x2.2 column		min	G + Q + Ex [1] (1.000)	0.150	(285)	0	–0.005	**–0.468**
20	1	69x80x2.2 column		max	G + Q + Ex [1] (1.000)	0.150	(21)	0	–0.005	**0.318**
122	1	69x80x2.2 column		max	G + Q + Ex [1] (1.000)	0.150	(284)	0	0.005	**0.318**

Sh.: Cross-section; **C:** Extremal component; **min. max.:** Extreme type; **Case:** Load case of extreme; **Loc.:** Cross-section local x position on the beam; **Nx:** Axial force;
Vy: Shear force in local y direction; **Vz:** Shear force in local z direction; **Tx:** Torsional moment; **My:** Flexural moment about local y axis; **Mz:** Flexural moment about local z axis;

69x80x2.2 Column Capacities:

ØMS,Y = 4.80 kNm
ØMS,Z = 3.19 kNm
ØNT = 187 kN
ØNC = 56.1 kN
Shear 60 kN

Note that for one column, it is slightly overloaded at 57kN at ULS load. There is no bending, so it should be ok and is only for the critical length due to singly symmetric torsional distortion.

Therefore, ok

Beam internal forces [Nonlin., Envelope (All ULS), Filtered]

	Sh.	*Cross-section name*	*C*	*min. max.*	*Case*	*Loc. [m]*	*Node*	*Nx [kN]*	*Vy [kN]*	*Vz [kN]*
Ext.										
4	2	90x50x3	Nx	min	G + 0.6Q + Ed [1] (1.000)	2.835	(10)	**–0.517**	0	1.280
8	2	90x50x3		min	G + 0.6Q + Ed [1] (1.000)	2.835	(20)	**–0.518**	0	1.280
3	2	90x50x3		max	G + 0.6Q + Ed [1] (1.000)	0	(2)	**0.497**	0	–0.825
7	2	90x50x3		max	G + 0.6Q + Ed [1] (1.000)	0	(17)	**0.497**	0	–0.825

	Sh.	*Cross-section name*	*C*	*min. max.*	*Case*	*Loc. [m]*	*Node*	*Tx [kNm]*	*My [kNm]*	*Mz [kNm]*
Ext.										
4	2	90x50x3	Nx	min	G + 0.6Q + Ed [1] (1.000)	2.835	(10)	0	0.397	0
8	2	90x50x3		min	G + 0.6Q + Ed [1] (1.000)	2.835	(20)	0	0.397	0
3	2	90x50x3		max	G + 0.6Q + Ed [1] (1.000)	0	(2)	0	–0.236	0
7	2	90x50x3		max	G + 0.6Q + Ed [1] (1.000)	0	(17)	0	–0.236	0

Beam internal forces [Nonlin., Envelope (All ULS), Filtered]

	Sh.	Cross-section name	C	min. max.	Case	Loc. [m]	Node	Nx [kN]	Vy [kN]	Vz [kN]
4	2	90x50x3	Vy	min	G + Q + Ex [1] (1.000)	2.835	(10)	–0.292	**–0.499**	1.785
8	2	90x50x3		min	G + Q + Ex [1] (1.000)	2.835	(20)	–0.303	**–0.499**	1.779
114	2	90x50x3		max	G + Q + Ex [1] (1.000)	0	(279)	–0.292	**0.499**	–1.785
118	2	90x50x3		max	G + Q + Ex [1] (1.000)	0	(283)	–0.303	**0.499**	–1.779
114	2	90x50x3	Vz	min	G + Q + Ex [1] (1.000)	0	(279)	–0.292	0.499	**–1.785**
4	2	90x50x3		max	G + Q + Ex [1] (1.000)	2.835	(10)	–0.292	–0.499	**1.785**
8	2	90x50x3	Tx	min	G + Q + Ex [1] (1.000)	0	(19)	–0.302	0.352	–1.773
118	2	90x50x3		max	G + Q + Ex [1] (1.000)	2.126	(1372)	–0.302	–0.141	0.885
4	2	90x50x3	My	min	G + Q + Ex [1] (1.000)	1.418	(336)	–0.299	–0.073	0.008
8	2	90x50x3		min	G + Q + Ex [1] (1.000)	1.418	(348)	–0.309	–0.072	0.003
114	2	90x50x3		min	G + Q + Ex [1] (1.000)	1.418	(1359)	–0.299	0.073	–0.008
118	2	90x50x3		min	G + Q + Ex [1] (1.000)	1.418	(1371)	–0.309	0.072	–0.003
111	2	90x50x3		max	G + 0.6Q + Ed [1] (1.000)	2.835	(301)	–0.261	0	1.484
115	2	90x50x3		max	G + 0.6Q + Ed [1] (1.000)	2.835	(305)	–0.260	0	1.484
8	2	90x50x3	Mz	min	G + Q + Ex [1] (1.000)	1.205		–0.309	–0.009	–0.263
118	2	90x50x3		min	G + Q + Ex [1] (1.000)	1.630		–0.309	0.009	0.263
4	2	90x50x3		max	G + Q + Ex [1] (1.000)	2.835	(10)	–0.292	–0.499	1.785
14	2	90x50x3		max	G + Q + Ex [1] (1.000)	0	(10)	–0.279	0.426	–1.775
104	2	90x50x3		max	G + Q + Ex [1] (1.000)	2.835	(279)	–0.279	–0.426	1.775
114	2	90x50x3		max	G + Q + Ex [1] (1.000)	0	(279)	–0.292	0.499	–1.785

	Sh.	Cross-section name	C	min. max.	Case	Loc. [m]	Node	Tx [kNm]	My [kNm]	Mz [kNm]
4	2	90x50x3	Vy	min	G + Q + Ex [1] (1.000)	2.835	(10)	–0.008	0.234	0.209
8	2	90x50x3		min	G + Q + Ex [1] (1.000)	2.835	(20)	–0.012	0.226	0.207
114	2	90x50x3		max	G + Q + Ex [1] (1.000)	0	(279)	0.008	0.234	0.209
118	2	90x50x3		max	G + Q + Ex [1] (1.000)	0	(283)	0.012	0.226	0.207
114	2	90x50x3	Vz	min	G + Q + Ex [1] (1.000)	0	(279)	0.008	0.234	0.209
4	2	90x50x3		max	G + Q + Ex [1] (1.000)	2.835	(10)	–0.008	0.234	0.209
8	2	90x50x3	Tx	min	G + Q + Ex [1] (1.000)	0	(19)	**–0.016**	0.217	0
118	2	90x50x3		max	G + Q + Ex [1] (1.000)	2.126	(1372)	**0.016**	–0.725	–0.175
4	2	90x50x3	My	min	G + Q + Ex [1] (1.000)	1.418	(336)	–0.011	**–1.038**	–0.197
8	2	90x50x3		min	G + Q + Ex [1] (1.000)	1.418	(348)	–0.014	**–1.038**	–0.199
114	2	90x50x3		min	G + Q + Ex [1] (1.000)	1.418	(1359)	0.011	**–1.038**	–0.197
118	2	90x50x3		min	G + Q + Ex [1] (1.000)	1.418	(1371)	0.014	**–1.038**	–0.199
111	2	90x50x3		max	G + 0.6Q + Ed [1] (1.000)	2.835	(301)	0	**0.709**	0
115	2	90x50x3		max	G + 0.6Q + Ed [1] (1.000)	2.835	(305)	0	**0.709**	0
8	2	90x50x3	Mz	min	G + Q + Ex [1] (1.000)	1.205		–0.014	–1.010	**–0.207**
118	2	90x50x3		min	G + Q + Ex [1] (1.000)	1.630		0.014	–1.010	**–0.207**
4	2	90x50x3		max	G + Q + Ex [1] (1.000)	2.835	(10)	–0.008	0.234	**0.209**
14	2	90x50x3		max	G + Q + Ex [1] (1.000)	0	(10)	0.001	0.229	**0.209**
104	2	90x50x3		max	G + Q + Ex [1] (1.000)	2.835	(279)	–0.001	0.229	**0.209**
114	2	90x50x3		max	G + Q + Ex [1] (1.000)	0	(279)	0.008	0.234	**0.209**

Sh.: Cross-section; **C:** Extremal component; **min. max.:** Extreme type; **Case:** Load case of extreme; **Loc.:** Cross-section local x position on the beam; **Nx:** Axial force; **Vy:** Shear force in local y direction; **Vz:** Shear force in local z direction; **Tx:** Torsional moment; **My:** Flexural moment about local y axis; **Mz:** Flexural moment about local z axis;

90x50 Beam Capacities:

Bending Section, y	5.01 kNm
Bending Section, z	2.66 kNm
Axial Tension	130 kN
Axial Compression	41.2 kN
Shear	13.9 kN

Therefore, ok

Nodal support internal forces [Nonlin., Envelope (All ULS)]

	Node	Type	C	min. max.	Case	Rx [kN]	Ry [kN]	Rz [kN]	Rr [kN]	Ryy [kNm]
Ext.										

Nodal support internal forces [Nonlin., Envelope (All ULS)]

	Node	*Type*	*C*	*min. max.*	*Case*	*Rx [kN]*	*Ry [kN]*	*Rz [kN]*	*Rr [kN]*	*Ryy [kNm]*
3	13	Glob.	Rx	min	G + Q + Ex [1] (1.000)	**–0.056**	0.775	–16.919	16.936	–0.006
2	3	Glob.		max	G + 0.6Q + Ed [1] (1.000)	**0.940**	0.033	–9.071	9.119	1.087
4	14	Glob.		max	G + 0.6Q + Ed [1] (1.000)	**0.939**	–0.032	–9.076	9.124	1.087
5	61	Glob.		max	G + 0.6Q + Ed [1] (1.000)	**0.940**	0.032	–9.042	9.091	1.087
6	62	Glob.		max	G + 0.6Q + Ed [1] (1.000)	**0.940**	–0.033	–9.037	9.086	1.087
7	86	Glob.		max	G + 0.6Q + Ed [1] (1.000)	**0.940**	0.033	–9.040	9.088	1.087
8	87	Glob.		max	G + 0.6Q + Ed [1] (1.000)	**0.940**	–0.032	–9.039	9.088	1.087
9	111	Glob.		max	G + 0.6Q + Ed [1] (1.000)	**0.940**	0.032	–9.040	9.089	1.087
10	112	Glob.		max	G + 0.6Q + Ed [1] (1.000)	**0.940**	–0.032	–9.039	9.087	1.087
11	136	Glob.		max	G + 0.6Q + Ed [1] (1.000)	**0.940**	0.032	–9.040	9.089	1.086
12	137	Glob.		max	G + 0.6Q + Ed [1] (1.000)	**0.940**	–0.032	–9.039	9.087	1.086
13	161	Glob.		max	G + 0.6Q + Ed [1] (1.000)	**0.939**	0.032	–9.040	9.088	1.086
14	162	Glob.		max	G + 0.6Q + Ed [1] (1.000)	**0.939**	–0.032	–9.039	9.087	1.086
6	62	Glob.	Ry	min	G + 0.6Q + Ed [1] (1.000)	0.940	**–0.033**	–9.037	9.086	1.087
2	3	Glob.		max	G + Q + Ex [1] (1.000)	0.011	**3.148**	10.275	10.746	0.006
23	286	Glob.		max	G + Q + Ex [1] (1.000)	–0.011	**3.148**	10.275	10.746	–0.006
4	14	Glob.	Rz	min	G + Q + Ex [1] (1.000)	0.006	1.982	**–39.348**	39.398	0.006
24	287	Glob.		min	G + Q + Ex [1] (1.000)	–0.006	1.982	**–39.348**	39.398	–0.006
2	3	Glob.		max	G + Q + Ex [1] (1.000)	0.011	3.148	**10.275**	10.746	0.006
23	286	Glob.		max	G + Q + Ex [1] (1.000)	–0.011	3.148	**10.275**	10.746	–0.006
3	13	Glob.	Ryy	min	G + Q + Ex [1] (1.000)	–0.056	0.775	–16.919	16.936	**–0.006**
23	286	Glob.		min	G + Q + Ex [1] (1.000)	–0.011	3.148	10.275	10.746	**–0.006**
2	3	Glob.		max	G + 0.6Q + Ed [1] (1.000)	0.940	0.033	–9.071	9.119	**1.087**
4	14	Glob.		max	G + 0.6Q + Ed [1] (1.000)	0.939	–0.032	–9.076	9.124	**1.087**
5	61	Glob.		max	G + 0.6Q + Ed [1] (1.000)	0.940	0.032	–9.042	9.091	**1.087**
6	62	Glob.		max	G + 0.6Q + Ed [1] (1.000)	0.940	–0.033	–9.037	9.086	**1.087**
7	86	Glob.		max	G + 0.6Q + Ed [1] (1.000)	0.940	0.033	–9.040	9.088	**1.087**
8	87	Glob.		max	G + 0.6Q + Ed [1] (1.000)	0.940	–0.032	–9.039	9.088	**1.087**
9	111	Glob.		max	G + 0.6Q + Ed [1] (1.000)	0.940	0.032	–9.040	9.089	**1.087**
10	112	Glob.		max	G + 0.6Q + Ed [1] (1.000)	0.940	–0.032	–9.039	9.087	**1.087**
11	136	Glob.		max	G + 0.6Q + Ed [1] (1.000)	0.940	0.032	–9.040	9.089	**1.086**
12	137	Glob.		max	G + 0.6Q + Ed [1] (1.000)	0.940	–0.032	–9.039	9.087	**1.086**

Node: Supported node; **Type:** Support type; **C:** Extremal component; **min. max.:** Extreme type; **Case:** Load case of extreme; **Rx:** Support reaction force x component; **Ry:** Support reaction force y component; **Rz:** Support reaction force z component; **Rr:** Resultant support reaction Force; **Ryy:** Support reaction moment y component;

Floor connections:	Ult. Moment	1.27 kNm - RAMSET specs
	Stiffness	100 kNm/rad
	RZ	10 kN

One trubolt or two dynabolts.

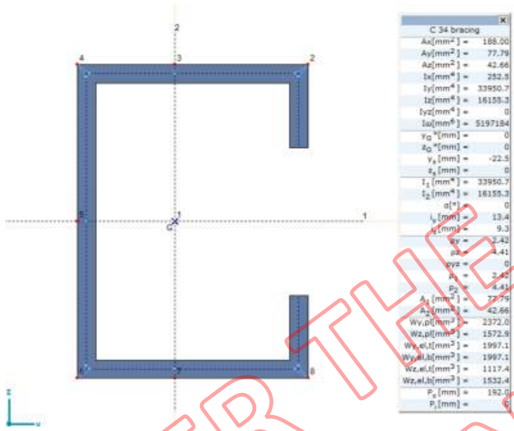
Therefore, ok

AS/NZS 4600 - COLD-FORMED STEEL STRUCTURES | INPUTS | JOB NUMBER: 16081149

CLIENT: Forbes and Davies | DATE: 11/08/2016 | SHEET: 1 of 2

2.1 SECTION PROPERTIES — C36 Brace

Young's Modulus	E =	200,000	MPa
Yield Strength	fy =	350	MPa
Ultimate Tensile Strength	fu =	480	MPa
Compression Flange Stress	f* =	350	MPa
Poisson's Ratio	v =	0.3	
Full Section Modulus (X)	Zfx =	1,997	mm^3
Full Section Modulus (Y)	Zfy =	1,117	mm^3
Full Cross-Sectional Area	Af =	188	mm^2
Full Second Moment of Area (X)	Ix =	33,950	mm^4
Full Second Moment of Area (Y)	Iy =	16,155	mm^4
Torsional Constant	J =	252	mm^4
Warping Constant	Iw =	5,197,184	mm^6
Shear Modulus of Elasticity	G =	80,000	MPa



Stress at compression flange for given scenario - using fy is a conservative approach. For more accuracy, this value should be updated when determining the critical stress values.

Effective Section Modulus (X)	Zex =	1,997	mm^3
Effective Section Modulus (Y)	Zey =	1,117	mm^3
Effective Cross-Sectional Area	Ae =	188	mm^2
Radius of Gyration (x)	rx =	13.4	mm
Radius of Gyration (y)	ry =	9.3	mm
Shear Centre X-Coordinate	x0 =	-22.5	mm
Shear Centre Y-Coordinate	y0 =	0.0	mm
Polar Radius of Gyration	r01 =	27.8	mm

2.2 EFFECTIVE WIDTHS OF STIFFENED ELEMENTS

This section shall be used to determine the effective section properties if not known. Using this in parallel with Spaceclaim, the effective section properties can be determined. The purpose of this section is to calculate the reduced compression element area. Refer to section 2.2 of AS/NZS 4600 for more information.

2.2.1 Uniformly Compressed Stiffened Elements

Thickness of Stiffened Element	t =	2.00	mm
Width of Flat Element	b =	25.0	mm
Plate Elastic Buckling Stress	fcr =	4628	MPa
Plate Buckling Coefficient	k =	4.00	
Slenderness Ratio	lamba =	0.275	
Effective Width Factor	p =	0.727	
Effective Width	**be =**	**25.0**	**mm**

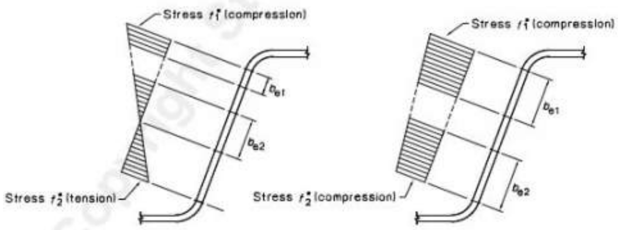
(a) Actual element

(b) Effective width (b_e) of element and design stress (f^*) on effective element

Effective width of element is essentially as if the area in the centre of the element has been cut-away. Use Spaceclaim to determine the effective section properties

2.2.3 Stiffened Elements With Stress Gradient

Thickness of Stiffened Element	t =	2	mm
Width of Flat Element	b =	36	mm
Compression Web Stress	f1* =	220	MPa
Compression/Tension Web Stres	f2* =	-220	MPa
Web Stress Ratio	Psi =	-1.000	
Plate Buckling Cofficient	k =	24.00	
Plate Elastic Buckling Stress	fcr =	13390	MPa
Slenderness Ratio	lamba =	0.128	
Effective Width Factor	p =	-5.59	
Effective Width	be =	36	mm
Effective Width 1	**be1 =**	**9.0**	**mm**
Effective Width 2	**be2 =**	**18**	**mm**



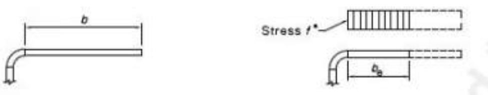
(b) Effective element and stress on effective element

Compressive stress is taken to be +ve and tension stress taken to be -ve

Note be1 + be2 shall not exceed the compression portion of the we

2.3.1 Uniformly Compressed Unstiffened Elements

Thickness of Unstiffened Elemen	t =	2	mm
Width of Flat Element	b =	6	mm
Plate Buckling Coefficient	k =	0.43	
Plate Elastic Buckling Stress	fcr =	8636	MPa
Slenderness Ratio	lamba =	0.201	
Effective Width Factor	p =	-0.461	



(a) Actual element

(b) Effective element and stress on effective element

Effective Width	**be =**	**6.0**	**mm**

2.3.2 Unstiffened Elements and Edge Stiffeners With Stress Gradient

Thickness of Unstiffened Elemen	t =	2	mm
Width of Flat Element	b =	10	mm
Compression Web Stress	f1* =	250	MPa
Compression/Tension Web Stres	f2* =	130	MPa
Web Stress Ratio	Psi =	0.52	
Type of Stress Gradient	Compression Decrease Fig 2.3.2(A)(i)		
Plate Buckling Coefficient	k =	0.672	
Plate Elastic Buckling Stress	fcr =	4860	MPa
Slenderness Ratio	lamba =	0.227	
Effective Width Factor	p =	0.13	
Effective Width	**be =**	**10.0**	**mm**

Figure 2.3.2(A)(i)

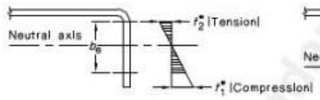


Figure 2.3.2(B)(i)

Figure 2.3.2(A)(ii)

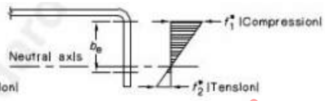
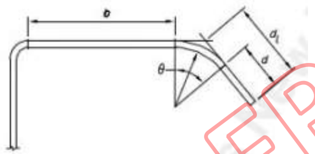
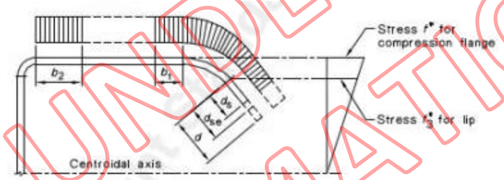


Figure 2.3.2(B)(ii)

2.4.2 Effective Width of Uniformly Compressed Elements With an Edge Stiffene

Thickness of Stiffened Element	t =	0	mm
Width of Flat Element	b =	0	mm
Width of Stiffened Element	dl =	0	mm
Width of Flat Stiffened Element	d =	0	mm
Angle of Edge Stiffener	theta =	0	degrees
Compression Flange Stress	f* =	0	MPa
Slenderness Factor	S =	#DIV/0!	
Exponent Factor	n =	#DIV/0!	
Second Moment Area Stiffener	Is =	#DIV/0!	mm^4
Adequate Second Moment Area	Ia =	#DIV/0!	mm^4
Plate Buckling Coefficient	k =	#DIV/0!	
Plate Elastic Buckling Stress	fcr =	#DIV/0!	MPa
Slenderness Factor	lamba =	#DIV/0!	
Effective Width Factor	p =	#DIV/0!	
Effective Width	be =	#DIV/0!	mm
Effective Width of Stiffener	dse =	32.0	mm
Reduced Effective Width	**b1 =**	**#DIV/0!**	**mm**
Reduced Effective Width	**b2 =**	**#DIV/0!**	**mm**
Reduced Effective Stiffener	**ds =**	**#DIV/0!**	**mm**



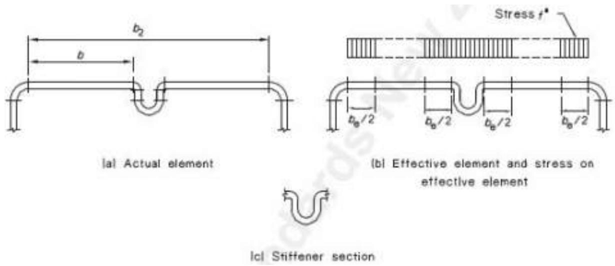


Calculate in accordance with 2.3.2 above

2.5.2 Effective Width of Uniformly Compressed Stiffened Elements With One Intermediate Stiffener

Thickness of Stiffened Element	t =	2	mm
Width of Element	b2 =	98	mm
Width of Flat Element	b =	34	mm
Stiffener Second Moment of Area	Is =	778	mm^4
Compression Flange Stress	f* =	250	MPa
Slenderness Factor	S =	36.2	
Adequate Second Moment Area	Ia =	282.8	mm^4
Exponent	n =	0.470	
Plate Buckling Coefficient	k =	4.00	
Plate Elastic Buckling Stress	fcr =	2502	MPa
Slenderness Ratio	lamba =	0.316	
Effective Width Factor	p =	0.96	
Effective Width	**be =**	**34.0**	**mm**

Second moment of area of the full stiffener about its own centroidal axis parallel to the element



CALCULATED ACTIONS

Bending Moment Action (X)	Mx* =	0	Nmm
Bending Moment Action (Y)	My* =	0	Nmm
Axial Compression	Nc* =	0	N
Axial Tension	Nt* =	0	N
Shear	Vv* =	0	N

3.2 MEMBERS SUBJECT TO AXIAL TENSION

Capacity Reduction Factor	phi_t =	0.90		*Table 1.6 - Tension capacity reduction factor*
Correction Factor	k_t =	0.75		*Table 3.2 - Correction factor for shaded element*
Area of Penetrations or Holes	Ar =	78.0	mm^2	*Reduction of cross-sectional area from penetrations or holes*
Nominal Section Capacity	Nt =	33,660	N	
Axial Tensile Force Limit	**phi_t*Nt =**	**30,294**	**N**	**OK**

3.3 MEMBERS SUBJECT TO BENDING

3.3.2 Nominal Section Moment Capacity

Capacity Reduction Factor	phi_t =	0.90		*Table 1.6 - Bending capacity reduction factor*
Section Moment Capacity (X)	Msx =	698,950	Nmm	*Based on initiation of yielding*
Section Moment Capacity (Y)	Msy =	390,950	Nmm	*Based on initiation of yielding*
Section Moment Limit (X)	**phi_t*Msx =**	**629,055**	**Nmm**	**OK**
Section Moment Limit (Y)	**phi_t*Msy =**	**351,855**	**Nmm**	**OK**

3.3.3.2 Members Subject to Lateral Buckling

Effective Buckling Length	ley =	610	mm	
Effective Section Modulus (X)	Zex =	1,997	mm^3	*Calculated at the critical stress of the member - using fy as critical stress is conservative*
Elastic Buckling Moment	Mo =	2,022,482	Nmm	
Moment Distribution Coefficient	Cb =	1.0		*Unity is a conservative approach - refer to equation 3.3.3.2(9) for more accuracy*
Flexural Elastic Buckling Stress	foy =	456	MPa	
Torsional Elastic Buckling Stress	foz =	329	MPa	
Initial Yield Moment	My =	698,950	Nmm	
Slenderness Ratio	lamba_b =	0.588		
Critical Moment	Mc =	698,950	Nmm	
Critical Stress	fc =	350	MPa	*Critical stress value used to determine effective section modulus for member moment capacity*
Member Moment Capacity (X)	Mbx =	698,950	Nmm	
Member Moment Limit (X)	**phi_t*Mbx =**	**629,055**	**Nmm**	**OK**

3.3.3.3 Members Subject to Distortional Buckling

Distortional buckling is local failure of the section - ie if the web was to crumple, or the flange were to rotate/distort under compression. The elastic distortional buckling stress is often too complex to calculate by hand, so ANSYS is recommended to be used to determine this stress. Appendix D gives formulas for specific sections.

Elastic Distortional Buckling Stress	fod =	1000	MPa	*Determined from rational elastic buckling analysis - usually ANSYS is best approach here*
Elastic Buckling Moment (X)	Modx =	1,997,000	Nmm	
Initial Yield Moment	My =	698,950	Nmm	
Slenderness Ratio	lamba_d =	0.59	Nmm	
Mode of Distortional Buckling		(a) Flange Rotation		*Refer to section 3.3.3.3 for more details*
Critical Moment (X)	Mcx =	698,950	Nmm	
Member Moment Limit (X)	**phi_t*Mcx =**	**629,055**	**Nmm**	**OK**

3.3.4 Shear Capacity of Webs

Capacity Reduction Factor	phi_v =	0.90		*Table 1.6 - Shear capacity reduction factor*
Thickness of Web	tw =	3.0	mm	
Depth of Flat Portion of Web	d1 =	212	mm	
Depth of Web Hole	dwh =	0	mm	*Web holes shall comply with the requirements setout in 3.3.4.2*
Web Hole Reduction Multiplier	qs =	1.000		
Shear Buckling Coefficient	kv =	5.34		*For unstiffened webs - refer to 3.3.4.1 for webs with stiffeners*
Nominal Shear Capacity	Vv =	111,363	N	

Nominal Shear Capacity Web Hole	Vvwh =	111,363	N	
Web Shear Limit	**phi_v*Vv =**	**100,227**	**N**	**OK**

3.4 CONCENTRICALLY LOADED COMPRESSION MEMBERS

3.4.2 Sections Not Subject to Torsional or Flexural-Torsional Buckling

Capacity Reduction Factor	phi_c =	0.85		*Table 1.6 - Concentrically loaded compression member capacity reduction factor*
Effective Buckling Length	le =	610	mm	
Effective Area	Ae =	188	mm^2	*Calculated at the critical stress of the member - using fy as critical stress is conservative*
Elastic Flexural Buckling Stress	foc =	456	MPa	
Slenderness Ratio	lamba_c =	0.88		
Critical Stress	fn =	254	MPa	*Critical stress value used to determine effective area of section under axial compression*
Nominal Member Capacity	Nc =	47,716	N	
Axial Compression Limit	**phi_c*Nc =**	**40,558**	**N**	**OK**

3.4.3 Doubly- or Singly-Symmetric Sections Subject to Torsional or Flexural-Torsional Buckling

Flexural Elastic Buckling Stress	fox =	958	MPa	
Torsional Elastic Buckling Stress	foz =	329	MPa	
Beta Factor	beta =	0.345		
Elastic Flexural Buckling Stress	foxz =	263	MPa	
Slenderness Ratio	lamba_c =	1.15		
Critical Stress	fn =	201	MPa	*Critical stress value used to determine effective area of section under axial compression*
Nominal Member Capacity	Nc =	37,715	N	
Axial Compression Limit	**phi_c*Nc =**	**32,058**	**N**	**OK**

3.4.6 Singly-Symmetric Section Subject to Distortional Buckling

Nominal Member Capacity	Nc =	60,043	N	*Note that this must be the lesser value of Nc calculated in accordance with 3.4.2 and 3.4.6*
Axial Compression Limit	**phi_c*Nc =**	**40,558**	**N**	**OK**

3.5 COMBINED AXIAL COMPRESSION OR TENSION, AND BENDING

3.5.1 Combined Axial Compression and Bending

Axial Compression Ratio	Nc*/phi_c*Nc =	0.000	
Unequal Moment Coefficient (X)	Cmx =	1.00	*Refer to section 3.5.1*
Unequal Moment Coefficient (Y)	Cmy =	1.00	*Refer to section 3.5.1*
Moment Amplification Factor (X)	alpha_nx =	1.00	
Moment Amplification Factor (Y)	alpha_ny =	1.00	
Unity Equation	**=**	**0.000**	**OK**

3.5.2 Combined Axial Tension and Bending

Unity Equation **=** **0.000** **OK**

3.3.5 Combined Bending and Shear

Unity Equation **=** **0.000** **OK**

2.1 SECTION PROPERTIES — Racking Beam 90x50

Young's Modulus	E =	200,000	MPa
Yield Strength	fy =	350	MPa
Ultimate Tensile Strength	fu =	410	MPa
Compression Flange Stress	f* =	350	MPa
Poisson's Ratio	v =	0.3	

Stress at compression flange for given scenario - using fy is a conservative approach. For more accuracy, this value should be updated when determining the critical stress values.

Full Section Modulus (X)	Zfx =	15,912	mm^3
Full Section Modulus (Y)	Zfy =	8,429	mm^3
Full Cross-Sectional Area	Af =	552	mm^2
Full Second Moment of Area (X)	Ix =	716,076	mm^4
Full Second Moment of Area (Y)	Iy =	210,739	mm^4
Torsional Constant	J =	488,857	mm^4
Warping Constant	Iw =	67,000,000	mm^6
Shear Modulus of Elasticity	G =	80,000	MPa

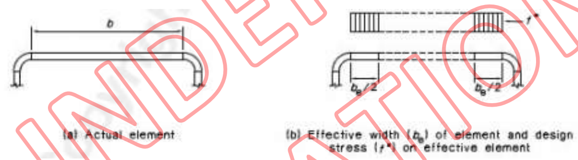
Effective Section Modulus (X)	Zex =	15,912	mm^3
Effective Section Modulus (Y)	Zey =	8,429	mm^3
Effective Cross-Sectional Area	Ae =	552	mm^2
Radius of Gyration (x)	rx =	36.0	mm
Radius of Gyration (y)	ry =	19.5	mm
Shear Centre X-Coordinate	x0 =	0.0	mm
Shear Centre Y-Coordinate	y0 =	0.0	mm
Polar Radius of Gyration	r01 =	41.0	mm

2.2 EFFECTIVE WIDTHS OF STIFFENED ELEMENTS

This section shall be used to determine the effective section properties if not known. Using this in parallel with Spaceclaim, the effective section properties can be determined. The purpose of this section is to calculate the reduced compression element area. Refer to section 2.2 of AS/NZS 4600 for more information.

2.2.1 Uniformly Compressed Stiffened Elements

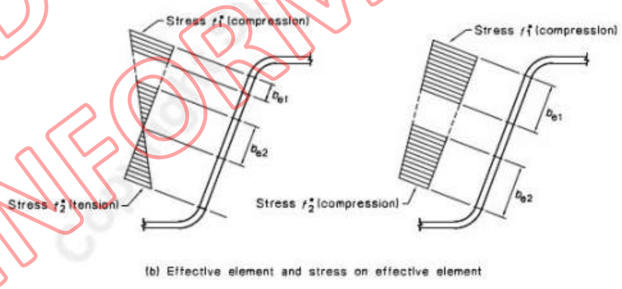
Thickness of Stiffened Element	t =	3.00	mm
Width of Flat Element	b =	50.0	mm
Plate Elastic Buckling Stress	fcr =	2603	MPa
Plate Buckling Coefficient	k =	4.00	
Slenderness Ratio	lamba =	0.367	
Effective Width Factor	p =	1.000	
Effective Width	**be =**	**50.0**	**mm**



Effective width of element is essentially as if the area in the centre of the element has been cut-away. Use Spaceclaim to determine the effective section properties

2.2.3 Stiffened Elements With Stress Gradient

Thickness of Stiffened Element	t =	1.5	mm
Width of Flat Element	b =	90	mm
Compression Web Stress	f1* =	220	MPa
Compression/Tension Web Stres	f2* =	-220	MPa
Web Stress Ratio	Psi =	-1.000	
Plate Buckling Cofficient	k =	24.00	
Plate Elastic Buckling Stress	fcr =	1205	MPa
Slenderness Ratio	lamba =	0.427	
Effective Width Factor	p =	1.00	
Effective Width	be =	90	mm
Effective Width 1	**be1 =**	**22.5**	**mm**
Effective Width 2	**be2 =**	**45**	**mm**

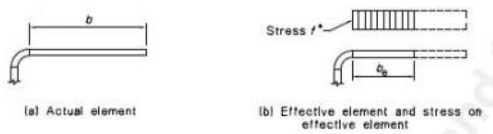


Compressive stress is taken to be +ve and tension stress taken to be -ve

Note be1 + be2 shall not exceed the compression portion of the we

2.3.1 Uniformly Compressed Unstiffened Elements

Thickness of Unstiffened Elemen	t =	0	mm
Width of Flat Element	b =	0	mm
Plate Buckling Coefficient	k =	0.43	
Plate Elastic Buckling Stress	fcr =	#DIV/0!	MPa
Slenderness Ratio	lamba =	#DIV/0!	
Effective Width Factor	p =	#DIV/0!	
Effective Width	**be =**	**#DIV/0!**	**mm**



2.3.2 Unstiffened Elements and Edge Stiffeners With Stress Gradient

Thickness of Unstiffened Elemen	t =	0	mm
Width of Flat Element	b =	0	mm
Compression Web Stress	f1* =	0	MPa
Compression/Tension Web Stres	f2* =	0	MPa
Web Stress Ratio	Psi =	#DIV/0!	
Type of Stress Gradient		Compression Increase Fig 2.3.2(A)(ii)	
Plate Buckling Coefficient	k =	#DIV/0!	
Plate Elastic Buckling Stress	fcr =	#DIV/0!	MPa
Slenderness Ratio	lamba =	#DIV/0!	

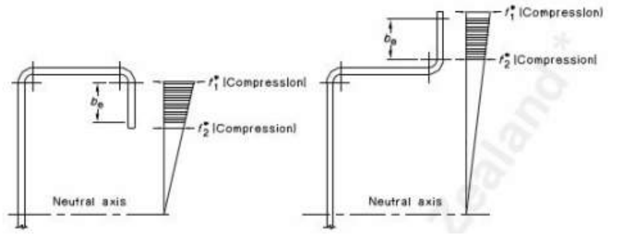
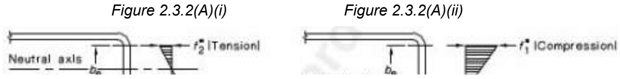


Figure 2.3.2(A)(i) *Figure 2.3.2(A)(ii)*

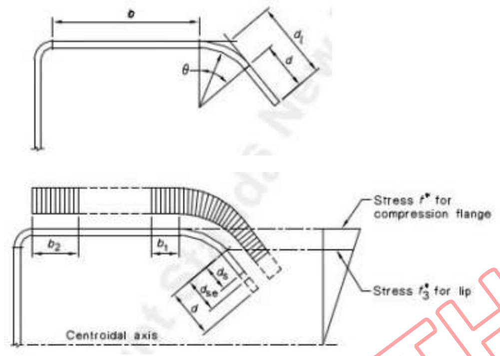


Effective Width Factor	p =	#DIV/0!	
Effective Width	**be =**	**#DIV/0!**	**mm**

Figure 2.3.2(B)(i) *Figure 2.3.2(B)(ii)*

2.4.2 Effective Width of Uniformly Compressed Elements With an Edge Stiffene

Thickness of Stiffened Element	t =	0	mm
Width of Flat Element	b =	0	mm
Width of Stiffened Element	dl =	0	mm
Width of Flat Stiffened Element	d =	0	mm
Angle of Edge Stiffener	theta =	0	degrees
Compression Flange Stress	f* =	0	MPa
Slenderness Factor	S =	#DIV/0!	
Exponent Factor	n =	#DIV/0!	
Second Moment Area Stiffener	Is =	#DIV/0!	mm^4
Adequate Second Moment Area	Ia =	#DIV/0!	mm^4
Plate Buckling Coefficient	k =	#DIV/0!	
Plate Elastic Buckling Stress	fcr =	#DIV/0!	MPa
Slenderness Factor	lamba =	#DIV/0!	
Effective Width Factor	p =	#DIV/0!	
Effective Width	be =	#DIV/0!	mm
Effective Width of Stiffener	dse =	32.0	mm



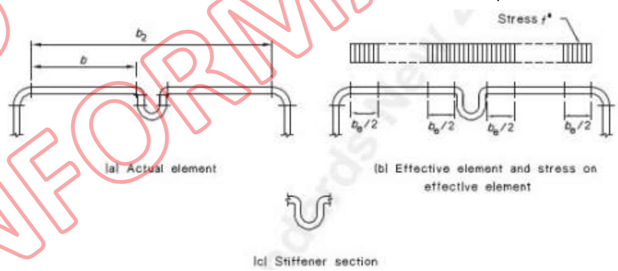
Calculate in accordance with 2.3.2 above

Reduced Effective Width	**b1 =**	**#DIV/0!**	**mm**
Reduced Effective Width	**b2 =**	**#DIV/0!**	**mm**
Reduced Effective Stiffener	**ds =**	**#DIV/0!**	**mm**

2.5.2 Effective Width of Uniformly Compressed Stiffened Elements With One Intermediate Stiffener

Thickness of Stiffened Element	t =	1.5	mm
Width of Element	b2 =	98	mm
Width of Flat Element	b =	34	mm
Stiffener Second Moment of Area	Is =	778	mm^4
Compression Flange Stress	f* =	250	MPa
Slenderness Factor	S =	36.2	
Adequate Second Moment Area	Ia =	203.7	mm^4
Exponent	n =	0.433	
Plate Buckling Coefficient	k =	4.00	
Plate Elastic Buckling Stress	fcr =	1407	MPa
Slenderness Ratio	lamba =	0.421	
Effective Width Factor	p =	1.00	
Effective Width	**be =**	**34.0**	**mm**

Second moment of area of the full stiffener about its own centroidal axis parallel to the element



CALCULATED ACTIONS

Bending Moment Action (X)	Mx* =	0	Nmm	
Bending Moment Action (Y)	My* =	0	Nmm	
Axial Compression	Nc* =	0	N	
Axial Tension	Nt* =	0	N	
Shear	Vv* =	0	N	

3.2 MEMBERS SUBJECT TO AXIAL TENSION

Capacity Reduction Factor	phi_t =	0.90		*Table 1.6 - Tension capacity reduction factor*
Correction Factor	k_t =	0.75		*Table 3.2 - Correction factor for shaded element*
Area of Penetrations or Holes	Ar =	0.0	mm^2	*Reduction of cross-sectional area from penetrations or holes*
Nominal Section Capacity	Nt =	144,279	N	
Axial Tensile Force Limit	**phi_t*Nt =**	**129,851**	**N**	**OK**

3.3 MEMBERS SUBJECT TO BENDING

3.3.2 Nominal Section Moment Capacity

Capacity Reduction Factor	phi_t =	0.90		*Table 1.6 - Bending capacity reduction factor*
Section Moment Capacity (X)	Msx =	5,569,200	Nmm	*Based on initiation of yielding*
Section Moment Capacity (Y)	Msy =	2,950,150	Nmm	*Based on initiation of yielding*
Section Moment Limit (X)	**phi_t*Msx =**	**5,012,280**	**Nmm**	**OK**
Section Moment Limit (Y)	**phi_t*Msy =**	**2,655,135**	**Nmm**	**OK**

3.3.3.2 Members Subject to Lateral Buckling

Effective Buckling Length	ley =	2,743	mm	
Effective Section Modulus (X)	Zex =	15,912	mm^3	*Calculated at the critical stress of the member - using fy as critical stress is conservative*
Elastic Buckling Moment	Mo =	46,509,857	Nmm	
Moment Distribution Coefficient	Cb =	1.0		*Unity is a conservative approach - refer to equation 3.3.3.2(9) for more accuracy*
Flexural Elastic Buckling Stress	foy =	100	MPa	
Torsional Elastic Buckling Stress	foz =	42216	MPa	
Initial Yield Moment	My =	5,569,200	Nmm	
Slenderness Ratio	lamba_b =	0.346		
Critical Moment	Mc =	5,569,200	Nmm	
Critical Stress	fc =	350	MPa	*Critical stress value used to determine effective section modulus for member moment capacity*
Member Moment Capacity (X)	Mbx =	5,569,200	Nmm	
Member Moment Limit (X)	**phi_t*Mbx =**	**5,012,280**	**Nmm**	**OK**

3.3.3.3 Members Subject to Distortional Buckling

Distortional buckling is local failure of the section - ie if the web was to crumple, or the flange were to rotate/distort under compression. The elastic distortional buckling stress is often too complex to calculate by hand, so ANSYS is recommended to be used to determine this stress. Appendix D gives formulas for specific sections.

Elastic Distortional Buckling Stress	fod =	1000	MPa	*Determined from rational elastic buckling analysis - usually ANSYS is best approach here*
Elastic Buckling Moment (X)	Modx =	15,912,000	Nmm	
Initial Yield Moment	My =	5,569,200	Nmm	
Slenderness Ratio	lamba_d =	0.59	Nmm	
Mode of Distortional Buckling		(a) Flange Rotation		*Refer to section 3.3.3.3 for more details*
Critical Moment (X)	Mcx =	5,569,200	Nmm	
Member Moment Limit (X)	**phi_t*Mcx =**	**5,012,280**	**Nmm**	**OK**

3.3.4 Shear Capacity of Webs

Capacity Reduction Factor	phi_v =	0.90		*Table 1.6 - Shear capacity reduction factor*
Thickness of Web	tw =	1.5	mm	
Depth of Flat Portion of Web	d1 =	90	mm	
Depth of Web Hole	dwh =	0	mm	*Web holes shall comply with the requirements setout in 3.3.4.2*
Web Hole Reduction Multiplier	qs =	0.556		
Shear Buckling Coefficient	kv =	5.34		*For unstiffened webs - refer to 3.3.4.1 for webs with stiffeners*
Nominal Shear Capacity	Vv =	27,841	N	

Nominal Shear Capacity Web Hole	Vvwh =	15,467	N	
Web Shear Limit	**phi_v*Vv =**	**13,920**	**N**	**OK**

3.4 CONCENTRICALLY LOADED COMPRESSION MEMBERS

3.4.2 Sections Not Subject to Torsional or Flexural-Torsional Buckling

Capacity Reduction Factor	phi_c =	0.85		*Table 1.6 - Concentrically loaded compression member capacity reduction factor*
Effective Buckling Length	le =	2,743	mm	
Effective Area	Ae =	552	mm^2	*Calculated at the critical stress of the member - using fy as critical stress is conservative*
Elastic Flexural Buckling Stress	foc =	100	MPa	
Slenderness Ratio	lamba_c =	1.87		
Critical Stress	fn =	88	MPa	*Critical stress value used to determine effective area of section under axial compression*
Nominal Member Capacity	Nc =	48,487	N	
Axial Compression Limit	**phi_c*Nc =**	**41,214**	**N**	**OK**

3.4.3 Doubly- or Singly-Symmetric Sections Subject to Torsional or Flexural-Torsional Buckling

Flexural Elastic Buckling Stress	fox =	340	MPa	
Torsional Elastic Buckling Stress	foz =	42216	MPa	
Beta Factor	beta =	1.000		
Elastic Flexural Buckling Stress	foxz =	340	MPa	
Slenderness Ratio	lamba_c =	1.87		
Critical Stress	fn =	88	MPa	*Critical stress value used to determine effective area of section under axial compression*
Nominal Member Capacity	Nc =	48,487	N	
Axial Compression Limit	**phi_c*Nc =**	**41,214**	**N**	**OK**

3.4.6 Singly-Symmetric Section Subject to Distortional Buckling

Nominal Member Capacity	Nc =	176,295	N	*Note that this must be the lesser value of Nc calculated in accordance with 3.4.2 and 3.4.6*
Axial Compression Limit	**phi_c*Nc =**	**41,214**	**N**	**OK**

3.5 COMBINED AXIAL COMPRESSION OR TENSION, AND BENDING

3.5.1 Combined Axial Compression and Bending

Axial Compression Ratio	Nc*/phi_c*Nc =	0.000	
Unequal Moment Coefficient (X)	Cmx =	1.00	*Refer to section 3.5.1*
Unequal Moment Coefficient (Y)	Cmy =	1.00	*Refer to section 3.5.1*
Moment Amplification Factor (X)	alpha_nx =	1.00	
Moment Amplification Factor (Y)	alpha_ny =	1.00	
Unity Equation	**=**	**0.000**	**OK**

3.5.2 Combined Axial Tension and Bending

Unity Equation	**=**	**0.000**	**OK**

3.3.5 Combined Bending and Shear

Unity Equation	**=**	**0.000**	**OK**

BVT Engineering Professional Services

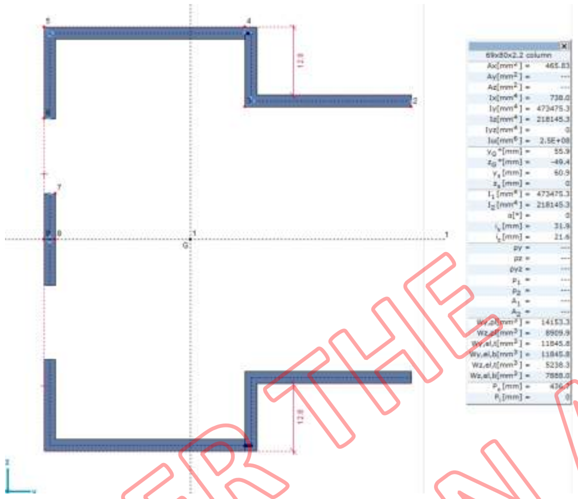
AS/NZS 4600 - COLD-FORMED STEEL STRUCTURES

INPUTS JOB NUMBER: 16081149

CLIENT: Forbes & Davies DATE: 11/08/2016 SHEET: 1 OF 2

2.1 SECTION PROPERTIES 68x80x2.2 Column

Young's Modulus	E =	200,000	MPa
Yield Strength	fy =	450	MPa
Ultimate Tensile Strength	fu =	525	MPa
Compression Flange Stress	f* =	450	MPa
Poisson's Ratio	v =	0.3	
Full Section Modulus (X)	Zfx =	11,845	mm^3
Full Section Modulus (Y)	Zfy =	7,888	mm^3
Full Cross-Sectional Area	Af =	465	mm^2
Full Second Moment of Area (X)	Ix =	473,475	mm^4
Full Second Moment of Area (Y)	Iy =	218,145	mm^4
Torsional Constant	J =	738	mm^4
Warping Constant	Iw =	250,000,000	mm^6
Shear Modulus of Elasticity	G =	80,000	MPa



Stress at compression flange for given scenario - using fy is a conservative approach. For more accuracy, this value should be updated when determining the critical stress values.

Effective Section Modulus (X)	Zex =	11,845	mm^3
Effective Section Modulus (Y)	Zey =	7,888	mm^3
Effective Cross-Sectional Area	Ae =	465	mm^2
Radius of Gyration (x)	rx =	31.9	mm
Radius of Gyration (y)	ry =	21.7	mm
Shear Centre X-Coordinate	x0 =	60.9	mm
Shear Centre Y-Coordinate	y0 =	0.0	mm
Polar Radius of Gyration	r01 =	72.1	mm

2.2 EFFECTIVE WIDTHS OF STIFFENED ELEMENTS

This section shall be used to determine the effective section properties if not known. Using this in parallel with Spaceclaim, the effective section properties can be determined. The purpose of this section is to calculate the reduced compression element area. Refer to section 2.2 of AS/NZS 4600 for more information.

2.2.1 Uniformly Compressed Stiffened Elements

Thickness of Stiffened Element	t =	2.20	mm
Width of Flat Element	b =	80.0	mm
Plate Elastic Buckling Stress	fcr =	547	MPa
Plate Buckling Coefficient	k =	4.00	
Slenderness Ratio	lamba =	0.907	
Effective Width Factor	p =	0.835	
Effective Width	**be =**	**66.8**	**mm**

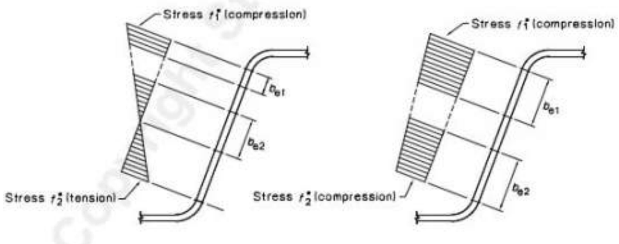
(a) Actual element

(b) Effective width (b_e) of element and design stress (f^*) on effective element

Effective width of element is essentially as if the area in the centre of the element has been cut-away. Use Spaceclaim to determine the effective section properties

2.2.3 Stiffened Elements With Stress Gradient

Thickness of Stiffened Element	t =	2.2	mm
Width of Flat Element	b =	49.8	mm
Compression Web Stress	f1* =	143	MPa
Compression/Tension Web Stres	f2* =	205	MPa
Web Stress Ratio	Psi =	1.434	
Plate Buckling Coefficient	k =	2.97	
Plate Elastic Buckling Stress	fcr =	1048	MPa
Slenderness Ratio	lamba =	0.369	
Effective Width Factor	p =	1.00	
Effective Width	be =	49.8	mm
Effective Width 1	**be1 =**	**31.8**	**mm**
Effective Width 2	**be2 =**	**18.0**	**mm**



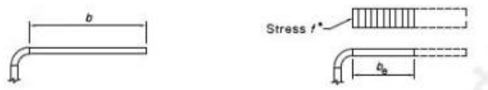
(b) Effective element and stress on effective element

Compressive stress is taken to be +ve and tension stress taken to be -ve

Note be1 + be2 shall not exceed the compression portion of the we

2.3.1 Uniformly Compressed Unstiffened Elements

Thickness of Unstiffened Elemen	t =	0	mm
Width of Flat Element	b =	0	mm
Plate Buckling Coefficient	k =	0.43	
Plate Elastic Buckling Stress	fcr =	#DIV/0!	MPa
Slenderness Ratio	lamba =	#DIV/0!	



(a) Actual element

(b) Effective element and stress on effective element

Effective Width Factor	p =	#DIV/0!	
Effective Width	**be =**	**#DIV/0!**	**mm**

2.3.2 Unstiffened Elements and Edge Stiffeners With Stress Gradient

Thickness of Unstiffened Elemen	t =	2.2	mm
Width of Flat Element	b =	18.2	mm
Compression Web Stress	f1* =	205	MPa
Compression/Tension Web Stres	f2* =	143	MPa
Web Stress Ratio	Psi =	0.70	
Type of Stress Gradient	Compression Increase Fig 2.3.2(A)(ii)		
Plate Buckling Coefficient	k =	0.458	
Plate Elastic Buckling Stress	fcr =	1209	MPa
Slenderness Ratio	lamba =	0.412	
Effective Width Factor	p =	1.00	
Effective Width	**be =**	**18.2**	**mm**

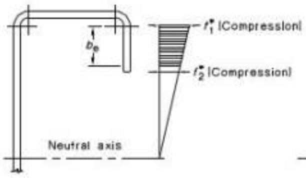


Figure 2.3.2(A)(i)

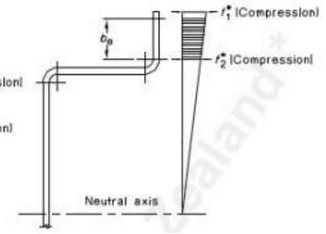


Figure 2.3.2(A)(ii)

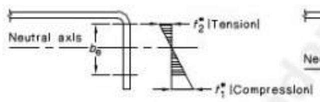
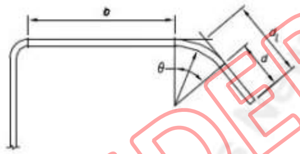


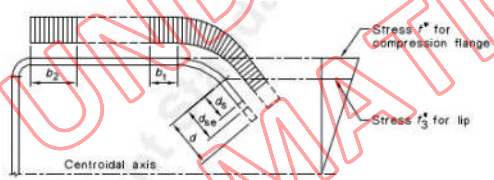
Figure 2.3.2(B)(i)

Figure 2.3.2(B)(ii)

2.4.2 Effective Width of Uniformly Compressed Elements With an Edge Stiffene

Thickness of Stiffened Element	t =	0	mm
Width of Flat Element	b =	0	mm
Width of Stiffened Element	dl =	0	mm
Width of Flat Stiffened Element	d =	0	mm
Angle of Edge Stiffener	theta =	0	degrees
Compression Flange Stress	f* =	0	MPa
Slenderness Factor	S =	#DIV/0!	
Exponent Factor	n =	#DIV/0!	
Second Moment Area Stiffener	Is =	#DIV/0!	mm^4
Adequate Second Moment Area	Ia =	#DIV/0!	mm^4
Plate Buckling Coefficient	k =	#DIV/0!	
Plate Elastic Buckling Stress	fcr =	#DIV/0!	MPa
Slenderness Factor	lamba =	#DIV/0!	
Effective Width Factor	p =	#DIV/0!	
Effective Width	be =	#DIV/0!	mm
Effective Width of Stiffener	dse =	0.0	mm
Reduced Effective Width	**b1 =**	**#DIV/0!**	**mm**
Reduced Effective Width	**b2 =**	**#DIV/0!**	**mm**
Reduced Effective Stiffener	**ds =**	**#DIV/0!**	**mm**



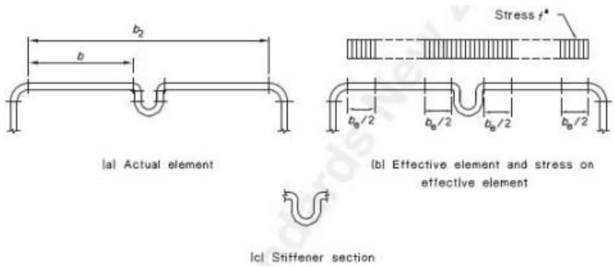


Calculate in accordance with 2.3.2 above

2.5.2 Effective Width of Uniformly Compressed Stiffened Elements With One Intermediate Stiffener

Thickness of Stiffened Element	t =	2.2	mm
Width of Element	b2 =	80	mm
Width of Flat Element	b =	37.5	mm
Stiffener Second Moment of Area	Is =	10	mm^4
Compression Flange Stress	f* =	250	MPa
Slenderness Factor	S =	36.2	
Adequate Second Moment Area	Ia =	5.2	mm^4
Exponent	n =	0.499	
Plate Buckling Coefficient	k =	4.00	
Plate Elastic Buckling Stress	fcr =	2489	MPa
Slenderness Ratio	lamba =	0.317	
Effective Width Factor	p =	0.97	
Effective Width	**be =**	**37.5**	**mm**

Second moment of area of the full stiffener about its own centroidal axis parallel to the element



CALCULATED ACTIONS

Bending Moment Action (X)	Mx* =	0	Nmm	
Bending Moment Action (Y)	My* =	0	Nmm	
Axial Compression	Nc* =	57,000	N	
Axial Tension	Nt* =	0	N	
Shear	Vv* =	0	N	

3.2 MEMBERS SUBJECT TO AXIAL TENSION

Capacity Reduction Factor	phi_t =	0.90		*Table 1.6 - Tension capacity reduction factor*
Correction Factor	k_t =	1.00		*Table 3.2 - Correction factor for shaded element*
Area of Penetrations or Holes	Ar =	0.0	mm^2	*Reduction of cross-sectional area from penetrations or holes*
Nominal Section Capacity	Nt =	207,506	N	
Axial Tensile Force Limit	**phi_t*Nt =**	**186,756**	**N**	**OK**

3.3 MEMBERS SUBJECT TO BENDING

3.3.2 Nominal Section Moment Capacity

Capacity Reduction Factor	phi_t =	0.90		*Table 1.6 - Bending capacity reduction factor*
Section Moment Capacity (X)	Msx =	5,330,250	Nmm	*Based on initiation of yielding*
Section Moment Capacity (Y)	Msy =	3,549,600	Nmm	*Based on initiation of yielding*
Section Moment Limit (X)	**phi_t*Msx =**	**4,797,225**	**Nmm**	**OK**
Section Moment Limit (Y)	**phi_t*Msy =**	**3,194,640**	**Nmm**	**OK**

3.3.3.2 Members Subject to Lateral Buckling

Effective Buckling Length	ley =	1,200	mm	
Effective Section Modulus (X)	Zex =	11,845	mm^3	*Calculated at the critical stress of the member - using fy as critical stress is conservative*
Elastic Buckling Moment	Mo =	10,960,387	Nmm	
Moment Distribution Coefficient	Cb =	1.0		*Unity is a conservative approach - refer to equation 3.3.3.2(9) for more accuracy*
Flexural Elastic Buckling Stress	foy =	643	MPa	
Torsional Elastic Buckling Stress	foz =	166	MPa	
Initial Yield Moment	My =	5,330,250	Nmm	
Slenderness Ratio	lamba_b =	0.697		
Critical Moment	Mc =	5,117,314	Nmm	
Critical Stress	fc =	432	MPa	*Critical stress value used to determine effective section modulus for member moment capacity*
Member Moment Capacity (X)	Mbx =	5,117,314	Nmm	
Member Moment Limit (X)	**phi_t*Mbx =**	**4,605,583**	**Nmm**	**OK**

3.3.3.3 Members Subject to Distortional Buckling

Distortional buckling is local failure of the section - ie if the web was to crumple, or the flange were to rotate/distort under compression. The elastic distortional buckling stress is often too complex to calculate by hand, so ANSYS is recommended to be used to determine this stress. Appendix D gives formulas for specific sections.

Elastic Distortional Buckling Stress	fod =	1000	MPa	*Determined from rational elastic buckling analysis - usually ANSYS is best approach here*
Elastic Buckling Moment (X)	Modx =	11,845,000	Nmm	
Initial Yield Moment	My =	5,330,250	Nmm	
Slenderness Ratio	lamba_d =	0.67	Nmm	
Mode of Distortional Buckling		(a) Flange Rotation		*Refer to section 3.3.3.3 for more details*
Critical Moment (X)	Mcx =	5,330,250	Nmm	
Member Moment Limit (X)	**phi_t*Mcx =**	**4,797,225**	**Nmm**	**OK**

3.3.4 Shear Capacity of Webs

Capacity Reduction Factor	phi_v =	0.90		*Table 1.6 - Shear capacity reduction factor*
Thickness of Web	tw =	2.2	mm	
Depth of Flat Portion of Web	d1 =	80	mm	
Depth of Web Hole	dwh =	0	mm	*Web holes shall comply with the requirements setout in 3.3.4.2*
Web Hole Reduction Multiplier	qs =	0.337		
Shear Buckling Coefficient	kv =	5.34		*For unstiffened webs - refer to 3.3.4.1 for webs with stiffeners*
Nominal Shear Capacity	Vv =	50,688	N	

Nominal Shear Capacity Web Hole	Vvwh =	17,067	N	
Web Shear Limit	**phi_v*Vv =**	**15,360**	**N**	**OK**

3.4 CONCENTRICALLY LOADED COMPRESSION MEMBERS

3.4.2 Sections Not Subject to Torsional or Flexural-Torsional Buckling

Capacity Reduction Factor	phi_c =	0.90		*Table 1.6 - Concentrically loaded compression member capacity reduction factor*
Effective Buckling Length	le =	1,200	mm	
Effective Area	Ae =	465	mm^2	*Calculated at the critical stress of the member - using fy as critical stress is conservative*
Elastic Flexural Buckling Stress	foc =	643	MPa	
Slenderness Ratio	lamba_c =	0.84		
Critical Stress	fn =	336	MPa	*Critical stress value used to determine effective area of section under axial compression*
Nominal Member Capacity	Nc =	156,123	N	
Axial Compression Limit	**phi_c*Nc =**	**140,510**	**N**	**OK**

3.4.3 Doubly- or Singly-Symmetric Sections Subject to Torsional or Flexural-Torsional Buckling

Flexural Elastic Buckling Stress	fox =	1396	MPa	
Torsional Elastic Buckling Stress	foz =	166	MPa	
Beta Factor	beta =	0.286		
Elastic Flexural Buckling Stress	foxz =	153	MPa	
Slenderness Ratio	lamba_c =	1.72		
Critical Stress	fn =	134	MPa	*Critical stress value used to determine effective area of section under axial compression*
Nominal Member Capacity	Nc =	62,333	N	
Axial Compression Limit	**phi_c*Nc =**	**56,100**	**N**	**NOT OK**

3.4.6 Singly-Symmetric Section Subject to Distortional Buckling

Nominal Member Capacity	Nc =	185,709	N	*Note that this must be the lesser value of Nc calculated in accordance with 3.4.2 and 3.4.6*
Axial Compression Limit	**phi_c*Nc =**	**140,510**	**N**	**OK**

3.5 COMBINED AXIAL COMPRESSION OR TENSION, AND BENDING

3.5.1 Combined Axial Compression and Bending

Axial Compression Ratio	Nc*/phi_c*Nc =	1.016	
Unequal Moment Coefficient (X)	Cmx =	1.00	*Refer to section 3.5.1*
Unequal Moment Coefficient (Y)	Cmy =	1.00	*Refer to section 3.5.1*
Moment Amplification Factor (X)	alpha_nx =	0.91	
Moment Amplification Factor (Y)	alpha_ny =	0.81	
Unity Equation	**=**	**1.02**	**NOT OK**

3.5.2 Combined Axial Tension and Bending

Unity Equation	**=**	**0.000**	**OK**

3.3.5 Combined Bending and Shear

Unity Equation	**=**	**0.000**	**OK**

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AS/NZS 4600 - COLD-FORMED STEEL STRUCTURES INPUTS **JOB NUMBER:** 16081149

CLIENT: Forbes and Davies **DATE:** 12/08/2016 **SHEET:** 1 OF 2

2.1 SECTION PROPERTIES — 120x50 Beam capacity

Young's Modulus	E =	200,000	MPa
Yield Strength	fy =	450	MPa
Ultimate Tensile Strength	fu =	490	MPa
Compression Flange Stress	f* =	450	MPa
Poisson's Ratio	v =	0.3	

Stress at compression flange for given scenario - using fy is a conservative approach. For more accuracy, this value should be updated when determining the critical stress values.

Full Section Modulus (X)	Zfx =	23,288	mm^3
Full Section Modulus (Y)	Zfy =	10,547	mm^3
Full Cross-Sectional Area	Af =	642	mm^2
Full Second Moment of Area (X)	Ix =	1,397,284	mm^4
Full Second Moment of Area (Y)	Iy =	263,680	mm^4
Torsional Constant	J =	686,587	mm^4
Warping Constant	Iw =	190,000,000	mm^6
Shear Modulus of Elasticity	G =	80,000	MPa

Effective Section Modulus (X)	Zex =	23,288	mm^3
Effective Section Modulus (Y)	Zey =	10,547	mm^3
Effective Cross-Sectional Area	Ae =	642	mm^2
Radius of Gyration (x)	rx =	46.7	mm
Radius of Gyration (y)	ry =	20.3	mm
Shear Centre X-Coordinate	x0 =	0.0	mm
Shear Centre Y-Coordinate	y0 =	0.0	mm
Polar Radius of Gyration	r01 =	50.9	mm

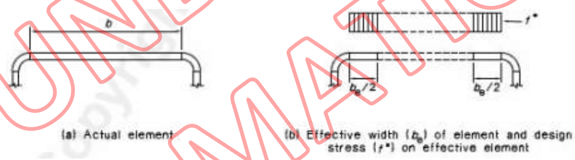
2.2 EFFECTIVE WIDTHS OF STIFFENED ELEMENTS

This section shall be used to determine the effective section properties if not known. Using this in parallel with Spaceclaim, the effective section properties can be determined. The purpose of this section is to calculate the reduced compression element area. Refer to section 2.2 of AS/NZS 4600 for more information.

Relevant

2.2.1 Uniformly Compressed Stiffened Elements

Thickness of Stiffened Element	t =	3.00	mm
Width of Flat Element	b =	50.0	mm
Plate Elastic Buckling Stress	fcr =	2603	MPa
Plate Buckling Coefficient	k =	4.00	
Slenderness Ratio	lamba =	0.416	
Effective Width Factor	p =	1.000	
Effective Width	**be =**	**50.0**	**mm**

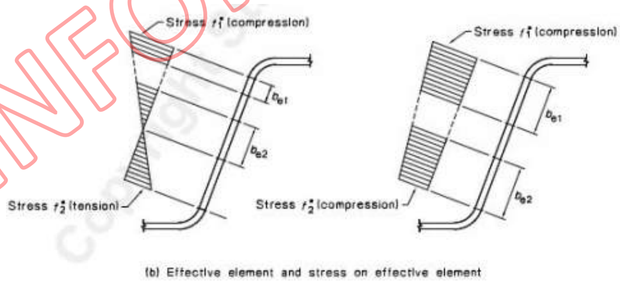


Effective width of element is essentially as if the area in the centre of the element has been cut-away. Use Spaceclaim to determine the effective section properties

Relevant

2.2.3 Stiffened Elements With Stress Gradient

Thickness of Stiffened Element	t =	1.5	mm
Width of Flat Element	b =	120.0	mm
Compression Web Stress	f1* =	350	MPa
Compression/Tension Web Stres	f2* =	-350	MPa
Web Stress Ratio	Psi =	-1.000	
Plate Buckling Coefficient	k =	24.00	
Plate Elastic Buckling Stress	fcr =	678	MPa
Slenderness Ratio	lamba =	0.719	
Effective Width Factor	p =	0.97	
Effective Width	be =	115.9	mm
Effective Width 1	**be1 =**	**29.0**	**mm**
Effective Width 2	**be2 =**	**57.9**	**mm**

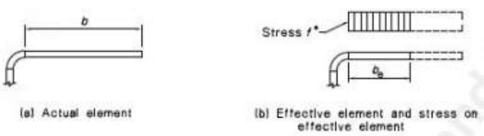


Compressive stress is taken to be +ve and tension stress taken to be -ve

Note be1 + be2 shall not exceed the compression portion of the we

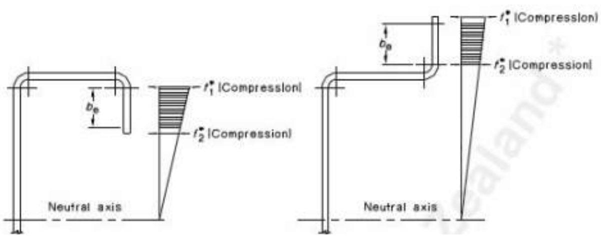
2.3.1 Uniformly Compressed Unstiffened Elements

Thickness of Unstiffened Elemen	t =	0	mm
Width of Flat Element	b =	0.0	mm
Plate Buckling Coefficient	k =	0.43	
Plate Elastic Buckling Stress	fcr =	#DIV/0!	MPa
Slenderness Ratio	lamba =	#DIV/0!	
Effective Width Factor	p =	#DIV/0!	
Effective Width	**be =**	**#DIV/0!**	**mm**



2.3.2 Unstiffened Elements and Edge Stiffeners With Stress Gradient

Thickness of Unstiffened Elemen	t =	0	mm
Width of Flat Element	b =	0	mm
Compression Web Stress	f1* =	0	MPa
Compression/Tension Web Stres	f2* =	0	MPa
Web Stress Ratio	Psi =	#DIV/0!	



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Type of Stress Gradient		Compression-Tension Fig 2.3.2(B)(i)	
Plate Buckling Coefficient	k =	#DIV/0!	
Plate Elastic Buckling Stress	fcr =	#DIV/0!	MPa
Slenderness Ratio	lamba =	#DIV/0!	
Effective Width Factor	p =	#DIV/0!	
Effective Width	**be =**	**#DIV/0!**	**mm**

Figure 2.3.2(A)(i) *Figure 2.3.2(A)(ii)*

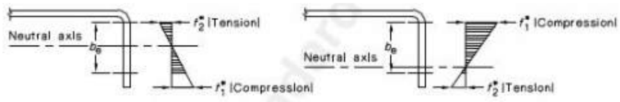
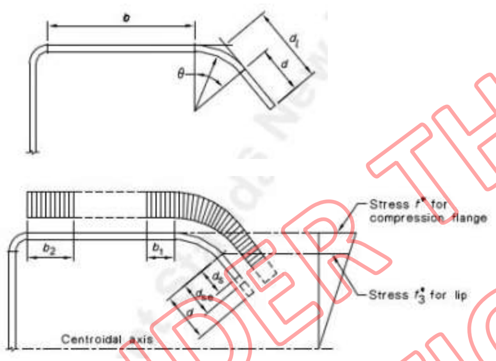


Figure 2.3.2(B)(i) *Figure 2.3.2(B)(ii)*

2.4.2 Effective Width of Uniformly Compressed Elements With an Edge Stiffene

Thickness of Stiffened Element	t =	0	mm
Width of Flat Element	b =	0	mm
Width of Stiffened Element	dl =	0	mm
Width of Flat Stiffened Element	d =	0	mm
Angle of Edge Stiffener	theta =	0	degrees
Compression Flange Stress	f* =	0	MPa
Slenderness Factor	S =	#DIV/0!	
Exponent Factor	n =	#DIV/0!	
Second Moment Area Stiffener	Is =	#DIV/0!	mm^4
Adequate Second Moment Area	Ia =	#DIV/0!	mm^4
Plate Buckling Coefficient	k =	#DIV/0!	
Plate Elastic Buckling Stress	fcr =	#DIV/0!	MPa
Slenderness Factor	lamba =	#DIV/0!	
Effective Width Factor	p =	#DIV/0!	
Effective Width	be =	#DIV/0!	mm
Effective Width of Stiffener	dse =	4.0	mm
Reduced Effective Width	**b1 =**	**#DIV/0!**	**mm**
Reduced Effective Width	**b2 =**	**#DIV/0!**	**mm**
Reduced Effective Stiffener	**ds =**	**#DIV/0!**	**mm**

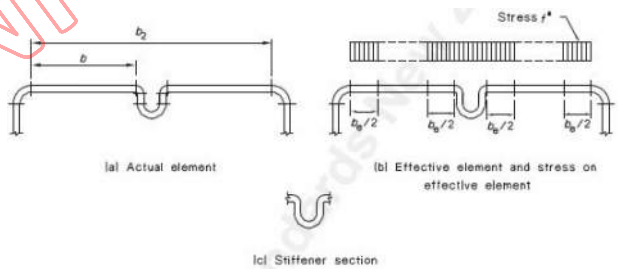


Calculate in accordance with 2.3.2 above

2.5.2 Effective Width of Uniformly Compressed Stiffened Elements With One Intermediate Stiffener

Thickness of Stiffened Element	t =	0.75	mm
Width of Element	b2 =	88	mm
Width of Flat Element	b =	36	mm
Stiffener Second Moment of Area	Is =	800	mm^4
Compression Flange Stress	f* =	14	MPa
Slenderness Factor	S =	153.0	
Adequate Second Moment Area	Ia =	0.0	mm^4
Exponent	n =	0.519	
Plate Buckling Coefficient	k =	4.00	
Plate Elastic Buckling Stress	fcr =	314	MPa
Slenderness Ratio	lamba =	0.211	
Effective Width Factor	p =	-0.20	
Effective Width	**be =**	**36.0**	**mm**

Second moment of area of the full stiffener about its own centroidal axis parallel to the element



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CALCULATED ACTIONS

Bending Moment Action (X)	Mx* =	0	Nmm
Bending Moment Action (Y)	My* =	0	Nmm
Axial Compression	Nc* =	0	N
Axial Tension	Nt* =	0	N
Shear	Vv* =	0	N

3.2 MEMBERS SUBJECT TO AXIAL TENSION

Capacity Reduction Factor	phi_t =	0.90		*Table 1.6 - Tension capacity reduction factor*
Correction Factor	k_t =	1.00		*Table 3.2 - Correction factor for shaded element*
Area of Penetrations or Holes	Ar =	0.0	mm^2	*Reduction of cross-sectional area from penetrations or holes*
Nominal Section Capacity	Nt =	267,393	N	
Axial Tensile Force Limit	**phi_t*Nt =**	**240,654**	**N**	**OK**

3.3 MEMBERS SUBJECT TO BENDING

3.3.2 Nominal Section Moment Capacity

Capacity Reduction Factor	phi_t =	0.90		*Table 1.6 - Bending capacity reduction factor*
Section Moment Capacity (X)	Msx =	10,479,600	Nmm	*Based on initiation of yielding*
Section Moment Capacity (Y)	Msy =	4,746,150	Nmm	*Based on initiation of yielding*
Section Moment Limit (X)	**phi_t*Msx =**	**9,431,640**	**Nmm**	**OK**
Section Moment Limit (Y)	**phi_t*Msy =**	**4,271,535**	**Nmm**	**OK**

3.3.3.2 Members Subject to Lateral Buckling

Effective Buckling Length	ley =	2,743	mm	
Effective Section Modulus (X)	Zex =	23,288	mm^3	*Calculated at the critical stress of the member - using fy as critical stress is conservative*
Elastic Buckling Moment	Mo =	61,669,072	Nmm	
Moment Distribution Coefficient	Cb =	1.0		*Unity is a conservative approach - refer to equation 3.3.3.2(9) for more accuracy*
Flexural Elastic Buckling Stress	foy =	108	MPa	
Torsional Elastic Buckling Stress	foz =	33099	MPa	
Initial Yield Moment	My =	10,479,600	Nmm	
Slenderness Ratio	lamba_b =	0.412		
Critical Moment	Mc =	10,479,600	Nmm	
Critical Stress	fc =	450	MPa	*Critical stress value used to determine effective section modulus for member moment capacity*
Member Moment Capacity (X)	Mbx =	10,479,600	Nmm	
Member Moment Limit (X)	**phi_t*Mbx =**	**9,431,640**	**Nmm**	**OK**

3.3.3.3 Members Subject to Distortional Buckling

Distortional buckling is local failure of the section - ie if the web was to crumple, or the flange were to rotate/distort under compression. The elastic distortional buckling stress is often too complex to calculate by hand, so ANSYS is recommended to be used to determine this stress. Appendix D gives formulas for specific sections.

Elastic Distortional Buckling Stress	fod =	1000	MPa	*Determined from rational elastic buckling analysis - usually ANSYS is best approach here*
Elastic Buckling Moment (X)	Modx =	23,288,000	Nmm	
Initial Yield Moment	My =	10,479,600	Nmm	
Slenderness Ratio	lamba_d =	0.67	Nmm	
Mode of Distortional Buckling		(a) Flange Rotation		*Refer to section 3.3.3.3 for more details*
Critical Moment (X)	Mcx =	10,479,600	Nmm	
Member Moment Limit (X)	**phi_t*Mcx =**	**9,431,640**	**Nmm**	**OK**

3.3.4 Shear Capacity of Webs

Capacity Reduction Factor	phi_v =	0.90		*Table 1.6 - Shear capacity reduction factor*
Thickness of Web	tw =	1.5	mm	
Depth of Flat Portion of Web	d1 =	120	mm	

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Depth of Web Hole	dwh =	0	mm	*Web holes shall comply with the requirements setout in 3.3.4.2*
Web Hole Reduction Multiplier	qs =	1.000		
Shear Buckling Coefficient	kv =	5.34		*For unstiffened webs - refer to 3.3.4.1 for webs with stiffeners*
Nominal Shear Capacity	Vv =	27,184	N	
Nominal Shear Capacity Web Hole	Vvwh =	27,184	N	
Web Shear Limit	**phi_v*Vv =**	**24,466**	**N**	**OK**

3.4 CONCENTRICALLY LOADED COMPRESSION MEMBERS

3.4.2 Sections Not Subject to Torsional or Flexural-Torsional Buckling

Capacity Reduction Factor	phi_c =	0.85		*Table 1.6 - Concentrically loaded compression member capacity reduction factor*
Effective Buckling Length	le =	2,743	mm	
Effective Area	Ae =	642	mm^2	*Calculated at the critical stress of the member - using fy as critical stress is conservative*
Elastic Flexural Buckling Stress	foc =	108	MPa	
Slenderness Ratio	lamba_c =	2.04		
Critical Stress	fn =	94	MPa	*Critical stress value used to determine effective area of section under axial compression*
Nominal Member Capacity	Nc =	60,667	N	
Axial Compression Limit	**phi_c*Nc =**	**51,567**	**N**	**OK**

3.4.3 Doubly- or Singly-Symmetric Sections Subject to Torsional or Flexural-Torsional Buckling

Flexural Elastic Buckling Stress	fox =	571	MPa	
Torsional Elastic Buckling Stress	foz =	33099	MPa	
Beta Factor	beta =	1.000		
Elastic Flexural Buckling Stress	foxz =	571	MPa	
Slenderness Ratio	lamba_c =	2.04		
Critical Stress	fn =	94	MPa	*Critical stress value used to determine effective area of section under axial compression*
Nominal Member Capacity	Nc =	60,667	N	
Axial Compression Limit	**phi_c*Nc =**	**51,567**	**N**	**OK**

3.4.6 Singly-Symmetric Section Subject to Distortional Buckling

Nominal Member Capacity	Nc =	256,399	N	*Note that this must be the lesser value of Nc calculated in accordance with 3.4.2 and 3.4.6*
Axial Compression Limit	**phi_c*Nc =**	**51,567**	**N**	**OK**

3.5 COMBINED AXIAL COMPRESSION OR TENSION, AND BENDING

3.5.1 Combined Axial Compression and Bending

Axial Compression Ratio	Nc*/phi_c*Nc =	0.000	
Unequal Moment Coefficient (X)	Cmx =	1.00	*Refer to section 3.5.1*
Unequal Moment Coefficient (Y)	Cmy =	1.00	*Refer to section 3.5.1*
Moment Amplification Factor (X)	alpha_nx =	1.00	
Moment Amplification Factor (Y)	alpha_ny =	1.00	
Unity Equation	**=**	**0.000**	**OK**

3.5.2 Combined Axial Tension and Bending

Unity Equation	**=**	**0.000**	**OK**

3.3.5 Combined Bending and Shear

Unity Equation	**=**	**0.000**	**OK**

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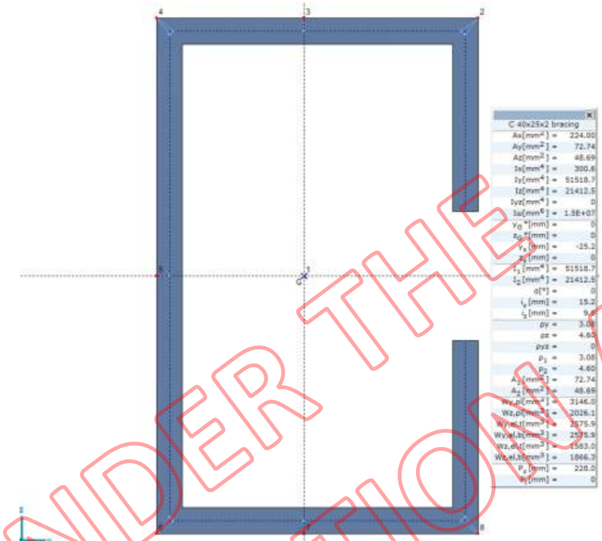
2.1 SECTION PROPERTIES

C40 Brace

Young's Modulus	E =	200,000	MPa
Yield Strength	fy =	350	MPa
Ultimate Tensile Strength	fu =	410	MPa
Compression Flange Stress	f* =	350	MPa
Poisson's Ratio	v =	0.3	
Full Section Modulus (X)	Zfx =	2,575	mm^3
Full Section Modulus (Y)	Zfy =	1,583	mm^3
Full Cross-Sectional Area	Af =	224	mm^2
Full Second Moment of Area (X)	Ix =	51,518	mm^4
Full Second Moment of Area (Y)	Iy =	21,412	mm^4
Torsional Constant	J =	300	mm^4
Warping Constant	Iw =	15,000,000	mm^6
Shear Modulus of Elasticity	G =	80,000	MPa



Stress at compression flange for given scenario - using fy is a conservative approach. For more accuracy, this value should be updated when determining the critical stress values.

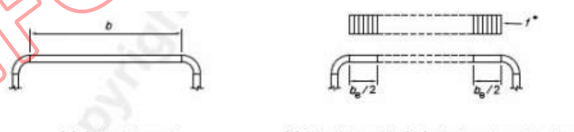
Effective Section Modulus (X)	Zex =	2,575	mm^3
Effective Section Modulus (Y)	Zey =	1,583	mm^3
Effective Cross-Sectional Area	Ae =	224	mm^2
Radius of Gyration (x)	rx =	15.2	mm
Radius of Gyration (y)	ry =	9.8	mm
Shear Centre X-Coordinate	x0 =	-25.2	mm
Shear Centre Y-Coordinate	y0 =	0.0	mm
Polar Radius of Gyration	r01 =	31.0	mm

2.2 EFFECTIVE WIDTHS OF STIFFENED ELEMENTS

This section shall be used to determine the effective section properties if not known. Using this in parallel with Spaceclaim, the effective section properties can be determined. The purpose of this section is to calculate the reduced compression element area. Refer to section 2.2 of AS/NZS 4600 for more information.

2.2.1 Uniformly Compressed Stiffened Elements

Thickness of Stiffened Element	t =	2.00	mm
Width of Flat Element	b =	40.0	mm
Plate Elastic Buckling Stress	fcr =	1808	MPa
Plate Buckling Coefficient	k =	4.00	
Slenderness Ratio	lamba =	0.440	
Effective Width Factor	p =	1.000	
Effective Width	**be =**	**40.0**	**mm**



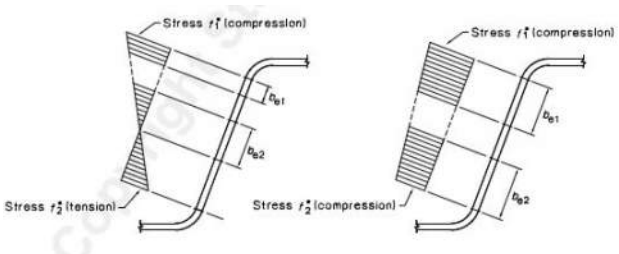
(a) Actual element

(b) Effective width (b_e) of element and design stress (f^*) on effective element

Effective width of element is essentially as if the area in the centre of the element has been cut-away. Use Spaceclaim to determine the effective section properties

2.2.3 Stiffened Elements With Stress Gradient

Thickness of Stiffened Element	t =	0	mm
Width of Flat Element	b =	0	mm
Compression Web Stress	f1* =	0	MPa
Compression/Tension Web Stres	f2* =	0	MPa
Web Stress Ratio	Psi =	#DIV/0!	
Plate Buckling Cofficient	k =	#DIV/0!	
Plate Elastic Buckling Stress	fcr =	#DIV/0!	MPa
Slenderness Ratio	lamba =	#DIV/0!	
Effective Width Factor	p =	#DIV/0!	
Effective Width	be =	#DIV/0!	mm
Effective Width 1	**be1 =**	**#DIV/0!**	**mm**
Effective Width 2	**be2 =**	**#DIV/0!**	**mm**



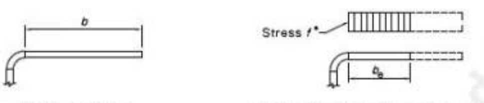
(b) Effective element and stress on effective element

Compressive stress is taken to be +ve and tension stress taken to be -ve

Note be1 + be2 shall not exceed the compression portion of the we

2.3.1 Uniformly Compressed Unstiffened Elements

Thickness of Unstiffened Elemen	t =	2	mm
Width of Flat Element	b =	15	mm
Plate Buckling Coefficient	k =	0.43	
Plate Elastic Buckling Stress	fcr =	1382	MPa
Slenderness Ratio	lamba =	0.503	



(a) Actual element

(b) Effective element and stress on effective element

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Effective Width Factor	p =	1.000	
Effective Width	**be =**	**15.0**	**mm**

2.3.2 Unstiffened Elements and Edge Stiffeners With Stress Gradient

Thickness of Unstiffened Elemen	t =	2	mm
Width of Flat Element	b =	15	mm
Compression Web Stress	f1* =	250	MPa
Compression/Tension Web Stres	f2* =	130	MPa
Web Stress Ratio	Psi =	0.52	
Type of Stress Gradient	Compression Decrease Fig 2.3.2(A)(i)		
Plate Buckling Coefficient	k =	0.672	
Plate Elastic Buckling Stress	fcr =	2160	MPa
Slenderness Ratio	lambda =	0.340	
Effective Width Factor	p =	1.00	
Effective Width	**be =**	**15.0**	**mm**

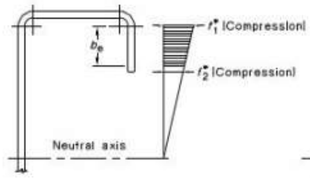


Figure 2.3.2(A)(i)

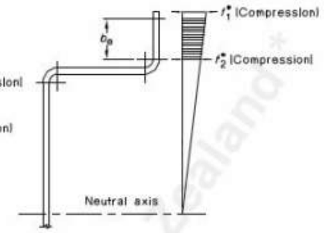


Figure 2.3.2(A)(ii)

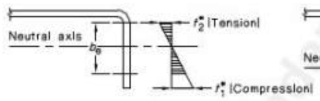
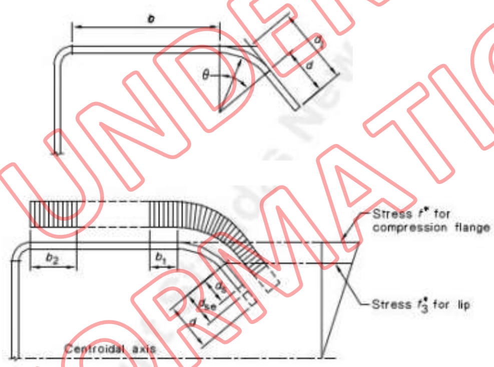


Figure 2.3.2(B)(i)

Figure 2.3.2(B)(ii)

2.4.2 Effective Width of Uniformly Compressed Elements With an Edge Stiffene

Thickness of Stiffened Element	t =	0	mm
Width of Flat Element	b =	0	mm
Width of Stiffened Element	dl =	0	mm
Width of Flat Stiffened Element	d =	0	mm
Angle of Edge Stiffener	theta =	0	degrees
Compression Flange Stress	f* =	0	MPa
Slenderness Factor	S =	#DIV/0!	
Exponent Factor	n =	#DIV/0!	
Second Moment Area Stiffener	Is =	#DIV/0!	mm^4
Adequate Second Moment Area	Ia =	#DIV/0!	mm^4
Plate Buckling Coefficient	k =	#DIV/0!	
Plate Elastic Buckling Stress	fcr =	#DIV/0!	MPa
Slenderness Factor	lambda =	#DIV/0!	
Effective Width Factor	p =	#DIV/0!	
Effective Width	be =	#DIV/0!	mm
Effective Width of Stiffener	dse =	32.0	mm
Reduced Effective Width	**b1 =**	**#DIV/0!**	**mm**
Reduced Effective Width	**b2 =**	**#DIV/0!**	**mm**
Reduced Effective Stiffener	**ds =**	**#DIV/0!**	**mm**

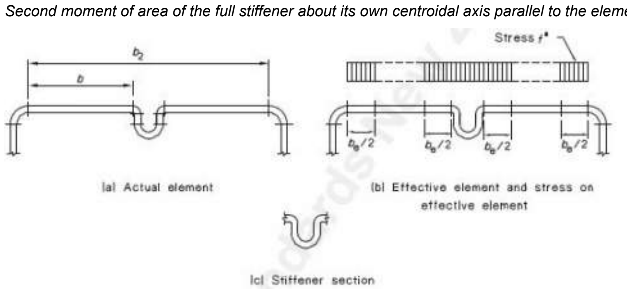


Calculate in accordance with 2.3.2 above

2.5.2 Effective Width of Uniformly Compressed Stiffened Elements With One Intermediate Stiffener

Thickness of Stiffened Element	t =	0	mm
Width of Element	b2 =	0	mm
Width of Flat Element	b =	0	mm
Stiffener Second Moment of Area	Is =	0	mm^4
Compression Flange Stress	f* =	0	MPa
Slenderness Factor	S =	#DIV/0!	
Adequate Second Moment Area	Ia =	#DIV/0!	mm^4
Exponent	n =	#DIV/0!	
Plate Buckling Coefficient	k =	#DIV/0!	
Plate Elastic Buckling Stress	fcr =	#DIV/0!	MPa
Slenderness Ratio	lambda =	#DIV/0!	
Effective Width Factor	p =	#DIV/0!	
Effective Width	**be =**	**#DIV/0!**	**mm**

Second moment of area of the full stiffener about its own centroidal axis parallel to the element



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CALCULATED ACTIONS

Bending Moment Action (X)	Mx* =	0	Nmm
Bending Moment Action (Y)	My* =	0	Nmm
Axial Compression	Nc* =	11,866	N
Axial Tension	Nt* =	9,761	N
Shear	Vv* =	0	N

3.2 MEMBERS SUBJECT TO AXIAL TENSION

Capacity Reduction Factor	phi_t =	0.90		*Table 1.6 - Tension capacity reduction factor*
Correction Factor	k_t =	0.75		*Table 3.2 - Correction factor for shaded element*
Area of Penetrations or Holes	Ar =	78.0	mm^2	*Reduction of cross-sectional area from penetrations or holes*
Nominal Section Capacity	Nt =	38,161	N	
Axial Tensile Force Limit	**phi_t*Nt =**	**34,345**	**N**	**OK**

3.3 MEMBERS SUBJECT TO BENDING

3.3.2 Nominal Section Moment Capacity

Capacity Reduction Factor	phi_t =	0.90		*Table 1.6 - Bending capacity reduction factor*
Section Moment Capacity (X)	Msx =	901,250	Nmm	*Based on initiation of yielding*
Section Moment Capacity (Y)	Msy =	554,050	Nmm	*Based on initiation of yielding*
Section Moment Limit (X)	**phi_t*Msx =**	**811,125**	**Nmm**	**OK**
Section Moment Limit (Y)	**phi_t*Msy =**	**498,645**	**Nmm**	**OK**

3.3.3.2 Members Subject to Lateral Buckling

Effective Buckling Length	ley =	960	mm	
Effective Section Modulus (X)	Zex =	2,575	mm^3	*Calculated at the critical stress of the member - using fy as critical stress is conservative*
Elastic Buckling Moment	Mo =	1,604,392	Nmm	
Moment Distribution Coefficient	Cb =	1.0		*Unity is a conservative approach - refer to equation 3.3.3.2(9) for more accuracy*
Flexural Elastic Buckling Stress	foy =	205	MPa	
Torsional Elastic Buckling Stress	foz =	261	MPa	
Initial Yield Moment	My =	901,250	Nmm	
Slenderness Ratio	lamba_b =	0.749		
Critical Moment	Mc =	844,288	Nmm	
Critical Stress	fc =	328	MPa	*Critical stress value used to determine effective section modulus for member moment capacity*
Member Moment Capacity (X)	Mbx =	844,288	Nmm	
Member Moment Limit (X)	**phi_t*Mbx =**	**759,860**	**Nmm**	**OK**

3.3.3.3 Members Subject to Distortional Buckling

Distortional buckling is local failure of the section - ie if the web was to crumple, or the flange were to rotate/distort under compression. The elastic distortional buckling stress is often too complex to calculate by hand, so ANSYS is recommended to be used to determine this stress. Appendix D gives formulas for specific sections.

Elastic Distortional Buckling Stress	fod =	1000	MPa	*Determined from rational elastic buckling analysis - usually ANSYS is best approach here*
Elastic Buckling Moment (X)	Modx =	2,575,000	Nmm	
Initial Yield Moment	My =	901,250	Nmm	
Slenderness Ratio	lamba_d =	0.59	Nmm	
Mode of Distortional Buckling		(a) Flange Rotation		*Refer to section 3.3.3.3 for more details*
Critical Moment (X)	Mcx =	901,250	Nmm	
Member Moment Limit (X)	**phi_t*Mcx =**	**811,125**	**Nmm**	**OK**

3.3.4 Shear Capacity of Webs

Capacity Reduction Factor	phi_v =	0.90		*Table 1.6 - Shear capacity reduction factor*
Thickness of Web	tw =	2.0	mm	
Depth of Flat Portion of Web	d1 =	40	mm	

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Depth of Web Hole	dwh =	0	mm	*Web holes shall comply with the requirements setout in 3.3.4.2*
Web Hole Reduction Multiplier	qs =	0.185		
Shear Buckling Coefficient	kv =	5.34		*For unstiffened webs - refer to 3.3.4.1 for webs with stiffeners*
Nominal Shear Capacity	Vv =	17,920	N	
Nominal Shear Capacity Web Hole	Vvwh =	3,319	N	
Web Shear Limit	**phi_v*Vv =**	**2,987**	**N**	**OK**

3.4 CONCENTRICALLY LOADED COMPRESSION MEMBERS

3.4.2 Sections Not Subject to Torsional or Flexural-Torsional Buckling

Capacity Reduction Factor	phi_c =	0.85		*Table 1.6 - Concentrically loaded compression member capacity reduction factor*
Effective Buckling Length	le =	960	mm	
Effective Area	Ae =	224	mm^2	*Calculated at the critical stress of the member - using fy as critical stress is conservative*
Elastic Flexural Buckling Stress	foc =	205	MPa	
Slenderness Ratio	lamba_c =	1.31		
Critical Stress	fn =	171	MPa	*Critical stress value used to determine effective area of section under axial compression*
Nominal Member Capacity	Nc =	38,333	N	
Axial Compression Limit	**phi_c*Nc =**	**32,583**	**N**	**OK**

3.4.3 Doubly- or Singly-Symmetric Sections Subject to Torsional or Flexural-Torsional Buckling

Flexural Elastic Buckling Stress	fox =	493	MPa	
Torsional Elastic Buckling Stress	foz =	261	MPa	
Beta Factor	beta =	0.339		
Elastic Flexural Buckling Stress	foxz =	186	MPa	
Slenderness Ratio	lamba_c =	1.37		
Critical Stress	fn =	159	MPa	*Critical stress value used to determine effective area of section under axial compression*
Nominal Member Capacity	Nc =	35,685	N	
Axial Compression Limit	**phi_c*Nc =**	**30,333**	**N**	**OK**

3.4.6 Singly-Symmetric Section Subject to Distortional Buckling

Nominal Member Capacity	Nc =	71,540	N	*Note that this must be the lesser value of Nc calculated in accordance with 3.4.2 and 3.4.6*
Axial Compression Limit	**phi_c*Nc =**	**32,583**	**N**	**OK**

3.5 COMBINED AXIAL COMPRESSION OR TENSION, AND BENDING

3.5.1 Combined Axial Compression and Bending

Axial Compression Ratio	Nc*/phi_c*Nc =	0.391	
Unequal Moment Coefficient (X)	Cmx =	1.00	*Refer to section 3.5.1*
Unequal Moment Coefficient (Y)	Cmy =	1.00	*Refer to section 3.5.1*
Moment Amplification Factor (X)	alpha_nx =	0.89	
Moment Amplification Factor (Y)	alpha_ny =	0.74	
Unity Equation	**=**	**0.391**	**OK**

3.5.2 Combined Axial Tension and Bending

Unity Equation	**=**	**0.284**	**OK**

3.3.5 Combined Bending and Shear

Unity Equation	**=**	**0.000**	**OK**

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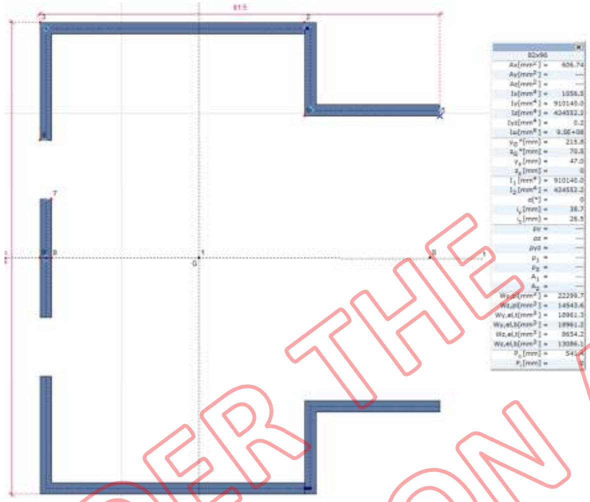
2.1 SECTION PROPERTIES — 96x82x2.3 Column

Young's Modulus	E =	200,000	MPa
Yield Strength	fy =	350	MPa
Ultimate Tensile Strength	fu =	430	MPa
Compression Flange Stress	f* =	160	MPa
Poisson's Ratio	v =	0.3	

Full Section Modulus (X)	Zfx =	19,064	mm^3
Full Section Modulus (Y)	Zfy =	8,964	mm^3
Full Cross-Sectional Area	Af =	661	mm^2
Full Second Moment of Area (X)	Ix =	915,082	mm^4
Full Second Moment of Area (Y)	Iy =	465,571	mm^4
Torsional Constant	J =	1,160	mm^4
Warping Constant	Iw =	1,400,000,000	mm^6
Shear Modulus of Elasticity	G =	80,000	MPa



Stress at compression flange for given scenario - using fy is a conservative approach. For more accuracy, this value should be updated when determining the critical stress values.

Effective Section Modulus (X)	Zex =	18,961	mm^3
Effective Section Modulus (Y)	Zey =	8,654	mm^3
Effective Cross-Sectional Area	Ae =	606	mm^2
Radius of Gyration (x)	rx =	37.2	mm
Radius of Gyration (y)	ry =	26.5	mm
Shear Centre X-Coordinate	x0 =	47.0	mm
Shear Centre Y-Coordinate	y0 =	0.0	mm
Polar Radius of Gyration	r01 =	65.6	mm

2.2 EFFECTIVE WIDTHS OF STIFFENED ELEMENTS

This section shall be used to determine the effective section properties if not known. Using this in parallel with Spaceclaim, the effective section properties can be determined. The purpose of this section is to calculate the reduced compression element area. Refer to section 2.2 of AS/NZS 4600 for more information.

2.2.1 Uniformly Compressed Stiffened Elements

Thickness of Stiffened Element	t =	2.30	mm
Width of Flat Element	b =	82.0	mm
Plate Elastic Buckling Stress	fcr =	569	MPa
Plate Buckling Coefficient	k =	4.00	
Slenderness Ratio	lamba =	0.530	
Effective Width Factor	p =	1.000	
Effective Width	**be =**	**82.0**	**mm**

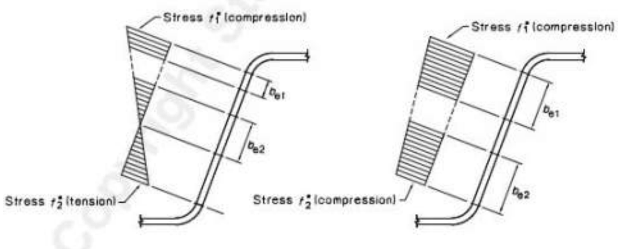
(a) Actual element

(b) Effective width (b_e) of element and design stress (f^*) on effective element

Effective width of element is essentially as if the area in the centre of the element has been cut-away. Use Spaceclaim to determine the effective section properties

2.2.3 Stiffened Elements With Stress Gradient

Thickness of Stiffened Element	t =	2.3	mm
Width of Flat Element	b =	49.8	mm
Compression Web Stress	f1* =	143	MPa
Compression/Tension Web Stres	f2* =	205	MPa
Web Stress Ratio	Psi =	1.434	
Plate Buckling Cofficient	k =	2.97	
Plate Elastic Buckling Stress	fcr =	1145	MPa
Slenderness Ratio	lamba =	0.353	
Effective Width Factor	p =	1.00	
Effective Width	be =	49.8	mm
Effective Width 1	**be1 =**	**31.8**	**mm**
Effective Width 2	**be2 =**	**18.0**	**mm**



(b) Effective element and stress on effective element

Compressive stress is taken to be +ve and tension stress taken to be -ve

Note be1 + be2 shall not exceed the compression portion of the we

2.3.1 Uniformly Compressed Unstiffened Elements

Thickness of Unstiffened Elemen	t =	0	mm

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Width of Flat Element	b =	0	mm
Plate Buckling Coefficient	k =	0.43	
Plate Elastic Buckling Stress	fcr =	#DIV/0!	MPa
Slenderness Ratio	lamba =	#DIV/0!	
Effective Width Factor	p =	#DIV/0!	
Effective Width	**be =**	**#DIV/0!**	**mm**

2.3.2 Unstiffened Elements and Edge Stiffeners With Stress Gradient

Thickness of Unstiffened Elemen	t =	2.3	mm
Width of Flat Element	b =	19	mm
Compression Web Stress	f1* =	205	MPa
Compression/Tension Web Stres	f2* =	143	MPa
Web Stress Ratio	Psi =	0.70	
Type of Stress Gradient	Compression Increase Fig 2.3.2(A)(ii)		
Plate Buckling Coefficient	k =	0.458	
Plate Elastic Buckling Stress	fcr =	1212	MPa
Slenderness Ratio	lamba =	0.411	
Effective Width Factor	p =	1.00	
Effective Width	**be =**	**19.0**	**mm**

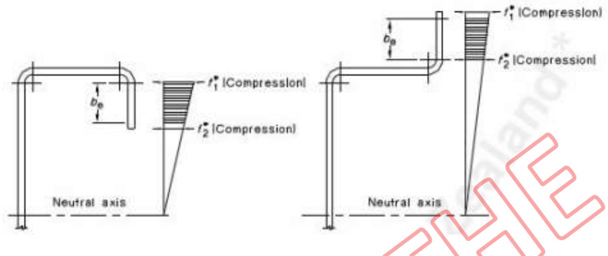


Figure 2.3.2(A)(i) *Figure 2.3.2(A)(ii)*

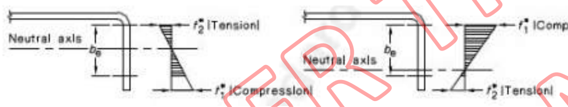
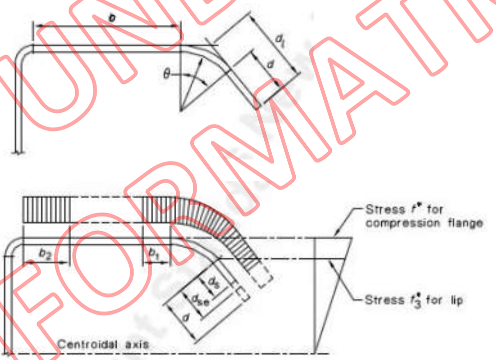


Figure 2.3.2(B)(i) *Figure 2.3.2(B)(ii)*

2.4.2 Effective Width of Uniformly Compressed Elements With an Edge Stiffene

Thickness of Stiffened Element	t =	0	mm
Width of Flat Element	b =	0	mm
Width of Stiffened Element	dl =	0	mm
Width of Flat Stiffened Element	d =	0	mm
Angle of Edge Stiffener	theta =	0	degrees
Compression Flange Stress	f* =	0	MPa
Slenderness Factor	S =	#DIV/0!	
Exponent Factor	n =	#DIV/0!	
Second Moment Area Stiffener	Is =	#DIV/0!	mm^4
Adequate Second Moment Area	Ia =	#DIV/0!	mm^4
Plate Buckling Coefficient	k =	#DIV/0!	
Plate Elastic Buckling Stress	fcr =	#DIV/0!	MPa
Slenderness Factor	lamba =	#DIV/0!	
Effective Width Factor	p =	#DIV/0!	
Effective Width	be =	#DIV/0!	mm
Effective Width of Stiffener	dse =	0.0	mm
Reduced Effective Width	**b1 =**	**#DIV/0!**	**mm**
Reduced Effective Width	**b2 =**	**#DIV/0!**	**mm**
Reduced Effective Stiffener	**ds =**	**#DIV/0!**	**mm**

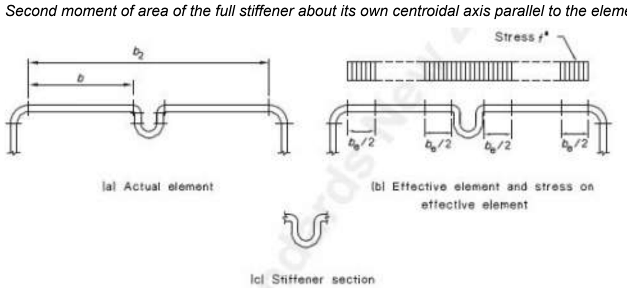


Calculate in accordance with 2.3.2 above

2.5.2 Effective Width of Uniformly Compressed Stiffened Elements With One Intermediate Stiffener

Thickness of Stiffened Element	t =	2.3	mm
Width of Element	b2 =	80	mm
Width of Flat Element	b =	37.5	mm
Stiffener Second Moment of Area	Is =	10	mm^4
Compression Flange Stress	f* =	250	MPa
Slenderness Factor	S =	36.2	
Adequate Second Moment Area	Ia =	0.0	mm^4
Exponent	n =	0.503	
Plate Buckling Coefficient	k =	4.00	
Plate Elastic Buckling Stress	fcr =	2720	MPa
Slenderness Ratio	lamba =	0.303	
Effective Width Factor	p =	0.90	
Effective Width	**be =**	**37.5**	**mm**

Second moment of area of the full stiffener about its own centroidal axis parallel to the element



AS/NZS 4600 96x81.5x2.3 Column Section capacity - SECTION 3 - MEMBERS

AS/NZS 4600 - COLD-FORMED STEEL STRUCTURES INPUTS **JOB NUMBER:** 16081149

BVT Engineering Professional Services

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CALCULATED ACTIONS

Bending Moment Action (X)	Mx* =	0	Nmm	
Bending Moment Action (Y)	My* =	0	Nmm	
Axial Compression	Nc* =	0	N	
Axial Tension	Nt* =	0	N	
Shear	Vv* =	0	N	

3.2 MEMBERS SUBJECT TO AXIAL TENSION

Capacity Reduction Factor	phi_t =	0.90		*Table 1.6 - Tension capacity reduction factor*
Correction Factor	k_t =	1.00		*Table 3.2 - Correction factor for shaded element*
Area of Penetrations or Holes	Ar =	0.0	mm^2	*Reduction of cross-sectional area from penetrations or holes*
Nominal Section Capacity	Nt =	231,350	N	
Axial Tensile Force Limit	**phi_t*Nt =**	**208,215**	**N**	**OK**

3.3 MEMBERS SUBJECT TO BENDING

3.3.2 Nominal Section Moment Capacity

Capacity Reduction Factor	phi_t =	0.90		*Table 1.6 - Bending capacity reduction factor*
Section Moment Capacity (X)	Msx =	6,636,350	Nmm	*Based on initiation of yielding*
Section Moment Capacity (Y)	Msy =	3,028,900	Nmm	*Based on initiation of yielding*
Section Moment Limit (X)	**phi_t*Msx =**	**5,972,715**	**Nmm**	**OK**
Section Moment Limit (Y)	**phi_t*Msy =**	**2,726,010**	**Nmm**	**OK**

3.3.3.2 Members Subject to Lateral Buckling

Effective Buckling Length	ley =	1,200	mm	
Effective Section Modulus (X)	Zex =	18,961	mm^3	*Calculated at the critical stress of the member - using fy as critical stress is conservative*
Elastic Buckling Moment	Mo =	35,832,629	Nmm	
Moment Distribution Coefficient	Cb =	1.0		*Unity is a conservative approach - refer to equation 3.3.3.2(9) for more accuracy*
Flexural Elastic Buckling Stress	foy =	965	MPa	
Torsional Elastic Buckling Stress	foz =	708	MPa	
Initial Yield Moment	My =	6,672,400	Nmm	
Slenderness Ratio	lamba_b =	0.432		
Critical Moment	Mc =	6,672,400	Nmm	
Critical Stress	fc =	350	MPa	*Critical stress value used to determine effective section modulus for member moment capacity*
Member Moment Capacity (X)	Mbx =	6,636,350	Nmm	
Member Moment Limit (X)	**phi_t*Mbx =**	**5,972,715**	**Nmm**	**OK**

3.3.3.3 Members Subject to Distortional Buckling

Distortional buckling is local failure of the section - ie if the web was to crumple, or the flange were to rotate/distort under compression. The elastic distortional buckling stress is often too complex to calculate by hand, so ANSYS is recommended to be used to determine this stress. Appendix D gives formulas for specific sections.

Elastic Distortional Buckling Stress	fod =	1000	MPa	*Determined from rational elastic buckling analysis - usually ANSYS is best approach here*
Elastic Buckling Moment (X)	Modx =	19,064,000	Nmm	
Initial Yield Moment	My =	6,672,400	Nmm	
Slenderness Ratio	lamba_d =	0.59	Nmm	
Mode of Distortional Buckling		(a) Flange Rotation		*Refer to section 3.3.3.3 for more details*
Critical Moment (X)	Mcx =	6,672,400	Nmm	
Member Moment Limit (X)	**phi_t*Mcx =**	**6,005,160**	**Nmm**	**OK**

3.3.4 Shear Capacity of Webs

Capacity Reduction Factor	phi_v =	0.90		*Table 1.6 - Shear capacity reduction factor*
Thickness of Web	tw =	2.3	mm	
Depth of Flat Portion of Web	d1 =	95	mm	

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Depth of Web Hole	dwh =	0	mm	*Web holes shall comply with the requirements setout in 3.3.4.2*
Web Hole Reduction Multiplier	qs =	0.382		
Shear Buckling Coefficient	kv =	5.34		*For unstiffened webs - refer to 3.3.4.1 for webs with stiffeners*
Nominal Shear Capacity	Vv =	48,944	N	
Nominal Shear Capacity Web Hole	Vvwh =	18,719	N	
Web Shear Limit	**phi_v*Vv =**	**16,847**	**N**	**OK**

3.4 CONCENTRICALLY LOADED COMPRESSION MEMBERS

3.4.2 Sections Not Subject to Torsional or Flexural-Torsional Buckling

Capacity Reduction Factor	phi_c =	0.90		*Table 1.6 - Concentrically loaded compression member capacity reduction factor*
Effective Buckling Length	le =	1,200	mm	
Effective Area	Ae =	606	mm^2	*Calculated at the critical stress of the member - using fy as critical stress is conservative*
Elastic Flexural Buckling Stress	foc =	965	MPa	
Slenderness Ratio	lamba_c =	0.60		
Critical Stress	fn =	301	MPa	*Critical stress value used to determine effective area of section under axial compression*
Nominal Member Capacity	Nc =	182,241	N	
Axial Compression Limit	**phi_c*Nc =**	**164,017**	**N**	**OK**

3.4.3 Doubly- or Singly-Symmetric Sections Subject to Torsional or Flexural-Torsional Buckling

Flexural Elastic Buckling Stress	fox =	1898	MPa	
Torsional Elastic Buckling Stress	foz =	708	MPa	
Beta Factor	beta =	0.486		
Elastic Flexural Buckling Stress	foxz =	578	MPa	
Slenderness Ratio	lamba_c =	0.78		
Critical Stress	fn =	272	MPa	*Critical stress value used to determine effective area of section under axial compression*
Nominal Member Capacity	Nc =	164,620	N	
Axial Compression Limit	**phi_c*Nc =**	**148,158**	**N**	**OK**

3.4.6 Singly-Symmetric Section Subject to Distortional Buckling

Nominal Member Capacity	Nc =	211,107	N	*Note that this must be the lesser value of Nc calculated in accordance with 3.4.2 and 3.4.6*
Axial Compression Limit	**phi_c*Nc =**	**164,017**	**N**	**OK**

3.5 COMBINED AXIAL COMPRESSION OR TENSION, AND BENDING

3.5.1 Combined Axial Compression and Bending

Axial Compression Ratio	Nc*/phi_c*Nc =	0.000	
Unequal Moment Coefficient (X)	Cmx =	1.00	*Refer to section 3.5.1*
Unequal Moment Coefficient (Y)	Cmy =	1.00	*Refer to section 3.5.1*
Moment Amplification Factor (X)	alpha_nx =	1.00	
Moment Amplification Factor (Y)	alpha_ny =	1.00	
Unity Equation	**=**	**0.00**	**OK**

3.5.2 Combined Axial Tension and Bending

Unity Equation	**=**	**0.000**	**OK**

3.3.5 Combined Bending and Shear

Unity Equation	**=**	**0.000**	**OK**