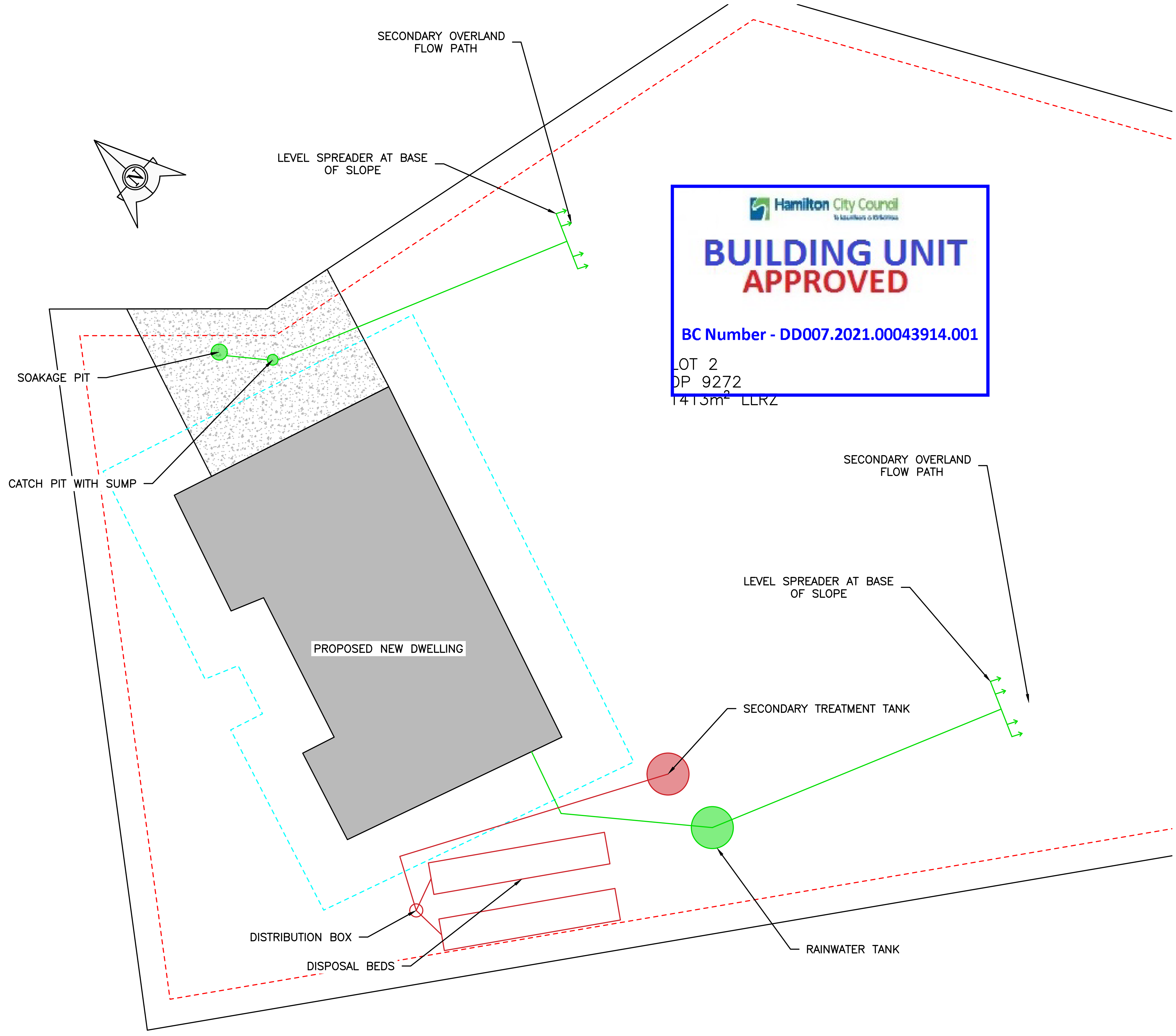

MINIMUM SEPARATION DISTANCES		
	PRIMARY	SECONDARY
BUILDINGS/HOUSES[1]	3m	1.5m TO 3m
PROPERTY/BOUNDARY[1]	1.5m	1.5m
SURFACE WATER[2]	10m	10m
WATER SUPPLY BORE[1]	20m	20m
GROUND WATER[2]	600mm	300mm
FLOOD PLAIN ARI[1]	100 YEAR	20 YEAR
EMBANKMENTS/ RETAINING WALLS[1]	3m FROM THE DRAINAGE MATERIAL.CUT BATTER INTERFACE OR 45° ANGLE FROM TOE OF WALL EXCAVATION (WHICH EVER IS GREATEST)	

NOTE 1: AS PER TP58 ON–SITE WASTEWATER SYSTEMS: DESIGN AND MANAGEMENT MANUAL

NOTE 2: AS PER WAIKATO REGIONAL PLAN

DISPOSAL FIELD CHARACTERISTICS	
SYSTEM	LPED BEDS
TREATMENT	SECONDARY
BASAL AREA	33m²
TOTAL AREA	42m²

NOTES:
1) PLAN IS INDICATIVE ONLY,
2) OFFSETS FOR SOAKAGE TRENCH SHALL BE ADHERED TO;
 2)1) 3.0m FROM BUILDINGS,
 2)2) 1.5m FROM PROPERTY BOUNDARIES,
 2)3) 0.5m FROM ROAD BOUNDARY
3) IF CONTRACTOR CANNOT ACHIEVE OFFSETS, ENGINEER SHALL BE NOTIFIED PRIOR TO CONSTRUCTION.
4) ALL RUNOFF TO ENTER A SUMP PRIOR TO SOAKAGE DEVICE.

NOTES:
1. WORKS TO COMPLY WITH BUILDING CODE, BUILD CONSENT CONDITIONS AND PRODUCT MANUFACTURERS SPECIFICATIONS
2. EARTHWORKS SHALL NOT CONCENTRATE SURFACE WATER NEAR DISPOSAL FIELD
3. FENCE DISPOSAL FIELD FROM STOCK IF REQUIRED
4. POSITION OF SEPTIC TANK MAY BE RELOCATED AT CLIENTS DISCRETION

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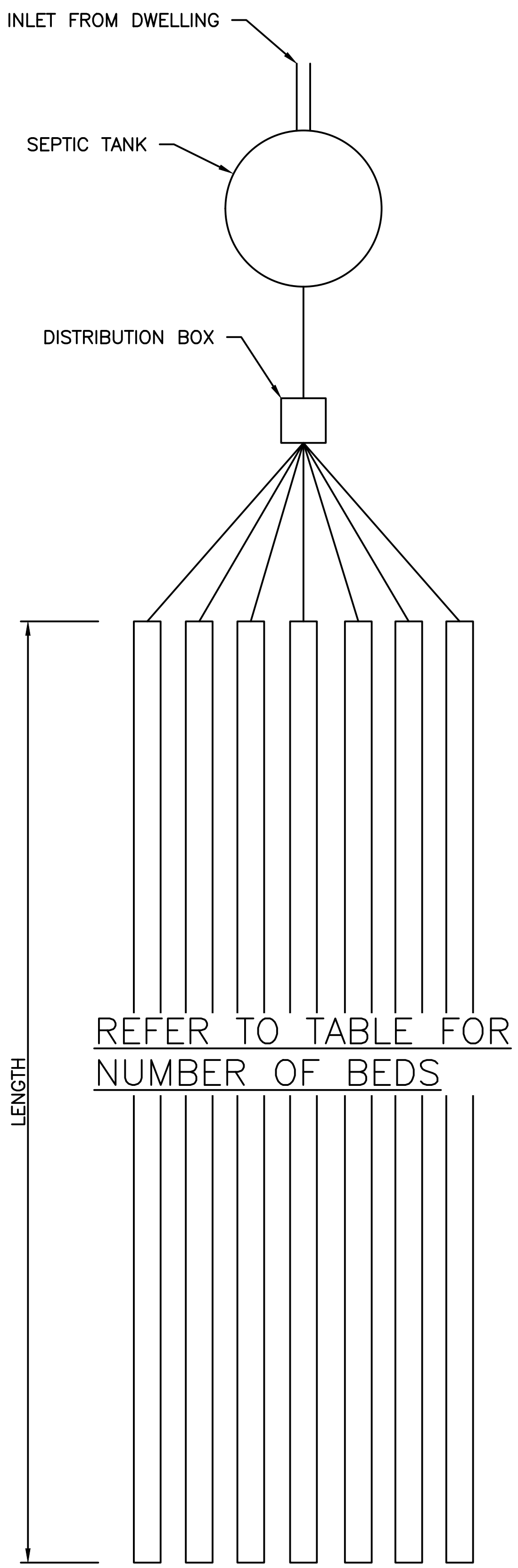
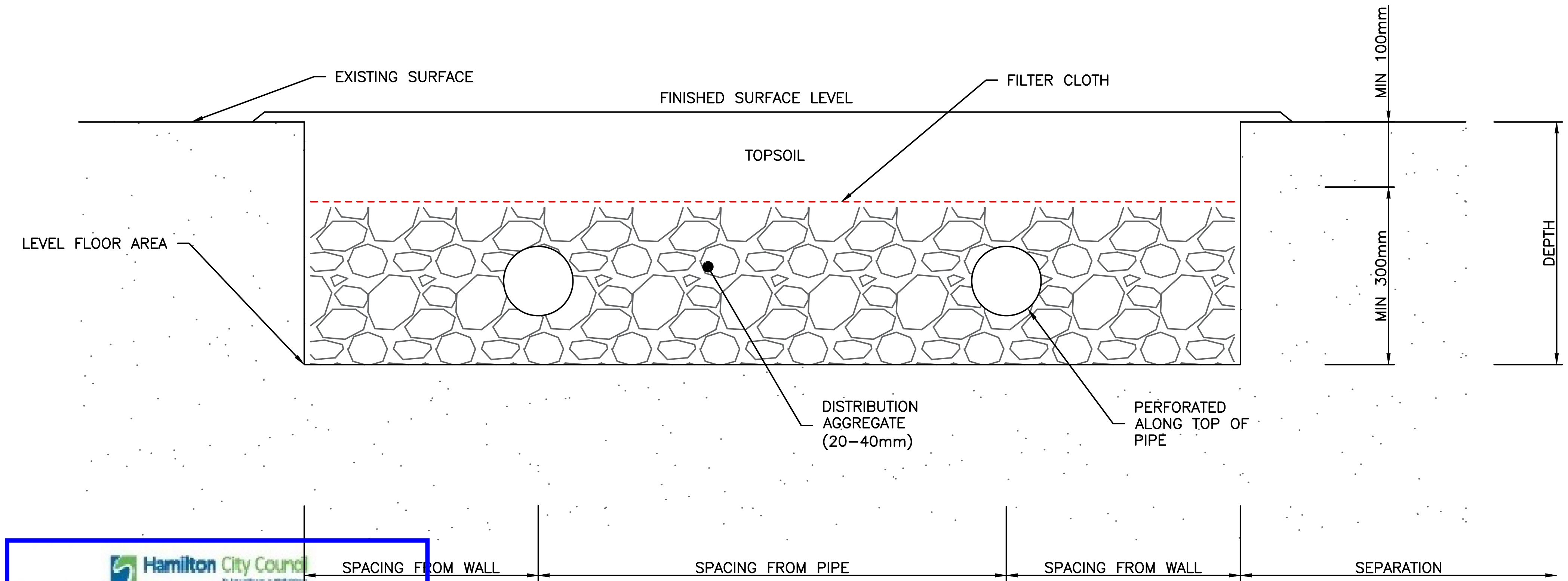
Tauranga Office: 250 Chadwick Road, Greerton, Tauranga, 3112
07 242 0017
www.tituscivil.co.nz
office@tituscivil.co.nz

DESIGNED	CHECKED	CLIENT	PROJECT	DRAWING	STATUS
RM	JV	MARTIN CAMERON	NEW DWELLING LOT 2 136 STATE HIGHWAY 26 HAMILTON	WASTEWATER PLAN	FOR APPROVAL FINAL AS APPROVED BY COUNCIL
DRAWN: RM	APPROVED: TR				DATE: 11/10/2021; SCALE (ORIGINAL SIZE A3): NTS
					DRAWING NUMBER: 11122–WW –0001; REVISION: B



PLAN VIEW



CROSS SECTION VIEW

Hamilton City Council
BUILDING UNIT
APPROVED
BC Number - DD007.2021.00043914.001

BED SPECIFICATIONS			
NUMBERS OF BEDS	2	SPACING FROM PIPE	0.6m
LENGTH	10m	BASE WIDTH	1.8m
DEPTH	MIN 0.4m MAX 0.6m	BASAL AREA	$33m^2$
SEPARATION	1.0m	TOTAL AREA	$42m^2$
SPACING FROM WALL	0.3m		

NOTES:
1. WORKS TO COMPLY WITH BUILDING, PLUMBING REGULATIONS AND MANUFACTURERS SPECIFICATIONS
2. EARTHWORKS SHALL NOT CONCENTRATE SURFACE WATER NEAR DISPOSAL FIELD
3. FENCE DISPOSAL FIELD FROM STOCK IF REQUIRED
4. THERE MUST BE AT LEAST 1 METER FROM SURFACE TO WINTER GROUND WATER MAXIMUM FOR TRENCHES/BEDS
5. THE TRENCHES/BEDS MUST BE LAID ON A LEVEL CONTOUR
6. TRENCH/BED BASES MUST BE LEVEL
7. A 100% RESERVE AREA IS REQUIRED FOR TRENCHES/BEDS AND A 50% RESERVE AREA FOR DRIP IRRIGATION

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DESIGNED	CHECKED
RM	JV
DRAWN	APPROVED
RM	TR

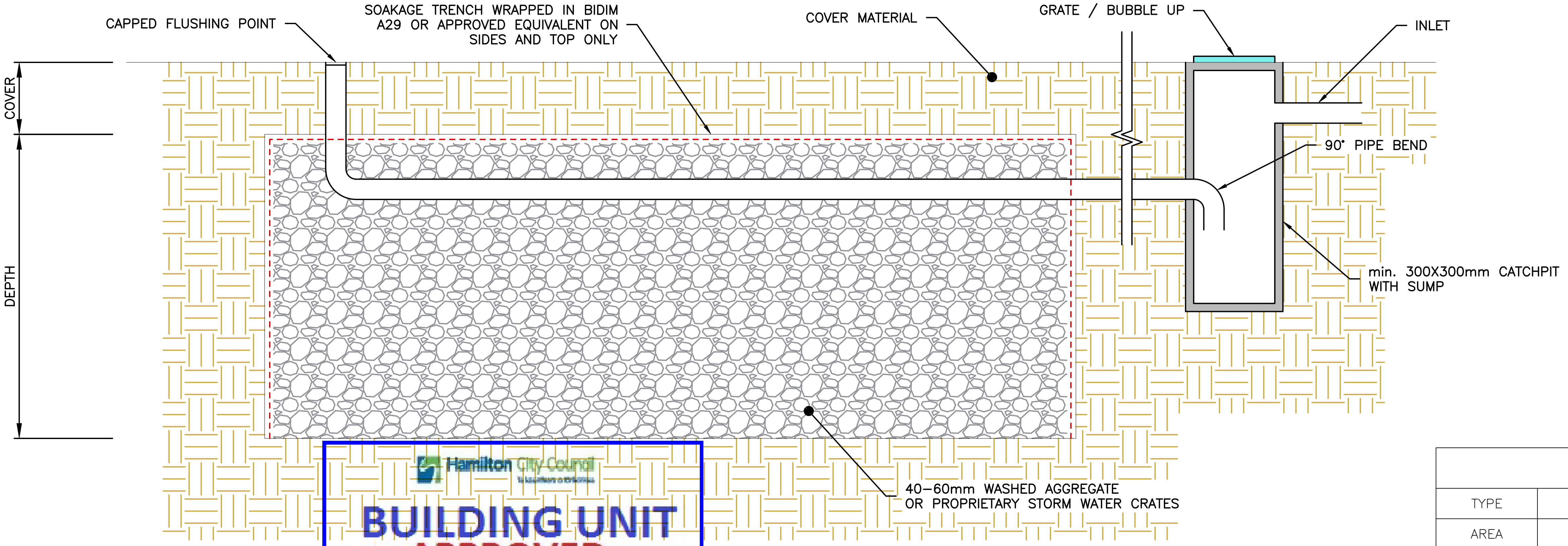
CLIENT: MARTIN CAMERON

PROJECT: NEW DWELLING
LOT 2 136 STATE HIGHWAY 26
HAMILTON

DRAWING: DISPOSAL FIELD

STATUS: FOR APPROVAL
FINAL AS APPROVED BY COUNCIL

DATE: 11/10/2021
SCALE (ORIGINAL SIZE A3): NTS
DRAWING NUMBER: 11122-WW-0006
REVISION: B



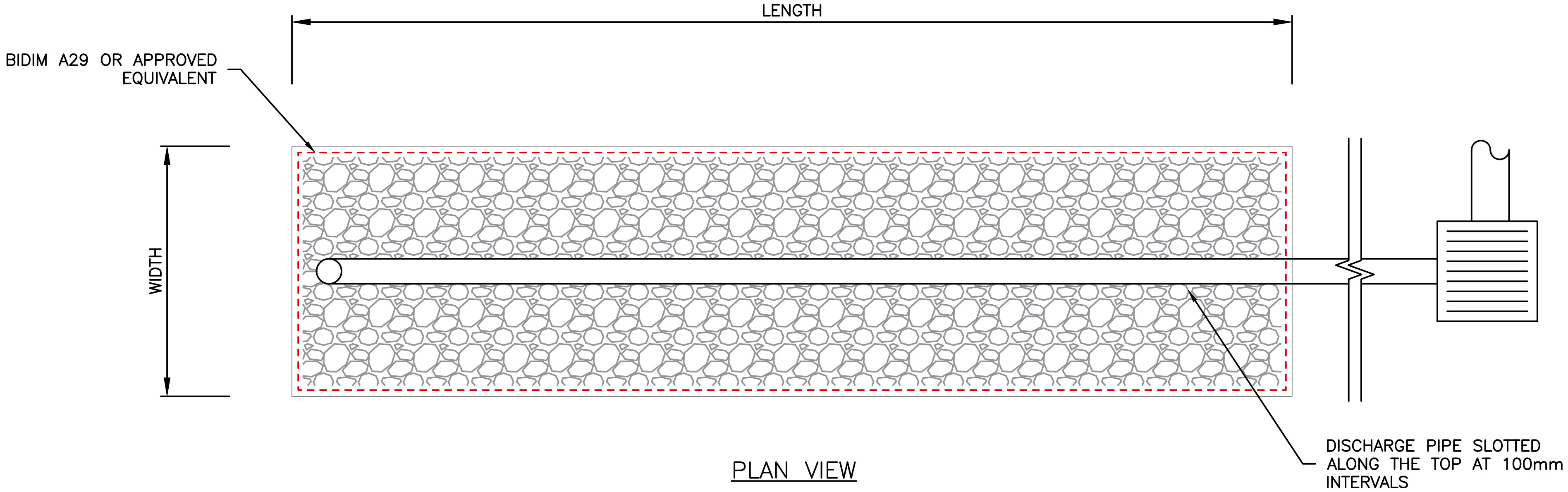
Hamilton City Council

BUILDING UNIT
APPROVED

BC Number - DD007.2021.00043914.001

SECTIONAL VIEW

SPECIFICATIONS		
TYPE	ROCK FILL	SW CRATES
AREA	$34.6m^2$	$20.1m^2$
DEPTH	1.0m	0.86m
WIDTH	1m (MIN)	1m (MIN)
LENGTH	AS REQUIRED	AS REQUIRED
COVER	0.3m (MINIMUM FOR NON–TRAFFICABLE AREAS)	AS PER MANUFACTURERS SPECIFICATION



PLAN VIEW

SOAKAGE TRENCH SETBACKS (RITS 4.2.15):

1. 3.0m FROM ANY BUILDING OR BOUNDARY (1.5m FROM BOUNDARY IF NEIGHBOURING BUILDING MUST BE 1.5m FROM BOUNDARY).
2. 0.5m FROM ROAD SIDE BOUNDARY'S.
3. FOR RETAINING WALLS LESS THAN 2m, THE SETBACK SHALL BE THE HEIGHT OF THE WALL PLUS 1.5m. FOR HIGHER WALLS A SPECIFIC DEIGN IS REQUIRED.
4. 2.0m FROM PUBLIC SANITARY SEWERS.
5. 1.0m FROM PRIVATE SEWER.
6. SOAKAGE TRENCH TO NOT BE POSITIONED ON UNSTABLE SLOPES AND AWAY FROM OVERLAND FLOW PATHS.

NOTES:

1. ALL WORKS TO BE CARRIED OUT IN ACCORDANCE WITH THE PRODUCT MANUFACTURERS SPECIFICATIONS, THE BUILDING CODE AND THE BUILDING CONSENT CONDITIONS.
2. ROCK FILL MUST BE 40–60mm OF CLEAN WASHED ROCK.
3. A PRE–TREATMENT DEVICE MUST BE USED TO MINIMISE SILT INGRESS INTO SOAK TRENCH.
4. CONTRACTORS SHALL PROVIDE AS–LAID PLANS AND A PS3 ON COMPLETION.

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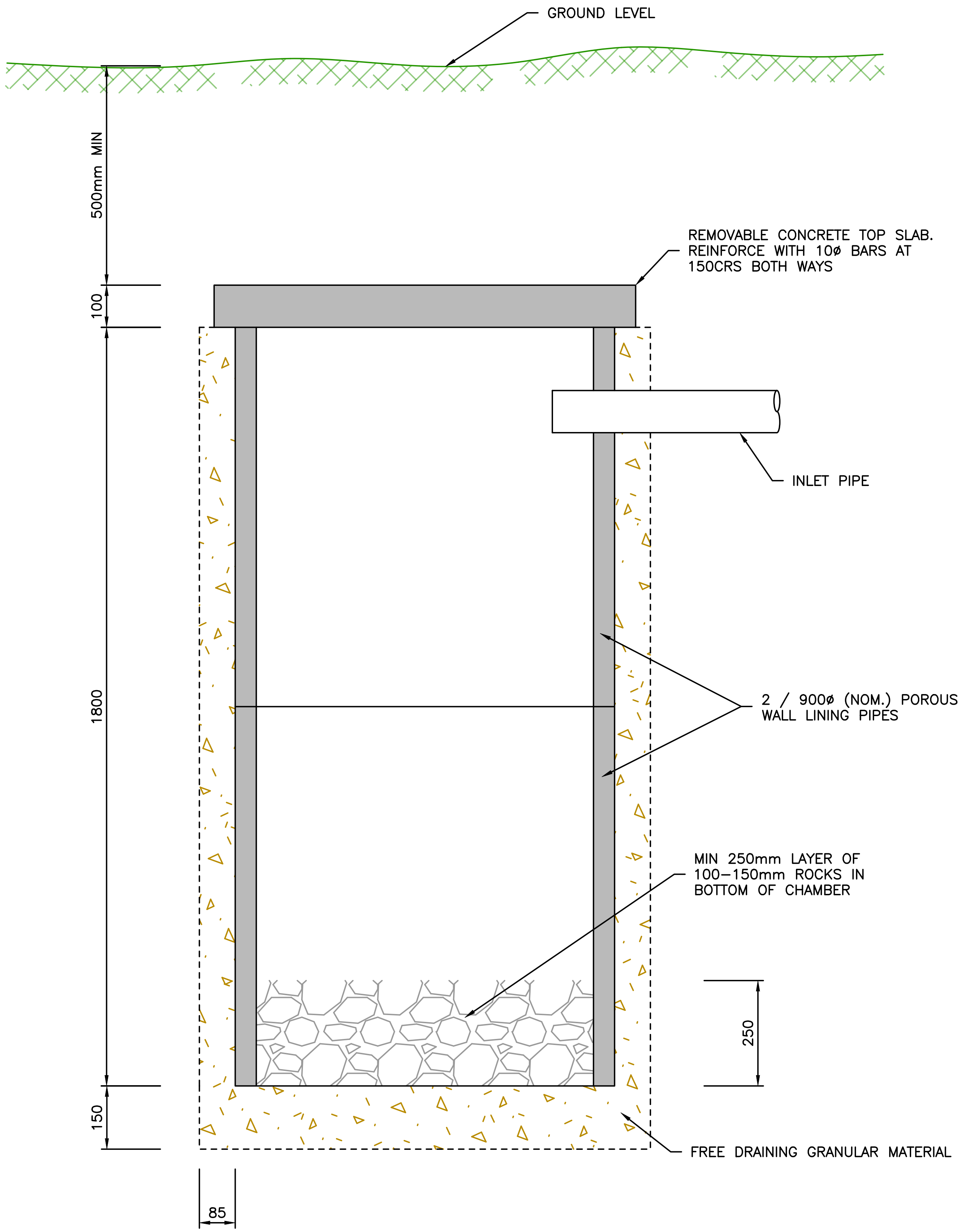
www.tituscivil.co.nz
office@tituscivil.co.nz

DESIGNED	CHECKED	CLIENT	PROJECT	DRAWING	STATUS
RM	JV	MARTIN CAMERON	NEW DWELLING	SOAKAGE TRENCH OPTION	FOR APPROVAL
DRAWN	APPROVED		LOT 2 136 STATE HIGHWAY 26 HAMILTON		FINAL AS APPROVED BY COUNCIL
RM	TR				DATE: 11/10/2021 — SCALE (ORIGINAL SIZE A3): NTS
					DRAWING NUMBER: 11122– SW –0010 — REVISION: A



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BUILDING UNIT APPROVED

BC Number - DD007.2021.00043914.001

SOAKAGE TRENCH SETBACKS (RITS 4.2.15):

1. 3.0m FROM ANY BUILDING OR BOUNDARY (1.5m FROM BOUNDARY IF NEIGHBOURING BUILDING MUST BE 1.5m FROM BOUNDARY).
2. 0.5m FROM ROAD SIDE BOUNDARY'S.
3. FOR RETAINING WALLS LESS THAN 2m, THE SETBACK SHALL BE THE HEIGHT OF THE WALL PLUS 1.5m. FOR HIGHER WALLS A SPECIFIC DEIGN IS REQUIRED.
4. 2.0m FROM PUBLIC SANITARY SEWERS.
5. 1.0m FROM PRIVATE SEWER.
6. SOAKAGE TRENCH TO NOT BE POSITIONED ON UNSTABLE SLOPES AND AWAY FROM OVERLAND FLOW PATHS.

NOTES:

1. ALL WORKS TO BE CARRIED OUT IN ACCORDANCE WITH THE PRODUCT MANUFACTURERS SPECIFICATIONS, THE BUILDING CODE AND THE BUILDING CONSENT CONDITIONS.
2. ROCK FILL MUST BE 100–150mm OF CLEAN WASHED ROCK.
3. A PRE–TREATMENT DEVICE MUST BE USED TO MINIMISE SILT INGRESS INTO SOAK TRENCH.
4. CONTRACTORS SHALL PROVIDE AS–LAID PLANS AND A PS3 ON COMPLETION.

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CAD FILE: C:\Users\TONY\AppData\Local\Temp\BricsCAD\bp_19\11122 – SW–0014 – SOAK pit.dwg

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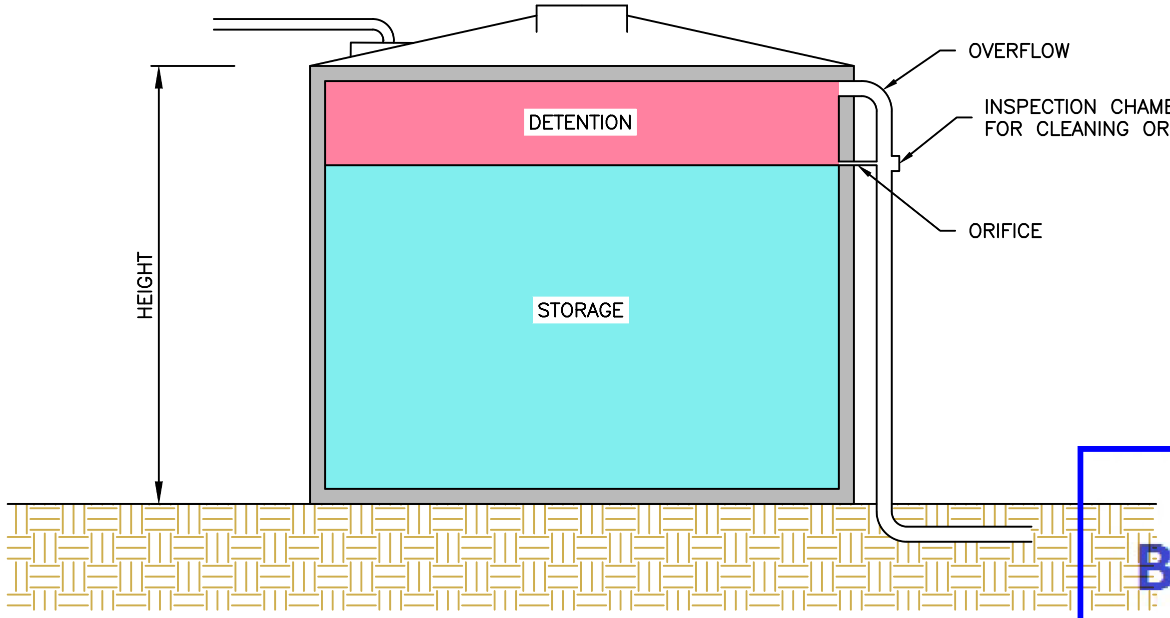
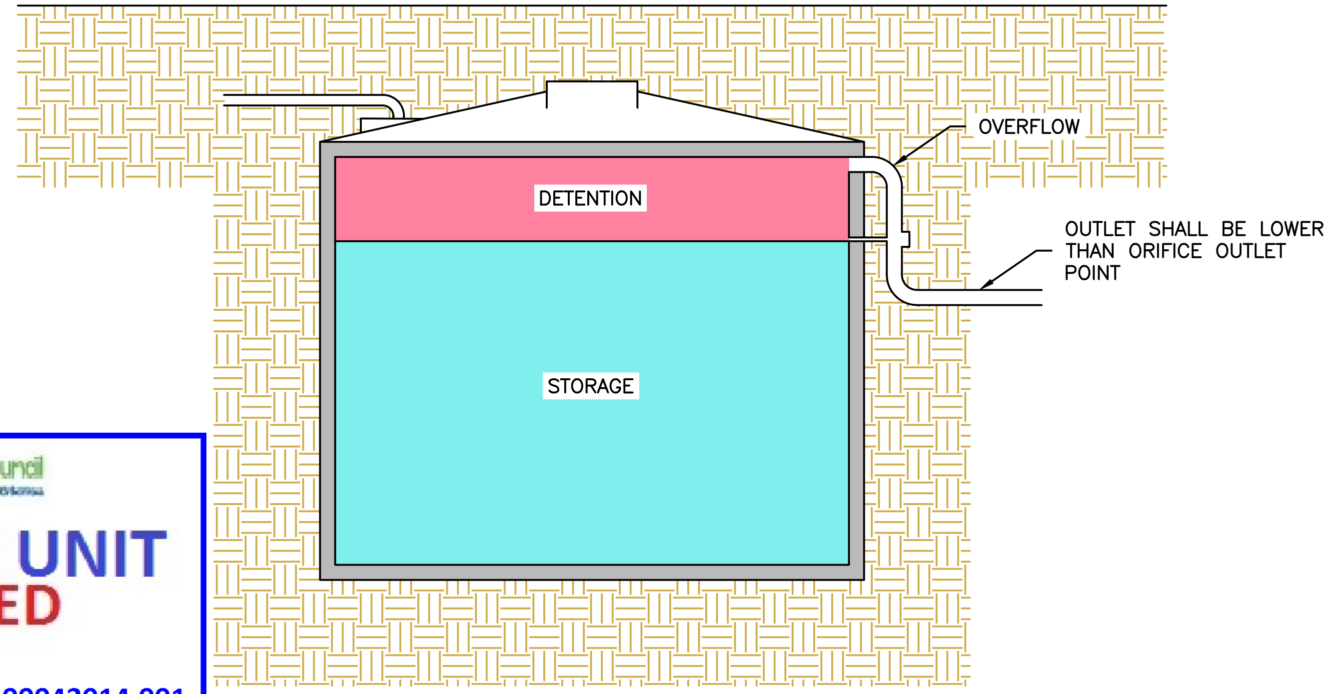


DESIGN 1: ABOVE–GROUND TANK

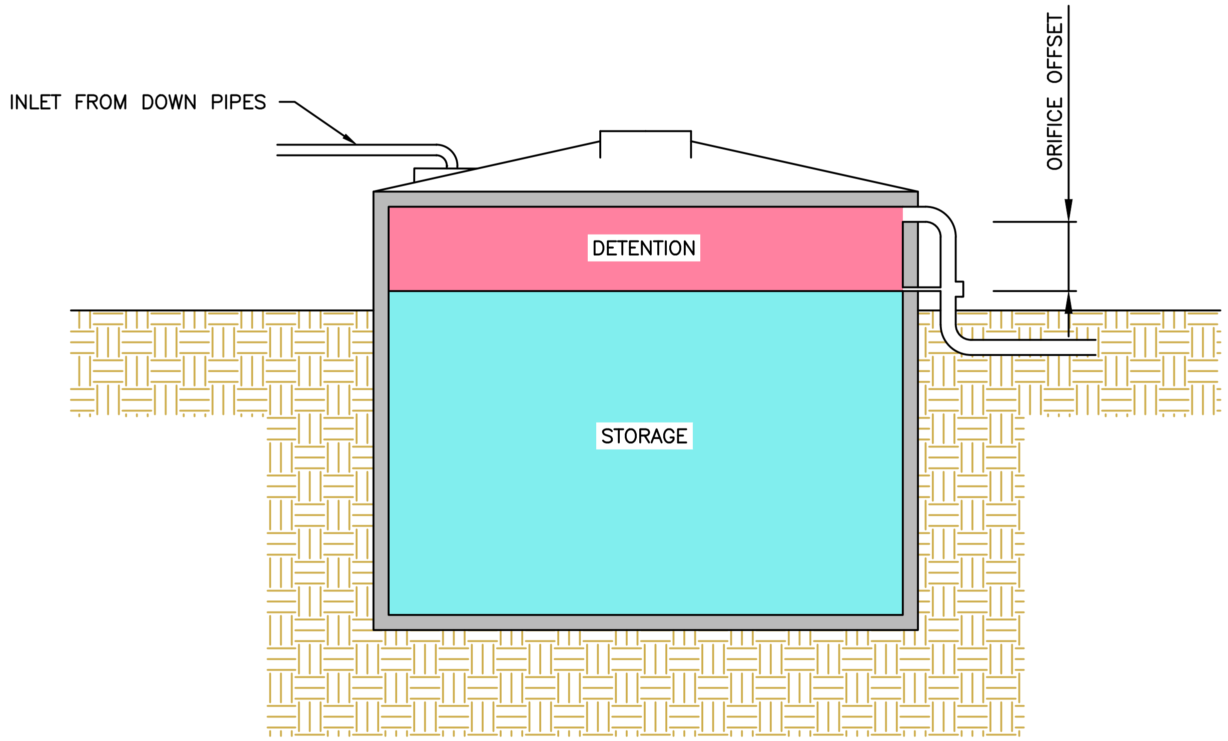
Hamilton City Council

BUILDING UNIT APPROVED

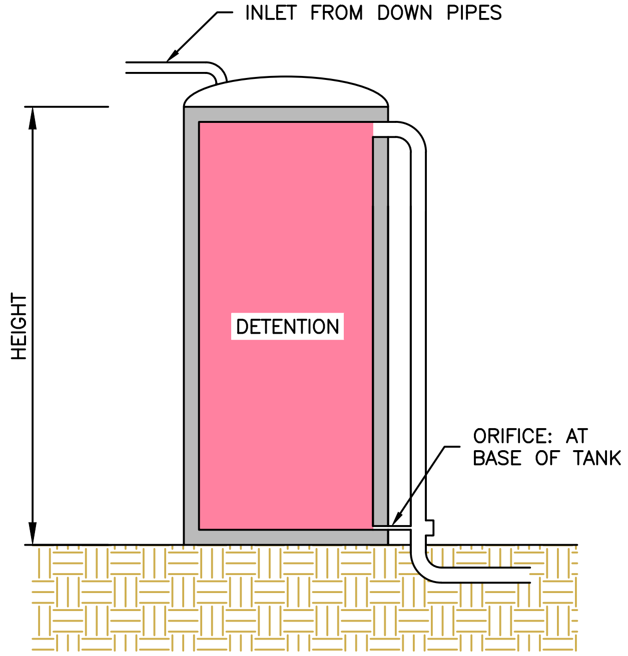
BC Number - DD007.2021.00043914.001



DESIGN 3: FULLY–BURIED TANK



DESIGN 2: SEMI–BURIED TANK



DESIGN 4: DETENTION–ONLY TANK

SPECIFICATIONS	
DESIGN OPTIONS AVAILABLE	1–3
VOLUME	1 x 25,000 L
HEIGHT	2,800 mm
ORIFICE	35 mm
ORIFICE OFFSET	560 mm
OVERFLOW	LEVEL SPREADER

NOTES:
1. ALL WORKS TO COMPLY WITH BUILDING, PLUMBING REGULATIONS AND MANUFACTURERS SPECIFICATIONS
2. ALL DRAINS TO BE 100mm PVC LAID AT 1:100 FALL UNLESS NOTED OTHERWISE.
3. CHECK LEVELS ON SITE BEFORE INSTALLATION.
4. APPROPRIATE COVER REQUIRED FOR OPTION 3 AS PER MANUFACTURERS SPECIFICATIONS.
5. FIRST FLUSH DEVICES AND LEAF DIVERTERS ARE RECOMMENDED.
6. TRICKLE FEED MAY BE SUITABLE DEPENDING ON SITE LOCATION.
7. OVERFLOW (INCLUDING FLOW FROM ORIFICE) SHALL BE GRAVITY FEED WITH NO BUBBLE UP DOWNSTREAM. OUTLET POINT SHALL BE LOWER THAN ORIFICE POINT.
8. TWO TANKS MAY BE UTILISED AND CONNECTED AT THE BASE.

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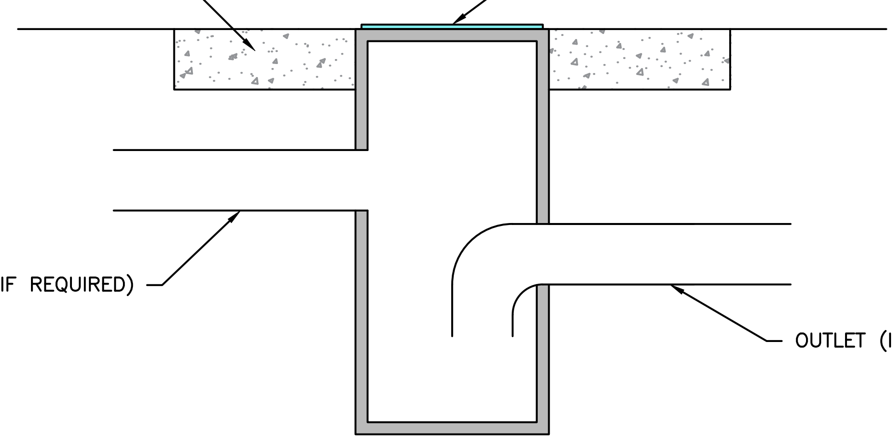
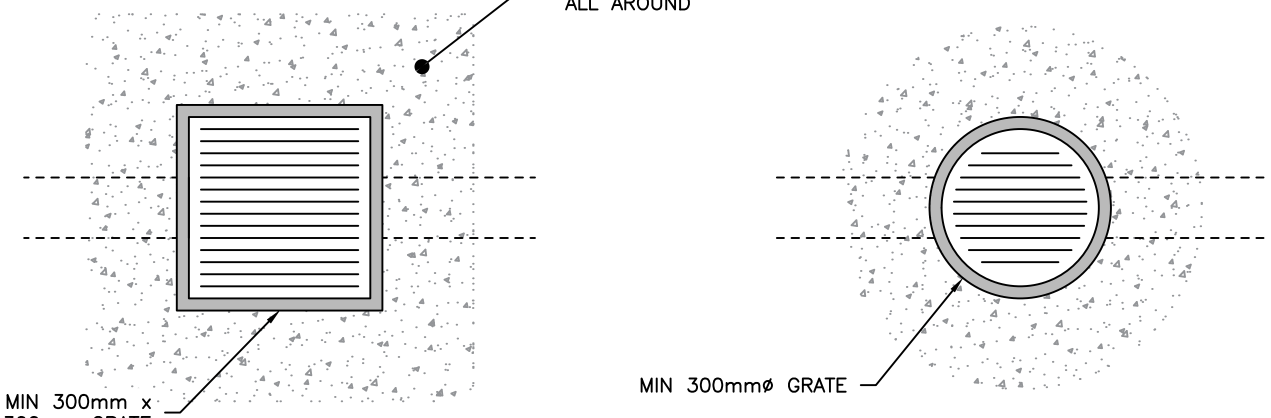
NOTES:
1. WORKS TO COMPLY WITH BUILDING, PLUMBING REGULATIONS AND MANUFACTURERS SPECIFICATIONS
2. CHAMBER TO BE LOCATED AS TO NOT CAUSE ANY STORM WATER NUISANCE



BUBBLE UP: SECTION VIEW



BUBBLE UP: PLAN VIEW

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office@tituscivil.co.nz

DESIGNED	CHECKED
RM	TR
DRAWN	APPROVED
RM	TR

CLIENT: MARTIN CAMERON

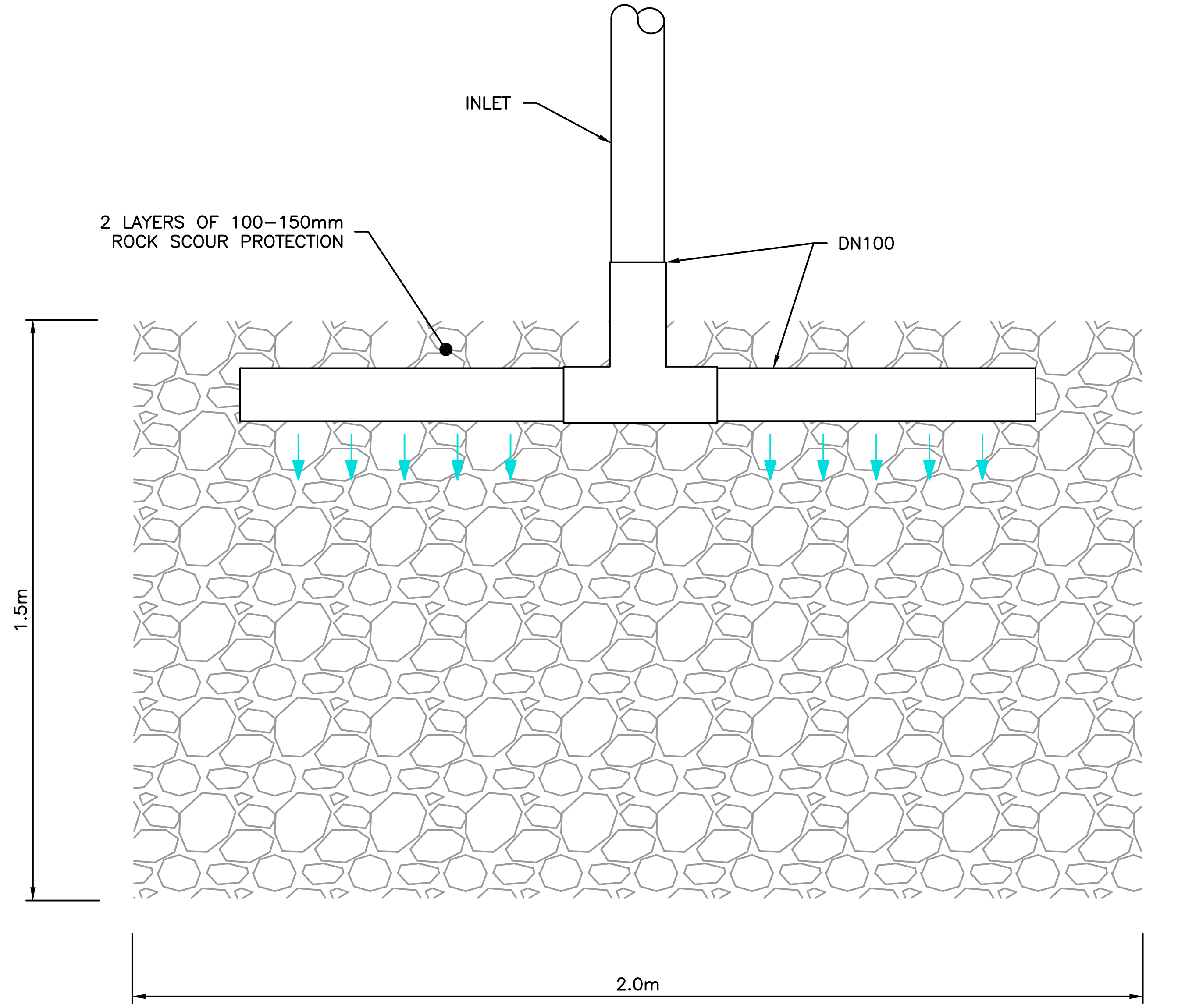
PROJECT: LOT 2, 136 STATE HIGHWAY 26 NEWSTEAD

DRAWING: BUBBLE UP

STATUS: FOR APPROVAL
FINAL AS APPROVED BY COUNCIL

DATE: 11/10/2021
SCALE (ORIGINAL SIZE A3): NTS
DRAWING NUMBER: 11122- SW -0030
REVISION: A

NOTES:
1. WORKS TO COMPLY WITH BUILDING, PLUMBING REGULATIONS AND MANUFACTURERS SPECIFICATIONS
2. CHAMBER TO BE LOCATED AS TO NOT CAUSE ANY STORM WATER NUISANCE

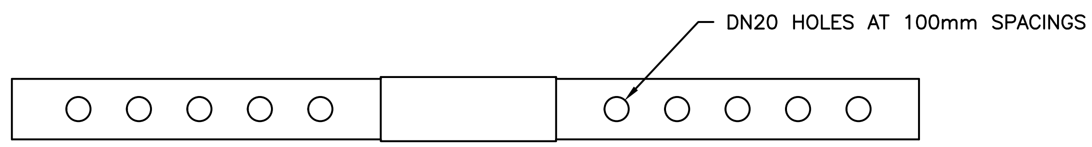


LEVEL SPREADER: PLAN VIEW

Hamilton City Council
Te kaunihera o Kirikiriroa

BUILDING UNIT APPROVED

BC Number - DD007.2021.00043914.001



LEVEL SPREADER: SECTION VIEW

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STATUS	FOR APPROVAL FINAL AS APPROVED BY COUNCIL	
DATE	11/10/2021	SCALE (ORIGINAL SIZE A3): NTS
DRAWING NUMBER	11122- SW -0031	REVISION: A

Native Plants Recommended for Effluent Fields (ETS Field).

Compiled by
Alter-Natives Wholesale Nursery. www.alter-natives.co.nz
571 Ormiston Road, Waipu, ph 09 4321333

The two main reasons for putting plants on effluent fields are:

1. To soak up the wastewater and transpire (*the plants version of perspiring*) it out through the plants foliage.
2. To absorb nutrients out of the wastewater so that it does not make its way into the underground water table and so on into stream systems.

Thus, plants that have a high transpiration rate, vigorous growth and are tolerant of wet soil conditions are best.

Plant spacings will vary depending upon species and mature size, but it's generally calculated at 1 plant per square metre of field. Thus a 200m square field should have 200 plants and so-on. When planting smaller grasses or lilies you may space them at 50-75cm while medium sized trees may be spaced at 1.2-1.5m, so overall with a mixed species planting it is typically 1 plant per metre.

The key differences between the two common types of effluent fields are:

1. Underground pipes (*usually 100mm diameter pipes with small seepage holes along its length and buried 600mm underground*)
2. Dripper lines on the ground surface (*usually 15mm diameter hoses with drippers laid under mulch*).

The biggest concern for the fields with underground pipes is the possibility of damage from large tree roots or blockage of the seepage holes along the pipes. Blockage is most likely to happen with deep rooting trees, or trees that will put out masses of feeder roots into water.

Many of the plants listed below can be viewed on our website. www.alter-natives.co.nz
Or by visiting our nursery: Alter-Natives Wholesale Nursery, 571 Ormiston Rd, Waipu

Sources of information:
Auckland Regional Council Technical Sheet G-1: List of water tolerant plants suitable for on-site wastewater disposal systems.
Northland Regional Council: Looking after your household sewage system.
Above documents have been used by most engineers around Whangarei and Northland.

Plants that are highly recommended. (Suitable for both sorts of effluent fields).

Grasses, Grass-like or Flaxes, Flax-like

Botanical name	Maori Name	Common Name	Typical height
Arthropodium cirratum	Rengarenga	Rock lily	20-40cm
Astelia grandis *		Swamp Astelia	1.5-2m
Carex dissita *			40-90cm
Carex flagellifera			50-70cm
Carex germinate			50cm-1m
Carex lessoniana			70cm-1.2m
Carex maorica *			50cm-1m
Carex secta	Purei, Pukio		70cm-1m
Carex tenuiculmis			50-70cm
Carex virgata			70cm-1m
Cortaderia fulvida	Toetoe, Kakaho	Cutty Grass	1-1.5m
Cortaderia toetoe	Toetoe, Kakaho	Cutty Grass	1.5-2m
Cyperus ustulatus *	Toetoe upokotangata	Giant umbrella sedge	60cm-1m
Dianella nigra	Turutu	NZ blue berry	30-60cm
Juncus gregiflorus *	Wiwi	Common rush	50cm-1.5m
Leptocarpus similis	Oioi	Jointed Rush	50-70cm
Machaerina sinclairi *	Pepepe, Toetoe tuhara		50cm-1m
Phormium tenax	Harakeke	Flax	1.5-2m
Phormium tenax purpurea		Purple Flax	1.5-2m
Uncinia unciniata *		Hook Grass	20-40cm

Spreading ground covers

Botanical name	Maori Name	Common Name	Typical height
Coprosma acerosa	Tataraheke	Sand Coprosma	30-40cm
Coprosma atropurpurea *			5-10cm
Coprosma 'Hawera'			10-20cm
Coprosma kirkii			30-50cm
Coprosma 'Poor Knights'			30-50cm
Coprosma prostrata			30-50cm
Coprosma 'Taiko'			20-30cm
Elatostema rugosum *	parataniwha		40-70cm
Fuchsia procumbins			10-20cm

Shrubs and small trees (typically less than 5m)

Botanical name	Maori Name	Common Name	Typical height
Brachyglottis repanda *	Rangiora		3-4m
Carpodetus serratus	Putaputaweta	Marble leaf	3-5m
Coprosma arborea *	Mamangi		3-4m
Coprosma areolate *		Thin-leaved coprosma	3-4m
Coprosma cultivars which are coloured (lots of options)			1-3m
Coprosma propinqua	Mingimingi		1-3m
Coprosma repens	Taupata	Mirror Plant	2-4m
Coprosma rhamnioides *			1-2m
Coprosma robusta	Karamu		2-4m
Clianthus puniceus	Kowhai ngutu-kaka	Kaka Beak	2-3m
Fuchsia excorticate *	Kotukutuku	Tree Fuchsia	3-5m
Geniostoma rupestre *	Hangehange		1-2m
Hebe Stricta	Koromiko		1-3m
Macropiper excelsus	Kawakawa		1-3m

Trees (typically 5m or more)

Botanical name	Maori Name	Common Name	Typical height
Kunzea ericoides	Kanuka	White Tea-tree	6-10m
Cordyline australis	Ti kouka	Cabbage Tree	5-10m
Leptospermum scoparium	Manuka	Tea-tree	4-8m

Ferns

Botanical name	Maori Name	Common Name	Typical height
Blechnum novaezealandiae *	kiokio		1-2m
Dicksonia squarrose	wheki	tree fern	3-5m
Hypolepis dicksonioides *			1-1.5m

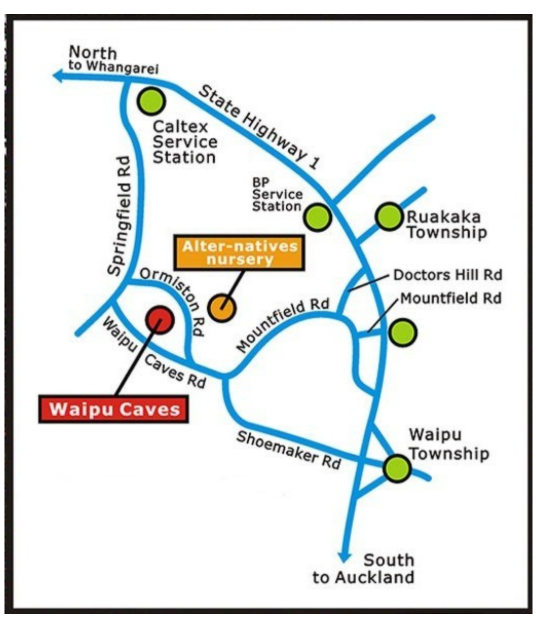
Large trees that are recommended for effluent fields using dripper lines; or only for on the fringe of underground systems (3m or more away from pipes because of the potential for the plant roots damaging your pipes).

Trees (typically 5m or more)

Botanical name	Maori Name	Common Name	Typical height
Laurelia novae-zealandiae	Pukatea		10-20m
Corynocarpus laevigata	Karaka		8-15m
Dacrycarpus dacrydioides	Kahikatea	White pine	20-30m
Pittosporum eugenoides	Tarata	Lemonwood	6-10m
Aristotelia serrata	Makomako	Wineberry	6-10m
Pittosporum tenuifolium	Kohuhu		5-8m
Plagianthus regius	Manatu	Ribbonwood	8-12m
Hoheria populnea	Houhere		6-8m
Melicytus ramiflorus	Mahoe	Whiteywood	4-6m
Rhopalostylis sapida	Nikau		6-12m
Pennantia corymbose *	Kaikomako		6-10m
Schflera digitata	Pate, Pata	Seven finger	4-6m
Vitex luscens	Puriri		10-20m

Note: Plants marked with a * can sometimes be hard to find in nurseries around Northland.

Good luck. From the team at Alter-Natives Nursery.



Building Code Clause(s) E1, G13

PRODUCER STATEMENT - PS1 - DESIGN

ISSUED BY: Titus Consulting Engineers
(Design Firm)

TO: Martin Cameron
(Owner/Developer)

TO BE SUPPLIED TO: Hamilton City Council
(Building Consent Authority)

IN RESPECT OF: Stormwater Design, Wastewater Design
(Description of Building Work)

AT: 2 / 136 State Highway 26
(Address)

Town/City: Hamilton (Address) **LOT** 2 **DP** 556335 **SO**

Hamilton City Council

BUILDING UNIT APPROVED

BC Number BD007.2021.00043914.001

We have been engaged by the owner/developer referred to above to provide:

Stormwater Design, Wastewater Design
(Extent of Engagement)

services in respect of the requirements of Clause(s) E1, G13 of the Building Code for:

☐ All or ☒ Part only (as specified in the attachment to this statement), of the proposed work.

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

☒ Compliance Documents issued by the Ministry of Business, Innovation & Employment E1/VM1, G13/VM4 (Verification methods/acceptable solutions) or

☐ Alternative solution as per the attached schedule

The proposed building work covered by this producer statement is described on the drawings titled:

Plan, Soakage Pit, Rainwater Tank, Level Spreader, Disposal Field, Bubble Up, and numbered 11122-P-1,SW-11,20,31,WW-2,SW-30

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

(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)

I, Anthony Richardson (Name of Design Professional) am: ☒ CPEng # 1026340

I am a member of: ☒ Engineering New Zealand and hold the following qualifications: BSc(Eng) Civil, MSc (Eng)

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 ACE New Zealand: ☒

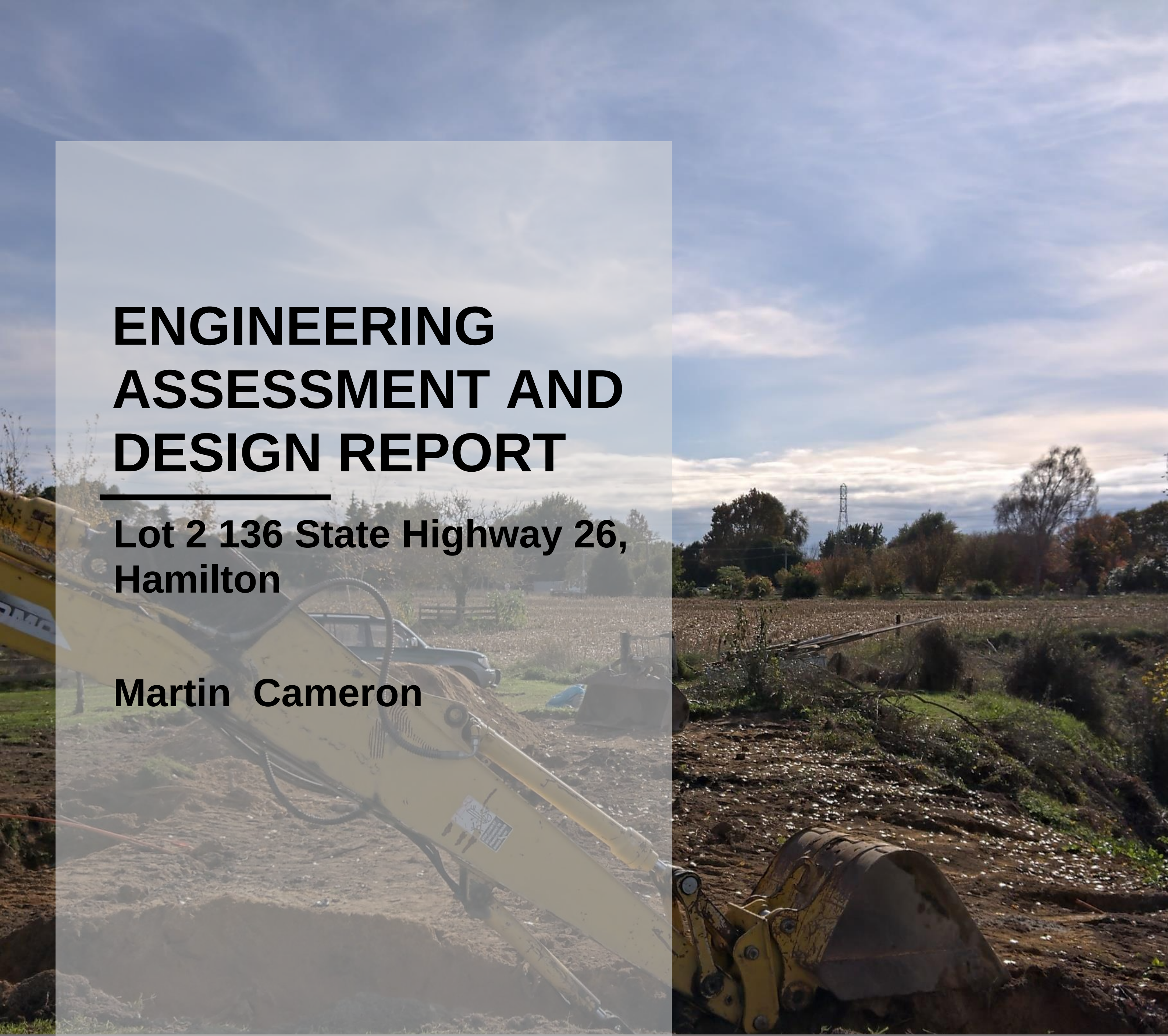
SIGNED BY Anthony Richardson (Name of Design Professional) (Signature) Tony Richardson

Digitally signed by Tony Richardson
DN: cn=Tony Richardson, c=NZ, o=Titus Civil Ltd, email=tonyr@tituscivil.co.nz
Date: 2021.10.01 21:28:21 +13'00'

ON BEHALF OF Titus Consulting Engineers (Design Firm) Date 1/10/2021

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.



ENGINEERING ASSESSMENT AND DESIGN REPORT

Lot 2 136 State Highway 26, Hamilton

Martin Cameron

11 OCTOBER 2021

PROJECT NO. 11122

Hamilton City Council

BUILDING UNIT APPROVED

BC Number - DD007.2021.00043914.001

TITUS

CONSULTING ENGINEERS

Approved for issue by:

X Tony Richardson

Digitally signed by Tony Richardson
DN: cn=Tony Richardson, c=NZ, o=Titus Civil Ltd, email=tonyr@tituscivil.co.nz
Date: 2021.10.11 14:01:04 +13'00'

Anthony Richardson CPEng 1026340
Principal Project Engineer

Hamilton City Council
Te kaunihera o Kirikiriroa

BUILDING UNIT APPROVED

BC Number - DD007.2021.00043914.001

DOCUMENT HISTORY AND STATUS

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A	Martin Cameron	20/05/2020	RM	MH/JV	TR
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E	Martin Cameron	2/02/2021	RM	TR	TR
F1	Martin Cameron	11/10/2021	RM / JV	JV / TR	TR

RECORD OF REVISION CHANGES

Revision B – Revised slope stability recommendations.

Revision C – Model revised to reduce height of slope by 2m through onsite earthworks.

Revision D – Section 5: Slope Stability Assessment updated with new design of basement. WW Design updated

Revision E – Section 2.4: Changed recommendation table title

Revision F – Revised dwelling orientation, SW/WW locations, and proposed rainwater tank

TITUS Consulting Engineers

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TABLE OF CONTENTS

1 INTRODUCTION

1.1 Overview

TITUS CIVIL Consulting Engineers has been engaged by Martin Cameron to perform an engineering assessment and design report for a storey timber-framed building at Lot 2 136 State Highway 26.

The report includes the following.

- Section 2: Site and Soils Assessment.
- Section 3: Stormwater Assessment and Design.
- Section 4: Wastewater Assessment and Design.
- Section 5: Slope Stability Assessment.

The assessments and design meet the requirements of the local authority, Hamilton City Council, and the following technical documents.

- The building code,
- NZS3604:2011,
- District Plan,
- Any current ICMP,
- Waikato Regional Council Plan, and
- AS/NZS 1547/2012

1.2 Site Details

The site is currently a newly subdivided lifestyle block with a large gully at the back of the section. The site is bordered by a gully system to the east, a field to the north (used for cultivation), and a residential house and garage to the south / south west. The area near the proposed foundation is gently sloping to the north. The location of the house is close to the top of a slope joining the gully system to the east.

The large gully system is approximately 3km upstream from where it enters the Waikato River.

Figure 1 shows a photo of the proposed dwelling location.

Figure 1: Site Photo

1.3 Planning Requirements

The following requirements based on the Regional Council Plan, Consent Notices and Subdivisional Reports are noted, and have been duly considered in the proposed recommendations.

The following is taken from the resource consent from HCC:

(2)(7) Any contaminated soil is to be removed under controlled conditions to a licensed waste facility or landfill for disposal in accordance with the RAP, and with the requirements of the disposal site and the relevant authority. Receipts of transport and disposal are required to be provided in the Site Validation Report.

- An area of lead contaminated ground from near the previously existing cow shed has been removed and disposed of on the day of the site investigation.

The following is taken from the Geotechnical report for lot 4 of the same subdivision and gives a setback of 7.5m for a dwelling along the same gully slope that lot 2 is on:

Slope Stability Assessment

- The conducted slope stability assessment indicates a development building setback of at least 7.5m from the crest of the slope. Building within the setback zone is feasible but specifically designed foundations (such as piles) will be required.

2 SITE AND SOILS ASSESSMENT

2.1 Assessment Parameters

This section details findings of a site and soils assessment in accordance with NZS3604:2011 cl. 3.1.3.1 Determination of 'Good Ground'. The investigation is in relation to the construction of a new single storey timber-framed building.

In particular the investigation focussed on assessing:

- The bearing capacity of the soil in accordance with NZBC – B1 (New Zealand Building Code),
- Checking for organic and peat soils,
- Checking for soft and very soft clays containing gravel or other hard material and,
- Checking for uncontrolled fill.

NZBC requires 5 blows per 100mm down to a depth of twice the footing width or 3 blows per 100mm at greater depths to establish good ground in terms of bearing capacity of soils.

Foundations outside of the scope of NZBC or proprietary specifications require *specific engineering design* (SED).

The proposed building has a floor area of approx. 250m^2 and various foundation options are being considered.

2.2 Soil Investigation

The site assessment conducted on 12th of May 2020 included the following:

- General site walkover
- Hand Auger Tests: 4
- Scala Penetrometer Tests: 4
- Shear Vane Tests: 7
- Soakage Test: 1

Test locations are shown in Appendix A.

Topsoil was found at a depth of 200mm on site in borehole 5 but not in boreholes 1 to 4 as they were located beneath the removed cowshed foundations. Underlying soils consist predominantly of sand. Overall, the boreholes showed interbedded layers of sand and silt with little correlation between boreholes.

No soft clays were found on the site (kPa < 25).

Organic material was found in borehole 2 under the propsed dwelling location. The material is suspected to be a dump site associated with the previously existing cow shed. This material was only found in an isolated area and was removed on the day of inspection of the removal of contaminated soil from the site.

Soakage testing yielded a raw soakage rate of 900mm/hr. An appropriate factor of safety shall be applied before use in design calculations.

The water table was not found in any borehole to a depth of 2.0m. The water table was found at 13.0m in the CPTs on the adjacent lot.

2.3 Preliminary Liquefaction Assessment

2.3.1 Geological Setting

According to GNS (GNS Science, 2019), the underlying geology of the site is classified as (Late Pleistocene) river deposits (Hinuera Formation), as shown in Appendix D. This is described as cross-bedded pumice sand, silt, and gravel with interbedded peat. The Late Pleistocene sediments are approximately up to 27,000 years old. The site sits on a geological boundary between Hinuera Formation and Holocene sediments. This boundary will sit somewhere on the slope where eroded sediments have been deposited. Given the nearby gully and the free draining nature of the Hinuera Formation it is assumed that the long-term water table is located near the base of the gully.

2.3.2 Seismic Parameters

Table 1 below summarises the seismic parameters adopted for the site:

Table 1: Seismic parameters (NZTA Bridge Manual, Third Edition)

Ground Acceleration (SLS)	
Hamilton	
Class D	
1/25	
f	1.00
R_u	0.25
$C_{0,1000}$	0.28
M_{eff}	-
PGA, a_{max} (g)	**0.05**

Ground Acceleration (ULS)	
Hamilton	
Class D	
1/500	
f	1.00
R_u	1
$C_{0,1000}$	0.28
M_{eff}	5.9
PGA, a_{max} (g)	**0.22**

The site is located within the Waikato Basin which is generally known for deep sedimentary soils and deep basement rock. Development of a preliminary model of the fundamental site period (T0) across the Waikato Basin has shown that most places within the Waikato Basin have fundamental periods longer than 0.6s and hence should be categorised as Site Class D. (Jeong & Wotherspoon, 2019)

Therefore, Subsoil Class D – Deep or Soft Soil (NZS 1170.5:2004) may be adopted for this site.

2.3.3 Liquefaction Susceptibility

A comparison between the ideal conditions for liquefaction occurrence and conditions found for each proposed lot assessed is shown in Table 2 below;

Table 2: Conditions for liquefaction occurrence

Soil conditions considered susceptible to liquefaction occurrence	Site
Holocene to Late Pleistocene sediments	Yes
Cohesionless	Yes
Non-cohesive silt to medium to fine sand	Yes*
Loosely packed	Yes*
Shallow water table (<4m)	No
Thick non-liquefiable crust at the ground surface	Unlikely

**Limited layers*

Due to underlying geology and according to Hamilton City Liquefaction Report prepared by Tonkin & Taylor it is indicated that liquefaction damage is possible. Due to the depth to water table and the free draining nature of the gully systems around Hamilton, liquefaction damage at the site is considered unlikely and no mitigation measures are recommended.

Note: In order to determine if any layers are susceptible to liquefaction below the base of the slope which may affect slope stability a detailed liquefaction assessment of the CPTs using CLIQ has been carried out, refer to Appendix I.

2.4 Recommendations

The following foundations options are suitable given the soil conditions on site, however, are subject to confirmation of the specific requirements of the recommended foundation, the slope on site and any filling proposed for the site.

2.4.1 SED Piled Foundation

An SED Piled Foundation shall be designed as summarised below and as per the slope stability assessment (in Section 5).

Table 3: Foundation Parameters

SED Piled Raft for House	
Minimum depth of excavation for sand pad to good ground	1200mm below original proposed ground level
Minimum Pile Depth	5.0m
Maximum Out-of-Plane Spacing	2.0m
Backfill material	Sand (Granular fill (brown rock) below 500mm)

Compaction standard	8 blows/300mm (Scala penetrometer) 270kPa
Inspections required	1 - Sub grade prior to back fill 2 - Compacted and finished sand pad
Foundation type	SED Piled raft
Comments	The foundation designer shall ensure the foundation is appropriate as per Section 5.
Piles for Concrete Foundation	
Minimum Pile Depth	5.0m
Maximum Out-of-Plane Spacing	2.0m
Inspections required	Pile driving / base of bored pile holes as applicable
Foundation type	SED pile foundation
Comments	The foundation designer shall ensure the foundation is appropriate as per Section 5

The piling and foundations shall be inspected in accordance with council and building code requriements.

3 STORMWATER ASSESSMENT AND DESIGN

3.1 Design Parameters

- Lot Size: 1,413m^2
- Proposed roof area: approx. 250m^2
- Design storms:
  - Primary: 10yr ARI
  - Secondary: 100yr ARI
- Rainfall data: Ruakura Rainfall data
- Climate change: 2.1 degrees warming
- Soakage rate: 900mm/hr (tested 12th of May 2020) – adopted 225mm/hr. Refer to Appendix C for results.
- Water table was determined to be 13.0m below the ground surface in the CPT logs from the neighbouring lot 4.

Figure 2 below summarises the catchment characteristics that have been adopted.

TANK DESIGN CALCULATIONS AND OUTPUTS

Rainfall Location	Event	ARI
Hamilton	Primary	10
	Secondary	100

Catchment	Area (m2) Existing	Area (m2) Proposed	C
Grass	280		0.30
Roof		280	0.95
Concrete			0.90
Gravel			0.70
Other			-
TOTAL	**280**	**280**	
Composite C	0.3	0.95	
Adopted C	**0.30**	**0.95**	

Existing	Input / Select
Proposed	Answer

Existing Catchment Characteristics, Time of Concentration (Tc)	
Average grassed surface	0.045
Length of flow path (m)	20.00
Slope (%)	3.00
Tc (min)	10.49
Existing Q(max) (l/s) (interpolated wrt Tc)	2.12
Proposed Q(max) (l/s)	7.89

Figure 2: Stormwater Design Parameters

3.1.1 Attenuation Tank

It is proposed that the roof runoff from the design storm is attenuated in a rainwater tank and released via a 35mm orifice to match the existing flow rate. The minimum detention storage is 5,000L. The proposed tank size is 1 X 25,000L and may be located at the Client's discretion given that Council's requirements are met. The orifice should be located 560mm below the invert of the overflow pipe.

The rainwater tank outlet and overflow shall discharge to the level spreader at the base of the slope.

Subsurface water drains shall be sized in accordance with Acceptable Solutions and Verification Methods for New Zealand Building Code Clause E1 Surface Water (E1/AS1) Section 3.

3.1.2 Soakage from other impermeable surfaces

The stormwater runoff from the impermeable driveway has been designed to be routed to the nearby soakage pit. The soakage pit is to be 900mm in diameter and a minimum of 1.8m deep. Overflow from this device shall flow to the level spreader at the base of the gully.

3.1.3 Secondary flow path

The stormwater runoff from impermeable surfaces has been designed to be routed via the rainwater tank and soakage pit. The overflow from these devices shall discharge to the bottom of the nearby gully as far as possible from the slope beneath the proposed dwelling.

3.2 Operation and maintenance

It is recommended that first flush devices are installed upstream of the rainwater tank and that these devices are regularly checked and cleaned along with the catchpit filters and overflow pipes.

3.3 Construction Monitoring

TITUS CIVIL Consulting Engineers have been engaged to perform inspections of the storm water system during construction.

4 WASTEWATER ASSESSMENT AND DESIGN

4.1 Design Parameters

The following design parameters have been adopted to design the system to meet the requirements:

- Water supply to the property will be reticulated community supply
- 5-bedroom home
- 8 people occupancy
- 145L/day/person
- Peak daily flow 1,160L/day
- The soil at the site is classified as a soil category 2 - Sandy loams (AS/NZS 1547:2012).
- Council planning maps show no flooding risk for the site.

4.1.1 Water Use Requirements

The following water use requirements are noted:

- Design information of 145L/day/person is based of AS/NZS 1547:2012. This requires the proposed building to have **FULL** water reduction fixtures.
- standard water reduction fixtures include reduced flush 6/3 litre water closets, shower flow restrictors, aerator faucets, front-load washing machines and flow/pressure control valves on all water-use outlets (9L/min maximum). Baths should also be discouraged.

4.2 Treatment Design

4.2.1 Secondary Treatment System

Both primary and secondary treatment will be provided by an Ecocycle Fusion Treatment Plant (or similar). This system includes a 4,500-litre chamber for primary treatment, and a 1,500-litre treatment unit chamber. It has an emergency storage of 2,000 litres. It can treat up to 1,600 litres of wastewater per day. This system has been tested by the On-site Effluent Treatment National Testing Programme (OSET) based at the Rotorua/ BOP wastewater plant and complies with the NZ Standards for on-site wastewater management and Waikato Regional Council conditions for rule 3.5.7.6 of the Waikato Regional Plan.

The proposed system has been designed as per the table below.

Table 4: System Specifications

Min Septic Tank (L)	24hr settling volume (L)	Scum and sludge capacity (L)	Max Pump out frequency (Yrs)
4500	1160	3200	5
DLR recommended (mm/d)	DLR adopted (mm/d)	Daily Flow (L/day)	Basal area (m^2)
10-25	35	1160	34

The Ecocycle Fusion Treatment Plant is an environmentally sustainable recirculating packed bed bio filter for aerobic secondary treatment of wastewater. The system performs significantly higher than the standard required by AS/NZS1547:2012.

The system has been tested and provides for the following:

- BOD_5 <10g/m^3 average
- Suspended solids <10g/m^3 average
- 10:10 Standard

Attached to this report is the following documentation:

- Certification confirming that the system has undergone testing to comply with the NZ Standards for on-site wastewater management and the Waikato Regional Plan rule 3.5.7.6.
- Manufacturer’s technical specifications for the tank and treatment plant.
- System warranty.
- Owner’s operation and maintenance guidelines.
- Planting guideline.

4.2.2 Wastewater Disposal

Primary and secondary treatment will be achieved using a septic tank and a treatment facility with disposal through LPED Beds. The design is outlined in the Table below.

Table 5: Disposal Method Specifications

Disposal Method / Specification	Beds
Number of beds	2
Length (m)	10
Width (m)	1.8
Spacing (m)	1
Basal area (m^2)	34
Total area (m^2)	43 + 43 Reserve Area

Appendix F provides an indicative layout of the proposed wastewater system and typical details.

4.3 Maintenance, Operation and Planting

Maintenance and Operation of the system shall be as per the manufacturers specifications, AS/NZS 1547:2012 and the recommendations contained in the appendices.

Planting shall be as per AS/NZS 1547:2012 and the recommendations contained in the appendices.

4.4 Inspections

TITUS CIVIL Consulting Engineers should be engaged to inspect the installation of the Septic Treatment and Land Disposal Systems prior to any excavations and pipe installations being buried.

5 SLOPE STABILITY ASSESSMENT

5.1 Assessment parameters

This slope stability assessment will consider the stability of the existing slope as well as the proposed plans with the basement cut into the slope, thus reducing the effective height of the slope. The assessment also considers strength loss in liquefiable layers following a ULS event.

The slope has been modelled using SLIDE 2018 software under several loading and ground water conditions. The report details the results of the assessment under the following loading conditions:

- Gravity (drained)
- Gravity (drained, elevated water table)
- SLS (Serviceability Limit State) – (drained)
- ULS (Ultimate Limit State) – (drained)
- Post Liquefied Conditions

The slope has been modelled in the three following scenarios:

- Existing conditions (prior to any earthworks undertaken on site)
- Proposed cutdown and dwelling
- Proposed cutdown and dwelling with strength loss layers due to liquefaction caused by a ULS earthquake.

Proposed slope cutting and dwelling foundation has been modelled to the specifications outlined in the latest engineering plans by Don Crowie Draughting & Design Services. Foundation Pile depths have been modelled to required depths to be founded below predicted failure arcs and to provide overall stability.

5.2 Historic Land Use

The site has previously been used as a milking shed that existed from pre-1938 until recent removal following subdivision of the land.

5.3 New Zealand Geotechnical Database

The New Zealand Geotechnical database has no entries close to the site. CPT logs from lot 4 of the subdivision have been used to determine the geological parameters in the slope model. The locations of the CPT logs are shown in Appendix A.

5.4 Geological Setting

Refer to section 2.3.1 *Geological Setting.*

5.5 Site Observations

The slope runs across the site from north to south. The slope separates flat (<5%) land above it to the west from the vegetated gully below it. Vegetation on the slope itself has been cleared in preparation for specialised planting. There were no outcrops of rock found on site. This is consistent with the geology of the Hamilton basin which has deep soils and deep bedrock.

The slope ranges in steepness from 7 degrees to a maximum of 40 degrees with an average slope of 27 degrees or 51% incline. Two large poplar trees are present at the top of the slope. Figure 3 below shows the slope below the proposed dwelling location. The loose material seen on the slope in Figure 3 is sand from the removal of the milking shed foundation. No evidence of slope instability was seen during the site inspection.

Figure 3: Photo of slope from below proposed dwelling location.

5.6 General

Slope stability modelling has been undertaken using Slide 2018 by RocScience using the Morgenstern-Price method to analyse the slope. The cross section of the slope was based on contour data taken from HCC 3 waters online mapping service. Location of the slope modelled is attached in Appendix A and Slope models are attached in Appendix H.

The factors of safety (FOS) as summarised in Table 7 has been adopted as appropriate for the loading conditions:

Table 7: FOS Standard Requirements

Modelled Loading Condition	FOS Required
Gravity Conditions	1.5
Gravity Conditions (elevated water table)	1.3
Seismic SLS (Serviceability Limit State)	1.45
Seismic ULS (Ultimate Limit State)	1.05
Post Liquefaction	1.1

5.7 Adopted Subsurface Conditions

The stratigraphy as determined by TITUS Consulting Engineers with reference to CPT logs for lot 4 undertaken by OPUS, has been separated into the different materials displayed in the Table below.

Table 8: Material characteristics

Material Name	Color	Unit Weight (kN/m3)	Sat. Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Vertical Strength Ratio	Minimum Shear Strength (kPa)	Water Surface	Hu Type	Hu
Coarse Sands		17.5	20.7	Mohr-Coulomb	2	36			Water Surface	Automatically Calculated	
Medium Sands		17	20.7	Mohr-Coulomb	2	34			Water Surface	Automatically Calculated	
Silty Sands and Sandy Silts		17	19.7	Mohr-Coulomb	6	32			Water Surface	Automatically Calculated	
Liquefied Layer		17		Vertical Stress Ratio			0.15	5	Water Surface	Custom	0
Concrete Slab		25		Mohr-Coulomb	700	0			Water Surface	Custom	1

5.8 Groundwater Model

The water table has been modelled at 12.0m below the ground surface at the top of the slope and 0.3m below the surface at the bottom except in the elevated water table conditions.

The elevated water table has been modelled at 8m below the ground surface at the top of the slope and at the ground surface at the bottom of the slope as the gully is expected to flood during a large storm event.

5.9 Loading

Loadings applied to each model are shown in the Table below. The location of loadings may be found in Appendix H.

Table 9: Surcharges to be present in slope profile.

Surcharge	Load	Load Type
Proposed Dwelling	25 kN/m^2	Uniformly Distributed
Deck	5 kN/m^2	Uniformly Distributed

5.10 Supports

The properties of supports modelled are displayed in the Table below.

Table 10: Support properties

Type	Out of plane Spacing	Shear Strength - Static	Shear Strength - Transient	Depth
200 mm SED High Density Timber Pile (with 8mm/m taper)	2.0 m	40 kN	67 kN	3.6 and 5.0m as applicable

5.11 Slope Stability Results

Under existing conditions, the model shows failure arcs below the required FOS up to 11.8m back from the crest of the slope during ULS and SLS conditions. The gravity condition had failure arcs below the required FOS up to 1.4m back from the crest of the slope.

Under the proposed slope cutting and dwelling foundation scenario the gravity and elevated water scenarios meet the required FOS required. The FOS reached for the dwelling under the SLS condition was 1.55 and the FOS reached under ULS conditions was 1.11. Both of these meet the required FOS for their conditions.

The strength loss scenario gave a FOS of 1.002 under ULS conditions.

Table 11 below shows the minimum FOS achieved for the modelled foundation under various seismic loading conditions as specified in Section 5.6 of this report.

Table 11: Worst Case failure plane FOS

Modelled Loading Condition	Minimum Global FOS (Existing)	FOS Reached (Proposed with house and piling)
Gravity Conditions	1.42	>1.5
Gravity Conditions (elevated water table)	1.42	>1.3
Seismic SLS (Serviceability Limit State)	1.28	>1.45
Seismic ULS (Ultimate Limit State)	0.93	>1.05
Liquefied Condition	>1.2	>1.1

5.12 Recommendations

It is proposed the site is cut down 3.0m as per the site development plans.

To improve stability of the slope the following recommendations have been made:

- The dwelling should be setback at least 5.5m from the new top of the slope after cutting down.
- The modelled foundation is based on 200mm diameter piles as per the Engineering Plans with a minimum embedment depth of 5.0m and 3.6m as modelled.
- The rest of the foundation piles will be designed by a suitably qualified engineer to be in accordance with suitable depths as outlined in section 2.4.1 of this report.
- Appropriate vegetation should be planted on the slope as to improve stability and avoid erosion.
- No overland flow paths should be directed onto or towards the slope.
- No undercutting of the slope should be undertaken without due consideration to slope stability.

6 LIMITATIONS

This report does not assess risk of contamination of soils. This report does not provide a foundation design.

Testing portrays a limited percentage of ground conditions at Lot 2 136 State Highway 26 and may not be representative of all soils present on site.

Assessment of the water table depth and moisture content is subject to seasonal variation.

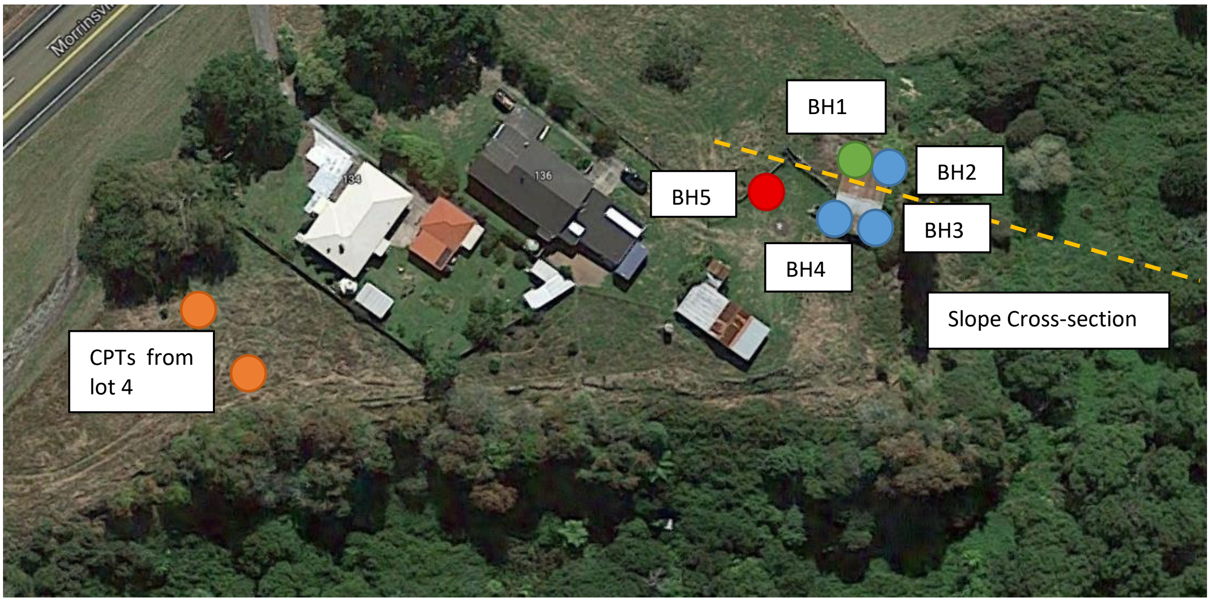
During excavation and construction, the site should be examined by a suitably qualified engineer in order to assess whether the exposed subsoils are compatible with the inferred soil conditions on which the recommendations have been based and potentially further investigation and design rationalisation may be required. Flooding and FFL requirements has not been assessed as part of this stormwater design.

This report has been prepared solely for Martin Cameron, its professional advisors, and local authorities in relation to Lot 2 136 State Highway 26. No liability is accepted for its use for any other purpose or by any other entity. Reliance by other parties or future owners of the property on the information or opinions contained in the report shall be verified with TITUS CIVIL Consulting Engineers.

Should you be in any doubt as to the recommendations of this report it is essential that you discuss these issues with TITUS CIVIL Consulting Engineers prior to proceeding with any work based on this report.

APPENDICES

APPENDIX A - Proposed Site Layout Plan



APPENDIX B - Soil Investigation Bore Logs

TITUS CONSULTING ENGINEERS

Log: **BH1**

Address: Lot 2 State Highway 26
Date: 12/05/2020
Testers: RM

Project №: 11122

Water Table:	Depth (mm):	Geology:	Graphic Log:	Material Description:	Blows /100mm:	Shear Strength (kPa): Undrained:	Remoulded:	Sensitivity:
Not Found	100	Hinuera Formation		Medium SAND, yellowish orange and brown, poorly graded, moist, very loose	0			
	200				0			
	300				0			
	400			SILT with some sand, yellowish grey, low plasticity, moist, stiff	3	91	53	1.7
	500				2			
	600				3			
	700				2			
	800			SILT with some sand, light yellowish brown, low plasticity, moist, stiff	3	91	30	3
	900				2			
	1000				2			
	1100				4			
	1200				6	91	61	1.5
	1300				4			
	1400				4			
	1500			*End of Borehole @1400mm*	5			
	1600				4			
	1700				6			
	1800				6			
	1900				5			
	2000							
	2100							

TITUS CONSULTING ENGINEERS

Log: **BH2**

Address: Lot 2 State Highway 26
Date: 12/05/2020
Testers: RM

Project №: 11122

Water Table:	Depth (mm):	Geology:	Material Description:	Blows /100mm:	Shear Strength (kPa): Undrained:	Remoulded:	Sensitivity:
Not Found	0–100	Hinuera Formation	Medium SAND, yellowish brown, poorly graded, moist, very loose	0			
	100–200			0			
	200–300		Black, organics	0			
	300–400			0			
	400–500			0			
	500–600			0			
	600–700			0			
	700–800			1			
	800–900		Medium SAND, yellowish brown, poorly graded, moist, very loose to loose	2			
	900–1000			1			
	1000–1100		SILT, grey, low plasticity, moist, stiff	3	91	61	1.5
	1100–1200			2			
	1200–1300			2			
	1300–1400			3			
	1400–1500		Medium to coarse SAND with some silt, brown, well graded, moist, medium dense	5			
	1500–1600			5			
	1600–1700			5			
	1700–1800			4			
	1800–1900			5			
	1900–2000						
	2000–2100		*End of Borehole @2000mm*				

TITUS CONSULTING ENGINEERS

Log: **BH3**

Address: Lot 2 State Highway 26
Date: 12/05/2020
Testers: RM
Project №: 11122

Water Table: Not Found
Geology: Hinuera Formation

Depth (mm)	Material Description	Blows /100mm	Undrained (kPa)	Remoulded (kPa)	Sensitivity
100	Fine SAND with minor silt, brown, well graded, moist, very loose	0			
200		0			
300		0			
400	SILT, brownish grey, low plasticity, moist, stiff	2	91	38	2.4
500		1			
600		3			
700	Medium SAND, yellowish brown, poorly graded, moist, loose to medium dense	2			
800		3			
900		2			
1000		5			
1100		3			
1200	Silty fine SAND, light brown, well graded, moist, loose	3			
1300		5			
1400		5			
1500	SILT with some sand, light yellowish brown, low plasticity, moist, very stiff	8	206	53	3.9
1600		5			
1700	Medium to coarse SAND, brownish orange, well graded, moist, medium dense	5			
1800		6			
1900					
2000					
2100	*End of Borehole @2000mm*				

TITUS CONSULTING ENGINEERS

Address: Lot 2 State Highway 26
Date: 12/05/2020
Testers: RM

Log: **BH4**

Project №: 11122

Water Table:	Depth (mm):	Geology:	Material Description:	Blows /100mm:	Undrained:	Remoulded:	Sensitivity:
Not Found	0–100	Hinuera Formation	Medium SAND, brownish orange, poorly graded, moist, very loose	0			
	100–200			0			
	200–300			0			
	300–400			1			
	400–500		Sandy SILT, yellowish brown, low plasticity, moist, stiff	2	84	38	2.2
	500–600			1			
	600–700			2			
	700–800			3	99	69	1.4
	800–900		Silty medium to coarse SAND, dark brownish orange, well graded, moist, very loose to medium dense	3			
	900–1000			2			
	1000–1100			1			
	1100–1200		Silty SAND, greyish brown, well graded, moist, loose	3			
	1200–1300			4			
	1300–1400			5			
	1400–1500			4			
	1500–1600		Coarse SAND, yellowish brown, poorly graded, moist, medium dense to dense	4			
	1600–1700			5			
	1700–1800			5			
	1800–1900			8			
	1900–2000						
	2000–2100		*End of Borehole @2000mm*				

TITUS CONSULTING ENGINEERS

Log: BH5

Address: Lot 2 State Highway 26
Date: 12/05/2020
Testers: RM

Project №: 11122

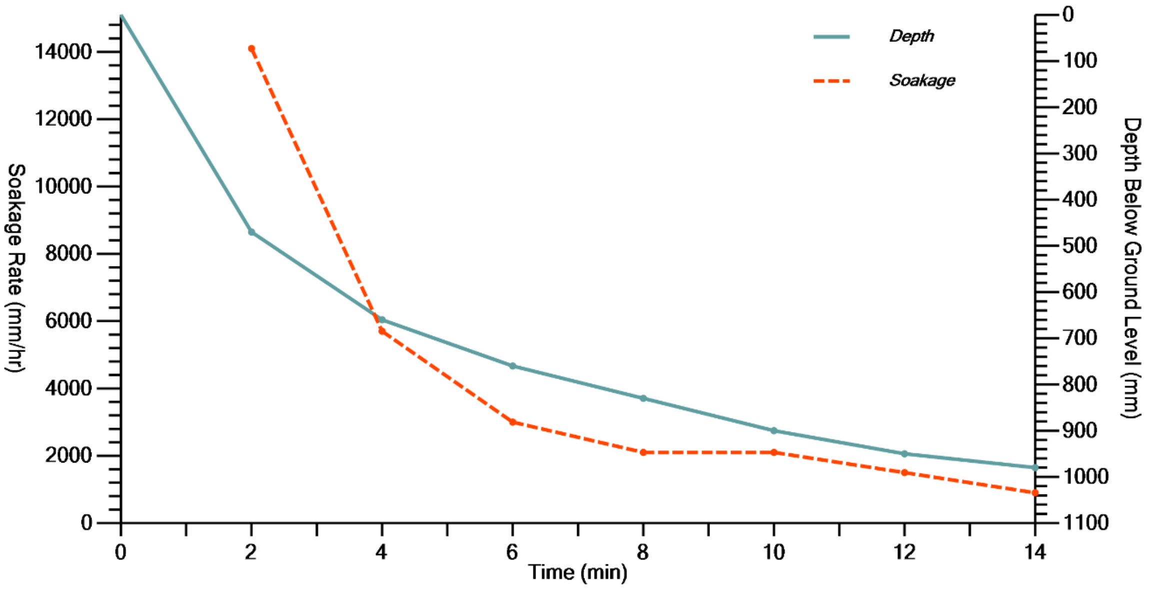
Water Table:	Depth (mm):	Geology:	Graphic Log:	Material Description:	Blows /100mm: (5, 10, 15)	Shear Strength (kPa): Undrained:	Remoulded:	Sensitivity:
Not Found	0–200	Undefined		Topsoil				
	200–500	Hinuera Formation		Medium SAND, yellowish brown, poorly graded, moist				
	500–900			SILT, light brown, low plasticity, moist				
	900–1300			Fine to medium SAND, brownish orange, well graded, moist				
	1300–2000			Medium to coarse SAND, brown, well graded, moist				
	2000			*End of Borehole @2000mm*				

APPENDIX C - Percolation Test

TITUS CONSULTING ENGINEERS

Percolation Test Sheet

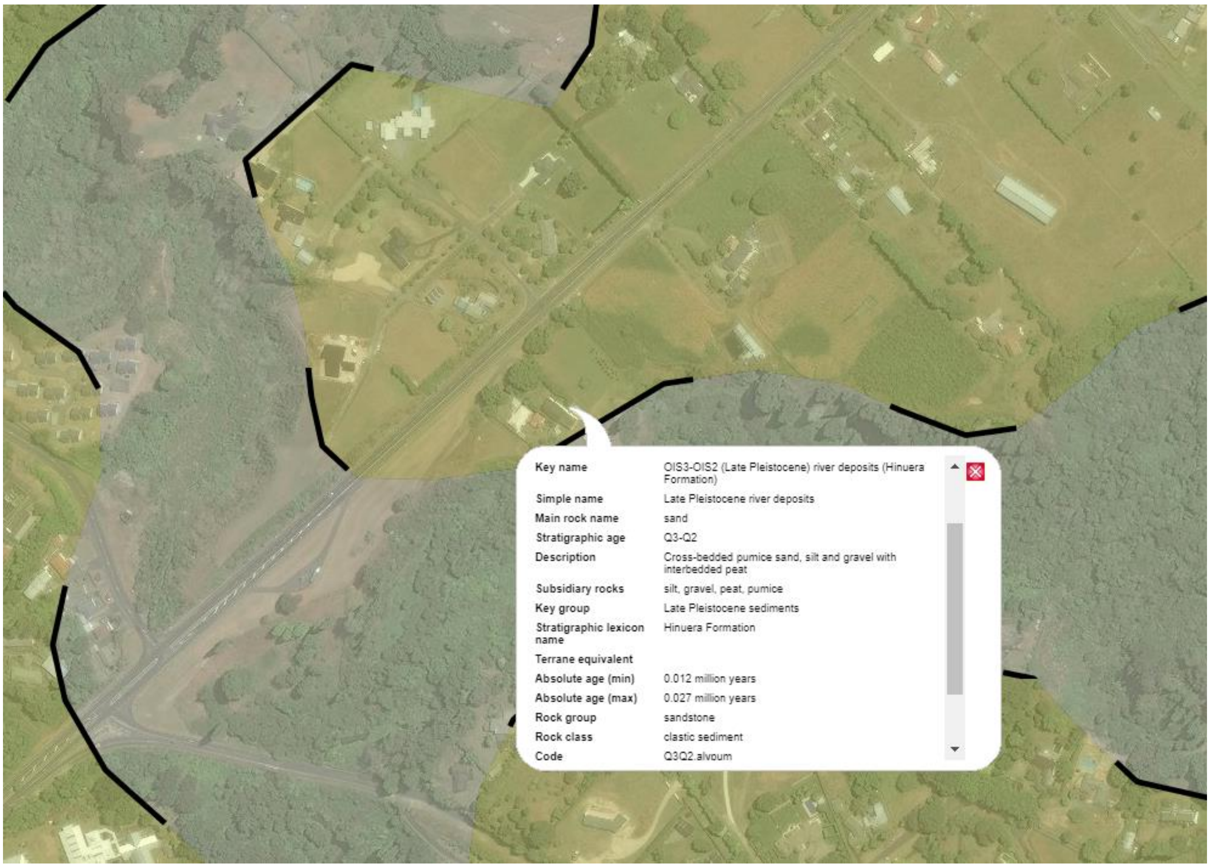
Project ID 11122
Address Lot 2 State Highway 26



Reading	Time Elapsed (min)	Drop (mm)	Soakage Rate (mm/hr)	Refill
1	2	470	14100	
2	4	190	5700	
3	6	100	3000	
4	8	70	2100	
5	10	70	2100	
6	12	50	1500	
7	14	30	900	

Log	BH1
Date	12/05/2020
Staff	RM
BH Depth	1400 mm
Ground Water	Not Encountered
Main Soil Type	SILT
Seasonal Variation	Conservative
Raw Soakage	900 mm/hr

APPENDIX D - Underlying Geology



APPENDIX E - Proposed Stormwater & Wastewater Layout Plans

Attached separately

APPENDIX F - Calculation Sheets

TANK DESIGN CALCULATIONS AND OUTPUTS

Legend	
Existing	Input / Select
Proposed	Answer

Rainfall Location	Event	ARI
Hamilton	Primary	10
	Secondary	100

Catchment	Area (m2)		C
	Existing	Proposed	
Grass	280		0.30
Roof		280	0.95
Concrete			0.90
Gravel			0.70
Other			-
TOTAL	**280**	**280**	
Composite C	0.3	0.95	
Adopted C	**0.30**	**0.95**	

Existing Catchment Characteristics, Time of Concentration (Tc)	
Average grassed surface	0.045
Length of flow path (m)	20.00
Slope (%)	3.00
Tc (min)	10.49

Existing Q(max) (l/s) (interpolated wrt Tc)	2.12
Proposed Q(max) (l/s)	7.89

ARI	10									
Duration(min)	10	20	30	60	120	360	720	1440	2880	4320
Delta t (min)	10	10	30	60	240	360	720	1440	1440	
Delta Q (l/s)	-0.5	-0.3	-0.4	-0.4	-0.3	-0.1	-0.1	0.0	0.0	
Intensity	91.9	69.1	57.1	38.7	23.6	10.6	6.7	4.1	2.5	1.8
Intensity CC	107.2	80.3	66.1	44.7	27.1	12.1	7.6	4.7	2.8	2.0
Existing Q (l/s)	2.1	1.6	1.3	0.9	0.6	0.2	0.2	0.1	0.1	0.0
Proposed Q (l/s)	7.9	5.9	4.9	3.3	2.0	0.9	0.6	0.3	0.2	0.1

ARI	10									
Duration	10m	20m	30m	60m	2h	6h	12h	24h	48h	72h
Depth EX	16.2	24.5	29.6	36.4	42.7	57.3	77.5	93.3	113.4	121.1
Depth CC	17.9	26.8	33.1	44.7	54.3	72.3	91.3	112.1	135.2	142.3
Existing Vol m3	1.4	2.1	2.5	3.1	3.6	4.8	6.5	7.8	9.5	10.2
Proposed Vol m3	4.8	7.1	8.8	11.9	14.4	19.2	24.3	29.8	36.0	37.9

Effective Tank Dia (m)	3.4	effective head m		0.56	Sharp edge					
ARI	10.0	orifice size mm		36.2	Cd	0.62				
Duration	10m	20m	30m	60m	2h	6h	12h	24h	48h	72h
Volume in	4.8	7.1	8.8	11.9	14.4	19.2	24.3	29.8	36.0	37.9
Volume out	1.3	2.5	3.8	7.6	15.3	45.8	91.5	183.0	366.0	549.0
Volume store	3.5	4.6	**5.0**	4.3	-0.8	-26.5	-67.2	-153.2	-330.1	-511.2

Tank Calc

			Capacities	
Height	2.80			
Volume (m3)	25.00		Run off reuse (m3)	20.0
Volume MAX (m3)	4.99		Detention (m3)	5.0
Diameter (m)	3.37			

APPENDIX G - Key Do's & Don'ts for the Householder

DO

- Minimise your water use.
- Minimise the length of showers.
- Use showers in preference to baths.
- Use bio-degradable soaps and cleaners
- Check all your cleaning products to see if they are suitable for septic tanks.
- Minimise use of strong toilet cleaners.
- Scrape all plates and dishes to remove as much fat and grease as possible. Clean with paper towels and place in the rubbish.
- Report/fix all leaking taps as soon as possible.
- Use phosphate free/low phosphorus based laundry detergents.

DO NOT

- Don't pour any toxic/strong chemicals (paint, oil, grease, paint thinners, pesticides down any drains).
- Don't flush any products down the toilet, other than standard toilet paper.
- Don't discard any drugs down the sink or toilet.
- Don't use strong cleaners.
- Don't tip chlorine cleaners or disinfectant based products into the system.
- Don't use huge amounts of cleaners.
- Don't use chemical drain cleaning products.
- Don't do all your laundry on one day.
- Don't install in-sink garbage grinders. If a grinder exists, don't discharge high volumes of scraps, especially carbohydrates or fats/oils down it.
- Don't put coffee grounds down the sink.

APPENDIX H - Maintenance, Operation and Planting Recommendations

Attached separately

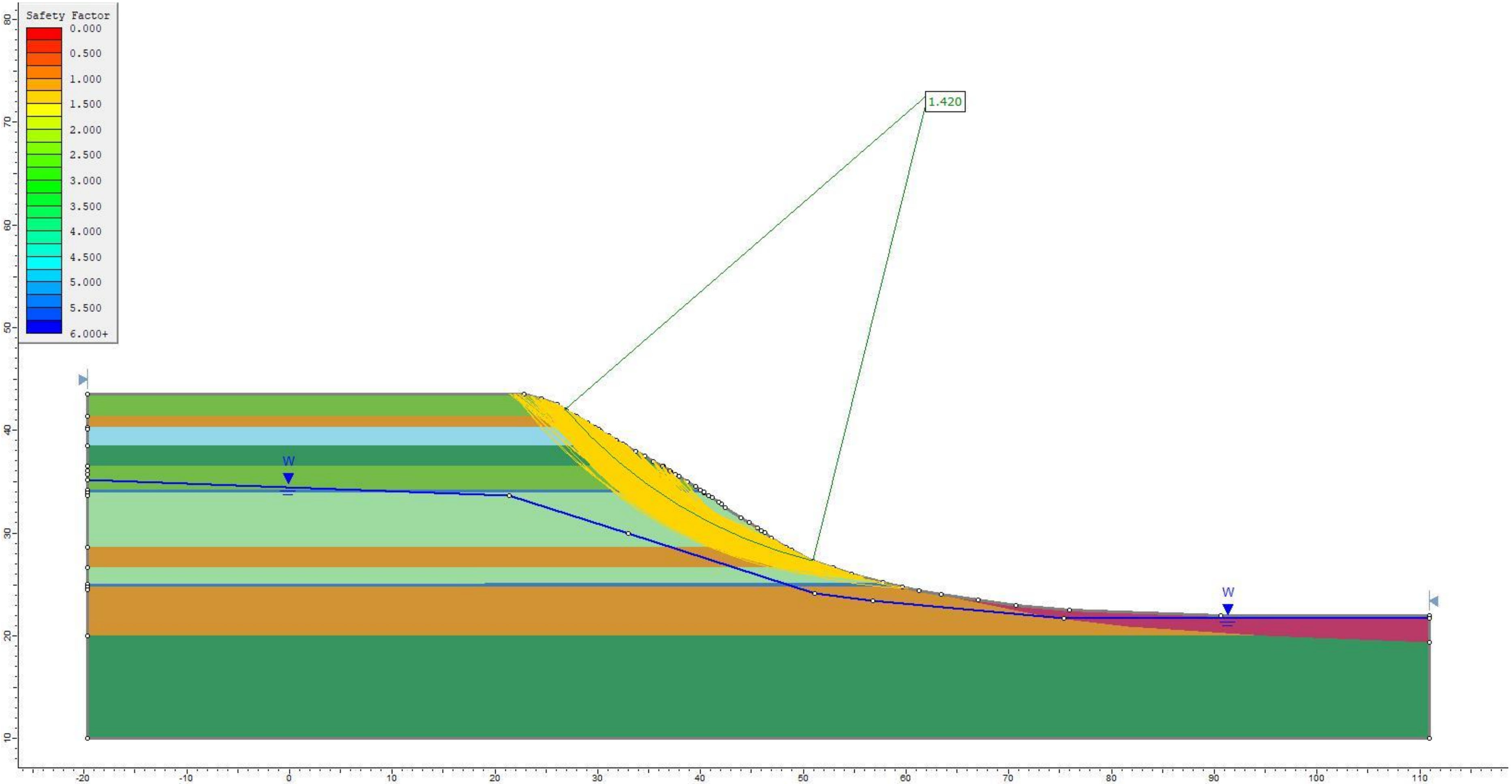
APPENDIX I – Liquefaction Assessment Results

Attached separately

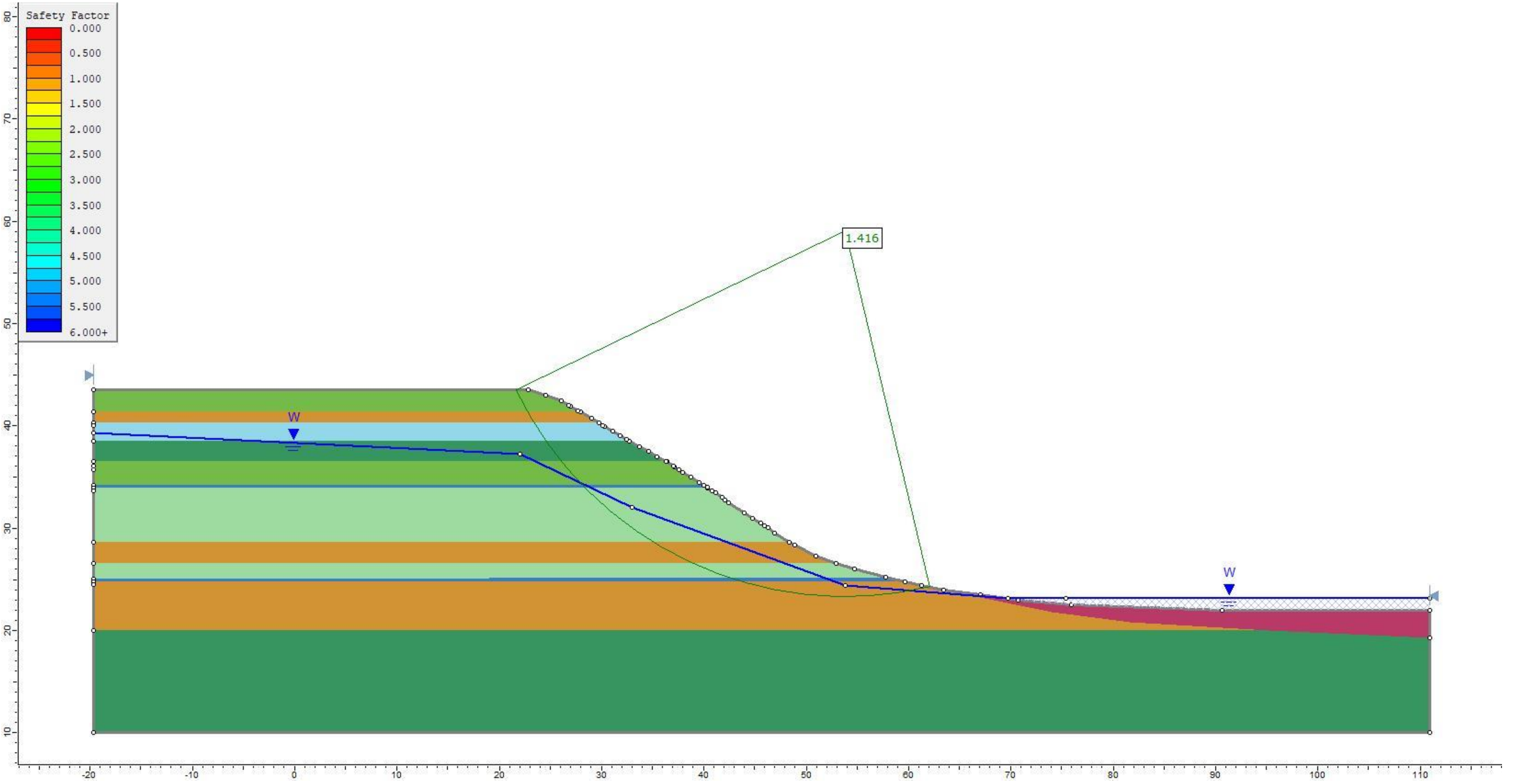
APPENDIX J – Slope Stability Models

EXISTING SLOPE (Modelled in 2020)

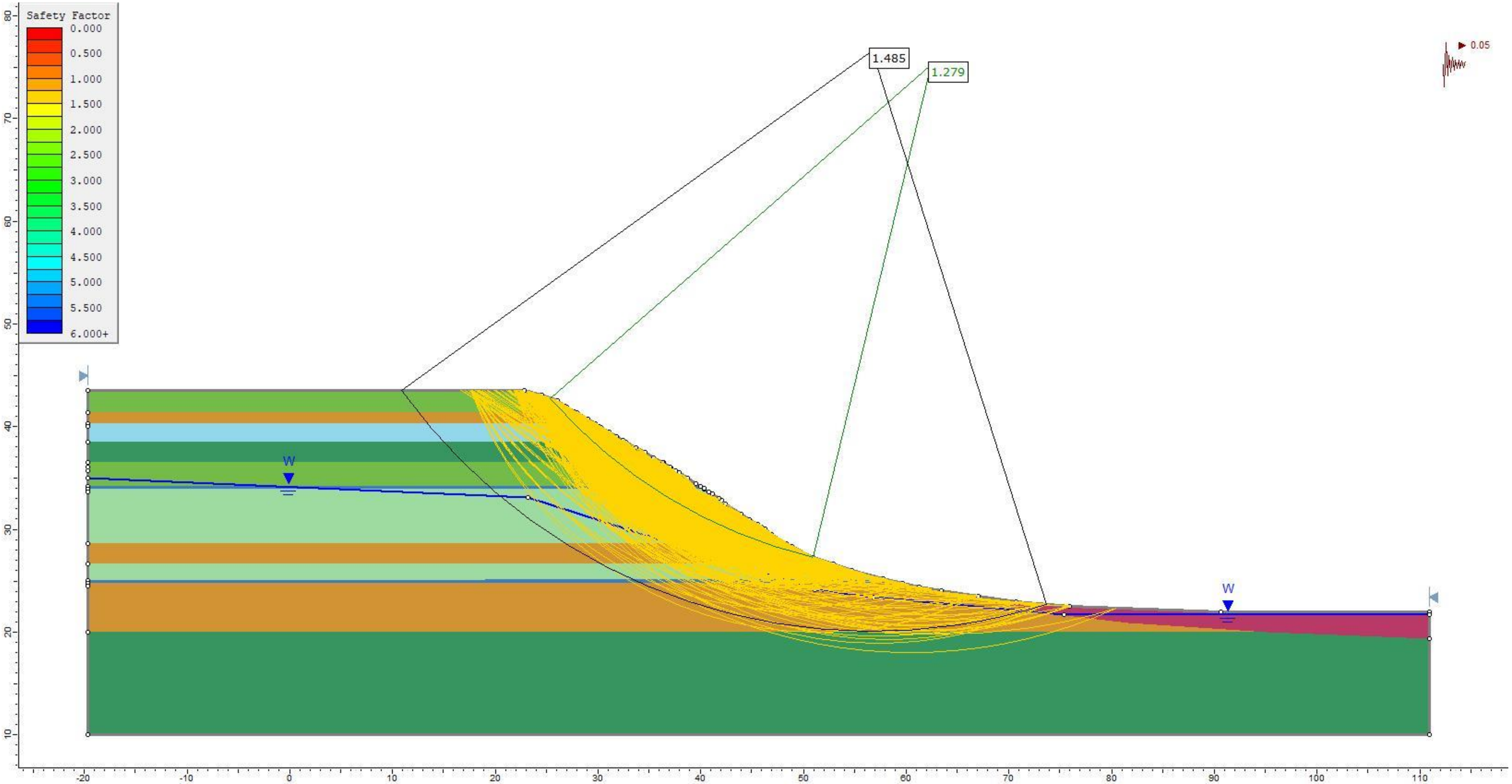
Material Name	Color	Unit Weight (kN/m3)	Sat. Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Ru
Topsoil		17	19.7	Mohr-Coulomb	2	28		Water Surface	Automatically Calculated	
Coarse Dense Sands		18	20.7	Mohr-Coulomb	2	40		Water Surface	Automatically Calculated	
Coarse Sands		18	20.7	Mohr-Coulomb	2	38		Water Surface	Automatically Calculated	
Medium to Coarse Sands		18	20.7	Mohr-Coulomb	2	37		Water Surface	Automatically Calculated	
Medium Sands		18	20.7	Mohr-Coulomb	2	36		Water Surface	Automatically Calculated	
Medium to Fine Sands		19	21	Mohr-Coulomb	2	35		Water Surface	Automatically Calculated	
Fine Silts		17	19.7	Mohr-Coulomb	3	32		Water Surface	Automatically Calculated	
Holocene Sediments		13	14	Mohr-Coulomb	0	32		Water Surface	Automatically Calculated	
Free Draining hardfill		18	20.7	Mohr-Coulomb	2	37		Water Surface	Automatically Calculated	
Concrete Retaining Wall		25		Undrained	650		Constant	None		0
Concrete Floor		24		Mohr-Coulomb	30	40		None		0
Liquefied Layer		18	20.7	Undrained	5		Constant	Water Surface	Automatically Calculated	



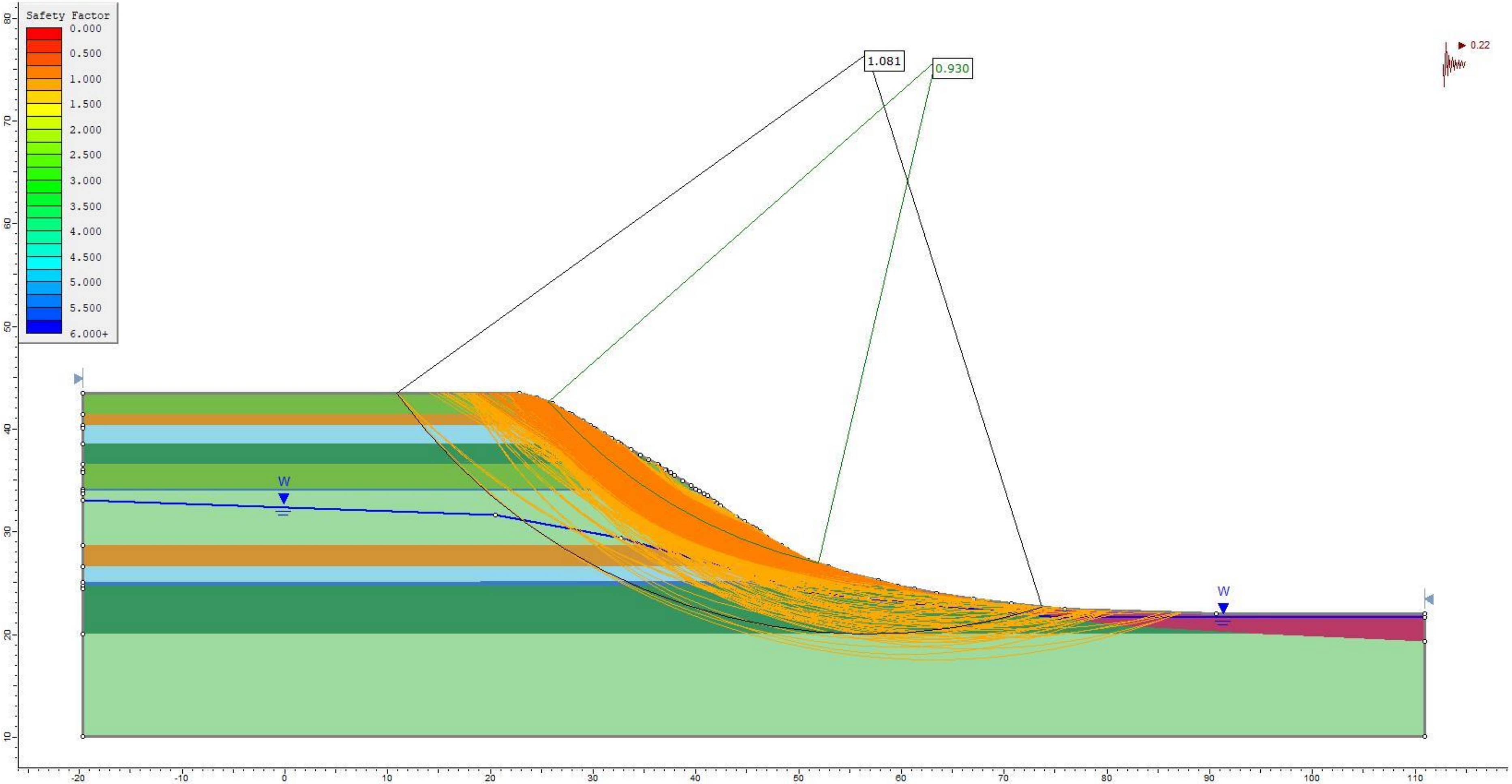
SLIDE 2018 model – Existing Site – Gravity Conditions – FOS required: 1.5



SLIDE 2018 model – Existing Site – Gravity Conditions –Elevated water table - FOS required: 1.3

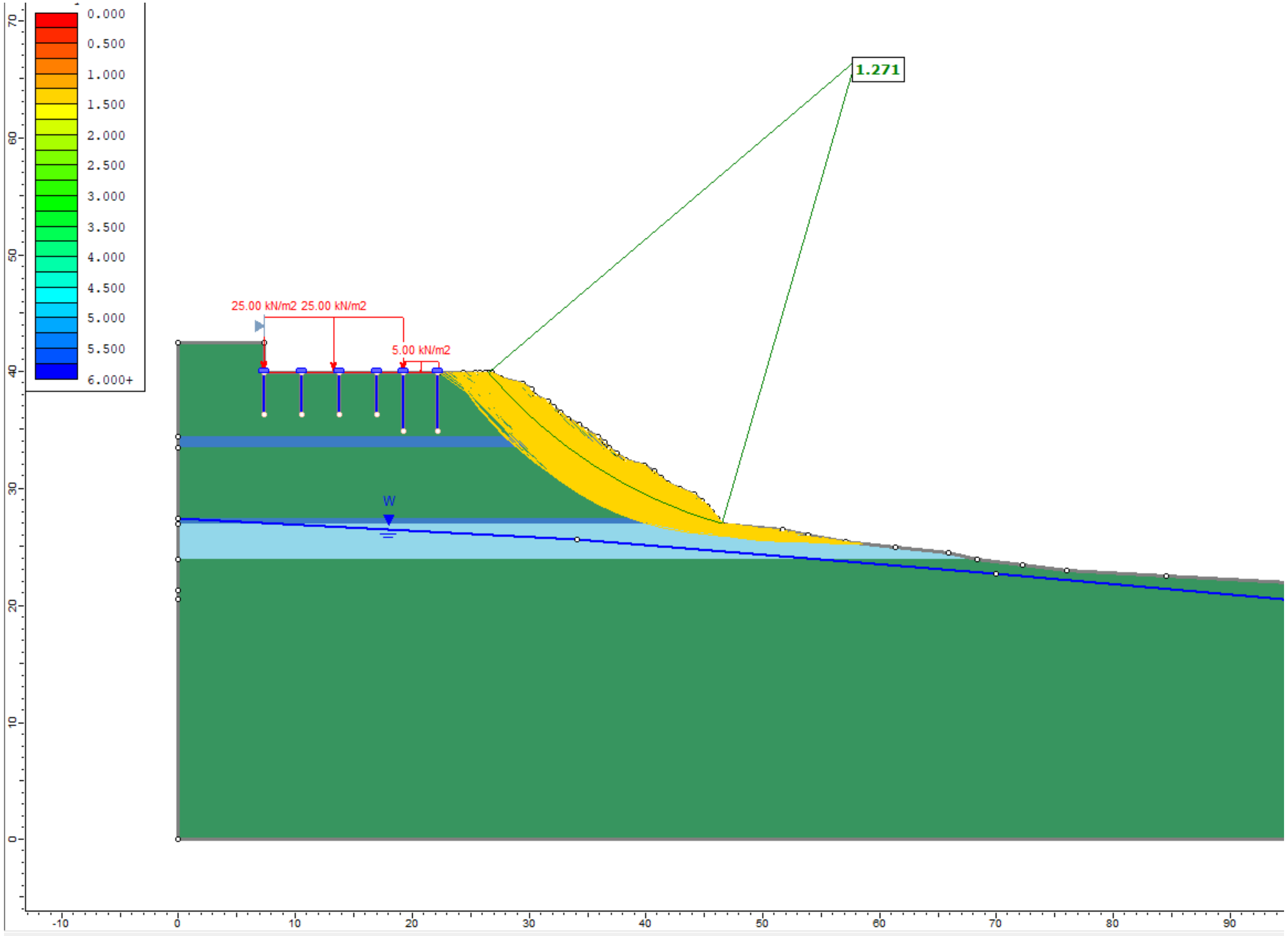


SLIDE 2018 model – Existing Site - SLS (Serviceability Limit State) – FOS required: 1.5

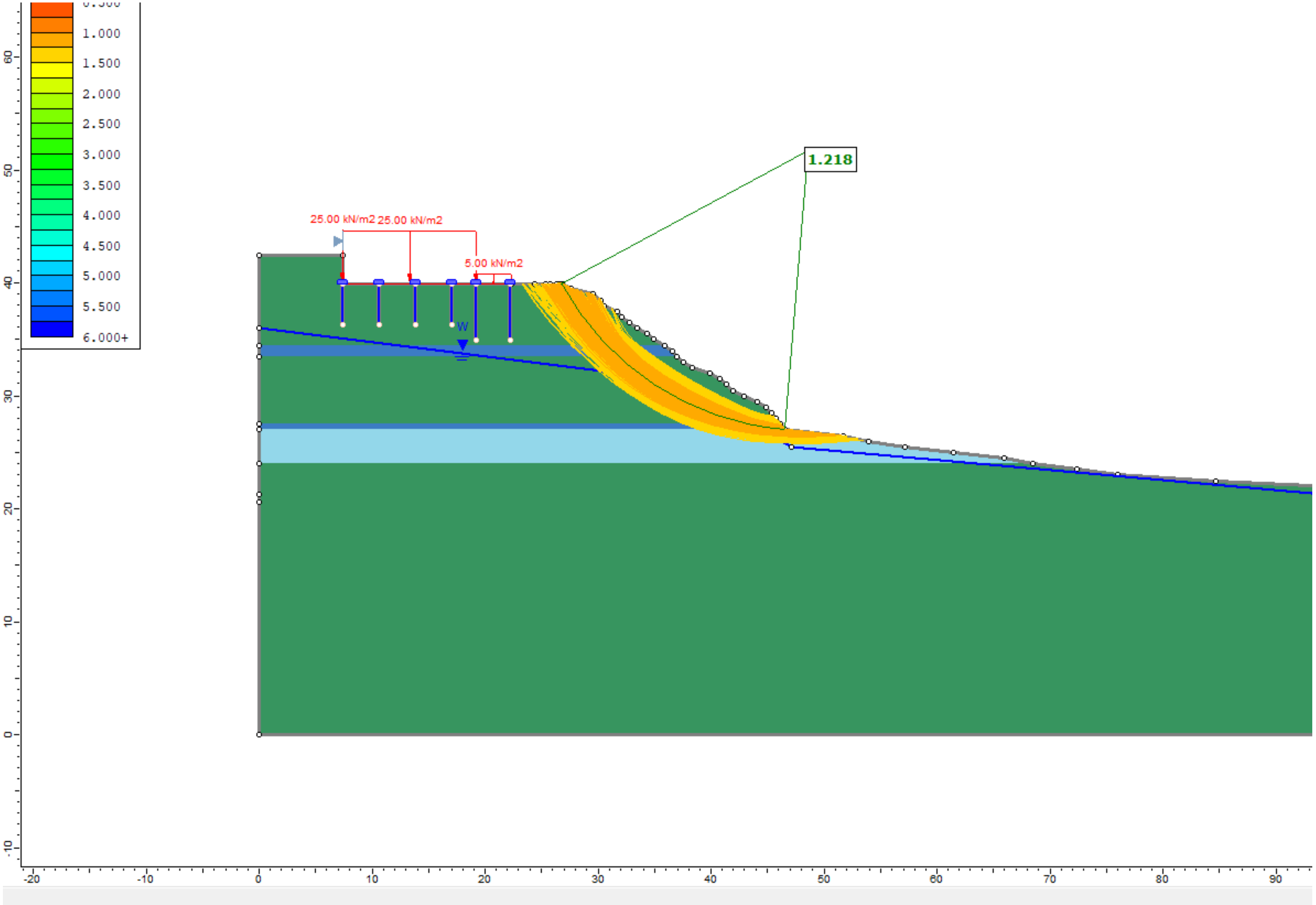


SLIDE 2018 model – Existing Site - ULS (Ultimate Limit State) – FOS required: 1.1

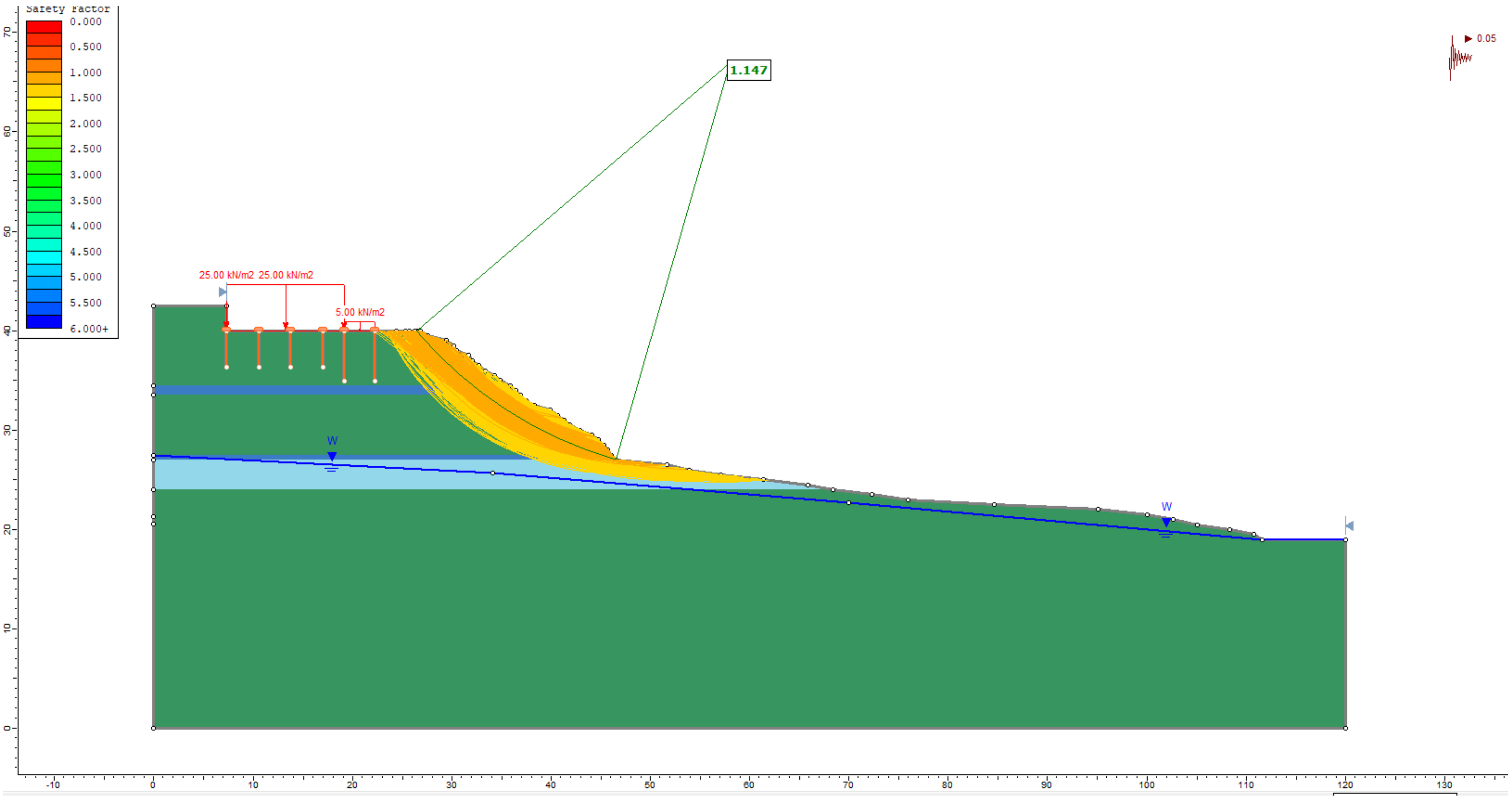
PROPOSED DWELLING (Modelled in 2021)



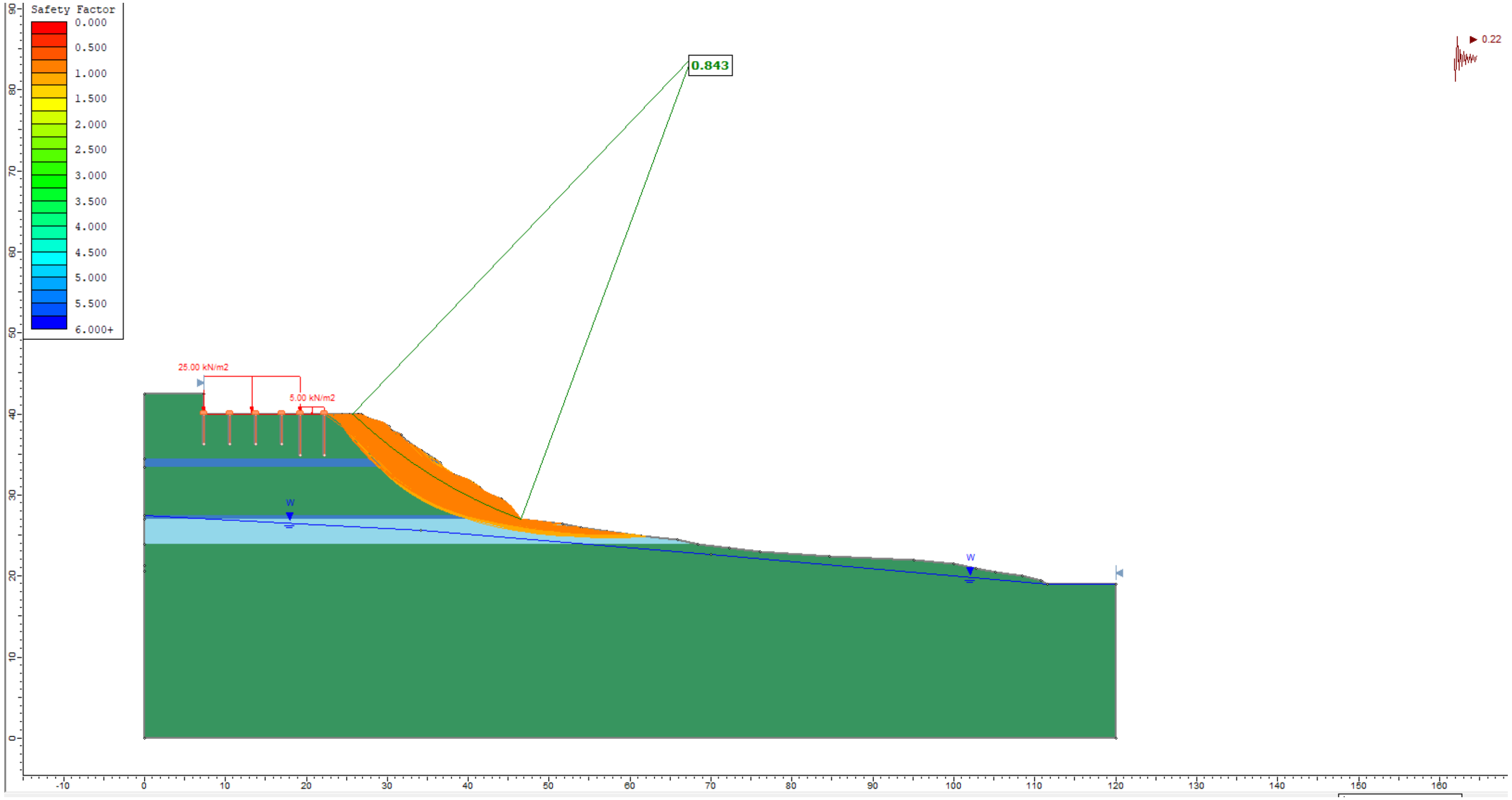
SLIDE model – Proposed Dwelling – Gravity Conditions – FOS required: 1.5



SLIDE model – Proposed Dwelling – Gravity Conditions –Elevated water table - FOS required: 1.3

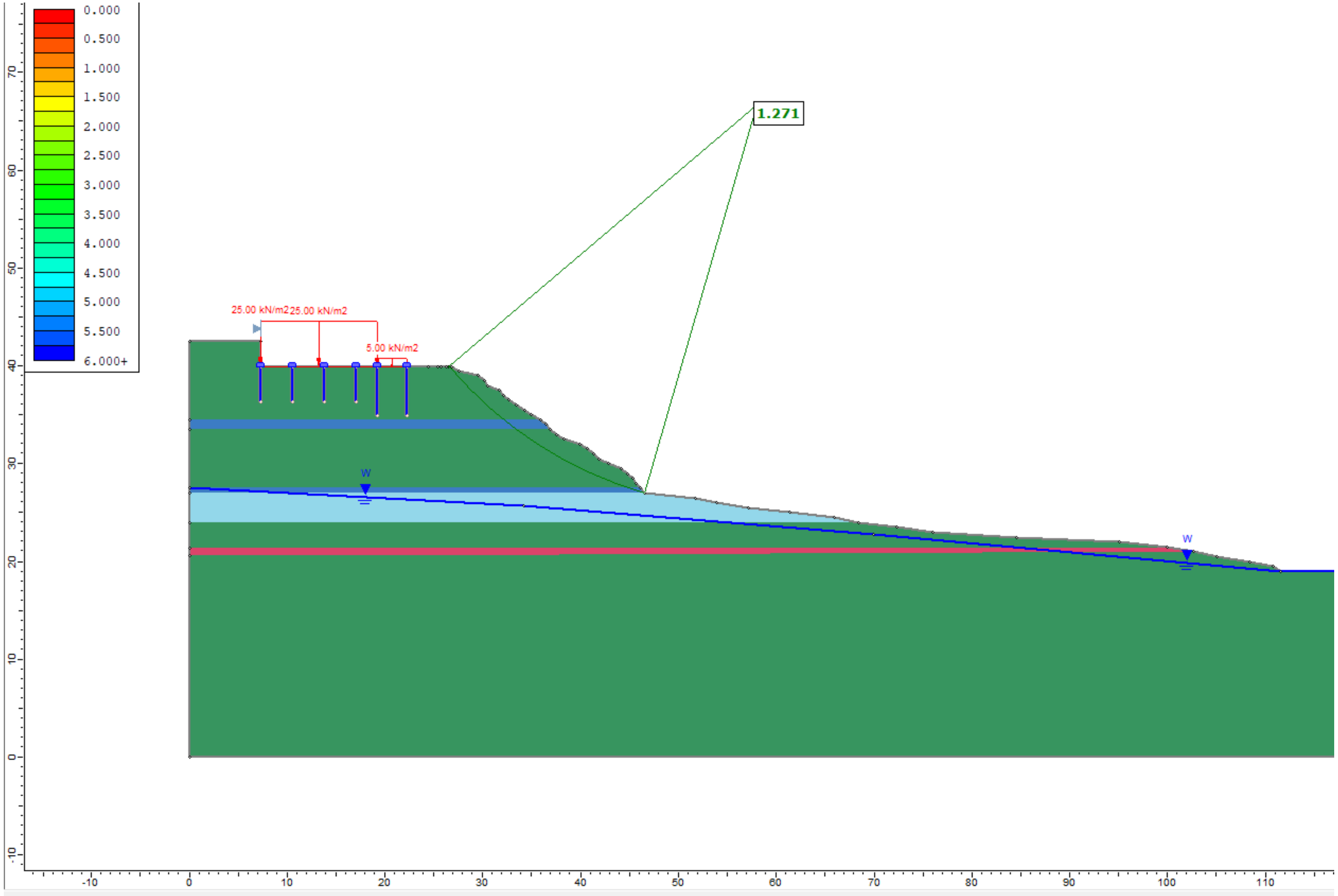


SLIDE model – Proposed Dwelling - SLS (Serviceability Limit State) – FOS required: 1.45



SLIDE model – Proposed Dwelling - ULS (Ultimate Limit State) – FOS required: 1.05

STRENGTH LOSS / Liquefied condition



SLIDE model – Proposed Dwelling – Strength Loss

Building Code Clause(s) B1

PRODUCER STATEMENT – PS1 – DESIGN

ISSUED BY: Micius Consultants Limited
(Design Firm)

TO: Steelhaus
(Owner/Developer)

TO BE SUPPLIED TO: Hamilton City Council
(Building Consent Authority)

IN RESPECT OF: New Residence Proposed Cameron Residence (Project#: 210176)
(Description of Building Work)

AT: 136 State Highway 26
(Address)

Town/City: Newstead (Address) **LOT** 2 **DP** ... **SO** ...

Hamilton City Council
Te kaunihera o Kirikiriroa
BUILDING UNIT APPROVED
BC Number - DD007.2021.00043914.001

We have been engaged by the owner/developer referred to above to provide:

Structural Engineering. Scopes refer Micius Structural Calculation #210176
(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 B1/VM1, B1/AS1 or
(verification method/acceptable solution)

☐ Alternative solution as per the attached schedule

The proposed building work covered by this producer statement is described on the drawings titled:

"New Residence Proposed Cameron Residence" by Steelhaus and numbered Cover, A01-A13;
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 foundation & other structural elements by others
(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)

I, Yang Benson Zhang am: ☒ CPEng # 1009967
(Name of Design Professional)

I am a member of: ☒ Engineering New Zealand and hold the following qualifications: CPEng, IntPE, MEngSt(Hons), BE

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 ACE New Zealand: ☐

SIGNED BY Yang Benson Zhang (Signature)
(Name of Design Professional)

14/02/2022

ON BEHALF OF Micius Consultants Limited Date
(Design Firm)

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.

CERTIFICATE OF DESIGN WORK
MEMORANDUM FROM LICENSED BUILDING PRACTITIONER

engineering new zealand

Section 30C and Section 45, Building Act 2004

The Building

Street address	136 State Highway 26		
Suburb	N/A	Town/city	Newstead
Postcode	3286	Building consent no.	N/A

The Owner

Name(s)	Martin Cameron		
Email	N/A	Phone	N/A
Address	N/A		

Hamilton City Council
BUILDING UNIT APPROVED
BC Number - DD007.2021.00043914.001

Basis for providing this memorandum

I am providing this memorandum in my role as the **specialist** designer who carried out or supervised specific Primary structure elements of restricted building work (RBW) design work as described in this memorandum. Other designers will provide memoranda covering the remaining RBW design work. Refer also to the attached PS1.

Identification of restricted building work (RBW) design work

I, Yang Benson Zhang carried out or supervised the following RBW design work:

Primary structure: B1

Design work that is RBW		Description (as required) and reference to plans and specifications	Carried out or supervised
Foundations and subfloor framing	✗	~~N/A~~	~~Carried out~~
Retaining walls	✗	~~N/A~~	~~Carried out~~
Beams	✓	Roof trusses, lightsteel/structural steel beams and post, lightsteel midfloor framing (Steelhaus Drawings and Detailing Ref#J000608)	Carried out
Portal	✗	~~N/A~~	~~Carried out~~
Bracing	✗	~~N/A~~	~~Carried out~~
Other (primary)	✓	Lightsteel Wall Framings (Steelhaus Drawings and Detailing Ref#J000608)	Carried out

Note: SED = Elements subject to Specific Engineering Design outside of the scope of NZS3604:2011, unless otherwise noted.

Initial ______ Date 14/02/2022

Waivers and modifications

Are waivers or modifications of the Building Code required? No

If yes, please provide details of the waivers or modifications:

Building Code clause	Waiver/modification required

Issued by

Name	Yang Benson Zhang	**Design entity/company**	Micius Consultants Limited
Chartered status	CPEng	**Chartered no.**	1009967
Email	benson.zhang@micius.co.nz	**Website**	www.micius.co.nz
Phone (daytime)	021 876206	**Phone (after hours)**	N/A
Mobile	021 876206		
Postal address	191-195 Onehunga Mall, Onehunga, Auckland		
Physical address	191-195 Onehunga Mall, Onehunga, Auckland		

Declaration

I, Yang Benson Zhang, LBP state that I have applied the skills and care reasonably required of a competent design professional in carrying out or supervising the RBW described in this memorandum and that based on this, I certify that the RBW described in this memorandum:

- complies with the Building Code; or
- ~~complies with the Building Code subject to any waiver or modification of the Building Code described in this memorandum.~~

Signature 14/02/2022 **Date**

Hamilton City Council
Te kaunihera o Kirikiriroa

BUILDING UNIT
APPROVED

BC Number - DD007.2021.00043914.001

Civil & Structural Engineering Consultants

Building Consents Team
Local Authority

Date: 2 November 2021

Attention: Building Consents

RE: B2 Durability Compliance for Proposed Dwelling at 136 State Highway 26, Newstead

Dear Sir/Madam,

The purpose of this letter is to demonstrate how compliance with Clause B2 (Durability) of the Building Code will be achieved for the above project. We can confirm that for specifically designed structural elements that are included within our design documentation:

Material	Means of Compliance	Details
Lightguage steel framing	Alternative Solution	The lightgauge steel framing have been specified as per B1/AS1 acceptable solution "NASH STANDARD PART 2 Light Steel-frame Buildings". The framing supplier has provided a NZ Steel 50-year durability statement for above-floor steel framing.
Mild steel structure	Alternative Solution	Protection for mild steel has been specified in accordance with SNZ TS 3404 – Durability requirements for steel structures and components and AS/NZS2312 – Guide to the protection of structural steel against atmospheric corrosion by the use of protective coatings. This guide works on a time to first maintenance basis and assumes on-going maintenance. Refer to the attached maintenance plan (optional but recommended).

We trust you find above points in order but should you have any further queries please do not hesitate to contact us directly.

Kind Regards

2/11/2021

Benson Zhang
CPEng, IntPE(NZ), CMEngNZ, MEngSt(Hons), BE
Senior Structural Engineer/Director

Hamilton City Council
BUILDING UNIT
APPROVED
BC Number - DD007.2021.00043914.001

MICIUS
Civil & Structural Engineering Consultants

136 State Highway 26, Newstead

STRUCTURAL MAINTENANCE SCHEDULE

This schedule of ongoing inspection and maintenance of structural elements shall be included with the O&M manuals and provided to the Owner/Body Corporate and building managers.

Inspection/Maintenance timeframe and item	
(a) Half-yearly	Wash down all exposed steelwork that is not in a fully interior environment including: • Veranda steelwork • Steel carpark structure (beams, columns, braces etc) • Deck and balcony steelwork • Exposed façade steelwork, both primary and secondary structure • Sub-ground floor mild-steel structures such as beams.
(b) 5-yearly	Inspect and repair sealant that encloses structural mild-steel components and/or timber with mild-steel fixings.
(c) 10-yearly	Check exposed timber fixings for corrosion, repair as required.
	Inspect/replace sealant that encloses structural mild-steel components and/or timber with mild-steel fixings. This will typically include sealants around the perimeter of precast panels. **Note that 10 years is the expected useful life for many sealants.**
	Check all exposed steelwork that is not in a fully interior environment for signs of corrosion. Repair protective coatings as required.
(d) 25-yearly	Inspect samples of structural steel that is hidden from view but not enclosed within a vapour barrier, and repair protective coatings as necessary. A typical example is a veranda with built-in steelwork. (Such steelwork should typically have duplex protective coatings). Inspection may typically require removal of claddings and/or the drilling of holes for borescope access. Repair as required.
	Inspect all exposed, external timber. Repair as required.
	Inspect all exposed, external reinforced concrete for signs of spalling. Repair as required.
Following seismic shaking > SLS1 event	Inspections and repair as per b), c) and d) above.

Hamilton City Council

BUILDING UNIT APPROVED

BC Number - DD007.2021.00043914.001

Hamilton City Council
BUILDING UNIT APPROVED
BC Number - DD007.2021.00043914.001

NEW ZEALAND STEEL

DURABILITY STATEMENT

For GALVSTEEL® (galvanised steel) manufactured by New Zealand Steel Limited and used for structural building elements

GALVSTEEL® material, when used for purlins, girts, battens or framing will have a durability of 50 years when used and maintained as referred to below.

Scope: GALVSTEEL® used to manufacture building components such as wall, roof, floor and sub-framing, purlins, girts and battens used in buildings built in New Zealand covered by the New Zealand Building Code with a 50 year design life.

This Durability Statement does not apply to Axxis® steel for framing used in building types and situations covered by the "Axxis® steel for framing" Durability Statement. This Durability Statement also excludes any other building components manufactured from GALVSTEEL® or other metal-coated products including nail plates and any composite wall, roof or floor systems. Composite systems include Galvsteel® embedded in concrete panels.

The above statements are subject to the following:

1. Specifications

Zinc coating weight;	Z275 (275g/m²) or Z450 (450g/m²)
Complying with;	AS 1397:2011.
Steel grade;	G250, G300, G450, G500 or G550.
Steel thickness range;	0.50-2.25 mm.
Bend diameter;	G250, G300; ≥ 2T. G450, G500, G550; ≥6T (where T = total coated thickness).

2. Fixing, Handling and Maintenance according to the following publications:

a) New Zealand Steel Limited, Specifiers and Builders Guide, and Installers Guide (refer www.nzsteel.co.nz for most current version).
b) NZ Metal Roof & Wall Cladding, Code of Practice, (refer www.metalroofing.org.nz for most current version and updates).
c) AS/NZS 2312:2014 (Incorporating Amendment 1;2017) Guide to the protection of structural steel against atmospheric corrosion by the use of protective coatings.
d) Instructions and literature published by individual purlin and steel framing manufacturers.
e) NASH Handbook Best Practice for Design and Construction of Residential and Low-Rise Steel Framing.
f) NASH N11 House Insulation Guide – version 2.2 April 2012.

Axxis® is a registered trademark of New Zealand Steel Limited.

3. Additional Fixing, Handling and Design Requirements.

a) Bottom plate details must ensure that the bottom plate remains dry in service and is not subject to water ingress from internal or external sources. Damp-proof course (DPC) must be used and be at least 10mm wider than the building element.
b) Separation methods as described within NZMRM Code of Practice 2.7 are required between any steel structural building element and incompatible materials which include, but are not limited to, timber treated with copper based preservatives, concrete, brickwork, copper and other dissimilar metals and also materials which may be moisture bearing during the life of the building.
c) Subfloor framing requires a minimum finished ground level clearance of 150mm. Ventilation must comply with the requirements of NZS3604:2011 Timber-framed buildings, including ground cover when specified. A minimum of 3500mm²/m² of floor space and a maximum of 7000mm²/m² of floor space is required.
d) Site storage conditions must ensure that the GALVSTEEL® is kept dry when in a stacked condition.
e) Prior to installation of external and internal lining the Galvsteel® must be clean, dry, free of corrosion, clear of debris and swarf.
f) During storage and erection the material should be kept as dry as possible and the building closed in as soon as practicable to limit exposure to the elements. As a guide, this should be within 3 weeks in marine or geothermal environments and with-in 12 weeks in moderate environments from the delivery of the Galvsteel® material to site.
g) GALVSTEEL® must be carried and not dragged when being moved.
h) GALVSTEEL® must not be exposed to spatter from any welding activities.
i) Wall wraps and roof underlays must comply with the requirements of NZS2295:2006 Pliable, permeable building underlays for use with steel framing.

4. Environment.

Initially the macroclimate in which the building is situated needs to be determined. Table 2 is broken down into broad geographical regions of New Zealand. Within the regions the corrosivity is further defined by the distance to the nearest coast, harbour or estuary.

For aggressive industrial environments either externally or internally, or buildings subject to heavy geothermal influence, expected corrosion rates and recommended coatings will need to be determined on a case by case basis using New Zealand Steelwork Corrosion Coatings Guide HERA Report R4-133:2005 [d].

5. Building Types

This statement classifies six different building situations where structural steel may be used (N.B. one building may contain more than one of these situations);

a) Residential/Dry
   Steelwork located in a dry internal environment, with an effective thermal break between external cladding and the structure, such as a fully enclosed office, an apartment building or a domestic house. This includes structures that are lined with building wrap and have internally controlled environments such as commercial shops and malls.
b) Internal
   Steelwork located in a damp or humid environment, with no effective thermal break between the external cladding and structure. For structures such as storage sheds, garages and workshops which are typically closed when not in use. These structures are distinguished in the following two cases;
   - Damp
     Steelwork located in a damp internal environment where condensation may occur, where the structure may be in an open sunny location (i.e. when the structure is exposed to the sun and not under any form of cover). This is for structures such as exhibition halls, vehicle depots and warehouses.

- High Humidity
  Steelwork located in an internal high humidity environment with some pollution, where the structure may be in a humid and shaded location (i.e. when the shed is under a tree shaded from the sun). This is for structures such as food processing plants, breweries and dairies. Steel in subfloor spaces is included in this building type.

c) Open Front
Steelwork located near permanent openings (such as near doors or windows that remain open under operating conditions), and may be exposed to the prevailing winds. For structures such as open front lean-to, gable structure closed in on three sides or warehouses with large openings. This building type has two options, which are only applicable to the internal steelwork close to the openings as defined in Section 5.5 of reference [d].

- Protected
  Structures that are protected from the wind coming off the closest sea.
- Open
  Structures that are open and exposed to the prevailing wind coming off the closest sea.

d) Awning
Steelwork that is exposed to the wind but is protected from the rain located in an open sided structure such as carports or structures closed in on one side only. The equivalent reference [b] designation is “Sheltered”. The corrosion rate of this building type and that of “Open Front; Open” are identical.

6. Coating Systems

The following coating systems are referenced in Table 2 of this document, alternative solutions are also available and may be identified by reference to HERA Report R4-133:2005 [d], or AS/NZS2312:2002 [c] or by discussions with paint suppliers or coatings specialists.

Table 1

<table>
<tr><th rowspan="2">System</th><th rowspan="2">Surface Preparation</th><th colspan="3">1st Coat</th><th colspan="3">2nd Coat</th><th colspan="3">3rd Coat</th><th rowspan="2">Total nominal DFT [3] (µm)</th></tr>
<tr><th>Type</th><th>PRN [1]</th><th>Nominal DFT [2] (µm)</th><th>Type</th><th>PRN[1]</th><th>Nominal DFT [2] (µm)</th><th>Type</th><th>PRN[1]</th><th>Nominal DFT [2] (µm)</th></tr>
<tr><td>P1</td><td rowspan="3">Degrease, wash and dry</td><td>Acrylic dispersion paint</td><td></td><td>40</td><td>Acrylic dispersion paint[4]</td><td></td><td>40</td><td></td><td></td><td></td><td>80</td></tr>
<tr><td>P2</td><td>Galvanised iron acrylic primer</td><td></td><td>40</td><td>Acrylic dispersion paint[4]</td><td></td><td>40</td><td></td><td></td><td></td><td>80</td></tr>
<tr><td>P3[5]</td><td>Etch primer</td><td></td><td>12</td><td>Acrylic elastomeric</td><td></td><td>350</td><td></td><td></td><td></td><td>362</td></tr>
<tr><td>P4[5]</td><td rowspan="2">Sweep abrasive blast</td><td rowspan="2">Polyamide cured epoxy primer</td><td rowspan="2">C10</td><td rowspan="2">75</td><td rowspan="2">High build epoxy</td><td rowspan="2">13</td><td rowspan="2">200</td><td>Acrylic 2-pack</td><td>C33</td><td>50</td><td>325</td></tr>
<tr><td>P5[5]</td><td>Polyurethane gloss</td><td>C26</td><td>50</td><td>325</td></tr>
</table>

Notes on Table 1

[1]PRN: Paint reference number as given in appendix C of reference [c].
[2]DFT; coating dry film thickness.
[3]The total nominal DFT does not include the galvanised coating thickness.
[4]Contact the coating supplier for feedback on the appropriate acrylic paint for its intended use. For example, for internal high humidity locations it is recommended to use acrylic enamel at the specified nominal DFT.
[5]P3, P4 and P5 coatings must be applied by a professional coating applicator to achieve the required durability performance.

7. Maintenance

Maintenance is necessary when the galvanised coating ceases to provide sacrificial protection to the steel base, or where the appearance is no longer aesthetically acceptable to the owner.

Rust staining or the growth of rust spots usually indicates the breakdown of galvanised coating. At the first sign of breakdown, the surface should be treated with an appropriate maintenance coating system. All maintenance should be carried out in accordance with AS/NZS 2312:2002 (Incorporating Amendment No. 1) [c] and HERA Report R4-133:2005 [d].

Regular inspections of the steel work and maintenance at the first signs of a breakdown in the galvanised coating will extend the durability of the sections.

8. Recommended coating systems to achieve 50 year durability.

Table 2 shows the recommended coating system to achieve 50 year durability for the different building conditions in the various marine environments throughout New Zealand.

9. Contacting New Zealand Steel

It is important you contact the Technical Manager at New Zealand Steel on 0800 100 523 if you require specialist advice, clarification or assessment in relation to the use of Galvsteel® within the scope of this Durability Statement. If you believe there is an issue with the durability of Galvsteel® used for a project within the scope of this Durability Statement, you are urged to advise New Zealand Steel as soon as you become aware of the issue and before proceeding with any project still under construction.

10. References

a) El Sarraf, R. and Hicks, S. – Extending the Durability Performance of Galvsteel® using a Protective Coating System, (HERA) Structural Systems Technical Report SSTR-001 2008.
b) NZS 3404 Part 1, Steel Structures Standard 2009; Standards New Zealand.
c) AS/NZS 2312:2014 (Incorporating Amendment No. 1;2017), Guide to the protection of structural steel against atmospheric corrosion by the use of protective coatings.
d) Clifton, G.C. and El Sarraf, R. New Zealand Steelwork Corrosion Coatings Guide (HERA Report R4-133) 2005.
e) Compliance Document for New Zealand Building Code – Clause E2 – External Moisture
f) Durability Statement – Axxis® Steel for Framing

Disclaimer

This edition of the Galvsteel® used for structural building elements Durability Statement supersedes all previous editions. It is important to check you have the latest edition of the Durability Statement by referring to www.nzsteel.co.nz or contacting New Zealand Steel Limited on 0800 100 523.

Table 2

Corrosion map to NZS3404.1	ISO 9223	Typically	Location	Characterised by	Residential /Dry	Internal		Open front		Awning
						Damp	High humidity	Protected	Open	
Seaspray	C5	Within 200m of breaking surf	West coast, South Island	Heavy salt deposits, almost constant smell of salt spray in the air.	1	3	4	4	4	4
		Within 100m of breaking surf	West coast, North Island		1	3	4	4	4	4
		Within 50m of breaking surf	Other coasts		1	3	4	4	4	4
		200m up to 500m or more inland from breaking surf. In the immediate vicinity of calm salt water such as harbour foreshores.	West coast, South Island	Medium salt deposits, Frequent smell of salt in the air.	1	3	4	4	4	4
	C4	50m up to 500m or more inland from breaking surf. In the immediate vicinity of calm salt water such as harbour foreshores.	All other coasts		1	1	3	4	4	4
		500m to 1km from breaking surf. In the immediate vicinity of calm salt water such as estuaries.	West coast of both islands and South coast of South Island.	Little salt deposits, occasional smell of salt in the air.	1	1	3	4	4	4
4	C3	500m to 1km from breaking surf. In the immediate vicinity of calm salt water such as estuaries.	East coast of both islands, South coast of North Island and all harbours.		1	1	2	3	4	4
		1km to 20 km from salt water	West coast of both islands and South coast of South Island	Minor salt deposits, no smell of salt in the air.	1	1	3	4	4	4
		1km to 5km from salt water	East coast of both islands, South coast of North Island and all harbours.		1	1	2	3	4	4
Zone 2	C2	20km to 50km from salt water.	West coast of both islands and South coast of South Island	No marine influence.	1	1	1	2	3	3
		5km to 50km from salt water	East coast of both islands, South coast of North Island and all harbours.		1	1	1	2	3	3
Zone 3		Inland more than 50km from salt water.	Both Islands		1	1	1	1	1	1

Note; all environments may be extended inland by prevailing winds and local conditions.

Key

1	Z275
2	Z275 and one of the paint systems P1 – P5 applied when new, <u>or</u> Z450.
3	Z275 and one of (P3, P4 or P5) applied when new, <u>or</u> P1 or P2 applied when new and recoated every 15 years.
4	Z275 and one of (P3, P4 or P5) applied when new and then recoated every 15 years

Hamilton City Council

BUILDING UNIT APPROVED

BC Number - DD007.2021.00043914.001

PRODUCER STATEMENT – PS1 DESIGN

Building Code Clause(s):	B1,	Job number: 12843
ISSUED BY: ***(Engineering Design Firm)***	Arnold and Johnstone 2015 Ltd	
TO: ***(Client)***	Martin Cameron	
TO BE SUPPLIED TO: ***(Building Consent Authority)***	Hamilton City Council	
IN RESPECT OF: ***(Description of building work))***	New build	
AT: ***(Address)***	136 State Highway 26, Newstead, Hamilton	
LEGAL DESCRIPTION	Lot 2 DP 556335	

Hamilton City Council
Te kaunihera o Kirikiriroa
BUILDING UNIT
APPROVED
BC Number - DD007.2021.00043914.001

We have been engaged by Martin Cameron to provide:

SED Bracing, SED Retaining Walls, SED Foundations, SED Beams, SED Posts

in respect of the requirements of the Clause(s) of the Building Code specified above for part only, as specified in the attached Schedule, of the proposed building work.

In this document SED means "Specific Engineering Design".

The design carried out by Arnold and Johnstone 2015 Ltd has been prepared in accordance with:

✓ compliance documents issued by the Ministry of Business, Innovation & Employment (Verification method /acceptable solution): B1/VM1, B1/VM4

The proposed building work covered by this producer statement is described in the drawings specified in the attached Schedule, together with the specification, and other documents set out in the attached Schedule.

On behalf of Arnold and Johnstone 2015 Ltd, and subject to:

- site verification of the following design assumptions:
  - In accordance with Titus Consulting Engineers report dated 2.2.21 Rev E, Ref 11122
- all proprietary products meeting their performance specification requirements;

I believe on reasonable grounds that:

- 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 specified above; and that
- the persons who have undertaken the design have the necessary competence to do so.

I recommend the CM3 level of construction monitoring.

I, Raman Forbes, am:

- CPEng number 228942
- and hold the following qualifications: B.E. (Hons)

Arnold and Johnstone 2015 Ltd holds a current policy of Professional Indemnity Insurance no less than $200,000.

Arnold and Johnstone 2015 Ltd is not a member of ACE New Zealand.

SIGNED BY: Raman Forbes

(Signature): Date: 21/2/22

ON BEHALF OF: Arnold and Johnstone 2015 Ltd

Note: This statement has been prepared solely for Hamilton City Council and shall not be relied upon by any other person or entity. Any liability in relation to this statement accrues to Arnold and Johnstone 2015 Ltd only. As a condition of reliance on this statement, Hamilton City Council accepts that the total maximum amount of liability of any kind arising from this statement and all other statements provided to Hamilton City Council in relation to this building work, whether in tort or otherwise, 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.

Hamilton City Council

BUILDING UNIT
APPROVED

BC Number - DD007.2021.00043914.001

Job Number: 12843
Job Address: 136 State Highway 26, Newstead, Hamilton

SCHEDULE TO PS1

Please include an itemised list of all referenced documents, drawings, or other supporting materials in relation to this producer statement below:

- Certificate of Design Work, Construction Monitoring Schedule, Structural Maintenance Schedule, B2 Letter in Lieu - Design
- Engineering Calculations: Structural Calculations
- Architectural Drawing Set: Sheet 3, 7 to 19 dated 17/2/22, all rev A

Limited Scope of Engagement

We have been engaged by Martin Cameron to provide services in respect of the requirements of the Clause(s) of the Building Code specified above for the following parts of the proposed building work:

SED Bracing, SED Retaining Walls, SED Foundations, SED Beams, SED Posts

Hamilton City Council

BUILDING UNIT
APPROVED

BC Number - DD007.2021.00043914.001

Job Number: 12843
Job Address: 136 State Highway 26, Newstead, Hamilton

GUIDANCE ON USE OF PRODUCER STATEMENTS

Information on the use of Producer Statements and Construction Monitoring Guidelines can be found on either the **ACE New Zealand** or **Engineering New Zealand** websites.

Producer statements were first introduced with the Building Act 1991. The producer statements were developed by a combined task committee consisting of members of the New Zealand Institute of Architects (NZIA), Institution of Professional Engineers New Zealand (now Engineering New Zealand), Association of Consulting and Engineering New Zealand (ACE NZ) in consultation with the Building Officials Institute of New Zealand (BOINZ). The original suite of producer statements has been revised at the date of this form to ensure standard use within the industry.

The producer statement system is intended to provide Building Consent Authorities (BCAs) with part of the reasonable grounds necessary for the issue of a Building Consent or a Code Compliance Certificate, without necessarily having to duplicate review of design or construction monitoring undertaken by others.

PS1 DESIGN: Intended for use by a suitably qualified independent engineering design professional in circumstances where the BCA accepts a producer statement for establishing reasonable grounds to issue a Building Consent;
PS2 DESIGN REVIEW: Intended for use by a suitably qualified independent engineering design review professional where the BCA accepts an independent design professional's review as the basis for establishing reasonable grounds to issue a Building Consent;
PS3 CONSTRUCTION: Forms commonly used as a certificate of completion of building work are Schedule 6 of NZS 3910:2013 or Schedules E1/E2 of NZIA's SCC 20112
PS4 CONSTRUCTION REVIEW: Intended for use by a suitably qualified independent engineering construction monitoring professional who either undertakes or supervises construction monitoring of the building works where the BCA requests a producer statement prior to issuing a Code Compliance Certificate.
This must be accompanied by a statement of completion of building work (Schedule 6).
The following guidelines are provided by ACE New Zealand and Engineering New Zealand to interpret the Producer Statement.

Competence of Engineering Professional

This statement is made by an engineering firm that has undertaken a contract of services for the services named, and is signed by a person authorised by that firm to verify the processes within the firm and competence of its personnel.
The person signing the Producer Statement on behalf of the engineering firm will have a professional qualification and proven current competence through registration on a national competence-based register such as a Chartered Professional Engineer (CPEng).
Membership of a professional body, such as Engineering New Zealand provides additional assurance of the designer's standing within the profession. If the engineering firm is a member of ACE New Zealand, this provides additional assurance about the standing of the firm.
Persons or firms meeting these criteria satisfy the term "suitably qualified independent engineering professional".

Professional Indemnity Insurance

As part of membership requirements, ACE New Zealand requires all member firms to hold Professional Indemnity Insurance to a minimum level.
The PI Insurance minimum stated on the front of this form reflects standard practice for the relationship between the BCA and the engineering firm.

Professional Services during Construction Phase

There are several levels of service that an engineering firm may provide during the construction phase of a project (CM1-CM5 for engineers3).
The BCA is encouraged to require that the service to be provided by the engineering firm is appropriate for the project concerned.

Requirement to provide Producer Statement PS4

BCAs should ensure that the applicant is aware of any requirement for producer statements for the construction phase of building work at the time the building consent is issued. No design professional should be expected to provide a producer statement unless such a requirement forms part of Arnold and Johnstone 2015 Ltd's engagement.

Refer Also:

1 Conditions of Contract for Building & Civil Engineering Construction NZS 3910: 2013
2 NZIA Standard Conditions of Contract SCC 2011
3 Guideline on the Briefing & Engagement for Consulting Engineering Services (ACE New Zealand Engineering New Zealand 2004)
4 PN01 Guidelines on Producer Statements

www.acenz.org.nz
www.engineeringnz.org

Hamilton City Council
BUILDING UNIT
APPROVED
BC Number - DD007.2021.00043914.001

CONSTRUCTION MONITORING SCHEDULE

ARNOLD & JOHNSTONE
CONSULTING STRUCTURAL ENGINEERS

Hamilton City Council

BUILDING UNIT APPROVED

BC Number - DD007.2021.00043914.001

SCHEDULE OF MONITORING FOR

Address: 136 State Highway 26, Newstead, Hamilton

Job number: 12843

We propose that at least the following site monitoring is undertaken to Engineering New Zealand/ACENZ CM3:

No.	Item of monitoring	Timeframe	To be monitored by
1.	Driven timber piles	Foundation pre-pour (Contractor to provide copy of pile sets to Engineer for review and approval prior to cutting off pile head to height.)	Engineer
2.	Foundation beams, pads and slabs	Pre-pour	Engineer
3.	Internal beams and connections	While all connections are clearly visible, pre-line and prior to building in to such an extent that remediation work could not be carried out.	Engineer
4.	Internal posts and connections	While all connections are clearly visible, pre-line and prior to building in to such an extent that remediation work could not be carried out.	Engineer
5.	External beam and connections	While all connections are clearly visible, pre-line and prior to building in to such an extent that remediation work could not be carried out.	Engineer
6.	External posts and connection	While all connections are clearly visible, pre-line and prior to building in to such an extent that remediation work could not be carried out.	Engineer
7.	Concrete block work	Pre-pour	Engineer
8.	Bracing walls	Pre & post-lining	Engineer

Notes:

a) The above items of monitoring are the minimum required to enable Arnold and Johnstone 2015 Ltd to issue a PS4 – Producer Statement Construction Review for the specific engineering design items.

b) The above items of monitoring do not cover work constructed in accordance with NZS 3604:2011, for which monitoring is to be undertaken by the Building Consent Authority.

c) The Contractor/Builder is to provide Arnold and Johnstone 2015 Ltd at least 24 hours' notice of the requirement for monitoring. The above timeframes are indicative, the Engineer and Contractor are to agree the timing of monitoring prior to work commencing on site.

d) A copy of this monitoring schedule is to be held on site during the works, and the Contractor/Builder is to provide reasonable and safe access to enable works to be monitored according to the schedule.

e) The above schedule does not necessarily represent the actual number of monitorings to be undertaken. The number of monitorings will depend on the construction method, sequence of the works and whether or not unforeseen conditions or difficulties are encountered on site.

Hamilton City Council

BUILDING UNIT
APPROVED

BC Number - DD007.2021.00043914.001

CERTIFICATE OF DESIGN WORK MEMORANDUM FROM LICENSED BUILDING PRACTITIONER

Hamilton City Council

BUILDING UNIT APPROVED

BC Number - DD007.2021.00043914.001

SECTION 30C AND SECTION 45, BUILDING ACT 2004

THE BUILDING			
Street Address	136 State Highway 26		
Suburb	Newstead	Town/City	Hamilton
Postcode			

THE OWNER			
Name(s)	Martin Cameron		
Email	martin@cameron.co.nz	Phone	0224444334
Address	136 State Highway 26, Newstead, Hamilton		

BASIS FOR PROVIDING THIS MEMORANDUM

I am providing this memorandum in my role as the specialist designer who carried out or supervised specific Primary structure elements of restricted building work (RBW) design work as described in this memorandum. Other designers will provide memoranda covering the remaining RBW design work. Refer also to the attached PS1.

IDENTIFICATION OF RESTRICTED BUILDING WORK (RBW) DESIGN WORK

I, Raman Forbes carried out or supervised the following RBW design work:

PRIMARY STRUCTURE: B1

Design work that is RBW		Description (as required) and reference to plans and specifications	Carried out or supervised
Foundations	✓	SED Foundations as per foundation plan Sheet 13, 16 & 17	Supervised
Retaining walls	✓	SED Retaining walls as per drawings and retaining wall plan Sheet 15 to 17	Supervised
Beams	✓	SED beams and posts as per beam layout plan Sheet 3, 7 to 12, 18 & 19	Supervised

Job Number: 12843
Job Address: 136 State Highway 26, Newstead, Hamilton

Portal	✗	Not applicable	Not applicable
Bracing	✓	SED basement floor bracing as per bracing plan Sheet 14	Supervised
Other (primary)	✗	Not applicable	Not applicable

Note: SED = Elements subject to Specific Engineering Design outside of the scope of NZS3604:2011, unless otherwise noted.

WAIVERS AND MODIFICATIONS

Are waivers or modifications of the Building Code required? No

If yes, please provide details of the waivers or modifications:

ISSUED BY

Name	Raman Forbes	**Design entity/company**	Arnold and Johnstone 2015 Ltd
Chartered status	Chartered Professional Engineer	**Chartered no.**	228942
Email	raman@ajeng.co.nz	**Website**	www.ajeng.co.nz
Phone (daytime)	07 5780921	**Phone (after hours)**	021 1298920
Mobile	021 1298920		
Postal address	55 Girven Road, Mt Maunganui		
Physical address	55 Girven Road, Mt Maunganui		

DECLARATION

I, Raman Forbes, LBP state that I have applied the skills and care reasonably required of a competent design professional in carrying out or supervising the RBW described in this memorandum and that based on this, I certify that the RBW described in this memorandum complies with the Building Code.

Signature______________________________ Date __ 21/2/22 ________________

Hamilton City Council

BUILDING UNIT
APPROVED

BC Number - DD007.2021.00043914.001

Job Number: 12843
Job Address: 136 State Highway 26, Newstead, Hamilton

NEW BUILD AT 136 STATE HIGHWAY 26, NEWSTEAD, HAMILTON

This schedule of ongoing inspection and maintenance of structural elements shall be included with the Operations and Maintenance manuals and provided to the Owner/Body Corporate and building managers.

Inspection/maintenance timeframe and item	
(a) Half-yearly	Wash down all exposed steelwork that is not in a fully interior environment including: • Steel Carpark structure (beams, columns, braces etc) • Deck and balcony steelwork • Exposed façade steelwork, both primary and secondary structure
(b) 5 yearly	• Inspect and repair sealant that encloses structural mild-steel components and/or timber with mild-steel fixings
(c) 10 yearly	• Check exposed timber fixings for corrosion, repair as required. • Inspect/replace sealant that encloses structural mild-steel components and/or timber with mild-steel fixings. This will typically include sealants around the perimeter of precast panels. Note that 10 years is the expected useful life for many sealants • Check all exposed steelwork that is not in a fully interior environment for signs of corrosion. Repair protective coatings as required.
(d) 25 yearly	• Inspect samples of structural steel that is hidden from view but not enclosed within a vapour barrier, and repair protective coatings as necessary. A typical example is a veranda with built-in steelwork. (Such steelwork should typically have duplex protective coatings). Inspection may typically require removal of claddings and/or the drilling of holes for borescope access. Repair as required. • Inspect all exposed, external timber. Repair as required. • Inspect all exposed, external reinforced concrete for signs of spalling or cracking. Repair as required.
(e) Following fit-out or alterations	Not applicable.
(f) Following seismic shaking > SLS1 event	Not applicable.

Hamilton City Council

BUILDING UNIT
APPROVED

BC Number - DD007.2021.00043914.001

Job Number: 12843
Job Address: 136 State Highway 26, Newstead, Hamilton

LETTER IN LIEU – DESIGN

ARNOLD & JOHNSTONE
CONSULTING STRUCTURAL ENGINEERS

To the Building Official,
Hamilton City Council
New build at 136 State Highway 26, Newstead, Hamilton

COMPLIANCE WITH BUILDING CODE CLAUSE B2 – DURABILITY

The purpose of this letter is to demonstrate how compliance with Clause B2 (Durability) of the Building Code will be achieved for the above project. We can confirm that for specifically designed structural elements that are included within our design documentation:

Material	Means of Compliance	Details
Reinforced concrete	BS2/AS1	Concrete cover to reinforcing has been selected in accordance with NZS3101, Part 1, Section 3
Structural timber	BS2/AS1	Timber treatment has been selected in accordance with Table 1A of B2/AS1
Mild steel structure	Acceptable Solution	Protection for mild steel has been specified in accordance with SNZ TS 3404 – Durability requirements for steel structures and components and AS/NZS2312 – Guide to the protection of structural steel against atmospheric corrosion by the use of protective coatings. This guide works on a time to first maintenance basis and assumes on-going maintenance.

Yours faithfully,

Raman Forbes

For and on behalf of

Arnold and Johnstone 2015 Ltd

Hamilton City Council

BUILDING UNIT
APPROVED

BC Number - DD007.2021.00043914.001

Job Number: 12843
Job Address: 136 State Highway 26, Newstead, Hamilton

136 State Highway 26, Hamilton
Proposed Cameron Residence
Structural Calculations

For Martin Cameron

3 December 2021

Ref: 12843

Hamilton City Council

BUILDING UNIT
APPROVED

BC Number - DD007.2021.00043914.001

ARNOLD & JOHNSTONE
CONSULTING STRUCTURAL ENGINEERS
TAURANGA | HAMILTON

PROJECT TITLE:	136 State Highway 26, Hamilton Proposed Cameron Residence	**PROJECT No:**	12843
CLIENT:	Martin Cameron	**ENGINEER:**	NAM
CALC TITLE:	Structural Calculations	**DATE:**	03/12/2021

<table>
<tr><td rowspan="1">SCOPE:</td><td colspan="3">Complete the structural calculations for the proposed Cameron Residence at 136 State Highway 26, Hamilton. The following SED items have been completed by Arnold & Johnstone:

▪ Basement Bracing Design
▪ Retaining Wall Design
▪ Pile Design
▪ Footing Beam Design
▪ Floor Slab Design
▪ Ground Floor Beam Design
▪ Block Lintel Design
▪ Deck Bearer Design

We note the structural design of elements above the basement floor level including the roof structure, support frames, steel wall framing and steel floor joists has been completed by Micius Consultants Limited.</td></tr>
<tr><td>CALCULATION METHOD:</td><td colspan="3">Hand calculations
Excel spreadsheets
Space Gass</td></tr>
<tr><td rowspan="15">DESIGN PARAMETERS:</td><td colspan="3">General</td></tr>
<tr><td>Importance Level</td><td>IL2</td><td>Normal structure</td></tr>
<tr><td>Design Life</td><td>50 years</td><td>IL2 Building</td></tr>
<tr><td>Location</td><td>Hamilton</td><td></td></tr>
<tr><td colspan="3">Seismic</td></tr>
<tr><td>Hazard Factor</td><td>Z = 0.16</td><td>Hamilton</td></tr>
<tr><td>Return Period Factor</td><td>R = 1.0</td><td>1/500 years</td></tr>
<tr><td>Near Fault Factor</td><td>N(T,D) = 1.0</td><td>No near faults</td></tr>
<tr><td>Site Subsoil Class</td><td>Type D</td><td>Deep or soft soil</td></tr>
<tr><td>Period</td><td>T = 0.4s</td><td>Both directions</td></tr>
<tr><td colspan="3">Wind</td></tr>
<tr><td>Wind Region</td><td>A7</td><td>Hamilton</td></tr>
<tr><td>Terrain Category</td><td>TC2</td><td>Open terrain</td></tr>
</table>

REFERENCES:	NZS1170:2002	Structural Design Actions
	NZS3101:2006	Concrete Structures Standard
	NZS3404:1997	Steel Structures Standard
	NZS3603:1993	Timber Structures Standard
	NZS3604:2011	Timber-framed Buildings
	NZS4230:2004	Design of reinforced concrete masonry structures
	Reference has also been made to specific manufacturer information throughout the structural calculations. Reference has also been made to the Engineering Assessment and Design Report by Titus Consulting Engineers Project No. 11122 dated 2 February 2021	

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1. BASEMENT BRACING DESIGN

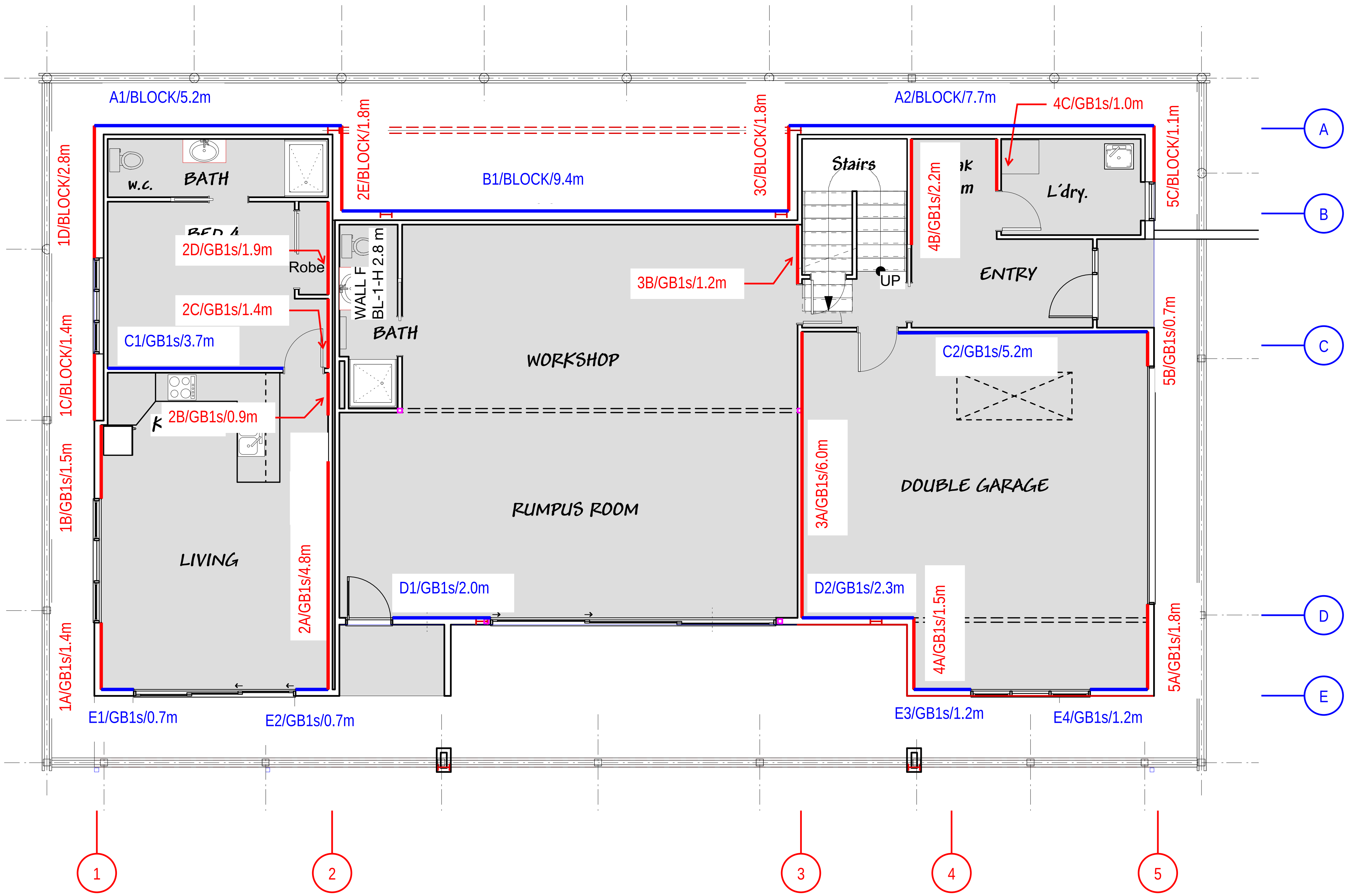
Ref: 12843

Hamilton City Council

BUILDING UNIT
APPROVED

BC Number - DD007.2021.00043914.001

BASEMENT BRACING WALL DESIGN



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JOB No 12843

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JOB 136 MORRINSVILLE ROAD

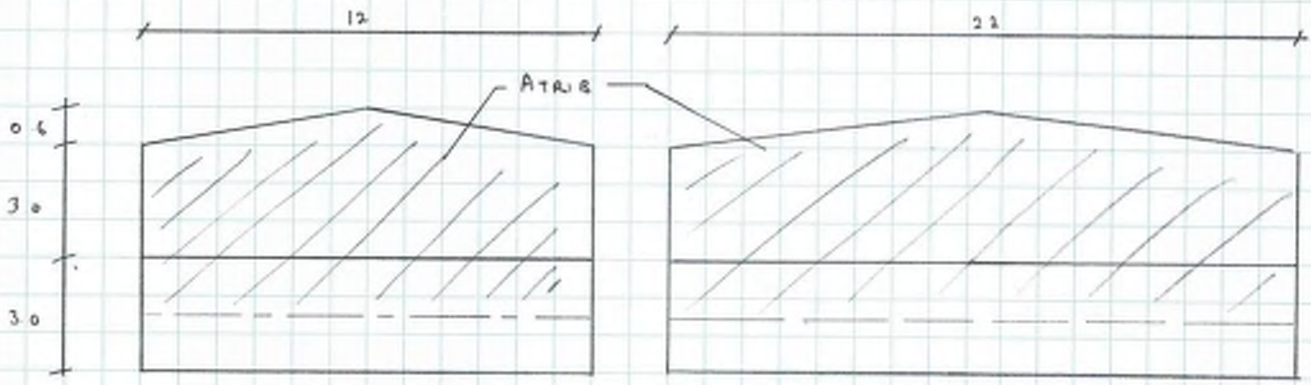
DATE 28.05.2021

BASEMENT BRACING DESIGN

BUILDING TO BE BRACED AT LOWER FLOOR LEVEL BY A COMBINATION OF CONCRETE BLOCK WALLS AND GIB-LINED STEEL-FRAMED WALLS.

WIND LOADING

BASIC DESIGN WIND PRESSURE $P_{005 ULS}$ = 1.13 kPa REFER ATTACHED SPREADSHEET



LONGITUDINAL DIRECTION TRANSVERSE DIRECTION

AERODYNAMIC SHAPE FACTOR C_{fig} : $0.7-(-0.5)$ = 1.2

TRIBUTARY AREA, LONG A_{trib} : $0.5\times0.6\times12+4.5\times12$ = 58 m^2

TRIBUTARY AREA, TRANS A_{trib} : $0.5\times0.6\times22+4.5\times22$ = 106 m^2

DESIGN WIND LOAD W_u : $P_{005}\,C_{fig}\,A_{trib}$

LONGITUDINAL DIRECTION W_L = $1.13\times1.2\times58$ = 79 kN

TRANSVERSE DIRECTION W_t = $1.13\times1.2\times106$ = 144 kN

SEISMIC LOADING

DEAD LOAD ROOF G_{roof} : 0.45 kPa × 348 m^2 = 157 kN

1ST FLOOR WALLS G_{1st} : 0.4 kPa × 3.0m × 0.5 × 114m = 68 kN

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SEISMIC WEIGHT ROOF $W_{Roof} = 157 + 68$ $= 225$ kN

DEAD LOAD STEEL FRAMED WALLS $G_{wall} = 68 \text{ kN} + 0.4 \times 3 \times 0.5 \times 75$ $= 113$ kN

DEAD LOAD FLOOR $G_{floor} = 0.5 \text{ kPa} \times 221 \text{ m}^2$ $= 111$ kN

DEAD LOAD DECK $G_{deck} = 0.5 \text{ kPa} \times 165 \text{ m}^2$ $= 83$ kN

DEAD LOAD BLOCK WALL $G_{bw} = 22 \text{ kN/m}^3 \times 0.19 \times 32 \times 3 \times 0.5$ $= 201$ kN

TOTAL DEAD LOAD $G = 113 + 111 + 83 + 201$ $= 508$ kN

LIVE LOAD FLOOR $Q_{floor} = 1.5 \times 221$ $= 332$ kN

LIVE LOAD DECK $Q_{deck} = 2.0 \times 165$ $= 330$ kN

TOTAL LIVE LOAD $Q = 332 + 330$ $= 662$ kN

NOTE - THE SEISMIC WEIGHT CALCULATED IS CONSERVATIVE, AS THE FINAL AREA OF THE DECK HAS BEEN REDUCED FROM THE ONE CALCULATED - OK BY INSPECTION

SEISMIC WEIGHT 1ST FLOOR $W_1 = G + \psi_a \psi_E Q$

$= 508 + 0.5 \times 0.3 \times 662$ $= 607$ kN

TOTAL SEISMIC WEIGHT $W_t = 225 + 607$ $= 832$ kN

DETERMINE HORIZONTAL SEISMIC SHEAR USING EQUIVALENT STATIC HORIZONTAL METHOD.

EQUIVALENT STATIC FORCE $F_i = F_t + 0.92V \dfrac{W_i H_i}{\sum W_i H_i}$

$F_t = 0.08V$ AT TOP AND 0 ELSEWHERE

LEVEL	W_i (kN)	H_i (m)	$W_i H_i$	F_i (kN)	M_i (kNm)
ROOF	225	6.0	1350	392	0
FIRST FLOOR	607	3.0	1821	440	1176
	832		3171	832	3672

THEREFORE BASEMENT WALLS DESIGNED FOR THE FOLLOWING

SHEAR DEMAND $V' = 832 \text{ kN} \times C_{0T}$

MOMENT DEMAND $M' = 3672 \text{ kNm} \times C_{0T}$

FOR SINGLY - REINFORCED BLOCK WALLS, ADOPT A DUCTILITY OF 1.25

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SEISMIC COEFFICIENT C_{o1} = 0.389 REFER ATTACHED SPREADSHEET

FOR NEW GIB-LINED WALLS, ADOPT A DUCTILITY OF 3.5

SEISMIC COEFFICIENT C_{oT} = 0.138 REFER ATTACHED SPREADSHEET

GIVEN PRIMARILY STEEL-FRAMED WALLS, CHECK GLOBAL CAPACITY USING μ = 3.5.

NEED TO SEPARATELY CHECK BLOCK WALLS FOR HIGHER μ = 1.25 LOADS LATER

LONGITUDINAL WIND W_L = 79 x 20 = 1580 BU

TRANSVERSE WIND W_T = 144 x 20 = 2880 BU

EARTHQUAKE DEMAND E_u = 832 x 0.138 x 20 = 2296 BU

FOR BLOCKWALLS ADOPT A BRACING CAPACITY OF 100 BU/m

REFER TO THE ATTACHED SPREADSHEET FOR FULL CALCULATIONS

ADOPT BRACING LAYOUT AS PER THE ATTACHED PLAN

BLOCKWALL CAPACITY

CHECK WHETHER A CAPACITY OF 100 BU/m IS REASONABLE. GIVEN A DUCTILITY OF 3.5 WAS ADOPTED, NEED TO INCREASE TO REFLECT AN EQUIVALENT CAPACITY FOR A DUCTILITY OF 1.25

EQUIVALENT CAPACITY = 100 BU/m x $\frac{0.389}{0.138}$ = 282 BU/m

OR IN KN = $\frac{282}{20}$ = 14 kN/m

FOR MOMENT = 14 x 3 = 42 kNm/m

ADOPT HD16-400 VERT & HD12-400 HOR REBAR. REFER TO THE ATTACHED SPREADSHEET FOR FULL CALCULATIONS

BLOCKWALLS OK

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JOB 136 MORRINSVILLE ROAD

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BRACING DESIGN SPREADSHEET

Wall Bracing Demand	Wind BU	EQ BU
Longitudinal	1580	2296
Transverse	2880	2296

	Long	Trans
Bracing Lines?	5	5
Timber Floor?	No	

Bracing Type	0.4m		1.2m	
	Wind	EQ	Wind	EQ
GS1-N	50	55	70	60
GS2-N	70	65	95	85
GS2-NOM	50	50	50	50
GSP-H	100	115	150	150
BL1-H	90	100	125	105
BLG-H	110	115	150	145
BLP-H	120	135	150	150
BLOCK	100	100	100	100
GB1s	70	85	125	105
EXTG	50	50	50	50

Longitudinal Direction

Brace Line	Wall ID	Length m	Height m	Wall Type	Wind BU	EQ BU
A	1	5.2 m	2.4 m	BLOCK	520	520
A	2	7.7 m	2.4 m	BLOCK	770	770
B	1	9.4 m	2.4 m	BLOCK	940	940
C	1	3.7 m	3.0 m	GB1s	370	311
C	2	5.2 m	3.0 m	GB1s	520	437
D	1	2.0 m	3.0 m	GB1s	200	168
D	2	2.3 m	3.0 m	GB1s	230	193
E	1	0.7 m	3.0 m	GB1s	39	48
E	2	0.7 m	3.0 m	GB1s	39	48
E	3	1.2 m	3.0 m	GB1s	120	101
E	4	1.2 m	3.0 m	GB1s	120	101

Longitudinal Direction

Brace Line	Ext. m	W_{trib} BU	Wind BU	Min Wind	Check	E_{trib} BU	EQ BU	Min EQ	Check
A	12.2 m	0	1290	183	OK	0	1290	230	OK
B	9.8 m	0	940	158	OK	0	940	230	OK
C	0.0 m	0	890	158	OK	0	748	230	OK
D	9.6 m	0	430	158	OK	0	361	230	OK
E	12.4 m	0	318	186	OK	0	297	230	OK
		0	**3868**	**1580**	**245%**	**0**	**3636**	**2296**	**158%**

Transverse Direction

Brace Line	Wall ID	Length m	Height m	Wall Type	Wind BU	EQ BU
1	A	1.4 m	3.0 m	GB1s	140	118
1	B	1.5 m	3.0 m	GB1s	150	126
1	C	1.4 m	2.4 m	BLOCK	140	140
1	D	2.8 m	2.4 m	BLOCK	280	280
2	A	4.8 m	3.0 m	GB1s	480	403
2	B	0.9 m	3.0 m	GB1s	50	61
2	C	1.4 m	3.0 m	GB1s	140	118
2	D	1.9 m	3.0 m	GB1s	190	160
2	E	1.8 m	2.4 m	BLOCK	180	180
3	A	6.0 m	3.0 m	GB1s	600	504
3	B	1.2 m	2.4 m	GB1s	150	126
3	C	1.8 m	2.4 m	BLOCK	180	180
4	A	1.5 m	3.0 m	GB1s	150	126
4	B	2.2 m	3.0 m	GB1s	220	185
4	C	1.0 m	3.0 m	GB1s	56	68
5	A	1.8 m	3.0 m	GB1s	180	151
5	B	0.7 m	3.0 m	GB1s	39	48
5	C	1.1 m	2.4 m	BLOCK	110	110

Transverse Direction

Brace Line	Ext. m	W_{trib} BU	Wind BU	Min Wind	Check	E_{trib} BU	EQ BU	Min EQ	Check
1	12.0 m	0	710	288	OK	0	664	230	OK
2	1.8 m	0	1040	288	OK	0	922	230	OK
3	1.8 m	0	930	288	OK	0	810	230	OK
4	1.4 m	0	426	288	OK	0	379	230	OK
5	12.0 m	0	329	288	OK	0	309	230	OK
		0	**3436**	**2880**	**119%**	**0**	**3083**	**2296**	**134%**

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JOB 136 MORRINSVILLE ROAD

This spreadsheet is for the calculation of basic wind actions for non-wind sensitive structures to AS/NZS 1170.2. The aerodynamic shape factor: Internal and external wind pressure coefficients ($C_{fig} = C_{p,i}, C_{p,e}$) are to be calculated separately to determine the design actions on individual elements.

Basic Wind Actions

to AS/NZS 1170.2:2002

Site Information

Region	A7	Hamilton	Refer Cl. 3.2
Terrain Category, ultimate	2	Open terrain with well-scattered obstructions	Refer Cl. 4.2.1
Terrain Category, serviceability	2	Open terrain with well-scattered obstructions	Refer Cl. 4.2.1

Building Information

Importance Level	2	(Normal Structures - 1170.0, refer Table 3.2)
Design Life	50 years	(from AS/NZS 1170.0, refer Table 3.3)
Height, z	8.0 m	(for z = h, average roof height)

Site Exposure Multipliers

Directional Multiplier, M_d	1.00	(for non-directional = 1.0)	Refer Cl. 3.3
Shielding Multiplier, M_s	1.00	(for no shielding = 1.0)	Refer Cl. 4.3
Topographic Multiplier, M_t	1.00	(for no topo influence = 1.0)	Refer Cl. 4.4

Site Wind Speed

$V_{sit,\beta} = V_R\, M_d\, (M_{z,cat}, M_s, M_t)$ 1 Refer Cl. 2.2

		ULS	SLS1			
where	R	500	25		(recurrence interval)	Refer 1170.0
	F_C , F_D	-	-	-	(factor for regions C & D)	Refer Cl. 3.4
	V_R	45 m/s	37 m/s		(regional wind speed)	Refer Cl. 3.2
	M_d	1.0	1.0			
	$M_{z,cat}$	0.96	0.96		(terrain/height multiplier)	Refer Cl. 4.2
	M_s	1.0	1.0			
	M_t	1.0	1.0			
therefore	$V_{sit,\beta}$	43.4 m/s	35.7 m/s		(NZS 3604 wind zone category = High)	

Design Wind Pressure

$P = 0.5\rho_{air}\,[V_{des\theta}]^2\, C_{fig}\, C_{dyn}\, x10^{-3}$ Refer Cl. 2.4.1

		ULS	SLS1			
where	ρ_{air}	1.2 kg/m³	1.2 kg/m³		(air density)	Refer Cl. 2.4.1
	$V_{des\theta}$	43.4 m/s	35.7 m/s		(for non directional = $V_{sit\beta}$)	Refer Cl. 2.3
	C_{fig}	1.0	1.0		(to be determined separately)	Refer Cl. 2.4.1
	C_{dyn}	1.0	1.0		(for non wind-sensitive)	Refer Cl. 2.4.1
therefore	P	1.13 KPa	0.76 KPa		*check ULS $V_{des\,\theta}$ >30m/s* OK	Refer Cl. 2.3

Summary

Site Wind Speed

- Ultimate	**V_{ULS}**	=	**43.4 m/s**
- Serviceability	**V_{SLS1}**	=	**35.7 m/s**

Basic Design Wind Pressure

- Ultimate	**$P_{Basic\ ULS}$**	=	**1.13 kPa**
- Serviceability	**$P_{Basic\ SLS1}$**	=	**0.76 kPa**

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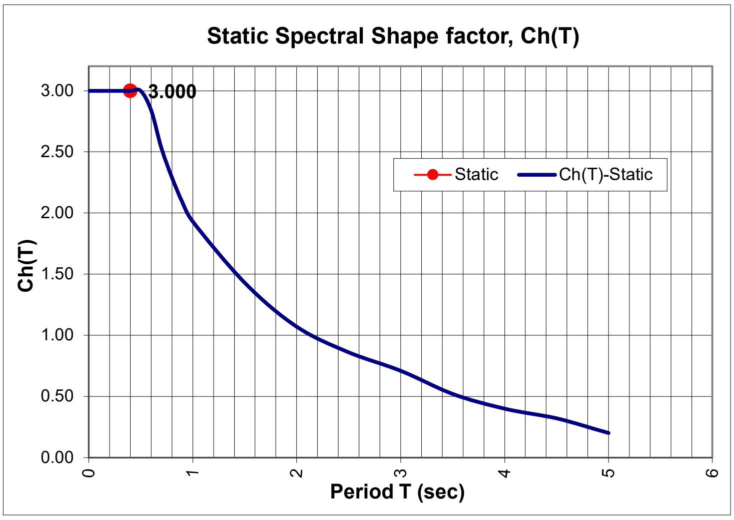
JOB 136 MORRINSVILLE ROAD

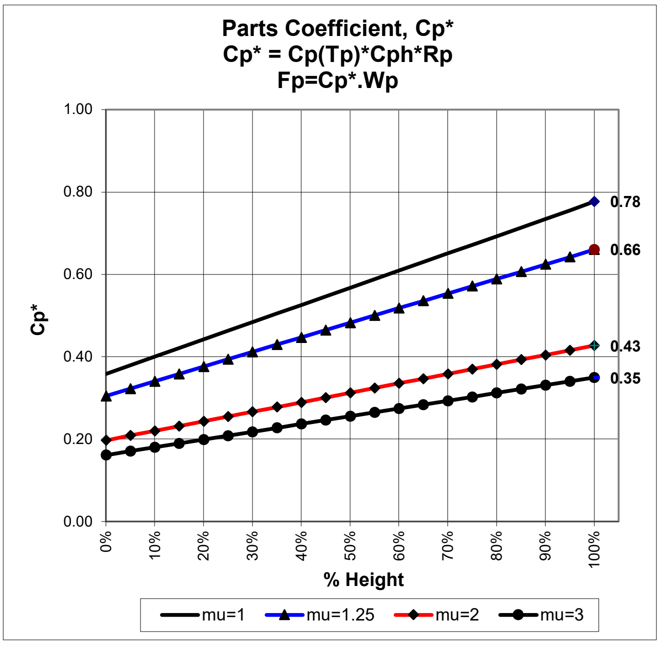
Input Data		
Design Working Life	50	years
Importance Level	2	
Soil Type	D	Deep or Soft Soil
Location	Hamilton	
Z	0.16	
Zuser		
Exceedence prob ULS, 1/K: K=	500	
Exceedence prob SLS, 1/K: K=	25	
Return Period Factor, Ru	1	
Return Period Factor, Rs	0.25	
SLS2 Return Period Factor, Rs2	#N/A	
D	25	km
Seismic Mass, Wt	0	kN
Building Period T	0.40	s
Cdmin	0.03	
Nmax	1	
Ultimate Design Spectra	**Static ULS**	
N(T,D)u	1	
Ch(T)	3.00	
C(T)	0.48	
Ductility μ	1.25	
Sp	0.925	
Kμ	1.14	
Seismic Coeff (ULS), Cdu	0.389	
Base Shear, Vbu	0	kN
Service Design Spectra	**Static SLS**	
N(T,D)s	1	
Ch(T)	3.00	
C(T)s	0.12	
Ductility μ	1	
Sp	0.7	
Kμ	1.00	
Seismic Coeff (SLS), Cds	0.084	
Parts		
Classification of parts	P.1	
Structure Limit State	ULS	
C(0) ULS	0.1792	
hn	7	m
Tp	0.4	s
Ci(Tp)	2	
Rp	1	

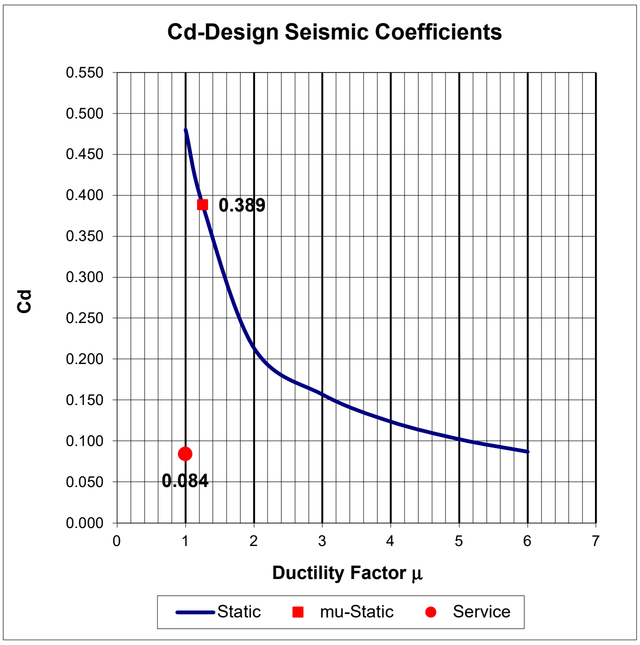


Design Base Shear				
Ultimate			Static	Base Shear
μ	Sp	Kμ	Cd	kN
1	1.00	1.00	0.480	0
1.25	0.93	1.14	0.389	0
2	0.70	1.57	0.214	0
3	0.70	2.14	0.157	0
4	0.70	2.71	0.124	0
5	0.70	3.29	0.102	0
6	0.70	3.86	0.087	0

Dynamic Scaling		
Structure Regular	Y	
Static Base Shear, Ve	0	kN
Dynamic Analysis Base Shear, V		kN
Scaling factor, K	1.000	
Scaled base Shear & % Static	0	#DIV/0!

Notes:





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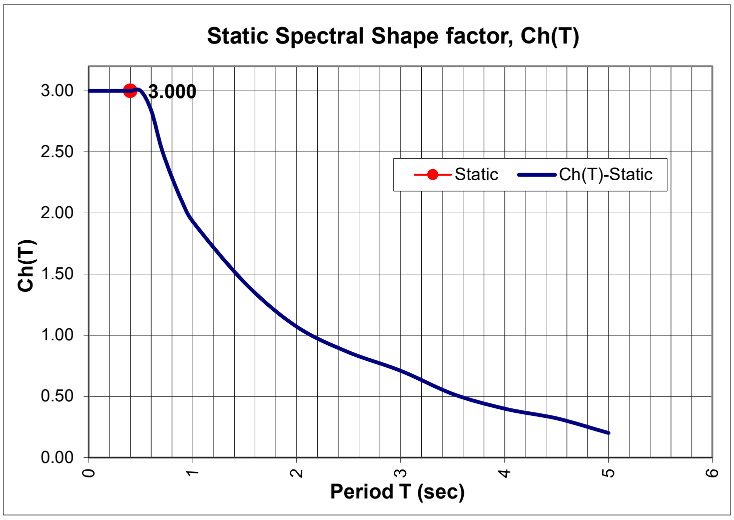
JOB 136 MORRINSVILLE ROAD

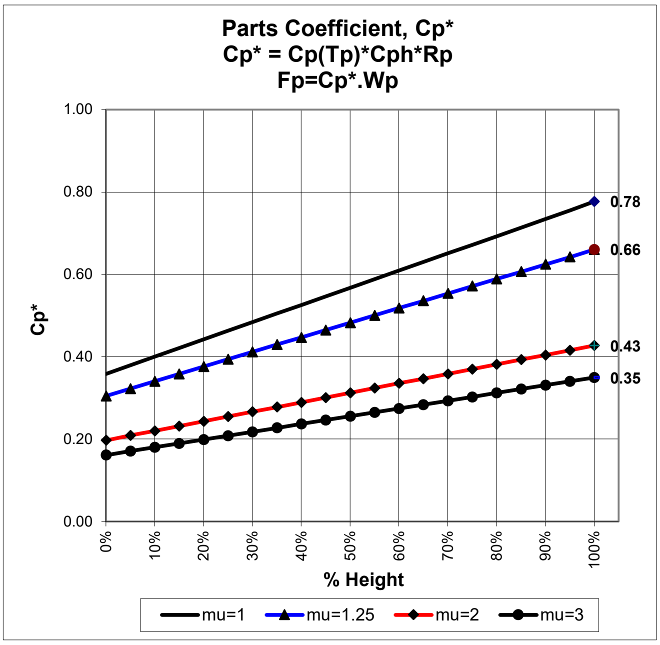
Input Data		
Design Working Life	50	years
Importance Level	2	
Soil Type	D	Deep or Soft Soil
Location	Hamilton	
Z	0.16	
Zuser		
Exceedence prob ULS, 1/K: K=	500	
Exceedence prob SLS, 1/K: K=	25	
Return Period Factor, Ru	1	
Return Period Factor, Rs	0.25	
SLS2 Return Period Factor, Rs2	#N/A	
D	25	km
Seismic Mass, Wt	0	kN
Building Period T	0.40	s
Cdmin	0.03	
Nmax	1	
Ultimate Design Spectra	**Static ULS**	
N(T,D)u	1	
Ch(T)	3.00	
C(T)	0.48	
Ductility µ	3.5	
Sp	0.7	
Kµ	2.43	
Seismic Coeff (ULS), Cdu	**0.138**	
Base Shear, Vbu	0	kN
Service Design Spectra	**Static SLS**	
N(T,D)s	1	
Ch(T)	3.00	
C(T)s	0.12	
Ductility µ	1	
Sp	0.7	
Kµ	1.00	
Seismic Coeff (SLS), Cds	**0.084**	
Parts		
Classification of parts	P.1	
Structure Limit State	ULS	
C(0) ULS	0.1792	
hn	7	m
Tp	0.4	s
Ci(Tp)	2	
Rp	1	

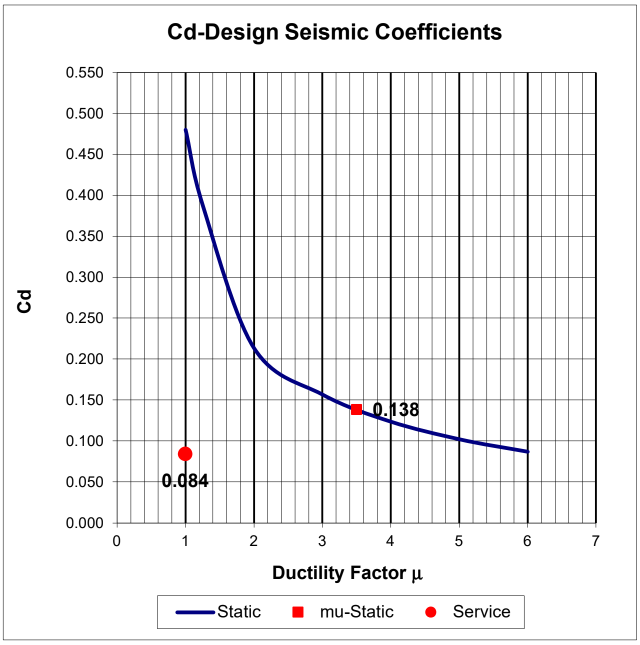


Design Base Shear				
Ultimate			Static	Base Shear
µ	Sp	Kµ	Cd	kN
1	1.00	1.00	0.480	0
1.25	0.93	1.14	0.389	0
2	0.70	1.57	0.214	0
3	0.70	2.14	0.157	0
4	0.70	2.71	0.124	0
5	0.70	3.29	0.102	0
6	0.70	3.86	0.087	0

Dynamic Scaling		
Structure Regular	Y	
Static Base Shear, Ve	0	kN
Dynamic Analysis Base Shear, V		kN
Scaling factor, K	1.000	
Scaled base Shear & % Static	0	#DIV/0!

Notes:





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JOB 136 MORRINSVILLE ROAD

CONCRETE BLOCKWALL CAPACITY CALCULATIONS

Masonry Compression Strengt	f'_m	=	12	MPa
Stirrup Reinforcement Strengtl	f_{yt}	=	500	MPa
Shear Reduction Factor	ϕ	=	0.75	
Factor	C_3	=	0.8	MPa

Longitudinal Direction

Wall ID	L_w mm	B_w mm	H_e mm	D_l mm	s mm	n	D_s mm	s mm	n	v_{bm} MPa	d mm	A_s mm^2	p_w	C_1	C_2	v_m MPa	A_v mm^2	v_s MPa	ϕV_n kN	v_g MPa	v_{glim} MPa	Shear kN	Shear BU/m	Moment kNm	Moment kNm/m	Check
Wall A1	5200	190	3000	16	400	14	12	400	1	0.69	4160	2815	0.0036	0.2	1.26	1.01	113	0.6	949	1.20	1.56	949	183	2556	492	OK
Wall A2	7700	190	3000	16	400	20	12	400	1	0.69	6160	4021	0.0034	0.19	1.39	1.1	113	0.6	1485	1.27	1.56	1485	193	5543	720	OK
Wall B1	9400	190	3000	16	400	24	12	400	1	0.69	7520	4825	0.0034	0.19	1.45	1.13	113	0.6	1849	1.29	1.56	1849	197	8038	855	OK

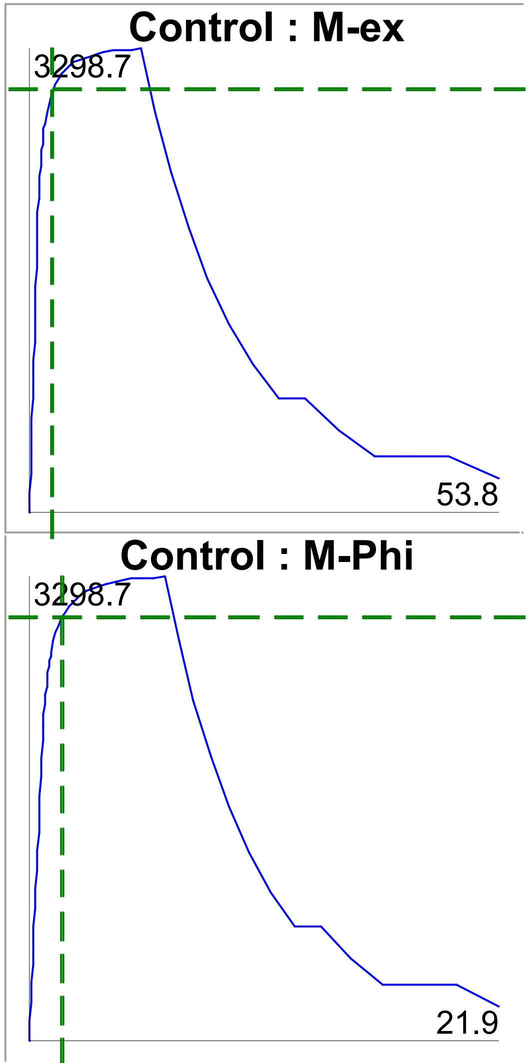
Transverse Direction

Wall ID	L_w mm	B_w mm	H_e mm	D_l mm	s mm	n	D_s mm	s mm	n	v_{bm} MPa	d mm	A_s mm^2	p_w	C_1	C_2	v_m MPa	A_v mm^2	v_s MPa	ϕV_n kN	v_g MPa	v_{glim} MPa	Shear kN	Shear BU/m	Moment kNm	Moment kNm/m	Check
Wall 1C	1400	190	3000	16	400	4	12	400	1	0.69	1120	804	0.0038	0.21	1.00	0.84	113	0.6	229	1.07	1.56	229	163	205	146	OK
Wall 1D	2800	190	3000	16	400	8	12	400	1	0.69	2240	1608	0.0038	0.21	1.00	0.84	113	0.6	457	1.07	1.56	457	163	798	285	OK
Wall 2D	1800	190	3000	16	400	5	12	400	1	0.69	1440	1005	0.0037	0.2	1.00	0.83	113	0.6	293	1.07	1.56	293	163	324	180	OK
Wall 3C	1800	190	3000	16	400	5	12	400	1	0.69	1440	1005	0.0037	0.2	1.00	0.83	113	0.6	293	1.07	1.56	293	163	324	180	OK
Wall 5C	1100	190	3000	16	400	3	12	400	1	0.69	880	603	0.0036	0.2	1.00	0.83	113	0.6	179	1.07	1.56	179	163	120	109	OK

Response-2000 v 1.0.5

Wall A1

NAM 2021/7/29 - 4:54 pm



ε_{x0} = 2.71 mm/m

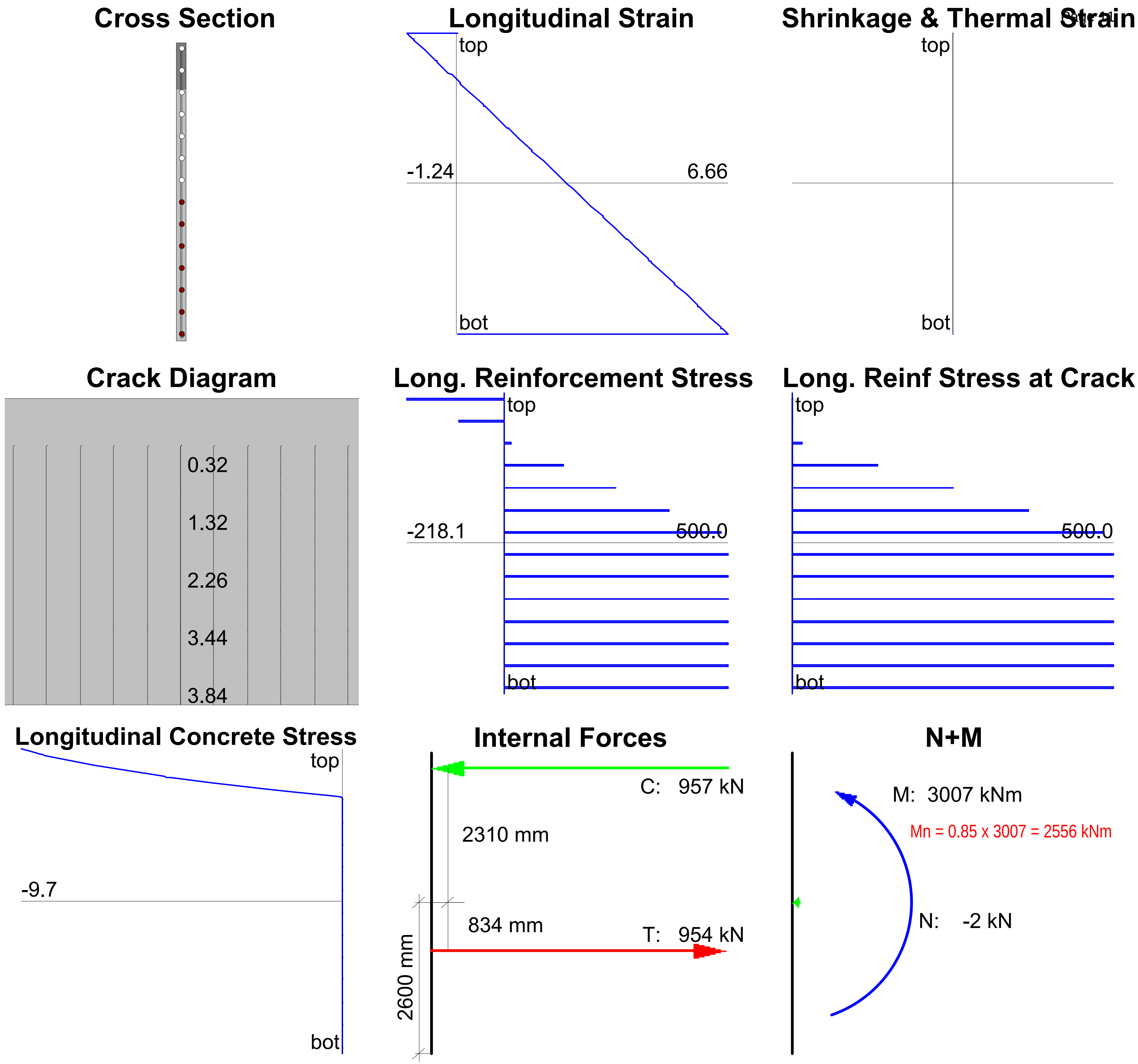
ϕ = 1.52 rad/km

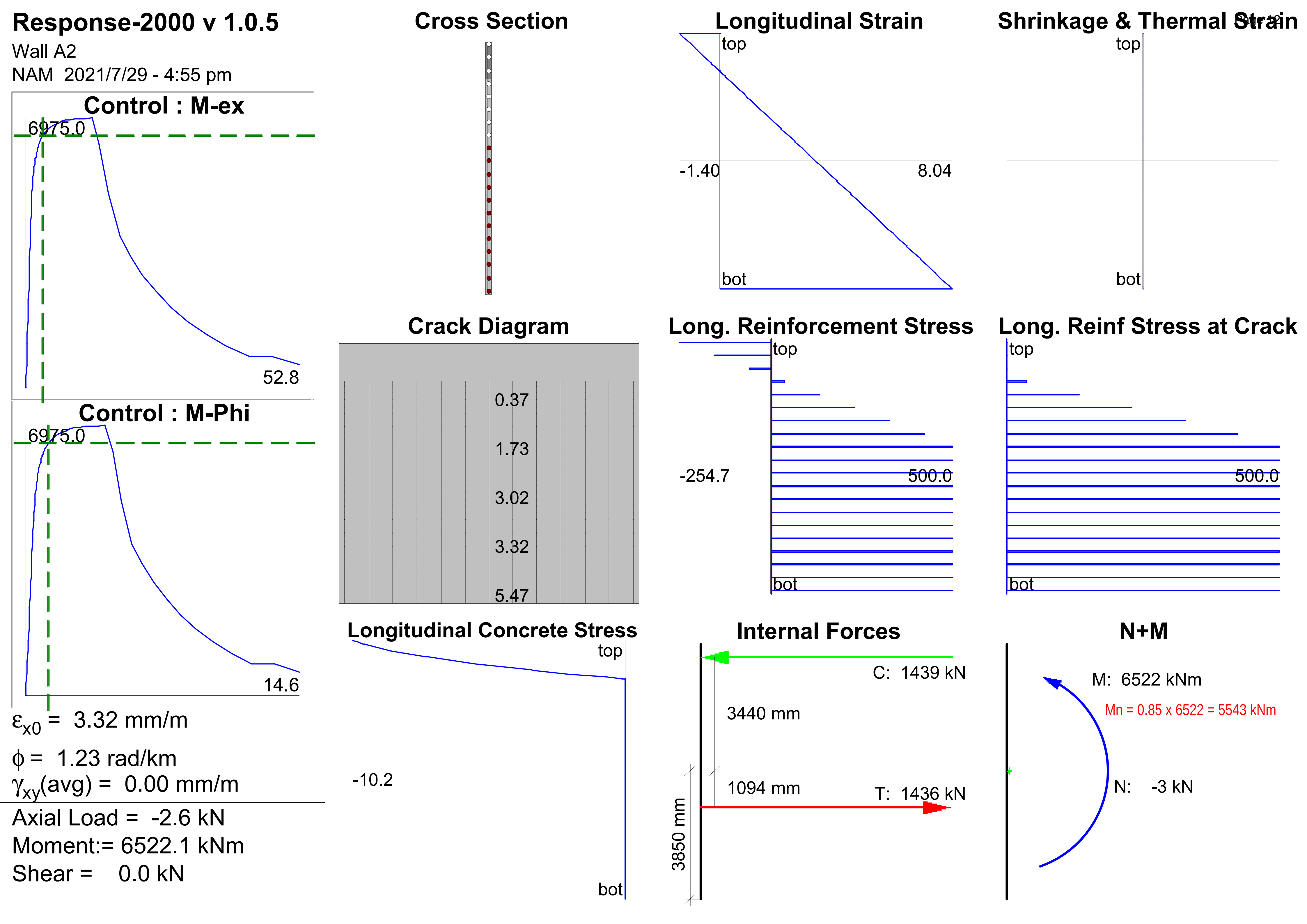
γ_{xy}(avg) = 0.00 mm/m

Axial Load = -2.5 kN

Moment:= 3006.8 kNm

Shear = 0.0 kN





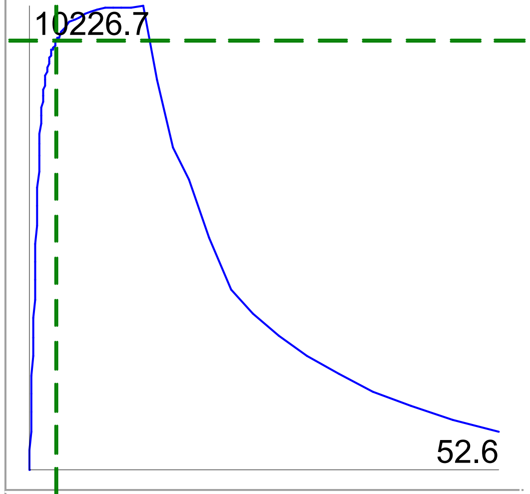
Response-2000 v 1.0.5
Wall A2
NAM 2021/7/29 - 4:55 pm
Control : M-ex
6975.0
52.8
Control : M-Phi
6975.0
14.6
εx0 = 3.32 mm/m
ϕ = 1.23 rad/km
γxy(avg) = 0.00 mm/m
Axial Load = -2.6 kN
Moment:= 6522.1 kNm
Shear = 0.0 kN
Cross Section
Longitudinal Strain
top
-1.40
8.04
bot
Shrinkage & Thermal Strain
top
bot
Crack Diagram
0.37
1.73
3.02
3.32
5.47
Long. Reinforcement Stress
top
-254.7
500.0
bot
Long. Reinf Stress at Crack
top
500.0
bot
Longitudinal Concrete Stress
top
-10.2
bot
Internal Forces
C: 1439 kN
3440 mm
1094 mm
T: 1436 kN
3850 mm
N+M
M: 6522 kNm
Mn = 0.85 x 6522 = 5543 kNm
N: -3 kN

Response-2000 v 1.0.5

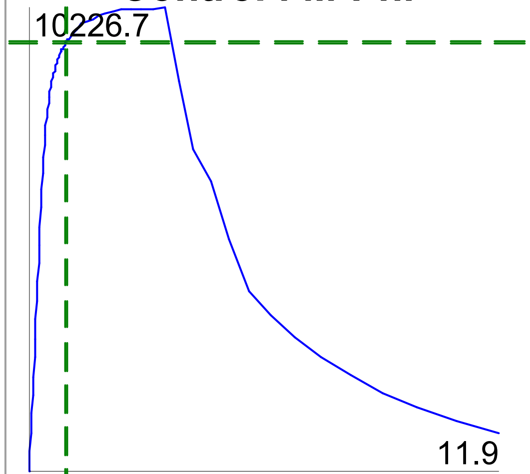
Wall A3

NAM 2021/7/29 - 4:56 pm

Control : M-ex



Control : M-Phi



ε_{x0} = 3.01 mm/m

ϕ = 0.91 rad/km

γ_{xy}(avg) = 0.00 mm/m

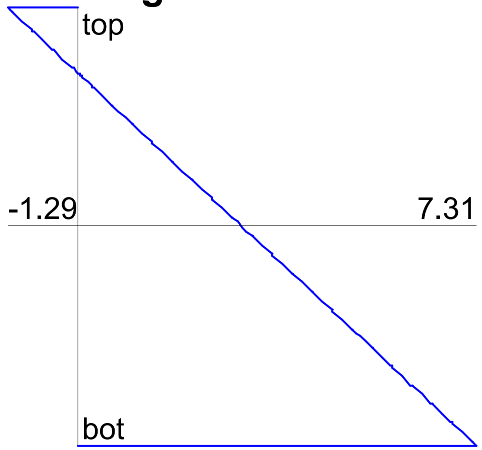
Axial Load = -5.7 kN

Moment:= 9457.4 kNm

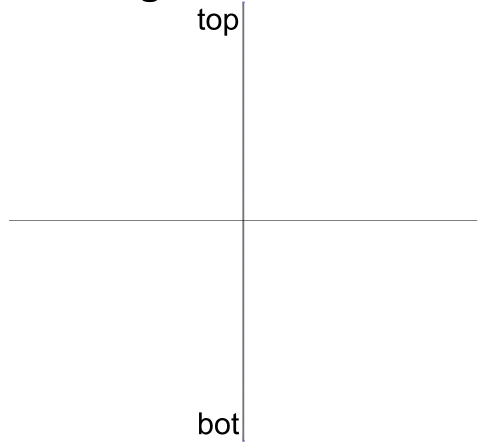
Shear = 0.0 kN

Cross Section

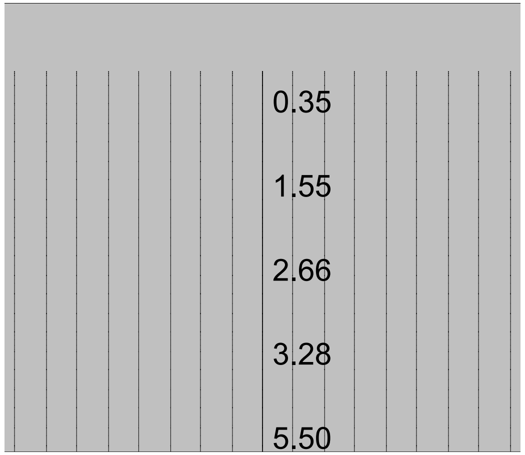
Longitudinal Strain



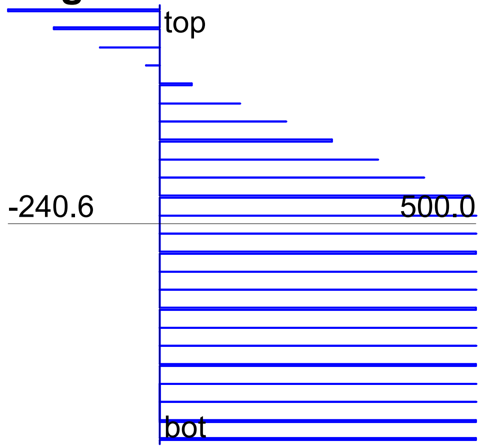
Shrinkage & Thermal Strain



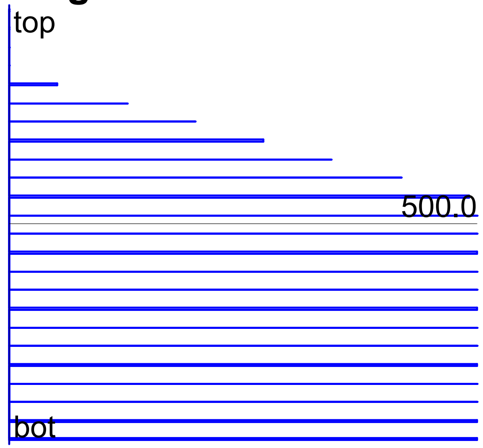
Crack Diagram



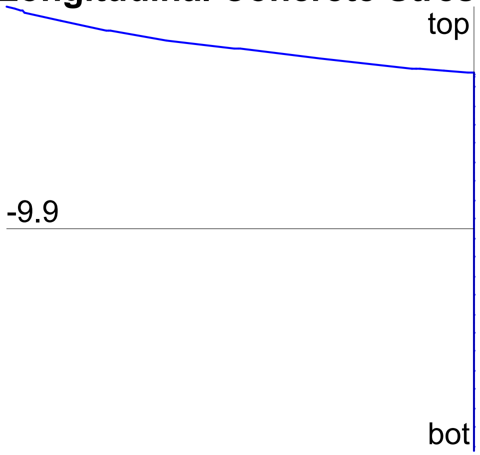
Long. Reinforcement Stress



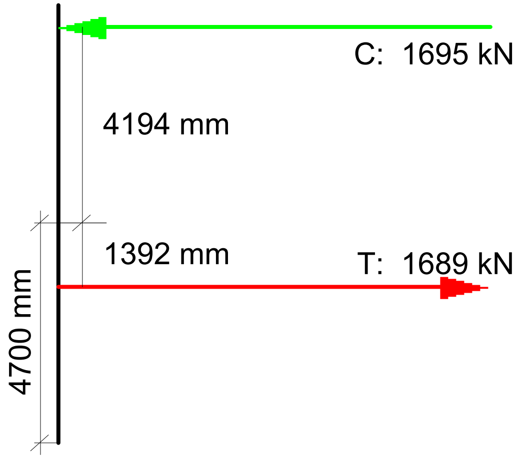
Long. Reinf Stress at Crack



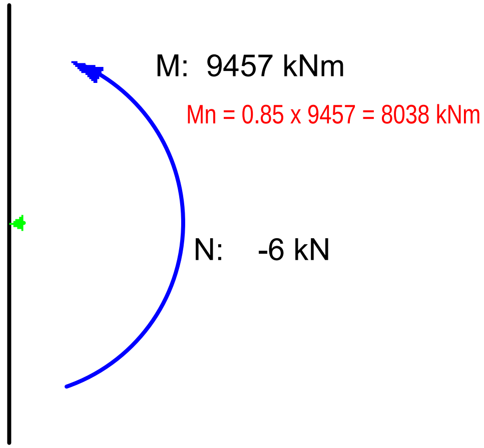
Longitudinal Concrete Stress



Internal Forces



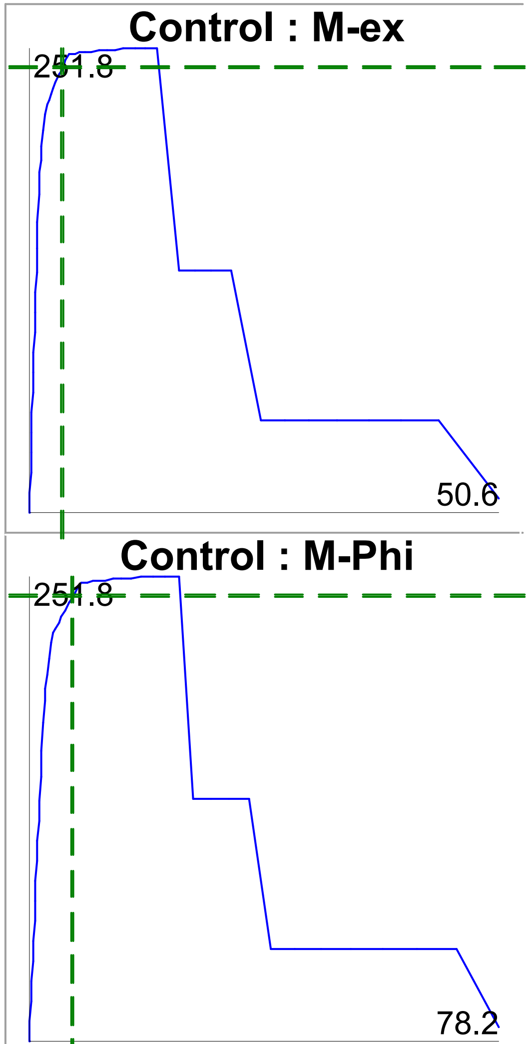
N+M



Response-2000 v 1.0.5

Wall 1C

NAM 2021/7/29 - 4:57 pm



ε_{x0} = 3.56 mm/m

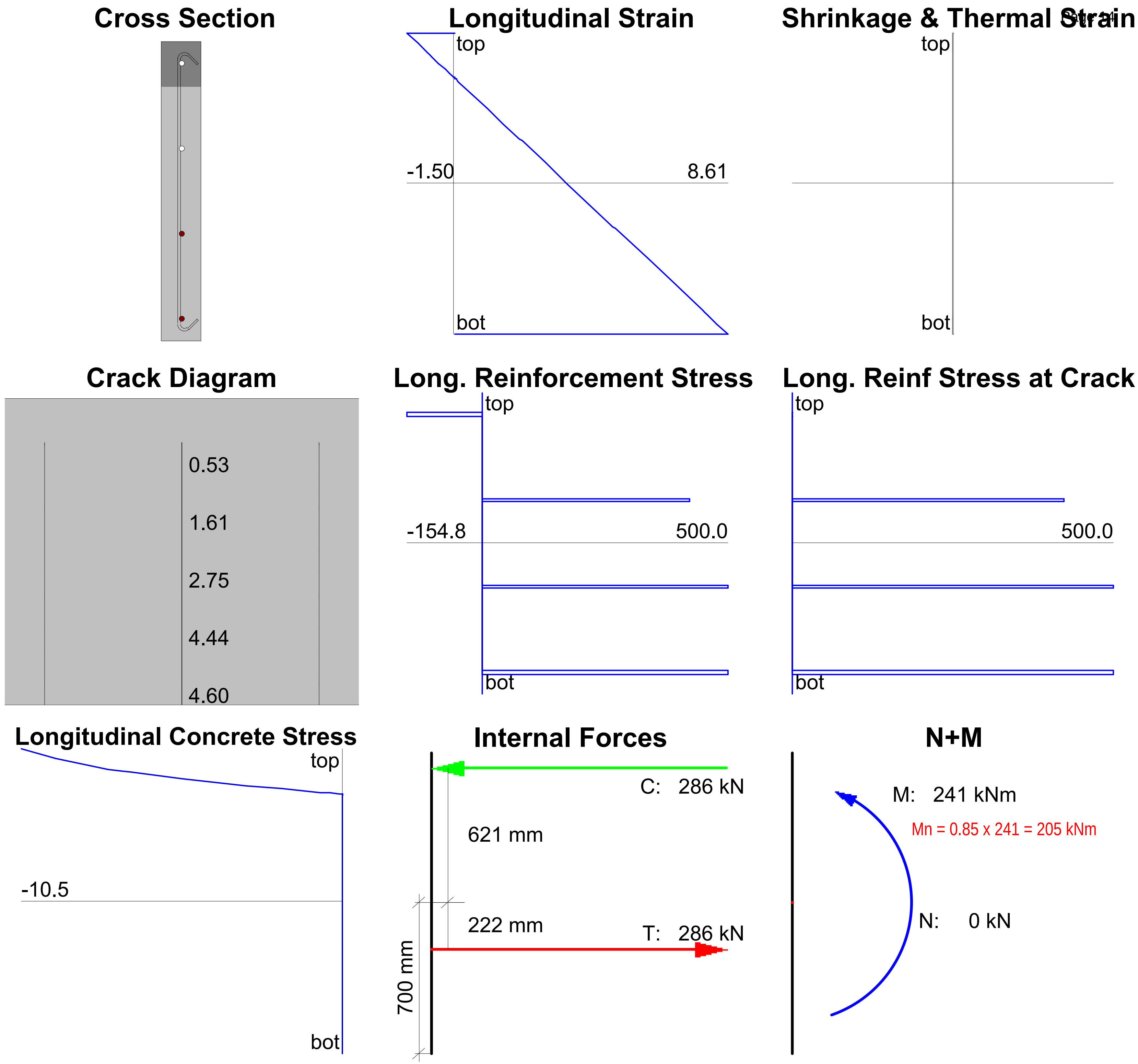
ϕ = 7.22 rad/km

γ_{xy}(avg) = 0.00 mm/m

Axial Load = 0.1 kN

Moment:= 241.1 kNm

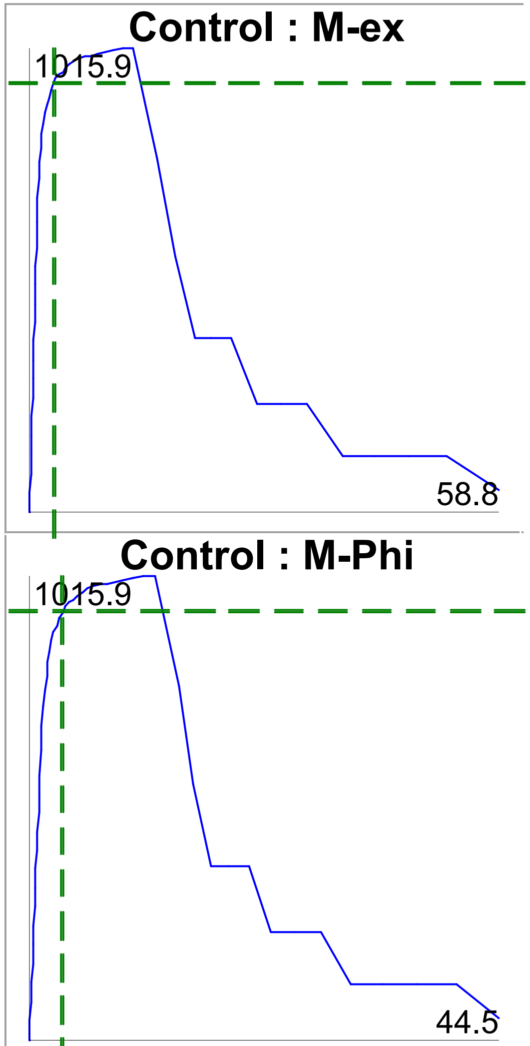
Shear = 0.0 kN



Response-2000 v 1.0.5

Wall 1D

NAM 2021/7/29 - 4:58 pm



ε_{x0} = 3.02 mm/m

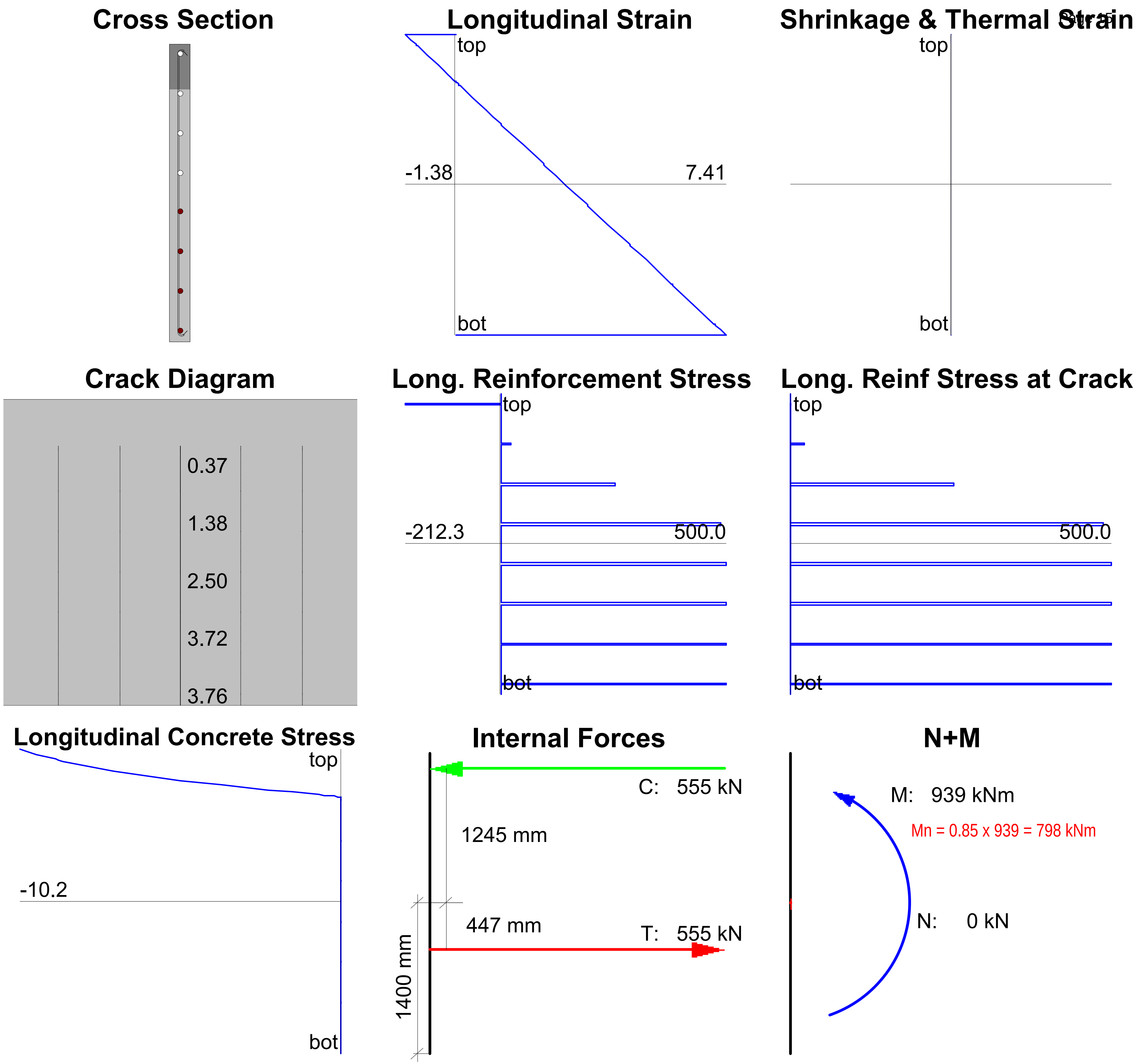
ϕ = 3.14 rad/km

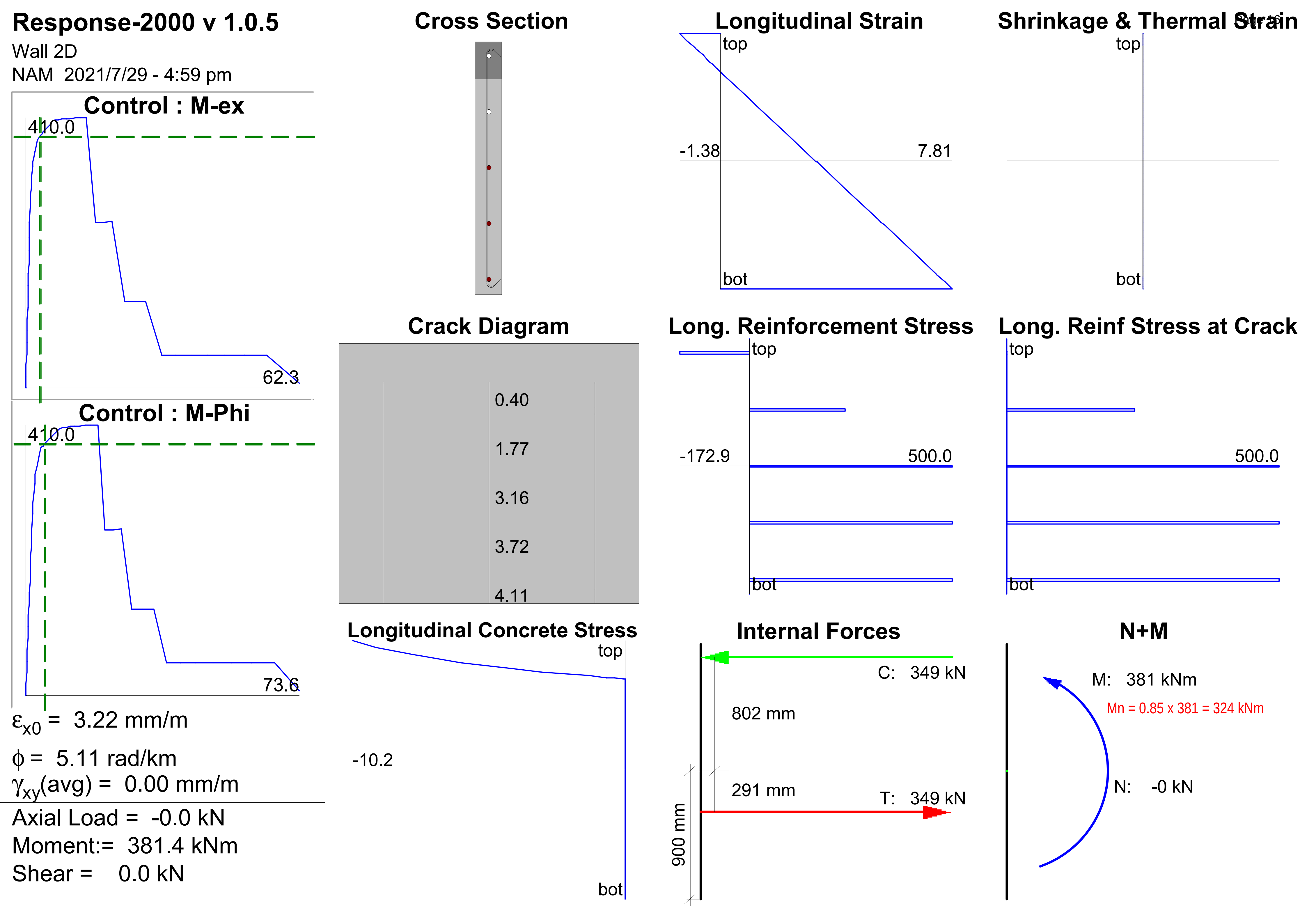
γ_{xy}(avg) = 0.00 mm/m

Axial Load = 0.5 kN

Moment:= 938.6 kNm

Shear = 0.0 kN

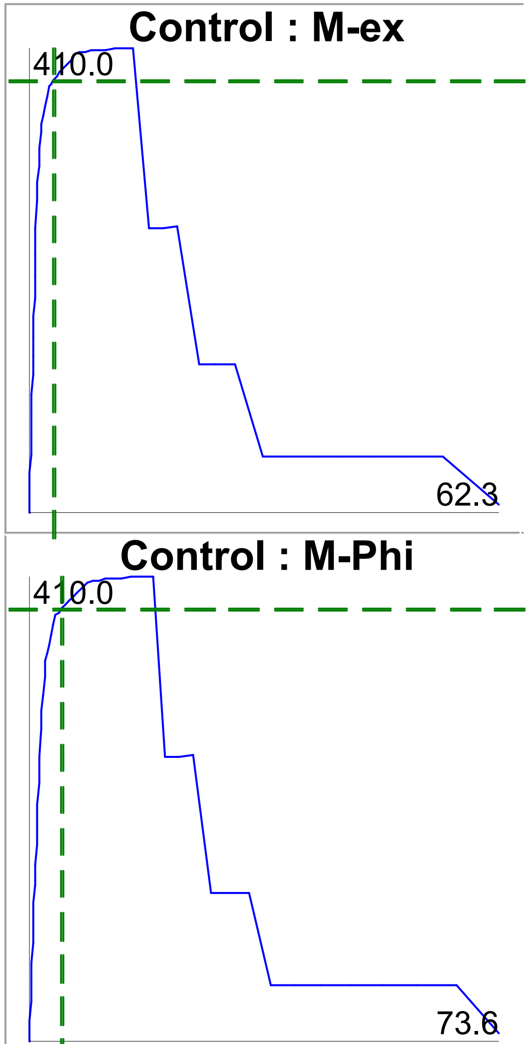



Response-2000 v 1.0.5
Wall 2D
NAM 2021/7/29 - 4:59 pm
Control : M-ex
410.0
62.3
Control : M-Phi
410.0
73.6
εx0 = 3.22 mm/m
φ = 5.11 rad/km
γxy(avg) = 0.00 mm/m
Axial Load = -0.0 kN
Moment:= 381.4 kNm
Shear = 0.0 kN
Cross Section
Longitudinal Strain
top
-1.38
7.81
bot
Shrinkage & Thermal Strain
top
bot
Crack Diagram
0.40
1.77
3.16
3.72
4.11
Long. Reinforcement Stress
top
-172.9
500.0
bot
Long. Reinf Stress at Crack
top
500.0
bot
Longitudinal Concrete Stress
top
-10.2
bot
Internal Forces
C: 349 kN
802 mm
291 mm
T: 349 kN
900 mm
N+M
M: 381 kNm
Mn = 0.85 x 381 = 324 kNm
N: -0 kN

Response-2000 v 1.0.5

Wall 3C

NAM 2021/7/29 - 5:00 pm



ε_{x0} = 3.22 mm/m

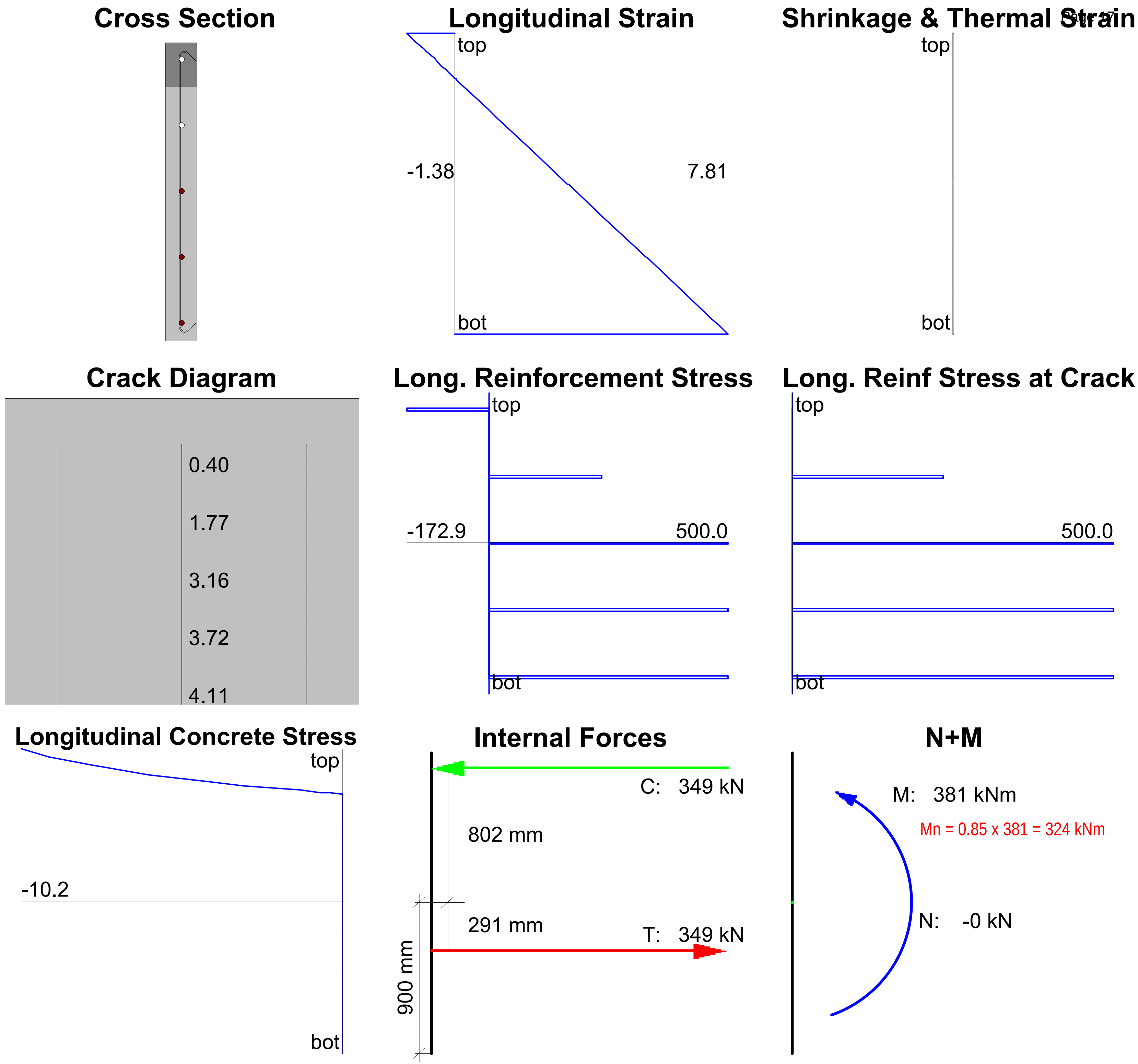
ϕ = 5.11 rad/km

γ_{xy}(avg) = 0.00 mm/m

Axial Load = -0.0 kN

Moment:= 381.4 kNm

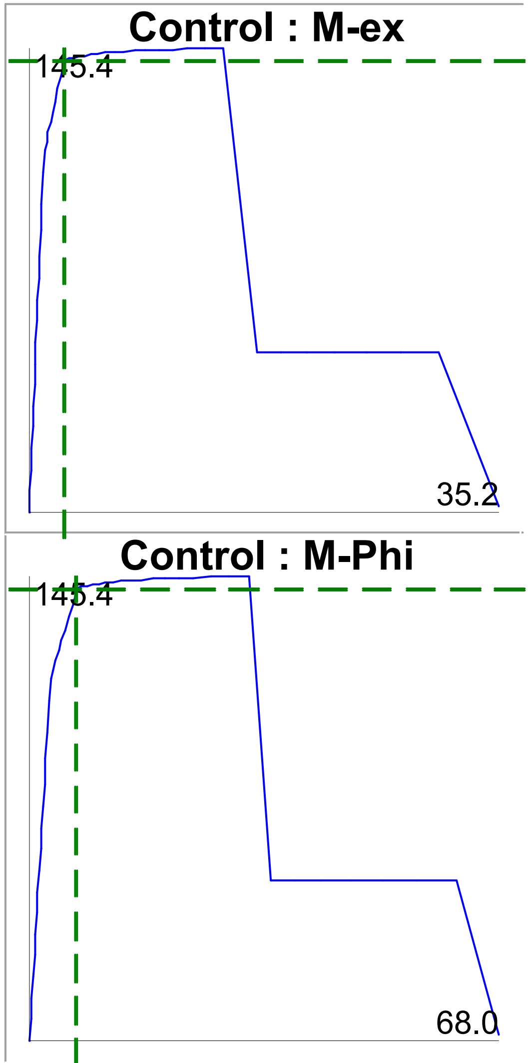
Shear = 0.0 kN



Response-2000 v 1.0.5

Wall 5C

NAM 2021/7/29 - 5:01 pm



ε_{x0} = 2.66 mm/m

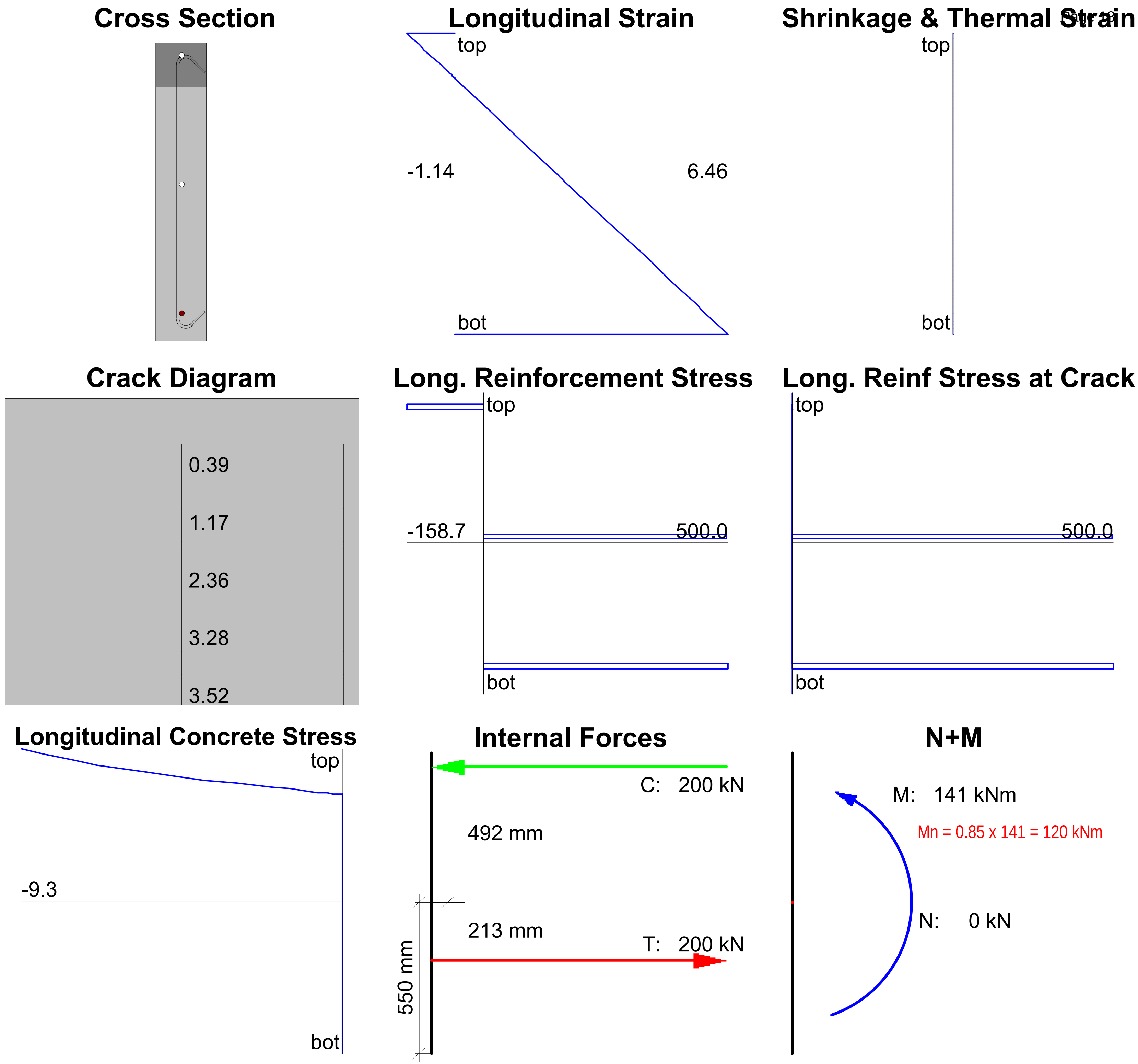
ϕ = 6.90 rad/km

γ_{xy}(avg) = 0.00 mm/m

Axial Load = 0.0 kN

Moment:= 141.4 kNm

Shear = 0.0 kN



GIB EzyBrace® Systems specification GB1s

Specification code	Minimum length (m)	Lining requirement	Other requirements	BU rating per metre	
				Wind	Earthquake
GB1s	0.4	10 mm GIB Braceline® plasterboard one side	Hold-down	70	85
	1.2			125	105

WALL FRAMING

Wall framing to comply with,

— NZBC B1 – Structure
— NZBC B2 – Durability

Steel framing dimensions and height as determined by Specific Engineering Design. C section studs shall have a minimum thickness of 0.75 mm and minimum nominal depth of 90 mm with 35 mm wide flanges.

BOTTOM PLATE FIXING

Timber floor

5mm washer as illustrated, fixed to timber floor framing using a 12 mm x 100 mm galvanized coach screw or 4 x 75mm Type 17 class 3 screws.

Concrete floor

5mm washer as illustrated, fixed to the concrete slab using a proprietary concrete anchor with a minimum uplift capacity of 12kN taking into consideration concrete slab thickness (internal walls) and edge distance (external walls).

WALL LINING

— One layer of 10 mm GIB Braceline® plasterboard.
— Vertical or horizontal fixing permitted.
— Sheet joints shall be touch fitted.
— Use full height sheets where possible.

PERMITTED ALTERNATIVES

The Bracing Unit ratings for system GB1s apply to 10 mm GIB Braceline®.

FASTENING THE LINING

Fasteners

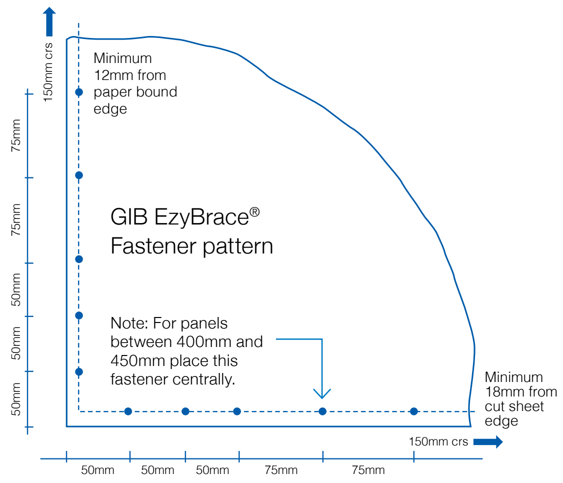
32mm x 6g GIB® Grabber® Drywall Screws.

Fastener Centres

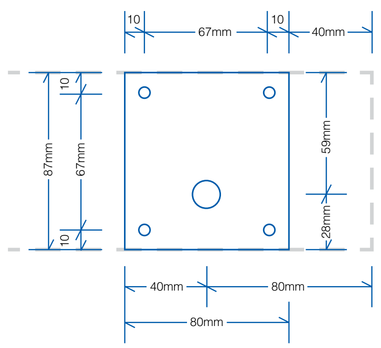
50,100,150,225,300mm from each corner and then 150mm thereafter around the perimeter of the bracing element. For vertically fixed sheets, place fasteners at 300mm centres to the intermediate sheet joints. For horizontally fixed sheets, place single fasteners to the sheet edge where it crosses the stud. Use daubs of GIBFix® adhesive at 300mm centres to intermediate studs. Place fasteners no closer than 12mm from paper bound sheet edges and 18mm from any sheet end or cut edge.

JOINTING

All fastener heads stopped and all sheet joints GIB® Joint Tape reinforced and stopped in accordance with the GIB® Site Guide.



Unless stated all fastener spacings are maximums.



GIB EzyBrace® end brace hold down washer (available from frame supplier)

In order for GIB® systems to perform as tested, all components must be installed exactly as prescribed. Substituting components produces an entirely different system and may seriously compromise performance. Follow the specifications. This specification sheet is issued in conjunction with the publication GIB EzyBrace® Systems

ARNOLD & JOHNSTONE 2015 LTD

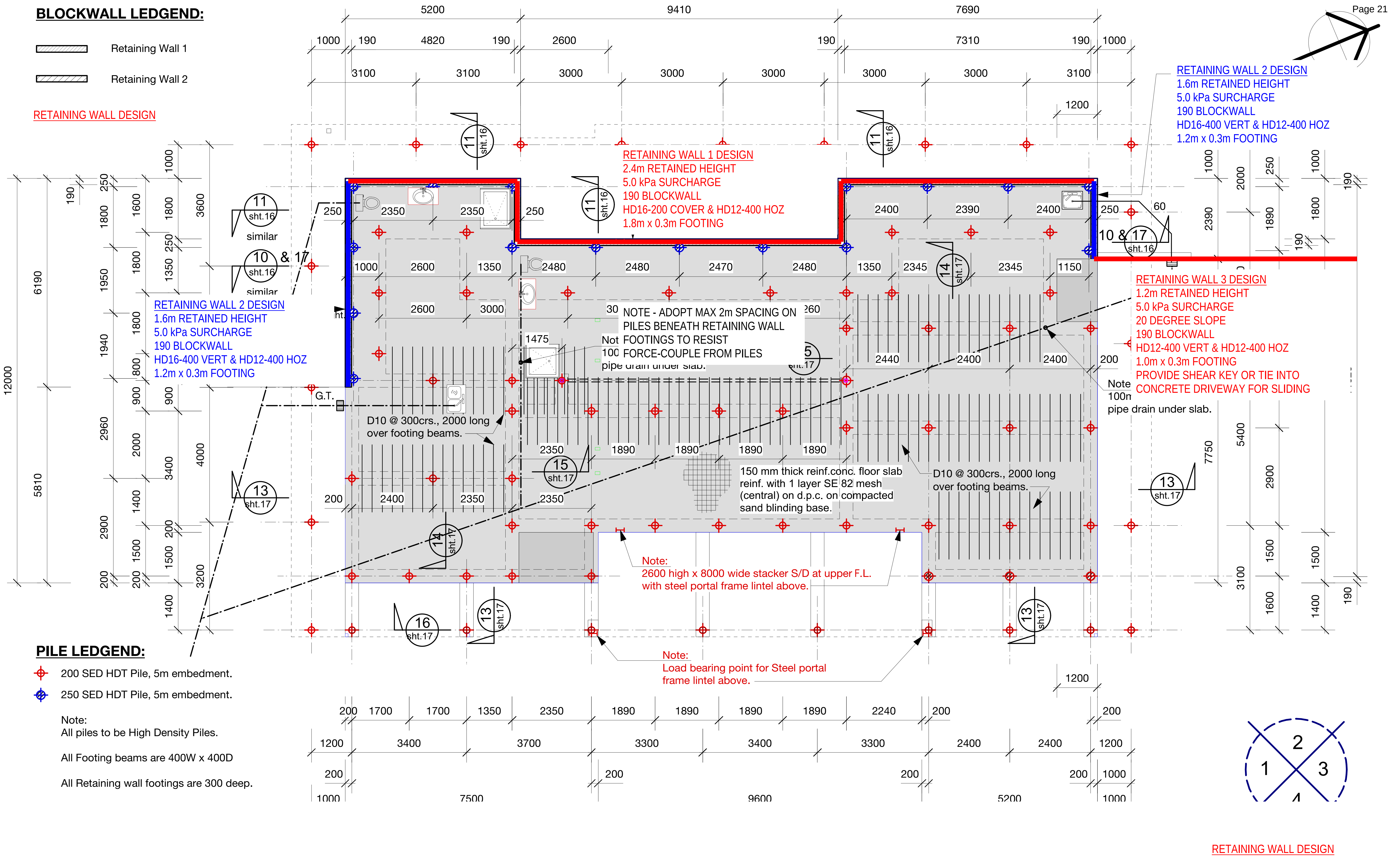
2. RETAINING WALL DESIGN

Ref: 12843

Hamilton City Council

BUILDING UNIT APPROVED

BC Number - DD007.2021.00043914.001


BLOCKWALL LEDGEND:
Retaining Wall 1
Retaining Wall 2
RETAINING WALL DESIGN
RETAINING WALL 1 DESIGN
2.4m RETAINED HEIGHT
5.0 kPa SURCHARGE
190 BLOCKWALL
HD16-200 COVER & HD12-400 HOZ
1.8m x 0.3m FOOTING
RETAINING WALL 2 DESIGN
1.6m RETAINED HEIGHT
5.0 kPa SURCHARGE
190 BLOCKWALL
HD16-400 VERT & HD12-400 HOZ
1.2m x 0.3m FOOTING
RETAINING WALL 3 DESIGN
1.2m RETAINED HEIGHT
5.0 kPa SURCHARGE
20 DEGREE SLOPE
190 BLOCKWALL
HD12-400 VERT & HD12-400 HOZ
1.0m x 0.3m FOOTING
PROVIDE SHEAR KEY OR TIE INTO
CONCRETE DRIVEWAY FOR SLIDING
NOTE - ADOPT MAX 2m SPACING ON
PILES BENEATH RETAINING WALL
FOOTINGS TO RESIST
FORCE-COUPLE FROM PILES
D10 @ 300crs., 2000 long
over footing beams.
150 mm thick reinf.conc. floor slab
reinf. with 1 layer SE 82 mesh
(central) on d.p.c. on compacted
sand blinding base.
Note
pipe drain under slab.
Note:
2600 high x 8000 wide stacker S/D at upper F.L.
with steel portal frame lintel above.
Note:
Load bearing point for Steel portal
frame lintel above.
G.T.
similar
PILE LEDGEND:
200 SED HDT Pile, 5m embedment.
250 SED HDT Pile, 5m embedment.
Note:
All piles to be High Density Piles.
All Footing beams are 400W x 400D
All Retaining wall footings are 300 deep.
RETAINING WALL DESIGN

RETAINING WALL 1 DESIGN

ARNOLD & JOHNSTONE
2015 LTD
CONSULTING CIVIL & STRUCTURAL ENGINEERS

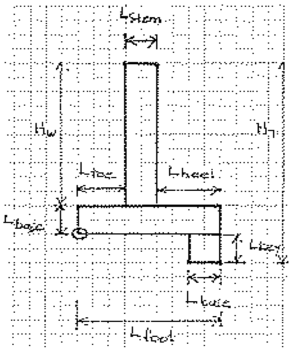
JOB 136 MORRINSVILLE ROAD

JOB No 12843
PAGE
BY NAM
DATE 09.08.2021

CANTILEVER RETAINING WALL DESIGN **COHESIONLESS SOIL**

Flexible Wall

Wall Parameters



Height of wall	H_w	2.4	m
Thickness of wall	T_{wall}	0.19	m
Length of Toe	L_{toe}	1.6	m
Foundation thickness	T_{found}	0.3	m
Length of heel	L_{heel}	0	m
Depth of shear key	D_{key}	0	m
Angle of backfill	β	0	°
Soil friction angle	$\varnothing$	30	°
interface friction angle	δ	20	°
Wall slope		0	°
Soil unit weight	γ_{soil}	18	kN/m^3
Concrete unit weight	γ_{wall}	24	kN/m^3
Surcharge, factored gravity case, de stabilising (1.2G+0.4Q)	S_g	5	kPa
Surcharge, factored EQ case, de stabilising (G+Eu+0.3Q)	S_g	5	kPa
Surcharge, factored gravity case, stabilising (0.9G)	S_{gs}	0	kPa
Effective stress angle of shearing (sliding)	$\varnothing'$	20	°
Width of footing	L_{foot}	1.79	m
Total height of structure	H_T	2.7	m
Weight of footing	W_{foot}	12.888	kn/m
Weight of key	W_{key}	0	kn/m
Weight of wall	W_{wall}	10.944	kn/m
Weight of soil above heel	W_{soil}	0	kn/m
Additional Weight (on wall)	W_{misc}	0	kn/m
Additional weight lever arm for inertia force EQ Stability	Lever arm Fbase W_{misc}	0	m

LRFD Parameters	Geotech Ult Bearing Pressure:	200 kPa	Design bearing pressures
Resistance factor for bearing capacity, gravity case	$\varnothing_{bc}$	0.5	100 kPa
Resistance factor for bearing capacity, Eq case	$\varnothing eq_{bc}$	0.8	160 kPa
Resistance factor for sliding, gravity case	$\varnothing_{sl}$	0.9	
Resistance factor for passive earth pressure, gravity case	$\varnothing_p$	0.5	
Resistance factor for sliding, Eq case	$\varnothing_{sl\ eq}$	0.9	
Resistance factor for passive earth pressure, Eq case	$\varnothing_{p\ eq}$	0.5	
Load factor for self weight (stabilising)	G_{stab}	0.9	
Load factor for self weight (de-stabilising)	G_{destab}	1.2	
Load factor for earth pressure, gravity case (de-stabilising)	EP_{static}	1.5	
Top of wall case Pinned P, Unsupported U		U	
Active Earth Pressure Coeffcient Ka or Ko	ka	0.333	
Passive Earth Pressure coeffcient	Kp	5.5	Navfac DM7

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JOB 136 MORRINSVILLE ROAD

Gravity Case - Stability

Active thrust, soil weight component	P_a=0.5 ka γ H_T^2	21.87	kn/m
horizontal component	Pah	20.55	kn/m
vertical component	Pav	7.48	kn/m
Active thrust surcharge component	P_{as}=Sg ka H_T	4.50	kn/m
horizontal component	Pash	4.23	kn/m
vertical component	Pasv	1.54	kn/m
Surcharge above heel	Pw=S_{gs} L_{heel}	0.00	kn/m
Factored moment from horizontal active pressure			
about bottom of toe	OTM	36.31	kn-m
Moment from vertical active pressure	RM	16.14	kn-m (-ve)
Moment from surcharge above heel	RM	0.00	kn-m
Factored moment from Self weight of wall and soil	RM	26.99	kn-m
Resultant moment	RM-OTM	6.82	kn-m
Ultimate factored horizontal load gravity case	1.5 Pa +1.5 Pas	37.17	kn/m
Vertical load on footing (wall and soil)	V_{total}	30.47	kn/m
Line of action from toe		0.22	m
effective footing width		0.45	m
Bearing pressure gravity case		68.02	kPa
	less than 100 kPa		**Bearing pressure o.k**

Gravity Case - Sliding

Weight of soil under footing	W_{slide}	0	kn/m
Passive resistance	P_p=0.5 kp γ Tfound+$Dkey^2$	4.46	kn/m
horizontal component	Pph	4.19	kn/m
vertical component	Ppv	1.52	kn/m
Friction under footing	Hs=($W_{slide*gstab)}$+V_{total}-ppv) tan Φ'	10.53	kn/m
Factored Gravity Sliding Resistance	Pp ϕp+Hs ϕsl	11.57	kn/m
			sliding Ngood

Bending moment in wall gravity case

Active thrust, soil weight component	P_a=0.5 ka γ Hw^2	17.28	kn/m
Active thrust surcharge component	P_{as}=Sg ka Hw	4.00	kn/m
Factored moment from horizontal active pressure			
about top of footing (wall design)	OTM	27.94	kn-m
Depth of section	*D*	190	mm
Distance to centre of tension steel	*d*	95	mm
Width of section	*Bw*	1000	mm
Area of steel mm2/m		1005	mm2
Steel yield strength	*fy*	500	Mpa
Concrete compressive strength	*f'c*	18	Mpa
	a	33	mm
Flexure strength reduction factor	*ϕ*	0.85	
	ϕMn	33.56	Kn-m
	Gravity BM in wall ok		

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Seismic Parameters

Passive earth coffcient Eq	KP eq	1	
Seismic coeffcient C(0)	CH(0) Z R	0.179	
Wd factor	Wd	0.7	wall displacement factor
Horizontal acceleration coeffcient	Kh=CH(0) Z R wd	0.125	
Wall is a flexible wall and Mononobe- Okabe equation used to calculate the earth pressure	K_{AE}	0.39	
Eq Active thrust increment flexible wall	Pae=0.5 Kae γ Ht^2	25.53	kn/m
horizontal component	Pah	23.99	kn/m
vertical component	Pav	8.73	kn/m
Eq Active thrust surcharge component	P_{aes}=Sg K Ht	5.25	kn/m
horizontal component	Pash	4.94	kn/m
vertical component	Pasv	1.80	kn/m
Surcharge above heel	Pw=S_{gs} L_{heel}	0.00	kn/m
Moment from horizontal EQ active pressure about bottom of toe	OTM	28.25	kn-m
Moment from structure inertia forces (including soil above the heel)	OTM	2.30	kn-m
Moment from vertical Eq pressure	RM	18.84	
Moment from surcharge above heel	RM	0.00	kn-m
Factored moment from Self weight of wall and soil	RM	29.98	kn-m
Resultant moment	RM-OTM	18.28	kn-m
Ultimate horizontal load Eq case		31.91	kn/m
Vertical load on footing (wall and soil)	V_{total}	34.36	kn/m
Line of action from toe		0.53	m
effective footing width		1.06	m
Bearing pressure Eq case		32.30	kPa
	less than 160 kPa		**Bearing pressure o.k**

Earthquake Case - Sliding short term

Passive resistance	P_p=0.5 kpeq γ(Tfound+Dkey)^2	0.81	kn/m	
Friction under footing	Hs=($W_{slide*gstab)}$+V_{total}-ppv) tan δ'	12.51	kn/m	δ' = Φ'
Factored EQ Sliding Resistance	Pp ϕp+Hs ϕsl	11.66	kn/m	
			sliding Ngood	

Bending moment in wall Eq case

Eq Active thrust increment	P_a=0.5 kae γ Hw^2	20.17	kn/m
Eq Active thrust increment surcharge component	P_{as}=Sg kh Hw	4.67	kn/m
Moment from structure inertia forces	OTM	1.65	kn-m
Factored moment from horizontal Eq thrust about base of wall	OTM	23.38	kn-m
Depth of section	*D*	190	mm
Distance to centre of tension steel	*d*	95	mm
Width of section	*Bw*	1000	mm
Area of steel mm2/m	*as*	1005	mm2
Steel yield strength	*fy*	500	Mpa
Concrete compressive strength	*f'c*	18	Mpa
	a	32.84	mm
Flexure strength reduction factor	ϕ	0.85	
	ϕMn	33.56	Kn-m
		EQ BM in wall ok	

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JOB 136 MORRINSVILLE ROAD

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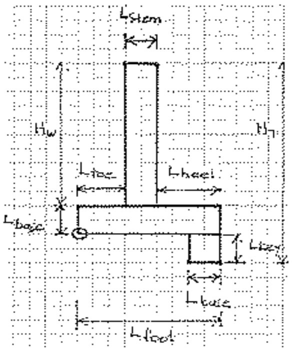
CANTILEVER RETAINING WALL DESIGN — **COHESIONLESS SOIL**

Flexible Wall

Wall Parameters



Height of wall	H_w	1.6	m
Thickness of wall	T_{wall}	0.19	m
Length of Toe	L_{toe}	1	m
Foundation thickness	T_{found}	0.3	m
Length of heel	L_{heel}	0	m
Depth of shear key	D_{key}	0	m
Angle of backfill	β	0	°
Soil friction angle	$\varnothing$	30	°
interface friction angle	δ	20	°
Wall slope		0	°
Soil unit weight	γ_{soil}	18	kN/m^3
Concrete unit weight	γ_{wall}	24	kN/m^3
Surcharge, factored gravity case, de stabilising (1.2G+0.4Q)	S_g	5	kPa
Surcharge, factored EQ case, de stabilising (G+Eu+0.3Q)	S_g	5	kPa
Surcharge, factored gravity case, stabilising (0.9G)	S_{gs}	0	kPa
Effective stress angle of shearing (sliding)	$\varnothing'$	20	°
Width of footing	L_{foot}	1.19	m
Total height of structure	H_T	1.9	m
Weight of footing	W_{foot}	8.568	kn/m
Weight of key	W_{key}	0	kn/m
Weight of wall	W_{wall}	7.296	kn/m
Weight of soil above heel	W_{soil}	0	kn/m
Additional Weight (on wall)	W_{misc}	0	kn/m
Additional weight lever arm for inertia force EQ Stability	Lever arm Fbase W_{misc}	0	m

LRFD Parameters	Geotech Ult Bearing Pressure:	200 kPa	Design bearing pressures
Resistance factor for bearing capacity, gravity case	$\varnothing_{bc}$	0.5	100 kPa
Resistance factor for bearing capacity, Eq case	$\varnothing eq_{bc}$	0.8	160 kPa
Resistance factor for sliding, gravity case	$\varnothing_{sl}$	0.9	
Resistance factor for passive earth pressure, gravity case	$\varnothing_p$	0.5	
Resistance factor for sliding, Eq case	$\varnothing_{sl\ eq}$	0.9	
Resistance factor for passive earth pressure, Eq case	$\varnothing_{p\ eq}$	0.5	
Load factor for self weight (stabilising)	G_{stab}	0.9	
Load factor for self weight (de-stabilising)	G_{destab}	1.2	
Load factor for earth pressure, gravity case (de-stabilising)	EP_{static}	1.5	
Top of wall case Pinned P, Unsupported U		U	
Active Earth Pressure Coeffcient Ka or Ko	ka	0.333	
Passive Earth Pressure coeffcient	Kp	5.5	Navfac DM7

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JOB 136 MORRINSVILLE ROAD

Gravity Case - Stability

Active thrust, soil weight component	P_a=0.5 ka γ H_T^2	10.83	kn/m
horizontal component	Pah	10.18	kn/m
vertical component	Pav	3.70	kn/m
Active thrust surcharge component	P_{as}=Sg ka H_T	3.17	kn/m
horizontal component	Pash	2.98	kn/m
vertical component	Pasv	1.08	kn/m
Surcharge above heel	Pw=S_{gs} L_{heel}	0.00	kn/m
Factored moment from horizontal active pressure			
about bottom of toe	OTM	13.91	kn-m
Moment from vertical active pressure	RM	5.70	kn-m (-ve)
Moment from surcharge above heel	RM	0.00	kn-m
Factored moment from Self weight of wall and soil	RM	11.69	kn-m
Resultant moment	RM-OTM	3.48	kn-m
Ultimate factored horizontal load gravity case	1.5 Pa +1.5 Pas	19.73	kn/m
Vertical load on footing (wall and soil)	V_{total}	19.06	kn/m
Line of action from toe		0.18	m
effective footing width		0.36	m
Bearing pressure gravity case		52.27	kPa
	less than 100 kPa		**Bearing pressure o.k**

Gravity Case - Sliding

Weight of soil under footing	W_{slide}	0	kn/m
Passive resistance	P_p=0.5 kp γ Tfound+$Dkey^2$	4.46	kn/m
horizontal component	Pph	4.19	kn/m
vertical component	Ppv	1.52	kn/m
Friction under footing	Hs=($W_{slide*gstab)}$+V_{total}-ppv) tan Φ'	6.38	kn/m
Factored Gravity Sliding Resistance	Pp ϕp+Hs ϕsl	7.84	kn/m
			sliding Ngood

Bending moment in wall gravity case

Active thrust, soil weight component	P_a=0.5 ka γ Hw^2	7.68	kn/m
Active thrust surcharge component	P_{as}=Sg ka Hw	2.67	kn/m
Factored moment from horizontal active pressure			
about top of footing (wall design)	OTM	9.34	kn-m
Depth of section	*D*	190	mm
Distance to centre of tension steel	*d*	95	mm
Width of section	*Bw*	1000	mm
Area of steel mm2/m	*as*	502	mm2
Steel yield strength	*fy*	500	Mpa
Concrete compressive strength	*f'c*	12	Mpa
	a	25	mm
Flexure strength reduction factor	*ϕ*	0.85	
	ϕMn	17.64	Kn-m
	Gravity BM in wall ok		

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Seismic Parameters

Passive earth coffcient Eq	KP eq	1		
Seismic coeffcient C(0)	CH(0) Z R	0.179		
Wd factor	Wd	0.7	wall displacement factor	
Horizontal acceleration coeffcient	Kh=CH(0) Z R wd	0.125		
Wall is a flexible wall and Mononobe- Okabe equation used to calculate the earth pressure	K_{AE}	0.39		
Eq Active thrust increment flexible wall	Pae=0.5 Kae γ Ht^2	12.64	kn/m	
horizontal component	Pah	11.88	kn/m	
vertical component	Pav	4.32	kn/m	
Eq Active thrust surcharge component	P_{aes}=Sg K Ht	3.70	kn/m	
horizontal component	Pash	3.47	kn/m	
vertical component	Pasv	1.26	kn/m	
Surcharge above heel	Pw=S_{gs} L_{heel}	0.00	kn/m	
Moment from horizontal EQ active pressure about bottom of toe	OTM	10.82	kn-m	
Moment from structure inertia forces (including soil above the heel)	OTM	1.17	kn-m	
Moment from vertical Eq pressure	RM	6.65		
Moment from surcharge above heel	RM	0.00	kn-m	
Factored moment from Self weight of wall and soil	RM	12.99	kn-m	
Resultant moment	RM-OTM	7.65	kn-m	
Ultimate horizontal load Eq case		17.34	kn/m	
Vertical load on footing (wall and soil)	V_{total}	21.45	kn/m	
Line of action from toe		0.36	m	
effective footing width		0.71	m	
Bearing pressure Eq case		30.09	kPa	
	less than	**160 kPa**	**Bearing pressure o.k**	
Earthquake Case - Sliding short term				
Passive resistance	P_p=0.5 kpeq γ(Tfound+Dkey)^2	0.81	kn/m	
Friction under footing	Hs=($W_{slide*gstab)}$+V_{total}-ppv) tan δ'	7.81	kn/m	$\delta' = \Phi'$
Factored EQ Sliding Resistance	Pp ϕp+Hs ϕsl	7.43	kn/m	
			sliding Ngood	
Bending moment in wall Eq case				
Eq Active thrust increment	P_a=0.5 kae γ Hw^2	8.96	kn/m	
Eq Active thrust increment surcharge component	P_{as}=Sg kh Hw	3.11	kn/m	
Moment from structure inertia forces	OTM	0.73	kn-m	
Factored moment from horizontal Eq thrust about base of wall	OTM	8.00	kn-m	
Depth of section	*D*	190	mm	
Distance to centre of tension steel	*d*	95	mm	
Width of section	*Bw*	1000	mm	
Area of steel mm2/m	*as*	502	mm2	
Steel yield strength	*fy*	500	Mpa	
Concrete compressive strength	*f'c*	12	Mpa	
	a	24.61	mm	
Flexure strength reduction factor	ϕ	0.85		
	ϕ*Mn*	17.64	Kn-m	
		EQ BM in wall ok		

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JOB 136 MORRINSVILLE ROAD

RETAINING WALL FOOTING DESIGN

TYPICAL FOOTING TO BE 300 THICK AND RESIST 28 kNm MOMENT

TRY HD16-400 TENSION REINFORCEMENT

MOMENT CAPACITY $\phi M_n = \phi A_s f_y jd$

REDUCTION FACTOR $\phi = 0.85$

AREA STEEL $A_s = \dfrac{\pi D^2}{4} \times \dfrac{1000}{s}$

$= \dfrac{\pi \times 16^2}{4} \times \dfrac{1000}{400}$ $= 503\ mm^2/m$

LEVER ARM $jd = 0.9d$

$= 0.9(300 - 75 - 8)$ $= 195\ mm$

$\phi M_n = 0.85 \times 503 \times 500 \times 195$ $= 42\ kNm$

$> 28\ kNm$ OK

ALONG FOOTING, MIN REINFORCEMENT SUFFICIENT

TRY HD12-300 REBAR TOP AND BOTTOM

AREA STEEL $A_{s,min} = \dfrac{1.4 b_w d}{f_y}$

$= \dfrac{1.4 \times 1000 \times 217}{500}$ $= 608\ mm^2/m$

AREA STEEL $A_s = \dfrac{\pi \times 12^2}{4} \times \dfrac{1000}{300} \times 2$ $= 754\ mm^2/m$

$> 608\ mm^2/m$ OK

ADOPT MIN HD12-300 STEEL TOP & BOTTOM ALONG FOOTING

ALTERNATIVELY, HD16-400 OK

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JOB 136 MORRINSVILLE ROAD

RETAINING WALL FOOTING DESIGN

TYPICAL FOOTING TO BE 300 THICK AND RESIST 28 kNm MOMENT

TRY HD16-400 TENSION REINFORCEMENT (OR HD12-200, $A_s = 565\ mm^2 > 503\ mm^2$ OK)

MOMENT CAPACITY $\phi M_n = \phi A_s f_y jd$

REDUCTION FACTOR $\phi = 0.85$

AREA STEEL $A_s = \frac{\pi D^2}{4} \times \frac{1000}{s}$

$= \frac{\pi \times 16^2}{4} \times \frac{1000}{400}$ $= 503\ mm^2/m$

LEVER ARM $jd = 0.9d$

$= 0.9(300 - 75 - 8)$ $= 195\ mm$

$\phi M_n = 0.85 \times 503 \times 500 \times 195$ $= 42\ kNm$

$> 28\ kNm$ OK

ALONG FOOTING, MIN REINFORCEMENT SUFFICIENT

TRY HD12-300 REBAR TOP AND BOTTOM

AREA STEEL $A_{s,min} = \frac{1.4 b_w d}{f_y}$

$= \frac{1.4 \times 1000 \times 217}{500}$ $= 608\ mm^2/m$

AREA STEEL $A_s = \frac{\pi \times 12^2}{4} \times \frac{1000}{300} \times 2$ $= 754\ mm^2/m$

$> 608\ mm^2/m$ OK

ADOPT MIN HD12-300 STEEL TOP & BOTTOM ALONG FOOTING

ALTERNATIVELY, HD16-400 OK

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CHECK FORCE-COUPLE IN PILES

FOR RETAINING WALL 1

MOMENT DEMAND $M^* = 28$ kNm/m

FOR PILES AT MAX 2.0m CENTRES

LEVER ARM $jd = 1.35$

FORCE COUPLE $N^*_c = \dfrac{28 \times 2}{1.35}$ = 41 kN

FOR LOAD CASE $1.2G + S_u + \psi_c Q$ FOR AN INTERNAL EDGE FOOTING

DEAD LOAD $G = 6.75 + 5.6$ = 12 kN/m

LIVE LOAD $Q = 7.5$ kN/m

ULS LOAD CASE $E_d = 1.2 \times 12 + 0.4 \times 7.5$ = 17 kN/m

TOTAL AXIAL LOAD $N^*_c = 41 + 17 \times 2$ = 75 kN

< 79 kN OK

FOR RETAINING WALL 2

MOMENT DEMAND $M^* = 9$ kNm/m

LEVER ARM $jd = 0.75$

FORCE COUPLE $N^*_c = \dfrac{9 \times 2}{0.75}$ = 24 kN

ULS LOAD CASE $E_d = 17$ kN/m

TOTAL AXIAL LOAD $N^*_c = 24 + 17 \times 2$ = 58 kN

< 79 kN OK

200SED PILES AT 2m SPACING OK FOR FORCE-COUPLE

RETAINING WALL 3 DESIGN

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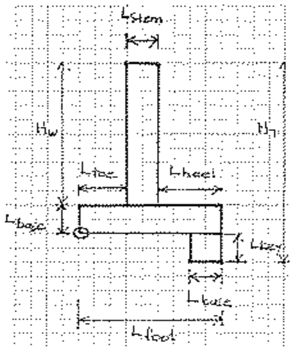
CANTILEVER RETAINING WALL DESIGN — **COHESIONLESS SOIL**

Flexible Wall

Wall Parameters



Height of wall	H_w	1.2	m
Thickness of wall	T_{wall}	0.19	m
Length of Toe	L_{toe}	0.8	m
Foundation thickness	T_{found}	0.3	m
Length of heel	L_{heel}	0	m
Depth of shear key	D_{key}	0	m
Angle of backfill	β	20	°
Soil friction angle	$\varnothing$	30	°
interface friction angle	δ	20	°
Wall slope		0	°
Soil unit weight	γ_{soil}	18	kN/m^3
Concrete unit weight	γ_{wall}	24	kN/m^3
Surcharge, factored gravity case, de stabilising (1.2G+0.4Q)	S_g	5	kPa
Surcharge, factored EQ case, de stabilising (G+Eu+0.3Q)	S_g	5	kPa
Surcharge, factored gravity case, stabilising (0.9G)	S_{gs}	0	kPa
Effective stress angle of shearing (sliding)	$\varnothing'$	20	°
Width of footing	L_{foot}	0.99	m
Total height of structure	H_T	1.5	m
Weight of footing	W_{foot}	7.128	kn/m
Weight of key	W_{key}	0	kn/m
Weight of wall	W_{wall}	5.472	kn/m
Weight of soil above heel	W_{soil}	0	kn/m
Additional Weight (on wall)	W_{misc}	0	kn/m
Additional weight lever arm for inertia force EQ Stability	Lever arm Fbase W_{misc}	0	m

LRFD Parameters	Geotech Ult Bearing Pressure:	200 kPa	Design bearing pressures
Resistance factor for bearing capacity, gravity case	$\varnothing_{bc}$	0.5	100 kPa
Resistance factor for bearing capacity, Eq case	$\varnothing eq_{bc}$	0.8	160 kPa
Resistance factor for sliding, gravity case	$\varnothing_{sl}$	0.9	
Resistance factor for passive earth pressure, gravity case	$\varnothing_p$	0.5	
Resistance factor for sliding, Eq case	$\varnothing_{sl\ eq}$	0.9	
Resistance factor for passive earth pressure, Eq case	$\varnothing_{p\ eq}$	0.5	
Load factor for self weight (stabilising)	G_{stab}	0.9	
Load factor for self weight (de-stabilising)	G_{destab}	1.2	
Load factor for earth pressure, gravity case (de-stabilising)	EP_{static}	1.5	
Top of wall case Pinned P, Unsupported U		U	
Active Earth Pressure Coeffcient Ka or Ko	ka	0.441	
Passive Earth Pressure coeffcient	Kp	5.5	Navfac DM7

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Gravity Case - Stability

Active thrust, soil weight component	P_a=0.5 ka γ H_T^2	8.93	kn/m
horizontal component	Pah	8.39	kn/m
vertical component	Pav	3.05	kn/m
Active thrust surcharge component	P_{as}=Sg ka H_T	3.31	kn/m
horizontal component	Pash	3.11	kn/m
vertical component	Pasv	1.13	kn/m
Surcharge above heel	Pw=S_{gs} L_{heel}	0.00	kn/m
Factored moment from horizontal active pressure			
about bottom of toe	OTM	9.79	kn-m
Moment from vertical active pressure	RM	4.14	kn-m (-ve)
Moment from surcharge above heel	RM	0.00	kn-m
Factored moment from Self weight of wall and soil	RM	7.49	kn-m
Resultant moment	RM-OTM	1.85	kn-m
Ultimate factored horizontal load gravity case	1.5 Pa +1.5 Pas	17.25	kn/m
Vertical load on footing (wall and soil)	V_{total}	15.53	kn/m
Line of action from toe		0.12	m
effective footing width		0.24	m
Bearing pressure gravity case		65.31	kPa
	less than 100 kPa		**Bearing pressure o.k**

Gravity Case - Sliding

Weight of soil under footing	W_{slide}	0	kn/m
Passive resistance	P_p=0.5 kp γ Tfound+$Dkey^2$	4.46	kn/m
horizontal component	Pph	4.19	kn/m
vertical component	Ppv	1.52	kn/m
Friction under footing	Hs=($W_{slide*gstab)}$+V_{total}-ppv) tan Φ'	5.10	kn/m
Factored Gravity Sliding Resistance	Pp ϕp+Hs ϕsl	6.68	kn/m
			sliding Ngood

Bending moment in wall gravity case

Active thrust, soil weight component	P_a=0.5 ka γ Hw^2	5.72	kn/m
Active thrust surcharge component	P_{as}=Sg ka Hw	2.65	kn/m
Factored moment from horizontal active pressure			
about top of footing (wall design)	OTM	5.81	kn-m
Depth of section	*D*	190	mm
Distance to centre of tension steel	*d*	95	mm
Width of section	*Bw*	1000	mm
Area of steel mm2/m	*as*	282	mm2
Steel yield strength	*fy*	500	Mpa
Concrete compressive strength	*f'c*	12	Mpa
	a	14	mm
Flexure strength reduction factor	*ϕ*	0.85	
	ϕMn	10.56	Kn-m
	Gravity BM in wall ok		

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Seismic Parameters

Passive earth coffcient Eq	KP eq	1	
Seismic coeffcient C(0)	CH(0) Z R	0.179	
Wd factor	Wd	0.7	wall displacement factor
Horizontal acceleration coeffcient	Kh=CH(0) Z R wd	0.125	
Wall is a flexible wall and Mononobe- Okabe equation used to calculate the earth pressure	K_{AE}	0.66	
Eq Active thrust increment flexible wall	Pae=0.5 Kae γ Ht^2	13.32	kn/m
horizontal component	Pah	12.52	kn/m
vertical component	Pav	4.56	kn/m
Eq Active thrust surcharge component	P_{aes}=Sg K Ht	4.93	kn/m
horizontal component	Pash	4.64	kn/m
vertical component	Pasv	1.69	kn/m
Surcharge above heel	Pw=S_{gs} L_{heel}	0.00	kn/m
Moment from horizontal EQ active pressure about bottom of toe	OTM	9.74	kn-m
Moment from structure inertia forces (including soil above the heel)	OTM	0.75	kn-m
Moment from vertical Eq pressure	RM	6.18	
Moment from surcharge above heel	RM	0.00	kn-m
Factored moment from Self weight of wall and soil	RM	8.33	kn-m
Resultant moment	RM-OTM	4.02	kn-m
Ultimate horizontal load Eq case		18.73	kn/m
Vertical load on footing (wall and soil)	V_{total}	18.84	kn/m
Line of action from toe		0.21	m
effective footing width		0.43	m
Bearing pressure Eq case		44.17	kPa
	less than	**160 kPa**	**Bearing pressure o.k**

Earthquake Case - Sliding short term

Passive resistance	P_p=0.5 kpeq γ(Tfound+Dkey)^2	0.81	kn/m
Friction under footing	Hs=($W_{slide*gstab)}$+V_{total}-ppv) tan δ'	6.86	kn/m $\delta' = \Phi'$
Factored EQ Sliding Resistance	Pp ϕp+Hs ϕsl	6.58	kn/m
			sliding Ngood

Bending moment in wall Eq case

Eq Active thrust increment	P_a=0.5 kae γ Hw^2	8.53	kn/m
Eq Active thrust increment surcharge component	P_{as}=Sg kh Hw	3.95	kn/m
Moment from structure inertia forces	OTM	0.41	kn-m
Factored moment from horizontal Eq thrust about base of wall	OTM	6.19	kn-m
Depth of section	*D*	190	mm
Distance to centre of tension steel	*d*	95	mm
Width of section	*Bw*	1000	mm
Area of steel mm2/m	*as*	282	mm2
Steel yield strength	*fy*	500	Mpa
Concrete compressive strength	*f'c*	12	Mpa
	a	13.82	mm
Flexure strength reduction factor	ϕ	0.85	
	ϕ*Mn*	10.56	Kn-m
		EQ BM in wall ok	

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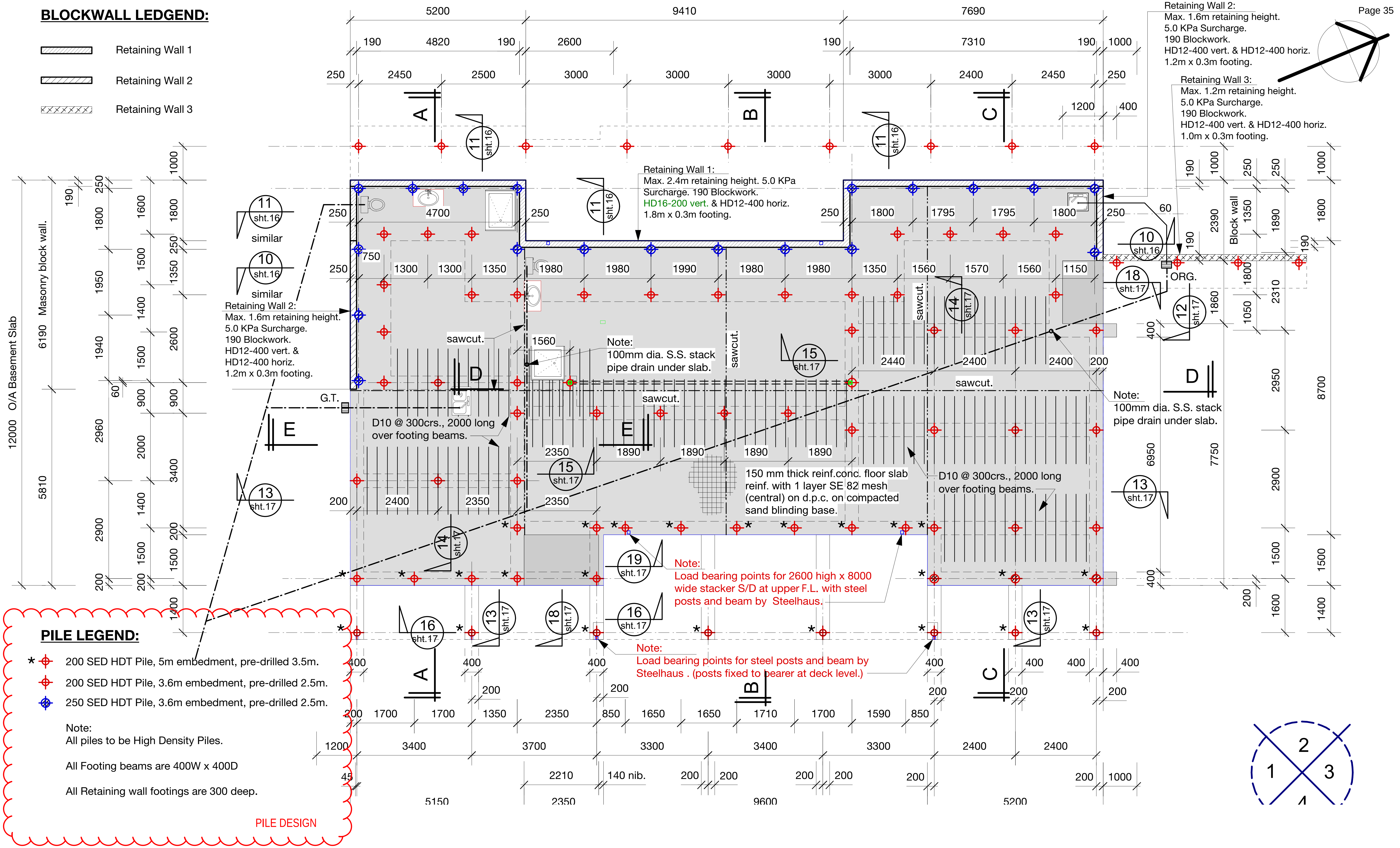
3. PILE DESIGN

Ref: 12843

Hamilton City Council

BUILDING UNIT
APPROVED

BC Number - DD007.2021.00043914.001


BLOCKWALL LEDGEND:
Retaining Wall 1
Retaining Wall 2
Retaining Wall 3
Retaining Wall 1:
Max. 2.4m retaining height. 5.0 KPa Surcharge. 190 Blockwork.
HD16-200 vert. & HD12-400 horiz.
1.8m x 0.3m footing.
Retaining Wall 2:
Max. 1.6m retaining height.
5.0 KPa Surcharge.
190 Blockwork.
HD12-400 vert. & HD12-400 horiz.
1.2m x 0.3m footing.
Retaining Wall 3:
Max. 1.2m retaining height.
5.0 KPa Surcharge.
190 Blockwork.
HD12-400 vert. & HD12-400 horiz.
1.0m x 0.3m footing.
Note:
100mm dia. S.S. stack pipe drain under slab.
D10 @ 300crs., 2000 long over footing beams.
150 mm thick reinf.conc. floor slab reinf. with 1 layer SE 82 mesh (central) on d.p.c. on compacted sand blinding base.
sawcut.
G.T.
ORG.
similar
12000 O/A Basement Slab
6190 Masonry block wall.
Block wall
Note:
Load bearing points for 2600 high x 8000 wide stacker S/D at upper F.L. with steel posts and beam by Steelhaus.
Note:
Load bearing points for steel posts and beam by Steelhaus. (posts fixed to bearer at deck level.)
140 nib.
PILE LEGEND:
200 SED HDT Pile, 5m embedment, pre-drilled 3.5m.
200 SED HDT Pile, 3.6m embedment, pre-drilled 2.5m.
250 SED HDT Pile, 3.6m embedment, pre-drilled 2.5m.
Note:
All piles to be High Density Piles.
All Footing beams are 400W x 400D
All Retaining wall footings are 300 deep.
PILE DESIGN

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JOB SH26 136 MORRINSVILLE ROAD

PILE DESIGN

BASEMENT TO BE SUPPORTED BY DRIVEN TIMBER PILES. ADOPT MIN 200 DIA × 7.6m DEEP PILES. AS PER TITUS REPORT, ADOPT A COHESIONLESS SOIL (SANDS).

REFER TO THE ATTACHED SPREADSHEET FOR FULL CALCULATIONS

PILE CAPACITY $\phi H_u = 80$ kN

CHECK CAPACITY USING HILEY FORMULA

$$\phi H_u = \frac{\phi W H n}{s + c/2}$$

FOR A 500 kg WEIGHT, 1m DROP, 15mm SET

$\phi H_u = 83$ kN REFER TO ATTACHED HILEY CALCULATIONS

> 180 kN

PILES SUPPORTING BASEMENT FLOOR ONLY

TRY PILES AT MAX 2.5m SPACING

DEAD LOAD FLOOR	$G_{floor} = 3.0 \times 0.15 \times 25$		$= 11.25$ kN/m
DEAD LOAD BEAM	$G_{beam} = 0.4 \times 0.4 \times 25$		$= 4$ kN/m
LIVE LOAD FLOOR	$Q_{floor} = 3.0 \times 2.5$		$= 7.5$ kN/m
ULS LOAD CASE	$E_d = 1.2G + 1.5Q$		
	$= (1.2 \times (11.25 + 4) + 1.5 \times 7.5) \times 2.5$		$= 74$ kN
	< 80 kN		OK

ADOPT PILES AT MAX 2.5m SPACING

PILES ON EASTERN BUILDING PERIMETER

CONSIDER WORST CASE LOADING ON EASTERN PERIMETER OF BUILDING

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JOB SH26 MORRINSVILLE ROAD

DEAD LOAD WALL	G_{wall}	= 0.4 kPa × 5.6 m	= 2.24 kN/m
DEAD LOAD FLOOR	G_{floor}	= 0.5 kPa × 2.2 m	= 1.1 kN/m
DEAD LOAD DECK	G_{deck}	= 0.5 kPa × 1.5 m	= 0.75 kN/m
DEAD LOAD SLAB	G_{slab}	= 1.5 × 0.15 × 25	= 5.63 kN/m
DEAD LOAD BEAM	G_{beam}	= 4 kN/m	
TOTAL DEAD LOAD	G	= 2.24 + 1.1 + 0.75 + 5.63 + 4	= 13.72 kN/m
LIVE LOAD FLOOR	Q_{floor}	= 1.5 × 2.2	= 3.3 kN/m
LIVE LOAD SLAB	Q_{slab}	= 2.5 × 1.9	= 4.75 kN/m
LIVE LOAD DECK	Q_{deck}	= 1.5 × 2.0	= 3 kN/m
TOTAL LIVE LOAD	Q	= 3.3 + 4.75 + 3	= 11.05 kN/m
ULS LOAD CASE	E_d	= 1.2 × 13.72 + 1.5 × 11.05	= 33 kN/m

MAX SPACING $s = \dfrac{\phi H_u}{E_d}$

$= \dfrac{80}{33}$ = 2.4 m

ADOPT PILES AT MAX 2.4 m CENTRES AROUND EASTERN PERIMETER IF SUPPORTING FLOOR AND ROOF ABOVE

PILES BENEATH POINT LOADS FROM BEAMS

TAKE POINT LOADS FROM BEAM DESIGN SPREADSHEET - TYPICALLY, 200 SED OK

BEAM ID	N^*_c (kN)
BEAM 01	52
BEAM 02	41
BEAM 03	27
BEAM 04	21
[illegible]	21
[illegible]	12
[illegible]	1

ADOPT 200 SED PILES BENEATH POINT LOADS

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JOB SH26 MORRINSVILLE ROAD

PILES SUPPORTING INTERNAL EDGE OF FOOTING

ADOPT THE FOLLOWING LOADS

DEADLOAD FOOTING $G_{foot} = 0.3 \times 0.9 \times 25 = 6.75$ kN/m

DEAD LOAD SLAB $G_{slab} = 0.15 \times 1.5 \times 25 = 5.6$ kN/m

LIVE LOAD SLAB $Q_{floor} = 2.5 \times 3.0 = 7.5$ kN/m

ULS LOAD CASE $E_d = 1.2 \times (6.75 + 5.6) + 1.5 \times 7.5 = 26.1$ kN/m

ADOPT 200 SED PILES

SPACING $s = \frac{80}{26.1} = 3.0m$

ADOPT PILES AT MAX 3.0m CENTRES

NOTE HAVE ADOPTED PILES AT MAX 2m CENTRES

$E_d = 26 \times 2 = 52$ kN OK

PILES SUPPORTING WESTERN BLOCKWALL

ADOPT THE FOLLOWING LOADS

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DEAD LOAD ROOF $G_{roof} = 0.45 \times 3.6$ DEAD LOAD DECK $= 1.6$ kN/m

DEAD LOAD FLOOR $G_{floor} = 0.5 \times 2.0$ $G_{deck} = 0.5 \times 1.4$ $= 1.0$ kN/m

DEAD LOAD WALL $G_{wall} = 0.4 \times 3.0$ $= 0.7$ kN/m $= 1.2$ kN/m

DEAD LOAD BLOCK WALL $G_{block} = 0.19 \times 3.0 \times 22$ $= 12.5$ kN/m

DEAD LOAD FOOTING $G_{foot} = 0.3 \times 0.9 \times 25$ $= 6.75$ kN/m

TOTAL DEAD LOAD $G = 1.6 + 1.0 + 1.2 + 12.5 + 6.75 + 0.7$ $= 24$ kN/m

LIVE LOAD ROOF $Q_{roof} = 0.25 \times 3.6$ LIVE LOAD DECK $= 0.9$ kN/m

LIVE LOAD FLOOR $Q_{floor} = 1.5 \times 2.0$ $Q_{deck} = 2.0 \times 1.4$ $= 3.0$ kN/m

LIVE LOAD FOOTING $Q_{foot} = 2.5 \times 0.9$ $= 2.8$ kN/m $= 2.3$ kN/m

TOTAL LIVE LOAD $Q = 0.9 + 3.0 + 2.3 + 2.8$ $= 9.0$ kN/m

ULS LOAD CASE $E_d = 1.2G + 1.5Q$

$= 1.2 \times 24 + 1.5 \times 9$ $= 42$ kN/m

ADOPT 250 SED PILES

PILE CAPACITY $\phi H_u = 124$ kN

FOR A 500 kg WEIGHT, 1m DROP, 10mm SET

$\phi H_u = 103$ kN REFER TO THE ATTACHED HILEY CALCULATIONS

ADOPT LOWER HILEY VALUE

MAX SPACING $= \frac{103}{42}$ $= 2.45$ m

ADOPT A MAX SPACING OF 2.0 m

PILE DEMAND = 42 x 2 = 84 kN

PILE AREA = PI x 0.25^2 / 4 = 0.05m2

BEARING PRESSURE (ASSUMING NO SKIN FRICTION) = 84/0.05 = 1700 kPa

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JOB SH26 136 MORRINSVILLE ROAD

HILEY DRIVING CRITERIA

ULTIMATE DRIVING RESISTANCE $R = \dfrac{W h \eta}{s + c/2}$

200 SED PILE

FOR A 500 kg HAMMER, 1m DROP,

WEIGHT $W = 500\,kg \times 9.81/1000 = 4.9\,kN$

FOR A WINCH OPERATED HAMMER

FREE FALL HEIGHT $h = 0.8 \times 1000 = 800\,mm$

FOR A 5m DEEP PILE WITH A 8mm/m TAPER

AVERAGE PILE AREA $A_p = \dfrac{\pi \times (0.2 + 0.5 \times 5 \times 0.008)^2}{4} = 0.038\,m^2$

PILE DENSITY $\gamma = 5\,kN/m^3$

PILE WEIGHT $P = 0.038 \times 5 \times 5 = 0.95\,kN$

COEFFICIENT OF RESTITUTION $e = 0.25$

$P_e = 0.95 \times 0.25 = 0.24$

$< 4.9\,kN$

EFFICIENCY $\eta = \dfrac{(W + Pe^2)}{W + P}$

$= \dfrac{4.9 + 0.95 \times 0.25^2}{4.9 + 0.95} = 0.85$

TEMPORARY COMPRESSIONS $c = 10\,mm$

REDUCTION FACTOR $\phi = 0.5$

FOR A 15mm SET

PILE CAPACITY $\phi R = \dfrac{0.5 \times 4.9 \times 800 \times 0.85}{15 + 10 \times 0.5} = 83\,kN$

ADOPT A 15mm SET FOR ALL 200SED PILES

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250 SED PILE

FOR A 5m DEEP PILE WITH A 8mm/m TAPER

AVERAGE PILE AREA $A_p = \dfrac{\pi(0.25 + 0.5 \times 5 \times 0.008)^2}{4}$ $= 0.057\ m^2$

PILE WEIGHT $P = 0.057 \times 5 \times 5$ $= 1.43\ kN$

EFFICIENCY $\eta = \dfrac{4.9 + 1.43 \times 0.25^2}{4.9 + 1.43}$ $= 0.79$

FOR A 10mm SET

PILE CAPACITY $\phi R = \dfrac{0.5 \times 4.9 \times 800 \times 0.79}{10 + 10 \times 0.5}$ $= 103\ kN$

ADOPT A 10mm SET FOR ALL 250 SED PILES

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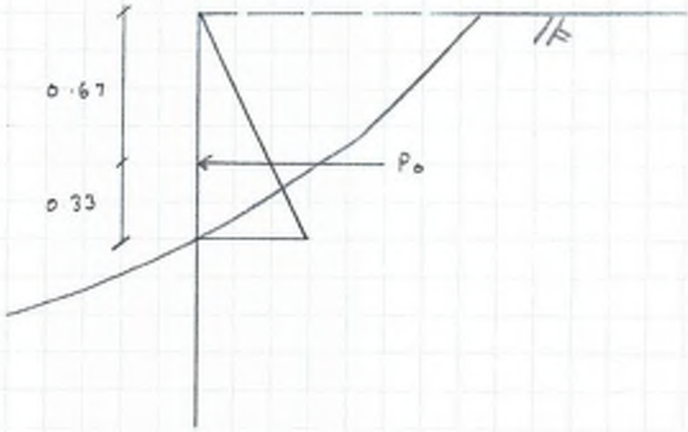
PILE STABILITY

PILES LOCATED NEAR SLOPE AS PER TITUS MODELLING, DESIGN LEADING EDGE PILES FOR A LOSS OF SUPPORT OF APPROX 1m. FOR 5m DEEP PILES ON THE LEADING EDGE



SLIP CIRCLE

AT THE FAILURE CASE THE PILES WILL BE SUPPORTING AT REST PRESSURE SOIL BEHIND PILE PREDOMINANTLY SANDS

SOIL DENSITY $\gamma = 18 \text{ kN/m}^3$

FRICTION ANGLE $\phi = 36$

LATERAL EARTH PRESSURE COEFFICIENT $K_o = 1 - \sin\phi$

$= 1 - \sin 36 \qquad = 0.41$

FOR 200 DIA PILES, ADOPT A 3x DIA SOIL WEDGE OF PRESSURE ACTING ON PILE

LATERAL LOAD $P_o = LF \times 0.5 \times K_o \times \gamma \times H^2 \times 3 \times DIA$

$= 1.5 \times 0.5 \times 0.41 \times 18 \times 1^2 \times 3 \times 0.2 \qquad = 3.3 \text{ kN}$

FOR A CANTILEVERED PILE

MOMENT DEMAND $M^* = 3.3 \times 0.33 \qquad = 1.1 \text{ kNm}$

FOR A 200 SED HOT PILE

MOMENT CAPACITY $\phi M_n = \frac{17 \times 52}{38} \qquad = 23 \text{ kNm}$

$> 1.1 \text{ kNm}$ ok

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WHAT IF LOSS OF SUPPORT WAS 2m (SAY)

LATERAL LOAD $P_0 = 1.5 \times 0.5 \times 0.41 \times 18 \times 2^2 \times 3 \times 0.2$ $= 13$ kN

MOMENT DEMAND $M^* = \dfrac{13 \times 2}{3}$ $= 8.9$ kNm

< 23 kNm OK

EVEN IF LOSS OF SUPPORT > 1m, PILE STILL OK

CHECK COMBINED AXIAL + MOMENT DEMANDS

AXIAL DEMAND $N^*_c = 80$ kN (SAY, CONSERVATIVE)

COMPRESSION CAPACITY $\phi N_c = 475$ kN

RATIO $\dfrac{M^*}{\phi M_n} + \dfrac{N^*_c}{\phi N_c} = \dfrac{8.9}{23} + \dfrac{80}{475}$ $= 0.56$

< 1.0 OK

CHECK WHETHER A 5m EMBEDMENT DEPTH IS ADEQUATE. ADOPT SOIL PROPERTIES AT APPROX LEVEL OF SLIP CIRCLE

PASSIVE PRESSURE COEFFICIENT $K_p = \dfrac{1 + \sin\phi}{1 - \sin\phi}$

$= \dfrac{1 + \sin 36}{1 - \sin 36}$ $= 3.85$

FOR A 2m EMBEDMENT (1m BELOW SLIP CIRCLE)

PILE CAPACITY $\phi H_u = 4.2$ kN REFER ATTACHED SPREADSHEET

> 3.3 kN OK

HOWEVER, CALCULATIONS ASSUMES LEVEL GROUND BEHIND PILE. SLIP CIRCLE WILL REDUCE AREA BEHIND PILE TO DEVELOP PRESSURE. DEEPEN PILE TO ACHIEVE REQUIRED AREA

ANGLE $\theta = 45 + \dfrac{36}{2}$ $= 63°$

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AREA BELOW LEVEL LINE = 0.92 m^2

FOR A 3m EMBEDMENT

AREA BELOW SLIP CIRCLE = 1.63 m^2

> 0.92 m^2 OK

A 5m EMBEDMENT IS ADEQUATE

Pile Vertical Strength B1/VM4

Cohesionless Soil

Vu=Vsu+Vbu

Base Resistance

Vbu=	(9c'+q'Nq+0.6DΓNϒ)Ab		=	155.1	kN
c'	effective stress cohesion is 0 for cohesionless soils		=	0	kPa
q'	Vertical stress	q'=Hϒ	=	64.8	kPa
	Pile depth	H	=	3.6	m
	Soil density	ϒ	=	18	kN/m^3
Nq	Bearing strength factor Figure 4	Nq	=	75	
ϕ	Angle of shearing resistance		=	36	
Ab	Area of base of pile		=	0.031	m^2
	Pile diameter	D	=	0.2	m
Γ	ϒ when the water tableis deeper than 2B beneath the underside of the foundation and ϒ' when the water table is above this.	ϒ or ϒ'	=	18	kN/m^3
Nϒ	Bearing strength factor Figure 3		=	35	

Shaft Resistance

Vsu=(σ'$K_s$tanδ')average*CL			=	5	kN
D	Pile diameter		=	0.2	
C	Pile circumference		=	0.63	m
L (H)	Pile shaft length		=	1.1	m
Ks	Factor from Table 2		=	1.5	
q'	Vertical stress	q'=Hϒ	=	19.8	kPa
	Pile depth	H	=	1.1	m
	Soil density	ϒ	=	18	kN/m^3
σ'	Vertical effective stress	q/2	=	9.9	
δ'	drained angle of shearing	2ϕ/3	=	23.76	degrees
		2ϕ/3	=	0.415	rads
ϕ	angle of shearing resistance		=	36	

ϕVu	ϕVu=ϕVsu+ϕVbu	=	80	
ϕ	Strength reduction factor	=	0.5	kN

Pile Vertical Strength B1/VM4

Cohesionless Soil

Vu=Vsu+Vbu

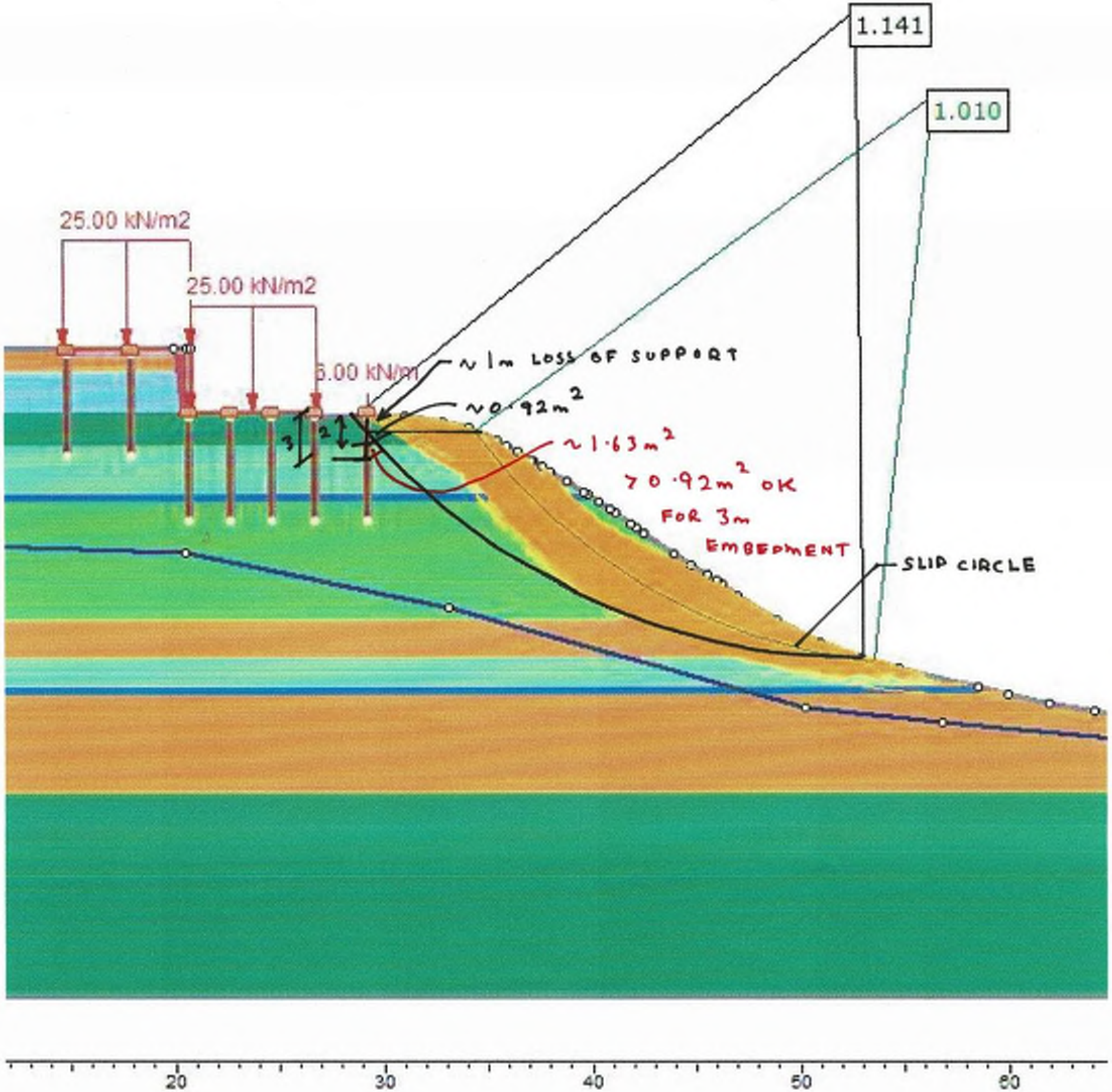
Base Resistance

Vbu=	(9c'+q'Nq+0.6DΓNϒ)Ab		=	242.7	kN
c'	effective stress cohesion is 0 for cohesionless soils		=	0	kPa
q'	Vertical stress	q'=Hϒ	=	64.8	kPa
	Pile depth	H	=	3.6	m
	Soil density	ϒ	=	18	kN/m^3
Nq	Bearing strength factor Figure 4	Nq	=	75	
ϕ	Angle of shearing resistance		=	36	
Ab	Area of base of pile		=	0.049	m^2
	Pile diameter	D	=	0.25	m
Γ	ϒ when the water tableis deeper than 2B beneath the underside of the foundation and ϒ' when the water table is above this.	ϒ or ϒ'	=	16	kN/m^3
Nϒ	Bearing strength factor Figure 3		=	35	

Shaft Resistance

Vsu=(σ'$K_s$tanδ')average*CL			=	6	kN
D	Pile diameter		=	0.25	
C	Pile circumference		=	0.79	m
L (H)	Pile shaft length		=	1.1	m
Ks	Factor from Table 2		=	1.5	
q'	Vertical stress	q'=Hϒ	=	19.8	kPa
	Pile depth	H	=	1.1	m
	Soil density	ϒ	=	18	kN/m^3
σ'	Vertical effective stress	q/2	=	9.9	
δ'	drained angle of shearing	2ϕ/3	=	23.76	degrees
		2ϕ/3	=	0.415	rads
ϕ	angle of shearing resistance		=	36	

ϕVu	ϕVu=ϕVsu+ϕVbu	=	124	
ϕ	Strength reduction factor	=	0.5	kN



ULS (Ultimate Limit State) – FOS required: 1.1

Table 7: FOS Standard Requirements

Modelled Loading Condition	FOS Required
Gravity Conditions	1.5
Gravity Conditions (elevated water table)	1.3
Seismic SLS (Serviceability Limit State)	1.5
Seismic ULS (Ultimate Limit State)	1.1

5.7 Adopted Subsurface Conditions

The stratigraphy as determined by TITUS CIVIL Consulting Engineers with reference to CPT logs for lot 4 undertaken by OPUS, has been separated into the different materials displayed in the Table below.

Table 8: Material characteristics

Material Name	Color	Unit Weight (kN/m3)	Sat. Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Hu	Ru
Topsoil		17	19.7	Mohr-Coulomb	2	28		Water Surface	Automatically Calculated		
Coarse Dense Sands		18	20.7	Mohr-Coulomb	2	40		Water Surface	Automatically Calculated		
Coarse Sands		18	20.7	Mohr-Coulomb	2	38		Water Surface	Automatically Calculated		
Medium to Coarse Sands		18	20.7	Mohr-Coulomb	2	37		Water Surface	Automatically Calculated		
Medium Sands		18	20.7	Mohr-Coulomb	2	36		Water Surface	Automatically Calculated		
Medium to Fine Sands		19	21	Mohr-Coulomb	2	35		Water Surface	Automatically Calculated		
Fine Silts		17	19.7	Mohr-Coulomb	3	32		Water Surface	Automatically Calculated		
Holocene Sediments		13	14	Mohr-Coulomb	0	32		Water Surface	Automatically Calculated		
Free Draining hardfill		18	20.7	Mohr-Coulomb	2	37		Water Surface	Automatically Calculated		
Concrete Retaining Wall		25		Undrained	650		Constant	None			0
Concrete Floor		24		Mohr-Coulomb	30	40		None			0
Liquefied Layer		18		Undrained	2		Constant	Water Surface	Custom	0	

5.8 Groundwater Model

The water table has been modelled at 12.0m below the ground surface at the top of the slope and 0.3m below the surface at the bottom except in the elevated water table conditions.

The elevated water table has been modelled at 0.9m below the ground surface at the top of the slope and 0.9m above the ground surface at the bottom of the slope as the gully is expected to flood during a large storm event.

5.9 Loading

Loadings applied to each model are shown in the Table below. The location of loadings may be found in Appendix H.

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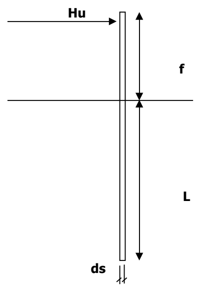
initials	**NAM**

client: **Martin Cameron** address: **SH26 136 Morrinsville Road**

Free Head Pile Capacity

Short Free head piles in cohesionless soil

Hu=Kp Ds $L^3\gamma$ / 2(L+F)



Φ	**0.8**	Strength reduction factor
F	**0.33**	(m) Distance above ground for horizontal shear
Kp	**3.85**	Passive Pressure coeffcient
Ds	**0.2**	(m) embedded pile diameter
γ	**18**	Unit weight of soil
L	**1**	(m) Length of pile shaft
2(L+F)	2.66	
Kp Ds $L^3\gamma$	13.86	
Hu	5.2	(Kn) Ultimate lateral resistance of pile
ΦHu	4.2	(Kn) Factored Ultimate lateral resistance of pile (short)
ΦMn	1.4	(Knm) Allowable Maximum moment (short) * based on max moment at glevel
Gc	0.5	(m) Location max pile moment from ground (short)
Mmax	3	(Kn-m) Maximum pile moment
Mult	23	(Kn-m) Pile section capacity

Long Free head piles in cohesionless soil

Hu	22.8	(Kn) Ultimate lateral resistance of pile
ΦHu	18.2	(Kn) Factored Ultimate lateral resistance of pile (long)
Gc	1.0	(m) Location max pile moment from ground (long)
Mmax Long	23	
	0	
Governing Equation		**Short Pile Governs**
ΦHu	4.2	Kn

2*hu	45.6
3*kp*ds*g	41.58
	0.698154
	1.03
	0.4

4. FOOTING BEAM DESIGN

Ref: 12843

Hamilton City Council

BUILDING UNIT
APPROVED

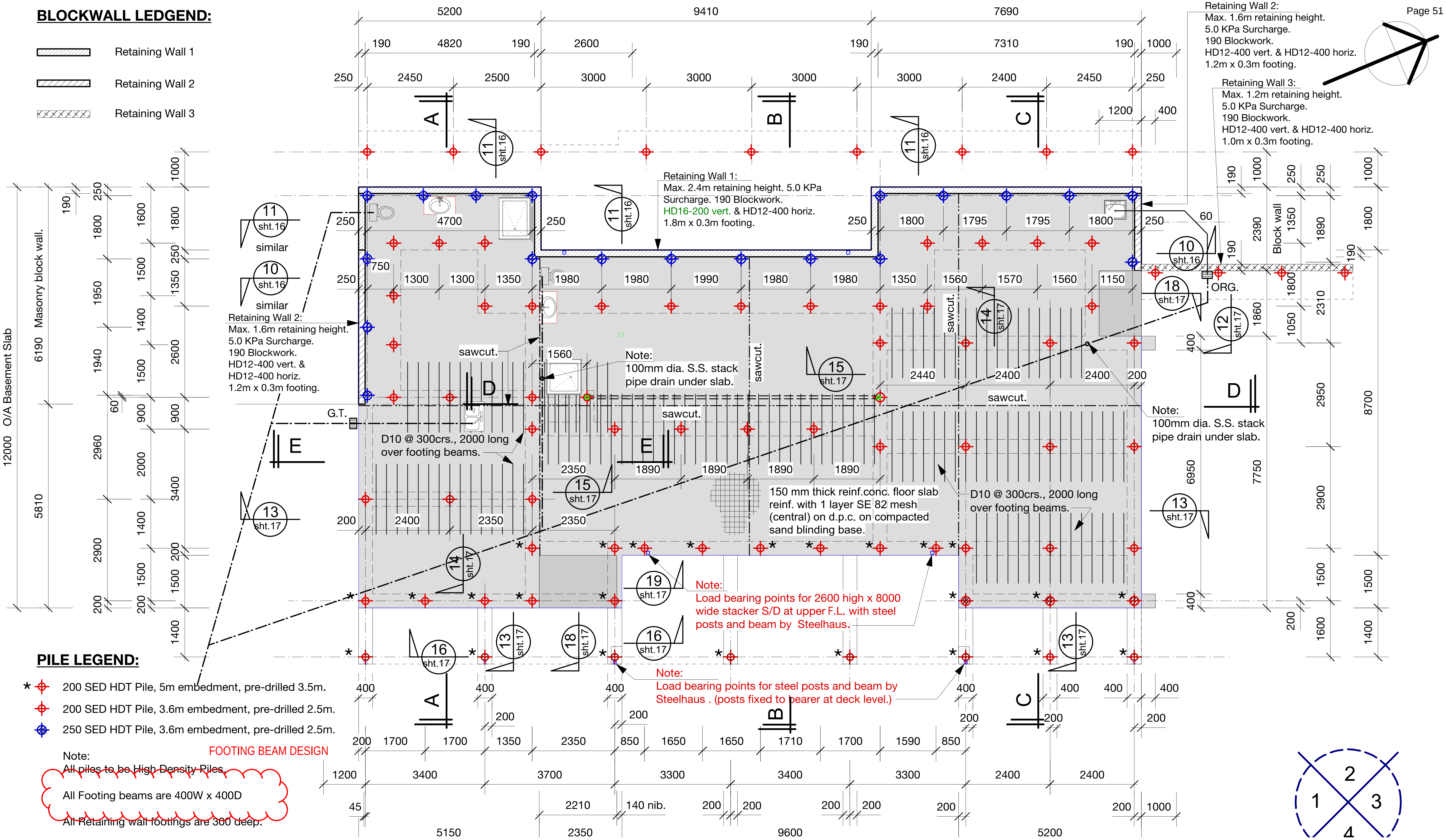
BC Number - DD007.2021.00043914.001

BLOCKWALL LEDGEND:

Retaining Wall 1

Retaining Wall 2

Retaining Wall 3



PILE LEGEND:

- * 200 SED HDT Pile, 5m embedment, pre-drilled 3.5m.
- 200 SED HDT Pile, 3.6m embedment, pre-drilled 2.5m.
- 250 SED HDT Pile, 3.6m embedment, pre-drilled 2.5m.

Note:

FOOTING BEAM DESIGN

All piles to be High Density Piles.

All Footing beams are 400W x 400D

All Retaining wall footings are 300 deep.

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JOB SH26 136 MORRINSVILLE ROAD

FOOTING BEAM DESIGN

CONSIDER WORST CASE FOOTING SPANNING 2·5m

ULS LOAD CASE $E_d = 33\ kN/m$

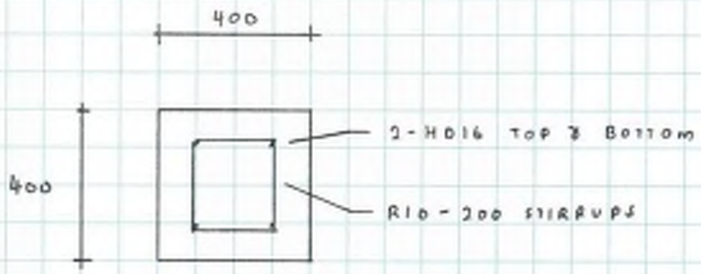
MOMENT DEMAND $M^* = \dfrac{wL^2}{8}$

$= \dfrac{33 \times 2.5^2}{8} = 26\ kNm$

SHEAR DEMAND $V^* = \dfrac{wL}{2}$

$= \dfrac{33 \times 2.5}{2} = 41\ kN$

TRY 400W × 400D FOOTING



MOMENT CAPACITY $\phi M_n = \phi A_s f_y jd$

REDUCTION FACTOR $\phi = 0.85$

AREA STEEL $A_s = \dfrac{\pi D^2}{4} \times n$

$= \dfrac{\pi \times 16^2}{4} \times 2 = 402\ mm^2$

LEVER ARM $jd = 0.9d$

$= 0.9 \times 307 = 276\ mm$

$\phi M_n = 0.85 \times 402 \times 500 \times 276 = 47\ kNm$

$> 26\ kNm$ OK

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BASIC SHEAR STRESS $v_b = (0.07 + 10\rho_w)\sqrt{f'_c}$

REINFORCEMENT RATIO $\rho_w = \dfrac{A_s}{b_w d}$

$= \dfrac{402}{400 \times 307} = 3.27 \times 10^{-3}$

FOR 20 MPa CONCRETE $v_b = (0.07 + 10 \times 3.27 \times 10^{-3})\sqrt{20} = 0.46$ MPa

CONCRETE MECHANISM $V_c = k_a k_d v_b b_w d$

FACTORS $k_a k_d = 1.0$

$V_c = 0.46 \times 400 \times 307 = 56$ kN

REBAR MECHANISM $V_s = \dfrac{A_v f_{yt} d}{s}$

STIRRUP AREA $A_v = \dfrac{\pi \times 10^2}{4} \times 2 = 157$ mm^2

$V_s = \dfrac{157 \times 300 \times 307}{200} = 72$ kN

REDUCTION FACTOR $\phi = 0.75$

SHEAR CAPACITY $\phi V_n = \phi(V_c + V_s)$

$= 0.75(56 + 72) = 96$ kN

> 41 kN OK

ADOPT 400 D × 400 W BEAM, WITH 2-HD16 TOP & BOTTOM & R10-200 STIRRUPS

BLOCKWALL FOOTING

CONSIDER WORST LOAD CASE

ULS LOAD CASE $E_d = 42$ kN/m AT 2m SPACING

MOMENT DEMAND $M^* = \dfrac{42 \times 2^2}{8} = 21$ kNm

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JOB SH26 MORRINSVILLE ROAD

SHEAR DEMAND $V' = \frac{42 \times 2}{2}$ $= 42$ kN

TRY 4-HD12 TOP & BOTTOM IN 1.2m WIDE × 0.3m DEEP FOOTING. SHEAR OK BY INSPECTION, CHECK MOMENT DEMAND.

MOMENT CAPACITY $\phi M_n = \phi A_s f_y jd$

REDUCTION FACTOR $\phi = 0.85$

AREA STEEL $A_s = \frac{\pi D^2}{4} \times n$

$= \frac{\pi \times 12^2}{4} \times 4$ $= 452$ mm²

LEVER ARM $jd = 0.9d$

$= 0.9 \times 225$ $= 203$ mm

$\phi M_n = 0.85 \times 452 \times 500 \times 203$ $= 39$ kNm

> 21 kNm OK

ADOPT MIN 4-HD12 T & B BARS IN BLOCKWALL FOOTINGS HD16 BARS ADOPTED

5. FLOOR SLAB DESIGN

Ref: 12843

Hamilton City Council
Te kaunihera o Kirikiriroa

BUILDING UNIT
APPROVED

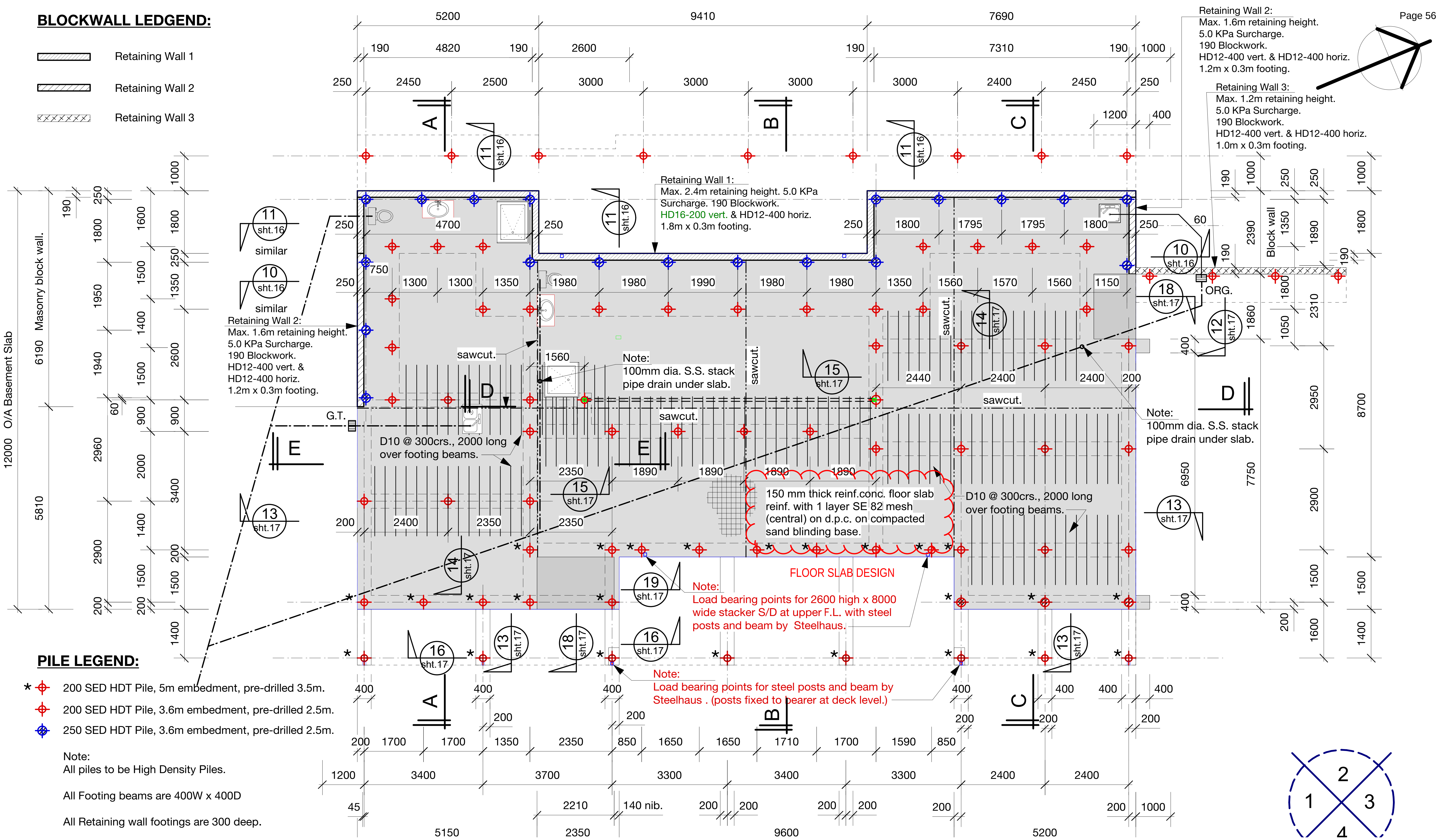
BC Number - DD007.2021.00043914.001

BLOCKWALL LEDGEND:

Retaining Wall 1

Retaining Wall 2

Retaining Wall 3



PILE LEGEND:

* 200 SED HDT Pile, 5m embedment, pre-drilled 3.5m.
200 SED HDT Pile, 3.6m embedment, pre-drilled 2.5m.
250 SED HDT Pile, 3.6m embedment, pre-drilled 2.5m.

Note:
All piles to be High Density Piles.

All Footing beams are 400W x 400D

All Retaining wall footings are 300 deep.

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FLOOR SLAB DESIGN

FLOOR SLAB TO COMPRISE SOLID 150 THK FLOOR SLAB. SLAB TO SPAN ONE-WAY BETWEEN FOOTING BEAMS

GARAGE SLAB

DEAD LOAD SLAB $G_{SLAB} = 0.15 \text{ kPa} \times 25 = 3.75 \text{ kPa}$

LIVE LOAD GARAGE $Q_{gar} = 2.5 \text{ kPa}$

FOR A MAX SPAN OF 2.9m, TRY ONE END CONTINUOUS

MIN DEPTH $d_{min} = \frac{L}{20}$

$= \frac{2900}{20} = 145 \text{ mm}$

ULS LOAD CASE $E_d = 1.2G + 1.5Q$

$= 1.2 \times 3.75 + 1.5 \times 2.5 = 8.25 \text{ kPa}$

SAGGING MOMENT $M^* = \frac{9wL^2}{128}$

$= \frac{9 \times 8.25 \times 2.9^2}{128} = 4.9 \text{ kNm/m}$

TRY SE82 MESH

MOMENT CAPACITY $\phi M_n = \phi A_s f_y jd$

REDUCTION FACTOR $\phi = 0.85$

AREA STEEL $A_s = 251 \text{ mm}^2/\text{m}$

STEEL YIELD STRENGTH $f_y = 500 \text{ MPa}$

LEVER ARM $jd = 0.9d$

$= 0.9 \times 0.5 \times 150 = 68 \text{ mm}$

$\phi M_n = 0.85 \times 251 \times 500 \times 68 = 7.3 \text{ kNm/m}$

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> 4.9 kNm/m OK

CHECK HOGGING MOMENT

MOMENT DEMAND $M' = \frac{wL^2}{8}$

$= \frac{8.25 \times 2.9^2}{8}$ $= 8.7$ kNm/m

> 7.3 kNm/m

PROVIDE ADDITIONAL HOGGING STEEL. TRY D10 - 300

AREA STEEL $A_s = \frac{\pi D^2}{4} \times \frac{1000}{s}$

$= \frac{\pi \times 10^2}{4} \times \frac{1000}{300}$ $= 262$ mm^2/m

MOMENT CAPACITY $\phi M_n = 7.3 + 0.85 \times 262 \times 300 \times 63$ $= 11.5$ kNm/m

> 8.7 kNm/m OK

PROVIDE SE82 MESH AND D10 - 300 HOGGING STEEL

RUMPUS SLAB

DEAD LOAD SLAB $G_{slab} = 3.75$ kPa

DEAD LOAD PARTITIONS $G_{par} = 0.5$ kPa

TOTAL DEAD LOAD $G = 3.75 + 0.5$ $= 4.25$ kPa

LIVE LOAD $Q = 2.5$ kPa

ULS LOAD CASE $E_d = 1.2 \times 4.25 + 1.5 \times 2.5$ $= 8.85$ kPa

FOR A MAX CLEAR SPAN OF 3.0m, TRY ONE END CONTINUOUS AND SE82 MESH

MIN DEPTH $d_{min} = \frac{3000}{20}$ $= 150$ mm

$= 150$ mm

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JOB No 12843

PAGE

BY NAM

JOB 136 MORRINSVILLE ROAD

DATE 31.05.2021

MOMENT DEMAND SAGGING $M^* = \dfrac{9 \times 8.85 \times 3^2}{128}$ = 5.6 kNm/m

< 7.3 kNm/m OK

HOGGING DEMAND $M^* = \dfrac{8.85 \times 3^2}{8}$ = 10 kNm/m

< 11.5 kNm/m OK

PROVIDE SE82 MESH AND D10-300 HOGGING STEEL

CHECK PUNCHING SHEAR

POINT LOAD $Q = 13$ kN

PUNCHING WIDTH $B_p = \sqrt{350}$ = 18.7 mm

FOR A 45° SPREAD PERIMETER AT MID DEPTH

PERIMETER $P_o = 4 \times (75 \times 2 + 18.7)$ = 675 mm

SHEAR STRESS $v = \dfrac{1.5Q}{P_o\, d}$

$= \dfrac{1.5 \times 13 \times 1000}{675 \times 150}$ = 0.19 MPa

MIN CONCRETE SHEAR STRESS $v_c = \phi\, 0.08 \sqrt{f'_c}$

$= 0.75 \times 0.08 \times \sqrt{20}$ = 0.27 MPa

> 0.19 MPa OK

SLAB OK FOR PUNCHING

ARNOLD & JOHNSTONE 2015 LTD

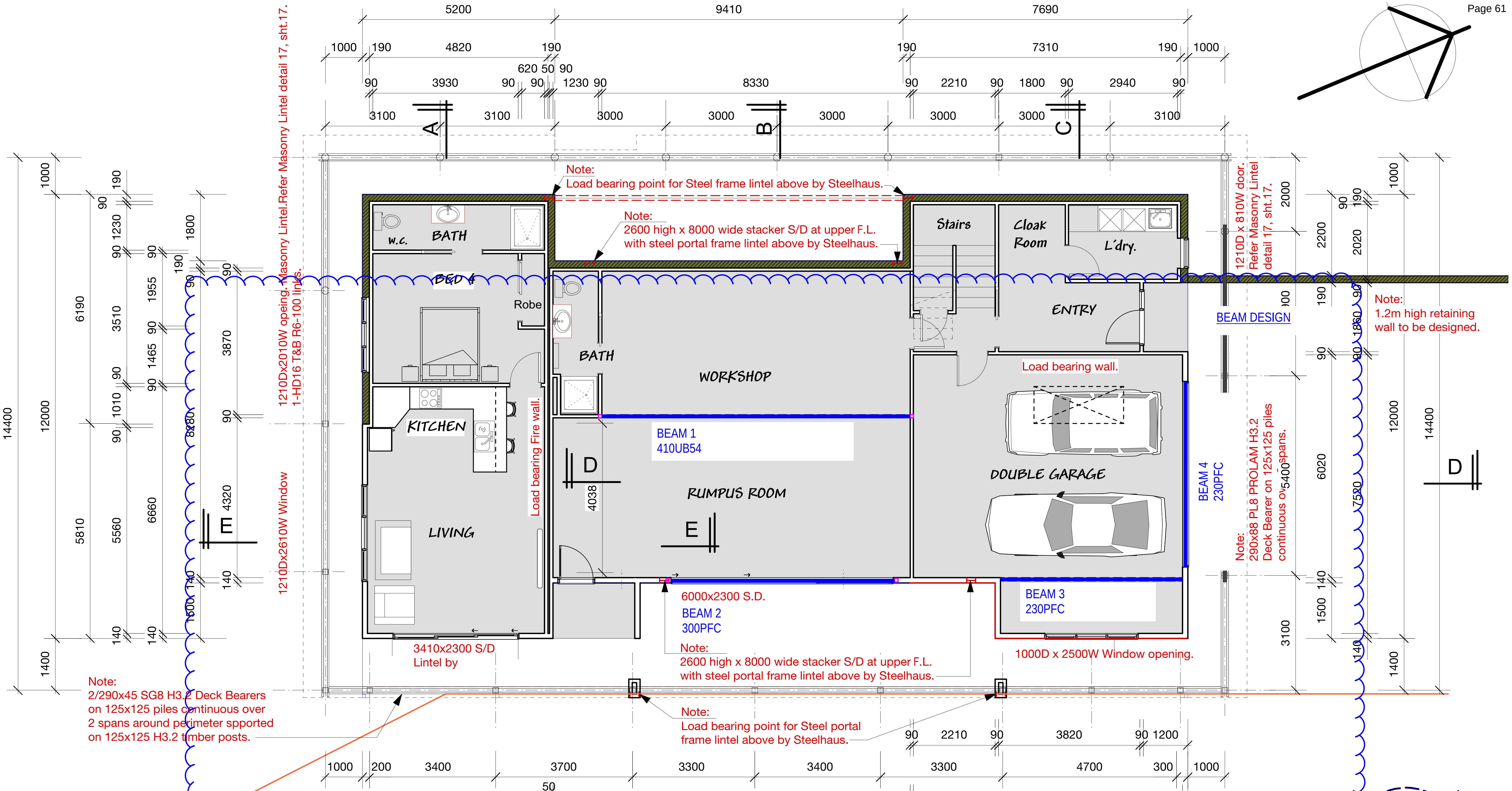
6. BEAM DESIGN

Ref: 12843

Hamilton City Council

BUILDING UNIT APPROVED

BC Number - DD007.2021.00043914.001



Entry / L'dry. / Stairs Area.....27.9 m2
Workshop / Rumpus Area.........83 m2
Ancillary Flat Area......................58 m2
Garage Area.............................54.7 m2
Total Basement Area....232.7 m2 (Outside block/Slab)

ARNOLD & JOHNSTONE
CONSULTING STRUCTURAL ENGINEERS

JOB No 12843
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JOB SH26 136 MORRINSVILLE ROAD

BEAM 01 DESIGN

ADOPT THE FOLLOWING LOADS

DEAD LOAD FLOOR G_{floor} = 0.5 kPa

LIVE LOAD FLOOR Q_{floor} = 1.5 kPa

TRIBUTARY WIDTH B_{trib} = 4.1 m

SPAN LENGTH L_s = 8.4 m

REFER TO THE ATTACHED SPREADSHEET FOR FULL CALCULATIONS

ADOPT MIN 410UB54

POST DESIGN

DESIGN POST TO SUPPORT BEAM 01

AXIAL LOAD N^*_c = 52

TRY 90x6 SHS

MOMENT DEMAND M^* = Pe

= 52 x 0.09 x 0.5 = 2.3 kNm

UTILISATION RATIO = 0.2 MEMDES

< 1.0 OK

ADOPT MIN 90x6 SHS POST SUPPORTS

ARNOLD & JOHNSTONE 2015 LTD
CONSULTING CIVIL & STRUCTURAL ENGINEERS

Job	136 MORRINSVILLE ROAD	Job No	12843
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Beam Design Beam 01 Design

Location: Beam 01 Design

Span = 8.40 m

Short Term Load Factor, ψ_s = 0.7
Long Term Load Factor, ψ_L = 0.4

UDL

Self Weight : 0.54 kN/m

	Trib(m)	G	Q	W	W_G	W_Q	W_W
					0.00 kN/m	0.00 kN/m	
					0.00 kN/m	0.00 kN/m	
Floor	4.10 m	0.50 kPA	1.50 kPA		2.05 kN/m	6.15 kN/m	
					0.00 kN/m	0.00 kN/m	
					0.00 kN/m	0.00 kN/m	
							0.00 kN/m
TOTAL					2.59 kN/m	6.15 kN/m	0.00 kN/m

UDL (ULS) , kN/m
1.2G+1.5Q= 12.33
0.9G+Wu= 2.33

UDL (SLS), kN/m
$G+\psi_sQ$ = 6.90
$G+\psi_LQ$ = 5.05 no roof live load
Ws = 0.00

Point Load At Position A= *(Measured from LHS)*

	Trib(m^2)	G	Q	W	W_G	W_Q	W_W
					0.00kN	0.00kN	
					0.00kN	0.00kN	
					0.00kN	0.00kN	
					0.00kN	0.00kN	
					0.00kN	0.00kN	
							0.00kN
TOTAL					0.00kN	0.00kN	0.00kN

1.2G+1.5Q= 0.00
0.9G+Wu= 0.00

$G+\psi_sQ$ = 0.00
$G+\psi_LQ$ = 0.00 no roof live load
Ws = 0

Bending Moment

Load Case	Mu*
1.2G+1.5Q =	108.78 kNm
0.9G+Wu =	20.56 kNm

Critical Bending Moment = 108.78 kNm

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CONSULTING CIVIL & STRUCTURAL ENGINEERS

Job	136 MORRINSVILLE ROAD	Job No	12843
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Try Timber Section

NZS3603

Width (b) = **90 mm** — L_{ay} = 4.00 m — Grade : Hyspan

Depth (d) = **360 mm**

φ_f =	0.8	CL 2.5
k_1 =	0.8	Table 2.4
k_2 (defl) =	2.0	Table 2.5
k_4 =	1	Table 2.7
k_5 =	1.000	Eq 2.5
k_8 =	0.81	Table 2.8
f_b =	42 MPa	
E =	13.2 GPa	
S_1 =	17.71	Eq 3.5

$\varphi M_n = \varphi k_1 k_4 k_5 k_8 f_b Z_{xx}$

= 42.29 kNm — **< Mu* therefore No Good**

Deflection :

Dead =	36.4 mm	= Span / 231
$G+\psi_s Q$ =	96.8 mm	= Span / 87
$G+\psi_L Q$ =	141.8 mm	= Span / 59
W_s =	0.0 mm	= Span / #DIV/0!

Try Steel Section

NZS3404

Section	I_{xx}	Z_{xx}
410 UB 53.7	188.00x10^6mm4	1060x10^3mm3

f_y =	320 MPa	
φ_f =	0.9	Table 3.3(1)
E =	200 GPa	
α_s	0.517	CL 5.6.1.1.2[c]
α_m	1.130	CL 5.6.1.1.1(b)
	PP	End Restraint Table 5.6.3(1)
k_t	1.07	Twist restraint factors Tb 5.6.3 (1)
k_r	1.00	Rotation restraint factors Tb 5.6.3 (3)
k_l	1.00	Load height factors Tb 5.6.3 (2)

$\varphi M_{sx} = \varphi Z_{xx} f_y$

= 305.28 — **> Mu* therefore O.K**

Segment Length 4.00 m — $L_e = k_t.k_r.k_l L$ 4.30 m

φM_{bx} = 305.28 kNm

$\varphi M_{bx} = \alpha_m \alpha_s M_{sx} \le M_{sx}$ 178.29 kNm — **> Mu* therefore O.K**

Deflection :

Dead =	4.5mm	= Span / 1881
$G+\psi_s Q$ =	11.9mm	= Span / 707
$G+\psi_L Q$ =	8.7mm	= Span / 965
W_s =	0.0mm	= Span / #DIV/0!

Reactions

	LHS	RHS
Dead (kn) =	10.9kN	10.9kN
Live (kn) =	25.8kN	25.8kN
Wind (kn) =	0.0kN	0.0kN
Rult (1.2G+1.5Q) =	51.8kN	51.8kN
(0.9G+Wu)=	9.8kN	9.8kN

Footing

Using square footing of size= 1.00 m

ϕ_{bc} 0.5

q_u 200 kPA

	LHS	RHS
fq (wkg) kPa =	36.7 kPA	36.7 kPA
fq (ULS) kPa =	51.8 kPA	51.8 kPA

Summary

Beam Location: **Beam 01 Design**

Beam Size:	**410 UB 53.7**	Span	**8.40 m**
Supports:	**89x5 SHS**		
RC Pad:			
Reinforcement:			

BEAM 1 POST DESIGN

MemDes Calculations @ 17:04:48 15-11-2021 by NAM
==
Project : 136 Morrinsville Road
Description : 90x6 SHS

Section : 089x089x6.0 SHS Grade 350

Major Axis Bending
Design Action M^*_x = 3.0 kNm
User provided value for α_m = 1.00
α_s = 1.01
α_m α_s >= 1.0, => Segment Fully Restrained
$M_{bx} = M_{sx}$ = 19.81 kNm
Major axis capacity Ratio = M^*_x / ϕM_{bx}
= 0.17, ---- OK ----

Shear Calculations (Unstiffened Web)
Design Action V^*_x = 0.0 kN
Nominal Shear Yield capacity V_w = 194.0 kN
α_v = 29.16 >= 1.0 => full web shear capacity
$V_u = V_w$ = 194.0 kN
Shear capacity ratio = V^*_x / ϕV_u
= 0.00, ---- OK ----

Axial Calculations
Design Action N_d = 52.0 kN [Comp], LeAxx = 3.00 m, LeAxy = 3.00 m
= 654.5 kN
Major axis buckling : Minor axis buckling : Minimum Capac. $N_c$min = 360.0
Axial buckling capac. Ratio = N_d / ϕN_cmin
= 0.160, ---- OK ----

Combined Actions Checks

Clause 8.3.3/4 :
$M_{ry} = M_{sy}$ (1 - (N^*/ ϕN_s)) * 1.18, =< M_{sy} [Alt. Prov. OK]
= 19.8
Load / Capacity Ratio = M^*_x / (0.9 M^r_x)
= 0.17, ---- OK ----

Clause 8.4.2.2 : Major : M_{ix} = 16.6
Load / Capacity Ratio = M^*_m/ ϕM_i
= 0.200 ---- OK ----
======================== SUMMARY =====================
**** U.L.S. Capacity Check Passed, Load Cap. Ratio = 0.20 ---- OK ----

==
SESOC MemDes v 3.8.2 : Calculations by NAM
Project : 136 Morrinsville Road at 5:04:48 pm on 15/11/2021
Description : 90x6 SHS

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CONSULTING STRUCTURAL ENGINEERS

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JOB SH26 136 MORRINSVILLE ROAD

BEAM 02 DESIGN

ADOPT THE FOLLOWING LOADS

DEAD LOAD FLOOR	G_{floor} =	0.5 kPa
LIVE LOAD FLOOR	Q_{floor} =	1.5 kPa
TRIBUTARY WIDTH	B_{trib} =	2.2 m
DEAD LOAD DECK	G_{deck} =	0.5 kPa
LIVE LOAD DECK	Q_{deck} =	2.0 kPa
TRIBUTARY WIDTH	B_{trib} =	1.6 m
DEAD LOAD WALL	G_{wall} =	0.4 kPa
TRIBUTARY HEIGHT	H_{trib} =	2.8 m
SPAN LENGTH	L_s =	6.0 m

REFER TO THE ATTACHED SPREADSHEET FOR FULL CALCULATIONS

ADOPT MIN 300 PFC BEAM

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CONSULTING CIVIL & STRUCTURAL ENGINEERS

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Beam Design Beam 02 Design
Location: Beam 02 Design

Span = 6.00 m

Short Term Load Factor, ψ_s = 0.7
Long Term Load Factor, ψ_L = 0.4

UDL

Self Weight : 0.40 kN/m

	Trib(m)	G	Q	W	W_G	W_Q	W_W
					0.00 kN/m	0.00 kN/m	
Wall	3.00 m	0.40 kPA			1.20 kN/m	0.00 kN/m	
Floor	2.20 m	0.50 kPA	1.50 kPA		1.10 kN/m	3.30 kN/m	
Deck	1.50 m	0.50 kPA	2.00 kPA		0.75 kN/m	3.00 kN/m	
					0.00 kN/m	0.00 kN/m	
							0.00 kN/m
TOTAL					3.45 kN/m	6.30 kN/m	0.00 kN/m

UDL (ULS) , kN/m
1.2G+1.5Q= 13.59
0.9G+Wu= 3.11

UDL (SLS), kN/m
$G+\psi_sQ$ = 7.86
$G+\psi_LQ$ = 5.97 no roof live load
Ws = 0.00

Point Load At Position A= *(Measured from LHS)*

	Trib(m^2)	G	Q	W	W_G	W_Q	W_W
					0.00kN	0.00kN	
					0.00kN	0.00kN	
					0.00kN	0.00kN	
					0.00kN	0.00kN	
					0.00kN	0.00kN	
							0.00kN
TOTAL					0.00kN	0.00kN	0.00kN

1.2G+1.5Q= 0.00
0.9G+Wu= 0.00

$G+\psi_sQ$ = 0.00
$G+\psi_LQ$ = 0.00 no roof live load
Ws = 0

Bending Moment

Load Case	Mu*
1.2G+1.5Q =	61.16 kNm
0.9G+Wu =	13.97 kNm

Critical Bending Moment = 61.16 kNm

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CONSULTING CIVIL & STRUCTURAL ENGINEERS

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Job 136 MORRINSVILLE ROAD

Try Timber Section

NZS3603

Width (b) = **90 mm**
Depth (d) = **360 mm**

L_{ay} = 4.00 m

Grade :	Hyspan	
φ_f =	0.8	CL 2.5
k_1 =	0.8	Table 2.4
k_2 (defl) =	2.0	Table 2.5
k_4 =	1	Table 2.7
k_5 =	1.000	Eq 2.5
k_8 =	0.81	Table 2.8
f_b =	42 MPa	
E =	13.2 GPa	
S_1 =	17.71	Eq 3.5

$\varphi M_n = \varphi k_1 k_4 k_5 k_8 f_b Z_{xx}$
= 42.29 kNm **< Mu* therefore No Good**

Deflection :

Dead =	12.6 mm	= Span / 476
$G+\psi_s Q$ =	28.7 mm	= Span / 209
$G+\psi_L Q$ =	43.6 mm	= Span / 138
W_s =	0.0 mm	= Span / #DIV/0!

Try Steel Section

NZS3404

Section	I_{xx}	Z_{xx}
300 PFC 40.1	72.40x10^6mm4	564x10^3mm3

f_y =	300 MPa	
φ_f =	0.9	Table 3.3(1)
E =	200 GPa	
α_s	0.913	CL 5.6.1.1.2[c]
α_m	1.130	CL 5.6.1.1.1(b)
	PP	End Restraint Table 5.6.3(1)
k_t	2.00	Twist restraint factors Tb 5.6.3 (1)
k_r	1.00	Rotation restraint factors Tb 5.6.3 (3)
k_l	1.00	Load height factors Tb 5.6.3 (2)

$\varphi M_{sx} = \varphi Z_{xx} f_y$
= 152.28 **> Mu* therefore O.K**

Segment Length 0.60 m $L_e = k_t.k_r.k_l L$ 1.20 m

φM_{bx} = 152.28 kNm
$\varphi M_{bx} = \alpha_m \alpha_s M_{sx} \leq M_{sx}$ 152.28 kNm **> Mu* therefore O.K**

Deflection :

Dead =	4.0mm	= Span / 1492
$G+\psi_s Q$ =	9.2mm	= Span / 655
$G+\psi_L Q$ =	7.0mm	= Span / 862
W_s =	0.0mm	= Span / #DIV/0!

Reactions

	LHS	RHS
Dead (kn) =	10.4kN	10.4kN
Live (kn) =	18.9kN	18.9kN
Wind (kn) =	0.0kN	0.0kN
Rult (1.2G+1.5Q) =	40.8kN	40.8kN
(0.9G+Wu)=	9.3kN	9.3kN

Footing

Using square footing of size= 1.00 m
ϕ_{bc} 0.5
q_u 200 kPA

	LHS	RHS
fq (wkg) kPa =	29.3 kPA	29.3 kPA
fq (ULS) kPa =	40.8 kPA	40.8 kPA

Summary

Beam Location: **Beam 02 Design**

Beam Size: **300 PFC 40.1** Span **6.00 m**
Supports: **75x5 SHS**
RC Pad:
Reinforcement:

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Beam Design Beam 03 Design

Location: Beam 03 Design

Span = 5.00 m

Short Term Load Factor, ψ_s = 0.7
Long Term Load Factor, ψ_L = 0.4

UDL

Self Weight : 0.25 kN/m

	Trib(m)	G	Q	W	W_G	W_Q	W_W
					0.00 kN/m	0.00 kN/m	
					0.00 kN/m	0.00 kN/m	
Floor	3.70 m	0.50 kPA	1.50 kPA		1.85 kN/m	5.55 kN/m	
					0.00 kN/m	0.00 kN/m	
					0.00 kN/m	0.00 kN/m	
							0.00 kN/m
TOTAL					2.10 kN/m	5.55 kN/m	0.00 kN/m

UDL (ULS) , kN/m
1.2G+1.5Q= 10.85
0.9G+Wu= 1.89

UDL (SLS), kN/m
$G+\psi_sQ$ = 5.99
$G+\psi_LQ$ = 4.32 no roof live load
Ws = 0.00

Point Load At Position A= *(Measured from LHS)*

	Trib(m^2)	G	Q	W	W_G	W_Q	W_W
					0.00kN	0.00kN	
					0.00kN	0.00kN	
					0.00kN	0.00kN	
					0.00kN	0.00kN	
					0.00kN	0.00kN	
							0.00kN
TOTAL					0.00kN	0.00kN	0.00kN

1.2G+1.5Q= 0.00
0.9G+Wu= 0.00

$G+\psi_sQ$ = 0.00
$G+\psi_LQ$ = 0.00 no roof live load
Ws = 0

Bending Moment

Load Case	Mu*
1.2G+1.5Q =	33.89 kNm
0.9G+Wu =	5.91 kNm

Critical Bending Moment = 33.89 kNm

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Try Timber Section

NZS3603

Width (b) = **90 mm** — L_{ay} = 3.40 m — Grade : Hyspan

Depth (d) = **290 mm**

φ_f = 0.8	CL 2.5
k_1 = 0.8	Table 2.4
k_2 (defl) = 2.0	Table 2.5
k_4 = 1	Table 2.7
k_5 = 1.000	Eq 2.5
k_8 = 0.93	Table 2.8
f_b = 42 MPa	
E = 13.2 GPa	
S_1 = 14.52	Eq 3.5

$\varphi M_n = \varphi k_1 k_4 k_5 k_8 f_b Z_{xx}$

= 31.38 kNm — **< Mu* therefore No Good**

Deflection :

Dead =	7.1 mm	= Span / 706
$G+\psi_s Q$ =	20.2 mm	= Span / 248
$G+\psi_L Q$ =	29.1 mm	= Span / 172
W_s =	0.0 mm	= Span / #DIV/0!

Try Steel Section

NZS3404

Section	I_{xx}	Z_{xx}
230 PFC 25.1	26.80x10^6mm4	271x10^3mm3

f_y = 300 MPa	
φ_f = 0.9	Table 3.3(1)
E = 200 GPa	
α_s 0.922	CL 5.6.1.1.2[c]
α_m 1.130	CL 5.6.1.1.1(b)
PP	End Restraint Table 5.6.3(1)
k_t 1.60	Twist restraint factors Tb 5.6.3 (1)
k_r 1.00	Rotation restraint factors Tb 5.6.3 (3)
k_l 1.00	Load height factors Tb 5.6.3 (2)

$\varphi M_{sx} = \varphi Z_{xx} f_y$

= 73.17 — **> Mu* therefore O.K**

Segment Length 0.60 m — $L_e = k_t . k_r . k_l L$ 0.96 m

φM_{bx} = 73.17 kNm

$\varphi M_{bx} = \alpha_m \alpha_s M_{sx} \le M_{sx}$ 73.17 kNm — **> Mu* therefore O.K**

Deflection :

Dead =	3.2mm	= Span / 1568
$G+\psi_s Q$ =	9.1mm	= Span / 550
$G+\psi_L Q$ =	6.6mm	= Span / 762
W_s =	0.0mm	= Span / #DIV/0!

Reactions

	LHS	RHS
Dead (kn) =	5.3kN	5.3kN
Live (kn) =	13.9kN	13.9kN
Wind (kn) =	0.0kN	0.0kN
Rult (1.2G+1.5Q) =	27.1kN	27.1kN
(0.9G+Wu)=	4.7kN	4.7kN

Footing

Using square footing of size= 1.00 m

Φ_{bc} 0.5

q_u 200 kPA

	LHS	RHS
fq (wkg) kPa =	19.1 kPA	19.1 kPA
fq (ULS) kPa =	27.1 kPA	27.1 kPA

Summary

Beam Location: **Beam 03 Design**

Beam Size:	**230 PFC 25.1**	Span	**5.00 m**
Supports:	**3/90x45 studs**		
RC Pad:			
Reinforcement:			

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Beam Design Beam 04 Design

Location: Beam 04 Design

Span = 5.20 m

Short Term Load Factor, ψ_s = 0.7
Long Term Load Factor, ψ_L = 0.4

UDL

Self Weight : 0.25 kN/m

NOTE BEAM 4 NO LONGER SUPPORTS A DECK. OK BY INSPECTION

	Trib(m)	G	Q	W	W_G	W_Q	W_W
Roof	3.60 m	0.45 kPA	0.25 kPA		1.62 kN/m	0.90 kN/m	
Wall	6.00 m	0.40 kPA			2.40 kN/m	0.00 kN/m	
					0.00 kN/m	0.00 kN/m	
Deck	0.50 m	0.50 kPA	2.00 kPA		0.25 kN/m	1.00 kN/m	
					0.00 kN/m	0.00 kN/m	
							0.00 kN/m
TOTAL					4.52 kN/m	1.90 kN/m	0.00 kN/m

UDL (ULS) , kN/m
1.2G+1.5Q= 8.27
0.9G+Wu= 4.07

UDL (SLS), kN/m
G+ψ_sQ = 5.85
G+ψ_LQ = 4.92 no roof live load
Ws = 0.00

Point Load At Position A= *(Measured from LHS)*

	Trib(m^2)	G	Q	W	W_G	W_Q	W_W
					0.00kN	0.00kN	
					0.00kN	0.00kN	
					0.00kN	0.00kN	
					0.00kN	0.00kN	
					0.00kN	0.00kN	
							0.00kN
TOTAL					0.00kN	0.00kN	0.00kN

1.2G+1.5Q= 0.00
0.9G+Wu= 0.00

G+ψ_sQ = 0.00
G+ψ_LQ = 0.00 no roof live load
Ws = 0

Bending Moment

Load Case	Mu*
1.2G+1.5Q =	27.97 kNm
0.9G+Wu =	13.75 kNm

Critical Bending Moment = 27.97 kNm

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Job 136 MORRINSVILLE ROAD

Try Timber Section

NZS3603

Width (b) = **90 mm** L_{ay} = 0.60 m Grade : SG8 Dry

Depth (d) = **290 mm**

φ_f =	0.8	CL 2.5
k_1 =	0.8	Table 2.4
k_2 (defl) =	2.0	Table 2.5
k_4 =	1	Table 2.7
k_5 =	1.000	Eq 2.5
k_8 =	1.00	Table 2.8
f_b =	14 MPa	
E =	6.7 GPa	
S_1 =	6.10	Eq 3.5

$\varphi M_n = \varphi k_1 k_4 k_5 k_8 f_b Z_{xx}$

= 11.30 kNm **< Mu* therefore No Good**

Deflection :

Dead =	35.1 mm	= Span / 148
$G+\psi_s Q$ =	45.4 mm	= Span / 114
$G+\psi_L Q$ =	76.4 mm	= Span / 68
W_s =	0.0 mm	= Span / #DIV/0!

Try Steel Section

NZS3404

Section	I_{xx}	Z_{xx}
230 PFC 25.1	26.80x10^6mm4	271x10^3mm3

f_y =	300 MPa	
φ_f =	0.9	Table 3.3(1)
E =	200 GPa	
α_s	0.922	CL 5.6.1.1.2[c]
α_m	1.130	CL 5.6.1.1.1(b)
	PP	End Restraint Table 5.6.3(1)
k_t	1.60	Twist restraint factors Tb 5.6.3 (1)
k_r	1.00	Rotation restraint factors Tb 5.6.3 (3)
k_l	1.00	Load height factors Tb 5.6.3 (2)

$\varphi M_{sx} = \varphi Z_{xx} f_y$

= 73.17 **> Mu* therefore O.K**

Segment Length 0.60 m $L_e = k_t . k_r . k_l L$ 0.96 m

φM_{bx} = 73.17 kNm

$\varphi M_{bx} = \alpha_m \alpha_s M_{sx} \leq M_{sx}$ 73.17 kNm **> Mu* therefore O.K**

Deflection :

Dead =	8.0mm	= Span / 648
$G+\psi_s Q$ =	10.4mm	= Span / 500
$G+\psi_L Q$ =	8.7mm	= Span / 595
W_s =	0.0mm	= Span / #DIV/0!

Reactions

	LHS	RHS
Dead (kn) =	11.8kN	11.8kN
Live (kn) =	4.9kN	4.9kN
Wind (kn) =	0.0kN	0.0kN
Rult (1.2G+1.5Q) =	21.5kN	21.5kN
(0.9G+Wu)=	10.6kN	10.6kN

Footing

Using square footing of size= 1.00 m

Φ_{bc} 0.5

q_u 200 kPA

	LHS	RHS
fq (wkg) kPa =	16.7 kPA	16.7 kPA
fq (ULS) kPa =	21.5 kPA	21.5 kPA

Summary

Beam Location: **Beam 04 Design**

Beam Size: **230 PFC 25.1** Span **5.20 m**

Supports: **3/90x45 studs**

RC Pad:

Reinforcement:

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CONSULTING STRUCTURAL ENGINEERS

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PAGE
BY NAM
JOB 136 MORRINSVILLE ROAD
DATE 09.08.2021

BEAM 4 POST DESIGN

89 x 6 SHS POSTS OK FOR VERTICAL LOADS. CHECK WIND FACE LOADING

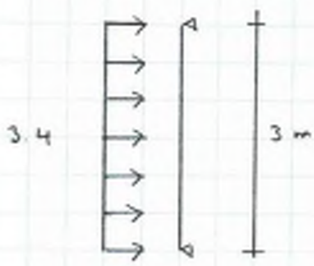
DESIGN WIND LOAD W_u = 1.36 kPa

TRIBUTARY WIDTH B_{trib} = 2.5 m

ULS WIND LOAD W_{ULS} = 1.36 x 2.5 = 3.4 kN/m

ASSUME SIMPLY SUPPORTED TOP & BOTTOM

MOMENT DEMAND $M^* = \frac{wL^2}{8}$



$M^* = \frac{3.4 \times 3^2}{8}$ = 3.8 kNm

UTILISATION RATIO = 0.224 MEMDES

< 1.0 OK

CHECK SLS DEFLECTIONS

SLS WIND LOAD W_{SLS} = 0.67 x 3.4 = 2.3 kN/m

DEFLECTION $\Delta^* = \frac{5wL^4}{384EI}$

$= \frac{5 \times 2.3 \times 3000^4}{384 \times 200\,000 \times 2.06 \times 10^6}$ = 6 mm

< 15 mm OK

ADOPT 89 x 6 SHS POSTS FOR BEAM 4

BEAM 1 POST DESIGN

MemDes Calculations @ 17:51:11 09-08-2021 by NAM
==

Project : 136 Morrinsville Road
Description : 89x6 SHS

Section : 089x089x6.0 SHS Grade 350

Major Axis Bending
Design Action M^*_x = 4.0 kNm
User provided value for α_m = 1.00
α_s = 1.01
α_m α_s >= 1.0, => Segment Fully Restrained
$M_{bx} = M_{sx}$ = 19.81 kNm
Major axis capacity Ratio = M^*_x / ϕ M_{bx}
= 0.22, ---- OK ----

Shear Calculations (Unstiffened Web)
Design Action V^*_x = 0.0 kN
Nominal Shear Yield capacity V_w = 194.0 kN
α_v = 29.16 >= 1.0 => full web shear capacity
$V_u = V_w$ = 194.0 kN
Shear capacity ratio = V^*_x / ϕ V_u
= 0.00, ---- OK ----
======================== SUMMARY ====================
**** U.L.S. Capacity Check Passed, Load Cap. Ratio = 0.22 ---- OK ----

==
SESOC MemDes v 3.8.2 : Calculations by NAM
Project : 136 Morrinsville Road at 5:51:11 pm on 9/08/2021
Description : 89x6 SHS

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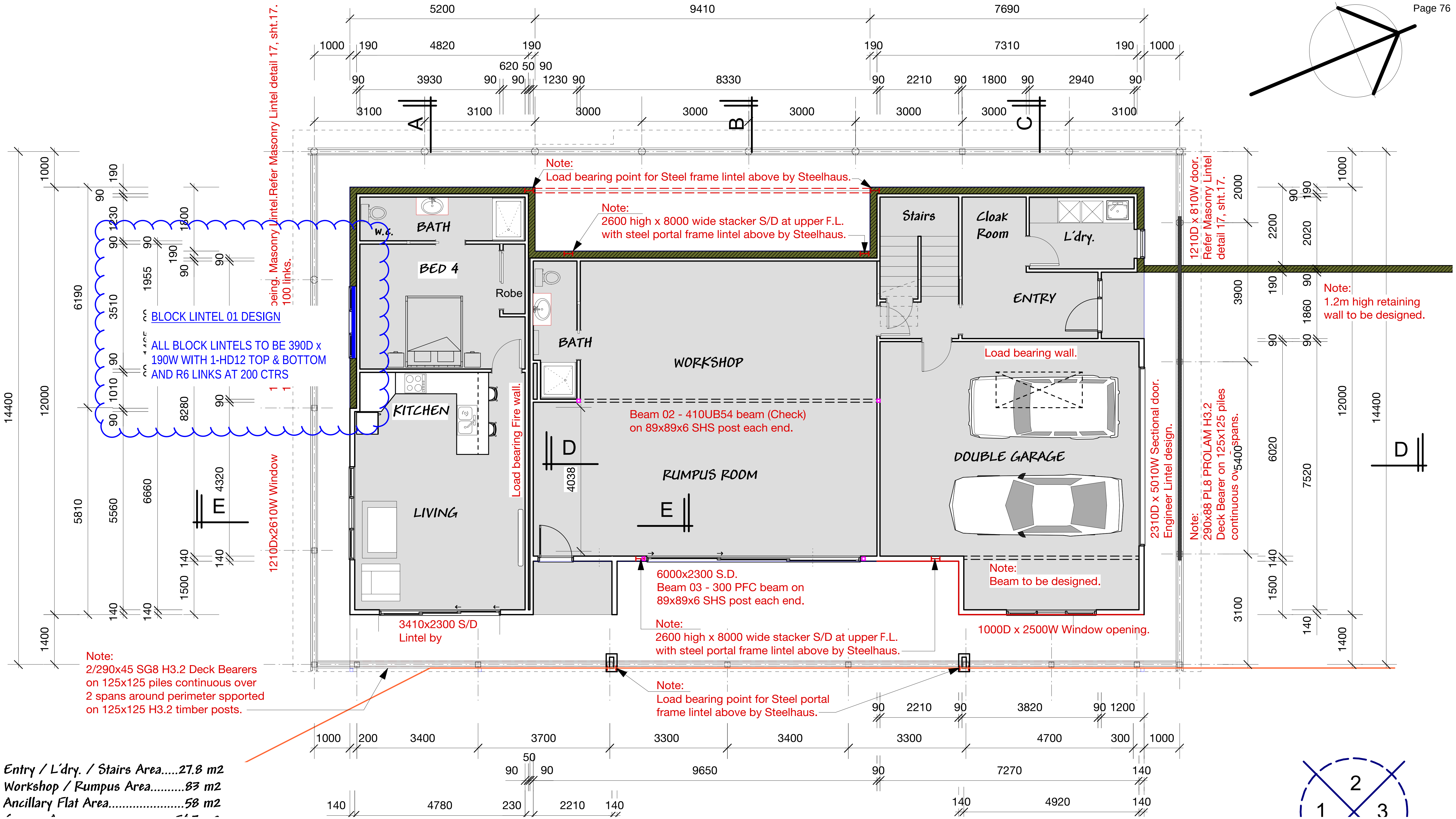
7. BLOCK LINTEL DESIGN

Ref: 12843

Hamilton City Council

BUILDING UNIT APPROVED

BC Number - DD007.2021.00043914.001



Note:
Load bearing point for Steel frame lintel above by Steelhaus.
Note:
2600 high x 8000 wide stacker S/D at upper F.L.
with steel portal frame lintel above by Steelhaus.
BLOCK LINTEL 01 DESIGN
ALL BLOCK LINTELS TO BE 390D x
190W WITH 1-HD12 TOP & BOTTOM
AND R6 LINKS AT 200 CTRS
Refer Masonry Lintel detail 17, sht.17.
W.C.
BATH
BED 4
Robe
KITCHEN
LIVING
Load bearing Fire wall.
1210Dx2610W Window
BATH
WORKSHOP
Beam 02 - 410UB54 beam (Check)
on 89x89x6 SHS post each end.
RUMPUS ROOM
Stairs
Cloak Room
L'dry.
ENTRY
Load bearing wall.
DOUBLE GARAGE
1210D x 810W door.
Refer Masonry Lintel
detail 17, sht.17.
Note:
1.2m high retaining
wall to be designed.
2310D x 5010W Sectional door.
Engineer Lintel design.
Note:
290x88 PL8 PROLAM H3.2
Deck Bearer on 125x125 piles
continuous over spans.
6000x2300 S.D.
Beam 03 - 300 PFC beam on
89x89x6 SHS post each end.
Note:
Beam to be designed.
3410x2300 S/D
Lintel by
Note:
2600 high x 8000 wide stacker S/D at upper F.L.
with steel portal frame lintel above by Steelhaus.
1000D x 2500W Window opening.
Note:
2/290x45 SG8 H3.2 Deck Bearers
on 125x125 piles continuous over
2 spans around perimeter spported
on 125x125 H3.2 timber posts.
Note:
Load bearing point for Steel portal
frame lintel above by Steelhaus.
Entry / L'dry. / Stairs Area.....27.8 m2
Workshop / Rumpus Area.........83 m2
Ancillary Flat Area......................58 m2

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JOB No 12843

PAGE

BY NAM

JOB 136 MORRINSVILLE ROAD

DATE 01.06.2021

BLOCK LINTEL 01 DESIGN

DESIGN BLOCK LINTEL OVER WINDOW ADOPT THE FOLLOWING LOADS

DEAD LOAD ROOF	G_{roof} =	0.45 kPa × 3.6 m	= 1.6 kN/m
DEAD LOAD WALL	G_{wall} =	0.4 kPa × 3.0 m	= 1.2 kN/m
DEAD LOAD DECK	G_{deck} =	0.5 kPa × 0.5 m	= 0.3 kN/m
DEAD LOAD FLOOR	G_{floor} =	0.5 kPa × 2.5 m	= 1.3 kN/m
DEAD LOAD BLOCK	G_{blo} =	0.39 × 0.19 × 22	= 1.6 kN/m
TOTAL DEAD LOAD	G =	1.6 + 1.2 + 0.3 + 1.3 + 1.6	= 6.0 kN/m
LIVE LOAD ROOF	Q_{roof} =	0.25 kPa × 3.6	= 0.9 kN/m
LIVE LOAD DECK	Q_{deck} =	2.0 kPa × 0.5	= 1.0 kN/m
LIVE LOAD FLOOR	Q_{floor} =	1.5 kPa × 2.5	= 3.8 kN/m
TOTAL LIVE LOAD	Q =	0.9 + 1.0 + 3.8	= 5.7 kN/m
ULS LOAD CASE	E_d =	1.2 × 6.0 + 1.5 × 5.7	= 15.8 kN/m

FOR A 2m SPAN, ADOPT AS PER NZS 4229: 2013 TABLE 11.5

ADOPT MIN 2-D12 LONG BARS, R6-200 STIRRUPS

NOTE - HD12 BARS HAVE BEEN ADOPTED

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8. BEARER DESIGN

Ref: 12843

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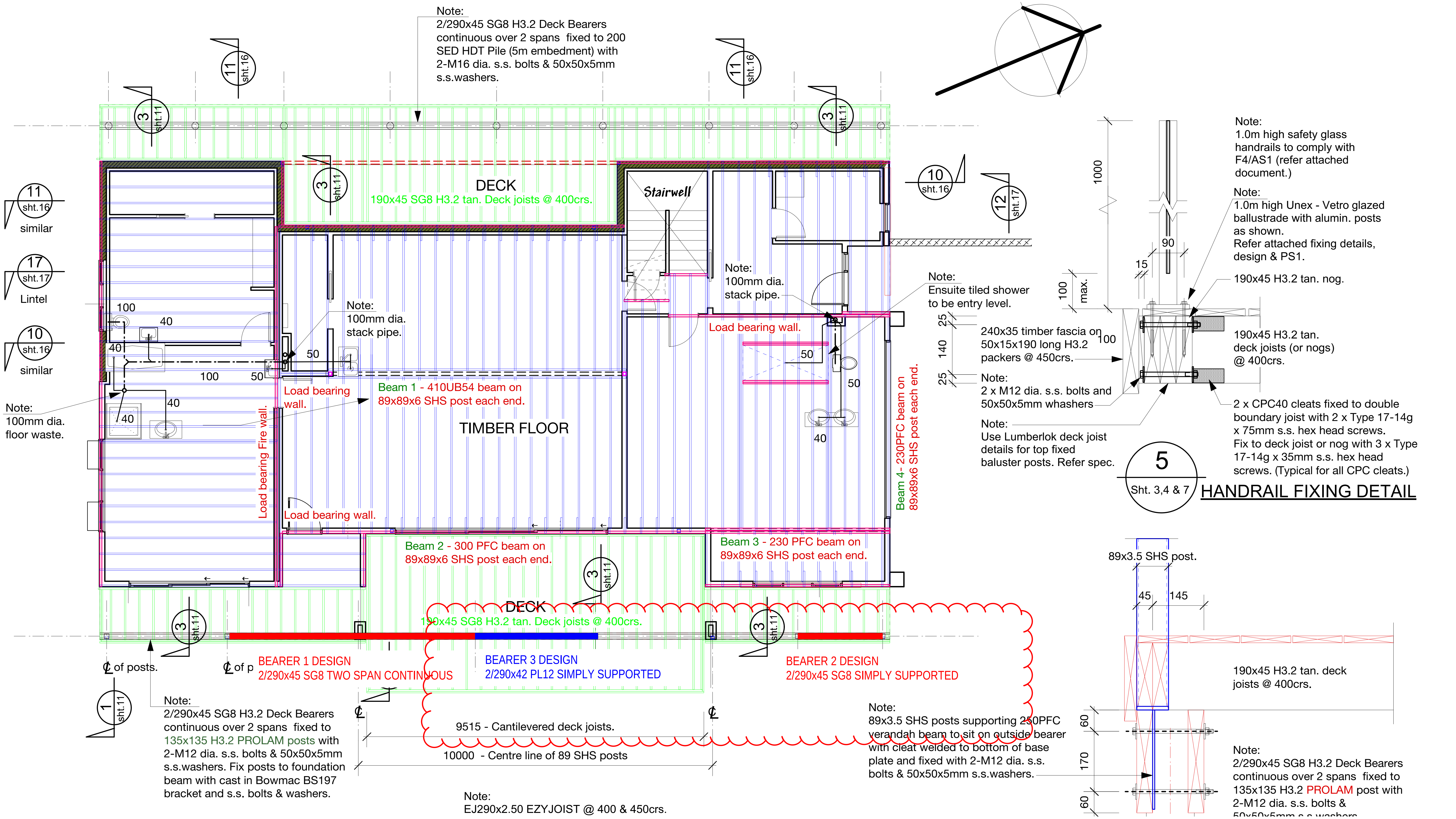
Note:
2/290x45 SG8 H3.2 Deck Bearers continuous over 2 spans fixed to 200 SED HDT Pile (5m embedment) with 2-M16 dia. s.s. bolts & 50x50x5mm s.s.washers.
DECK
190x45 SG8 H3.2 tan. Deck joists @ 400crs.
Stairwell
11 sht.16 similar
17 sht.17 Lintel
10 sht.16 similar
Note:
100mm dia. floor waste.
Note:
100mm dia. stack pipe.
Note:
100mm dia. stack pipe.
Load bearing wall.
Beam 1 - 410UB54 beam on 89x89x6 SHS post each end.
TIMBER FLOOR
Load bearing Fire wall.
Load bearing wall.
Load bearing wall.
Beam 2 - 300 PFC beam on 89x89x6 SHS post each end.
Beam 3 - 230 PFC beam on 89x89x6 SHS post each end.
Beam 4 - 230PFC beam on 89x89x6 SHS post each end.
Note:
Ensuite tiled shower to be entry level.
DECK
190x45 SG8 H3.2 tan. Deck joists @ 400crs.
℄ of posts.
℄ of p
BEARER 1 DESIGN
2/290x45 SG8 TWO SPAN CONTINUOUS
BEARER 3 DESIGN
2/290x42 PL12 SIMPLY SUPPORTED
BEARER 2 DESIGN
2/290x45 SG8 SIMPLY SUPPORTED
Note:
2/290x45 SG8 H3.2 Deck Bearers continuous over 2 spans fixed to 135x135 H3.2 PROLAM posts with 2-M12 dia. s.s. bolts & 50x50x5mm s.s.washers. Fix posts to foundation beam with cast in Bowmac BS197 bracket and s.s. bolts & washers.
9515 - Cantilevered deck joists.
10000 - Centre line of 89 SHS posts
Note:
EJ290x2.50 EZYJOIST @ 400 & 450crs.
Note:
89x3.5 SHS posts supporting 250PFC verandah beam to sit on outside bearer with cleat welded to bottom of base plate and fixed with 2-M12 dia. s.s. bolts & 50x50x5mm s.s.washers.
Note:
1.0m high safety glass handrails to comply with F4/AS1 (refer attached document.)
Note:
1.0m high Unex - Vetro glazed ballustrade with alumin. posts as shown.
Refer attached fixing details, design & PS1.
190x45 H3.2 tan. nog.
240x35 timber fascia on 50x15x190 long H3.2 packers @ 450crs.
190x45 H3.2 tan. deck joists (or nogs) @ 400crs.
Note:
2 x M12 dia. s.s. bolts and 50x50x5mm whashers
Note:
Use Lumberlok deck joist details for top fixed baluster posts. Refer spec.
2 x CPC40 cleats fixed to double boundary joist with 2 x Type 17-14g x 75mm s.s. hex head screws.
Fix to deck joist or nog with 3 x Type 17-14g x 35mm s.s. hex head screws. (Typical for all CPC cleats.)
5
Sht. 3,4 & 7
HANDRAIL FIXING DETAIL
89x3.5 SHS post.
190x45 H3.2 tan. deck joists @ 400crs.
Note:
2/290x45 SG8 H3.2 Deck Bearers continuous over 2 spans fixed to 135x135 H3.2 PROLAM post with 2-M12 dia. s.s. bolts & 50x50x5mm s.s.washers

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DATE 10.05.2021

JOB 136 MORRINSVILLE ROAD

BEARER DESIGN

DESIGN DECK BEARERS AS TWO SPAN CONTINUOUS. CHECK CRITICAL LOAD CASES

BEARER 1

DEAD LOAD DECK $\quad G_{deck} = 0.5\ kPa \times 1.5\ m \quad = 0.75\ kN/m$

LIVE LOAD DECK $\quad Q_{deck} = 2.0\ kPa \times 1.5\ m \quad = 3.0\ kN/m$

MOMENT DEMAND $\quad M^* = 7.6\ kNm$

TRY 2/290x45 SG8 FOR BEARER 1

MOMENT CAPACITY $\quad \phi M_n = \phi k_1 k_8 f_b Z$

SECTION MODULUS $\quad Z = \frac{bd^2}{6}$

$$= \frac{90 \times 290^2}{6} \quad = 1.26 \times 10^6\ mm^3$$

$$\phi M_n = 0.8 \times 0.8 \times 1.0 \times 11.7 \times 1.26 \times 10^6 \quad = 9.4\ kNm$$

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> 7.6 kNm   OK

CHECK SLS DEFLECTIONS LIMIT TO SPAN/300

BEARER 1 $\Delta' = 2.5$ mm

$= \frac{SPAN}{1400}$ OK

FOR TYPICAL BEARER ADOPT 2/290x45 SG8 TWO-SPAN CONTINUOUS

```
SPACE GASS 14.00 - ARNOLD && JOHNSTONE 2015 LTD
Path: C:\Users\nick\Dropbox (Arnold and ...\Space Gass\12843 - Bearer Design
Designer:  Date: Tuesday, August 10, 2021 10:45 AM, Page: 1
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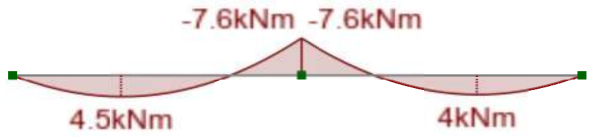
Load case 10

10 1.2G + 1.5Q



Viewpoint (0,0), Moments

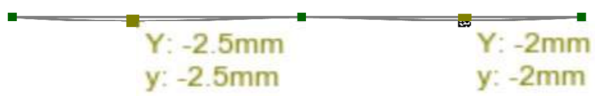
Materials:
1 SG8 Wet
2 PL12 Wet

Sections:
1 2/290x45
2 290x88

SPACE GASS 14.00 - ARNOLD && JOHNSTONE 2015 LTD
Path: C:\Users\nick\Dropbox (Arnold and ...\Space Gass\12843 - Bearer Design
Designer: Date: Tuesday, August 10, 2021 10:46 AM, Page: 1

Load case 21

21 G + wlQ



Viewpoint (0,0), Displacements

Materials:
1 SG8 Wet
2 PL12 Wet

Sections:
1 2/290x45
2 290x88

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CONSULTING CIVIL & STRUCTURAL ENGINEERS

Job	136 MORRINSVILLE ROAD	Job No	12843
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Beam Design Bearer 02 Design

Location: Bearer 02 Design

Span = 2.40 m

Short Term Load Factor, ψ_s = 0.7
Long Term Load Factor, ψ_L = 0.4

UDL

Self Weight : 0.13 kN/m

	Trib(m)	G	Q	W	W_G	W_Q	W_W
					0.00 kN/m	0.00 kN/m	
					0.00 kN/m	0.00 kN/m	
					0.00 kN/m	0.00 kN/m	
Deck	1.00 m	0.40 kPA	2.00 kPA		0.40 kN/m	2.00 kN/m	
					0.00 kN/m	0.00 kN/m	
							0.00 kN/m
TOTAL					0.53 kN/m	2.00 kN/m	0.00 kN/m

UDL (ULS) , kN/m
1.2G+1.5Q= 3.64
0.9G+Wu= 0.48

UDL (SLS), kN/m
$G+\psi_sQ$ = 1.93
$G+\psi_LQ$ = 1.33 no roof live load
Ws = 0.00

Point Load At Position A= *(Measured from LHS)*

	Trib(m^2)	G	Q	W	W_G	W_Q	W_W
					0.00kN	0.00kN	
					0.00kN	0.00kN	
					0.00kN	0.00kN	
					0.00kN	0.00kN	
					0.00kN	0.00kN	
							0.00kN
TOTAL					0.00kN	0.00kN	0.00kN

1.2G+1.5Q= 0.00
0.9G+Wu= 0.00

$G+\psi_sQ$ = 0.00
$G+\psi_LQ$ = 0.00 no roof live load
Ws = 0

Bending Moment

Load Case	Mu*
1.2G+1.5Q =	2.62 kNm
0.9G+Wu =	0.34 kNm

Critical Bending Moment = 2.62 kNm

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CONSULTING CIVIL & STRUCTURAL ENGINEERS

Job	136 MORRINSVILLE ROAD	Job No	12843
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Try Timber Section

NZS3603

Width (b) = **90 mm** L_{ay} = 2.40 m Grade : SG8 Wet

Depth (d) = **290 mm**

φ_f =	0.8	CL 2.5
k_1 =	0.8	Table 2.4
k_2 (defl) =	2.0	Table 2.5
k_4 =	1	Table 2.7
k_5 =	1.000	Eq 2.5
k_8 =	0.98	Table 2.8
f_b =	12 MPa	
E =	5.5 GPa	
S_1 =	12.20	Eq 3.5

$\varphi M_n = \varphi k_1 k_4 k_5 k_8 f_b Z_{xx}$
= 9.26 kNm **> Mu* therefore O.K**

Deflection :

Dead =	0.2 mm	= Span / 10546
$G+\psi_s Q$ =	0.8 mm	= Span / 2896
$G+\psi_L Q$ =	1.1 mm	= Span / 2101
W_s =	0.0 mm	= Span / #DIV/0!

Try Steel Section

NZS3404

Section	I_{xx}	Z_{xx}
200 PFC 22.9	19.10x10^6mm4	221x10^3mm3

f_y =	300 MPa	
φ_f =	0.9	Table 3.3(1)
E =	200 GPa	
α_s	0.471	CL 5.6.1.1.2[c]
α_m	1.130	CL 5.6.1.1.1(b)
	PP	End Restraint Table 5.6.3(1)
k_t	1.11	Twist restraint factors Tb 5.6.3 (1)
k_r	1.00	Rotation restraint factors Tb 5.6.3 (3)
k_l	1.00	Load height factors Tb 5.6.3 (2)

$\varphi M_{sx} = \varphi Z_{xx} f_y$
= 59.67 **> Mu* therefore O.K**

Segment Length 3.60 m $L_e = k_t.k_r.k_l L$ 4.00 m

φM_{bx} = 59.67 kNm
$\varphi M_{bx} = \alpha_m \alpha_s M_{sx} \le M_{sx}$ 31.78 kNm **> Mu* therefore O.K**

Deflection :

Dead =	0.1mm	= Span / 40042
$G+\psi_s Q$ =	0.2mm	= Span / 10996
$G+\psi_L Q$ =	0.2mm	= Span / 15957
W_s =	0.0mm	= Span / #DIV/0!

Reactions

	LHS	RHS
Dead (kn) =	0.6kN	0.6kN
Live (kn) =	2.4kN	2.4kN
Wind (kn) =	0.0kN	0.0kN
Rult (1.2G+1.5Q) =	4.4kN	4.4kN
(0.9G+Wu)=	0.6kN	0.6kN

Footing

Using square footing of size= 1.00 m
Φ_{bc} 0.5
q_u 200 kPA

	LHS	RHS
fq (wkg) kPa =	3.0 kPA	3.0 kPA
fq (ULS) kPa =	4.4 kPA	4.4 kPA

Summary

Beam Location: **Bearer 02 Design**

Beam Size:	**90**	**x 290mm**	**SG8 Wet**	Span	**2.40 m**
Supports:	**2/90x45 studs**				
RC Pad:					
Reinforcement:					

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Job	136 MORRINSVILLE ROAD	Job No	12843
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Beam Design Bearer 03 Design

Location: Bearer 03 Design

Span = 3.40 m

Short Term Load Factor, ψ_s = 0.7
Long Term Load Factor, ψ_L = 0.4

UDL

Self Weight : 0.12 kN/m

	Trib(m)	G	Q	W	W_G	W_Q	W_W
					0.00 kN/m	0.00 kN/m	
					0.00 kN/m	0.00 kN/m	
					0.00 kN/m	0.00 kN/m	
Deck	2.90 m	0.40 kPA	2.00 kPA		1.16 kN/m	5.80 kN/m	
					0.00 kN/m	0.00 kN/m	
							0.00 kN/m
TOTAL					1.28 kN/m	5.80 kN/m	0.00 kN/m

UDL (ULS) , kN/m
1.2G+1.5Q= 10.24
0.9G+Wu= 1.15

UDL (SLS), kN/m
$G+\psi_sQ$ = 5.34
$G+\psi_LQ$ = 3.60 no roof live load
Ws = 0.00

Point Load At Position A= *(Measured from LHS)*

	Trib(m^2)	G	Q	W	W_G	W_Q	W_W
					0.00kN	0.00kN	
					0.00kN	0.00kN	
					0.00kN	0.00kN	
					0.00kN	0.00kN	
					0.00kN	0.00kN	
							0.00kN
TOTAL					0.00kN	0.00kN	0.00kN

1.2G+1.5Q= 0.00
0.9G+Wu= 0.00

$G+\psi_sQ$ = 0.00
$G+\psi_LQ$ = 0.00 no roof live load
Ws = 0

Bending Moment

Load Case	Mu*
1.2G+1.5Q =	14.79 kNm
0.9G+Wu =	1.66 kNm

Critical Bending Moment = 14.79 kNm

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CONSULTING CIVIL & STRUCTURAL ENGINEERS

Job	136 MORRINSVILLE ROAD	Job No	12843
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Try Timber Section

NZS3603

Width (b) = **84 mm** L_{ay} = 0.60 m Grade : PL12 Wet
Depth (d) = **290 mm**

φ_f =	0.8	CL 2.5
k_1 =	0.8	Table 2.4
k_2 (defl) =	2.0	Table 2.5
k_4 =	1	Table 2.7
k_5 =	1.000	Eq 2.5
k_8 =	1.00	Table 2.8
f_b =	20 MPa	
E =	9.2 GPa	
S_1 =	6.56	Eq 3.5

$\varphi M_n = \varphi k_1 k_4 k_5 k_8 f_b Z_{xx}$
= 15.07 kNm **> Mu* therefore O.K**

Deflection :

Dead =	1.4 mm	= Span / 2398
$G+\psi_s Q$ =	5.9 mm	= Span / 575
$G+\psi_L Q$ =	8.0 mm	= Span / 426
W_s =	0.0 mm	= Span / #DIV/0!

Try Steel Section

NZS3404

Section	**I_{xx}**	**Z_{xx}**
200 PFC 22.9	19.10x10^6mm4	221x10^3mm3

f_y =	300 MPa	
φ_f =	0.9	Table 3.3(1)
E =	200 GPa	
α_s	0.471	CL 5.6.1.1.2[c]
α_m	1.130	CL 5.6.1.1.1(b)
	PP	End Restraint Table 5.6.3(1)
k_t	1.11	Twist restraint factors Tb 5.6.3 (1)
k_r	1.00	Rotation restraint factors Tb 5.6.3 (3)
k_l	1.00	Load height factors Tb 5.6.3 (2)

$\varphi M_{sx} = \varphi Z_{xx} f_y$
= 59.67 **> Mu* therefore O.K**

Segment Length 3.60 m $L_e = k_t.k_r.k_l L$ 4.00 m

φM_{bx} = 59.67 kNm
$\varphi M_{bx} = \alpha_m \alpha_s M_{sx} \le M_{sx}$ 31.78 kNm **> Mu* therefore O.K**

Deflection :

Dead =	0.6mm	= Span / 5831
$G+\psi_s Q$ =	2.4mm	= Span / 1398
$G+\psi_L Q$ =	1.6mm	= Span / 2073
W_s =	0.0mm	= Span / #DIV/0!

Reactions

	LHS	RHS
Dead (kn) =	2.2kN	2.2kN
Live (kn) =	9.9kN	9.9kN
Wind (kn) =	0.0kN	0.0kN
Rult (1.2G+1.5Q) =	17.4kN	17.4kN
(0.9G+Wu)=	2.0kN	2.0kN

Footing

Using square footing of size= 1.00 m
Φ_{bc} 0.5
q_u 200 kPA

	LHS	RHS
fq (wkg) kPa =	12.0 kPA	12.0 kPA
fq (ULS) kPa =	17.4 kPA	17.4 kPA

Summary

Beam Location: **Bearer 03 Design**

Beam Size:	**84**	**x 290mm**	**PL12 Wet**	Span	**3.40 m**
Supports:	**3/90x45 studs**				
RC Pad:					
Reinforcement:					

MICIUS
Civil & Structural Engineering Consultants

Proposed Cameron Residence

At

Lot 2 136 SH 26, Newstead

Reference: 210176

Document

STRUCTURAL CALCULATION

Contents

Prepared by: BZ
Reviewed by: HT

Revision: A
Date: November 2021
Status: Building consent

Hamilton City Council
BUILDING UNIT
APPROVED
BC Number - DD007.2021.00043914.001

S1.0 Structurl Summary

Building Description
The proposed two storey dwelling is a lightsteel framed structure with lightweight longrun iron roofing with lightweight vertical weatherboard cladding. The building is found on concrete slab foundation supported on SED driven piles. This PS1 and calculation covers the lightsteel framing and the roof beams only. The rest of the structural elements and overall bracing are designed by others.

Scope

- Lightsteel framing include roof trusses, lintels and wall studs.
- First Floor roof beams and posts
- Lightsteel midfloor joists
- Lightsteel connection for balustrade

- Exclude: Overall bracing design, ground floor structural steels beams and posts, foundation and retaining wall, timber decks, timber beams are designed by others.

Design Codes

- Elements designed to support loadings of AS/NZS 1170-2002.
- Light steel designed to AS/NZS 4600-2018
- Light steel framing designed to NASH Standard Part 1 V2: October 2010 and NASH Standard Part 2: May 2019.
- Steel designed to NZS 3404-1997

Design Loadings

- Building Important Level = IL 2
- Design life = 50 years
- Floor Live Loads = 1.5kPa (Domestic floor)
- Design Wind Speed = 42.4m/s (High Wind Zone)
- Seismic Z-Factor = 0.16 (Hamilton)
- Seismic Soil Class = D
- Ground Snow Load = 0kPa

Corrosion

- External Areas: Not applicable, all design elements are enclosed

- Sheltered Areas:

	Timber	**Structural Steel**	**Lightsteel**	**Concrete**
Zone	**B**	**C1**	**C2**	**B1**
Proviosion	N/A	ALK1	Z275	N/A

Hamilton City Council
Te kaunihera o Kirikiriroa

BUILDING UNIT APPROVED

BC Number - DD007.2021.00043914.001

Construction Monitoring
The design is based on the verification of specific design aspects of the construction by a suitably qualified Chartered Professional Engineer or by a qualified building inspector from the local authority in accordance with ACENZ/IPENZ level CM 3.

- Superstructure pre-line inspections of lightsteel framing and connections.
- Superstructure pre-line inspections of SED beams, posts and connections.

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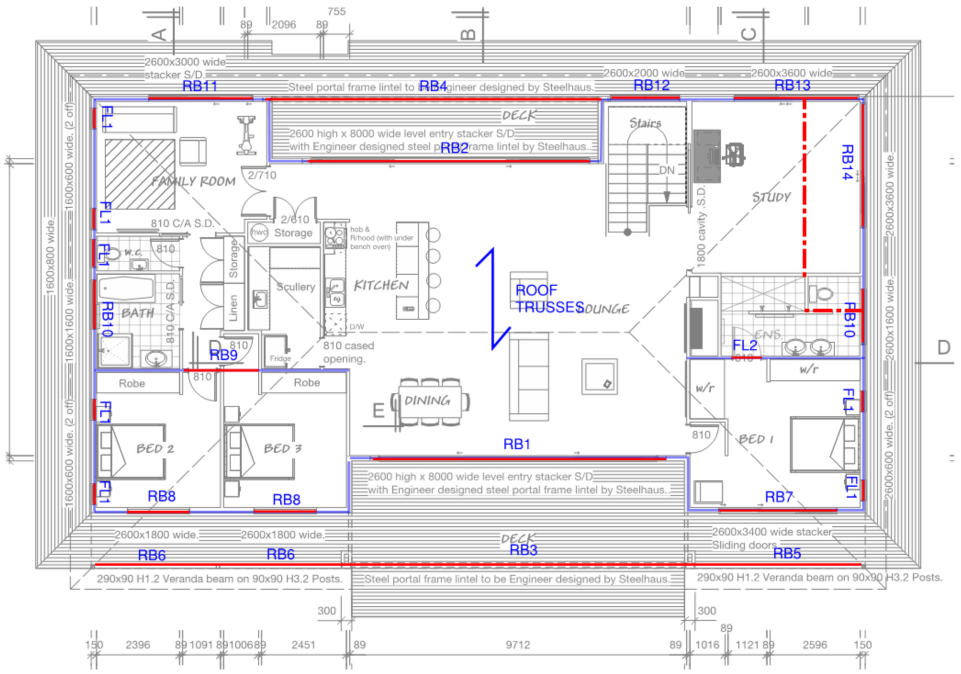
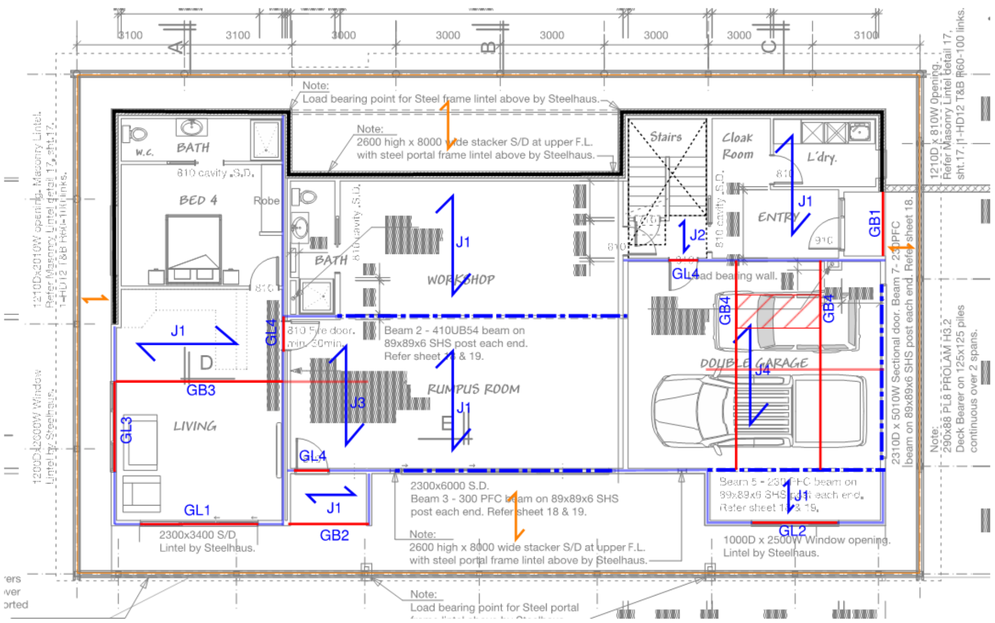
BC Number - DD007.2021.00043914.001

S2.0 Key Plans





:Floor beams by others

:Lightsteel load bearing walls

:Load bearing lintel/beam

:Timber balcony by others

S3.0 Design Loadings

Dead and Live Loads

Roof Type A

Dead	0.55BMT Steel roofing + steel purlins	=	0.07	kPa
	Light trusses @ 900crs	=	0.05	kPa
	Ceiling Batterns + 13mm Gib Ceiling + Services	=	0.15	kPa
Live	Non accesiable	=	0.25	kPa
	Total	DL =	**0.27**	**kPa**
		LL =	**0.25**	**kPa**

Floor Type A

Dead	19mm particleboard flooring	=	0.20	kPa
	Steel Joists @ 450crs	=	0.15	kPa
	Ceiling Batterns + 13mm Gib Ceiling + Services	=	0.15	kPa
Live	Self-contain dwelling	=	1.50	kPa
	Total	DL =	**0.50**	**kPa**
		LL =	**1.50**	**kPa**

External Wall

Dead	JH weatherboard over batten + TB	=	0.26	kPa
	Steel Studs @ 600crs	=	0.07	kPa
	10mm Gib wall lining	=	0.09	kPa
Live		=	0.00	kPa
	Total	DL =	**0.42**	**kPa**
		LL =	**0.00**	**kPa**

Important Level & Design Life

Building Important Level	=	2	Comment:	Self-containt residential dwellin
Design Life	=	50 years		

	Recurrence, R		
Actions	ULS	SLS	SLS2
Wind	500	25	N/A
Snow	150	25	N/A
EQ	500	25	N/A

Wind Loads

Design Wind Speeds

-Determine design wind speeds using NZS1170.2

Critical directional wind		=	West	
Region		=	A7	
Reference height	z	=	6.8	m
Terrain category		=	Category 2	
Lee Zone		=	No	
Height of hill	H	=	0	m
Horiz. Distance to H/2	Lu	=	200	m
Horiz. Distance crest to site	x	=	200	m

Regional V,R	ULS	=	45 m/s	*(section 3, Table 3.1)*
	SLS	=	37 m/s	
Wind directional multiplier	Md	=	1.00	*(section 4, Table3.2)*
Terrain/height multiplier	Mz,cat	=	0.94	*(section4)*
Shielding multiplier	Ms	=	1.00	*(section4)*
Topographic multiplier	Mt	=	1.00	*(section4)*

Design Site Wind Speed, Vsit = V,R x Md x (Mz,cat x Ms x Mt)

ULS,Vsit	=	**42.4**	**m/s**
SLS,Vsit	=	**34.9**	**m/s**

Wind Pressure Coefficients:

Internal	Cpi =	0.0	or	-0.3	
Windward wall	Cpe =	0.7			*(Table 5.2(a))*
Leeward wall	Cpe =	-0.5			*(Table 5.2(b))*
Side wall	Cpe =	-0.7			*(Table 5.2(C))*
Up-wind, roof	Cpe =	-1.2	or	-0.5	*(Table 5.3)*
Down-wind, roof	Cpe =	-1.2	or	-0.5	*(Table 5.3)*
Cross-wind, roof	Cpe =	-0.2	or	0.2	*(Table 5.3)*
Along-wind, canopy	Cp,n =	-0.4	or	0.4	*(Table D4)*
Across-wind, canopy	Cp,n =	-1.5	or	0.2	*(Table D8)*

Design Wind Pressures:

Area reduction factor	Ka =	1.0
External combination factor	Kc,e =	1.0
Internal combination factor	Kc,i =	1.0
Local pressure factor	Kl =	1.0

Critical Design Wind Pressures:

	ULS Vsit	SLS Vsit	Net Cp	ULS,pz		SLS,pz	
Roof, up	42.408	34.8688	-1.2	**-1.29**	**kPa**	**-0.88**	**kPa**
Roof, down	42.408	34.8688	0.5	**0.54**	**kPa**	**0.36**	**kPa**
Wall (absolute)	42.408	34.8688	1.0	**1.08**	**kPa**	**0.73**	**kPa**
Canopy, up	42.408	34.8688	-1.5	**-1.62**	**kPa**	**-1.09**	**kPa**
Canopy, down	42.408	34.8688	0.4	**0.43**	**kPa**	**0.29**	**kPa**

Snow Loads

Snow Region		=	N1	
Site Elevation	h_0	=	40	m
Alpine / Sub-Alpine / No Snow		=	No Snow	
Exposure Category		=	Sub-Alpine	
Roof angle	α	=	3	degrees
Snow load on the roof	S	=	**0.00**	kPa

(See Clause 4.2.1: 0.4kPA if Sg < 0.75kPA in Sub-Alpine)

Seismic Loads

Primary Lateral System: The steel framed dwelling is stabilized by Gib Wall Bracing system designed by others

Site Subsoil Class: Class D-Deep or soft soil sites

Specific Design Bracing Elements:

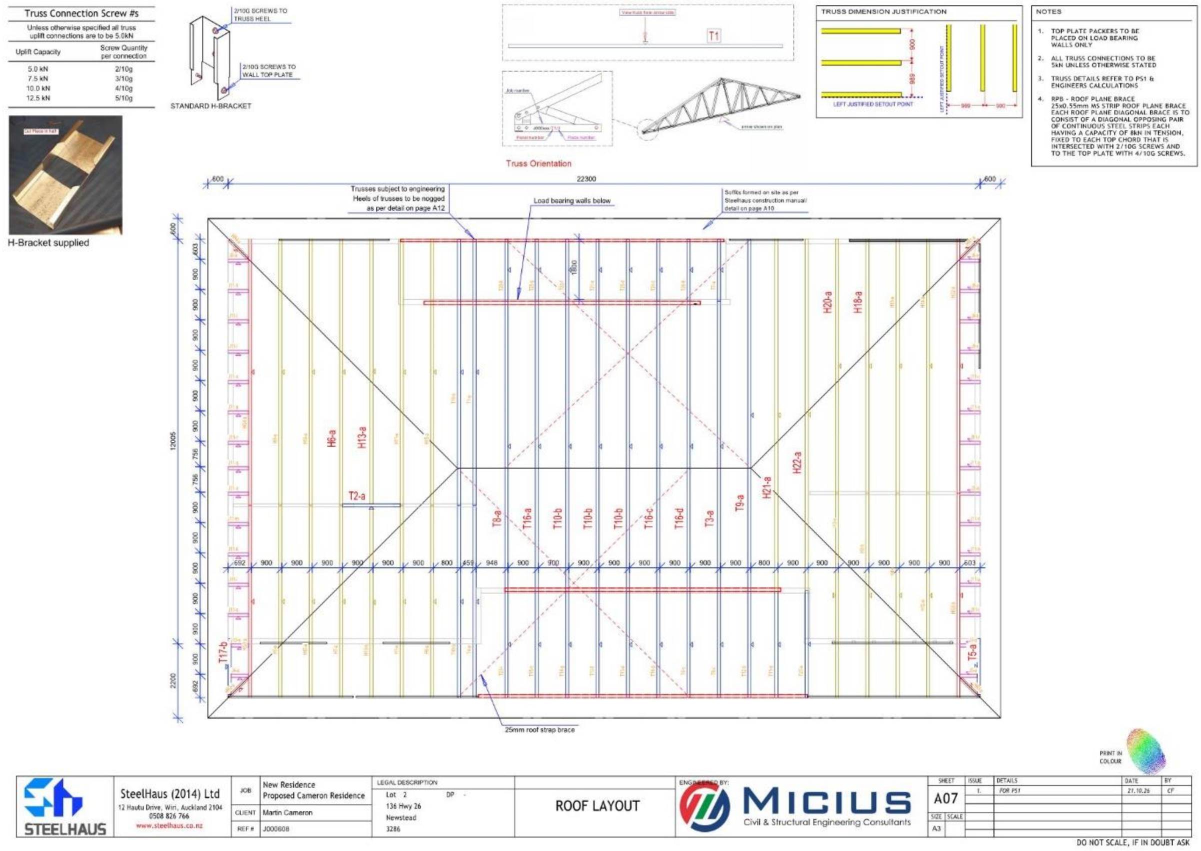
SED Elements

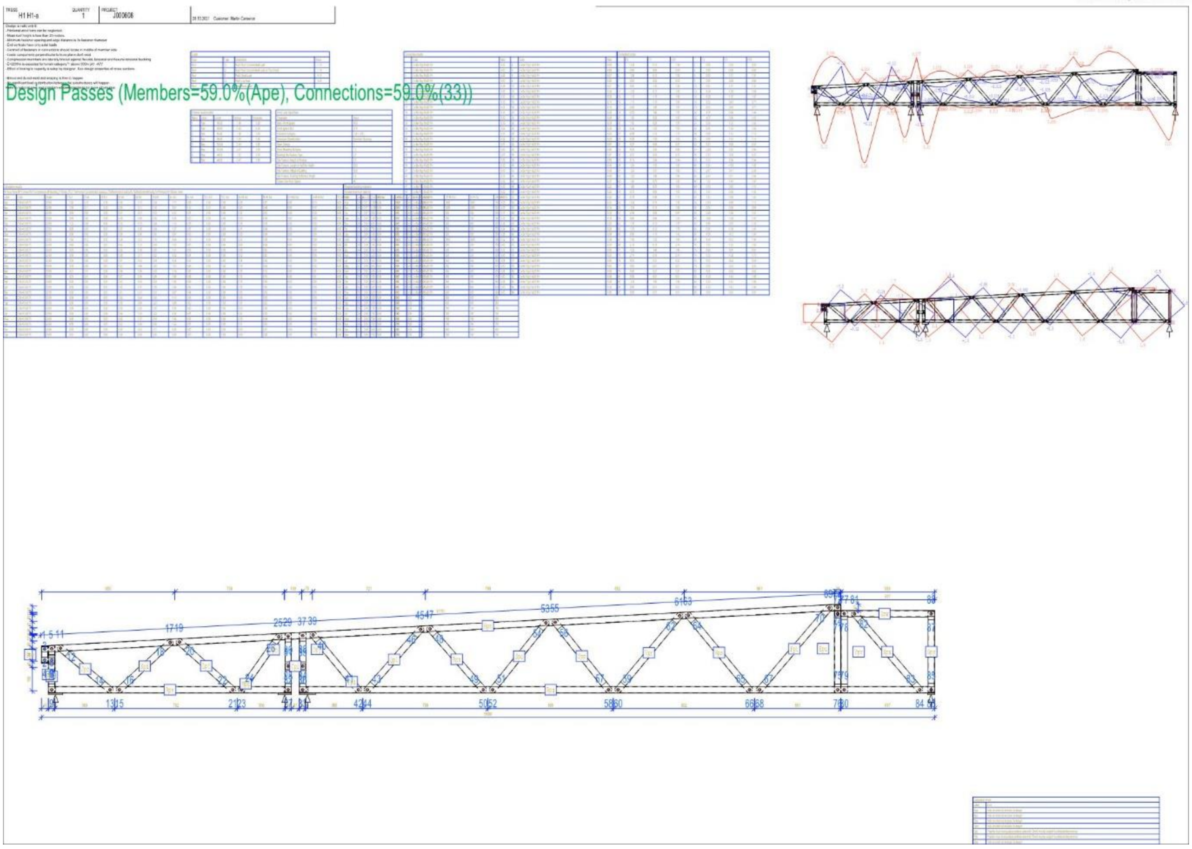
Period =	0.40	sec	Ch(T) =	3.00		*(Table 3.1)*
N(T,D) =	1.00		Z =	0.16	Hamilton	*(Table 3.3)*

Ultimate limit State:		Serviceability limit State	
Ductility, u =	1.25	Ductility, u =	1.00
Ru =	1.00	Rs =	0.25
Sp =	0.925	Sp =	0.7
kμ =	1.14	kμ =	1.14

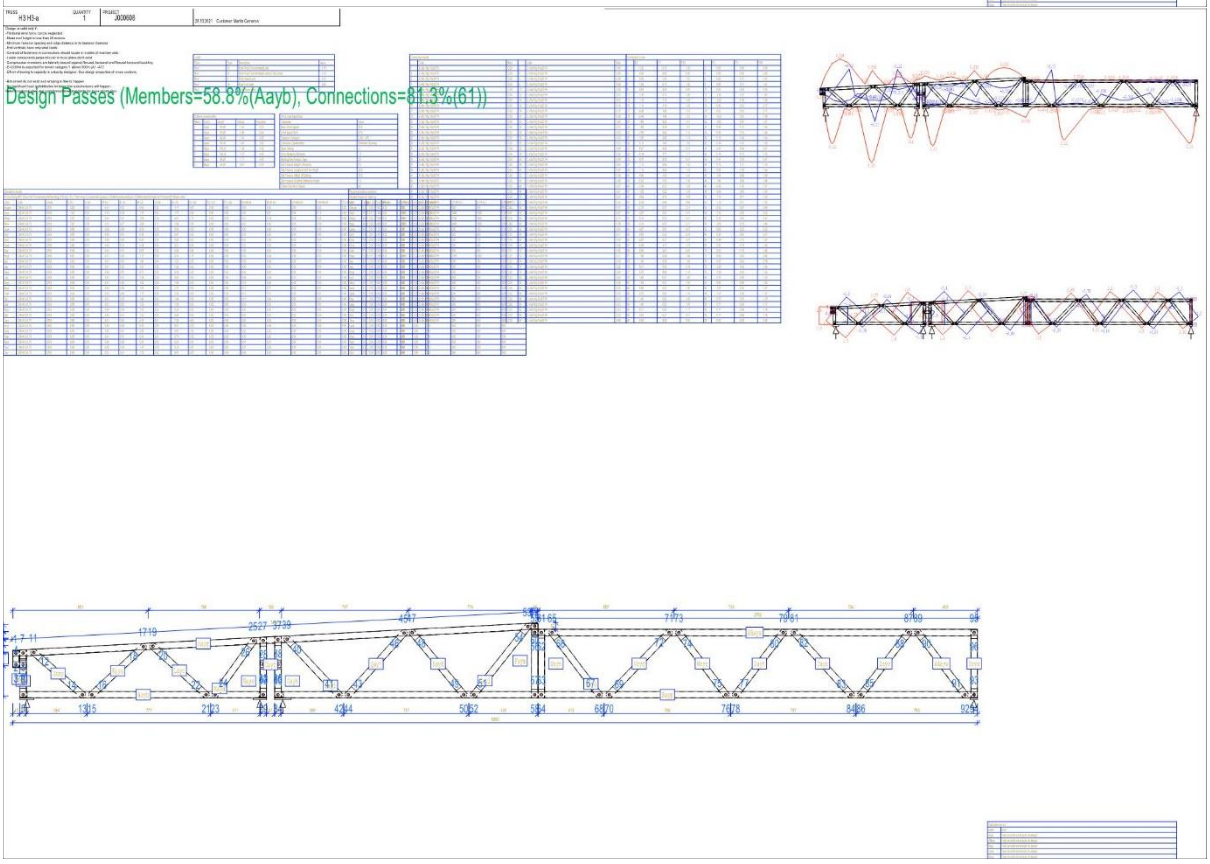
hence,	C(T) = Ch(T) Z R N(T,D) =	0.48	
	ULS Cd(T1) = C(T) Sp / kμ =	**0.39**	*(5.2.1.1)*
	SLS Cd(T1) =	**0.07**	*(5.2.1.2)*

S4.0 Roof Framing

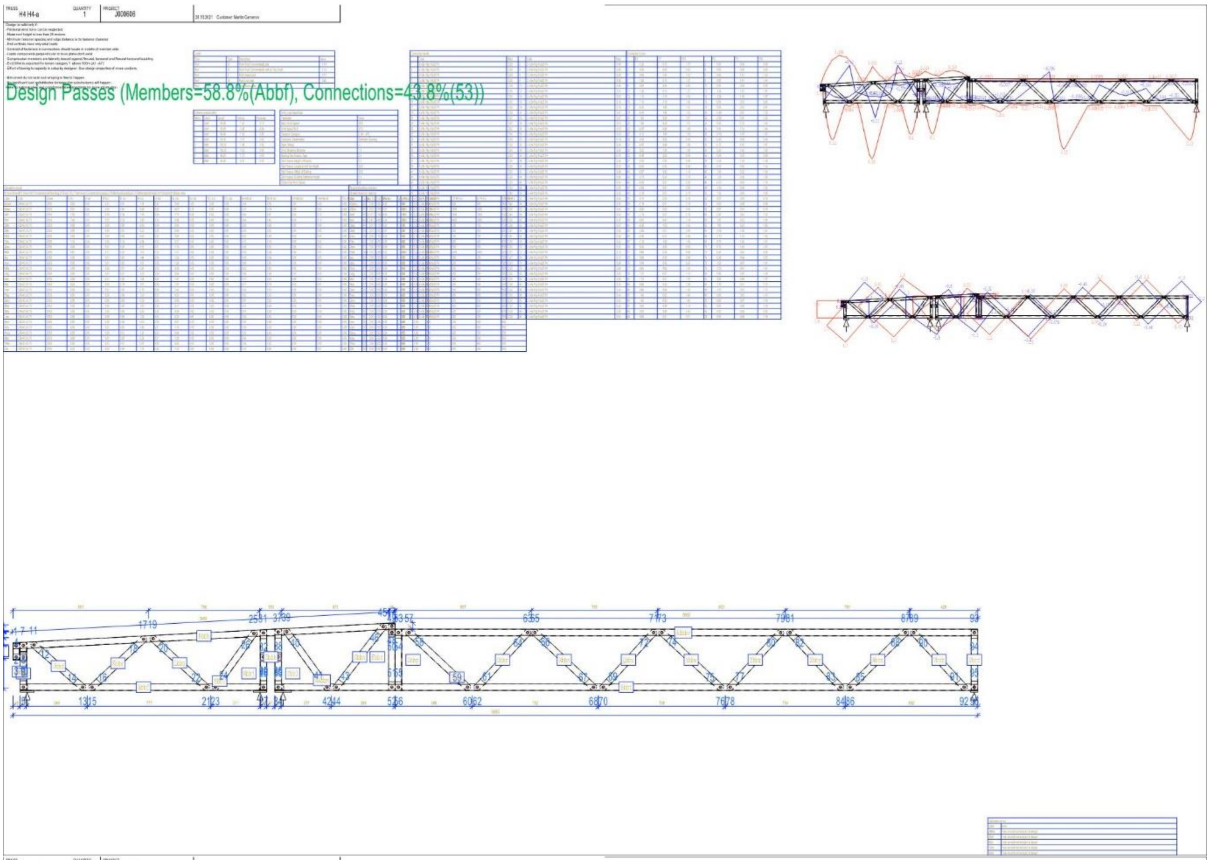


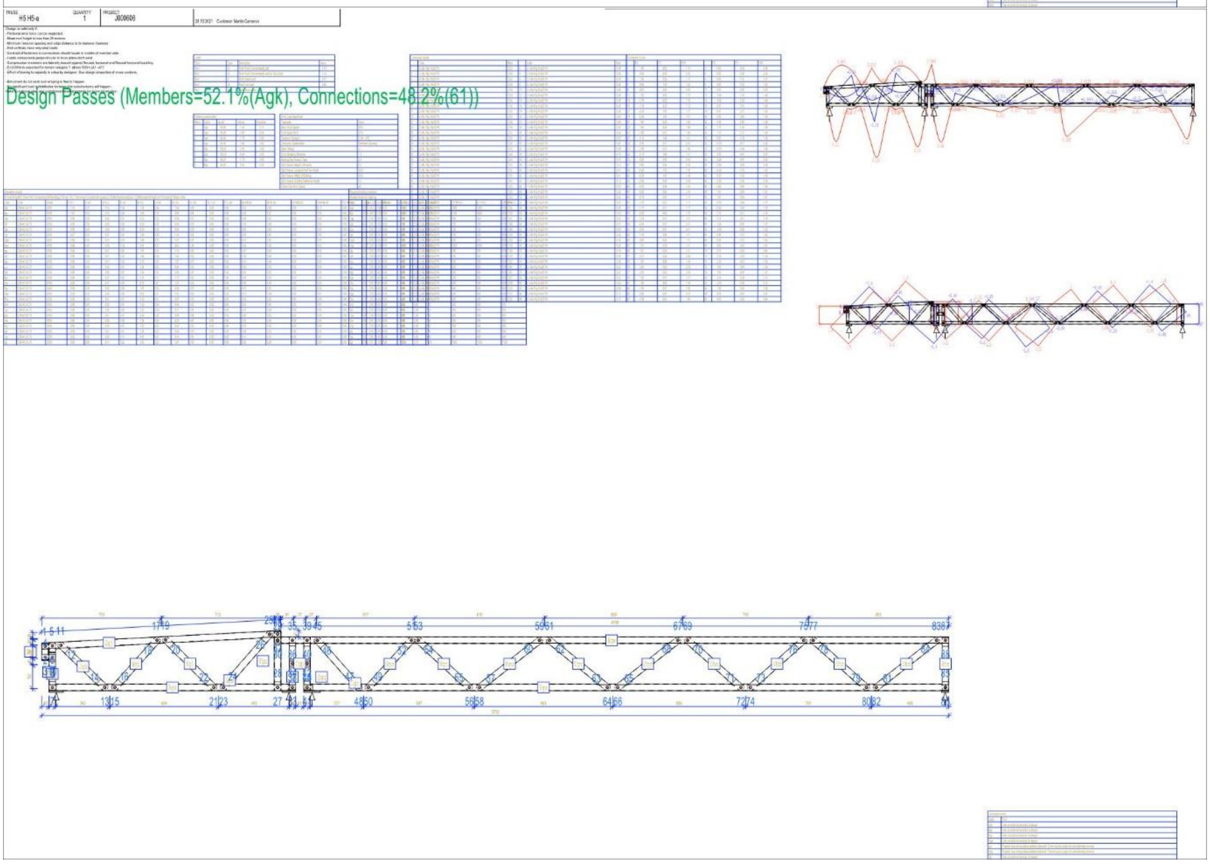


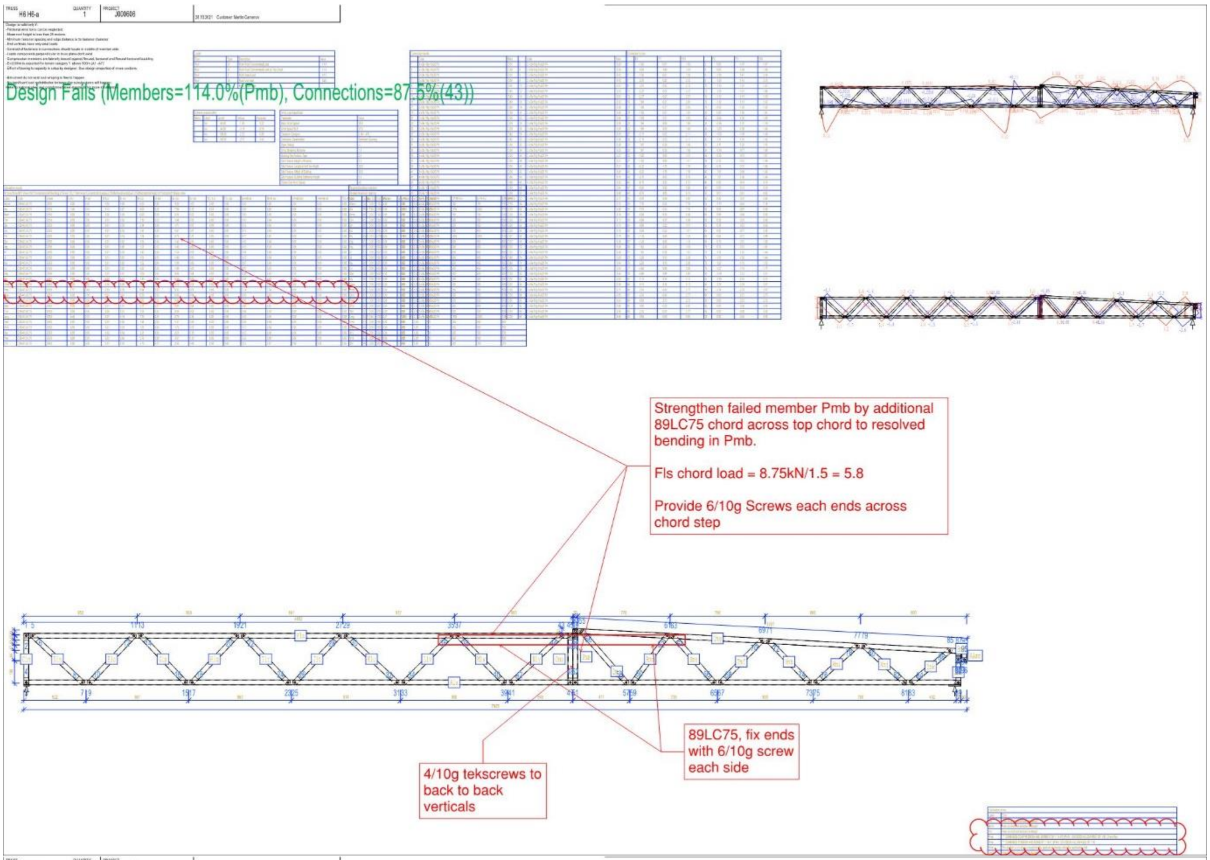
H2 H2-a
Design Passes (Members=58.8%(Aavy), Connections=74.6%(27))



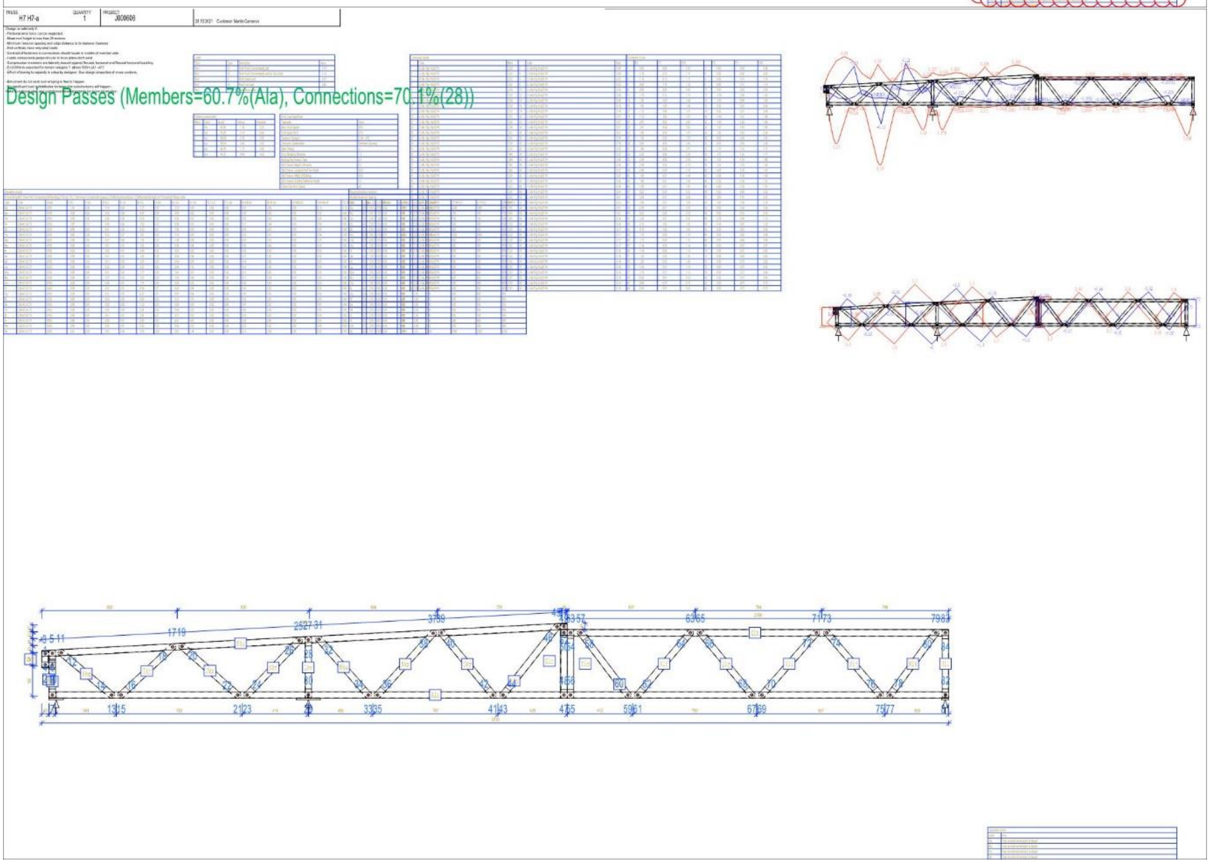
H3 H3-a
Design Passes (Members=58.8%(Aayb), Connections=81.3%(61))


Design Passes (Members=58.8%(Abbf), Connections=43.8%(53))

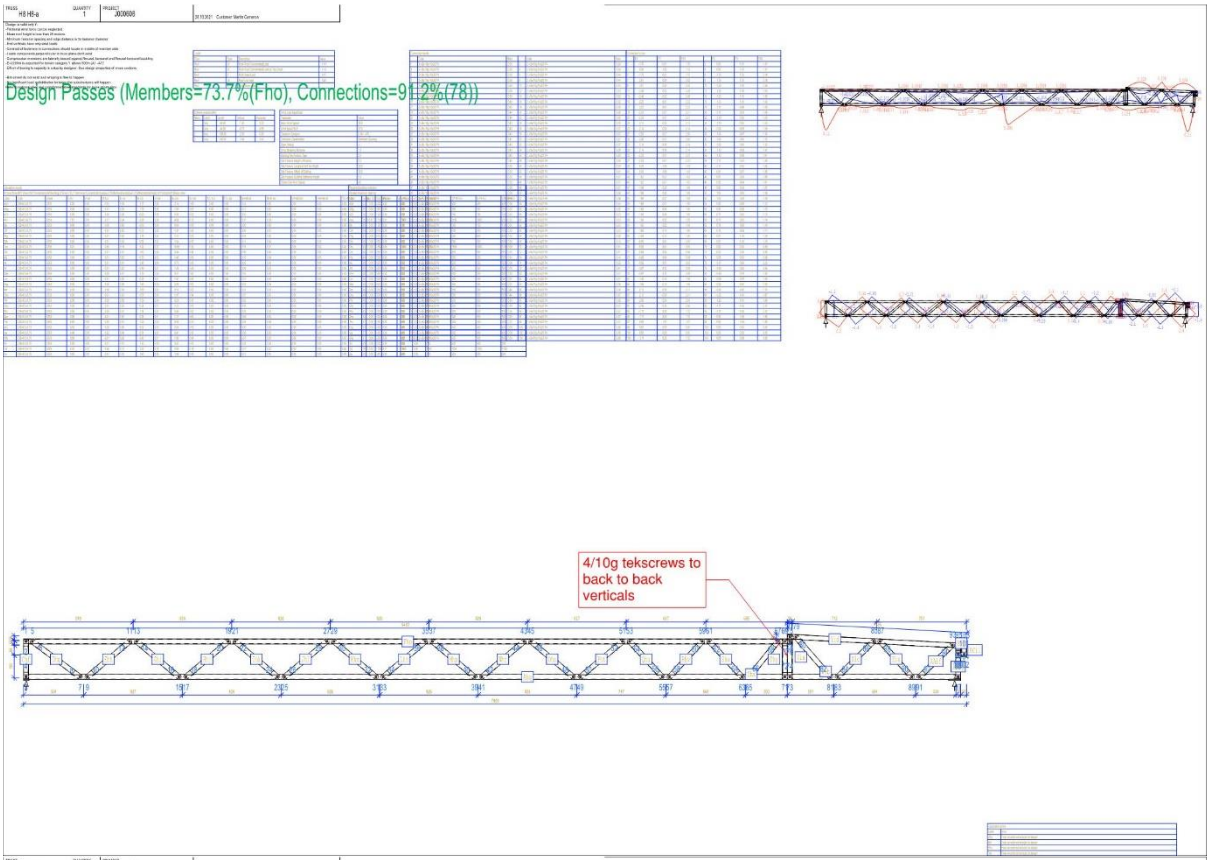

Design Passes (Members=52.1%(Agk), Connections=48.2%(61))

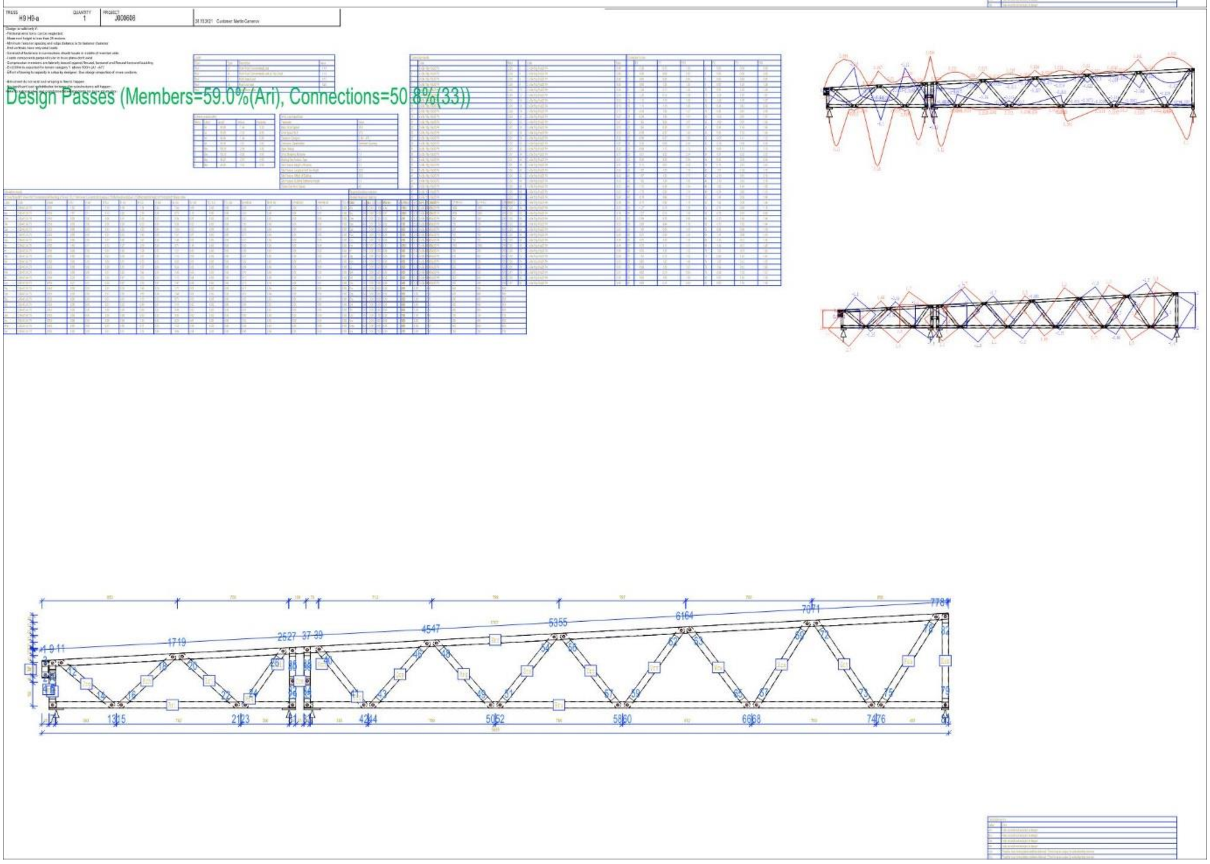


Design Fails (Members=114.0%(Pmb), Connections=87.5%(43))
Strengthen failed member Pmb by additional 89LC75 chord across top chord to resolved bending in Pmb.
Fls chord load = 8.75kN/1.5 = 5.8
Provide 6/10g Screws each ends across chord step
89LC75, fix ends with 6/10g screw each side
4/10g tekscrews to back to back verticals

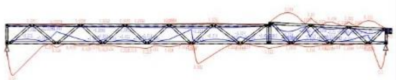


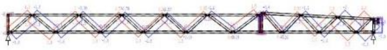
Design Passes (Members=60.7%(Ala), Connections=70.1%(28))

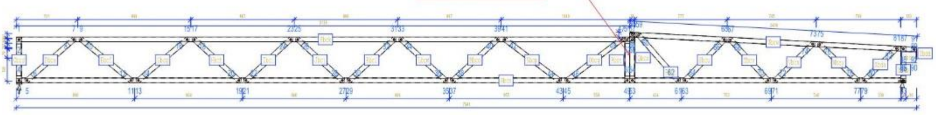

Design Passes (Members=73.7%(Fho), Connections=91.2%(78))
4/10g tekscrews to back to back verticals


Design Passes (Members=59.0%(Ari), Connections=50.8%(33))

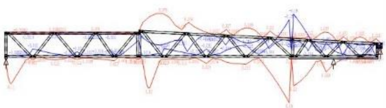
Design Passes (Members=86.2%(Qbcs), Connections=72.9%(77))

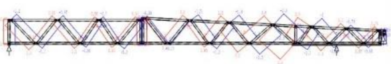


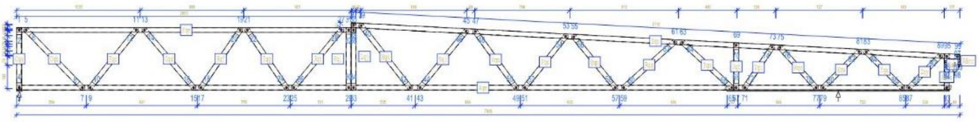


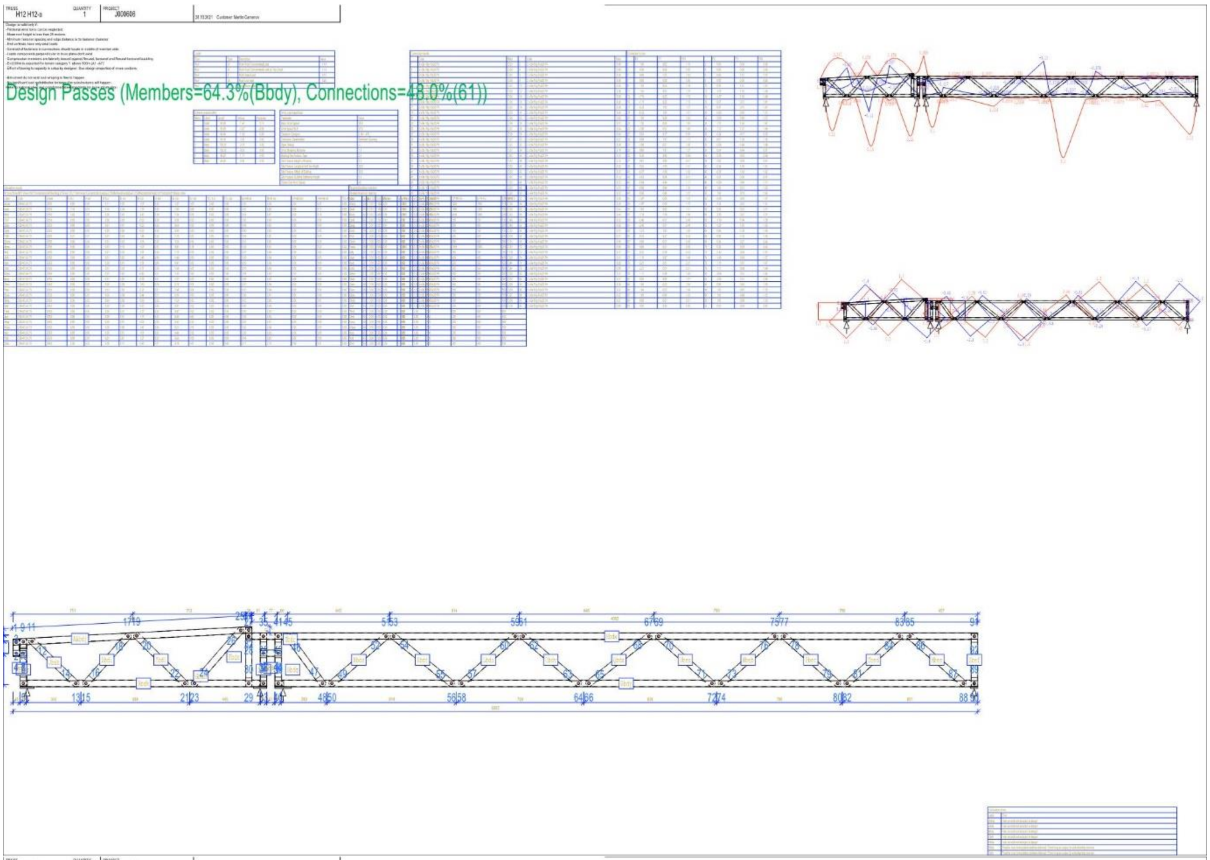


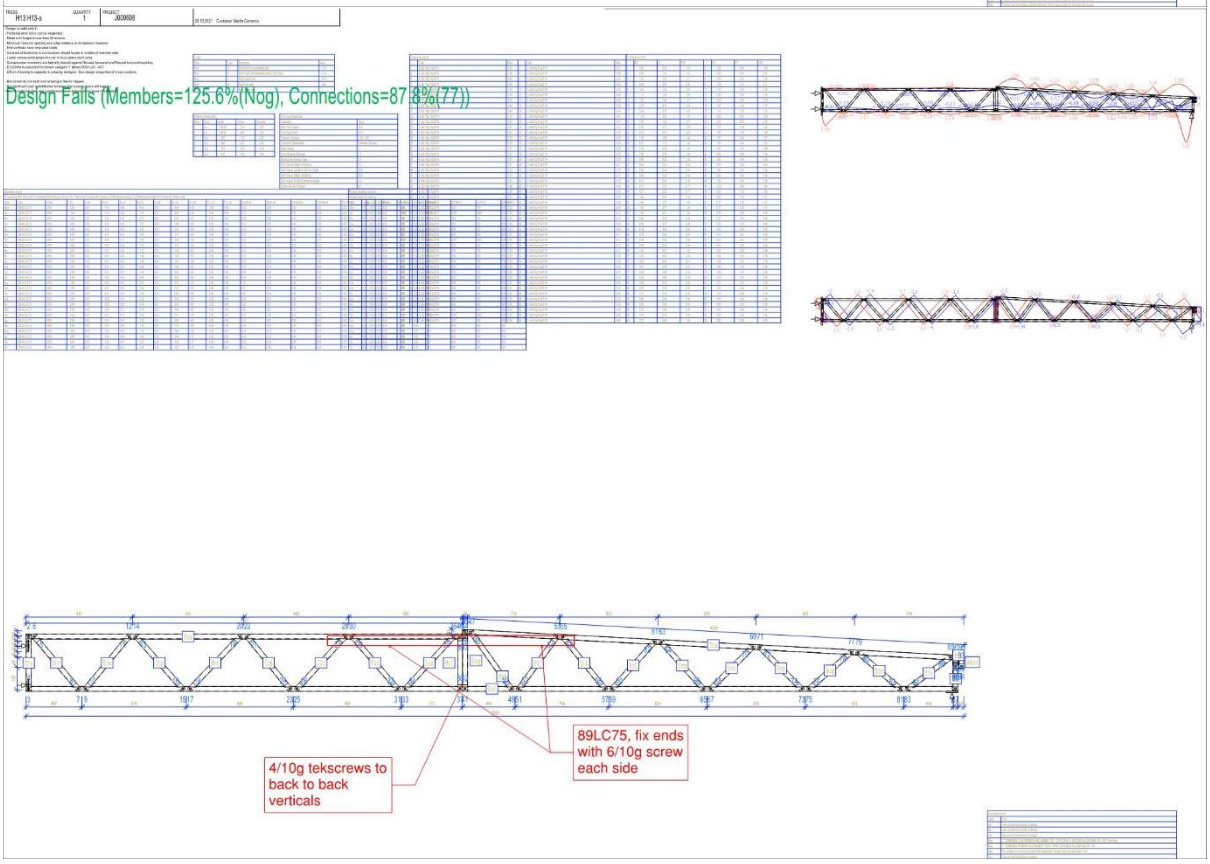
Design Passes (Members=80.6%(Aqm), Connections=87.1%(67))

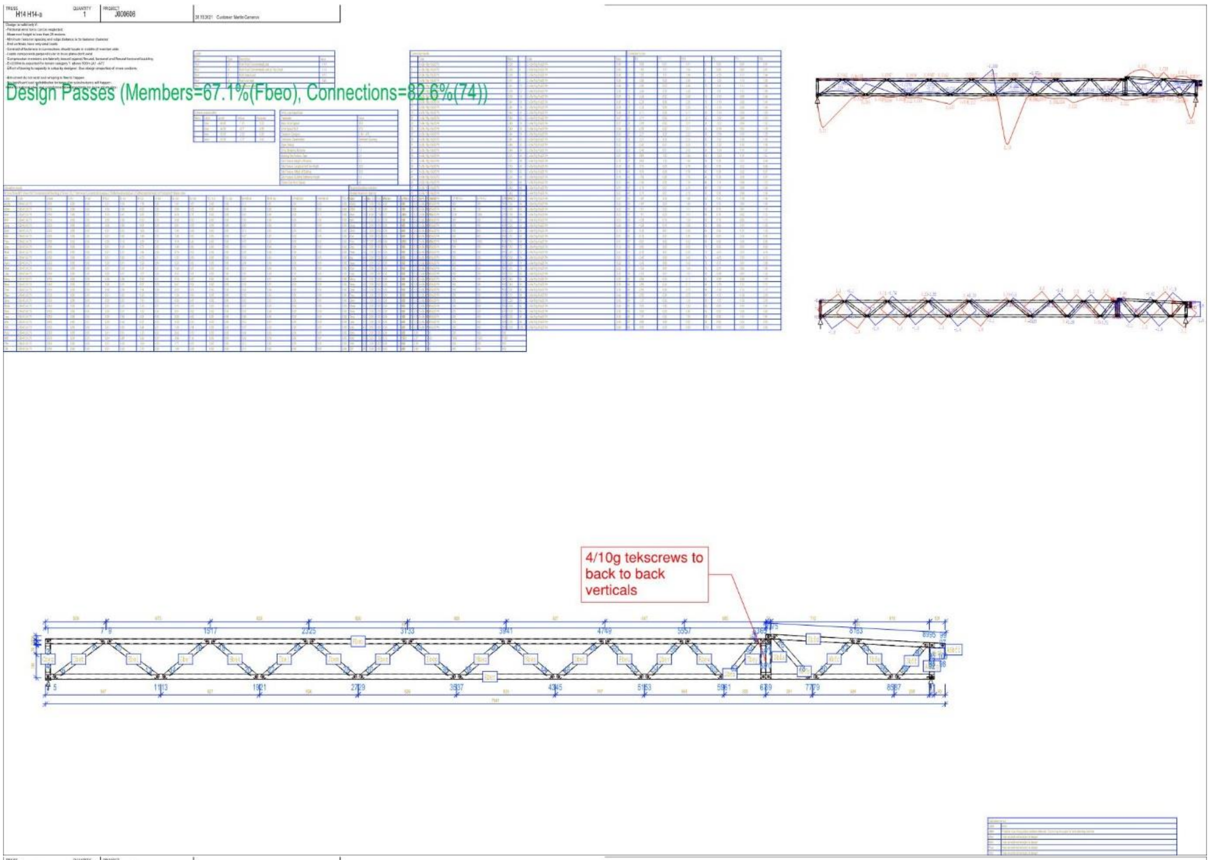




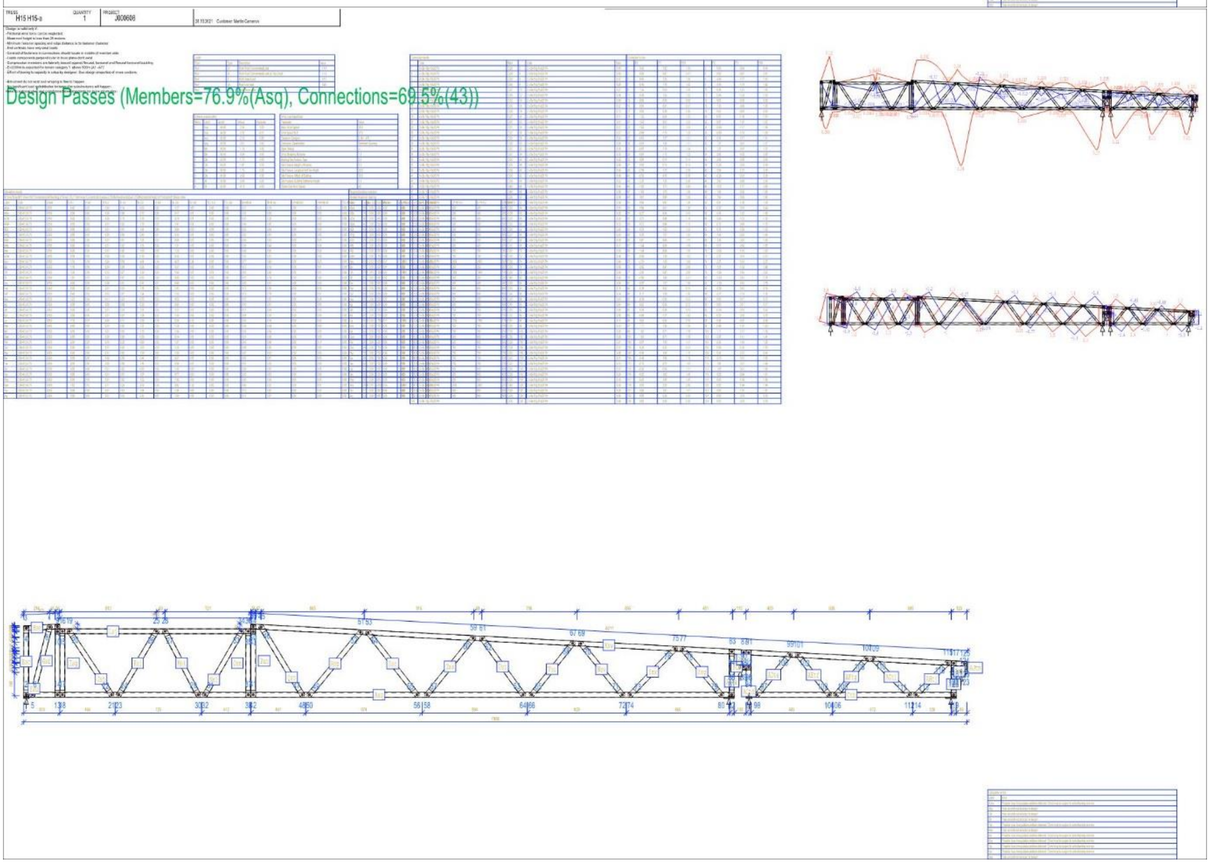



Design Passes (Members=64.3%(Bbdy), Connections=48.0%(61))

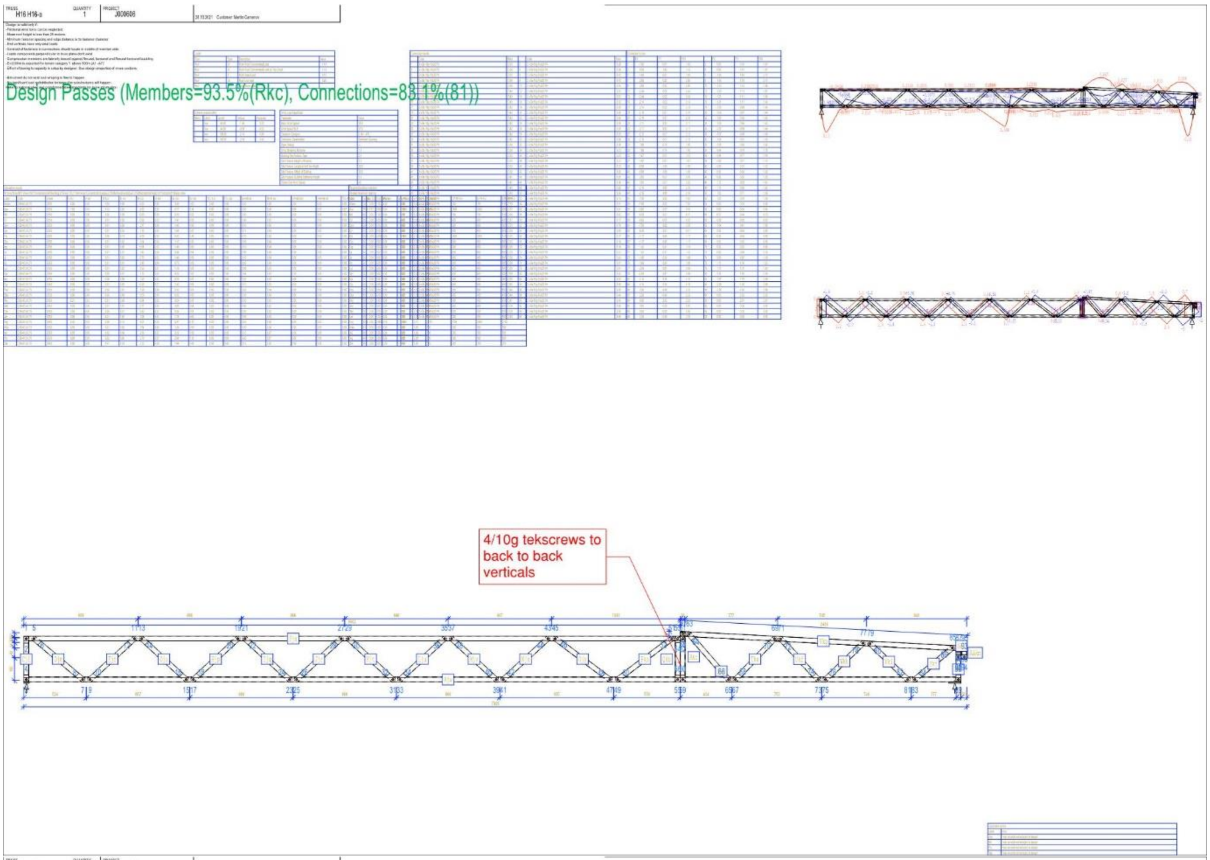

Design Fails (Members=125.6%(Nog), Connections=87.8%(77))
89LC75, fix ends with 6/10g screw each side
4/10g tekscrews to back to back verticals



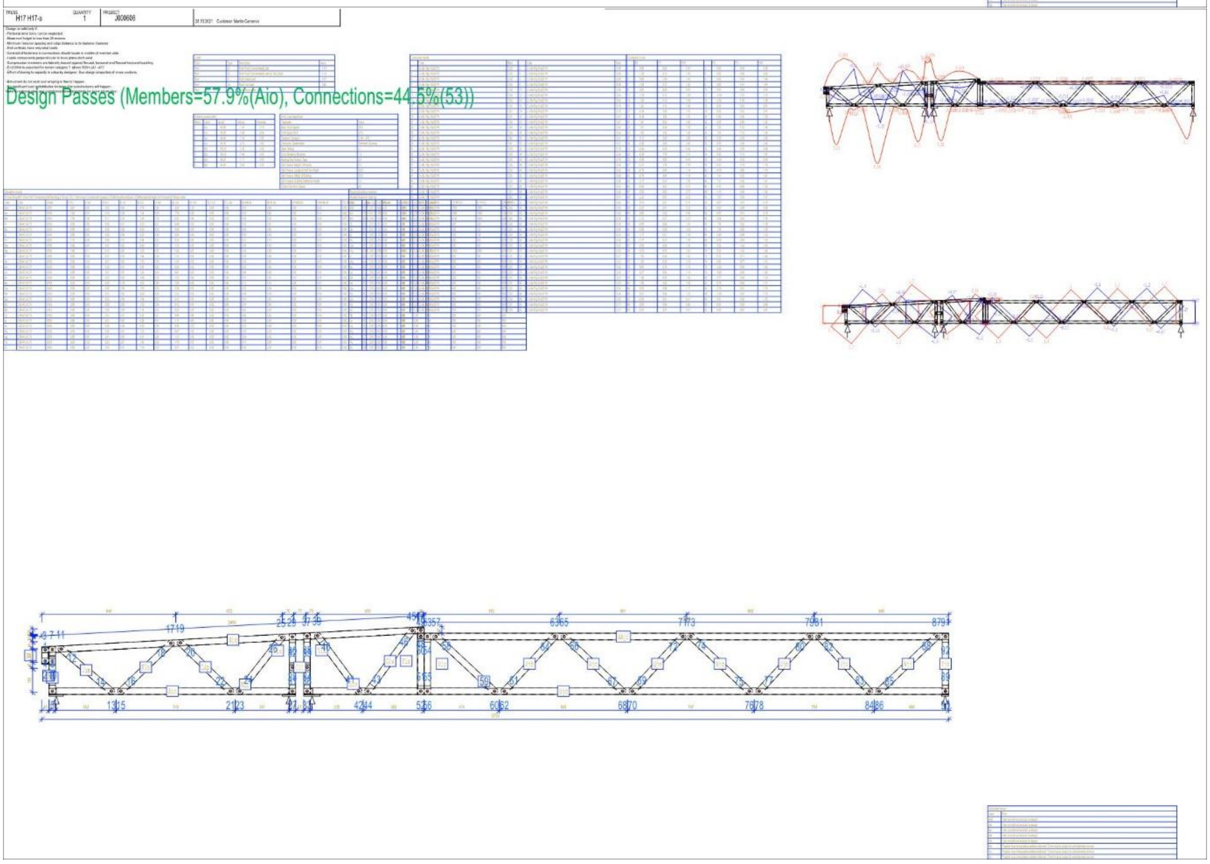
Design Passes (Members=67.1%(Fbeo), Connections=82.6%(74))
4/10g tekscrews to back to back verticals



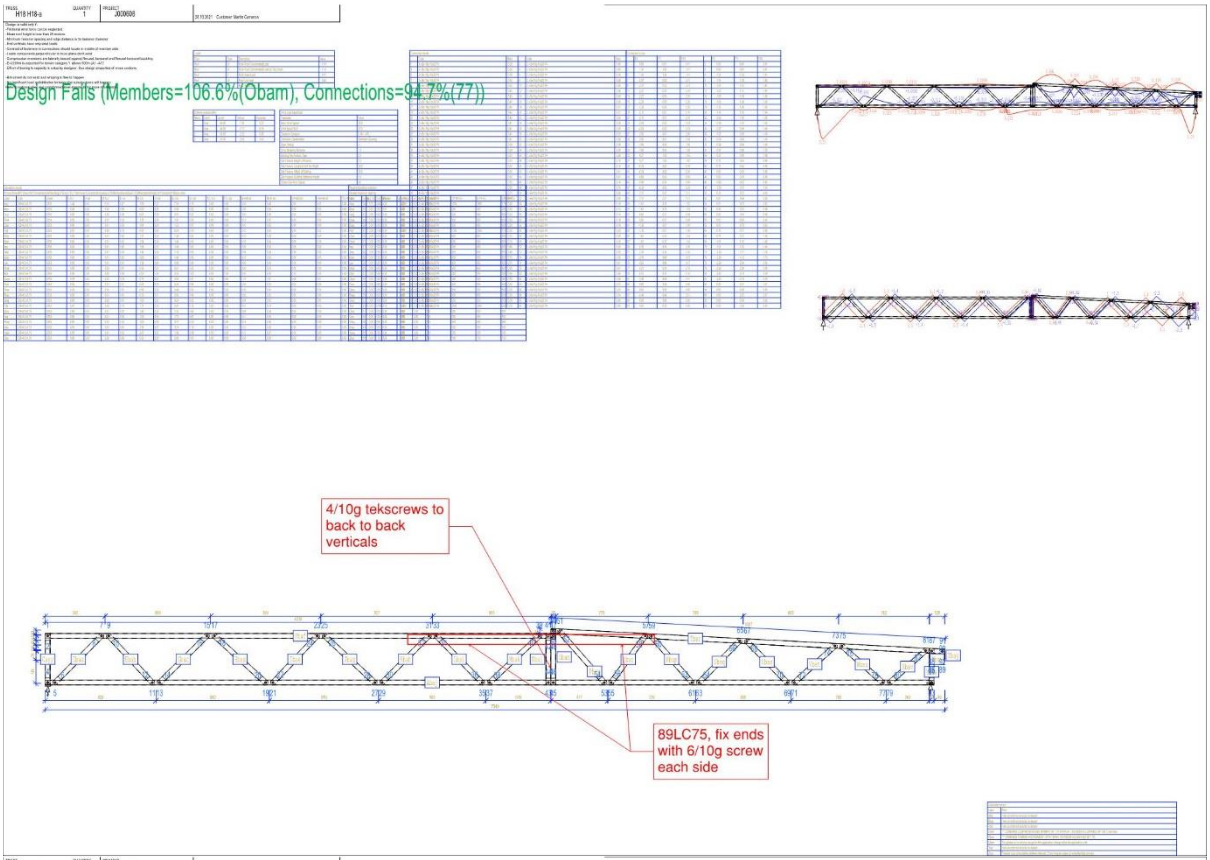
Design Passes (Members=76.9%(Asq), Connections=69.5%(43))



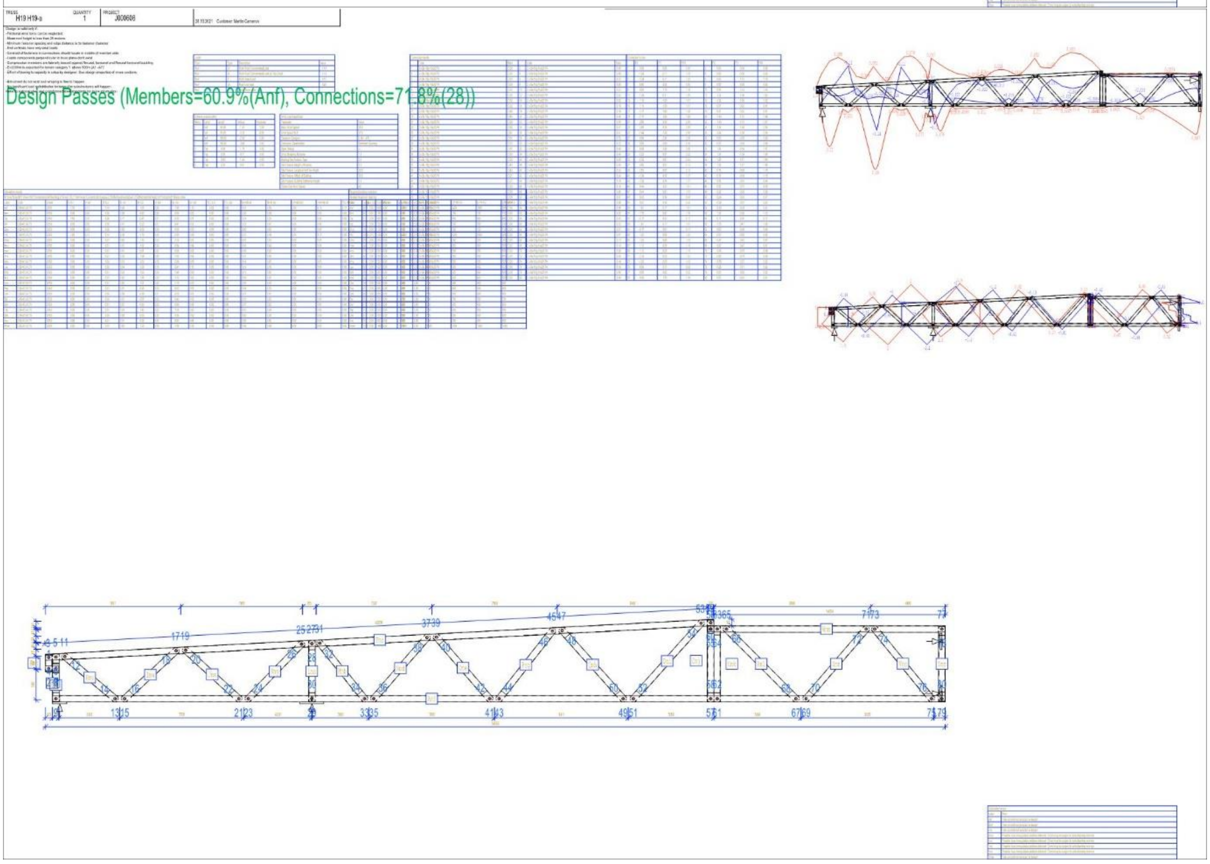
TRUSS H16 H16-a
QUANTITY 1
PROJECT J000606
Customer Martin Cameron
Design Passes (Members=93.5%(Rkc), Connections=83.1%(81))
4/10g tekscrews to back to back verticals



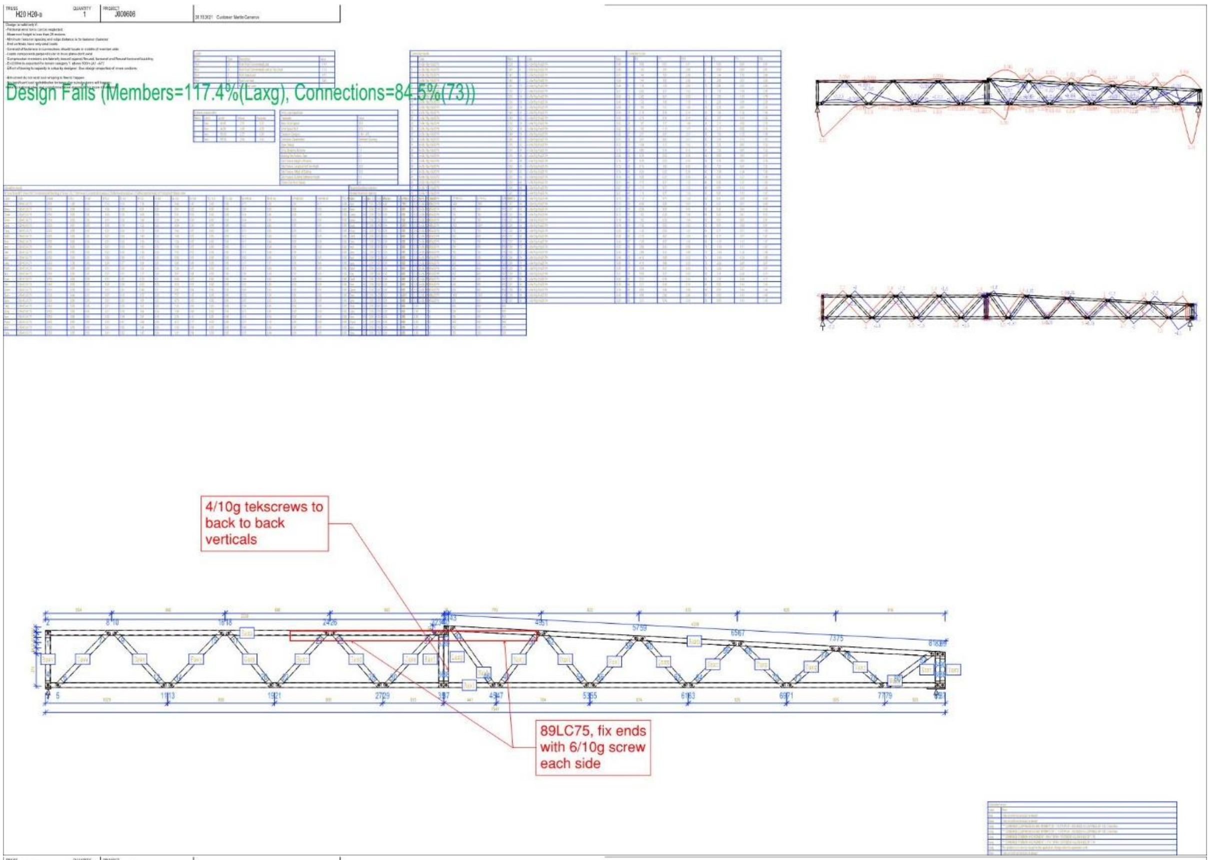
TRUSS H17 H17-a
QUANTITY 1
PROJECT J000606
Customer Martin Cameron
Design Passes (Members=57.9%(Aio), Connections=44.5%(53))



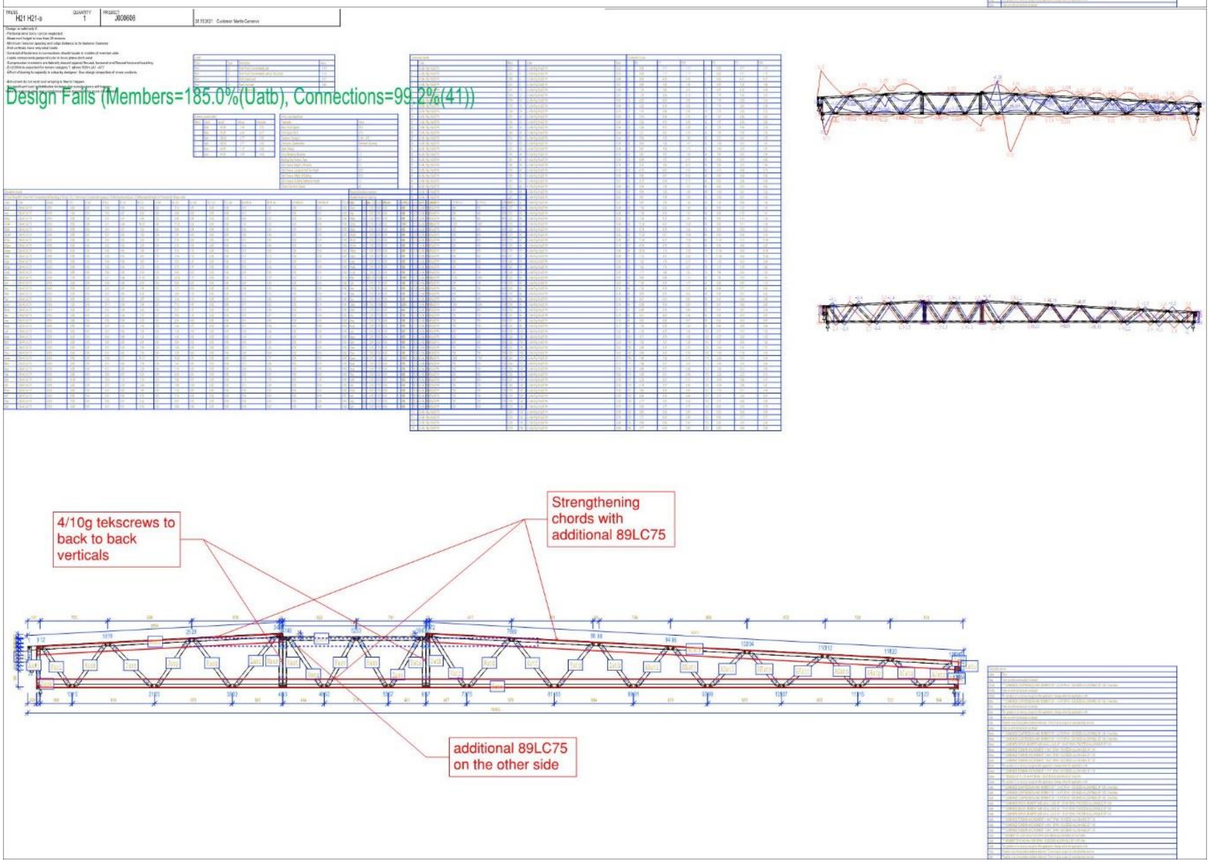
Design Fails (Members=106.6%(Obam), Connections=94.7%(77))
4/10g tekscrews to back to back verticals
89LC75, fix ends with 6/10g screw each side



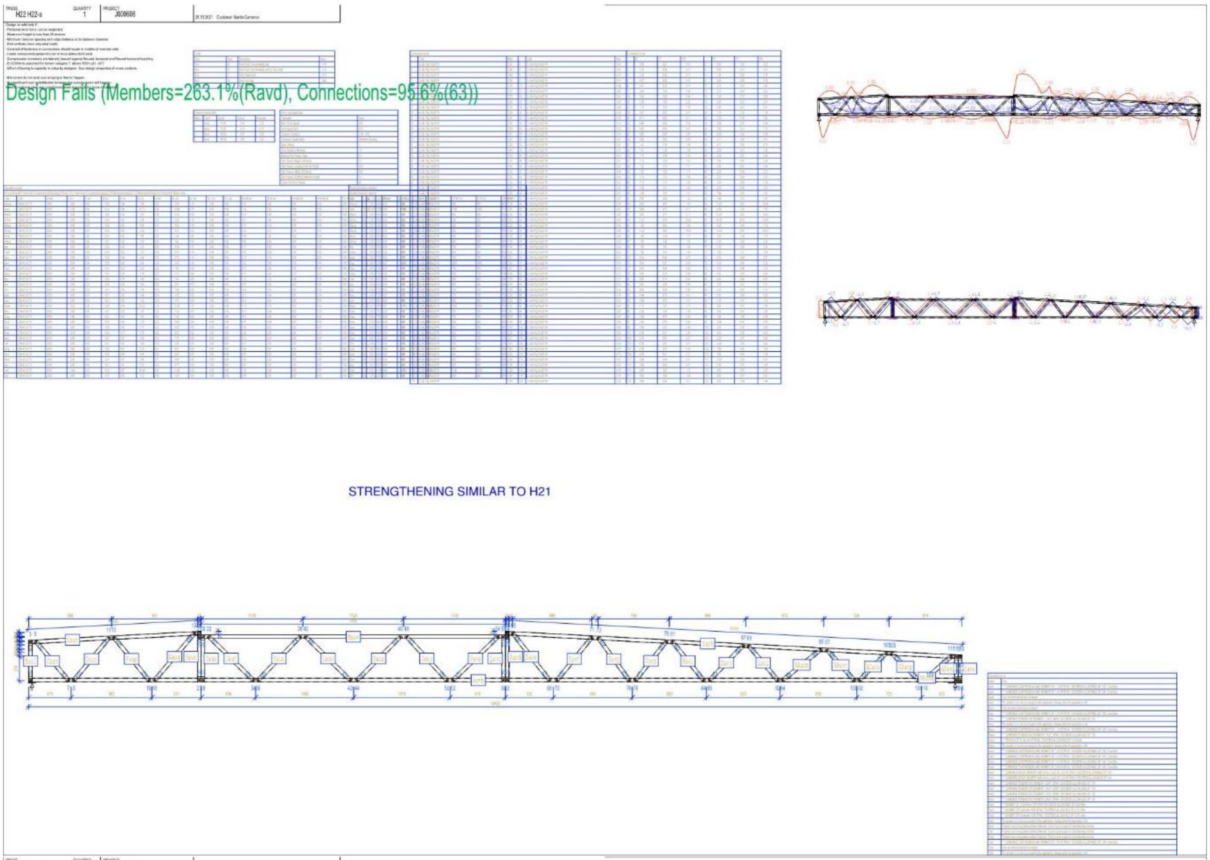
Design Passes (Members=60.9%(Anf), Connections=71.8%(28))



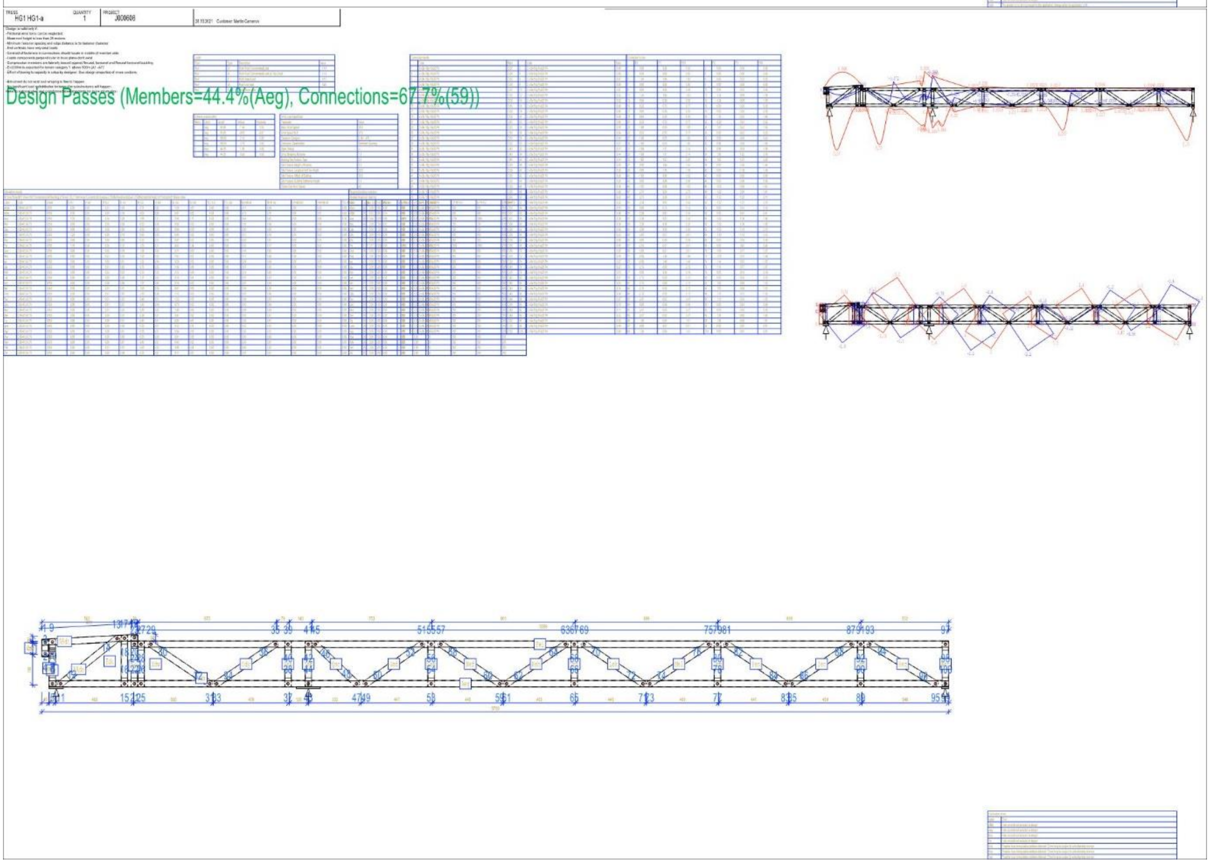
Design Fails (Members=117.4%(Laxg), Connections=84.5%(73))
4/10g tekscrews to back to back verticals
89LC75, fix ends with 6/10g screw each side



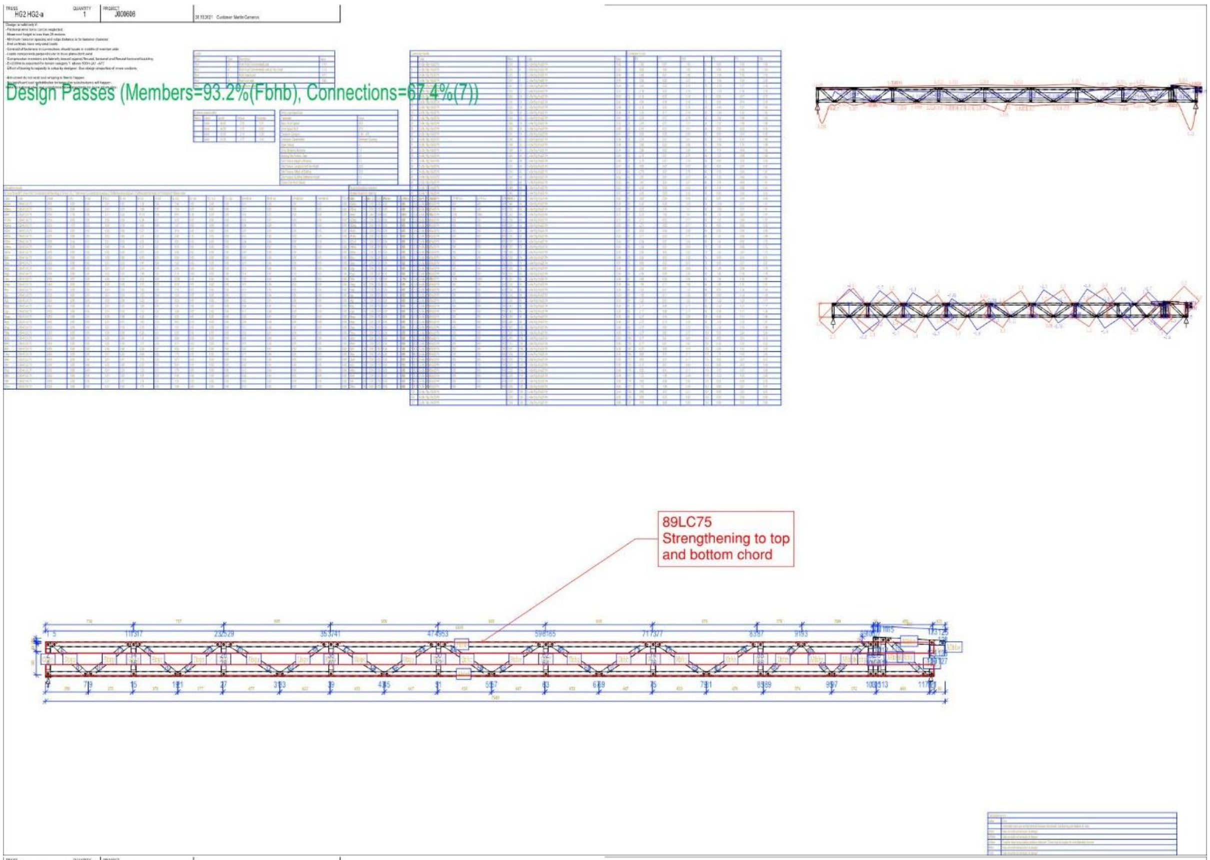
Design Fails (Members=185.0%(Uatb), Connections=99.2%(41))
4/10g tekscrews to back to back verticals
Strengthening chords with additional 89LC75
additional 89LC75 on the other side



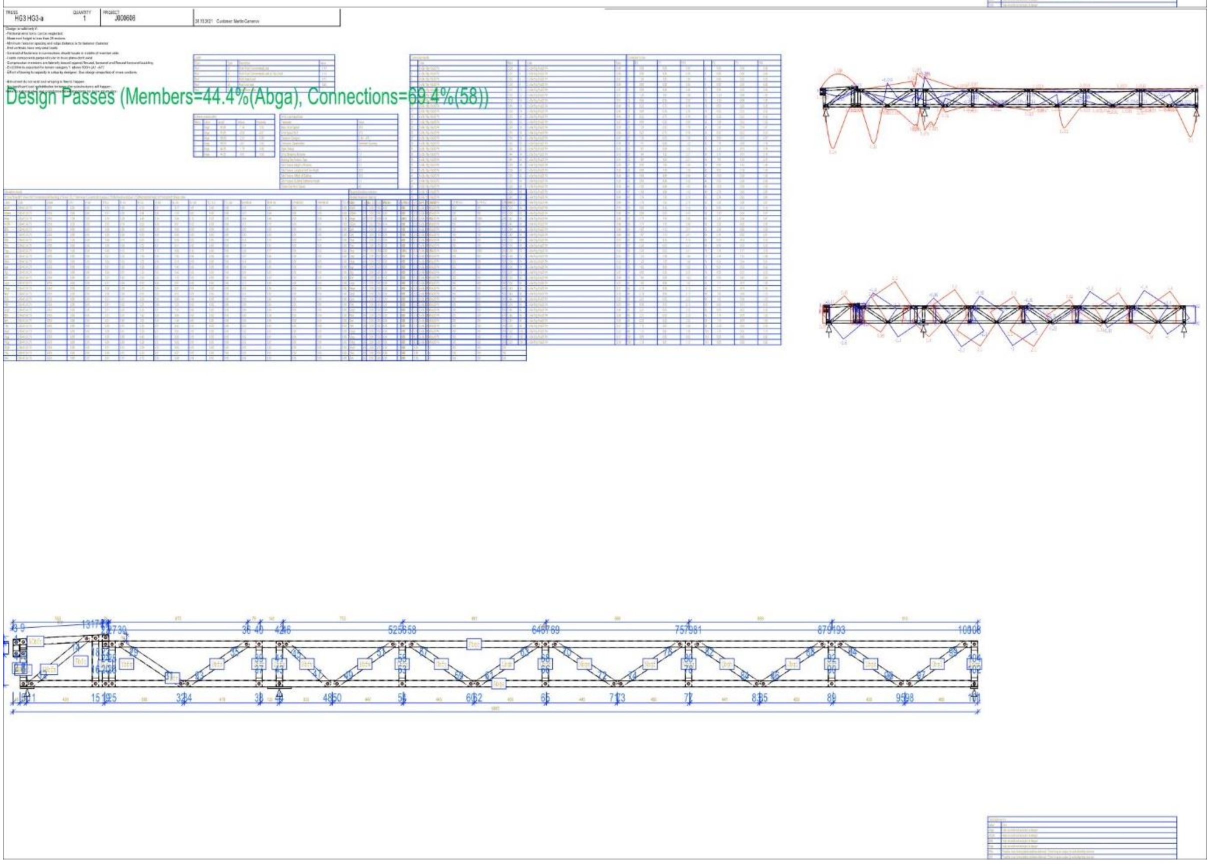
Design Fails (Members=263.1%(Ravd), Connections=95.6%(63))
STRENGTHENING SIMILAR TO H21



Design Passes (Members=44.4%(Aeg), Connections=67.7%(59))

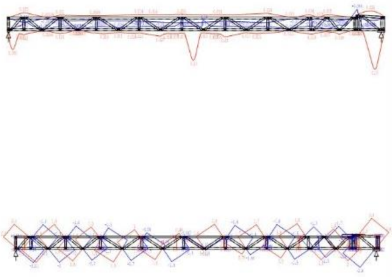


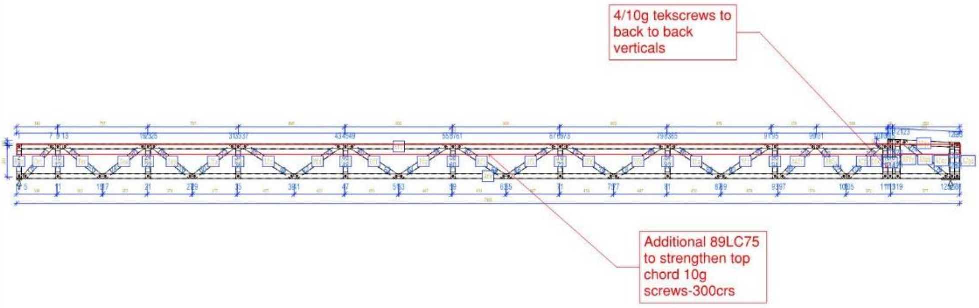
HG2 HG2-a
1
Design Passes (Members=93.2%(Fbhb), Connections=67.4%(7))
89LC75
Strengthening to top
and bottom chord



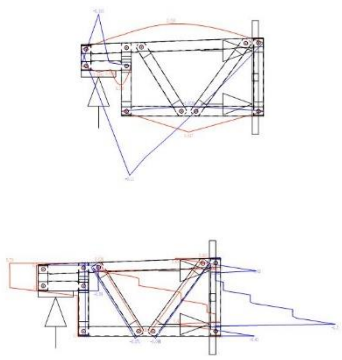
HG3 HG3-a
1
Design Passes (Members=44.4%(Abga), Connections=69.4%(58))

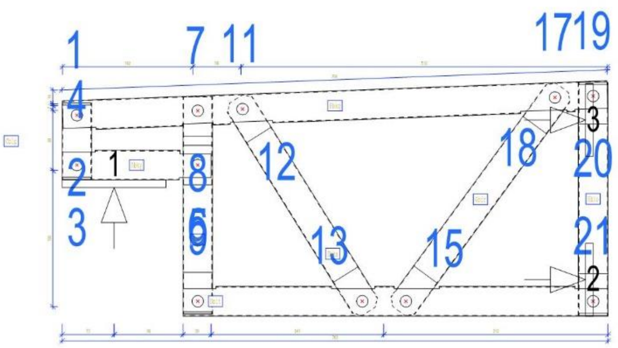
Design Passes (Members=97.4%(Efi), Connections=79.5%(109))

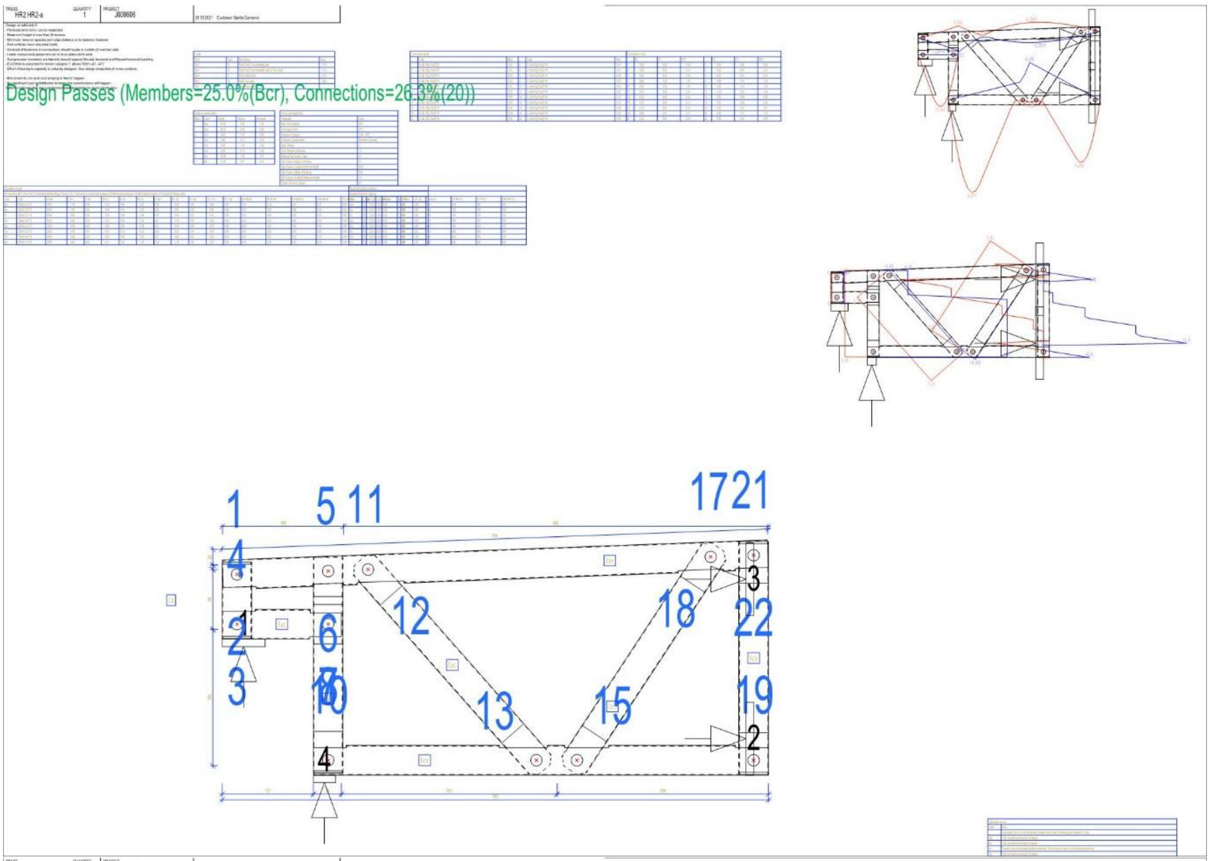


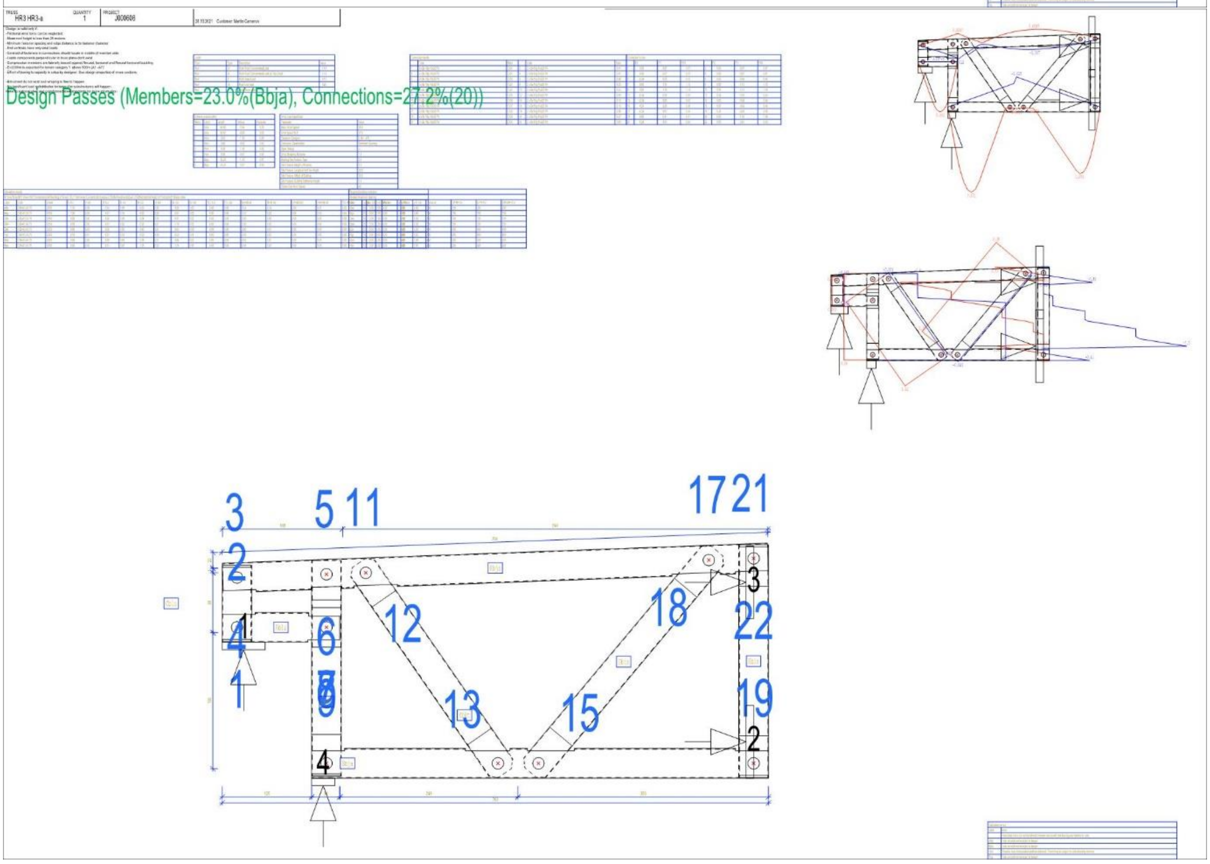


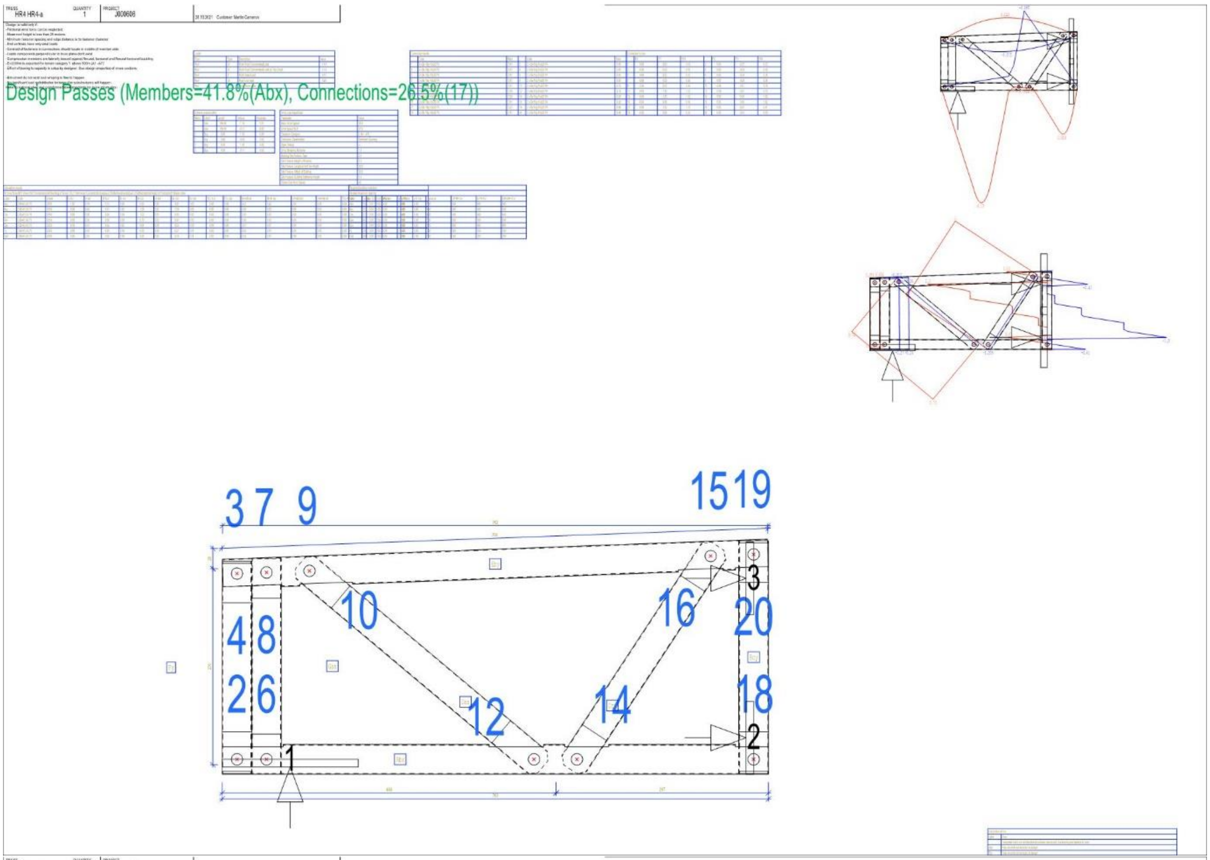
Design Passes (Members=24.9%(Fbkq), Connections=15.9%(7))



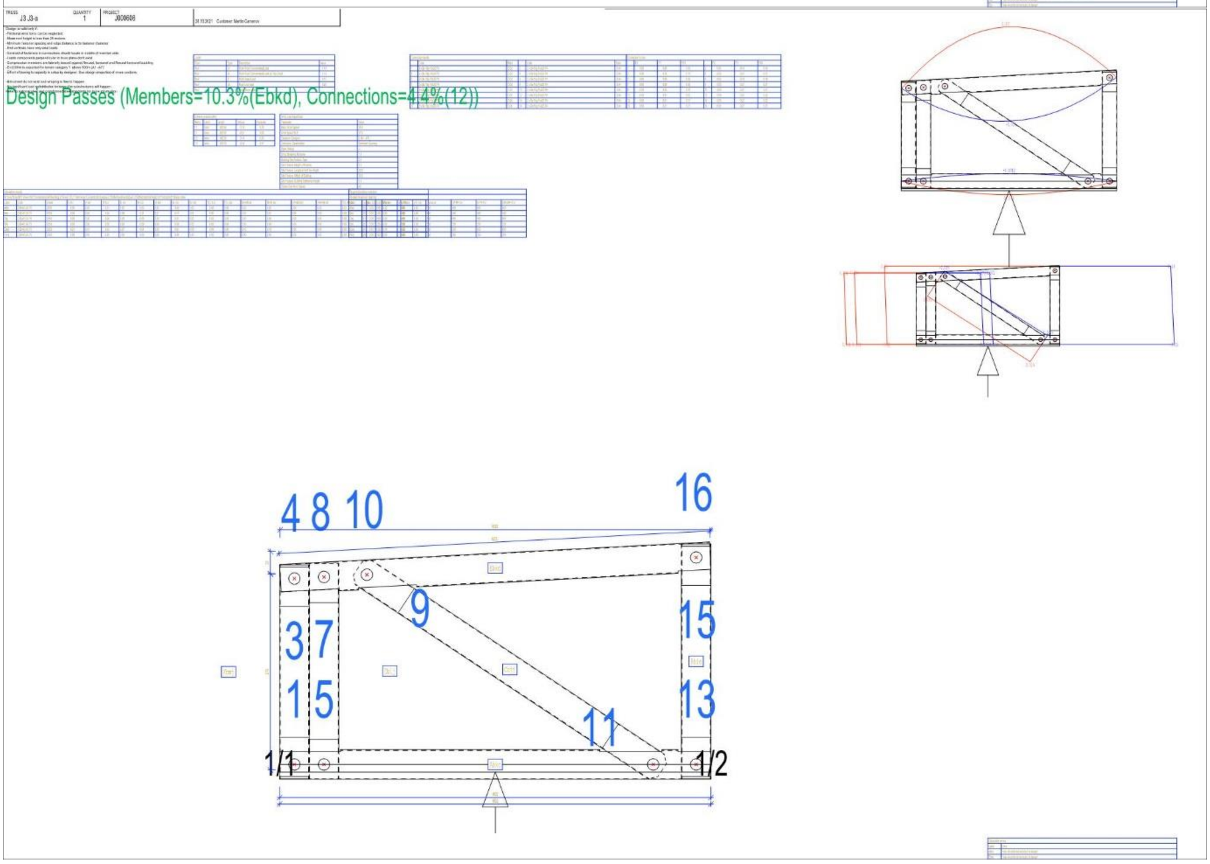



HR2 HR2-a
Design Passes (Members=25.0%(Bcr), Connections=26.3%(20))

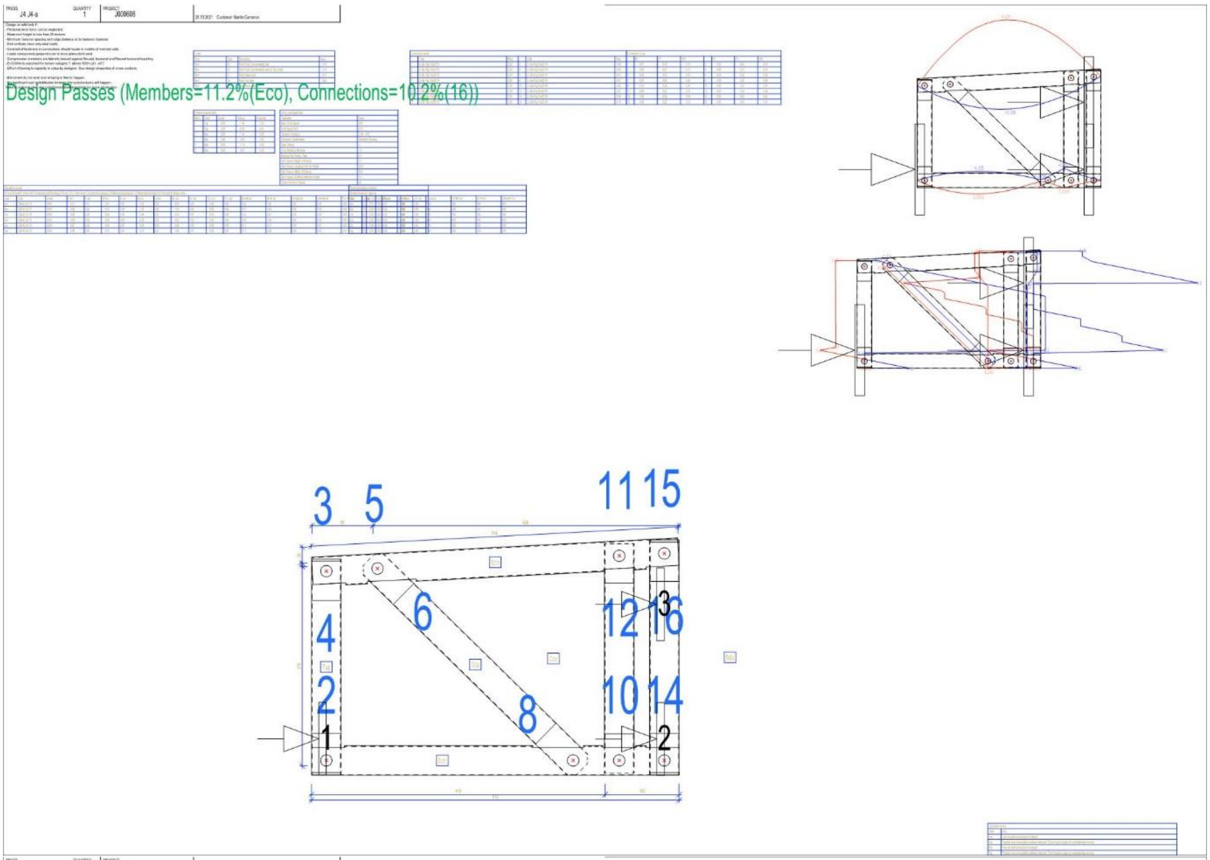

HR3 HR3-a
Design Passes (Members=23.0%(Bbja), Connections=27.2%(20))

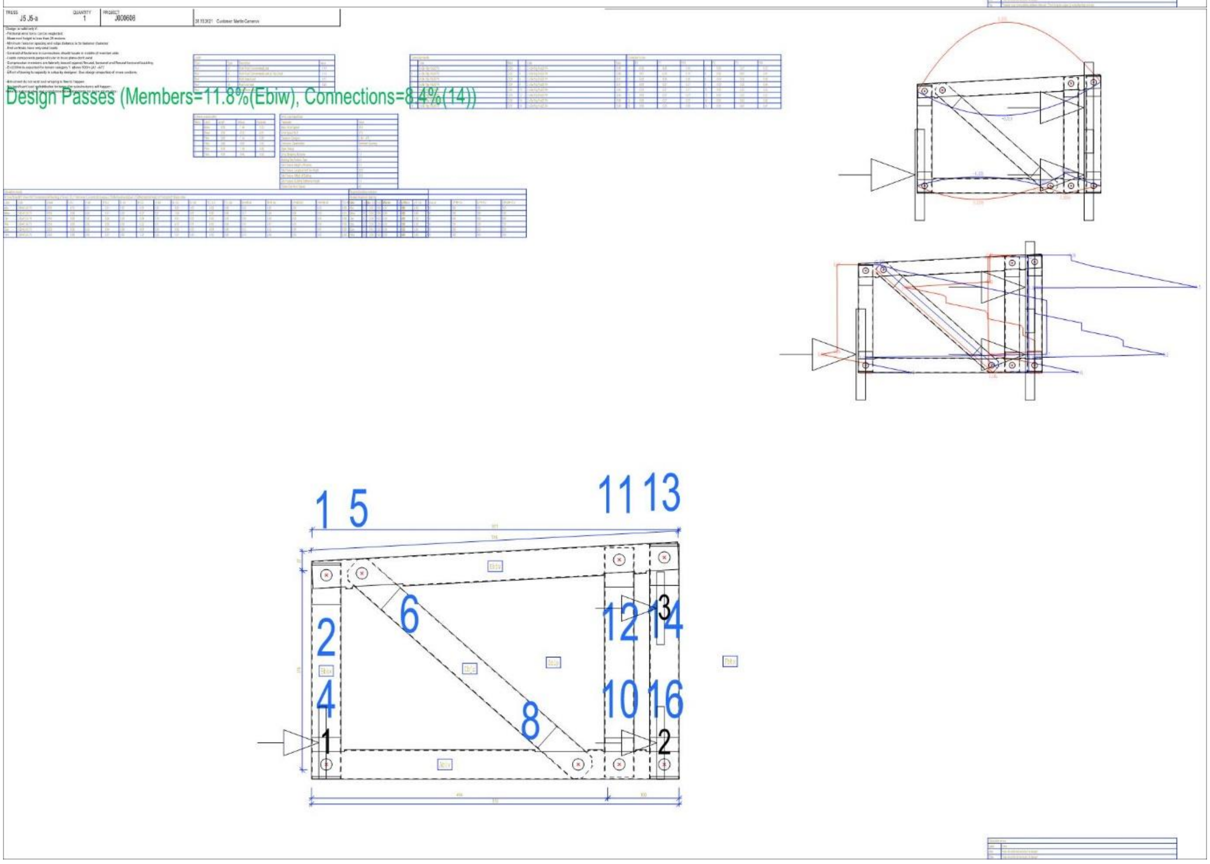


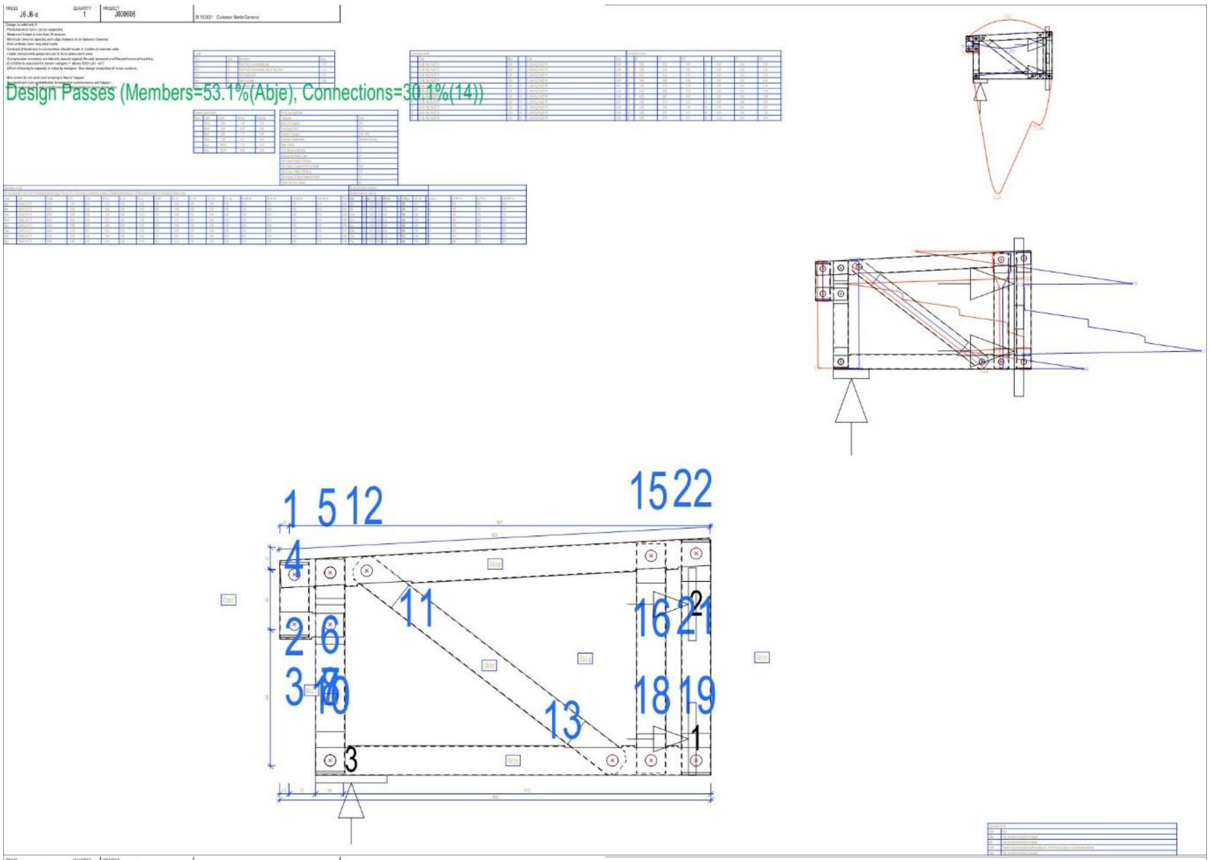
Design Passes (Members=41.8%(Abx), Connections=26.5%(17))



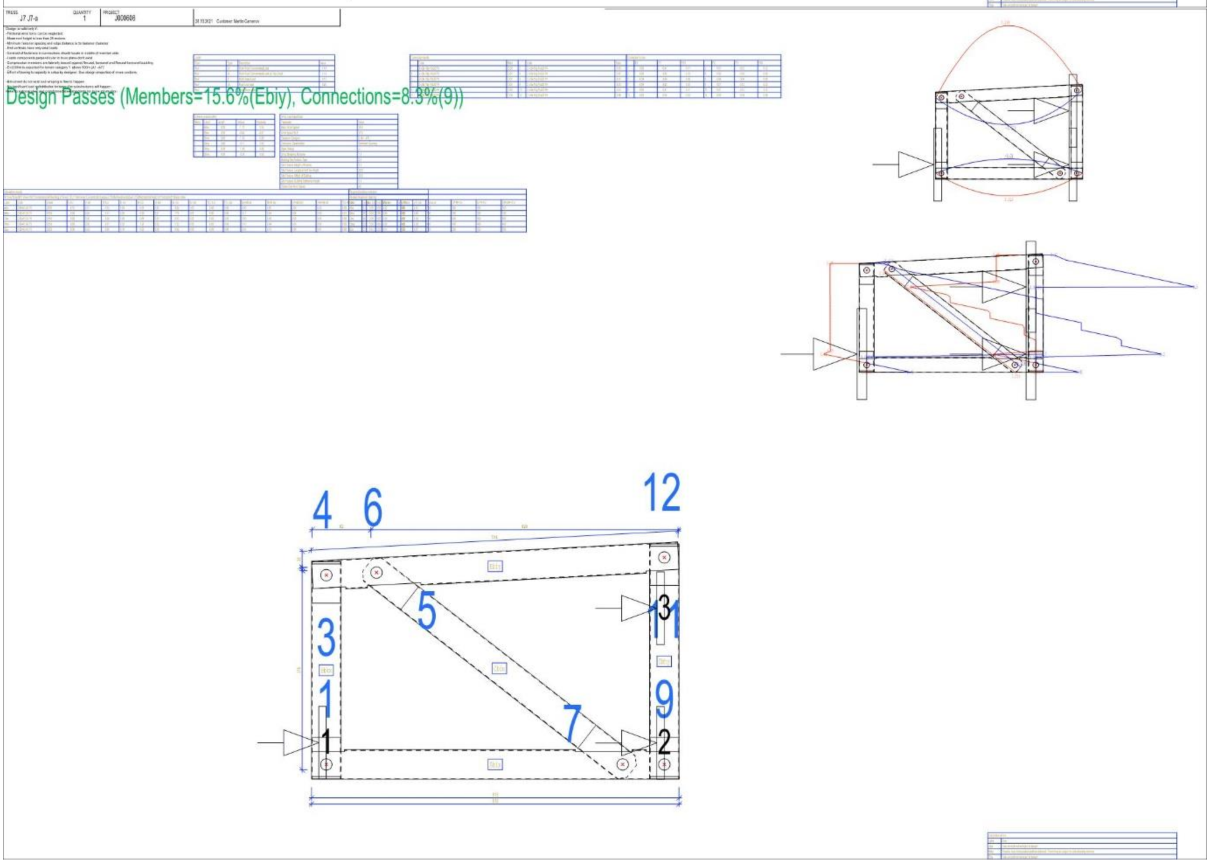
Design Passes (Members=10.3%(Ebkd), Connections=4.4%(12))


Design Passes (Members=11.2%(Eco), Connections=10.2%(16))
3 5
11 15
4
2
1
6
8
12
16
3
10
14
2

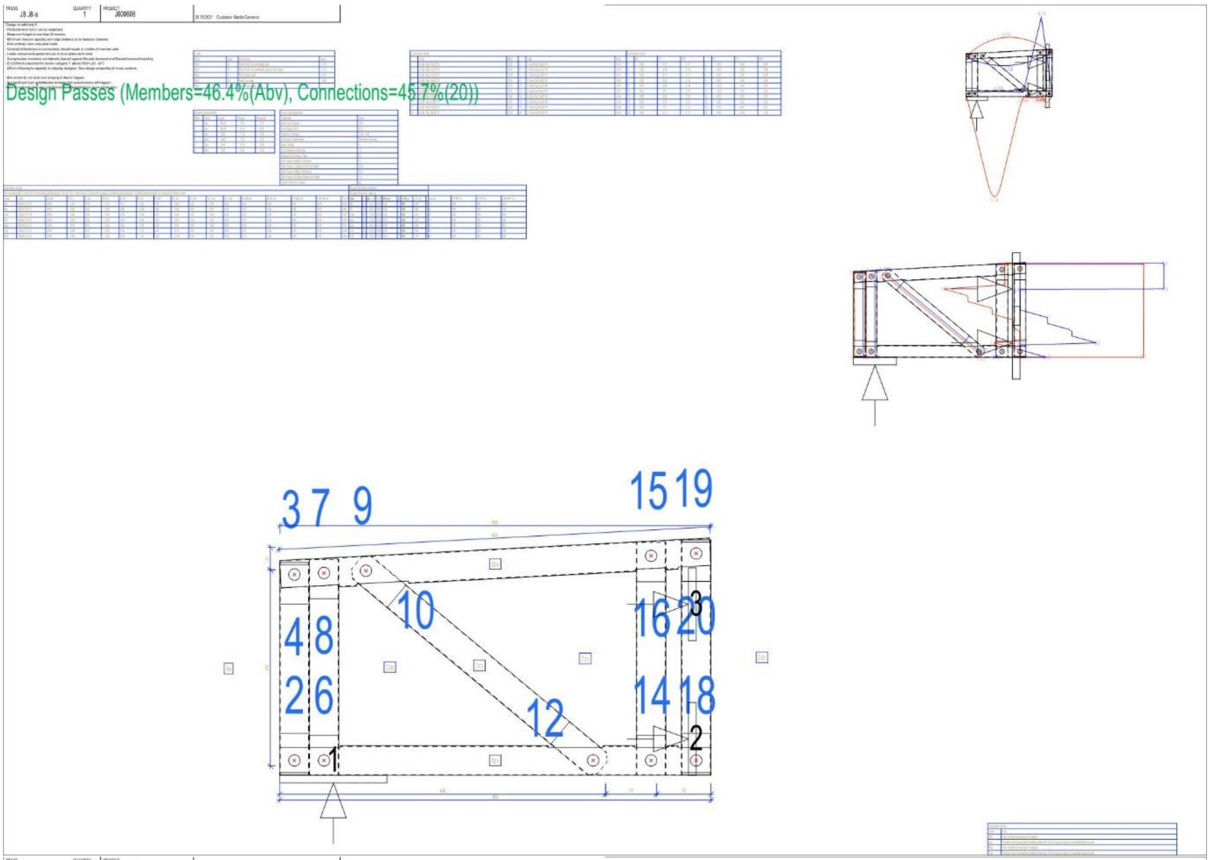

Design Passes (Members=11.8%(Ebiw), Connections=8.4%(14))
1 5
11 13
2
4
1
6
8
12
14
3
10
16
2

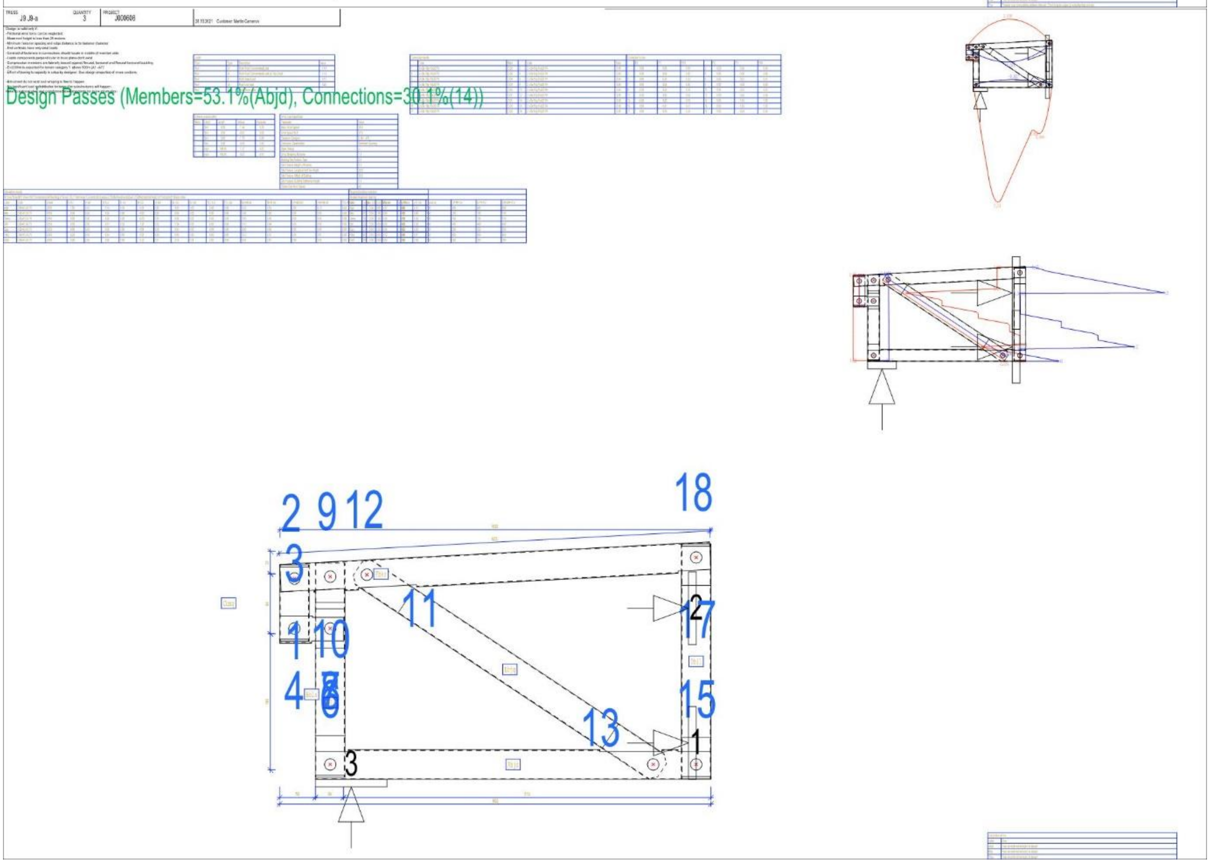


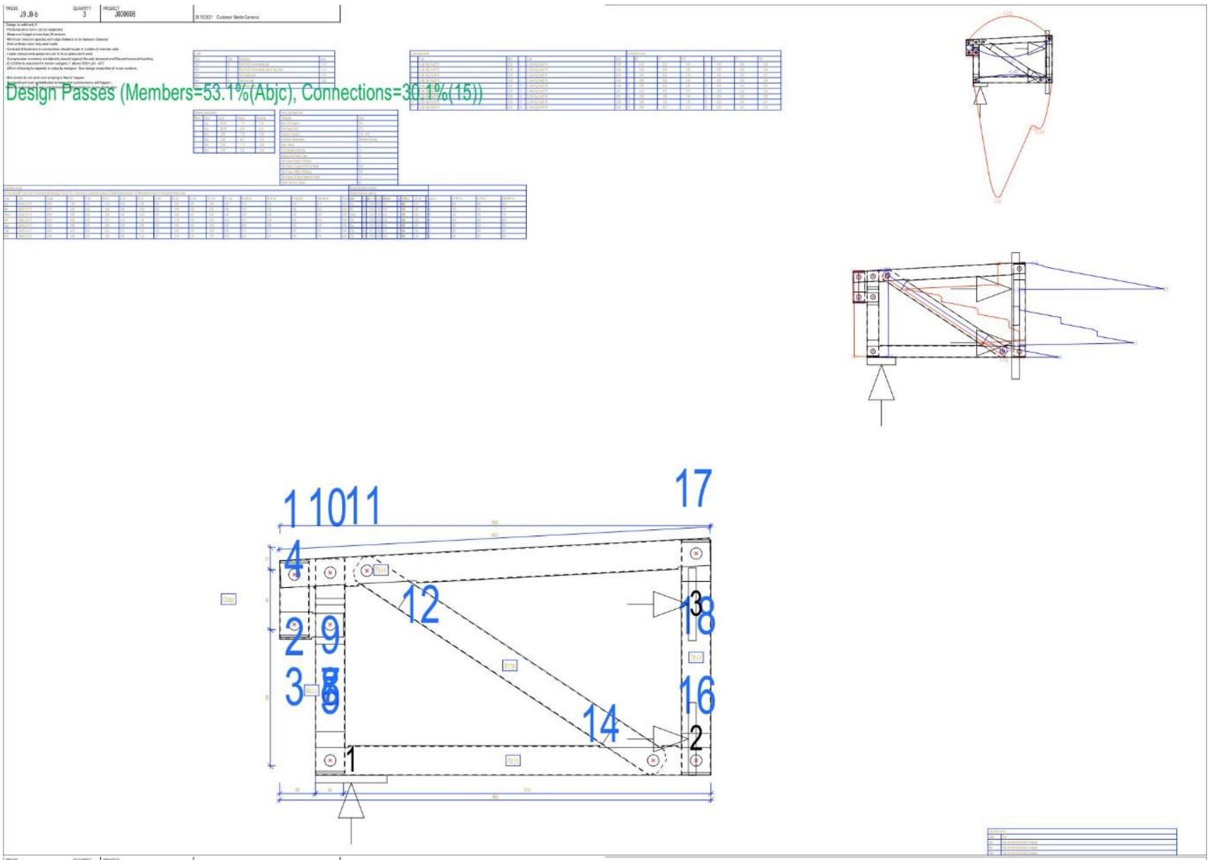
Design Passes (Members=53.1%(Abje), Connections=30.1%(14))



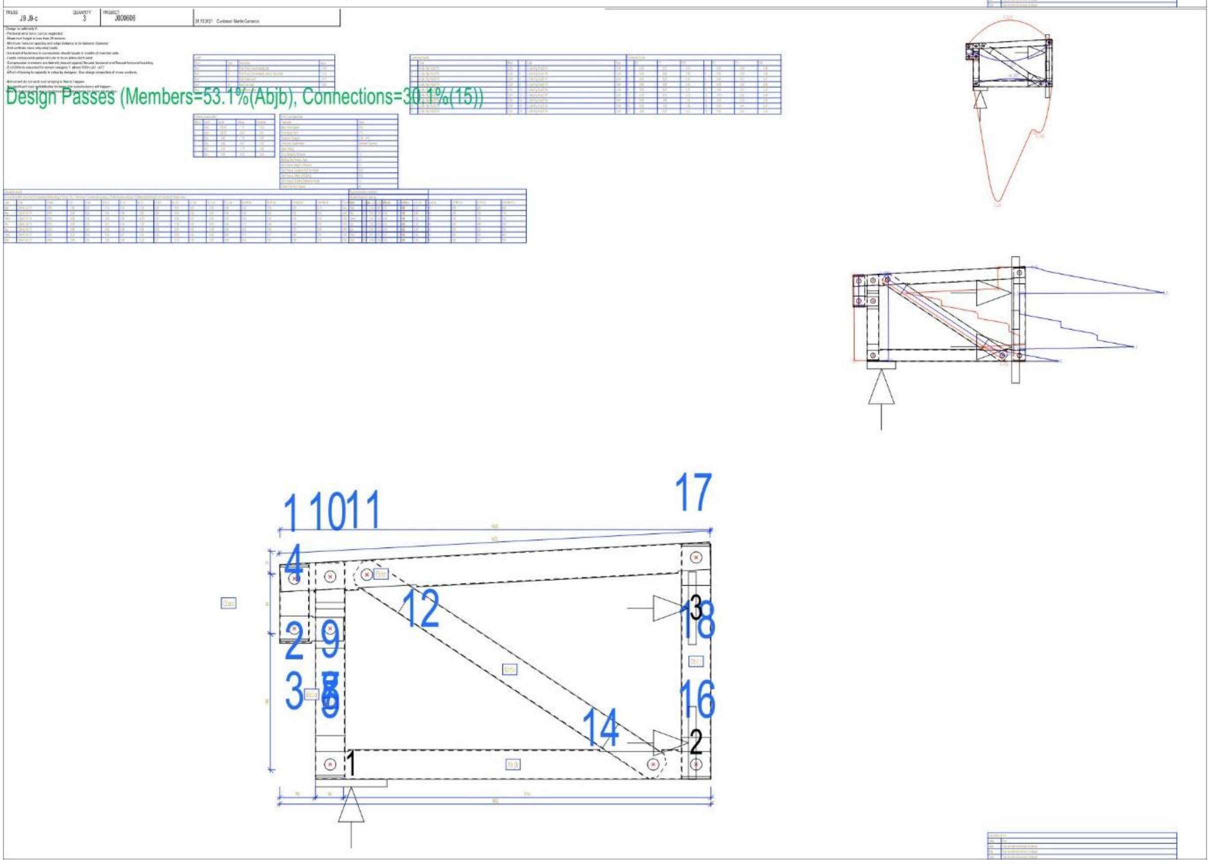
Design Passes (Members=15.6%(Ebiy), Connections=8.3%(9))


Design Passes (Members=46.4%(Abv), Connections=45.7%(20))

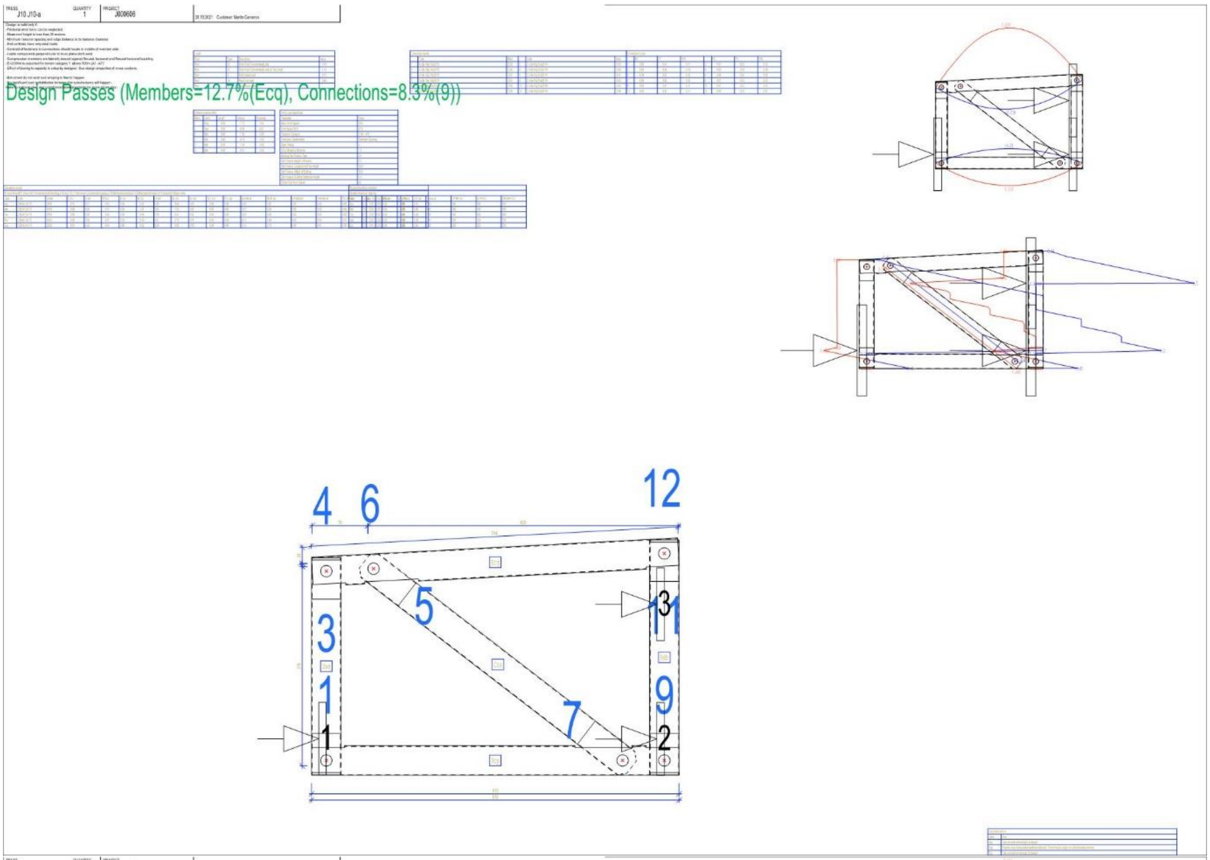

Design Passes (Members=53.1%(Abjd), Connections=30.1%(14))

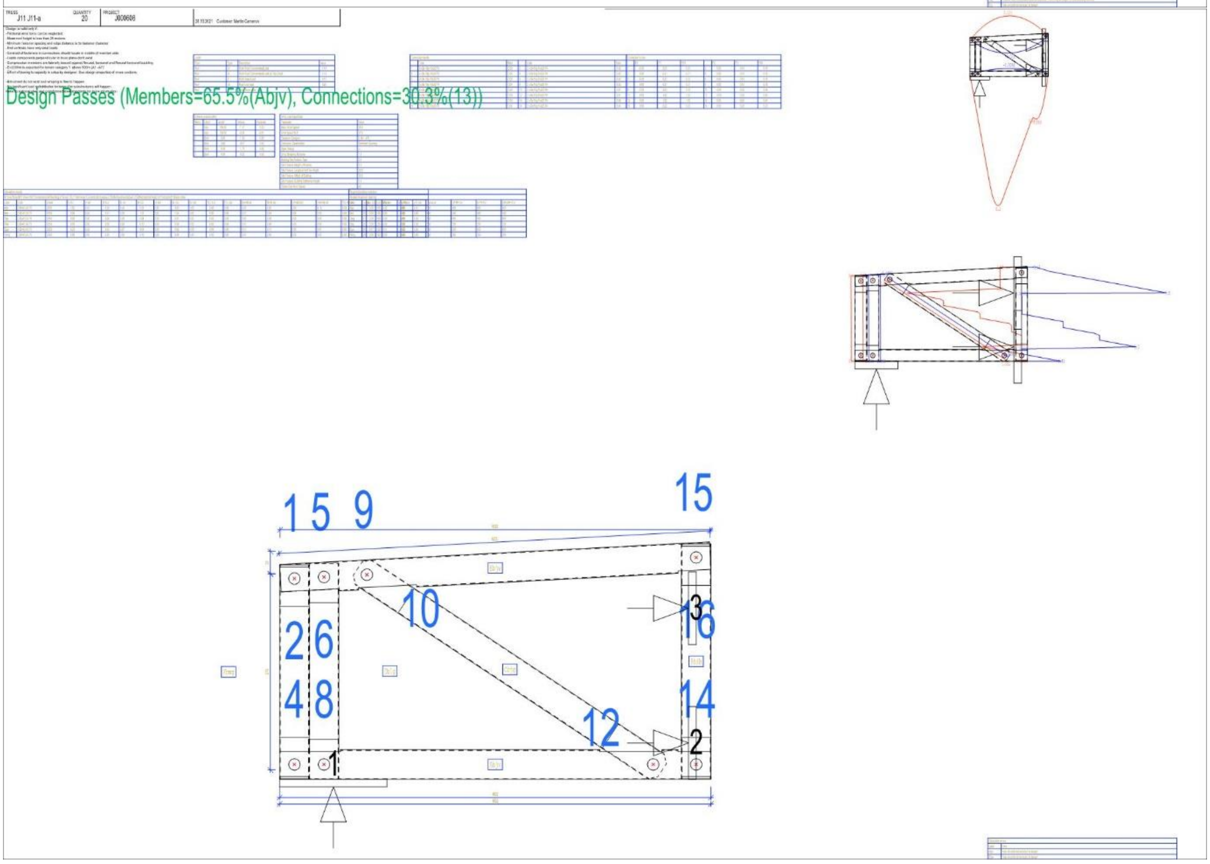


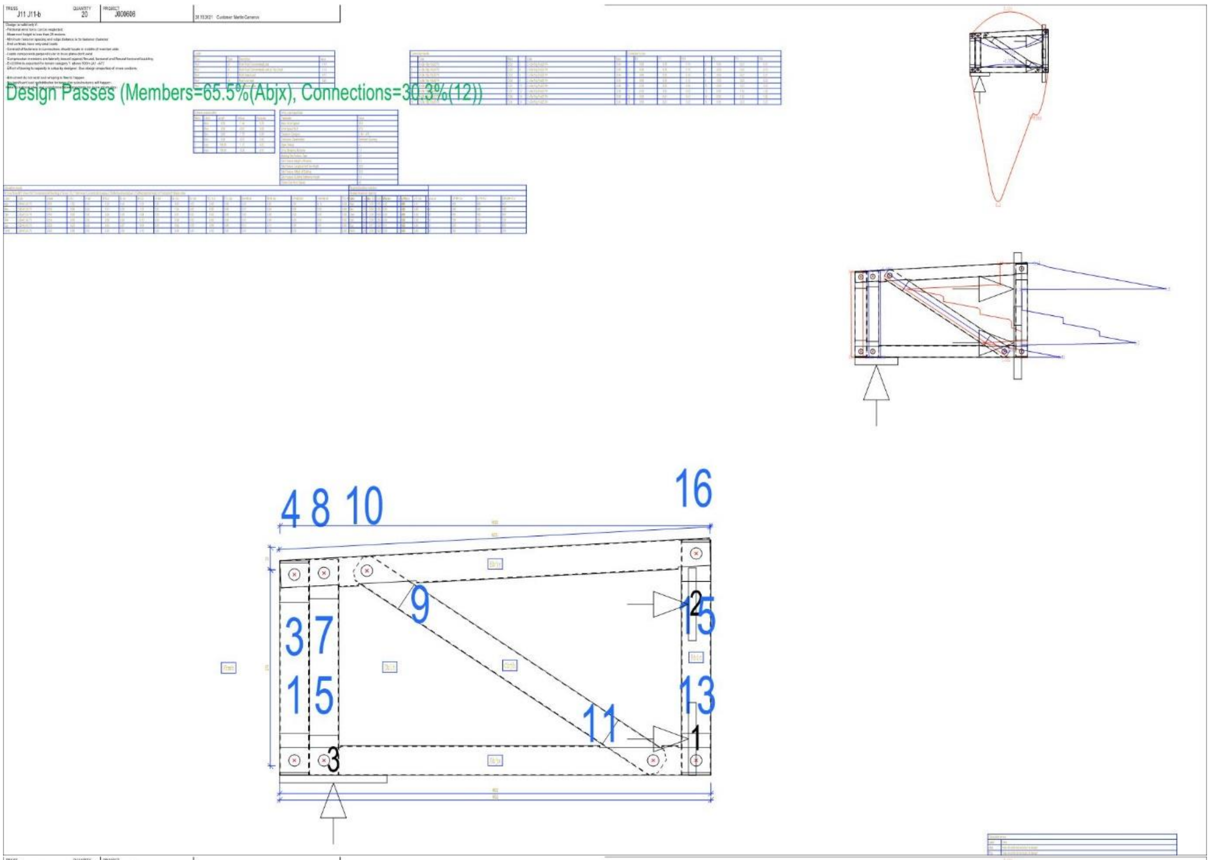
Design Passes (Members=53.1%(Abjc), Connections=30.1%(15))
1 1011
17
4
12
3
18
2
9
3
5
16
14
2
1

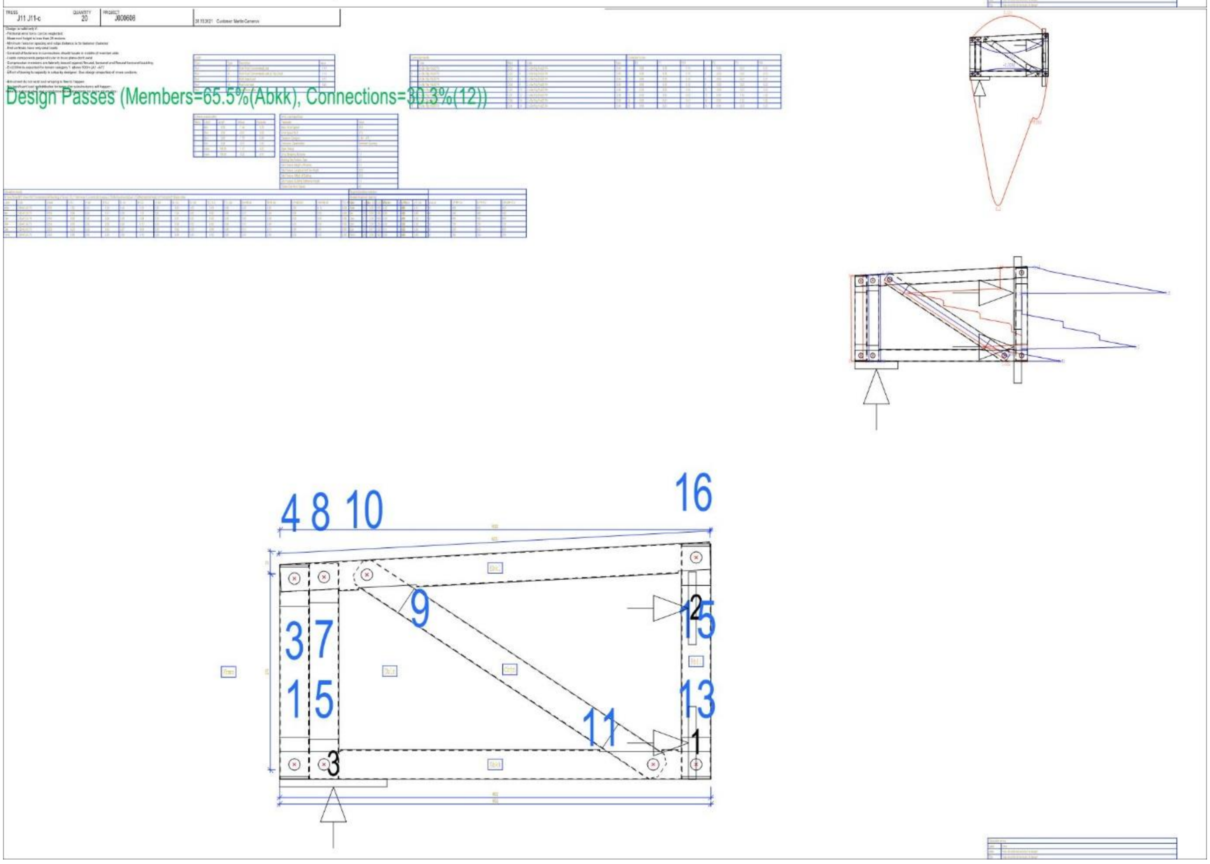


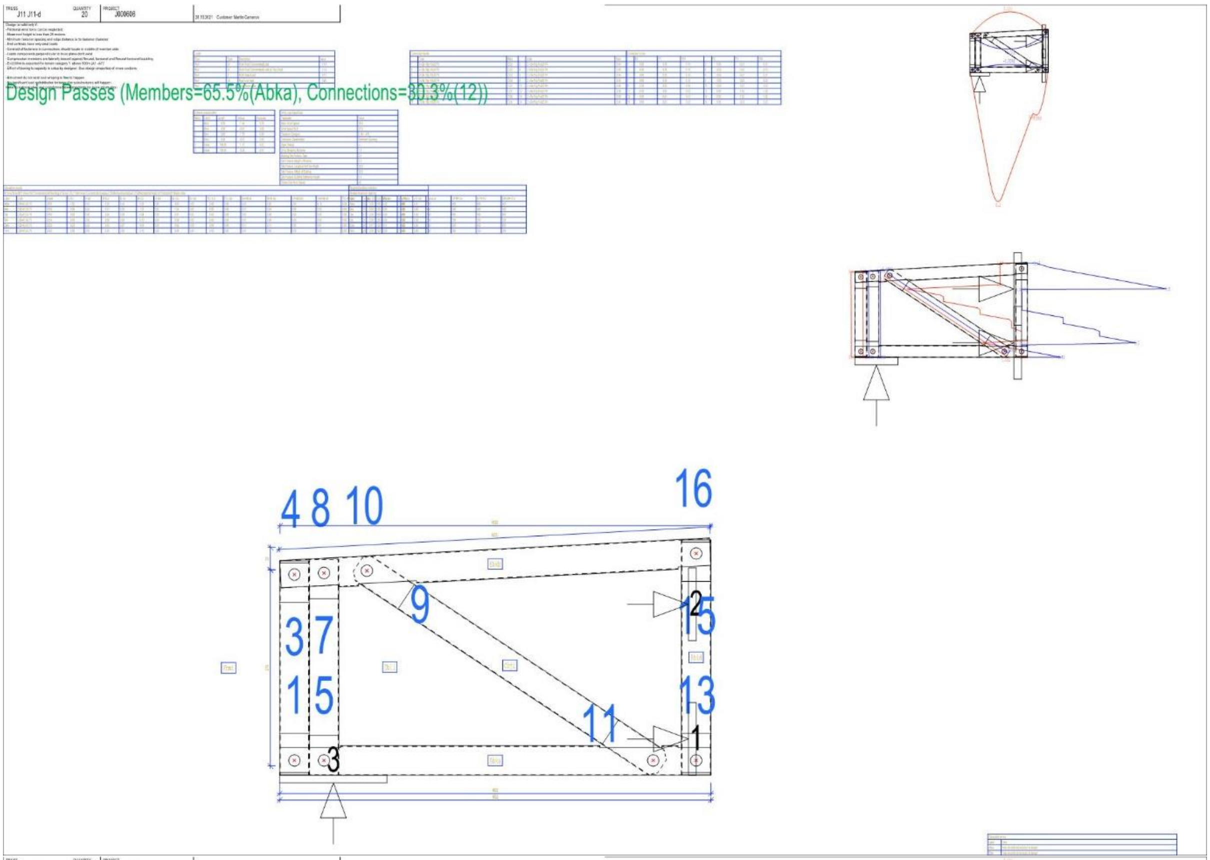
Design Passes (Members=53.1%(Abjb), Connections=30.1%(15))
1 1011
17
4
12
3
18
2
9
3
5
16
14
2
1

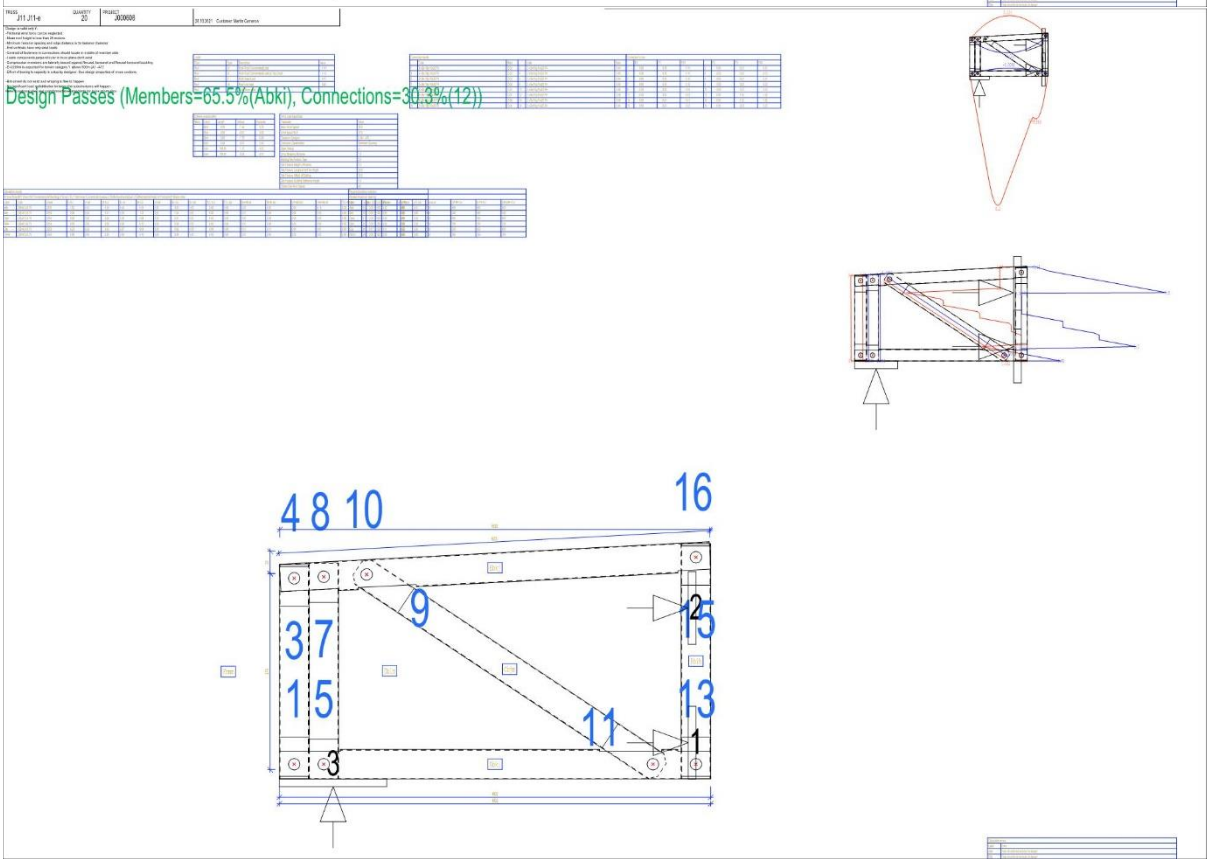

TRUSS
J10 J10-a
QUANTITY
1
PROJECT
J000606
Customer Martin Cameron
Design Passes (Members=12.7%(Ecq), Connections=8.3%(9))
4
6
12
5
3
11
1
9
7
1
2
3

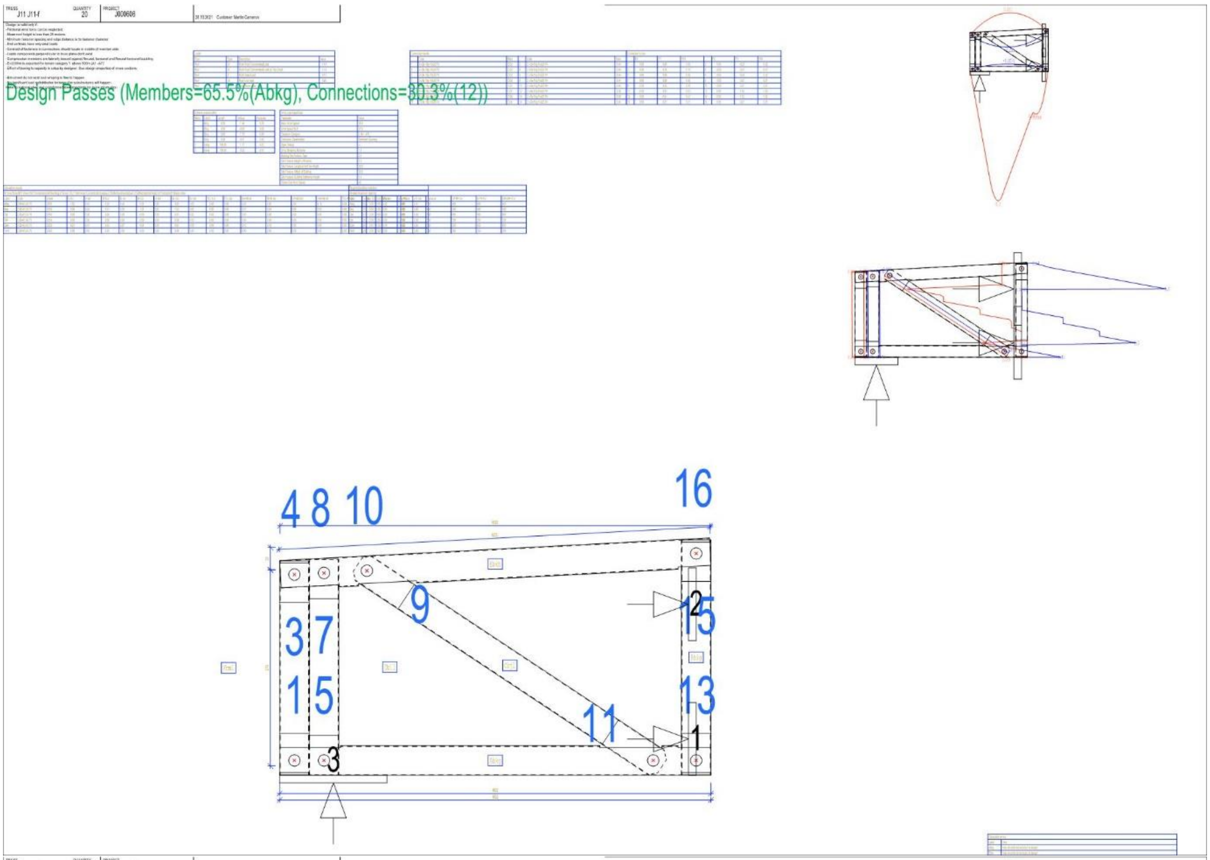

TRUSS
J11 J11-a
QUANTITY
20
PROJECT
J000606
Customer Martin Cameron
Design Passes (Members=65.5%(Abjv), Connections=30.3%(13))
1
5
9
15
10
2
6
16
4
8
14
12
1
2
3


Design Passes (Members=65.5%(Abjx), Connections=30.3%(12))
4 8 10
16
9
2
15
3 7
1 5
13
11
1
3

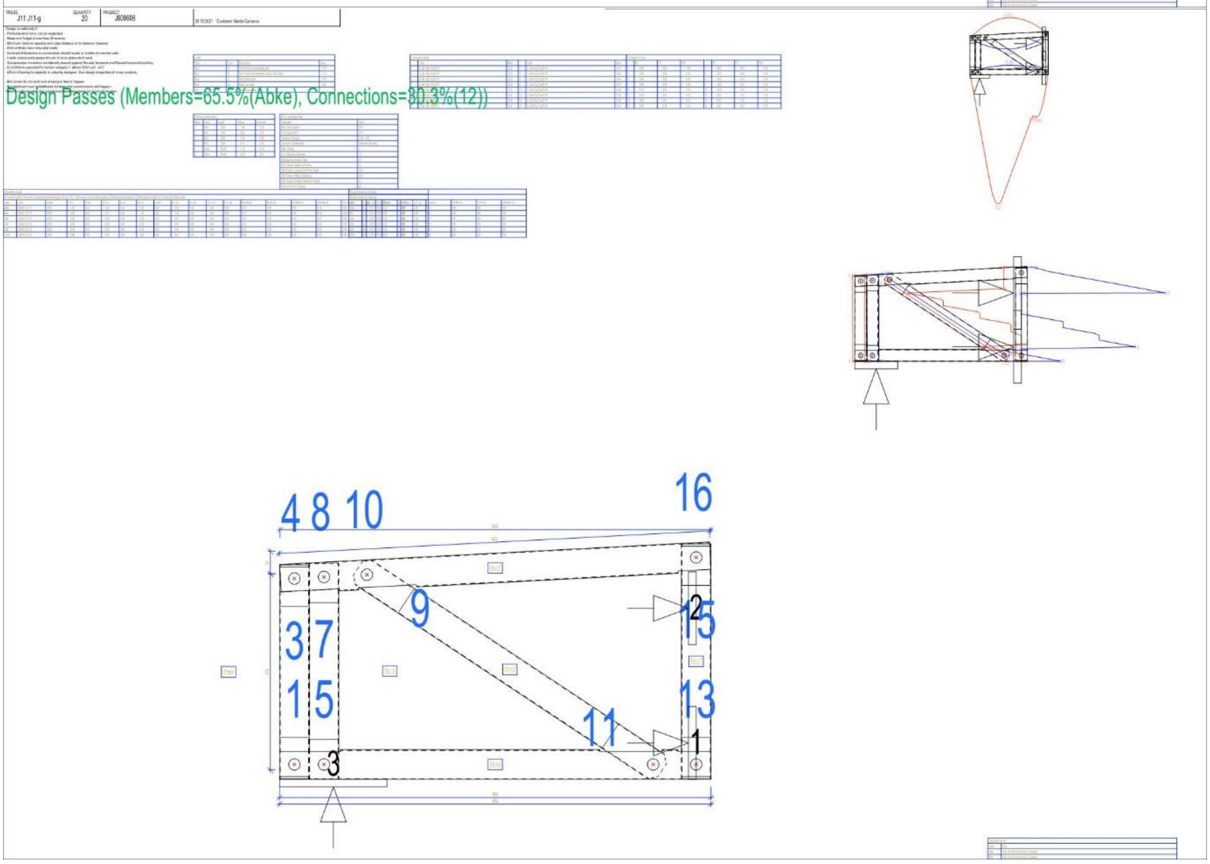

Design Passes (Members=65.5%(Abkk), Connections=30.3%(12))
4 8 10
16
9
2
15
3 7
1 5
13
11
1
3


Design Passes (Members=65.5%(Abka), Connections=30.3%(12))
4 8 10
16
9
2
15
3 7
1 5
13
11
1
3

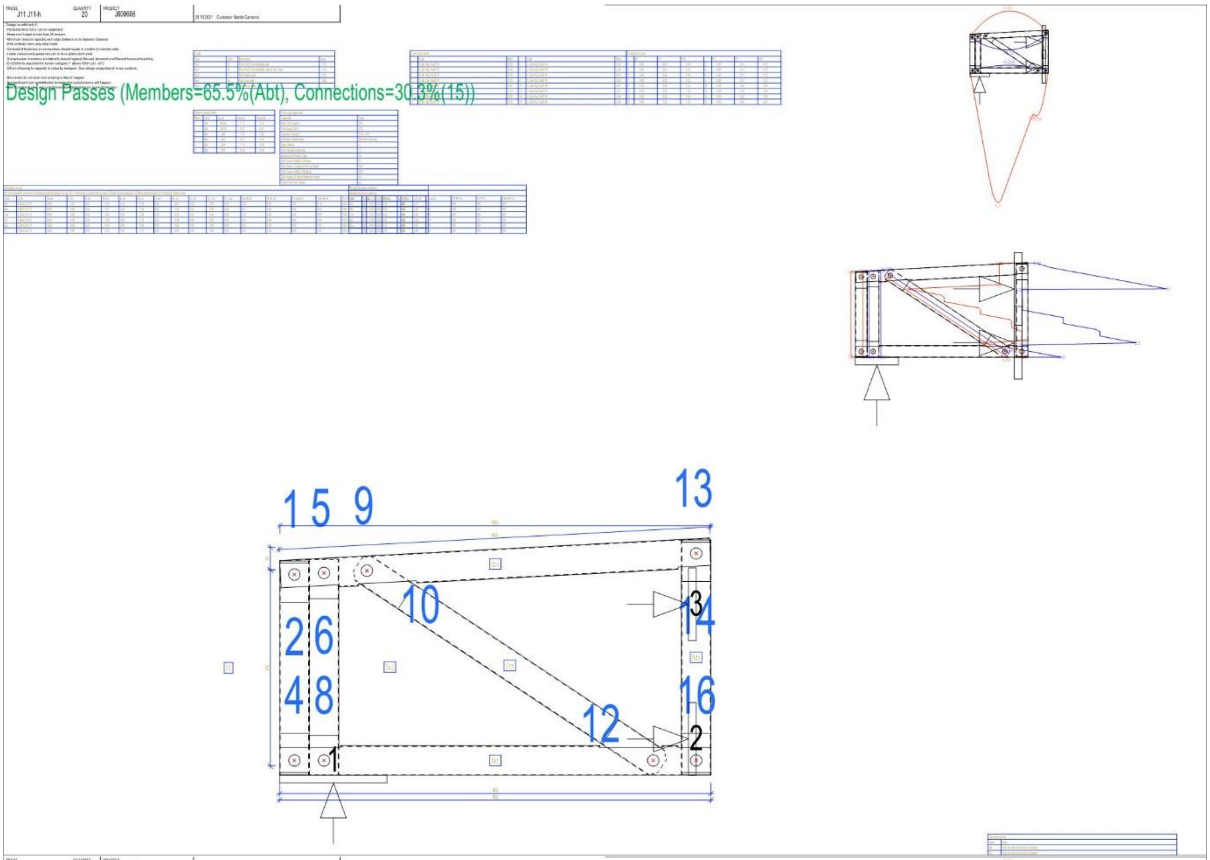

Design Passes (Members=65.5%(Abki), Connections=30.3%(12))
4 8 10
16
9
2
15
3 7
1 5
13
11
1
3

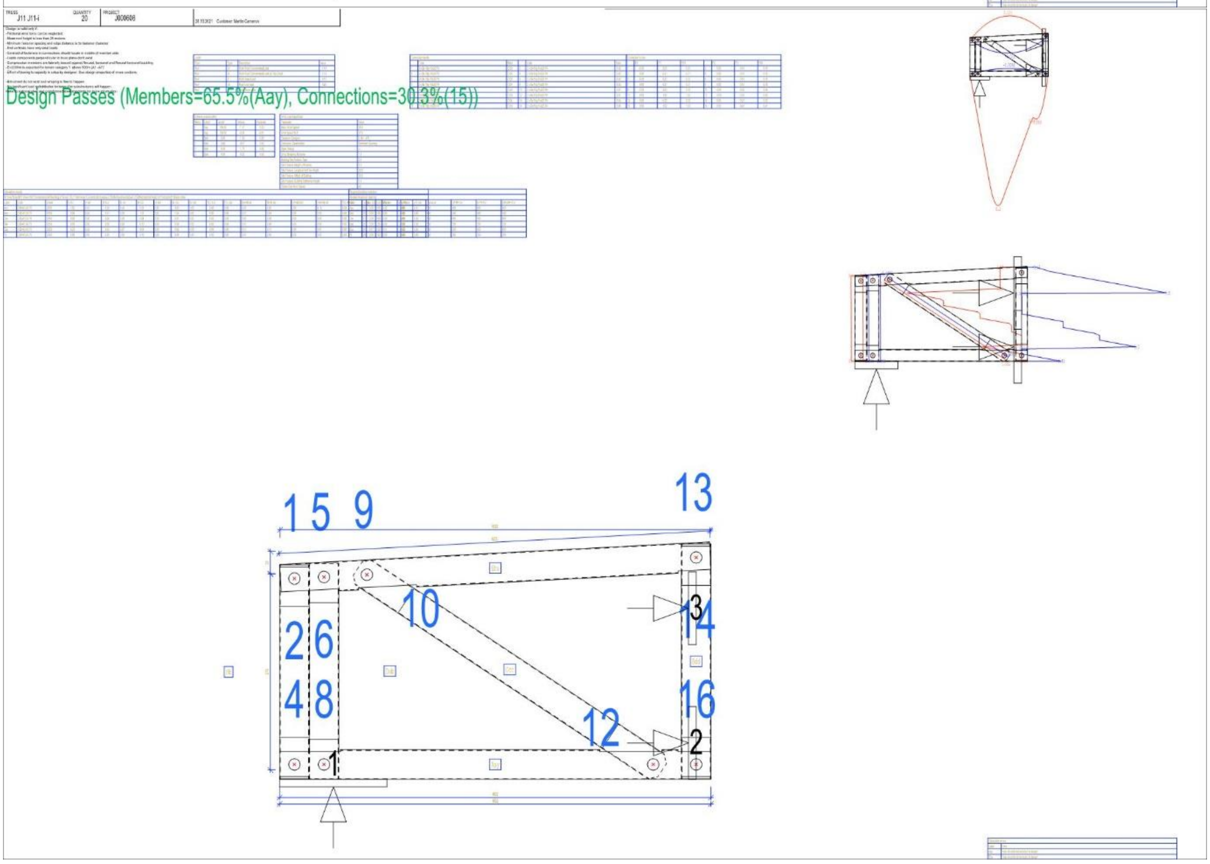


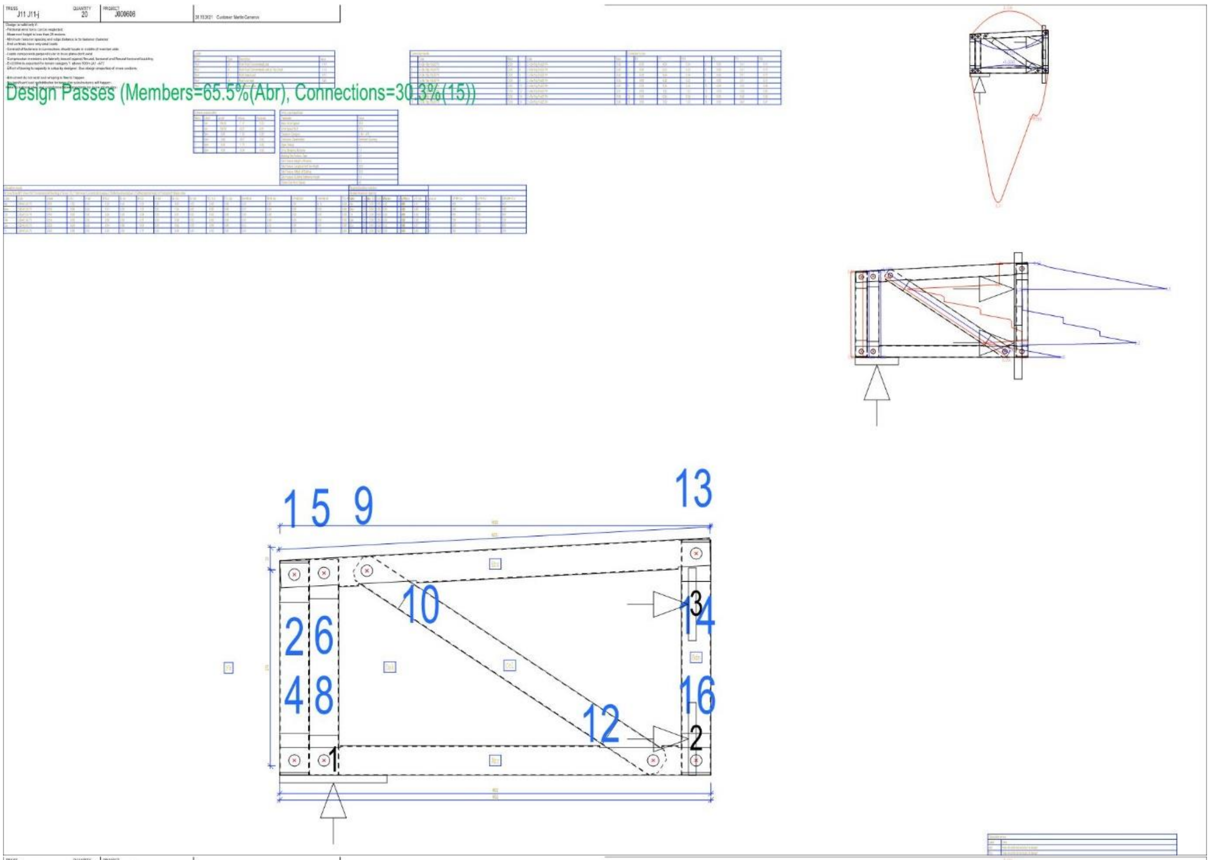
Design Passes (Members=65.5%(Abkg), Connections=80.3%(12))
4 8 10
16
9
2
15
3 7
1 5
13
11
1
3



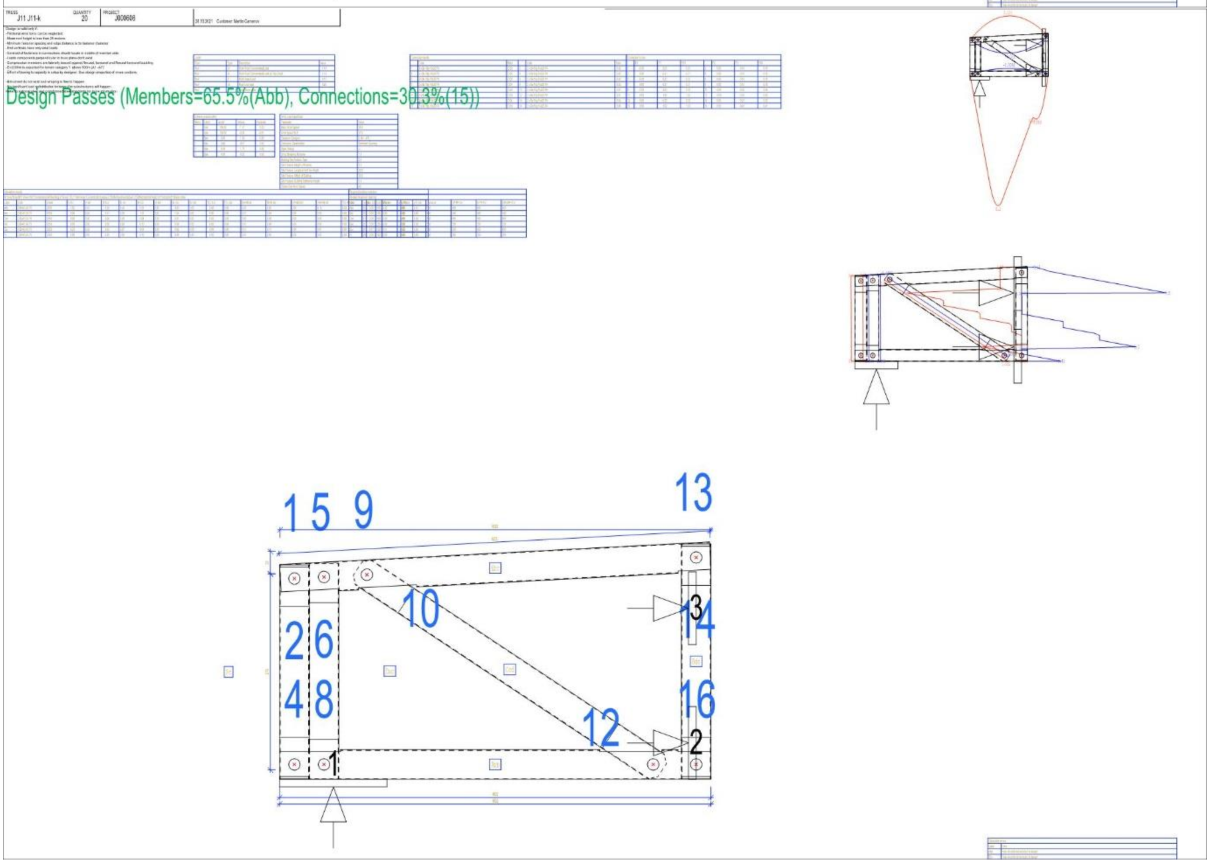
Design Passes (Members=65.5%(Abke), Connections=80.3%(12))
4 8 10
16
9
2
15
3 7
1 5
13
11
1
3


TRUSS
J11 J11-h
QUANTITY
20
PROJECT
Customer Martin Cameron
Design Passes (Members=65.5%(Abt), Connections=30.3%(15))
1 5 9
13
10
3
14
2 6
4 8
16
12
2
1

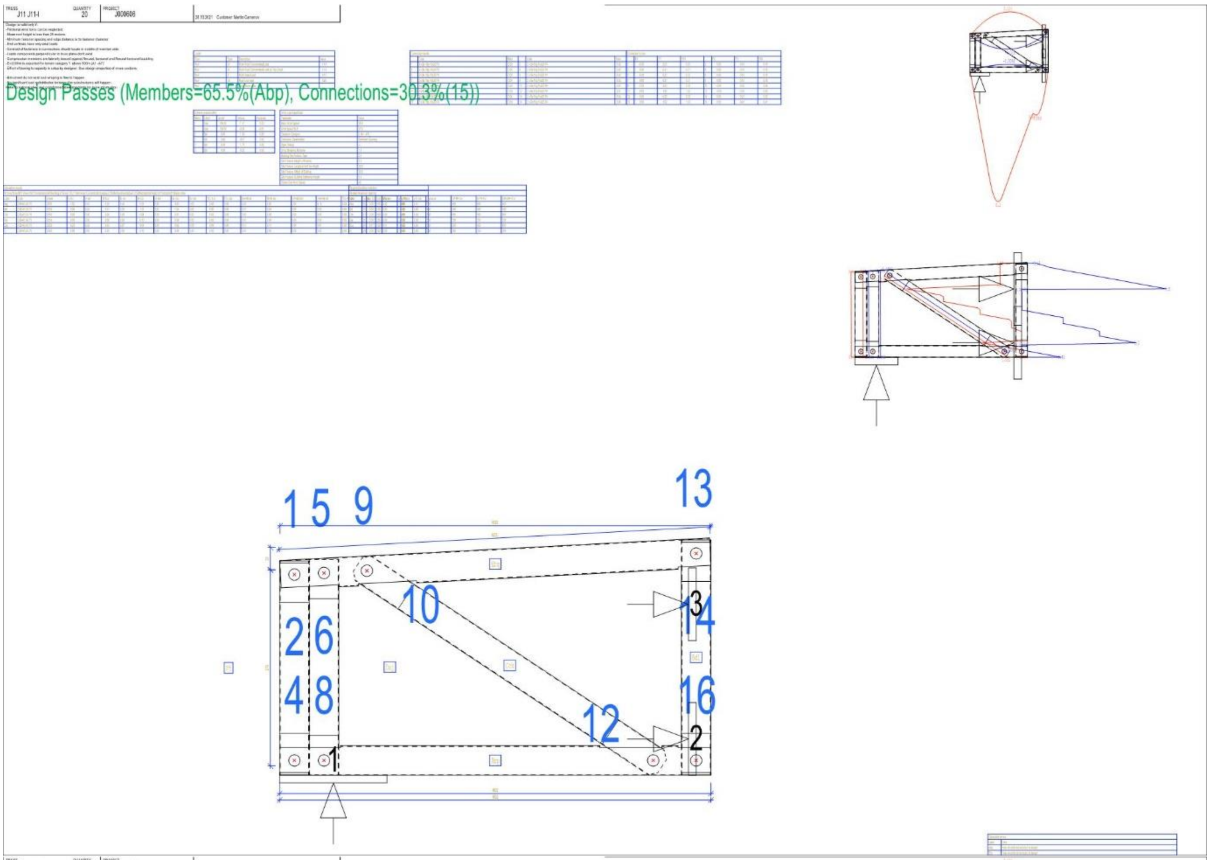

TRUSS
QUANTITY
20
PROJECT
Customer Martin Cameron
Design Passes (Members=65.5%(Aay), Connections=30.3%(15))
1 5 9
13
10
3
14
2 6
4 8
16
12
2
1



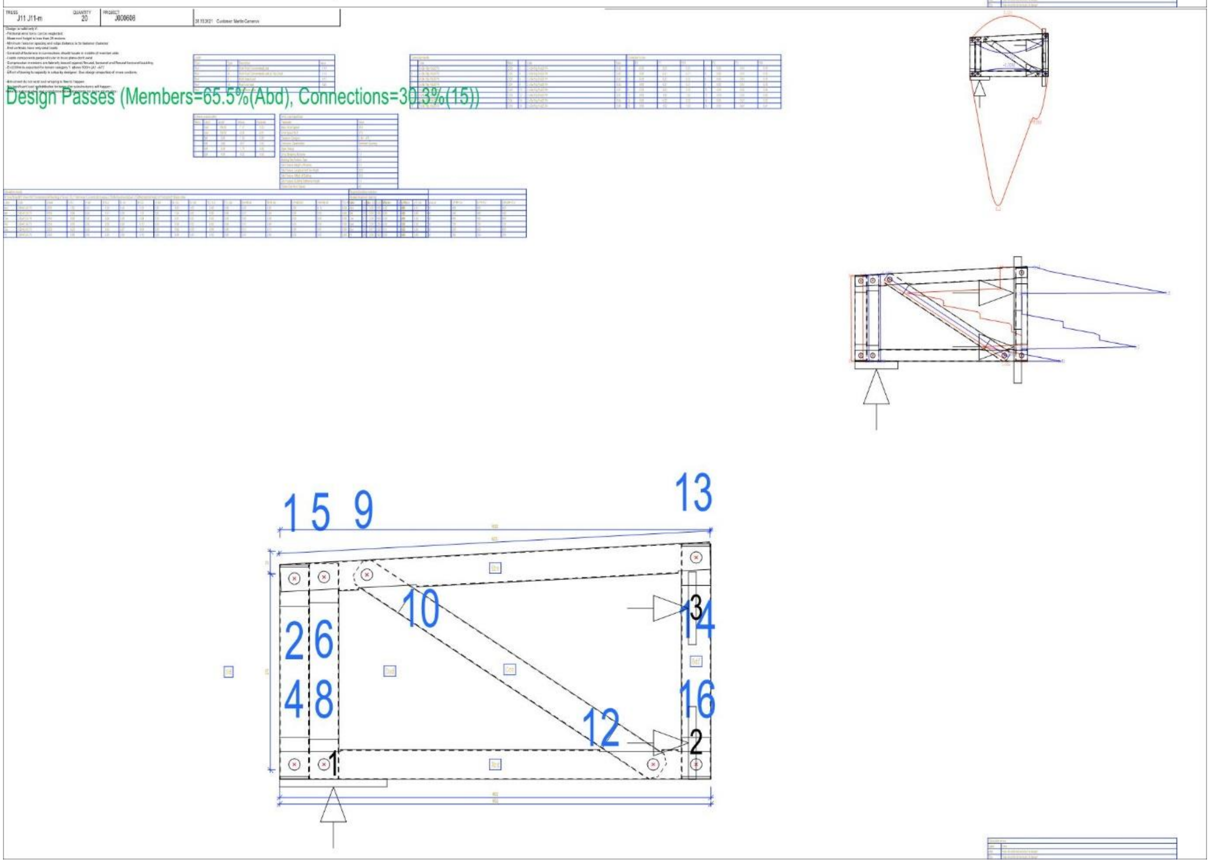
Design Passes (Members=65.5%(Abr), Connections=30.3%(15))
1 5 9
13
10
2 6
4 8
14
16
12
1
2
3



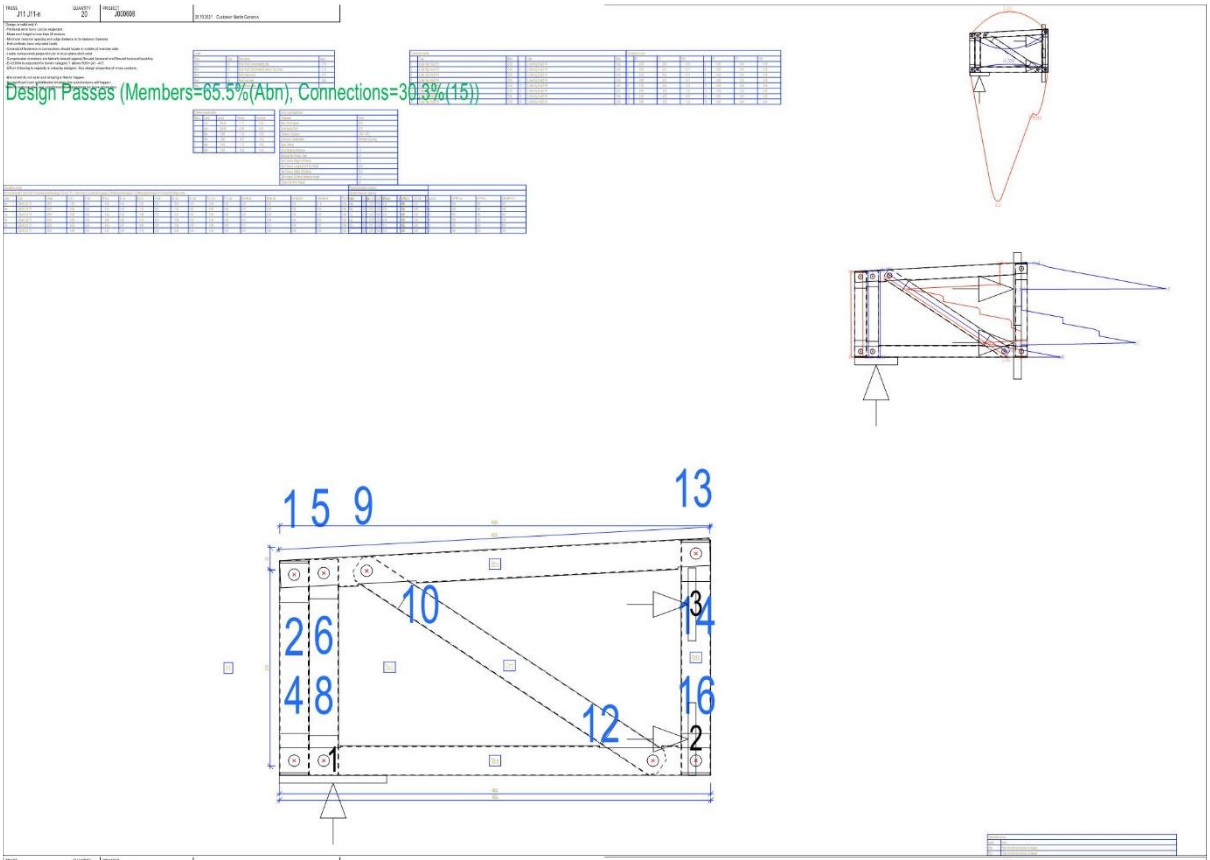
Design Passes (Members=65.5%(Abb), Connections=30.3%(15))
1 5 9
13
10
2 6
4 8
14
16
12
1
2
3



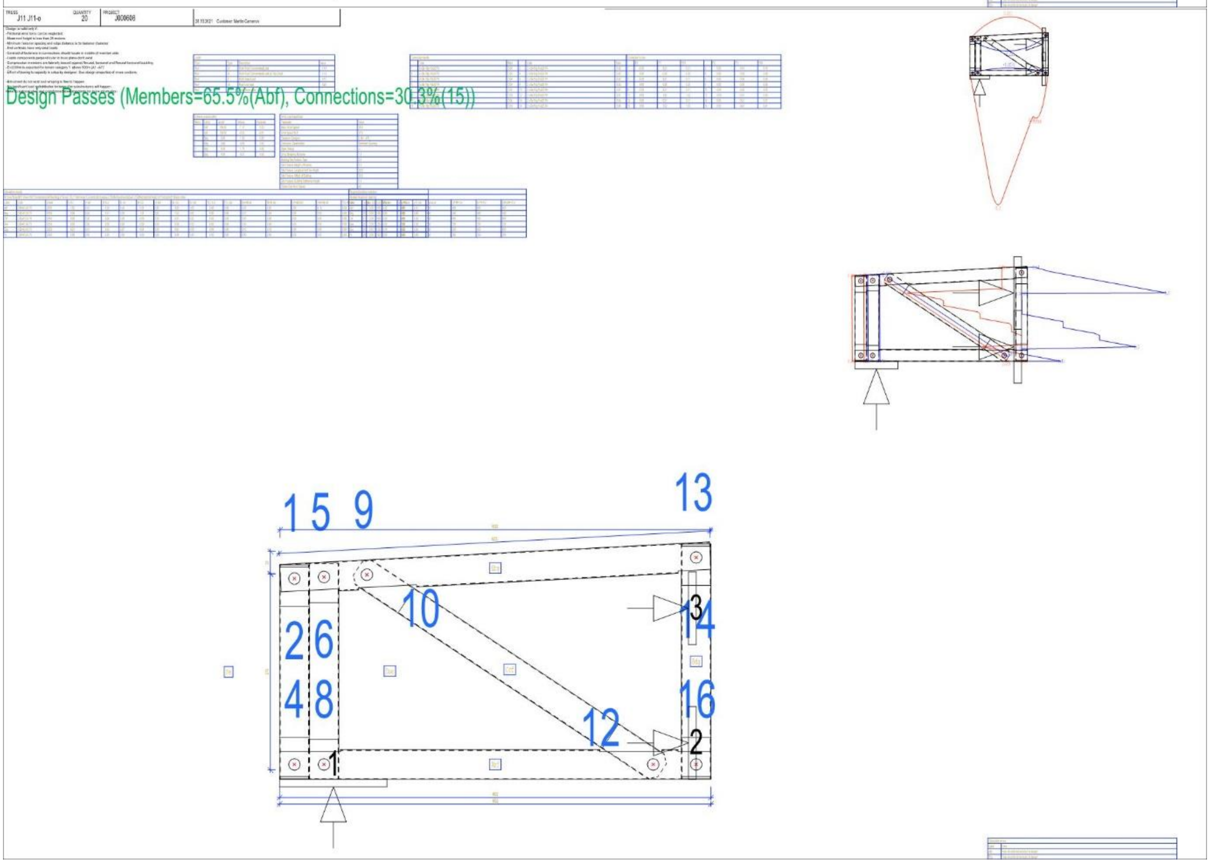
TRUSS
J11 J11-l
QUANTITY
20
PROJECT
J000608
Design Passes (Members=65.5%(Abp), Connections=30.3%(15))
1 5 9
13
2 6
4 8
10
12
14
16
3
2
1



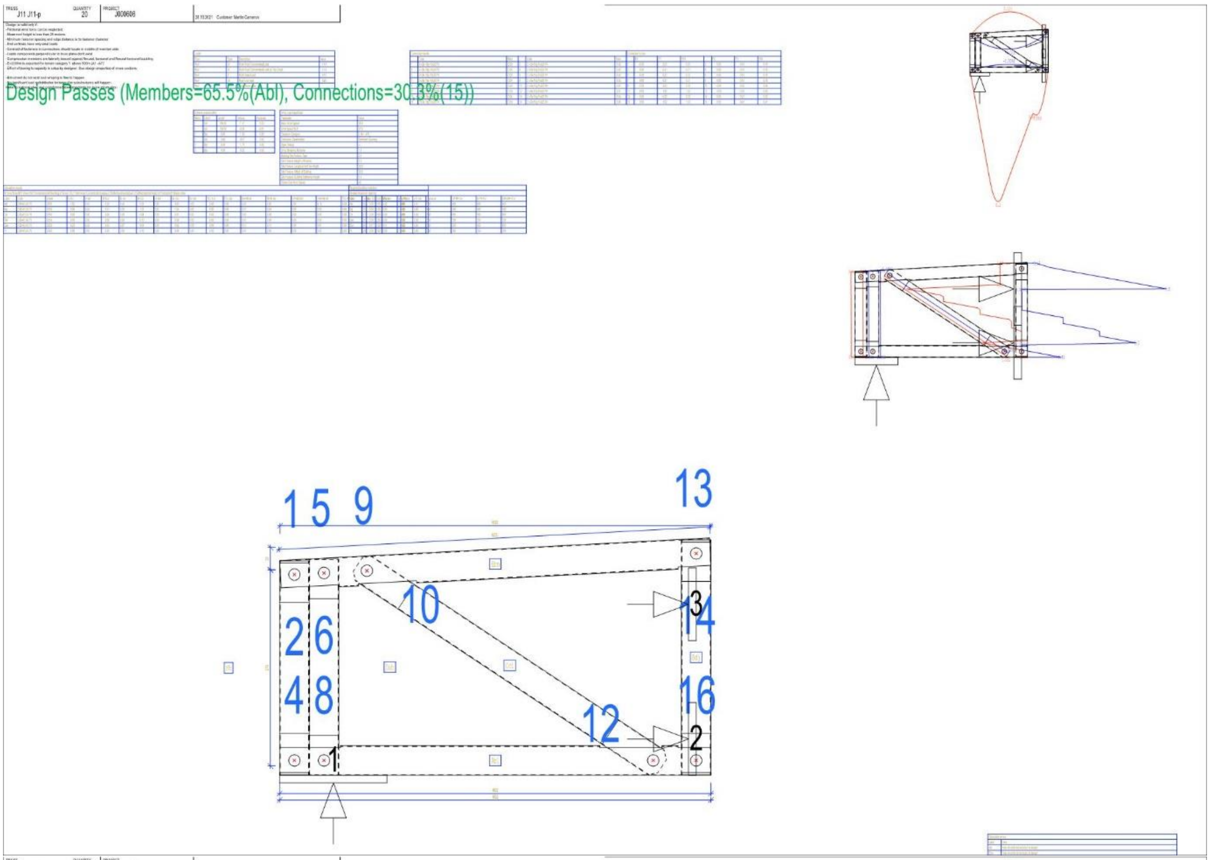
TRUSS
J11 J11-m
QUANTITY
20
PROJECT
J000608
Design Passes (Members=65.5%(Abd), Connections=30.3%(15))
1 5 9
13
2 6
4 8
10
12
14
16
3
2
1

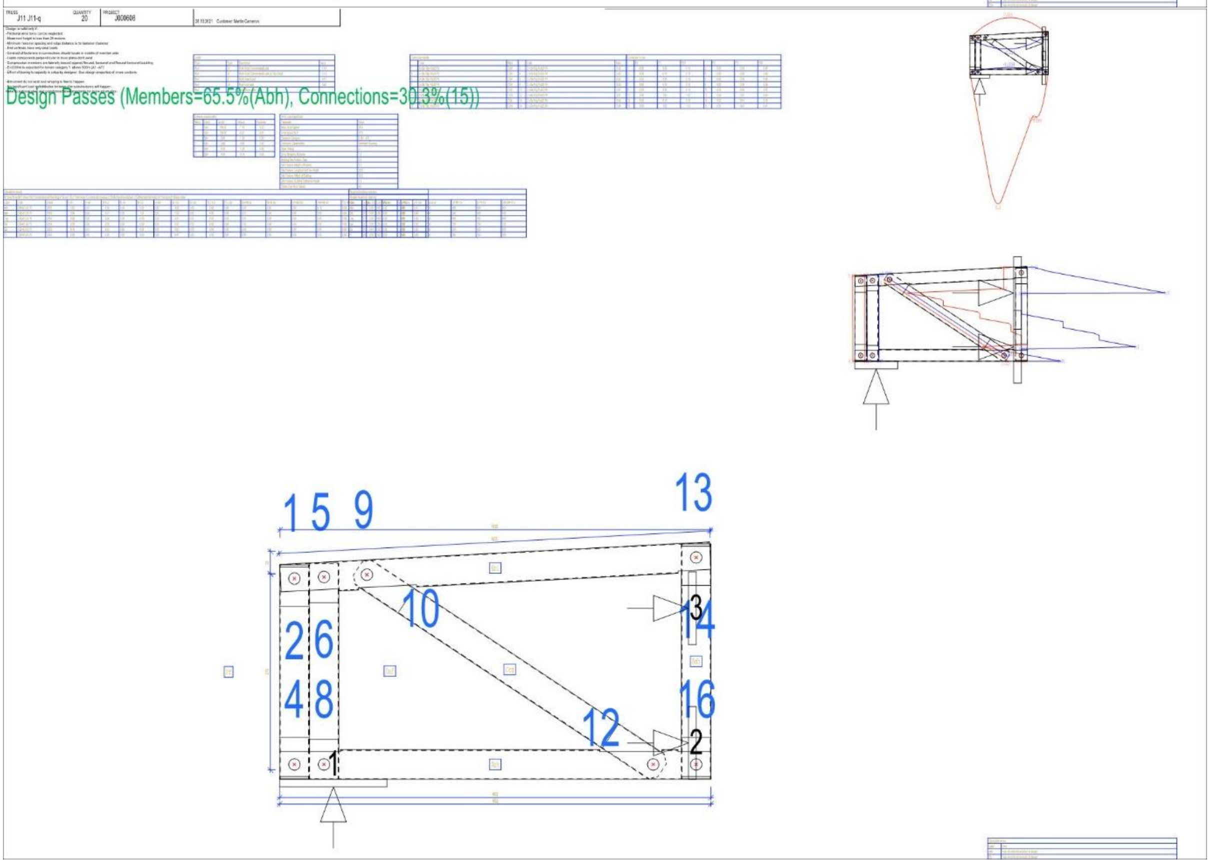


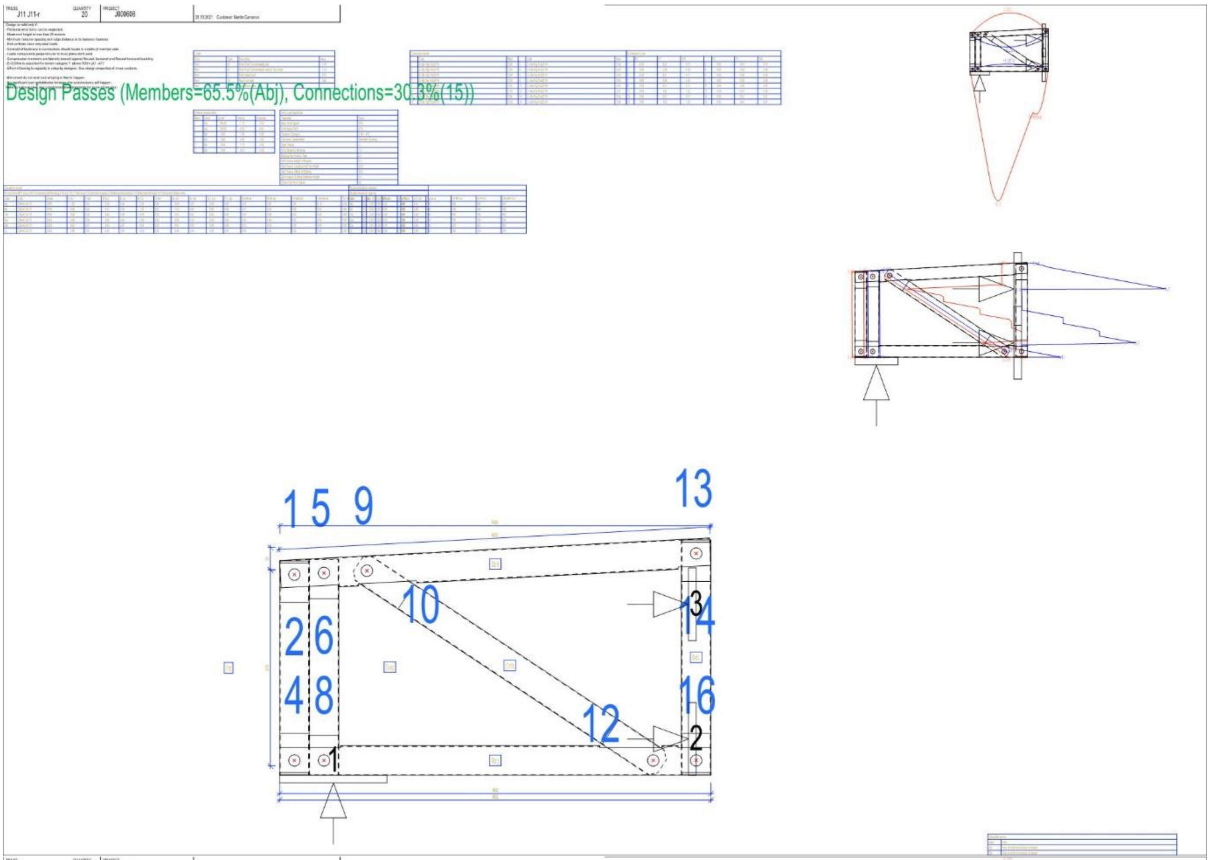
Design Passes (Members=65.5%(Abn), Connections=30.3%(15))
1 5 9
13
10
3
14
2 6
4 8
16
12
2
1



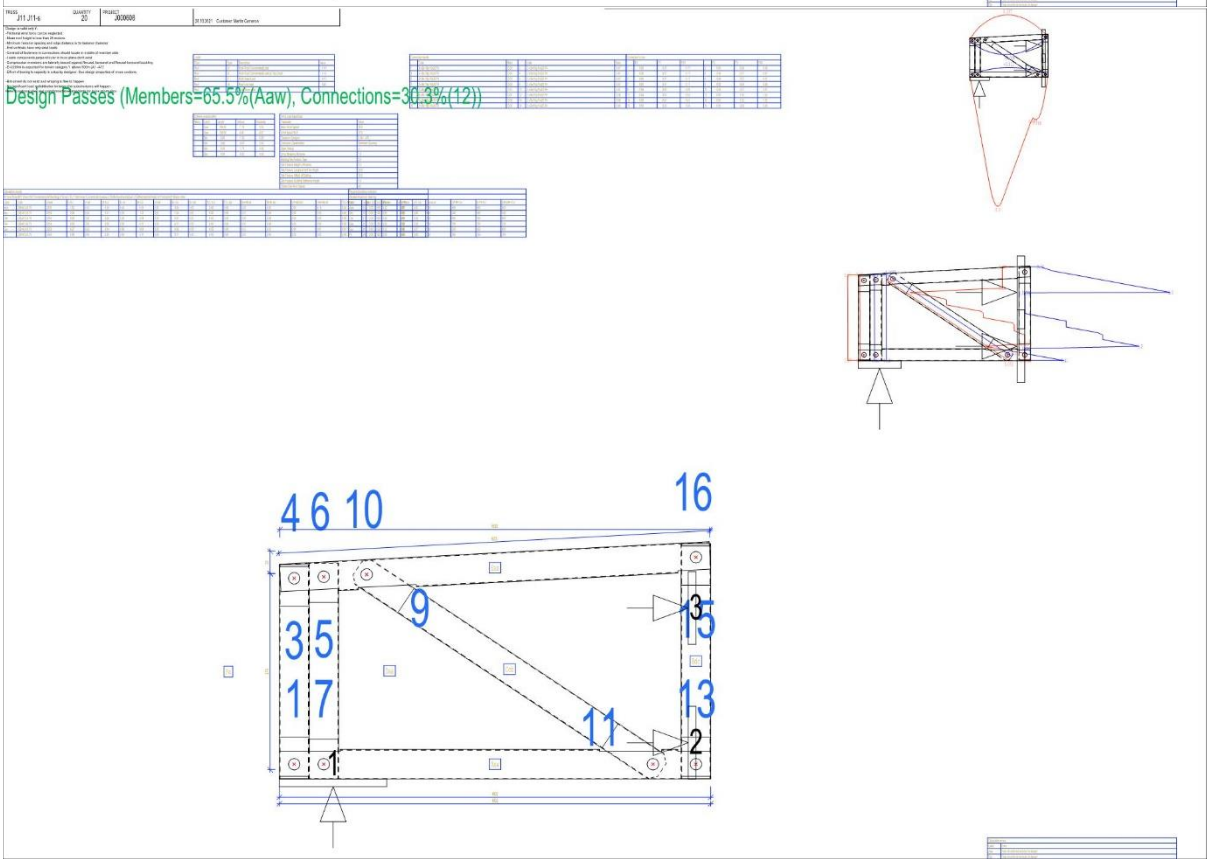
Design Passes (Members=65.5%(Abf), Connections=30.3%(15))
1 5 9
13
10
3
14
2 6
4 8
16
12
2
1


Design Passes (Members=65.5%(Abl), Connections=30.3%(15))
1 5 9
13
10
3
14
2 6
4 8
16
12
2
1

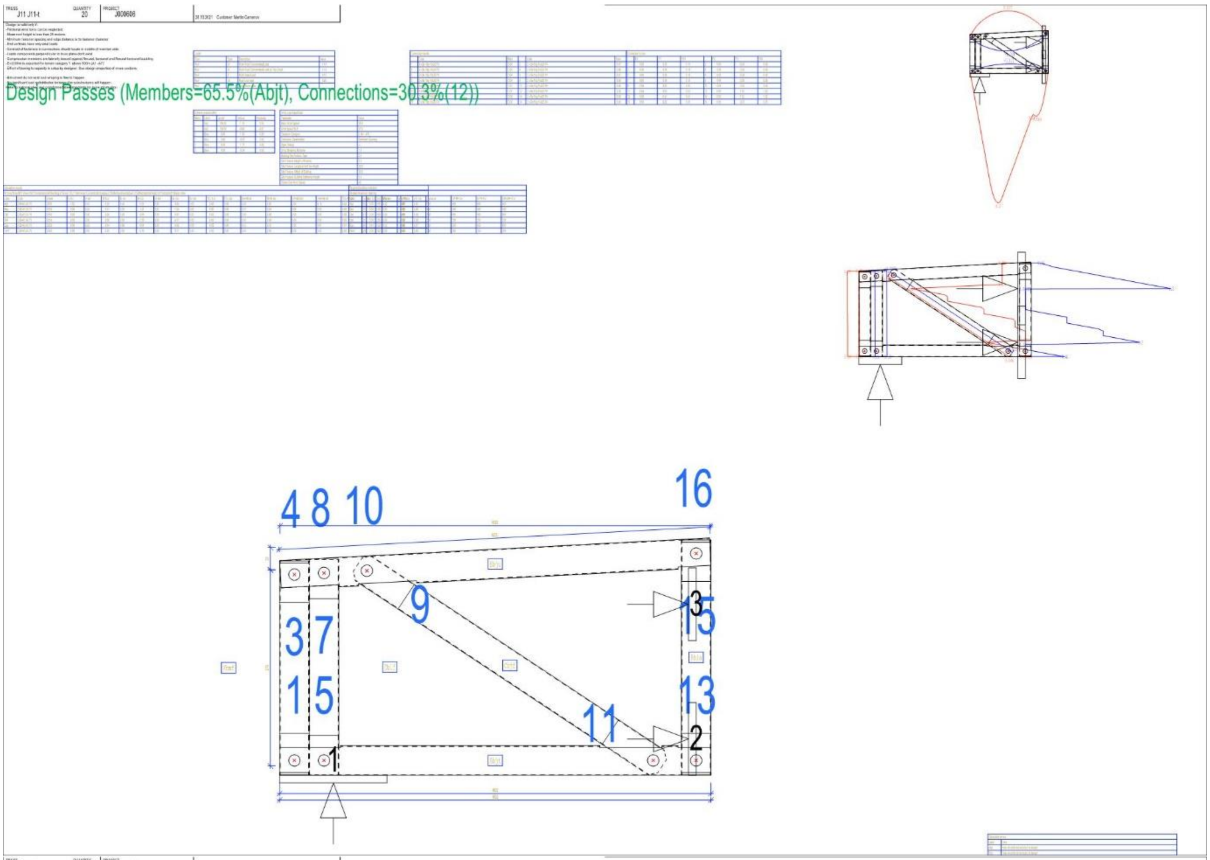

Design Passes (Members=65.5%(Abh), Connections=30.3%(15))
1 5 9
13
10
3
14
2 6
4 8
16
12
2
1



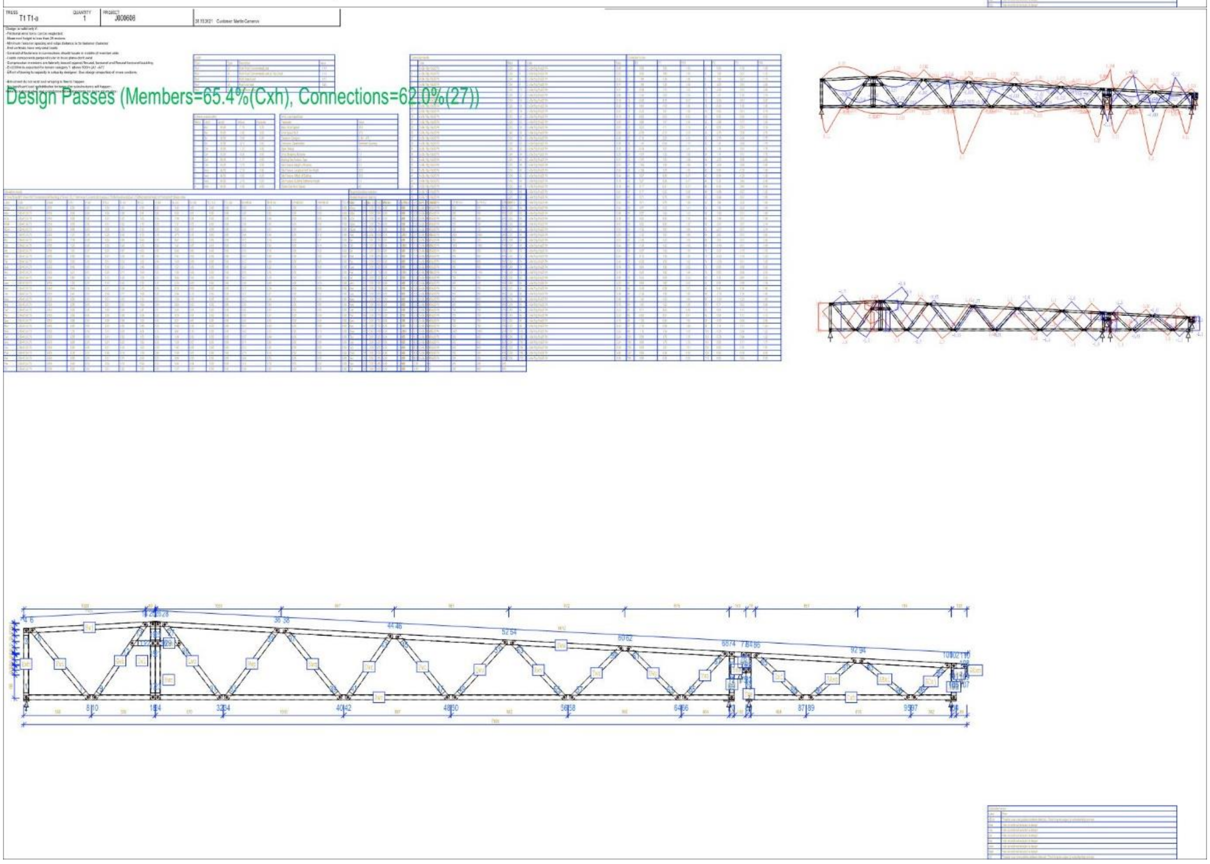
Design Passes (Members=65.5%(Abj), Connections=30.3%(15))
1 5 9
13
10
2 6
4 8
14
16
12
3
2
1



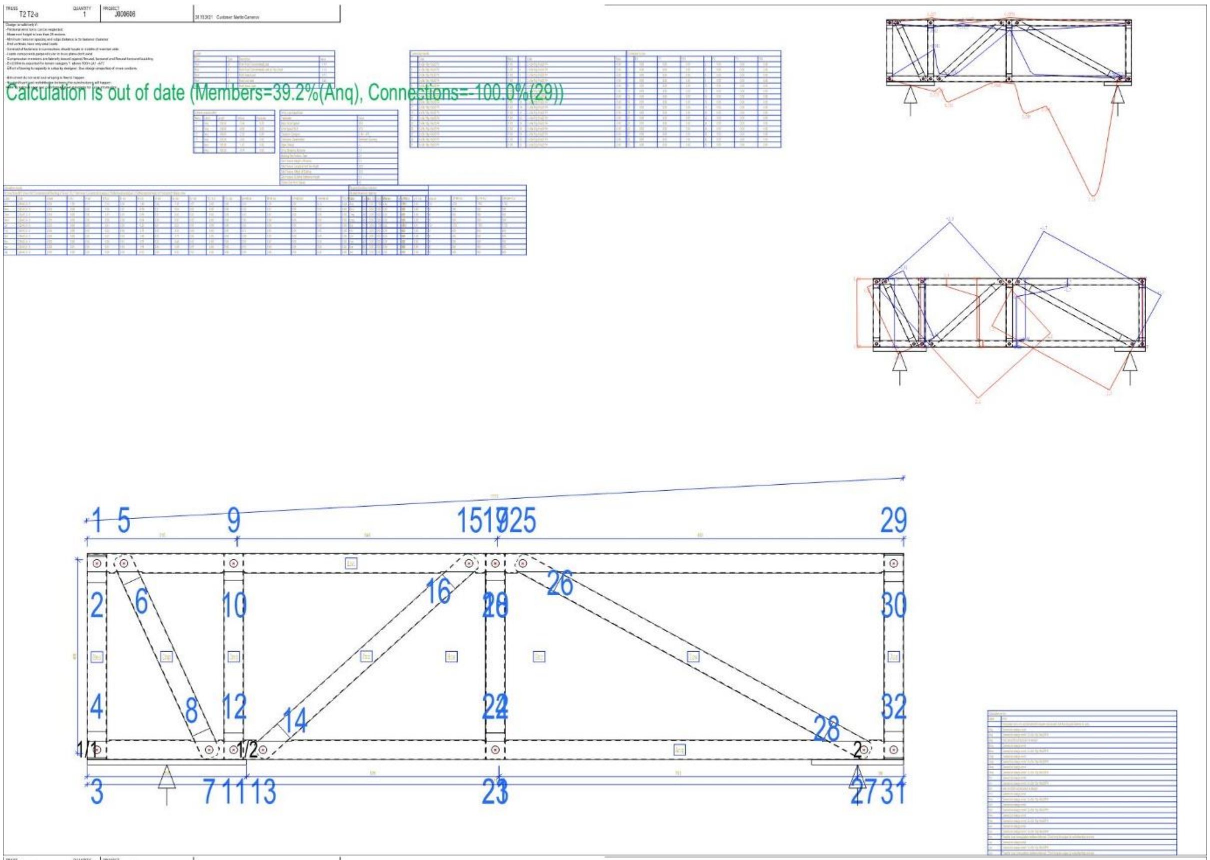
Design Passes (Members=65.5%(Aaw), Connections=30.3%(12))
4 6 10
16
9
3 5
1 7
15
13
11
3
2
1

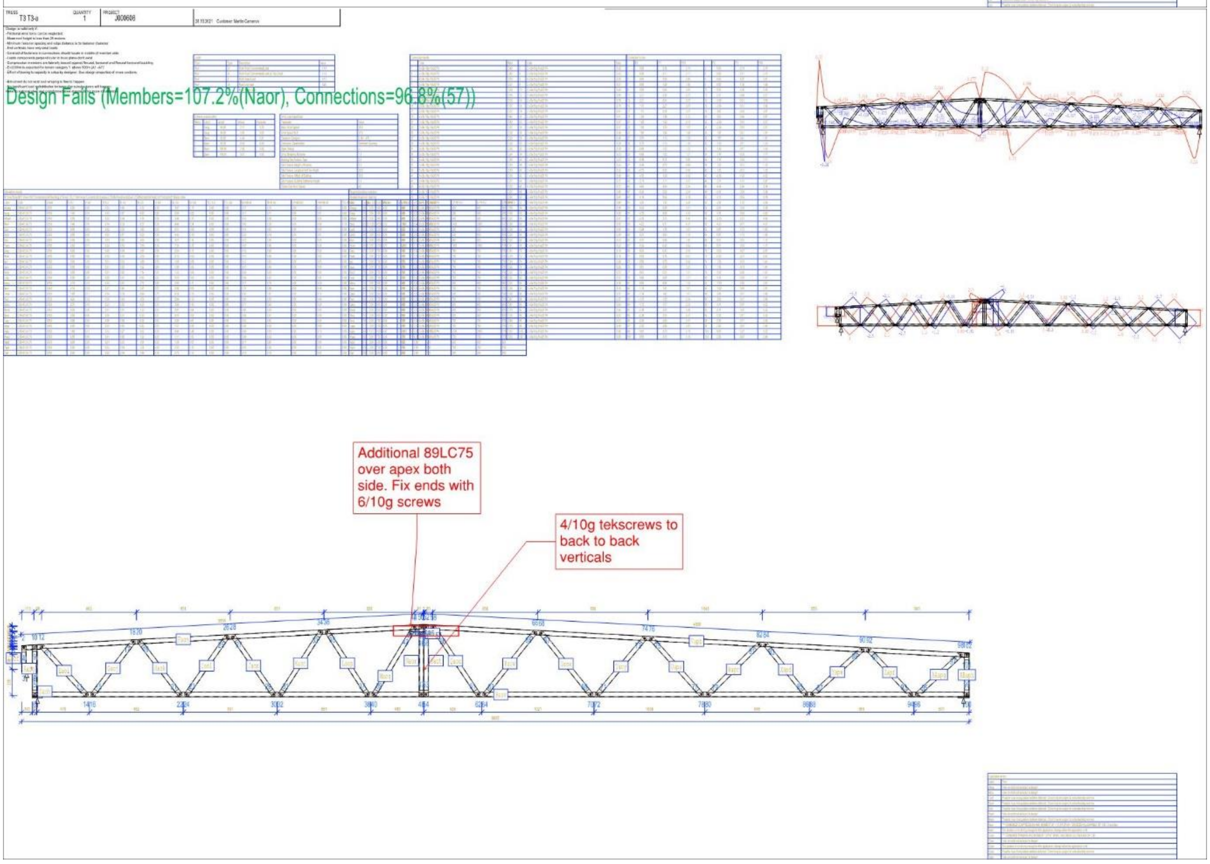


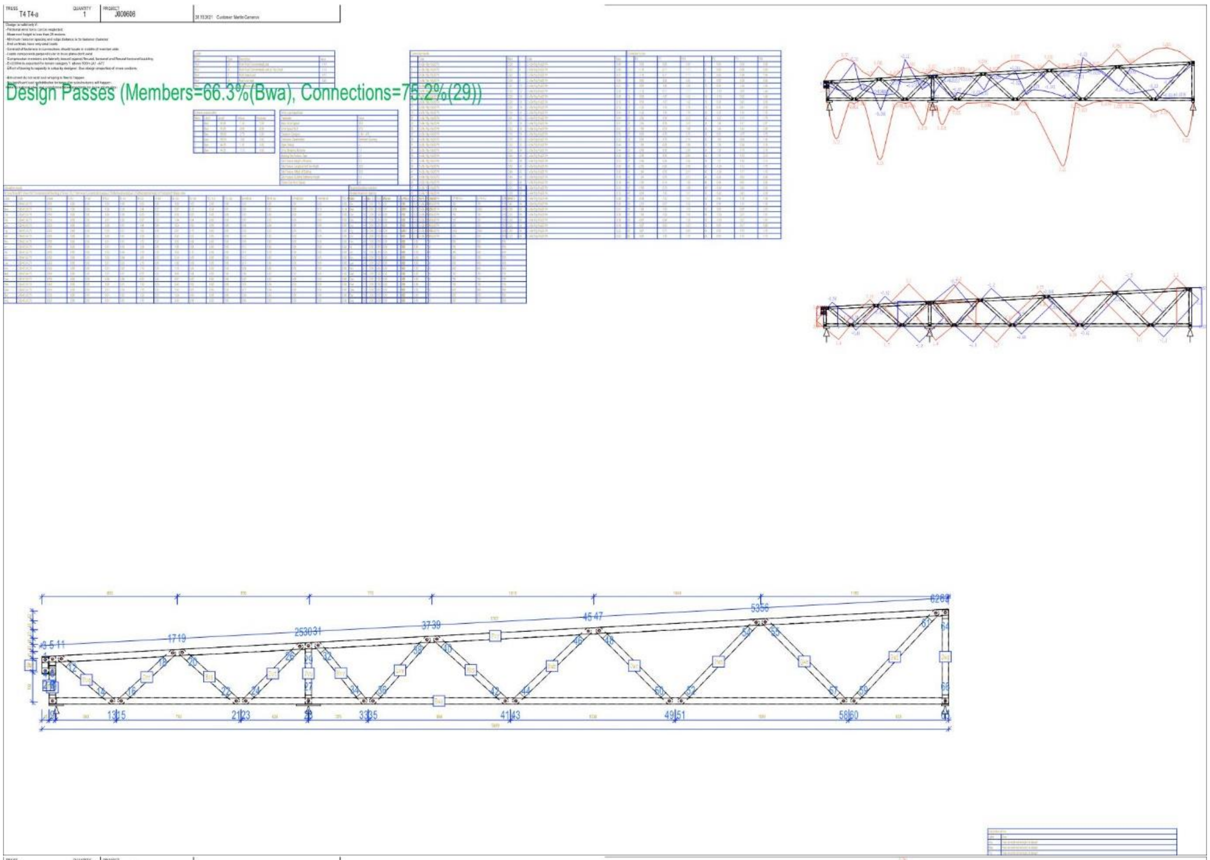
Design Passes (Members=65.5%(Abjt), Connections=30.3%(12))
4 8 10
16
9
3 7
1 5
3
15
13
11
2
1



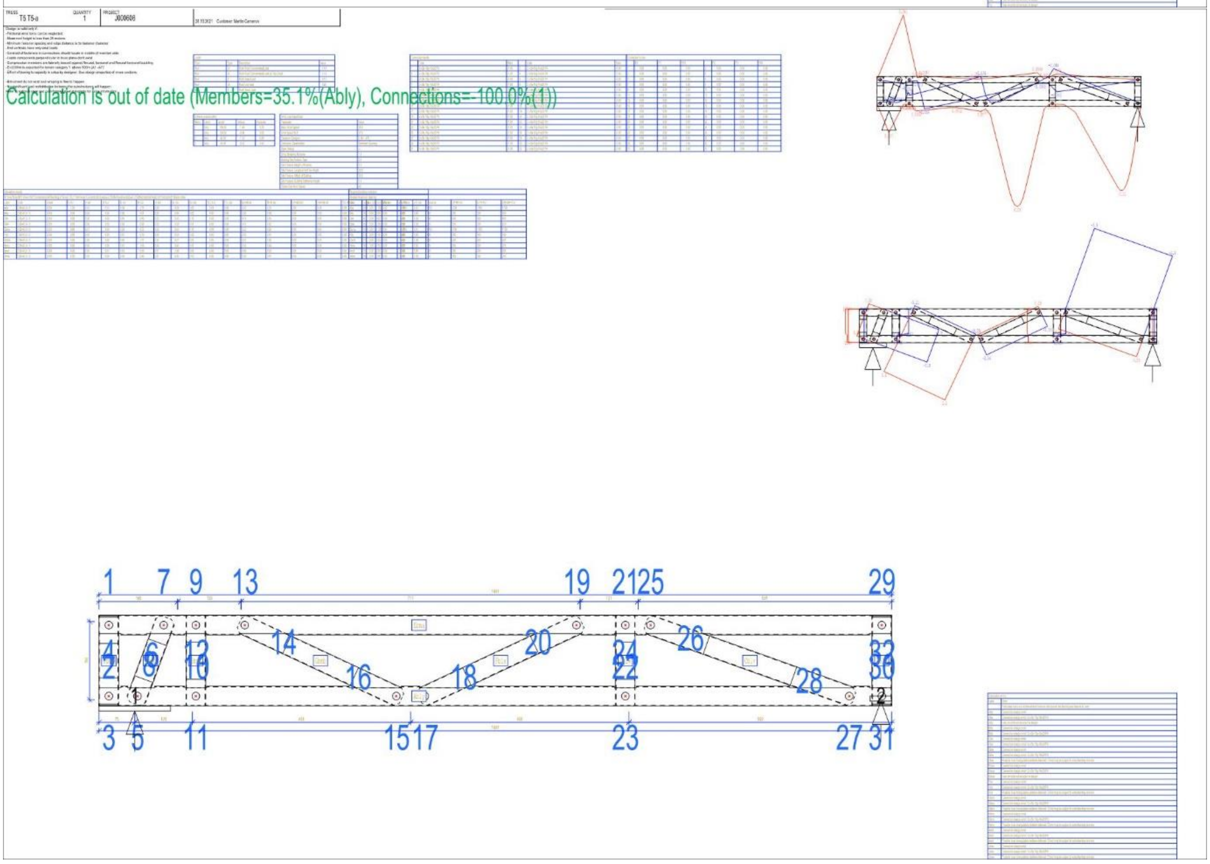
Design Passes (Members=65.4%(Cxh), Connections=62.0%(27))


T2 T2-a
J609686
Calculation is out of date (Members=39.2%(Anq), Connections=100.0%(29))

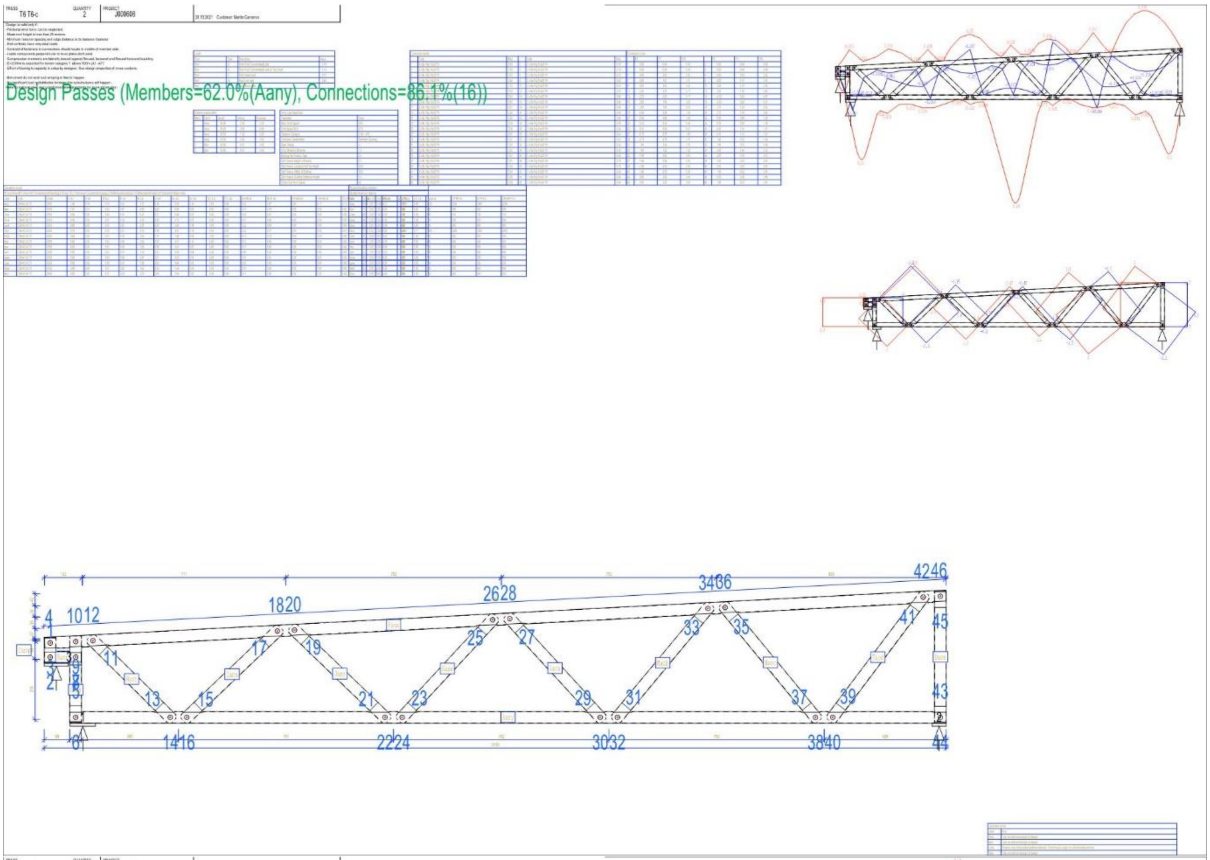

T3 T3-a
J609686
Design Fails (Members=107.2%(Naor), Connections=96.8%(57))
Additional 89LC75 over apex both side. Fix ends with 6/10g screws
4/10g tekscrews to back to back verticals



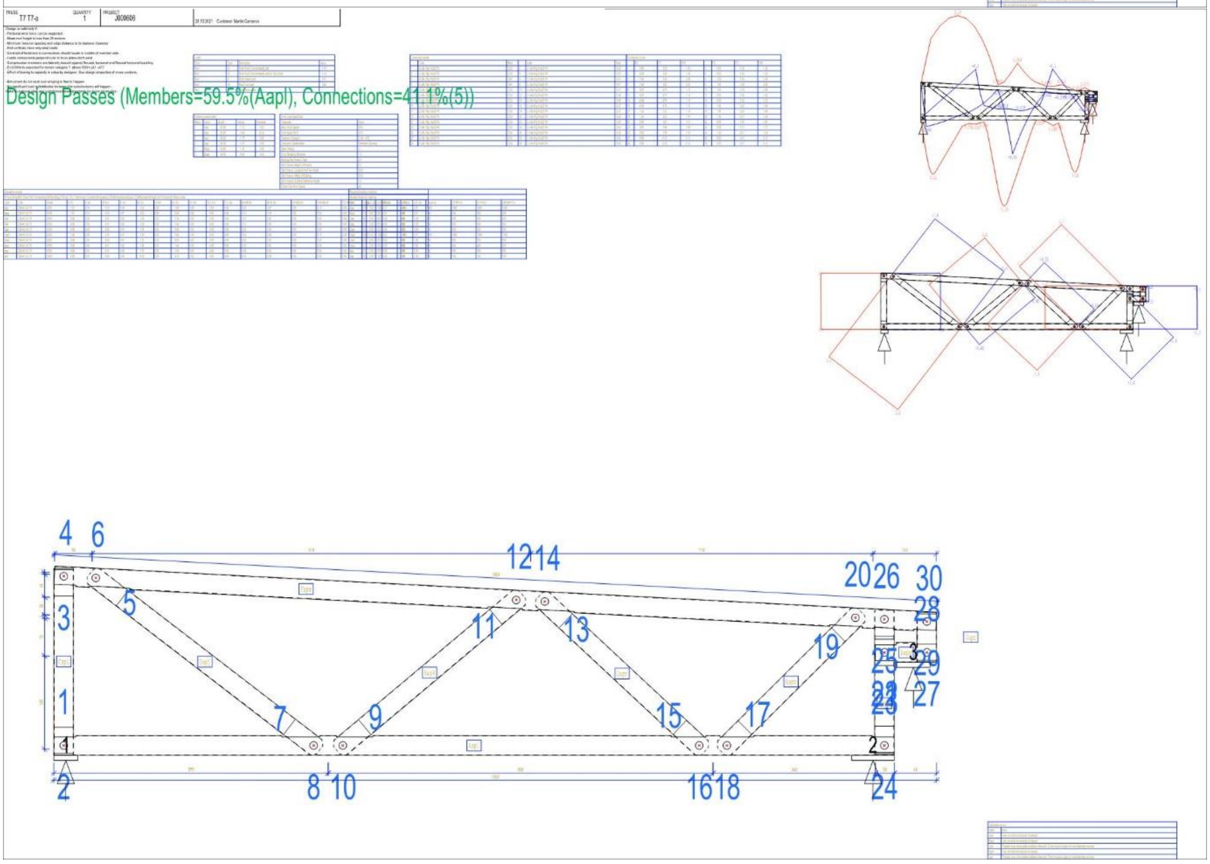
Design Passes (Members=66.3%(Bwa), Connections=75.2%(29))



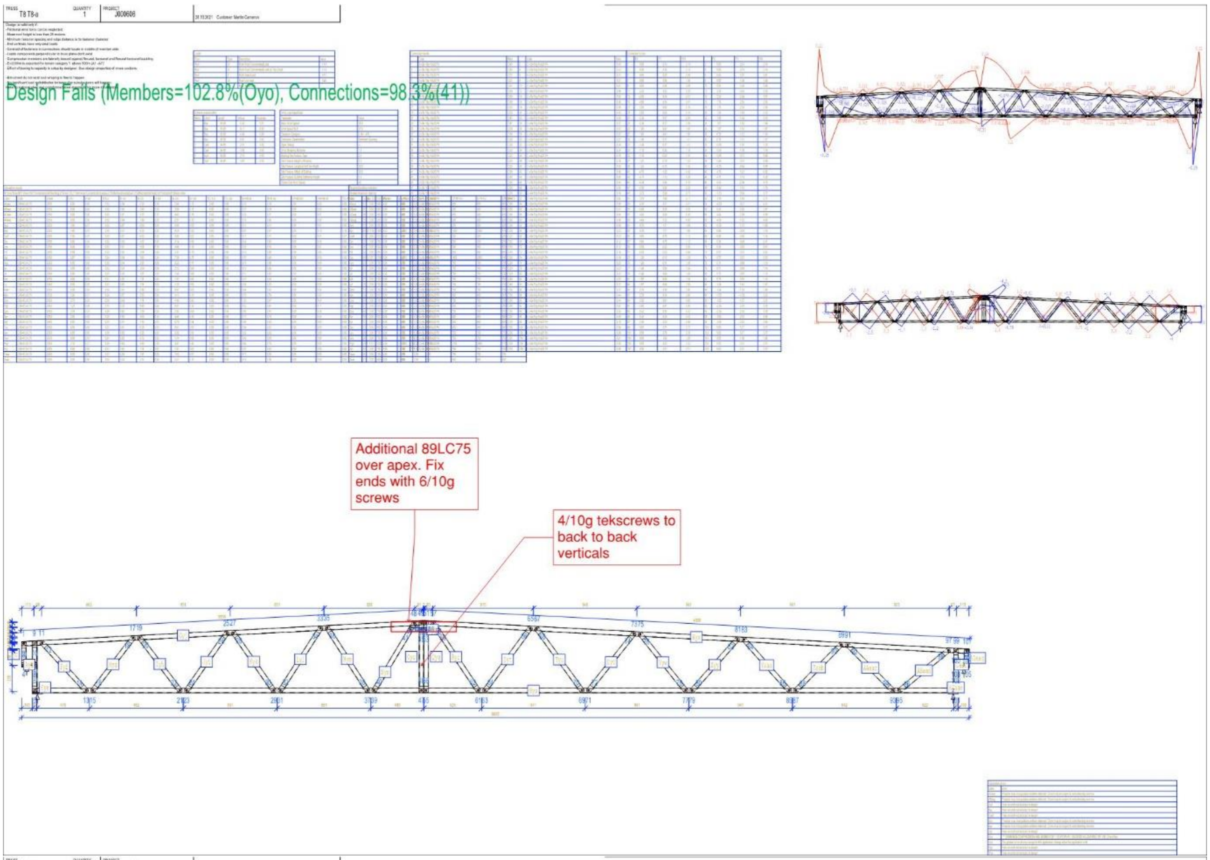
Calculation is out of date (Members=35.1%(Ably), Connections=-100.0%(1))



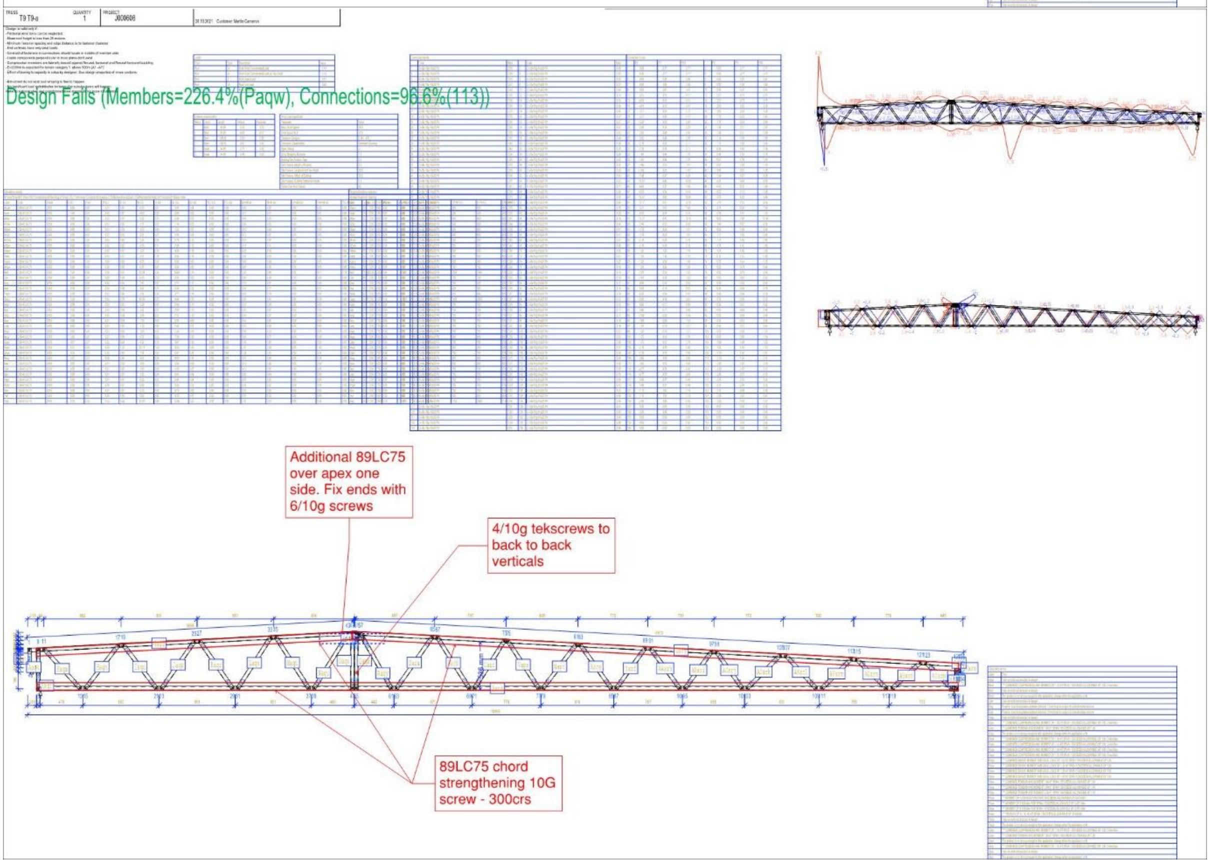
Design Passes (Members=62.0%(Aany), Connections=86.1%(16))



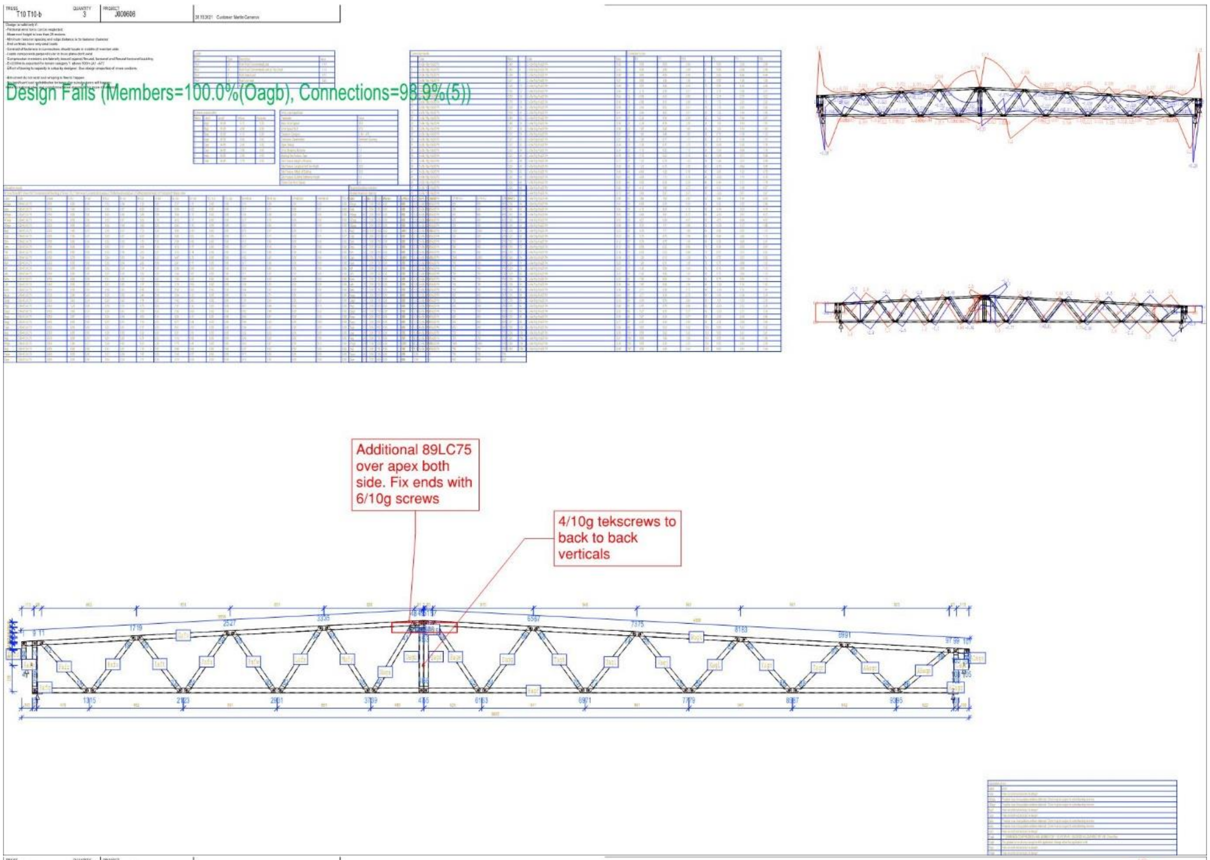
Design Passes (Members=59.5%(Aapl), Connections=41.1%(5))



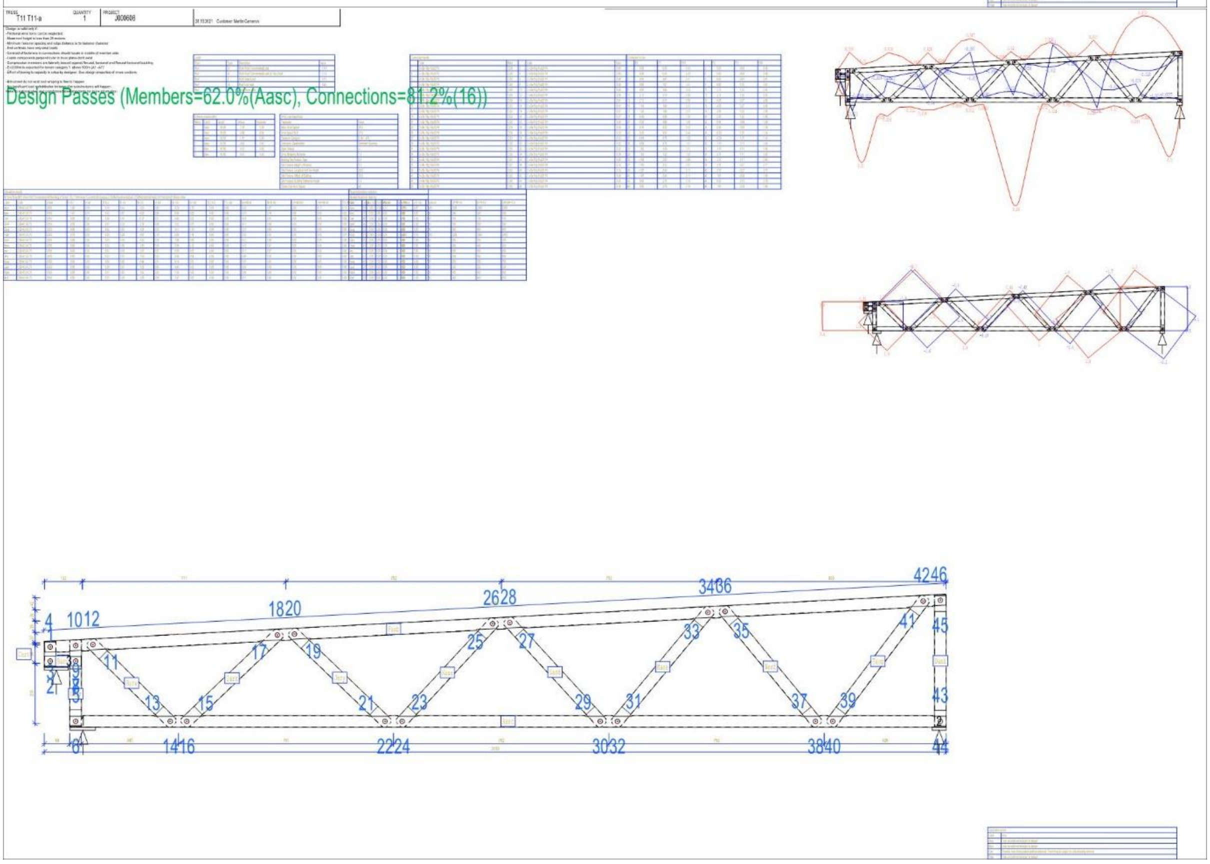
Design Fails (Members=102.8%(Oyo), Connections=98.3%(41))
Additional 89LC75 over apex. Fix ends with 6/10g screws
4/10g tekscrews to back to back verticals



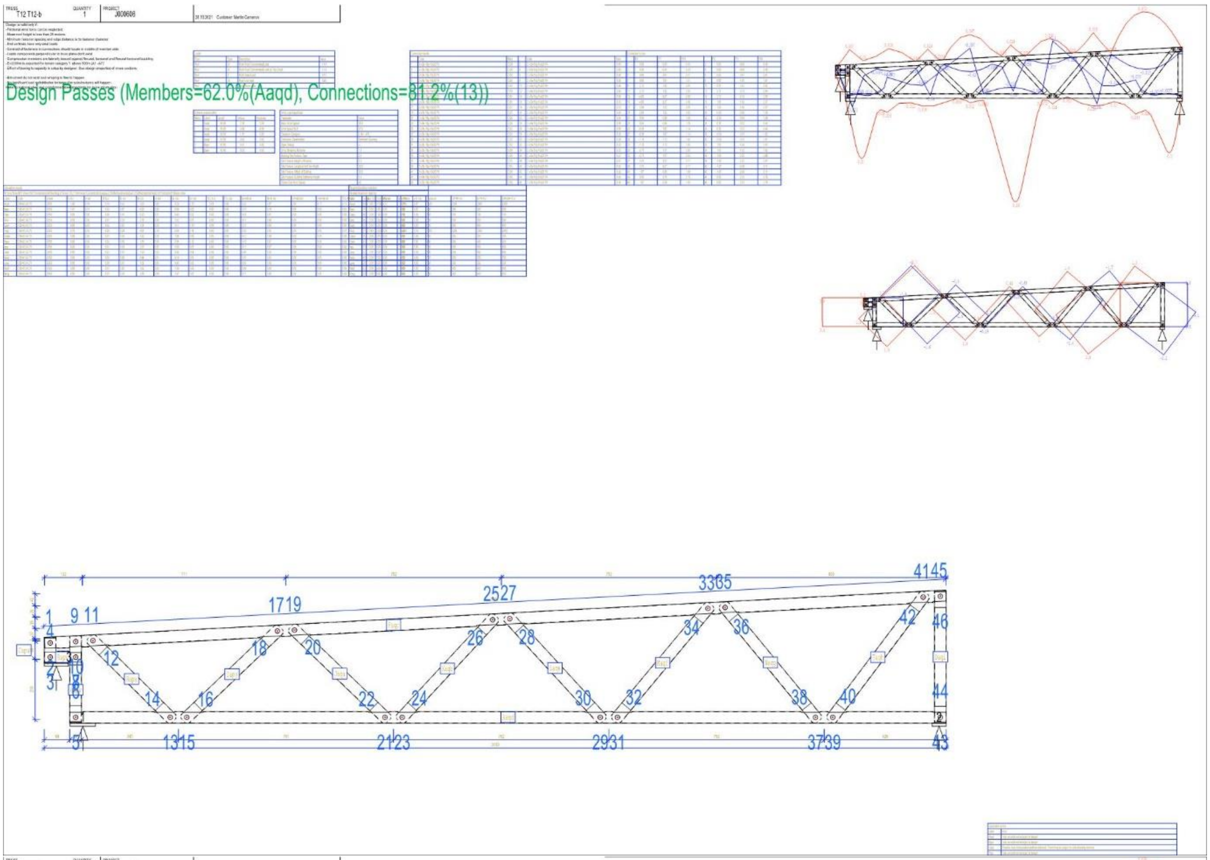
Design Fails (Members=226.4%(Paqw), Connections=96.6%(113))
Additional 89LC75 over apex one side. Fix ends with 6/10g screws
4/10g tekscrews to back to back verticals
89LC75 chord strengthening 10G screw - 300crs



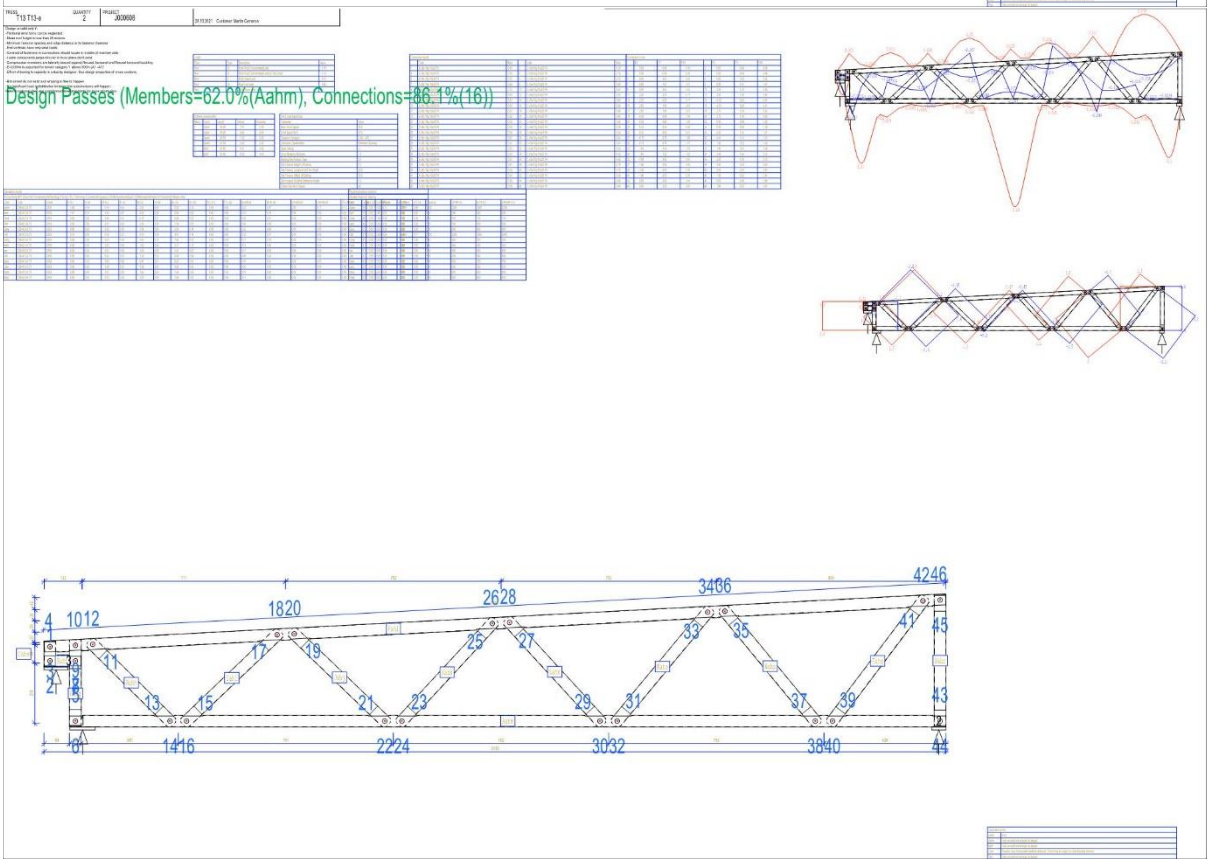
T10 T10-b
J608686
Design Fails (Members=100.0%(Oagb), Connections=98.9%(5))
Additional 89LC75 over apex both side. Fix ends with 6/10g screws
4/10g tekscrews to back to back verticals



T11 T11-a
J608686
Design Passes (Members=62.0%(Aasc), Connections=81.2%(16))
1012
1820
2628
3406
4246
1416
2224
3032
3840

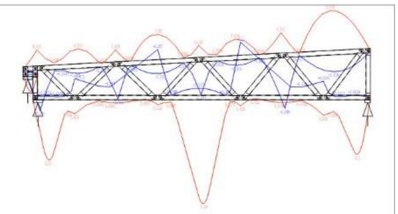


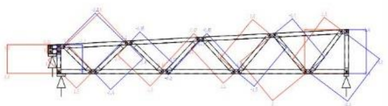
Design Passes (Members=62.0%(Aaqd), Connections=81.2%(13))

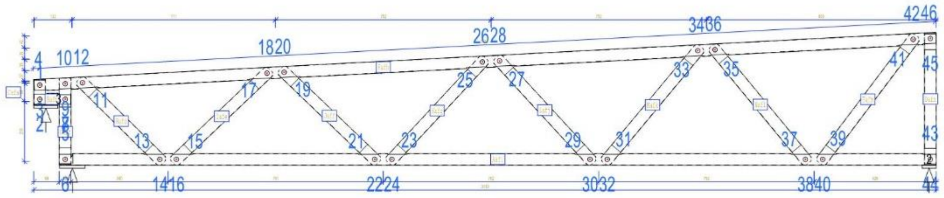


Design Passes (Members=62.0%(Aahm), Connections=86.1%(16))

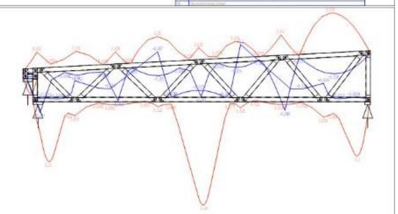
Design Passes (Members=62.0%(Aafi), Connections=86.1%(16))

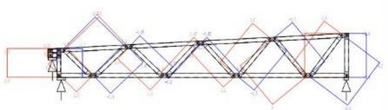


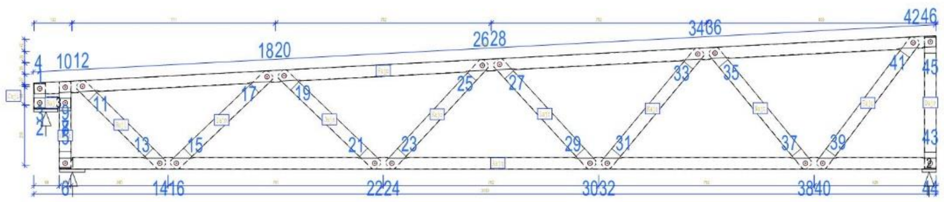


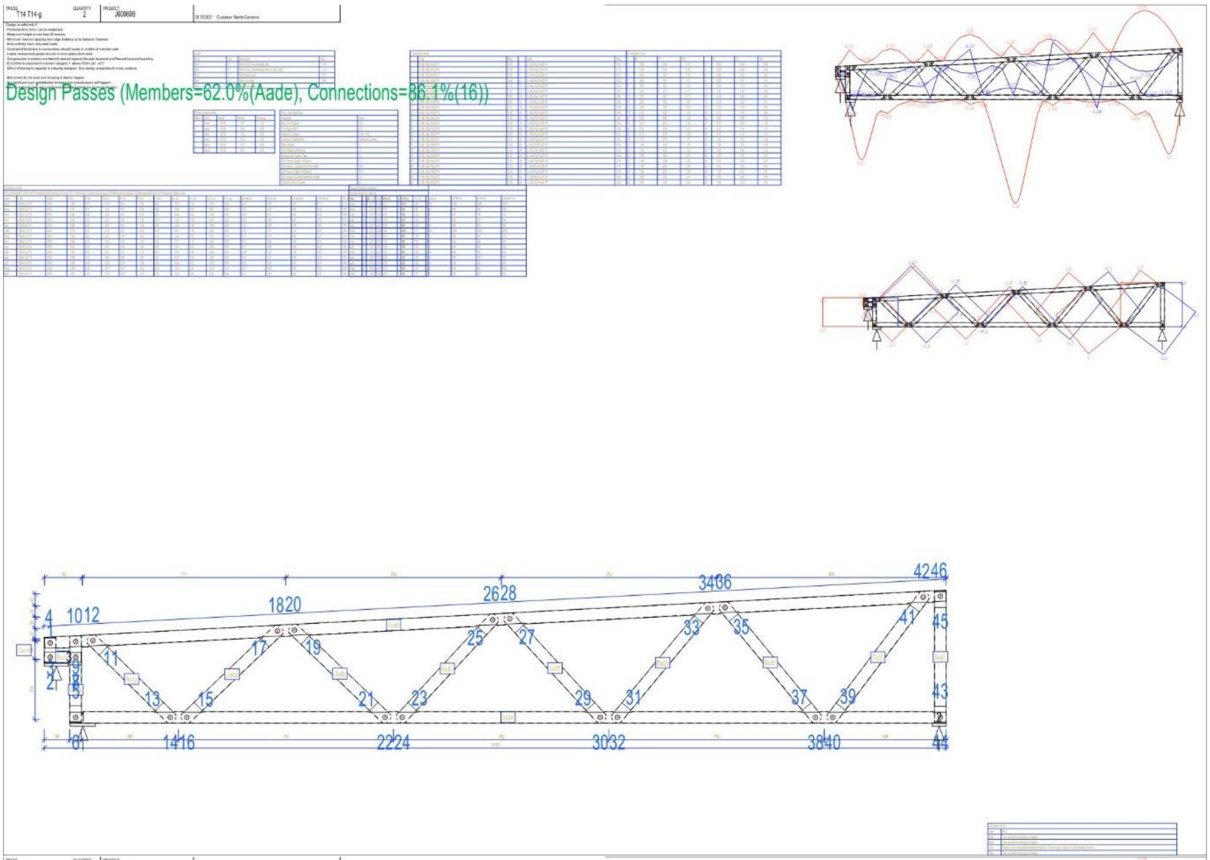


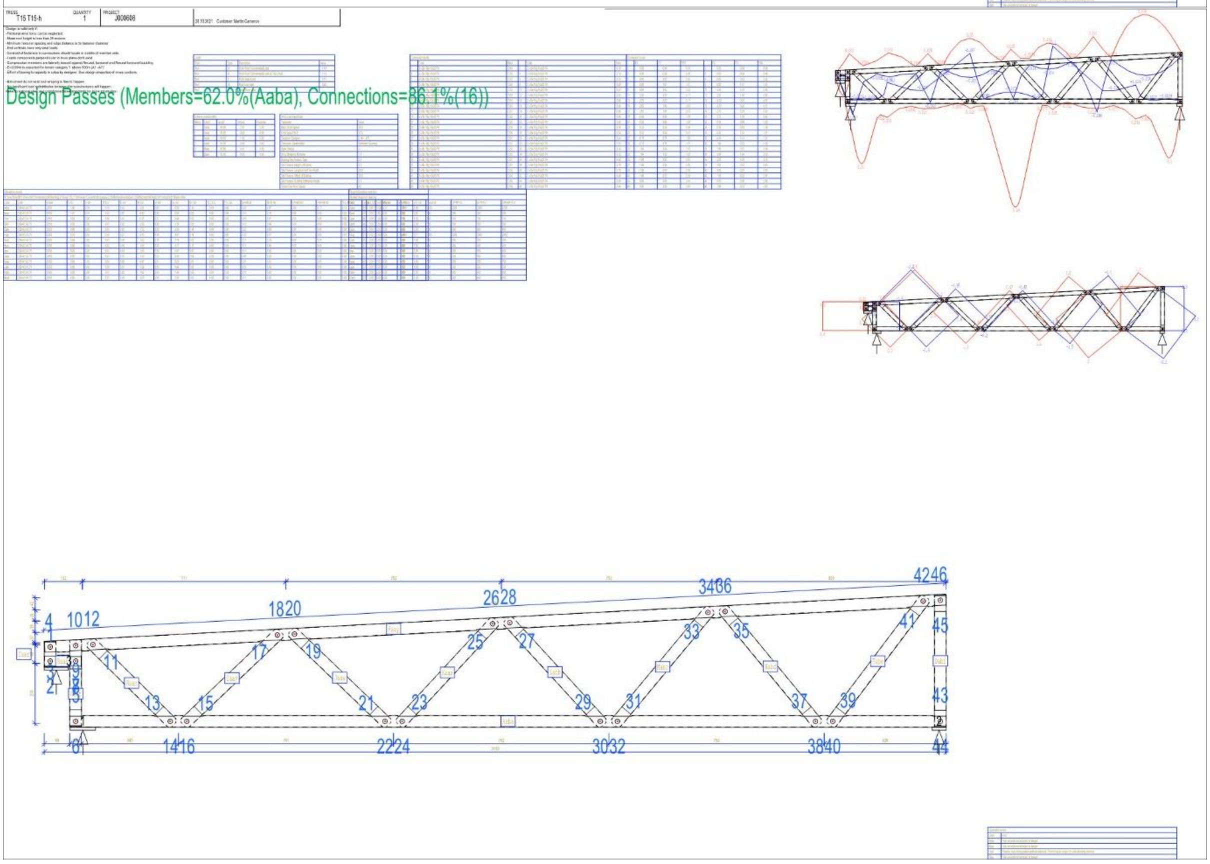
Design Passes (Members=62.0%(Aajq), Connections=86.1%(16))

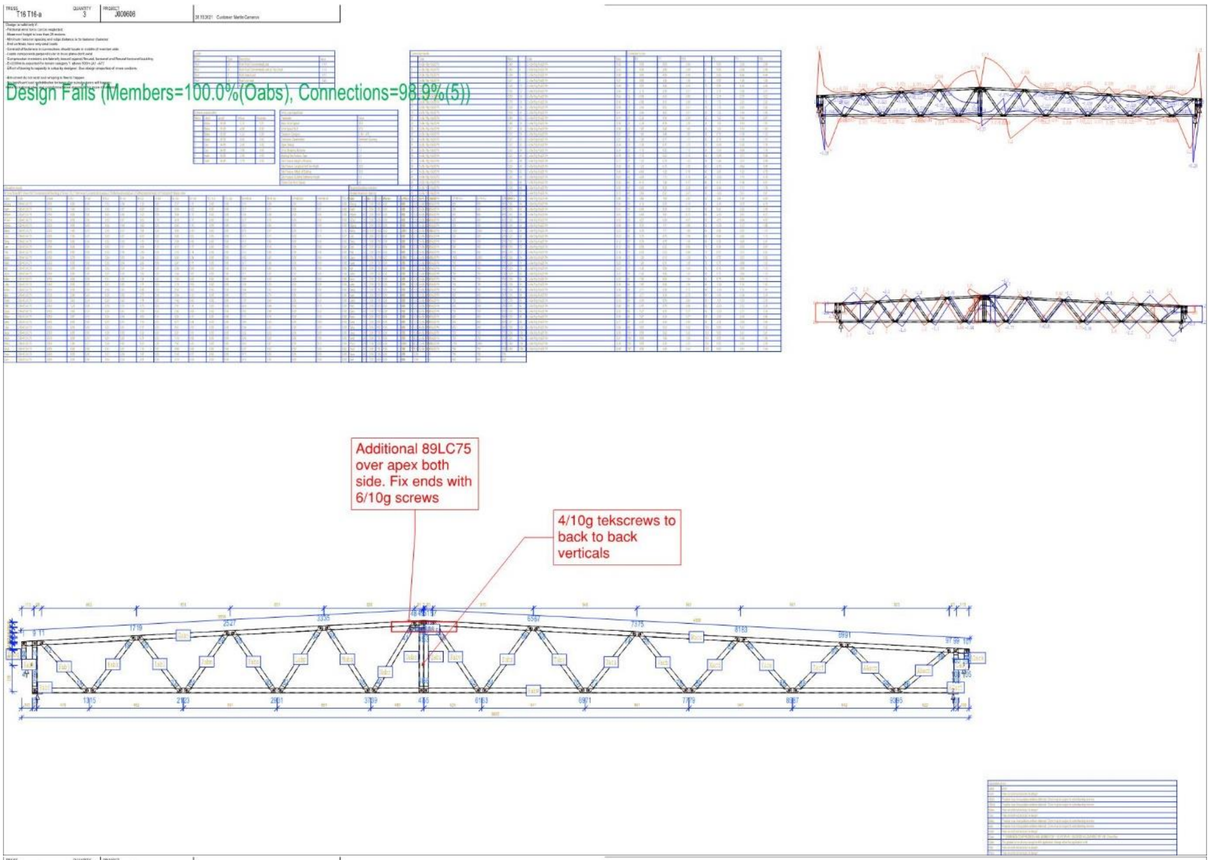




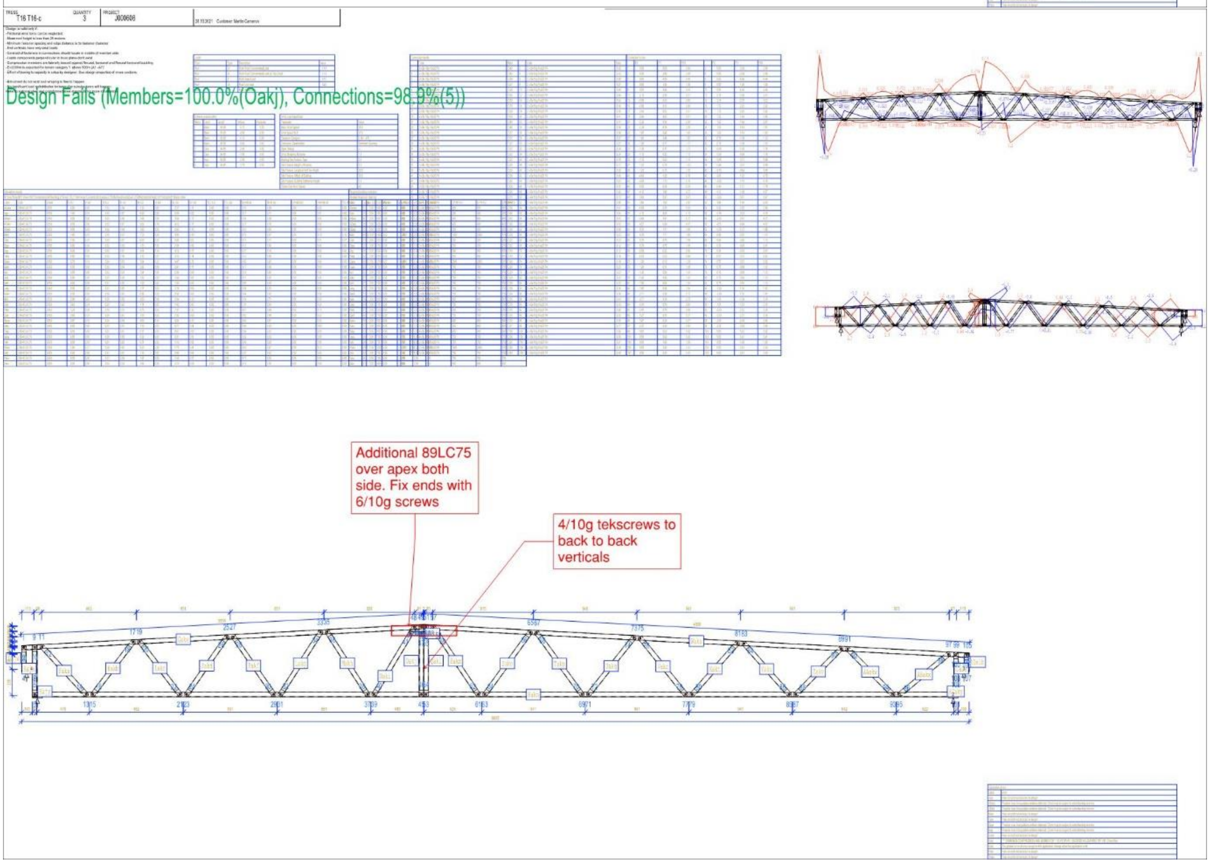



T14 T14-g
2
J608686
Design Passes (Members=62.0%(Aade), Connections=86.1%(16))
1012
1820
2628
3436
4246
1416
2224
3032
3840

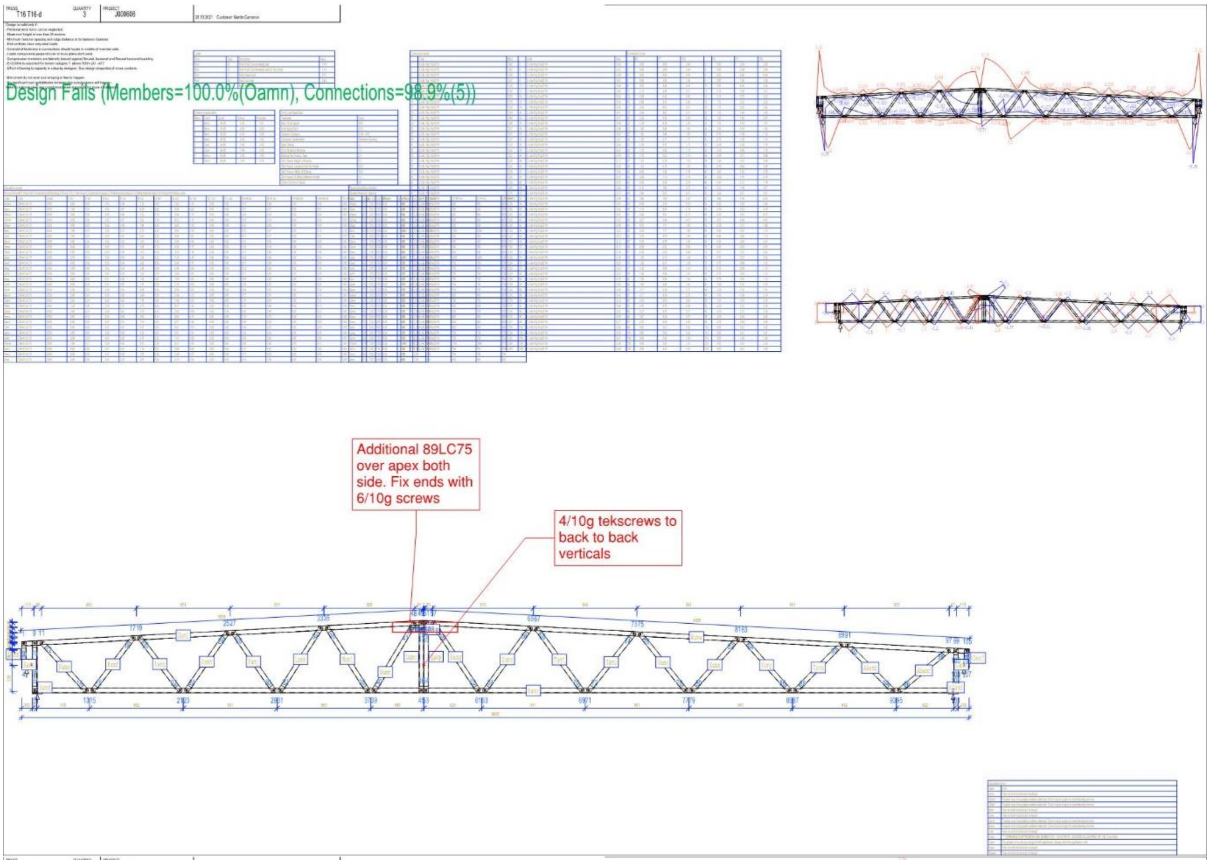

T15 T15-h
1
J608686
Design Passes (Members=62.0%(Aaba), Connections=86.1%(16))
1012
1820
2628
3436
4246
1416
2224
3032
3840



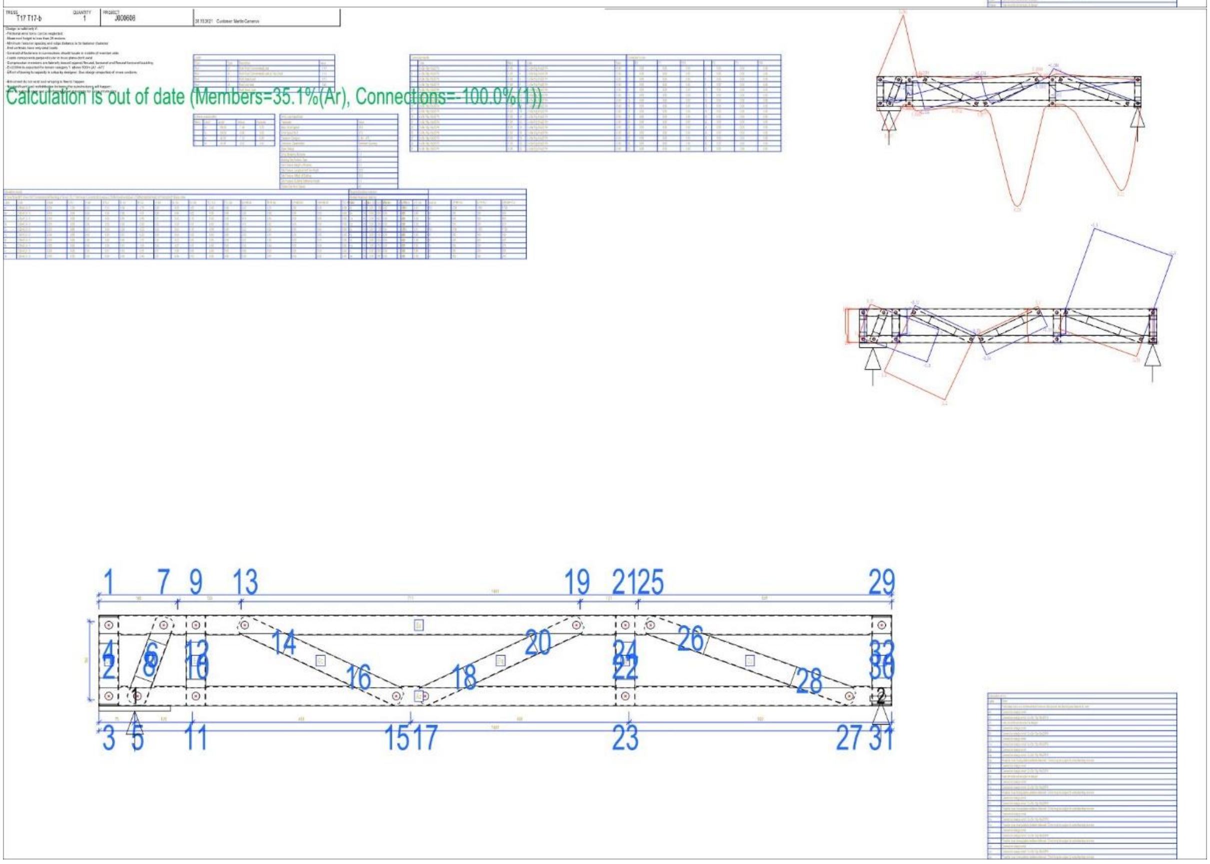
T16 T16-a
J606686
Design Fails (Members=100.0%(Oabs), Connections=98.9%(5))
Additional 89LC75 over apex both side. Fix ends with 6/10g screws
4/10g tekscrews to back to back verticals



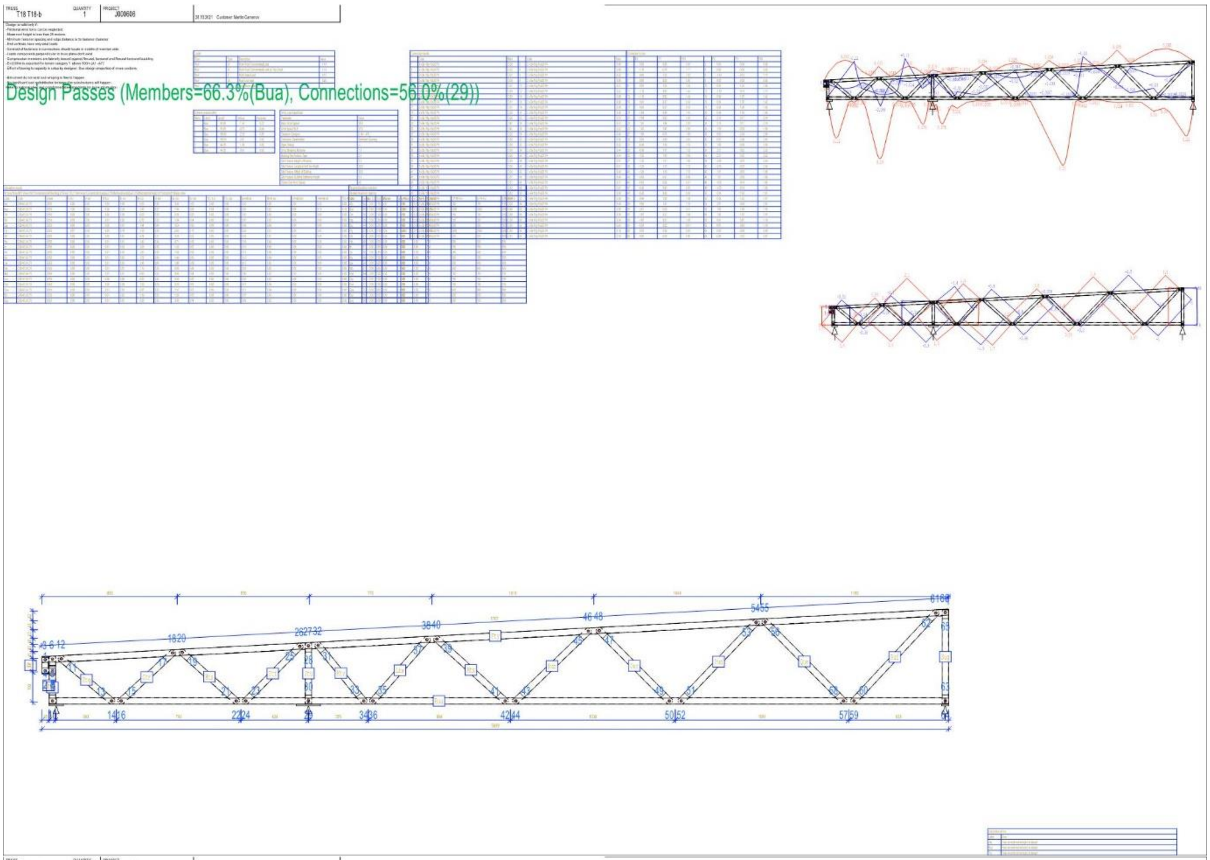
T16 T16-c
J606686
Design Fails (Members=100.0%(Oakj), Connections=98.9%(5))
Additional 89LC75 over apex both side. Fix ends with 6/10g screws
4/10g tekscrews to back to back verticals

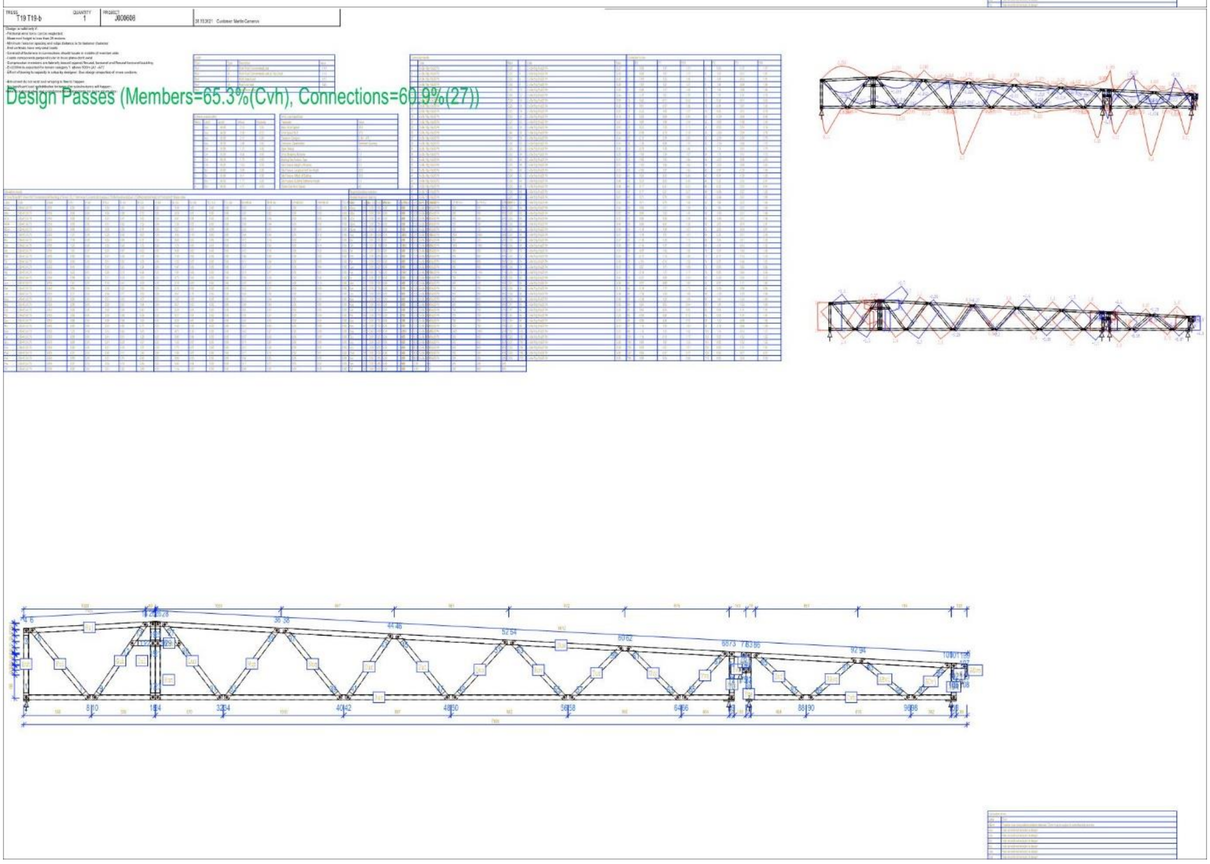


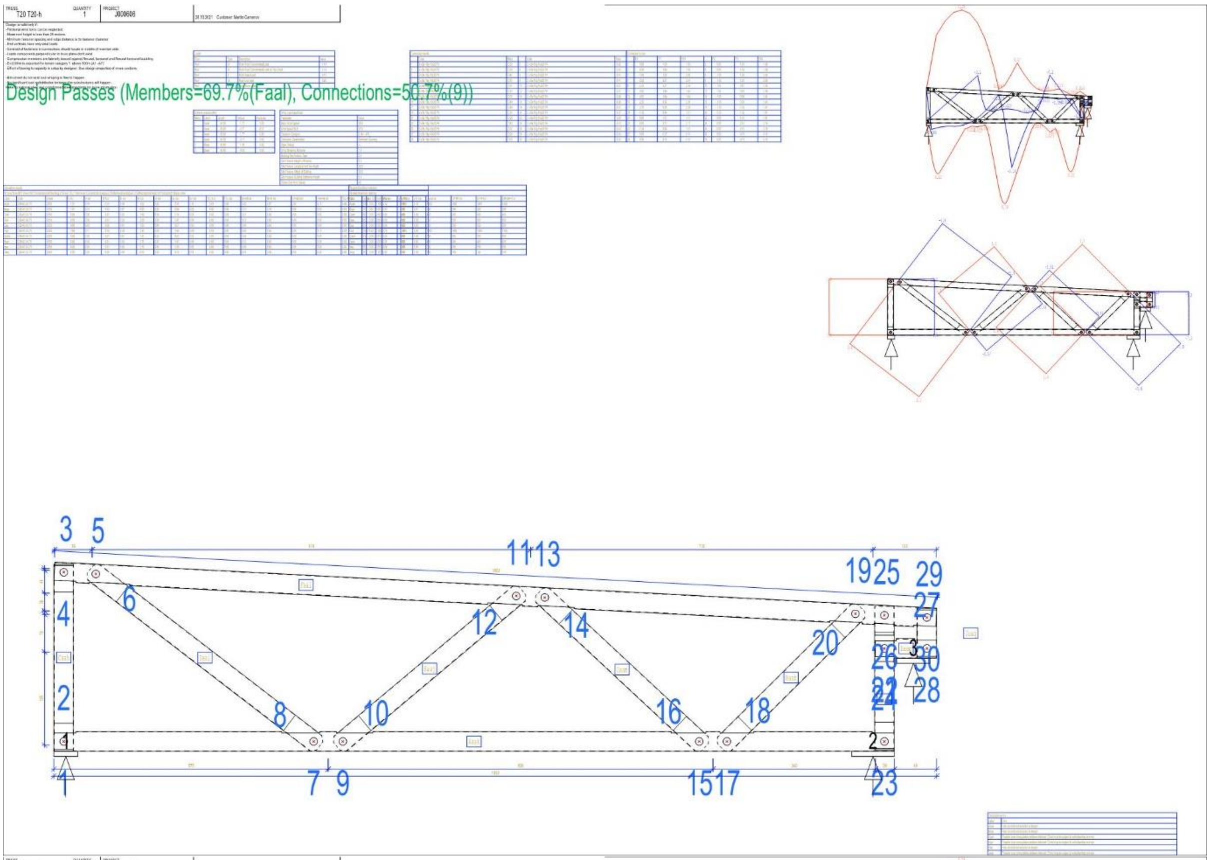
TRUSS T16 T16-d
QUANTITY 3
PROJECT J000606
Customer Martin Cameron
Design Fails (Members=100.0%(Oamn), Connections=98.9%(5))
Additional 89LC75 over apex both side. Fix ends with 6/10g screws
4/10g tekscrews to back to back verticals



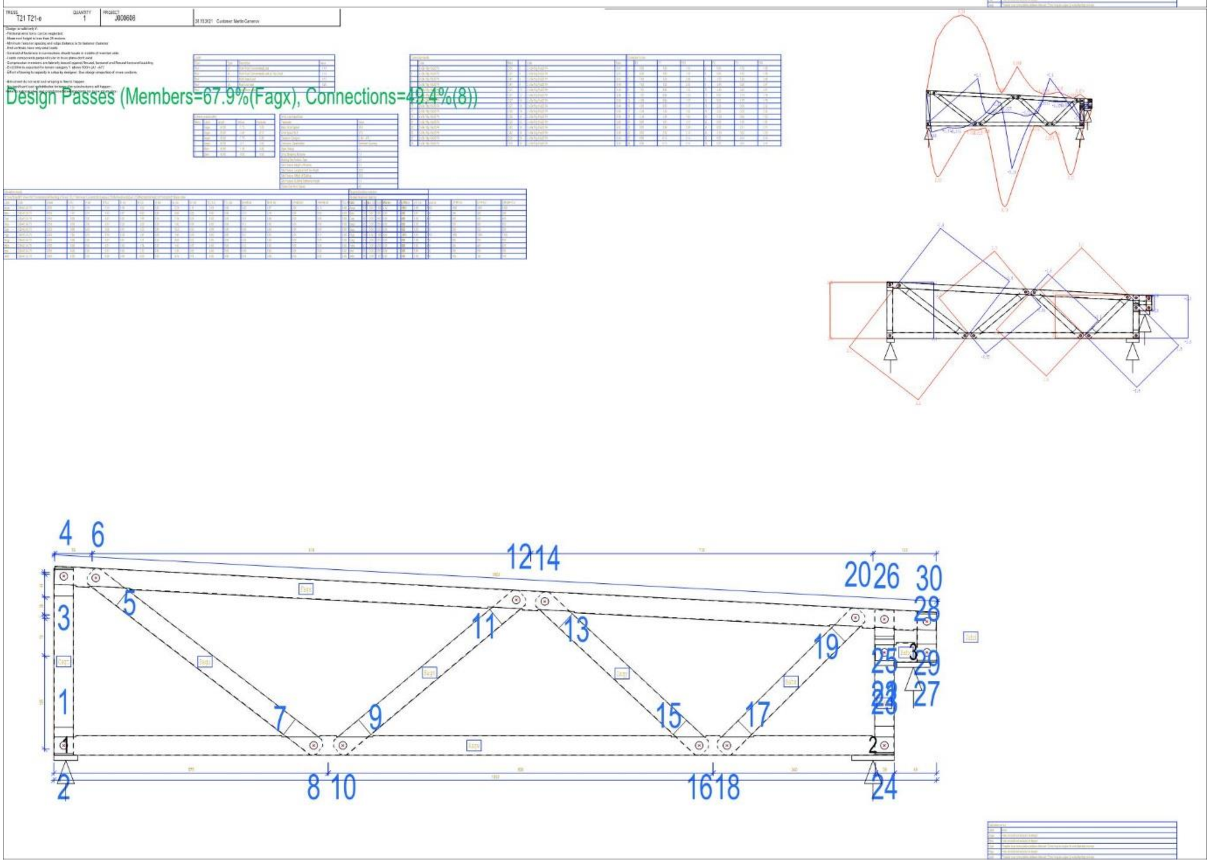
TRUSS T17 T17-b
QUANTITY 1
PROJECT J000606
Customer Martin Cameron
Calculation is out of date (Members=35.1%(Ar), Connections=-100.0%(1))


T18 T18-b
J606686
Design Passes (Members=66.3%(Bua), Connections=56.0%(29))

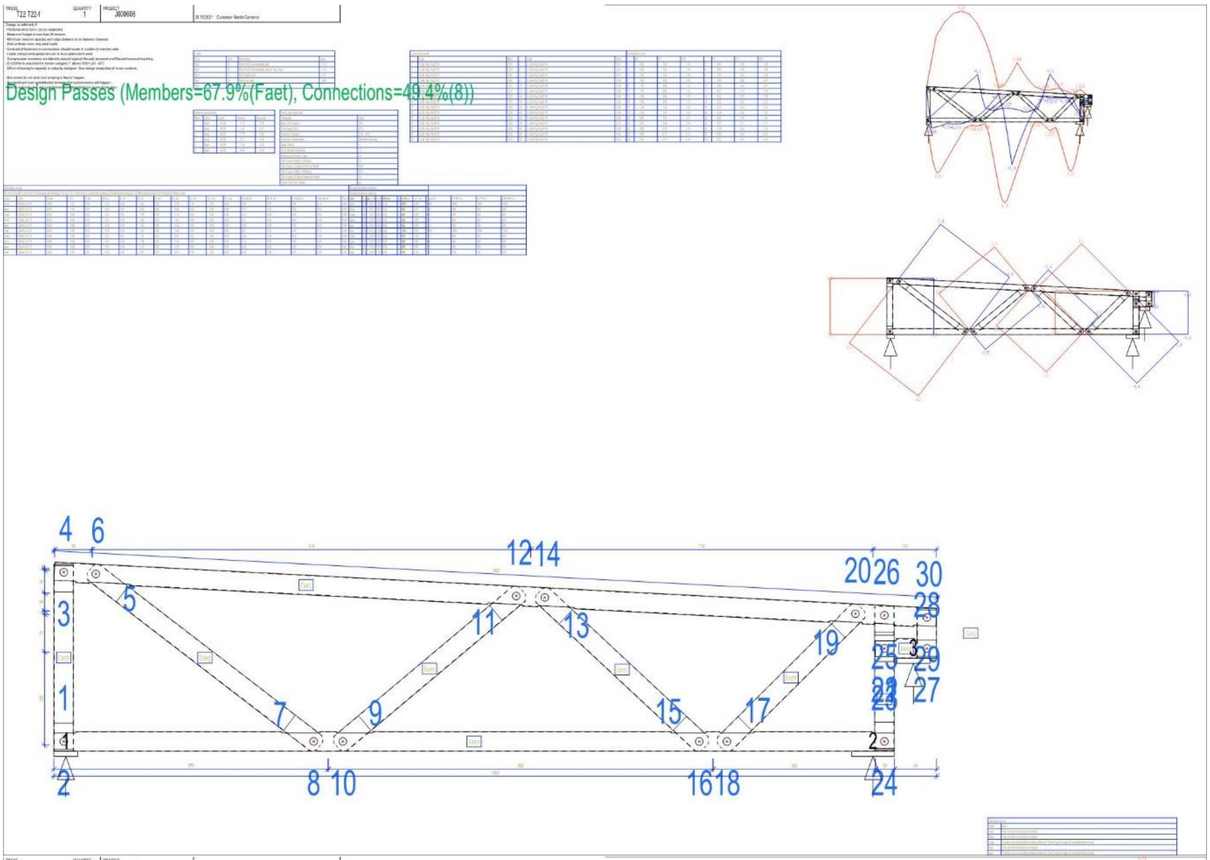

T19 T19-b
J606686
Design Passes (Members=65.3%(Cvh), Connections=60.9%(27))



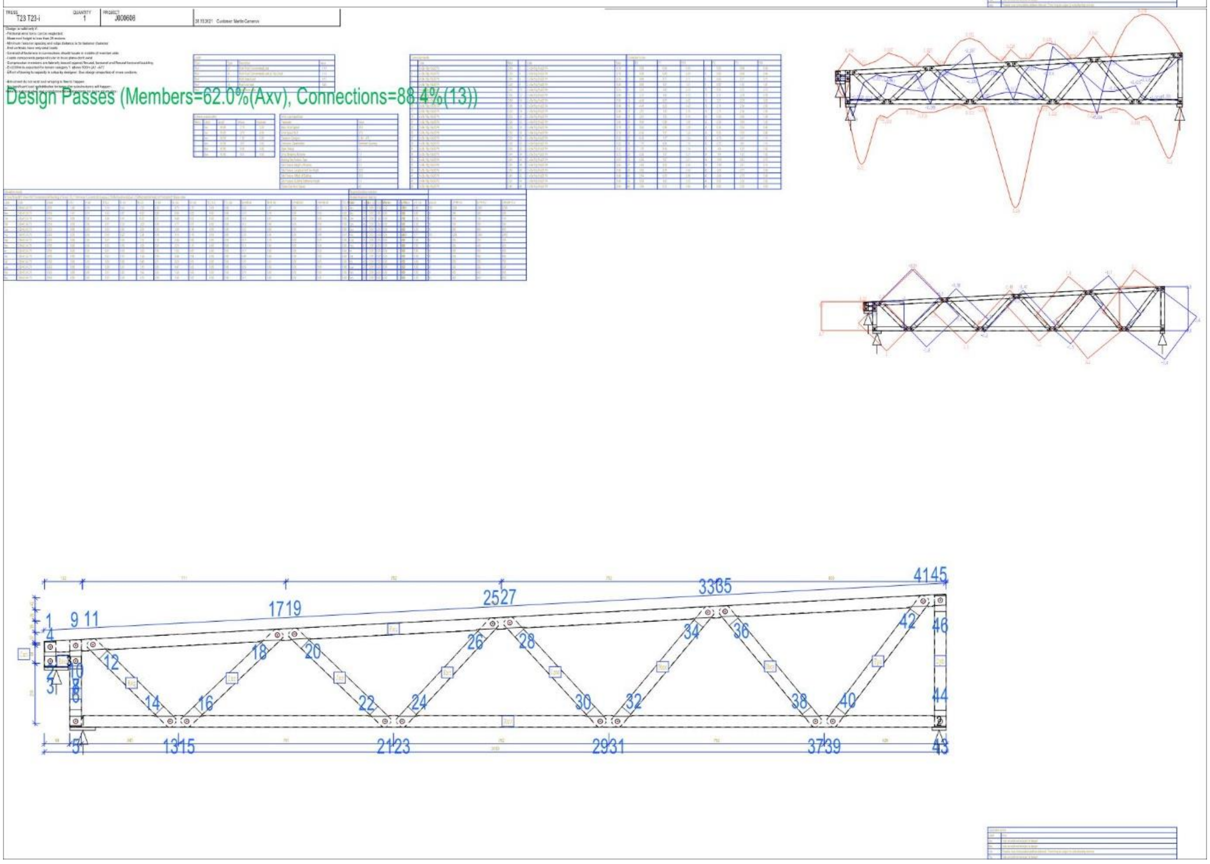
Design Passes (Members=69.7%(Faal), Connections=50.7%(9))



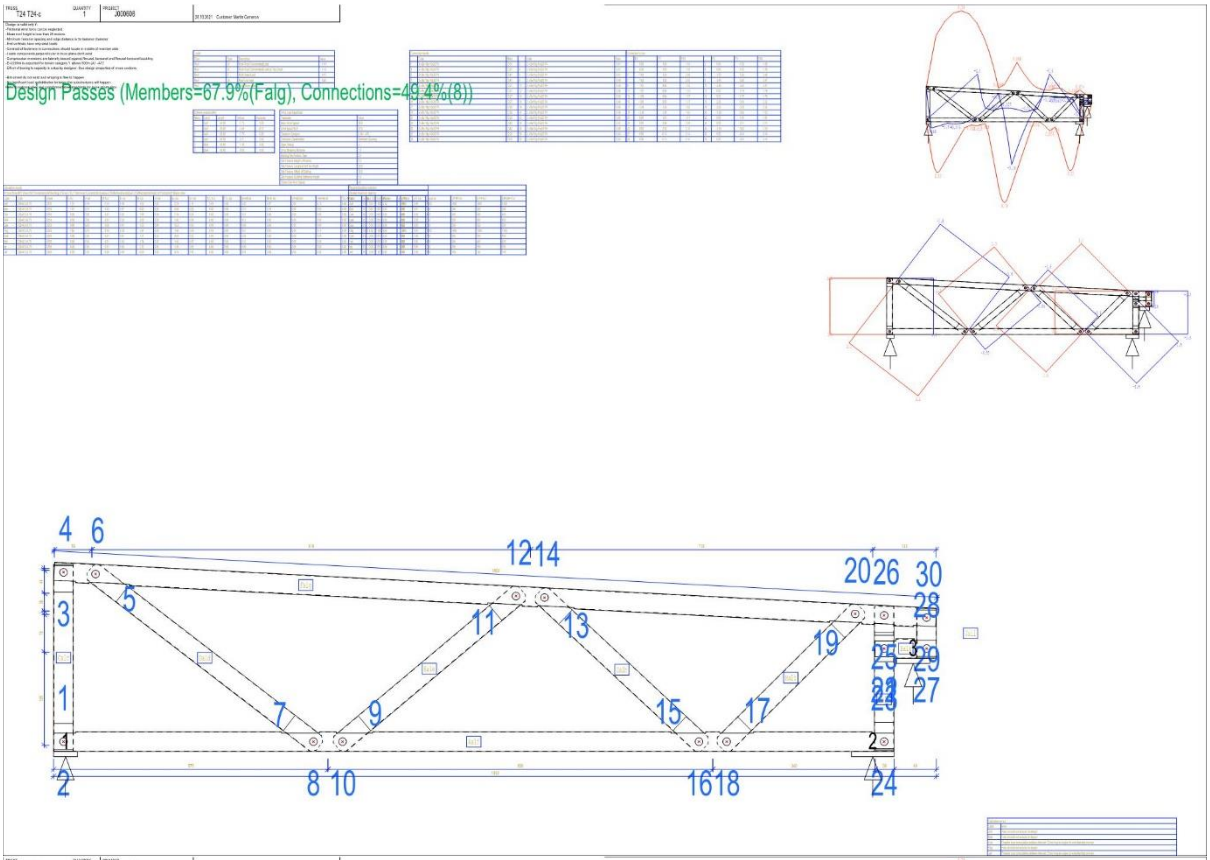
Design Passes (Members=67.9%(Fagx), Connections=49.4%(8))



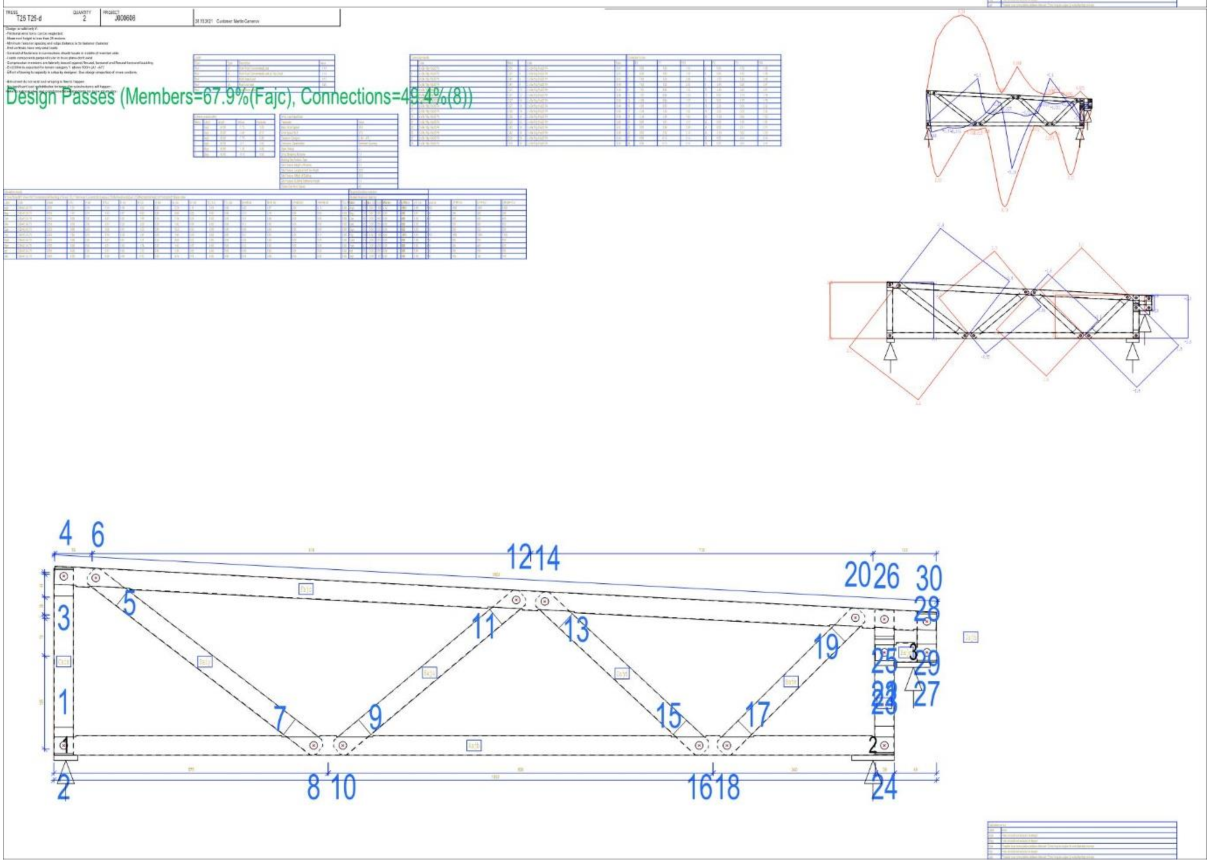
T22 T22-I
J606686
Design Passes (Members=67.9%(Faet), Connections=49.4%(8))



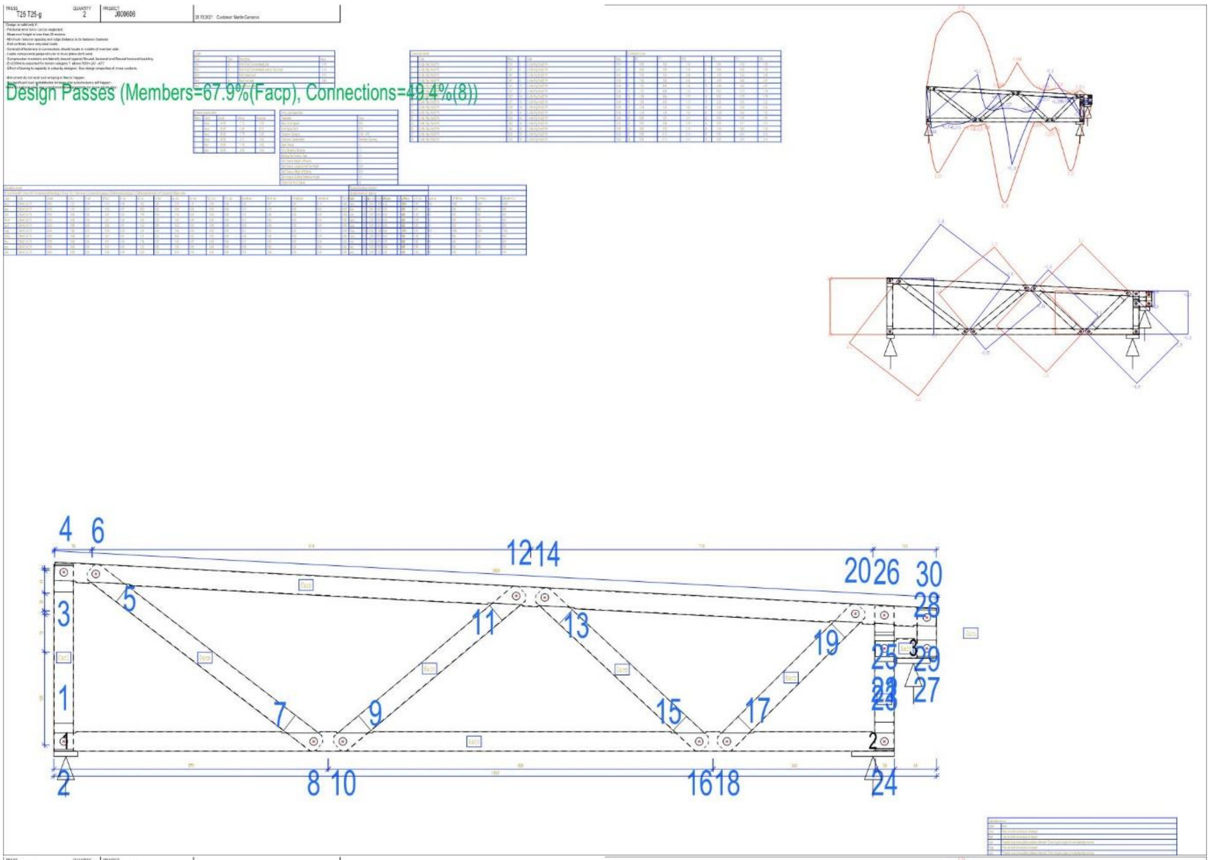
T23 T23-I
J606686
Design Passes (Members=62.0%(Axv), Connections=88.4%(13))



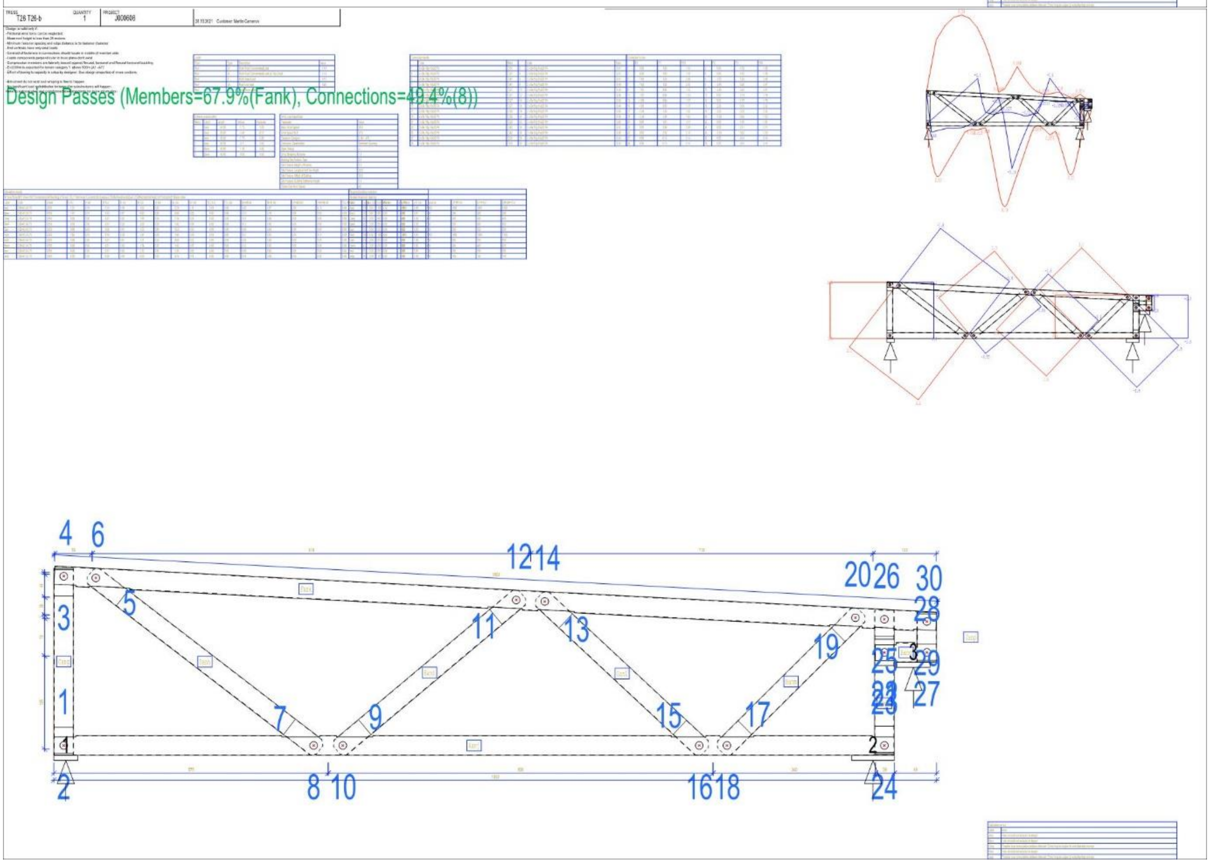
TRUSS T24 T24-c
QUANTITY 1
PROJECT J000606
Customer Martin Cameron
Design Passes (Members=67.9%(Falg), Connections=49.4%(8))



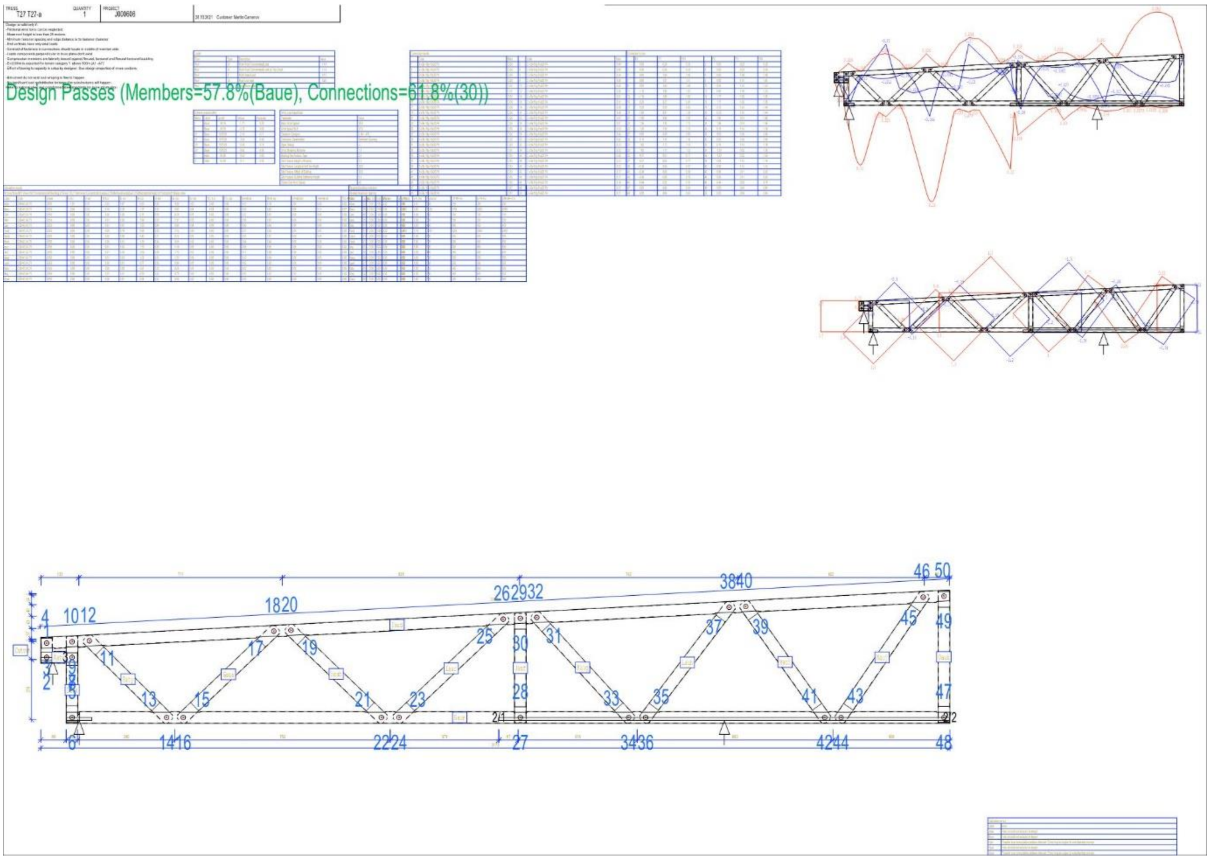
TRUSS T25 T25-d
QUANTITY 2
PROJECT J000606
Customer Martin Cameron
Design Passes (Members=67.9%(Fajc), Connections=49.4%(8))



Design Passes (Members=67.9%(Facp), Connections=49.4%(8))



Design Passes (Members=67.9%(Fank), Connections=49.4%(8))



Design Passes (Members=57.8%(Baue), Connections=61.8%(30))

Beam Label:	**RB1**	Location:	**Dinning Stacker**	**Bending**	**OK**
Span (L):	**8.1m**	Section:	**380 PFC 55.2**	**Shear**	**OK**
		Fire Rated:	**N/A**	**Deflection**	**OK**
				Limited Temp	**NA**

W = **5.7kN/m**

8.1m

Max	23.3kN	=R*	R* =	23.3kN	Max
Min	-23.0kN			-23.0kN	Min

Loadings: SW = 0.54 kN/m (Tributy Area)

Roof Type A	Dead	=	0.27	kPa	x	5.9m	=	1.6	kN/m	
	Live	=	0.25	kPa	x	5.9m	=	1.5	kN/m	
	Wind, up	=	-1.29	kPa	x	5.9m	=	-7.6	kN/m	
	Wind, down	=	0.54	kPa	x	5.9m	=	3.2	kN/m	
	Snow	=		kPa	x		=	0.0	kN/m	
Wall Type B	Dead	=		kPa	x		=	0.0	kN/m	Live load factors:
	Live	=		kPa	x		=	0.0	kN/m	Ψ,c = 0.4
Floor Type A	Dead	=		kPa	x		=	0.0	kN/m	Ψ,l = 0.4
	Live	=		kPa	x		=	0.0	kN/m	Ψ,s = 0.7

Load Cases:

Ultimate Limit State:			(w)		Serviceability Limit State:			(w)	
1.	1.2G + 1.5 Q	=	4.8	kN/m	6.	G + Ψ,l Q	=	2.1	kN/m
2.	0.9G + Wu,up	=	-5.7	kN/m	7.	G + Ψ,s Q	=	3.2	kN/m
3.	1.2G+Wu,dw+ΨcQ	=	5.7	kN/m	8.	Ws	=	5.1	kN/m
4.	1.2G+Su+ΨcQ	=	2.6	kN/m	9.	1kN Vibration			
5.	1.35G	=	2.9	kN/m					

Design Actions:

Cases:		1.	2.	3.	4.	5.	**Max**	**Min**
Moment, M*= W x L^2/	=	39.2	-46.7	47.1	21.0	23.6	**47.1kNm**	**-46.7kNm**
Shear, V* = W x L /2	=	19.3	-23.0	23.3	10.4	11.7	**23.3kN**	**-23.0kN**

Capacity:

Restraint Conditions:	(LL) Lateral-Lateral		Comments:			
Load Height Position:	Shear Centre		Restrains provided by floor joists at 600crs			
Segment Length L =	0.6	m				
Bending: ØMsx =	238.4	kNm	Shear:	ØVw =	591.0	kN
ØMbx =	334.5	kNm	Caqpacity Ratio	=	0.0	OK
Min (ØMsx, ØMbx) =	238.4	kNm				
Caqpacity Ratio =	0.2	OK				

Deflection:

	G + Ψ,l Q =	4mm	=	Span/2058	
Beams where line-of-sight is across soffit, aceess by operators and mainten		Span/250	Y	OK	
	Ws =	9mm	=	Span/862	
Ceilings with plaster finish		Span/200	Y	OK	
Lintel Beam (vertical sag), <12mm		Span/240	Y	OK	

Beam Label:	RB2	Location:	Kitchen Stacker	Bending	OK
Span (L):	8.1m	Section:	380 PFC 55.2	Shear	OK
		Fire Rated:	N/A	Deflection	OK
				Limited Temp	NA

W = 5.2kN/m

8.1m

Max	21.2kN	=R*		R* =	21.2kN	Max
Min	-20.5kN				-20.5kN	Min

Loadings: SW = 0.54 kN/m (Tributy Area)

Roof Type A	Dead	=	0.27	kPa	x	5.3m	=	1.4	kN/m			
	Live	=	0.25	kPa	x	5.3m	=	1.3	kN/m			
	Wind, up	=	-1.29	kPa	x	5.3m	=	-6.8	kN/m			
	Wind, down	=	0.54	kPa	x	5.3m	=	2.9	kN/m			
	Snow	=		kPa	x		=	0.0	kN/m			
Wall Type B	Dead	=		kPa	x		=	0.0	kN/m	Live load factors:		
	Live	=		kPa	x		=	0.0	kN/m	Ψ,c	=	0.4
Floor Type A	Dead	=		kPa	x		=	0.0	kN/m	Ψ,l	=	0.4
	Live	=		kPa	x		=	0.0	kN/m	Ψ,s	=	0.7

Load Cases:

Ultimate Limit State:			(w)		Serviceability Limit State:			(w)	
1.	1.2G + 1.5 Q	=	4.4	kN/m	6.	G + Ψ,l Q	=	2.0	kN/m
2.	0.9G + Wu,up	=	-5.1	kN/m	7.	G + Ψ,s Q	=	2.9	kN/m
3.	1.2G+Wu,dw+ΨcQ	=	5.2	kN/m	8.	Ws	=	4.6	kN/m
4.	1.2G+Su+ΨcQ	=	2.4	kN/m	9.	1kN Vibration			
5.	1.35G	=	2.7	kN/m					

Design Actions:

Cases:		1.	2.	3.	4.	5.	Max	Min
Moment, M*= W x L^2/	=	35.7	-41.5	42.9	19.4	21.8	**42.9kNm**	**-41.5kNm**
Shear, V* = W x L /2	=	17.6	-20.5	21.2	9.6	10.8	**21.2kN**	**-20.5kN**

Capacity:

Restraint Conditions:		(LL) Lateral-Lateral		Comments:			
Load Height Position:		Shear Centre		Restrains provided by floor joists at 600crs			
Segment Length	L =	0.6	m				
Bending: ØMsx	=	238.4	kNm	Shear: ØVw	=	591.0	kN
ØMbx	=	334.5	kNm	Caqpacity Ratio	=	0.0	OK
Min (ØMsx, ØMbx) =		238.4	kNm				
Caqpacity Ratio	=	0.2	OK				

Deflection:

	G + Ψ,l Q =	4mm	=	Span/2227	
Beams where line-of-sight is across soffit, aceess by operators and mainten		Span/250	Y	OK	
	Ws =	8mm	=	Span/959	
Ceilings with plaster finish		Span/200	Y	OK	
Lintel Beam (vertical sag), <12mm		Span/240	Y	OK	

Beam Label:	**RB3**	Location:	**Kitchen Stacker**	**Bending**	**OK**
Span (L):	**9.8m**	Section:	**250 PFC 35.5**	**Shear**	**OK**
		Fire Rated:	**N/A**	**Deflection**	**OK**
				Limited Temp	**NA**

W = **-2.7kN/m**

9.8m

Max	10.2kN	=R*	R* =	10.2kN	Max
Min	-13.3kN			-13.3kN	Min

Loadings: SW = 0.35 kN/m (Tributy Area)

Roof Type A	Dead	=	0.27	kPa	x	2.2m	=	0.6	kN/m
	Live	=	0.25	kPa	x	2.2m	=	0.6	kN/m
	Wind, up	=	-1.62	kPa	x	2.2m	=	-3.6	kN/m
	Wind, down	=	0.43	kPa	x	2.2m	=	0.9	kN/m
	Snow	=		kPa	x		=	0.0	kN/m
Wall Type B	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m
Floor Type A	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m

Live load factors:		
Ψ,c	=	0.4
Ψ,l	=	0.4
Ψ,s	=	0.7

Load Cases:

Ultimate Limit State:		(w)		Serviceability Limit State:		(w)	
1.	1.2G + 1.5 Q =	2.0	kN/m	6.	G + Ψ,l Q =	0.9	kN/m
2.	0.9G + Wu,up =	-2.7	kN/m	7.	G + Ψ,s Q =	1.3	kN/m
3.	1.2G+Wu,dw+ΨcQ =	2.1	kN/m	8.	Ws =	2.4	kN/m
4.	1.2G+Su+ΨcQ =	1.1	kN/m	9.	1kN Vibration		
5.	1.35G =	1.3	kN/m				

Design Actions:

Cases:	1.	2.	3.	4.	5.	**Max**	**Min**
Moment, M*= W x L^2/ =	23.5	-32.6	24.9	13.6	15.3	**24.9kNm**	**-32.6kNm**
Shear, V* = W x L /2 =	9.6	-13.3	10.2	5.5	6.2	**10.2kN**	**-13.3kN**

Capacity:

Restraint Conditions: (LL) Lateral-Lateral

Load Height Position: Shear Centre

Segment Length L = 0.6 m

Comments: Restrains provided by floor joists at 600crs

Bending: ØMsx =	113.7	kNm	Shear: ØVw =	311.0	kN		
ØMbx =	158.3	kNm	Caqpacity Ratio =	0.0	OK		
Min (ØMsx, ØMbx) =	113.7	kNm					
Caqpacity Ratio =	0.3	OK					

Deflection:

G + Ψ,l Q =	13mm	=	Span/781	
Beams where line-of-sight is across soffit, aceess by operators and mainten	Span/250	Y	OK	
Ws =	32mm	=	Span/308	
Ceilings with plaster finish	Span/200	Y	OK	
Lintel Beam (vertical sag), <12mm	Span/240	N/A	N/A	

Beam Label:	**RB4**	Location:	**Kitchen Stacker**	**Bending**	**OK**
Span (L):	**9.8m**	Section:	**230 PFC 25.1**	**Shear**	**OK**
		Fire Rated:	**N/A**	**Deflection**	**OK**
				Limited Temp	**NA**

W = **-2.0kN/m**

9.8m

Max	7.4kN	=R*	R* =	7.4kN	Max
Min	-9.7kN			-9.7kN	Min

Loadings: SW = 0.25 kN/m (Tributy Area)

Roof Type A	Dead	=	0.27	kPa	x	1.6m	=	0.4	kN/m
	Live	=	0.25	kPa	x	1.6m	=	0.4	kN/m
	Wind, up	=	-1.62	kPa	x	1.6m	=	-2.6	kN/m
	Wind, down	=	0.43	kPa	x	1.6m	=	0.7	kN/m
	Snow	=		kPa	x		=	0.0	kN/m
Wall Type B	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m
Floor Type A	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m

Live load factors:		
Ψ,c	=	0.4
Ψ,l	=	0.4
Ψ,s	=	0.7

Load Cases:

Ultimate Limit State:			(w)		Serviceability Limit State:			(w)	
1.	1.2G + 1.5 Q	=	1.4	kN/m	6.	G + Ψ,l Q	=	0.7	kN/m
2.	0.9G + Wu,up	=	-2.0	kN/m	7.	G + Ψ,s Q	=	1.0	kN/m
3.	1.2G+Wu,dw+ΨcQ	=	1.5	kN/m	8.	Ws	=	1.7	kN/m
4.	1.2G+Su+ΨcQ	=	0.8	kN/m	9.	1kN Vibration			
5.	1.35G	=	0.9	kN/m					

Design Actions:

Cases:		1.	2.	3.	4.	5.	**Max**	**Min**
Moment, M*= W x L^2/	=	17.0	-23.8	18.0	9.8	11.0	**18.0kNm**	**-23.8kNm**
Shear, V* = W x L /2	=	6.9	-9.7	7.4	4.0	4.5	**7.4kN**	**-9.7kN**

Capacity:

Restraint Conditions: (LL) Lateral-Lateral

Load Height Position: Shear Centre

Segment Length L = 0.6 m

Comments: Restrains provided by floor joists at 600crs

Bending:	ØMsx	=	73.2	kNm	Shear:	ØVw	=	232.5 kN
	ØMbx	=	100.4	kNm	Caqpacity Ratio		=	0.0 OK
Min (ØMsx, ØMbx) =			73.2	kNm				
Caqpacity Ratio		=	0.3	OK				

Deflection:

G + Ψ,l Q =	15mm	=	Span/645	
Beams where line-of-sight is across soffit, aceess by operators and mainten	Span/250	Y	OK	
Ws =	39mm	=	Span/252	
Ceilings with plaster finish	Span/200	Y	OK	
Lintel Beam (vertical sag), <12mm	Span/240	N/A	N/A	

Beam Label:	RB5	Location:	Bed 1 Veranda	Bending	OK
Span (L):	5.0m	Beam Section:	LC190x45x15x1.55	Shear	OK
			Double (Boxed or BTB)	Combined	OK
				Deflection	OK

Total Span (L) = 5m

Max	2.7kN	=RA*	RB* =	2.7kN	Max
Min	-4.8kN			-4.8kN	Min

Loadings: sw = 0.01kN/m

Roof	Dead	=	0.27	kPa	x	1.4m	=	0.4	kN/m
	Live	=	0.25	kPa	x	1.4m	=	0.4	kN/m
	Wind, up	=	-1.62	kPa	x	1.4m	=	-2.3	kN/m
	Wind, down	=	0.43	kPa	x	1.4m	=	0.6	kN/m
	Snow	=		kPa	x		=	0.0	kN/m
Wall	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m
Floor	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m

Live load factors:		
Ψ,c	=	0.4
Ψ,l	=	0.7
Ψ,s	=	0.7

Load Cases:

Ultimate Limit State:			(w)		Serviceability Limit State:			(w)	
1.	1.2G + 1.5 Q	=	1.0	kN/m	6.	G + Ψ,l Q	=	0.4	kN/m
2.	0.9G + Wu,up	=	-1.9	kN/m	7.	G + Ψ,s Q	=	0.6	kN/m
3.	1.2G+Wu,dw+ΨcQ	=	1.1	kN/m	8.	Ws	=	1.5	kN/m
4.	1.2G+Su+ΨcQ	=	0.5	kN/m	9.	1kN Vibration			
5.	1.35G	=	0.5	kN/m					

Design Actions:

Cases:		1.	2.	3.	4.	5.	Max	Min
Moment, M*= W x L^2/	=	3.1	-6.0	3.3	1.5	1.6	**3.3kNm**	**-6.0kNm**
Shear, V* = W x L /2	=	2.5	-4.8	2.7	1.2	1.3	**2.7kN**	**-4.8kN**

Capacity:

Bending:

Cb =	1.0	*(eq D2.1.1(2))*		Mo =	15.1	kNm	*(D2.1 Appendix D)*
Ley =	900	mm	*eff. length for buckling about y-axes*	Mol =	9.7	kNm	*eq. 7.2.2.3(4)*
Lez =	900	mm	*eff. length for twisting*	Mod =	9.7	kNm	*eq. 7.2.2.4(4)*
ϕ_b =	0.9	*Clause 7.2.2.1*		**ØMb =**	**ØMin(Mbe, Mbl, Mbd)**		
fy =	550	Mpa		=	**10.0**	kNm	
			Capacity Ratio	=	**0.6**	OK	

Shear:

a =	5000	mm	*shear panel length*
d1 =	187.70	mm	*depth of web*
kv =	6.30	*eq. D3(4)*	
ØVv =	**41.1**	kN	OK

Combined bending and shear:

Maximum bending and shear at same location ? No

$(M^*/\phi_b M_s)^2+(V^*/\phi_v V_v)^2$ = **0.21** N/A

Deflection:

Limits: **I,efftive = 4.55 x10^6mm4**

	G + Ψ,l Q =	3mm	=	Span/1437	
Roof members (trusses, rafters, etc)		Span/300		Y	OK
	Ws =	14mm	=	Span/368	
Ceilings with plaster finish		Span/200		Y	OK
Lintel Beam (vertical sag), <12mm		Span/240		N/A	N/A

Beam Label:	**RB6**	Location:	**Bed 2&3 Veranda**	**Bending**	**OK**
Span (L):	**3.6m**	Beam Section:	**LC190x45x15x1.55**	**Shear**	**OK**
			Single Member	**Combined**	**OK**
				Deflection	**OK**

Total Span (L) = 3.6m

Max	1.2kN	=RA*	RB* =	1.2kN	Max
Min	-2.2kN			-2.2kN	Min

Loadings: sw = 0.01kN/m

Roof	Dead	=	0.27	kPa	x	0.9m	=	0.2	kN/m
	Live	=	0.25	kPa	x	0.9m	=	0.2	kN/m
	Wind, up	=	-1.62	kPa	x	0.9m	=	-1.5	kN/m
	Wind, down	=	0.43	kPa	x	0.9m	=	0.4	kN/m
	Snow	=		kPa	x		=	0.0	kN/m
Wall	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m
Floor	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m

Live load factors:		
Ψ,c	=	0.4
Ψ,l	=	0.7
Ψ,s	=	0.7

Load Cases:

Ultimate Limit State:		(w)		Serviceability Limit State:		(w)	
1. 1.2G + 1.5 Q	=	0.6	kN/m	6. G + Ψ,l Q	=	0.3	kN/m
2. 0.9G + Wu,up	=	-1.2	kN/m	7. G + Ψ,s Q	=	0.4	kN/m
3. 1.2G+Wu,dw+ΨcQ	=	0.7	kN/m	8. Ws	=	1.0	kN/m
4. 1.2G+Su+ΨcQ	=	0.3	kN/m	9. 1kN Vibration			
5. 1.35G	=	0.3	kN/m				

Design Actions:

Cases:		1.	2.	3.	4.	5.	Max	Min
Moment, M*= W x L^2/	=	1.0	-2.0	1.1	0.5	0.6	**1.1kNm**	**-2.0kNm**
Shear, V* = W x L /2	=	1.2	-2.2	1.2	0.5	0.6	**1.2kN**	**-2.2kN**

Capacity:

Bending:

Cb =	1.0	*(eq D2.1.1(2))*		Mo =	15.1	kNm	*(D2.1 Appendix D)*
Ley =	900	mm	*eff. length for buckling about y-axes*	Mol =	9.7	kNm	*eq. 7.2.2.3(4)*
Lez =	900	mm	*eff. length for twisting*	Mod =	9.7	kNm	*eq. 7.2.2.4(4)*
$Ø_b$ =	0.9	*Clause 7.2.2.1*		**ØMb =**	**ØMin(Mbe, Mbl, Mbd)**		
fy =	550	Mpa		=	**5.0**	**kNm**	
				Capacity Ratio =	**0.4**	OK	

Shear:

a =	3600	mm	*shear panel length*
d1 =	187.70	mm	*depth of web*
kv =	6.30	*eq. D3(4)*	
ØVv =	**20.7**	**kN**	OK

Combined bending and shear:

Maximum bending and shear at same location ? No

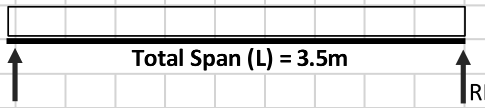
$(M^*/Ø_bMs)^2+(V^*/ØvVv)^2$ = **0.08** N/A

Deflection:

Limits: **I,efftive** = **2.32** **x10^6mm4**

	G + Ψ,l Q =	1mm	=	Span/3008	
Roof members (trusses, rafters, etc)		Span/300		Y	OK
	Ws =	5mm	=	Span/782	
Ceilings with plaster finish		Span/200		Y	OK
Lintel Beam (vertical sag), <12mm		Span/240		N/A	N/A

Beam Label:	**RB7**	Location:	**Bed 1 Stacker**	**Bending**	**OK**
Span (L):	**3.5m**	Beam Section:	**LC190x45x15x1.55**	**Shear**	**OK**
			Single Member	**Combined**	**OK**
				Deflection	**OK**



Max	4.6kN	=RA*	RB* =	4.6kN	Max
Min	-5.5kN			-5.5kN	Min

Loadings: sw = 0.01kN/m

Roof	Dead	=	0.27	kPa	x	3.0m	=	0.8	kN/m
	Live	=	0.25	kPa	x	3.0m	=	0.8	kN/m
	Wind, up	=	-1.29	kPa	x	3.0m	=	-3.9	kN/m
	Wind, down	=	0.54	kPa	x	3.0m	=	1.6	kN/m
	Snow	=		kPa	x		=	0.0	kN/m
Wall	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m
Floor	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m

Live load factors:		
Ψ,c	=	0.4
Ψ,l	=	0.7
Ψ,s	=	0.7

Load Cases:

Ultimate Limit State:			(w)		Serviceability Limit State:			(w)	
1.	1.2G + 1.5 Q	=	2.1	kN/m	6.	G + Ψ,l Q	=	0.8	kN/m
2.	0.9G + Wu,up	=	-3.1	kN/m	7.	G + Ψ,s Q	=	1.3	kN/m
3.	1.2G+Wu,dw+ΨcQ	=	2.6	kN/m	8.	Ws	=	2.6	kN/m
4.	1.2G+Su+ΨcQ	=	1.0	kN/m	9.	1kN Vibration			
5.	1.35G	=	1.1	kN/m					

Design Actions:

Cases:		1.	2.	3.	4.	5.	**Max**	**Min**
Moment, M*= W x L^2/	=	3.2	-4.8	4.0	1.5	1.7	**4.0kNm**	**-4.8kNm**
Shear, V* = W x L /2	=	3.7	-5.5	4.6	1.7	1.9	**4.6kN**	**-5.5kN**

Capacity:

Bending:

Cb =	1.0	*(eq D2.1.1(2))*		Mo =	15.1	kNm	*(D2.1 Appendix D)*
Ley =	900	mm	*eff. length for buckling about y-axes*	Mol =	9.7	kNm	*eq. 7.2.2.3(4)*
Lez =	900	mm	*eff. length for twisting*	Mod =	9.7	kNm	*eq. 7.2.2.4(4)*
$Ø_b$ =	0.9	*Clause 7.2.2.1*		**ØMb =**	**ØMin(Mbe, Mbl, Mbd)**		
fy =	550	Mpa		=	**5.0**	**kNm**	
			Capacity Ratio	=	**1.0**	OK	

Shear:

a =	3500	mm	*shear panel length*
d1 =	187.70	mm	*depth of web*
kv =	6.30	*eq. D3(4)*	
ØVv =	**20.7**	**kN**	OK

Combined bending and shear:

Maximum bending and shear at same location ? No

$(M^*/Ø_bMs)^2+(V^*/ØvVv)^2$ = **0.23** N/A

Deflection:

Limits: **I,efftive** = **2.32** **x10^6mm4**

	G + Ψ,l Q =	3mm	=	Span/1012	
Roof members (trusses, rafters, etc)		Span/300		Y	OK
	Ws =	11mm	=	Span/321	
Ceilings with plaster finish		Span/200		Y	OK
Lintel Beam (vertical sag), <12mm		Span/240		Y	OK

Beam Label:	**RB8**	Location:	**Bed 2&3 Stacker**
Span (L):	**1.9m**	Beam Section:	**LC150x41x11x0.75**
			Single Member

Bending	**OK**
Shear	**OK**
Combined	**OK**
Deflection	**OK**

Total Span (L) = 1.9m

Max	2.4kN	=RA*	RB* =	2.4kN	Max
Min	-2.9kN			-2.9kN	Min

Loadings: sw = 0.01kN/m

Roof	Dead	=	0.27	kPa	x	2.9m	=	0.8	kN/m
	Live	=	0.25	kPa	x	2.9m	=	0.7	kN/m
	Wind, up	=	-1.29	kPa	x	2.9m	=	-3.7	kN/m
	Wind, down	=	0.54	kPa	x	2.9m	=	1.6	kN/m
	Snow	=		kPa	x		=	0.0	kN/m
Wall	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m
Floor	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m

Live load factors:		
Ψ,c	=	0.4
Ψ,l	=	0.7
Ψ,s	=	0.7

Load Cases:

Ultimate Limit State:			(w)		Serviceability Limit State:			(w)	
1.	1.2G + 1.5 Q	=	2.0	kN/m	6.	G + Ψ,l Q	=	0.8	kN/m
2.	0.9G + Wu,up	=	-3.0	kN/m	7.	G + Ψ,s Q	=	1.3	kN/m
3.	1.2G+Wu,dw+ΨcQ	=	2.5	kN/m	8.	Ws	=	2.5	kN/m
4.	1.2G+Su+ΨcQ	=	1.0	kN/m	9.	1kN Vibration			
5.	1.35G	=	1.1	kN/m					

Design Actions:

Cases:		1.	2.	3.	4.	5.	**Max**	**Min**
Moment, M*= W x L^2/	=	0.9	-1.4	1.1	0.4	0.5	**1.1kNm**	**-1.4kNm**
Shear, V* = W x L /2	=	1.9	-2.9	2.4	0.9	1.0	**2.4kN**	**-2.9kN**

Capacity:

Bending:

Cb =	1.0	*(eq D2.1.1(2))*		Mo =	15.1	kNm	*(D2.1 Appendix D)*
Ley =	900	mm	*eff. length for buckling about y-axes*	Mol =	1.2	kNm	*eq. 7.2.2.3(4)*
Lez =	900	mm	*eff. length for twisting*	Mod =	1.8	kNm	*eq. 7.2.2.4(4)*
$Ø_b$ =	0.9	*Clause 7.2.2.1*			**ØMb = ØMin(Mbe, Mbl, Mbd)**		
fy =	550	Mpa		=	**2.0**	**kNm**	
			Capacity Ratio	=	**0.7**	OK	

Shear:

a =	1900	mm	*shear panel length*
d1 =	148.50	mm	*depth of web*
kv =	6.30	*eq. D3(4)*	
ØVv =	**3.0**	**kN**	OK

Combined bending and shear:

Maximum bending and shear at same location ? No

$(M^*/ØbMs)^2+(V^*/ØvVv)^2$ = **1.00** N/A

Deflection:

Limits: **I,efftive = 0.53 x10^6mm4**

	G + Ψ,l Q =	1mm	=	Span/1481
Roof members (trusses, rafters, etc)		Span/300	Y	OK
	Ws =	4mm	=	Span/469
Ceilings with plaster finish		Span/200	Y	OK
Lintel Beam (vertical sag), <12mm		Span/240	Y	OK

Beam Label:	**RB9**	Location:	**Bed 2&3 Door**	**Bending**	**OK**
Span (L):	**2.2m**	Beam Section:	**LC150x46x15x1.6**	**Shear**	**OK**
			Single Member	**Combined**	**OK**
				Deflection	**OK**

Total Span (L) = 2.2m

Max	5.7kN	=RA*	RB* =	5.7kN	Max
Min	-6.9kN			-6.9kN	Min

Loadings: sw = 0.01kN/m

Roof	Dead	=	0.27	kPa	x	6.0m	=	1.6	kN/m
	Live	=	0.25	kPa	x	6.0m	=	1.5	kN/m
	Wind, up	=	-1.29	kPa	x	6.0m	=	-7.7	kN/m
	Wind, down	=	0.54	kPa	x	6.0m	=	3.2	kN/m
	Snow	=		kPa	x		=	0.0	kN/m
Wall	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m
Floor	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m

Live load factors:		
Ψ,c	=	0.4
Ψ,l	=	0.7
Ψ,s	=	0.7

Load Cases:

Ultimate Limit State:			(w)		Serviceability Limit State:			(w)	
1.	1.2G + 1.5 Q	=	4.2	kN/m	6.	G + Ψ,l Q	=	1.6	kN/m
2.	0.9G + Wu,up	=	-6.3	kN/m	7.	G + Ψ,s Q	=	2.7	kN/m
3.	1.2G+Wu,dw+ΨcQ	=	5.2	kN/m	8.	Ws	=	5.2	kN/m
4.	1.2G+Su+ΨcQ	=	2.0	kN/m	9.	1kN Vibration			
5.	1.35G	=	2.2	kN/m					

Design Actions:

Cases:		1.	2.	3.	4.	5.	**Max**	**Min**
Moment, M*= W x L^2/	=	2.5	-3.8	3.1	1.2	1.3	**3.1kNm**	**-3.8kNm**
Shear, V* = W x L /2	=	4.6	-6.9	5.7	2.2	2.4	**5.7kN**	**-6.9kN**

Capacity:

Bending:

Cb =	1.0	*(eq D2.1.1(2))*		Mo =	15.1	kNm	*(D2.1 Appendix D)*
Ley =	900	mm	*eff. length for buckling about y-axes*	Mol =	11.5	kNm	*eq. 7.2.2.3(4)*
Lez =	900	mm	*eff. length for twisting*	Mod =	11.5	kNm	*eq. 7.2.2.4(4)*
$Ø_b$ =	0.9	*Clause 7.2.2.1*		**ØMb =**	**ØMin(Mbe, Mbl, Mbd)**		
fy =	550	Mpa		=	**4.0**	**kNm**	
				Capacity Ratio =	**1.0**	OK	

Shear:

a =	2200	mm	*shear panel length*
d1 =	146.00	mm	*depth of web*
kv =	6.30	*eq. D3(4)*	
ØVv =	**25.4**	**kN**	OK

Combined bending and shear:

Maximum bending and shear at same location ? No

$(M^*/Ø_bM_s)^2+(V^*/Ø_vV_v)^2$ = **0.32** N/A

Deflection:

Limits: **I,efftive** = **1.32** **x10^6mm4**

	G + Ψ,l Q =	2mm	=	Span/1167	
Roof members (trusses, rafters, etc)		Span/300		Y	OK
	Ws =	6mm	=	Span/367	
Ceilings with plaster finish		Span/200		Y	OK
Lintel Beam (vertical sag), <12mm		Span/240		Y	OK

Beam Label:	**RB10**	Location:	**Toilet**	**Bending**	**OK**
Span (L):	**1.7m**	Beam Section:	**LC89x40x11x0.75**	**Shear**	**OK**
			Single Member	**Combined**	**OK**
				Deflection	**OK**

Total Span (L) = 1.7m

Max	0.9kN	=RA*	RB* =	0.9kN	Max
Min	-1.1kN			-1.1kN	Min

Loadings: sw = 0.01kN/m

Roof	Dead	=	0.27	kPa	x	1.2m	=	0.3	kN/m
	Live	=	0.25	kPa	x	1.2m	=	0.3	kN/m
	Wind, up	=	-1.29	kPa	x	1.2m	=	-1.5	kN/m
	Wind, down	=	0.54	kPa	x	1.2m	=	0.6	kN/m
	Snow	=		kPa	x		=	0.0	kN/m
Wall	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m
Floor	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m

Live load factors:		
Ψ,c	=	0.4
Ψ,l	=	0.7
Ψ,s	=	0.7

Load Cases:

Ultimate Limit State:			(w)		Serviceability Limit State:			(w)	
1.	1.2G + 1.5 Q	=	0.9	kN/m	6.	G + Ψ,l Q	=	0.3	kN/m
2.	0.9G + Wu,up	=	-1.2	kN/m	7.	G + Ψ,s Q	=	0.5	kN/m
3.	1.2G+Wu,dw+ΨcQ	=	1.0	kN/m	8.	Ws	=	1.0	kN/m
4.	1.2G+Su+ΨcQ	=	0.4	kN/m	9.	1kN Vibration			
5.	1.35G	=	0.5	kN/m					

Design Actions:

Cases:		1.	2.	3.	4.	5.	**Max**	**Min**
Moment, M*= W x L^2/	=	0.3	-0.5	0.4	0.1	0.2	**0.4kNm**	**-0.5kNm**
Shear, V* = W x L /2	=	0.7	-1.1	0.9	0.3	0.4	**0.9kN**	**-1.1kN**

Capacity:

Bending:

Cb =	1.0		*(eq D2.1.1(2))*	Mo =	15.1	kNm	*(D2.1 Appendix D)*
Ley =	900	mm	*eff. length for buckling about y-axes*	Mol =	1.2	kNm	*eq. 7.2.2.3(4)*
Lez =	900	mm	*eff. length for twisting*	Mod =	1.2	kNm	*eq. 7.2.2.4(4)*
$Ø_b$ =	0.9		*Clause 7.2.2.1*	**ØMb =**	**ØMin(Mbe, Mbl, Mbd)**		
fy =	550	Mpa		=	**1.2**	**kNm**	
				Capacity Ratio =	**0.4**	OK	

Shear:

a =	1700	mm	*shear panel length*
d1 =	87.00	mm	*depth of web*
kv =	6.30		*eq. D3(4)*
ØVv =	**5.1**	**kN**	OK

Combined bending and shear:

Maximum bending and shear at same location ? No

$(M^*/Ø_bMs)^2+(V^*/ØvVv)^2$ = 0.17 N/A

Deflection:

Limits: **I,efftive = 0.17 x10^6mm4**

	G + Ψ,l Q =	1mm	=	Span/1587	
Roof members (trusses, rafters, etc)		Span/300		Y	OK
	Ws =	3mm	=	Span/512	
Ceilings with plaster finish		Span/200		Y	OK
Lintel Beam (vertical sag), <12mm		Span/240		Y	OK

Beam Label:	**RB11**	Location:	**Family Room**	**Bending**	**OK**
Span (L):	**3.1m**	Beam Section:	**LC190x45x15x1.55**	**Shear**	**OK**
			Single Member	**Combined**	**OK**
				Deflection	**OK**

Total Span (L) = 3.05m

Max	5.3kN	=RA*	RB* =	5.3kN	Max
Min	-6.4kN			-6.4kN	Min

Loadings: sw = 0.01kN/m

Roof	Dead	=	0.27	kPa	x	4.0m	=	1.1	kN/m
	Live	=	0.25	kPa	x	4.0m	=	1.0	kN/m
	Wind, up	=	-1.29	kPa	x	4.0m	=	-5.2	kN/m
	Wind, down	=	0.54	kPa	x	4.0m	=	2.2	kN/m
	Snow	=		kPa	x		=	0.0	kN/m
Wall	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m
Floor	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m

Live load factors:		
Ψ,c	=	0.4
Ψ,l	=	0.7
Ψ,s	=	0.7

Load Cases:

Ultimate Limit State:		(w)		Serviceability Limit State:		(w)	
1.	1.2G + 1.5 Q =	2.8	kN/m	6.	G + Ψ,l Q =	1.1	kN/m
2.	0.9G + Wu,up =	-4.2	kN/m	7.	G + Ψ,s Q =	1.8	kN/m
3.	1.2G+Wu,dw+ΨcQ =	3.5	kN/m	8.	Ws =	3.5	kN/m
4.	1.2G+Su+ΨcQ =	1.3	kN/m	9.	1kN Vibration		
5.	1.35G =	1.5	kN/m				

Design Actions:

Cases:	1.	2.	3.	4.	5.	Max	Min
Moment, M*= W x L^2/ =	3.3	-4.9	4.0	1.5	1.7	**4.0kNm**	**-4.9kNm**
Shear, V* = W x L /2 =	4.3	-6.4	5.3	2.0	2.2	**5.3kN**	**-6.4kN**

Capacity:

Bending:

Cb =	1.0	*(eq D2.1.1(2))*		Mo =	15.1	kNm	*(D2.1 Appendix D)*
Ley =	900	mm	*eff. length for buckling about y-axes*	Mol =	9.7	kNm	*eq. 7.2.2.3(4)*
Lez =	900	mm	*eff. length for twisting*	Mod =	9.7	kNm	*eq. 7.2.2.4(4)*
$Ø_b$ =	0.9	*Clause 7.2.2.1*		**ØMb =**	**ØMin(Mbe, Mbl, Mbd)**		
fy =	550	Mpa		=	**5.0**	**kNm**	
				Capacity Ratio =	**1.0**	OK	

Shear:

a =	3050	mm	*shear panel length*
d1 =	187.70	mm	*depth of web*
kv =	6.30	*eq. D3(4)*	
ØVv =	**20.8**	**kN**	OK

Combined bending and shear:

Maximum bending and shear at same location ? No

$(M^*/Ø_bMs)^2+(V^*/ØvVv)^2$ = **0.25** N/A

Deflection:

Limits: **I,efftive = 2.32 x10^6mm4**

	G + Ψ,l Q =	3mm =	Span/1151	
Roof members (trusses, rafters, etc)		Span/300	Y	OK
	Ws =	8mm =	Span/363	
Ceilings with plaster finish		Span/200	Y	OK
Lintel Beam (vertical sag), <12mm		Span/240	Y	OK

Beam Label:	RB12	Location:	Stair	Bending	OK
Span (L):	2.1m	Beam Section:	LC190x45x15x1.55	Shear	OK
			Single Member	Combined	OK
				Deflection	OK

Total Span (L) = 2.1m

Max	5.5kN	=RA*	RB* =	5.5kN	Max
Min	-6.7kN			-6.7kN	Min

Loadings: sw = 0.01kN/m

Roof	Dead	=	0.27	kPa	x	6.1m	=	1.6	kN/m
	Live	=	0.25	kPa	x	6.1m	=	1.5	kN/m
	Wind, up	=	-1.29	kPa	x	6.1m	=	-7.9	kN/m
	Wind, down	=	0.54	kPa	x	6.1m	=	3.3	kN/m
	Snow	=		kPa	x		=	0.0	kN/m
Wall	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m
Floor	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m

Live load factors:		
Ψ,c	=	0.4
Ψ,l	=	0.7
Ψ,s	=	0.7

Load Cases:

Ultimate Limit State:			(w)		Serviceability Limit State:			(w)	
1.	1.2G + 1.5 Q	=	4.3	kN/m	6.	G + Ψ,l Q	=	1.7	kN/m
2.	0.9G + Wu,up	=	-6.4	kN/m	7.	G + Ψ,s Q	=	2.7	kN/m
3.	1.2G+Wu,dw+ΨcQ	=	5.3	kN/m	8.	Ws	=	5.3	kN/m
4.	1.2G+Su+ΨcQ	=	2.0	kN/m	9.	1kN Vibration			
5.	1.35G	=	2.2	kN/m					

Design Actions:

Cases:		1.	2.	3.	4.	5.	Max	Min
Moment, M*= W x L^2/	=	2.4	-3.5	2.9	1.1	1.2	2.9kNm	-3.5kNm
Shear, V* = W x L /2	=	4.5	-6.7	5.5	2.1	2.4	5.5kN	-6.7kN

Capacity:

Bending:

Cb =	1.0	*(eq D2.1.1(2))*		Mo =	15.1	kNm	*(D2.1 Appendix D)*
Ley =	900	mm	*eff. length for buckling about y-axes*	Mol =	9.7	kNm	*eq. 7.2.2.3(4)*
Lez =	900	mm	*eff. length for twisting*	Mod =	9.7	kNm	*eq. 7.2.2.4(4)*
$Ø_b$ =	0.9	*Clause 7.2.2.1*		ØMb =	ØMin(Mbe, Mbl, Mbd)		
fy =	550	Mpa		=	5.0	kNm	
				Capacity Ratio =	0.7	OK	

Shear:

a =	2100	mm	*shear panel length*
d1 =	187.70	mm	*depth of web*
kv =	6.30	*eq. D3(4)*	
ØVv =	21.1	kN	OK

Combined bending and shear:

Maximum bending and shear at same location ? No

$(M^*/ØbMs)^2+(V^*/ØvVv)^2$ = 0.22 N/A

Deflection:

Limits: I,efftive = 2.32 x10^6mm4

	G + Ψ,l Q =	1mm	=	Span/2321	
Roof members (trusses, rafters, etc)		Span/300		Y	OK
	Ws =	3mm	=	Span/730	
Ceilings with plaster finish		Span/200		Y	OK
Lintel Beam (vertical sag), <12mm		Span/240		Y	OK

Beam Label:	**RB13**	Location:	**Study**	**Bending**	**OK**
Span (L):	**3.7m**	Beam Section:	**LC190x45x15x1.55**	**Shear**	**OK**
			Double (Boxed or BTB)	**Combined**	**OK**
				Deflection	**OK**

Total Span (L) = 3.7m

Max	7.1kN	=RA*	RB* =	7.1kN	Max
Min	-8.5kN			-8.5kN	Min

Loadings: sw = 0.01kN/m

Roof	Dead	=	0.27	kPa	x	4.4m	=	1.2	kN/m
	Live	=	0.25	kPa	x	4.4m	=	1.1	kN/m
	Wind, up	=	-1.29	kPa	x	4.4m	=	-5.7	kN/m
	Wind, down	=	0.54	kPa	x	4.4m	=	2.4	kN/m
	Snow	=		kPa	x		=	0.0	kN/m
Wall	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m
Floor	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m

Live load factors:		
Ψ,c	=	0.4
Ψ,l	=	0.7
Ψ,s	=	0.7

Load Cases:

Ultimate Limit State:			(w)	
1.	1.2G + 1.5 Q	=	3.1	kN/m
2.	0.9G + Wu,up	=	-4.6	kN/m
3.	1.2G+Wu,dw+ΨcQ	=	3.8	kN/m
4.	1.2G+Su+ΨcQ	=	1.4	kN/m
5.	1.35G	=	1.6	kN/m

Serviceability Limit State:			(w)	
6.	G + Ψ,l Q	=	1.2	kN/m
7.	G + Ψ,s Q	=	2.0	kN/m
8.	Ws	=	3.8	kN/m
9.	1kN Vibration			

Design Actions:

Cases:		1.	2.	3.	4.	5.	Max	Min
Moment, M*= W x L^2/	=	5.3	-7.9	6.5	2.5	2.8	**6.5kNm**	**-7.9kNm**
Shear, V* = W x L /2	=	5.7	-8.5	7.1	2.7	3.0	**7.1kN**	**-8.5kN**

Capacity:

Bending:

Cb =	1.0		*(eq D2.1.1(2))*
Ley =	900	mm	*eff. length for buckling about y-axes*
Lez =	900	mm	*eff. length for twisting*
$Ø_b$ =	0.9		*Clause 7.2.2.1*
fy =	550	Mpa	

Mo =	15.1	kNm	*(D2.1 Appendix D)*
Mol =	9.7	kNm	*eq. 7.2.2.3(4)*
Mod =	9.7	kNm	*eq. 7.2.2.4(4)*

ØMb = ØMin(Mbe, Mbl, Mbd)

= **10.0** **kNm**

Capacity Ratio = **0.8** OK

Shear:

a =	3700	mm	*shear panel length*
d1 =	187.70	mm	*depth of web*
kv =	6.30		*eq. D3(4)*
ØVv =	**41.4**	**kN**	OK

Combined bending and shear:

Maximum bending and shear at same location ? No

$(M^*/Ø_bMs)^2+(V^*/ØvVv)^2$ = **0.30** N/A

Deflection:

Limits: **I,efftive** = **4.14** **x10^6mm4**

	G + Ψ,l Q =	4mm	=	Span/1048
Roof members (trusses, rafters, etc)		Span/300	Y	OK
	Ws =	11mm	=	Span/330
Ceilings with plaster finish		Span/200	Y	OK
Lintel Beam (vertical sag), <12mm		Span/240	Y	OK

Beam Label:	**RB14**	Location:	**Study**	**Bending**	**OK**
Span (L):	**3.7m**	Beam Section:	**LC190x45x15x1.55**	**Shear**	**OK**
			Single Member	**Combined**	**OK**
				Deflection	**OK**

Total Span (L) = 3.7m

Max	1.9kN	=RA*	RB* =	1.9kN	Max
Min	-2.3kN			-2.3kN	Min

Loadings: sw = 0.01kN/m

Roof	Dead	=	0.27	kPa	x	1.2m	=	0.3	kN/m
	Live	=	0.25	kPa	x	1.2m	=	0.3	kN/m
	Wind, up	=	-1.29	kPa	x	1.2m	=	-1.5	kN/m
	Wind, down	=	0.54	kPa	x	1.2m	=	0.6	kN/m
	Snow	=		kPa	x		=	0.0	kN/m
Wall	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m
Floor	Dead	=		kPa	x		=	0.0	kN/m
	Live	=		kPa	x		=	0.0	kN/m

Live load factors:		
Ψ,c	=	0.4
Ψ,l	=	0.7
Ψ,s	=	0.7

Load Cases:

Ultimate Limit State:		(w)		Serviceability Limit State:		(w)	
1. 1.2G + 1.5 Q	=	0.9	kN/m	6. G + Ψ,l Q	=	0.3	kN/m
2. 0.9G + Wu,up	=	-1.2	kN/m	7. G + Ψ,s Q	=	0.5	kN/m
3. 1.2G+Wu,dw+ΨcQ	=	1.0	kN/m	8. Ws	=	1.0	kN/m
4. 1.2G+Su+ΨcQ	=	0.4	kN/m	9. 1kN Vibration			
5. 1.35G	=	0.5	kN/m				

Design Actions:

Cases:		1.	2.	3.	4.	5.	**Max**	**Min**
Moment, M*= W x L^2/	=	1.5	-2.1	1.8	0.7	0.8	**1.8kNm**	**-2.1kNm**
Shear, V* = W x L /2	=	1.6	-2.3	1.9	0.7	0.8	**1.9kN**	**-2.3kN**

Capacity:

Bending:

Cb =	1.0	*(eq D2.1.1(2))*		Mo =	15.1	kNm	*(D2.1 Appendix D)*
Ley =	900	mm	*eff. length for buckling about y-axes*	Mol =	9.7	kNm	*eq. 7.2.2.3(4)*
Lez =	900	mm	*eff. length for twisting*	Mod =	9.7	kNm	*eq. 7.2.2.4(4)*
$Ø_b$ =	0.9	*Clause 7.2.2.1*		**ØMb =**	**ØMin(Mbe, Mbl, Mbd)**		
fy =	550	Mpa		=	**5.0**	**kNm**	
			Capacity Ratio	=	**0.4**	OK	

Shear:

a =	3700	mm	*shear panel length*
d1 =	187.70	mm	*depth of web*
kv =	6.30	*eq. D3(4)*	
ØVv =	**20.7**	**kN**	OK

Combined bending and shear:

Maximum bending and shear at same location ? No

$(M^*/Ø_bMs)^2+(V^*/ØvVv)^2$ = **0.08** N/A

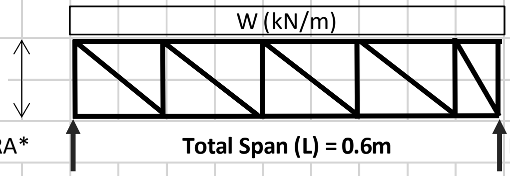
Deflection:

Limits: **I,efftive** = **2.32** **x10^6mm4**	G + Ψ,l Q =	2mm	=	Span/2100	
Roof members (trusses, rafters, etc)		Span/300		Y	OK
	Ws =	5mm	=	Span/678	
Ceilings with plaster finish		Span/200		Y	OK
Lintel Beam (vertical sag), <12mm		Span/240		Y	OK

Beam Label:	**FL1**	Location:	**Lintel**	**Bending**	**OK**
Span (L):	**0.6m**	Plate Section:	**89LC75**	**Shear**	**OK**
Depth (h):	**110mm**	Web Angle:	**90degree**	**Deflection**	**OK**

Top Chord Restraint = **0.9m**

Bottom Chord Restraint = **0.6m**



Max	0.9kN	=RA*	**Total Span (L) = 0.6m**	RB* =	0.9kN	Max
Min	-1.1kN				-1.1kN	Min

Loadings: Incoming Truss/Joists Spacing = 0.9m crs — No. of Point Load on Beam = **1**

Roof	Dead	=	0.27	kPa	x	1.2m	=	0.3	kN/m
	Live	=	0.25	kPa	x	1.2m	=	0.3	kN/m
	Wind, up	=	-1.29	kPa	x	1.2m	=	-1.5	kN/m
	Wind, down	=	0.54	kPa	x	1.2m	=	0.6	kN/m

Load Cases:

Ultimate Limit State:			(w)		Serviceability Limit State:			(w)	
1.	1.2G + 1.5 Q	=	0.8	kN/m	6.	G + Ψ,l Q	=	0.3	kN/m
2.	0.9G + Wu,up	=	-1.3	kN/m	7.	Ws	=	1.0	kN/m
3.	1.2G+Wu,dw+ΨcQ	=	1.0	kN/m	8.	Q2 = 1.1kN			
4.	1.2G+Su+ΨcQ	=	0.4	kN/m					
5.	1.35G	=	0.4	kN/m					

Design Actions: (P, point load = w x incoming truss or joist spacing)

Cases:		1.	2.	3.	4.	5.	**Max**	**Min**
Moment(M*)	W x L^2/8 =	0.0	-0.1	0.0	0.0	0.0	**0.1kNm**	**-0.2kNm**
	PL/4 =	0.1	-0.2	0.1	0.1	0.1		
Shear(V*)	WL/2 =	0.3	-0.4	0.3	0.1	0.1	**0.9kN**	**-1.1kN**
	P =	0.8	-1.1	0.9	0.3	0.4		

Capacity:

Bending:

Lintel plate section	=	89LC75 G550					
Effective Depth	=	60mm	*(Lintel depth - chord depth)*				
Top Chord Restraint	=	0.90	m	Bottom Chord Restraint	=	0.60	m
Chord Axial Capacity	=	16.00	kN	Chord Axial Capacity	=	16	kN
+ ØMn	=	**0.96**	**kNm**	**- ØMn**	=	**0.96**	**kNm**
Capacity Ratio	=	**0.15**	OK	**Capacity Ratio**	=	**0.18**	OK

Shear:

Lintel is too shallow to have diagonal, provide 0.75BMT G550 webplates on both sides

Fix to end studs with 2/10g screws, fix to top and bottom chord with 10gscrews - 200crs

Deflection:

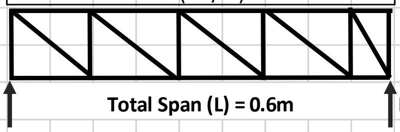
Limits: **I,efftive = 0.19 x10^6mm4**

	G + Ψ,l Q =	0mm	=	Span/40612	
Roof members (trusses, rafters, etc)		Span/300		Y	OK
	Ws =	0mm	=	Span/12687	
Ceilings with plaster finish		Span/200		Y	OK
Lintel Beam (vertical sag), <12mm		Span/240		Y	OK

Beam Label:	**FL2**	Location:	**Roof Beam**	**Bending**	**OK**
Span (L):	**0.6m**	Plate Section:	**89LC75**	**Shear**	**OK**
Depth (h):	**400mm**	Web Angle:	**45degree**	**Deflection**	**OK**

W (kN/m)

Depth (h)



Top Chord Restraint = **0.9m**

Bottom Chord Restraint = **0.9m**

Max 4.7kN	=RA*	**Total Span (L) = 0.6m**	RB* =	4.7kN	Max
Min -5.7kN				-5.7kN	Min

Loadings: Incoming Truss/Joists Spacing = 0.9m crs No. of Point Load on Beam = **1**

Roof								
	Dead	=	0.27	kPa	x	6.0m	=	1.6 kN/m
	Live	=	0.25	kPa	x	6.0m	=	1.5 kN/m
	Wind, up	=	-1.29	kPa	x	6.0m	=	-7.7 kN/m
	Wind, down	=	0.54	kPa	x	6.0m	=	3.2 kN/m
	Snow	=		kPa	x		=	0.0 kN/m

Load Cases:

Ultimate Limit State:		(w)		Serviceability Limit State:		(w)	
1.	1.2G + 1.5 Q =	4.2	kN/m	6.	G + Ψ,l Q =	1.6	kN/m
2.	0.9G + Wu,up =	-6.3	kN/m	7.	Ws =	5.2	kN/m
3.	1.2G+Wu,dw+ΨcQ =	5.2	kN/m	8.	Q2 = 1.1kN		
4.	1.2G+Su+ΨcQ =	1.9	kN/m				
5.	1.35G =	2.2	kN/m				

Design Actions: (P, point load = w x incoming truss or joist spacing)

Cases:		1.	2.	3.	4.	5.	Max	Min
Moment(M*)	W x L^2/8 =	0.2	-0.3	0.2	0.1	0.1	**0.7kNm**	**-0.8kNm**
	PL/4 =	0.6	-0.8	0.7	0.3	0.3		
Shear(V*)	WL/2 =	1.3	-1.9	1.6	0.6	0.7	**4.7kN**	**-5.7kN**
	P =	3.8	-5.7	4.7	1.7	2.0		

Capacity:

Bending:

Lintel plate section =	89LC75 G550					
Effective Depth =	350mm	*(Lintel depth - chord depth)*				
Top Chord Restraint =	0.90	m	Bottom Chord Restraint =	0.90	m	
Chord Axial Capacity =	16.00	kN	Chord Axial Capacity =	16	kN	
+ ØMn =	**5.60**	**kNm**	**- ØMn** =	**5.60**	**kNm**	
Capacity Ratio =	**0.12**	OK	**Capacity Ratio** =	**0.15**	OK	

Shear:

Web Diagonal Restraint =	0.57	m	*(Lintel depth / SIN (web angle)*
Diagonal Axial Capacity =	16	kN	
Web Screw Shear Capacity =	2.4	kN/screw	*(10g framing screw on 0.75BMT)*
No.of Screw Per Joint =	4		
Joint Connection Capacity =	9.6	kN	

ØVn = Min (Diagonal Axial Capacity, Joint Connection Capacity) x SIN (Web Angle)

ØVn = **6.79** kN

Capacity Ratio = **0.69** OK

Deflection:

Limits: **I,efftive = 6.30 x10^6mm4**

	G + Ψ,l Q =	0mm	=	Span/276390	
Roof members (trusses, rafters, etc)		Span/300		Y	OK
	Ws =	0mm	=	Span/86342	
Ceilings with plaster finish		Span/200		Y	OK

Beam Label:	**GB1**	Location:	**Entry**	**Bending**	**OK**
Span (L):	**2.1m**	Beam Section:	**LC290x45x15x2.5**	**Shear**	**OK**
			Single Member	**Combined**	**OK**
				Deflection	**OK**

Total Span (L) = 2.1m

Max	4.7kN	=RA*	RB* =	4.7kN	Max
Min	0.1kN			0.1kN	Min

Loadings: sw = 0.01kN/m

Roof	Dead	=	0.27	kPa	x	1.2m	=	0.3	kN/m	
	Live	=	0.25	kPa	x	1.2m	=	0.3	kN/m	
	Wind, up	=	-1.29	kPa	x	1.2m	=	-1.5	kN/m	
	Wind, down	=	0.54	kPa	x	1.2m	=	0.6	kN/m	
	Snow	=		kPa	x		=	0.0	kN/m	
Wall	Dead	=	0.42	kPa	x	2.7m	=	1.1	kN/m	Live load factors:
	Live	=		kPa	x		=	0.0	kN/m	Ψ,c = 0.4
Floor+Deck	Dead	=	0.50	kPa	x	0.8m	=	0.4	kN/m	Ψ,l = 0.7
	Live	=	1.50	kPa	x	0.8m	=	1.2	kN/m	Ψ,s = 0.7

Load Cases:

Ultimate Limit State:			(w)		Serviceability Limit State:			(w)	
1.	1.2G + 1.5 Q	=	4.5	kN/m	6.	G + Ψ,l Q	=	2.7	kN/m
2.	0.9G + Wu,up	=	0.1	kN/m	7.	G + Ψ,s Q	=	2.9	kN/m
3.	1.2G+Wu,dw+ΨcQ	=	3.4	kN/m	8.	Ws	=	1.0	kN/m
4.	1.2G+Su+ΨcQ	=	2.7	kN/m	9.	1kN Vibration			
5.	1.35G	=	2.5	kN/m					

Design Actions:

Cases:		1.	2.	3.	4.	5.	**Max**	**Min**
Moment, M*= W x L^2/	=	2.5	0.1	1.9	1.5	1.4	**2.5kNm**	**0.1kNm**
Shear, V* = W x L /2	=	4.7	0.1	3.5	2.9	2.6	**4.7kN**	**0.1kN**

Capacity:

Bending:

Cb =	1.0	*(eq D2.1.1(2))*		Mo =	15.1	kNm	*(D2.1 Appendix D)*
Ley =	900	mm	*eff. length for buckling about y-axes*	Mol =	17.8	kNm	*eq. 7.2.2.3(4)*
Lez =	900	mm	*eff. length for twisting*	Mod =	25.9	kNm	*eq. 7.2.2.4(4)*
$Ø_b$ =	0.9	*Clause 7.2.2.1*		**ØMb =**	**ØMin(Mbe, Mbl, Mbd)**		
fy =	550	Mpa		=	**13.6**	**kNm**	
				Capacity Ratio =	**0.2**	OK	

Shear:

a =	2100	mm	*shear panel length*
d1 =	285.00	mm	*depth of web*
kv =	6.30	*eq. D3(4)*	
ØVv =	**60.1**	**kN**	OK

Combined bending and shear:

Maximum bending and shear at same location ? No

$(M^*/Ø_bMs)^2+(V^*/ØvVv)^2$ = 0.02 N/A

Deflection:

Limits:	**I,efftive = 10.30 x10^6mm4**	G + Ψ,l Q =	0mm	=	Span/6306	
Roof members (trusses, rafters, etc)			Span/300		Y	OK
		Ws =	0mm	=	Span/16471	
Ceilings with plaster finish			Span/200		Y	OK
Lintel Beam (vertical sag), <12mm			Span/240		N/A	N/A

Beam Label:	**GB2**	Location:	**Rumpus**	**Bending**	**OK**
Span (L):	**2.4m**	Beam Section:	**LC290x45x15x2.5**	**Shear**	**OK**
			Single Member	**Combined**	**OK**
				Deflection	**OK**

Total Span (L) = 2.4m

Max	5.1kN	=RA*	RB* =	5.1kN	Max
Min	-2.2kN			-2.2kN	Min

Loadings: sw = 0.01kN/m

Roof	Dead	=	0.27	kPa	x	2.9m	=	0.8	kN/m	
	Live	=	0.25	kPa	x	2.9m	=	0.7	kN/m	
	Wind, up	=	-1.29	kPa	x	2.9m	=	-3.7	kN/m	
	Wind, down	=	0.54	kPa	x	2.9m	=	1.6	kN/m	
	Snow	=		kPa	x		=	0.0	kN/m	
Wall	Dead	=	0.42	kPa	x	2.7m	=	1.1	kN/m	Live load factors:
	Live	=		kPa	x		=	0.0	kN/m	Ψ,c = 0.4
Floor	Dead	=	0.50	kPa	x	0.3m	=	0.2	kN/m	Ψ,l = 0.7
	Live	=	1.50	kPa	x	0.3m	=	0.5	kN/m	Ψ,s = 0.7

Load Cases:

Ultimate Limit State:			(w)		Serviceability Limit State:			(w)	
1.	1.2G + 1.5 Q	=	4.3	kN/m	6.	G + Ψ,l Q	=	2.4	kN/m
2.	0.9G + Wu,up	=	-1.9	kN/m	7.	G + Ψ,s Q	=	2.9	kN/m
3.	1.2G+Wu,dw+ΨcQ	=	4.2	kN/m	8.	Ws	=	2.5	kN/m
4.	1.2G+Su+ΨcQ	=	2.7	kN/m	9.	1kN Vibration			
5.	1.35G	=	2.8	kN/m					

Design Actions:

Cases:		1.	2.	3.	4.	5.	**Max**	**Min**
Moment, M*= W x L^2/	=	3.1	-1.3	3.1	1.9	2.0	**3.1kNm**	**-1.3kNm**
Shear, V* = W x L /2	=	5.1	-2.2	5.1	3.2	3.4	**5.1kN**	**-2.2kN**

Capacity:

Bending:

Cb =	1.0	*(eq D2.1.1(2))*		Mo =	15.1	kNm	*(D2.1 Appendix D)*
Ley =	900	mm	*eff. length for buckling about y-axes*	Mol =	17.8	kNm	*eq. 7.2.2.3(4)*
Lez =	900	mm	*eff. length for twisting*	Mod =	25.9	kNm	*eq. 7.2.2.4(4)*
$Ø_b$ =	0.9	*Clause 7.2.2.1*		**ØMb =**	**ØMin(Mbe, Mbl, Mbd)**		
fy =	550	Mpa		=	**13.6**	**kNm**	
			Capacity Ratio	=	**0.2**	OK	

Shear:

a =	2400	mm	*shear panel length*
d1 =	285.00	mm	*depth of web*
kv =	6.30	*eq. D3(4)*	
ØVv =	**59.5**	**kN**	OK

Combined bending and shear:

Maximum bending and shear at same location ? No

$(M^*/Ø_bMs)^2+(V^*/ØvVv)^2$ = **0.02** N/A

Deflection:

Limits: **I,efftive = 10.30 x10^6mm4** G + Ψ,l Q =	1mm	=	Span/4783	
Beams where line-of-sight is across soffit, aceess by operators and mainten	Span/250		Y	OK
Floor Joists/Beams Sag, Flooring ripple, Floor Supports plaster lined walls	Span/300		Y	OK
Floor Vibration	<2mm		Y	OK
Ws =	1mm	=	Span/4566	
Ceilings with plaster finish	Span/200		Y	OK
Lintel Beam (vertical sag), <12mm	Span/240		Y	OK

Beam Label:	**GL1**	Location:	**Living**	**Bending**	**OK**
Span (L):	**3.5m**	Beam Section:	**LC290x45x15x2.5**	**Shear**	**OK**
			Single Member	**Combined**	**OK**
				Deflection	**OK**

Total Span (L) = 3.5m

Max	7.6kN	=RA*	RB* =	7.6kN	Max
Min	-3.5kN			-3.5kN	Min

Loadings: sw = 0.01kN/m

Roof	Dead	=	0.27	kPa	x	3.0m	=	0.8	kN/m
	Live	=	0.25	kPa	x	3.0m	=	0.8	kN/m
	Wind, up	=	-1.29	kPa	x	3.0m	=	-3.9	kN/m
	Wind, down	=	0.54	kPa	x	3.0m	=	1.6	kN/m
	Snow	=		kPa	x		=	0.0	kN/m
Wall	Dead	=	0.42	kPa	x	2.7m	=	1.1	kN/m
	Live	=		kPa	x		=	0.0	kN/m
Floor	Dead	=	0.50	kPa	x	0.3m	=	0.2	kN/m
	Live	=	1.50	kPa	x	0.3m	=	0.5	kN/m

Live load factors:		
Ψ,c	=	0.4
Ψ,l	=	0.7
Ψ,s	=	0.7

Load Cases:

Ultimate Limit State:		(w)		Serviceability Limit State:		(w)	
1.	1.2G + 1.5 Q =	4.3	kN/m	6.	G + Ψ,l Q =	2.4	kN/m
2.	0.9G + Wu,up =	-2.0	kN/m	7.	G + Ψ,s Q =	2.9	kN/m
3.	1.2G+Wu,dw+ΨcQ =	4.3	kN/m	8.	Ws =	2.6	kN/m
4.	1.2G+Su+ΨcQ =	2.7	kN/m	9.	1kN Vibration		
5.	1.35G =	2.8	kN/m				

Design Actions:

Cases:		1.	2.	3.	4.	5.	Max	Min
Moment, M*= W x L^2/	=	6.6	-3.0	6.6	4.1	4.4	**6.6kNm**	**-3.0kNm**
Shear, V* = W x L /2	=	7.6	-3.5	7.6	4.7	5.0	**7.6kN**	**-3.5kN**

Capacity:

Bending:

Cb =	1.0		*(eq D2.1.1(2))*	Mo =	15.1	kNm	*(D2.1 Appendix D)*
Ley =	900	mm	*eff. length for buckling about y-axes*	Mol =	17.8	kNm	*eq. 7.2.2.3(4)*
Lez =	900	mm	*eff. length for twisting*	Mod =	25.9	kNm	*eq. 7.2.2.4(4)*
$Ø_b$ =	0.9		*Clause 7.2.2.1*	**ØMb =**	**ØMin(Mbe, Mbl, Mbd)**		
fy =	550	Mpa		=	**13.6**	**kNm**	
				Capacity Ratio =	**0.5**	OK	

Shear:

a =	3500	mm	*shear panel length*	
d1 =	285.00	mm	*depth of web*	
kv =	6.30		*eq. D3(4)*	
ØVv =	**58.4**	**kN**	OK	

Combined bending and shear:

Maximum bending and shear at same location ? No

$(M^*/Ø_bMs)^2+(V^*/ØvVv)^2$ = 0.04 N/A

Deflection:

Limits: **I,efftive = 10.30 x10^6mm4** G + Ψ,l Q = 2mm = Span/1525

Beams where line-of-sight is across soffit, aceess by operators and mainten	Span/250	Y	OK
Floor Joists/Beams Sag, Flooring ripple, Floor Supports plaster lined walls	Span/300	Y	OK
Floor Vibration	<2mm	Y	OK

Ws = 2mm = Span/1423

Ceilings with plaster finish	Span/200	Y	OK
Lintel Beam (vertical sag), <12mm	Span/240	Y	OK

Beam Label:	GB2	Location:	Living	Bending	OK
Span (L):	2.4m	Beam Section:	LC290x45x15x2.5	Shear	OK
			Double (Boxed or BTB)	Combined	OK
				Deflection	OK

Total Span (L) = 2.4m

Max	7.2kN	=RA*	RB* =	7.2kN	Max
Min	-1.9kN			-1.9kN	Min

Loadings: sw = 0.01kN/m

Roof	Dead	=	0.27	kPa	x	2.9m	=	0.8	kN/m
	Live	=	0.25	kPa	x	2.9m	=	0.7	kN/m
	Wind, up	=	-1.29	kPa	x	2.9m	=	-3.7	kN/m
	Wind, down	=	0.54	kPa	x	2.9m	=	1.6	kN/m
	Snow	=		kPa	x		=	0.0	kN/m
Wall	Dead	=	0.42	kPa	x	2.7m	=	1.1	kN/m
	Live	=		kPa	x		=	0.0	kN/m
Floor + Deck	Dead	=	0.50	kPa	x	0.9m	=	0.5	kN/m
	Live	=	1.50	kPa	x	0.9m	=	1.4	kN/m

Live load factors:		
Ψ,c	=	0.4
Ψ,l	=	0.7
Ψ,s	=	0.7

Load Cases:

Ultimate Limit State:			(w)		Serviceability Limit State:			(w)	
1.	1.2G + 1.5 Q	=	6.0	kN/m	6.	G + Ψ,l Q	=	3.3	kN/m
2.	0.9G + Wu,up	=	-1.6	kN/m	7.	G + Ψ,s Q	=	3.8	kN/m
3.	1.2G+Wu,dw+ΨcQ	=	5.0	kN/m	8.	Ws	=	2.5	kN/m
4.	1.2G+Su+ΨcQ	=	3.4	kN/m	9.	1kN Vibration			
5.	1.35G	=	3.2	kN/m					

Design Actions:

Cases:		1.	2.	3.	4.	5.	Max	Min
Moment, M*= W x L^2/	=	4.3	-1.2	3.6	2.4	2.3	**4.3kNm**	**-1.2kNm**
Shear, V* = W x L /2	=	7.2	-1.9	6.0	4.1	3.9	**7.2kN**	**-1.9kN**

Capacity:

Bending:

Cb =	1.0	*(eq D2.1.1(2))*		Mo =	15.1	kNm	*(D2.1 Appendix D)*
Ley =	900	mm	*eff. length for buckling about y-axes*	Mol =	17.8	kNm	*eq. 7.2.2.3(4)*
Lez =	900	mm	*eff. length for twisting*	Mod =	25.9	kNm	*eq. 7.2.2.4(4)*
$Ø_b$ =	0.9	*Clause 7.2.2.1*		**ØMb =**	**ØMin(Mbe, Mbl, Mbd)**		
fy =	550	Mpa		=	**27.2**	**kNm**	
				Capacity Ratio =	**0.2**	OK	

Shear:

a =	2400	mm	*shear panel length*
d1 =	285.00	mm	*depth of web*
kv =	6.30	*eq. D3(4)*	
ØVv =	**119.1**	**kN**	OK

Combined bending and shear:

Maximum bending and shear at same location ? No

$(M^*/Ø_bMs)^2+(V^*/ØvVv)^2$ = 0.02 N/A

Deflection:

Limits: **I,efftive = 20.60 x10^6mm4** G + Ψ,l Q = 0mm = Span/6888

Beams where line-of-sight is across soffit, aceess by operators and mainten	Span/250	Y	OK
Floor Joists/Beams Sag, Flooring ripple, Floor Supports plaster lined walls	Span/300	Y	OK
Floor Vibration	<2mm	Y	OK

Ws = 0mm = Span/9132

Ceilings with plaster finish	Span/200	Y	OK
Lintel Beam (vertical sag), <12mm	Span/240	Y	OK

Beam Label:	**GB3**	Location:	**Living**	**Bending**	**OK**
Span (L):	**5.0m**	Beam Section:	**LC290x45x15x2.5**	**Shear**	**OK**
			Double (Boxed or BTB)	**Combined**	**OK**
				Deflection	**OK**

Total Span (L) = 5m

Max	17.2kN	=RA*	RB* =	17.2kN	Max
Min	-13.2kN			-13.2kN	Min

Loadings: sw = 0.01kN/m

Roof	Dead	=	0.27	kPa	x	6.0m	=	1.6	kN/m
	Live	=	0.25	kPa	x	6.0m	=	1.5	kN/m
	Wind, up	=	-1.29	kPa	x	6.0m	=	-7.7	kN/m
	Wind, down	=	0.54	kPa	x	6.0m	=	3.2	kN/m
	Snow	=		kPa	x		=	0.0	kN/m
Wall	Dead	=	0.30	kPa	x	2.7m	=	0.8	kN/m
	Live	=		kPa	x		=	0.0	kN/m
Floor	Dead	=	0.50	kPa	x	0.6m	=	0.3	kN/m
	Live	=	1.50	kPa	x	0.6m	=	0.9	kN/m

Live load factors:		
Ψ,c	=	0.4
Ψ,l	=	0.7
Ψ,s	=	0.7

Load Cases:

Ultimate Limit State:			(w)		Serviceability Limit State:			(w)	
1.	1.2G + 1.5 Q	=	6.9	kN/m	6.	G + Ψ,l Q	=	3.4	kN/m
2.	0.9G + Wu,up	=	-5.3	kN/m	7.	G + Ψ,s Q	=	4.4	kN/m
3.	1.2G+Wu,dw+ΨcQ	=	6.9	kN/m	8.	Ws	=	5.2	kN/m
4.	1.2G+Su+ΨcQ	=	3.6	kN/m	9.	1kN Vibration			
5.	1.35G	=	3.7	kN/m					

Design Actions:

Cases:		1.	2.	3.	4.	5.	**Max**	**Min**
Moment, M*= W x L^2/	=	21.5	-16.5	21.5	11.4	11.6	**21.5kNm**	**-16.5kNm**
Shear, V* = W x L /2	=	17.2	-13.2	17.2	9.1	9.3	**17.2kN**	**-13.2kN**

Capacity:

Bending:

Cb =	1.0	*(eq D2.1.1(2))*		Mo =	15.1	kNm	*(D2.1 Appendix D)*
Ley =	900	mm	*eff. length for buckling about y-axes*	Mol =	17.8	kNm	*eq. 7.2.2.3(4)*
Lez =	900	mm	*eff. length for twisting*	Mod =	25.9	kNm	*eq. 7.2.2.4(4)*
$\varnothing_b$ =	0.9	*Clause 7.2.2.1*		**ØMb =**	**ØMin(Mbe, Mbl, Mbd)**		
fy =	550	Mpa		=	**27.2**	**kNm**	
				Capacity Ratio =	**0.8**	OK	

Shear:

a =	5000	mm	*shear panel length*	
d1 =	285.00	mm	*depth of web*	
kv =	6.30	*eq. D3(4)*		
ØVv =	**115.4**	**kN**	OK	

Combined bending and shear:

Maximum bending and shear at same location ? No

$(M^*/\varnothing_b M_s)^2+(V^*/\varnothing_v V_v)^2$ = **0.11** N/A

Deflection:

Limits: **I,efftive = 18.04 x10^6mm4** G + Ψ,l Q = 8mm = Span/658

Beams where line-of-sight is across soffit, aceess by operators and mainten	Span/250	Y	OK
Floor Joists/Beams Sag, Flooring ripple, Floor Supports plaster lined walls	Span/300	Y	OK
Floor Vibration	<2mm	Y	OK

Ws = 12mm = Span/428

Ceilings with plaster finish	Span/200	Y	OK
Lintel Beam (vertical sag), <12mm	Span/240	Y	OK

Beam Label:	**GL1**	Location:	**Living**	**Bending**	**OK**
Span (L):	**3.5m**	Beam Section:	**LC290x45x15x2.5**	**Shear**	**OK**
			Single Member	**Combined**	**OK**
				Deflection	**OK**

Total Span (L) = 3.5m

Max	11.6kN	=RA*	RB* =	11.6kN	Max
Min	-2.8kN			-2.8kN	Min

Loadings: sw = 0.01kN/m

Roof	Dead	=	0.27	kPa	x	3.0m	=	0.8	kN/m
	Live	=	0.25	kPa	x	3.0m	=	0.8	kN/m
	Wind, up	=	-1.29	kPa	x	3.0m	=	-3.9	kN/m
	Wind, down	=	0.54	kPa	x	3.0m	=	1.6	kN/m
	Snow	=		kPa	x		=	0.0	kN/m
Wall	Dead	=	0.42	kPa	x	2.7m	=	1.1	kN/m
	Live	=		kPa	x		=	0.0	kN/m
Floor+Deck	Dead	=	0.50	kPa	x	1.1m	=	0.6	kN/m
	Live	=	1.50	kPa	x	1.1m	=	1.7	kN/m

Live load factors:		
Ψ,c	=	0.4
Ψ,l	=	0.7
Ψ,s	=	0.7

Load Cases:

Ultimate Limit State:		(w)		Serviceability Limit State:		(w)	
1. 1.2G + 1.5 Q	=	6.6	kN/m	6. G + Ψ,l Q	=	3.7	kN/m
2. 0.9G + Wu,up	=	-1.6	kN/m	7. G + Ψ,s Q	=	4.2	kN/m
3. 1.2G+Wu,dw+ΨcQ	=	5.3	kN/m	8. Ws	=	2.6	kN/m
4. 1.2G+Su+ΨcQ	=	3.7	kN/m	9. 1kN Vibration			
5. 1.35G	=	3.4	kN/m				

Design Actions:

Cases:		1.	2.	3.	4.	5.	**Max**	**Min**
Moment, M*= W x L^2/	=	10.1	-2.5	8.1	5.6	5.2	**10.1kNm**	**-2.5kNm**
Shear, V* = W x L /2	=	11.6	-2.8	9.3	6.4	5.9	**11.6kN**	**-2.8kN**

Capacity:

Bending:

Cb =	1.0	*(eq D2.1.1(2))*		Mo =	15.1	kNm	*(D2.1 Appendix D)*
Ley =	900	mm	*eff. length for buckling about y-axes*	Mol =	17.8	kNm	*eq. 7.2.2.3(4)*
Lez =	900	mm	*eff. length for twisting*	Mod =	25.9	kNm	*eq. 7.2.2.4(4)*
$Ø_b$ =	0.9	*Clause 7.2.2.1*		**ØMb =**	**ØMin(Mbe, Mbl, Mbd)**		
fy =	550	Mpa		=	**13.6**	**kNm**	
				Capacity Ratio =	**0.7**	OK	

Shear:

a =	3500	mm	*shear panel length*
d1 =	285.00	mm	*depth of web*
kv =	6.30	*eq. D3(4)*	
ØVv =	**58.4**	**kN**	OK

Combined bending and shear:

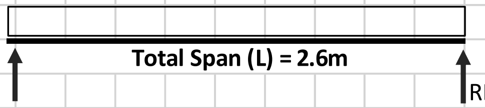
Maximum bending and shear at same location ? No

$(M^*/Ø_bMs)^2+(V^*/ØvVv)^2$ = **0.08** N/A

Deflection:

Limits: **I,efftive = 10.30 x10^6mm4**	G + Ψ,l Q =	3mm	=	Span/1008
Beams where line-of-sight is across soffit, aceess by operators and mainten	Span/250		Y	OK
Floor Joists/Beams Sag, Flooring ripple, Floor Supports plaster lined walls	Span/300		Y	OK
Floor Vibration	<2mm		Y	OK
	Ws =	2mm	=	Span/1423
Ceilings with plaster finish	Span/200		Y	OK
Lintel Beam (vertical sag), <12mm	Span/240		Y	OK

Beam Label:	**GL2**	Location:	**Garage Window**	**Bending**	**OK**
Span (L):	**2.6m**	Beam Section:	**PC240x40x1.85**	**Shear**	**OK**
			Single Member	**Combined**	**OK**
				Deflection	**#DIV/0!**



Max	7.7kN	=RA*	RB* =	7.7kN	Max
Min	2.3kN			2.3kN	Min

Loadings: sw = 0.01kN/m

Roof	Dead	=	0.27	kPa	x		=	0.0	kN/m
	Live	=	0.25	kPa	x		=	0.0	kN/m
	Wind, up	=	-1.29	kPa	x		=	0.0	kN/m
	Wind, down	=	0.54	kPa	x		=	0.0	kN/m
	Snow	=		kPa	x		=	0.0	kN/m
Wall	Dead	=	0.42	kPa	x	2.7m	=	1.1	kN/m
	Live	=		kPa	x		=	0.0	kN/m
Floor + Deck	Dead	=	0.50	kPa	x	1.6m	=	0.8	kN/m
	Live	=	1.50	kPa	x	1.6m	=	2.4	kN/m

Live load factors:		
Ψ,c	=	0.4
Ψ,l	=	0.7
Ψ,s	=	0.7

Load Cases:

Ultimate Limit State:		(w)		Serviceability Limit State:		(w)	
1. 1.2G + 1.5 Q	=	5.9	kN/m	6. G + Ψ,l Q	=	3.6	kN/m
2. 0.9G + Wu,up	=	1.8	kN/m	7. G + Ψ,s Q	=	3.6	kN/m
3. 1.2G+Wu,dw+ΨcQ	=	3.3	kN/m	8. Ws	=	0.0	kN/m
4. 1.2G+Su+ΨcQ	=	3.3	kN/m	9. 1kN Vibration			
5. 1.35G	=	2.6	kN/m				

Design Actions:

Cases:		1.	2.	3.	4.	5.	Max	Min
Moment, M*= W x L^2/	=	5.0	1.5	2.8	2.8	2.2	**5.0kNm**	**1.5kNm**
Shear, V* = W x L /2	=	7.7	2.3	4.3	4.3	3.4	**7.7kN**	**2.3kN**

Capacity:

Bending:

Cb =	1.0	*(eq D2.1.1(2))*		Mo =	15.1	kNm	*(D2.1 Appendix D)*
Ley =	450	mm	*eff. length for buckling about y-axes*	Mol =	8.5	kNm	*eq. 7.2.2.3(4)*
Lez =	450	mm	*eff. length for twisting*	Mod =	8.5	kNm	*eq. 7.2.2.4(4)*
$Ø_b$ =	0.9	*Clause 7.2.2.1*		**ØMb =**	**ØMin(Mbe, Mbl, Mbd)**		
fy =	550	Mpa		=	**6.0**	**kNm**	
				Capacity Ratio =	**0.8**	OK	

Shear:

a =	2600	mm	*shear panel length*
d1 =	236.00	mm	*depth of web*
kv =	6.30	*eq. D3(4)*	
ØVv =	**28.7**	**kN**	OK

Combined bending and shear:

Maximum bending and shear at same location ? No

$(M^*/Ø_bMs)^2+(V^*/ØvVv)^2$ = **0.16** N/A

Deflection:

Limits: **I,efftive = 4.07 x10^6mm4** G + Ψ,l Q = 3mm = Span/981

Beams where line-of-sight is across soffit, aceess by operators and mainten	Span/250	Y	OK
Floor Joists/Beams Sag, Flooring ripple, Floor Supports plaster lined walls	Span/300	Y	OK
Floor Vibration	<2mm	Y	OK

Beam Label:	GL3	Location:	Living	Bending	OK
Span (L):	2.7m	Beam Section:	LC290x45x15x2.5	Shear	OK
			Single Member	Combined	OK
				Deflection	OK

Total Span (L) = 2.7m

Max	15.7kN	=RA*	RB* =	15.7kN	Max
Min	1.7kN			1.7kN	Min

Loadings: sw = 0.01kN/m

Roof	Dead	=	0.27	kPa	x	1.2m	=	0.3	kN/m	
	Live	=	0.25	kPa	x	1.2m	=	0.3	kN/m	
	Wind, up	=	-1.29	kPa	x	1.2m	=	-1.5	kN/m	
	Wind, down	=	0.54	kPa	x	1.2m	=	0.6	kN/m	
	Snow	=		kPa	x		=	0.0	kN/m	Live load factors:
Wall	Dead	=	0.42	kPa	x	2.7m	=	1.1	kN/m	Ψ,c = 0.4
	Live	=		kPa	x		=	0.0	kN/m	Ψ,l = 0.7
Floor + Deck	Dead	=	0.50	kPa	x	3.3m	=	1.7	kN/m	Ψ,s = 0.7
	Live	=	1.50	kPa	x	3.3m	=	5.0	kN/m	

Load Cases:

Ultimate Limit State:		(w)		Serviceability Limit State:		(w)	
1. 1.2G + 1.5 Q	=	11.6	kN/m	6. G + Ψ,l Q	=	6.6	kN/m
2. 0.9G + Wu,up	=	1.3	kN/m	7. G + Ψ,s Q	=	6.8	kN/m
3. 1.2G+Wu,dw+ΨcQ	=	6.4	kN/m	8. Ws	=	1.0	kN/m
4. 1.2G+Su+ΨcQ	=	5.7	kN/m	9. 1kN Vibration			
5. 1.35G	=	4.2	kN/m				

Design Actions:

Cases:		1.	2.	3.	4.	5.	Max	Min
Moment, M*= W x L^2/	=	10.6	1.1	5.8	5.2	3.8	**10.6kNm**	**1.1kNm**
Shear, V* = W x L /2	=	15.7	1.7	8.6	7.7	5.7	**15.7kN**	**1.7kN**

Capacity:

Bending:

Cb =	1.0	*(eq D2.1.1(2))*		Mo =	15.1	kNm	*(D2.1 Appendix D)*
Ley =	900	mm	*eff. length for buckling about y-axes*	Mol =	17.8	kNm	*eq. 7.2.2.3(4)*
Lez =	900	mm	*eff. length for twisting*	Mod =	25.9	kNm	*eq. 7.2.2.4(4)*
$Ø_b$ =	0.9	*Clause 7.2.2.1*		**ØMb =**	**ØMin(Mbe, Mbl, Mbd)**		
fy =	550	Mpa		=	**13.6**	**kNm**	
			Capacity Ratio	=	**0.8**	OK	

Shear:

a =	2700	mm	*shear panel length*
d1 =	285.00	mm	*depth of web*
kv =	6.30	*eq. D3(4)*	
ØVv =	**59.1**	**kN**	OK

Combined bending and shear:

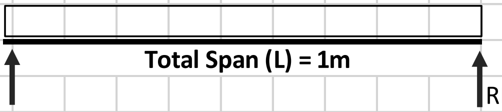
Maximum bending and shear at same location ? No

$(M^*/Ø_bMs)^2+(V^*/ØvVv)^2$ = **0.11** N/A

Deflection:

Limits: **I,efftive** = **10.30** **x10^6mm4**	G + Ψ,l Q =	2mm	=	Span/1221	
Beams where line-of-sight is across soffit, aceess by operators and mainten		Span/250		Y	OK
Floor Joists/Beams Sag, Flooring ripple, Floor Supports plaster lined walls		Span/300		Y	OK
Floor Vibration		<2mm		Y	OK
	Ws =	0mm	=	Span/7750	
Ceilings with plaster finish		Span/200		Y	OK
Lintel Beam (vertical sag), <12mm		Span/240		Y	OK

Beam Label:	GL4	Location:	Garage	Bending	OK
Span (L):	1.0m	Beam Section:	PC290x40x2.50	Shear	OK
			Single Member	Combined	OK
				Deflection	OK



Total Span (L) = 1m

Max	6.3kN	=RA*	RB* =	6.3kN Max
Min	1.0kN			1.0kN Min

Loadings: sw = 0.01kN/m

Roof	Dead	=		kPa	x		=	0.0 kN/m
	Live	=		kPa	x		=	0.0 kN/m
	Wind, up	=		kPa	x		=	0.0 kN/m
	Wind, down	=		kPa	x		=	0.0 kN/m
	Snow	=		kPa	x		=	0.0 kN/m
Wall	Dead	=		kPa	x		=	0.0 kN/m
	Live	=		kPa	x		=	0.0 kN/m
Floor	Dead	=	0.50	kPa	x	4.4m	=	2.2 kN/m
	Live	=	1.50	kPa	x	4.4m	=	6.6 kN/m

Live load factors:		
Ψ,c	=	0.4
Ψ,l	=	0.7
Ψ,s	=	0.7

Load Cases:

Ultimate Limit State:		(w)		Serviceability Limit State:		(w)	
1. 1.2G + 1.5 Q	=	12.6	kN/m	6. G + Ψ,l Q	=	6.8	kN/m
2. 0.9G + Wu,up	=	2.0	kN/m	7. G + Ψ,s Q	=	6.8	kN/m
3. 1.2G+Wu,dw+ΨcQ	=	5.3	kN/m	8. Ws	=	0.0	kN/m
4. 1.2G+Su+ΨcQ	=	5.3	kN/m	9. 1kN Vibration			
5. 1.35G	=	3.0	kN/m				

Design Actions:

Cases:		1.	2.	3.	4.	5.	Max	Min
Moment, M*= W x L^2/	=	1.6	0.2	0.7	0.7	0.4	1.6kNm	0.2kNm
Shear, V* = W x L /2	=	6.3	1.0	2.6	2.6	1.5	6.3kN	1.0kN

Capacity:

Bending:

Cb =	1.0	*(eq D2.1.1(2))*	Mo =	15.1	kNm	*(D2.1 Appendix D)*
Ley =	900	mm *eff. length for buckling about y-axes*	Mol =	19.7	kNm	*eq. 7.2.2.3(4)*
Lez =	900	mm *eff. length for twisting*	Mod =	19.7	kNm	*eq. 7.2.2.4(4)*
$Ø_b$ =	0.9	*Clause 7.2.2.1*	**ØMb =**	**ØMin(Mbe, Mbl, Mbd)**		
fy =	550	Mpa	=	**11.7**	**kNm**	
			Capacity Ratio =	**0.1**	OK	

Shear:

a =	1000	mm *shear panel length*
d1 =	285.00	mm *depth of web*
kv =	6.30	*eq. D3(4)*
ØVv =	**64.9**	**kN** OK

Combined bending and shear:

Maximum bending and shear at same location ? No

$(M^*/Ø_bMs)^2+(V^*/ØvVv)^2$ = **0.02** N/A

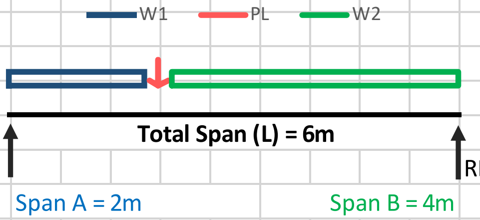
Deflection:

Limits: **I,efftive = 8.81 x10^6mm4** G + Ψ,l Q = 0mm = Span/19810

Beams where line-of-sight is across soffit, aceess by operators and mainten	Span/250	Y	OK
Floor Joists/Beams Sag, Flooring ripple, Floor Supports plaster lined walls	Span/300	Y	OK
Floor Vibration	<2mm	Y	OK

Beam Label:	**GB4**	Location:	**Shower Trimmer**	**Bending**	**OK**
Span (L):	**6.0m**	Beam Section:	**2/LC290x45x15x2.5**	**Shear**	**OK**
PL Location, a =	**2.0m**	Fire Rated:	**N/A**	**Deflection**	**OK**
				Limited Temp	**NA**

sw = 0.01kN/m

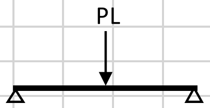


	RA*	RB*	
Max	13.2kN	7.6kN	Max
Min	-2.7kN	-1.1kN	Min

Loadings:

Point Load, PL: (Tributy Area)

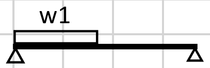
Roof	Dead	=	0.27	kPa	x	5.9m	x	1.2m	=	1.9	kN
	Live	=	0.25	kPa	x	5.9m	x	1.2m	=	1.8	kN
	Wind, up	=	-1.29	kPa	x	5.9m	x	1.2m	=	-9.1	kN
	Wind, down	=	0.54	kPa	x	5.9m	x	1.2m	=	3.8	kN
	Snow	=		kPa	x		x		=	0.0	kN
Wall	Dead	=	0.30	kPa	x	2.7m	x	1.2m	=	1.0	kN
	Live	=		kPa	x		x		=	0.0	kN
Floor	Dead	=	0.60	kPa	x	3.0m	x	1.2m	=	2.2	kN
	Live	=	1.50	kPa	x	3.0m	x	1.2m	=	5.4	kN



Dead	=	5.0	kN
Live	=	7.2	kN
Wind, up	=	-9.1	kN
Wind, down	=	3.8	kN
Snow	=	0.0	kN

UDL Load, W1: (Tributy Area)

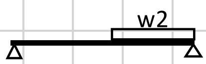
Floor	Dead	=	0.60	kPa	x	0.2m	=	0.1	kN/m
	Live	=	1.50	kPa	x	0.2m	=	0.3	kN/m



Dead	=	0.1	kN/m
Live	=	0.3	kN/m
Wind, up	=	0.0	kN/m
Wind, down	=	0.0	kN/m
Snow	=	0.0	kN/m

UDL Load, W2: (Tributy Area)

Floor	Dead	=	0.60	kPa	x	0.2m	=	0.1	kN/m
	Live	=	1.50	kPa	x	0.2m	=	0.3	kN/m

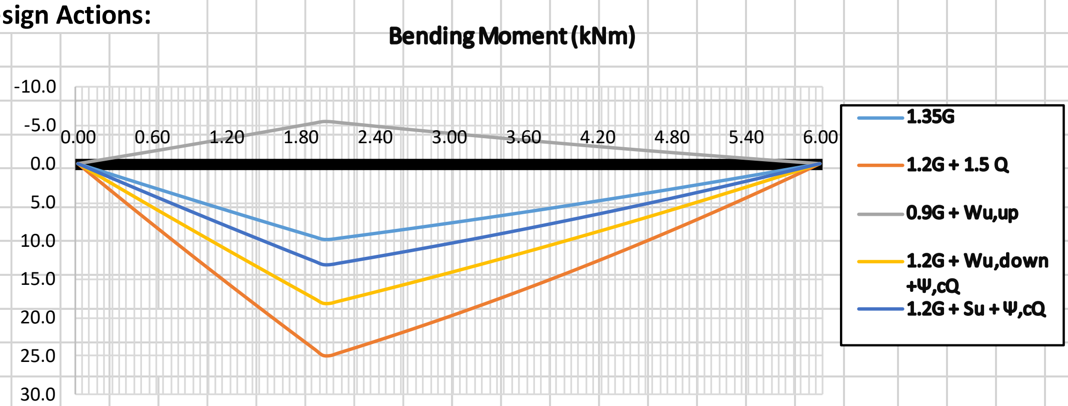


Dead	=	0.1	kN/m
Live	=	0.3	kN/m
Wind, up	=	0.0	kN/m
Wind, down	=	0.0	kN/m
Snow	=	0.0	kN/m

Load Cases:

Ultimate Limit State:		Serviceability Limit State:	
1.	1.2G + 1.5 Q	6.	G + Ψ,l Q (Ψ,l = 0.40)
2.	0.9G + Wu,up	7.	G + Ψ,s Q (Ψ,s = 0.70)
3.	1.2G + Wu,down +Ψ,cQ (Ψ,c = 0.40)	8.	Ws
4.	1.2G + Su + Ψ,cQ	9.	1kN Vibration
5.	1.35G		

Design Actions:

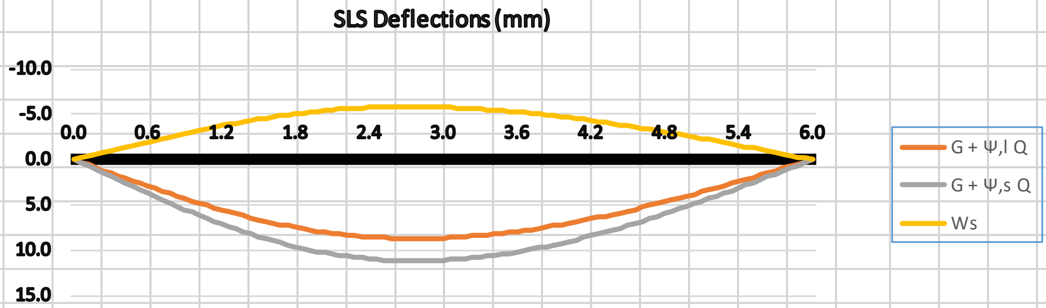


			Critical case:	1.2G + 1.5 Q					
Cases:			1.	2.	3.	4.	5.	**Max**	**Min**
Moment, M*		=	24.9	-5.5	18.1	13.0	9.8	**24.9kNm**	**-5.5kNm**
Shear, V*		=	13.2	2.7	9.4	6.9	5.1	**13.2kN**	
Reactions	RA*	=	13.2	-2.7	9.4	6.9	5.1	**13.2kN**	**-2.7kN**
	RB*	=	7.6	-1.1	5.2	3.9	2.9	**7.6kN**	**-1.1kN**

Capacity:

Restraint Conditions:		(LL) Lateral-Lateral		Comments:			
Load Height Position:		Shear Centre		Restrains provided by floor joists at 400crs			
Segment Length	L =	0.4	m				
Bending: ØMsx	=	27.2	kNm	Shear: ØVw	=	114.8	kN
ØMbx	=	27.2	kNm	Caqpacity Ratio	=	0.1	OK
Min (ØMsx, ØMbx)	=	27.2	kNm				
Caqpacity Ratio	=	0.9	OK				

Deflection:



Limits:	G + Ψ,l Q = 9mm	=	Span/690	
Beams where line-of-sight is across soffit, aceess by operators and mainten	Span/250	Y	OK	
Floor Joists/Beams Sag, Flooring ripple, Floor Supports plaster lined walls	Span/300	Y	OK	
Normal floor systems	Span/400	Y	OK	
Floor Vibration	<2mm	Y	OK	

S5.0 Wall Framing

Light Steel Frame Checks

- Design residential steelframing as per NASH Standard Part 2 within the scope
- Supplied framing is 89LC75 G550, plate classification = PC

Roof Rafters/Purlins Summary:

Location	Label	Span	Spacing	Wind Zone	Roof Type	Design Method	Type
Typ. Purlin		0.9m	0.9m	High	Light	Table 9.1	40RB55

Jamb Stud Summary:

Location	Stud Height	Loaded Dimension	Opening	Design Method	Number of jamb studs
Upper F	2.7	6.0m	0-0.7m	Table 7.15	1
Upper F	2.7	6.0m	0.7-2.0m	Table 7.15	2
Upper F	2.7	6.0m	2.0-3.3m	Table 7.15	3
Upper F	2.7	6.0m	3.3-4.6m	Table 7.15	4
Lower F	2.7	6.0m	0-1.4m	Table 7.17	2
Lower F	2.7	6.0m	1.4-2.1m	Table 7.17	3
Lower F	2.7	6.0m	2.1-2.7m	Table 7.17	4

Window Sill :

- Window sill and wind beam designs are deflections governs
- Limit out-of-plane deflection to Span/150 as per NASH Standard Part 1

Opening Size	=	3.50	m					
Tributry Width	=	1.35	m		Limiting Deflection =	23	mm	
SLS Wind Pressure	=	0.73	kPa		Deflection =	16	mm	OK
150LC75			Ix =	0.60	x10^6 mm4	E =	200	Gpa
Number of sill section required			=	**1**				

Typ. Single sill member is okay for opening up to 3.5m

Top Plates:

- Upper floor studs are required to be aligned with trusses above to avoid loading on top plate
- Lower floor floor joists loading are supporitng by boundary joists

Wall Studs:

Location	Stud Height	Loaded Dimension	Spacing	Design Method	Stud Type	
Upper Ex	2.7m	Max 6.0m	600crs	Table 7.5	SC	
Upper Int	2.7m	Max 6.0m	600crs	Table 7.7	SC	
Lower Ex	2.7m	Max 6.0m	600crs	Table 7.9	SC	
Lower Int	2.7m	Max 6.0m	600crs	SED	SC	(with double screws)

Lightsteel-Wall Studs

- Limit out-of-plane deflections as per NASH Standard Part 1
- Assume Q=0.7kN Soft body impact is shared by two studs

Label: LBW 2.7
Location: Internal

Stud Height	=	2.70	m						
Stud Spacing	=	0.60	m	Limiting deflectior	=	18	mm	(Ws, Height/150)	
SLS Wind Pressure	=	0.50	kPa		=	12	mm	(Impact, Height/200)	
89LC75 G550			Iy =	0.17	x10^6 mm4	E =	200	Gpa	
			SLS, w	=	0.30	kN/m			
		Ws	Deflection		6	mm OK	Impact	5	mm OK
			ULS, w	=	0.45	kN/m			
			My*	=	0.41	kNm			
				=	0.20	kNm per section			

Axial Loads:

Floor	DL	=	0.50	kPa	x	5.20	m	=	2.6	kN/m
	LL	=	1.50	kPa	x	5.20	m	=	7.8	kN/m
						1.2DL+ 1.5LL		=	15	kN/m
							Nc*	=	8.9	kN/stud

89LC75 provided fyNc= 9.3kN > 8.9kN, therefore Okay

Provide double screw fixing to top and bottom plate

S6.0 Floor Framing

Beam Label:	**J1**	Location:	**Floor Joists**	**Bending**	**OK**
Span (L):	**5.0m**	Beam Section:	**LC290x45x15x2.5**	**Shear**	**OK**
			Single Member	**Combined**	**N/A**
				Deflection	**OK**

Total Span (L) = 5m

Max	4.5kN	=RA*	RB* =	4.5kN	Max
Min	0.8kN			0.8kN	Min

Loadings: sw = 0.07kN/m

Floor	Dead	=	0.50	kPa	x	0.6m	=	0.3	kN/m	Ψ,l	=	0.4
	Live	=	1.50	kPa	x	0.6m	=	0.9	kN/m	Ψ,s	=	0.7

Load Cases:

Ultimate Limit State:			(w)		Serviceability Limit State:			(w)	
1.	1.2G + 1.5 Q	=	1.8	kN/m	6.	G + Ψ,l Q	=	0.7	kN/m
2.	0.9G + Wu,up	=	0.3	kN/m	7.	G + Ψ,s Q	=	1.0	kN/m
3.	1.2G+Wu,dw+ΨcQ	=	0.8	kN/m	8.	Ws	=	0.0	kN/m
4.	1.2G+Su+ΨcQ	=	0.8	kN/m	9.	G + Q (Vibration)	=	1.2	kN/m
5.	1.35G	=	0.5	kN/m					

Design Actions:

Cases:		1.	2.	3.	4.	5.	Max	Min
Moment, M*= W x L^2/	=	5.6	1.1	2.5	2.5	1.6	**5.6kNm**	**1.1kNm**
Shear, V* = W x L /2	=	4.5	0.8	2.0	2.0	1.3	**4.5kN**	**0.8kN**

Capacity:

Bending:

Cb =	1.0	*(eq D2.1.1(2))*		Mo =	28.8	kNm	*(D2.1 Appendix D)*
Ley =	600	mm	*eff. length for buckling about y-axes*	Mol =	17.8	kNm	*eq. 7.2.2.3(4)*
Lez =	600	mm	*eff. length for twisting*	Mod =	17.8	kNm	*eq. 7.2.2.4(4)*
ϕ_b =	0.9	*Clause 7.2.2.1*		**ØMb =**	**ØMin(Mbe, Mbl, Mbd)**		
fy =	250	Mpa		=	**13.6**	**kNm**	
			Capacity Ratio	=	**0.4**	OK	

Shear:

a =	5000	mm	*shear panel length*
d1 =	290	mm	*depth of web*
kv =	6.31	*eq. D3(4)*	
ØVv =	**57.7**	**kN**	OK

Combined bending and shear:

Maximum bending and shear at same location ? No

$(M^*/\phi_b M_s)^2+(V^*/\phi_v V_v)^2$ = **0.05** N/A

Deflection:

Limits: **I,efftive = 10.30 x10^6mm4** G + Ψ,l Q = 3mm = Span/1722

Beams where line-of-sight is across soffit, aceess by operators and mainten	Span/250	Y	OK
Floor Joists/Beams Sag, Flooring ripple, Floor Supports plaster lined walls	Span/300	Y	OK

Floor Vibration: G + Q = 5mm = Span/1055

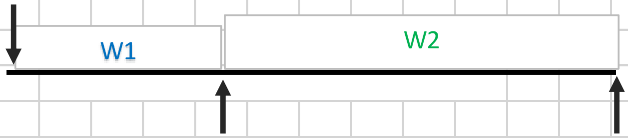
Floor Vibration L/500, <12mm	Span/500	Y	OK

Joist section effective Ie =	10.30	x10^6mm4	Width of floor, B =	12.0m
Joist modulus of elasticity =	200	Gpa	Floor thickness =	19mm
Floor modulus of elasticity =	2.7	Gpa	Joists Spacing =	600mm

Kd = 0.883 - 0.34 log10[(kc/kb)+0.44] =	0.74			w = 81.5494 kg/m2
kc = Ec x tf^3 x L/12/ s^3 =	35.72			
kb = Eb x Ib /L^3 =	16.48			
Kx = Eb * Ib/ s =	3433333			
Ky = Ef x tf^3 /12 =	1543			
Δ = kd x (L^3/48 Eb Ib) =	0.9	<	2mm	OK
Natural frequency =	12.9	>	8Hz	OK

Beam Label:	**J2**	Location:	**Cantilever Joist**		**Bending**	**OK**
BackSpan (L)	**6.0m**	Beam Section:	**LC290x45x15x2.5**		**Shear**	**OK**
Cant.Span (C)	**1.4m**	Fire Rated:	**N/A**		**Deflection**	**OK**
					Limited Temp	**NA**

sw = 0.07kN/m



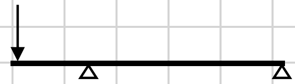
C.Span A = 1.4m   B.Span B = 6m

RA* =	DL =	2.5kN	Max	11.6kN	RB* =	DL =	0.9kN	4.4kN	Max
	LL =	5.8kN	Min	2.2kN		LL =	2.2kN	0.8kN	Min
	-Wu =	0.0kN				-Wu =	0.0kN		
	Wu =	0.0kN				Wu =	0.0kN		
	Su =	0.0kN				Su =	0.0kN		

Loadings:

Point Load, PL: (Tributy Area)

Wall	Dead =	0.30	kPa	x	1.0m	x	0.6m	=	0.2	kN
	Live =		kPa	x		x		=	0.0	kN
Stair	Dead =	0.50	kPa	x	1.5m	x	0.6m	=	0.5	kN
	Live =	1.50	kPa	x	1.5m	x	0.6m	=	1.4	kN



Dead =	0.63	kN
Live =	1.35	kN
Wind, up =	0.00	kN
Wind, down =	0.00	kN
Snow =	0.00	kN

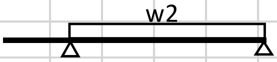
UDL Load, W1: (Tributy Area)

Floor	Dead =	0.50	kPa	x	0.6m	=	0.3	kN/m
	Live =	1.50	kPa	x	0.6m	=	0.9	kN/m

Dead =	0.37	kN/m
Live =	0.90	kN/m
Wind, up =	0.00	kN/m
Wind, down =	0.00	kN/m
Snow =	0.00	kN/m

UDL Load, W2: (Tributy Area)

Floor	Dead =	0.50	kPa	x	0.6m	=	0.3	kN/m
	Live =	1.50	kPa	x	0.6m	=	0.9	kN/m



Dead =	0.37	kN/m
Live =	0.90	kN/m
Wind, up =	0.00	kN/m
Wind, down =	0.00	kN/m
Snow =	0.00	kN/m

Load Cases:

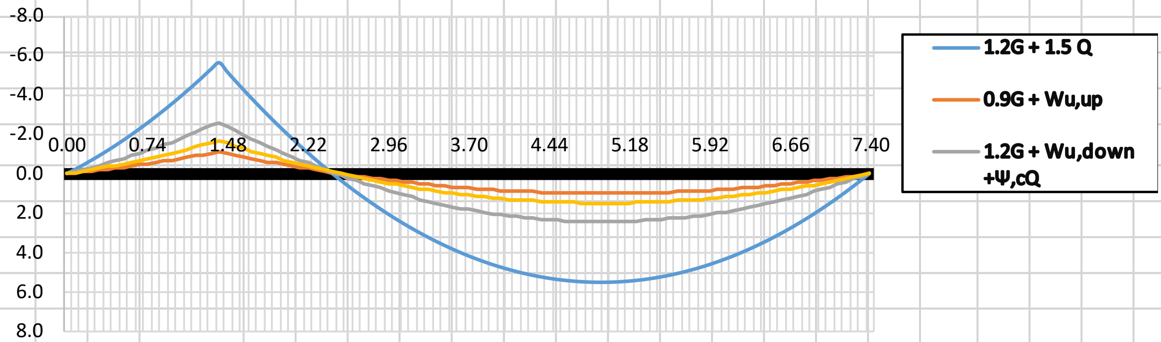
Ultimate Limit State:

1. 1.2G + 1.5 Q
2. 0.9G + Wu,up
3. 1.2G + Wu,down +Ψ,cQ (Ψ,c = 0.40)
4. 1.2G + Su + Ψ,cQ
5. 1.35G

Serviceability Limit State:

6. G + Ψ,l Q (Ψ,l = 0.40)
7. G + Ψ,s Q (Ψ,s = 0.70)
8. Ws
9. 1kN Vibration

Design Actions:

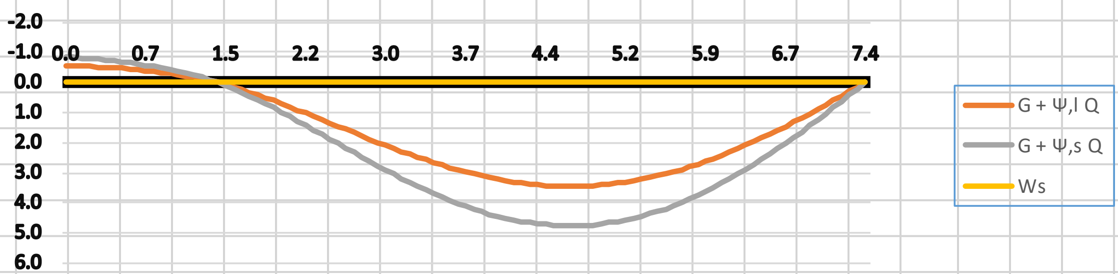


	Critical case:		1.2G + 1.5 Q						
Cases:			1.	2.	3.	4.	5.	**Max**	**Min**
Moment	M*(-)	=	-5.6	-1.1	-2.6	-2.6	-1.7		**-5.6kNm**
	M*(+)	=	5.5	1.0	2.4	2.4	1.5	**5.5kNm**	
Shear, V*		=	5.3	1.0	2.4	2.4	0.0	**5.3kN**	
Reactions	RA*	=	11.6	2.2	5.3	5.3	3.3	**11.6kN**	**2.2kN**
	RB*	=	4.4	0.8	2.0	2.0	1.2	**4.4kN**	**0.8kN**

Capacity:

Restraint Conditions:	(LL) Lateral-Lateral		Comments:			
Load Height Position:	Top Flange					
Segment Length L =	0.6	m				
Bending: ØMsx =	13.6	kNm	Shear:	ØVw =	57.7	kN
ØMbx =	13.6	kNm	Caqpacity Ratio	=	0.1	OK
Min (ØMsx, ØMbx) =	13.6	kNm				
Caqpacity Ratio =	0.4	OK				

Deflection:

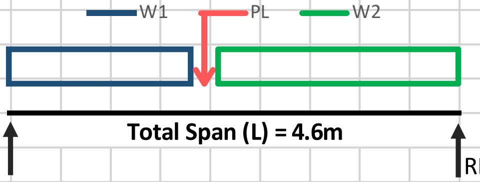


Limits:	G + Ψ,l Q @Cantilever End	=	3mm	=	Spanx2/3480	
	@Backspan	=	3mm	=	Span/1740	
Beams where line-of-sight is across soffit, aceess by operators and mainten			Span/250		Y	OK
Floor Joists/Beams Sag, Flooring ripple, Floor Supports plaster lined walls			Span/300		Y	OK
Floor Vibration (shared by two joists)			<2mm		Y	OK

Beam Label:	J3	Location:	Over Rumpus
Span (L):	4.6m	Beam Section:	LC290x45x15x2.5
PL Location, a =	2.0m	Fire Rated:	N/A

Bending	**OK**
Shear	**OK**
Deflection	**OK**
Limited Temp	**NA**

sw = 0.07kN/m



Total Span (L) = 4.6m

Max	5.7kN	=RA*	RB* =	5.3kN	Max
Min	-0.8kN			-0.4kN	Min

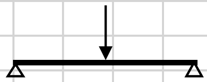
Span A = 2m — Span B = 2.6m

Loadings:

Point Load, PL: (Tributy Area)

Roof	Dead	=	0.27	kPa	x	5.1m	x	0.6m	=	0.8	kN
	Live	=	0.25	kPa	x	5.1m	x	0.6m	=	0.8	kN
	Wind, up	=	-1.29	kPa	x	5.1m	x	0.6m	=	-3.9	kN
	Wind, down	=	0.54	kPa	x	5.1m	x	0.6m	=	1.7	kN
	Snow	=		kPa	x		x		=	0.0	kN
Wall	Dead	=	0.30	kPa	x	2.7m	x	0.6m	=	0.5	kN
	Live	=		kPa	x		x		=	0.0	kN
Floor	Dead	=	0.50	kPa	x		x		=	0.0	kN
	Live	=	1.50	kPa	x		x		=	0.0	kN

PL

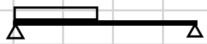


Dead	=	1.3	kN
Live	=	0.8	kN
Wind, up	=	-3.9	kN
Wind, down	=	1.7	kN
Snow	=	0.0	kN

UDL Load, W1: (Tributy Area)

Floor	Dead	=	0.50	kPa	x	0.6m	=	0.3	kN/m
	Live	=	1.50	kPa	x	0.6m	=	0.9	kN/m

w1

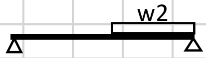


Dead	=	0.4	kN/m
Live	=	0.9	kN/m
Wind, up	=	0.0	kN/m
Wind, down	=	0.0	kN/m
Snow	=	0.0	kN/m

UDL Load, W2: (Tributy Area)

Floor	Dead	=	0.50	kPa	x	0.6m	=	0.3	kN/m
	Live	=	1.50	kPa	x	0.6m	=	0.9	kN/m

w2



Dead	=	0.4	kN/m
Live	=	0.9	kN/m
Wind, up	=	0.0	kN/m
Wind, down	=	0.0	kN/m
Snow	=	0.0	kN/m

Load Cases:

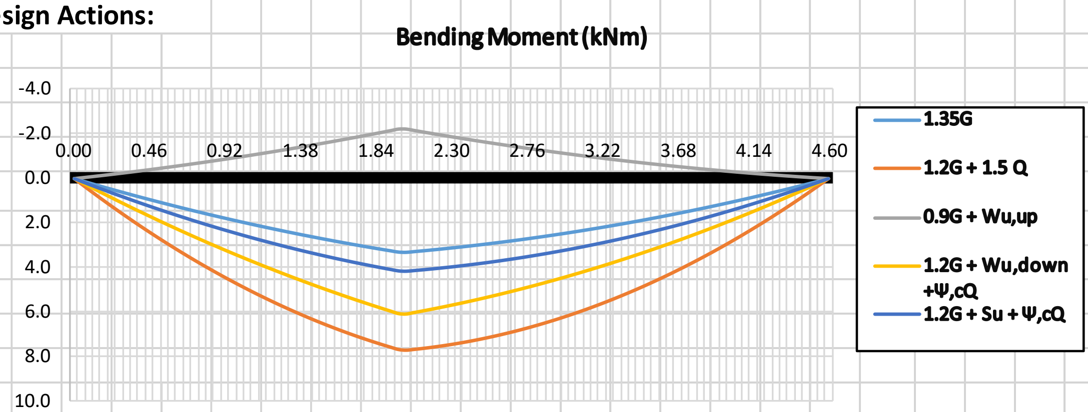
Ultimate Limit State:

1. 1.2G + 1.5 Q
2. 0.9G + Wu,up
3. 1.2G + Wu,down +Ψ,cQ (Ψ,c = 0.40)
4. 1.2G + Su + Ψ,cQ
5. 1.35G

Serviceability Limit State:

6. G + Ψ,l Q (Ψ,l = 0.40)
7. G + Ψ,s Q (Ψ,s = 0.70)
8. Ws
9. 1kN Vibration

Design Actions:

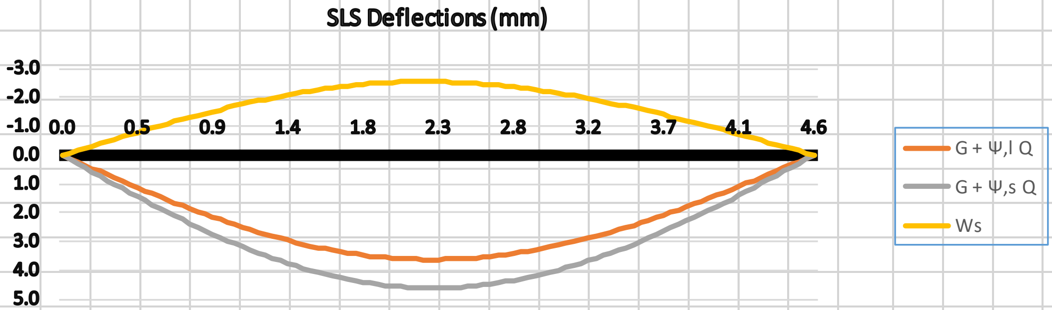


	Critical case:		1.2G + 1.5 Q						
Cases:			1.	2.	3.	4.	5.	**Max**	**Min**
Moment, M*		=	7.7	-2.2	6.1	4.2	3.3	**7.7kNm**	**-2.2kNm**
Shear, V*		=	5.7	0.8	3.8	2.9	2.2	**5.7kN**	
Reactions	RA*	=	5.7	-0.8	3.8	2.9	2.2	**5.7kN**	**-0.8kN**
	RB*	=	5.3	-0.4	3.4	2.7	1.9	**5.3kN**	**-0.4kN**

Capacity:

Restraint Conditions:		(LL) Lateral-Lateral		Comments:			
Load Height Position:		Top Flange					
Segment Length	L =	0.6	m				
Bending: ØMsx	=	13.6	kNm	Shear: ØVw	=	57.7	kN
ØMbx	=	13.6	kNm	Caqpacity Ratio	=	0.1	OK
Min (ØMsx, ØMbx) =		13.6	kNm				
Caqpacity Ratio	=	0.6	OK				

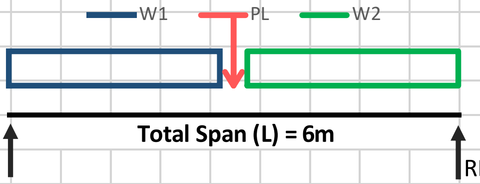
Deflection:



Limits: G + Ψ,l Q =	4mm	=	Span/1270	
Beams where line-of-sight is across soffit, aceess by operators and mainten	Span/250		Y	OK
Floor Joists/Beams Sag, Flooring ripple, Floor Supports plaster lined walls	Span/300		Y	OK
Floor Vibration	<2mm		Y	OK
Ws =	3mm	=	Span/1807	
Ceilings with plaster finish	Span/200		Y	OK

Beam Label:	**J4**	Location:	**Over Rumpus**		**Bending**	**OK**
Span (L):	**6.0m**	Beam Section:	**LC290x45x15x2.5**		**Shear**	**OK**
PL Location, a =	**3.0m**	Fire Rated:	**N/A**		**Deflection**	**OK**
					Limited Temp	**NA**

sw = 0.07kN/m



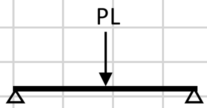
Max	6.9kN	=RA*	RB* =	6.9kN	Max
Min	-0.7kN			-0.7kN	Min

Span A = 3m — Span B = 3m

Loadings:

Point Load, PL: (Tributy Area)

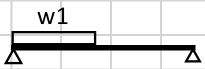
Roof	Dead	=	0.27	kPa	x	6.0m	x	0.6m	= 1.0	kN
	Live	=	0.25	kPa	x	6.0m	x	0.6m	= 0.9	kN
	Wind, up	=	-1.29	kPa	x	6.0m	x	0.6m	= -4.6	kN
	Wind, down	=	0.54	kPa	x	6.0m	x	0.6m	= 1.9	kN
	Snow	=		kPa	x		x		= 0.0	kN
Wall	Dead	=	0.30	kPa	x	2.7m	x	0.6m	= 0.5	kN
	Live	=		kPa	x		x		= 0.0	kN
Floor	Dead	=	0.50	kPa	x		x		= 0.0	kN
	Live	=	1.50	kPa	x		x		= 0.0	kN



Dead =	1.5	kN
Live =	0.9	kN
Wind, up =	-4.6	kN
Wind, down =	1.9	kN
Snow =	0.0	kN

UDL Load, W1: (Tributy Area)

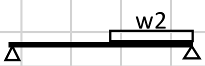
Floor	Dead	=	0.50	kPa	x	0.6m	= 0.3	kN/m
	Live	=	1.50	kPa	x	0.6m	= 0.9	kN/m



Dead =	0.4	kN/m
Live =	0.9	kN/m
Wind, up =	0.0	kN/m
Wind, down =	0.0	kN/m
Snow =	0.0	kN/m

UDL Load, W2: (Tributy Area)

Floor	Dead	=	0.50	kPa	x	0.6m	= 0.3	kN/m
	Live	=	1.50	kPa	x	0.6m	= 0.9	kN/m

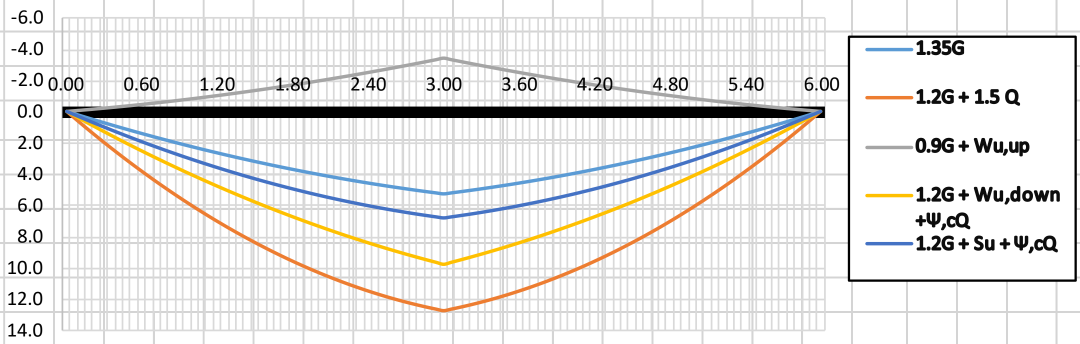


Dead =	0.4	kN/m
Live =	0.9	kN/m
Wind, up =	0.0	kN/m
Wind, down =	0.0	kN/m
Snow =	0.0	kN/m

Load Cases:

Ultimate Limit State:		Serviceability Limit State:	
1.	1.2G + 1.5 Q	6.	G + Ψ,l Q (Ψ,l = 0.40)
2.	0.9G + Wu,up	7.	G + Ψ,s Q (Ψ,s = 0.70)
3.	1.2G + Wu,down +Ψ,cQ (Ψ,c = 0.40)	8.	Ws
4.	1.2G + Su + Ψ,cQ	9.	1kN Vibration
5.	1.35G		

Design Actions:

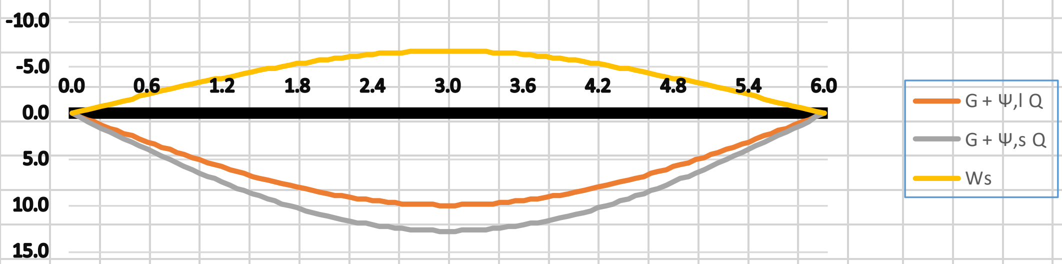


			Critical case:	1.2G + 1.5 Q						
Cases:				1.	2.	3.	4.	5.	Max	Min
Moment, M*		=		12.7	-3.5	9.7	6.8	5.2	**12.7kNm**	**-3.5kNm**
Shear, V*		=		6.9	0.7	4.4	3.5	2.5	**6.9kN**	
Reactions	RA*	=		6.9	-0.7	4.4	3.5	2.5	**6.9kN**	**-0.7kN**
	RB*	=		6.9	-0.7	4.4	3.5	2.5	**6.9kN**	**-0.7kN**

Capacity:

Restraint Conditions:		(LL) Lateral-Lateral		Comments:				
Load Height Position:		Top Flange						
Segment Length	L =	0.6	m					
Bending: ØMsx	=	13.6	kNm	Shear:	ØVw	=	57.7	kN
ØMbx	=	13.6	kNm	Caqpacity Ratio		=	0.1	OK
Min (ØMsx, ØMbx) =		13.6	kNm					
Caqpacity Ratio	=	0.9	OK					

Deflection:



Limits: G + Ψ,l Q =	10mm	=	Span/603	
Beams where line-of-sight is across soffit, aceess by operators and mainten	Span/250		Y	OK
Floor Joists/Beams Sag, Flooring ripple, Floor Supports plaster lined walls	Span/300		Y	OK
Ws =	7mm	=	Span/883	
Ceilings with plaster finish	Span/200		Y	OK

S7.0 Miscellaneous

Lightsteel - Balustrade

Loading:

AS/NZS1170.1 Table 3.3:

Type of Occupancy	Specific Uses	Top Edge			Infill	
		Horiz.	Vert.	PL	Horiz.	PL
A	Stair	0.35kN/m	0.35kN/m	0.60kN	0.50kPa	0.25kN

Vertical balustrade spacing = 0.60 m

Balustrade Height = 1.20 m

Top Rail: (Load Factor)

Cases:	1.	M*	=	0.35	x	1.50	x	0.60	^2/8	=	0.02	kNm
	2.	M*	=	0.60	x	1.50	x	0.60	/4	=	0.14	kNm
	2.	M*	=	0.50	x	1.50	x	1.20	x L^2/8	=	0.02	kNm
									M*	=	0.14	kNm

Try 89LC75 G550

Capacity Ratio = **0.2** OK

Post: (Load Factor)

Cases:	1.	M*	=	0.35	x	1.50	x	0.60	x H	=	0.38	kNm
	2.	M*	=	0.60	x	1.50	x	1.20	/2	=	0.54	kNm
	2.	M*	=	0.50	x	1.50	x	1.20	/2 x L x H	=	0.32	kNm
									M*	=	0.54	kNm

Try 89LC75 G550

Capacity Ratio = **0.5** OK

Base Connection:

M* = 0.54 kNm jd = 210 mm

Tension Nt* = M* / jd = 2.6 kN

<u>Provide 3/Type 17 10g screws, fyNt on 0.75BMT = 1kN/screw x 3 = 3kN</u>

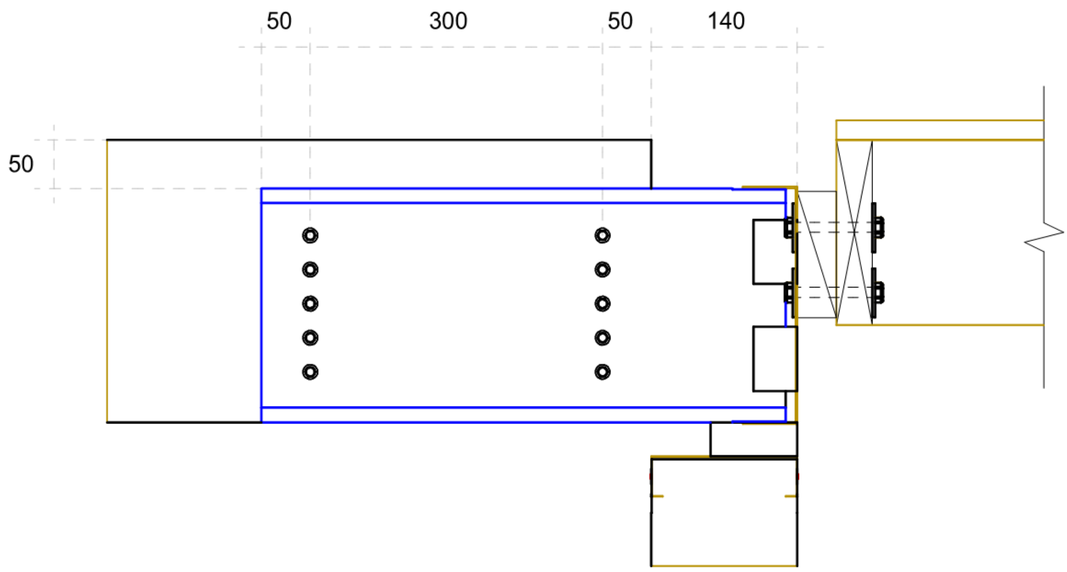
FLOOR JOIST STEP END CONNECTION:

End shear force from joist:
1.2DL + 1.5LL: (1.2 x 0.5kPa + 1.5 x 1.5kPa) x 6m span x 0.45m spacing / 2 = 3.85kN
V* = 3.85kN x 0.5m / 0.3m = 6.4kN/line

Provided two lines of 5/12g screws, V*,screw = 6.4kN/5 = 1.3kN/screw, 12Gscrew on 1.8BMT steel are ok by inspection

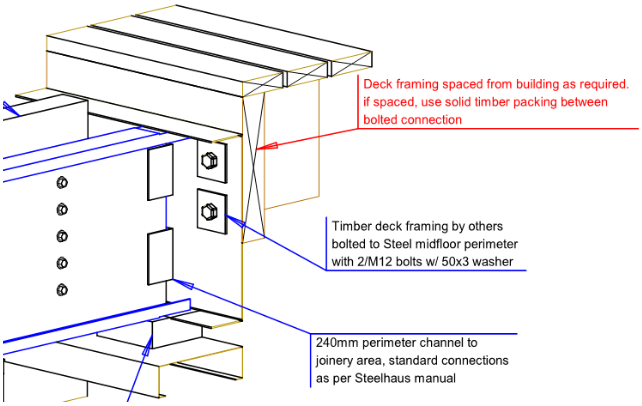
End shear force from joist:
1.2DL + 1.5LL: (1.2 x 0.5kPa + 1.5 x 1.5kPa) x 6m span x 0.45m spacing / 2 = 3.85kN
V* = 3.85kN x 0.5m / 0.3m = 6.4kN/line

Provided two lines of 5/12g screws, V*,screw = 6.4kN/5 = 1.3kN/screw, 12Gscrew on 1.8BMT steel are ok by inspection



TIMBER DECK CONNECTION:

End shear force from DECK:
1.2DL + 1.5LL: (1.2 x 0.3kPa + 1.5 x 2.0kPa) x 3.2m span/ 2 = 5.4kN



Provided two M12 bolts, bearing on lightsteel:

AS/NZS 4600:2005 Clause 5.3.4.2 - Bearing capacity without considering bolt hole deformation				
	Steel grade	G250		
	t	1.85	mm	
	d_f	12	mm	
	α	1.00		table 5.3.4.2(a)
	ϕ	0.6		
	d_f/t	6.49		
	C	3.00		table 5.3.4.2(b)
	f_u	320	MPa	
	ϕV_b	12.79	kN	
AS/NZS 4600:2005 Clause 5.3.4.3 - Bearing capacity at a bolt hole deformation of 6mm				
	Steel grade	G250		
	t	1.85	mm	
	d_f	12	mm	
	ϕ	0.65		
	f_u	320	MPa	
	ϕV_b	8.63	kN	

Shear capacity on timber:
fyQn = 0.7 x 0.8 (k1) x 0.7 (green) x 6.97kN/bolt = 2.7kN, timber capacity governs

5.4kN / (2 x 2.7) = 1

Provide 2/M12 bolts at 900crs

Design Navigator

Design Navigator H1 Compliance Report

Project Summary

H1 Report created by:	Don Cowie Draughting & Design Ltd
Project Name:	Copy of 2/136 SH26
Client:	M. Cameron
Lot No:	Lot 2 DP556335
Comment:	
Project Id:	152854
Report Date:	22/02/2022

Compliance Result

This report shows compliance of the design with Clause H1 Fourth edition Amendment 4 from November 2019 and the R-value targets of Clause E3 Second edition Amendment 7 from November 2020.

This building complies with H1 via the following methods:

- the Calculation Method in NZS4218:2009

H1 Compliance Details

NZS4218:2009 Calculation Method Compliance

The use of the Calculation Method is permitted .

In order to comply the Actual Heat Loss must be the same or smaller than the Reference Heat Loss AND all component R-values must be the same or larger than 50% of the R-values in the '50% Rule' table below. This design **complies** with the NZS4218:2009 Calculation Method.

HeatLoss:

Reference building	Proposed building
636	589

Minimum R-values ("50% rule"):

	Permitted Minimum	Proposed Minimum	
Floor:	0.65	3.2	✓
Non-solid Walls:	0.95	2.4	✓
Roof:	1.45	2.46	✓

Hamilton City Council
Te kaunihera o Kirikiriroa

BUILDING UNIT APPROVED

BC Number - DD007.2021.00043914.001

The Reference building has the following areas and R-values.

	Area (m²)		Non-solid	Solid Timber	Other Solid
			100.0	0.0%	0.0%
Floor:	Area: 225 m²	R-values:	1.3	1.3	1.5
Walls excl. glazing:	Area: 101.2 m²	R-values:	1.9	1.2	1
Glazing (up to 30%):	Area: 60.4 m²	R-values:	0.26	0.26	0.26
Glazing (surplus of 30%):	Area: 39.8 m²	R-values:	0.4	0.34	0.31
Roof:	Area: 225 m²	R-values:	2.9	3.5	3.5
Heat Loss:			636	671	676

For mixed constructions the heat loss of the reference building is calculated as the sum of the heat losses for each type of wall construction multiplied by the fraction of the wall area of each type. This approach is based on clause 4.2.6 of NZS4218:2009.
There are no skylights in the reference building. The reference building roof area is the sum of the proposed building roof and skylight areas.

Compliance with Clause E3

This building complies with the R-value targets in NZBC Clause E3 .

Component	Minimum R-value	Project R-value
Framed wall constructions with cavities	1.5	2.4
Single skin masonry wall without a cavity	0.6	
Solid timber wall no less than 60 mm thick	0.6	
Roof or ceilings	1.5	2.46

Hamilton City Council
Te kaunihera o Kirikiriroa

BUILDING UNIT
APPROVED

BC Number - DD007.2021.00043914.001

Design Details

Building Dimensions

Floor Area	225	m²
Gross Wall Area	201.4	m²
Net Wall Area	101.2	m²
Wall (North) Area	30.9	m²
Wall (East, South and West) Area	70.3	m²
Gross Roof Area	225	m²
Net Roof Area	225	m²
Glazing Area	100.2	m²
Window (North) Area	10.4	m²
Window (East, South and West) Area	89.8	m²
Skylight Area	0	m²

Glazing Areas

Total Vertical Glazing Percentage	49.8	%
East, South and West Window Percentage	56.1	%
Total over 30%	yes	
East, South and West over 30%	yes	
Total over 50%	no	
max. Skylight Area for Schedule Method	3.38	m²
Skylights over Schedule Method Limit	no	
Decorative Glazing	0	m²
Decorative Glazing over 3m²	no	

Information required for BPI calculation

Living Floor Area	225	m² Note: This includes also internal floors.
Average Room Height	2.7	m
Thermal Mass Level	Medium weight	Slab floor with some carpeting or direct glued timber, timber framed walls.

Climate

Location	Hamilton & Ruakura
Climate Zone	2

Heat Loss Details

	ID	Or.	Width	Height	Gross Area	Net Area	R*	Heat Loss	Shad. Coeff **	Solid ***
Floors										
Floor 1	Ground Floor				225.0	225.0	3.20	70.3		
Walls										
Wall 1	Wall 1	N	15.3	2.7	41.3	30.9	2.40	12.9		C
Window 1-1	N1		0.6	2.6		1.6	0.26	6.0	0.00	
Window 1-2	N2		0.6	2.6		1.6	0.26	6.0	0.00	
Window 1-3	N3		1.6	2.6		4.2	0.26	16.0	0.00	
Window 1-4	N4		0.6	2.6		1.6	0.26	6.0	0.00	
Window 1-5	N5		0.6	2.6		1.6	0.26	6.0	0.00	
Wall 2	Wall 2	W	22.0	2.7	59.4	16.2	2.40	6.8		C
Window 2-1	W1		3.0	2.6		7.8	0.26	30.0	0.00	
Window 2-2	W2		8.0	2.6		20.8	0.26	80.0	0.00	
Window 2-3	W3		2.0	2.6		5.2	0.26	20.0	0.00	
Window 2-4	W4		3.6	2.6		9.4	0.26	36.0	0.00	
Wall 3	Wall 3	S	15.3	2.7	41.3	33.6	2.40	14.0		C
Window 3-1	S1		0.6	1.6		1.0	0.26	3.7	0.00	
Window 3-2	S2		0.6	1.6		1.0	0.26	3.7	0.00	
Window 3-3	S3		1.6	1.6		2.6	0.26	9.8	0.00	
Window 3-4	S4		0.8	1.6		1.3	0.26	4.9	0.00	
Window 3-5	S5		0.6	1.6		1.0	0.26	3.7	0.00	
Window 3-6	S6		0.6	1.6		1.0	0.26	3.7	0.00	
Wall 4	Wall 4	E	22.0	2.7	59.4	20.4	2.40	8.5		C
Window 4-1	E1		1.8	2.6		4.7	0.26	18.0	0.00	
Window 4-2	E2		1.8	2.6		4.7	0.26	18.0	0.00	
Window 4-3	E3		8.0	2.6		20.8	0.26	80.0	0.00	
Window 4-4	E4		3.4	2.6		8.8	0.26	34.0	0.00	
Roofs										
Roof 1	Roof				225.0	225.0	2.46	91.5		
Total Heat Loss								589.5		

* Any concrete slab-on-ground floor regardless of its dimensions can be assumed to have an R-value of at least R-1.3 (H1/AS1 2.1.5).
** The Shading Coefficient is only required for BPI calculations.
*** C: Cavity Construction (any construction that is not solid), T: Solid Timber, S: Other Solid Construction (Note that the use of solid timber and other solid construction types is discretional, i.e. solid timber walls and other solid walls can be treated as if they are non-solid (NZS4218:2009 section 4.1.3.).)

Wall Construction Details

Name: Walltype 1

2.40 m²°C/W

Type: Wall: Steel Frame with vented Cavity

external surface 0.03

Cladding : Weathertex 9.5mm
R-value: 0.05

Air Barrier : IBS RigidRAP XT
R-value: 0.39

Steel Frame & Cavity : 150x50x1.15mm, studs @ 600mm, dwangs @ 800mm

Wall Frame Area: 16.7%	*Cavity Area: 83.3%*
15-90mm vented cavity (all R-values on ext. side of cavity will be halved), R: 0.08	15-90mm vented cavity (all R-values on ext. side of cavity will be halved), R: 0.08
Thermal Break : IBS RigidRAP XT *R-value: 0.39*	Insulation : 2.8
Framing : *R-value: 0.19*	still Airgap: none *R-value: 0.00*

Wall Lining : Gypsum plasterboard 10mm
R-value: 0.04

internal surface 0.09

Roof Construction Details

Name: Rooftype 1

2.46 m²°C/W

Type: Roof: Steel framed Roof, battened flat Ceiling

external surface 0.03

Roofing : Corrugate iron with building paper
R-value: 0.01

Insulation : 0

Steel Frame & Cavity : 89x41x0.75mm rafters or joists @ 900mm, battens covered with insulation

Roof Frame Area: 4.6%	*Cavity Area: 95.4%*
Roof space (still air) 0.11	Roof space (still air) 0.11
Framing : *R-value: 0.16*	Insulation : 3.6
Thermal Break : none *R-value: 0.00*	

Roof Lining : none
R-value: 0.00

internal surface 0.09

Non-IC-rated recessed downlights

Ceiling Area [m²]: 225 Number of downlights: Clearance from lamp holder side [m]:

Hamilton City Council

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BC Number - DD007.2021.00043914.001

Glazing Details

Name: Glazingtype 1

0.26 m²°C/W

Type: Aluminium Frame

Glass Type : IGU Grey - 12mm gap

R-value: 0.26

Solar Heat Gain Coefficient (SHGC): 0.50

Hamilton City Council

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BC Number - DD007.2021.00043914.001

UNEX
ARCHITECTURAL BARRIERS

Specifically prepared on:	21-02-2022
Product | Fixture:	Vetro(NoRail) | FS.2T.02.00-Wet Timber, Top Fix, 90mm CRS
Indicative Linear Meterage:	Approx. 37 m
Project Name:	Cameron Residence
Site Address:	Lot 2 SH26 Hamilton
Legal Description:	Lot 2 DP 556335

SPECIFIER DETAILS

Area where balustrade is installed:	First Floor level deck
Company:	Don Cowie Draughting & Design Services Ltd
Phone:	0276041392
Email:	dcowie_design@hotmail.com
Website:	

Hamilton City Council

BUILDING UNIT
APPROVED

BC Number - DD007.2021.00043914.001

PRODUCER STATEMENT – PS1 - DESIGN

Issued By: ALAN H. THOMAS – CONSULTING ENGINEER

Issued To: TO WHOM IT MAY CONCERN

Date: 21-02-2022

Project: Cameron Residence

Site Address: Lot 2 SH26 Hamilton

Legal Description: Lot 2 DP 556335

This statement applies to the Vetro balustrade style as supplied by Unex Systems (NZ) Ltd for the

(1) First Floor level deck

Construction details are to be in accordance with the **UNEX Fabricator's Manual – Semi-frameless Glass NZBAL-C12.0**. The maximum post spacing must not exceed the distance given on the following Manual pages, copies enclosed.

(2) Vetro(NoRail) | FS.2T.02.00-Wet Timber, Top Fix, 90mm CRS

I, Alan H Thomas hold a current Professional Indemnity Insurance policy for no less than $200,000 and have been engaged by Unex Systems (NZ) Ltd to provide design services for their **UNEX** balustrading in respect of Clauses B1 and F2 of the Building Regulations. The design has been prepared in accordance with Clauses B1/VM1 and B1/AS1. The design of the load carrying members and their connections have been verified by load testing where applicable.

Materials and corrosion consultants have been engaged by Unex Systems (NZ) Ltd to provide a Durability Appraisal in respect of the requirements of clause B2 of the New Zealand Building Code. The appraisal has been prepared in accordance with verification method B2/VM1 of the approved documents issued by the Ministry of Business, Innovation & Employment, compliance with Clause B2 relies entirely on this appraisal.

I believe on reasonable grounds that subject to:

1. All proprietary products meeting their performance specification requirements.
2. The general arrangement and dimensions of the balustrade members, post spacing, fixing details and assembly methods being in accordance with the instructions in the current **UNEX Fabricator's Manual - Semi-frameless Glass NZBAL-C12.0** and the above details.

The design of the balustrade and its fixings (excluding the supporting structure) complies with clauses B1, B2, F2 and F4 of the New Zealand Building Code.

Signed by ALAN H THOMAS AH Thomas 18/9/2020

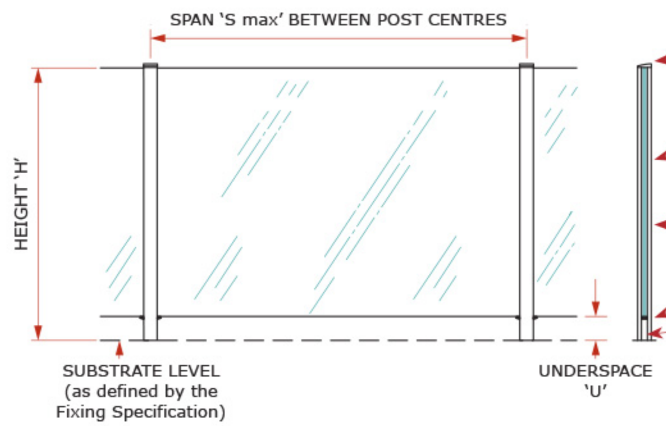
(Qualifying Engineer in accordance with Clause 1.0.3(e) of B1/VM1). Auckland Council registration number 1838.
CONSULTING ENGINEER Southern Building Controls Group Author number PSA/2010/45.
42 Bryant Road, Te Rapa, HAMILTON 3200
PO Box 92, HAMILTON 3240
Telephone: 07 850 9464

Note: This Producer Statement must be accompanied by the relevant Style Specification, Fixing Specification and Assembly Specification pages when submitted for Building Consent purposes.

'VETRO' (NO RAIL)

This specification details the members to be used, glass thicknesses required and the maximum spacing for the various posts for this style. A separate specification must be referred to for fixing to the substrate (refer to Chapter 3). Post spacing must not exceed the lesser of the spacing from both Chapter 2 and Chapter 3. NOTE: For the Vetro Style we recommend always using 4 Substrate fixings per baseplate to give lateral stability to the post, even if 2 fixings is structurally adequate. Refer to Page 56 for notes on balustrade deflection.

1. Glass shall be GRADE A EVA TOUGHENED LAMINATED SAFETY GLASS suitable for exterior applications in accordance with AS/NZS 2208, with a minimum thickness as determined from the Table below. Glass shall be supported and glazed in accordance with NZS 4223. All exposed edges to be Flat Polished.
2. Fabrication and Installation to be in accordance with Assembly Specification AS.35.00 on Page 119, the Installation Guides in Chapter 5, and all other relevant portions of the UNEX Fabricators Manual.
3. APS and APE may be used for either "top fixed" or "side fixed". API & APT "side fixed" ONLY situations.
4. Both the vertical edges of the glass shall fully attach to the post with structural silicone, for the full height and thickness of the glass.



The balustrade shall be fabricated from the following items:

POST CAPS: ACFIS, ACEIS, ACPR, or ACPE. Refer to Chapter 1 for options.

POSTS: APS or APE for in-line posts, API or APT for side fixing only, APQ2 at 90º corners (see Note 3 above).

EVA TOUGHENED LAMINATED GLASS PANELS. See table below

BRACKET: AKS2

INSERT: AIA

MAXIMUM POST CENTRES 'S max' (metres)

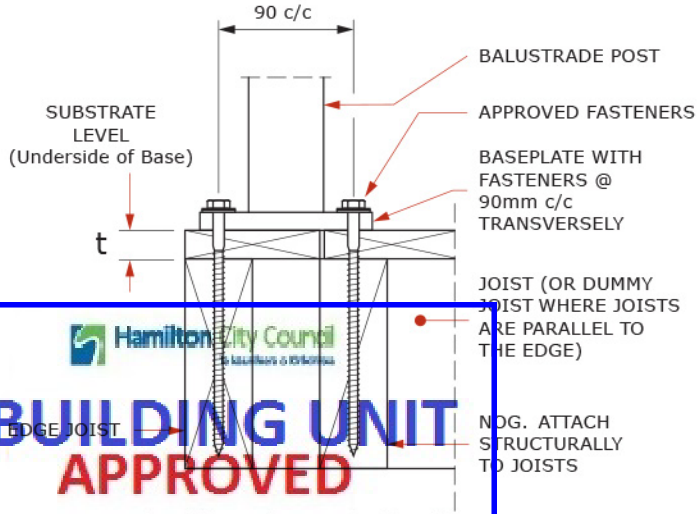
ALWAYS TAKE THE LESSER OF THE VALUE BELOW AND THE VALUE FROM THE FIXING SPECIFICATION

HEIGHT(3)	Post Type(2)	Toughened Glass Thickness	Line No.	LOADING CLASS(1)																		
				N07C/N07R								N03R	Not Preventing Falls									
				Design Wind Speed(4)									Design Wind Speed(4)									
				VH	EH								M		H				VH		EH	
				50	52	54	56	58	60	62	64	N/A	38	40	42	44	46	48	50	52	54	56
1.0	APR2 or APS	10	2	1.39	1.39	1.39	1.38	1.29	1.20	1.13	1.06	1.65	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.49	1.38
		12	3	1.39	1.39	1.39	1.38	1.29	1.20	1.13	1.06	2.00	1.80	1.80	1.80	1.80	1.80	1.80	1.73	1.60	1.49	1.38
	APE	12	4	1.77	1.77	1.77	1.76	1.64	1.53	1.44	1.35	2.00	1.80	1.80	1.80	1.80	1.80	1.80	1.80	1.80	1.80	1.76
		15	5	1.77	1.77	1.77	1.76	1.64	1.53	1.44	1.35	2.00	2.40	2.40	2.40	2.33	2.24	2.15	2.07	1.98	1.90	1.76
1.1	APR2 or APS	10	7	1.26	1.26	1.22	1.14	1.06	0.99	0.93	0.87	1.65	1.50	1.50	1.50	1.50	1.50	1.48	1.39	1.30	1.22	1.14
		12	8	1.26	1.26	1.22	1.14	1.06	0.99	0.93	0.87	2.00	1.80	1.80	1.80	1.69	1.59	1.48	1.39	1.30	1.22	1.14
	APE	12	9	1.61	1.61	1.55	1.46	1.36	1.27	1.19	1.11	2.00	1.80	1.80	1.80	1.80	1.80	1.80	1.73	1.64	1.55	1.46
		15	10	1.61	1.61	1.55	1.46	1.36	1.27	1.19	1.11	2.00	2.36	2.24	2.13	2.02	1.92	1.82	1.73	1.64	1.55	1.46
1.2	APR2 or APS	10	12	1.03	0.96	0.89	0.83	0.78	0.73	0.68	0.64	1.65	1.50	1.50	1.41	1.30	1.20	1.11	1.03	0.96	0.89	0.83
		12	13	1.03	0.96	0.89	0.83	0.78	0.73	0.68	0.64	1.96	1.66	1.53	1.41	1.30	1.20	1.11	1.03	0.96	0.89	0.83
	APE	12	14	1.37	1.28	1.20	1.12	1.06	0.99	0.93	0.88	2.00	1.80	1.80	1.78	1.66	1.56	1.46	1.37	1.28	1.20	1.12
		15	15	1.37	1.28	1.20	1.12	1.06	0.99	0.93	0.88	2.00	2.02	1.90	1.78	1.66	1.56	1.46	1.37	1.28	1.20	1.12

1. LOADING CLASS: Refer to Page 176 for the scope of the Loading Class designations.
2. POST TYPES: Refer to Chapter 1 for details.
3. HEIGHT 'H': is the overall height of the balustrade above the substrate level shown. Interpolate for Heights between those shown.
4. DESIGN WIND SPEED: in m/s, Refer to Pages 51 to 52 for details of applicable wind codes and the methods for determining the Design Wind Speed for any particular site.

WET TIMBER - TOP FIXING, 90MM CRS

Refer to all notes on Pages 72 and 73 which shall apply to this specification and the relevant pages in Chapter 5 Installation Guides. Refer also to Chapter 2 for the Style Specification.



t = distance from underside of Base Plate to top of structural timber

1. For details of approved fasteners refer to General Notes on Page 72 note 3.
2. Washers to be fitted under screw and bolt heads shall be as follows
   - For 6mm fasteners - washer supplied with fasteners.
   - For FC8-165 fasteners - washer supplied with fasteners.
   - For 8mm bolts - 22mm O.D. S/S washer (Part No. FW8-22) with a polymer washer (Part No. FWP8-22G) between the S/S and the aluminium.
   - For Washers bearing against timber use 50 x 50 x 3mm stainless steel washers Part No FW10-50SQ.
3. Substrate design including waterproofing and the structural design of the timber substrate and its connections are not included in this specification and must be carried out by others.
4. Important, the FC8-165 coachscrews in this specification are to be used with the "**Sika Supergrip 2 Hour**" adhesive system (TASG).

MAXIMUM POST CENTRES 'S max' (metres)

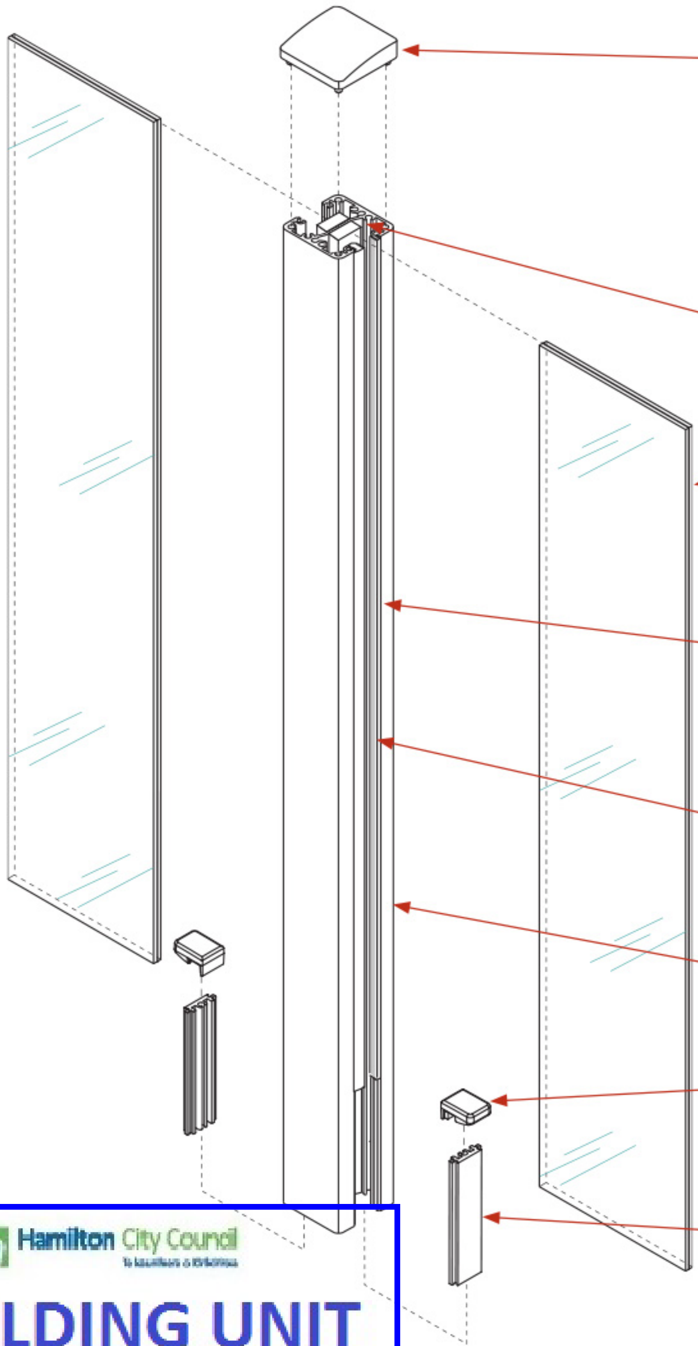
ALWAYS TAKE THE LESSER OF THE VALUE BELOW AND THE VALUE FROM THE STYLE SPECIFICATION

Height(3)	Baseplate Size D x W	Fasteners - Qty and Type(2)	't' (See diagram)	Line No.	LOADING CLASS(1)																	
					N07C/N07R								N03R	Not Preventing Fall									
					Design Wind Speed(4)									Design Wind Speed(4)									
					VH	EH								M		H				VH		EH	
					50	52	54	56	58	60	62	64	N/A	38	40	42	44	46	48	50	52	54	56
1.0	115 x 105	4 x FC8-165	19	1	1.51	1.51	1.51	1.51	1.41	1.31	1.23	1.16	3.25	2.84	2.84	2.68	2.44	2.24	2.05	1.89	1.75	1.62	1.51
	115 x 105	4 x FC8-165	25	2	1.46	1.46	1.46	1.45	1.35	1.27	1.19	1.11	3.12	2.73	2.73	2.58	2.35	2.15	1.98	1.82	1.68	1.56	1.45
	115 x 105	4 x FC8-165	32	3	1.39	1.39	1.39	1.39	1.29	1.21	1.13	1.06	2.98	2.61	2.61	2.46	2.24	2.05	1.89	1.74	1.61	1.49	1.39
	115 x 105	4 x M8 Bolts	N/A	4	1.84	1.84	1.84	1.84	1.71	1.60	1.50	1.41	3.95	3.46	3.46	3.27	2.98	2.72	2.50	2.31	2.13	1.98	1.84
1.1	115 x 105	4 x FC8-165	19	5	1.38	1.38	1.34	1.25	1.16	1.09	1.02	0.96	2.95	2.35	2.35	2.22	2.02	1.85	1.70	1.56	1.45	1.34	1.25
	115 x 105	4 x FC8-165	25	6	1.33	1.33	1.29	1.20	1.12	1.05	0.98	0.92	2.84	2.26	2.26	2.13	1.94	1.78	1.63	1.51	1.39	1.29	1.20
	115 x 105	4 x FC8-165	32	7	1.26	1.26	1.23	1.14	1.07	1.00	0.93	0.88	2.71	2.15	2.15	2.04	1.85	1.70	1.56	1.44	1.33	1.23	1.14
	115 x 105	4 x M8 Bolts	N/A	8	1.68	1.68	1.63	1.52	1.42	1.32	1.24	1.16	3.59	2.86	2.86	2.70	2.46	2.25	2.07	1.91	1.76	1.63	1.52
1.2	115 x 105	4 x FC8-165	19	9	1.26	1.22	1.13	1.05	0.98	0.91	0.86	0.80	2.70	1.97	1.97	1.86	1.70	1.55	1.43	1.31	1.22	1.13	1.05
	115 x 105	4 x FC8-165	25	10	1.21	1.17	1.08	1.01	0.94	0.88	0.82	0.77	2.60	1.90	1.90	1.79	1.63	1.50	1.37	1.27	1.17	1.08	1.01
	115 x 105	4 x FC8-165	32	11	1.16	1.12	1.03	0.96	0.90	0.84	0.78	0.74	2.48	1.81	1.81	1.71	1.56	1.43	1.31	1.21	1.12	1.03	0.96
	115 x 105	4 x M8 Bolts	N/A	12	1.54	1.48	1.37	1.28	1.19	1.11	1.04	0.98	3.29	2.40	2.40	2.27	2.07	1.89	1.74	1.60	1.48	1.37	1.28

1. LOADING CLASS: Refer to Page 176 for the scope of the Loading Class designations.
2. FASTENER DESIGNATIONS: beginning with 'F' are part numbers for fasteners supplied by UNEX eg. FC8-165: FC = Coach Screw Stainless Steel. 8 = 8mm diameter, 165 = length in mm; Substitution with other fasteners is not permitted.
3. HEIGHT 'H': is the overall height of the balustrade above the substrate level shown. Interpolate for Heights between those shown.
4. DESIGN WIND SPEED: in m/s, Refer to Pages 51 to 52 for details of applicable wind codes and the methods for determining the Design Wind Speed.

'VETRO' (NO RAIL)

Refer elsewhere for corners, slopes and other situations not illustrated here.



ACPR POST CAP (Illustrated) for 50 x 50 post, or ACPE for 50 x 60 post. Alternatively, use ACEFS as flat blank for 50 x 50 APE post or ACFFS on 50 x 60 APS post. Push fit to top of post after installation of glass. Secure to posts with a small amount of silicon or adhesive on the prongs.

STRUCTURAL GLAZING: Both the vertical edges of the glass shall attach to the post with structural silicone sealant, for the full height and thickness of the glass.

EVA TOUGHENED LAMINATED GLASS, of appropriate thickness, depending on the post spacing. Refer to Style Specification. Exposed horizontal edges to be flat polished.

SRG40 RETAINED GASKET to the outer face of glass 10mm or 12mm glass. SRG70 RETAINED GASKET used for 8mm glass. Must be slid into post prior to assembly.

WEDGE GASKET to the inner face of glass, SWE50 (with SRG70 RETAINED GASKET) for 8mm glass, SWE55 for 10mm glass, or SWE35 for 12mm glass.

APR2 (illustrated), APS or APE RECESSED POST. On 90° corners use APQ2 CORNER RECESSED POST

AKS2 BRACKET, to support glass. Attach to AIA insert (press fit). For further detail of bracket attachment for situations, refer to "AKS attachment" pages in Chapter 5.

AIA INSERT for supporting glass by transferring weight down to base plate or substrate.

UNEX LOADING CLASSIFICATION

Below is an extract from AS/NZS 1170.1:2002, a standard which is cited in the NZBC. This table gives the minimum imposed actions on barriers for various occupancy types. To the right hand side of this table are the applicable UNEX Loading Classifications (ULC) which are stated in the specifications contained in this catalogue.

AS/NZS 1170.1:2002 TABLE 3.3
MINIMUM IMPOSED ACTIONS FOR BARRIERS

	Type of occupancy for part of the building or structure	Specific Uses	Top Edge			Infill		**ULC** (UNEX Loading Classifi-cations)
			Horizontal kN/m	Vertical kN/m	Inwards, Outwards, or Downwards kN	Horizontal kPa	Any Direction kN	
A	**Domestic and residential activities**	All areas within or serving exclusively one dwelling including stairs, landings, etc., but excluding external balconies and edges of roofs (see C3)	0.35	0.35	0.6	0.5	0.25	**N03R**
		Other residential (see C3)	0.75	0.75	0.6	1.0	0.5	**N07R**
B, E	**Offices and work areas not included elsewhere including storage areas**	Light access stairs and gangways not more than 600mm wide	0.22	0.22	0.6	N/A	N/A	**N02**
		Fixed platforms, walkways, stairways and ladders for access	0.35	0.35	0.6	N/A	N/A	**N03C**
		Areas not susceptible to overcrowding in office and institutional buildings also industrial and storage buildings	0.75	0.75	0.6	1.0	0.5	**N07C**
AREAS WHERE PEOPLE MAY CONGREGATE								
C1/C2	**Areas with tables or fixed seating**	Areas with fixed seating adjacent to a balustrade, restaurants, bars. etc.	1.5	0.75	0.6	1.5	1.5	**N15**
C3	**Areas without obstacles for moving people not susceptible to over-crowding**	Stairs, landings, external balconies, edges of roofs, etc.	0.75	0.75	0.6	1.0	0.5	**N07C**
C5	**Areas susceptible to over-crowding**	Theatres, cinemas, grandstands, disctheques, bars, clubs, auditoria, shopping malls (see also D), assembly areas, studios, etc	3.0*	0.75	0.6	1.5	1.5	**N30** **N20**(1)
D	**Retail areas**	All retails areas including public areas of banks/building societies (see C5 for areas where over-crowding may occur)	1.5	0.75	0.6	1.5	1.5	**N15**

(1) N20 is only applicable where 2.0kN/m horizontal top edge load is acceptable by the Building Consent Authorities.

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