

20218200 14 St Clements Road



20218200

BDI Structural Information Pack

14 St Clements Road

Prepared for:

Belverdale Building Contractors Ltd

228 Compstall
Road Romiley
SK6 4JG
United Kingdom

Subject Property Address:

14 St Clements Road
Chorlton
Manchester
M219HU

United Kingdom

Issue Date: May 21

Revision: A0

Revision Date: May 21

Information on the use of the BDI structural information pack

This information pack has been prepared to determine and detail the required structure for the proposed project.

The information pack should be read in conjunction with the relevant architects/building designers' drawings and details.

The information is provided such that your builder/contractor has adequate information to construct and install the structure.

The information pack should also be forwarded to Building Control to demonstrate that you have considered the requirements of Part A (Structure) of the Building Regulations. Building Control will want to review the structural proposals and supporting calculations that we have prepared.

The Information pack has been prepared for the use of the named client only in relation to the subject property only and may only be used by the client, the Building Control organisation for the project and the client's builder/contractor.

The information pack is separated into four sections as follows

Section 1: General project information

Background and information on the project and on BDI Structural Solutions services provided.

Section 2: Structural Layouts, details and guidance notes

Showing the layout/location of the proposed structure and typical details, guidance on materials, etc.

Section 3: Health and Safety

Risk assessments in relation to the design we have completed.

Section 4: Structural Design

Basis of structural design completed, loadings, structural codes used and copies of the structural calculations.

Section 1: General project information

1.1 Project Scope and scope of BDI Structural Solutions Instructions

The proposed project entails side and rear extension and associated internal alterations to existing house.

BDI Structural Solutions Ltd have been engaged by the client to provide structural engineering advice in respect of the following elements of the proposed project

- Pitched Roof Timbers
- Steel beams and padstones
- Steel columns
- Lintels to external walls
- Stability framing
- Internal wall lintels
- Preliminary Foundation design

The general arrangement of the structure we have designed is indicated in section 2 within this information pack.

We have not been instructed to visit site, we have derived information from the provided architects/building designers' drawings and details.

We have provided preliminary foundation design. A site investigation has not been completed and we have based the preliminary foundation design on assumed ground conditions likely to prevail on site, based on available BGS soils data and/or local knowledge and is based on a relatively conservative bearing capacity of 100kN/m². The actual ground conditions will need to be confirmed on site and the adequacy of the bearing capacity used in the preliminary foundation design confirmed.

We have not been provided any information regarding the existing structure; therefore, we have taken a conservative approach to any structural design based on our inspection or our knowledge of the structural arrangement of buildings of this type and age.

Due care and attention has been given in the production of the structural information pack, however, any discrepancies should be clarified prior to commencing work.

In relation to this project we have only designed the structural elements requested and as detailed within the information pack.

We have no information regarding setting out of structure on plan or in relation to required levels/heights of structure and these should be determined on site or from the architects/building designers' drawings and details.

1.2 Existing Building Overview

The existing building consists of a semi detached two storey house.

For the purposes of these calculations we have assumed the existing structure is a traditional load bearing structure, comprising of a timber traditional cut roof, suspended timber floors and load bearing masonry.

Stability is presumed to be provided by plate action provided by floors and roofs acting as rigid diaphragms and buttressing walls.

1.3 Impact of proposed works on the existing building.

The proposed works do not impact on the stability of the existing building.

1.4 BDI Project Team.

The lead Engineer for this project was

Mariam Yousaf MSc.

If you have any queries in relation to the project or the information in the document please call Mariam on 0161 969 3886

Or alternately email: Mariam@bdistructuralsolutions.co.uk

The project was reviewed by

Micaela Maccan MEng

Section 2: Structural Layouts, details and technical guidance notes

2.1 Structural Layouts

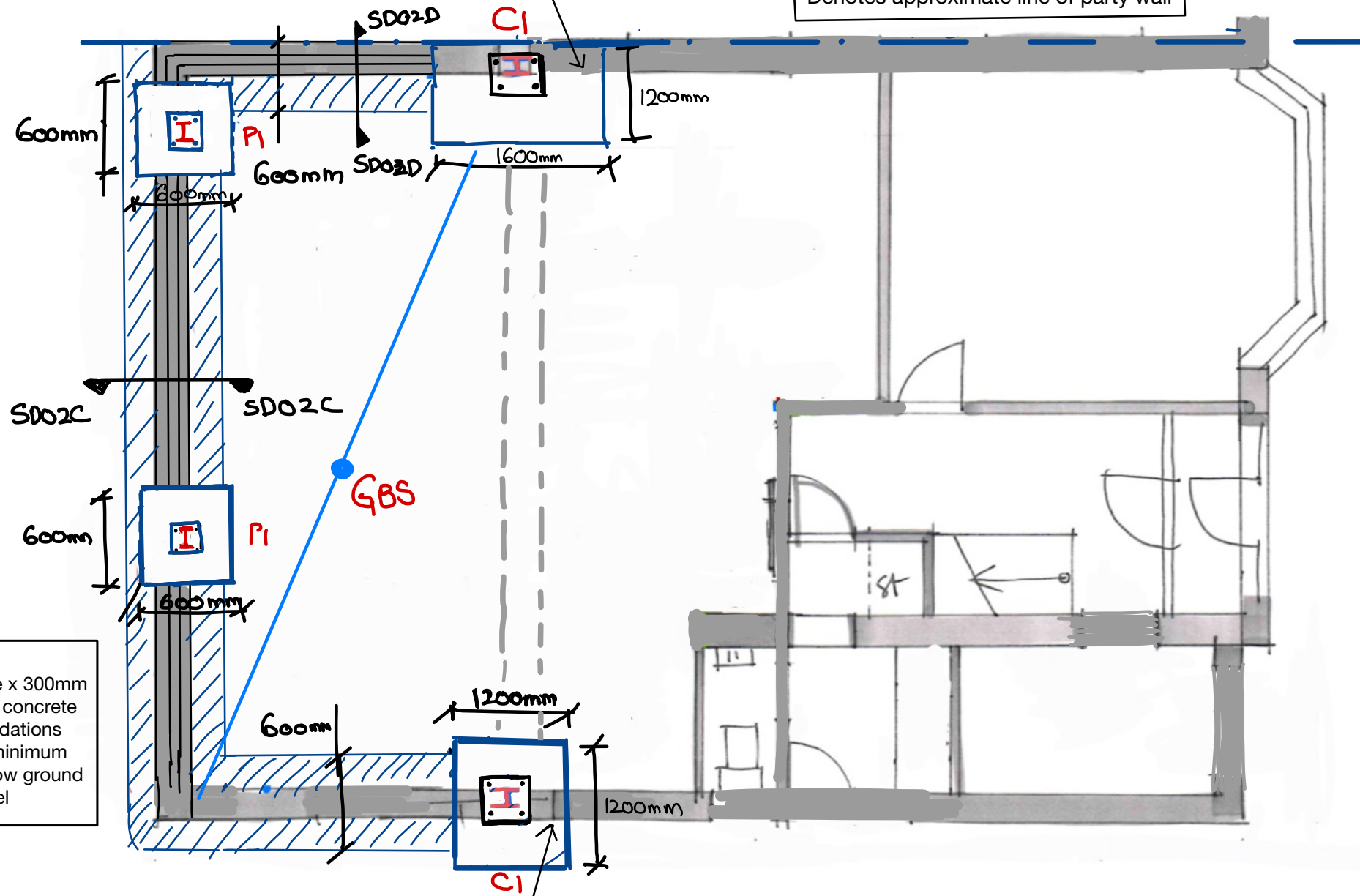
Appended you will find a structural layout sketch or marked up layout taken from the architects/building designers' drawings.

The appended structural layouts provides a key plans to indicate the location of the required structure. Structural member sizes are either indicated on the layout or referenced to the member table below the layout.

Strengthen existing strip foundation by providing 1200x1600x300mm pad foundation with A393 mesh in bottom face, min cover 40 mm. Concrete to flood up sides of existing strip foundation.
Note: min depth of concrete below existing strip to be min 300mm.
Provide adequate temporary propping below ex foundation to limit span of masonry to max 675 mm

Client to abide to Party Wall Act
Denotes approximate line of party wall

A site investigation has not been completed.
Foundation scheme is guidance only.
The suitability of bearing strata should be verified by the building control officer. Assumed bearing capacity 100KN/Sq. m



KEY

- PAD TO C1 (LHS)** : 1200x1600x300mm (DEEP RC PAD w/A393 MESH IN BOTTOM FACE (MIN COVER 40mm))
- PAD TO C1 (RHS)** : 1200x1200x300mm (DEEP RC PAD w/A393 MESH IN BOTTOM FACE (MIN COVER 40mm))
- PAD TO P1** : 600 x 600 x 300mm (DEEP RC PAD w/A393 MESH IN BOTTOM FACE (MIN COVER 40mm))
- GBS** : 125mm RC GBS w/A252 MESH IN TOP FACE
- STRIP FOOTING** : 600x300mm MASS CONCRETE STRIP FOOTING

600mm wide x 300mm deep mass concrete strip foundations founded minimum 750mm below ground level

All elements are indicatively set out, contractor to determine all setting out onsite in conjunction with architectural proposals and existing structure. All levels are assumed

Drawings to be read in conjunction with design calculation, advice notes and guidance. All stated guidance to be adhered to.

Strengthen existing strip foundation by providing 1200x1200x300mm pad foundation with A393 mesh in bottom face, min cover 40 mm. Concrete to flood up sides of existing strip foundation.
Note: min depth of concrete below existing strip to be min 300mm.
Provide adequate temporary propping below ex foundation to limit span of masonry to max 675 mm

**Proposed Foundation Layout
Indicating Structure Over
N.T.S.**

For Approval



Project 14 St. Clements Road		Date April 21
Title Foundation Layout		Drawn MY
Project No 20218200	Sheet SK01	Rev A0

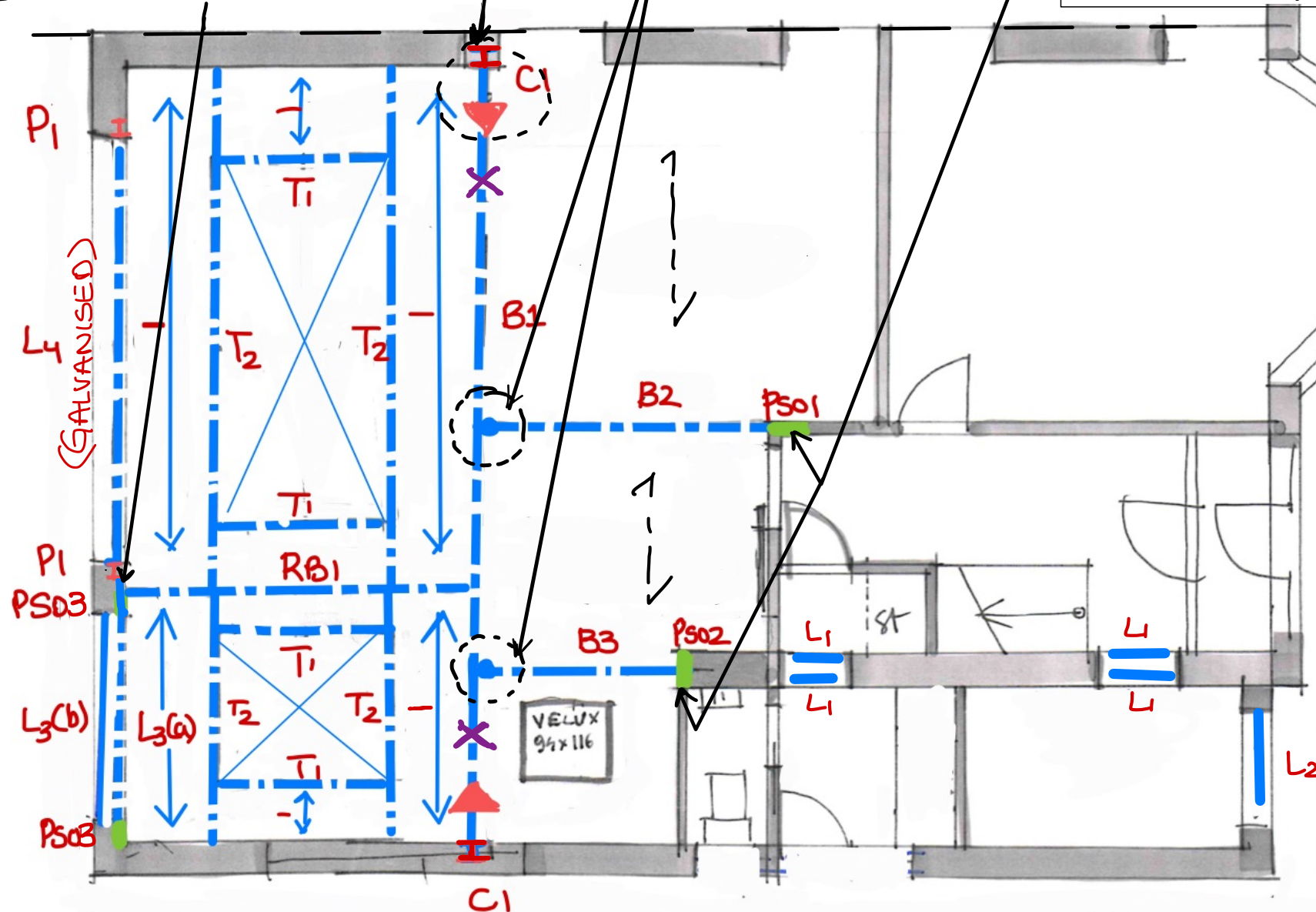
WELD 100x100x8 RSA TO P1
W/6mm FPFW. SIT L3(a) ONTO
RSA & FIX DOWN USING 2No.
M12 GRADE

B1-C1 CONNECTION
DETAIL AS PER
SK04D

TYPICAL CONNECTION
DETAIL SDO1A

TYPICAL PADSTONE
DETAIL AS PER
SDOIR

Client to abide to Party Wall Act



**Proposed Ground Floor Layout
Indicating Structure Over
N.T.S.**

For Schematic elevations &
Details on frames B1 & C1
and L4 & P1 see drawing
SK03 & SK04

✕ Approximate Crank
Position

Fix new timber roof rafters
and wall plate down to
wall as per detail SD01P

Strap new timber roof
joists to masonry wall
(parallel) as detail SD01O

Ensure multiple timber
members forming beams
are fixed together using
M12 coach bolts at 400c/c

All proprietary lintels to
have minimum 150mm
bearing and to suit wall
construction

Contractor to determine
levels on site prior to
construction/fabrication

Do not scale off this
drawing.

Drawings to be read in conjunction
with design calculation, advice
notes and guidance. All stated
guidance to be adhered to.

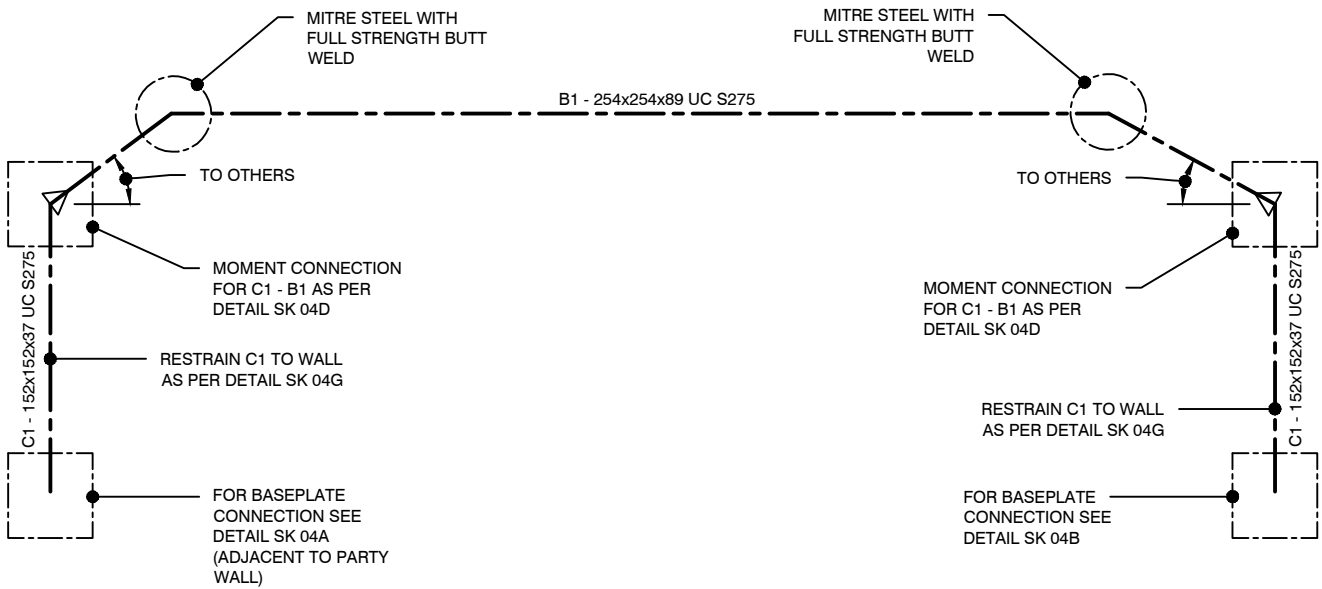
For Approval

KEY

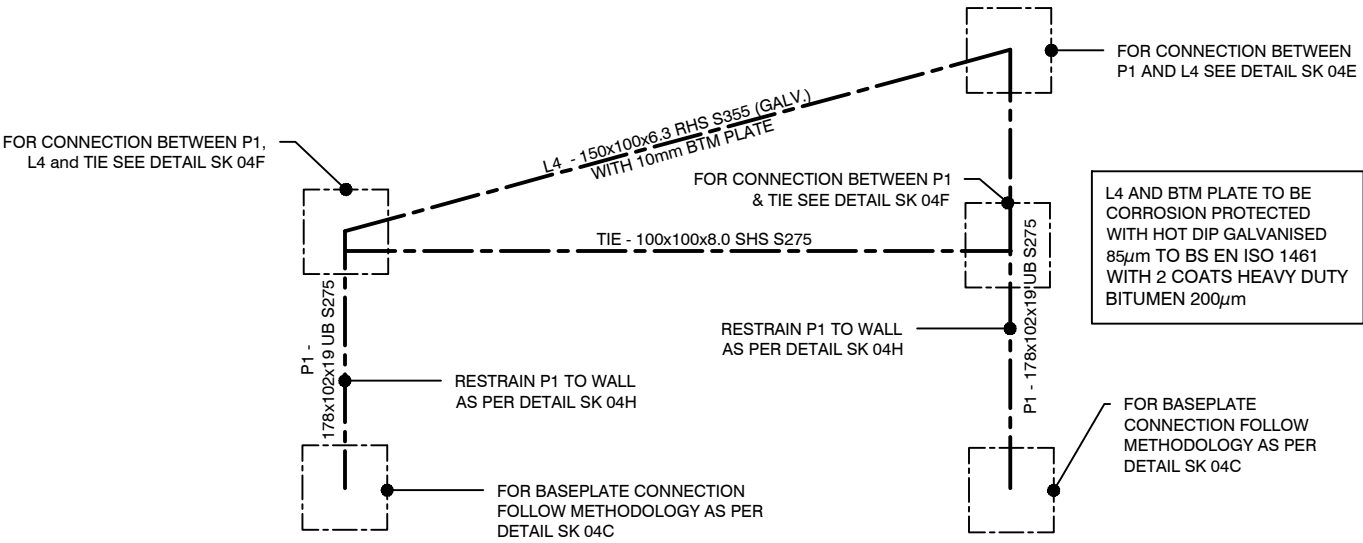
- I** : 75x225mm C24 @ 400mm c/c
- RB1** : UB 178x102x19 S275
- T₁** : 2/75x225mm C24
- T₂** : 3/75x225mm C24
- B₁** : UC 254x254x89 S275
- B₂** : UC 152x152x23 S275
- B₃** : UC 152x152x23 S275
W/10mm THICK STEEL BOTTOM PLATE
- C₁** : UC 152x152x37 S275
- P₁** : UB 178x102x19 S275
- TIE1** : SHS 100x100x8 S355
- L₁** : NAYLOR S4 LINTEL
- L₂** : CATNIC CG LINTEL
- L_{3(a)}** : UB 152x89x16 S275
- L_{3(b)}** : CATNIC ANG LINTEL
- L₄** : RHS 150x100x6.3 S355
W/10mm THICK STEEL BOTTOM PLATE
(GALVANISED)
- PS01** : 330x100x215mm
MASS CONCRETE PADSTONE
- PS02** : 215x200x215mm
MASS CONCRETE PADSTONE
- PS03** : 215x100x215mm
MASS CONCRETE PADSTONE

ALL ELEMENTS ARE INDICATIVELY SET OUT,
CONTRACTOR TO DETERMINE ALL SETTING OUT
ONSITE IN CONJUNCTION WITH ARCHITECTURAL
PROPOSALS AND EXISTING STRUCTURE.
ALL LEVELS ARE ASSUMED.

This drawing is to be read in conjunction with all other relevant
documentation.
Do not Scale
All setting out by contractor. Report all discrepancies to
bdi Structural Solutions.
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Client required to comply with requirements of Party Wall Act as
required by nature of work.



PROPOSED C1 - B1 STEEL ELEVATION
NOT TO SCALE



PROPOSED P1, L4 & TIE STEEL ELEVATION
NOT TO SCALE

Steel / Timber Beam Notation

Steel Reference
(See key for sizes)

Denotes moment connection

Steel References DO NOT represent actual section size

Moment connections are critical connections which must be fabricated and installed as detailed. The requirement for the column and connection is essential to the global stability of the structure. Under no circumstances should the column and connection be omitted.

FOR APPROVAL

Rev	Date	Rev	Date
A0	05.05.21	OO	MM

FIRST APPROVAL ISSUE

bdi
Structural Engineering Solutions

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Client
Rhian Cooke

Project
14 St Clements Road

Drawing Title
Steel Elevations

Scale
As shown at A3

Drawn
OO

Chk'd by
MM

Date
05.05.21

Date Chk'd
05.05.21

Drg No.
20218200
SK 03

Rev
A0

This drawing is to be read in conjunction with all other relevant documentation.

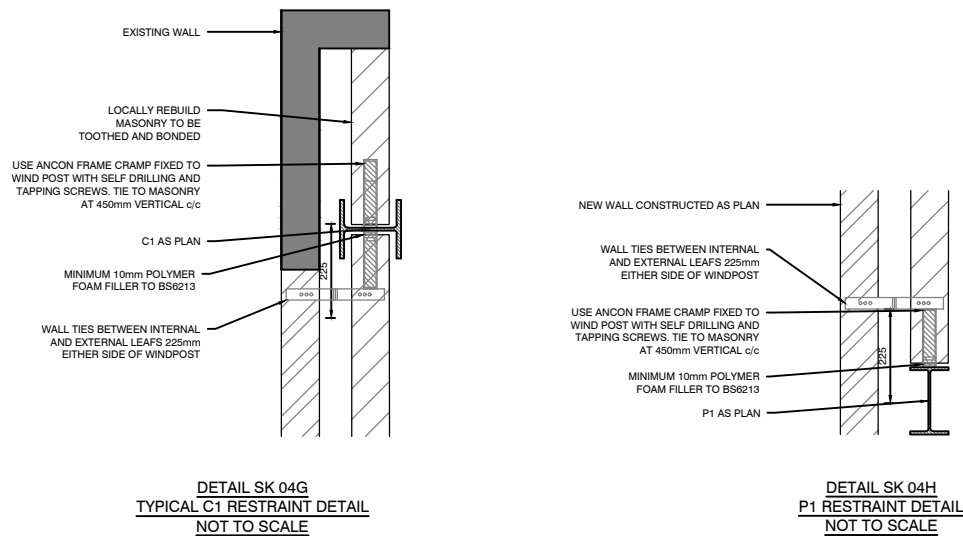
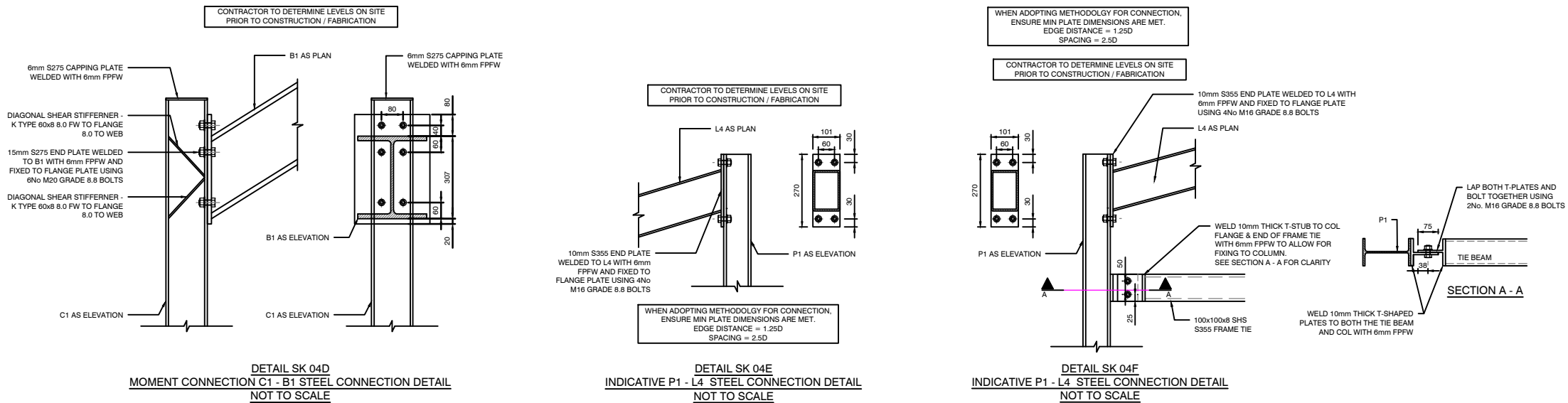
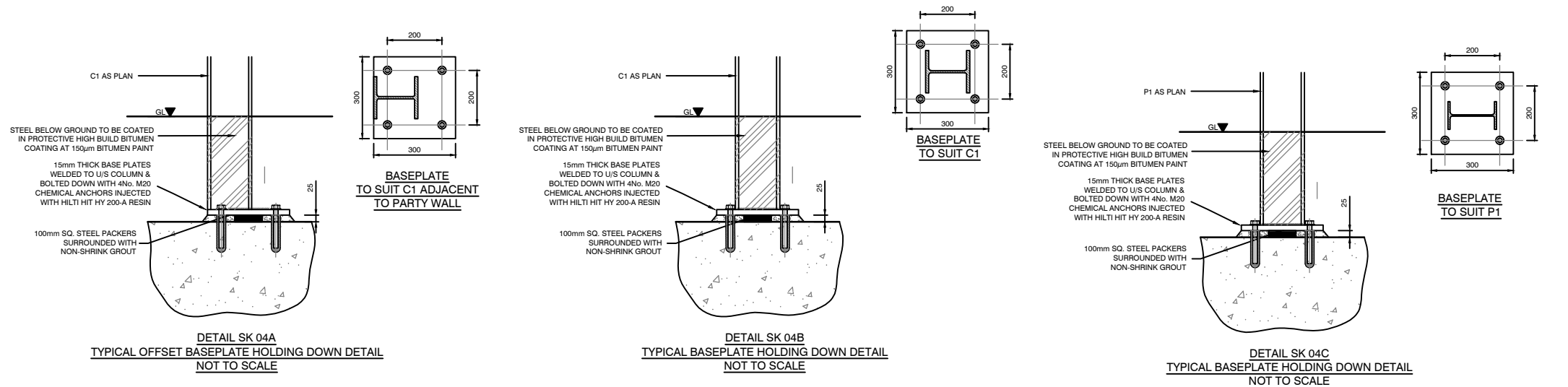
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ALL ELEMENTS ARE INDICATIVELY SET OUT,
CONTRACTOR TO DETERMINE ALL SETTING OUT
ONSITE IN CONJUNCTION WITH ARCHITECTURAL
PROPOSALS AND EXISTING STRUCTURE.
ALL LEVELS ARE ASSUMED.



FOR APPROVAL

Rev	Date	Drawn	Appr'd
A0	05.05.21	OO	MM

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Client
Rhian Cooke

Project
14 St Clements Road

Drawing Title
Structural Details

Scale
As shown at A3

Drawn
OO

Date
05.05.21

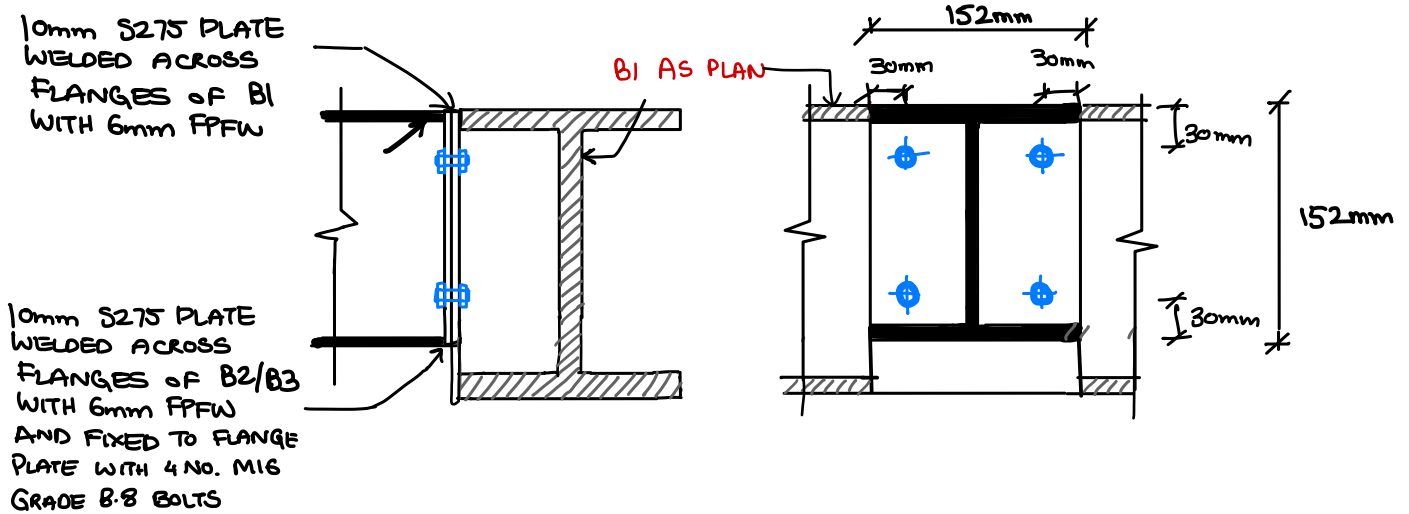
Rev
A0

20218200
SK 04

2.2 Structural details relevant to project

Within this section we appended structural details that are relevant the project.

Project 14 St Clements Road		Title SD 01A-Connection Detail		
Project No 20218200	Sheet SD01	Rev A0	Date April 21	Drawn MY

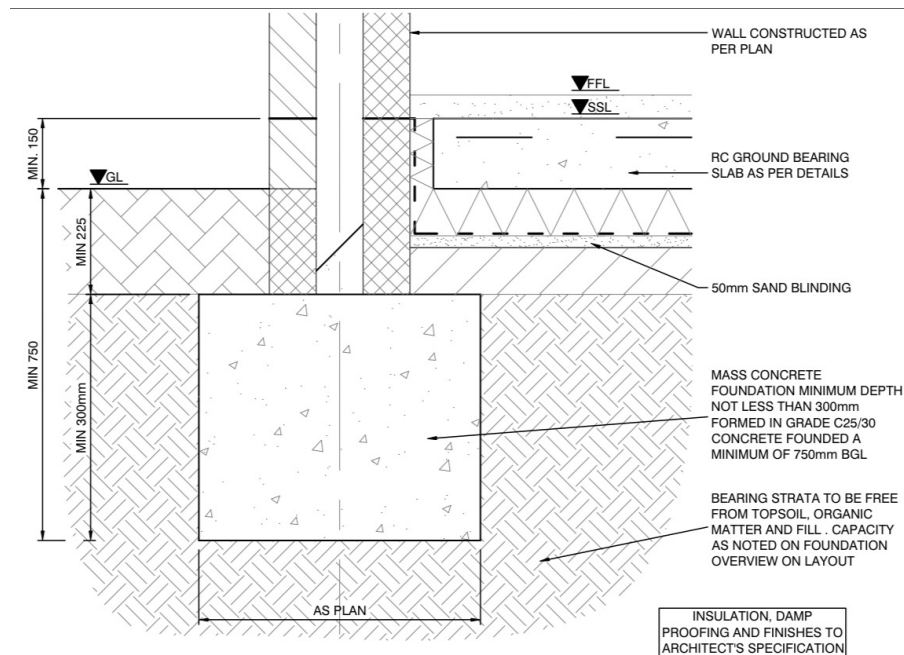


DETAIL SDO1A
INDICATIVE Beam1- Beam 2/3 CONNECTION DETAIL
N.T.S.

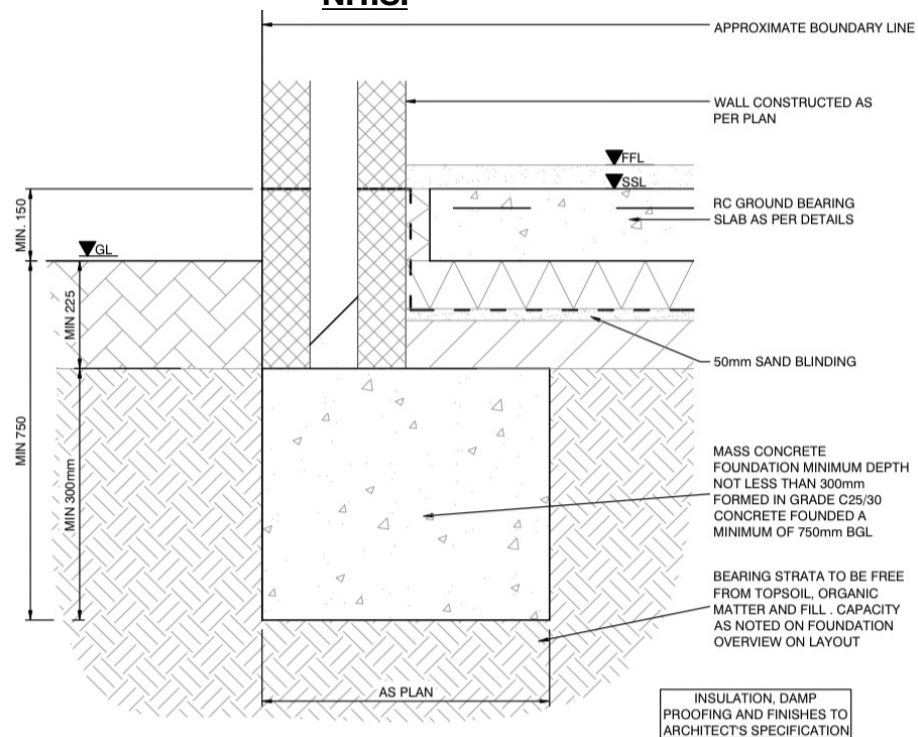
Drawings to be read in conjunction with design calculation, advice notes and guidance. All stated guidance to be adhered to.

For Approval

Project	Title			
14 St Clements Road	Strip Foundation Details			
Project No	Sheet	Rev	Date	Drawn
20218200	SD02	A0	April 21	MY



DETAIL SDO2C
TYPICAL SECTION THROUGH STRIP FOOTING
N.T.S.



DETAIL SDO2D
TYPICAL SECTION THROUGH STRIP FOOTING (OFFSET)
N.T.S.

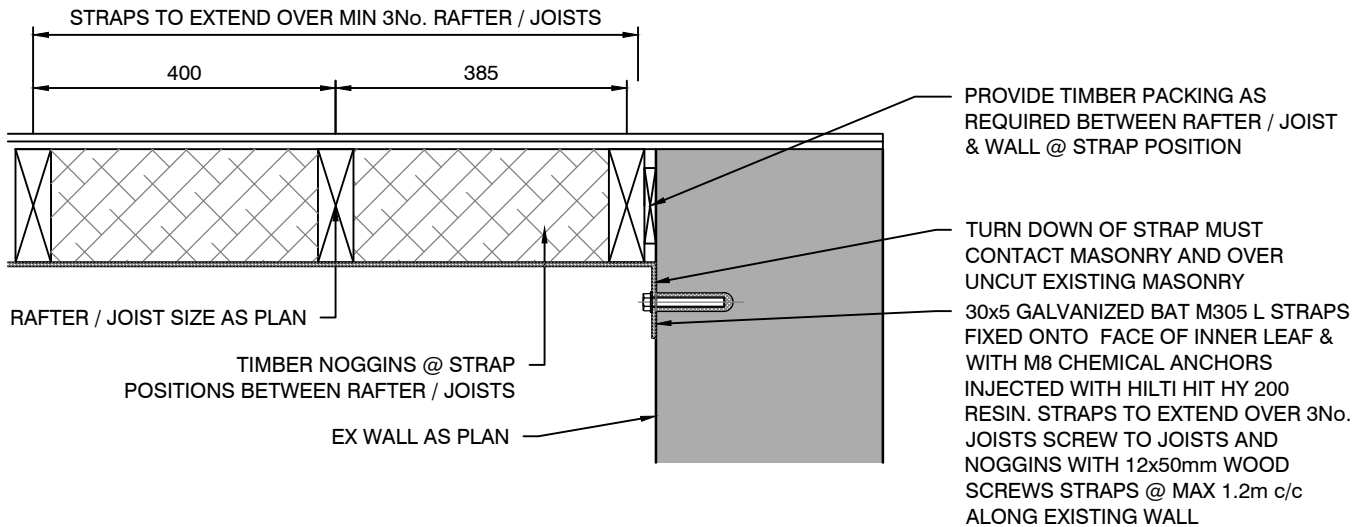
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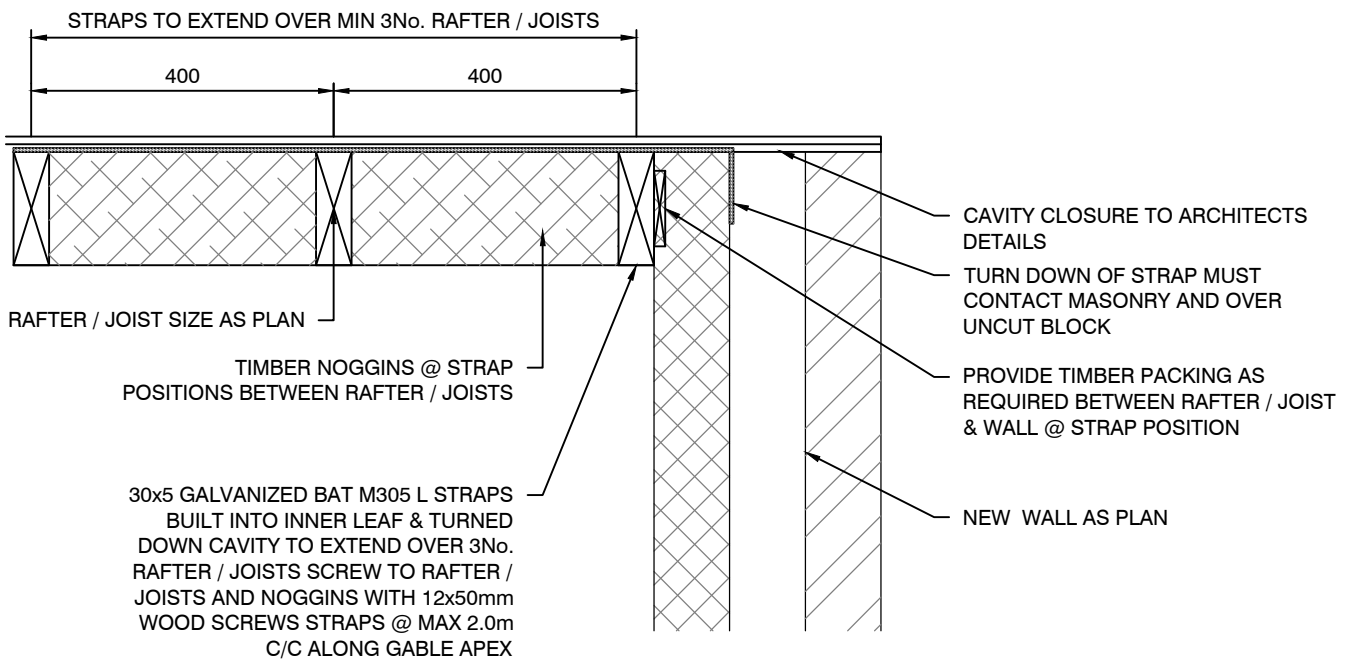
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Client required to comply with requirements of Party Wall Act as required by nature of work.



RAFTER / JOIST STRAPPING TO EX WALL



RAFTER / JOIST STRAPPING TO NEW WALL

TYPICAL RAFTER / JOIST STRAPPING DETAILS TO NEW & EX WALLS

1:10

Rev A0 Date FIRST ISSUE, FOR APPROVAL

Drawn JDH Apprd NF

Drawing Title
Typical Rafter / Joist Strapping Detail

Scale
As shown at A4

Drg No.

SD 010

Rev A0

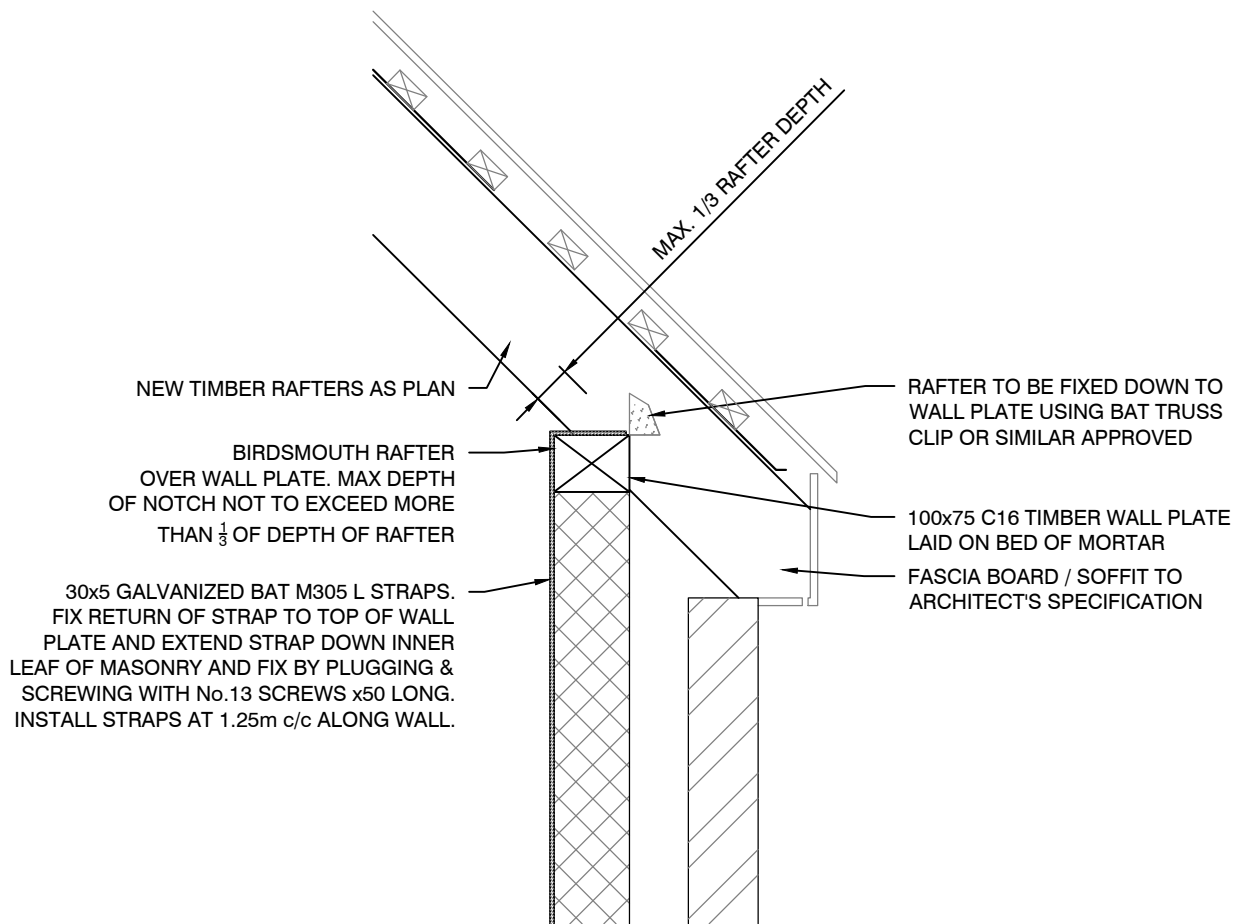
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TYPICAL STRAPPING TO WALL PLATE

1:10

Rev A0 Date FIRST ISSUE, FOR APPROVAL

Drawn JDH Apprd NF

Drawing Title
Typical Wall Plate Strapping / Rafter Fixing Detail

Scale
As shown at A4

Drg No.

SD 01P

Drawn JDH

Chkd by NF

Rev A0

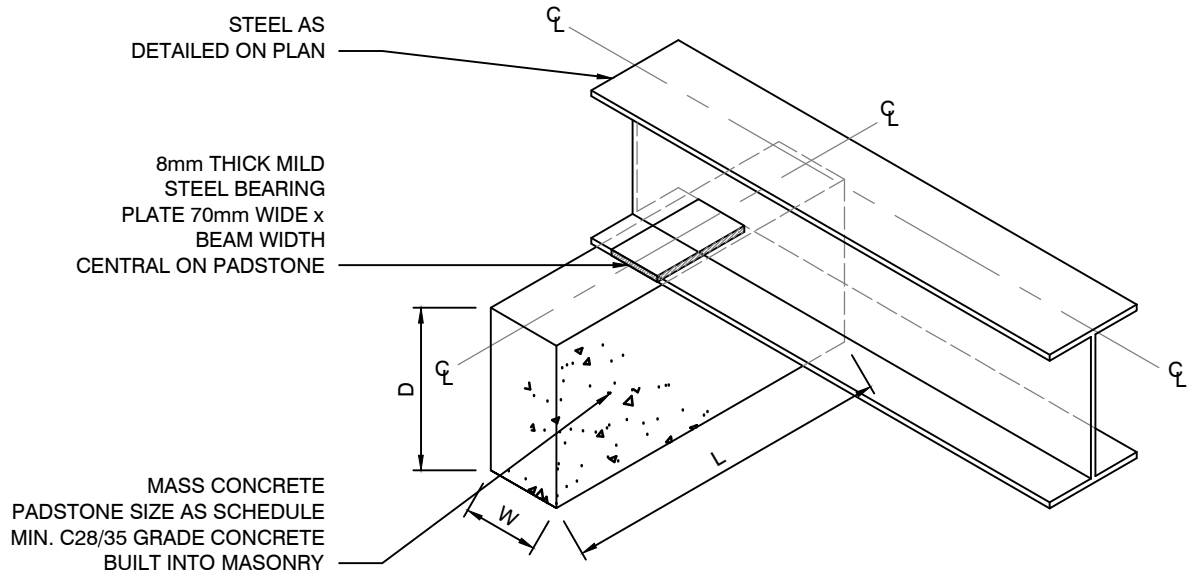
This drawing is to be read in conjunction with all other relevant documentation.

Do not Scale

All setting out by contractor. Report all discrepancies to bdi Structural Solutions.

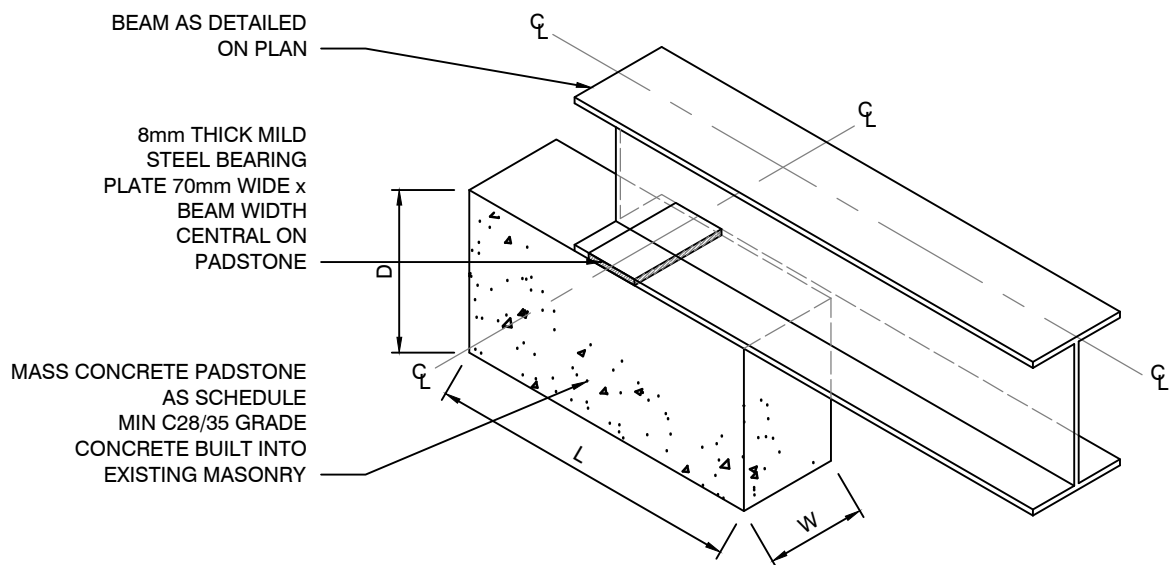
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Client required to comply with requirements of Party Wall Act as required by nature of work.



TYPICAL PADSTONE DETAIL (PERPENDICULAR)

1:10



TYPICAL PADSTONE DETAIL (IN LINE)

1:10

Rev A0	Date	FIRST ISSUE, FOR APPROVAL
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Drawn JDH	Appr'd NF
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Drawing Title Typical Padstone Details

Scale As shown at A4

Drg No.

Drawn JDH

Chkd by NF

SD 01R

Rev A0

2.3 Technical Guidance notes

These guidance notes should be read in conjunction with the structural layouts and details of this information pack and the details and drawings provided by your architect/designer.

Structural Steelwork

Where we have specified steel beams these are, unless stated otherwise, grade S275 steel and British Standard Sections.

We recommend that all steel sections be adequately corrosion protected this is important for the steel structure to achieve the required design life.

Required corrosion protection is based on systems provided by TATA/Corus steel and these are available to review at:

www.steelconstruction.info/standard_corrosion_protection_systems_for_buildings

This guidance is based internal steel being in a C2 Low Atmospheric Environment and external steel being in a C3 Medium Atmospheric Environment.

Hidden Steelwork IH-C2-C

Surface Preparation: Shot Blasted to SA2.5

Factory Applied Coatings: 2 coats water based Zinc epoxy primer 2 x 60um = 120um total

External Exterior Steelwork E-C3-A

Hot Dip galvanised to BS EN ISO 1461

Steelwork in exterior walls but 25mm minimum separation from outer leaf PW-C2-S

Surface Preparation: Shot Blasted to SA2.5

Factory Applied Coatings: Zinc phosphate epoxy primer 80um

Steelwork in exterior walls in contact with outer leaf PW-C2-B

Hot Dip galvanised to BS EN ISO 1461 85um PLUS

2no coats heavy duty bitumen, 200um total.

Where lengths of beams are detailed within calculations these lengths are for design purposes only. Exact required lengths of steel beams should be measured on site. Where required the steel beams will need to be fire protected as required by the building regulations.

Padstones

All steel beams should be provided with padstones. Where we have specified a padstone size, the size should be considered the minimum required.

The exact size should be determined on site. The steel beam should be seated centrally of the padstone and an 8mm thick steel bearing plate should be placed between the bottom of the beam and the padstone. Mass concrete padstones should be made from minimum grade RC30 Concrete.

Brick Piers

Where we have specified brick pier sizes we will have indicated whether this can be formed in existing brickwork or whether the pier should be formed in Class B Engineering Brick. The quality of existing brick work in piers will need to be assessed on site with your contractor and the BCO.

The plan size of piers given is the minimum required and assumes the masonry meets the required quality and that the required padstone has been provided. Where a pier abuts a new or existing wall the pier should be adequately bonded or tied to enhance stability and provide load transfer.

New load bearing walls

Where we have been asked to specify load bearing masonry such as block walls, cavity walls, etc. We will have specified a block strength and width.

The block strength should be considered as a minimum and the contractor should ensure that the blocks used meet or exceed the required compressive strength. The Mortar designation should be adhered to.

Where brickwork is used in exposed location such as parapets or chimney stacks, or below DPC a brick with Low Soluble Salts and a high frost resistance should be used such as an FL grade brick.

Foundations

Unless we have been asked to specify and design foundations, these will need to be assessed on site. A site investigation will be required to accurately design the required foundations. In the absence of a site investigation prior to design we will design foundations based upon assumed typical or average ground for the area based upon any local knowledge we may have. Exact foundations will need to be determined on site with the BCO. Particular care will be required where piers are formed in existing walls supporting concentrated loads.

Structural Timber

Where we have been asked to design and specify timber members, ie floor joists, rafters, etc we will have specified structural graded timber.

You should ensure that the timber used in the construction is clearly stamped with the correct structural grade and that the timber is kept dry prior and during construction.

Timber located in ground floors, roof structures and embedded or supported in external walls the timber should be preservative treated. Joist ends should be wrapped in DPC.

Lintels

Where we have been asked to design Lintels, we will where ever possible specify proprietary lintels. In such instances the lintel should be installed in accordance with the Manufacturers recommendations and instructions.

A minimum bearing of 150mm should be provided. A cavity tray and weepholes should be provided over all lintels. Any one of or special fabricated lintels we have specified should be galvanised.

Reinforced Concrete

Where we have designed and specified reinforced concrete we will have provided a concrete mix designation that should be used.

In addition, where cut and bent reinforcement is specified, we will have provided bending schedule.

Cover to all reinforcement should be minimum 40mm.

The contractor and BCO should establish whether ground conditions require the use of Sulphate Resisting Portland Cement (SRPC) where concrete structures are in contact with the ground.

All concrete should be vibrated into position with a 50mm poker vibrator. The building contractor will be required to design and implement adequate temporary support to enable the structural works to be completed.

Fire Protection

The structure should be adequately fire protected in accordance with the requirements of the building regulations.

Depending on the required level of fire protection this can take the form of 1 or 2 No layers of plasterboard or fire line to timber and steel structures or intumescent paint to any exposed steel structure.

Temporary works

We have designed the required permanent structure for the project.

Temporary works are often required to undertake a project and install/construct the permanent structure. Such temporary works might involve scaffolding, temporary propping, shoring, concrete formwork/shuttering, crane bases, etc.

In respect of temporary works, we always recommend that the responsibility for the design, erection, maintenance and removal of all temporary works remains with the contractor at all times.

BDI Structural Solutions Ltd undertakes temporary works design for contractors and we would be pleased to discuss designing any temporary works for this project with your contractor/builder.

Party Wall Act

You should satisfy yourself that you have complied with the requirements of the Party Wall Act in respect of any elements of the works that relate to the party wall or building boundaries. You have a legal responsibility to comply with the act and if required we would always recommend a Party Wall Surveyor be engaged. If you require any further information on this aspect please contact us.

Section 3: Health and Safety

3.1 Health and Safety Risk Assessment

In designing the elements of the structure we have considered issues of health and safety in respect of the implementation of the works as required by the CDM Regulations. All attempts have been made to design out significant health and safety risks. Where unusual procedures or processes are required we will provide a method statement to assist the contractor in establishing a safe method of working.

In undertaking the structural design we have aimed to satisfy the requirements of the CDM regulations.

As a designer of the building the CDM regulations requires us to consider the impact our design has on health and safety.

In considering the impact the design has on health and safety we have considered measures we can adopt to reduce the hazards and risks.

Appended to this section is a Designers risk assessment that details the hazards and risks associated with our design and measures we have taken to reduce risk.

Your contractor should consider the risk assessment when planning and undertaking the works.

Designer's Risk Assessment

Activity Element	Potential Hazard	Popu latio n at Risk	Likelihood	Severity	Risk		Design Stage Risk Management	Likelihood Post Management	Post Management Risk		Risk (Temporary / Permanent)	Residual?	Responsibility Handover
Installation of new steel beams and columns	Heavy lift/injury/collapse	All	2	2	4	Medium	Steel beam designed to minimum weight possible and contractor to use lifting aids if necessary. Splice if required BDI can provide splice details	2	4	Medium	Temporary	Yes	Contractor
Installation of new steel beams	Deflection in beams leading to damage of existing brittle finishes supported over.	Client	2	2	4	Medium	The new beams have been designed to limit deflection to prescribed limits within the current codes of practice. However there still remains a residual risk.	1	2	Low	Temporary	Yes	NA

20218200 14 St Clements Road



Removal of existing walls.	Possible collapse of structure and potential excessive vertical movement of supporting structure, causing damage of finishes supported over	All	2	3	6	Medium	Contractor to design, install and maintain adequate temporary works.	1	3	Low	Temporary	Yes	Contractor
Unknown ground conditions	Ground conditions are unknown Consideration may need to be given to ground water, clay soils or contamination	Client	2	3	6	Medium	Contractor to investigate ground conditions in conjunction with BCO and client	2	6	Medium	Temporary	Yes	Contractor

Section 4: Structural Design

In completing this project, we have undertaken detailed structural design involving calculations to relevant standards. This section outlines the basis of the structural design completed.

The calculations are provided as a record of the technical analysis and design we have completed and to provide adequate information for Building Regulations review and approval.

The calculations are not provided for use by your builder, the required structure is detailed on the layouts and details within Section 2.

Do not use design dimensions contained within the calculations to order or fabricate steelwork or other structural members

4.1 Design Life

The structure that we have designed and specified has been designed for a 50 year design life.

4.2 Codes of Practice / Statutory Approval

The calculations have been completed such that the proposed structural elements of the above mentioned project are in compliance with the Approved Document A: Structure, The Building Regulations, and the referenced and associated codes of practice where applicable.

The following British Standards have been referred to:

- BS5268-2: 2002 Structural use of timber
- BS5628-1: 2005 Structural use of unreinforced masonry
- BS5950-1:2000 Structural use of steelwork in building
- BS6399-1: 1996 Code of practice for dead and imposed loads
- BS6399-2: 1997 Code of practice for wind loads
- BS6399-3: 1988 Code of practice for imposed roof loads
- BS8004:1986 Foundations
- BS8110-1:1997 Structural use of concrete

4.3 Method of Analysis

A combination of hand calculations, using established methods, computer analysis and design, and published data has been used in the design.

- Tekla TEDDS
- CADS A3D Max: 3D analysis
- CADS SW Moment Connections: Moment connection design
- CADS SW Member Designer: Steel member design

4.4 Foundation Overview

Site Investigations

A site investigation has not been completed and therefore the foundation scheme indicated is guidance only. The foundation guidance is based on an assumed bearing capacity of 100kNm-2. The bearing capacity and suitability of the existing and proposed foundations should be confirmed through a site investigation. If a site investigation is not provided then the suitability of the ground bearing strata should be verified by building control.

Ground Conditions

We have based this design upon a traditional ground bearing foundation solution. This comprises of mass concrete pad foundations, strip footings, and a ground bearing slab.

In some cases ground conditions may require more advance foundation systems such as Raft Foundations or Piles. If investigations on site determine that a more advanced foundation system is required we would be pleased to design and detail this.

Ground Water

The water table has not been identified. Ground water can impact on foundation design, installation and on the design of retaining structures.

4.5 Progressive Collapse

The building has been designed to be in accordance with requirement A3 of Approved Document A: Structure.

The building is a single occupancy two storey house and is a Class 1.

The project and the proposed works do not impact on the classification of the building and the proposed project does not impact on the robustness of the building.

4.6 Fire Resistance

The structural elements of the above project will be protected to the requirements of Approved Document B: Fire Safety of the Building Regulations

5 Loading

Where specified (unless noted otherwise), the below loading has been assumed for the purposes of the following calculations.

Roof		
Type	Gk (kNm ⁻²)	Qk (kNm ⁻²)
Timber Pitched Underdrawn (Slates, timber, insulation, ceiling)	1.25	0.60
Timber Pitched (Slates, timber, insulation, ceiling, ceiling joists, services)	1.31	0.85

Floors		
Type	Gk (kNm ⁻²)	Qk (kNm ⁻²)
Timber, Modern (Boarding, Joist, Insulation, Ceiling, Services)	0.60	1.50

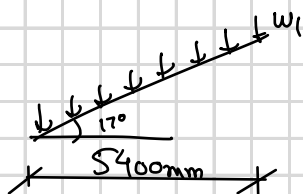
Walls		
Type	Gk (kNm ⁻²)	Qk (kNm ⁻²)
External (100 Brick, 100 Brick, Plaster)	4.56	NA
External (100 Brick, 100 Block, Plaster)	3.76	NA
Internal (100 Brick, Plaster either side)	2.61	NA

Misc.		
Type	Gk	Qk
Roof lights	1.10kNm ⁻²	0.60 kNm ⁻²

	Job No	Job Title	Calculation	Page No
	20218200	14 St. Clements Road	Rafters 1, RB1 & Trimmer 1	1
Date April 21	Made By MY	Checked By MM	Revision A0	

RAFTERS 1

GEOMETRY



LOADING

w_1

PITCHED ROOF

$$G_k = 1.25 \text{ kN/m}^2$$

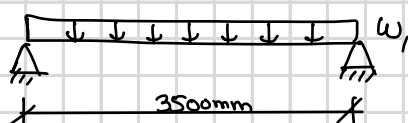
$$Q_k = 0.6 \text{ kN/m}^2$$

ADOPT.

[75 x 225mm C24 TIMBER RAFTERS @ 400mm c/c]

RIDGE BEAM (RB1)

GEOMETRY



LOADING

w_1

PITCHED ROOF

$$G_k = 1.25 \text{ kN/m}^2 \times (5.4/2 \times \cos 17^\circ + 2.7/2 \times \cos 29^\circ) = 5.45 \text{ kN/m}$$

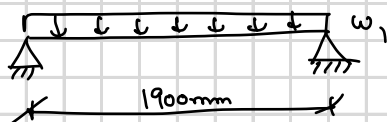
$$Q_k = 0.6 \text{ kN/m}^2 \times (5.4/2 + 2.7/2) = 2.43 \text{ kN/m}$$

ADOPT.

[UB 178 x 102 x 19 S275]

TRIMMER 1

GEOMETRY



LOADING

w_1

SKY LIGHT,

$$G_k = 1.1 \text{ kN/m}^2 \times 3.8/2 \times \cos 17^\circ = 2.1 \text{ kN/m}$$

$$Q_k = 0.6 \text{ kN/m}^2 \times 3.8/2 \text{ m} = 1.14 \text{ kN/m}$$

w_2

PITCHED ROOF,

$$G_k = 1.25 \text{ kN/m}^2 \times 1/2 \text{ m} = 0.625 \text{ kN/m}$$

$$Q_k = 0.6 \text{ kN/m}^2 \times 1/2 \text{ m} = 0.3 \text{ kN/m}$$

$$\begin{aligned} \Sigma G_k &= 2.73 \text{ kN/m} \\ \Sigma Q_k &= 1.44 \text{ kN/m} \end{aligned}$$

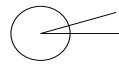
ADOPT

[2/75 x 225 mm C24 TIMBER TRIMMER]

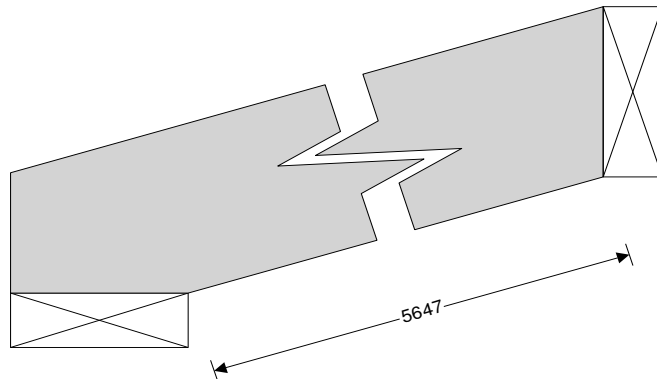
Project 14 St. Clement's Road				Job no. 20218200	
Calcs for Rafters 1				Start page no./Revision 2 A0	
Calcs by MY	Calcs date 25/04/2021	Checked by MM	Checked date	Approved by MM	Approved date

TIMBER RAFTER DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.03



17 degrees



Rafter details

Breadth of timber sections	b = 75 mm	Depth of timber sections	h = 225 mm
Rafter spacing	s = 400 mm	Rafter span	Single span
Clear length of span on slope	L _{cl} = 5647 mm	Rafter slope	α = 17.0 deg
Timber strength class	C24		

Section properties

Cross sectional area of rafter	A = 16875 mm²	Section modulus	Z = 632813 mm³
Radius of gyration	r = 65 mm	Second moment of area	I = 71191406 mm⁴

Loading details

Rafter self weight	F _j = 0.06 kN/m	Dead load on slope	F _d = 1.05 kN/m²
Imposed snow load on plan	F _u = 0.60 kN/m²	Imposed point load	F _p = 0.90 kN

Modification factors

Section depth factor	K ₇ = 1.03	Load sharing factor	K ₈ = 1.10
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Consider long term load condition

Load duration factor	K ₃ = 1.00	Total UDL perp. to rafter	F = 0.457 kN/m
Notional bearing length	L _b = 7 mm	Effective span	L _{eff} = 5653 mm

Check bending stress

Permissible bending stress	σ _{m_adm} = 8.515 N/mm²	Applied bending stress	σ _{m_max} = 2.885 N/mm²
PASS - Applied bending stress within permissible limits			

Check compressive stress parallel to grain

Permissible comp. stress	σ _{c_adm} = 4.147 N/mm²	Applied compressive stress	σ _{c_max} = 0.321 N/mm²
PASS - Applied compressive stress within permissible limits			

Check combined bending and compressive stress parallel to grain

Combined loading check	0.425 < 1	PASS - Combined compressive and bending stresses are within permissible limits	
------------------------	---------------------	---	--

Check shear stress

Permissible shear stress	τ _{adm} = 0.781 N/mm²	Applied shear stress	τ _{max} = 0.115 N/mm²
PASS - Applied shear stress within permissible limits			

Check deflection

Permissible deflection	δ _{adm} = 16.960 mm	Total deflection	δ _{max} = 8.098 mm
------------------------	-------------------------------------	------------------	------------------------------------

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PASS - Total deflection within permissible limits

Consider medium term load condition

Load duration factor	$K_3 = 1.25$	Total UDL perp. to rafter	$F = 0.677 \text{ kN/m}$
Notional bearing length	$L_b = 10 \text{ mm}$	Effective span	$L_{eff} = 5656 \text{ mm}$

Check bending stress

Permissible bending stress	$\sigma_{adm} = 10.644 \text{ N/mm}^2$	Applied bending stress	$\sigma_{max} = 4.276 \text{ N/mm}^2$
----------------------------	--	------------------------	---------------------------------------

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Permissible comp. stress	$\sigma_{c_adm} = 4.640 \text{ N/mm}^2$	Applied compressive stress	$\sigma_{c_max} = 0.475 \text{ N/mm}^2$
--------------------------	--	----------------------------	--

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Combined loading check **0.518 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Permissible shear stress	$\tau_{adm} = 0.976 \text{ N/mm}^2$	Applied shear stress	$\tau_{max} = 0.170 \text{ N/mm}^2$
--------------------------	-------------------------------------	----------------------	-------------------------------------

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection	$\delta_{adm} = 16.969 \text{ mm}$	Total deflection	$\delta_{max} = 12.013 \text{ mm}$
------------------------	------------------------------------	------------------	------------------------------------

PASS - Total deflection within permissible limits

Consider short term load condition

Load duration factor	$K_3 = 1.50$	Total UDL perp. to rafter	$F = 0.457 \text{ kN/m}$
Notional bearing length	$L_b = 9 \text{ mm}$	Effective span	$L_{eff} = 5655 \text{ mm}$

Check bending stress

Permissible bending stress	$\sigma_{adm} = 12.773 \text{ N/mm}^2$	Applied bending stress	$\sigma_{max} = 4.810 \text{ N/mm}^2$
----------------------------	--	------------------------	---------------------------------------

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Permissible comp. stress	$\sigma_{c_adm} = 5.007 \text{ N/mm}^2$	Applied compressive stress	$\sigma_{c_max} = 0.336 \text{ N/mm}^2$
--------------------------	--	----------------------------	--

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Combined loading check **0.452 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Permissible shear stress	$\tau_{adm} = 1.172 \text{ N/mm}^2$	Applied shear stress	$\tau_{max} = 0.191 \text{ N/mm}^2$
--------------------------	-------------------------------------	----------------------	-------------------------------------

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection	$\delta_{adm} = 16.966 \text{ mm}$	Total deflection	$\delta_{max} = 12.457 \text{ mm}$
------------------------	------------------------------------	------------------	------------------------------------

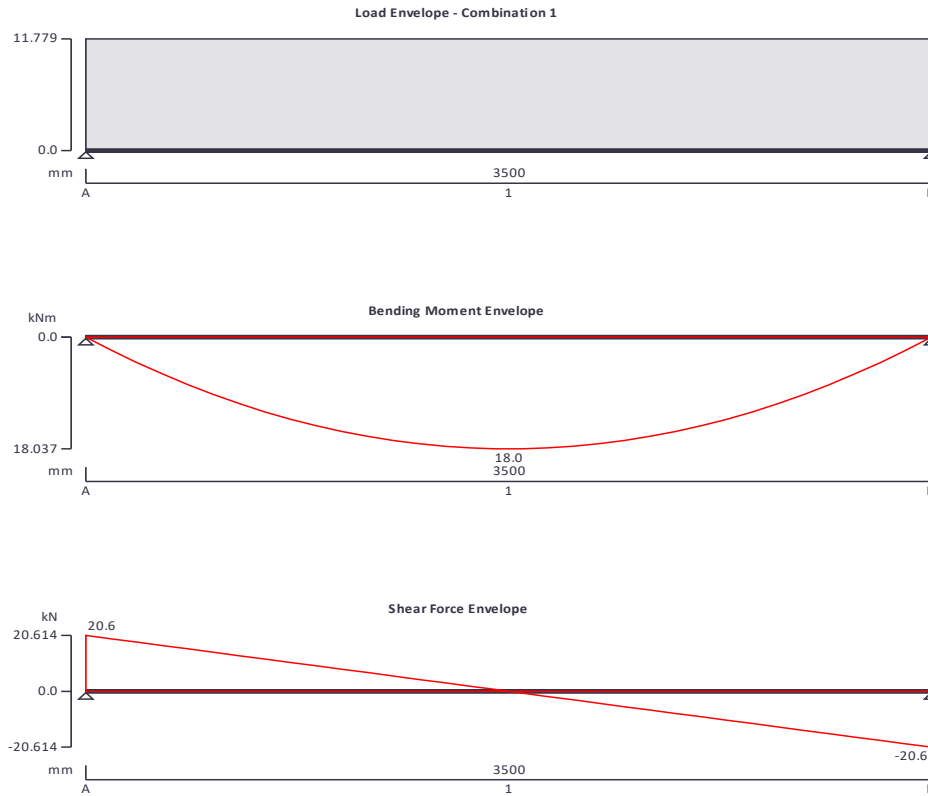
PASS - Total deflection within permissible limits

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Calcs for Ridge Beam 1				Start page no./Revision 4 A0	
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STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

w1 - Dead full UDL 5.45 kN/m

w1 - Imposed full UDL 2.43 kN/m

Load combinations

Load combination 1

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

Dead $\times 1.40$

Imposed $\times 1.60$

Support B

Dead $\times 1.40$

Imposed $\times 1.60$

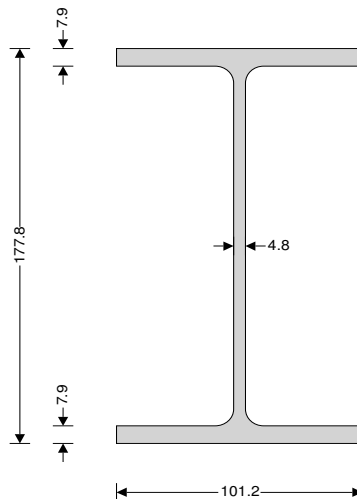
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Calcs for Ridge Beam 1				Start page no./Revision 5 A0	
Calcs by MY	Calcs date 26/04/2021	Checked by MM	Checked date	Approved by MM	Approved date

Analysis results

Maximum moment	$M_{max} = 18 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 20.6 \text{ kN}$	$V_{min} = -20.6 \text{ kN}$
Deflection	$\delta_{max} = 5.7 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{max}} = 20.6 \text{ kN}$	$R_{A_{min}} = 20.6 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_{Dead}} = 9.9 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_{Imposed}} = 4.3 \text{ kN}$	
Maximum reaction at support B	$R_{B_{max}} = 20.6 \text{ kN}$	$R_{B_{min}} = 20.6 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_{Dead}} = 9.9 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_{Imposed}} = 4.3 \text{ kN}$	

Section details

Section type	UKB 178x102x19 (Tata Steel Advance)	Steel grade	S275
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Classification of cross sections - Section 3.5

Tensile strain coefficient	$\epsilon = 1.00$	Section classification	Plastic
----------------------------	-------------------	------------------------	----------------

Shear capacity - Section 4.2.3

Design shear force	$F_v = 20.6 \text{ kN}$	Design shear resistance	$P_v = 140.8 \text{ kN}$
PASS - Design shear resistance exceeds design shear force			

Moment capacity - Section 4.2.5

Design bending moment	$M = 18 \text{ kNm}$	Moment capacity low shear	$M_c = 47.1 \text{ kNm}$
-----------------------	----------------------	---------------------------	--------------------------

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment	$M_b = 17.2 \text{ kNm}$	$M_b / m_{LT} = 18.6 \text{ kNm}$
PASS - Buckling resistance moment exceeds design bending moment		

Check vertical deflection - Section 2.5.2

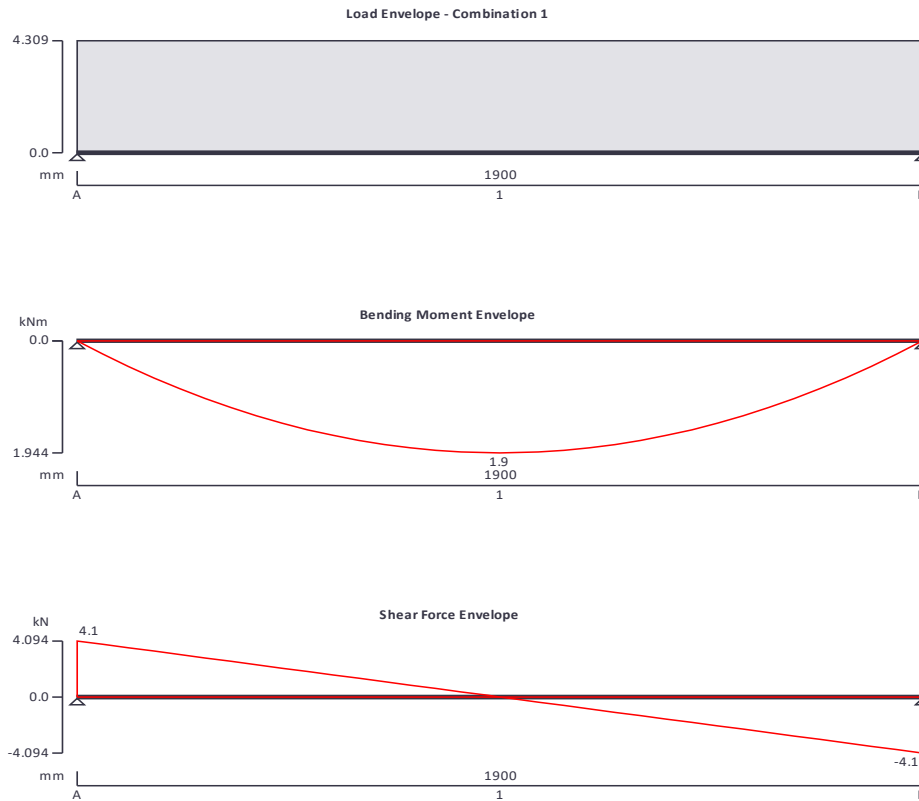
Consider deflection due to dead and imposed loads

Limiting deflection	$\delta_{lim} = 14 \text{ mm}$	Maximum deflection	$\delta = 5.67 \text{ mm}$
PASS - Maximum deflection does not exceed deflection limit			

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TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.02



Applied loading

Beam loads

gk

Dead self weight of beam $\times 1$

qk

Dead full UDL 2.730 kN/m

Imposed full UDL 1.440 kN/m

Load combinations

Load combination 1

Support A

Dead $\times 1.00$

Imposed $\times 1.00$

Span 1

Dead $\times 1.00$

Imposed $\times 1.00$

Support B

Dead $\times 1.00$

Imposed $\times 1.00$

Analysis results

Design moment

M = **1.944 kNm**

Design shear

F = **4.094 kN**

Total load on beam

W_{tot} = **8.187 kN**

Reactions at support A

R_{A_max} = **4.094 kN**

R_{A_min} = **4.094 kN**

Unfactored dead load reaction at support A

R_{A_Dead} = **2.726 kN**

Unfactored imposed load reaction at support A

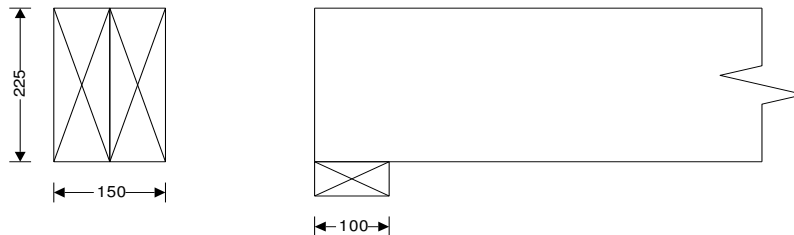
R_{A_Imposed} = **1.368 kN**

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Reactions at support B $R_{B_max} = 4.094$ kN $R_{B_min} = 4.094$ kN

Unfactored dead load reaction at support B $R_{B_Dead} = 2.726$ kN

Unfactored imposed load reaction at support B $R_{B_Imposed} = 1.368$ kN



Timber section details

Breadth of section $b = 75$ mm Depth of section $h = 225$ mm
 Number of sections $N = 2$ Breadth of beam $b_b = 150$ mm
 Timber strength class **C24**

Member details

Service class of timber **1** Load duration **Long term**
 Length of span $L_{s1} = 1900$ mm
 Length of bearing $L_b = 100$ mm

Lateral support - cl.2.10.8

Permiss.depth-to-breadth ratio **3.00** Actual depth-to-breadth ratio **1.50**
PASS - Lateral support is adequate

Check bearing stress

Permissible bearing stress $\sigma_{c_adm} = 2.640$ N/mm² Applied bearing stress $\sigma_{c_a} = 0.273$ N/mm²
PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress $\sigma_{m_adm} = 8.515$ N/mm² Applied bending stress $\sigma_{m_a} = 1.536$ N/mm²
PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress $\tau_{adm} = 0.781$ N/mm² Applied shear stress $\tau_a = 0.182$ N/mm²
PASS - Applied shear stress is less than permissible shear stress

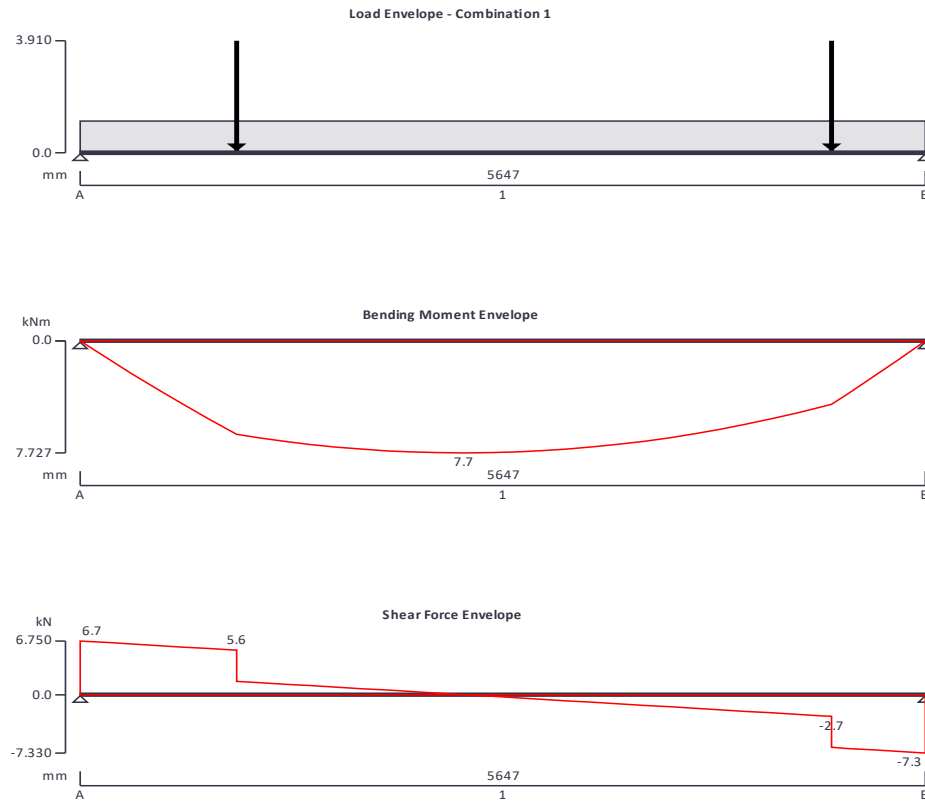
Deflection

Permissible deflection $\delta_{adm} = 5.700$ mm Total deflection $\delta_a = 0.760$ mm
PASS - Total deflection is less than permissible deflection

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TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.02



Applied loading

Beam loads

w1	Dead self weight of beam $\times 1$
w1	Dead full UDL 0.300 kN/m
t1	Imposed full UDL 0.600 kN/m
t1	Dead point load 2.600 kN at 1046 mm
t1	Imposed point load 1.310 kN at 1046 mm
t1	Dead point load 2.600 kN at 5020 mm
t1	Imposed point load 1.310 kN at 5020 mm
Axial load	Factored axial compression 2.060 kN

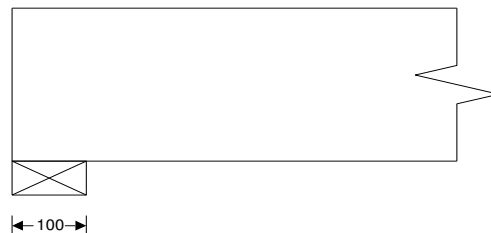
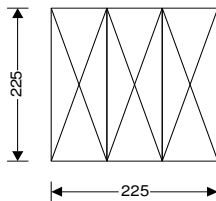
Load combinations

Load combination 1	Support A	Dead $\times 1.00$ Imposed $\times 1.00$
	Span 1	Dead $\times 1.00$ Imposed $\times 1.00$
	Support B	Dead $\times 1.00$ Imposed $\times 1.00$

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Analysis results

Design moment	M = 7.727 kNm	Design shear	F = 7.330 kN
Total load on beam	W _{tot} = 14.080 kN		
Reactions at support A	R _{A_max} = 6.750 kN	R _{A_min} = 6.750 kN	
Unfactored dead load reaction at support A	R _{A_Dead} = 3.843 kN		
Unfactored imposed load reaction at support A	R _{A_Imposed} = 2.907 kN		
Reactions at support B	R _{B_max} = 7.330 kN	R _{B_min} = 7.330 kN	
Unfactored dead load reaction at support B	R _{B_Dead} = 4.229 kN		
Unfactored imposed load reaction at support B	R _{B_Imposed} = 3.101 kN		



Timber section details

Breadth of section	b = 75 mm	Depth of section	h = 225 mm
Number of sections	N = 3	Breadth of beam	b _b = 225 mm
Timber strength class	C24		

Member details

Service class of timber	1	Load duration	Long term
Length of span	L _{s1} = 5647 mm		
Length of bearing	L _b = 100 mm		
Unbraced length in x-axis	L _x = 5647 mm	Unbraced length in y-axis	L _y = 5647 mm Effective
length factor in x-axis	K _x = 1	Effective length factor in y-axis	K _y = 1
Effective length in x-axis	L _{ex} = 5647 mm	Effective length in y-axis	L _{ey} = 5647 mm

Lateral support - cl.2.10.8

Permiss.depth-to-breadth ratio	3.00	Actual depth-to-breadth ratio	1.00
			PASS - Lateral support is adequate

Slenderness ratio - cl.2.11.4

Slenderness ratio	λ = 86.941	Permissible slenderness ratio	λ _{max} = 180
			PASS - Slenderness ratio is less than permissible slenderness ratio

Check bearing stress

Permissible bearing stress	σ _{c_adm} = 2.640 N/mm ²	Applied bearing stress	σ _{c_a} = 0.326 N/mm ²
			PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress	σ _{m_adm} = 8.515 N/mm ²	Applied bending stress	σ _{m_a} = 4.070 N/mm ²
			PASS - Applied bending stress is less than permissible bending stress

Compression parallel to grain

Permissible comp.stress	σ _{c_adm} = 4.481 N/mm ²	Applied compressive stress	σ _{c_a} = 0.041 N/mm ²
			PASS - Applied compressive stress is less than permissible compressive stress

Members subject to axial compression and bending - cl.2.11.6

Comb.comp.and bending	0.488 < 1
-----------------------	-----------

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PASS - Combined compressive and bending stresses are within permissible limits

Shear parallel to grain

Permissible shear stress

$$\tau_{adm} = 0.781 \text{ N/mm}^2$$

Applied shear stress

$$\tau_a = 0.217 \text{ N/mm}^2$$

PASS - Applied shear stress is less than permissible shear stress

Deflection

Permissible deflection

$$\delta_{adm} = 15.011 \text{ mm}$$

Total deflection

$$\delta_a = 14.991 \text{ mm}$$

PASS - Total deflection is less than permissible deflection

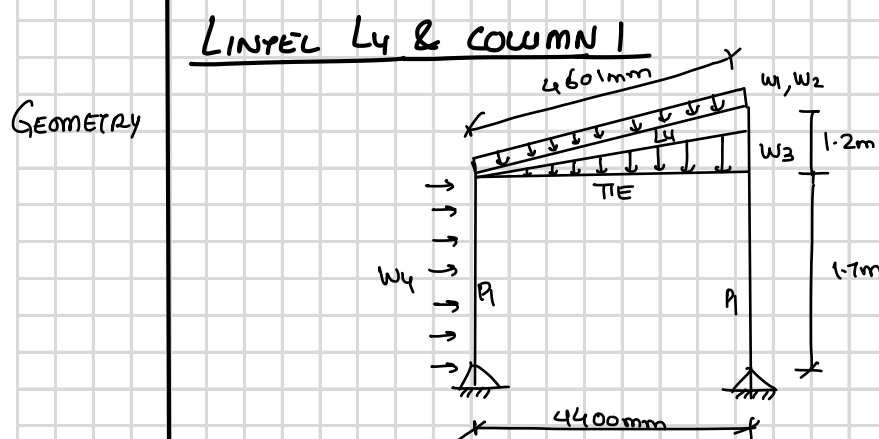
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ADOPT LINTEL 1
[BY INSPECTION - NAYLOR S4 LINTEL]

ADOPT LINTEL 2
[BY INSPECTION - CARNIC CG LINTEL]

ADOPT LINTEL 3(a)
[BY INSPECTION - UB 152x89x16 S275]

ADOPT LINTEL 3(b)
[BY INSPECTION - CARNIC ANG LINTEL]



LOADING

W₁ NOMINAL PITCHED ROOF,
 $G_k = 1.25 \text{ kN/m}^2 \times 0.4 \text{ m} = 0.5 \text{ kN/m}$
 $Q_k = 0.6 \text{ kN/m}^2 \times 0.4 \text{ m} = 0.24 \text{ kN/m}$

W₂ MASONRY
 $G_k = 3.76 \text{ kN/m}^2 \times 0.6 \text{ m} = 2.26 \text{ kN/m}$

W₃ GLAZING
 $G_k = 0.6 \text{ kN/m}^2 \times 1.2 \text{ m} = 0.72 \text{ kN/m}$

W₄ WIND LOAD
 $W_k = 0.8 \text{ kN/m}^2 \times \frac{3.5 \text{ m}}{2} = 1.4 \text{ kN/m}$

TORSION = $22 \times 0.1 \times 0.2 \times \frac{44}{\cos 17^\circ} \times 0.6 \text{ m} = 1.21 \text{ kNm}$
 ANALYSIS = $1.6 = 1.94 \text{ kNm}$

MAX BM = 2.1 kNm
 MAX V = 3.3 kN
 MAX S = 1.4 mm

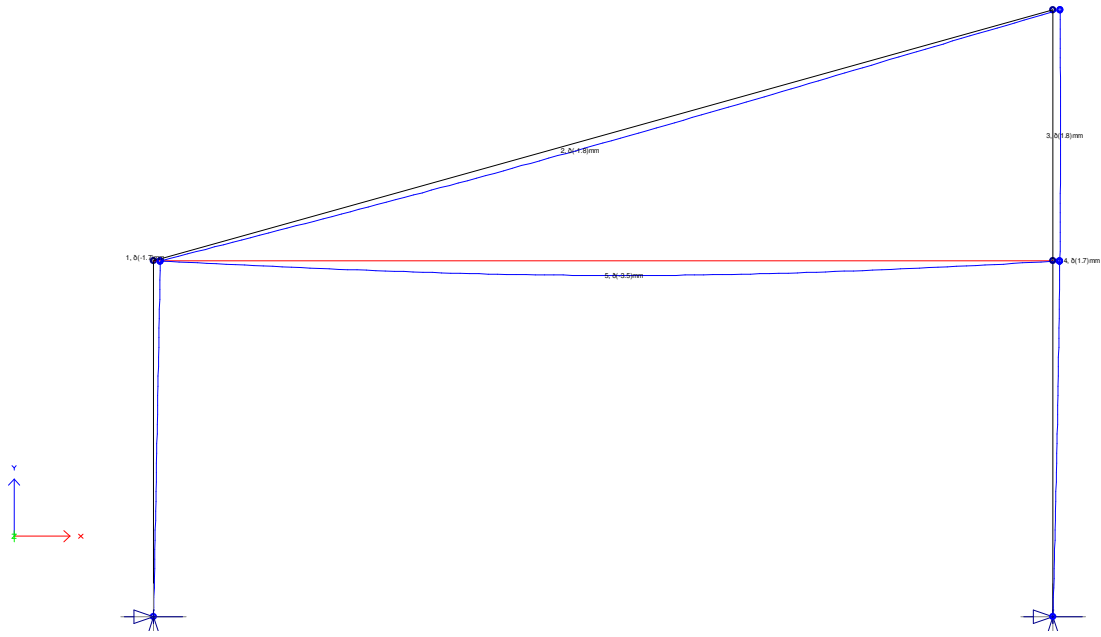
ADOPT

L4 RAFTER	: RHS 150x100x6.3 S355 w/10mm STEEL S275 BOTTOM PLATE
TIE 1	: UB 178x102x19 S275
P1	: SHS 100x100x8 S355

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DEFLECTION DIAGRAM

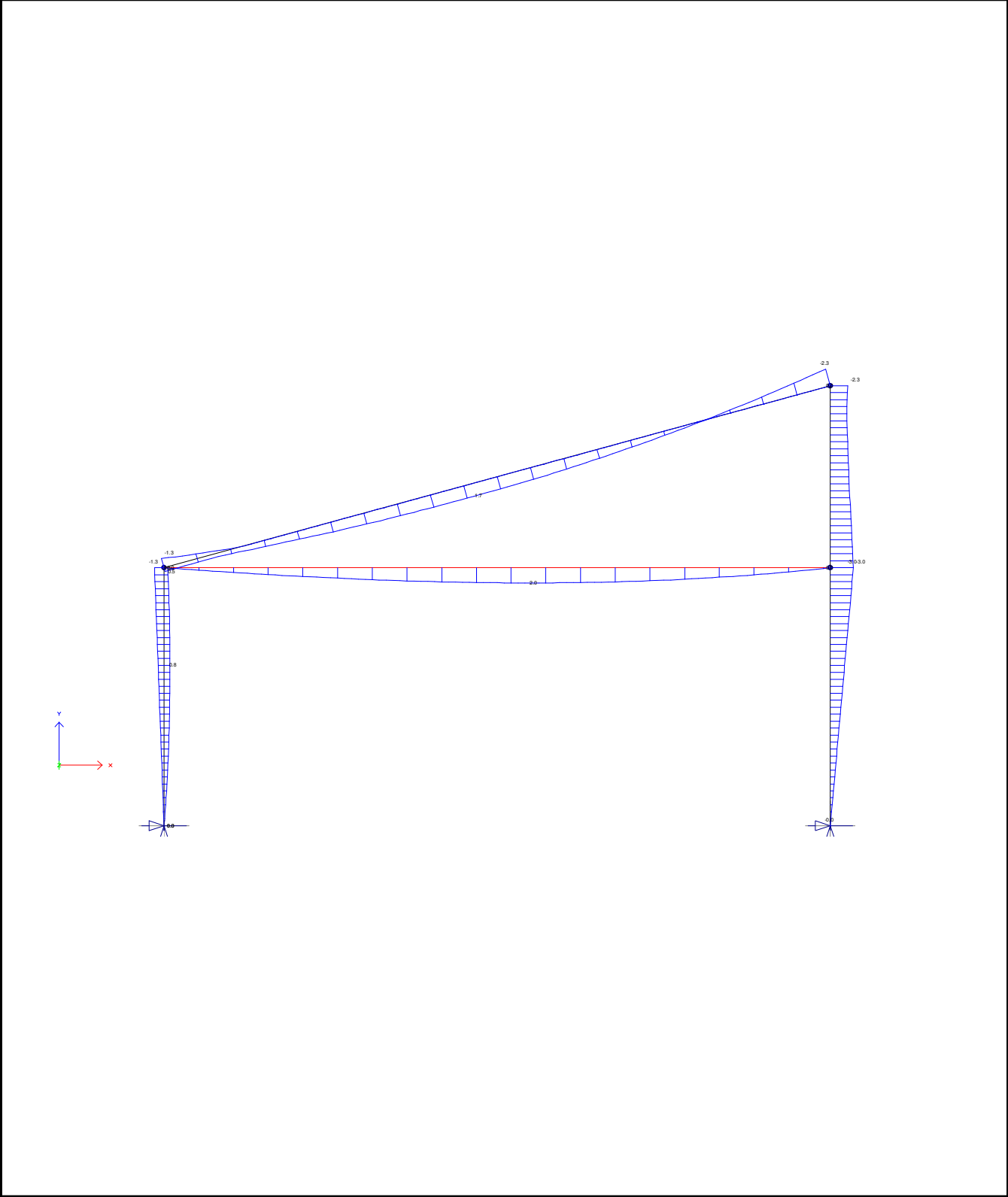


Max Deflection = 3.5mm

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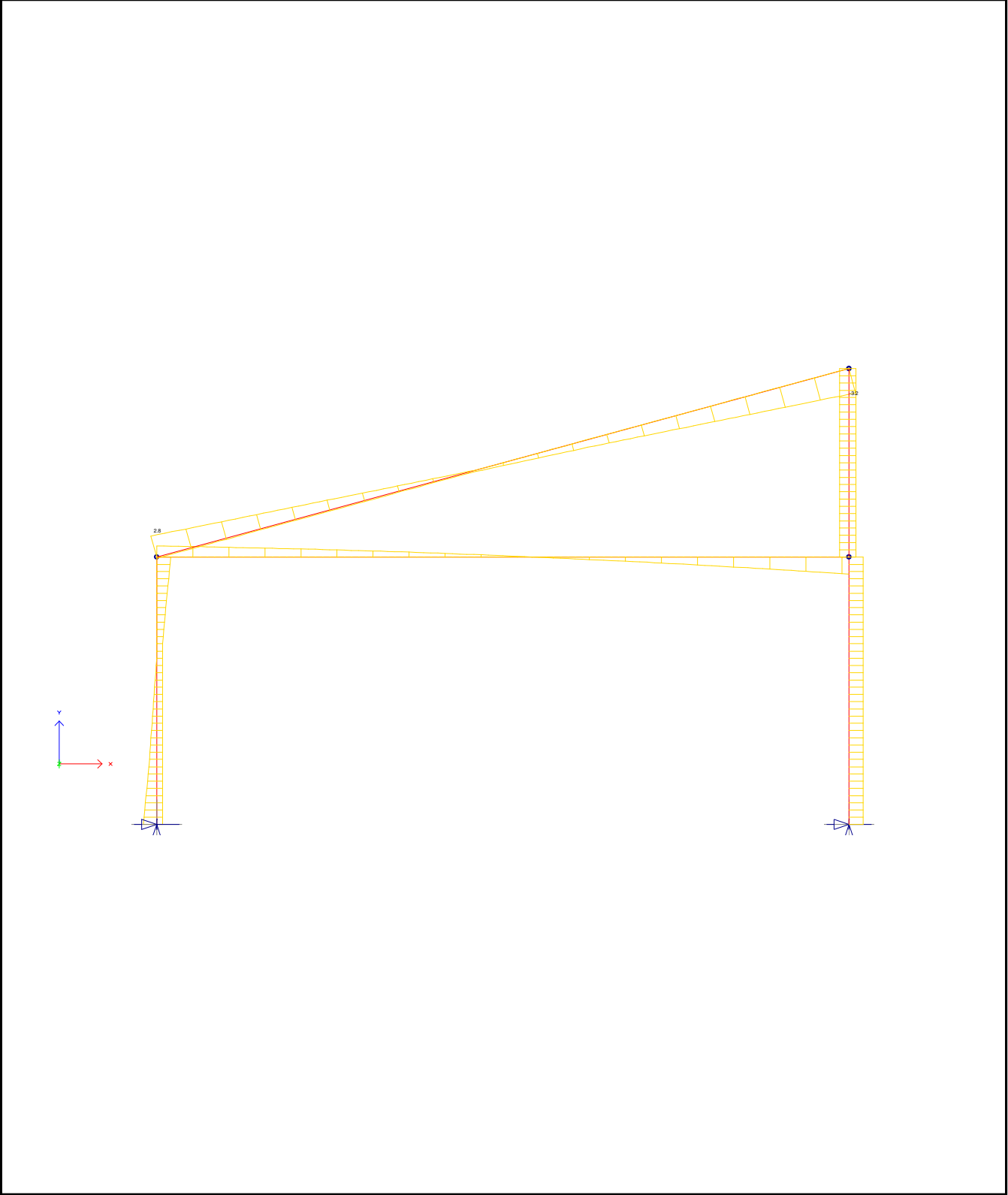
MOMENT ENVELOPE



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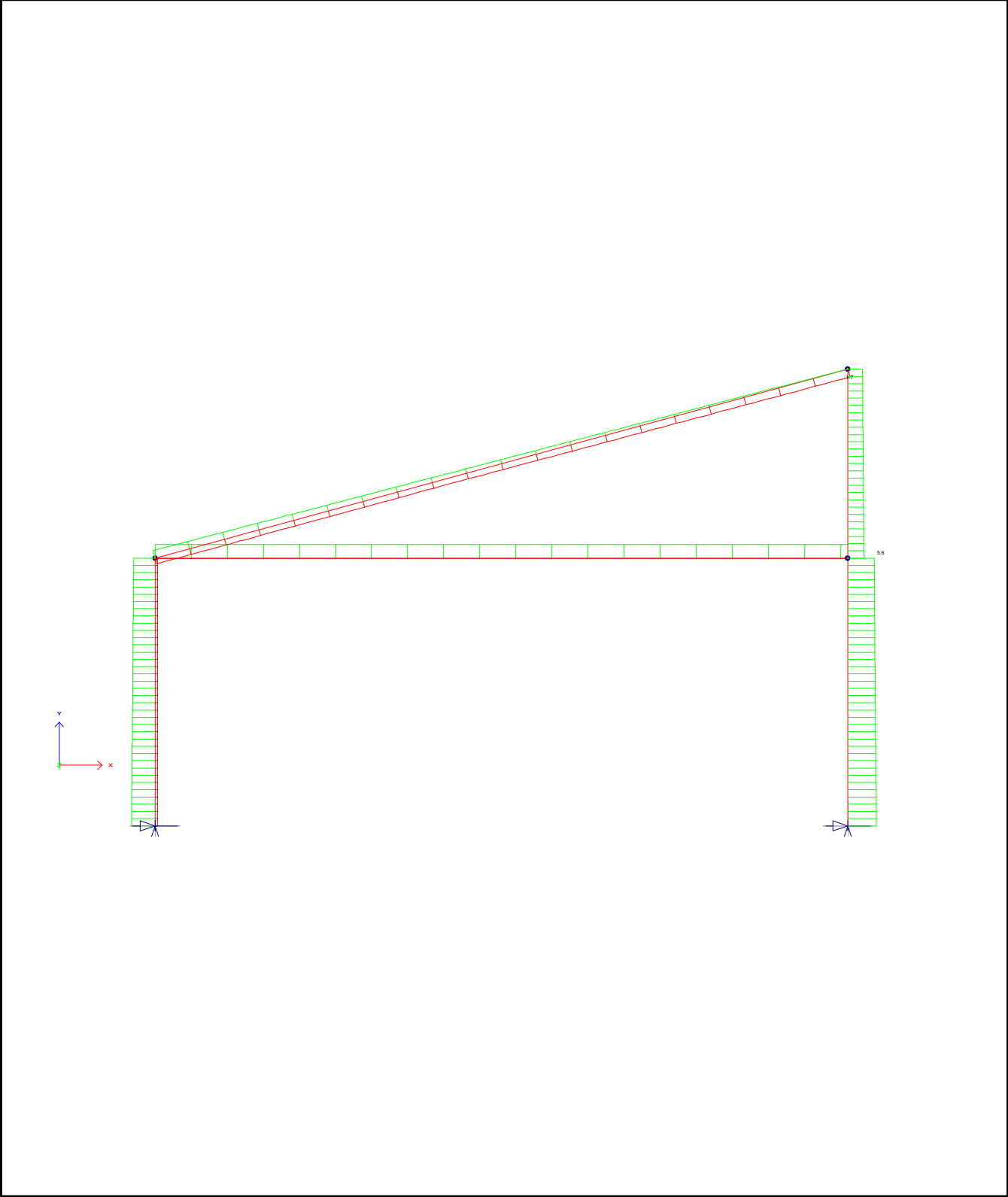
SHEAR DIAGRAM



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AXIAL DIAGRAM



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Member Loads

Load reference	Load type	Start pos'n (m)	Start intensity (kN) & (m)	End pos'n (m)	End intensity (kN) & (m)	Direction	Category
Loads on member 1 (Length 1.700m)							
W4	UL		1.400			Horiz. (+X)	Wind X
Loads on member 2 (Length 4.561m)							
W1	UL		0.500			Vertical (-Y)	Dead
W1	UL		0.240			Vertical (-Y)	Imposed
Loads on member 5 (Length 4.400m)							
W3	DL	0.000	0.000	4.400	0.720	Vertical (-Y)	Dead

Support Reactions for Combination GK

Analysis Type : Linear elastic

Joint reference	Support reactions (kN)			Support moments (kNm)		
	Px	Py	Pz	Mx	My	Mz
1	0.383	2.952	0.000	0.000	0.000	0.000
5	-0.383	3.704	0.000	0.000	0.000	0.000

Support Reactions for Combination QK

Analysis Type : Linear elastic

Joint reference	Support reactions (kN)			Support moments (kNm)		
	Px	Py	Pz	Mx	My	Mz
1	0.125	0.528	0.000	0.000	0.000	0.000
5	-0.125	0.528	0.000	0.000	0.000	0.000

Support Reactions for Combination WK

Analysis Type : Linear elastic

Joint reference	Support reactions (kN)			Support moments (kNm)		
	Px	Py	Pz	Mx	My	Mz
1	-1.484	-0.460	0.000	0.000	0.000	0.000
5	-0.896	0.460	0.000	0.000	0.000	0.000

Steelwork Design Results

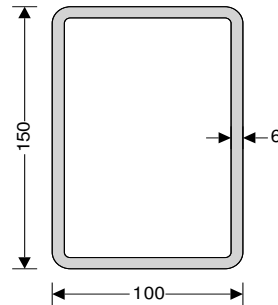
Design object ref	Memb ref	Design parameter	Analysis section	Design section	Utilisation factors				Design status
					Local capacity	Lateral buckling	Torsion	Deflection	
Group Reference - Beam									
3	2	Beam	150x100x6.3 RHS		0.058	0.043	n / a	0.073	Passed
Group Reference - TIE									
5	5	TIE	100x100x8.0 SHS		0.046	0.051	n / a	0.283	Passed
Group Reference - POST 1									
7	3-4	POST 1	178x102 UKB19		0.050	0.106	n / a	0.173	Passed
8	1	POST 1	178x102 UKB19		0.021	0.021	n / a	0.294	Passed

Project 14 St. Clement's Road				Job no. 20218200	
Calcs for L4 Torsion Analysis				Start page no./Revision 17 A0	
Calcs by MY	Calcs date 25/04/2021	Checked by MM	Checked date	Approved by MM	Approved date

STEEL BEAM TORSION DESIGN

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

Tedds calculation version 2.0.02



Section details

Section type	RHS 150x100x6.3	Steel grade	S355
Design strength	$p_{yw} = p_y = 355 \text{ N/mm}^2$	Constant	$\varepsilon = 0.880$

Calculated torsional properties

Height of web	$d_x = b_y = D - 3 \times t = 131.1 \text{ mm}$
Width of flange	$b_x = d_y = B - 3 \times t = 81.1 \text{ mm}$
Shear area - y	$A_{vy} = A \times D / (D + B) = 17.7 \text{ cm}^2$
Shear area - x	$A_{vx} = A \times B / (D + B) = 11.8 \text{ cm}^2$
	$d_{vx} = A_{vx} / (2 \times t) = 93.6 \text{ mm}$
	$d_{vy} = A_{vy} / (2 \times t) = 140.4 \text{ mm}$
Plastic modulus of web	$S_{vx} = 2 \times t \times d_{vy}^2 / 4 = 62.1 \text{ cm}^3$
Plastic modulus of flange	$S_{vy} = 2 \times t \times d_{vx}^2 / 4 = 27.6 \text{ cm}^3$
Statical moment at mid-depth	$Q_w = S_x / 2 = 73 \text{ cm}^3$

Geometry - Beam unrestrained against lateral-torsional buckling between supports.

Effective span	$L = 4601 \text{ mm}$		
Length of segment LTB	$L_{LT} = 4601 \text{ mm}$	Effective length for LTB	$L_{E_LT} = 5821 \text{ mm}$

Loading - Torsional loading comprises only full-length uniformly distributed load(s)

Internal forces & moments on member under factored loading for ult design

Applied shear force	$F_{vy} = 3.3 \text{ kN}$	Maximum bending moment	$M_{LT} = M_x = 2.10 \text{ kNm}$
Applied torque	$T_q = 2.00 \text{ kNm}$	Minor axis bending moment	$M_y = 0 \text{ kNm}$
Compression force	$F_c = 0 \text{ kN}$		

Equivalent uniform moment factors

EUM factor (Cl.4.3.6.6 & T18) $m_{LT} = 1.000$

Torsional deflection analysis

Beam is torsion fixed at each end. (as defined in SCI-P-057 section 2.1.6)

Max torque (at supports)	$T_o = 1.00 \text{ kNm}$	Avg torque support & CL	$T_{av} = 0.50 \text{ kNm}$
Max. angle of twist (midspan)	$\phi = 0.001 \text{ rads}$		

Section classification

$b_x / t = 12.9$	$d_x / t = 20.8$
$b_y / t = 20.8$	$d_y / t = 12.9$
$r_{1sx} = 0.000$	$r_{1sy} = 0.000$

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 $r_{2s} = 0.000$

Section classification is plastic

Shear capacity (parallel to y-axis)

Design shear force $F_{vy} = 3.3$ kN

Design shear resist (cl. 4.2.3) $P_{vy} = 376.8$ kN

Pass - Shear

Moment capacity (x-axis)

Design bending moment $M_x = 2.1$ kNm

Mnt cap low shear (cl. 4.2.5.1) $M_{cx} = 51.0$ kNm

Pass - Moment capacity exceeds design bending moment

Lateral torsional buckling

Effective length for LTB $L_{E_LT} = 5821$ mm

Slenderness ratio - cl 4.3.6.5 $\lambda = 145$

$D / B = 1.5$

LTB check not required

Buckling resistance mnt $M_b = M_{cx} = 51.0$ kNm

Buckling under combined bending & torsion - SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Max angle of twist $\phi = 0.001$ rads

Induced minor axis moment $M_{yt} = 0.00$ kNm

Norm stress corner due to M_{yt} $\sigma_{byt} = 0$ N/mm²

Interaction index $i_b = 0.04$

Pass - Combined bending and torsion check satisfied

Local capacity under combined bending & torsion

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Max. direct stress due to M_x $\sigma_{bx} = M_x / Z_x = 18$ N/mm²

Combined stress - eqn 2.22 $\sigma_{bx} + \sigma_{byt} = 18$ N/mm²

Design strength

$p_y = 355$ N/mm²

Pass - Local capacity

Combined shear stresses SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum shear stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Max. shear stress bending $\tau_{bw} = 2$ N/mm²

Max. shear stresses torsion $\tau_t = 7$ N/mm²

Amplified shear stress torsion $\tau_{vt} = 7$ N/mm²

Combined shear bend & tors $\tau = 9$ N/mm²

Shear strength $p_v = 213$ N/mm²

Pass - Combined shear stresses

Twist check

Total applied torque $T_{qu} = 1.21$ kNm

Max twist under sls loading $\phi_{sls} = 0.05$ deg

Twist limit

$\phi_{lim} = 2.00$ deg

Pass - Twist

Deflection

Maximum y-axis deflection $\delta_{y_max} = 1.4$ mm

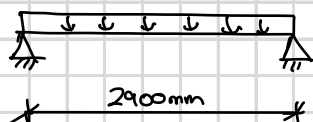
Deflection limit - cl. 2.5.2 $\delta_{lim} = 17.6$ mm

Pass - Deflection within specified limit

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	20218200	14 St. Clements Road	Beam 2 & 3	19
Date April 21	Made By MY	Checked By MM	Revision A0	

BEAM 2

GEOMETRY



LOADING

W₁

PITCHED ROOF VIA PURLIN

$$G_k = 1.31 \text{ kN/m}^2 \times (4/3 + 2.3/2) \times 3.9/2 \times \cos 30^\circ = 7.3 \text{ kN}$$

$$Q_k = 0.85 \text{ kN/m}^2 \times (4/3 + 2.3/2) \times 3.9/2 = 4.11 \text{ kN}$$

LOAD ON B2

$$G_k = 7.3/2.9 = 2.52 \text{ kN/m}$$

$$Q_k = 4.11/2.9 = 1.42 \text{ kN/m}$$

W₂

MASONRY WALL ABOVE

$$G_k = 2.61 \text{ kN/m}^2 \times 2.4 \text{ m} = 6.3 \text{ kN/m}$$

W₃

FLOOR

$$G_k = 0.6 \text{ kN/m}^2 \times (4 + 2.3)/2 \text{ m} = 1.9 \text{ kN/m} \quad \Sigma G_k = 10.72 \text{ kN/m}$$

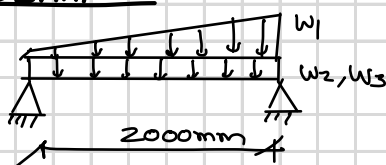
$$Q_k = 1.5 \text{ kN/m}^2 \times (4 + 2.3)/2 \text{ m} = 4.73 \text{ kN/m} \quad \Sigma Q_k = 6.15 \text{ kN/m}$$

ADOPT

[UC 152x152x23 S275]

BEAM 3

GEOMETRY



LOADING

W₁

PITCHED ROOF

$$G_k = 1.31 \text{ kN/m}^2 \times 2.3/2 \times \cos 30^\circ = 1.74 \text{ kN/m}$$

$$Q_k = 0.85 \text{ kN/m}^2 \times 2.3/2 = 0.98 \text{ kN/m}$$

W₂

PITCHED ROOF (EXTENSION)

$$G_k = 1.25 \text{ kN/m}^2 \times 1.7/2 \text{ m} = 1.1 \text{ kN/m}$$

$$Q_k = 0.6 \text{ kN/m}^2 \times 1.7/2 \text{ m} = 0.51 \text{ kN/m}$$

W₃

MASONRY

$$G_k = 4.56 \text{ kN/m}^2 \times 2.4 \text{ m} = 10.94 \text{ kN/m}$$

W₄

FLOOR

$$G_k = 0.6 \text{ kN/m}^2 \times 2.3/2 \text{ m} = 0.7 \text{ kN/m}$$

$$Q_k = 1.5 \text{ kN/m}^2 \times 2.3/2 \text{ m} = 1.73 \text{ kN/m}$$

ADOPT.

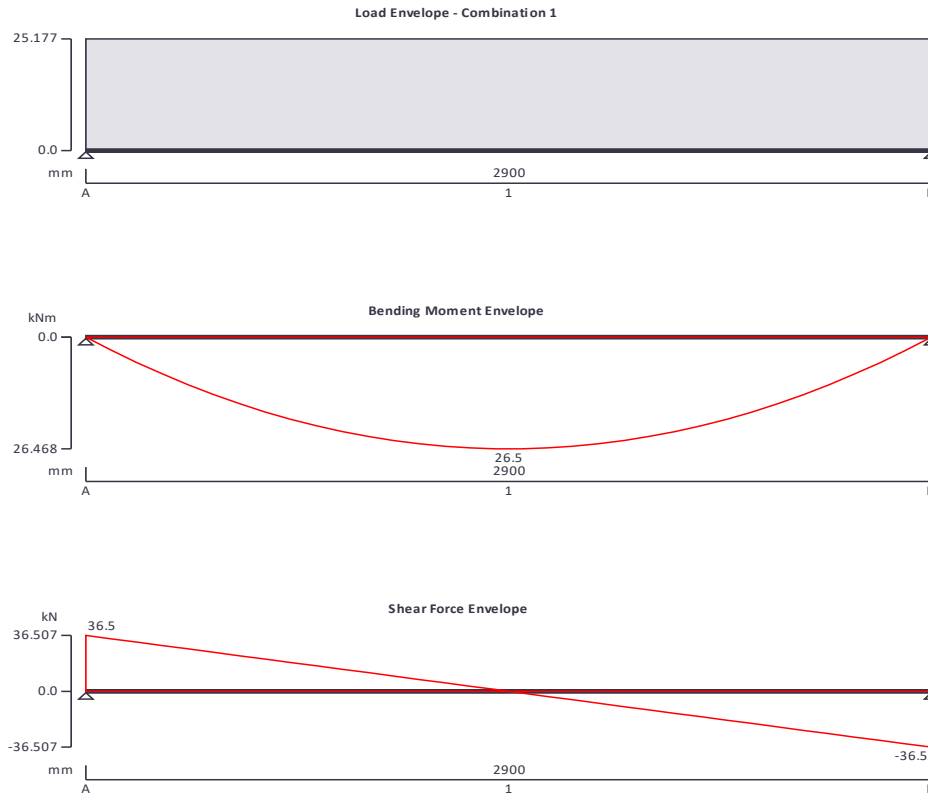
[UC 152x152x23 S275 w/10mm THICK STEEL TOP PLATE]

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Calcs for Beam 2				Start page no./Revision 20 A0	
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STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

gk - Dead full UDL 10.73 kN/m

qk - Imposed full UDL 6.15 kN/m

Load combinations

Load combination 1

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

Dead $\times 1.40$

Imposed $\times 1.60$

Support B

Dead $\times 1.40$

Imposed $\times 1.60$

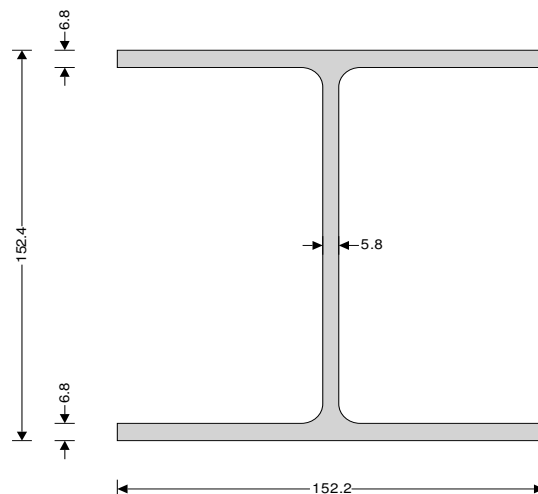
Project 14 St. Clement's Road				Job no. 20218200	
Calcs for Beam 2				Start page no./Revision 21 A0	
Calcs by MY	Calcs date 26/04/2021	Checked by MM	Checked date	Approved by MM	Approved date

Analysis results

Maximum moment	$M_{max} = 26.5 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 36.5 \text{ kN}$	$V_{min} = -36.5 \text{ kN}$
Deflection	$\delta_{max} = 6.1 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{max}} = 36.5 \text{ kN}$	$R_{A_{min}} = 36.5 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_{Dead}} = 15.9 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_{Imposed}} = 8.9 \text{ kN}$	
Maximum reaction at support B	$R_{B_{max}} = 36.5 \text{ kN}$	$R_{B_{min}} = 36.5 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_{Dead}} = 15.9 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_{Imposed}} = 8.9 \text{ kN}$	

Section details

Section type **UKC 152x152x23 (Tata Steel Advance)** Steel grade **S275**



Classification of cross sections - Section 3.5

Tensile strain coefficient $\epsilon = 1.00$ Section classification **Semi-compact**

Shear capacity - Section 4.2.3

Design shear force $F_v = 36.5 \text{ kN}$ Design shear resistance $P_v = 145.8 \text{ kN}$
PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment $M = 26.5 \text{ kNm}$ Moment capacity low shear $M_c = 48.5 \text{ kNm}$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment $M_b = 33.5 \text{ kNm}$ $M_b / m_{LT} = 36.2 \text{ kNm}$
PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

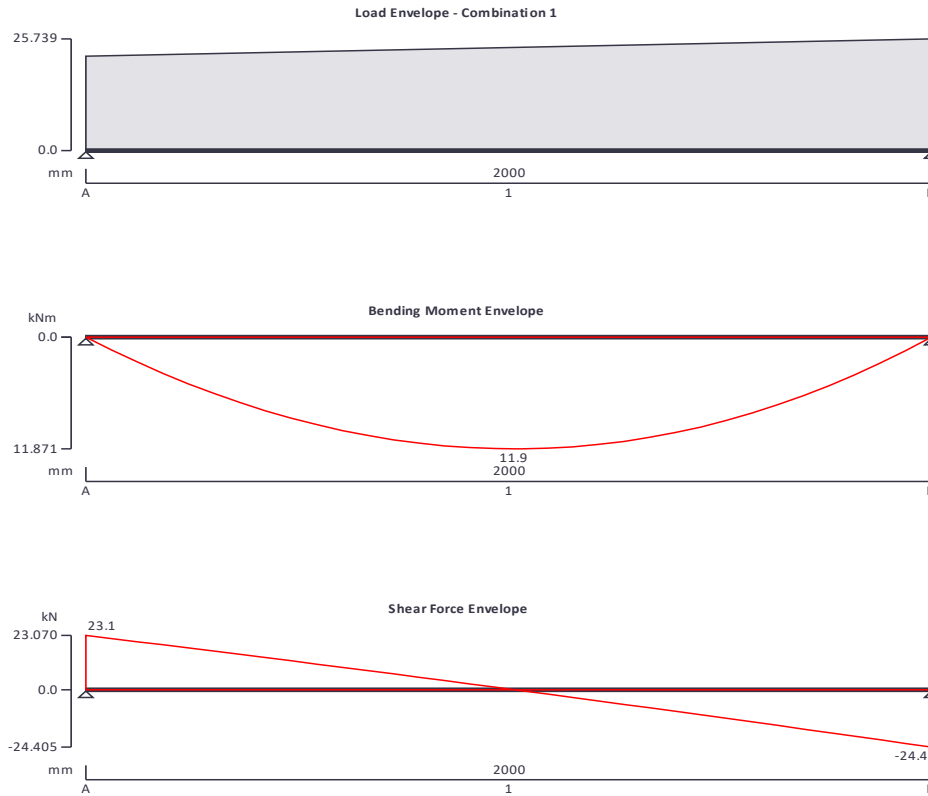
Limiting deflection $\delta_{lim} = 11.6 \text{ mm}$ Maximum deflection $\delta = 6.148 \text{ mm}$
PASS - Maximum deflection does not exceed deflection limit

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STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

w1 - Dead full VDL 0 kN/m to 1.74 kN/m

w1 - Imposed full VDL 0 kN/m to 0.98 kN/m

w2 - Dead full UDL 1.1 kN/m

w2 - Imposed full UDL 0.51 kN/m

w3 - Dead full UDL 10.94 kN/m

w4 - Dead full UDL 0.7 kN/m

w4 - Imposed full UDL 1.73 kN/m

Load combinations

Load combination 1

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

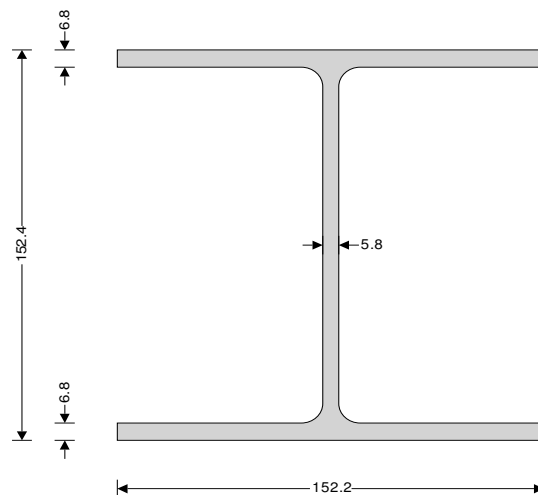
Project 14 St. Clement's Road				Job no. 20218200	
Calcs for Beam 3				Start page no./Revision 23 A0	
Calcs by MY	Calcs date 26/04/2021	Checked by MM	Checked date	Approved by MM	Approved date

Analysis results

Maximum moment	$M_{max} = 11.9 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 23.1 \text{ kN}$	$V_{min} = -24.4 \text{ kN}$
Deflection	$\delta_{max} = 1.3 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{max}} = 23.1 \text{ kN}$	$R_{A_{min}} = 23.1 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_{Dead}} = 13.5 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_{Imposed}} = 2.6 \text{ kN}$	
Maximum reaction at support B	$R_{B_{max}} = 24.4 \text{ kN}$	$R_{B_{min}} = 24.4 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_{Dead}} = 14.1 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_{Imposed}} = 2.9 \text{ kN}$	

Section details

Section type **UKC 152x152x23 (Tata Steel Advance)** Steel grade **S275**



Classification of cross sections - Section 3.5

Tensile strain coefficient $\epsilon = 1.00$ Section classification **Semi-compact**

Shear capacity - Section 4.2.3

Design shear force $F_v = 24.4 \text{ kN}$ Design shear resistance $P_v = 145.8 \text{ kN}$
PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment $M = 11.9 \text{ kNm}$ Moment capacity low shear $M_c = 48.5 \text{ kNm}$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment $M_b = 40.4 \text{ kNm}$ $M_b / m_{LT} = 43.7 \text{ kNm}$
PASS - Buckling resistance moment exceeds design bending moment

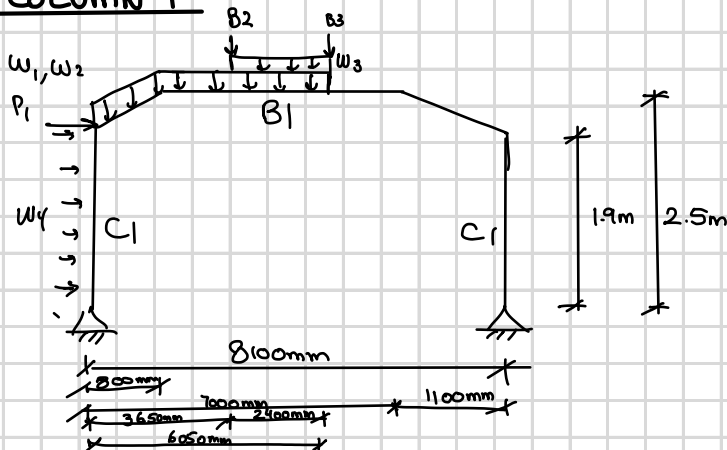
Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection $\delta_{lim} = 8 \text{ mm}$ Maximum deflection $\delta = 1.347 \text{ mm}$
PASS - Maximum deflection does not exceed deflection limit

BEAM 1 & COLUMN 1

GEOMETRY



LOADING
W1

MASONRY

$$G_k = 4.56 \text{ kN/m}^2 \times 2.4 \text{ m} = 10.94 \text{ kN/m}$$

W2

PITCHED ROOF,

$$G_k = 1.31 \text{ kN/m}^2 \times 3.9 / 4 \times \cos 30^\circ = 1.5 \text{ kN/m}$$

$$Q_k = 0.85 \text{ kN/m}^2 \times 3.9 / 4 \text{ m} = 0.83 \text{ kN/m}$$

W3

LOAD FROM RB1

$$G_k = 9.9 \text{ kN} / 1.2 \times 2 = 4.04 \text{ kN/m}$$

$$Q_k = 4.3 \text{ kN} / 1.2 \times 2 = 1.75 \text{ kN/m}$$

B2

END REACTION

$$G_k = 15.9 \text{ kN}$$

$$Q_k = 8.9 \text{ kN}$$

B3

END REACTION

$$G_k = 13.5 \text{ kN}$$

$$Q_k = 2.6 \text{ kN}$$

W4

WIND LOAD

$$W_k = 0.8 \text{ kN/m}^2 \times (3.9 + 3.5) / 2 = 2.96 \text{ kN/m}$$

P1

WIND LOAD

$$W_k = 0.8 \text{ kN/m}^2 \times 3.9 \times 2.4 \text{ m} / 2 = 3.74 \text{ kN/m}$$

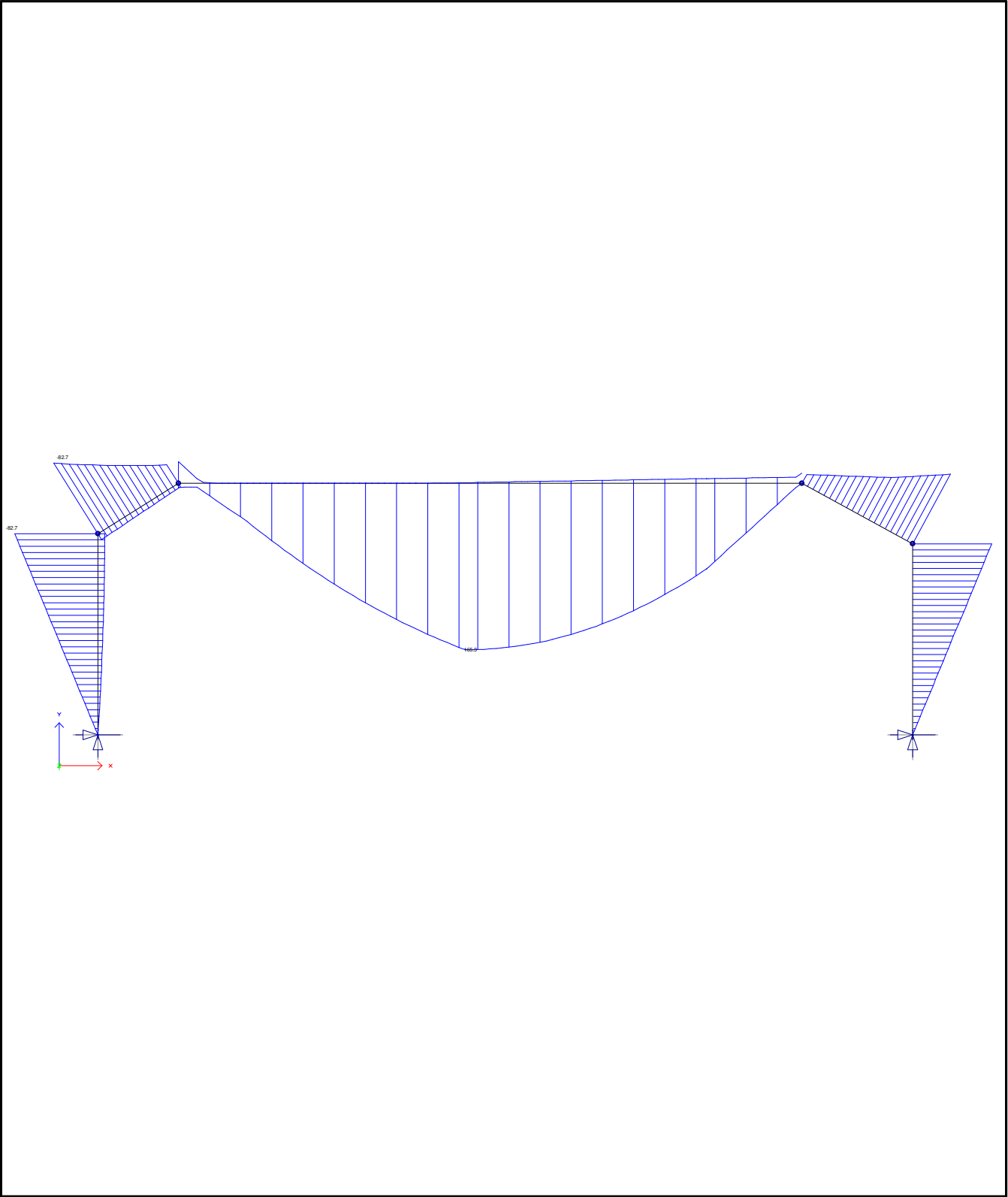
ADOPT

$$\left[\begin{array}{l} B1 : UC 254 \times 254 \times 89 \text{ S275} \\ C1 : UC 152 \times 152 \times 37 \text{ S275} \end{array} \right]$$

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		Checked By	Mm
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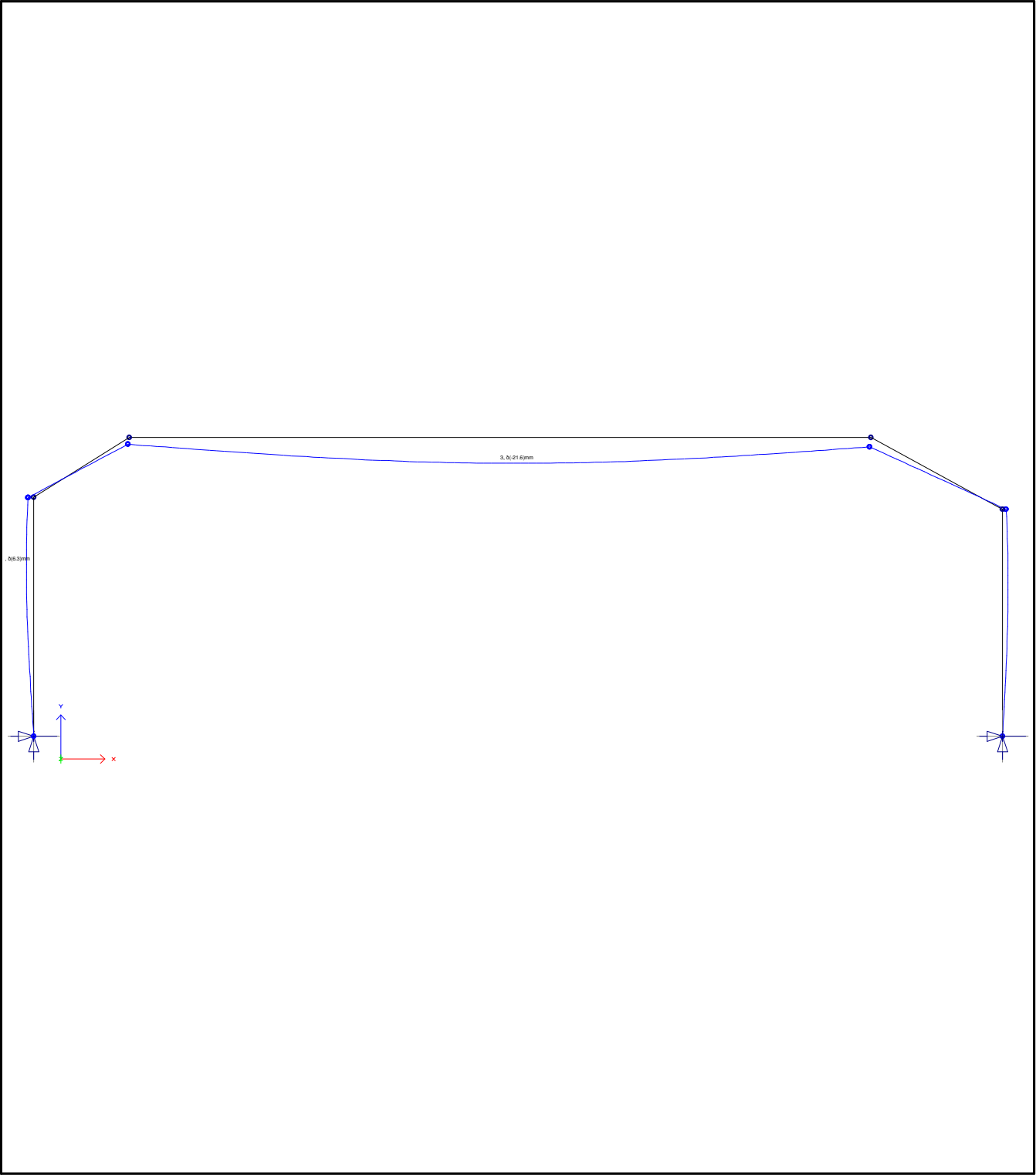
Moment Envelope



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Deflection diagram (SLS)

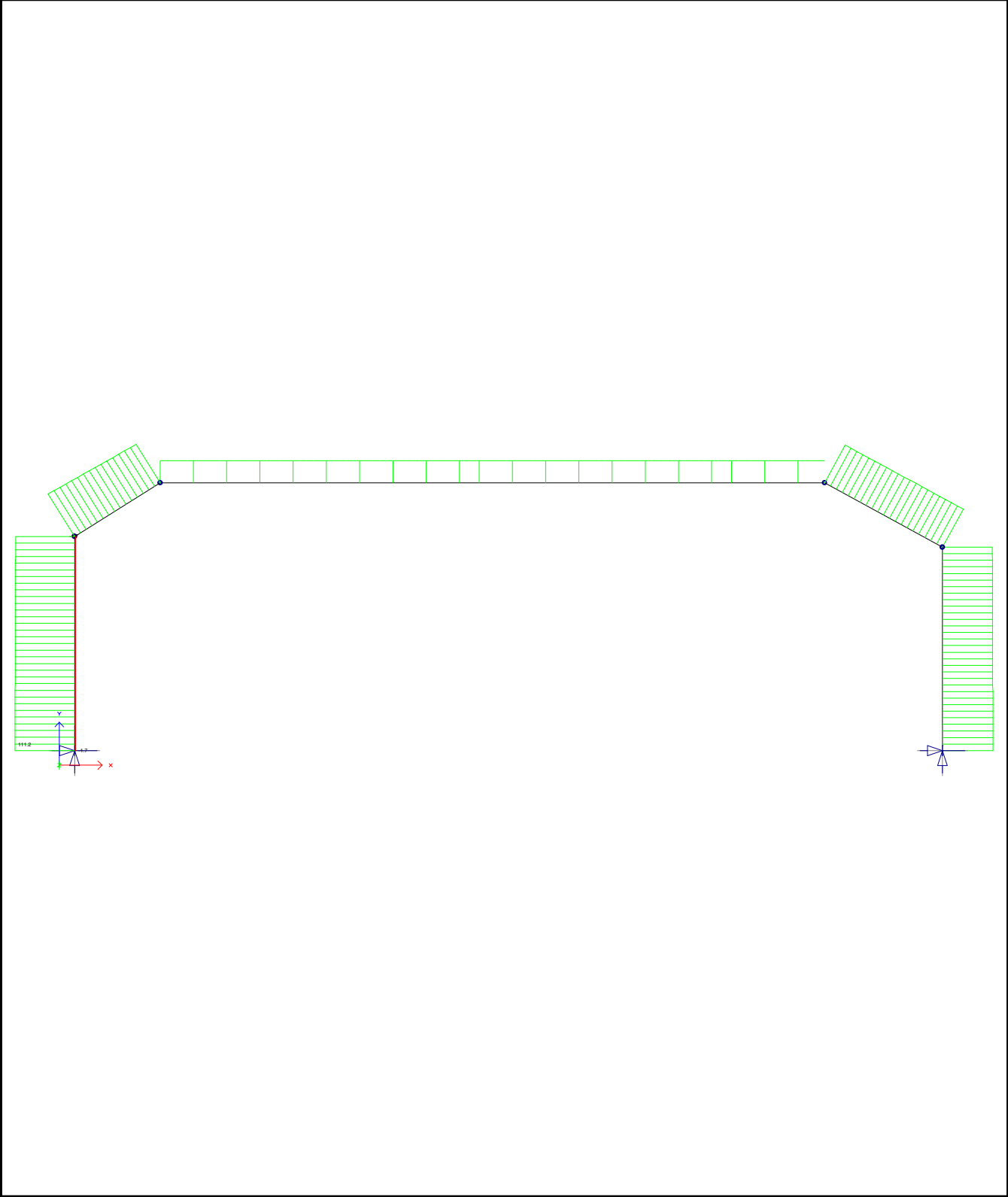


Max= 21.6mm

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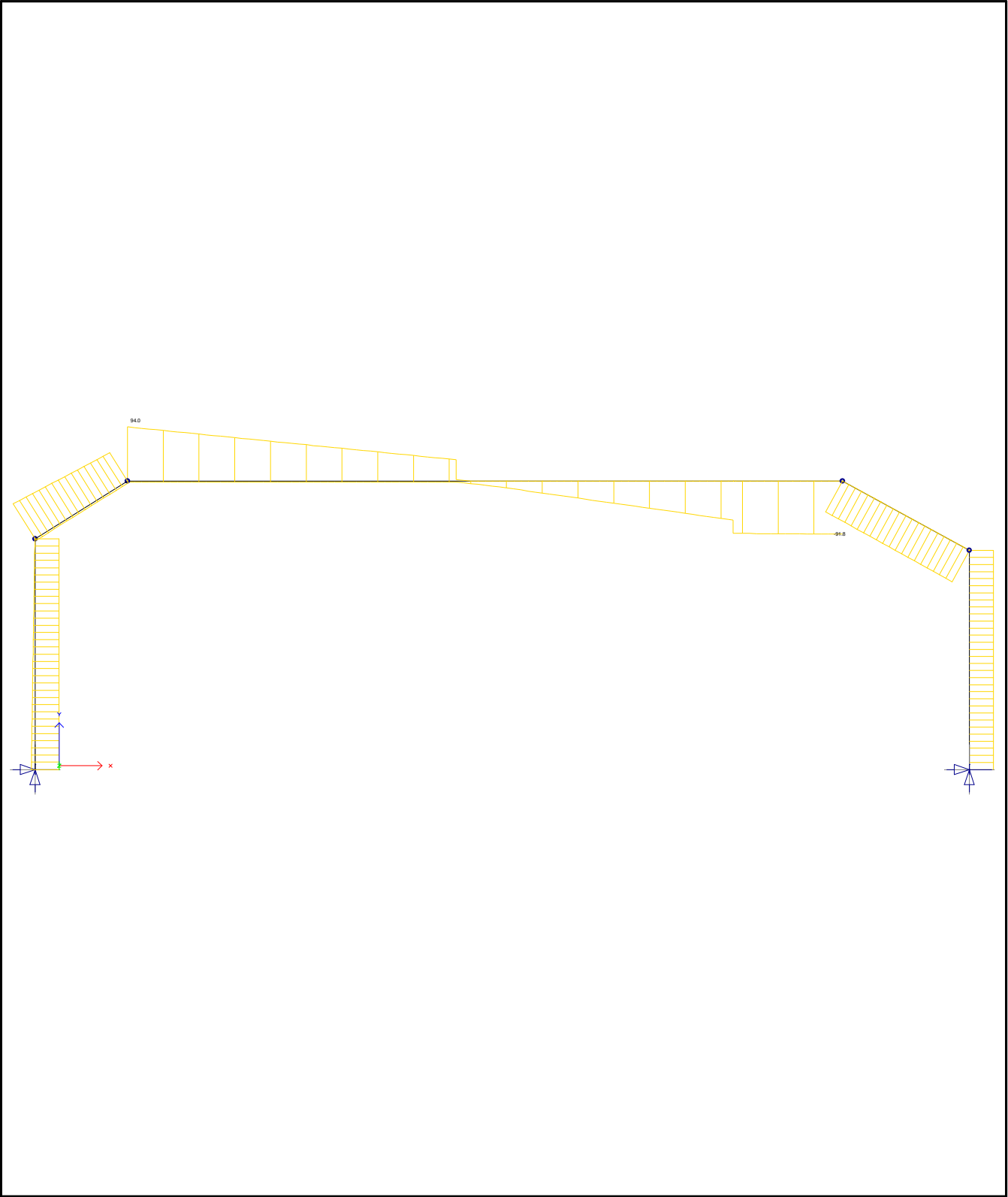
Axiial Diagram



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Shear Diagram



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Joint Loads

Load reference	Load type	Intensity (kN) & (m)	Direction	Category (type)
Loads on Joint 2				
p1	JL	3.740	Horiz. (+X)	Wind X (Imposed)

Member Loads

Load reference	Load type	Start pos'n (m)	Start intensity (kN) & (m)	End pos'n (m)	End intensity (kN) & (m)	Direction	Category
Loads on member 1 (Length 2.000m)							
w4	UL		2.960			Horiz. (+X)	Wind X
Loads on member 2 (Length 0.943m)							
w2	UL		1.500			Vertical (-Y)	Dead
w1	UL		10.940			Vertical (-Y)	Dead
w2	UL		0.830			Vertical (-Y)	Imposed
Loads on member 3 (Length 6.200m)							
w3	DL	2.850	4.040	2.400	4.040	Vertical (-Y)	Dead
w3	DL	2.850	1.750	2.400	1.750	Vertical (-Y)	Imposed
b2	PL	2.850	15.900			Vertical (-Y)	Dead
b2	PL	2.850	8.900			Vertical (-Y)	Imposed
b3	PL	5.250	13.500			Vertical (-Y)	Dead
b3	PL	5.250	2.600			Vertical (-Y)	Imposed
w1	DL	0.000	10.940	5.250	10.940	Vertical (-Y)	Dead
w2	DL	0.000	1.500	5.250	1.500	Vertical (-Y)	Dead
w2	DL	0.000	0.830	5.250	0.830	Vertical (-Y)	Imposed

Support Reactions for Combination GK

Analysis Type : Linear elastic

Joint reference	Support reactions (kN)			Support moments (kNm)		
	Px	Py	Pz	Mx	My	Mz
1	24.233	67.572	0.000	0.000	0.000	0.000
6	-24.233	55.503	0.000	0.000	0.000	0.000

Support Reactions for Combination QK

Analysis Type : Linear elastic

Joint reference	Support reactions (kN)			Support moments (kNm)		
	Px	Py	Pz	Mx	My	Mz
1	4.649	10.379	0.000	0.000	0.000	0.000
6	-4.649	10.343	0.000	0.000	0.000	0.000

Support Reactions for Combination WK

Analysis Type : Linear elastic

Joint reference	Support reactions (kN)			Support moments (kNm)		
	Px	Py	Pz	Mx	My	Mz
1	-6.413	-1.654	0.000	0.000	0.000	0.000

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		Job Ref	14 st clements
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		Checked By	Mm
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A3D MAX V4.55(Build 924) Copyright © Computer and Design Services Ltd. (Analysis No. 2)

Support Reactions for Combination WK (continued)

Analysis Type : Linear elastic

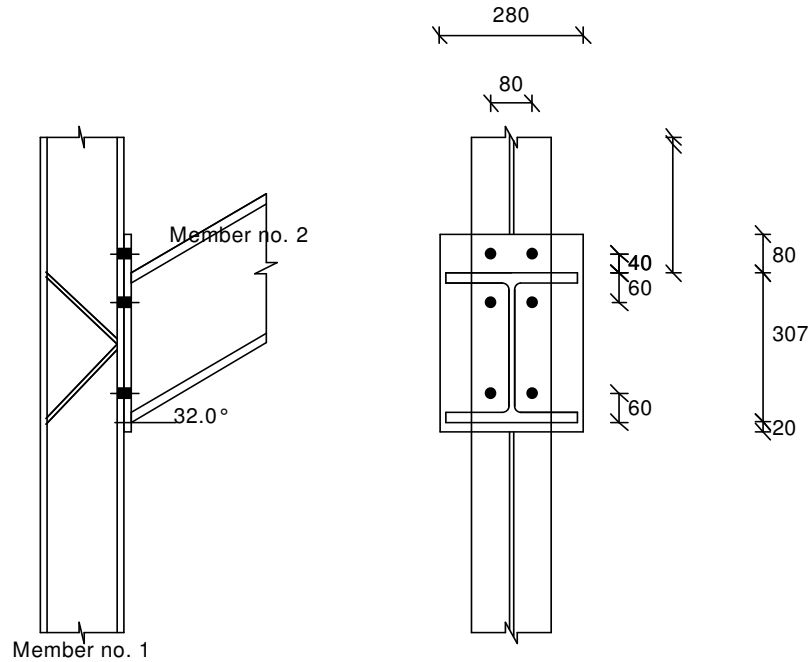
Joint reference	Support reactions (kN)			Support moments (kNm)		
	Px	Py	Pz	Mx	My	Mz
6	-3.247	1.654	0.000	0.000	0.000	0.000

Steelwork Design Results

Desig object ref	Memb ref	Design parameter	Analysis section	Design section	Utilisation factors				Design status
					Local capacity	Lateral buckling	Torsion	Deflection	
Group Reference - column									
2	1	Column	152x152 UKC37		0.990	0.974	n / a	0.695	Passed
3	5	Column	152x152 UKC37		0.936	0.925	n / a	0.859	Passed
Group Reference - Beam									
6	2	Beam	254x254 UKC89		0.256	0.255	n / a	0.202	Passed
7	3	Beam	254x254 UKC89		0.510	0.613	n / a	0.457	Passed
8	4	Beam	254x254 UKC89		0.243	0.242	n / a	0.269	Passed

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CADS SW Moment Connections V1.84 (Build 476) Copyright © Computer and Design Services Ltd.



Support member 152x152 UKC37 grade S275

Connected member	254x254 UKC89	grade S275
End plate	407x280x15	grade S275
	Upper Lower	
Flange welds	8.0 8.0	
Web welds	8.0 8.0	
Bolts	6 No. M20 grade 8.8	
Diagonal shear stiffener:	K - type	60x8
	8.0 FW to flange	8.0 FW to web

Results summary

Effect	Units	Load comb.	Required value	Resistance value	Utilisation ratio	Status report
Right connection						Passed
Shear :	kN	ULS1	110.20	502.73	0.219	Passed
Axial compression :	kN	ULS1	41.35	699.17	0.059	Passed
Axial tension :	kN	-	-	-	-	-
Hogging moment :	kNm	ULS1	82.73	97.19	0.851	Passed
Sagging moment :	kNm	-	-	-	-	-

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FOUNDATION OVERVIEW

- NO SI HAS BEEN CARRIED OUT
- ASSUMED $ACBC = 100 \text{ kN/m}^2$
- ASSUMED GROUND PROFILE = GRANULAR
(TO BE CONFIRMED ONSITE BY BCO)

STRIP FOOTING

ADOPT [BY INSPECTION - 600x300mm MASS CONCRETE STRIP FOOTING]

GROUND BEARING SLAB

ADOPT [BY INSPECTION - 125mm RC GBS W/A252 MESH IN TOP]

PAD TO C1

LOADING

$$\begin{array}{lcl}
 \downarrow V & G_k = 67.6 \text{ kN} & \rightarrow H & G_k = 24.23 \text{ kN} \\
 & Q_k = 10.4 \text{ kN} & & Q_k = 4.65 \text{ kN} \\
 & W_k = -1.65 \text{ kN} & & W_k = -6.41 \text{ kN}
 \end{array}
 \left. \vphantom{\begin{array}{lcl}} \right\} \text{LHS (NEXT TO PARTY WALL)}$$

$$\begin{array}{lcl}
 \downarrow V & G_k = 55.5 \text{ kN} & \rightarrow H & G_k = 24.23 \text{ kN} \\
 & Q_k = 10.34 \text{ kN} & & Q_k = 4.65 \text{ kN} \\
 & W_k = 1.65 \text{ kN} & & W_k = 3.25 \text{ kN}
 \end{array}
 \left. \vphantom{\begin{array}{lcl}} \right\} \text{RHS}$$

ADOPT. LHS - 1200x(600x300mm - RC PAD FOOTING W/A393 MESH IN BOTTOM

RHS - 1200x1200x300mm " " " " " "

PAD TO P1

LOADING

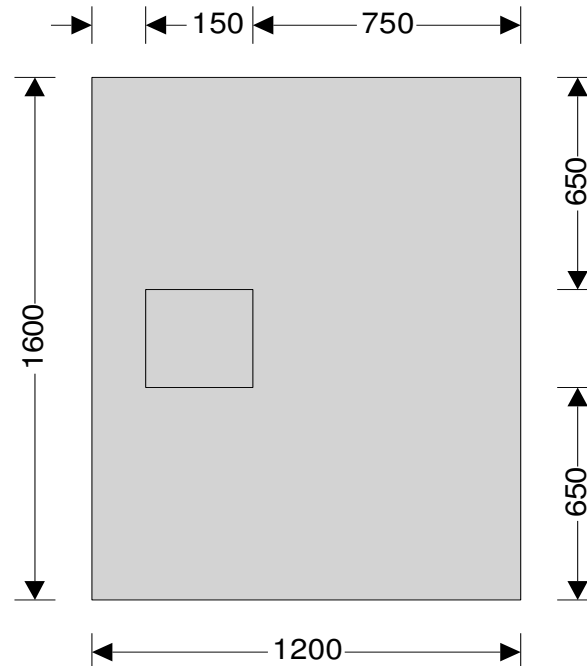
$$\begin{array}{lcl}
 \downarrow V & G_k = 3.8 \text{ kN} & \rightarrow H & G_k = 0.185 \text{ kN} \\
 & Q_k = 0.53 \text{ kN} & & Q_k = 0.06 \text{ kN} \\
 & W_k = 0.46 \text{ kN} & & W_k = 0.74 \text{ kN}
 \end{array}$$

ADOPT [600x600x300mm - RC PAD FOOTING W/A393 MESH IN BOTTOM]

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PAD FOOTING ANALYSIS AND DESIGN (BS8110-1:1997)

Tedds calculation version 2.0.07



Pad footing details

Length of pad footing	$L = 1200$ mm	Width of pad footing	$B = 1600$ mm
Depth of pad footing	$h = 300$ mm	Depth of soil over pad footing	$h_{\text{soil}} = 200$ mm
Density of concrete	$\rho_{\text{conc}} = 24.0$ kN/m ³		

Column details

Column base length	$l_A = 300$ mm	Column base width	$b_A = 300$ mm
Column eccentricity in x	$e_{Px} = -300$ mm	Column eccentricity in y	$e_{Py} = 0$ mm

Soil details

Depth of soil over pad footing	$h_{\text{soil}} = 200$ mm	Density of soil	$\rho_{\text{soil}} = 20.0$ kN/m ³
Allowable bearing pressure	$P_{\text{bearing}} = 100$ kN/m ²		

Axial loading on column

Dead axial load	$P_{GA} = 67.6$ kN	Imposed axial load	$P_{QA} = 10.4$ kN
Wind axial load	$P_{WA} = -1.6$ kN	Total axial load	$P_A = 76.4$ kN

Foundation loads

Dead surcharge load	$F_{G\text{sur}} = 1.500$ kN/m ²	Imposed surcharge load	$F_{Q\text{sur}} = 1.500$ kN/m ²
Pad footing self weight	$F_{\text{swt}} = 7.200$ kN/m ²		
Soil self weight	$F_{\text{soil}} = 4.000$ kN/m ²	Total foundation load	$F = 27.3$ kN

Horizontal loading on pad footing

Dead load in x direction	$H_{GxA} = 24.2$ kN	Dead load in y direction	$H_{GyA} = 0.0$ kN
Imposed load in x direction	$H_{QxA} = 4.7$ kN	Imposed load in y direction	$H_{QyA} = 0.0$ kN
Wind load in x direction	$H_{WxA} = -6.4$ kN	Wind load in y direction	$H_{WyA} = 0.0$ kN
Total load in x direction	$H_{xA} = 22.5$ kN	Total load in y direction	$H_{yA} = 0.0$ kN

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Check stability against sliding

Passive pressure coefficient $K_p = 2.464$

Stability against sliding in x direction

Total sliding resistance in x dir $H_{xres} = 39.9$ kN

PASS - Resistance to sliding is greater than horizontal load in x direction

Check stability against overturning in x direction

Total overturning moment $M_{xOT} = 6.741$ kNm

Total restoring moment $M_{xres} = 73.985$ kNm

PASS - Restoring moment is greater than overturning moment in x direction

Calculate pad base reaction

Total base reaction $T = 103.6$ kN

Base reaction eccentricity in x $e_{Tx} = -156$ mm

Base reaction eccentricity in y $e_{Ty} = 0$ mm

Base reaction acts within middle third of base

Calculate pad base pressures

$q_1 = 96.059$ kN/m²

$q_2 = 96.059$ kN/m²

$q_3 = 11.872$ kN/m²

$q_4 = 11.872$ kN/m²

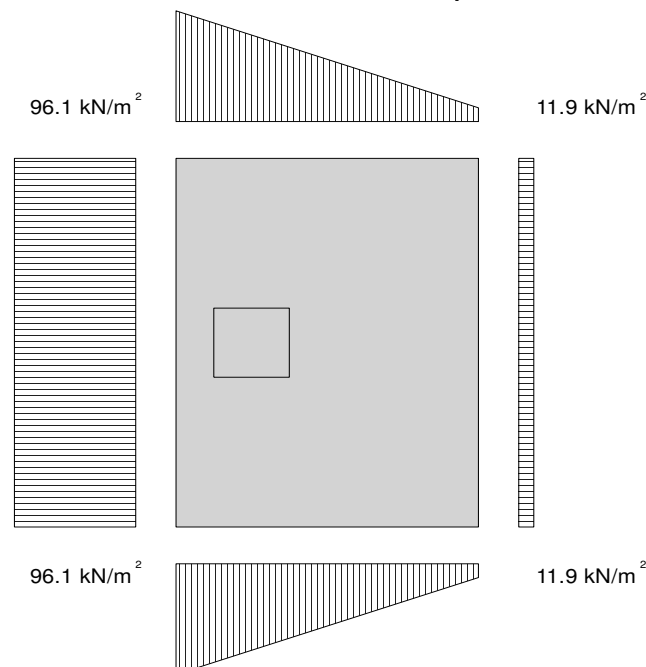
Minimum base pressure

$q_{min} = 11.872$ kN/m²

Maximum base pressure

$q_{max} = 96.059$ kN/m²

PASS - Maximum base pressure is less than allowable bearing pressure



Partial safety factors for loads

Dead loads $\gamma_{IG} = 1.40$

Imposed loads

$\gamma_{IQ} = 1.60$

Wind loads $\gamma_{IW} = 0.00$

Ultimate axial loading on column

Ultimate axial load on column $P_{uA} = 111.3$ kN

Ultimate foundation loads

Ultimate foundation load $F_u = 38.7$ kN

Ultimate horizontal loading on column

Ult. horizontal load in x dir $H_{xUA} = 41.4$ kN

Ult. horizontal load in y dir

$H_{yUA} = 0.0$ kN

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Ultimate moment on column

Ult.moment on column in x dir $M_{xuA} = 0.000$ kNm

Ult.moment on column in y dir $M_{yuA} = 0.000$ kNm

Ultimate pad base reaction

Ultimate base reaction $T_u = 150.0$ kN

Ecc.of ult.base reaction in x $e_{Txu} = -140$ mm

Ecc.of ult.base reaction in y $e_{Tyu} = 0$ mm

Calculate ultimate pad base pressures

$q_{1u} = 132.762$ kN/m²

$q_{2u} = 132.762$ kN/m²

$q_{3u} = 23.515$ kN/m²

$q_{4u} = 23.515$ kN/m²

Minimum ult.base pressure $q_{minu} = 23.515$ kN/m²

Maximum ult.base pressure

$q_{maxu} = 132.762$ kN/m²

Library item: Ultimate pressures summary

Ultimate moment in x dir $M_x = 19.859$ kNm

Ultimate moment in y dir $M_y = 22.256$ kNm

Material details

Char.strength of concrete $f_{cu} = 30$ N/mm²

Char.strength of reinf $f_y = 500$ N/mm²

Char.strength of shear reinf $f_{yv} = 500$ N/mm²

Nom.cover to reinforcement $C_{nom} = 30$ mm

Moment design in x direction

Tens.reinforcement diameter $\phi_{xB} = 10$ mm

Tens.reinforcement depth $d_x = 255$ mm

Design formula for rectangular beams (cl 3.4.4.4)

$K_x = 0.006$ $K_x' = 0.156$

$K_x < K_x'$ compression reinforcement is not required

Tens.reinforcement required $A_{s_x_req} = 188$ mm²

Minimum tens.reinforcement $A_{s_x_min} = 624$ mm²

Tens.reinforcement provided **8 No. 10 dia. bars btm**

$A_{s_xB_prov} = 628$ mm²

PASS - Tension reinforcement provided exceeds tension reinforcement required

Moment design in y direction

Tens.reinforcement diameter $\phi_{yB} = 10$ mm

Tens.reinforcement depth $d_y = 265$ mm

Design formula for rectangular beams (cl 3.4.4.4)

$K_y = 0.009$ $K_y' = 0.156$

$K_y < K_y'$ compression reinforcement is not required

Tens.reinforcement required $A_{s_y_req} = 203$ mm²

Minimum tens.reinforcement $A_{s_y_min} = 468$ mm²

Tens.reinforcement provided **7 No. 10 dia. bars btm**

$A_{s_yB_prov} = 550$ mm²

PASS - Tension reinforcement provided exceeds tension reinforcement required

Calculate ultimate shear force at d from top face of column

Ult.pressure for shear $q_{su} = 78.138$ kN/m²

Area loaded for shear $A_s = 0.462$ m²

Ult.shear force $V_{su} = 26.777$ kN

Shear stresses at d from top face of column (cl 3.5.5.2)

Design shear stress $v_{su} = 0.084$ N/mm²

Design concrete shear stress $v_c = 0.409$ N/mm²

Allowable design shear stress $v_{max} = 4.382$ N/mm²

PASS - $v_{su} < v_c$ - No shear reinforcement required

Calculate ultimate punching shear force at face of column

Ult.press.for punching shear $q_{puA} = 105.450$ kN/m²

Avg.effective reinf.depth $d = 260$ mm

Area loaded $A_{pA} = 0.090$ m²

Length of shear perimeter $U_{pA} = 1200$ mm

Ult.punching shear force $V_{puA} = 103.606$ kN

Eff.punching shear force $V_{puAeff} = 103.606$ kN

Punching shear stresses at face of column (cl 3.7.7.2)

Design shear stress $v_{puA} = 0.332$ N/mm²

PASS - Design shear stress is less than allowable design shear stress

Calculate ultimate punching shear force at perimeter of 1.5 d from face of column

Ult.press.for punching shear $q_{puA1.5d} = 94.525$ kN/m²

Avg.effective reinf.depth $d = 260$ mm

Area loaded $A_{pA1.5d} = 0.907$ m²

Length of shear perimeter $U_{pA1.5d} = 2760$ mm

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Ult.punching shear force

$V_{puA1.5d} = 43.834$ kN

Eff.punching shear force

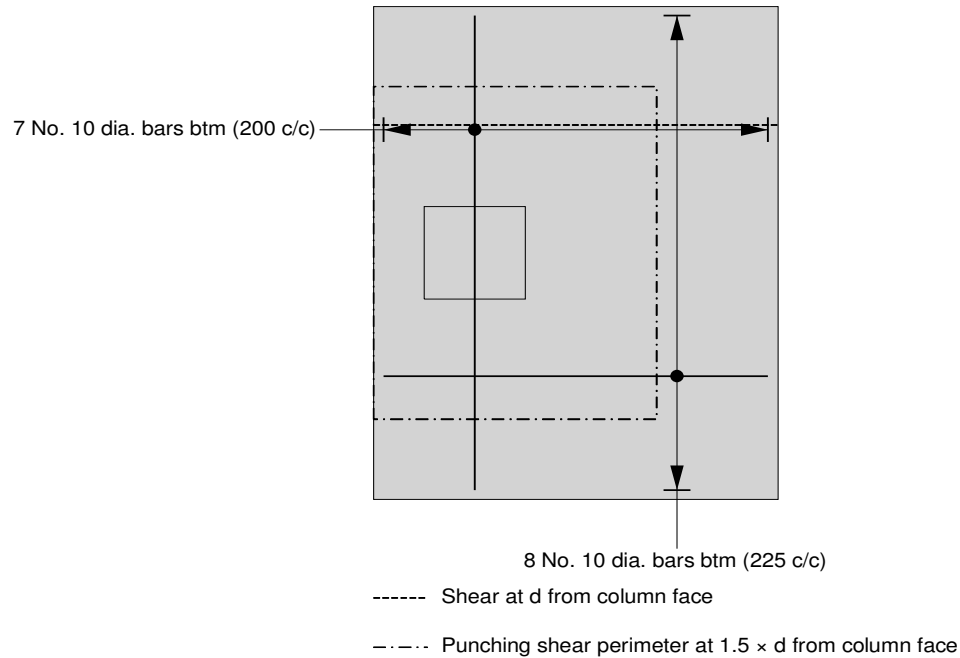
$V_{puA1.5deff} = 54.792$ kN

Punching shear stresses at perimeter of 1.5 d from face of column (cl 3.7.7.2)

Design shear stress

$V_{puA1.5d} = 0.076$ N/mm²

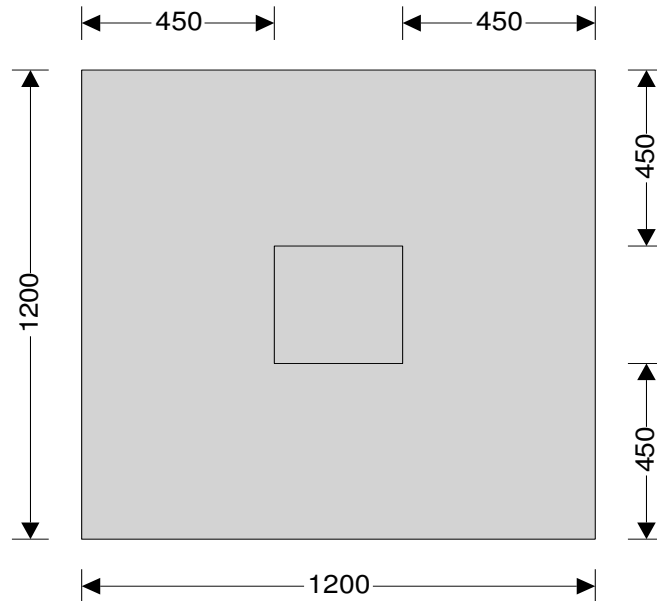
PASS - $v_{puA1.5d} < v_c$ - No shear reinforcement required



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PAD FOOTING ANALYSIS AND DESIGN (BS8110-1:1997)

Tedds calculation version 2.0.07



Pad footing details

Length of pad footing	$L = 1200 \text{ mm}$	Width of pad footing	$B = 1200 \text{ mm}$
Depth of pad footing	$h = 300 \text{ mm}$	Depth of soil over pad footing	$h_{\text{soil}} = 200 \text{ mm}$
Density of concrete	$\rho_{\text{conc}} = 24.0 \text{ kN/m}^3$		

Column details

Column base length	$l_A = 300 \text{ mm}$	Column base width	$b_A = 300 \text{ mm}$
Column eccentricity in x	$e_{Px} = 0 \text{ mm}$	Column eccentricity in y	$e_{Py} = 0 \text{ mm}$

Soil details

Depth of soil over pad footing	$h_{\text{soil}} = 200 \text{ mm}$	Density of soil	$\rho_{\text{soil}} = 20.0 \text{ kN/m}^3$
Allowable bearing pressure	$P_{\text{bearing}} = 100 \text{ kN/m}^2$		

Axial loading on column

Dead axial load	$P_{GA} = 55.5 \text{ kN}$	Imposed axial load	$P_{QA} = 10.3 \text{ kN}$
Wind axial load	$P_{WA} = 1.7 \text{ kN}$	Total axial load	$P_A = 67.5 \text{ kN}$

Foundation loads

Dead surcharge load	$F_{G\text{sur}} = 1.500 \text{ kN/m}^2$	Imposed surcharge load	$F_{Q\text{sur}} = 1.500 \text{ kN/m}^2$
Pad footing self weight	$F_{\text{swt}} = 7.200 \text{ kN/m}^2$		
Soil self weight	$F_{\text{soil}} = 4.000 \text{ kN/m}^2$	Total foundation load	$F = 20.4 \text{ kN}$

Horizontal loading on pad footing

Dead load in x direction	$H_{GxA} = 24.2 \text{ kN}$	Dead load in y direction	$H_{GyA} = 0.0 \text{ kN}$
Imposed load in x direction	$H_{QxA} = 4.7 \text{ kN}$	Imposed load in y direction	$H_{QyA} = 0.0 \text{ kN}$
Wind load in x direction	$H_{WxA} = 3.3 \text{ kN}$	Wind load in y direction	$H_{WyA} = 0.0 \text{ kN}$
Total load in x direction	$H_{xA} = 32.1 \text{ kN}$	Total load in y direction	$H_{yA} = 0.0 \text{ kN}$

Check stability against sliding

Passive pressure coefficient	$K_p = 2.464$
------------------------------	---------------

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Stability against sliding in x direction

Total sliding resistance in x dir $H_{xres} = 32.0$ kN

FAIL - Resistance to sliding is less than horizontal load in x direction

Check stability against overturning in x direction

Total overturning moment $M_{xOT} = 9.639$ kNm

Total restoring moment $M_{xres} = 44.273$ kNm

PASS - Restoring moment is greater than overturning moment in x direction

Calculate pad base reaction

Total base reaction $T = 87.9$ kN

Base reaction eccentricity in x $e_{Tx} = 110$ mm

Base reaction eccentricity in y $e_{Ty} = 0$ mm

Base reaction acts within middle third of base

Calculate pad base pressures

$q_1 = 27.599$ kN/m²

$q_2 = 27.599$ kN/m²

$q_3 = 94.537$ kN/m²

$q_4 = 94.537$ kN/m²

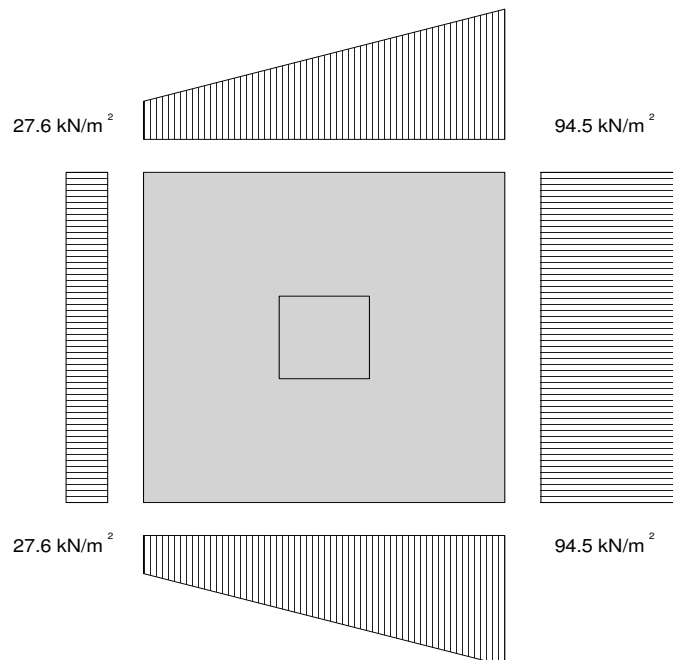
Minimum base pressure

$q_{min} = 27.599$ kN/m²

Maximum base pressure

$q_{max} = 94.537$ kN/m²

PASS - Maximum base pressure is less than allowable bearing pressure



Partial safety factors for loads

Dead loads $\gamma_{fG} = 1.20$

Imposed loads $\gamma_{fQ} = 1.20$

Wind loads $\gamma_{fW} = 1.20$

Ultimate axial loading on column

Ultimate axial load on column $P_{uA} = 81.0$ kN

Ultimate foundation loads

Ultimate foundation load $F_u = 24.5$ kN

Ultimate horizontal loading on column

Ult. horizontal load in x dir $H_{xuA} = 38.6$ kN

Ult. horizontal load in y dir $H_{yuA} = 0.0$ kN

Ultimate moment on column

Ult. moment on column in x dir $M_{xuA} = 0.000$ kNm

Ult. moment on column in y dir $M_{yuA} = 0.000$ kNm

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Ultimate pad base reaction

Ultimate base reaction $T_u = 105.5$ kN

Ecc.of ult.base reaction in x $e_{Txu} = 110$ mm

Ecc.of ult.base reaction in y $e_{Tyu} = 0$ mm

Calculate ultimate pad base pressures

$q_{1u} = 33.119$ kN/m²

$q_{2u} = 33.119$ kN/m²

$q_{3u} = 113.444$ kN/m²

$q_{4u} = 113.444$ kN/m²

Minimum ult.base pressure $q_{minu} = 33.119$ kN/m²

Maximum ult.base pressure

$q_{maxu} = 113.444$ kN/m²

Library item: Ultimate pressures summary

Ultimate moment in x dir $M_x = 17.932$ kNm

Ultimate moment in y dir

$M_y = 12.148$ kNm

Material details

Char.strength of concrete $f_{cu} = 30$ N/mm²

Char.strength of reinf $f_y = 500$ N/mm²

Char.strength of shear reinf $f_{yv} = 500$ N/mm²

Nom.cover to reinforcement $C_{nom} = 40$ mm

Moment design in x direction

Tens.reinforcement diameter $\phi_{xB} = 10$ mm

Tens.reinforcement depth $d_x = 255$ mm

Design formula for rectangular beams (cl 3.4.4.4)

$K_x = 0.008$

$K_x' = 0.156$

$K_x < K_x'$ compression reinforcement is not required

Tens.reinforcement required $A_{s_x_req} = 170$ mm²

Minimum tens.reinforcement $A_{s_x_min} = 468$ mm²

Tens.reinforcement provided **6 No. 10 dia. bars btm**

$A_{s_xB_prov} = 471$ mm²

PASS - Tension reinforcement provided exceeds tension reinforcement required

Moment design in y direction

Tens.reinforcement diameter $\phi_{yB} = 10$ mm

Tens.reinforcement depth $d_y = 245$ mm

Design formula for rectangular beams (cl 3.4.4.4)

$K_y = 0.006$

$K_y' = 0.156$

$K_y < K_y'$ compression reinforcement is not required

Tens.reinforcement required $A_{s_y_req} = 120$ mm²

Minimum tens.reinforcement $A_{s_y_min} = 468$ mm²

Tens.reinforcement provided **6 No. 10 dia. bars btm**

$A_{s_yB_prov} = 471$ mm²

PASS - Tension reinforcement provided exceeds tension reinforcement required

Calculate ultimate shear force at d from right face of column

Ult.pressure for shear $q_{su} = 106.918$ kN/m²

Area loaded for shear $A_s = 0.234$ m²

Ult.shear force

$V_{su} = 21.031$ kN

Shear stresses at d from right face of column (cl 3.5.5.2)

Design shear stress $v_{su} = 0.069$ N/mm²

Design concrete shear stress $v_c = 0.408$ N/mm²

Allowable design shear stress $v_{max} = 4.382$ N/mm²

PASS - $v_{su} < v_c$ - No shear reinforcement required

Calculate ultimate punching shear force at face of column

Ult.press.for punching shear $q_{puA} = 73.282$ kN/m²

Area loaded $A_{pA} = 0.090$ m²

Avg.effective reinf.depth

$d = 250$ mm

Ult.punching shear force $V_{puA} = 75.926$ kN

Length of shear perimeter

$u_{pA} = 1200$ mm

Eff.punching shear force

$V_{puAeff} = 75.926$ kN

Punching shear stresses at face of column (cl 3.7.7.2)

Design shear stress $v_{puA} = 0.253$ N/mm²

PASS - Design shear stress is less than allowable design shear stress

Calculate ultimate punching shear force at perimeter of 1.5 d from face of column

Ult.press.for punching shear $q_{puA1.5d} = 73.282$ kN/m²

Area loaded $A_{pA1.5d} = 1.260$ m²

Avg.effective reinf.depth

$d = 250$ mm

Ult.punching shear force $V_{puA1.5d} = 10.124$ kN

Length of shear perimeter

$u_{pA1.5d} = 2400$ mm

Eff.punching shear force

$V_{puA1.5deff} = 12.654$ kN

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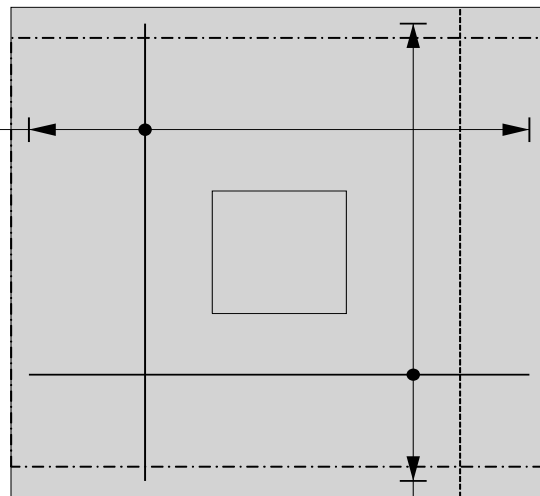
Punching shear stresses at perimeter of 1.5 d from face of column (cl 3.7.7.2)

Design shear stress

$$V_{puA1.5d} = 0.021 \text{ N/mm}^2$$

PASS - $v_{puA1.5d} < v_c$ - No shear reinforcement required

6 No. 10 dia. bars btm (225 c/c)



6 No. 10 dia. bars btm (225 c/c)

----- Shear at d from column face

- . - . - Punching shear perimeter at $1.5 \times d$ from column face

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PADSTONES

	ELEMENT	REACTION	Allow	WIDTH	LENGTH	PADSTONE
PS01	B2	36.5kN	1.54 N/mm ²	100mm	234mm	330x100x245mm
PS02	B3	24.4kN	11	200mm	79mm	215x100x215
PS03	L3			100mm		215x100x150mm

MOMENT CONNECTION FOR P1 & L4 RAFTER

$$\text{MAX MOMENT} = 2.3 \text{ kNm.}$$

$$\text{LEVER ARM} = (150 + 20 + 20) = 190 \text{ mm}$$

$$\text{TENSION (REQUIRED)} = \frac{2.3}{0.19} = 12.1 \text{ kN}$$

$$\text{CAPACITY OF M16 (GRADE 8.8) / UNIT} = 35.2 \text{ kN.}$$

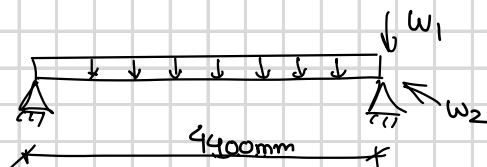
ADOPT

[CONNECT L4 RAFTER W/ POST P1 USING 4 No. M16
 GRADE 8.8 BOLTS.
 - EDGE DISTANCE = $1.25(\phi) = 20 \text{ mm (min)}$
 - DISTANCE B/W BOLTS = $2.5(\phi) = 40 \text{ mm (min)}$

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BIAXIAL BENDING CHECK FOR TIE 1

GEOMETRY



LOADING
W1

VERTICAL LOAD

$$\text{GLAZING } G_k = 0.6 \times 1.2 / 2 \text{ m} = 0.36 \text{ kN/m}$$

W2

HORIZONTAL LOAD

AREA OF WIND LOAD APPLYING ON TIE FROM DOOR.

$$= \left(\frac{4.4 \times 1.7}{2} - \frac{1}{2} (0.85 \times 0.85 \times 2) \right)$$

$$= 3 \text{ m}^2$$

AREA OF LOAD APPLYING ON TIE FROM GLAZING.

$$= \frac{1}{2} (4.4 \times 1.2) = 2.64 \text{ m}^2$$

$$\text{TOTAL LOAD AREA} = (2.64 + 3) \text{ m}^2 = 5.64 \text{ m}^2$$

$$\text{TOTAL WIND LOAD ON TIE } - W_k = (0.8 \text{ kN/m}^2 \times 5.64) / 4.4 = 1.02 \text{ kN/m}$$

BIAXIAL BENDING - SECTION S45 100x100x8 S355

$$f_y = 355 \text{ N/mm}^2$$

$$\text{MULT} = f_y W_{pl} = (355 \times 98.2 \times 10^3)$$

$$\text{MULT} = 35 \text{ kNm}$$

$$M_{app} (x-x') = \frac{WL^2}{8} = \frac{0.36 \times 4.4^2}{8} = 0.87 \text{ kNm}$$

$$M_{app} (y-y') = \frac{W_k L^2}{8} = \frac{1.02 \times 4.4^2}{8} = 2.47 \text{ kNm}$$

$$\frac{M_{app}(x-x')}{\text{MULT}} + \frac{M_{app}(y-y')}{\text{MULT}} < 1.0$$

$$\frac{0.87}{35} + \frac{2.47}{35} = 0.02 + 0.07 = 0.09 < 1.0 \quad \boxed{\text{OK}}$$

RESULT

[TIE IS OK UNDER BIAXIAL BENDING]