Addendum to BIA Submission

in connection with proposed development at

No. 52 Eton Avenue
Camden
London
NW3 3HN

for

Natalie Matalon & Izzy Tepekoylu

LBH4564a Ver. 1.0

May 2019

LBH WEMBLEY ENGINEERING

Document Control

Version	Date	Comment		Authorised
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1.0	16th May 2019			

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Contents

Foreword-Guidance Notes

GENERAL

This report has been prepared for a specific client and to meet a specific brief. The preparation of this report may have been affected by limitations of scope, resources or time scale required by the client. Should any part of this report be relied on by a third party, that party does so wholly at its own risk and LBH Wembley Engineering disclaims any liability to such parties.

The observations and conclusions described in this report are based solely upon the agreed scope of work. LBH Wembley Engineering has not performed any observations, investigations, studies or testing not specifically set out in the agreed scope of work and cannot accept any liability for the existence of any condition, the discovery of which would require performance of services beyond the agreed scope of work.

VALIDITY

Should the purpose for which the report is used, or the proposed use of the site change, this report may no longer be valid and any further use of or reliance upon the report in those circumstances shall be at the client's sole and own risk. The passage of time may result in changes in site conditions, regulatory or other legal provisions, technology or economic conditions which could render the report inaccurate or unreliable. The information and conclusions contained in this report should therefore not be relied upon in the future and any such reliance on the report in the future shall again be at the client's own and sole risk.

THIRD PARTY INFORMATION

The report may present an opinion based upon information received from third parties. However, no liability can be accepted for any inaccuracies or omissions in that information.

1. Introduction

1.1 Background

It is proposed to construct a basement beneath the entire footprint of an existing three storey property at No. 52 Eton Avenue that will extend to approximately 3m depth. The basement will also laterally extend to the front and side of the existing building.

A Basement Impact Assessment was prepared in January 2019 to support a full planning application to the London Borough of Camden.

As a result of site ownership issues, access to the site was not possible in January and hence the assessment was submitted for initial review by the council's retained BIA consultant Campbell Reith Hill Structural Engineers (CRH) without the benefit of any site specific ground investigation.

1.2 Policy

According to their (own) terms of reference[1] for their services to Camden, where Campbell Reith Hill (CRH) "*have concerns about the technical content or considerations of the audit material which should be addressed prior to planning permission being granted, then <u>they will advise the case officer and the applicant's consultants of their concerns and any suggested remedy</u>. The applicant will be required to submit details to address the concerns to Campbell Reith's satisfaction <u>prior to completion of the Initial Audit Report</u>.*"

In this case it seems that CRH have proceeded prematurely to the completion and issue of an initial audit report [2]

Although the audit has been based upon the specification provided by the CGHHS[3] , the terms of reference for this audit have exceeded the originally conceived non-technical evaluation of the information supplied and include elements of technical evaluation rather than a simple audit.

This technical evaluation is classed under the guidance as an independent technical verification rather than a non-technical audit of the information supplied, and is applicable where CRH "Category B" conditions apply.

> *"Category B:*
>
> *Residential single basement or commercial development with single or double basement where the Screening Stage of the Basement Impact Assessment identifies matters of concern which need further investigation: Submitted BIA anticipates potential impact:*
>
> - *to a listed building;*
> - *on land stability;*

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[1] Campbell Reith Hill: BIA Terms of Reference & Audit Process14th May 2015.
[2] Campbell Reith Hill: BIA Audit Ref: 12985-48 Rev: D1 dated 29th April 2019.
[3] London Borough of Camden: Camden geological, hydrogeological and hydrological study: Guidance for subterranean Development Issue01 | Ove Arup & Partners Limited (November 2010)

- *on groundwater flow;*
- *on potential for surface water flooding ;*
- *on underground tunnels or infrastructure; and*
- *cumulative impact on ground stability and the water environment. "*

The process for this independent verification that Camden requires where a scheme requires applicants to proceed beyond the Screening stage of the Basement Impact Assessment (i.e. where a matter of concern has been identified which requires the preparation of a full Basement Impact Assessment) is described by Section 4.36 of the guidance[4] and it may be reasonably expected that this independent verification demands the services of a basement expert who should as a minimum match the detailed competency levels that have been laid down by Camden for BIA work.

[4] London Borough of Camden: Camden Planning Guidance: Basements (March 2018)

2. Response to CRH initial Audit Report

A detailed response to the CRH audit checklist comments is appended to this report. For the purposes of enabling CRH to complete a positive report it will be necessary to "close out" the seven CRH audit queries which are listed as follows:

1. Drawings with clear indication of the dimensions of the existing and proposed structures to be presented.
2. Outline design calculations and drawings for the proposed temporary and permanent works.
3. A construction works programme should be provided for the main works including any proposed works for the western perimeter wall.
4. Contradictory information presented in the BIA about protection of trees
5. Consultation with drainage/sewer utility owners should be confirmed
6. A ground investigation should be carried out to establish ground conditions, ground water level, soil parameters, bearing capacity of founding stratum, the details regarding foundations of neighbouring structures and the feasibility of underpinning through backfilled demolition material.
7. The ground movement assessment should be revisited once the information from the ground investigation is available.

2.1 Drawings

See attached Structural Engineer's Drawings.

2.2 Calculations

See attached Structural Engineer's Calculations.

2.3 Programme

See attached construction works programme.

2.4 Tree Protection

It is confirmed that the upper 1m of excavation is to be undertaken under arboricultural supervision and, where necessary, root pruning undertaken.

2.5 Sewer

It is confirmed that Thames Water Developer Services were consulted on 15th April 2019 to discuss the proposed scheme including the drainage diversion.

However, it is noted that this query has been raised as a Hydrological issue and that the surface water report has demonstrated all that needs to have been demonstrated for BIA validation purposes that surface water flood risks will be satisfactory and not adversely impacted.

2.6 Ground Investigation

As described in the initial BIA, a programme of trial pits has now been undertaken to confirm the ground conditions and to expose the configuration of the existing foundations.

The records are appended to this addendum.

2.6.1 Ground Conditions

As expected, a backfilled basement is present beneath the existing property and garden areas.

2.6.1.1 Made Ground

The fill material generally comprises brown silty sandy clay and brick fill with stones.

The fill material extends to up to approximately 3m beneath the building beneath the existing property and to a lesser depth (<2m) in the garden area.

2.6.1.2 London Clay

The London Clay Formation is present beneath the fill material and comprises typical firm to stiff pale brown silty clay, confirming that any capping of downwash deposits overlying the clay had been removed during the construction of the former villa.

2.6.1.3 Groundwater

The pits have confirmed that there is no groundwater table present beneath the site.

2.6.2 Soil Parameters

The following parameters may be considered in the design of the retaining walls:-

Stratum	Bulk Unity Weight	Effective Cohesion	Effective Friction Angle
	(kg/m^3)	(c' - kN/m^2)	(ϕ'- degrees)
Made Ground	17	Zero	18
London Clay Formation	20	Zero	25

2.6.3 Bearing Capacity

A presumed net bearing capacity of 120kN/m^2 may be adopted for the firm to stiff London Clay that is to be adopted as the founding soil for the new basement underpinning.

2.6.4 Excavation Support

The trial pit excavations themselves have demonstrated the feasibility of underpinning through the backfilled basement material.

2.7 Ground Movement Assessment

The ground movement assessment presented in section 7 of audited report was based upon the worst credible conditions with soil parameters drawn from accepted sources.

The ground investigation information confirms the assumed conditions and thus the ground movement assessment undertaken as part of the initial impact assessment remains valid, resulting in no worse than Burland Category 1 damage.

3. Conclusion

The BIA has demonstrated that the proposed scheme will not

- adversely affect the drainage and run off or cause other damage to the water
- result in cumulative impacts on structural stability or the water environment in the local area
- put at risk the structural stability of the building or any neighbouring properties

As explained above, the ground investigation information confirms that the proposed development will not result in an unacceptable result in no worse than Burland Category 1 damage.

Audit Query No.	Audit Query	Location of additional Information Submitted	Status
1	Drawings with clear indication of the dimensions of the existing and proposed structures to be presented.	Appendix	Addressed
2	Outline design calculations and drawings for the proposed temporary and permanent works.	Appendix	Addressed
3	A construction works programme should be provided for the main works including any proposed works for the western perimeter wall.	Appendix	Addressed
4	Contradictory information presented in the BIA about protection of trees	See section 2.4	Addressed
5	Consultation with drainage/sewer utility owners should be confirmed	See section 2.5	Addressed
6	A ground investigation should be carried out to establish ground conditions, ground water level, soil parameters, bearing capacity of founding stratum, the details regarding foundations of neighbouring structures and the feasibility of underpinning through backfilled demolition material.	See sections 2.6 and Appendix	Addressed
7	The ground movement assessment should be revisited once the information from the ground investigation is available.	See section 2.7	Addressed

Appendix

Structural Engineer’s Drawings

Structural Engineer’s Calculations

Programme

Ground Investigation

Full Response to CRH Audit Checklist comments

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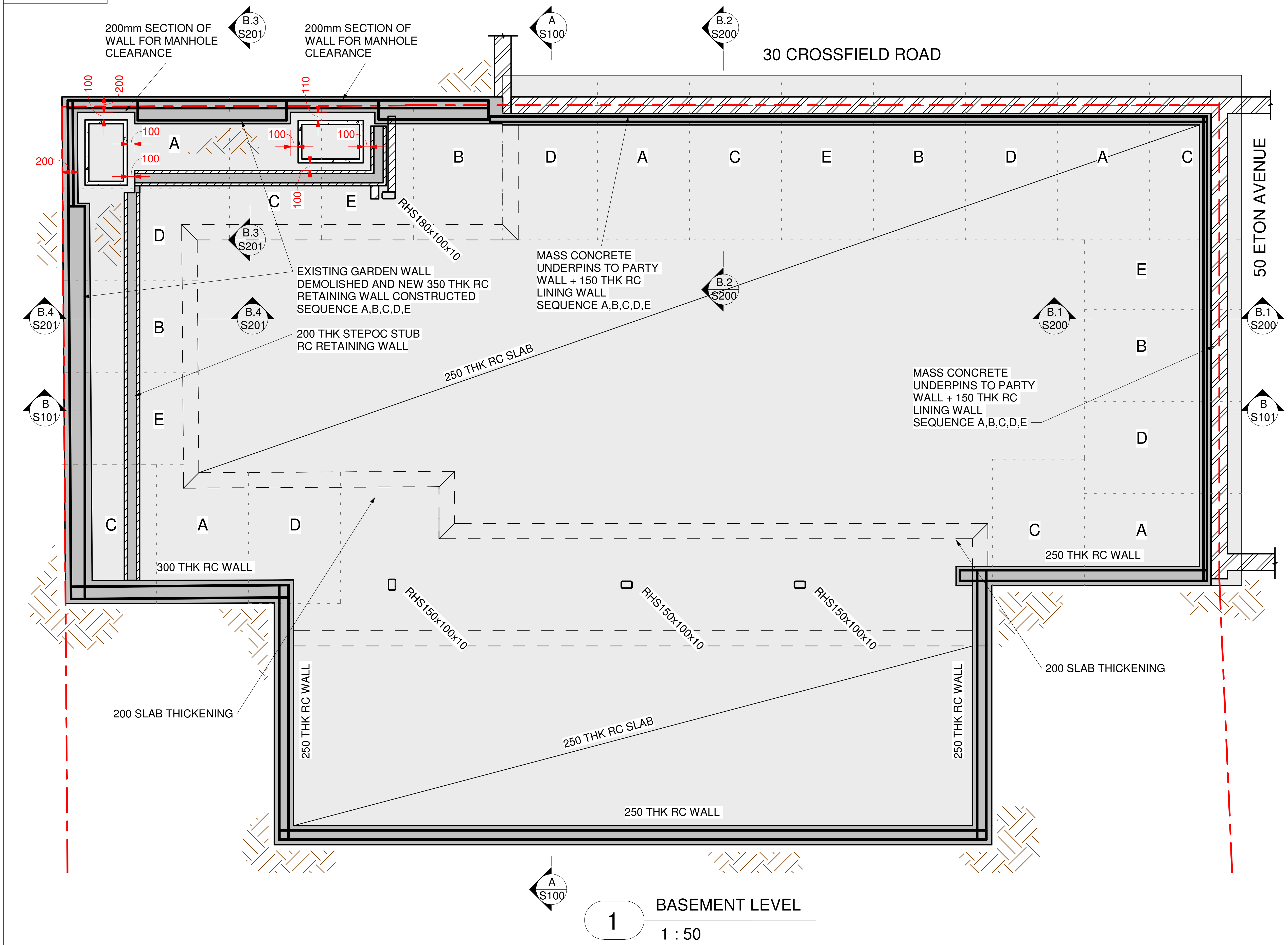

UNDERPINNING NOTES

1. CONTRACTOR TO BE RESPONSIBLE TO ENSURE THAT ALL WORKS ARE CARRIED OUT IN A SAFE AND WORKMANLIKE MANNER, TO PROVIDE SATISFACTORY TEMPORARY PROPPING TO MAINTAIN THE STABILITY OF ALL THE EXISTING STRUCTURAL ELEMENTS THROUGHOUT THE DURATION OF THE PROPOSED WORKS.
2. CONTRACTOR IS TO ENSURE THAT THE SITE IS KEPT CLEAN AND TIDY AT ALL TIMES THROUGHOUT THE PROPOSED WORKS. ALL DEBRIS IS TO BE BAGGED UP AT THE END OF EACH DAY AND REMOVED FROM SITE.
3. SECTIONS WITH THE SAME LETTER CAN BE EXCAVATED SIMULTANEOUSLY, CAST BASES MARKED 'A' COMPLETING ALL BASES BEFORE PROCEEDING WITH BASES MARKED 'B' ETC.
4. THE NEXT LETTERED SECTION IS NOT TO BE EXCAVATED UNTIL THE PREVIOUS SECTION HAS BEEN PINNED AND CURED,
5. THE UNDERSIDE OF THE EXISTING FOUNDATION IS TO BE THOROUGHLY CLEANED OF ALL SOIL BEFORE UNDERPINNING COMMENCES.
6. ALL EXCAVATED SECTIONS TO BE CONCRETED THE SAME DAY. PLYWOOD SHEETS ARE TO BE PROVIDED OVER SHALLOW TRENCHES LEFT OPEN OVERNIGHT.
7. BOTTOMS OF ALL EXCAVATIONS TO BE INSPECTED AND AGREED BY THE BUILDING INSPECTOR BEFORE CONCRETE IS POURED.
8. ALL EXCAVATIONS AND THE SURROUNDING SITE TO BE KEPT FREE OF WATER.
9. CONCRETE TO UNDERPINNING TO BE GRADE C30 (MAXIMUM NOMINAL SIZE OF AGGREGATE TO BE 20MM) AND CAST UP TO WITHIN 50MM OF THE UNDERSIDE OF THE EXISTING FOUNDATION.
10. A PERIOD OF AT LEAST FOUR DAYS SHALL BE ALLOWED BEFORE PINNING UP THE LAST 50MM.
11. USING A CLUB HAMMER AND A 25MM SQ. STEEL ROD, 1:3 SEMI-DRY SAND/CEMENT MORTAR WITH A PROPRIETARY NON-SHRINK ADDITIVE, TO BE WELL RAMMED HOME BETWEEN TOP OF UNDERPINNING AND UNDERSIDE OF EXISTING FOUNDATION.
12. BEFORE CASTING AGAINST A PREVIOUSLY CAST PIN, THE SURFACE OF THE EXISTING CONCRETE SHALL BE THOROUGHLY ROUGHENED TO REMOVE ALL LAITANCE AND TO EXPOSE THE COURSE AGGREGATE. THE PREPARED SURFACE MUST BE THOROUGHLY CLEANED OF ALL LOOSE MATERIAL INCLUDING DIRT AND DUST (WITHOUT USING WATER) BEFORE CASTING ADJACENT PINS.
13. A FOUR DAY PERIOD MUST PASS BEFORE TWO BASES ARE CAST AGAINST EACH OTHER TO ALLOW FOR CONCRETE TO CURE.
14. ON COMPLETION OF UNDERPINNING BACKFILL EXCAVATED TRENCHES IN 150MM LAYERS, WELL COMPACTING EACH LAYER.

NOTES
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DESIGN STAGE
PRELIMINARY

REV.	DATE	DESCRIPTION	REV.	DATE	DESCRIPTION

CLIENT
N. Matalon and I. Tepekoylu
CONTRACT
Eton Avenue
TITLE
Basement floor plan

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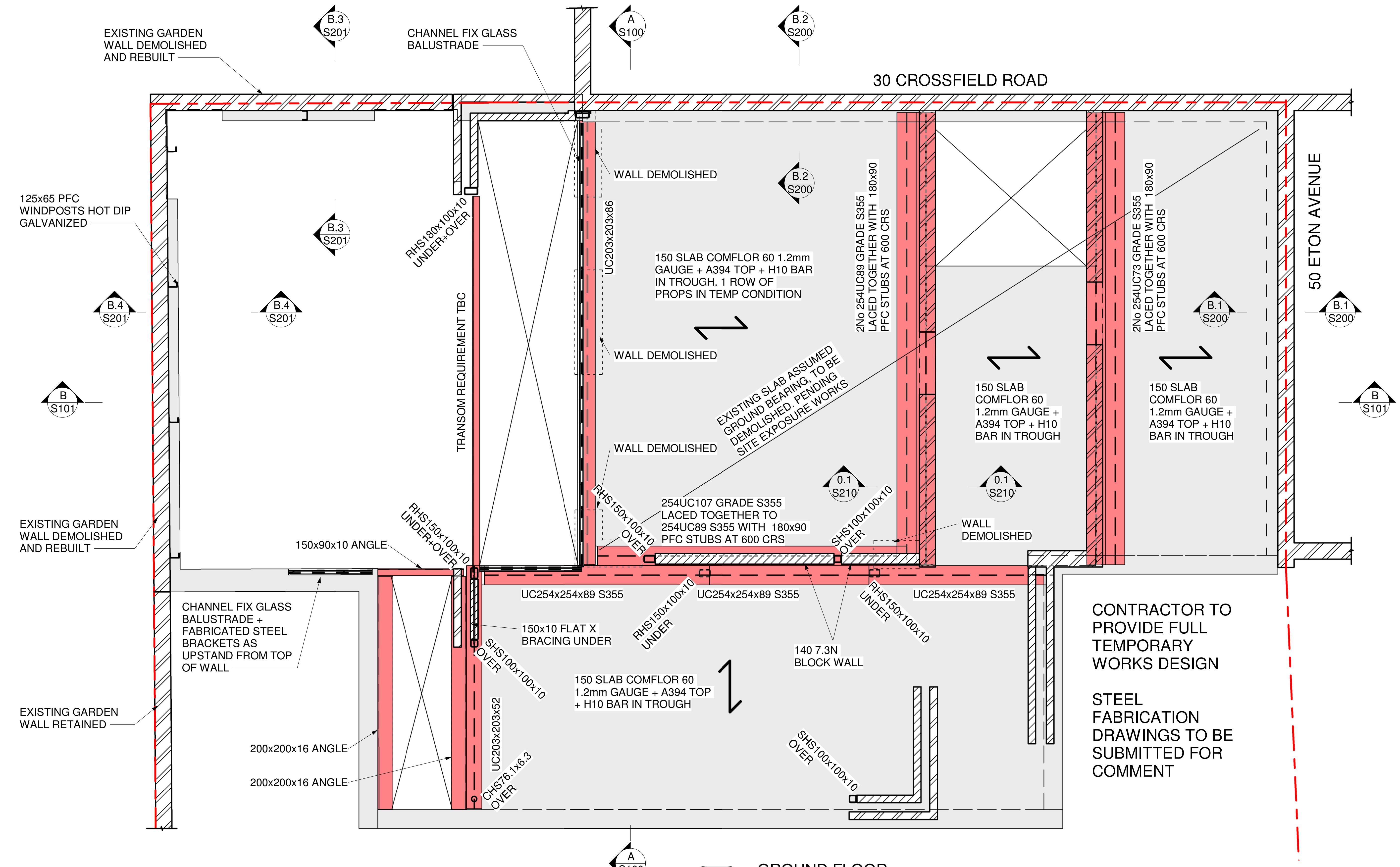

1 GROUND FLOOR
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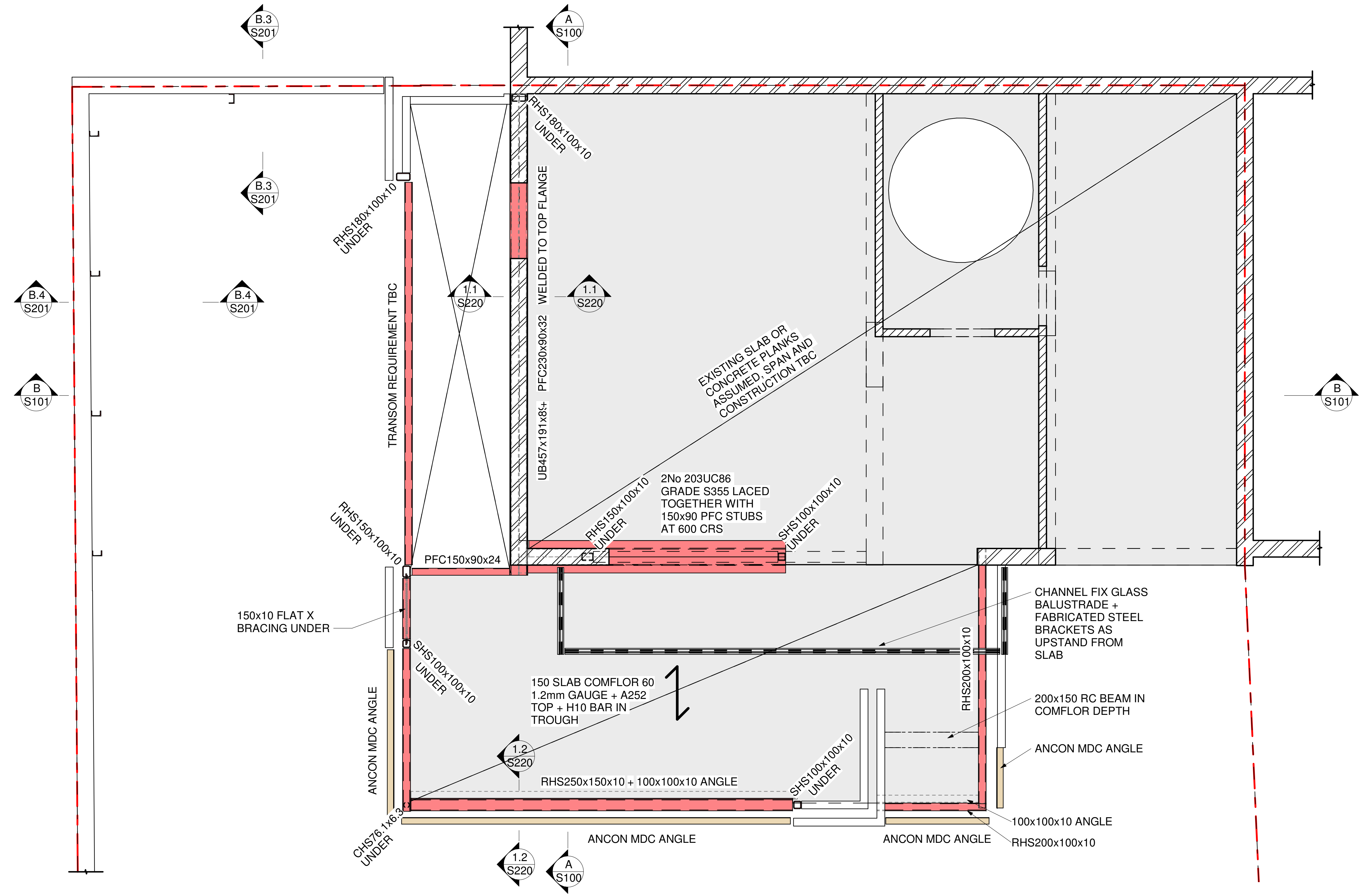

1 1ST FLOOR

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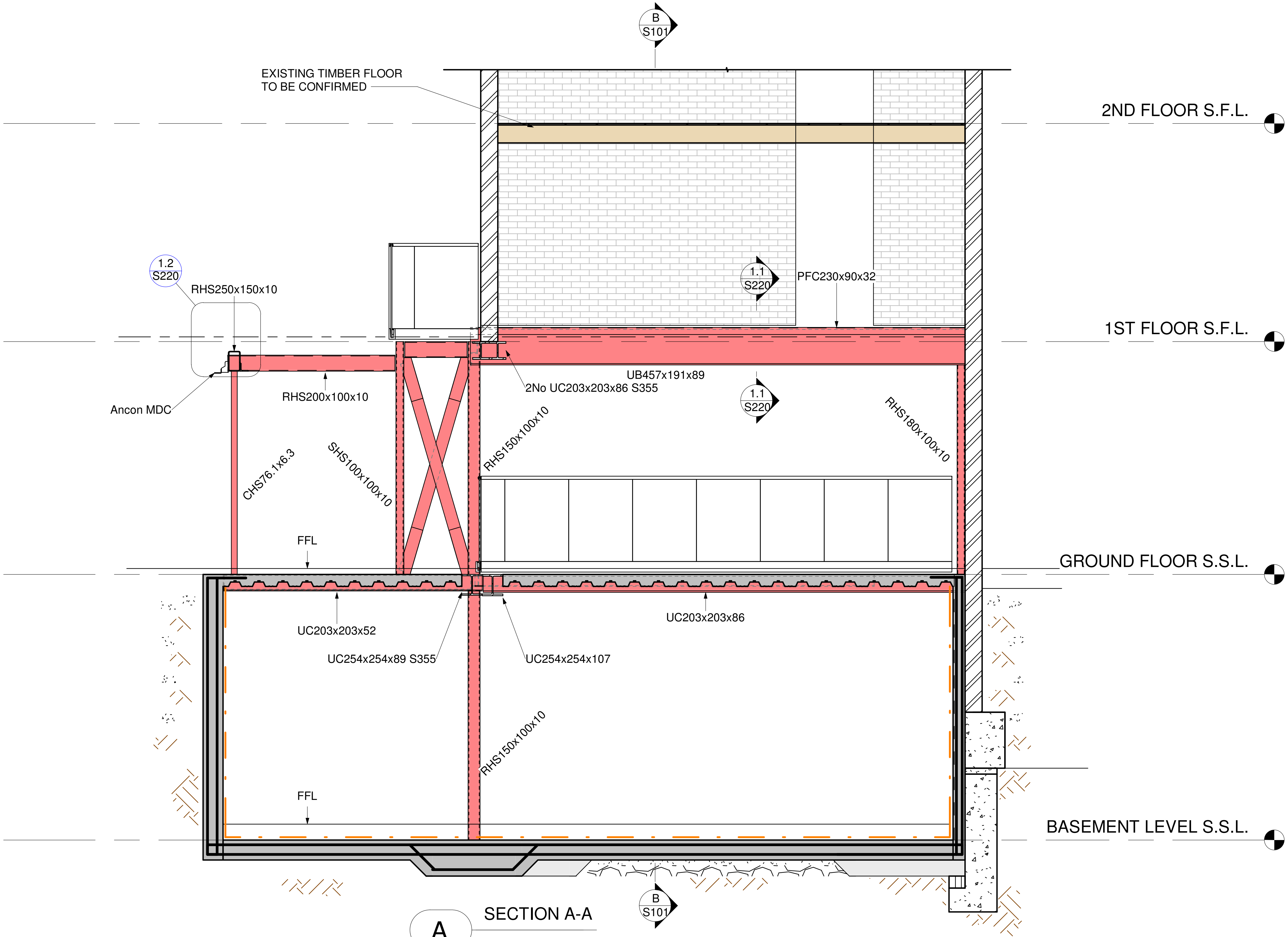

A SECTION A-A

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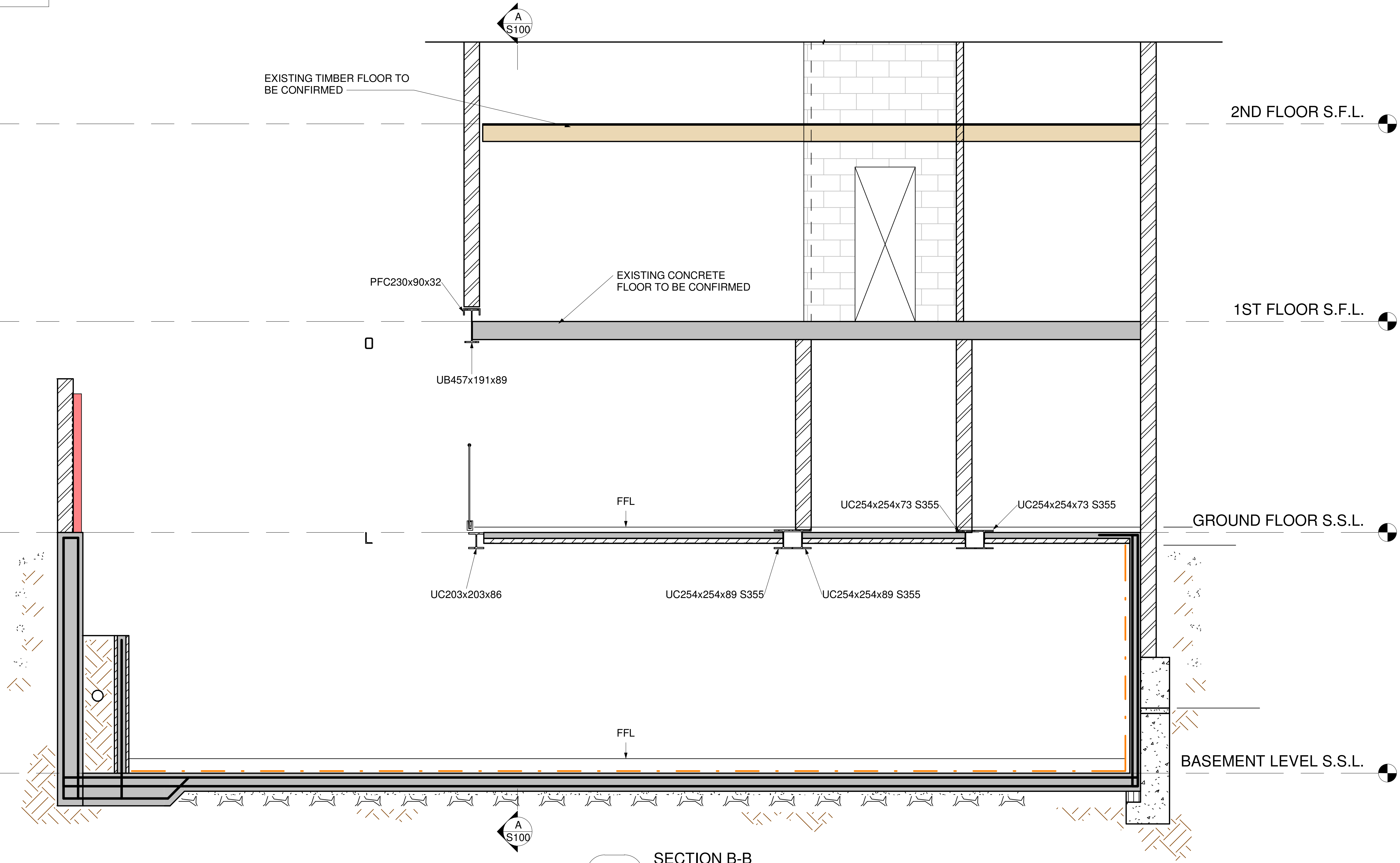

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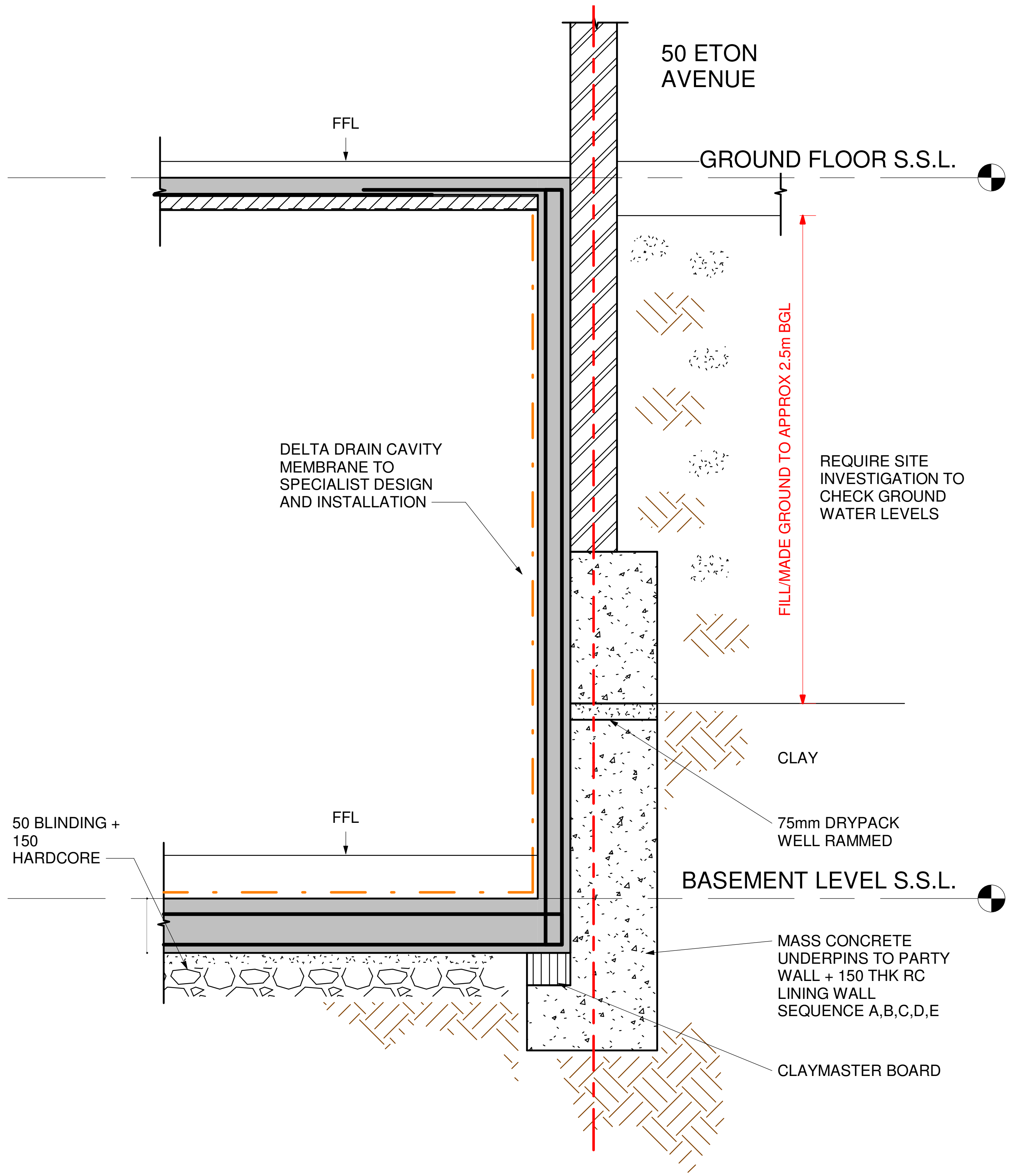

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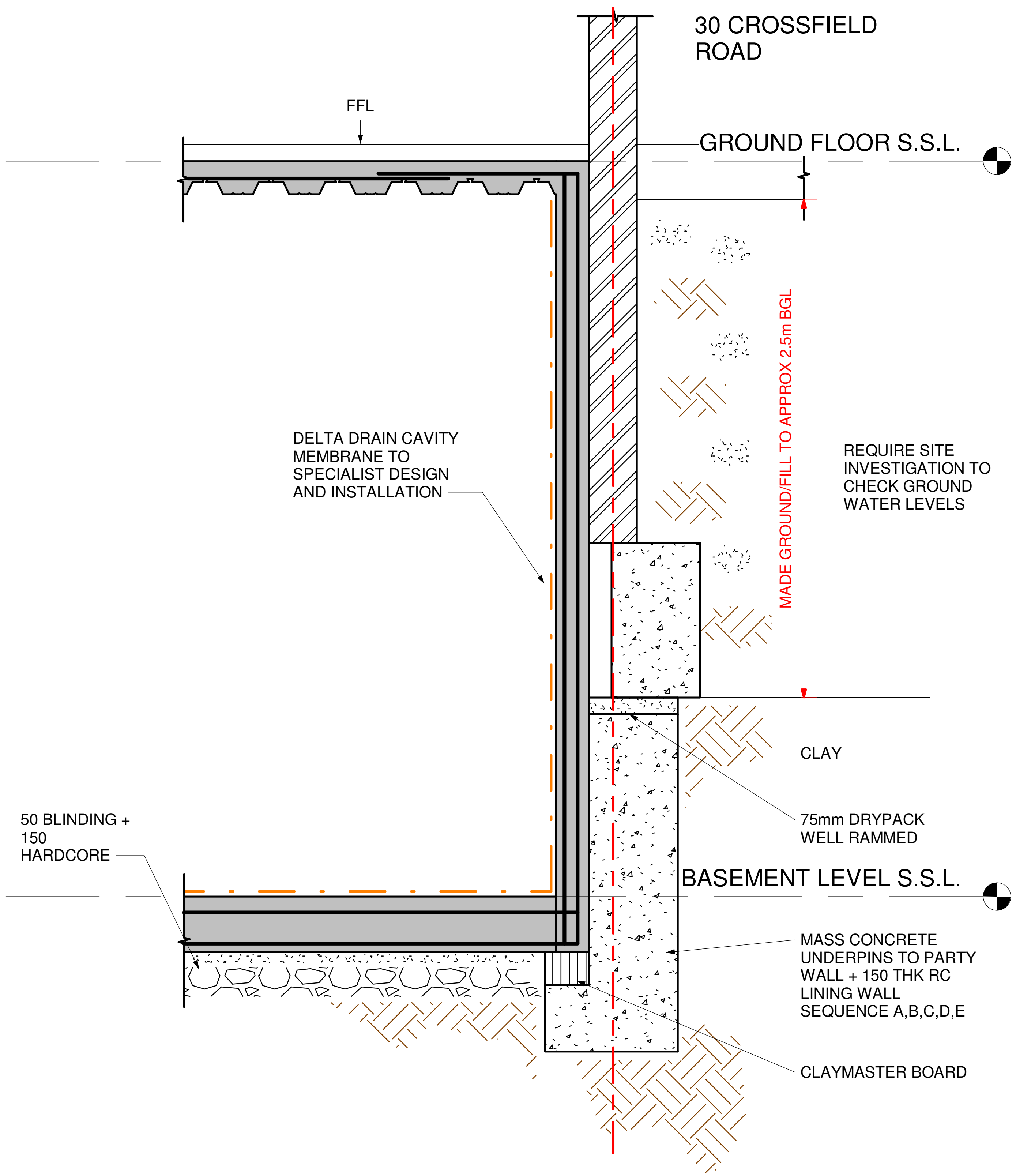

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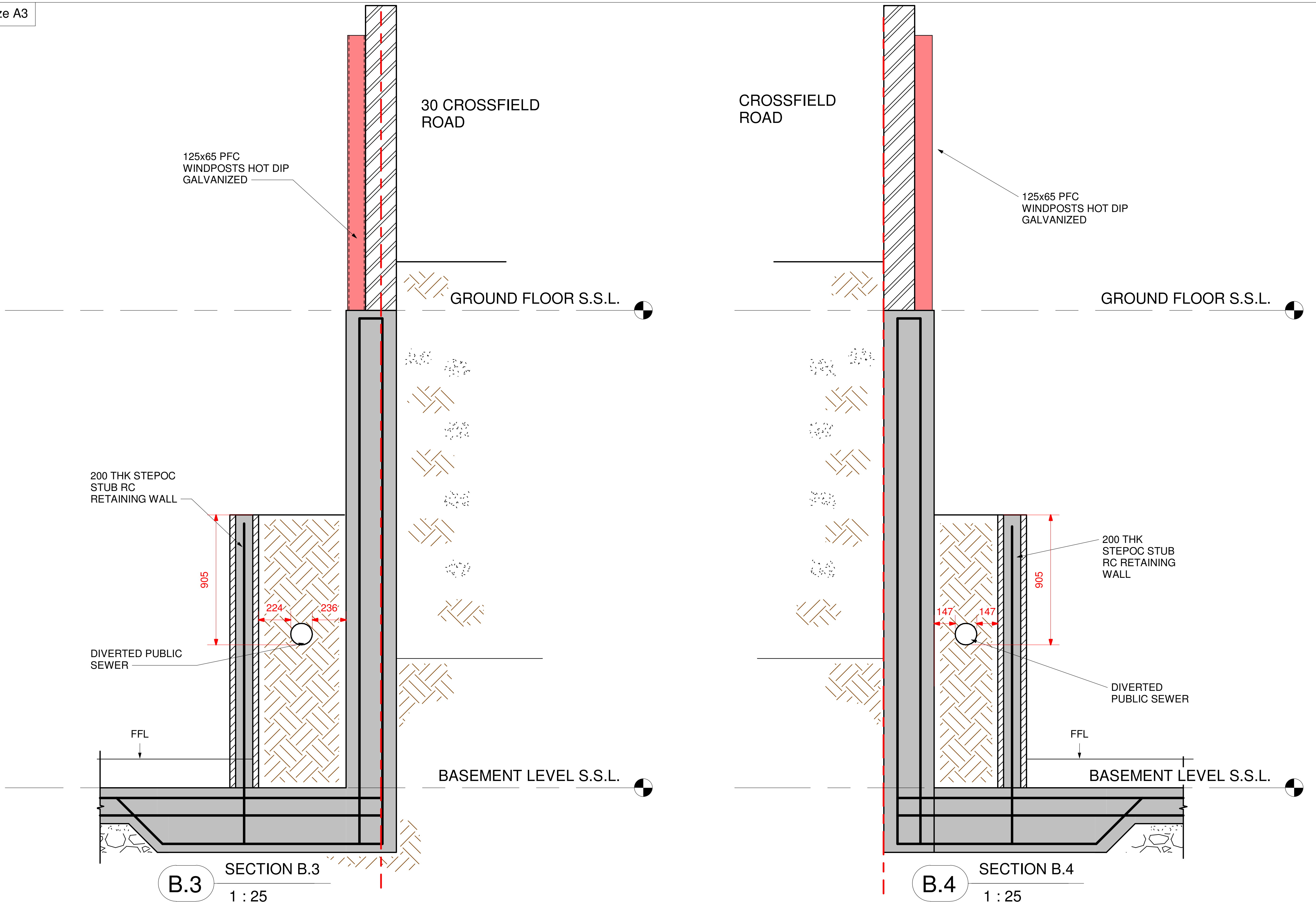

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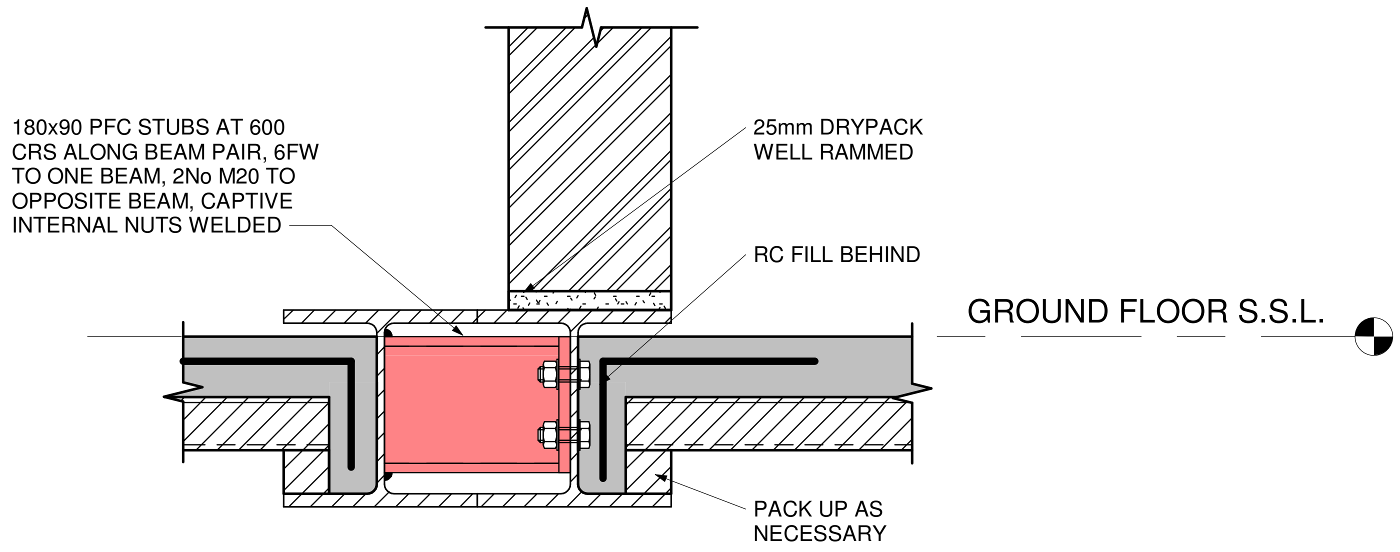

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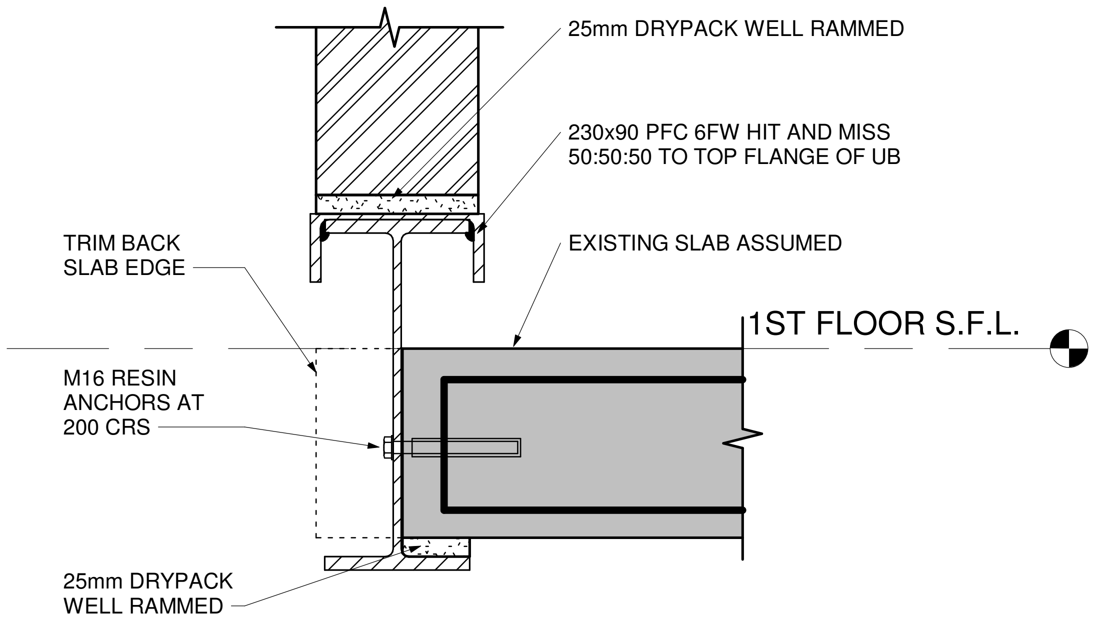

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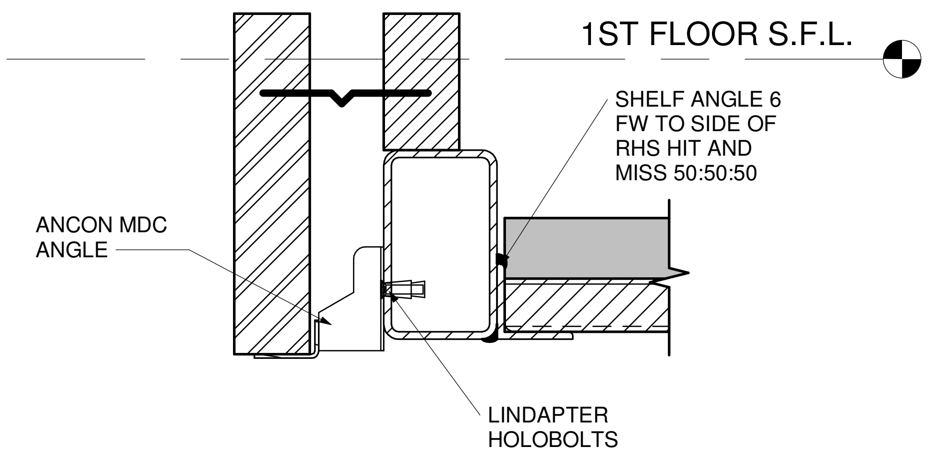

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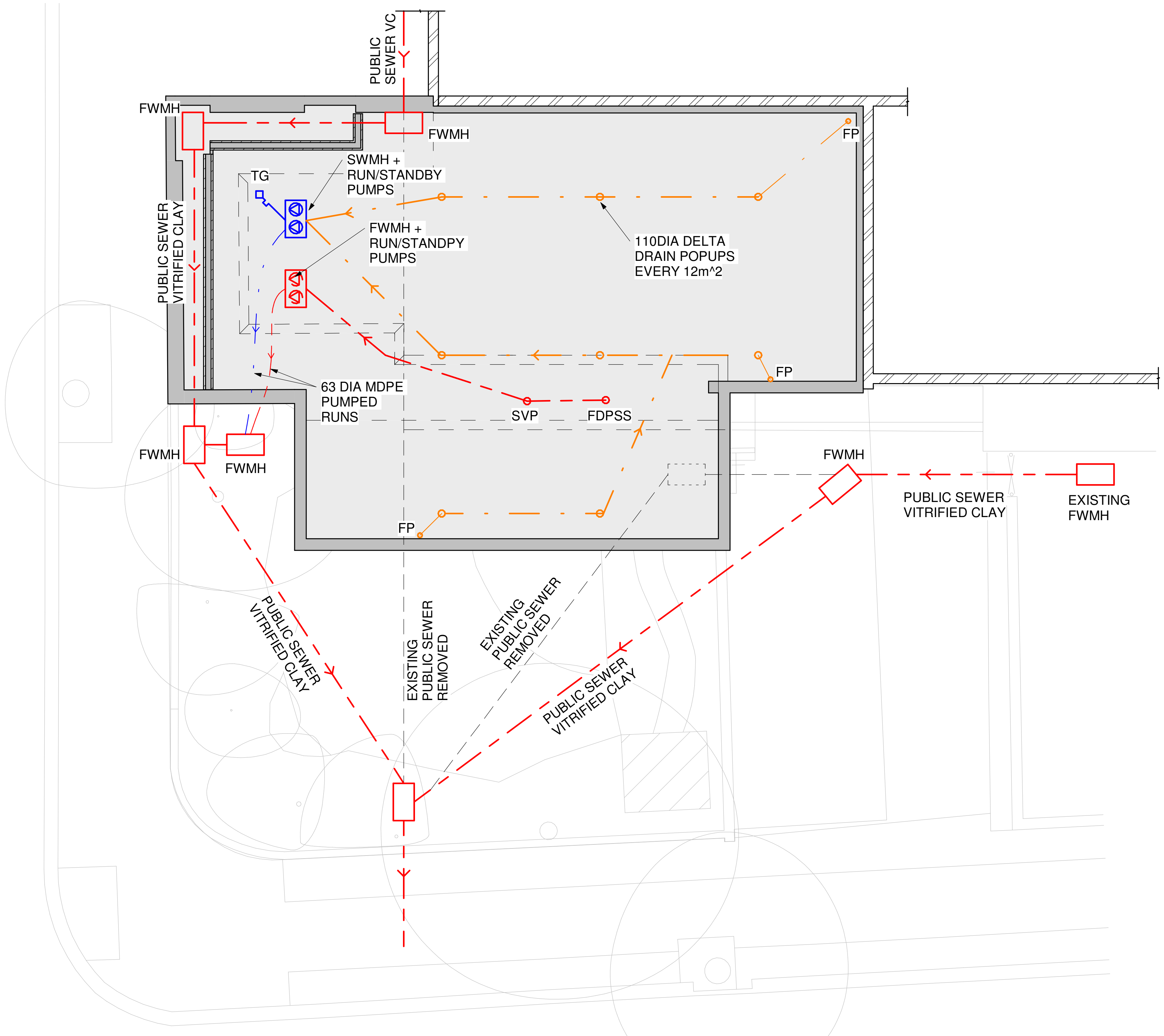
DRAINAGE LAYOUT
INDICATIVE, FULL
DESIGN BY SPECIALIST



1 DRAINAGE GA
1 : 100

= SURFACE WATER DRAIN
= FOUL DRAIN
= DELTA DRAIN

RK = FLEXIBLE ROCKER COUPLING
RE = RODDING EYE
FDPSS = FOUL DRAIN POINT STUB STACK
IC = INSPECTION CHAMBER
MH = MANHOLE
RWDP = RAINWATER DOWNPIPE
SVP = SOIL VENT PIPE
FP = DELTA DRAIN FLUSHING PORT
TG = TRAPPED GULLEY
IL = INVERT LEVEL
CL = COVER LEVEL
SKW = SOAKAWAY
RG = ROAD GULLEY
ACO = ACO TYPE SLOT DRAIN CHANNEL

NOTES
1. IT IS RECOMMENDED THAT NO WORK IS CARRIED OUT UNTIL BUILDING REGULATIONS APPROVAL HAS BEEN OBTAINED.
2. IT IS ESSENTIAL THAT TZG ARE NOTIFIED OF ANY DISCREPANCIES OR SUBSEQUENT CHANGES PRIOR TO THEM BEING IMPLEMENTED.
3. DO NOT SCALE FROM THIS DRAWING.
4. THIS DRG IS TO BE READ IN CONJUNCTION WITH THE SPECIFICATION AND ARCHITECTS DRAWINGS.
5. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE STATED.
6. A CONTRACTOR USED TO WORKING WITH MATERIALS SHOWN ON THIS DRAWING SHOULD FIND NO UNEXPECTED HAZARDS. ANY ADDITIONAL RISKS ARE NOTED ON THE DRAWING

DESIGN STAGE
PRELIMINARY

REV.	DATE	DESCRIPTION	REV.	DATE	DESCRIPTION

CLIENT
N. Matalon and I. Tepekoylu

CONTRACT
Eton Avenue

TITLE
Underground drainage GA

TZG PARTNERSHIP
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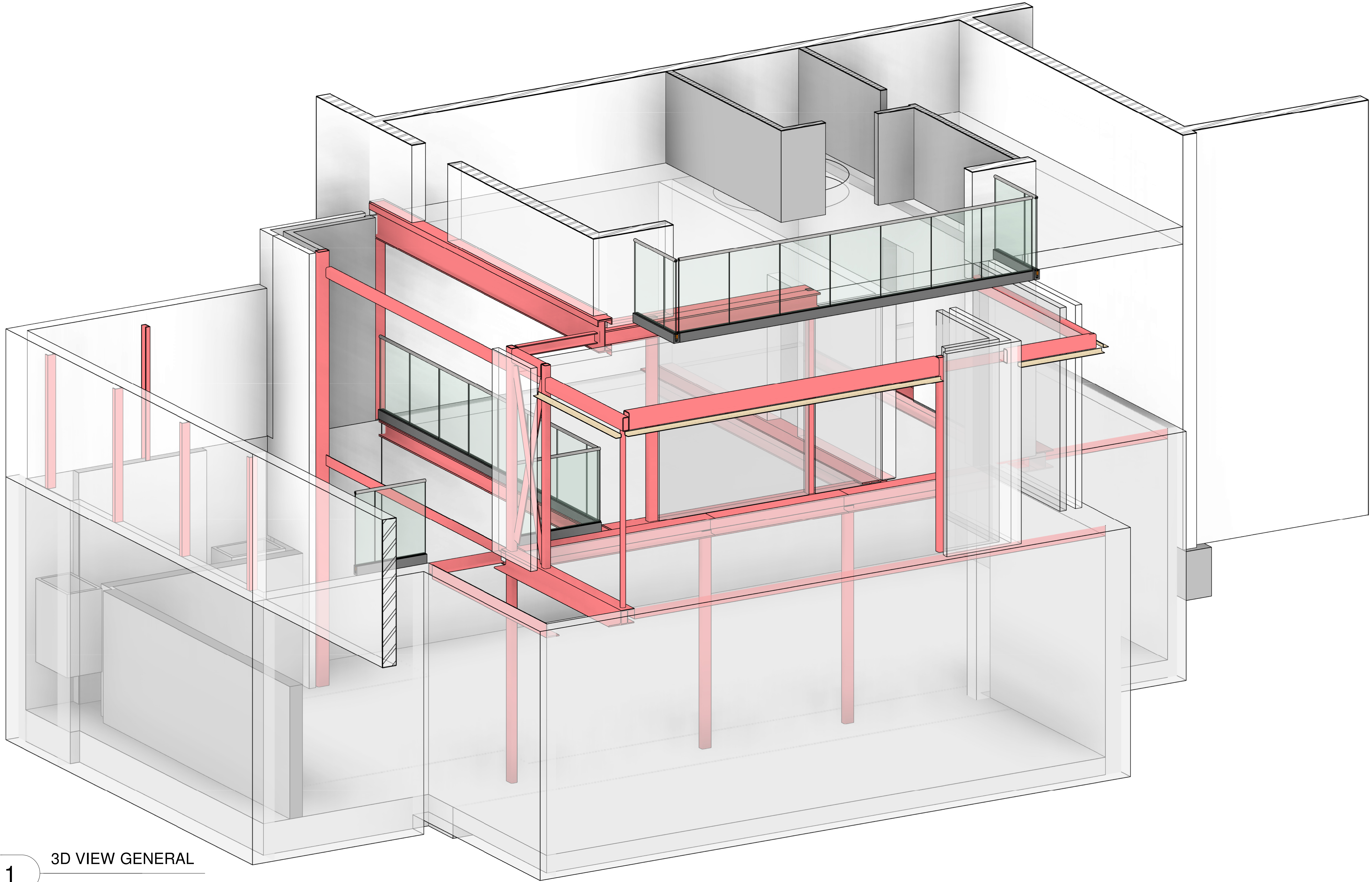
SCALE	DRAWN	DATE
AS SHOWN	GH	03.19
CONTRACT No. 6722	DRG. No. S400	REVISION

Original size A3



1 3D VIEW GENERAL

NOTES
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DESIGN STAGE
PRELIMINARY

REV.	DATE	DESCRIPTION	REV.	DATE	DESCRIPTION

CLIENT
N. Matalon and I. Tepekoylu

CONTRACT
Eton Avenue

TITLE
3D Views

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SCALE	DRAWN	DATE
AS SHOWN	GH	03.19
CONTRACT No. 6722	DRG. No. S300	REVISION

TZG PARTNERSHIP
ENGINEERING CONSULTANTS

CONTRACT: Eton Avenue

CONTRACT NO. 6722

DATE: 04/2019

BY: GH

PRELIMINARY STRUCTURAL CALCULATIONS FOR PLANNING

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INTRODUCTION

Eton Avenue is an existing 3 storey end of terrace house constructed with loadbearing masonry, timber floors and timber flat roof. The following calculations are for a basement extension and alterations to the existing property above ground

Calculations make reference to the following design codes

BS EN 1990 – Eurocode 0: Basis of Structural Design

BS EN 1991 – Eurocode 1: Actions on Structures

BS EN 1992 – Eurocode 2: Design on Concrete Structure

BS EN 1993 – Eurocode 3: Design of Steel Structures

BS EN 1995 – Eurocode 5: Design of Timber Structures

Frame calculations are performed with Nemetschek SCIA Engineer and code checks performed in Fitzroy SAND

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LOADINGS

Dead and Live Load

Pitch Roofs

Tiles	= 0.60 kN/m²
Battens + Felt	= 0.10 kN/m²
Plasterboard	= 0.15 kN/m²
Rafters	= <u>0.15 kN/m²</u>
Total dead along rafter	= 1.00 kN/m²
On plan	$= \frac{1.00}{cos30} = 1.15$ kN/m²
Live on plan	= 0.75 kN/m²

Existing roof Space

Joists + Insulation	= 0.15 kN/m²
Soffit	= <u>0.15 kN/m²</u>
Total dead	= 0.30 kN/m²
Live	= 0.25 kN/m²

Pitch Roof to extension

Tiles	= 0.60 kN/m²
Battens + Felt	= 0.10 kN/m²
Joists + Insulation	= 0.15 kN/m²
Soffit	= <u>0.15 kN/m²</u>
Total dead along rafter	= 1.00 kN/m²
On plan	$= \frac{1.00}{cos30} = 1.15$ kN/m²
Live on plan	= 0.75 kN/m² `

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Flat Roof to extension

Chippings + Felt	= 0.35 kN/m²
Boards + Joists + Fittings	= 0.30 kN/m²
Soffit + Insulation	= 0.15 kN/m²
Total dead	= 0.80 kN/m²
Live on plan	= 0.75 kN/m²

Exg timber Floors

Boards + Joists	= 0.35 kN/m²
20 lath and plaster	= 0.44 kN/m²
Total dead	= 0.80 kN/m²
Live	= 1.50 kN/m²

New timber Floors

Boards + Joists	= 0.35 kN/m²
Plasterboard soffit	= 0.15 kN/m²
Total dead	= 0.50 kN/m²
Live	= 1.50 kN/m²

Existing First floor slab

250 RC slab	= 6.00 kN/m²
Insulation	= 0.05 kN/m²
50 screed	= 1.20 kN/m²
Tiles	= 0.20 kN/m²
Total dead	= 7.45 kN/m²
Live	= 1.50 kN/m²

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First floor comflor slab

150 Comflor 60	= 2.96 kN/m²
Insulation	= 0.05 kN/m²
50 screed	= 1.20 kN/m²
Tiles	= 0.20 kN/m²
Total dead	= 4.41 kN/m²
Live	= 1.50 kN/m²

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Walls

Timber stud internal

Plasterboard	= 0.15 kN/m²
Studs	= 0.10 kN/m²
Plasterboard	= 0.15 kN/m²
Total dead	= 0.40 kN/m²

Block Partition internal

Block 100mm	= 1.00 kN/m²
Plaster (two faces)	= 0.50 kN/m²
Total dead	= 1.50 kN/m²

Brick and Blockwork

Brick 102.5mm	= 2.00 kN/m²
Block 100mm	= 1.60 kN/m²
Plaster (one face)	= 0.30 kN/m²
Total dead	= 3.90 kN/m²

Brick 215mm

Brick 215mm	= 4.00 kN/m²
Plaster (2 faces)	= 0.60 kN/m²
Total dead	= 4.60 kN/m²

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Brick 330mm

Brick 330mm	= 6.00 kN/m²
Plaster (one face)	= 0.30 kN/m²
Total dead	= 6.30 kN/m²

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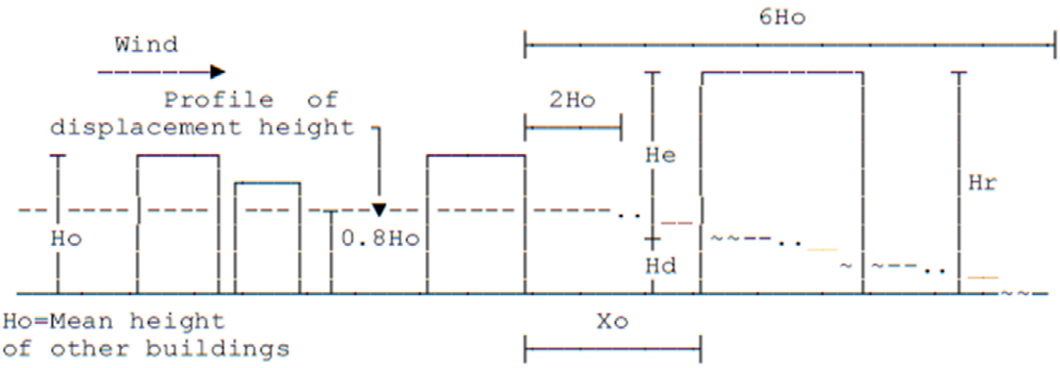
WIND LOAD

Displacement heights



Ho=Mean height
of other buildings

Building reference height	Hr=7.5 m
upwind of site	Ho=7.500 m
Upwind spacing of building	Xo=15 m
Displacement height	Hdw0=6 m
Displacement height	Hdw90=6 m
Building type factor	Kb=0.5
Building height	H=7.5 m

Standard wind loads

Basic wind speed	Vb=21 m/s
Site altitude above mean sea level	deltaS=44 m
Direction factor	Sdw0=1
Direction factor	Sdw90=1

DESIGN SUMMARY			
	Effective height (long face)	Hew0	3 m
	Effective height (short face)	Hew90	3 m
	Altitude factor (long face)	Saw0	1.044
	Altitude factor (short face)	Saw90	1.044
	Direction factor (long face)	Sdw0	1
	Direction factor (short face)	Sdw90	1
	Seasonal factor	Ss	1
	Probability factor	Sp	1
	Dynamic wind pressure (long face)	qsw0	0.4013 kN/m²
	Dynamic wind pressure (short face)	qsw90	0.4013 kN/m²

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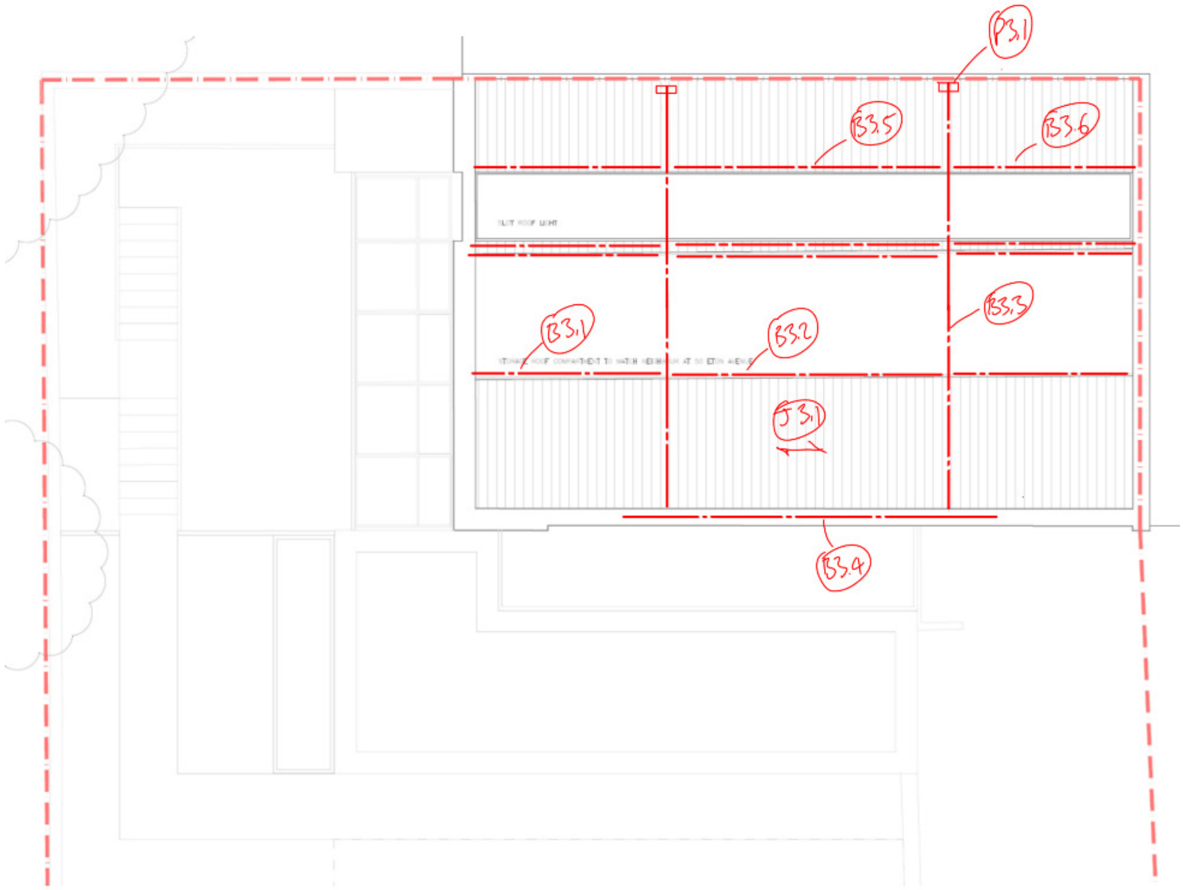
CONTRACT	Eton Ave
PAGE No.	FD13
CONTRACT No.	6722
DATE	04/19
BY	GH

FRAMING DIAGRAMS



Roof level

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PAGE No.	FD14
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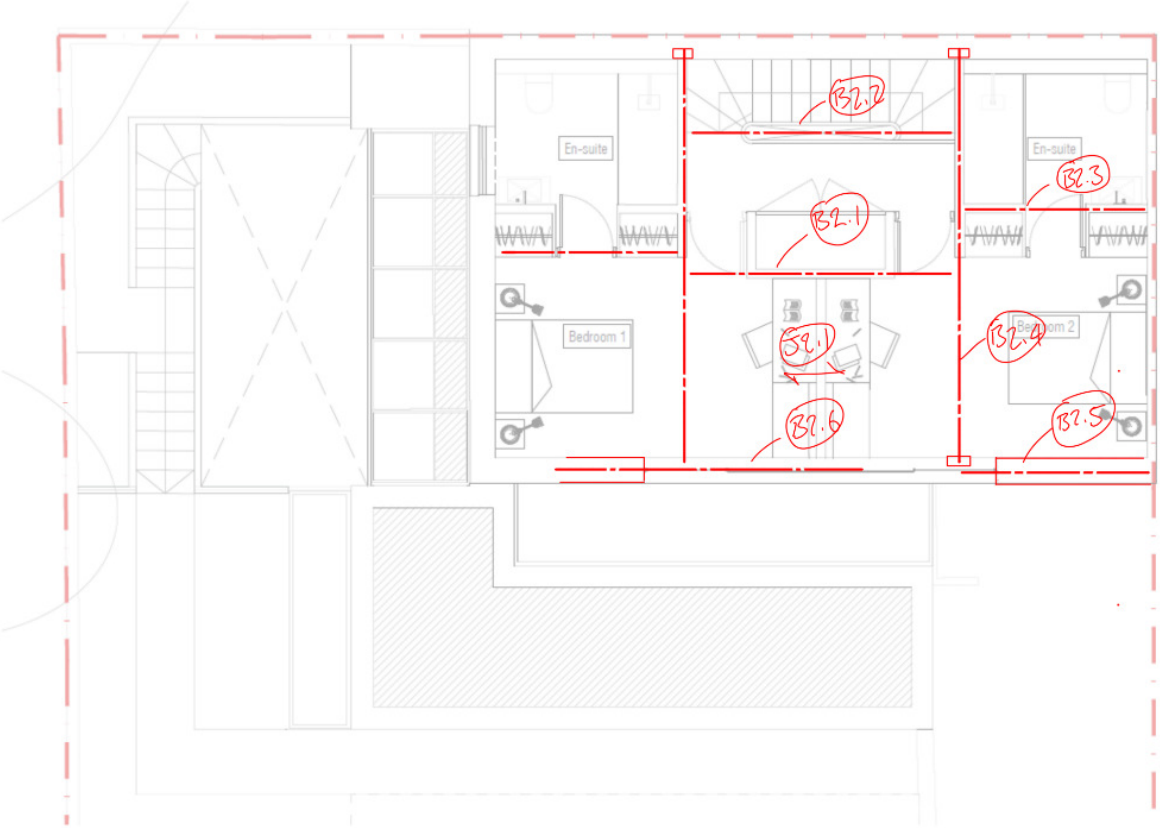

Second Floor

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CONTRACT No.	6722
DATE	04/19
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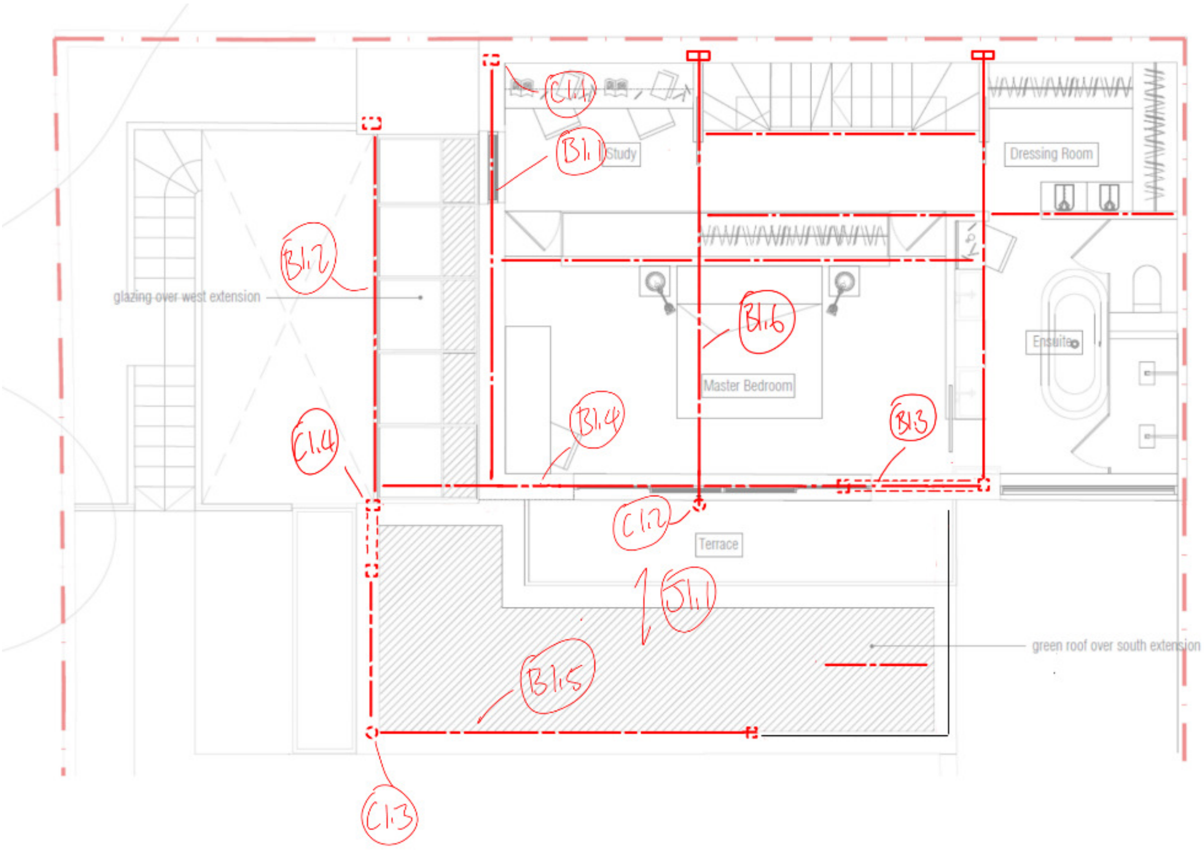

First Floor

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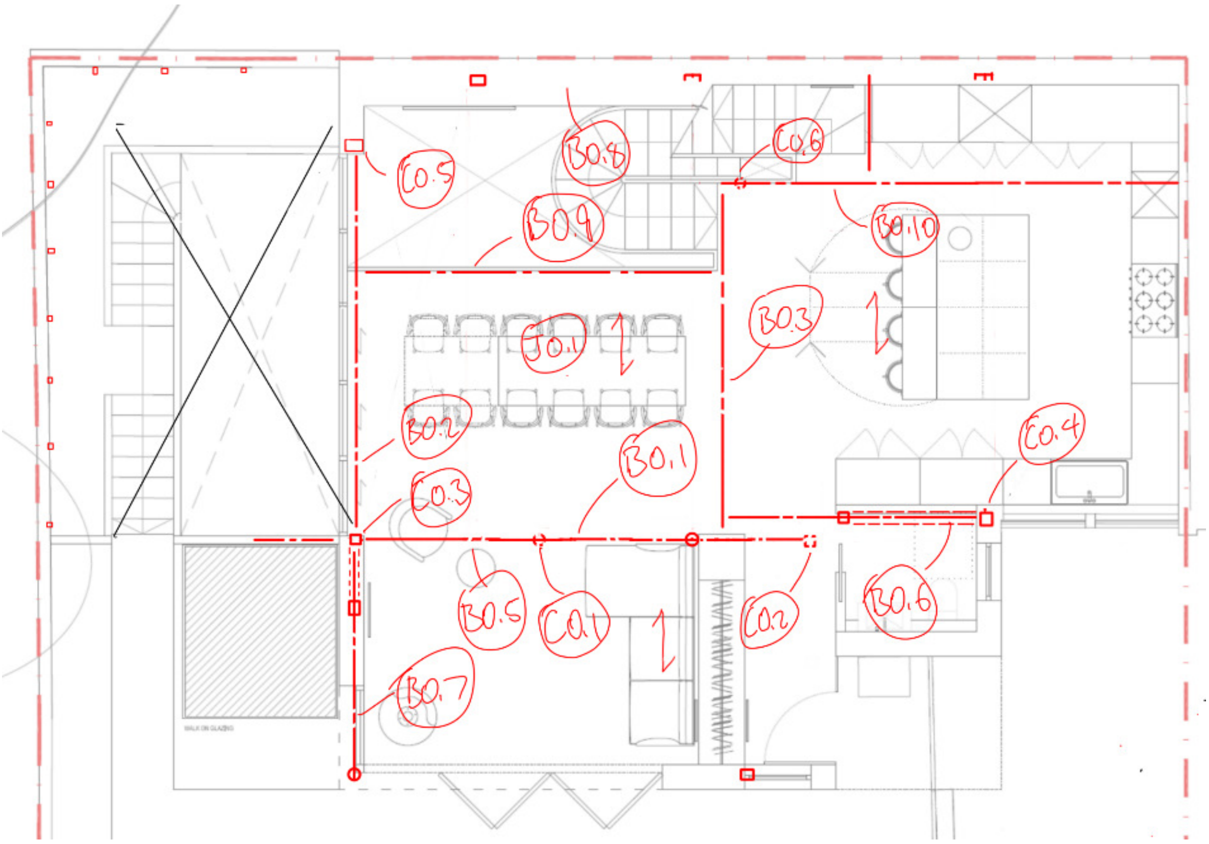
PAGE No.	FD16
CONTRACT No.	6722
DATE	04/19
BY	GH



Ground Floor

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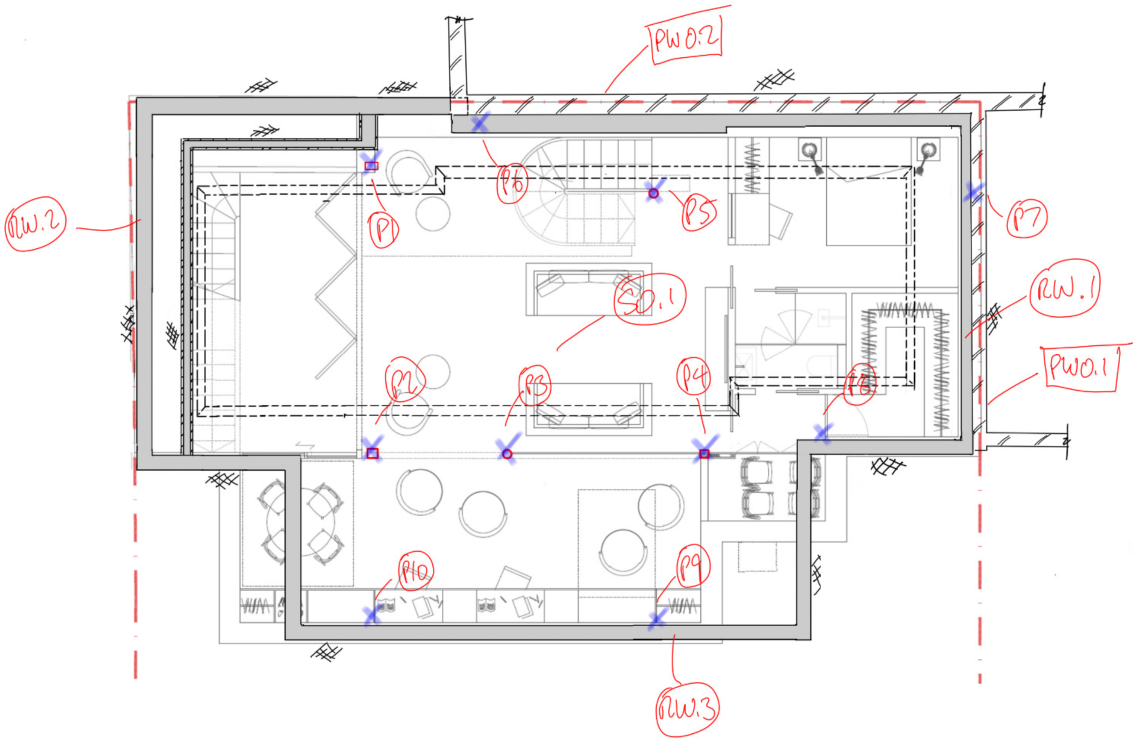
CONTRACT	Eton Ave
PAGE No.	FD17
CONTRACT No.	6722
DATE	04/19
BY	GH



Basement Floor

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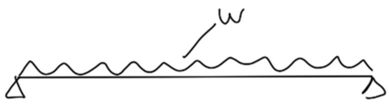
Roof Load Takedown

J3.1

New roof joist span = 3.9m

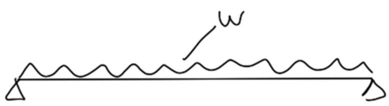


	Dead	Live
$w_D = 0.8 \times 0.4 =$	0.32kN/m	
$w_L = 1.5 \times 0.4 =$		0.6kN/m

200x50 C24@400

B3.1

New roof beam supports storage wall and roof span = 2.7m



	Dead	Live
$w_D = 0.8 \times 0.4 + 0.8 \times 1.8 + 0.8 \times \frac{1.7}{2} =$	2.5kN/m	
$w_L = 1.5 \times 0.4 + 1.5 \times \frac{1.7}{2} =$		1.9kN/m

2No 200x50 C24

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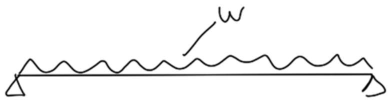
B3.2

New roof beam supports storage wall and roof span = 3.9m



	Dead	Live
$w_D = 0.8 \times 0.4 + 0.8 \times 1.8 + 0.8 \times \frac{1.7}{2} =$	2.5kN/m	
$w_L = 1.5 \times 0.4 + 1.5 \times \frac{1.7}{2} =$		1.9kN/m

152UC23

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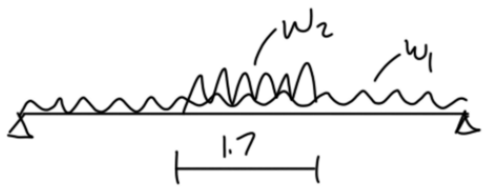
B3.3

New roof beam supports span = 5.8m



	Dead	Live
$w_{1D} = 0.8 \times \frac{6.5}{2} =$	2.6kN/m	
$w_{1L} = 1.5 \times \frac{6.5}{2} =$		4.9kN/m
$w_{2D} = 1.6 \times \frac{6.5}{2} =$	5.2kN/m	
$w_{2L} = 0.75 \times \frac{6.5}{2} =$		2.5kN/m

203UC46 + 500x100x215 padstone

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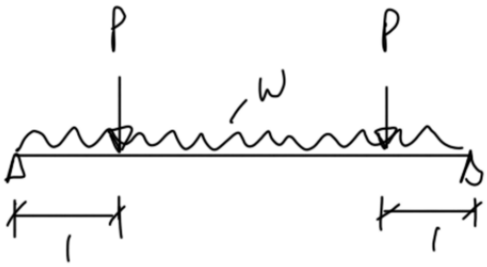
B3.4

Lintel supports wall and B3.3 span = 5m



	Dead	Live
$w_D = 6.6 \times 0.5 =$	3.3kN/m	
$P_D =$	13.3kN	
$P_L =$		16.3kN

203UC46 + 300 bearing

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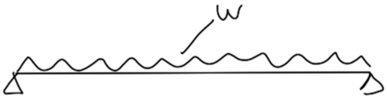
B3.5

New roof beam supports rooflight span = 3.9m

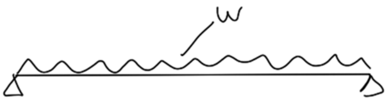


	Dead	Live
$w_D = 0.8 \times \frac{1.4}{2} =$	0.6kN/m	
$w_L = 1.5 \times \frac{1.4}{2} =$		1.1kN/m

4No 200x50 C24

B3.6

New roof beam supports rooflight span = 2.7m



	Dead	Live
$w_D = 0.8 \times \frac{1.4}{2} =$	0.6kN/m	
$w_L = 1.5 \times \frac{1.4}{2} =$		1.1kN/m

2No 200x50 C24

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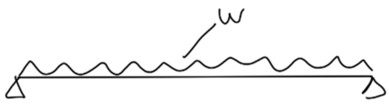
Second floor Load Takedown

J2.1

New floor joist span = 3.9m

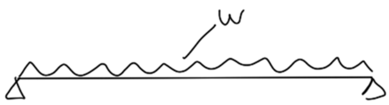


	Dead	Live
$w_D = 0.8 \times 0.4 =$	0.32kN/m	
$w_L = 1.5 \times 0.4 =$		0.6kN/m

200x50 C24@400

B2.1

New floor beam supports partition span = 3.9m

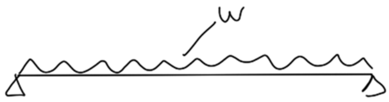


	Dead	Live
$w_D = 0.8 \times 0.4 + 0.5 \times 3 =$	1.82kN/m	
$w_L = 1.5 \times 0.4 =$		0.6kN/m

3No 200x50 C24

B2.2

New trimmer span = 3.9m

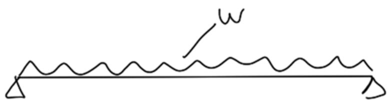


	Dead	Live
$w_D = 0.8 \times \frac{1.2}{2} + 0.8 \times 1.1 =$	1.4kN/m	
$w_L = 1.5 \times \frac{1.2}{2} =$		0.9kN/m

3No 200x50 C24

B2.3

New floor beam supports partition span = 2.9m



	Dead	Live
$w_D = 0.8 \times 0.4 + 0.5 \times 3 =$	1.82kN/m	
$w_L = 1.5 \times 0.4 =$		0.6kN/m

2No 200x50 C24

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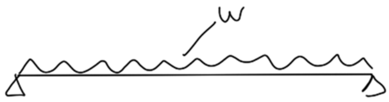
B2.4

Floor beam span = 5.8m

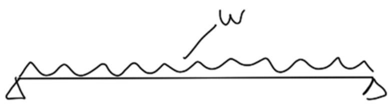


	Dead	Live
$w_D = 1.6 \times \frac{6.5}{2} =$	5.2kN/m	
$w_L = 1.5 \times \frac{6.5}{2} =$		4.9kN/m

203UC52

B2.5

Lintel supports new wall span = 5m



	Dead	Live
$w_D = 6.6 \times 3.5 =$	23kN/m	

2No IG HD BOX 140

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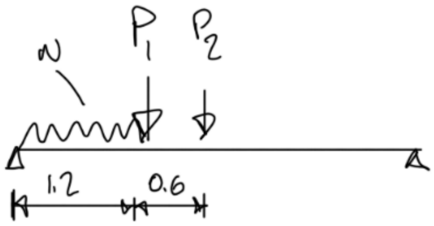
B2.6

Lintel supports new wall and reaction from B2.4 and B3.4 span = 5.7m



	Dead	Live
$w_D = 6.6 \times 3.5 =$	23kN/m	
$P_{1D} =$	22.7kN	
$P_{1L} =$		16.3kN
B3.4		
$P_{2D} =$	16.6kN	
$P_{2L} =$		14.2kN
B2.4		

203UC86

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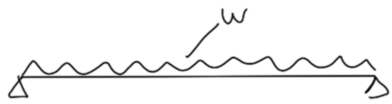
First floor Load Takedown

B1.1

Beam supports wall, glass roof, 1st 2nd and roof span = 6m

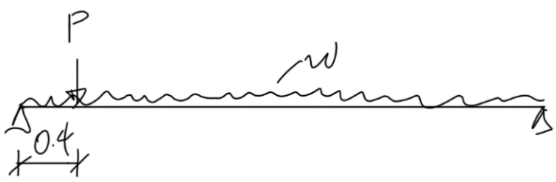


	Dead	Live
$w_D = 6.5 \times 5.5 \times 0.85 + 0.8 \times \frac{1.6}{2} + 0.5 \times \frac{2.8}{2} \times 2 + 1.6 \times \frac{2.8}{2} =$	34kN/m	
$w_L = 0.75 \times \frac{1.6}{2} + 1.5 \times \frac{2.8}{2} \times 2 + 1.5 \times \frac{2.8}{2} =$		7kN/m

457x191x67UB + 300x100 PFC to top flange

B1.2

Beam supports glass roof and B1.4 span = 5.1m



	Dead	Live
$w_D = 0.8 \times \frac{1.6}{2} =$	0.64kN/m	
$w_L = 0.75 \times \frac{1.6}{2} =$		0.6kN/m
$P_D =$	131kN	
$P_L =$		31kN

200x100x12.5 RHS

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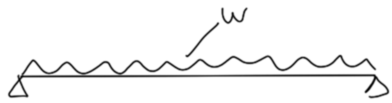
J1.1

Comflor span = 3.1m



	Dead	Live
$w_D =$	4.4kN/m^2	
$w_L =$		2.5kN/m^2

150 slab Comflor 60 0.9mm gauge + A193 top

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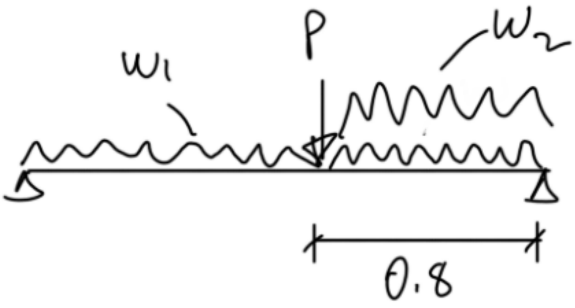
B1.3

Beam supports wall, 1st, B2.6 span = 2m



	Dead	Live
$w_{1D} = 4.4 \times \frac{3.1}{2} =$	6.9kN/m	
$w_{1L} = 2.5 \times \frac{3.1}{2} =$		3.9kN/m
$w_{2D} = 6.5 \times 5.5 =$	36.3kN/m	
$P_D =$	16kN	
$P_L =$		8kN

203UC46

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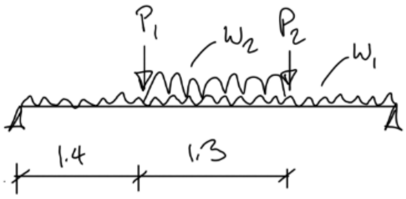
B1.4

Beam supports wall, 1st B1.1, B2.6 span = 4.2m



	Dead	Live
$w_{1D} = 4.4 \times \frac{3.1}{2} =$	6.9kN/m	
$w_{1L} = 2.5 \times \frac{3.1}{2} =$		3.9kN/m
$w_{2D} = 6.5 \times 5.5 =$	36.3kN/m	
$P_{1D} = 34 \times \frac{6}{2} =$	102kN	
$P_{1L} = 7 \times \frac{6}{2} =$		21kN
$P_{2D} =$	57kN	
$P_{2L} =$		23kN

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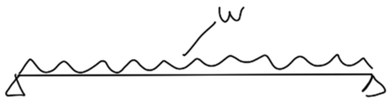
B1.5

Beam supports floor and masonry span = 5.1m

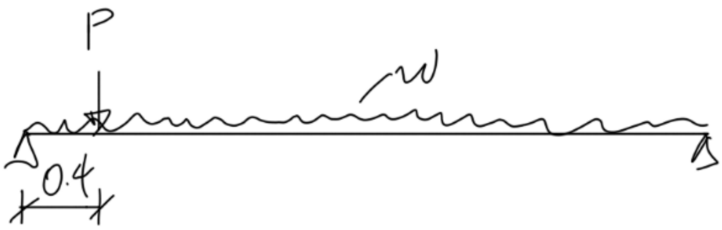


	Dead	Live
$w_D = 4.4 \times \frac{3.1}{2} + 4.8 \times 0.45 =$	9kN/m	
$w_L = 2.5 \times \frac{3.1}{2} =$		4kN/m

250x150x10 RHS

B1.6

Floor beam supports B1.4 and B1.3span = 6m



	Dead	Live
$w_D = 1.6 \times \frac{6.5}{2} =$	5.2kN/m	
$w_L = 1.5 \times \frac{6.5}{2} =$		4.9kN/m
$P_D = 112 + 20 =$	132kN	
$P_L = 30 + 7.1 =$		37kN

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C1.1
Post supports B1.1

	Dead	Live
$P_{1D} = 34 \times \frac{6}{2} =$	102kN	
$P_{1L} = 7 \times \frac{6}{2} =$		21kN

160x80x10 RHS

C1.2
Post supports B1.6

	Dead	Live
$P_D =$	141kN	
$P_L =$		50kN

139.7x8 CHS

C1.3
Post supports B1.5

	Dead	Live
$P_D = 25 + 4.8 \times 0.45 \times \frac{2.2}{2}$	28kN	
$P_L =$		11kN

76.1x6.3 CHS

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C1.4

Post supports B1.2

	Dead	Live
$P_D =$	125kN	
$P_L =$		30kN

100x100x8 SHS

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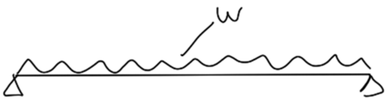
GROUND FLOOR LOAD TAKEDOWN

J0.1

Comflor span = 4.6m



	Dead	Live
$w_D =$	4.4kN/m^2	
$w_L =$		2.5kN/m^2

Tata Comflor 60 1.2mm gauge 150 slab A393 top H8 in trough single row of props in temporary condition

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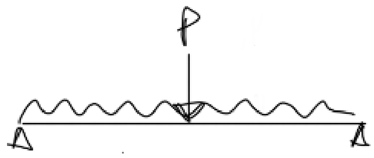
B0.1

Beam supports C1.2 and floor span = 3.6m



	Dead	Live
$w_D = 4.4 \times \frac{6.5}{2} =$	15kN/m	
$w_L = 1.5 \times \frac{6.5}{2} =$		5kN/m
$P_D =$	141kN	
$P_L =$		50kN

203UC86 S355

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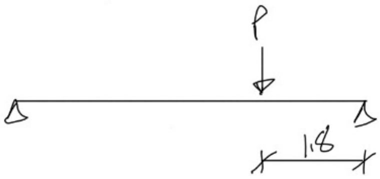
B0.2

Beam supports B0.9 span = 5m



	Dead	Live
P_{2D} =	20N	
P_{2L} =		10kN

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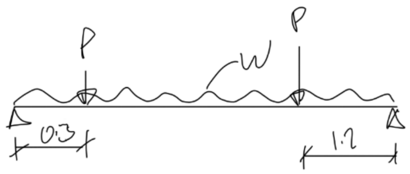
B0.3

Beam B0.6 and B0.9 span = 4.7m



	Dead	Live
P_{1D} =	45N	
P_{1L} =		15kN
P_{2D} =	20N	
P_{2L} =		10kN

152UC30

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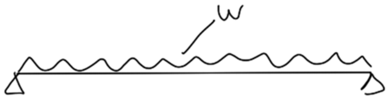
B0.5

Beam supports floorspan = 2.5m



	Dead	Live
$w_D = 4.4 \times \frac{6.5}{2} =$	15kN/m	
$w_L = 1.5 \times \frac{6.5}{2} =$		5kN/m

152UC23

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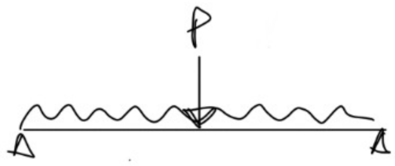
B0.6

Beam supports floor and B1.3 span = 3.5m

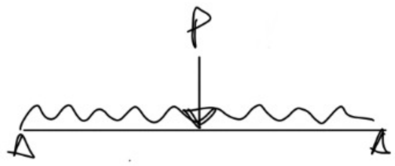


	Dead	Live
$w_D = 4.4 \times \frac{4.5}{2} =$	10kN/m	
$w_L = 1.5 \times \frac{4.5}{2} =$		3.5kN/m
$P_D =$	45N	
$P_L =$		15kN

203UC52

B0.7

Beam supports braced frame span = 3.1m



	Dead	Live
$P_w = C_p \times q_s \times A = 1.2 \times 0.41 \times \frac{5.7}{2} \times 7.2 =$	10.1kN	
$P = C_p \times q_s \times A = 10.1 \times \frac{2.9}{0.95} =$		30kN
$P_{0.5\%} = \frac{1}{200} \times \left(1.35 \times \left(10 \times \frac{11}{2} \times \frac{6}{2}\right) \times 3 + 1.5 \times \left(11.5 \times \frac{11}{2} \times \frac{6}{2}\right) \times 3\right) =$		4kN

152UC30

C0.1

Post supports B0.1 and B0.5

	Dead	Live
$P_D = 100 + 20 =$	120kN	
$P_L = 40 + 10 =$		50kN

100x100x10 RHS

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C0.2

Post supports B0.1

	Dead	Live
$P_D =$	100N	
$P_L =$		40kN

100x100x10 RHS

C0.3

Post supports B0.5, B0.7, B0.2 and B1.2

	Dead	Live
$P_D = 20 + 8 + 10 + 130$	170N	
$P_L = 10 + 17 + 5 + 30$		62kN

150x100x10 RHS

C0.4

Post supports B0.6

	Dead	Live
$P_D =$	45N	
$P_L =$		20kN

100x100x10 RHS

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C0.5
Post supports B0.2 and B1.2

	Dead	Live
$P_D = 15 + 15 =$	30N	
$P_L = 5 + 8 =$		13kN

100x100x10 RHS

C0.6
Post supports B0.3 and B0.10

	Dead	Live
$P_D = 20 + 35 =$	55N	
$P_L = 10 + 15 =$		25kN

88.9x6.3CHS

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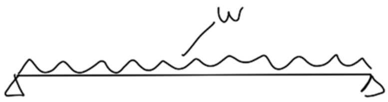
B0.8

Waler to top of lining wall span = 5.6m



Prop force from retaining wall design

$w_{ULS} =$ 15.7kN/m

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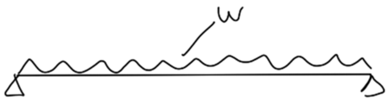
B0.9

Beam supports floor span = 4.8m

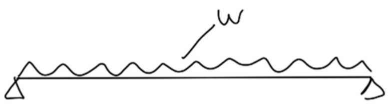


	Dead	Live
$w_D = 4.4 \times \frac{3.5}{2} =$	7.7kN/m	
$w_L = 1.5 \times \frac{3.5}{2} =$		2.7kN/m

152UC37

B0.10

Beam supports floor span = 5m



	Dead	Live
$w_D = 4.4 \times \frac{5.5}{2} =$	12.1kN/m	
$w_L = 1.5 \times \frac{5.5}{2} =$		4.2kN/m

152UC37

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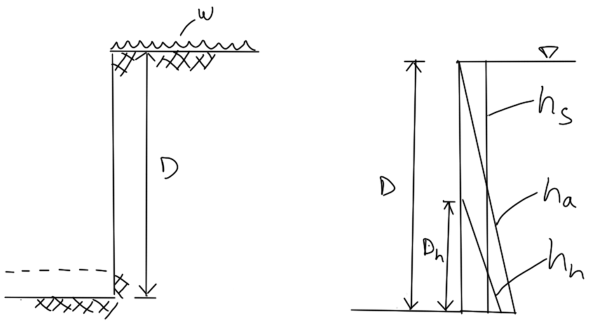
BASEMENT LOAD TAKEDOWN

RW.1

Lining RC wall to underpins D = 3.4m



Active pressure coefficient

$K_a = 0.48$

Density of earth

$\rho = 18\mathrm{kN/m^3}$

150THK RC

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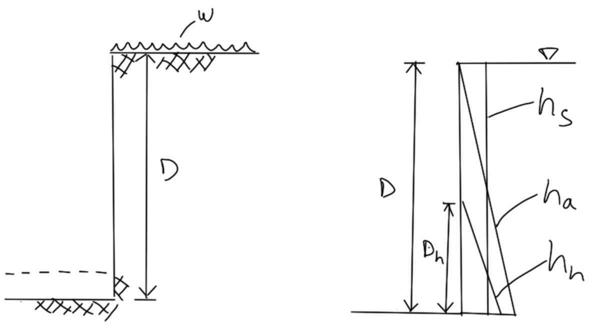
RW.2

Retaining wall adjacent to highway D = 3.6m



Active pressure coefficient

$K_a = 0.48$

Density of earth

$\rho = 18\text{kN/m}^3$

350THK RC

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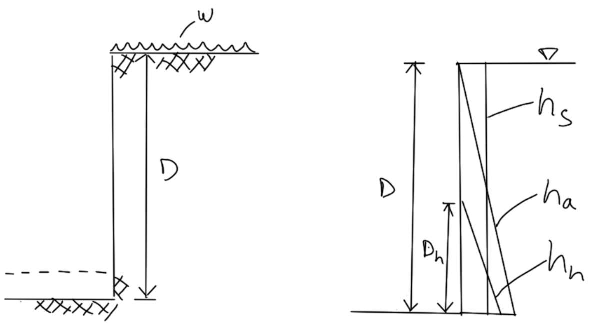
RW.3

Retaining wall adjacent to garden D = 3.4m



Active pressure coefficient

$K_a = 0.48$

Density of earth

$\rho = 18\text{kN/m}^3$

250THK RC

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S0.1

Basement slab supports point loads

	Dead	Live
C0.5		
$P_{1D} = 15 + 15 =$	30N	
$P_{1L} = 5 + 8 =$		13kN
C0.3		
$P_{2D} = 20 + 8 + 10 + 130$	170N	
$P_{2L} = 10 + 17 + 5 + 30$		62kN
C0.1		
$P_{3D} = 100 + 20 =$	120kN	
$P_{3L} = 40 + 10 =$		50kN
C0.2		
$P_{4D} =$	100N	
$P_{4L} =$		40kN
C0.6		
$P_{5D} = 20 + 35 =$	55N	
$P_{5L} = 10 + 15 =$		25kN
B1.1		
$P_{6D} =$	110N	
$P_{6L} =$		25kN
B0.10		
$P_{7D} =$	32N	
$P_{7L} =$		12kN

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	Dead	Live
B0.6		
$P_{8D} =$	45N	
$P_{8L} =$		15kN
B1.5		
$P_{9D} =$	35N	
$P_{9L} =$		15kN
B1.5+B0.7		
$P_{10D} = 35 + 8 =$	43N	
$P_{10L} = 15 + 20$		35kN

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PW0.1 existing

Party wall between 52 Eton avenue and 50 Eton Avenue existing load takedown

	Dead	Live
$w_D = 7.5 \times \frac{5.5}{2} + 7.5 \times \frac{5.5}{2} + 0.5 \times \frac{10}{2} + 4.6 \times 11.3 =$	95.7kN/m	
Ground First Second 215 wall		
$w_L = 1.5 \times \frac{5.5}{2} + 1.5 \times \frac{5.5}{2} + 1.5 \times \frac{10}{2} =$		16kN/m

PW0.1 proposed

Party wall between 52 Eton avenue and 50 Eton Avenue new load takedown

	Dead	Live
$w_D = 7.5 \times \frac{2.75}{2} + 0.5 \times \frac{5.7}{2} + 0.5 \times \frac{7.5}{2} + 0.8 \times \frac{2.7}{2} + 4.6 \times 11.3 + 0.4 \times 1.7 \times 24 =$		
Ground First Second Roof 215 wall Underpin		
	83kN/m	
$w_L = 1.5 \times \frac{2.75}{2} + 1.5 \times \frac{5.7}{2} + 1.5 \times \frac{7.5}{2} + 0.75 \times \frac{2.7}{2} =$		13kN/m

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PW0.2 existing

Party wall between 52 Eton avenue and 30 Crossfield road existing load takedown

	Dead	Live
$w_D = 7.5 \times \frac{6}{2} + 7.5 \times \frac{6}{2} + 0.5 \times \frac{6}{2} + 0.8 \times \frac{12}{2} + 6.6 \times 7.4 + 4.6 \times 3.2 =$	115kN/m	
Ground First Second Roof 330 wall 215 wall		
Floor loads are allowance from AO		
$w_L = 1.5 \times \frac{6}{2} + 1.5 \times \frac{6}{2} + 1.5 \times \frac{6}{2} + 0.75 \times \frac{6}{2} =$		18kN/m

PW0.2 proposed upper bound

Party wall between Eton avenue and 30 Crossfield road new load takedown

	Dead	Live
$w_D = 7.5 \times \frac{6}{2} + 0.5 \times \frac{5.5}{2} \times \frac{6}{9} + 7.5 \times \frac{6}{2} + 0.5 \times \frac{5.5}{2} \times \frac{6}{9} + 0.5 \times \frac{6}{2} + 0.8 \times \frac{12}{2} + 6.6 \times 7.4 + 4.6 \times 3.2 + 0.4 \times 1.7 \times 24 =$	133kN/m	
Ground New first First New second Second Roof 330 wall 215 wall Underpin		
$w_L = 1.5 \times \frac{6}{2} + 1.5 \times \frac{5.5}{2} \times \frac{6}{9} + 1.5 \times \frac{6}{2} + 1.5 \times \frac{5.5}{2} \times \frac{6}{9} + 1.5 \times \frac{6}{2} + 0.75 \times \frac{12}{2} =$		24kN/m

$$Increase = \frac{133 + 24}{115 + 18} - 1 = 18\%$$

Considered okay

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PW0.2 proposed lower bound

Party wall between Eton avenue and 30 Crossfield road new load takedown

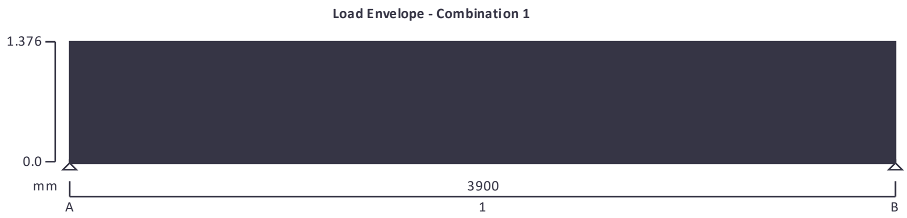
	Dead	Live
$w_D = 0.5 \times \frac{5.5}{2} \times \frac{6}{9} + 0.5 \times \frac{5.5}{2} \times \frac{6}{9} + 0.8 \times \frac{6}{2} + 6.6 \times 7.4 + 4.6 \times 3.2 + 0.4 \times 1.7 \times 24 =$	85kN/m	
New first New second Second Roof 330 wall		
215 wall Underpin		
$w_L = 1.5 \times \frac{5.5}{2} \times \frac{6}{9} + 1.5 \times \frac{5.5}{2} \times \frac{6}{9} + 0.75 \times \frac{6}{2} =$		8kN/m

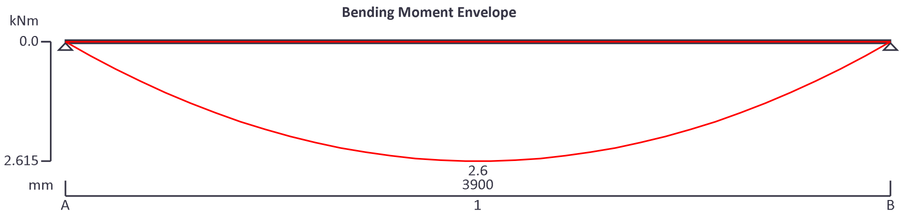
Magnitude of loads equivalent to PW01 therefore considered okay

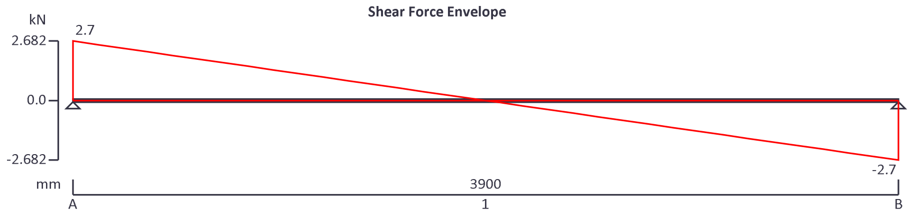
Roof Member Design

J3.1 Design

Timber beam analysis & design to EN1995-1-1:2004
In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1
Tedds calculation version 1.7.02







Applied loading
Beam loads
Permanent self weight of beam × 1
Permanent full UDL 0.320 kN/m
Variable full UDL 0.600 kN/m
Load combinations
Load combination 1 Support A Permanent × 1.35
Variable × 1.50
Span 1 Permanent × 1.35
Variable × 1.50
Support B Permanent × 1.35
Variable × 1.50

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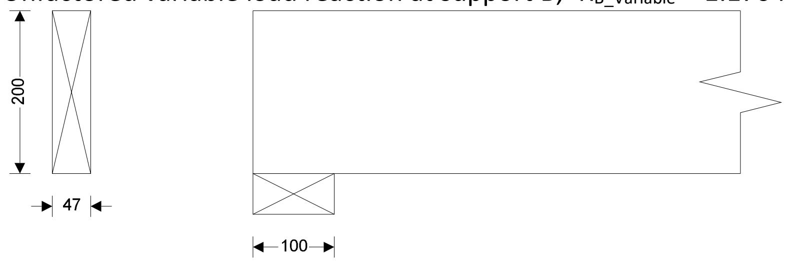
Analysis results
Design moment; $M = 2.615$ kNm; Design shear; $F = 2.682$ kN
Total load on member; $W_{tot} = 5.365$ kN
Reactions at support A; $R_{A\_max} = 2.682$ kN; $R_{A\_min} = 2.682$ kN
Unfactored permanent load reaction at support A; $R_{A\_Permanent} = 0.687$ kN
Unfactored variable load reaction at support A; $R_{A\_Variable} = 1.170$ kN
Reactions at support B; $R_{B\_max} = 2.682$ kN; $R_{B\_min} = 2.682$ kN
Unfactored permanent load reaction at support B; $R_{B\_Permanent} = 0.687$ kN
Unfactored variable load reaction at support B; $R_{B\_Variable} = 1.170$ kN



Timber section details
Breadth of section; $b = 47$ mm; Depth of section; $h = 200$ mm
Number of sections; $N = 1$; Breadth of member; $b_b = 47$ mm
Timber strength class; C24
Member details
Service class of timber; 2; Load duration; Long-term
Length of span; $L_{s1} = 3900$ mm
Length of bearing; $L_b = 100$ mm
In accordance with cl.6.6 the member is one of several similar and equally spaced members laterally connected by a continuous load distribution system capable of transferring loads from one member to the neighboring members.
Compression perpendicular to grain - cl.6.1.4
Design compressive stress; $\sigma_{c.90.d} = 0.571$ N/mm^2; Design compressive strength; $f_{c.90.d} =$ 1.481 N/mm^2
PASS - Design compressive strength exceeds design compressive stress at bearing
Bending - cl 6.1.6
Design bending stress; $\sigma_{m.d} = 8.347$ N/mm^2; Design bending strength; $f_{m.d} = 8.611$ N/mm^2
PASS - Design bending strength exceeds design bending stress
Shear - cl.6.1.7
Applied shear stress; $\tau_d = 0.639$ N/mm^2; Permissible shear stress; $f_{v.d} = 2.369$ N/mm^2
PASS - Design shear strength exceeds design shear stress
Deflection - cl.7.2
Deflection limit; $\delta_{lim} = 14.000$ mm; Total final deflection; $\delta_{fin} = 12.529$ mm
PASS - Total final deflection is less than the deflection limit

Therefore use 200x50 C24@400

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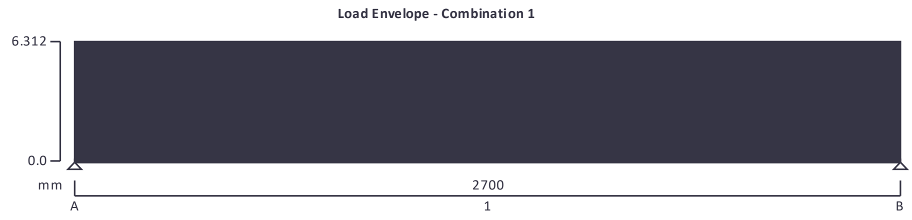
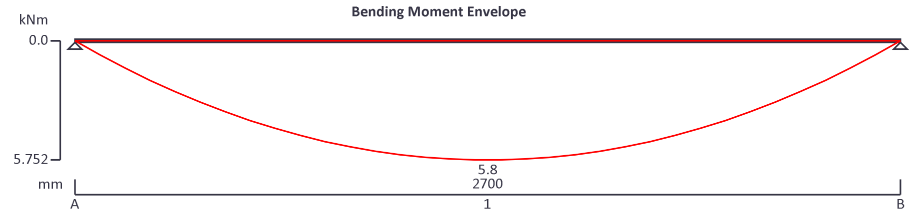
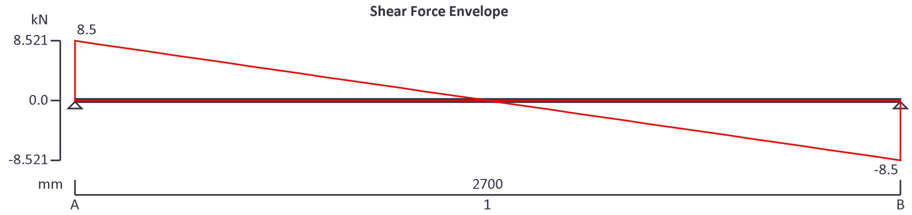
B3.1 Design

Timber beam analysis & design to EN1995-1-1:2004
In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1
Tedds calculation version 1.7.02







Applied loading
Beam loads
Permanent self weight of beam × 1
Permanent full UDL 2.500 kN/m
Variable full UDL 1.900 kN/m
Load combinations
Load combination 1 Support A Permanent × 1.35
Variable × 1.50
Span 1 Permanent × 1.35
Variable × 1.50
Support B Permanent × 1.35
Variable × 1.50
Analysis results
Design moment; M = 5.752 kNm; Design shear; F = 8.521 kN
Total load on member; W_{tot} = 17.043 kN

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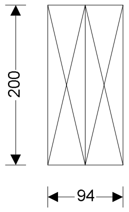
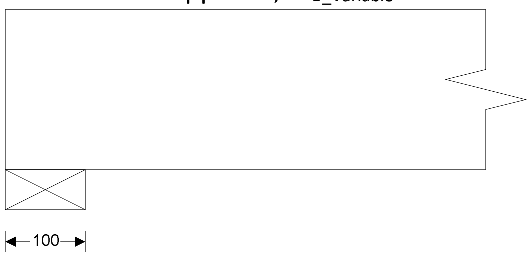
Reactions at support A; R_{A_max} = 8.521 kN; R_{A_min} = 8.521 kN
Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = 3.462 kN
Unfactored variable load reaction at support A; $R_{A\_Variable}$ = 2.565 kN
Reactions at support B; R_{B_max} = 8.521 kN; R_{B_min} = 8.521 kN
Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = 3.462 kN
Unfactored variable load reaction at support B; $R_{B\_Variable}$ = 2.565 kN



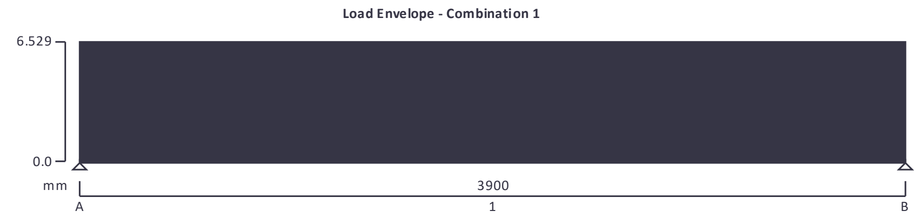


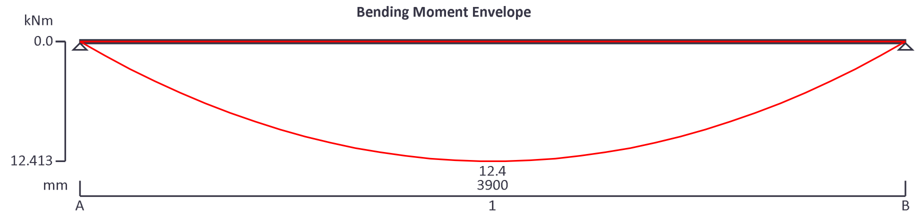
Timber section details
Breadth of section; b = 47 mm; Depth of section; h = 200 mm
Number of sections; N = 2; Breadth of member; b_b = 94 mm
Timber strength class; C24
Member details
Service class of timber; 2; Load duration; Long-term
Length of span; L_{s1} = 2700 mm
Length of bearing; L_b = 100 mm
Compression perpendicular to grain - cl.6.1.4
Design compressive stress; $\sigma_{c.90.d}$ = 0.907 N/mm^2; Design compressive strength; $f_{c.90.d}$ = 1.346 N/mm^2
PASS - Design compressive strength exceeds design compressive stress at bearing
Bending - cl 6.1.6
Design bending stress; $\sigma_{m.d}$ = 9.179 N/mm^2; Design bending strength; $f_{m.d}$ = 12.923 N/mm^2
PASS - Design bending strength exceeds design bending stress
Shear - cl.6.1.7
Applied shear stress; τ_d = 1.015 N/mm^2; Permissible shear stress; $f_{v.d}$ = 2.154 N/mm^2
PASS - Design shear strength exceeds design shear stress
Deflection - cl.7.2
Deflection limit; δ_{lim} = 10.800 mm; Total final deflection; δ_{fin} = 7.587 mm
PASS - Total final deflection is less than the deflection limit

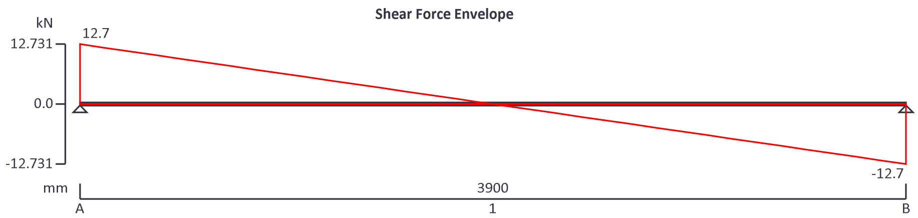
Therefore use 2No 200x50 C24

B3.2 Design

Steel beam analysis & design (EN1993-1-1:2005)
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex
TEDDS calculation version 3.0.13

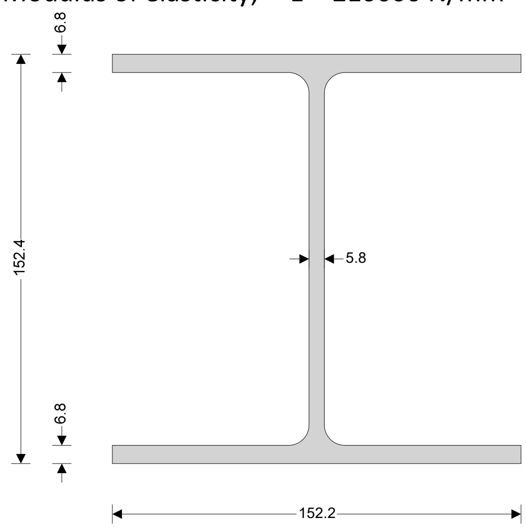






Support conditions
Support A Vertically restrained
Rotationally free
Support B Vertically restrained
Rotationally free
Applied loading
Beam loads Permanent self weight of beam × 1
Permanent full UDL 2.5 kN/m
Variable full UDL 1.9 kN/m
Load combinations
Load combination 1 Support A Permanent × 1.35
Variable × 1.50
Permanent × 1.35
Variable × 1.50
Support B Permanent × 1.35

Variable × 1.50
Analysis results
Maximum moment; M_{max} = 12.4 kNm; M_{min} = 0 kNm
Maximum shear; V_{max} = 12.7 kN; V_{min} = -12.7 kN
Deflection; δ_{max} = 5.3 mm; δ_{min} = 0 mm
Maximum reaction at support A; R_{A_max} = 12.7 kN; R_{A_min} = 12.7 kN
Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = 5.3 kN
Unfactored variable load reaction at support A; $R_{A\_Variable}$ = 3.7 kN
Maximum reaction at support B; R_{B_max} = 12.7 kN; R_{B_min} = 12.7 kN
Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = 5.3 kN
Unfactored variable load reaction at support B; $R_{B\_Variable}$ = 3.7 kN
Section details
Section type; UC 152x152x23 (BS4-1)
Steel grade; S275
EN 10025-2:2004 - Hot rolled products of structural steels
Nominal thickness of element; $t = \max(t_f, t_w)$ = 6.8 mm
Nominal yield strength; f_y = 275 N/mm^2
Nominal ultimate tensile strength; f_u = 410 N/mm^2
Modulus of elasticity; E = 210000 N/mm^2



Partial factors - Section 6.1
Resistance of cross-sections; γ_{M0} = 1.00
Resistance of members to instability; γ_{M1} = 1.00
Resistance of tensile members to fracture; γ_{M2} = 1.10
Lateral restraint
Span 1 has lateral restraint at supports only
Effective length factors
Effective length factor in major axis; K_y = 1.000
Effective length factor in minor axis; K_z = 1.000
Effective length factor for torsion; $K_{LT.A}$ = 1.400; + 2 × h
$K_{LT.B}$ = 1.400; + 2 × h
Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.92$
Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)
Width of section; $c = d = 123.6$ mm
$c / t_w = 23.1 \times \varepsilon <= 72 \times \varepsilon$; Class 1
Outstand flanges - Table 5.2 (sheet 2 of 3)
Width of section; $c = (b - t_w - 2 \times r) / 2 = 65.6$ mm
$c / t_f = 10.4 \times \varepsilon <= 14 \times \varepsilon$; Class 3
Section is class 3
Check shear - Section 6.2.6
Height of web; $h_w = h - 2 \times t_f = 138.8$ mm
Shear area factor; $\eta = 1.000$
$h_w / t_w < 72 \times \varepsilon / \eta$
Shear buckling resistance can be ignored
Design shear force; $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = 12.7$ kN
Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 997 \text{ mm}^2$
Design shear resistance - cl 6.2.6(2); $V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = 158.4$ kN
PASS - Design shear resistance exceeds design shear force
Check bending moment major (y-y) axis - Section 6.2.5
Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = 12.4$ kNm
Design bending resistance moment - eq 6.14; $M_{c,Rd} = M_{el,Rd} = W_{el.y} \times f_y / \gamma_{M0} = 45.1$ kNm
Slenderness ratio for lateral torsional buckling
Correction factor - Table 6.6; $k_c = 1$
$C_1 = 1 / k_c^2 = 1$
Destabilised load condition factor; $D = 1.5$
Curvature factor; $g = \sqrt{[1 - (I_z / I_y)]} = 0.825$
Poissons ratio; $\nu = 0.3$
Shear modulus; $G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$
Unrestrained length; $L = 1.4 \times L_{s1} + 2 \times h = 5765$ mm
Elastic critical buckling moment;$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = 43.1 kNm
Slenderness ratio for lateral torsional buckling; $\overline{\lambda}_{LT} = D \times \sqrt{(W_{el.y} \times f_y / M_{cr})} = 1.534$
Limiting slenderness ratio; $\overline{\lambda}_{LT,0} = 0.4$
$\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored
Design resistance for buckling - Section 6.3.2.1
Buckling curve - Table 6.5; b
Imperfection factor - Table 6.3; $\alpha_{LT} = 0.34$
Correction factor for rolled sections; $\beta = 0.75$
LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}) + \beta \times \overline{\lambda}_{LT}^2] = 1.576$
LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \overline{\lambda}_{LT}^2)}], 1, 1 / \overline{\lambda}_{LT}^2) = 0.413$
Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\overline{\lambda}_{LT} - 0.8)^2], 1) = 1.000$
Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.413$
Design buckling resistance moment - eq 6.55; $M_{b,Rd} = \chi_{LT,mod} \times W_{el.y} \times f_y / \gamma_{M1} = 18.6$ kNm
PASS - Design buckling resistance moment exceeds design bending moment
Check vertical deflection - Section 7.2.1
Consider deflection due to permanent and variable loads
Limiting deflection; $\delta_{lim} = L_{s1} / 250 = 15.6$ mm

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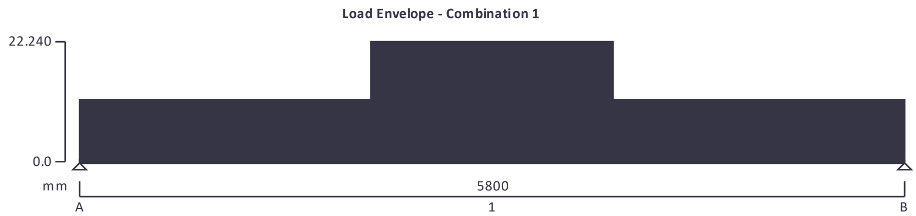
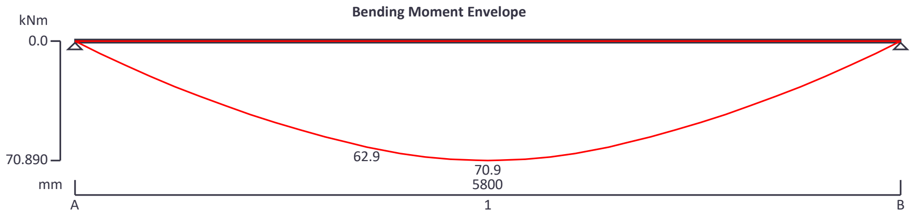
Maximum deflection span 1; δ = max(abs(δ_{max}), abs(δ_{min})) = 5.308 mm
PASS - Maximum deflection does not exceed deflection limit

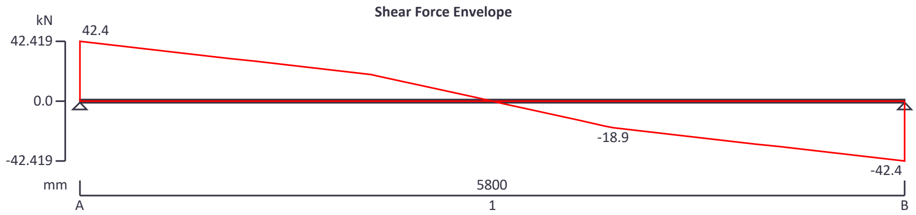
Therefore use 152UC23

B3.3 Design

Steel beam analysis & design (EN1993-1-1:2005)
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex
TEDDS calculation version 3.0.13







Support conditions
Support A Vertically restrained
Rotationally free
Support B Vertically restrained
Rotationally free
Applied loading
Beam loads Permanent self weight of beam × 1
Permanent full UDL 2.6 kN/m
Variable full UDL 4.9 kN/m
Permanent partial UDL 5.2 kN/m from 2050 mm to 3750 mm
Variable partial UDL 2.5 kN/m from 2050 mm to 3750 mm
Load combinations
Load combination 1 Support A Permanent × 1.35
Variable × 1.50
Permanent × 1.35

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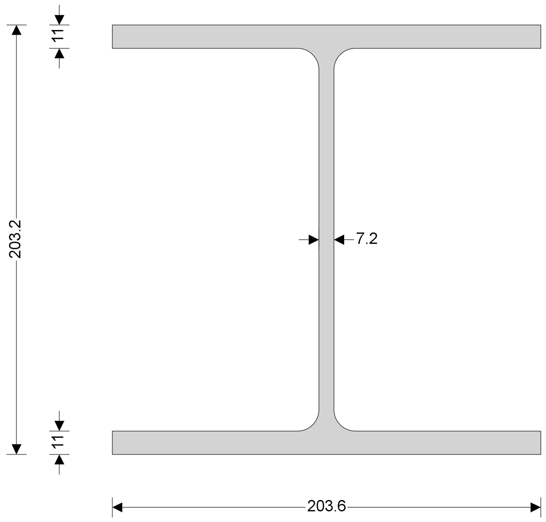
Variable × 1.50
Support B Permanent × 1.35
Variable × 1.50

Analysis results
Maximum moment; M_{max} = 70.9 kNm; M_{min} = 0 kNm
Maximum moment span 1 segment 1; $M_{s1\_seg1\_max}$ = 60.6 kNm; $M_{s1\_seg1\_min}$ = 0 kNm
Maximum moment span 1 segment 2; $M_{s1\_seg2\_max}$ = 70.9 kNm; $M_{s1\_seg2\_min}$ = 0 kNm
Maximum moment span 1 segment 3; $M_{s1\_seg3\_max}$ = 60.6 kNm; $M_{s1\_seg3\_min}$ = 0 kNm
Maximum shear; V_{max} = 42.4 kN; V_{min} = -42.4 kN
Maximum shear span 1 segment 1; $V_{s1\_seg1\_max}$ = 42.4 kN; $V_{s1\_seg1\_min}$ = 0 kN
Maximum shear span 1 segment 2; $V_{s1\_seg2\_max}$ = 20.2 kN; $V_{s1\_seg2\_min}$ = -20.2 kN
Maximum shear span 1 segment 3; $V_{s1\_seg3\_max}$ = 0 kN; $V_{s1\_seg3\_min}$ = -42.4 kN
Deflection segment 4; δ_{max} = 17.5 mm; δ_{min} = 0 mm
Maximum reaction at support A; R_{A_max} = 42.4 kN; R_{A_min} = 42.4 kN
Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = 13.3 kN
Unfactored variable load reaction at support A; $R_{A\_Variable}$ = 16.3 kN
Maximum reaction at support B; R_{B_max} = 42.4 kN; R_{B_min} = 42.4 kN
Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = 13.3 kN
Unfactored variable load reaction at support B; $R_{B\_Variable}$ = 16.3 kN

Section details
Section type; UKC 203x203x46 (Tata Steel Advance)
Steel grade; S275
EN 10025-2:2004 - Hot rolled products of structural steels
Nominal thickness of element; $t = \max(t_f, t_w)$ = 11.0 mm
Nominal yield strength; f_y = 275 N/mm^2
Nominal ultimate tensile strength; f_u = 410 N/mm^2
Modulus of elasticity; E = 210000 N/mm^2



Partial factors - Section 6.1
Resistance of cross-sections; γ_{M0} = 1.00
Resistance of members to instability; γ_{M1} = 1.00
Resistance of tensile members to fracture; γ_{M2} = 1.10

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Lateral restraint

Span 1 has lateral restraint at supports plus third points

Effective length factors

Effective length factor in major axis; $K_y = 1.000$

Effective length factor in minor axis; $K_z = 1.000$

Effective length factor for torsion; $K_{LT.A} = 1.400; + 2 \times h$

$K_{LT.B} = 1.400; + 2 \times h$

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.92$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section; $c = d = 160.8$ mm

$c / t_w = 24.2 \times \varepsilon <= 72 \times \varepsilon$; Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section; $c = (b - t_w - 2 \times r) / 2 = 88$ mm

$c / t_f = 8.7 \times \varepsilon <= 9 \times \varepsilon$; Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f = 181.2$ mm

Shear area factor; $\eta = 1.000$

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = 42.4$ kN

Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1698 \text{ mm}^2$

Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = 269.5$ kN

PASS - Design shear resistance exceeds design shear force

Check bending moment at span 1 segment 2 major (y-y) axis - Section 6.2.5

Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s1\_seg2\_max}), \text{abs}(M_{s1\_seg2\_min})) = 70.9$ kNm

Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = 136.8$ kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6; $k_c = 1$

$C_1 = 1 / k_c^2 = 1$

Destabilised load condition factor; $D = 1.5$

Curvature factor; $g = \sqrt{[1 - (I_z / I_y)]} = 0.813$

Poissons ratio; $\nu = 0.3$

Shear modulus; $G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$

Unrestrained length; $L = 1.2 \times L_{s1\_seg2} = 2320$ mm

Elastic critical buckling moment;$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = 811.1 kNm

Slenderness ratio for lateral torsional buckling; $\overline{\lambda}_{LT} = D \times \sqrt{(W_{pl.y} \times f_y / M_{cr})} = 0.616$

Limiting slenderness ratio; $\overline{\lambda}_{LT,0} = 0.4$

$\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5; b

Imperfection factor - Table 6.3; $\alpha_{LT} = 0.34$

Correction factor for rolled sections; $\beta = 0.75$

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}) + \beta \times \overline{\lambda}_{LT}^2] = 0.679$

LTB reduction factor - eq 6.57; χ_{LT} = min(1 / [ϕ_{LT} + $\sqrt{}$(ϕ_{LT}^2 - $\beta \times \bar{\lambda}_{LT}^2$)], 1, 1 / $\bar{\lambda}_{LT}^2$) = 0.910
Modification factor; f = min(1 - 0.5 $\times$ (1 - k_c)$\times$ [1 - 2 $\times$ ($\bar{\lambda}_{LT}$ - 0.8)^2], 1) = 1.000
Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod}$ = min(χ_{LT} / f, 1) = 0.910
Design buckling resistance moment - eq 6.55; $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = 124.5 kNm
PASS - Design buckling resistance moment exceeds design bending moment
Check vertical deflection - Section 7.2.1
Consider deflection due to permanent and variable loads
Limiting deflection; $\delta_{lim} = L_{s1}$ / 250 = 23.2 mm
Maximum deflection span 1; δ = max(abs(δ_{max}), abs(δ_{min})) = 17.541 mm
PASS - Maximum deflection does not exceed deflection limit

Therefore use 203UC46 + 500x100x215 padstone

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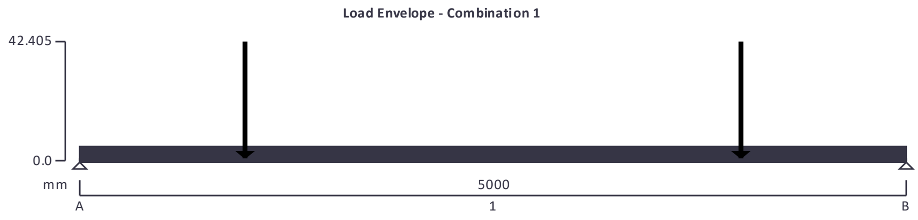
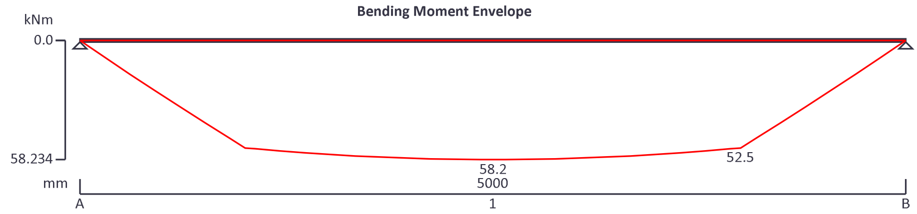
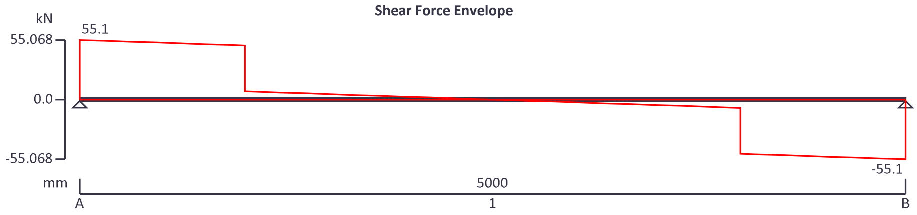
B3.4 Design

Steel beam analysis & design (EN1993-1-1:2005)
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex
TEDDS calculation version 3.0.13







Support conditions
Support A Vertically restrained
Rotationally free
Support B Vertically restrained
Rotationally free
Applied loading
Beam loads Permanent self weight of beam × 1
Permanent full UDL 3.3 kN/m
Permanent point load 13.3 kN at 1000 mm
Variable point load 16.3 kN at 1000 mm
Permanent point load 13.3 kN at 4000 mm
Variable point load 16.3 kN at 4000 mm
Load combinations
Load combination 1 Support A Permanent × 1.35
Variable × 1.50

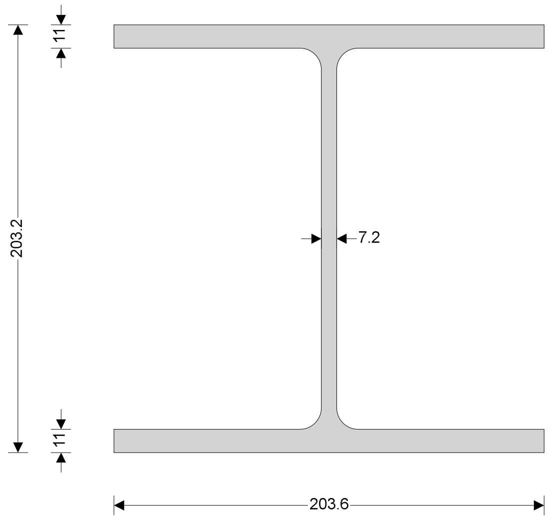
Permanent × 1.35
Variable × 1.50
Support B Permanent × 1.35
Variable × 1.50

Analysis results

Maximum moment; M_{max} = 58.2 kNm; M_{min} = 0 kNm
Maximum shear; V_{max} = 55.1 kN; V_{min} = -55.1 kN
Deflection; δ_{max} = 12.3 mm; δ_{min} = 0 mm
Maximum reaction at support A; R_{A_max} = 55.1 kN; R_{A_min} = 55.1 kN
Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = 22.7 kN
Unfactored variable load reaction at support A; $R_{A\_Variable}$ = 16.3 kN
Maximum reaction at support B; R_{B_max} = 55.1 kN; R_{B_min} = 55.1 kN
Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = 22.7 kN
Unfactored variable load reaction at support B; $R_{B\_Variable}$ = 16.3 kN

Section details

Section type; UKC 203x203x46 (Tata Steel Advance)
Steel grade; S275
EN 10025-2:2004 - Hot rolled products of structural steels
Nominal thickness of element; $t = \max(t_f, t_w)$ = 11.0 mm
Nominal yield strength; f_y = 275 N/mm^2
Nominal ultimate tensile strength; f_u = 410 N/mm^2
Modulus of elasticity; E = 210000 N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections; γ_{M0} = 1.00
Resistance of members to instability; γ_{M1} = 1.00
Resistance of tensile members to fracture; γ_{M2} = 1.10

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis; K_y = 1.000
Effective length factor in minor axis; K_z = 1.000

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Effective length factor for torsion; $K_{LT.A}$ = 1.400; + 2 × h
$K_{LT.B}$ = 1.400; + 2 × h
Classification of cross sections - Section 5.5
$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.92$
Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)
Width of section; c = d = 160.8 mm
$c / t_w = 24.2 \times \varepsilon <= 72 \times \varepsilon$; Class 1
Outstand flanges - Table 5.2 (sheet 2 of 3)
Width of section; $c = (b - t_w - 2 \times r) / 2 = 88$ mm
$c / t_f = 8.7 \times \varepsilon <= 9 \times \varepsilon$; Class 1
Section is class 1
Check shear - Section 6.2.6
Height of web; $h_w = h - 2 \times t_f = 181.2$ mm
Shear area factor; $\eta = 1.000$
$h_w / t_w < 72 \times \varepsilon / \eta$
Shear buckling resistance can be ignored
Design shear force; $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = 55.1$ kN
Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1698 \text{ mm}^2$
Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = 269.5$ kN
PASS - Design shear resistance exceeds design shear force
Check bending moment major (y-y) axis - Section 6.2.5
Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = 58.2$ kNm
Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = 136.8$ kNm
Slenderness ratio for lateral torsional buckling
Correction factor - Table 6.6; $k_c = 1$
$C_1 = 1 / k_c^2 = 1$
Destabilised load condition factor; D = 1.5
Curvature factor; $g = \sqrt{[1 - (I_z / I_y)]} = 0.813$
Poissons ratio; $\nu = 0.3$
Shear modulus; $G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$
Unrestrained length; $L = 1.4 \times L_{s1} + 2 \times h = 7406$ mm
Elastic critical buckling moment;$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = 143.6 kNm
Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = D \times \sqrt{(W_{pl.y} \times f_y / M_{cr})} = 1.464$
Limiting slenderness ratio; $\bar{\lambda}_{LT,0} = 0.4$
$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - *Lateral torsional buckling cannot be ignored*
Design resistance for buckling - Section 6.3.2.1
Buckling curve - Table 6.5; b
Imperfection factor - Table 6.3; $\alpha_{LT} = 0.34$
Correction factor for rolled sections; $\beta = 0.75$
LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 1.485$
LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.443$
Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 1.000$
Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.443$
Design buckling resistance moment - eq 6.55; $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = 60.6$ kNm
PASS - Design buckling resistance moment exceeds design bending moment

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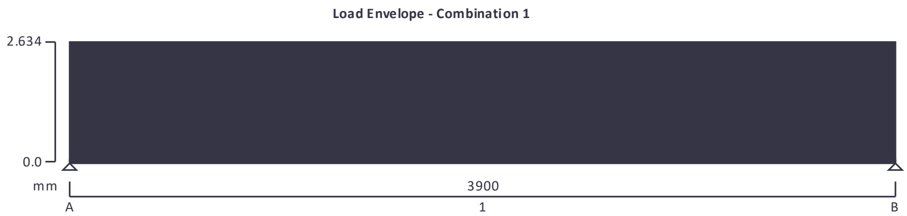
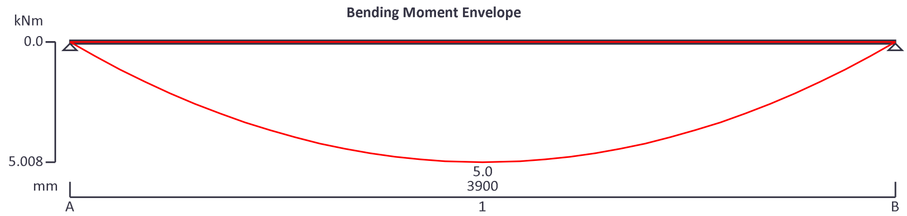
Check vertical deflection - Section 7.2.1
Consider deflection due to permanent and variable loads
Limiting deflection; $\delta_{lim} = L_{s1} / 250 = 20$ mm
Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 12.312$ mm
PASS - Maximum deflection does not exceed deflection limit

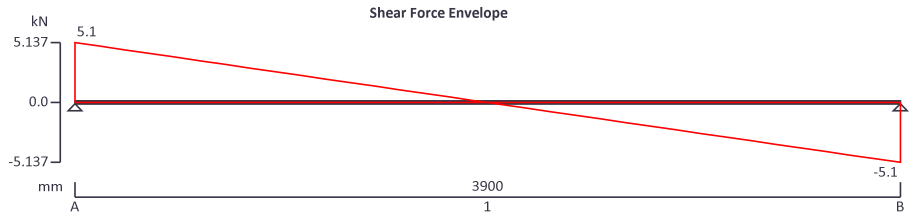
Therefore use 203UC46 +300 bearing

B3.5 Design

Timber beam analysis & design to EN1995-1-1:2004
In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1
Tedds calculation version 1.7.02







Applied loading
Beam loads
Permanent self weight of beam × 1
Permanent full UDL 0.600 kN/m
Variable full UDL 1.100 kN/m
Load combinations
Load combination 1 Support A Permanent × 1.35
Variable × 1.50
Span 1 Permanent × 1.35
Variable × 1.50
Support B Permanent × 1.35
Variable × 1.50
Analysis results
Design moment; M = 5.008 kNm; Design shear; F = 5.137 kN
Total load on member; W_{tot} = 10.273 kN

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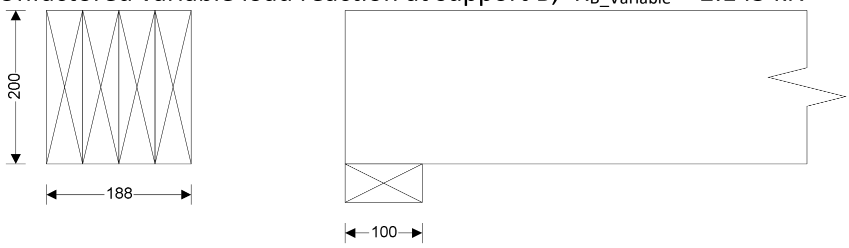
Reactions at support A; R_{A_max} = 5.137 kN; R_{A_min} = 5.137 kN
Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = 1.422 kN
Unfactored variable load reaction at support A; $R_{A\_Variable}$ = 2.145 kN
Reactions at support B; R_{B_max} = 5.137 kN; R_{B_min} = 5.137 kN
Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = 1.422 kN
Unfactored variable load reaction at support B; $R_{B\_Variable}$ = 2.145 kN



Timber section details
Breadth of section; b = 47 mm; Depth of section; h = 200 mm
Number of sections; N = 4; Breadth of member; b_b = 188 mm
Timber strength class; C24
Member details
Service class of timber; 2; Load duration; Long-term
Length of span; L_{s1} = 3900 mm
Length of bearing; L_b = 100 mm
Compression perpendicular to grain - cl.6.1.4
Design compressive stress; $\sigma_{c.90.d}$ = 0.273 N/mm^2; Design compressive strength; $f_{c.90.d}$ = 1.346 N/mm^2
PASS - Design compressive strength exceeds design compressive stress at bearing
Bending - cl 6.1.6
Design bending stress; $\sigma_{m.d}$ = 3.996 N/mm^2; Design bending strength; $f_{m.d}$ = 12.923 N/mm^2
PASS - Design bending strength exceeds design bending stress
Shear - cl.6.1.7
Applied shear stress; τ_d = 0.306 N/mm^2; Permissible shear stress; $f_{v.d}$ = 2.154 N/mm^2
PASS - Design shear strength exceeds design shear stress
Deflection - cl.7.2
Deflection limit; δ_{lim} = 14.000 mm; Total final deflection; δ_{fin} = 6.083 mm
PASS - Total final deflection is less than the deflection limit

Therefore use 4No 200x50 C24

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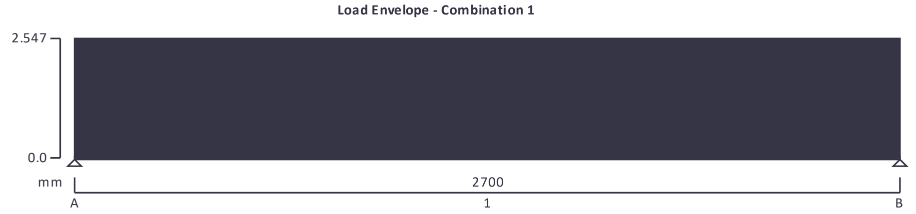
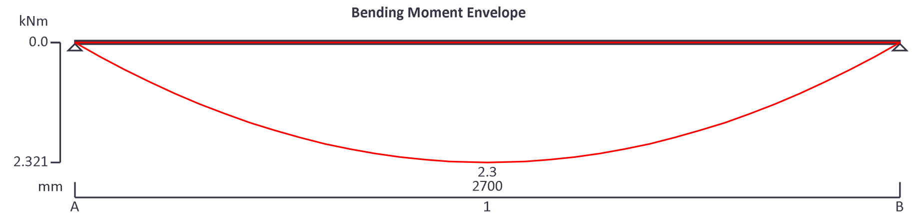
B3.6 Design

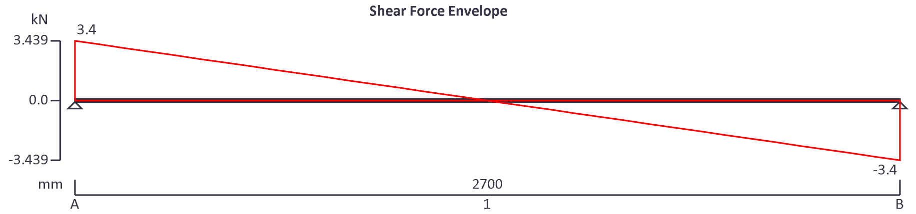
TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 1.7.02







Applied loading

Beam loads

Permanent self weight of beam × 1

Permanent full UDL 0.600 kN/m

Variable full UDL 1.100 kN/m

Load combinations

Load combination 1 Support A Permanent × 1.35

Variable × 1.50

Span 1 Permanent × 1.35

Variable × 1.50

Support B Permanent × 1.35

Variable × 1.50

Analysis results

Design moment; M = 2.321 kNm; Design shear; F = 3.439 kN

Total load on member; W_{tot} = 6.877 kN

Reactions at support A; R_{A_max} = 3.439 kN; R_{A_min} = 3.439 kN

Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = 0.897 kN

Unfactored variable load reaction at support A; $R_{A\_Variable}$ = 1.485 kN

Reactions at support B; R_{B_max} = 3.439 kN; R_{B_min} = 3.439 kN

Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = 0.897 kN

Unfactored variable load reaction at support B; $R_{B\_Variable}$ = 1.485 kN

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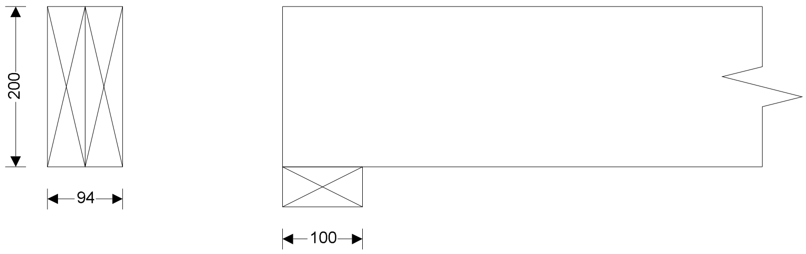

Timber section details
Breadth of section; $b = 47$ mm; Depth of section; $h = 200$ mm
Number of sections; $N = 2$; Breadth of member; $b_b = 94$ mm
Timber strength class; C24
Member details
Service class of timber; 2; Load duration; Long-term
Length of span; $L_{s1} = 2700$ mm
Length of bearing; $L_b = 100$ mm
Compression perpendicular to grain - cl.6.1.4
Design compressive stress; $\sigma_{c.90.d} = 0.366$ N/mm^2; Design compressive strength; $f_{c.90.d} = 1.346$ N/mm^2
PASS - Design compressive strength exceeds design compressive stress at bearing
Bending - cl 6.1.6
Design bending stress; $\sigma_{m.d} = 3.704$ N/mm^2; Design bending strength; $f_{m.d} = 12.923$ N/mm^2
PASS - Design bending strength exceeds design bending stress
Shear - cl.6.1.7
Applied shear stress; $\tau_d = 0.409$ N/mm^2; Permissible shear stress; $f_{v.d} = 2.154$ N/mm^2
PASS - Design shear strength exceeds design shear stress
Deflection - cl.7.2
Deflection limit; $\delta_{lim} = 10.800$ mm; Total final deflection; $\delta_{fin} = 2.786$ mm
PASS - Total final deflection is less than the deflection limit

Therefore use 2No 200x50 C24

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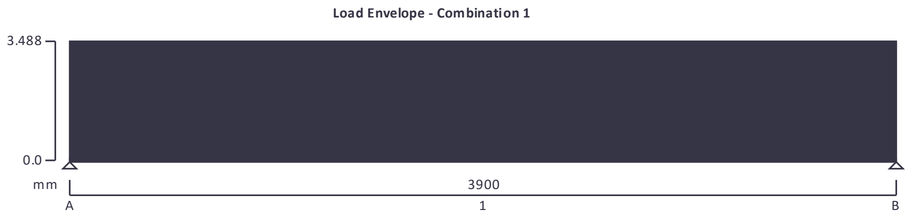
Second Floor Member Design

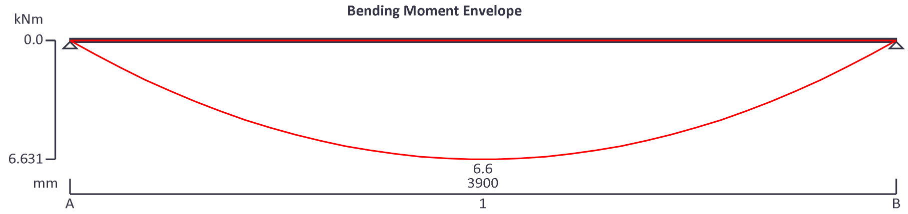
B2.1 Design

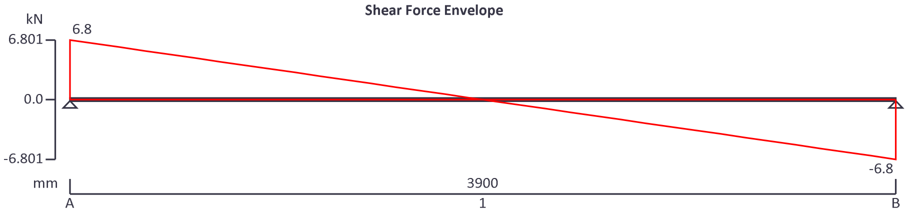
Timber beam analysis & design to EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 1.7.02







Applied loading

Beam loads

Permanent self weight of beam × 1

Permanent full UDL 1.820 kN/m

Variable full UDL 0.600 kN/m

Load combinations

Load combination 1 Support A Permanent × 1.35

Variable × 1.50

Span 1 Permanent × 1.35

Variable × 1.50

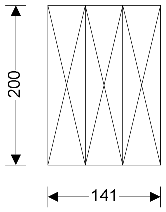
Support B Permanent × 1.35

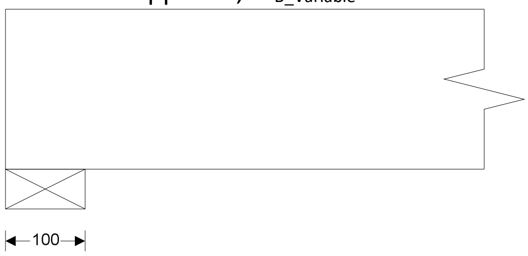
Variable × 1.50

Analysis results

Design moment; M = 6.631 kNm;Design shear; F = 6.801 kN

Total load on member; W_{tot} = 13.602 kN
Reactions at support A; R_{A_max} = 6.801 kN; R_{A_min} = 6.801 kN
Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = 3.738 kN
Unfactored variable load reaction at support A; $R_{A\_Variable}$ = 1.170 kN
Reactions at support B; R_{B_max} = 6.801 kN; R_{B_min} = 6.801 kN
Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = 3.738 kN
Unfactored variable load reaction at support B; $R_{B\_Variable}$ = 1.170 kN



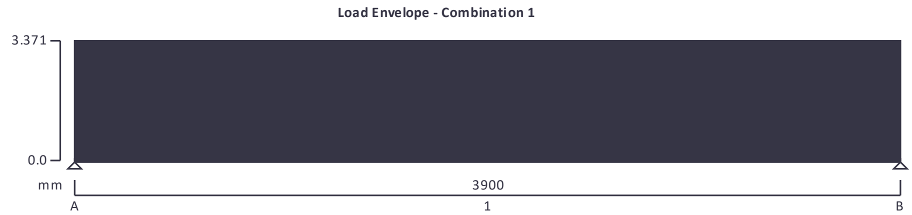


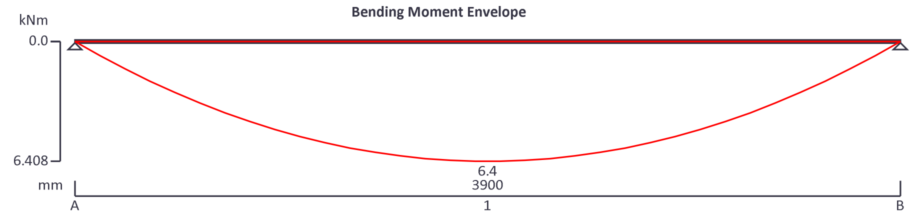
Timber section details
Breadth of section; b = 47 mm; Depth of section; h = 200 mm
Number of sections; N = 3; Breadth of member; b_b = 141 mm
Timber strength class; C24
Member details
Service class of timber; 2; Load duration; Long-term
Length of span; L_{s1} = 3900 mm
Length of bearing; L_b = 100 mm
In accordance with cl.6.6 the member is one of several similar and equally spaced members laterally connected by a continuous load distribution system capable of transferring loads from one member to the neighboring members.
Compression perpendicular to grain - cl.6.1.4
Design compressive stress; $\sigma_{c.90.d}$ = 0.482 N/mm^2; Design compressive strength; $f_{c.90.d}$ = 1.481 N/mm^2
PASS - Design compressive strength exceeds design compressive stress at bearing
Bending - cl 6.1.6
Design bending stress; $\sigma_{m.d}$ = 7.054 N/mm^2; Design bending strength; $f_{m.d}$ = 14.215 N/mm^2
PASS - Design bending strength exceeds design bending stress
Shear - cl.6.1.7
Applied shear stress; τ_d = 0.540 N/mm^2; Permissible shear stress; $f_{v.d}$ = 2.369 N/mm^2
PASS - Design shear strength exceeds design shear stress
Deflection - cl.7.2
Deflection limit;δ_{lim} = 14.000 mm; Total final deflection; δ_{fin} = 12.711 mm
PASS - Total final deflection is less than the deflection limit

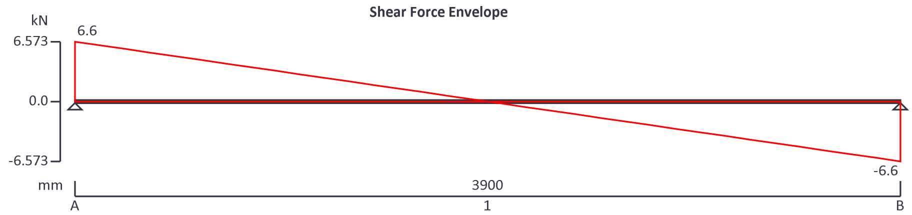
Therefore use 3No 200x50 C24

B2.2 Design

Timber beam analysis & design to EN1995-1-1:2004
In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1
Tedds calculation version 1.7.02







Applied loading

Beam loads

Permanent self weight of beam × 1
Permanent full UDL 1.400 kN/m
Variable full UDL 0.900 kN/m

Load combinations

Load combination 1 Support A Permanent × 1.35
Variable × 1.50
Span 1 Permanent × 1.35
Variable × 1.50
Support B Permanent × 1.35
Variable × 1.50

Analysis results

Design moment; M = 6.408 kNm; Design shear; F = 6.573 kN
Total load on member; W_{tot} = 13.146 kN
Reactions at support A; R_{A_max} = 6.573 kN; R_{A_min} = 6.573 kN
Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = 2.919 kN

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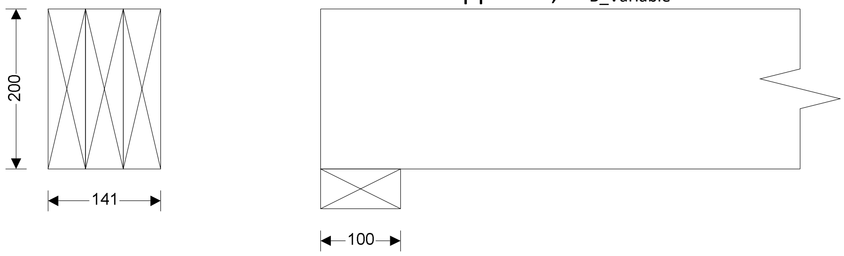
Unfactored variable load reaction at support A; $R_{A\_Variable}$ = 1.755 kN
Reactions at support B; R_{B_max} = 6.573 kN; R_{B_min} = 6.573 kN
Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = 2.919 kN
Unfactored variable load reaction at support B; $R_{B\_Variable}$ = 1.755 kN

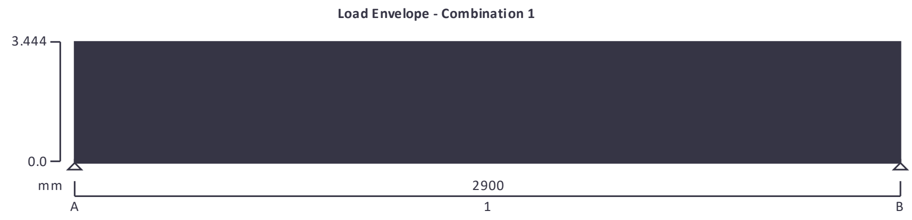


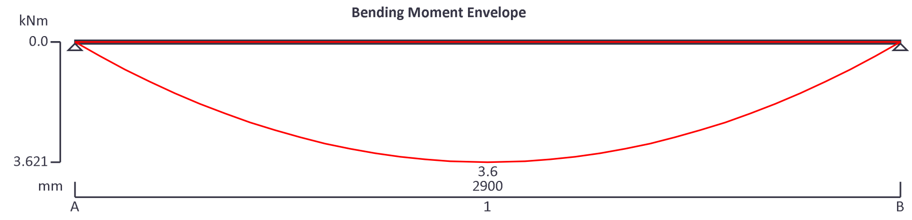
Timber section details
Breadth of section; b = 47 mm; Depth of section; h = 200 mm
Number of sections; N = 3; Breadth of member; b_b = 141 mm
Timber strength class; C24
Member details
Service class of timber; 2; Load duration; Long-term
Length of span; L_{s1} = 3900 mm
Length of bearing; L_b = 100 mm
Compression perpendicular to grain - cl.6.1.4
Design compressive stress; $\sigma_{c.90.d}$ = 0.466 N/mm^2; Design compressive strength; $f_{c.90.d}$ = 1.346 N/mm^2
PASS - Design compressive strength exceeds design compressive stress at bearing
Bending - cl 6.1.6
Design bending stress; $\sigma_{m.d}$ = 6.818 N/mm^2; Design bending strength; $f_{m.d}$ = 12.923 N/mm^2
PASS - Design bending strength exceeds design bending stress
Shear - cl.6.1.7
Applied shear stress; τ_d = 0.522 N/mm^2; Permissible shear stress; $f_{v.d}$ = 2.154 N/mm^2
PASS - Design shear strength exceeds design shear stress
Deflection - cl.7.2
Deflection limit; δ_{lim} = 14.000 mm; Total final deflection; δ_{fin} = 11.547 mm
PASS - Total final deflection is less than the deflection limit

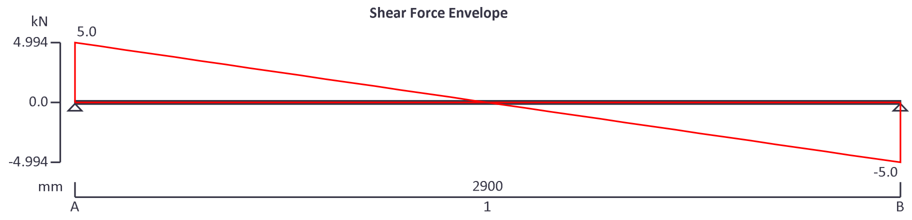
Therefore use 3No 200x50 C24

B2.3 Design

Timber beam analysis & design to EN1995-1-1:2004
In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1
Tedds calculation version 1.7.02

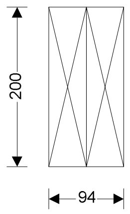


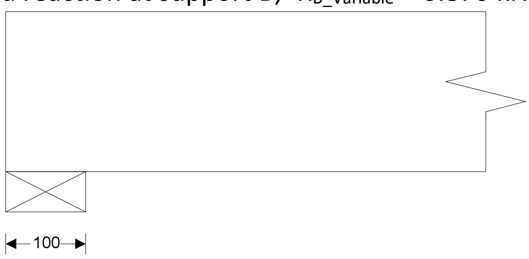




Applied loading
Beam loads
Permanent self weight of beam × 1
Permanent full UDL 1.820 kN/m
Variable full UDL 0.600 kN/m
Load combinations
Load combination 1 Support A Permanent × 1.35
Variable × 1.50
Span 1 Permanent × 1.35
Variable × 1.50
Support B Permanent × 1.35
Variable × 1.50
Analysis results
Design moment; M = 3.621 kNm;Design shear; F = 4.994 kN
Total load on member; W_{tot} = 9.988 kN

Reactions at support A; R_{A_max} = 4.994 kN; R_{A_min} = 4.994 kN
Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = 2.733 kN
Unfactored variable load reaction at support A; $R_{A\_Variable}$ = 0.870 kN
Reactions at support B; R_{B_max} = 4.994 kN; R_{B_min} = 4.994 kN
Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = 2.733 kN
Unfactored variable load reaction at support B; $R_{B\_Variable}$ = 0.870 kN



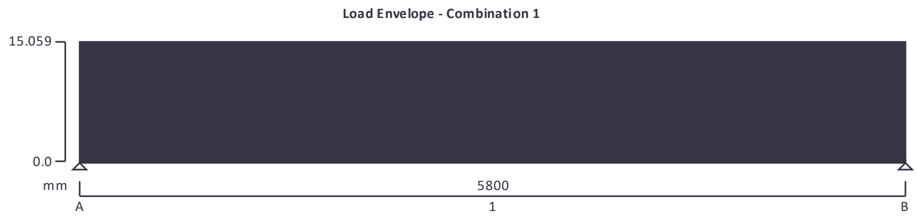


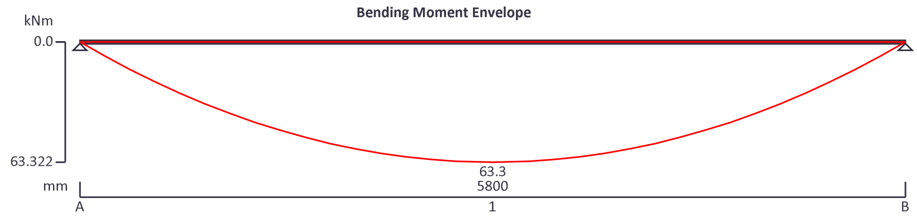
Timber section details
Breadth of section; b = 47 mm; Depth of section; h = 200 mm
Number of sections; N = 2; Breadth of member; b_b = 94 mm
Timber strength class; C24
Member details
Service class of timber; 2; Load duration; Long-term
Length of span; L_{s1} = 2900 mm
Length of bearing; L_b = 100 mm
In accordance with cl.6.6 the member is one of several similar and equally spaced members laterally connected by a continuous load distribution system capable of transferring loads from one member to the neighboring members.
Compression perpendicular to grain - cl.6.1.4
Design compressive stress; $\sigma_{c.90.d}$ = 0.531 N/mm^2; Design compressive strength; $f_{c.90.d}$ = 1.481 N/mm^2
PASS - Design compressive strength exceeds design compressive stress at bearing
Bending - cl 6.1.6
Design bending stress; $\sigma_{m.d}$ = 5.778 N/mm^2; Design bending strength; $f_{m.d}$ = 14.215 N/mm^2
PASS - Design bending strength exceeds design bending stress
Shear - cl.6.1.7
Applied shear stress; τ_d = 0.595 N/mm^2; Permissible shear stress; $f_{v.d}$ = 2.369 N/mm^2
PASS - Design shear strength exceeds design shear stress
Deflection - cl.7.2
Deflection limit;δ_{lim} = 11.600 mm; Total final deflection; δ_{fin} = 5.928 mm
PASS - Total final deflection is less than the deflection limit

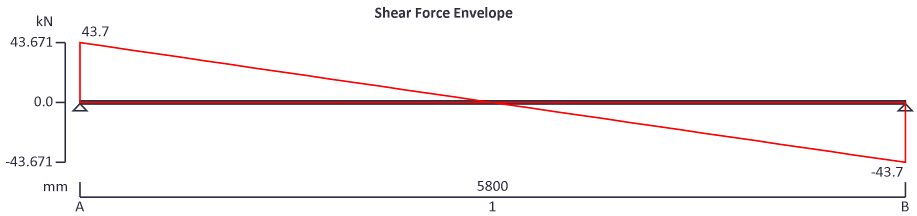
Therefore use 2No 200x50 C24

B2.4 Design

Steel beam analysis & design (EN1993-1-1:2005)
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex
TEDDS calculation version 3.0.13







Support conditions
Support A Vertically restrained
Rotationally free
Support B Vertically restrained
Rotationally free
Applied loading
Beam loads Permanent self weight of beam × 1
Permanent full UDL 5.2 kN/m
Variable full UDL 4.9 kN/m
Load combinations
Load combination 1 Support A Permanent × 1.35
Variable × 1.50
Permanent × 1.35
Variable × 1.50
Support B Permanent × 1.35

Variable $\times$ 1.50

Analysis results

Maximum moment; M_{max} = 63.3 kNm; M_{min} = 0 kNm

Maximum shear; V_{max} = 43.7 kN; V_{min} = -43.7 kN

Deflection; δ_{max} = 14.2 mm; δ_{min} = 0 mm

Maximum reaction at support A; R_{A_max} = 43.7 kN; R_{A_min} = 43.7 kN

Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = 16.6 kN

Unfactored variable load reaction at support A; $R_{A\_Variable}$ = 14.2 kN

Maximum reaction at support B; R_{B_max} = 43.7 kN; R_{B_min} = 43.7 kN

Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = 16.6 kN

Unfactored variable load reaction at support B; $R_{B\_Variable}$ = 14.2 kN

Section details

Section type; UKC 203x203x52 (Tata Steel Advance)

Steel grade; S275

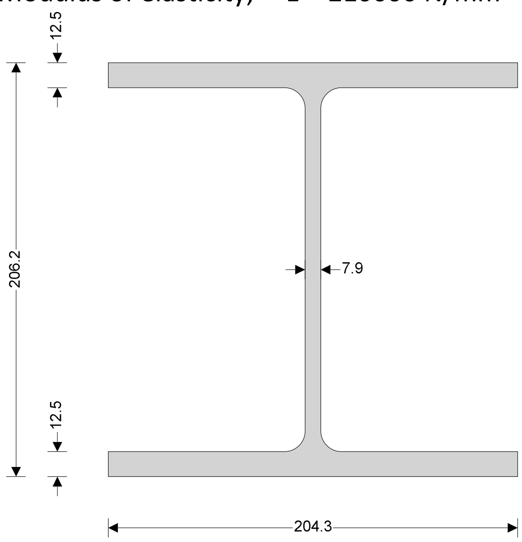
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element; $t = \max(t_f, t_w)$ = 12.5 mm

Nominal yield strength; f_y = 275 N/mm^2

Nominal ultimate tensile strength; f_u = 410 N/mm^2

Modulus of elasticity; E = 210000 N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections; γ_{M0} = 1.00

Resistance of members to instability; γ_{M1} = 1.00

Resistance of tensile members to fracture; γ_{M2} = 1.10

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis; K_y = 1.000

Effective length factor in minor axis; K_z = 1.000

Effective length factor for torsion; $K_{LT.A}$ = 1.400; + 2 $\times$ h

$K_{LT.B}$ = 1.400; + 2 $\times$ h

Classification of cross sections - Section 5.5

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$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.92$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section; $c = d = 160.8$ mm

$c / t_w = 22.0 \times \varepsilon <= 72 \times \varepsilon$; Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section; $c = (b - t_w - 2 \times r) / 2 = 88$ mm

$c / t_f = 7.6 \times \varepsilon <= 9 \times \varepsilon$; Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f = 181.2$ mm

Shear area factor; $\eta = 1.000$

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = 43.7$ kN

Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1875 \text{ mm}^2$

Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = 297.6$ kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = 63.3$ kNm

Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = 156$ kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6; $k_c = 1$

$C_1 = 1 / k_c^2 = 1$

Destabilised load condition factor; $D = 1.5$

Curvature factor; $g = \sqrt{[1 - (I_z / I_y)]} = 0.814$

Poissons ratio; $\nu = 0.3$

Shear modulus; $G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$

Unrestrained length; $L = 1.4 \times L_{s1} + 2 \times h = 8532$ mm

Elastic critical buckling moment; $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = 152.4$ kNm

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = D \times \sqrt{(W_{pl.y} \times f_y / M_{cr})} = 1.518$

Limiting slenderness ratio; $\bar{\lambda}_{LT,0} = 0.4$

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5; b

Imperfection factor - Table 6.3; $\alpha_{LT} = 0.34$

Correction factor for rolled sections; $\beta = 0.75$

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 1.554$

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.420$

Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 1.000$

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.420$

Design buckling resistance moment - eq 6.55; $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = 65.5$ kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection; $\delta_{lim} = L_{s1} / 250 = 23.2$ mm

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 14.157$ mm

PASS - Maximum deflection does not exceed deflection limit

Therefore use 203UC52

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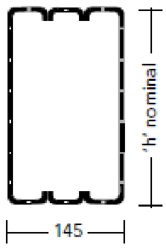
B2.5 Design

HD BOX 140

Manufactured length 150mm increments	600-1200	1350-1800	1950-2400	2550-2700
Height 'h'	150	150	215	215
Thickness	2.5	2.5	2.5	2.5
Total UDL kN	50	50	50	45

For heavy duty loading conditions to support concrete floors and point loads. Used to support internal and external openings in 150mm wide walls.

Heavy Duty Load



Therefore use 2No IG HD BOX 140

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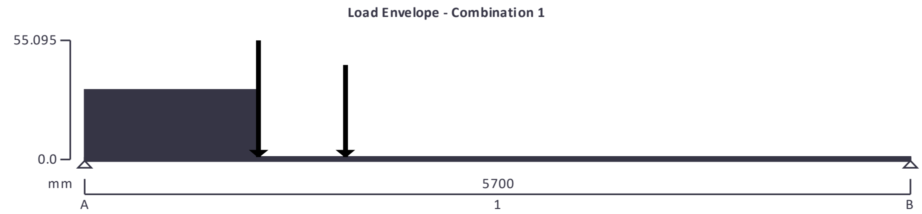
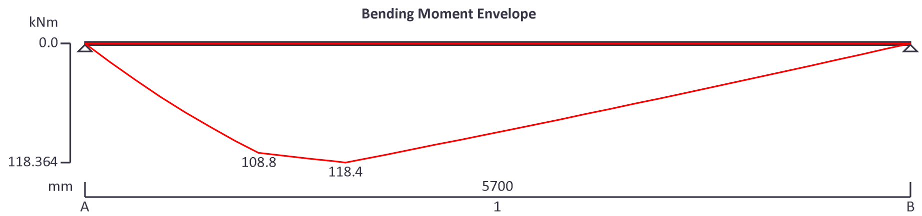
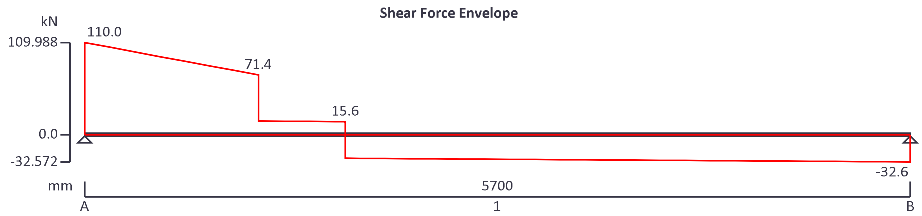
B2.6 Design

Steel beam analysis & design (EN1993-1-1:2005)
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex
TEDDS calculation version 3.0.13





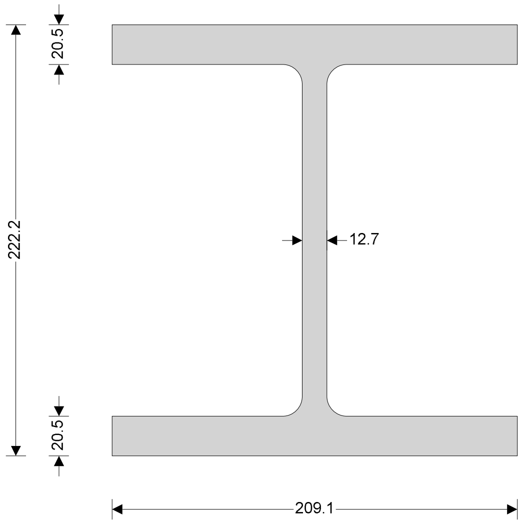


Support conditions
Support A Vertically restrained
Rotationally free
Support B Vertically restrained
Rotationally free
Applied loading
Beam loads Permanent self weight of beam × 1
Permanent partial UDL 23 kN/m from 0 mm to 1200 mm
Permanent point load 22.7 kN at 1200 mm
Variable point load 16.3 kN at 1200 mm
Permanent point load 16.6 kN at 1800 mm
Variable point load 14.2 kN at 1800 mm
Load combinations
Load combination 1 Support A Permanent × 1.35
Variable × 1.50
Permanent × 1.35
Variable × 1.50

Support B Permanent × 1.35
Variable × 1.50

Analysis results
Maximum moment; M_{max} = 118.4 kNm; M_{min} = 0 kNm
Maximum shear; V_{max} = 110 kN; V_{min} = -32.6 kN
Deflection; δ_{max} = 11.8 mm; δ_{min} = 0 mm
Maximum reaction at support A; R_{A_max} = 110 kN; R_{A_min} = 110 kN
Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = 56.4 kN
Unfactored variable load reaction at support A; $R_{A\_Variable}$ = 22.6 kN
Maximum reaction at support B; R_{B_max} = 32.6 kN; R_{B_min} = 32.6 kN
Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = 15.3 kN
Unfactored variable load reaction at support B; $R_{B\_Variable}$ = 7.9 kN

Section details
Section type; UKC 203x203x86 (Tata Steel Advance)
Steel grade; S275
EN 10025-2:2004 - Hot rolled products of structural steels
Nominal thickness of element; $t = max(t_f, t_w)$ = 20.5 mm
Nominal yield strength; f_y = 265 N/mm^2
Nominal ultimate tensile strength; f_u = 410 N/mm^2
Modulus of elasticity; E = 210000 N/mm^2



Partial factors - Section 6.1
Resistance of cross-sections; γ_{M0} = 1.00
Resistance of members to instability; γ_{M1} = 1.00
Resistance of tensile members to fracture; γ_{M2} = 1.10

Lateral restraint
Span 1 has lateral restraint at supports only

Effective length factors
Effective length factor in major axis; K_y = 1.000
Effective length factor in minor axis; K_z = 1.000
Effective length factor for torsion; $K_{LT.A}$ = 1.400; + 2 × h
$K_{LT.B}$ = 1.400; + 2 × h

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Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.94$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section; $c = d = 160.8$ mm

$c / t_w = 13.4 \times \varepsilon <= 72 \times \varepsilon$; Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section; $c = (b - t_w - 2 \times r) / 2 = 88$ mm

$c / t_f = 4.6 \times \varepsilon <= 9 \times \varepsilon$; Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f = 181.2$ mm

Shear area factor; $\eta = 1.000$

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = 110$ kN

Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 3069 \text{ mm}^2$

Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = 469.6$ kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = 118.4$ kNm

Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = 258.8$ kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6; $k_c = 1$

$C_1 = 1 / k_c^2 = 1$

Destabilised load condition factor; $D = 1.5$

Curvature factor; $g = \sqrt{[1 - (I_z / I_y)]} = 0.818$

Poissons ratio; $\nu = 0.3$

Shear modulus; $G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$

Unrestrained length; $L = 1.4 \times L_{s1} + 2 \times h = 8424$ mm

Elastic critical buckling moment;$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = 404.4 kNm

Slenderness ratio for lateral torsional buckling; $\overline{\lambda}_{LT} = D \times \sqrt{(W_{pl.y} \times f_y / M_{cr})} = 1.2$

Limiting slenderness ratio; $\overline{\lambda}_{LT,0} = 0.4$

$\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5; b

Imperfection factor - Table 6.3; $\alpha_{LT} = 0.34$

Correction factor for rolled sections; $\beta = 0.75$

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}) + \beta \times \overline{\lambda}_{LT}^2] = 1.176$

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \overline{\lambda}_{LT}^2)}], 1, 1 / \overline{\lambda}_{LT}^2) = 0.579$

Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\overline{\lambda}_{LT} - 0.8)^2], 1) = 1.000$

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.579$

Design buckling resistance moment - eq 6.55; $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = 149.9$ kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

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Limiting deflection; $\delta_{lim} = L_{s1} / 250 = 22.8$ mm
Maximum deflection span 1; $\delta = \max(\mathrm{abs}(\delta_{max}), \mathrm{abs}(\delta_{min})) = 11.82$ mm
PASS - Maximum deflection does not exceed deflection limit

Therefore use 203UC86

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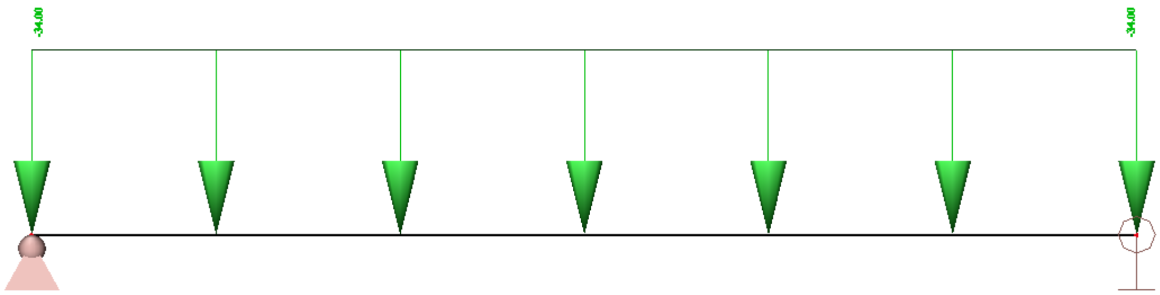
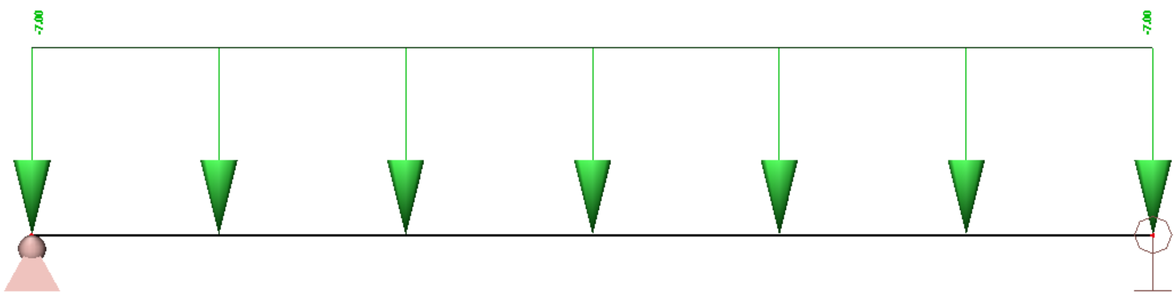
First Floor Member Design

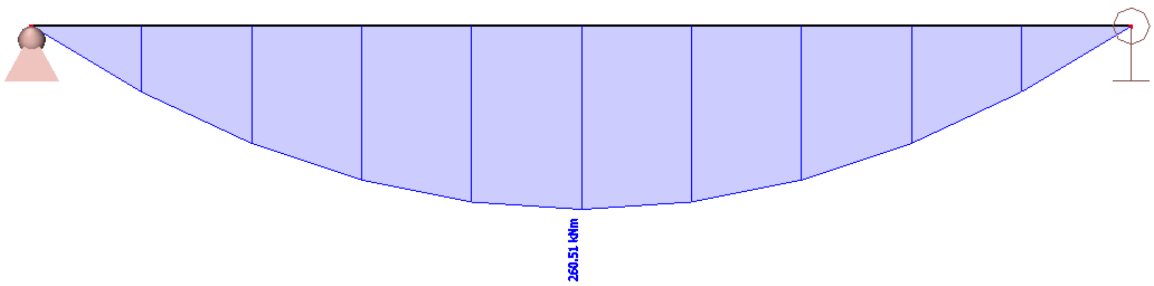
B1.1 Design



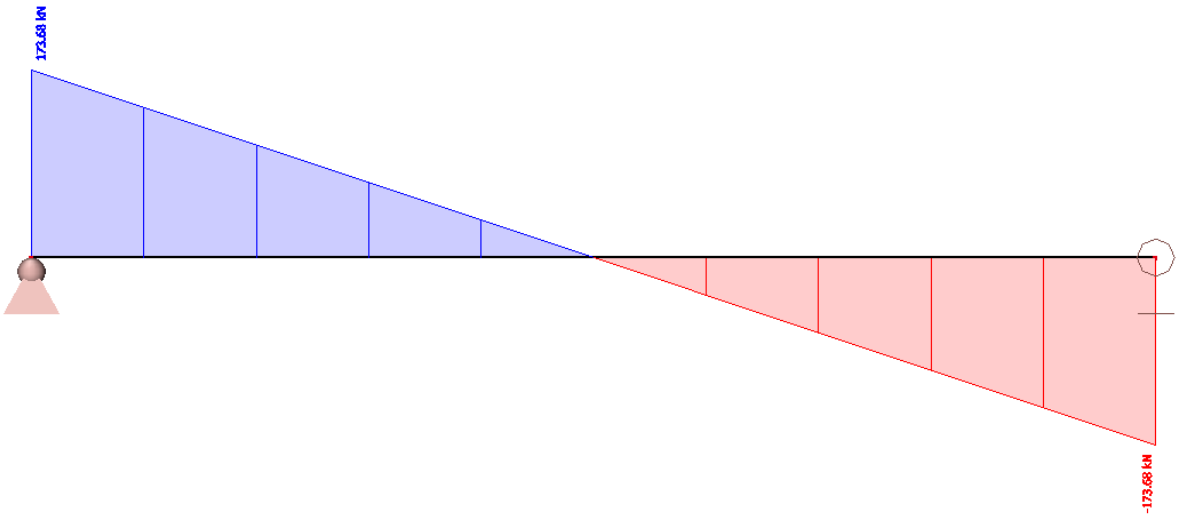
Dead load



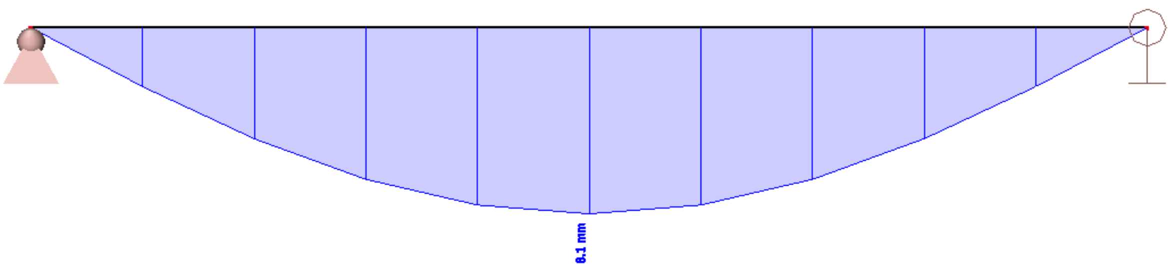
Live load



Bending moment



Shear force



Deflection SLS

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Dead reaction

Live reaction

EC-EN 1993 Steel check ULS

Linear calculation
Combination: ULS D+L
Coordinate system: Principal
Extreme 1D: Global
Selection: B3

EN 1993-1-1 Code Check
National annex: Standard EN

Member B3	**3.000 / 6.000 m**	**I + Ud (UB457/191/67, UKPFC300/100/45.50)**	**S 275**	**ULS D+L**	**0.66 -**

Combination key
ULS D+L / 1.35*LC1 + 1.35*Dead + 1.50*Live

Partial safety factors	
γ_{M0} for resistance of cross-sections	1.00
γ_{M1} for resistance to instability	1.00
γ_{M2} for resistance of net sections	1.25

Material		
Yield strength f_y	275.0	MPa
Ultimate strength f_u	430.0	MPa
Fabrication	Welded	

...::SECTION CHECK::...

The critical check is on position 3.000 m

Internal forces	Calculated	Unit
N_{Ed}	0.00	kN
$V_{y,Ed}$	0.00	kN
$V_{z,Ed}$	0.00	kN
T_{Ed}	0.00	kNm
$M_{y,Ed}$	260.51	kNm
$M_{z,Ed}$	0.00	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m^2]	σ_2 [kN/m^2]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	UO	95	13	-1.772e+05	-1.772e+05								
2	I	95	22	8.386e+04	8.386e+04	1.0		1.0	4.4	30.5	35.1	38.8	1
3	I	95	22	8.386e+04	8.386e+04	1.0		1.0	4.4	30.5	35.1	38.8	1
4	UO	95	13	-1.772e+05	-1.772e+05								
5	I	6	9	-1.772e+05	-1.735e+05								
6	I	428	9	-1.735e+05	7.750e+04	-2.2		0.3	50.4	107.8	124.3	277.8	1
7	I	6	9	7.750e+04	8.122e+04	1.0		1.0	0.7	30.5	35.1	39.4	1
8	UO	96	17	3.158e+04	8.759e+04	0.4	0.8	1.0	5.8	8.3	9.2	17.6	1
9	I	47	9	8.759e+04	8.759e+04	1.0		1.0	5.2	30.5	35.1	38.8	1
10	UO	95	17	8.759e+04	3.158e+04	0.4	0.8	1.0	5.8	8.3	9.2	17.6	1
11	I	47	9	8.759e+04	8.759e+04	1.0		1.0	5.2	30.5	35.1	38.8	1

The cross-section is classified as Class 1

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

$W_{pl,y}$	1.9574e-03	m^3
$M_{pl,y,Rd}$	538.30	kNm
Unity check	0.48	-

Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.19)

$\tau_{Vz,Ed}$	0.0	MPa
τ_{Rd}	158.8	MPa
Unity check	0.00	-

Note: No shear area is given for this section/fabrication, therefore the plastic shear resistance cannot be determined. As a result the elastic shear resistance according to EN 1993-1-1 article 6.2.6(4) is verified.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.1(5) and formula (6.1)

Elastic verification		
Fibre	2	
$\sigma_{N,Ed}$	0.0	MPa
$\sigma_{My,Ed}$	-181.2	MPa
$\sigma_{Mz,Ed}$	0.0	MPa
$\sigma_{tot,Ed}$	-181.2	MPa
$\tau_{Vy,Ed}$	0.0	MPa
$\tau_{Vz,Ed}$	0.0	MPa
$\tau_{t,Ed}$	0.0	MPa
$\tau_{tot,Ed}$	0.0	MPa
$\sigma_{von\ Mises,Ed}$	181.2	MPa

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Elastic verification		
Unity check	0.66	-

Note: For this section no plastic shear resistance and corresponding Rho value can be determined.
Therefore the elastic yield criterion according to EN 1993-1-1 article 6.2.1(5) is verified.

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 3.000 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	UO	95	13	-1.772e+05	-1.772e+05								
2	I	95	22	8.386e+04	8.386e+04	1.0		1.0	4.4	30.5	35.1	38.8	1
3	I	95	22	8.386e+04	8.386e+04	1.0		1.0	4.4	30.5	35.1	38.8	1
4	UO	95	13	-1.772e+05	-1.772e+05								
5	I	6	9	-1.772e+05	-1.735e+05								
6	I	428	9	-1.735e+05	7.750e+04	-2.2		0.3	50.4	107.8	124.3	277.8	1
7	I	6	9	7.750e+04	8.122e+04	1.0		1.0	0.7	30.5	35.1	39.4	1
8	UO	96	17	3.158e+04	8.759e+04	0.4	0.8	1.0	5.8	8.3	9.2	17.6	1
9	I	47	9	8.759e+04	8.759e+04	1.0		1.0	5.2	30.5	35.1	38.8	1
10	UO	95	17	8.759e+04	3.158e+04	0.4	0.8	1.0	5.8	8.3	9.2	17.6	1
11	I	47	9	8.759e+04	8.759e+04	1.0		1.0	5.2	30.5	35.1	38.8	1

The cross-section is classified as Class 1

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.2 and formula (6.54)

LTB parameters		
Method for LTB curve	General case	
Plastic section modulus $W_{pl,y}$	1.9574e-03	m³
Elastic critical moment M_{cr}	2006.24	kNm
Relative slenderness $\lambda_{rel,LT}$	0.52	
Limit slenderness $\lambda_{rel,LT,0}$	0.20	
LTB curve	d	
Imperfection α_{LT}	0.76	
Reduction factor χ_{LT}	0.77	
Design buckling resistance $M_{b,Rd}$	412.72	kNm
Unity check	0.63	-

Mcr parameters		
LTB length L	6.000	m
Influence of load position	no influence	
Correction factor k	1.00	
Correction factor k_w	1.00	
LTB moment factor C_1	1.13	
LTB moment factor C_2	0.45	
LTB moment factor C_3	0.53	
Shear center distance d_z	139	mm
Distance of load application z_g	0	mm
Mono-symmetry constant β_y	-407	mm
Mono-symmetry constant z_j	204	mm

Note: C parameters are determined according to ECCS 119 2006 / Galea 2002.

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The member satisfies the stability check.

Therefore use 457x191x89UB + 230x90 PFC to top flange

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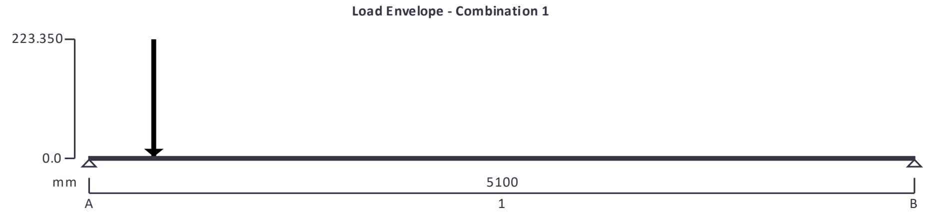
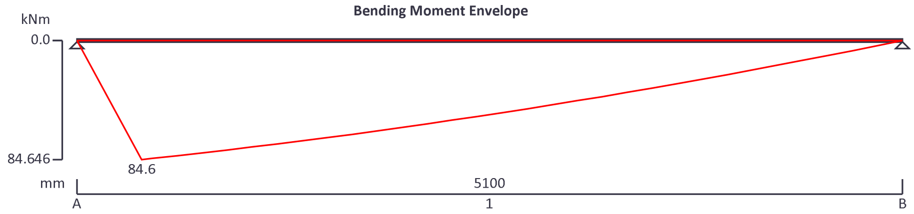
B1.2 Design

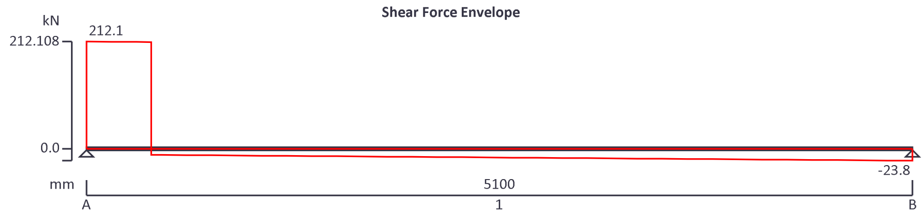
Steel beam analysis & design (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.13







Support conditions

Support A Vertically restrained
Rotationally free
Support B Vertically restrained
Rotationally free

Applied loading

Beam loads Permanent self weight of beam × 1
Permanent full UDL 0.64 kN/m
Variable full UDL 0.6 kN/m
Permanent point load 131 kN at 400 mm
Variable point load 31 kN at 400 mm

Load combinations

Load combination 1 Support A Permanent × 1.35
Variable × 1.50
Permanent × 1.35
Variable × 1.50
Support B Permanent × 1.35
Variable × 1.50

Analysis results

Maximum moment; M_{max} = **84.6** kNm; M_{min} = **0** kNm
Maximum shear; V_{max} = **212.1** kN; V_{min} = **-23.8** kN
Deflection; δ_{max} = **18.6** mm; δ_{min} = **0** mm
Maximum reaction at support A; R_{A_max} = **212.1** kN; R_{A_min} = **212.1** kN

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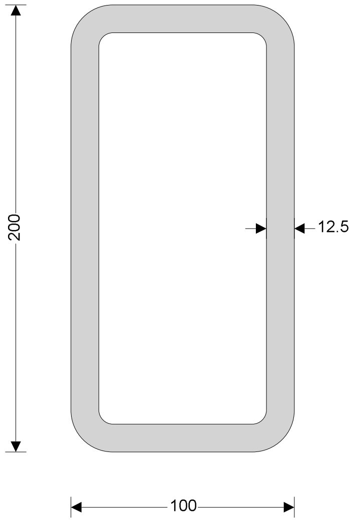
Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = **123.7** kN
Unfactored variable load reaction at support A; $R_{A\_Variable}$ = **30.1** kN
Maximum reaction at support B; R_{B_max} = **23.8** kN; R_{B_min} = **23.8** kN
Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = **13.2** kN
Unfactored variable load reaction at support B; $R_{B\_Variable}$ = **4** kN

Section details
Section type; **RHS 200x100x12.5 (Tata Steel Celsius)**
Steel grade; **S355H**

EN 10210-1:2006 - Hot finished structural hollow sections of non-alloy and fine grain steels
Nominal thickness of element; t = **12.5** mm
Nominal yield strength; f_y = **355** N/mm^2
Nominal ultimate tensile strength; f_u = **470** N/mm^2
Modulus of elasticity; E = **210000** N/mm^2



Partial factors - Section 6.1
Resistance of cross-sections; γ_{M0} = **1.00**
Resistance of members to instability; γ_{M1} = **1.00**
Resistance of tensile members to fracture; γ_{M2} = **1.10**

Lateral restraint
Span 1 has lateral restraint at supports only

Effective length factors
Effective length factor in major axis; K_y = **1.000**
Effective length factor in minor axis; K_z = **1.000**
Effective length factor for torsion; $K_{LT.A}$ = **1.400**; + 2 × h
$K_{LT.B}$ = **1.400**; + 2 × h

Classification of cross sections - Section 5.5
$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.81**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)
Width of section; c = h - 3 × t = **162.5** mm
$c / t = 16.0 \times \varepsilon <= 72 \times \varepsilon$; Class 1

Internal compression parts subject to compression only - Table 5.2 (sheet 1 of 3)
Width of section; c = b - 3 × t = **62.5** mm
$c / t = 6.1 \times \varepsilon <= 33 \times \varepsilon$; Class 1

Section is class 1

Check shear - Section 6.2.6
Height of web; h_w = h - 2 × t = **175** mm
Shear area factor; η = **1.000**
$h_w / t < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored
Design shear force; $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **212.1** kN
Shear area - cl 6.2.6(3); $A_v = A \times h / (b + h)$ = **4472** mm^2

Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **916.5** kN
PASS - Design shear resistance exceeds design shear force
Check bending moment major (y-y) axis - Section 6.2.5
Design bending moment; $M_{Ed} = \max(\mathrm{abs}(M_{s1\_max}), \mathrm{abs}(M_{s1\_min}))$ = **84.6** kNm
Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **144.9** kNm
PASS - Design bending resistance moment exceeds design bending moment
Check vertical deflection - Section 7.2.1
Consider deflection due to permanent and variable loads
Limiting deflection; $\delta_{lim} = L_{s1} / 250$ = **20.4** mm
Maximum deflection span 1; $\delta = \max(\mathrm{abs}(\delta_{max}), \mathrm{abs}(\delta_{min}))$ = **18.561** mm
PASS - Maximum deflection does not exceed deflection limit

Therefore use 200x100x12.5 RHS

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J1.1 Design

ComFlor® 60 normal weight concrete / using mesh / unpropped / Eurocode

Single span deck continuous slab (m) - Mesh and Deck Fire Method - Beam width 152mm
(Note: Single span deck single span slab is only permitted using Bar Fire Method.)

Props	Fire period	Slab depth (mm)	Mesh 0.2% min.reqd*	Total applied load (kN/m²) 0.90mm 5.00	0.90mm 7.50	0.90mm 10.00	1.00mm 5.00	1.00mm 7.50	1.00mm 10.00	1.20mm 5.00	1.20mm 7.50	1.20mm 10.00
None	60 minutes	120***	A142	3.50 (A142)	3.09 (A193)	2.51 (A193)	3.68 (A193)	3.09 (A193)	2.55 (A193)	3.94 (A193)	3.18 (A193)	2.64 (A193)
		130	A142	3.41 (A142)	3.40 (A252)	3.39 (A393)	3.59 (A142)	3.58 (A252)	3.56 (A393)	3.84 (A142)	3.82 (A393)	3.81 (2xA252)
		140	A193	3.32 (A193)	3.32 (A193)	3.30 (A393)	3.50 (A193)	3.50 (A193)	3.48 (A393)	3.74 (A193)	3.74 (A252)	3.73 (A393)
		150	A193	3.24 (A193)	3.24 (A193)	3.23 (A252)	3.42 (A193)	3.42 (A193)	3.40 (A393)	3.66 (A193)	3.66 (A193)	3.65 (A393)
		160	A252	3.16 (A252)	3.16 (A252)	3.16 (A252)	3.34 (A252)	3.34 (A252)	3.34 (A252)	3.58 (A252)	3.58 (A252)	3.57 (A252)
		170	A252	3.09 (A252)	3.09 (A252)	3.09 (A252)	3.27 (A252)	3.27 (A252)	3.27 (A252)	3.51 (A252)	3.51 (A252)	3.51 (A252)
		180	A252	3.03 (A252)	3.03 (A252)	3.03 (A252)	3.21 (A252)	3.21 (A252)	3.21 (A252)	3.44 (A252)	3.44 (A252)	3.44 (A252)
		190	A393	2.96 (A393)	2.96 (A393)	2.96 (A393)	3.15 (A393)	3.15 (A393)	3.15 (A393)	3.37 (A393)	3.37 (A393)	3.37 (A393)
		200	A393	2.90 (A393)	2.90 (A393)	2.90 (A393)	3.10 (A393)	3.10 (A393)	3.10 (A393)	3.32 (A393)	3.32 (A393)	3.32 (A393)
None	90 minutes	130	A142	3.40 (A252)	3.39 (A393)	3.38 (2xA252)	3.58 (A252)	3.57 (A393)	3.32 (2xA252)	3.83 (A252)	3.82 (A393)	3.19 (2xA252)
		140	A193	3.32 (A193)	3.30 (A393)	3.30 (2xA252)	3.50 (A193)	3.48 (A393)	3.48 (2xA252)	3.74 (A252)	3.73 (A393)	3.72 (2xA252)
		150	A193	3.24 (A193)	3.23 (A252)	3.23 (A393)	3.42 (A193)	3.40 (A393)	3.40 (2xA252)	3.66 (A193)	3.65 (A393)	3.64 (2xA252)
		160	A252	3.16 (A252)	3.16 (A252)	3.15 (A393)	3.34 (A252)	3.34 (A252)	3.33 (A393)	3.58 (A252)	3.57 (A393)	3.56 (2xA252)
		170	A252	3.09 (A252)	3.09 (A252)	3.09 (A252)	3.27 (A252)	3.27 (A252)	3.27 (A252)	3.51 (A252)	3.51 (A252)	3.50 (A393)
		180	A252	3.03 (A252)	3.03 (A252)	3.03 (A252)	3.21 (A252)	3.21 (A252)	3.21 (A252)	3.44 (A252)	3.44 (A252)	3.44 (A252)
		190	A393	2.96 (A393)	2.96 (A393)	2.96 (A393)	3.15 (A393)	3.15 (A393)	3.15 (A393)	3.37 (A393)	3.37 (A393)	3.37 (A393)
		200	A393	2.90 (A393)	2.90 (A393)	2.90 (A393)	3.10 (A393)	3.10 (A393)	3.10 (A393)	3.32 (A393)	3.32 (A393)	3.32 (A393)
None	120 minutes	140	A193	3.31 (A252)	3.30 (A393)	3.28 (2xA393)	3.49 (A252)	3.48 (A393)	3.46 (2xA393)	3.73 (A393)	3.72 (2xA252)	3.70 (2xA393)
		150	A193	3.24 (A193)	3.23 (A393)	3.22 (2xA252)	3.41 (A252)	3.40 (A393)	3.40 (2xA252)	3.65 (A252)	3.65 (A393)	3.62 (2xA393)
		160	A252	3.16 (A252)	3.16 (A252)	3.15 (A393)	3.34 (A252)	3.34 (A252)	3.33 (2xA252)	3.58 (A252)	3.57 (A393)	3.56 (2xA252)
		170	A252	3.09 (A252)	3.09 (A252)	3.09 (A252)	3.27 (A252)	3.27 (A252)	3.27 (A393)	3.51 (A252)	3.51 (A252)	3.50 (A393)
		180	A252	3.03 (A252)	3.03 (A252)	3.03 (A252)	3.21 (A252)	3.21 (A252)	3.21 (A252)	3.44 (A252)	3.44 (A252)	3.43 (A393)
		190	A393	2.96 (A393)	2.96 (A393)	2.96 (A393)	3.15 (A393)	3.15 (A393)	3.15 (A393)	3.37 (A393)	3.37 (A393)	3.37 (A393)
		200	A393	2.90 (A393)	2.90 (A393)	2.90 (A393)	3.10 (A393)	3.10 (A393)	3.10 (A393)	3.32 (A393)	3.32 (A393)	3.32 (A393)

Therefore use 150 slab Comflor 60 0.9mm gauge + A193 top

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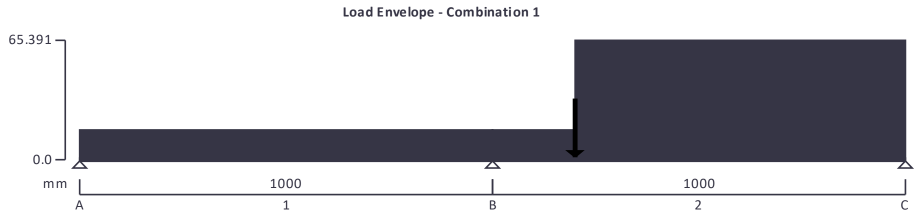
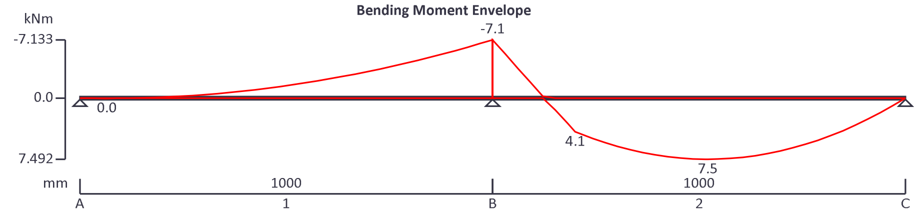
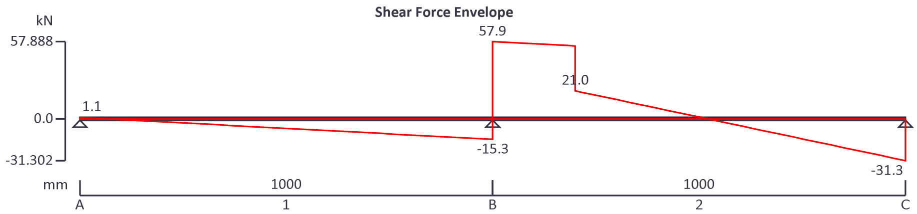
B1.3 Design

Steel beam analysis & design (EN1993-1-1:2005)
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex
TEDDS calculation version 3.0.13







Support conditions
Support A Vertically restrained
Rotationally free
Support B Vertically restrained
Rotationally free
Support C Vertically restrained
Rotationally free
Applied loading
Beam loads Permanent self weight of beam × 1
Permanent full UDL 6.9 kN/m
Variable full UDL 3.9 kN/m
Permanent partial UDL 36.3 kN/m from 1200 mm to 2000 mm
Permanent point load 16 kN at 1200 mm
Variable point load 8 kN at 1200 mm
Load combinations

Load combination 1 Support A Permanent × 1.35
Variable × 1.50
Permanent × 1.35
Variable × 1.50
Support B Permanent × 1.35
Variable × 1.50
Permanent × 1.35
Variable × 1.50
Support C Permanent × 1.35
Variable × 1.50

Analysis results

Maximum moment; M_{max} = 7.5 kNm; M_{min} = -7.1 kNm
Maximum moment span 1; M_{s1_max} = 0 kNm; M_{s1_min} = -7.1 kNm
Maximum moment span 2; M_{s2_max} = 7.5 kNm; M_{s2_min} = -7.1 kNm
Maximum shear; V_{max} = 57.9 kN; V_{min} = -31.3 kN
Maximum shear span 1; V_{s1_max} = 1.1 kN; V_{s1_min} = -15.3 kN
Maximum shear span 2; V_{s2_max} = 57.9 kN; V_{s2_min} = -31.3 kN
Deflection; δ_{max} = 0 mm; δ_{min} = 0 mm
Deflection span 1; δ_{s1_max} = 0 mm; δ_{s1_min} = 0 mm
Deflection span 2; δ_{s2_max} = 0 mm; δ_{s2_min} = 0 mm
Maximum reaction at support A; R_{A_max} = 1.1 kN; R_{A_min} = 1.1 kN
Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = -0.2 kN
Unfactored variable load reaction at support A; $R_{A\_Variable}$ = 0.9 kN
Maximum reaction at support B; R_{B_max} = 73.2 kN; R_{B_min} = 73.2 kN
Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = 40.4 kN
Unfactored variable load reaction at support B; $R_{B\_Variable}$ = 12.4 kN
Maximum reaction at support C; R_{C_max} = 31.3 kN; R_{C_min} = 31.3 kN
Unfactored permanent load reaction at support C; $R_{C\_Permanent}$ = 20.4 kN
Unfactored variable load reaction at support C; $R_{C\_Variable}$ = 2.5 kN

Section details

Section type; 2 x UKC 203x203x46 (Tata Steel Advance)
Steel grade; S275
EN 10025-2:2004 - Hot rolled products of structural steels
Nominal thickness of element; $t = \max(t_f, t_w)$ = 11.0 mm
Nominal yield strength; f_y = 275 N/mm^2
Nominal ultimate tensile strength; f_u = 410 N/mm^2
Modulus of elasticity; E = 210000 N/mm^2

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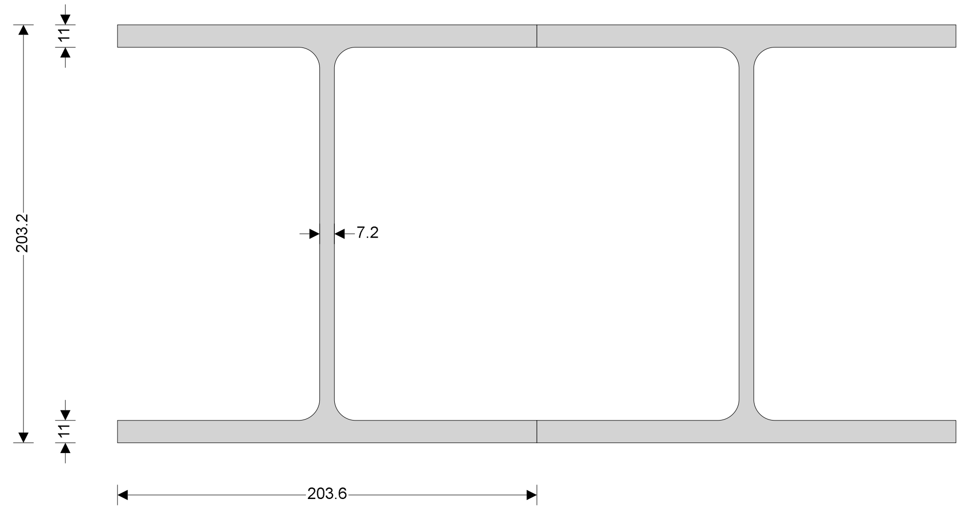

Partial factors - Section 6.1

Resistance of cross-sections; $\gamma_{M0} = 1.00$

Resistance of members to instability; $\gamma_{M1} = 1.00$

Resistance of tensile members to fracture; $\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Span 2 has full lateral restraint

Effective length factors

Effective length factor in major axis; $K_y = 1.000$

Effective length factor in minor axis; $K_z = 1.000$

Effective length factor for torsion; $K_{LT.A} = 2.400$;

$K_{LT.B} = 1.400$; $+ 2 \times h$

$K_{LT.C} = 1.000$;

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.92$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section; $c = d = 160.8$ mm

$c / t_w = 24.2 \times \varepsilon <= 72 \times \varepsilon$; Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section; $c = (b - t_w - 2 \times r) / 2 = 88$ mm

$c / t_f = 8.7 \times \varepsilon <= 9 \times \varepsilon$; Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f = 181.2$ mm

Shear area factor; $\eta = 1.000$

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = 57.9$ kN

Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1698 \text{ mm}^2$

Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = N \times A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = 539$ kN

PASS - Design shear resistance exceeds design shear force

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Check bending moment at span 2 major (y-y) axis - Section 6.2.5

Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s2\_max}), \text{abs}(M_{s2\_min})) = 7.5$ kNm

Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} = N \times W_{pl.y} \times f_y / \gamma_{M0} = 273.6$ kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection; $\delta_{lim} = L_{s2} / 180 = 5.6$ mm

Maximum deflection span 2; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 0.028$ mm

PASS - Maximum deflection does not exceed deflection limit

Therefore use 203UC46

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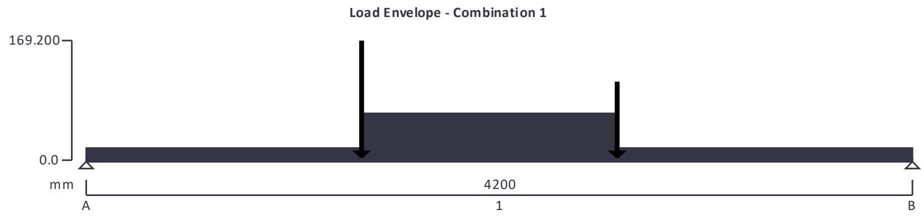
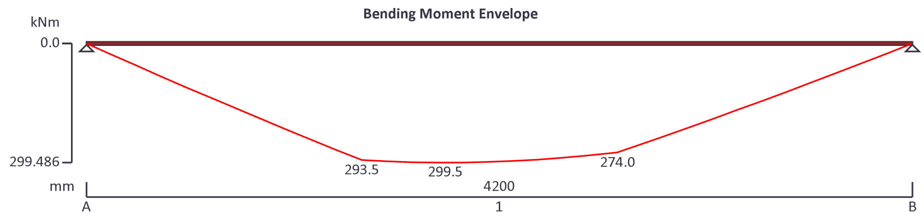
B1.4 Design

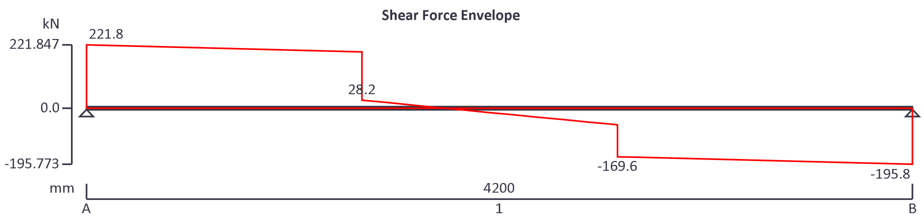
Steel beam analysis & design (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.13







Support conditions

Support A Vertically restrained

Rotationally free

Support B Vertically restrained

Rotationally free

Applied loading

Beam loads Permanent self weight of beam × 1

Permanent full UDL 6.9 kN/m

Variable full UDL 3.9 kN/m

Permanent partial UDL 36.3 kN/m from 1400 mm to 2700 mm

Permanent point load 102 kN at 1400 mm

Variable point load 21 kN at 1400 mm

Permanent point load 57 kN at 2700 mm

Variable point load 23 kN at 2700 mm

Load combinations

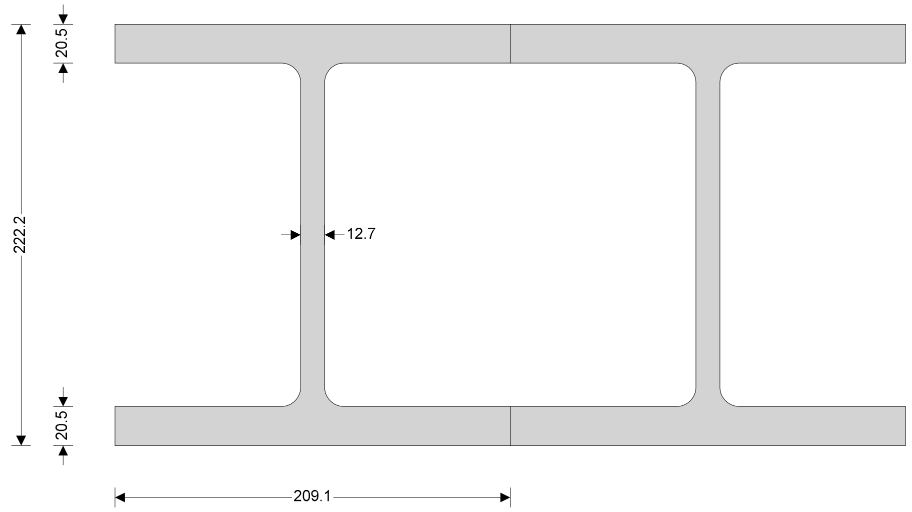
Load combination 1 Support A Permanent × 1.35

Variable × 1.50

Permanent × 1.35
Variable × 1.50
Support B Permanent × 1.35
Variable × 1.50

Analysis results
Maximum moment; M_{max} = 299.5 kNm; M_{min} = 0 kNm
Maximum shear; V_{max} = 221.8 kN; V_{min} = -195.8 kN
Deflection; δ_{max} = 9.9 mm; δ_{min} = 0 mm
Maximum reaction at support A; R_{A_max} = 221.8 kN; R_{A_min} = 221.8 kN
Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = 130.5 kN
Unfactored variable load reaction at support A; $R_{A\_Variable}$ = 30.4 kN
Maximum reaction at support B; R_{B_max} = 195.8 kN; R_{B_min} = 195.8 kN
Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = 111.7 kN
Unfactored variable load reaction at support B; $R_{B\_Variable}$ = 30 kN

Section details
Section type; 2 x UKC 203x203x86 (Tata Steel Advance)
Steel grade; S355
EN 10025-2:2004 - Hot rolled products of structural steels
Nominal thickness of element; $t = \max(t_f, t_w)$ = 20.5 mm
Nominal yield strength; f_y = 345 N/mm^2
Nominal ultimate tensile strength; f_u = 470 N/mm^2
Modulus of elasticity; E = 210000 N/mm^2



Partial factors - Section 6.1
Resistance of cross-sections; γ_{M0} = 1.00
Resistance of members to instability; γ_{M1} = 1.00
Resistance of tensile members to fracture; γ_{M2} = 1.10

Lateral restraint
Span 1 has lateral restraint at supports only

Effective length factors
Effective length factor in major axis; K_y = 1.000
Effective length factor in minor axis; K_z = 1.000

Effective length factor for torsion; $K_{LT.A}$ = 2.400;

$K_{LT.B}$ = 1.400; + 2 × h

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.83$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section; c = d = 160.8 mm

$c / t_w = 15.3 \times \varepsilon <= 72 \times \varepsilon$; Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section; $c = (b - t_w - 2 \times r) / 2 = 88$ mm

$c / t_f = 5.2 \times \varepsilon <= 9 \times \varepsilon$; Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f = 181.2$ mm

Shear area factor; $\eta = 1.000$

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = 221.8$ kN

Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 3069 \text{ mm}^2$

Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = N \times A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = 1222.6$ kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = 299.5$ kNm

Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} = N \times W_{pl.y} \times f_y / \gamma_{M0} = 673.9$ kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6; $k_c = 1$

$C_1 = 1 / k_c^2 = 1$

Destabilised load condition factor; D = 1.5

Curvature factor; $g = \sqrt{[1 - (I_z / I_y)]} = 0.818$

Poissons ratio; $\nu = 0.3$

Shear modulus; $G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$

Unrestrained length; $L = (1.4 \times L_{s1} + 1.4 \times L_{s1} + 2 \times h) / 2 = 6102$ mm

Elastic critical buckling moment;$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = 577.6 kNm

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = D \times \sqrt{(W_{pl.y} \times f_y / M_{cr})} = 1.146$

Limiting slenderness ratio; $\bar{\lambda}_{LT,0} = 0.4$

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5; b

Imperfection factor - Table 6.3; $\alpha_{LT} = 0.34$

Correction factor for rolled sections; $\beta = 0.75$

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 1.119$

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.611$

Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 1.000$

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.611$

Design buckling resistance moment - eq 6.55; $M_{b,Rd} = \chi_{LT,mod} \times N \times W_{pl.y} \times f_y / \gamma_{M1} = 411.8$ kNm

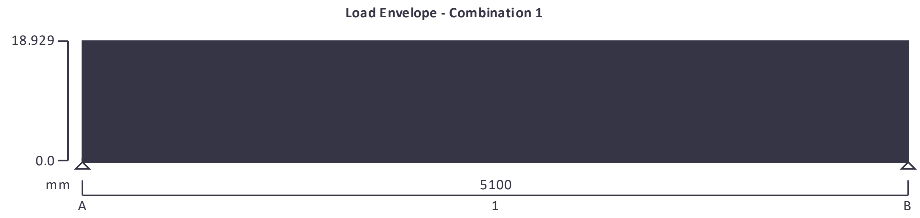
PASS - Design buckling resistance moment exceeds design bending moment

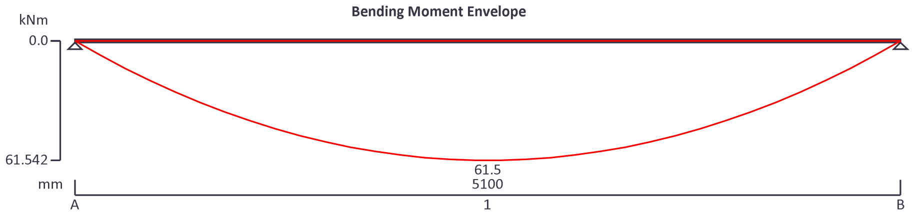
Check vertical deflection - Section 7.2.1
Consider deflection due to permanent and variable loads
Limiting deflection; $\delta_{lim} = L_{s1} / 180 = 23.3$ mm
Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 9.873$ mm
PASS - Maximum deflection does not exceed deflection limit

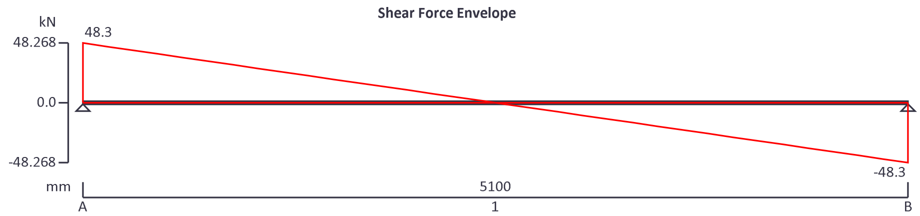
Therefore use 203UC86

B1.5 Design

Steel beam analysis & design (EN1993-1-1:2005)
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex
TEDDS calculation version 3.0.13







Support conditions
Support A Vertically restrained
Rotationally free
Support B Vertically restrained
Rotationally free
Applied loading
Beam loads Permanent self weight of beam × 1
Permanent full UDL 9 kN/m
Variable full UDL 4 kN/m
Load combinations
Load combination 1 Support A Permanent × 1.35
Variable × 1.50
Permanent × 1.35
Variable × 1.50
Support B Permanent × 1.35

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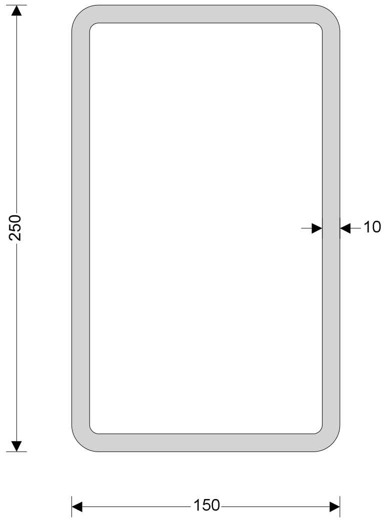
Variable × 1.50
Analysis results
Maximum moment; M_{max} = 61.5 kNm; M_{min} = 0 kNm
Maximum shear; V_{max} = 48.3 kN; V_{min} = -48.3 kN
Deflection; δ_{max} = 9.2 mm; δ_{min} = 0 mm
Maximum reaction at support A; R_{A_max} = 48.3 kN; R_{A_min} = 48.3 kN
Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = 24.4 kN
Unfactored variable load reaction at support A; $R_{A\_Variable}$ = 10.2 kN
Maximum reaction at support B; R_{B_max} = 48.3 kN; R_{B_min} = 48.3 kN
Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = 24.4 kN
Unfactored variable load reaction at support B; $R_{B\_Variable}$ = 10.2 kN
Section details
Section type; RHS 250x150x10.0 (Tata Steel Celsius)
Steel grade; S355H
EN 10210-1:2006 - Hot finished structural hollow sections of non-alloy and fine grain steels
Nominal thickness of element; t = 10.0 mm
Nominal yield strength; f_y = 355 N/mm^2
Nominal ultimate tensile strength; f_u = 470 N/mm^2
Modulus of elasticity; E = 210000 N/mm^2



Partial factors - Section 6.1
Resistance of cross-sections; γ_{M0} = 1.00
Resistance of members to instability; γ_{M1} = 1.00
Resistance of tensile members to fracture; γ_{M2} = 1.10
Lateral restraint
Span 1 has lateral restraint at supports only
Effective length factors
Effective length factor in major axis; K_y = 1.000
Effective length factor in minor axis; K_z = 1.000
Effective length factor for torsion; $K_{LT.A}$ = 1.400; + 2 × h
$K_{LT.B}$ = 1.400; + 2 × h
Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]} = 0.81$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section; $c = h - 3 \times t = 220$ mm

$c / t = 27.0 \times \varepsilon <= 72 \times \varepsilon$; Class 1

Internal compression parts subject to compression only - Table 5.2 (sheet 1 of 3)

Width of section; $c = b - 3 \times t = 120$ mm

$c / t = 14.7 \times \varepsilon <= 33 \times \varepsilon$; Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t = 230$ mm

Shear area factor; $\eta = 1.000$

$h_w / t < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = 48.3$ kN

Shear area - cl 6.2.6(3); $A_v = A \times h / (b + h) = 4683\ mm^2$

Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = 959.8$ kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = 61.5$ kNm

Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = 216.8$ kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection; $\delta_{lim} = L_{s1} / 250 = 20.4$ mm

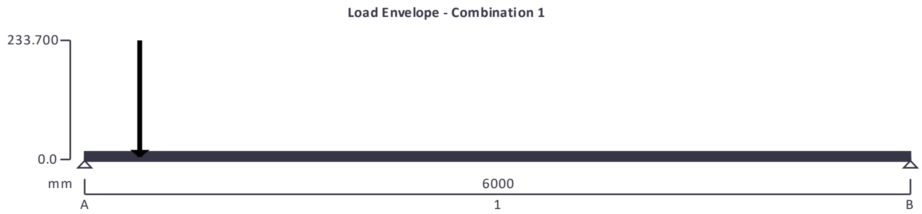
Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 9.225$ mm

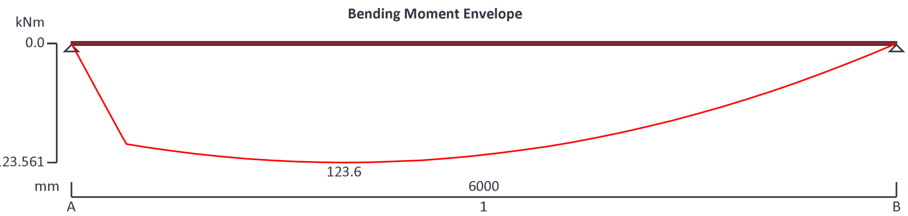
PASS - Maximum deflection does not exceed deflection limit

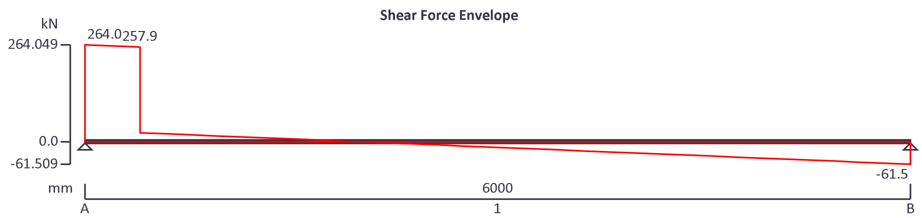
Therefore use 250x150x10 RHS

B1.6 Design

Steel beam analysis & design (EN1993-1-1:2005)
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex
TEDDS calculation version 3.0.13







Support conditions
Support A Vertically restrained
Rotationally free
Support B Vertically restrained
Rotationally free
Applied loading
Beam loads Permanent self weight of beam × 1
Permanent full UDL 5.2 kN/m
Variable full UDL 4.9 kN/m
Permanent point load 132 kN at 400 mm
Variable point load 37 kN at 400 mm
Load combinations
Load combination 1 Support A Permanent × 1.35
Variable × 1.50
Permanent × 1.35
Variable × 1.50
Support B Permanent × 1.35

Variable × 1.50

Analysis results

Maximum moment; M_{max} = 123.6 kNm; M_{min} = 0 kNm

Maximum shear; V_{max} = 264 kN; V_{min} = -61.5 kN

Deflection; δ_{max} = 21 mm; δ_{min} = 0 mm

Maximum reaction at support A; R_{A_max} = 264 kN; R_{A_min} = 264 kN

Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = 140.9 kN

Unfactored variable load reaction at support A; $R_{A\_Variable}$ = 49.2 kN

Maximum reaction at support B; R_{B_max} = 61.5 kN; R_{B_min} = 61.5 kN

Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = 26.5 kN

Unfactored variable load reaction at support B; $R_{B\_Variable}$ = 17.2 kN

Section details

Section type; UKC 203x203x71 (Tata Steel Advance)

Steel grade; S355

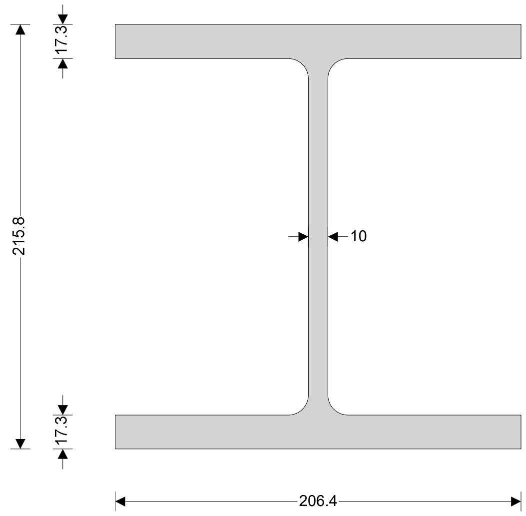
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element; t = max(t_f, t_w) = 17.3 mm

Nominal yield strength; f_y = 345 N/mm^2

Nominal ultimate tensile strength; f_u = 470 N/mm^2

Modulus of elasticity; E = 210000 N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections; γ_{M0} = 1.00

Resistance of members to instability; γ_{M1} = 1.00

Resistance of tensile members to fracture; γ_{M2} = 1.10

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis; K_y = 1.000

Effective length factor in minor axis; K_z = 1.000

Effective length factor for torsion; $K_{LT.A}$ = 1.000;

$K_{LT.B}$ = 1.400; + 2 × h

Classification of cross sections - Section 5.5

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$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.83$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section; $c = d = 160.8$ mm

$c / t_w = 19.5 \times \varepsilon <= 72 \times \varepsilon$; Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section; $c = (b - t_w - 2 \times r) / 2 = 88$ mm

$c / t_f = 6.2 \times \varepsilon <= 9 \times \varepsilon$; Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f = 181.2$ mm

Shear area factor; $\eta = 1.000$

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = 264$ kN

Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 2427 \text{ mm}^2$

Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = 483.5$ kN

PASS - Design shear resistance exceeds design shear force

Combined bending and shear - Section 6.2.8

Reduction factor - cl.6.2.8(3); $\rho_v = [(2 \times V_{Ed} / V_{pl,Rd}) - 1]^2 = 0.009$

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = 123.6$ kNm

Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} = [(W_{pl.y} - t_w \times h^2 / 4) + (t_w \times h^2 / 4) \times (1 - \rho_v)] \times f_y / \gamma_{M0} = 275.2$ kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6; $k_c = 1$

$C_1 = 1 / k_c^2 = 1$

Destabilised load condition factor; $D = 1.5$

Curvature factor; $g = \sqrt{[1 - (I_z / I_y)]} = 0.817$

Poissons ratio; $\nu = 0.3$

Shear modulus; $G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$

Unrestrained length; $L = (1.0 \times L_{s1} + 1.4 \times L_{s1} + 2 \times h) / 2 = 7416$ mm

Elastic critical buckling moment; $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = 326.2$ kNm

Slenderness ratio for lateral torsional buckling; $\overline{\lambda}_{LT} = D \times \sqrt{([(W_{pl.y} - t_w \times h^2 / 4) + (t_w \times h^2 / 4) \times (1 - \rho_v)] \times f_y / M_{cr})} = 1.378$

Limiting slenderness ratio; $\overline{\lambda}_{LT,0} = 0.4$

$\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5; b

Imperfection factor - Table 6.3; $\alpha_{LT} = 0.34$

Correction factor for rolled sections; $\beta = 0.75$

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}) + \beta \times \overline{\lambda}_{LT}^2] = 1.378$

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \overline{\lambda}_{LT}^2)}], 1, 1 / \overline{\lambda}_{LT}^2) = 0.484$

Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\overline{\lambda}_{LT} - 0.8)^2], 1) = 1.000$

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.484$

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Design buckling resistance moment - eq 6.55; $M_{b,Rd} = \chi_{LT,mod} \times [(W_{pl.y} - t_w \times h^2 / 4) + (t_w \times h^2 / 4) \times (1 - \rho_v)] \times f_y / \gamma_{M1}$ = 133.1 kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection; $\delta_{lim} = L_{s1} / 250$ = 24 mm

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = 20.951 mm

PASS - Maximum deflection does not exceed deflection limit

Therefore use 203UC71

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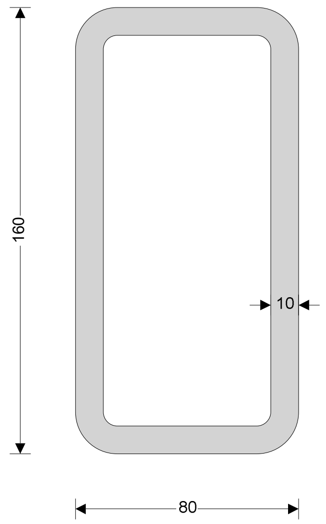
C1.1 Design

Steel member design (EN1993-1-1:2005)
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex
Tedds calculation version 4.3.01
Partial factors - Section 6.1
Resistance of cross-sections; $\gamma_{M0} = 1$
Resistance of members to instability; $\gamma_{M1} = 1$
Resistance of tensile members to fracture; $\gamma_{M2} = 1.1$
Design section 1
Section details
Section type; RHS 160x80x10.0 (Tata Steel Celsius)
Steel grade - EN 10210-1:2006; S275H
Nominal thickness of element; $t_{nom} = t = 10$ mm
Nominal yield strength; $f_y = 275$ N/mm^2
Nominal ultimate tensile strength; $f_u = 410$ N/mm^2
Modulus of elasticity; $E = 210000$ N/mm^2



RHS 160x80x10.0 (Tata Steel Celsius)
Section depth, h, 160 mm
Section breadth, b, 80 mm
Mass of section, Mass, 33.7 kg/m
Section thickness, t, 10 mm
Area of section, A, 4293 mm^2
Radius of gyration about y-axis, i_y, 54.702 mm
Radius of gyration about z-axis, i_z, 30.951 mm
Elastic section modulus about y-axis, $W_{el.y}$, 160561 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 102805 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 209013 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 125305 mm^3
Second moment of area about y-axis, I_y, 12844910 mm^4
Second moment of area about z-axis, I_z, 4112196 mm^4

Analysis results
Design bending moment - Minor axis; $M_{z,Ed} = 12$ kNm
Design axial compression force; $N_{Ed} = 225$ kN
Restraint spacing
Major axis lateral restraint; $L_y = 3000$ mm
Minor axis lateral restraint; $L_z = 3000$ mm
Torsional restraint; $L_T = 3000$ mm
Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.92$

Internal compression parts subject to bending and compression - Table 5.2 (sheet 1 of 3)
Width of section; $c = b - 3 \times t = 50$ mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times 2 \times t \times f_y) - 3 \times t / 2] / c, 1) = 1.000$

$c / t = 5 = 5.4 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$; Class 1

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Internal compression parts subject to compression - Table 5.2 (sheet 1 of 3)
Width of section; $c = h - 3 \times t = 130$ mm
$c / t = 13 = 14.1 \times \varepsilon <= 33 \times \varepsilon$; Class 1
Section is class 1
Check compression - Section 6.2.4
Design compression force; $N_{Ed} = 225$ kN
Design resistance of section - eq 6.10; $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0} = 1180.5$ kN
$N_{Ed} / N_{c,Rd} = 0.191$
PASS - Design compression resistance exceeds design compression
Slenderness ratio for y-y axis flexural buckling - Section 6.3.1.3
Critical buckling length; $L_{cr,y} = L_{y\_s1} = 3000$ mm
Critical buckling force; $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 = 2958.1$ kN
Slenderness ratio for buckling - eq 6.50; $\overline{\lambda}_y = \sqrt{}(A \times f_y / N_{cr,y}) = 0.632$
Check y-y axis flexural buckling resistance - Section 6.3.1.1
Buckling curve - Table 6.2; a
Imperfection factor - Table 6.1; $\alpha_y = 0.21$
Buckling reduction determination factor; $\phi_y = 0.5 \times (1 + \alpha_y \times (\overline{\lambda}_y - 0.2) + \overline{\lambda}_y^2) = 0.745$
Buckling reduction factor - eq 6.49; $\chi_y = \min(1 / (\phi_y + \sqrt{}(\phi_y^2 - \overline{\lambda}_y^2)), 1) = 0.878$
Design buckling resistance - eq 6.47; $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} = 1036$ kN
$N_{Ed} / N_{b,y,Rd} = 0.217$
PASS - Design buckling resistance exceeds design compression
Slenderness ratio for z-z axis flexural buckling - Section 6.3.1.3
Critical buckling length; $L_{cr,z} = L_{z\_s1} = 3000$ mm
Critical buckling force; $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2 = 947$ kN
Slenderness ratio for buckling - eq 6.50; $\overline{\lambda}_z = \sqrt{}(A \times f_y / N_{cr,z}) = 1.116$
Check z-z axis flexural buckling resistance - Section 6.3.1.1
Buckling curve - Table 6.2; a
Imperfection factor - Table 6.1; $\alpha_z = 0.21$
Buckling reduction determination factor; $\phi_z = 0.5 \times (1 + \alpha_z \times (\overline{\lambda}_z - 0.2) + \overline{\lambda}_z^2) = 1.22$
Buckling reduction factor - eq 6.49; $\chi_z = \min(1 / (\phi_z + \sqrt{}(\phi_z^2 - \overline{\lambda}_z^2)), 1) = 0.585$
Design buckling resistance - eq 6.47; $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} = 690.3$ kN
$N_{Ed} / N_{b,z,Rd} = 0.326$
PASS - Design buckling resistance exceeds design compression
Check bending moment - Section 6.2.5
Design bending moment; $M_{z,Ed} = 12$ kNm
Design bending resistance moment - eq 6.13; $M_{c,z,Rd} = M_{pl,z,Rd} = W_{pl.z} \times f_y / \gamma_{M0} = 34.5$ kNm
$M_{z,Ed} / M_{c,z,Rd} = 0.348$
PASS - Design bending resistance moment exceeds design bending moment
Check bending and axial force - Section 6.2.9
Normal force to plastic resistance force ratio; $n = N_{Ed} / N_{pl,Rd} = 0.191$
$a_f = \min((A - 2 \times h \times t) / A, 0.5) = 0.255$
Reduced plastic moment resistance - Eq.6.40; $M_{N,z,Rd} = M_{pl,z,Rd} \times \min((1 - n) / (1 - 0.5 \times a_f), 1) = 32.0$ kNm
$M_{z,Ed} / M_{N,z,Rd} = 0.375$
PASS - Reduced bending resistance moment exceeds design bending moment
Check combined bending and compression - Section 6.3.3

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Equivalent uniform moment factors - Table B.3; $C_{my} = 1.000$

$C_{mz} = 1.000$

$C_{mLT} = 1.000$

Interaction factors k_{ij} for members not susceptible to torsional deformations - Table B.1

Characteristic moment resistance; $M_{y,Rk} = W_{pl.y} \times f_y = 57.5$ kNm

Characteristic moment resistance; $M_{z,Rk} = W_{pl.z} \times f_y = 34.5$ kNm

Characteristic resistance to normal force; $N_{Rk} = A \times f_y = 1180.5$ kN

Interaction factors; $k_{zz} = C_{mz} \times (1 + \min(\overline{\lambda}_z - 0.2, 0.8) \times N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1})) = 1.261$

$k_{yz} = 0.6 \times k_{zz} = 0.756$

; $\chi_{LT} = 1.000$

Interaction formulae - eq 6.61 & eq 6.62; $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yz} \times M_{z,Ed} / (M_{z,Rk} / \gamma_{M1}) = 0.481$

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zz} \times M_{z,Ed} / (M_{z,Rk} / \gamma_{M1}) = 0.765$

PASS - Combined bending and compression checks are satisfied

Therefore use 160x80x10 RHS

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C1.2 Design

Steel member design (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

Tedds calculation version 4.3.01

Partial factors - Section 6.1

Resistance of cross-sections; $\gamma_{M0} = 1$

Resistance of members to instability; $\gamma_{M1} = 1$

Resistance of tensile members to fracture; $\gamma_{M2} = 1.1$

Design section 1

Section details

Section type; CHS 139.7x8.0 (Tata Steel Celsius)

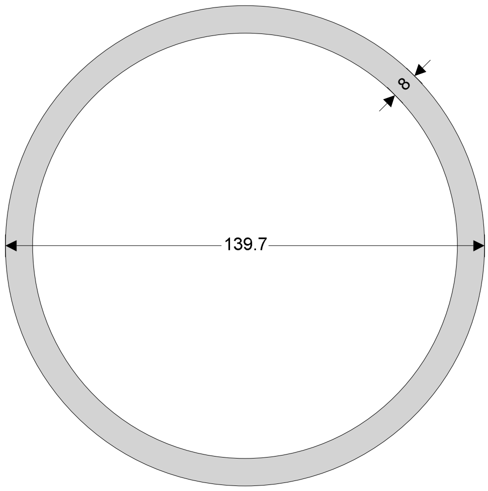
Steel grade - EN 10210-1:2006; S355H

Nominal thickness of element; $t_{nom} = t = 8$ mm

Nominal yield strength; $f_y = 355$ N/mm^2

Nominal ultimate tensile strength; $f_u = 470$ N/mm^2

Modulus of elasticity; $E = 210000$ N/mm^2



CHS 139.7x8.0 (Tata Steel Celsius)
Diameter, d, 139.7 mm
Mass of section, Mass, 26 kg/m
Section thickness, t, 8 mm
Area of section, A, 3310 mm^2
Radius of gyration about y-axis, i_y, 46.649 mm
Radius of gyration about z-axis, i_z, 46.649 mm
Elastic section modulus about y-axis, $W_{el.y}$, 103119 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 103119 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 138930 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 138930 mm^3
Second moment of area about y-axis, I_y, 7202889 mm^4
Second moment of area about z-axis, I_z, 7202889 mm^4

Analysis results

Design bending moment - Major axis; $M_{y,Ed} = 14$ kNm

Design axial compression force; $N_{Ed} = 270$ kN

Restraint spacing

Major axis lateral restraint; $L_y = 2700$ mm

Minor axis lateral restraint; $L_z = 2700$ mm

Torsional restraint; $L_T = 2700$ mm

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.81$

Tubular sections - Table 5.2 (sheet 3 of 3)

$d / t = 17.5 = 26.4 \times \varepsilon^2 <= 50 \times \varepsilon^2$; Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force; $N_{Ed} = 270$ kN

Design resistance of section - eq 6.10; $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0} = 1175$ kN

$N_{Ed} / N_{c,Rd} = 0.23$

PASS - Design compression resistance exceeds design compression

Slenderness ratio for y-y axis flexural buckling - Section 6.3.1.3

Critical buckling length; $L_{cr,y} = L_{y\_s1} = 2700$ mm

Critical buckling force; $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 = 2047.9$ kN

Slenderness ratio for buckling - eq 6.50; $\bar{\lambda}_y = \sqrt{}(A \times f_y / N_{cr,y}) = 0.757$

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2; a

Imperfection factor - Table 6.1; $\alpha_y = 0.21$

Buckling reduction determination factor; $\phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) = 0.845$

Buckling reduction factor - eq 6.49; $\chi_y = \min(1 / (\phi_y + \sqrt{}(\phi_y^2 - \bar{\lambda}_y^2)), 1) = 0.819$

Design buckling resistance - eq 6.47; $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} = 962.5$ kN

$N_{Ed} / N_{b,y,Rd} = 0.281$

PASS - Design buckling resistance exceeds design compression

Slenderness ratio for z-z axis flexural buckling - Section 6.3.1.3

Critical buckling length; $L_{cr,z} = L_{z\_s1} = 2700$ mm

Critical buckling force; $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2 = 2047.9$ kN

Slenderness ratio for buckling - eq 6.50; $\bar{\lambda}_z = \sqrt{}(A \times f_y / N_{cr,z}) = 0.757$

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2; a

Imperfection factor - Table 6.1; $\alpha_z = 0.21$

Buckling reduction determination factor; $\phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2) = 0.845$

Buckling reduction factor - eq 6.49; $\chi_z = \min(1 / (\phi_z + \sqrt{}(\phi_z^2 - \bar{\lambda}_z^2)), 1) = 0.819$

Design buckling resistance - eq 6.47; $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} = 962.5$ kN

$N_{Ed} / N_{b,z,Rd} = 0.281$

PASS - Design buckling resistance exceeds design compression

Check bending moment - Section 6.2.5

Design bending moment; $M_{y,Ed} = 14$ kNm

Design bending resistance moment - eq 6.13; $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = 49.3$ kNm

$M_{y,Ed} / M_{c,y,Rd} = 0.284$

PASS - Design bending resistance moment exceeds design bending moment

Check bending and axial force - Section 6.2.9

Normal force to plastic resistance force ratio; $n = N_{Ed} / N_{pl,Rd} = 0.23$

Reduced plastic moment resistance - Eq.6.39; $M_{N,y,Rd} = M_{pl,y,Rd} \times (1 - n^{1.7}) = 45.3$ kNm

$M_{y,Ed} / M_{N,y,Rd} = 0.309$

PASS - Reduced bending resistance moment exceeds design bending moment

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3; $C_{my} = 1.000$

$C_{mz} = 1.000$

$C_{mLT} = 1.000$

Interaction factors k_{ij} for members not susceptible to torsional deformations - Table B.1

Characteristic moment resistance; $M_{y,Rk} = W_{pl.y} \times f_y = 49.3$ kNm

Characteristic moment resistance; $M_{z,Rk} = W_{pl.z} \times f_y = 49.3$ kNm

Characteristic resistance to normal force; $N_{Rk} = A \times f_y = 1175$ kN

Interaction factors; $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) = 1.156$

$k_{zy} = 0.6 \times k_{yy} = 0.694$

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; χ_{LT} = 1.000

Interaction formulae - eq 6.61 & eq 6.62; $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = 0.609

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = 0.477

PASS - Combined bending and compression checks are satisfied

Therefore use 139.7x8 CHS

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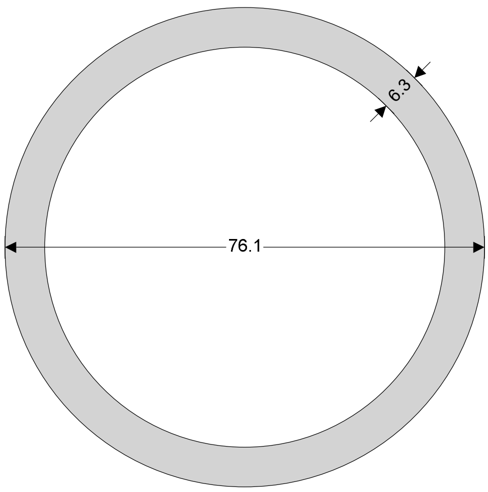
C1.3 Design

Steel member design (EN1993-1-1:2005)
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex
Tedds calculation version 4.3.01
Partial factors - Section 6.1
Resistance of cross-sections; $\gamma_{M0} = 1$
Resistance of members to instability; $\gamma_{M1} = 1$
Resistance of tensile members to fracture; $\gamma_{M2} = 1.1$
Design section 1
Section details
Section type; CHS 76.1x6.3 (Tata Steel Celsius)
Steel grade - EN 10210-1:2006; S355H
Nominal thickness of element; $t_{nom} = t = 6.3$ mm
Nominal yield strength; $f_y = 355$ N/mm^2
Nominal ultimate tensile strength; $f_u = 470$ N/mm^2
Modulus of elasticity; $E = 210000$ N/mm^2



CHS 76.1x6.3 (Tata Steel Celsius)
Diameter, d, 76.1 mm
Mass of section, Mass, 10.8 kg/m
Section thickness, t, 6.3 mm
Area of section, A, 1381 mm^2
Radius of gyration about y-axis, i_y, 24.778 mm
Radius of gyration about z-axis, i_z, 24.778 mm
Elastic section modulus about y-axis, $W_{el.y}$, 22291 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 22291 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 30777 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 30777 mm^3
Second moment of area about y-axis, I_y, 848185 mm^4
Second moment of area about z-axis, I_z, 848185 mm^4

Analysis results
Design bending moment - Major axis; $M_{y,Ed} = 2$ kNm
Design axial compression force; $N_{Ed} = 55$ kN
Restraint spacing
Major axis lateral restraint; $L_y = 3000$ mm
Minor axis lateral restraint; $L_z = 3000$ mm
Torsional restraint; $L_T = 3000$ mm
Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.81$

Tubular sections - Table 5.2 (sheet 3 of 3)

$d / t = 12.1 = 18.2 \times \varepsilon^2 <= 50 \times \varepsilon^2$; Class 1

Section is class 1
Check compression - Section 6.2.4

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Design compression force; N_{Ed} = 55 kN
Design resistance of section - eq 6.10; $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0}$ = 490.4 kN
$N_{Ed} / N_{c,Rd}$ = 0.112
PASS - Design compression resistance exceeds design compression
Slenderness ratio for y-y axis flexural buckling - Section 6.3.1.3
Critical buckling length; $L_{cr,y} = L_{y\_s1}$ = 3000 mm
Critical buckling force; $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2$ = 195.3 kN
Slenderness ratio for buckling - eq 6.50; $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = 1.585
Check y-y axis flexural buckling resistance - Section 6.3.1.1
Buckling curve - Table 6.2; a
Imperfection factor - Table 6.1; α_y = 0.21
Buckling reduction determination factor; $\phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = 1.901
Buckling reduction factor - eq 6.49; $\chi_y = \min(1 / (\phi_y + \sqrt{(\phi_y^2 - \bar{\lambda}_y^2)}), 1)$ = 0.339
Design buckling resistance - eq 6.47; $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = 166.2 kN
$N_{Ed} / N_{b,y,Rd}$ = 0.331
PASS - Design buckling resistance exceeds design compression
Slenderness ratio for z-z axis flexural buckling - Section 6.3.1.3
Critical buckling length; $L_{cr,z} = L_{z\_s1}$ = 3000 mm
Critical buckling force; $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2$ = 195.3 kN
Slenderness ratio for buckling - eq 6.50; $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})}$ = 1.585
Check z-z axis flexural buckling resistance - Section 6.3.1.1
Buckling curve - Table 6.2; a
Imperfection factor - Table 6.1; α_z = 0.21
Buckling reduction determination factor; $\phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2)$ = 1.901
Buckling reduction factor - eq 6.49; $\chi_z = \min(1 / (\phi_z + \sqrt{(\phi_z^2 - \bar{\lambda}_z^2)}), 1)$ = 0.339
Design buckling resistance - eq 6.47; $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1}$ = 166.2 kN
$N_{Ed} / N_{b,z,Rd}$ = 0.331
PASS - Design buckling resistance exceeds design compression
Check bending moment - Section 6.2.5
Design bending moment; $M_{y,Ed}$ = 2 kNm
Design bending resistance moment - eq 6.13; $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = 10.9 kNm
$M_{y,Ed} / M_{c,y,Rd}$ = 0.183
PASS - Design bending resistance moment exceeds design bending moment
Check bending and axial force - Section 6.2.9
Normal force to plastic resistance force ratio; $n = N_{Ed} / N_{pl,Rd}$ = 0.112
Reduced plastic moment resistance - Eq.6.39; $M_{N,y,Rd} = M_{pl,y,Rd} \times (1 - n^{1.7})$ = 10.7 kNm
$M_{y,Ed} / M_{N,y,Rd}$ = 0.188
PASS - Reduced bending resistance moment exceeds design bending moment
Check combined bending and compression - Section 6.3.3
Equivalent uniform moment factors - Table B.3; C_{my} = 1.000
C_{mz} = 1.000
C_{mLT} = 1.000
Interaction factors k_{ij} for members not susceptible to torsional deformations - Table B.1
Characteristic moment resistance; $M_{y,Rk} = W_{pl.y} \times f_y$ = 10.9 kNm
Characteristic moment resistance; $M_{z,Rk} = W_{pl.z} \times f_y$ = 10.9 kNm
Characteristic resistance to normal force; $N_{Rk} = A \times f_y$ = 490.4 kN

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Interaction factors; $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) = 1.265$
$k_{zy} = 0.6 \times k_{yy} = 0.759$
; $\chi_{LT} = 1.000$
Interaction formulae - eq 6.61 & eq 6.62; $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) = 0.562$
$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) = 0.47$
PASS - Combined bending and compression checks are satisfied

Therefore use 76.1x6.3 CHS

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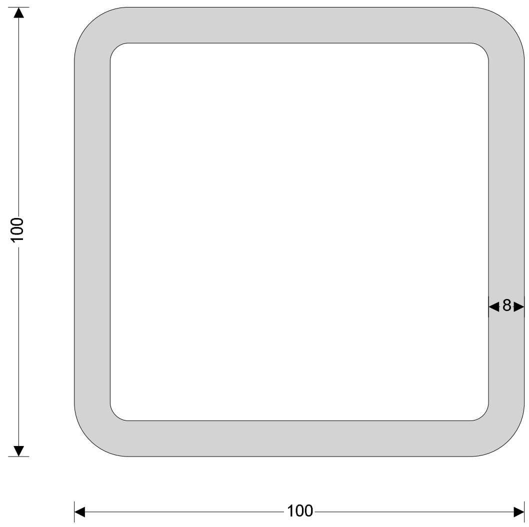
C1.4 Design

Steel member design (EN1993-1-1:2005)
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex
Tedds calculation version 4.3.01

Partial factors - Section 6.1
Resistance of cross-sections; $\gamma_{M0} = 1$
Resistance of members to instability; $\gamma_{M1} = 1$
Resistance of tensile members to fracture; $\gamma_{M2} = 1.1$

Design section 1
Section details
Section type; SHS 100x100x8.0 (Tata Steel Celsius)
Steel grade - EN 10210-1:2006; S355H
Nominal thickness of element; $t_{nom} = t = 8$ mm
Nominal yield strength; $f_y = 355$ N/mm^2
Nominal ultimate tensile strength; $f_u = 470$ N/mm^2
Modulus of elasticity; $E = 210000$ N/mm^2



SHS 100x100x8.0 (Tata Steel Celsius)
Section depth, h, 100 mm
Section breadth, b, 100 mm
Mass of section, Mass, 22.6 kg/m
Section thickness, t, 8 mm
Area of section, A, 2875 mm^2
Radius of gyration about y-axis, i_y, 37.279 mm
Radius of gyration about z-axis, i_z, 37.279 mm
Elastic section modulus about y-axis, $W_{el.y}$, 79919 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 79919 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 98184 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 98184 mm^3
Second moment of area about y-axis, I_y, 3995961 mm^4
Second moment of area about z-axis, I_z, 3995961 mm^4

Analysis results
Design bending moment - Major axis; $M_{y,Ed} = 11$ kNm
Design axial compression force; $N_{Ed} = 240$ kN
Restraint spacing
Major axis lateral restraint; $L_y = 2700$ mm
Minor axis lateral restraint; $L_z = 2700$ mm
Torsional restraint; $L_T = 2700$ mm
Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.81$

Internal compression parts subject to bending and compression - Table 5.2 (sheet 1 of 3)
Width of section; $c = h - 3 \times t = 76$ mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times 2 \times t \times f_y) - 3 \times t / 2] / c, 1) = 0.778$

$c / t = 9.5 = 11.7 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$; Class 1

Internal compression parts subject to compression - Table 5.2 (sheet 1 of 3)

Width of section; $c = b - 3 \times t = 76$ mm

$c / t = 9.5 = 11.7 \times \varepsilon <= 33 \times \varepsilon$; Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force; $N_{Ed} = 240$ kN

Design resistance of section - eq 6.10; $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0} = 1020.7$ kN

$N_{Ed} / N_{c,Rd} = 0.235$

PASS - Design compression resistance exceeds design compression

Slenderness ratio for y-y axis flexural buckling - Section 6.3.1.3

Critical buckling length; $L_{cr,y} = L_{y\_s1} = 2700$ mm

Critical buckling force; $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 = 1136.1$ kN

Slenderness ratio for buckling - eq 6.50; $\bar{\lambda}_y = \sqrt{}(A \times f_y / N_{cr,y}) = 0.948$

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2; a

Imperfection factor - Table 6.1; $\alpha_y = 0.21$

Buckling reduction determination factor; $\phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) = 1.028$

Buckling reduction factor - eq 6.49; $\chi_y = \min(1 / (\phi_y + \sqrt{}(\phi_y^2 - \bar{\lambda}_y^2)), 1) = 0.702$

Design buckling resistance - eq 6.47; $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} = 716.3$ kN

$N_{Ed} / N_{b,y,Rd} = 0.335$

PASS - Design buckling resistance exceeds design compression

Slenderness ratio for z-z axis flexural buckling - Section 6.3.1.3

Critical buckling length; $L_{cr,z} = L_{z\_s1} = 2700$ mm

Critical buckling force; $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2 = 1136.1$ kN

Slenderness ratio for buckling - eq 6.50; $\bar{\lambda}_z = \sqrt{}(A \times f_y / N_{cr,z}) = 0.948$

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2; a

Imperfection factor - Table 6.1; $\alpha_z = 0.21$

Buckling reduction determination factor; $\phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2) = 1.028$

Buckling reduction factor - eq 6.49; $\chi_z = \min(1 / (\phi_z + \sqrt{}(\phi_z^2 - \bar{\lambda}_z^2)), 1) = 0.702$

Design buckling resistance - eq 6.47; $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} = 716.3$ kN

$N_{Ed} / N_{b,z,Rd} = 0.335$

PASS - Design buckling resistance exceeds design compression

Check bending moment - Section 6.2.5

Design bending moment; $M_{y,Ed} = 11$ kNm

Design bending resistance moment - eq 6.13; $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = 34.9$ kNm

$M_{y,Ed} / M_{c,y,Rd} = 0.316$

PASS - Design bending resistance moment exceeds design bending moment

Check bending and axial force - Section 6.2.9

Normal force to plastic resistance force ratio; $n = N_{Ed} / N_{pl,Rd} = 0.235$

$a_w = \min((A - 2 \times b \times t) / A, 0.5) = 0.444$

Reduced plastic moment resistance - Eq.6.39; $M_{N,y,Rd} = M_{pl,y,Rd} \times \min((1 - n) / (1 - 0.5 \times a_w), 1) = 34.3$ kNm

$M_{y,Ed} / M_{N,y,Rd} = 0.321$

PASS - Reduced bending resistance moment exceeds design bending moment

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Check combined bending and compression - Section 6.3.3
Equivalent uniform moment factors - Table B.3; C_{my} = 1.000
C_{mz} = 1.000
C_{mLT} = 1.000
Interaction factors k_{ij} for members not susceptible to torsional deformations - Table B.1
Characteristic moment resistance; $M_{y,Rk} = W_{pl.y} \times f_y$ = 34.9 kNm
Characteristic moment resistance; $M_{z,Rk} = W_{pl.z} \times f_y$ = 34.9 kNm
Characteristic resistance to normal force; $N_{Rk} = A \times f_y$ = 1020.7 kN
Interaction factors; $k_{yy} = C_{my} \times (1 + \min(\overline{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = 1.251
$k_{zy} = 0.6 \times k_{yy}$ = 0.750
; χ_{LT} = 1.000
Interaction formulae - eq 6.61 & eq 6.62; $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = 0.73
$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = 0.572
PASS - Combined bending and compression checks are satisfied

Therefore use 100x100x6.3 SHS

Ground Floor Member Design

J0.1 Design

SCI	Tata Steel		v9.0.30.0
Job Reference:		Date:	13/3/2019
Deck Reference:	CF60/1.2_350	Time:	15:38:22
Company Name:		Job No:	
Client Name:		Calcs By:	
Checked By:		File Name:	J0.1.pmd

Summary Output

Note: Section Designed to Eurocodes, United Kingdom National Annex

Construction Stage:	PASS	**Max Unity Factor:**	0.52
Normal Stage:	PASS	**Max Unity Factor:**	0.48
Fire Condition:	PASS	**Max Unity Factor:**	0.00
Serviceability:	SATISFACTORY	**Max Unity Factor:**	0.86

***** Section Adequate *****

Floor Plan Data (propped composite construction with ComFlor 60/1.2/S350 decking)

Beam centres	4.60 m	Profile span type	Single
Beam or wall width	152 mm	Propping	Single (at 1/2 span)
Prop width	100 mm		
		Concrete span type	Single

Profile Data (ComFlor 60/1.2/S350 decking. Grade C30/37)

Depth	60 mm	Pitch of deck ribs	300 mm
Trough width	120 mm	Crest width	130.7 mm
Nominal sheet thickness	1.20 mm	Design sheet thickness	1.16 mm
Deck weight	0.14 kN/m²	Yield strength	350 N/mm²

Concrete Slab (Normal Weight Concrete ; Mesh : A393)

Overall slab depth	150 mm		
Concrete characteristic strength	30 N/mm²	Concrete wet density	2550 kg/m³
Modular ratio	10	Concrete dry density	2450 kg/m³
Bar reinforcement :			
Diameter	8 mm	Yield strength	500 N/mm²
Distance from slab soffit	30 mm		
Mesh reinforcement :			
Mesh	A393	Yield strength	500 N/mm²
Cover to Mesh	30 mm	Mesh Layers	Single
Account for End Anchorage	No	Shear connectors per rib	N/A
Diameter of Shear Connectors	N/A		
Screed depth	75 mm	Screed density	2000 kg/m³

Section Properties

*** Note - 1: All values of inertia are expressed in steel units
*** Note - 2: Average inertia is used for deflection calculations for the composite stage
*** Note - 3: Cracked dynamic inertia is used for natural frequency calculations

Deck Profile

Sagging Inertia, Iy	132.910 cm4/m	Area of profile (Net), Ap	1721 mm²/m
Hogging Inertia, Iy	121.600 cm4/m	Effective area of profile	1576.00 mm²/m

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Composite

Inertia, Iy - Uncracked	2472 cm4/m	Inertia, Iy - Cracked	1385 cm4/m
Average inertia	1928 cm4/m	Cracked inertia (dynamic)	1569 cm4/m
Shear bond coefficients - Mr	178.39	Kr	0.099400
Concrete volume	0.118 m³/m/m		

Loads Acting on Slab (Actions)

*** Note: Slab subjected to uniformly distributed loads (UDL) ONLY

Imposed (occupancy)	1.50 kN/m²	Partitions	0.00 kN/m²
Ceilings and services	0.20 kN/m²	Finishes	0.80 kN/m²
Self weight of concrete slab (wet)	2.96 kN/m²	Self weight of decking	0.14 kN/m²
Self weight of concrete slab (dry)	2.84 kN/m²	Self weight of screeds	1.47 kN/m²
Construction load	1.50 kN/m²		

Line Loads Perpendicular to Deck Span (Actions)

None

Line Loads Parallel to Deck Span (Actions)

None

Fire Data

Design method	Bar Method	Fire resistance period	30 mins
Non-permanent imposed loads	N/A		

Partial Safety Factors

Actions		**Materials**	
Permanent, gamma G	1.35	Structural steel - elastic, gamma M0	1.00
Permanent - accidental, gamma GA	N/A	Structural steel - buckling, gamma M1	1.00
Variable, gamma Q	1.50	Concrete, gamma C	1.50
Combination factor - Fire, psi 1	0.70	Reinforcement, gamma S	1.15
Combination factor, psi 0	0.70	Combination factor, psi 2	0.60

Therefore use Tata Comflor 60 1.2mm gauge 150 slab A393 top H8 in trough single row of props in temporary condition

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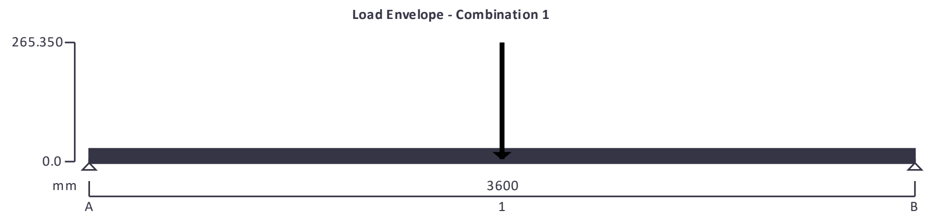
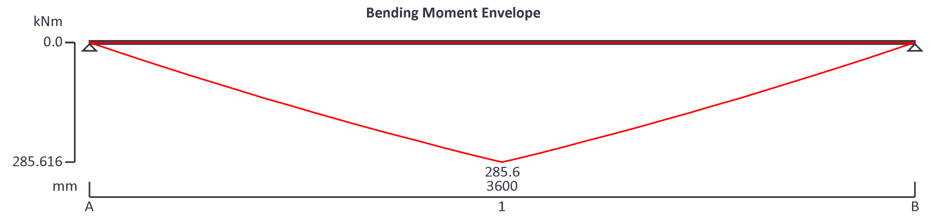
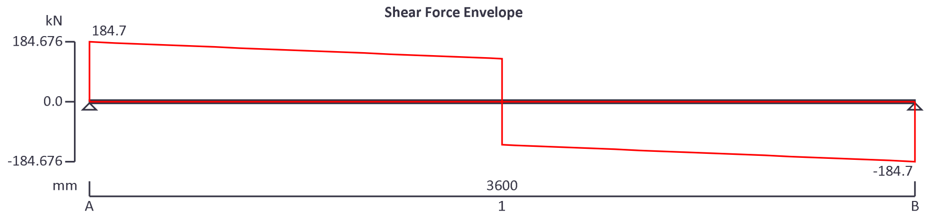
B0.1 Design

Steel beam analysis & design (EN1993-1-1:2005)
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex
TEDDS calculation version 3.0.13







Support conditions
Support A Vertically restrained
Rotationally free
Support B Vertically restrained
Rotationally free
Applied loading
Beam loads Permanent self weight of beam × 1
Permanent full UDL 15 kN/m
Variable full UDL 5 kN/m
Permanent point load 141 kN at 1800 mm
Variable point load 50 kN at 1800 mm
Load combinations
Load combination 1 Support A Permanent × 1.35
Variable × 1.50
Permanent × 1.35

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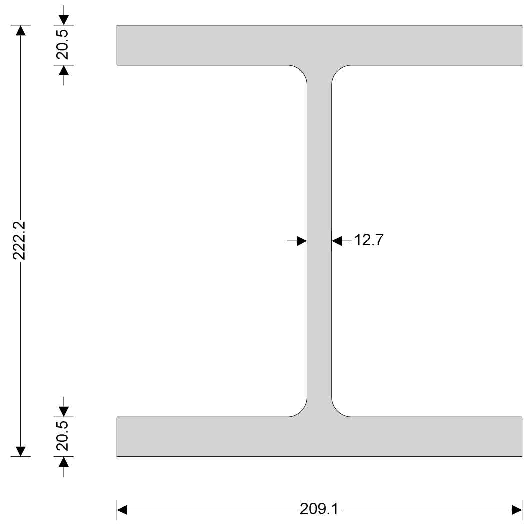
Variable × 1.50
Support B Permanent × 1.35
Variable × 1.50

Analysis results
Maximum moment; M_{max} = 285.6 kNm; M_{min} = 0 kNm
Maximum shear; V_{max} = 184.7 kN; V_{min} = -184.7 kN
Deflection; δ_{max} = 11.7 mm; δ_{min} = 0 mm
Maximum reaction at support A; R_{A_max} = 184.7 kN; R_{A_min} = 184.7 kN
Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = 99 kN
Unfactored variable load reaction at support A; $R_{A\_Variable}$ = 34 kN
Maximum reaction at support B; R_{B_max} = 184.7 kN; R_{B_min} = 184.7 kN
Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = 99 kN
Unfactored variable load reaction at support B; $R_{B\_Variable}$ = 34 kN

Section details
Section type; UKC 203x203x86 (Tata Steel Advance)
Steel grade; S355
EN 10025-2:2004 - Hot rolled products of structural steels
Nominal thickness of element; $t = \max(t_f, t_w)$ = 20.5 mm
Nominal yield strength; f_y = 345 N/mm^2
Nominal ultimate tensile strength; f_u = 470 N/mm^2
Modulus of elasticity; E = 210000 N/mm^2



Partial factors - Section 6.1
Resistance of cross-sections; γ_{M0} = 1.00
Resistance of members to instability; γ_{M1} = 1.00
Resistance of tensile members to fracture; γ_{M2} = 1.10

Lateral restraint
Span 1 has lateral restraint at supports only

Effective length factors
Effective length factor in major axis; K_y = 1.000
Effective length factor in minor axis; K_z = 1.000
Effective length factor for torsion; $K_{LT.A}$ = 0.800;

$K_{LT.B}$ = 0.800;
Classification of cross sections - Section 5.5
$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.83$
Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)
Width of section; c = d = 160.8 mm
$c / t_w = 15.3 \times \varepsilon <= 72 \times \varepsilon$; Class 1
Outstand flanges - Table 5.2 (sheet 2 of 3)
Width of section; $c = (b - t_w - 2 \times r) / 2 = 88$ mm
$c / t_f = 5.2 \times \varepsilon <= 9 \times \varepsilon$; Class 1
Section is class 1
Check shear - Section 6.2.6
Height of web; $h_w = h - 2 \times t_f = 181.2$ mm
Shear area factor; $\eta = 1.000$
$h_w / t_w < 72 \times \varepsilon / \eta$
Shear buckling resistance can be ignored
Design shear force; $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = 184.7$ kN
Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 3069 \text{ mm}^2$
Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = 611.3$ kN
PASS - Design shear resistance exceeds design shear force
Check bending moment major (y-y) axis - Section 6.2.5
Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = 285.6$ kNm
Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = 337$ kNm
Slenderness ratio for lateral torsional buckling
Correction factor - Table 6.6; $k_c = 1$
$C_1 = 1 / k_c^2 = 1$
Destabilised load condition factor; D = 1.5
Curvature factor; $g = \sqrt{[1 - (I_z / I_y)]} = 0.818$
Poissons ratio; $\nu = 0.3$
Shear modulus; $G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$
Unrestrained length; $L = 0.8 \times L_{s1} = 2880$ mm
Elastic critical buckling moment;$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = 1489.7 kNm
Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = D \times \sqrt{(W_{pl.y} \times f_y / M_{cr})} = 0.713$
Limiting slenderness ratio; $\bar{\lambda}_{LT,0} = 0.4$
$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored
Design resistance for buckling - Section 6.3.2.1
Buckling curve - Table 6.5; b
Imperfection factor - Table 6.3; $\alpha_{LT} = 0.34$
Correction factor for rolled sections; $\beta = 0.75$
LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 0.744$
LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.863$
Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 1.000$
Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.863$
Design buckling resistance moment - eq 6.55; $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = 290.8$ kNm
PASS - Design buckling resistance moment exceeds design bending moment
Check vertical deflection - Section 7.2.1

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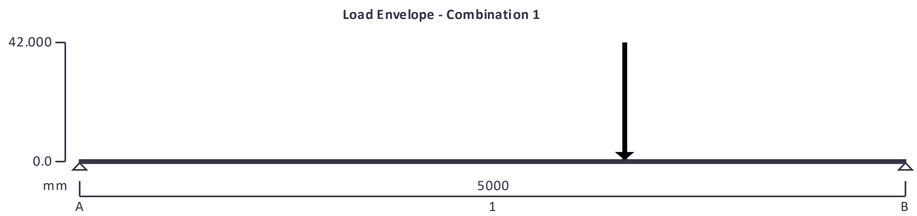
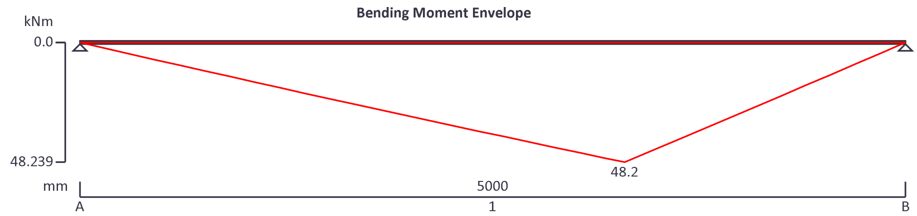
Consider deflection due to permanent and variable loads
Limiting deflection; $\delta_{lim} = L_{s1} / 250 = 14.4$ mm
Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 11.654$ mm
PASS - Maximum deflection does not exceed deflection limit

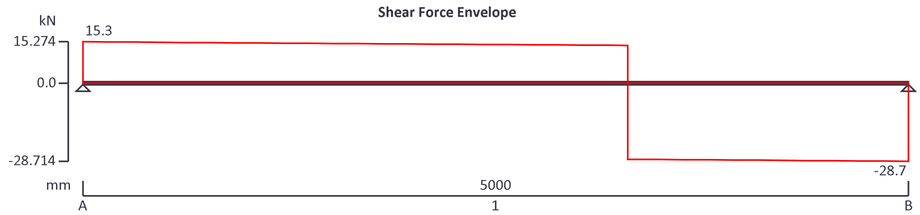
Therefore use 203UC86 S355

B0.2 Design

Steel beam analysis & design (EN1993-1-1:2005)
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex
TEDDS calculation version 3.0.13







Support conditions
Support A Vertically restrained
Rotationally free
Support B Vertically restrained
Rotationally free
Applied loading
Beam loads Permanent self weight of beam × 1
Permanent point load 20 kN at 3300 mm
Variable point load 10 kN at 3300 mm
Load combinations
Load combination 1 Support A Permanent × 1.35
Variable × 1.50
Permanent × 1.35
Variable × 1.50
Support B Permanent × 1.35

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Variable × 1.50

Analysis results

Maximum moment; M_{max} = 48.2 kNm; M_{min} = 0 kNm

Maximum shear; V_{max} = 15.3 kN; V_{min} = -28.7 kN

Deflection; δ_{max} = 6.2 mm; δ_{min} = 0 mm

Maximum reaction at support A; R_{A_max} = 15.3 kN; R_{A_min} = 15.3 kN

Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = 7.5 kN

Unfactored variable load reaction at support A; $R_{A\_Variable}$ = 3.4 kN

Maximum reaction at support B; R_{B_max} = 28.7 kN; R_{B_min} = 28.7 kN

Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = 13.9 kN

Unfactored variable load reaction at support B; $R_{B\_Variable}$ = 6.6 kN

Section details

Section type; UKC 152x152x30 (Tata Steel Advance)

Steel grade; S275

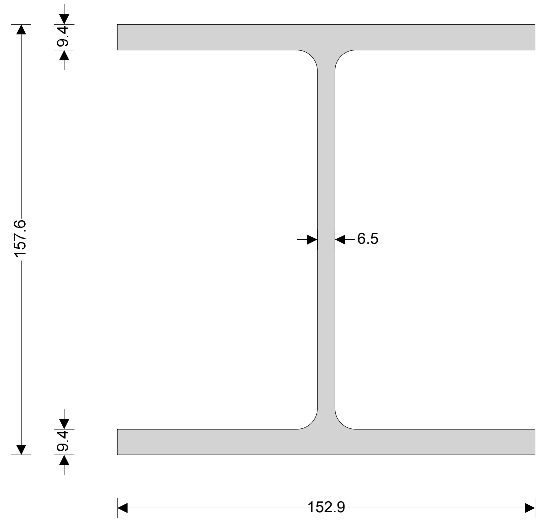
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element; $t = \max(t_f, t_w)$ = 9.4 mm

Nominal yield strength; f_y = 275 N/mm^2

Nominal ultimate tensile strength; f_u = 410 N/mm^2

Modulus of elasticity; E = 210000 N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections; γ_{M0} = 1.00

Resistance of members to instability; γ_{M1} = 1.00

Resistance of tensile members to fracture; γ_{M2} = 1.10

Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis; K_y = 1.000

Effective length factor in minor axis; K_z = 1.000

Effective length factor for torsion; $K_{LT.A}$ = 0.800;

$K_{LT.B}$ = 0.800;

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.92$
Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)
Width of section; $c = d = 123.6$ mm
$c / t_w = 20.6 \times \varepsilon <= 72 \times \varepsilon$; Class 1
Outstand flanges - Table 5.2 (sheet 2 of 3)
Width of section; $c = (b - t_w - 2 \times r) / 2 = 65.6$ mm
$c / t_f = 7.5 \times \varepsilon <= 9 \times \varepsilon$; Class 1
Section is class 1
Check shear - Section 6.2.6
Height of web; $h_w = h - 2 \times t_f = 138.8$ mm
Shear area factor; $\eta = 1.000$
$h_w / t_w < 72 \times \varepsilon / \eta$
Shear buckling resistance can be ignored
Design shear force; $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = 28.7$ kN
Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1156 \text{ mm}^2$
Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = 183.5$ kN
PASS - Design shear resistance exceeds design shear force
Check bending moment major (y-y) axis - Section 6.2.5
Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = 48.2$ kNm
Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = 68.1$ kNm
PASS - Design bending resistance moment exceeds design bending moment
Check vertical deflection - Section 7.2.1
Consider deflection due to variable loads
Limiting deflection; $\delta_{lim} = L_{s1} / 360 = 13.9$ mm
Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 6.177$ mm
PASS - Maximum deflection does not exceed deflection limit

Therefore use 152UC30

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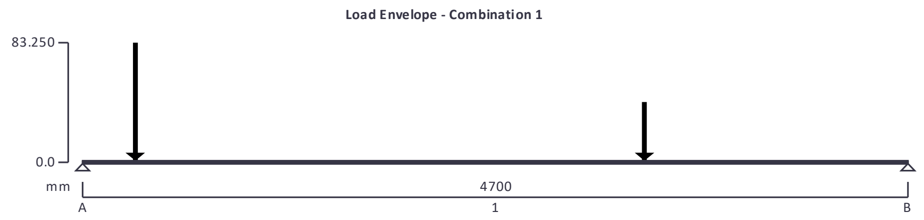
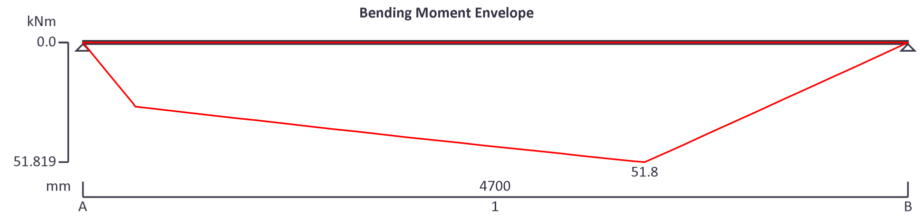
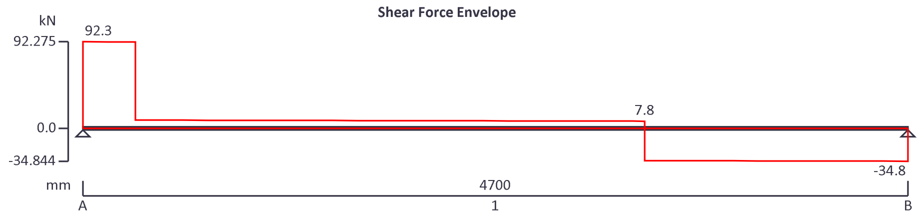
B0.3 Design

Steel beam analysis & design (EN1993-1-1:2005)
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex
TEDDS calculation version 3.0.13

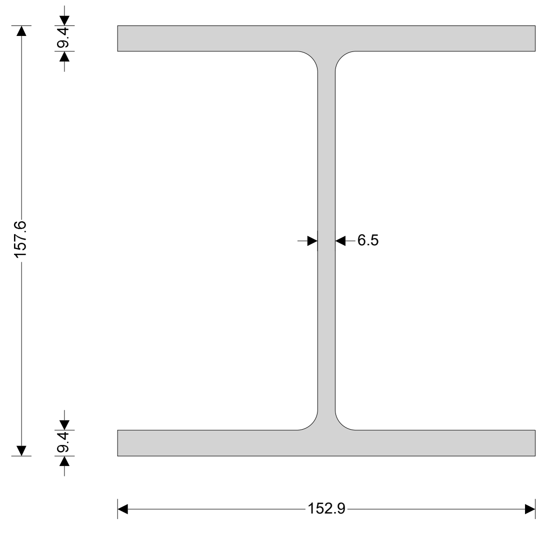






Support conditions
Support A Vertically restrained
Rotationally free
Support B Vertically restrained
Rotationally free
Applied loading
Beam loads Permanent self weight of beam × 1
Permanent point load 45 kN at 300 mm
Variable point load 15 kN at 300 mm
Permanent point load 20 kN at 3200 mm
Variable point load 10 kN at 3200 mm
Load combinations
Load combination 1 Support A Permanent × 1.35
Variable × 1.50
Permanent × 1.35

Variable × 1.50
Support B Permanent × 1.35
Variable × 1.50
Analysis results
Maximum moment; M_{max} = 51.8 kNm; M_{min} = 0 kNm
Maximum shear; V_{max} = 92.3 kN; V_{min} = -34.8 kN
Deflection; δ_{max} = 6.6 mm; δ_{min} = 0 mm
Maximum reaction at support A; R_{A_max} = 92.3 kN; R_{A_min} = 92.3 kN
Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = 49.2 kN
Unfactored variable load reaction at support A; $R_{A\_Variable}$ = 17.2 kN
Maximum reaction at support B; R_{B_max} = 34.8 kN; R_{B_min} = 34.8 kN
Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = 17.2 kN
Unfactored variable load reaction at support B; $R_{B\_Variable}$ = 7.8 kN
Section details
Section type; UKC 152x152x30 (Tata Steel Advance)
Steel grade; S275
EN 10025-2:2004 - Hot rolled products of structural steels
Nominal thickness of element; $t = \max(t_f, t_w)$ = 9.4 mm
Nominal yield strength; f_y = 275 N/mm^2
Nominal ultimate tensile strength; f_u = 410 N/mm^2
Modulus of elasticity; E = 210000 N/mm^2



Partial factors - Section 6.1
Resistance of cross-sections; γ_{M0} = 1.00
Resistance of members to instability; γ_{M1} = 1.00
Resistance of tensile members to fracture; γ_{M2} = 1.10
Lateral restraint
Span 1 has full lateral restraint
Effective length factors
Effective length factor in major axis; K_y = 1.000
Effective length factor in minor axis; K_z = 1.000
Effective length factor for torsion; $K_{LT.A}$ = 0.800;

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$K_{LT.B}$ = 0.800;

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.92$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section; c = d = 123.6 mm

$c / t_w = 20.6 \times \varepsilon <= 72 \times \varepsilon$; Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section; $c = (b - t_w - 2 \times r) / 2 = 65.6$ mm

$c / t_f = 7.5 \times \varepsilon <= 9 \times \varepsilon$; Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f = 138.8$ mm

Shear area factor; $\eta = 1.000$

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = 92.3$ kN

Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1156 \text{ mm}^2$

Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = 183.5$ kN

PASS - Design shear resistance exceeds design shear force

Combined bending and shear - Section 6.2.8

Reduction factor - cl.6.2.8(3); $\rho_v = [(2 \times V_{Ed} / V_{pl,Rd}) - 1]^2 = 0$

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = 51.8$ kNm

Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} = [(W_{pl.y} - t_w \times h^2 / 4) + (t_w \times h^2 / 4) \times (1 - \rho_v)] \times f_y / \gamma_{M0} = 68.1$ kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection; $\delta_{lim} = L_{s1} / 360 = 13.1$ mm

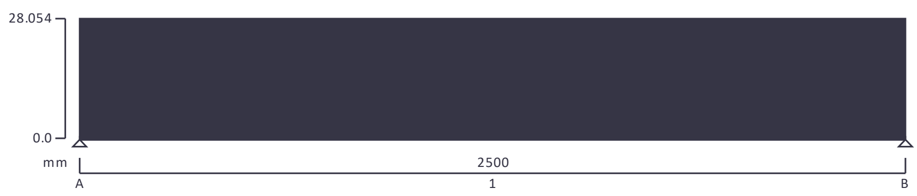
Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 6.569$ mm

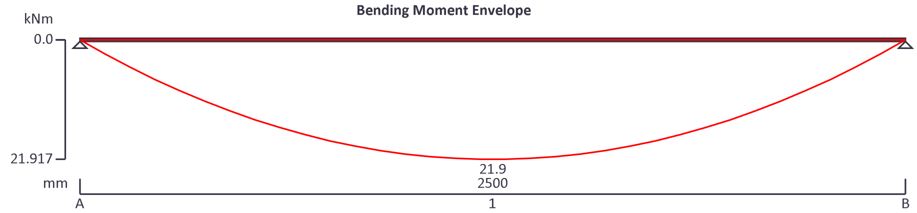
PASS - Maximum deflection does not exceed deflection limit

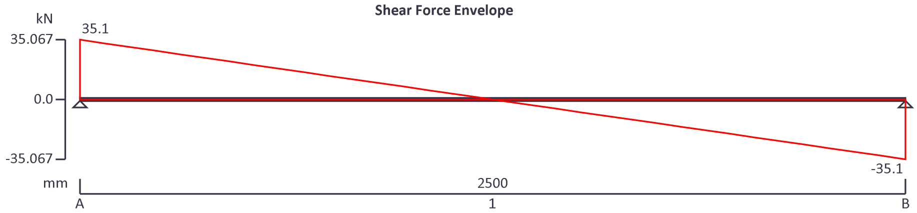
Therefore use 152UC30

B0.5 Design

Steel beam analysis & design (EN1993-1-1:2005)
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex
TEDDS calculation version 3.0.13







Support conditions
Support A Vertically restrained
Rotationally free
Support B Vertically restrained
Rotationally free
Applied loading
Beam loads Permanent self weight of beam × 1
Permanent full UDL 15 kN/m
Variable full UDL 5 kN/m
Load combinations
Load combination 1 Support A Permanent × 1.35
Variable × 1.50
Permanent × 1.35
Variable × 1.50
Support B Permanent × 1.35

Variable × 1.50

Analysis results

Maximum moment; M_{max} = 21.9 kNm; M_{min} = 0 kNm

Maximum shear; V_{max} = 35.1 kN; V_{min} = -35.1 kN

Deflection; δ_{max} = 1 mm; δ_{min} = 0 mm

Maximum reaction at support A; R_{A_max} = 35.1 kN; R_{A_min} = 35.1 kN

Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = 19 kN

Unfactored variable load reaction at support A; $R_{A\_Variable}$ = 6.3 kN

Maximum reaction at support B; R_{B_max} = 35.1 kN; R_{B_min} = 35.1 kN

Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = 19 kN

Unfactored variable load reaction at support B; $R_{B\_Variable}$ = 6.3 kN

Section details

Section type; UKC 152x152x23 (Tata Steel Advance)

Steel grade; S355

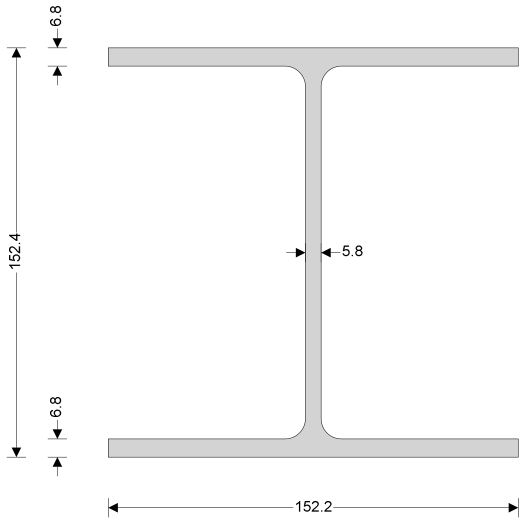
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element; $t = \max(t_f, t_w)$ = 6.8 mm

Nominal yield strength; f_y = 355 N/mm^2

Nominal ultimate tensile strength; f_u = 470 N/mm^2

Modulus of elasticity; E = 210000 N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections; γ_{M0} = 1.00

Resistance of members to instability; γ_{M1} = 1.00

Resistance of tensile members to fracture; γ_{M2} = 1.10

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis; K_y = 1.000

Effective length factor in minor axis; K_z = 1.000

Effective length factor for torsion; $K_{LT.A}$ = 1.200; + 2 × h

$K_{LT.B}$ = 1.200; + 2 × h

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]} = 0.81$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section; $c = d = 123.6$ mm

$c / t_w = 26.2 \times \varepsilon <= 72 \times \varepsilon$; Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section; $c = (b - t_w - 2 \times r) / 2 = 65.6$ mm

$c / t_f = 11.9 \times \varepsilon <= 14 \times \varepsilon$; Class 3

Section is class 3

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f = 138.8$ mm

Shear area factor; $\eta = 1.000$

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $V_{Ed} = \max(\mathrm{abs}(V_{max}), \mathrm{abs}(V_{min})) = 35.1$ kN

Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 997\ mm^2$

Design shear resistance - cl 6.2.6(2); $V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = 204.4$ kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment; $M_{Ed} = \max(\mathrm{abs}(M_{s1\_max}), \mathrm{abs}(M_{s1\_min})) = 21.9$ kNm

Design bending resistance moment - eq 6.14; $M_{c,Rd} = M_{el,Rd} = W_{el.y} \times f_y / \gamma_{M0} = 58.2$ kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6; $k_c = 1$

$C_1 = 1 / k_c^2 = 1$

Destabilised load condition factor; $D = 1.5$

Curvature factor; $g = \sqrt{[1 - (I_z / I_y)]} = 0.825$

Poissons ratio; $\nu = 0.3$

Shear modulus; $G = E / [2 \times (1 + \nu)] = 80769\ N/mm^2$

Unrestrained length; $L = 1.2 \times L_{s1} + 2 \times h = 3305$ mm

Elastic critical buckling moment; $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = 93.1 kNm

Slenderness ratio for lateral torsional buckling; $\overline{\lambda}_{LT} = D \times \sqrt{(W_{el.y} \times f_y / M_{cr})} = 1.186$

Limiting slenderness ratio; $\overline{\lambda}_{LT,0} = 0.4$

$\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5; b

Imperfection factor - Table 6.3; $\alpha_{LT} = 0.34$

Correction factor for rolled sections; $\beta = 0.75$

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}) + \beta \times \overline{\lambda}_{LT}^2] = 1.161$

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \overline{\lambda}_{LT}^2)}], 1, 1 / \overline{\lambda}_{LT}^2) = 0.587$

Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\overline{\lambda}_{LT} - 0.8)^2], 1) = 1.000$

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.587$

Design buckling resistance moment - eq 6.55; $M_{b,Rd} = \chi_{LT,mod} \times W_{el.y} \times f_y / \gamma_{M1} = 34.2$ kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

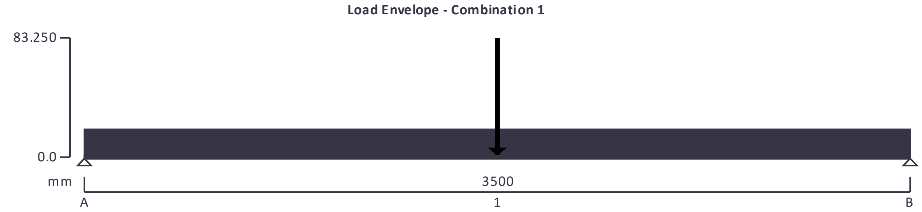
Limiting deflection; $\delta_{lim} = L_{s1} / 360 = 6.9$ mm

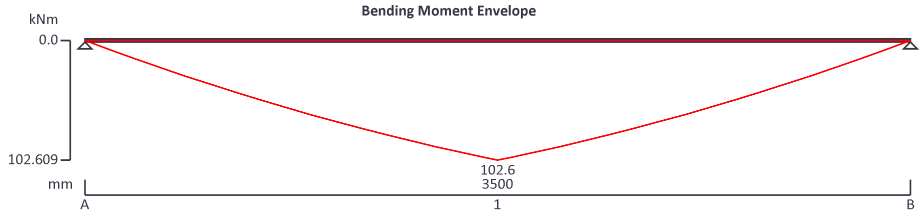
Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 0.969$ mm
PASS - Maximum deflection does not exceed deflection limit

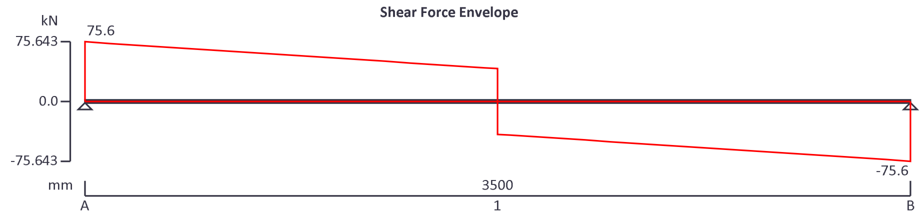
Therefore use 152UC23

B0.6 Design

Steel beam analysis & design (EN1993-1-1:2005)
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex
TEDDS calculation version 3.0.13

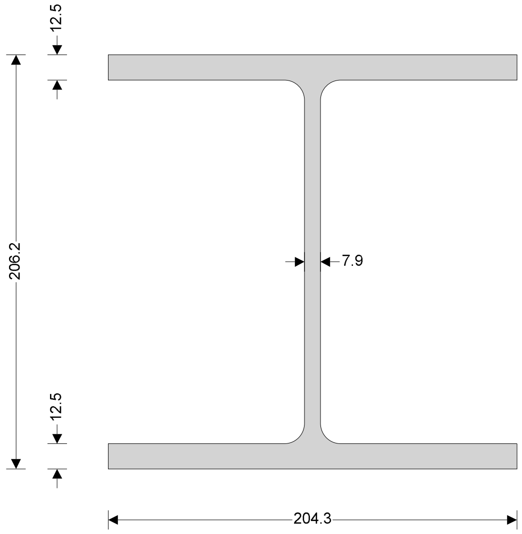






Support conditions
Support A Vertically restrained
Rotationally free
Support B Vertically restrained
Rotationally free
Applied loading
Beam loads Permanent self weight of beam × 1
Permanent full UDL 10 kN/m
Variable full UDL 3.5 kN/m
Permanent point load 45 kN at 1750 mm
Variable point load 15 kN at 1750 mm
Load combinations
Load combination 1 Support A Permanent × 1.35
Variable × 1.50
Permanent × 1.35

Variable $\times$ 1.50
Support B Permanent $\times$ 1.35
Variable $\times$ 1.50
Analysis results
Maximum moment; M_{max} = 102.6 kNm; M_{min} = 0 kNm
Maximum shear; V_{max} = 75.6 kN; V_{min} = -75.6 kN
Deflection; δ_{max} = 1.8 mm; δ_{min} = 0 mm
Maximum reaction at support A; R_{A_max} = 75.6 kN; R_{A_min} = 75.6 kN
Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = 40.9 kN
Unfactored variable load reaction at support A; $R_{A\_Variable}$ = 13.6 kN
Maximum reaction at support B; R_{B_max} = 75.6 kN; R_{B_min} = 75.6 kN
Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = 40.9 kN
Unfactored variable load reaction at support B; $R_{B\_Variable}$ = 13.6 kN
Section details
Section type; UKC 203x203x52 (Tata Steel Advance)
Steel grade; S275
EN 10025-2:2004 - Hot rolled products of structural steels
Nominal thickness of element; $t = \max(t_f, t_w)$ = 12.5 mm
Nominal yield strength; f_y = 275 N/mm^2
Nominal ultimate tensile strength; f_u = 410 N/mm^2
Modulus of elasticity; E = 210000 N/mm^2



Partial factors - Section 6.1
Resistance of cross-sections; γ_{M0} = 1.00
Resistance of members to instability; γ_{M1} = 1.00
Resistance of tensile members to fracture; γ_{M2} = 1.10
Lateral restraint
Span 1 has lateral restraint at supports only
Effective length factors
Effective length factor in major axis; K_y = 1.000
Effective length factor in minor axis; K_z = 1.000
Effective length factor for torsion; $K_{LT.A}$ = 1.200; + 2 $\times$ h

$K_{LT.B}$ = 1.200; + 2 × h

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]} = 0.92$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section; c = d = 160.8 mm

$c / t_w = 22.0 \times \varepsilon <= 72 \times \varepsilon$; Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section; $c = (b - t_w - 2 \times r) / 2 = 88$ mm

$c / t_f = 7.6 \times \varepsilon <= 9 \times \varepsilon$; Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f = 181.2$ mm

Shear area factor; $\eta = 1.000$

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = 75.6$ kN

Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1875\ mm^2$

Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = 297.6$ kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = 102.6$ kNm

Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = 156$ kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6; $k_c = 1$

$C_1 = 1 / k_c^2 = 1$

Destabilised load condition factor; D = 1.5

Curvature factor; $g = \sqrt{[1 - (I_z / I_y)]} = 0.814$

Poissons ratio; $\nu = 0.3$

Shear modulus; $G = E / [2 \times (1 + \nu)] = 80769\ N/mm^2$

Unrestrained length; $L = 1.2 \times L_{s1} + 2 \times h = 4612$ mm

Elastic critical buckling moment;$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = 331.1 kNm

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = D \times \sqrt{(W_{pl.y} \times f_y / M_{cr})} = 1.03$

Limiting slenderness ratio; $\bar{\lambda}_{LT,0} = 0.4$

$\bar{\lambda}_{LT}$ > $\bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5; b

Imperfection factor - Table 6.3; $\alpha_{LT} = 0.34$

Correction factor for rolled sections; $\beta = 0.75$

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 1.005$

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.681$

Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 1.000$

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.681$

Design buckling resistance moment - eq 6.55; $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = 106.3$ kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

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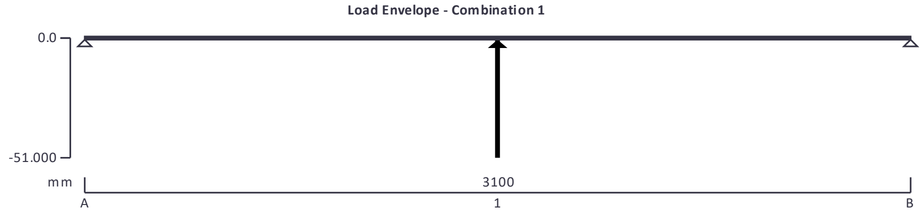
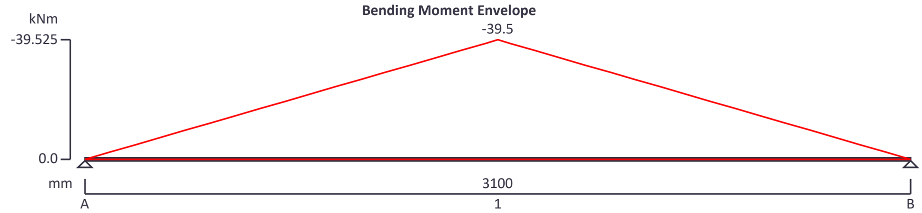
Consider deflection due to variable loads
Limiting deflection; $\delta_{lim} = L_{s1} / 360 = 9.7$ mm
Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 1.832$ mm
PASS - Maximum deflection does not exceed deflection limit

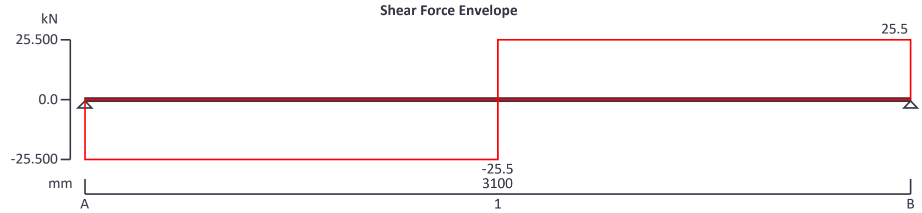
Therefore use 203UC52

B0.7 Design

Steel beam analysis & design (EN1993-1-1:2005)
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex
TEDDS calculation version 3.0.13







Support conditions
Support A Vertically restrained
Rotationally free
Support B Vertically restrained
Rotationally free
Applied loading
Beam loads Variable point load -34 kN at 1550 mm
Load combinations
Load combination 1 Support A Permanent × 1.35
Variable × 1.50
Permanent × 1.35
Variable × 1.50
Support B Permanent × 1.35
Variable × 1.50
Analysis results

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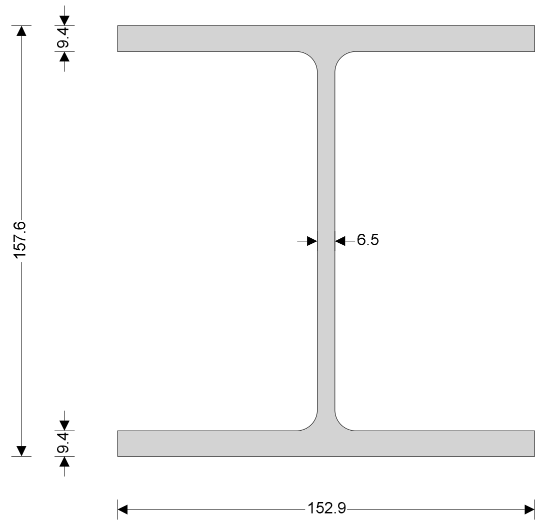
Maximum moment; M_{max} = 0 kNm; M_{min} = -39.5 kNm
Maximum shear; V_{max} = 25.5 kN; V_{min} = -25.5 kN
Deflection; δ_{max} = 0 mm; δ_{min} = 5.7 mm
Maximum reaction at support A; R_{A_max} = -25.5 kN; R_{A_min} = -25.5 kN
Unfactored variable load reaction at support A; $R_{A\_Variable}$ = -17 kN
Maximum reaction at support B; R_{B_max} = -25.5 kN; R_{B_min} = -25.5 kN
Unfactored variable load reaction at support B; $R_{B\_Variable}$ = -17 kN
Section details
Section type; UKC 152x152x30 (Tata Steel Advance)
Steel grade; S275
EN 10025-2:2004 - Hot rolled products of structural steels
Nominal thickness of element; $t = \max(t_f, t_w)$ = 9.4 mm
Nominal yield strength; f_y = 275 N/mm^2
Nominal ultimate tensile strength; f_u = 410 N/mm^2
Modulus of elasticity; E = 210000 N/mm^2



Partial factors - Section 6.1
Resistance of cross-sections; γ_{M0} = 1.00
Resistance of members to instability; γ_{M1} = 1.00
Resistance of tensile members to fracture; γ_{M2} = 1.10
Lateral restraint
Span 1 has lateral restraint at supports only
Effective length factors
Effective length factor in major axis; K_y = 1.000
Effective length factor in minor axis; K_z = 1.000
Effective length factor for torsion; $K_{LT.A}$ = 0.850;
$K_{LT.B}$ = 0.850;
Classification of cross sections - Section 5.5
$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.92$
Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)
Width of section; c = d = 123.6 mm
$c / t_w = 20.6 \times \varepsilon <= 72 \times \varepsilon$; Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)
Width of section; $c = (b - t_w - 2 \times r) / 2 = 65.6$ mm
$c / t_f = 7.5 \times \varepsilon <= 9 \times \varepsilon$; Class 1
Section is class 1
Check shear - Section 6.2.6
Height of web; $h_w = h - 2 \times t_f = 138.8$ mm
Shear area factor; $\eta = 1.000$
$h_w / t_w < 72 \times \varepsilon / \eta$
Shear buckling resistance can be ignored
Design shear force; $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = 25.5$ kN
Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1156$ mm^2
Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = 183.5$ kN
PASS - Design shear resistance exceeds design shear force
Check bending moment major (y-y) axis - Section 6.2.5
Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = 39.5$ kNm
Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = 68.1$ kNm
Slenderness ratio for lateral torsional buckling
Correction factor - Table 6.6; $k_c = 1$
$C_1 = 1 / k_c^2 = 1$
Destabilised load condition factor; $D = 1.5$
Curvature factor; $g = \sqrt{[1 - (I_z / I_y)]} = 0.824$
Poissons ratio; $\nu = 0.3$
Shear modulus; $G = E / [2 \times (1 + \nu)] = 80769$ N/mm^2
Unrestrained length; $L = 0.85 \times L_{s1} = 2635$ mm
Elastic critical buckling moment; $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = 208.7 kNm
Slenderness ratio for lateral torsional buckling; $\overline{\lambda}_{LT} = D \times \sqrt{(W_{pl.y} \times f_y / M_{cr})} = 0.857$
Limiting slenderness ratio; $\overline{\lambda}_{LT,0} = 0.4$
$\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored
Design resistance for buckling - Section 6.3.2.1
Buckling curve - Table 6.5; b
Imperfection factor - Table 6.3; $\alpha_{LT} = 0.34$
Correction factor for rolled sections; $\beta = 0.75$
LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}) + \beta \times \overline{\lambda}_{LT}^2] = 0.853$
LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \overline{\lambda}_{LT}^2)}], 1, 1 / \overline{\lambda}_{LT}^2) = 0.785$
Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\overline{\lambda}_{LT} - 0.8)^2], 1) = 1.000$
Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.785$
Design buckling resistance moment - eq 6.55; $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = 53.5$ kNm
PASS - Design buckling resistance moment exceeds design bending moment
Check vertical deflection - Section 7.2.1
Consider deflection due to permanent and variable loads
Limiting deflection; $\delta_{lim} = L_{s1} / 250 = 12.4$ mm
Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 5.748$ mm
PASS - Maximum deflection does not exceed deflection limit

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Therefore use 152UC30

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C0.1 Design

Steel member design (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

Tedds calculation version 4.3.01

Partial factors - Section 6.1

Resistance of cross-sections; γ_{M0} = **1**

Resistance of members to instability; γ_{M1} = **1**

Resistance of tensile members to fracture; γ_{M2} = **1.1**

<u>Design section 1</u>

Section details

Section type; SHS 100x100x10.0 (Tata Steel Celsius)

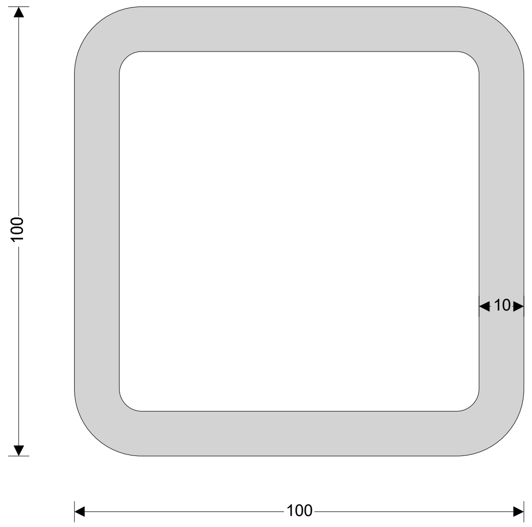
Steel grade - EN 10210-1:2006; S275H

Nominal thickness of element; t_{nom} = t = **10** mm

Nominal yield strength; f_y = **275** N/mm^2

Nominal ultimate tensile strength; f_u = **410** N/mm^2

Modulus of elasticity; E = **210000** N/mm^2



SHS 100x100x10.0 (Tata Steel Celsius)
Section depth, h, 100 mm
Section breadth, b, 100 mm
Mass of section, Mass, 27.4 kg/m
Section thickness, t, 10 mm
Area of section, A, 3493 mm^2
Radius of gyration about y-axis, i_y, 36.373 mm
Radius of gyration about z-axis, i_z, 36.373 mm
Elastic section modulus about y-axis, $W_{el.y}$, 92418 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 92418 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 116232 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 116232 mm^3
Second moment of area about y-axis, I_y, 4620898 mm^4
Second moment of area about z-axis, I_z, 4620898 mm^4

Analysis results

Design bending moment - Major axis; $M_{y,Ed}$ = **12** kNm

Design axial compression force; N_{Ed} = **240** kN

Restraint spacing

Major axis lateral restraint; L_y = **3300** mm

Minor axis lateral restraint; L_z = **3300** mm

Torsional restraint; L_T = **3300** mm

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.92**

Internal compression parts subject to bending and compression - Table 5.2 (sheet 1 of 3)

Width of section; $c = h - 3 \times t$ = **70** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times 2 \times t \times f_y) - 3 \times t / 2] / c, 1)$ = **0.812**

$c / t = 7 = 7.6 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$; Class 1

Internal compression parts subject to compression - Table 5.2 (sheet 1 of 3)

Width of section; $c = b - 3 \times t$ = **70** mm

$c / t = 7 = 7.6 \times \varepsilon <= 33 \times \varepsilon$; Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force; N_{Ed} = **240** kN

Design resistance of section - eq 6.10; $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0}$ = **960.5** kN

N_{Ed} / $N_{c,Rd}$ = **0.25**

PASS - Design compression resistance exceeds design compression

Slenderness ratio for y-y axis flexural buckling - Section 6.3.1.3

Critical buckling length; $L_{cr,y} = L_{y\_s1}$ = **3300** mm

Critical buckling force; $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2$ = **879.5** kN

Slenderness ratio for buckling - eq 6.50; $\overline{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **1.045**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2; a

Imperfection factor - Table 6.1; α_y = **0.21**

Buckling reduction determination factor; $\phi_y = 0.5 \times (1 + \alpha_y \times (\overline{\lambda}_y - 0.2) + \overline{\lambda}_y^2)$ = **1.135**

Buckling reduction factor - eq 6.49; $\chi_y = \min(1 / (\phi_y + \sqrt{(\phi_y^2 - \overline{\lambda}_y^2)}), 1)$ = **0.634**

Design buckling resistance - eq 6.47; $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **609** kN

N_{Ed} / $N_{b,y,Rd}$ = **0.394**

PASS - Design buckling resistance exceeds design compression

Slenderness ratio for z-z axis flexural buckling - Section 6.3.1.3

Critical buckling length; $L_{cr,z} = L_{z\_s1}$ = **3300** mm

Critical buckling force; $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2$ = **879.5** kN

Slenderness ratio for buckling - eq 6.50; $\overline{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})}$ = **1.045**

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2; a

Imperfection factor - Table 6.1; α_z = **0.21**

Buckling reduction determination factor; $\phi_z = 0.5 \times (1 + \alpha_z \times (\overline{\lambda}_z - 0.2) + \overline{\lambda}_z^2)$ = **1.135**

Buckling reduction factor - eq 6.49; $\chi_z = \min(1 / (\phi_z + \sqrt{(\phi_z^2 - \overline{\lambda}_z^2)}), 1)$ = **0.634**

Design buckling resistance - eq 6.47; $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1}$ = **609** kN

N_{Ed} / $N_{b,z,Rd}$ = **0.394**

PASS - Design buckling resistance exceeds design compression

Check bending moment - Section 6.2.5

Design bending moment; $M_{y,Ed}$ = **12** kNm

Design bending resistance moment - eq 6.13; $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **32** kNm

$M_{y,Ed}$ / $M_{c,y,Rd}$ = **0.375**

PASS - Design bending resistance moment exceeds design bending moment

Check bending and axial force - Section 6.2.9

Normal force to plastic resistance force ratio; $n = N_{Ed} / N_{pl,Rd}$ = **0.25**

$a_w = \min((A - 2 \times b \times t) / A, 0.5)$ = **0.427**

Reduced plastic moment resistance - Eq.6.39; $M_{N,y,Rd} = M_{pl,y,Rd} \times \min((1 - n) / (1 - 0.5 \times a_w), 1)$ = **30.5** kNm

$M_{y,Ed}$ / $M_{N,y,Rd}$ = **0.394**

PASS - Reduced bending resistance moment exceeds design bending moment

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3; C_{my} = **1.000**

C_{mz} = **1.000**

C_{mLT} = **1.000**

Interaction factors k_{ij} for members not susceptible to torsional deformations - Table B.1

Characteristic moment resistance; $M_{y,Rk} = W_{pl.y} \times f_y$ = **32** kNm

Characteristic moment resistance; $M_{z,Rk} = W_{pl.z} \times f_y$ = **32** kNm

Characteristic resistance to normal force; $N_{Rk} = A \times f_y$ = **960.5** kN

Interaction factors; $k_{yy} = C_{my} \times (1 + \min(\overline{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **1.315**

$k_{zy} = 0.6 \times k_{yy}$ = **0.789**

; χ_{LT} = **1.000**

Interaction formulae - eq 6.61 & eq 6.62; $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.888**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.69**

PASS - Combined bending and compression checks are satisfied

Therefore use 100x100x10 RHS

C0.3 Design

Steel member design (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

Tedds calculation version 4.3.01

Partial factors - Section 6.1

Resistance of cross-sections; $\gamma_{M0} = 1$

Resistance of members to instability; $\gamma_{M1} = 1$

Resistance of tensile members to fracture; $\gamma_{M2} = 1.1$

Design section 1

Section details

Section type; RHS 150x100x10.0 (Tata Steel Celsius)

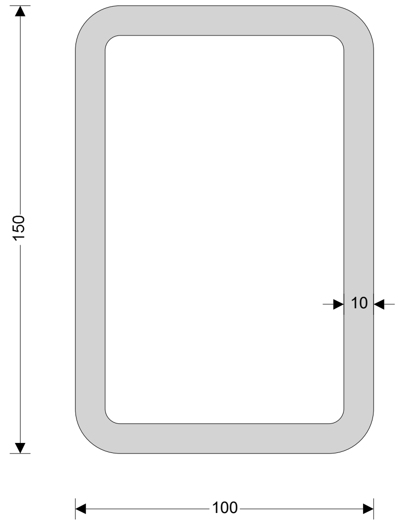
Steel grade - EN 10210-1:2006; S275H

Nominal thickness of element; $t_{nom} = t = 10$ mm

Nominal yield strength; $f_y = 275$ N/mm^2

Nominal ultimate tensile strength; $f_u = 410$ N/mm^2

Modulus of elasticity; $E = 210000$ N/mm^2



RHS 150x100x10.0 (Tata Steel Celsius)

Section depth, h, 150 mm

Section breadth, b, 100 mm

Mass of section, Mass, 35.3 kg/m

Section thickness, t, 10 mm

Area of section, A, 4493 mm^2

Radius of gyration about y-axis, i_y, 53.426 mm

Radius of gyration about z-axis, i_z, 38.485 mm

Elastic section modulus about y-axis, $W_{el.y}$, 170984 mm^3

Elastic section modulus about z-axis, $W_{el.z}$, 133085 mm^3

Plastic section modulus about y-axis, $W_{pl.y}$, 216049 mm^3

Plastic section modulus about z-axis, $W_{pl.z}$, 161232 mm^3

Second moment of area about y-axis, I_y, 12823765 mm^4

Second moment of area about z-axis, I_z, 6654232 mm^4

Analysis results

Design bending moment - Major axis; $M_{y,Ed} = 16$ kNm

Design axial compression force; $N_{Ed} = 322$ kN

Restraint spacing

Major axis lateral restraint; $L_y = 3300$ mm

Minor axis lateral restraint; $L_z = 3300$ mm

Torsional restraint; $L_T = 3300$ mm

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.92$

Internal compression parts subject to bending and compression - Table 5.2 (sheet 1 of 3)

Width of section; $c = h - 3 \times t = 120$ mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times 2 \times t \times f_y) - 3 \times t / 2] / c, 1) = 0.744$

$c / t = 12 = 13 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$; Class 1

Internal compression parts subject to compression - Table 5.2 (sheet 1 of 3)

Width of section; $c = b - 3 \times t = 70$ mm

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$c / t = 7 = 7.6 \times \varepsilon <= 33 \times \varepsilon$; Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force; $N_{Ed} = 322$ kN

Design resistance of section - eq 6.10; $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0} = 1235.5$ kN

$N_{Ed} / N_{c,Rd} = 0.261$

PASS - Design compression resistance exceeds design compression

Slenderness ratio for y-y axis flexural buckling - Section 6.3.1.3

Critical buckling length; $L_{cr,y} = L_{y\_s1} = 3300$ mm

Critical buckling force; $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 = 2440.7$ kN

Slenderness ratio for buckling - eq 6.50; $\overline{\lambda}_y = \sqrt{}(A \times f_y / N_{cr,y}) = 0.711$

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2; a

Imperfection factor - Table 6.1; $\alpha_y = 0.21$

Buckling reduction determination factor; $\phi_y = 0.5 \times (1 + \alpha_y \times (\overline{\lambda}_y - 0.2) + \overline{\lambda}_y^2) = 0.807$

Buckling reduction factor - eq 6.49; $\chi_y = \min(1 / (\phi_y + \sqrt{}(\phi_y^2 - \overline{\lambda}_y^2)), 1) = 0.842$

Design buckling resistance - eq 6.47; $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} = 1040.6$ kN

$N_{Ed} / N_{b,y,Rd} = 0.309$

PASS - Design buckling resistance exceeds design compression

Slenderness ratio for z-z axis flexural buckling - Section 6.3.1.3

Critical buckling length; $L_{cr,z} = L_{z\_s1} = 3300$ mm

Critical buckling force; $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2 = 1266.5$ kN

Slenderness ratio for buckling - eq 6.50; $\overline{\lambda}_z = \sqrt{}(A \times f_y / N_{cr,z}) = 0.988$

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2; a

Imperfection factor - Table 6.1; $\alpha_z = 0.21$

Buckling reduction determination factor; $\phi_z = 0.5 \times (1 + \alpha_z \times (\overline{\lambda}_z - 0.2) + \overline{\lambda}_z^2) = 1.07$

Buckling reduction factor - eq 6.49; $\chi_z = \min(1 / (\phi_z + \sqrt{}(\phi_z^2 - \overline{\lambda}_z^2)), 1) = 0.674$

Design buckling resistance - eq 6.47; $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} = 833$ kN

$N_{Ed} / N_{b,z,Rd} = 0.387$

PASS - Design buckling resistance exceeds design compression

Check bending moment - Section 6.2.5

Design bending moment; $M_{y,Ed} = 16$ kNm

Design bending resistance moment - eq 6.13; $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = 59.4$ kNm

$M_{y,Ed} / M_{c,y,Rd} = 0.269$

PASS - Design bending resistance moment exceeds design bending moment

Check bending and axial force - Section 6.2.9

Normal force to plastic resistance force ratio; $n = N_{Ed} / N_{pl,Rd} = 0.261$

$a_w = \min((A - 2 \times b \times t) / A, 0.5) = 0.5$

Reduced plastic moment resistance - Eq.6.39; $M_{N,y,Rd} = M_{pl,y,Rd} \times \min((1 - n) / (1 - 0.5 \times a_w), 1) = 58.6$ kNm

$M_{y,Ed} / M_{N,y,Rd} = 0.273$

PASS - Reduced bending resistance moment exceeds design bending moment

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3; $C_{my} = 1.000$

$C_{mz} = 1.000$

C_{mLT} = 1.000
Interaction factors k_{ij} for members not susceptible to torsional deformations - Table B.1
Characteristic moment resistance; $M_{y,Rk} = W_{pl.y} \times f_y$ = 59.4 kNm
Characteristic moment resistance; $M_{z,Rk} = W_{pl.z} \times f_y$ = 44.3 kNm
Characteristic resistance to normal force; $N_{Rk} = A \times f_y$ = 1235.5 kN
Interaction factors; $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = 1.158
$k_{zy} = 0.6 \times k_{yy}$ = 0.695
; χ_{LT} = 1.000
Interaction formulae - eq 6.61 & eq 6.62; $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = 0.621
$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = 0.574
PASS - Combined bending and compression checks are satisfied

Therefore use 150x100x10 RHS

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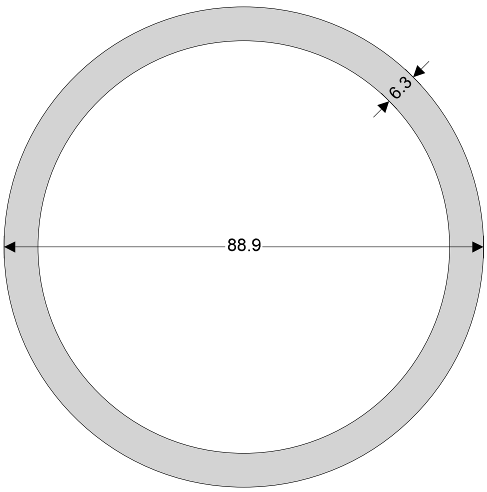
C0.6 Design

Steel member design (EN1993-1-1:2005)
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex
Tedds calculation version 4.3.01
Partial factors - Section 6.1
Resistance of cross-sections; $\gamma_{M0} = 1$
Resistance of members to instability; $\gamma_{M1} = 1$
Resistance of tensile members to fracture; $\gamma_{M2} = 1.1$
Design section 1
Section details
Section type; CHS 88.9x6.3 (Tata Steel Celsius)
Steel grade - EN 10210-1:2006; S355H
Nominal thickness of element; $t_{nom} = t = 6.3$ mm
Nominal yield strength; $f_y = 355$ N/mm^2
Nominal ultimate tensile strength; $f_u = 470$ N/mm^2
Modulus of elasticity; $E = 210000$ N/mm^2



CHS 88.9x6.3 (Tata Steel Celsius)
Diameter, d, 88.9 mm
Mass of section, Mass, 12.8 kg/m
Section thickness, t, 6.3 mm
Area of section, A, 1635 mm^2
Radius of gyration about y-axis, i_y, 29.288 mm
Radius of gyration about z-axis, i_z, 29.288 mm
Elastic section modulus about y-axis, $W_{el.y}$, 31549 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 31549 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 43067 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 43067 mm^3
Second moment of area about y-axis, I_y, 1402361 mm^4
Second moment of area about z-axis, I_z, 1402361 mm^4

Analysis results
Design bending moment - Major axis; $M_{y,Ed} = 5.5$ kNm
Design axial compression force; $N_{Ed} = 115$ kN
Restraint spacing
Major axis lateral restraint; $L_y = 3000$ mm
Minor axis lateral restraint; $L_z = 3000$ mm
Torsional restraint; $L_T = 3000$ mm
Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.81$

Tubular sections - Table 5.2 (sheet 3 of 3)

$d / t = 14.1 = 21.3 \times \varepsilon^2 <= 50 \times \varepsilon^2$; Class 1

Section is class 1
Check compression - Section 6.2.4

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Design compression force; N_{Ed} = 115 kN
Design resistance of section - eq 6.10; $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0}$ = 580.4 kN
$N_{Ed} / N_{c,Rd}$ = 0.198
PASS - Design compression resistance exceeds design compression
Slenderness ratio for y-y axis flexural buckling - Section 6.3.1.3
Critical buckling length; $L_{cr,y} = L_{y\_s1}$ = 3000 mm
Critical buckling force; $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2$ = 323 kN
Slenderness ratio for buckling - eq 6.50; $\bar{\lambda}_y = \sqrt{}(A \times f_y / N_{cr,y})$ = 1.341
Check y-y axis flexural buckling resistance - Section 6.3.1.1
Buckling curve - Table 6.2; a
Imperfection factor - Table 6.1; α_y = 0.21
Buckling reduction determination factor; $\phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = 1.518
Buckling reduction factor - eq 6.49; $\chi_y = \min(1 / (\phi_y + \sqrt{}(\phi_y^2 - \bar{\lambda}_y^2)), 1)$ = 0.448
Design buckling resistance - eq 6.47; $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = 260.1 kN
$N_{Ed} / N_{b,y,Rd}$ = 0.442
PASS - Design buckling resistance exceeds design compression
Slenderness ratio for z-z axis flexural buckling - Section 6.3.1.3
Critical buckling length; $L_{cr,z} = L_{z\_s1}$ = 3000 mm
Critical buckling force; $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2$ = 323 kN
Slenderness ratio for buckling - eq 6.50; $\bar{\lambda}_z = \sqrt{}(A \times f_y / N_{cr,z})$ = 1.341
Check z-z axis flexural buckling resistance - Section 6.3.1.1
Buckling curve - Table 6.2; a
Imperfection factor - Table 6.1; α_z = 0.21
Buckling reduction determination factor; $\phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2)$ = 1.518
Buckling reduction factor - eq 6.49; $\chi_z = \min(1 / (\phi_z + \sqrt{}(\phi_z^2 - \bar{\lambda}_z^2)), 1)$ = 0.448
Design buckling resistance - eq 6.47; $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1}$ = 260.1 kN
$N_{Ed} / N_{b,z,Rd}$ = 0.442
PASS - Design buckling resistance exceeds design compression
Check bending moment - Section 6.2.5
Design bending moment; $M_{y,Ed}$ = 5.5 kNm
Design bending resistance moment - eq 6.13; $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = 15.3 kNm
$M_{y,Ed} / M_{c,y,Rd}$ = 0.36
PASS - Design bending resistance moment exceeds design bending moment
Check bending and axial force - Section 6.2.9
Normal force to plastic resistance force ratio; $n = N_{Ed} / N_{pl,Rd}$ = 0.198
Reduced plastic moment resistance - Eq.6.39; $M_{N,y,Rd} = M_{pl,y,Rd} \times (1 - n^{1.7})$ = 14.3 kNm
$M_{y,Ed} / M_{N,y,Rd}$ = 0.384
PASS - Reduced bending resistance moment exceeds design bending moment
Check combined bending and compression - Section 6.3.3
Equivalent uniform moment factors - Table B.3; C_{my} = 1.000
C_{mz} = 1.000
C_{mLT} = 1.000
Interaction factors k_{ij} for members not susceptible to torsional deformations - Table B.1
Characteristic moment resistance; $M_{y,Rk} = W_{pl.y} \times f_y$ = 15.3 kNm
Characteristic moment resistance; $M_{z,Rk} = W_{pl.z} \times f_y$ = 15.3 kNm
Characteristic resistance to normal force; $N_{Rk} = A \times f_y$ = 580.4 kN

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Interaction factors; $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) = 1.354$

$k_{zy} = 0.6 \times k_{yy} = 0.812$

; $\chi_{LT} = 1.000$

Interaction formulae - eq 6.61 & eq 6.62; $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) = 0.929$

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) = 0.734$

PASS - Combined bending and compression checks are satisfied

Therefore use 88.9x6.3CHS

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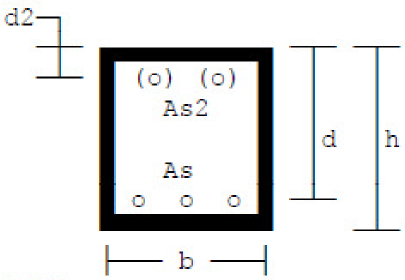
B0.8 Design

Bending in rectangular beams

Calculations are in accordance with EC2: Design of concrete structures and assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.45*d.



Design BM before redistribution	Mbef=62.0 kNm
Beam being analysed is considered as non-continuous.	
Char yield strength of reinf'ment	fyk=500 N/mm²
Max.aggregate size (for bar spc.)	hagg=20 mm
Diameter of tension bars	dia=25 mm
Diameter of link legs	dial=10 mm

Section properties

Effective depth of section	d=238 mm
Number of bars to be used	nbart=5

DESIGN	Overall depth	300 mm
SUMMARY	Effective depth	238 mm
	Width of section	200 mm
FLEXURE	Parameter K	0.1368
	Parameter K'	0.1684
	Lever arm ratio z/d	0.8596
TENSION REINFORCEMENT	Steel area required	697 mm²
	Steel area provided	2454 mm²
	Diameter of bars	25 mm
	Number of bars	5
	Steel percentage req.	1.464 %
	Minimum area of steel	86.85 mm²
	Maximum area of steel	2400 mm²

Deflection check

Effective span of beam	L=5.6 m
Actual span to depth ratio	l'd=L*1000/d=23.53
Allowable span/depth ratio	l'da=k*N*F1*F2*F3=29.44

DESIGN	Actual l/d ratio	23.53
SUMMARY	Basic l/d ratio	15.1
DEFLECTION	Span factor	1.3
	Flange beam factor F1	1
	Long spans factor F2	1
	Steel stress factor F3	1.5
	Allowable l/d ratio	29.44

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Location for shear calculation:	
Shear force due to ultimate load	VEd=44 kN
Design shear stress	vEd=VEd*10^3/(0.9*bw*d)=1.027 N/mm²
Concrete strut capacity	vRdm=0.124*fck*(1-fck/250)=4.166 N/mm²
Angle of inclination of strut	theta'=22°
Number of shear legs	nsl=2

SHEAR SUMMARY

Design shear force	44 kN
Design shear stress	1.027 N/mm²
Concrete strut capacity	4.166 N/mm²
Area/spacing ratio	0.2024
Diameter of links	10 mm
Area of links	157 mm²
Maximum spacing (based on 0.75d)	178.5 mm
Actual spacing	178.5 mm

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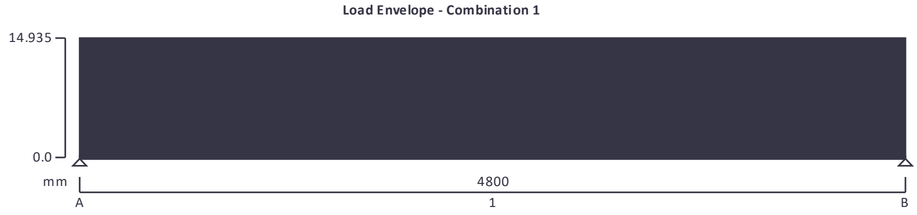
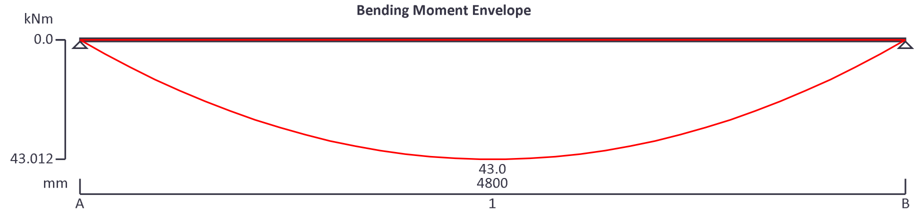
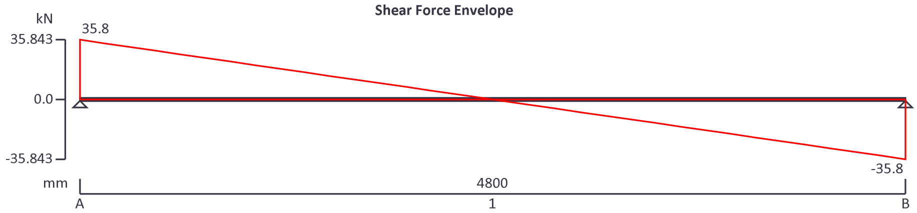
B0.9 Design

Steel beam analysis & design (EN1993-1-1:2005)
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex
TEDDS calculation version 3.0.13

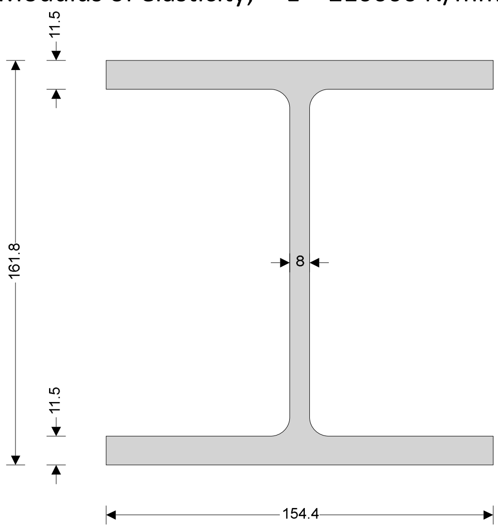






Support conditions
Support A Vertically restrained
Rotationally free
Support B Vertically restrained
Rotationally free
Applied loading
Beam loads Permanent self weight of beam × 1
Permanent full UDL 7.7 kN/m
Variable full UDL 2.7 kN/m
Load combinations
Load combination 1 Support A Permanent × 1.35
Variable × 1.50
Permanent × 1.35
Variable × 1.50
Support B Permanent × 1.35

Variable × 1.50
Analysis results
Maximum moment; M_{max} = 43 kNm; M_{min} = 0 kNm
Maximum shear; V_{max} = 35.8 kN; V_{min} = -35.8 kN
Deflection; δ_{max} = 4 mm; δ_{min} = 0 mm
Maximum reaction at support A; R_{A_max} = 35.8 kN; R_{A_min} = 35.8 kN
Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = 19.4 kN
Unfactored variable load reaction at support A; $R_{A\_Variable}$ = 6.5 kN
Maximum reaction at support B; R_{B_max} = 35.8 kN; R_{B_min} = 35.8 kN
Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = 19.4 kN
Unfactored variable load reaction at support B; $R_{B\_Variable}$ = 6.5 kN
Section details
Section type; UKC 152x152x37 (Tata Steel Advance)
Steel grade; S355
EN 10025-2:2004 - Hot rolled products of structural steels
Nominal thickness of element; $t = \max(t_f, t_w)$ = 11.5 mm
Nominal yield strength; f_y = 355 N/mm^2
Nominal ultimate tensile strength; f_u = 470 N/mm^2
Modulus of elasticity; E = 210000 N/mm^2



Partial factors - Section 6.1
Resistance of cross-sections; γ_{M0} = 1.00
Resistance of members to instability; γ_{M1} = 1.00
Resistance of tensile members to fracture; γ_{M2} = 1.10
Lateral restraint
Span 1 has lateral restraint at supports only
Effective length factors
Effective length factor in major axis; K_y = 1.000
Effective length factor in minor axis; K_z = 1.000
Effective length factor for torsion; $K_{LT.A}$ = 1.200; + 2 × h
$K_{LT.B}$ = 1.200; + 2 × h
Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.81$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section; $c = d = 123.6$ mm

$c / t_w = 19.0 \times \varepsilon <= 72 \times \varepsilon$; Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section; $c = (b - t_w - 2 \times r) / 2 = 65.6$ mm

$c / t_f = 7.0 \times \varepsilon <= 9 \times \varepsilon$; Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f = 138.8$ mm

Shear area factor; $\eta = 1.000$

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = 35.8$ kN

Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1427 \text{ mm}^2$

Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = 292.4$ kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = 43$ kNm

Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = 109.6$ kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6; $k_c = 1$

$C_1 = 1 / k_c^2 = 1$

Destabilised load condition factor; $D = 1.5$

Curvature factor; $g = \sqrt{[1 - (I_z / I_y)]} = 0.825$

Poissons ratio; $\nu = 0.3$

Shear modulus; $G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$

Unrestrained length; $L = 1.2 \times L_{s1} + 2 \times h = 6084$ mm

Elastic critical buckling moment;$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = 101.5 kNm

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = D \times \sqrt{(W_{pl.y} \times f_y / M_{cr})} = 1.559$

Limiting slenderness ratio; $\bar{\lambda}_{LT,0} = 0.4$

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5; b

Imperfection factor - Table 6.3; $\alpha_{LT} = 0.34$

Correction factor for rolled sections; $\beta = 0.75$

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 1.608$

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.403$

Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 1.000$

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.403$

Design buckling resistance moment - eq 6.55; $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = 44.2$ kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection; $\delta_{lim} = L_{s1} / 360 = 13.3$ mm

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 4.02$ mm
PASS - Maximum deflection does not exceed deflection limit

Therefore use 152UC37

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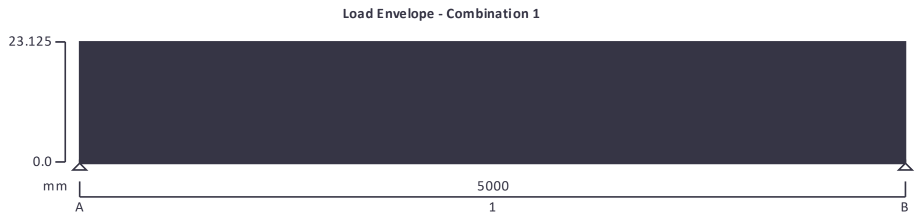
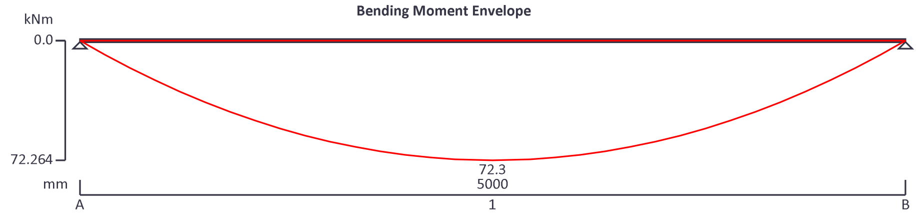
B0.10 Design

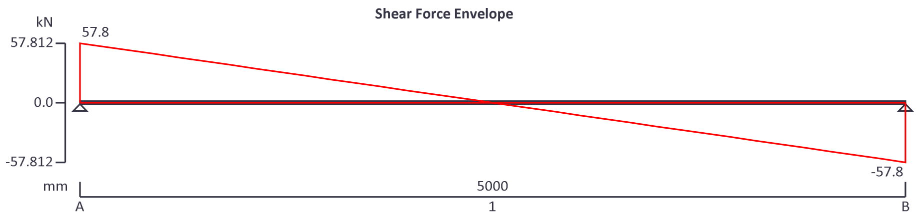
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.13







Support conditions

Support A Vertically restrained
Rotationally free

Support B Vertically restrained
Rotationally free

Applied loading

Beam loads Permanent self weight of beam × 1
Permanent full UDL 12.1 kN/m
Variable full UDL 4.2 kN/m

Load combinations

Load combination 1 Support A Permanent × 1.35
Variable × 1.50
Permanent × 1.35
Variable × 1.50
Support B Permanent × 1.35
Variable × 1.50

Analysis results

Maximum moment; M_{max} = 72.3 kNm; M_{min} = 0 kNm

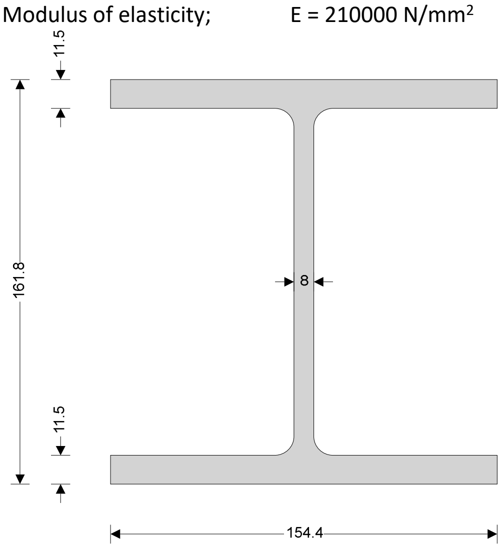
Maximum shear; V_{max} = 57.8 kN; V_{min} = -57.8 kN

Deflection; δ_{max} = 7.4 mm; δ_{min} = 0 mm

Maximum reaction at support A; R_{A_max} = 57.8 kN; R_{A_min} = 57.8 kN

Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = 31.2 kN

Unfactored variable load reaction at support A; $R_{A\_Variable}$ = 10.5 kN
Maximum reaction at support B; R_{B_max} = 57.8 kN; R_{B_min} = 57.8 kN
Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = 31.2 kN
Unfactored variable load reaction at support B; $R_{B\_Variable}$ = 10.5 kN
Section details
Section type; UKC 152x152x37 (Tata Steel Advance)
Steel grade; S355
EN 10025-2:2004 - Hot rolled products of structural steels
Nominal thickness of element; $t = \max(t_f, t_w)$ = 11.5 mm
Nominal yield strength; f_y = 355 N/mm^2
Nominal ultimate tensile strength; f_u = 470 N/mm^2
Modulus of elasticity; E = 210000 N/mm^2



Partial factors - Section 6.1
Resistance of cross-sections; γ_{M0} = 1.00
Resistance of members to instability; γ_{M1} = 1.00
Resistance of tensile members to fracture; γ_{M2} = 1.10
Lateral restraint
Span 1 has full lateral restraint
Effective length factors
Effective length factor in major axis; K_y = 1.000
Effective length factor in minor axis; K_z = 1.000
Effective length factor for torsion; $K_{LT.A} = 1.200; + 2 \times h$
$K_{LT.B} = 1.200; + 2 \times h$
Classification of cross sections - Section 5.5
$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.81$
Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)
Width of section; c = d = 123.6 mm
$c / t_w = 19.0 \times \varepsilon <= 72 \times \varepsilon$; Class 1
Outstand flanges - Table 5.2 (sheet 2 of 3)
Width of section; $c = (b - t_w - 2 \times r) / 2$ = 65.6 mm
$c / t_f = 7.0 \times \varepsilon <= 9 \times \varepsilon$; Class 1
Section is class 1
Check shear - Section 6.2.6
Height of web; $h_w = h - 2 \times t_f$ = 138.8 mm
Shear area factor; η = 1.000
$h_w / t_w < 72 \times \varepsilon / \eta$
Shear buckling resistance can be ignored
Design shear force; $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = 57.8 kN

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Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = 1427 mm^2
Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = 292.4 kN
PASS - Design shear resistance exceeds design shear force
Check bending moment major (y-y) axis - Section 6.2.5
Design bending moment; $M_{Ed} = \max(\mathrm{abs}(M_{s1\_max}), \mathrm{abs}(M_{s1\_min}))$ = 72.3 kNm
Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = 109.6 kNm
PASS - Design bending resistance moment exceeds design bending moment
Check vertical deflection - Section 7.2.1
Consider deflection due to variable loads
Limiting deflection; $\delta_{lim} = L_{s1} / 360$ = 13.9 mm
Maximum deflection span 1; $\delta = \max(\mathrm{abs}(\delta_{max}), \mathrm{abs}(\delta_{min}))$ = 7.363 mm
PASS - Maximum deflection does not exceed deflection limit

Therefore use 152UC37

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Basement Member Design

RW.1 Design

Retaining wall analysis

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.04

Retaining wall details

Stem type; Propped cantilever

Stem height; h_{stem} = 3400 mm

Prop height; h_{prop} = 3300 mm

Stem thickness; t_{stem} = 150 mm

Angle to rear face of stem; α = 90 deg

Stem density; γ_{stem} = 25 kN/m^3

Toe length; l_{toe} = 6000 mm

Base thickness; t_{base} = 300 mm

Base density; γ_{base} = 25 kN/m^3

Height of retained soil; h_{ret} = 3400 mm

Angle of soil surface; β = 0 deg

Depth of cover; d_{cover} = 0 mm

Height of water; h_{water} = 2200 mm

Water density; γ_w = 9.8 kN/m^3

Retained soil properties

Soil type; Organic clay

Moist density; γ_{mr} = 18 kN/m^3

Saturated density; γ_{sr} = 18 kN/m^3

Characteristic effective shear resistance angle; $\phi'_{r.k}$ = 18 deg

Characteristic wall friction angle; $\delta_{r.k}$ = 9 deg

Base soil properties

Soil type; Firm clay

Soil density; γ_b = 18 kN/m^3

Characteristic effective shear resistance angle; $\phi'_{b.k}$ = 18 deg

Characteristic wall friction angle; $\delta_{b.k}$ = 9 deg

Characteristic base friction angle; $\delta_{bb.k}$ = 12 deg

Presumed bearing capacity; $P_{bearing}$ = 100 kN/m^2

Loading details

Permanent surcharge load; $Surcharge_G$ = 7.8 kN/m^2

Variable surcharge load; $Surcharge_Q$ = 4 kN/m^2

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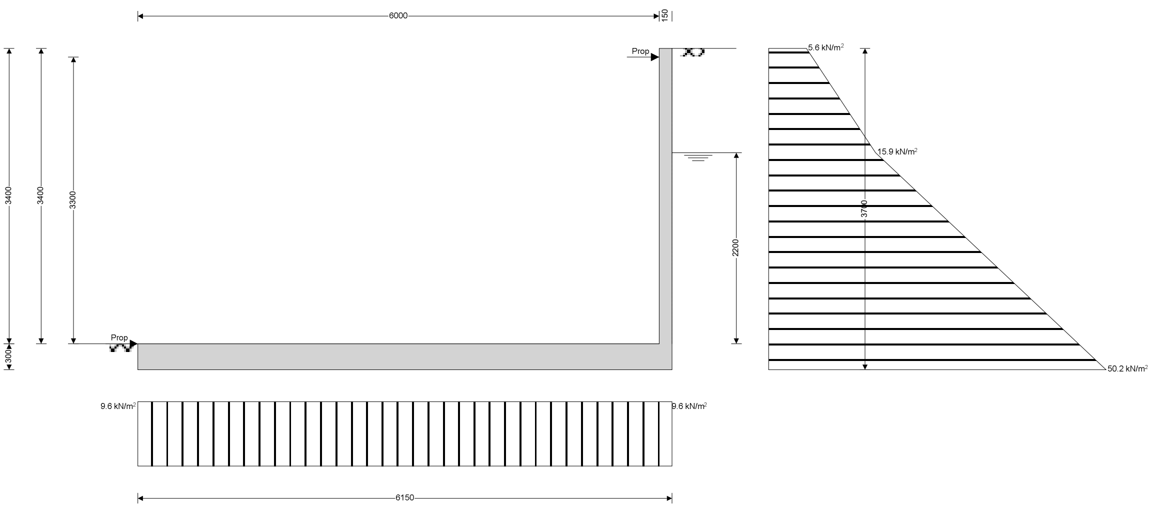


General arrangement

Calculate retaining wall geometry

Base length; $l_{base} = l_{toe} + t_{stem} = 6150$ mm

Saturated soil height; $h_{sat} = h_{water} + d_{cover} = 2200$ mm

Moist soil height; $h_{moist} = h_{ret} - h_{water} = 1200$ mm

Length of surcharge load; $l_{sur} = l_{heel} = 0$ mm

- Distance to vertical component; $x_{sur\_v} = l_{base} - l_{heel} / 2 = 6150$ mm

Effective height of wall; $h_{eff} = h_{base} + d_{cover} + h_{ret} = 3700$ mm

- Distance to horizontal component; $x_{sur\_h} = h_{eff} / 2 = 1850$ mm

Area of wall stem; $A_{stem} = h_{stem} \times t_{stem} = 0.51$ m^2

- Distance to vertical component; $x_{stem} = l_{toe} + t_{stem} / 2 = 6075$ mm

Area of wall base; $A_{base} = l_{base} \times t_{base} = 1.845$ m^2

- Distance to vertical component; $x_{base} = l_{base} / 2 = 3075$ mm

Using Coulomb theory

Active pressure coefficient; $K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[}\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]]^2) = 0.483$

Passive pressure coefficient; $K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[}\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]]^2) = 2.359$

Bearing pressure check

Vertical forces on wall

Wall stem; $F_{stem} = A_{stem} \times \gamma_{stem} = 12.8$ kN/m

Wall base; $F_{base} = A_{base} \times \gamma_{base} = 46.1$ kN/m

Total; $F_{total\_v} = F_{stem} + F_{base} + F_{water\_v} = 58.9$ kN/m

Horizontal forces on wall

Surcharge load; $F_{sur\_h} = K_A \times \cos(\delta_{r.k}) \times (Surcharge_G + Surcharge_Q) \times h_{eff} = 20.8$ kN/m

Saturated retained soil; $F_{sat\_h} = K_A \times \cos(\delta_{r.k}) \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2 = 12.2$ kN/m

Water; $F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 = 30.7$ kN/m

Moist retained soil; $F_{moist\_h} = K_A \times \cos(\delta_{r.k}) \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) = 31.9$ kN/m

Base soil; $F_{pass\_h} = -K_P \times \cos(\delta_{b.k}) \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 = -1.9$ kN/m

Total; $F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h}$ = 93.8 kN/m

Moments on wall

Wall stem; $M_{stem} = F_{stem} \times x_{stem}$ = 77.5 kNm/m

Wall base; $M_{base} = F_{base} \times x_{base}$ = 141.8 kNm/m

Surcharge load; $M_{sur} = -F_{sur\_h} \times x_{sur\_h}$ = -38.5 kNm/m

Saturated retained soil; $M_{sat} = -F_{sat\_h} \times x_{sat\_h}$ = -10.2 kNm/m

Water; $M_{water} = -F_{water\_h} \times x_{water\_h}$ = -25.5 kNm/m

Moist retained soil; $M_{moist} = -F_{moist\_h} \times x_{moist\_h}$ = -50.1 kNm/m

Total; $M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{water} + M_{sur}$ = 94.9 kNm/m

Check bearing pressure

Propping force to stem; $F_{prop\_stem} = (F_{total\_v} \times l_{base} / 2 - M_{total}) / (h_{prop} + t_{base})$ = 23.9 kN/m

Propping force to base; $F_{prop\_base} = F_{total\_h} - F_{prop\_stem}$ = 69.8 kN/m

Moment from propping force; $M_{prop} = F_{prop\_stem} \times (h_{prop} + t_{base})$ = 86.1 kNm/m

Distance to reaction; $\bar{x} = (M_{total} + M_{prop}) / F_{total\_v}$ = 3075 mm

Eccentricity of reaction; $e = \bar{x} - l_{base} / 2$ = 0 mm

Loaded length of base; $l_{load} = l_{base}$ = 6150 mm

Bearing pressure at toe; $q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base})$ = 9.6 kN/m^2

Bearing pressure at heel; $q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base})$ = 9.6 kN/m^2

Factor of safety; $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel})$ = 10.446

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

Retaining wall design

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 2.9.04

Concrete details - Table 3.1 - Strength and deformation characteristics for concrete

Concrete strength class; C30/37

Characteristic compressive cylinder strength; f_{ck} = 30 N/mm^2

Characteristic compressive cube strength; $f_{ck,cube}$ = 37 N/mm^2

Mean value of compressive cylinder strength; $f_{cm} = f_{ck} + 8$ N/mm^2 = 38 N/mm^2

Mean value of axial tensile strength; $f_{ctm} = 0.3\text{ N/mm}^2 \times (f_{ck} / 1\text{ N/mm}^2)^{2/3}$ = 2.9 N/mm^2

5% fractile of axial tensile strength; $f_{ctk,0.05} = 0.7 \times f_{ctm}$ = 2.0 N/mm^2

Secant modulus of elasticity of concrete; $E_{cm} = 22\text{ kN/mm}^2 \times (f_{cm} / 10\text{ N/mm}^2)^{0.3}$ = 32837 N/mm^2

Partial factor for concrete - Table 2.1N; γ_C = 1.50

Compressive strength coefficient - cl.3.1.6(1); α_{cc} = 0.85

Design compressive concrete strength - exp.3.15; $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = 17.0 N/mm^2

Maximum aggregate size; h_{agg} = 20 mm

Ultimate strain - Table 3.1; ε_{cu2} = 0.0035

Shortening strain - Table 3.1; ε_{cu3} = 0.0035

Effective compression zone height factor; λ = 0.80

Effective strength factor; η = 1.00

Bending coefficient k_1; K_1 = 0.40

Bending coefficient k_2; $K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ = 1.00

Bending coefficient k_3; K_3 =0.40

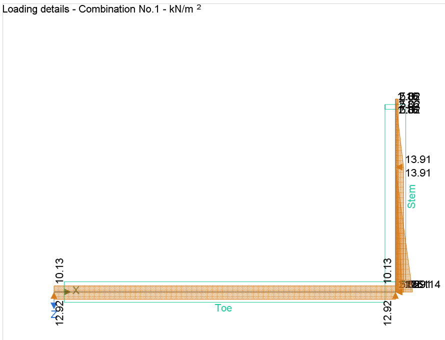
Bending coefficient k_4; $K_4 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ =1.00

Reinforcement details

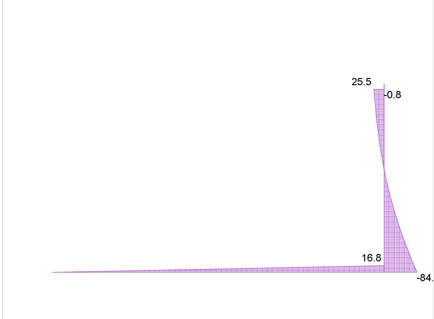
Characteristic yield strength of reinforcement; f_{yk} = 500 N/mm^2

Modulus of elasticity of reinforcement; $E_s = 200000 \text{ N/mm}^2$
Partial factor for reinforcing steel - Table 2.1N; $\gamma_S = 1.15$
Design yield strength of reinforcement; $f_{yd} = f_{yk} / \gamma_S = 435 \text{ N/mm}^2$
Cover to reinforcement
Front face of stem; $c_{sf} = 25$ mm
Rear face of stem; $c_{sr} = 50$ mm
Top face of base; $c_{bt} = 50$ mm
Bottom face of base; $c_{bb} = 75$ mm

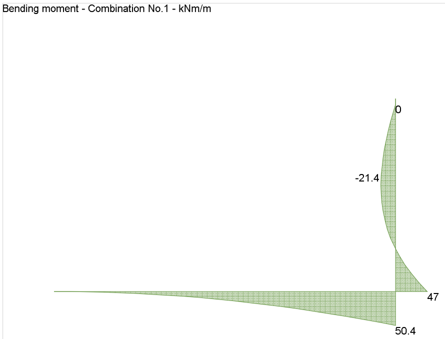
Loading details - Combination No.1 - kN/m 2



Shear force - Combination No.1 - kN/m



Bending moment - Combination No.1 - kNm/m



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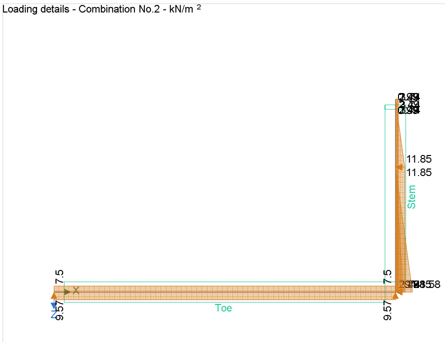

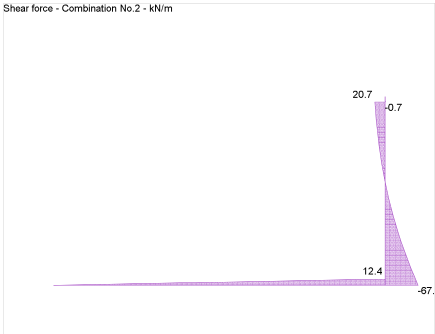

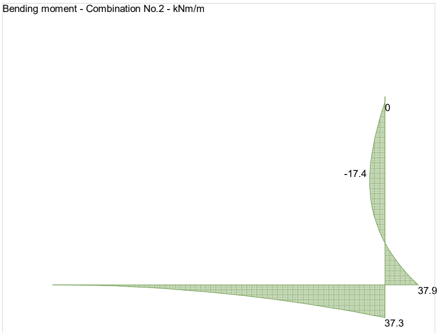

Check stem design at 1860 mm

Depth of section; $h = 150$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; $M = 21.4$ kNm/m

Depth to tension reinforcement; $d = h - c_{sf} - \phi_{sx} - \phi_{sfM} / 2 = 107$ mm

$K = M / (d^2 \times f_{ck}) = 0.062$

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

$K' = 0.207$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = 101$ mm

Depth of neutral axis; $x = 2.5 \times (d – z) = 16$ mm

Area of tension reinforcement required; $A_{sfM.req} = M / (f_{yd} \times z) = 489$ mm^2/m

Tension reinforcement provided; 12 dia.bars @ 150 c/c

Area of tension reinforcement provided; $A_{sfM.prov} = \pi \times \phi_{sfM}^2 / (4 \times s_{sfM}) = 754$ mm^2/m

Minimum area of reinforcement - exp.9.1N; $A_{sfM.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 161$ mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{sfM.max} = 0.04 \times h = 6000$ mm^2/m

$\max(A_{sfM.req}, A_{sfM.min}) / A_{sfM.prov} = 0.648$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio; $\rho_0 = \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) / 1000 = 0.005$

Required tension reinforcement ratio; $\rho = A_{sfM.req} / d = 0.005$

Required compression reinforcement ratio; $\rho' = A_{sfM.2.req} / d_2 = 0.000$

Structural system factor - Table 7.4N; $K_b = 1$

Reinforcement factor - exp.7.17; $K_s = \min(500 \text{ N/mm}^2 / (f_{yk} \times A_{sfM.req} / A_{sfM.prov}), 1.5) = 1.5$

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Limiting span to depth ratio - exp.7.16.a; $\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{}(f_{ck} / 1\ N/mm^2) \times \rho_0 / \rho + 3.2 \times \sqrt{}(f_{ck} / 1\ N/mm^2) \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b) = 33.6$
Actual span to depth ratio; $h_{prop} / d = 30.8$
PASS - Span to depth ratio is less than deflection control limit
Crack control - Section 7.3
Limiting crack width; $w_{max} = 0.3$ mm
Variable load factor - EN1990 – Table A1.1; $\psi_2 = 0.6$
Serviceability bending moment; $M_{sls} = 15.1$ kNm/m
Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{sfM.prov} \times z) = 199.3\ N/mm^2$
Load duration; Long term
Load duration factor; $k_t = 0.4$
Effective area of concrete in tension; $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$
$A_{c.eff} = 44790\ mm^2/m$
Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} = 2.9\ N/mm^2$
Reinforcement ratio; $\rho_{p.eff} = A_{sfM.prov} / A_{c.eff} = 0.017$
Modular ratio; $\alpha_e = E_s / E_{cm} = 6.091$
Bond property coefficient; $k_1 = 0.8$
Strain distribution coefficient; $k_2 = 0.5$
$k_3 = 3.4$
$k_4 = 0.425$
Maximum crack spacing - exp.7.11; $s_{r.max} = k_3 \times c_{sf} + k_1 \times k_2 \times k_4 \times \phi_{sfM} / \rho_{p.eff} = 206$ mm
Maximum crack width - exp.7.8; $w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
$w_k = 0.127$ mm
$w_k / w_{max} = 0.424$
PASS - Maximum crack width is less than limiting crack width
Check stem design at base of stem
Depth of section; $h = 150$ mm
Rectangular section in flexure - Section 6.1
Design bending moment combination 1; $M = 47$ kNm/m
Depth to tension reinforcement; $d = h - c_{sr} - \phi_{sr} / 2 = 90$ mm
$K = M / (d^2 \times f_{ck}) = 0.194$
$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$
$K' = 0.207$
K' > K - No compression reinforcement is required
Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = 70$ mm
Depth of neutral axis; $x = 2.5 \times (d - z) = 49$ mm
Area of tension reinforcement required; $A_{sr.req} = M / (f_{yd} \times z) = 1538\ mm^2/m$
Tension reinforcement provided; 20 dia.bars @ 150 c/c
Area of tension reinforcement provided; $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = 2094\ mm^2/m$
Minimum area of reinforcement - exp.9.1N; $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 136\ mm^2/m$
Maximum area of reinforcement - cl.9.2.1.1(3); $A_{sr.max} = 0.04 \times h = 6000\ mm^2/m$
$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} = 0.734$
PASS - Area of reinforcement provided is greater than area of reinforcement required
Library item: Rectangular single output
Deflection control - Section 7.4
Reference reinforcement ratio; $\rho_0 = \sqrt{}(f_{ck} / 1\ N/mm^2) / 1000 = 0.005$

Required tension reinforcement ratio; $\rho = A_{sr.req} / d = 0.017$
Required compression reinforcement ratio; $\rho' = A_{sr.2.req} / d_2 = 0.000$
Structural system factor - Table 7.4N; $K_b = 1$
Reinforcement factor - exp.7.17; $K_s = \min(500\ N/mm^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5) = 1.362$
Limiting span to depth ratio - exp.7.16.b; $\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times \rho_0 / (\rho - \rho') + \sqrt{(f_{ck} / 1\ N/mm^2)} \times \sqrt{(\rho' / \rho_0)} / 12], 40 \times K_b) = 18.6$
Actual span to depth ratio; $h_{prop} / d = 36.7$
FAIL - Span to depth ratio exceeds deflection control limit
Crack control - Section 7.3
Limiting crack width; $w_{max} = 0.3$ mm
Variable load factor - EN1990 – Table A1.1; $\psi_2 = 0.6$
Serviceability bending moment; $M_{sls} = 33.5$ kNm/m
Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{sr.prov} \times z) = 227.5\ N/mm^2$
Load duration; Long term
Load duration factor; $k_t = 0.4$
Effective area of concrete in tension; $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$
$A_{c.eff} = 33609\ mm^2/m$
Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} = 2.9\ N/mm^2$
Reinforcement ratio; $\rho_{p.eff} = A_{sr.prov} / A_{c.eff} = 0.062$
Modular ratio; $\alpha_e = E_s / E_{cm} = 6.091$
Bond property coefficient; $k_1 = 0.8$
Strain distribution coefficient; $k_2 = 0.5$
$k_3 = 3.4$
$k_4 = 0.425$
Maximum crack spacing - exp.7.11; $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff} = 225$ mm
Maximum crack width - exp.7.8; $w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
$w_k = 0.227$ mm
$w_k / w_{max} = 0.756$
PASS - Maximum crack width is less than limiting crack width
Rectangular section in shear - Section 6.2
Design shear force; $V = 84.3$ kN/m
$C_{Rd,c} = 0.18 / \gamma_C = 0.120$
$k = \min(1 + \sqrt{(200\ mm / d)}, 2) = 2.000$
Longitudinal reinforcement ratio; $\rho_l = \min(A_{sr.prov} / d, 0.02) = 0.020$
$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} = 0.542\ N/mm^2$
Design shear resistance - exp.6.2a & 6.2b; $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
$V_{Rd.c} = 84.6$ kN/m
$V / V_{Rd.c} = 0.997$
PASS - Design shear resistance exceeds design shear force
Check stem design at prop
Depth of section; $h = 150$ mm
Rectangular section in flexure - Section 6.1
Design bending moment combination 1; $M = 0$ kNm/m
Depth to tension reinforcement; $d = h - c_{sr} - \phi_{sr1} / 2 = 94$ mm
$K = M / (d^2 \times f_{ck}) = 0.000$

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

$K' = 0.207$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = 89$ mm

Depth of neutral axis; $x = 2.5 \times (d – z) = 12$ mm

Area of tension reinforcement required; $A_{sr1.req} = M / (f_{yd} \times z) = 1$ mm^2/m

Tension reinforcement provided; 12 dia.bars @ 200 c/c

Area of tension reinforcement provided; $A_{sr1.prov} = \pi \times \phi_{sr1}^2 / (4 \times s_{sr1}) = 565$ mm^2/m

Minimum area of reinforcement - exp.9.1N; $A_{sr1.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 142$ mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{sr1.max} = 0.04 \times h = 6000$ mm^2/m

$\max(A_{sr1.req}, A_{sr1.min}) / A_{sr1.prov} = 0.25$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio; $\rho_0 = \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) / 1000 = 0.005$

Required tension reinforcement ratio; $\rho = A_{sr1.req} / d = 0.000$

Required compression reinforcement ratio; $\rho' = A_{sr1.2.req} / d_2 = 0.000$

Structural system factor - Table 7.4N; $K_b = 0.4$

Reinforcement factor - exp.7.17; $K_s = \min(500 \text{ N/mm}^2 / (f_{yk} \times A_{sr1.req} / A_{sr1.prov}), 1.5) = 1.5$

Limiting span to depth ratio - exp.7.16.a; $\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) \times \rho_0 / \rho + 3.2 \times \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b) = 16$

Actual span to depth ratio; $(h_{stem} - h_{prop}) / d = 1.1$

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width; $w_{max} = 0.3$ mm

Variable load factor - EN1990 – Table A1.1; $\psi_2 = 0.6$

Serviceability bending moment; $M_{sls} = 0$ kNm/m

Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{sr1.prov} \times z) = 0.5$ N/mm^2

Load duration; Long term

Load duration factor; $k_t = 0.4$

Effective area of concrete in tension; $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$

$A_{c.eff} = 46083$ mm^2/m

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} = 2.9$ N/mm^2

Reinforcement ratio; $\rho_{p.eff} = A_{sr1.prov} / A_{c.eff} = 0.012$

Modular ratio; $\alpha_e = E_s / E_{cm} = 6.091$

Bond property coefficient; $k_1 = 0.8$

Strain distribution coefficient; $k_2 = 0.5$

$k_3 = 3.4$

$k_4 = 0.425$

Maximum crack spacing - exp.7.11; $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr1} / \rho_{p.eff} = 336$ mm

Maximum crack width - exp.7.8; $w_k = s_{r.max} \times \max(\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

$w_k = 0$ mm

$w_k / w_{max} = 0.002$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force; $V = 25.5$ kN/m

$C_{Rd,c} = 0.18 / \gamma_C = 0.120$

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = 2.000$

Longitudinal reinforcement ratio; $\rho_l = \min(A_{sr1.prov} / d, 0.02) = 0.006$

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = 0.542 \text{ N/mm}^2$

Design shear resistance - exp.6.2a & 6.2b; $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c} = 59.2 \text{ kN/m}$

$V / V_{Rd.c} = 0.430$

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1); $A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem}) = 524 \text{ mm}^2/\text{m}$

Maximum spacing of reinforcement – cl.9.6.3(2); $s_{sx\_max} = 400 \text{ mm}$

Transverse reinforcement provided; 12 dia.bars @ 200 c/c

Area of transverse reinforcement provided; $A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx}) = 565 \text{ mm}^2/\text{m}$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Check base design at toe

Depth of section; $h = 300 \text{ mm}$

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; $M = 50.4 \text{ kNm/m}$

Depth to tension reinforcement; $d = h - c_{bb} - \phi_{bb} / 2 = 215 \text{ mm}$

$K = M / (d^2 \times f_{ck}) = 0.036$

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

$K' = 0.207$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = 204 \text{ mm}$

Depth of neutral axis; $x = 2.5 \times (d - z) = 27 \text{ mm}$

Area of tension reinforcement required; $A_{bb.req} = M / (f_{yd} \times z) = 567 \text{ mm}^2/\text{m}$

Tension reinforcement provided; 20 dia.bars @ 200 c/c

Area of tension reinforcement provided; $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) = 1571 \text{ mm}^2/\text{m}$

Minimum area of reinforcement - exp.9.1N; $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 324 \text{ mm}^2/\text{m}$

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{bb.max} = 0.04 \times h = 12000 \text{ mm}^2/\text{m}$

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov} = 0.361$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single output

Crack control - Section 7.3

Limiting crack width; $w_{max} = 0.3 \text{ mm}$

Variable load factor - EN1990 – Table A1.1; $\psi_2 = 0.6$

Serviceability bending moment; $M_{sls} = 37.3 \text{ kNm/m}$

Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{bb.prov} \times z) = 116.3 \text{ N/mm}^2$

Load duration; Long term

Load duration factor; $k_t = 0.4$

Effective area of concrete in tension; $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$

$A_{c.eff} = 91042 \text{ mm}^2/\text{m}$

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} = 2.9 \text{ N/mm}^2$

Reinforcement ratio; $\rho_{p.eff} = A_{bb.prov} / A_{c.eff} = 0.017$

Modular ratio; $\alpha_e = E_s / E_{cm} = 6.091$
Bond property coefficient; $k_1 = 0.8$
Strain distribution coefficient; $k_2 = 0.5$
$k_3 = 3.4$
$k_4 = 0.425$
Maximum crack spacing - exp.7.11; $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff} = 452$ mm
Maximum crack width - exp.7.8; $w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
$w_k = 0.158$ mm
$w_k / w_{max} = 0.526$
PASS - Maximum crack width is less than limiting crack width
Rectangular section in shear - Section 6.2
Design shear force; $V = 16.8$ kN/m
$C_{Rd,c} = 0.18 / \gamma_C = 0.120$
$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = 1.964$
Longitudinal reinforcement ratio; $\rho_l = \min(A_{bb.prov} / d, 0.02) = 0.007$
$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = 0.528 \text{ N/mm}^2$
Design shear resistance - exp.6.2a & 6.2b; $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
$V_{Rd.c} = 141.8$ kN/m
$V / V_{Rd.c} = 0.118$
PASS - Design shear resistance exceeds design shear force
Secondary transverse reinforcement to base - Section 9.3
Minimum area of reinforcement – cl.9.3.1.1(2); $A_{bx.req} = 0.2 \times A_{bb.prov} = 314 \text{ mm}^2/\text{m}$
Maximum spacing of reinforcement – cl.9.3.1.1(3); $s_{bx\_max} = 450$ mm
Transverse reinforcement provided; 12 dia.bars @ 200 c/c
Area of transverse reinforcement provided; $A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx}) = 565 \text{ mm}^2/\text{m}$
PASS - Area of reinforcement provided is greater than area of reinforcement required

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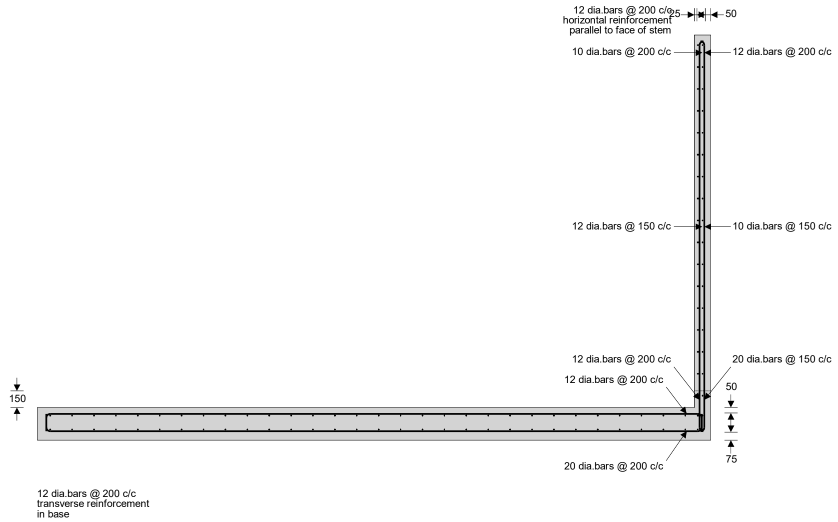

Reinforcement details

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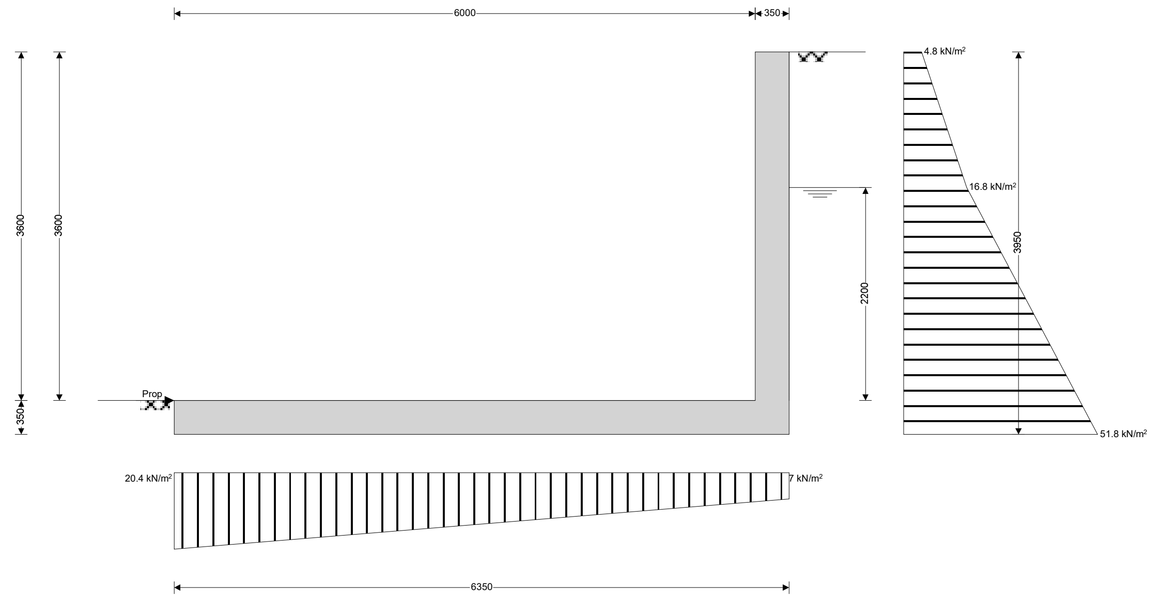
RW.2 Design

Retaining wall analysis
In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1
Tedds calculation version 2.9.04
Retaining wall details
Stem type; Cantilever
Stem height; h_{stem} = 3600 mm
Stem thickness; t_{stem} = 350 mm
Angle to rear face of stem; α = 90 deg
Stem density; γ_{stem} = 25 kN/m^3
Toe length; l_{toe} = 6000 mm
Base thickness; t_{base} = 350 mm
Base density; γ_{base} = 25 kN/m^3
Height of retained soil; h_{ret} = 3600 mm
Angle of soil surface; β = 0 deg
Depth of cover; d_{cover} = 0 mm
Height of water; h_{water} = 2200 mm
Water density; γ_w = 9.8 kN/m^3
Retained soil properties
Soil type; Organic clay
Moist density; γ_{mr} = 18 kN/m^3
Saturated density; γ_{sr} = 18 kN/m^3
Characteristic effective shear resistance angle; $\phi'_{r.k}$ = 18 deg
Characteristic wall friction angle; $\delta_{r.k}$ = 9 deg
Base soil properties
Soil type; Firm clay
Soil density; γ_b = 18 kN/m^3
Characteristic effective shear resistance angle; $\phi'_{b.k}$ = 18 deg
Characteristic wall friction angle; $\delta_{b.k}$ = 9 deg
Characteristic base friction angle; $\delta_{bb.k}$ = 12 deg
Presumed bearing capacity; $P_{bearing}$ = 100 kN/m^2
Loading details
Variable surcharge load; $Surcharge_Q$ = 10 kN/m^2



General arrangement

Calculate retaining wall geometry

Base length; $l_{base} = l_{toe} + t_{stem} = 6350$ mm

Saturated soil height; $h_{sat} = h_{water} + d_{cover} = 2200$ mm

Moist soil height; $h_{moist} = h_{ret} - h_{water} = 1400$ mm

Length of surcharge load; $l_{sur} = l_{heel} = 0$ mm

- Distance to vertical component; $x_{sur\_v} = l_{base} - l_{heel} / 2 = 6350$ mm

Effective height of wall; $h_{eff} = h_{base} + d_{cover} + h_{ret} = 3950$ mm

- Distance to horizontal component; $x_{sur\_h} = h_{eff} / 2 = 1975$ mm

Area of wall stem; $A_{stem} = h_{stem} \times t_{stem} = 1.26\ m^2$

- Distance to vertical component; $x_{stem} = l_{toe} + t_{stem} / 2 = 6175$ mm

Area of wall base; $A_{base} = l_{base} \times t_{base} = 2.223\ m^2$

- Distance to vertical component; $x_{base} = l_{base} / 2 = 3175$ mm

Using Coulomb theory

Active pressure coefficient; $K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \surd[\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]]^2) = 0.483$

Passive pressure coefficient; $K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \surd[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]]^2) = 2.359$

Bearing pressure check

Vertical forces on wall

Wall stem; $F_{stem} = A_{stem} \times \gamma_{stem} = 31.5$ kN/m

Wall base; $F_{base} = A_{base} \times \gamma_{base} = 55.6$ kN/m

Total; $F_{total\_v} = F_{stem} + F_{base} + F_{water\_v} = 87.1$ kN/m

Horizontal forces on wall

Surcharge load; $F_{sur\_h} = K_A \times \cos(\delta_{r.k}) \times Surcharge_Q \times h_{eff} = 18.8$ kN/m

Saturated retained soil; $F_{sat\_h} = K_A \times \cos(\delta_{r.k}) \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2 = 12.7$ kN/m

Water; $F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 = 31.9$ kN/m

Moist retained soil; $F_{moist\_h} = K_A \times \cos(\delta_{r.k}) \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) = 39.1$ kN/m

Base soil; $F_{pass\_h} = -K_P \times \cos(\delta_{b.k}) \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 = -2.6$ kN/m

Total; $F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h} = 100$ kN/m

Moments on wall

Wall stem; $M_{stem} = F_{stem} \times x_{stem} = 194.5$ kNm/m

Wall base; $M_{base} = F_{base} \times x_{base} = 176.4$ kNm/m

Surcharge load; $M_{sur} = -F_{sur\_h} \times x_{sur\_h} = -37.2$ kNm/m

Saturated retained soil; $M_{sat} = -F_{sat\_h} \times x_{sat\_h} = -10.8$ kNm/m

Water; $M_{water} = -F_{water\_h} \times x_{water\_h} = -27.1$ kNm/m

Moist retained soil; $M_{moist} = -F_{moist\_h} \times x_{moist\_h} = -64.5$ kNm/m

Total; $M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{water} + M_{sur} = 231.3$ kNm/m

Check bearing pressure

Propping force; $F_{prop\_base} = F_{total\_h} = 100$ kN/m

Distance to reaction; $\bar{x} = M_{total} / F_{total\_v} = 2657$ mm

Eccentricity of reaction; $e = \bar{x} - l_{base} / 2 = -518$ mm

Loaded length of base; $l_{load} = l_{base} = 6350$ mm

Bearing pressure at toe; $q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) = 20.4$ kN/m^2

Bearing pressure at heel; $q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) = 7$ kN/m^2

Factor of safety; $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = 4.896$

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

Retaining wall design

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 2.9.04

Concrete details - Table 3.1 - Strength and deformation characteristics for concrete

Concrete strength class; C30/37

Characteristic compressive cylinder strength; $f_{ck} = 30$ N/mm^2

Characteristic compressive cube strength; $f_{ck,cube} = 37$ N/mm^2

Mean value of compressive cylinder strength; $f_{cm} = f_{ck} + 8$ N/mm^2 = 38 N/mm^2

Mean value of axial tensile strength; $f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck} / 1$ N/mm$^2)^{2/3} = 2.9$ N/mm^2

5% fractile of axial tensile strength; $f_{ctk,0.05} = 0.7 \times f_{ctm} = 2.0$ N/mm^2

Secant modulus of elasticity of concrete; $E_{cm} = 22$ kN/mm$^2 \times (f_{cm} / 10$ N/mm$^2)^{0.3} = 32837$ N/mm^2

Partial factor for concrete - Table 2.1N; $\gamma_C = 1.50$

Compressive strength coefficient - cl.3.1.6(1); $\alpha_{cc} = 0.85$

Design compressive concrete strength - exp.3.15; $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = 17.0$ N/mm^2

Maximum aggregate size; $h_{agg} = 20$ mm

Ultimate strain - Table 3.1; $\varepsilon_{cu2} = 0.0035$

Shortening strain - Table 3.1; $\varepsilon_{cu3} = 0.0035$

Effective compression zone height factor; $\lambda = 0.80$

Effective strength factor; $\eta = 1.00$

Bending coefficient k_1; $K_1 = 0.40$

Bending coefficient k_2; $K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2}) = 1.00$

Bending coefficient k_3; $K_3 = 0.40$

Bending coefficient k_4; $K_4 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2}) = 1.00$

Reinforcement details

Characteristic yield strength of reinforcement; $f_{yk} = 500$ N/mm^2

Modulus of elasticity of reinforcement; $E_s = 200000$ N/mm^2

Partial factor for reinforcing steel - Table 2.1N; $\gamma_S = 1.15$

Design yield strength of reinforcement; $f_{yd} = f_{yk} / \gamma_S = 435$ N/mm^2

Cover to reinforcement

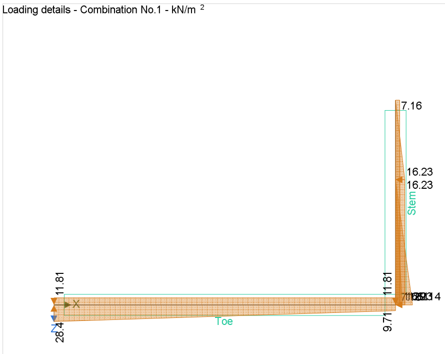
Front face of stem; $c_{sf} = 30$ mm

Rear face of stem; $c_{sr} = 35$ mm

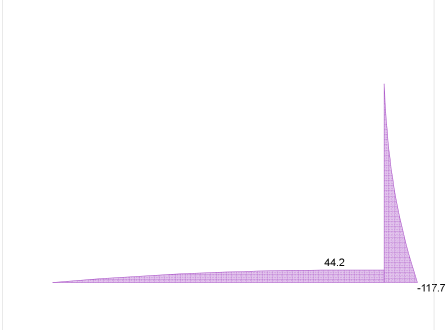
Top face of base; $c_{bt} = 50$ mm

Bottom face of base; $c_{bb} = 75$ mm

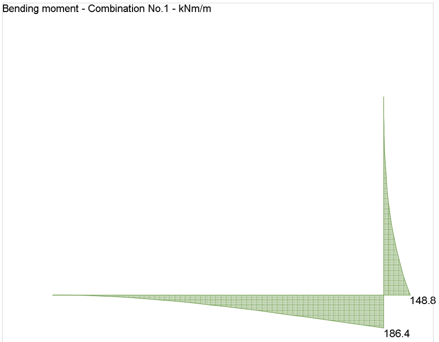
Loading details - Combination No.1 - kN/m ^{2}

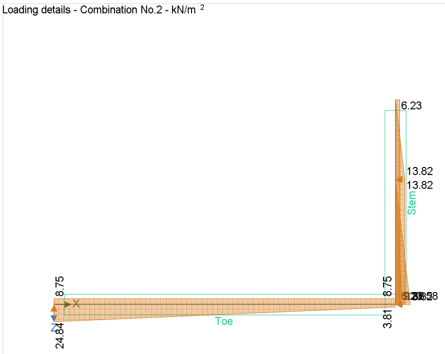


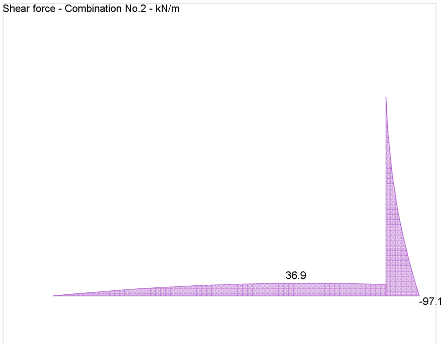
Shear force - Combination No.1 - kN/m

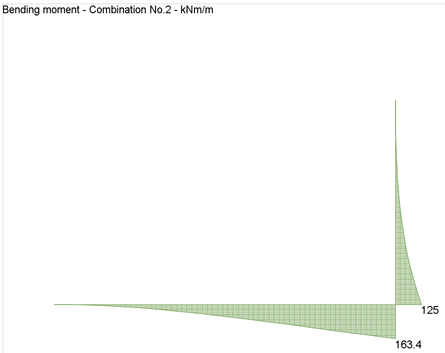


Bending moment - Combination No.1 - kNm/m









Check stem design at base of stem

Depth of section; h = 350 mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; M = 148.8 kNm/m

Depth to tension reinforcement; $d = h - c_{sr} - \phi_{sr} / 2 = 305$ mm

$K = M / (d^2 \times f_{ck}) = 0.053$

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

$K' = 0.207$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = 290$ mm

Depth of neutral axis; $x = 2.5 \times (d - z) = 38$ mm

Area of tension reinforcement required; $A_{sr.req} = M / (f_{yd} \times z) = 1181$ mm²/m

Tension reinforcement provided; 20 dia.bars @ 200 c/c

Area of tension reinforcement provided; $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = 1571$ mm²/m

Minimum area of reinforcement - exp.9.1N; $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 459$ mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{sr.max} = 0.04 \times h = 14000$ mm²/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} = 0.752$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio; $\rho_0 = \sqrt{}(f_{ck} / 1\ \text{N/mm}^2) / 1000 = 0.005$

Required tension reinforcement ratio; $\rho = A_{sr.req} / d = 0.004$

Required compression reinforcement ratio; $\rho' = A_{sr.2.req} / d_2 = 0.000$

Structural system factor - Table 7.4N; $K_b = 0.4$

Reinforcement factor - exp.7.17; $K_s = \min(500\ \text{N/mm}^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5) = 1.33$

Limiting span to depth ratio - exp.7.16.a; $\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{}(f_{ck} / 1\ N/mm^2) \times \rho_0 / \rho + 3.2 \times \sqrt{}(f_{ck} / 1\ N/mm^2) \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b) = 14.5$
Actual span to depth ratio; $h_{stem} / d = 11.8$
PASS - Span to depth ratio is less than deflection control limit
Rectangular section in shear - Section 6.2
Design shear force; V = 117.7 kN/m
$C_{Rd,c} = 0.18 / \gamma_C = 0.120$
$k = \min(1 + \sqrt{}(200\ mm / d), 2) = 1.810$
Longitudinal reinforcement ratio; $\rho_l = \min(A_{sr.prov} / d, 0.02) = 0.005$
$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} = 0.467\ N/mm^2$
Design shear resistance - exp.6.2a & 6.2b; $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
$V_{Rd.c} = 165\ kN/m$
$V / V_{Rd.c} = 0.713$
PASS - Design shear resistance exceeds design shear force
Horizontal reinforcement parallel to face of stem - Section 9.6
Minimum area of reinforcement – cl.9.6.3(1); $A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem}) = 393\ mm^2/m$
Maximum spacing of reinforcement – cl.9.6.3(2); $s_{sx\_max} = 400\ mm$
Transverse reinforcement provided; 10 dia.bars @ 200 c/c
Area of transverse reinforcement provided; $A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx}) = 393\ mm^2/m$
PASS - Area of reinforcement provided is greater than area of reinforcement required
Check base design at toe
Depth of section; h = 350 mm
Rectangular section in flexure - Section 6.1
Design bending moment combination 1;M = 186.4 kNm/m
Depth to tension reinforcement; $d = h - c_{bb} - \phi_{bb} / 2 = 263\ mm$
$K = M / (d^2 \times f_{ck}) = 0.090$
$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$
$K' = 0.207$
K' > K - No compression reinforcement is required
Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = 240\ mm$
Depth of neutral axis; $x = 2.5 \times (d – z) = 57\ mm$
Area of tension reinforcement required;$A_{bb.req} = M / (f_{yd} \times z) = 1790\ mm^2/m$
Tension reinforcement provided; 25 dia.bars @ 200 c/c
Area of tension reinforcement provided; $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) = 2454\ mm^2/m$
Minimum area of reinforcement - exp.9.1N; $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 395\ mm^2/m$
Maximum area of reinforcement - cl.9.2.1.1(3); $A_{bb.max} = 0.04 \times h = 14000\ mm^2/m$
$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov} = 0.729$
PASS - Area of reinforcement provided is greater than area of reinforcement required
Library item: Rectangular single output
Rectangular section in shear - Section 6.2
Design shear force; V = 44.2 kN/m
$C_{Rd,c} = 0.18 / \gamma_C = 0.120$
$k = \min(1 + \sqrt{}(200\ mm / d), 2) = 1.873$
Longitudinal reinforcement ratio; $\rho_l = \min(A_{bb.prov} / d, 0.02) = 0.009$

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$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} = 0.491\ N/mm^2$

Design shear resistance - exp.6.2a & 6.2b; $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c} = 179.3\ kN/m$

$V / V_{Rd.c} = 0.246$

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

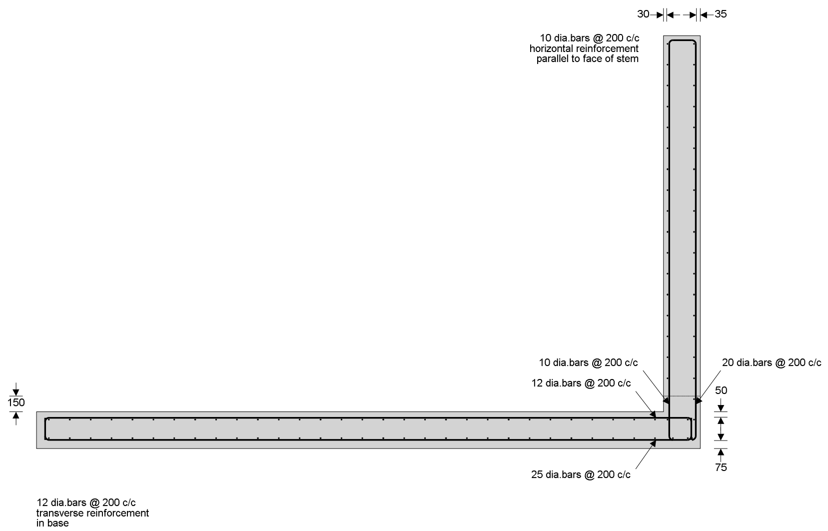
Minimum area of reinforcement – cl.9.3.1.1(2); $A_{bx.req} = 0.2 \times A_{bb.prov} = 491\ mm^2/m$

Maximum spacing of reinforcement – cl.9.3.1.1(3); $s_{bx\_max} = 450\ mm$

Transverse reinforcement provided; 12 dia.bars @ 200 c/c

Area of transverse reinforcement provided; $A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx}) = 565\ mm^2/m$

PASS - Area of reinforcement provided is greater than area of reinforcement required



Reinforcement details

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RW.3 Design

Retaining wall analysis

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.04

Retaining wall details

Stem type; Propped cantilever

Stem height; h_{stem} = 3400 mm

Prop height; h_{prop} = 3300 mm

Stem thickness; t_{stem} = 250 mm

Angle to rear face of stem; α = 90 deg

Stem density; γ_{stem} = 25 kN/m^3

Toe length; l_{toe} = 6000 mm

Base thickness; t_{base} = 350 mm

Base density; γ_{base} = 25 kN/m^3

Height of retained soil; h_{ret} = 3400 mm

Angle of soil surface; β = 0 deg

Depth of cover; d_{cover} = 0 mm

Height of water; h_{water} = 2200 mm

Water density; γ_w = 9.8 kN/m^3

Retained soil properties

Soil type; Organic clay

Moist density; γ_{mr} = 18 kN/m^3

Saturated density; γ_{sr} = 18 kN/m^3

Characteristic effective shear resistance angle; $\phi'_{r.k}$ = 18 deg

Characteristic wall friction angle; $\delta_{r.k}$ = 9 deg

Base soil properties

Soil type; Firm clay

Soil density; γ_b = 18 kN/m^3

Characteristic effective shear resistance angle; $\phi'_{b.k}$ = 18 deg

Characteristic wall friction angle; $\delta_{b.k}$ = 9 deg

Characteristic base friction angle; $\delta_{bb.k}$ = 12 deg

Presumed bearing capacity; $P_{bearing}$ = 100 kN/m^2

Loading details

Variable surcharge load; $Surcharge_Q$ = 5 kN/m^2

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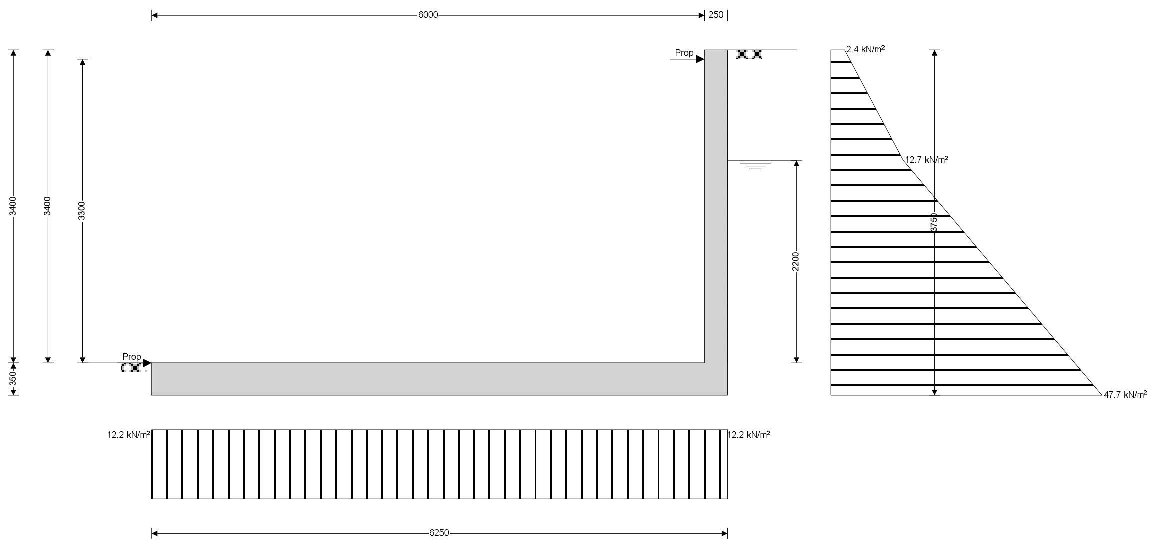


General arrangement

Calculate retaining wall geometry

Base length; $l_{base} = l_{toe} + t_{stem} = 6250$ mm

Saturated soil height; $h_{sat} = h_{water} + d_{cover} = 2200$ mm

Moist soil height; $h_{moist} = h_{ret} - h_{water} = 1200$ mm

Length of surcharge load; $l_{sur} = l_{heel} = 0$ mm

- Distance to vertical component; $x_{sur\_v} = l_{base} - l_{heel} / 2 = 6250$ mm

Effective height of wall; $h_{eff} = h_{base} + d_{cover} + h_{ret} = 3750$ mm

- Distance to horizontal component; $x_{sur\_h} = h_{eff} / 2 = 1875$ mm

Area of wall stem; $A_{stem} = h_{stem} \times t_{stem} = 0.85\ m^2$

- Distance to vertical component; $x_{stem} = l_{toe} + t_{stem} / 2 = 6125$ mm

Area of wall base; $A_{base} = l_{base} \times t_{base} = 2.188\ m^2$

- Distance to vertical component; $x_{base} = l_{base} / 2 = 3125$ mm

Using Coulomb theory

Active pressure coefficient; $K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[}\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]]^2) = 0.483$

Passive pressure coefficient; $K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[}\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]]^2) = 2.359$

Bearing pressure check

Vertical forces on wall

Wall stem; $F_{stem} = A_{stem} \times \gamma_{stem} = 21.3$ kN/m

Wall base; $F_{base} = A_{base} \times \gamma_{base} = 54.7$ kN/m

Total; $F_{total\_v} = F_{stem} + F_{base} + F_{water\_v} = 75.9$ kN/m

Horizontal forces on wall

Surcharge load; $F_{sur\_h} = K_A \times \cos(\delta_{r.d}) \times Surcharge_Q \times h_{eff} = 9$ kN/m

Saturated retained soil; $F_{sat\_h} = K_A \times \cos(\delta_{r.d}) \times (\gamma_{sr}' - \gamma_w') \times (h_{sat} + h_{base})^2 / 2 = 12.8$ kN/m

Water; $F_{water\_h} = \gamma_w' \times (h_{water} + d_{cover} + h_{base})^2 / 2 = 31.9$ kN/m

Moist retained soil; $F_{moist\_h} = K_A \times \cos(\delta_{r.d}) \times \gamma_{mr}' \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) = 32.6$ kN/m
Base soil; $F_{pass\_h} = -K_P \times \cos(\delta_{b.d}) \times \gamma_b' \times (d_{cover} + h_{base})^2 / 2 = -2.6$ kN/m
Total; $F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h} = 83.7$ kN/m
Moments on wall
Wall stem; $M_{stem} = F_{stem} \times x_{stem} = 130.2$ kNm/m
Wall base; $M_{base} = F_{base} \times x_{base} = 170.9$ kNm/m
Surcharge load; $M_{sur} = -F_{sur\_h} \times x_{sur\_h} = -16.8$ kNm/m
Saturated retained soil; $M_{sat} = -F_{sat\_h} \times x_{sat\_h} = -10.8$ kNm/m
Water; $M_{water} = -F_{water\_h} \times x_{water\_h} = -27.1$ kNm/m
Moist retained soil; $M_{moist} = -F_{moist\_h} \times x_{moist\_h} = -52$ kNm/m
Total; $M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{water} + M_{sur} = 194.3$ kNm/m
Check bearing pressure
Propping force to stem; $F_{prop\_stem} = (F_{total\_v} \times l_{base} / 2 - M_{total}) / (h_{prop} + t_{base}) = 11.8$ kN/m
Propping force to base; $F_{prop\_base} = F_{total\_h} - F_{prop\_stem} = 71.9$ kN/m
Moment from propping force; $M_{prop} = F_{prop\_stem} \times (h_{prop} + t_{base}) = 43$ kNm/m
Distance to reaction; $\bar{x} = (M_{total} + M_{prop}) / F_{total\_v} = 3125$ mm
Eccentricity of reaction; $e = \bar{x} - l_{base} / 2 = 0$ mm
Loaded length of base; $l_{load} = l_{base} = 6250$ mm
Bearing pressure at toe; $q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) = 12.2$ kN/m^2
Bearing pressure at heel; $q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) = 12.2$ kN/m^2
Factor of safety; $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = 8.23$
PASS - Allowable bearing pressure exceeds maximum applied bearing pressure
Retaining wall design
In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1
Tedds calculation version 2.9.04
Concrete details - Table 3.1 - Strength and deformation characteristics for concrete
Concrete strength class;C30/37
Characteristic compressive cylinder strength; $f_{ck} = 30$ N/mm^2
Characteristic compressive cube strength; $f_{ck,cube} = 37$ N/mm^2
Mean value of compressive cylinder strength; $f_{cm} = f_{ck} + 8$ N/mm^2 = 38 N/mm^2
Mean value of axial tensile strength; $f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck} / 1$ N/mm$^2)^{2/3} = 2.9$ N/mm^2
5% fractile of axial tensile strength; $f_{ctk,0.05} = 0.7 \times f_{ctm} = 2.0$ N/mm^2
Secant modulus of elasticity of concrete; $E_{cm} = 22$ kN/mm$^2 \times (f_{cm} / 10$ N/mm$^2)^{0.3} = 32837$ N/mm^2
Partial factor for concrete - Table 2.1N; $\gamma_C = 1.50$
Compressive strength coefficient - cl.3.1.6(1); $\alpha_{cc} = 0.85$
Design compressive concrete strength - exp.3.15; $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = 17.0$ N/mm^2
Maximum aggregate size; $h_{agg} = 20$ mm
Ultimate strain - Table 3.1; $\varepsilon_{cu2} = 0.0035$
Shortening strain - Table 3.1; $\varepsilon_{cu3} = 0.0035$
Effective compression zone height factor; $\lambda = 0.80$
Effective strength factor; $\eta = 1.00$
Bending coefficient k$_1$; $K_1 = 0.40$
Bending coefficient k$_2$; $K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2}) = 1.00$

Bending coefficient k_3; K_3 =0.40

Bending coefficient k_4; $K_4 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2}) = 1.00$

Reinforcement details

Characteristic yield strength of reinforcement; f_{yk} = 500 N/mm^2

Modulus of elasticity of reinforcement; E_s = 200000 N/mm^2

Partial factor for reinforcing steel - Table 2.1N; γ_S = 1.15

Design yield strength of reinforcement; $f_{yd} = f_{yk} / \gamma_S$ = 435 N/mm^2

Cover to reinforcement

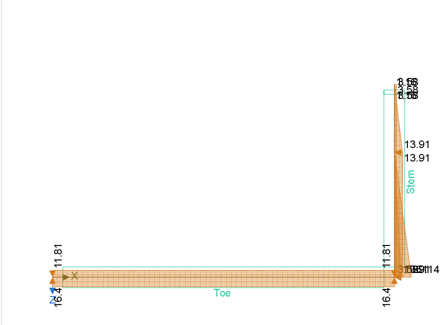
Front face of stem; c_{sf} = 35 mm

Rear face of stem; c_{sr} = 75 mm

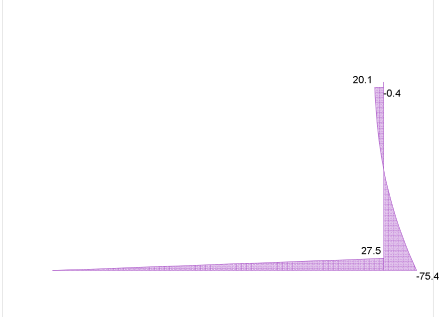
Top face of base; c_{bt} = 50 mm

Bottom face of base; c_{bb} = 75 mm

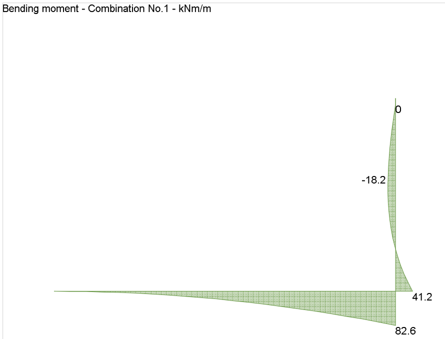
Loading details - Combination No.1 - kN/m ^{2}



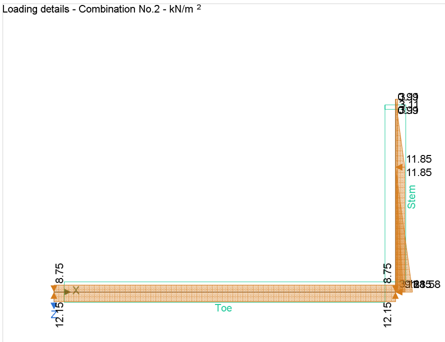
Shear force - Combination No.1 - kN/m



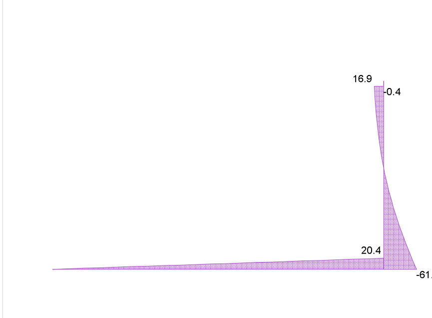
Bending moment - Combination No.1 - kNm/m



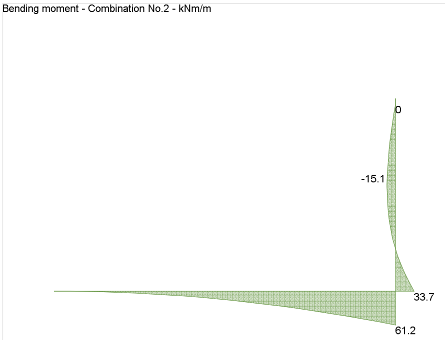
Loading details - Combination No.2 - kN/m ²



Shear force - Combination No.2 - kN/m



Bending moment - Combination No.2 - kNm/m



Check stem design at 1824 mm

Depth of section; $h = 250$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; $M = 18.2$ kNm/m

Depth to tension reinforcement; $d = h - c_{sf} - \phi_{sx} - \phi_{sfM} / 2 = 200$ mm

$K = M / (d^2 \times f_{ck}) = 0.015$

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

$K' = 0.207$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = 190$ mm

Depth of neutral axis; $x = 2.5 \times (d – z) = 25$ mm

Area of tension reinforcement required; $A_{sfM.req} = M / (f_{yd} \times z) = 221$ mm^2/m

Tension reinforcement provided; 10 dia.bars @ 200 c/c

Area of tension reinforcement provided; $A_{sfM.prov} = \pi \times \phi_{sfM}^2 / (4 \times s_{sfM}) = 393$ mm^2/m

Minimum area of reinforcement - exp.9.1N; $A_{sfM.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 301$ mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{sfM.max} = 0.04 \times h = 10000$ mm^2/m

$\max(A_{sfM.req}, A_{sfM.min}) / A_{sfM.prov} = 0.767$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio; $\rho_0 = \sqrt{}(f_{ck} / 1\ \text{N/mm}^2) / 1000 = 0.005$

Required tension reinforcement ratio; $\rho = A_{sfM.req} / d = 0.001$

Required compression reinforcement ratio; $\rho' = A_{sfM.2.req} / d_2 = 0.000$

Structural system factor - Table 7.4N; $K_b = 1$

Reinforcement factor - exp.7.17; $K_s = \min(500\ \text{N/mm}^2 / (f_{yk} \times A_{sfM.req} / A_{sfM.prov}), 1.5) = 1.5$

Limiting span to depth ratio - exp.7.16.a; $\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{}(f_{ck} / 1\ N/mm^2) \times \rho_0 / \rho + 3.2 \times \sqrt{}(f_{ck} / 1\ N/mm^2) \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b) = 40$

Actual span to depth ratio; $h_{prop} / d = 16.5$

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width; $w_{max} = 0.3$ mm

Variable load factor - EN1990 – Table A1.1; $\psi_2 = 0.6$

Serviceability bending moment; $M_{sls} = 12.6$ kNm/m

Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{sfM.prov} \times z) = 169\ N/mm^2$

Load duration; Long term

Load duration factor; $k_t = 0.4$

Effective area of concrete in tension; $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$

$A_{c.eff} = 75000\ mm^2/m$

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} = 2.9\ N/mm^2$

Reinforcement ratio; $\rho_{p.eff} = A_{sfM.prov} / A_{c.eff} = 0.005$

Modular ratio; $\alpha_e = E_s / E_{cm} = 6.091$

Bond property coefficient; $k_1 = 0.8$

Strain distribution coefficient; $k_2 = 0.5$

$k_3 = 3.4$

$k_4 = 0.425$

Maximum crack spacing - exp.7.11; $s_{r.max} = k_3 \times c_{sf} + k_1 \times k_2 \times k_4 \times \phi_{sfM} / \rho_{p.eff} = 444$ mm

Maximum crack width - exp.7.8; $w_k = s_{r.max} \times \max(\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

$w_k = 0.225$ mm

$w_k / w_{max} = 0.75$

PASS - Maximum crack width is less than limiting crack width

Check stem design at base of stem

Depth of section; $h = 250$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; $M = 41.2$ kNm/m

Depth to tension reinforcement; $d = h - c_{sr} - \phi_{sr} / 2 = 167$ mm

$K = M / (d^2 \times f_{ck}) = 0.049$

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

$K' = 0.207$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = 159$ mm

Depth of neutral axis; $x = 2.5 \times (d – z) = 21$ mm

Area of tension reinforcement required; $A_{sr.req} = M / (f_{yd} \times z) = 597\ mm^2/m$

Tension reinforcement provided; 16 dia.bars @ 200 c/c

Area of tension reinforcement provided; $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = 1005\ mm^2/m$

Minimum area of reinforcement - exp.9.1N; $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 252\ mm^2/m$

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{sr.max} = 0.04 \times h = 10000\ mm^2/m$

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} = 0.594$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio; $\rho_0 = \sqrt{}(f_{ck} / 1\ N/mm^2) / 1000 = 0.005$

Required tension reinforcement ratio; $\rho = A_{sr.req} / d = 0.004$
Required compression reinforcement ratio; $\rho' = A_{sr.2.req} / d_2 = 0.000$
Structural system factor - Table 7.4N; $K_b = 1$
Reinforcement factor - exp.7.17; $K_s = \min(500\ N/mm^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5) = 1.5$
Limiting span to depth ratio - exp.7.16.a; $\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b) = 40$
Actual span to depth ratio; $h_{prop} / d = 19.8$
PASS - Span to depth ratio is less than deflection control limit
Crack control - Section 7.3
Limiting crack width; $w_{max} = 0.3$ mm
Variable load factor - EN1990 – Table A1.1; $\psi_2 = 0.6$
Serviceability bending moment; $M_{sls} = 28.8$ kNm/m
Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{sr.prov} \times z) = 180.9\ N/mm^2$
Load duration; Long term
Load duration factor; $k_t = 0.4$
Effective area of concrete in tension; $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$
$A_{c.eff} = 76375\ mm^2/m$
Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} = 2.9\ N/mm^2$
Reinforcement ratio; $\rho_{p.eff} = A_{sr.prov} / A_{c.eff} = 0.013$
Modular ratio; $\alpha_e = E_s / E_{cm} = 6.091$
Bond property coefficient; $k_1 = 0.8$
Strain distribution coefficient; $k_2 = 0.5$
$k_3 = 3.4$
$k_4 = 0.425$
Maximum crack spacing - exp.7.11; $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff} = 462$ mm
Maximum crack width - exp.7.8; $w_k = s_{r.max} \times \max(\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
$w_k = 0.25$ mm
$w_k / w_{max} = 0.835$
PASS - Maximum crack width is less than limiting crack width
Rectangular section in shear - Section 6.2
Design shear force; $V = 75.4$ kN/m
$C_{Rd,c} = 0.18 / \gamma_C = 0.120$
$k = \min(1 + \sqrt{(200\ mm / d)}, 2) = 2.000$
Longitudinal reinforcement ratio; $\rho_l = \min(A_{sr.prov} / d, 0.02) = 0.006$
$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} = 0.542\ N/mm^2$
Design shear resistance - exp.6.2a & 6.2b; $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
$V_{Rd.c} = 105.2$ kN/m
$V / V_{Rd.c} = 0.717$
PASS - Design shear resistance exceeds design shear force
Check stem design at prop
Depth of section; $h = 250$ mm
Rectangular section in flexure - Section 6.1
Design bending moment combination 1; $M = 0$ kNm/m
Depth to tension reinforcement; $d = h - c_{sr} - \phi_{sr1} / 2 = 170$ mm
$K = M / (d^2 \times f_{ck}) = 0.000$

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$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$
$K' = 0.207$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = 161 mm

Depth of neutral axis; $x = 2.5 \times (d - z)$ = 21 mm

Area of tension reinforcement required; $A_{sr1.req} = M / (f_{yd} \times z)$ = 0 mm²/m

Tension reinforcement provided; 10 dia.bars @ 200 c/c

Area of tension reinforcement provided; $A_{sr1.prov} = \pi \times \phi_{sr1}^2 / (4 \times s_{sr1})$ = 393 mm²/m

Minimum area of reinforcement - exp.9.1N; $A_{sr1.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = 256 mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{sr1.max} = 0.04 \times h$ = 10000 mm²/m

$\max(A_{sr1.req}, A_{sr1.min}) / A_{sr1.prov} = 0.652$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio; $\rho_0 = \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) / 1000 = 0.005$

Required tension reinforcement ratio; $\rho = A_{sr1.req} / d = 0.000$

Required compression reinforcement ratio; $\rho' = A_{sr1.2.req} / d_2 = 0.000$

Structural system factor - Table 7.4N; $K_b = 0.4$

Reinforcement factor - exp.7.17; $K_s = \min(500 \text{ N/mm}^2 / (f_{yk} \times A_{sr1.req} / A_{sr1.prov}), 1.5) = 1.5$

Limiting span to depth ratio - exp.7.16.a; $\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) \times \rho_0 / \rho + 3.2 \times \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b) = 16$

Actual span to depth ratio; $(h_{stem} - h_{prop}) / d = 0.6$

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width; w_{max} = 0.3 mm

Variable load factor - EN1990 – Table A1.1; $\psi_2 = 0.6$

Serviceability bending moment; M_{sls} = 0 kNm/m

Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{sr1.prov} \times z)$ = 0.1 N/mm²

Load duration; Long term

Load duration factor; $k_t = 0.4$

Effective area of concrete in tension; $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$

$A_{c.eff}$ = 76250 mm²/m

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm}$ = 2.9 N/mm²

Reinforcement ratio; $\rho_{p.eff} = A_{sr1.prov} / A_{c.eff} = 0.005$

Modular ratio; $\alpha_e = E_s / E_{cm} = 6.091$

Bond property coefficient; $k_1 = 0.8$

Strain distribution coefficient; $k_2 = 0.5$

$k_3 = 3.4$

$k_4 = 0.425$

Maximum crack spacing - exp.7.11; $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr1} / \rho_{p.eff}$ = 585 mm

Maximum crack width - exp.7.8; $w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

w_k = 0 mm

$w_k / w_{max} = 0.001$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force; V = 20.1 kN/m

$C_{Rd,c} = 0.18 / \gamma_C = 0.120$

$k = \min(1 + \sqrt{(200\ mm / d)}, 2) = 2.000$

Longitudinal reinforcement ratio; $\rho_l = \min(A_{sr1.prov} / d, 0.02) = 0.002$

$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} = 0.542\ N/mm^2$

Design shear resistance - exp.6.2a & 6.2b; $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c} = 92.2\ kN/m$

$V / V_{Rd.c} = 0.218$

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1); $A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem}) = 251\ mm^2/m$

Maximum spacing of reinforcement – cl.9.6.3(2); $s_{sx\_max} = 400\ mm$

Transverse reinforcement provided; 10 dia.bars @ 200 c/c

Area of transverse reinforcement provided; $A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx}) = 393\ mm^2/m$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Check base design at toe

Depth of section; $h = 350\ mm$

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; $M = 82.6\ kNm/m$

Depth to tension reinforcement; $d = h - c_{bb} - \phi_{bb} / 2 = 265\ mm$

$K = M / (d^2 \times f_{ck}) = 0.039$

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

$K' = 0.207$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = 252\ mm$

Depth of neutral axis; $x = 2.5 \times (d – z) = 33\ mm$

Area of tension reinforcement required; $A_{bb.req} = M / (f_{yd} \times z) = 755\ mm^2/m$

Tension reinforcement provided; 20 dia.bars @ 200 c/c

Area of tension reinforcement provided; $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) = 1571\ mm^2/m$

Minimum area of reinforcement - exp.9.1N; $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 399\ mm^2/m$

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{bb.max} = 0.04 \times h = 14000\ mm^2/m$

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov} = 0.481$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single output

Crack control - Section 7.3

Limiting crack width; $w_{max} = 0.3\ mm$

Variable load factor - EN1990 – Table A1.1; $\psi_2 = 0.6$

Serviceability bending moment; $M_{sls} = 61.2\ kNm/m$

Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{bb.prov} \times z) = 154.8\ N/mm^2$

Load duration; Long term

Load duration factor; $k_t = 0.4$

Effective area of concrete in tension; $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$

$A_{c.eff} = 105625\ mm^2/m$

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} = 2.9\ N/mm^2$

Reinforcement ratio; $\rho_{p.eff} = A_{bb.prov} / A_{c.eff} = 0.015$

Modular ratio; $\alpha_e = E_s / E_{cm} = 6.091$
Bond property coefficient; $k_1 = 0.8$
Strain distribution coefficient; $k_2 = 0.5$
$k_3 = 3.4$
$k_4 = 0.425$
Maximum crack spacing - exp.7.11; $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff} = 484$ mm
Maximum crack width - exp.7.8; $w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
$w_k = 0.225$ mm
$w_k / w_{max} = 0.748$
PASS - Maximum crack width is less than limiting crack width
Rectangular section in shear - Section 6.2
Design shear force; $V = 27.5$ kN/m
$C_{Rd,c} = 0.18 / \gamma_C = 0.120$
$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = 1.869$
Longitudinal reinforcement ratio; $\rho_l = \min(A_{bb.prov} / d, 0.02) = 0.006$
$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = 0.490 \text{ N/mm}^2$
Design shear resistance - exp.6.2a & 6.2b; $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
$V_{Rd.c} = 155.1$ kN/m
$V / V_{Rd.c} = 0.178$
PASS - Design shear resistance exceeds design shear force
Secondary transverse reinforcement to base - Section 9.3
Minimum area of reinforcement – cl.9.3.1.1(2); $A_{bx.req} = 0.2 \times A_{bb.prov} = 314 \text{ mm}^2/\text{m}$
Maximum spacing of reinforcement – cl.9.3.1.1(3); $s_{bx\_max} = 450$ mm
Transverse reinforcement provided; 10 dia.bars @ 200 c/c
Area of transverse reinforcement provided; $A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx}) = 393 \text{ mm}^2/\text{m}$
PASS - Area of reinforcement provided is greater than area of reinforcement required

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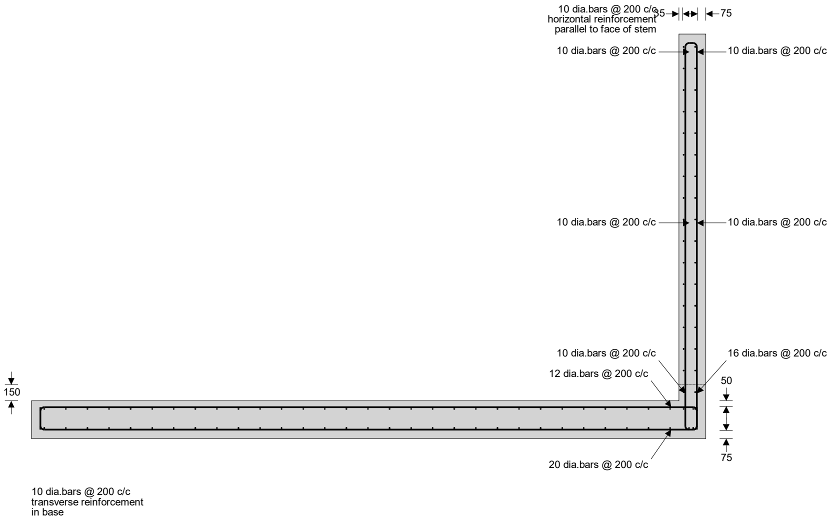

Reinforcement details

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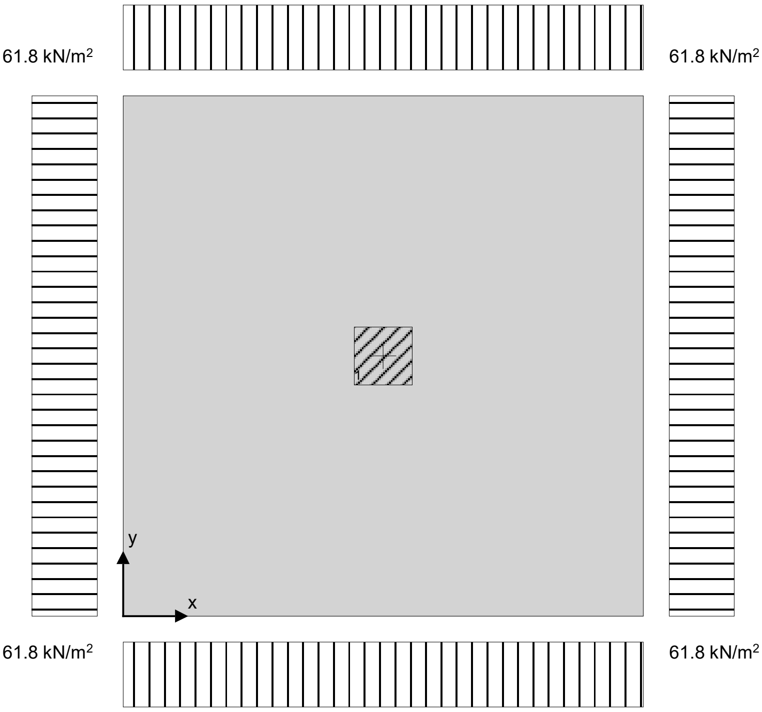
S0.1 Design

P1 pad

Foundation analysis (EN1997-1:2004)
In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1
TEDDS calculation version 3.2.15

Pad foundation details
Length of foundation; L_x = 900 mm
Width of foundation; L_y = 900 mm
Foundation area; $A = L_x \times L_y = 0.810\ m^2$
Depth of foundation; h = 350 mm
Depth of soil over foundation; h_{soil} = 0 mm
Level of water; h_{water} = 0 mm
Density of water; $\gamma_{water} = 9.8\ kN/m^3$
Density of concrete; $\gamma_{conc} = 25.0\ kN/m^3$



Column no.1 details
Length of column; l_{x1} = 100 mm
Width of column; l_{y1} = 100 mm
position in x-axis; x_1 = 450 mm
position in y-axis; y_1 = 450 mm
Soil properties
Density of soil; $\gamma_{soil} = 18.0\ kN/m^3$
Characteristic cohesion; $c'_k = 0\ kN/m^2$

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Characteristic effective shear resistance angle; $\phi'_k = 30$ deg
Characteristic friction angle; $\delta_k = 20$ deg
Foundation loads
Self weight; $F_{swt} = h \times \gamma_{conc} = 8.8$ kN/m^2
Column no.1 loads
Permanent load in z; $F_{Gz1} = 30.0$ kN
Variable load in z; $F_{Qz1} = 13.0$ kN
Bearing resistance (Section 6.5.2)
Forces on foundation
Force in z-axis; $F_{dz} = A \times F_{swt} + F_{Gz1} + F_{Qz1} = 50.1$ kN
Moments on foundation
Moment in x-axis; $M_{dx} = A \times F_{swt} \times L_x / 2 + F_{Gz1} \times x_1 + F_{Qz1} \times x_1 = 22.5$ kNm
Moment in y-axis; $M_{dy} = A \times F_{swt} \times L_y / 2 + F_{Gz1} \times y_1 + F_{Qz1} \times y_1 = 22.5$ kNm
Eccentricity of base reaction
Eccentricity of base reaction in x-axis; $e_x = M_{dx} / F_{dz} - L_x / 2 = 0$ mm
Eccentricity of base reaction in y-axis; $e_y = M_{dy} / F_{dz} - L_y / 2 = 0$ mm
Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_x / L_x - 6 \times e_y / L_y) / (L_x \times L_y) = 61.8$ kN/m^2

$q_2 = F_{dz} \times (1 - 6 \times e_x / L_x + 6 \times e_y / L_y) / (L_x \times L_y) = 61.8$ kN/m^2

$q_3 = F_{dz} \times (1 + 6 \times e_x / L_x - 6 \times e_y / L_y) / (L_x \times L_y) = 61.8$ kN/m^2

$q_4 = F_{dz} \times (1 + 6 \times e_x / L_x + 6 \times e_y / L_y) / (L_x \times L_y) = 61.8$ kN/m^2

Minimum base pressure; $q_{min} = \min(q_1, q_2, q_3, q_4) = 61.8$ kN/m^2
Maximum base pressure; $q_{max} = \max(q_1, q_2, q_3, q_4) = 61.8$ kN/m^2
Presumed bearing capacity
Presumed bearing capacity; $P_{bearing} = 75.0$ kN/m^2
PASS - Presumed bearing capacity exceeds design base pressure
Design approach 1
Partial factors on actions - Combination1
Partial factor set; A1
Permanent unfavourable action - Table A.3; $\gamma_G = 1.35$
Permanent favourable action - Table A.3; $\gamma_{Gf} = 1.00$
Variable unfavourable action - Table A.3; $\gamma_Q = 1.50$
Variable favourable action - Table A.3; $\gamma_{Qf} = 0.00$
Partial factors for spread foundations - Combination1
Resistance factor set; R1
Bearing - Table A.5; $\gamma_{R.v} = 1.00$
Sliding - Table A.5; $\gamma_{R.h} = 1.00$
Forces on foundation
Force in z-axis; $F_{dz} = \gamma_G \times (A \times F_{swt} + F_{Gz1}) + \gamma_Q \times F_{Qz1} = 69.6$ kN
Moments on foundation
Moment in x-axis; $M_{dx} = \gamma_G \times (A \times F_{swt} \times L_x / 2 + F_{Gz1} \times x_1) + \gamma_Q \times F_{Qz1} \times x_1 = 31.3$ kNm
Moment in y-axis; $M_{dy} = \gamma_G \times (A \times F_{swt} \times L_y / 2 + F_{Gz1} \times y_1) + \gamma_Q \times F_{Qz1} \times y_1 = 31.3$ kNm
Eccentricity of base reaction
Eccentricity of base reaction in x-axis; $e_x = M_{dx} / F_{dz} - L_x / 2 = 0$ mm
Eccentricity of base reaction in y-axis; $e_y = M_{dy} / F_{dz} - L_y / 2 = 0$ mm
Effective area of base

Effective length; $L'_x = L_x - 2 \times e_x = 900$ mm
Effective width; $L'_y = L_y - 2 \times e_y = 900$ mm
Effective area; $A' = L'_x \times L'_y = 0.810$ m^2
Pad base pressure
Design base pressure; $f_{dz} = F_{dz} / A' = 85.9$ kN/m^2
Design approach 1
Partial factors on actions - Combination2
Partial factor set; A2
Permanent unfavourable action - Table A.3; $\gamma_G = 1.00$
Permanent favourable action - Table A.3; $\gamma_{Gf} = 1.00$
Variable unfavourable action - Table A.3; $\gamma_Q = 1.30$
Variable favourable action - Table A.3; $\gamma_{Qf} = 0.00$
Partial factors for spread foundations - Combination2
Resistance factor set; R1
Bearing - Table A.5; $\gamma_{R.v} = 1.00$
Sliding - Table A.5; $\gamma_{R.h} = 1.00$
Forces on foundation
Force in z-axis; $F_{dz} = \gamma_G \times (A \times F_{swt} + F_{Gz1}) + \gamma_Q \times F_{Qz1} = 54.0$ kN
Moments on foundation
Moment in x-axis; $M_{dx} = \gamma_G \times (A \times F_{swt} \times L_x / 2 + F_{Gz1} \times x_1) + \gamma_Q \times F_{Qz1} \times x_1 = 24.3$ kNm
Moment in y-axis; $M_{dy} = \gamma_G \times (A \times F_{swt} \times L_y / 2 + F_{Gz1} \times y_1) + \gamma_Q \times F_{Qz1} \times y_1 = 24.3$ kNm
Eccentricity of base reaction
Eccentricity of base reaction in x-axis; $e_x = M_{dx} / F_{dz} - L_x / 2 = 0$ mm
Eccentricity of base reaction in y-axis; $e_y = M_{dy} / F_{dz} - L_y / 2 = 0$ mm
Effective area of base
Effective length; $L'_x = L_x - 2 \times e_x = 900$ mm
Effective width; $L'_y = L_y - 2 \times e_y = 900$ mm
Effective area; $A' = L'_x \times L'_y = 0.810$ m^2
Pad base pressure
Design base pressure; $f_{dz} = F_{dz} / A' = 66.7$ kN/m^2
Foundation design (EN1992-1-1:2004)
In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1
TEDDS calculation version 3.2.15
Concrete details (Table 3.1 - Strength and deformation characteristics for concrete)
Concrete strength class;C30/37
Characteristic compressive cylinder strength; $f_{ck} = 30$ N/mm^2
Characteristic compressive cube strength; $f_{ck,cube} = 37$ N/mm^2
Mean value of compressive cylinder strength; $f_{cm} = f_{ck} + 8$ N/mm^2 = 38 N/mm^2
Mean value of axial tensile strength; $f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck}/ 1$ N/mm$^2)^{2/3}$ = 2.9 N/mm^2
5% fractile of axial tensile strength; $f_{ctk,0.05} = 0.7 \times f_{ctm} = 2.0$ N/mm^2
Secant modulus of elasticity of concrete; $E_{cm} = 22$ kN/mm$^2 \times [f_{cm}/10$ N/mm$^2]^{0.3}$ = 32837 N/mm^2
Partial factor for concrete (Table 2.1N); $\gamma_C = 1.50$
Compressive strength coefficient (cl.3.1.6(1)); $\alpha_{cc} = 0.85$
Design compressive concrete strength (exp.3.15); $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = 17.0$ N/mm^2
Tens.strength coeff.for plain concrete (cl.12.3.1(1)); $\alpha_{ct,pl} = 0.80$

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Des.tens.strength for plain concrete (exp.12.1); $f_{ctd,pl} = \alpha_{ct,pl} \times f_{ctk,0.05} / \gamma_C$ = 1.1 N/mm^2
Maximum aggregate size; h_{agg} = 20 mm
Ultimate strain - Table 3.1; ε_{cu2} = 0.0035
Shortening strain - Table 3.1; ε_{cu3} = 0.0035
Effective compression zone height factor; λ = 0.80
Effective strength factor; η = 1.00
Bending coefficient k_1; K_1 = 0.40
Bending coefficient k_2; $K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ =1.00
Bending coefficient k_3; K_3 =0.40
Bending coefficient k_4; $K_4 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ = 1.00
Reinforcement details
Characteristic yield strength of reinforcement; f_{yk} = 500 N/mm^2
Modulus of elasticity of reinforcement; E_s = 210000 N/mm^2
Partial factor for reinforcing steel (Table 2.1N); γ_S = 1.15
Design yield strength of reinforcement; $f_{yd} = f_{yk} / \gamma_S$ = 435 N/mm^2
Nominal cover to reinforcement; c_{nom} = 30 mm
Strip and pad footings (Section 12.9.3)
Design base pressure; f_{dz} = 85.9 kN/m^2
Projection from column face; a = 400 mm
Max.projection from column face - (exp.12.13); $a_{max} = 0.85 \times h / \sqrt{[3 \times f_{dz} / f_{ctd,pl}]}$ = 609 mm
PASS - Projection from the column face doesn't exceed permissible limit for plain concrete

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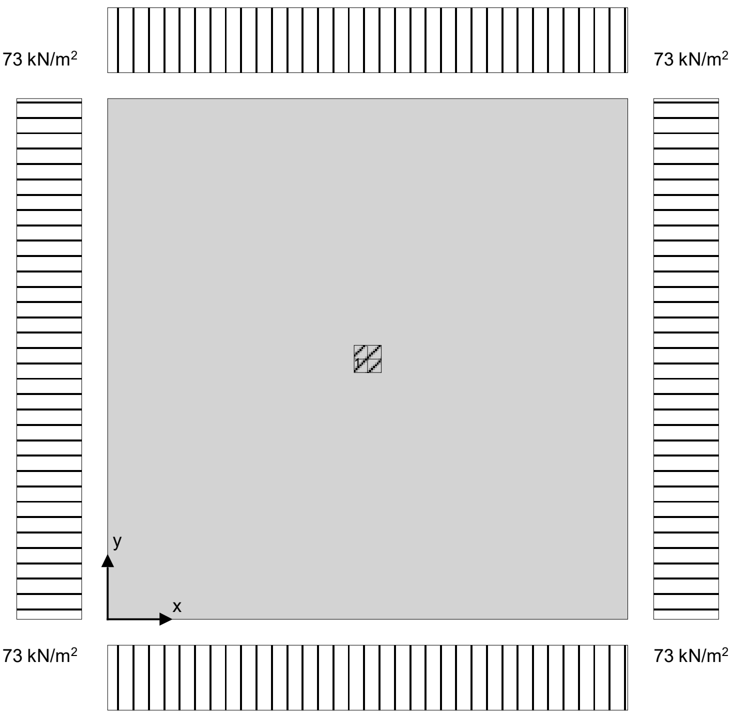
P2 pad

Foundation analysis (EN1997-1:2004)
In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1
TEDDS calculation version 3.2.15
Pad foundation details
Length of foundation; L_x = 1900 mm
Width of foundation; L_y = 1900 mm
Foundation area; $A = L_x \times L_y = 3.610\ m^2$
Depth of foundation; h = 350 mm
Depth of soil over foundation; h_{soil} = 0 mm
Level of water; h_{water} = 0 mm
Density of water; $\gamma_{water} = 9.8\ kN/m^3$
Density of concrete; $\gamma_{conc} = 25.0\ kN/m^3$



Column no.1 details
Length of column; l_{x1} = 100 mm
Width of column; l_{y1} = 100 mm
position in x-axis; x_1 = 950 mm
position in y-axis; y_1 = 950 mm
Soil properties
Density of soil; $\gamma_{soil} = 18.0\ kN/m^3$
Characteristic cohesion; $c'_k = 0\ kN/m^2$
Characteristic effective shear resistance angle; ϕ'_k = 30 deg
Characteristic friction angle; δ_k = 20 deg
Foundation loads

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Self weight; $F_{swt} = h \times \gamma_{conc} = 8.8$ kN/m^2
Column no.1 loads
Permanent load in z; $F_{Gz1} = 170.0$ kN
Variable load in z; $F_{Qz1} = 62.0$ kN
Bearing resistance (Section 6.5.2)
Forces on foundation
Force in z-axis; $F_{dz} = A \times F_{swt} + F_{Gz1} + F_{Qz1} = 263.6$ kN
Moments on foundation
Moment in x-axis; $M_{dx} = A \times F_{swt} \times L_x / 2 + F_{Gz1} \times x_1 + F_{Qz1} \times x_1 = 250.4$ kNm
Moment in y-axis; $M_{dy} = A \times F_{swt} \times L_y / 2 + F_{Gz1} \times y_1 + F_{Qz1} \times y_1 = 250.4$ kNm
Eccentricity of base reaction
Eccentricity of base reaction in x-axis; $e_x = M_{dx} / F_{dz} - L_x / 2 = 0$ mm
Eccentricity of base reaction in y-axis; $e_y = M_{dy} / F_{dz} - L_y / 2 = 0$ mm
Pad base pressures
$q_1 = F_{dz} \times (1 - 6 \times e_x / L_x - 6 \times e_y / L_y) / (L_x \times L_y) = 73$ kN/m^2
$q_2 = F_{dz} \times (1 - 6 \times e_x / L_x + 6 \times e_y / L_y) / (L_x \times L_y) = 73$ kN/m^2
$q_3 = F_{dz} \times (1 + 6 \times e_x / L_x - 6 \times e_y / L_y) / (L_x \times L_y) = 73$ kN/m^2
$q_4 = F_{dz} \times (1 + 6 \times e_x / L_x + 6 \times e_y / L_y) / (L_x \times L_y) = 73$ kN/m^2
Minimum base pressure; $q_{min} = \min(q_1, q_2, q_3, q_4) = 73$ kN/m^2
Maximum base pressure; $q_{max} = \max(q_1, q_2, q_3, q_4) = 73$ kN/m^2
Presumed bearing capacity
Presumed bearing capacity; $P_{bearing} = 75.0$ kN/m^2
PASS - Presumed bearing capacity exceeds design base pressure
Design approach 1
Partial factors on actions - Combination1
Partial factor set; A1
Permanent unfavourable action - Table A.3; $\gamma_G = 1.35$
Permanent favourable action - Table A.3; $\gamma_{Gf} = 1.00$
Variable unfavourable action - Table A.3; $\gamma_Q = 1.50$
Variable favourable action - Table A.3; $\gamma_{Qf} = 0.00$
Partial factors for spread foundations - Combination1
Resistance factor set; R1
Bearing - Table A.5; $\gamma_{R.v} = 1.00$
Sliding - Table A.5; $\gamma_{R.h} = 1.00$
Forces on foundation
Force in z-axis; $F_{dz} = \gamma_G \times (A \times F_{swt} + F_{Gz1}) + \gamma_Q \times F_{Qz1} = 365.1$ kN
Moments on foundation
Moment in x-axis; $M_{dx} = \gamma_G \times (A \times F_{swt} \times L_x / 2 + F_{Gz1} \times x_1) + \gamma_Q \times F_{Qz1} \times x_1 = 346.9$ kNm
Moment in y-axis; $M_{dy} = \gamma_G \times (A \times F_{swt} \times L_y / 2 + F_{Gz1} \times y_1) + \gamma_Q \times F_{Qz1} \times y_1 = 346.9$ kNm
Eccentricity of base reaction
Eccentricity of base reaction in x-axis; $e_x = M_{dx} / F_{dz} - L_x / 2 = 0$ mm
Eccentricity of base reaction in y-axis; $e_y = M_{dy} / F_{dz} - L_y / 2 = 0$ mm
Effective area of base
Effective length; $L'_x = L_x - 2 \times e_x = 1900$ mm
Effective width; $L'_y = L_y - 2 \times e_y = 1900$ mm
Effective area; $A' = L'_x \times L'_y = 3.610$ m^2

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Pad base pressure
Design base pressure; $f_{dz} = F_{dz} / A' = 101.1$ kN/m^2
Design approach 1
Partial factors on actions - Combination2
Partial factor set; A2
Permanent unfavourable action - Table A.3; $\gamma_G = 1.00$
Permanent favourable action - Table A.3; $\gamma_{Gf} = 1.00$
Variable unfavourable action - Table A.3; $\gamma_Q = 1.30$
Variable favourable action - Table A.3; $\gamma_{Qf} = 0.00$
Partial factors for spread foundations - Combination2
Resistance factor set; R1
Bearing - Table A.5; $\gamma_{R.v} = 1.00$
Sliding - Table A.5; $\gamma_{R.h} = 1.00$
Forces on foundation
Force in z-axis; $F_{dz} = \gamma_G \times (A \times F_{swt} + F_{Gz1}) + \gamma_Q \times F_{Qz1} = 282.2$ kN
Moments on foundation
Moment in x-axis; $M_{dx} = \gamma_G \times (A \times F_{swt} \times L_x / 2 + F_{Gz1} \times x_1) + \gamma_Q \times F_{Qz1} \times x_1 = 268.1$ kNm
Moment in y-axis; $M_{dy} = \gamma_G \times (A \times F_{swt} \times L_y / 2 + F_{Gz1} \times y_1) + \gamma_Q \times F_{Qz1} \times y_1 = 268.1$ kNm
Eccentricity of base reaction
Eccentricity of base reaction in x-axis; $e_x = M_{dx} / F_{dz} - L_x / 2 = 0$ mm
Eccentricity of base reaction in y-axis; $e_y = M_{dy} / F_{dz} - L_y / 2 = 0$ mm
Effective area of base
Effective length; $L'_x = L_x - 2 \times e_x = 1900$ mm
Effective width; $L'_y = L_y - 2 \times e_y = 1900$ mm
Effective area; $A' = L'_x \times L'_y = 3.610$ m^2
Pad base pressure
Design base pressure; $f_{dz} = F_{dz} / A' = 78.2$ kN/m^2
Foundation design (EN1992-1-1:2004)
In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1
TEDDS calculation version 3.2.15
Concrete details (Table 3.1 - Strength and deformation characteristics for concrete)
Concrete strength class;C30/37
Characteristic compressive cylinder strength; $f_{ck} = 30$ N/mm^2
Characteristic compressive cube strength; $f_{ck,cube} = 37$ N/mm^2
Mean value of compressive cylinder strength; $f_{cm} = f_{ck} + 8$ N/mm^2 = 38 N/mm^2
Mean value of axial tensile strength; $f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck}/ 1$ N/mm$^2)^{2/3} = 2.9$ N/mm^2
5% fractile of axial tensile strength; $f_{ctk,0.05} = 0.7 \times f_{ctm} = 2.0$ N/mm^2
Secant modulus of elasticity of concrete; $E_{cm} = 22$ kN/mm$^2 \times [f_{cm}/10$ N/mm$^2]^{0.3} = 32837$ N/mm^2
Partial factor for concrete (Table 2.1N); $\gamma_C = 1.50$
Compressive strength coefficient (cl.3.1.6(1)); $\alpha_{cc} = 0.85$
Design compressive concrete strength (exp.3.15); $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = 17.0$ N/mm^2
Tens.strength coeff.for plain concrete (cl.12.3.1(1)); $\alpha_{ct,pl} = 0.80$
Des.tens.strength for plain concrete (exp.12.1); $f_{ctd,pl} = \alpha_{ct,pl} \times f_{ctk,0.05} / \gamma_C = 1.1$ N/mm^2
Maximum aggregate size; $h_{agg} = 20$ mm
Ultimate strain - Table 3.1; $\varepsilon_{cu2} = 0.0035$

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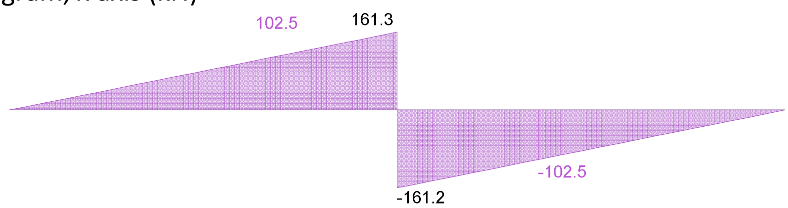
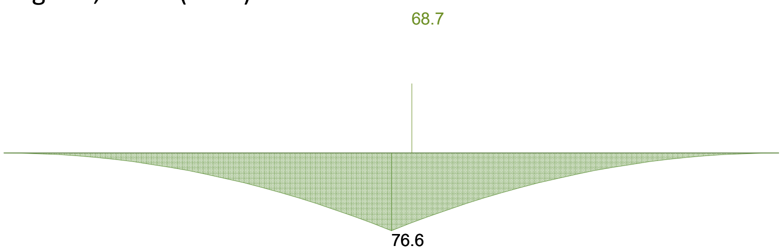
Shortening strain - Table 3.1; $\varepsilon_{cu3} = 0.0035$
Effective compression zone height factor; $\lambda = 0.80$
Effective strength factor; $\eta = 1.00$
Bending coefficient k_1; $K_1 = 0.40$
Bending coefficient k_2; $K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2}) = 1.00$
Bending coefficient k_3; $K_3 = 0.40$
Bending coefficient k_4; $K_4 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2}) = 1.00$
Reinforcement details
Characteristic yield strength of reinforcement; $f_{yk} = 500$ N/mm^2
Modulus of elasticity of reinforcement; $E_s = 210000$ N/mm^2
Partial factor for reinforcing steel (Table 2.1N); $\gamma_S = 1.15$
Design yield strength of reinforcement; $f_{yd} = f_{yk} / \gamma_S = 435$ N/mm^2
Nominal cover to reinforcement; $c_{nom} = 30$ mm
Shear diagram, x axis (kN)



Moment diagram, x axis (kNm)



Rectangular section in flexure (Section 6.1)
Design bending moment; $M_{Ed.x.max} = 68.7$ kNm
Depth to tension reinforcement; $d = h - c_{nom} - \phi_{x.bot} / 2 = 312$ mm

$K = M_{Ed.x.max} / (L_y \times d^2 \times f_{ck}) = 0.012$

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

$K' = 0.207$

K' > K - No compression reinforcement is required
Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = 296$ mm
Depth of neutral axis; $x = 2.5 \times (d - z) = 39$ mm
Area of tension reinforcement required; $A_{sx.bot.req} = M_{Ed.x.max} / (f_{yd} \times z) = 533$ mm^2
Tension reinforcement provided; 8 No.16 dia.bars bottom (260 c/c)
Area of tension reinforcement provided; $A_{sx.bot.prov} = 1608$ mm^2
Minimum area of reinforcement (exp.9.1N); $A_{s.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times L_y \times d = 893$ mm^2
Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s.max} = 0.04 \times L_y \times d = 23712$ mm^2
PASS - Area of reinforcement provided is greater than area of reinforcement required
Crack control (Section 7.3)

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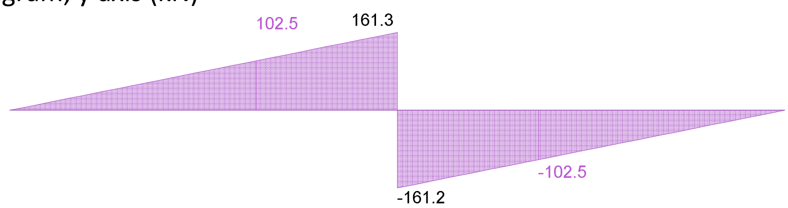
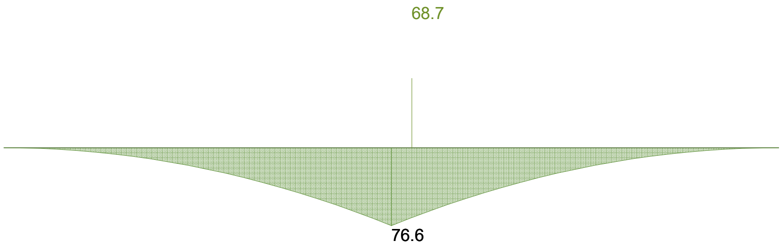
Limiting crack width; w_{max} = 0.3 mm
Variable load factor (EN1990 – Table A1.1); ψ_2 = 0.3
Serviceability bending moment; $M_{sls.x.max}$ = 40.2 kNm
Tensile stress in reinforcement; $\sigma_s = M_{sls.x.max} / (A_{sx.bot.prov} \times z)$ = 84.3 N/mm^2
Load duration factor; k_t = 0.4
Effective depth of concrete in tension; $h_{c.ef} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = 95 mm
Effective area of concrete in tension; $A_{c.eff} = h_{c.ef} \times L_y$ = 180500 mm^2
Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm}$ = 2.9 N/mm^2
Reinforcement ratio; $\rho_{p.eff} = A_{sx.bot.prov} / A_{c.eff}$ = 0.009
Modular ratio; $\alpha_e = E_s / E_{cm}$ = 6.395
Bond property coefficient; k_1 = 0.8
Strain distribution coefficient; k_2 = 0.5
k_3 = 3.4 =3.4
k_4 = 0.425
Maximum crack spacing (exp.7.11); $s_{r.max} = k_3 \times c_{nom} + k_1 \times k_2 \times k_4 \times \phi_{x.bot} / \rho_{p.eff}$ = 407 mm
Maximum crack width (exp.7.8); $w_k = s_{r.max} \times \max([\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff})] / E_s, 0.6 \times \sigma_s / E_s)$ = 0.098 mm
PASS - Maximum crack width is less than limiting crack width
Library item: Crack width output
Rectangular section in shear (Section 6.2)
Design shear force; $V_{Ed.x.max}$ = 102.5 kN
$C_{Rd,c} = 0.18 / \gamma_C$ = 0.120
$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = 1.822
Longitudinal reinforcement ratio; $\rho_l = \min(A_{sx.bot.prov} / (L_y \times d), 0.02)$ = 0.003
$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = 0.471 N/mm^2
Design shear resistance (exp.6.2a & 6.2b); $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times L_y \times d$
$V_{Rd.c}$ = 265.2 kN
PASS - Design shear resistance exceeds design shear force
Shear diagram, y axis (kN)



Moment diagram, y axis (kNm)



Rectangular section in flexure (Section 6.1)
Design bending moment; $M_{Ed.y.max}$ = 68.7 kNm

Depth to tension reinforcement; $d = h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2 = 296$ mm

$K = M_{Ed.y.max} / (L_x \times d^2 \times f_{ck}) = 0.014$

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

$K' = 0.207$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = 281$ mm

Depth of neutral axis; $x = 2.5 \times (d - z) = 37$ mm

Area of tension reinforcement required; $A_{sy.bot.req} = M_{Ed.y.max} / (f_{yd} \times z) = 562\ mm^2$

Tension reinforcement provided; 8 No.16 dia.bars bottom (260 c/c)

Area of tension reinforcement provided; $A_{sy.bot.prov} = 1608\ mm^2$

Minimum area of reinforcement (exp.9.1N); $A_{s.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times L_x \times d = 847\ mm^2$

Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s.max} = 0.04 \times L_x \times d = 22496\ mm^2$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control (Section 7.3)

Limiting crack width; $w_{max} = 0.3$ mm

Variable load factor (EN1990 – Table A1.1); $\psi_2 = 0.3$

Serviceability bending moment; $M_{sls.y.max} = 40.2$ kNm

Tensile stress in reinforcement; $\sigma_s = M_{sls.y.max} / (A_{sy.bot.prov} \times z) = 88.9\ N/mm^2$

Load duration factor; $k_t = 0.4$

Effective depth of concrete in tension; $h_{c.ef} = \min(2.5 \times (h - d), (h - x) / 3, h / 2) = 104$ mm

Effective area of concrete in tension; $A_{c.eff} = h_{c.ef} \times L_x = 198233\ mm^2$

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} = 2.9\ N/mm^2$

Reinforcement ratio; $\rho_{p.eff} = A_{sy.bot.prov} / A_{c.eff} = 0.008$

Modular ratio; $\alpha_e = E_s / E_{cm} = 6.395$

Bond property coefficient; $k_1 = 0.8$

Strain distribution coefficient; $k_2 = 0.5$

$k_3 = 3.4 = 3.4$

$k_4 = 0.425$

Maximum crack spacing (exp.7.11); $s_{r.max} = k_3 \times (c_{nom} + \phi_{x.bot}) + k_1 \times k_2 \times k_4 \times \phi_{y.bot} / \rho_{p.eff} = 492$ mm

Maximum crack width (exp.7.8); $w_k = s_{r.max} \times \max([\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff})] / E_s, 0.6 \times \sigma_s / E_s) = 0.125$ mm

PASS - Maximum crack width is less than limiting crack width

Library item: Crack width output

Rectangular section in shear (Section 6.2)

Design shear force; $V_{Ed.y.max} = 102.5$ kN

$C_{Rd,c} = 0.18 / \gamma_C = 0.120$

$k = \min(1 + \sqrt{(200\ mm / d)}, 2) = 1.822$

Longitudinal reinforcement ratio; $\rho_l = \min(A_{sy.bot.prov} / (L_x \times d), 0.02) = 0.003$

$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} = 0.471\ N/mm^2$

Design shear resistance (exp.6.2a & 6.2b); $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times L_x \times d$

$V_{Rd.c} = 265.2$ kN

PASS - Design shear resistance exceeds design shear force

Punching shear (Section 6.4)

Strength reduction factor (exp 6.6N); $v = 0.6 \times [1 - f_{ck} / 250\ N/mm^2] = 0.528$

Average depth to reinforcement; $d = 304$ mm

Maximum punching shear resistance (cl.6.4.5(3)); $v_{Rd.max} = 0.5 \times v \times f_{cd} = 4.488$ N/mm^2

$k = \min(1 + \sqrt{(200\text{ mm} / d)}, 2) = 1.811$

Longitudinal reinforcement ratio (cl.6.4.4(1)); $\rho_{lx} = A_{sx.bot.prov} / (L_y \times d) = 0.003$

$\rho_{ly} = A_{sy.bot.prov} / (L_x \times d) = 0.003$

$\rho_l = \min(\sqrt{(\rho_{lx} \times \rho_{ly})}, 0.02) = 0.003$

$C_{Rd,c} = 0.18 / \gamma_C = 0.120$

$v_{min} = 0.035\text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = 0.467$ N/mm^2

Design punching shear resistance (exp.6.47); $v_{Rd.c} = \max(C_{Rd.c} \times k \times (100\text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min})$ = 0.467 N/mm^2

Design punching shear resistance at 1d (exp. 6.50); $v_{Rd.c1} = (2 \times d / d) \times v_{Rd.c} = 0.934$ N/mm^2

Column No.1 - Punching shear perimeter at column face

Punching shear perimeter; $u_0 = 400$ mm

Area within punching shear perimeter; $A_0 = 0.010$ m^2

Maximum punching shear force; $V_{Ed.max} = 321.6$ kN

Punching shear stress factor (fig 6.21N); $\beta = 1.000$

Maximum punching shear stress (exp 6.38); $v_{Ed.max} = \beta \times V_{Ed.max} / (u_0 \times d) = 2.645$ N/mm^2

PASS - Maximum punching shear resistance exceeds maximum punching shear stress

Column No.1 - Punching shear perimeter at 1d from column face

Punching shear perimeter; $u_1 = 2310$ mm

Area within punching shear perimeter; $A_1 = 0.422$ m^2

Design punching shear force; $V_{Ed.1} = 284.8$ kN

Punching shear stress factor (fig 6.21N); $\beta = 1.000$

Design punching shear stress (exp 6.38); $v_{Ed.1} = \beta \times V_{Ed.1} / (u_1 \times d) = 0.406$ N/mm^2

PASS - Design punching shear resistance exceeds increased design punching shear stress

Column No.1 - Punching shear perimeter at 2d from column face

Punching shear perimeter; $u_2 = 4220$ mm

Area within punching shear perimeter; $A_2 = 1.415$ m^2

Design punching shear force; $V_{Ed.2} = 196.1$ kN

Punching shear stress factor (fig 6.21N); $\beta = 1.000$

Design punching shear stress (exp 6.38); $v_{Ed.2} = \beta \times V_{Ed.2} / (u_2 \times d) = 0.153$ N/mm^2

PASS - Design punching shear resistance exceeds design punching shear stress

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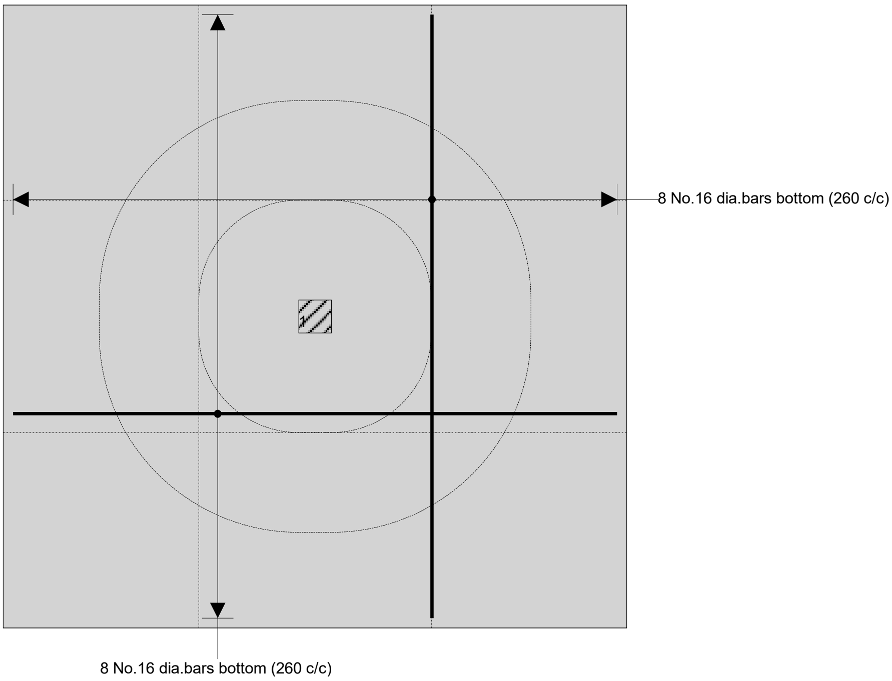

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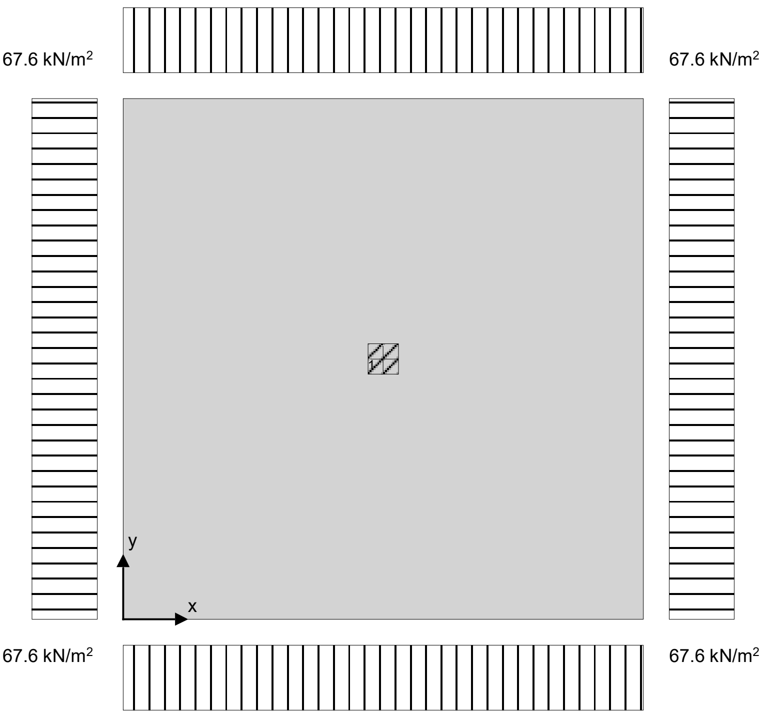
P3 pad

Foundation analysis (EN1997-1:2004)
In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1
TEDDS calculation version 3.2.15
Pad foundation details
Length of foundation; L_x = 1700 mm
Width of foundation; L_y = 1700 mm
Foundation area; $A = L_x \times L_y$ = 2.890 m^2
Depth of foundation; h = 350 mm
Depth of soil over foundation; h_{soil} = 0 mm
Level of water; h_{water} = 0 mm
Density of water; γ_{water} = 9.8 kN/m^3
Density of concrete; γ_{conc} = 25.0 kN/m^3



Column no.1 details
Length of column; l_{x1} = 100 mm
Width of column; l_{y1} = 100 mm
position in x-axis; x_1 = 850 mm
position in y-axis; y_1 = 850 mm
Soil properties
Density of soil; γ_{soil} = 18.0 kN/m^3
Characteristic cohesion; c'_k = 0 kN/m^2
Characteristic effective shear resistance angle; ϕ'_k = 30 deg
Characteristic friction angle; δ_k = 20 deg
Foundation loads

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Self weight; $F_{swt} = h \times \gamma_{conc} = 8.8$ kN/m^2
Column no.1 loads
Permanent load in z; $F_{Gz1} = 120.0$ kN
Variable load in z; $F_{Qz1} = 50.0$ kN
Bearing resistance (Section 6.5.2)
Forces on foundation
Force in z-axis; $F_{dz} = A \times F_{swt} + F_{Gz1} + F_{Qz1} = 195.3$ kN
Moments on foundation
Moment in x-axis; $M_{dx} = A \times F_{swt} \times L_x / 2 + F_{Gz1} \times x_1 + F_{Qz1} \times x_1 = 166.0$ kNm
Moment in y-axis; $M_{dy} = A \times F_{swt} \times L_y / 2 + F_{Gz1} \times y_1 + F_{Qz1} \times y_1 = 166.0$ kNm
Eccentricity of base reaction
Eccentricity of base reaction in x-axis; $e_x = M_{dx} / F_{dz} - L_x / 2 = 0$ mm
Eccentricity of base reaction in y-axis; $e_y = M_{dy} / F_{dz} - L_y / 2 = 0$ mm
Pad base pressures
$q_1 = F_{dz} \times (1 - 6 \times e_x / L_x - 6 \times e_y / L_y) / (L_x \times L_y) = 67.6$ kN/m^2
$q_2 = F_{dz} \times (1 - 6 \times e_x / L_x + 6 \times e_y / L_y) / (L_x \times L_y) = 67.6$ kN/m^2
$q_3 = F_{dz} \times (1 + 6 \times e_x / L_x - 6 \times e_y / L_y) / (L_x \times L_y) = 67.6$ kN/m^2
$q_4 = F_{dz} \times (1 + 6 \times e_x / L_x + 6 \times e_y / L_y) / (L_x \times L_y) = 67.6$ kN/m^2
Minimum base pressure; $q_{min} = \min(q_1, q_2, q_3, q_4) = 67.6$ kN/m^2
Maximum base pressure; $q_{max} = \max(q_1, q_2, q_3, q_4) = 67.6$ kN/m^2
Presumed bearing capacity
Presumed bearing capacity; $P_{bearing} = 75.0$ kN/m^2
PASS - Presumed bearing capacity exceeds design base pressure
Design approach 1
Partial factors on actions - Combination1
Partial factor set; A1
Permanent unfavourable action - Table A.3; $\gamma_G = 1.35$
Permanent favourable action - Table A.3; $\gamma_{Gf} = 1.00$
Variable unfavourable action - Table A.3; $\gamma_Q = 1.50$
Variable favourable action - Table A.3; $\gamma_{Qf} = 0.00$
Partial factors for spread foundations - Combination1
Resistance factor set; R1
Bearing - Table A.5; $\gamma_{R.v} = 1.00$
Sliding - Table A.5; $\gamma_{R.h} = 1.00$
Forces on foundation
Force in z-axis; $F_{dz} = \gamma_G \times (A \times F_{swt} + F_{Gz1}) + \gamma_Q \times F_{Qz1} = 271.1$ kN
Moments on foundation
Moment in x-axis; $M_{dx} = \gamma_G \times (A \times F_{swt} \times L_x / 2 + F_{Gz1} \times x_1) + \gamma_Q \times F_{Qz1} \times x_1 = 230.5$ kNm
Moment in y-axis; $M_{dy} = \gamma_G \times (A \times F_{swt} \times L_y / 2 + F_{Gz1} \times y_1) + \gamma_Q \times F_{Qz1} \times y_1 = 230.5$ kNm
Eccentricity of base reaction
Eccentricity of base reaction in x-axis; $e_x = M_{dx} / F_{dz} - L_x / 2 = 0$ mm
Eccentricity of base reaction in y-axis; $e_y = M_{dy} / F_{dz} - L_y / 2 = 0$ mm
Effective area of base
Effective length; $L'_x = L_x - 2 \times e_x = 1700$ mm
Effective width; $L'_y = L_y - 2 \times e_y = 1700$ mm
Effective area; $A' = L'_x \times L'_y = 2.890$ m^2

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Pad base pressure
Design base pressure; $f_{dz} = F_{dz} / A' = 93.8$ kN/m^2
Design approach 1
Partial factors on actions - Combination2
Partial factor set; A2
Permanent unfavourable action - Table A.3; $\gamma_G = 1.00$
Permanent favourable action - Table A.3; $\gamma_{Gf} = 1.00$
Variable unfavourable action - Table A.3; $\gamma_Q = 1.30$
Variable favourable action - Table A.3; $\gamma_{Qf} = 0.00$
Partial factors for spread foundations - Combination2
Resistance factor set; R1
Bearing - Table A.5; $\gamma_{R.v} = 1.00$
Sliding - Table A.5; $\gamma_{R.h} = 1.00$
Forces on foundation
Force in z-axis; $F_{dz} = \gamma_G \times (A \times F_{swt} + F_{Gz1}) + \gamma_Q \times F_{Qz1} = 210.3$ kN
Moments on foundation
Moment in x-axis; $M_{dx} = \gamma_G \times (A \times F_{swt} \times L_x / 2 + F_{Gz1} \times x_1) + \gamma_Q \times F_{Qz1} \times x_1 = 178.7$ kNm
Moment in y-axis; $M_{dy} = \gamma_G \times (A \times F_{swt} \times L_y / 2 + F_{Gz1} \times y_1) + \gamma_Q \times F_{Qz1} \times y_1 = 178.7$ kNm
Eccentricity of base reaction
Eccentricity of base reaction in x-axis; $e_x = M_{dx} / F_{dz} - L_x / 2 = 0$ mm
Eccentricity of base reaction in y-axis; $e_y = M_{dy} / F_{dz} - L_y / 2 = 0$ mm
Effective area of base
Effective length; $L'_x = L_x - 2 \times e_x = 1700$ mm
Effective width; $L'_y = L_y - 2 \times e_y = 1700$ mm
Effective area; $A' = L'_x \times L'_y = 2.890$ m^2
Pad base pressure
Design base pressure; $f_{dz} = F_{dz} / A' = 72.8$ kN/m^2
Foundation design (EN1992-1-1:2004)
In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1
TEDDS calculation version 3.2.15
Concrete details (Table 3.1 - Strength and deformation characteristics for concrete)
Concrete strength class;C30/37
Characteristic compressive cylinder strength; $f_{ck} = 30$ N/mm^2
Characteristic compressive cube strength; $f_{ck,cube} = 37$ N/mm^2
Mean value of compressive cylinder strength; $f_{cm} = f_{ck} + 8$ N/mm^2 = 38 N/mm^2
Mean value of axial tensile strength; $f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck}/ 1$ N/mm$^2)^{2/3}$ = 2.9 N/mm^2
5% fractile of axial tensile strength; $f_{ctk,0.05} = 0.7 \times f_{ctm} = 2.0$ N/mm^2
Secant modulus of elasticity of concrete; $E_{cm} = 22$ kN/mm$^2 \times [f_{cm}/10$ N/mm$^2]^{0.3}$ = 32837 N/mm^2
Partial factor for concrete (Table 2.1N); $\gamma_C = 1.50$
Compressive strength coefficient (cl.3.1.6(1)); $\alpha_{cc} = 0.85$
Design compressive concrete strength (exp.3.15); $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = 17.0$ N/mm^2
Tens.strength coeff.for plain concrete (cl.12.3.1(1)); $\alpha_{ct,pl} = 0.80$
Des.tens.strength for plain concrete (exp.12.1); $f_{ctd,pl} = \alpha_{ct,pl} \times f_{ctk,0.05} / \gamma_C = 1.1$ N/mm^2
Maximum aggregate size; $h_{agg} = 20$ mm
Ultimate strain - Table 3.1; $\varepsilon_{cu2} = 0.0035$

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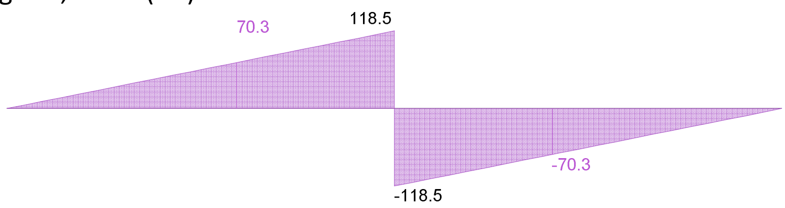
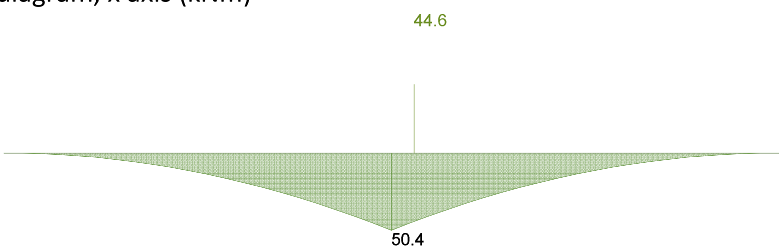
Shortening strain - Table 3.1; ε_{cu3} = 0.0035
Effective compression zone height factor; λ = 0.80
Effective strength factor; η = 1.00
Bending coefficient k_1; K_1 = 0.40
Bending coefficient k_2; $K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2}) = 1.00$
Bending coefficient k_3; K_3 =0.40
Bending coefficient k_4; $K_4 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2}) = 1.00$
Reinforcement details
Characteristic yield strength of reinforcement; f_{yk} = 500 N/mm^2
Modulus of elasticity of reinforcement; E_s = 210000 N/mm^2
Partial factor for reinforcing steel (Table 2.1N); γ_S = 1.15
Design yield strength of reinforcement; $f_{yd} = f_{yk} / \gamma_S$ = 435 N/mm^2
Nominal cover to reinforcement; c_{nom} = 30 mm
Shear diagram, x axis (kN)



Moment diagram, x axis (kNm)



Rectangular section in flexure (Section 6.1)
Design bending moment; $M_{Ed.x.max}$ = 44.6 kNm
Depth to tension reinforcement; $d = h - c_{nom} - \phi_{x.bot} / 2$ = 312 mm

$K = M_{Ed.x.max} / (L_y \times d^2 \times f_{ck}) = 0.009$

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

$K' = 0.207$

K' > K - No compression reinforcement is required
Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = 296 mm
Depth of neutral axis; $x = 2.5 \times (d - z)$ = 39 mm
Area of tension reinforcement required; $A_{sx.bot.req} = M_{Ed.x.max} / (f_{yd} \times z)$ = 346 mm^2
Tension reinforcement provided; 8 No.16 dia.bars bottom (230 c/c)
Area of tension reinforcement provided; $A_{sx.bot.prov}$ = 1608 mm^2
Minimum area of reinforcement (exp.9.1N); $A_{s.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times L_y \times d$ = 799 mm^2
Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s.max} = 0.04 \times L_y \times d$ = 21216 mm^2
PASS - Area of reinforcement provided is greater than area of reinforcement required
Crack control (Section 7.3)

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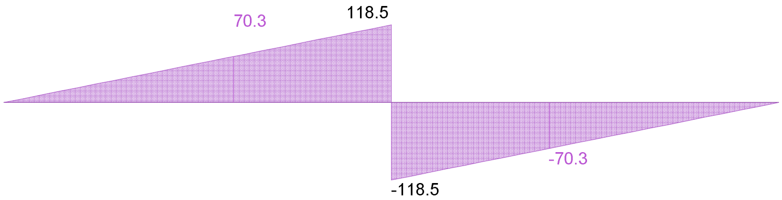
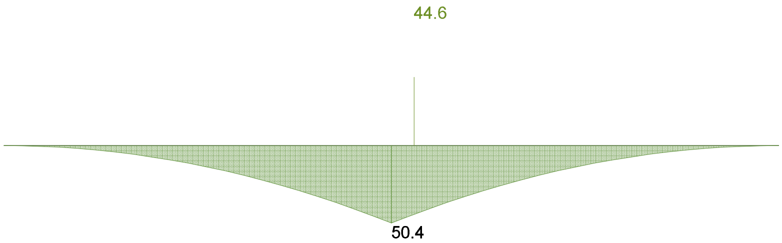
Limiting crack width; w_{max} = 0.3 mm
Variable load factor (EN1990 – Table A1.1); ψ_2 = 0.3
Serviceability bending moment; $M_{sls.x.max}$ = 25.4 kNm
Tensile stress in reinforcement; $\sigma_s = M_{sls.x.max} / (A_{sx.bot.prov} \times z)$ = 53.3 N/mm^2
Load duration factor; k_t = 0.4
Effective depth of concrete in tension; $h_{c.ef}$ = min(2.5 × (h - d), (h - x) / 3, h / 2) = 95 mm
Effective area of concrete in tension; $A_{c.eff} = h_{c.ef} \times L_y$ = 161500 mm^2
Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm}$ = 2.9 N/mm^2
Reinforcement ratio; $\rho_{p.eff} = A_{sx.bot.prov} / A_{c.eff}$ = 0.010
Modular ratio; $\alpha_e = E_s / E_{cm}$ = 6.395
Bond property coefficient; k_1 = 0.8
Strain distribution coefficient; k_2 = 0.5
k_3 = 3.4 =3.4
k_4 = 0.425
Maximum crack spacing (exp.7.11); $s_{r.max} = k_3 \times c_{nom} + k_1 \times k_2 \times k_4 \times \phi_{x.bot} / \rho_{p.eff}$ = 375 mm
Maximum crack width (exp.7.8); $w_k = s_{r.max} \times \max([\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff})] / E_s, 0.6 \times \sigma_s / E_s)$ = 0.057 mm
PASS - Maximum crack width is less than limiting crack width
Library item: Crack width output
Rectangular section in shear (Section 6.2)
Design shear force; $V_{Ed.x.max}$ = 70.3 kN
$C_{Rd,c} = 0.18 / \gamma_C$ = 0.120
k = min(1 + √(200 mm / d), 2) = 1.822
Longitudinal reinforcement ratio; $\rho_l = \min(A_{sx.bot.prov} / (L_y \times d), 0.02)$ = 0.003
v_{min} = 0.035 N$^{1/2}$/mm × $k^{3/2} \times f_{ck}^{0.5}$ = 0.471 N/mm^2
Design shear resistance (exp.6.2a & 6.2b); $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times L_y \times d$
$V_{Rd.c}$ = 237.2 kN
PASS - Design shear resistance exceeds design shear force
Shear diagram, y axis (kN)



Moment diagram, y axis (kNm)



Rectangular section in flexure (Section 6.1)
Design bending moment; $M_{Ed.y.max}$ = 44.6 kNm

Depth to tension reinforcement; $d = h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2 = 296$ mm

$K = M_{Ed.y.max} / (L_x \times d^2 \times f_{ck}) = 0.010$

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

$K' = 0.207$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = 281$ mm

Depth of neutral axis; $x = 2.5 \times (d - z) = 37$ mm

Area of tension reinforcement required; $A_{sy.bot.req} = M_{Ed.y.max} / (f_{yd} \times z) = 365$ mm^2

Tension reinforcement provided; 8 No.16 dia.bars bottom (230 c/c)

Area of tension reinforcement provided; $A_{sy.bot.prov} = 1608$ mm^2

Minimum area of reinforcement (exp.9.1N); $A_{s.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times L_x \times d = 758$ mm^2

Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s.max} = 0.04 \times L_x \times d = 20128$ mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control (Section 7.3)

Limiting crack width; $w_{max} = 0.3$ mm

Variable load factor (EN1990 – Table A1.1); $\psi_2 = 0.3$

Serviceability bending moment; $M_{sls.y.max} = 25.4$ kNm

Tensile stress in reinforcement; $\sigma_s = M_{sls.y.max} / (A_{sy.bot.prov} \times z) = 56.2$ N/mm^2

Load duration factor; $k_t = 0.4$

Effective depth of concrete in tension; $h_{c.ef} = \min(2.5 \times (h - d), (h - x) / 3, h / 2) = 104$ mm

Effective area of concrete in tension; $A_{c.eff} = h_{c.ef} \times L_x = 177367$ mm^2

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} = 2.9$ N/mm^2

Reinforcement ratio; $\rho_{p.eff} = A_{sy.bot.prov} / A_{c.eff} = 0.009$

Modular ratio; $\alpha_e = E_s / E_{cm} = 6.395$

Bond property coefficient; $k_1 = 0.8$

Strain distribution coefficient; $k_2 = 0.5$

$k_3 = 3.4 = 3.4$

$k_4 = 0.425$

Maximum crack spacing (exp.7.11); $s_{r.max} = k_3 \times (c_{nom} + \phi_{x.bot}) + k_1 \times k_2 \times k_4 \times \phi_{y.bot} / \rho_{p.eff} = 456$ mm

Maximum crack width (exp.7.8); $w_k = s_{r.max} \times \max([\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff})] / E_s, 0.6 \times \sigma_s / E_s) = 0.073$ mm

PASS - Maximum crack width is less than limiting crack width

Library item: Crack width output

Rectangular section in shear (Section 6.2)

Design shear force; $V_{Ed.y.max} = 70.3$ kN

$C_{Rd,c} = 0.18 / \gamma_C = 0.120$

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = 1.822$

Longitudinal reinforcement ratio; $\rho_l = \min(A_{sy.bot.prov} / (L_x \times d), 0.02) = 0.003$

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = 0.471$ N/mm^2

Design shear resistance (exp.6.2a & 6.2b); $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times L_x \times d$

$V_{Rd.c} = 237.2$ kN

PASS - Design shear resistance exceeds design shear force

Punching shear (Section 6.4)

Strength reduction factor (exp 6.6N); $v = 0.6 \times [1 - f_{ck} / 250 \text{ N/mm}^2] = 0.528$

Average depth to reinforcement; $d = 304$ mm

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Maximum punching shear resistance (cl.6.4.5(3)); $v_{Rd.max} = 0.5 \times v \times f_{cd} = 4.488$ N/mm^2

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = 1.811$

Longitudinal reinforcement ratio (cl.6.4.4(1)); $\rho_{lx} = A_{sx.bot.prov} / (L_y \times d) = 0.003$

$\rho_{ly} = A_{sy.bot.prov} / (L_x \times d) = 0.003$

$\rho_l = \min(\sqrt{(\rho_{lx} \times \rho_{ly})}, 0.02) = 0.003$

$C_{Rd,c} = 0.18 / \gamma_C = 0.120$

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = 0.467$ N/mm^2

Design punching shear resistance (exp.6.47); $v_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min})$ = 0.467 N/mm^2

Design punching shear resistance at 1d (exp. 6.50); $v_{Rd.c1} = (2 \times d / d) \times v_{Rd.c} = 0.934$ N/mm^2

Column No.1 - Punching shear perimeter at column face

Punching shear perimeter; $u_0 = 400$ mm

Area within punching shear perimeter; $A_0 = 0.010$ m^2

Maximum punching shear force; $V_{Ed.max} = 236.2$ kN

Punching shear stress factor (fig 6.21N); $\beta = 1.000$

Maximum punching shear stress (exp 6.38); $v_{Ed.max} = \beta \times V_{Ed.max} / (u_0 \times d) = 1.942$ N/mm^2

PASS - Maximum punching shear resistance exceeds maximum punching shear stress

Column No.1 - Punching shear perimeter at 1d from column face

Punching shear perimeter; $u_1 = 2310$ mm

Area within punching shear perimeter; $A_1 = 0.422$ m^2

Design punching shear force; $V_{Ed.1} = 202.4$ kN

Punching shear stress factor (fig 6.21N); $\beta = 1.000$

Design punching shear stress (exp 6.38); $v_{Ed.1} = \beta \times V_{Ed.1} / (u_1 \times d) = 0.288$ N/mm^2

PASS - Design punching shear resistance exceeds increased design punching shear stress

Column No.1 - Punching shear perimeter at 2d from column face

Punching shear perimeter; $u_2 = 4220$ mm

Area within punching shear perimeter; $A_2 = 1.415$ m^2

Design punching shear force; $V_{Ed.2} = 121$ kN

Punching shear stress factor (fig 6.21N); $\beta = 1.000$

Design punching shear stress (exp 6.38); $v_{Ed.2} = \beta \times V_{Ed.2} / (u_2 \times d) = 0.094$ N/mm^2

PASS - Design punching shear resistance exceeds design punching shear stress

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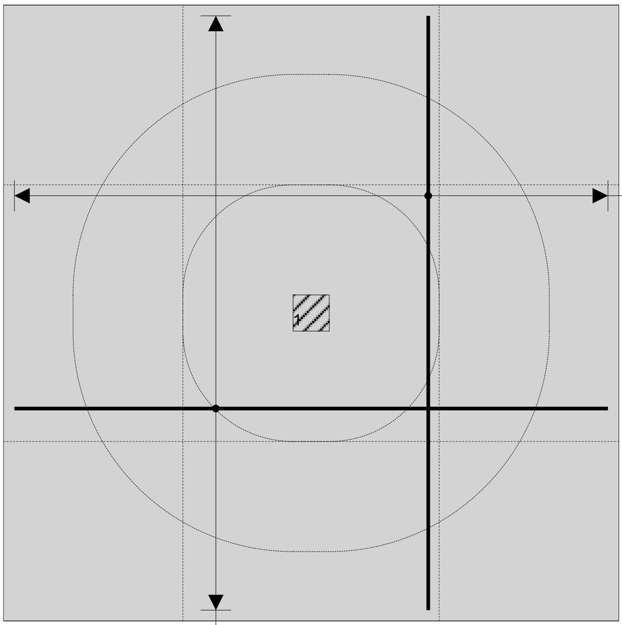

8 No.16 dia.bars bottom (230 c/c)

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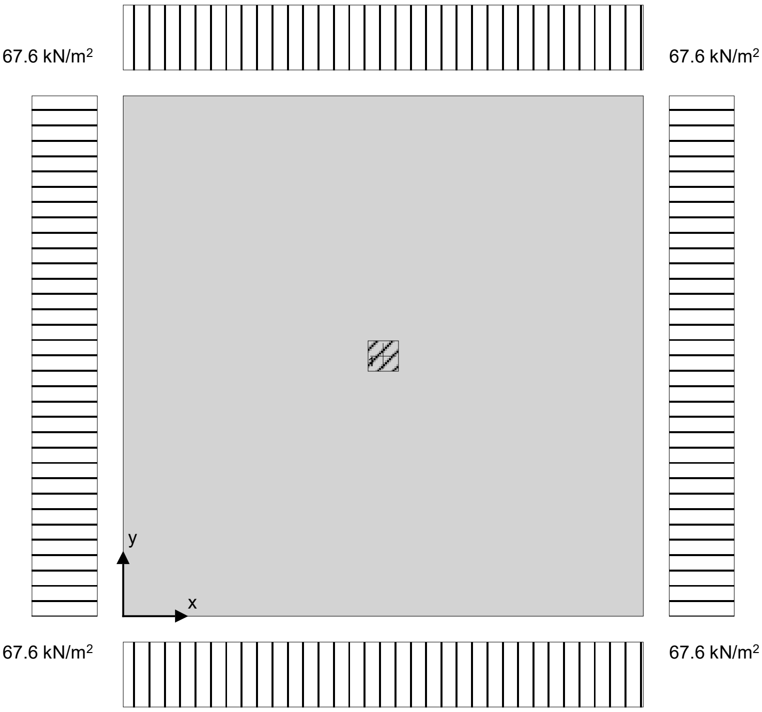
P4 pad

Foundation analysis (EN1997-1:2004)
In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1
TEDDS calculation version 3.2.15

Pad foundation details
Length of foundation; L_x = 1700 mm
Width of foundation; L_y = 1700 mm
Foundation area; $A = L_x \times L_y$ = 2.890 m^2
Depth of foundation; h = 350 mm
Depth of soil over foundation; h_{soil} = 0 mm
Level of water; h_{water} = 0 mm
Density of water; γ_{water} = 9.8 kN/m^3
Density of concrete; γ_{conc} = 25.0 kN/m^3



Column no.1 details
Length of column; l_{x1} = 100 mm
Width of column; l_{y1} = 100 mm
position in x-axis; x_1 = 850 mm
position in y-axis; y_1 = 850 mm

Soil properties
Density of soil; γ_{soil} = 18.0 kN/m^3
Characteristic cohesion; c'_k = 0 kN/m^2
Characteristic effective shear resistance angle; ϕ'_k = 30 deg
Characteristic friction angle; δ_k = 20 deg

Foundation loads

Self weight; $F_{swt} = h \times \gamma_{conc} = 8.8$ kN/m^2

Column no.1 loads

Permanent load in z; $F_{Gz1} = 120.0$ kN

Variable load in z; $F_{Qz1} = 50.0$ kN

Bearing resistance (Section 6.5.2)

Forces on foundation

Force in z-axis; $F_{dz} = A \times F_{swt} + F_{Gz1} + F_{Qz1} = 195.3$ kN

Moments on foundation

Moment in x-axis; $M_{dx} = A \times F_{swt} \times L_x / 2 + F_{Gz1} \times x_1 + F_{Qz1} \times x_1 = 166.0$ kNm

Moment in y-axis; $M_{dy} = A \times F_{swt} \times L_y / 2 + F_{Gz1} \times y_1 + F_{Qz1} \times y_1 = 166.0$ kNm

Eccentricity of base reaction

Eccentricity of base reaction in x-axis; $e_x = M_{dx} / F_{dz} - L_x / 2 = 0$ mm

Eccentricity of base reaction in y-axis; $e_y = M_{dy} / F_{dz} - L_y / 2 = 0$ mm

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_x / L_x - 6 \times e_y / L_y) / (L_x \times L_y) = 67.6$ kN/m^2

$q_2 = F_{dz} \times (1 - 6 \times e_x / L_x + 6 \times e_y / L_y) / (L_x \times L_y) = 67.6$ kN/m^2

$q_3 = F_{dz} \times (1 + 6 \times e_x / L_x - 6 \times e_y / L_y) / (L_x \times L_y) = 67.6$ kN/m^2

$q_4 = F_{dz} \times (1 + 6 \times e_x / L_x + 6 \times e_y / L_y) / (L_x \times L_y) = 67.6$ kN/m^2

Minimum base pressure; $q_{min} = \min(q_1, q_2, q_3, q_4) = 67.6$ kN/m^2

Maximum base pressure; $q_{max} = \max(q_1, q_2, q_3, q_4) = 67.6$ kN/m^2

Presumed bearing capacity

Presumed bearing capacity; $P_{bearing} = 75.0$ kN/m^2

PASS - Presumed bearing capacity exceeds design base pressure

Design approach 1

Partial factors on actions - Combination1

Partial factor set; A1

Permanent unfavourable action - Table A.3; $\gamma_G = 1.35$

Permanent favourable action - Table A.3; $\gamma_{Gf} = 1.00$

Variable unfavourable action - Table A.3; $\gamma_Q = 1.50$

Variable favourable action - Table A.3; $\gamma_{Qf} = 0.00$

Partial factors for spread foundations - Combination1

Resistance factor set; R1

Bearing - Table A.5; $\gamma_{R.v} = 1.00$

Sliding - Table A.5; $\gamma_{R.h} = 1.00$

Forces on foundation

Force in z-axis; $F_{dz} = \gamma_G \times (A \times F_{swt} + F_{Gz1}) + \gamma_Q \times F_{Qz1} = 271.1$ kN

Moments on foundation

Moment in x-axis; $M_{dx} = \gamma_G \times (A \times F_{swt} \times L_x / 2 + F_{Gz1} \times x_1) + \gamma_Q \times F_{Qz1} \times x_1 = 230.5$ kNm

Moment in y-axis; $M_{dy} = \gamma_G \times (A \times F_{swt} \times L_y / 2 + F_{Gz1} \times y_1) + \gamma_Q \times F_{Qz1} \times y_1 = 230.5$ kNm

Eccentricity of base reaction

Eccentricity of base reaction in x-axis; $e_x = M_{dx} / F_{dz} - L_x / 2 = 0$ mm

Eccentricity of base reaction in y-axis; $e_y = M_{dy} / F_{dz} - L_y / 2 = 0$ mm

Effective area of base

Effective length; $L'_x = L_x - 2 \times e_x = 1700$ mm

Effective width; $L'_y = L_y - 2 \times e_y = 1700$ mm

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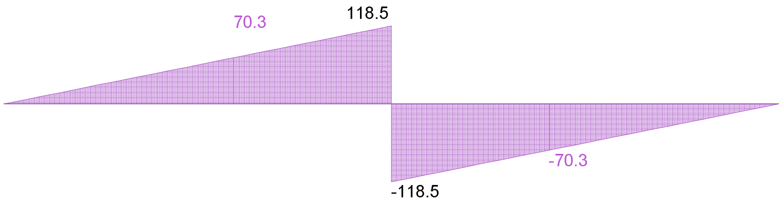
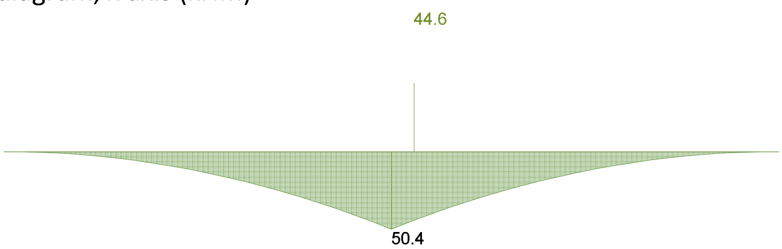
Effective area; $A' = L'_x \times L'_y = 2.890$ m^2
Pad base pressure
Design base pressure; $f_{dz} = F_{dz} / A' = 93.8$ kN/m^2
Design approach 1
Partial factors on actions - Combination2
Partial factor set; A2
Permanent unfavourable action - Table A.3; $\gamma_G = 1.00$
Permanent favourable action - Table A.3; $\gamma_{Gf} = 1.00$
Variable unfavourable action - Table A.3; $\gamma_Q = 1.30$
Variable favourable action - Table A.3; $\gamma_{Qf} = 0.00$
Partial factors for spread foundations - Combination2
Resistance factor set; R1
Bearing - Table A.5; $\gamma_{R.v} = 1.00$
Sliding - Table A.5; $\gamma_{R.h} = 1.00$
Forces on foundation
Force in z-axis; $F_{dz} = \gamma_G \times (A \times F_{swt} + F_{Gz1}) + \gamma_Q \times F_{Qz1} = 210.3$ kN
Moments on foundation
Moment in x-axis; $M_{dx} = \gamma_G \times (A \times F_{swt} \times L_x / 2 + F_{Gz1} \times x_1) + \gamma_Q \times F_{Qz1} \times x_1 = 178.7$ kNm
Moment in y-axis; $M_{dy} = \gamma_G \times (A \times F_{swt} \times L_y / 2 + F_{Gz1} \times y_1) + \gamma_Q \times F_{Qz1} \times y_1 = 178.7$ kNm
Eccentricity of base reaction
Eccentricity of base reaction in x-axis; $e_x = M_{dx} / F_{dz} - L_x / 2 = 0$ mm
Eccentricity of base reaction in y-axis; $e_y = M_{dy} / F_{dz} - L_y / 2 = 0$ mm
Effective area of base
Effective length; $L'_x = L_x - 2 \times e_x = 1700$ mm
Effective width; $L'_y = L_y - 2 \times e_y = 1700$ mm
Effective area; $A' = L'_x \times L'_y = 2.890$ m^2
Pad base pressure
Design base pressure; $f_{dz} = F_{dz} / A' = 72.8$ kN/m^2
Foundation design (EN1992-1-1:2004)
In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1
TEDDS calculation version 3.2.15
Concrete details (Table 3.1 - Strength and deformation characteristics for concrete)
Concrete strength class;C30/37
Characteristic compressive cylinder strength; $f_{ck} = 30$ N/mm^2
Characteristic compressive cube strength; $f_{ck,cube} = 37$ N/mm^2
Mean value of compressive cylinder strength; $f_{cm} = f_{ck} + 8$ N/mm^2 = 38 N/mm^2
Mean value of axial tensile strength; $f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck}/ 1$ N/mm$^2)^{2/3} = 2.9$ N/mm^2
5% fractile of axial tensile strength; $f_{ctk,0.05} = 0.7 \times f_{ctm} = 2.0$ N/mm^2
Secant modulus of elasticity of concrete; $E_{cm} = 22$ kN/mm$^2 \times [f_{cm}/10$ N/mm$^2]^{0.3} = 32837$ N/mm^2
Partial factor for concrete (Table 2.1N); $\gamma_C = 1.50$
Compressive strength coefficient (cl.3.1.6(1)); $\alpha_{cc} = 0.85$
Design compressive concrete strength (exp.3.15); $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = 17.0$ N/mm^2
Tens.strength coeff.for plain concrete (cl.12.3.1(1)); $\alpha_{ct,pl} = 0.80$
Des.tens.strength for plain concrete (exp.12.1); $f_{ctd,pl} = \alpha_{ct,pl} \times f_{ctk,0.05} / \gamma_C = 1.1$ N/mm^2
Maximum aggregate size; $h_{agg} = 20$ mm

Ultimate strain - Table 3.1; $\varepsilon_{cu2} = 0.0035$
Shortening strain - Table 3.1; $\varepsilon_{cu3} = 0.0035$
Effective compression zone height factor; $\lambda = 0.80$
Effective strength factor; $\eta = 1.00$
Bending coefficient k_1; $K_1 = 0.40$
Bending coefficient k_2; $K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2}) = 1.00$
Bending coefficient k_3; $K_3 = 0.40$
Bending coefficient k_4; $K_4 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2}) = 1.00$
Reinforcement details
Characteristic yield strength of reinforcement; $f_{yk} = 500$ N/mm^2
Modulus of elasticity of reinforcement; $E_s = 210000$ N/mm^2
Partial factor for reinforcing steel (Table 2.1N); $\gamma_S = 1.15$
Design yield strength of reinforcement; $f_{yd} = f_{yk} / \gamma_S = 435$ N/mm^2
Nominal cover to reinforcement; $c_{nom} = 30$ mm
Shear diagram, x axis (kN)



Moment diagram, x axis (kNm)



Rectangular section in flexure (Section 6.1)
Design bending moment; $M_{Ed.x.max} = 44.6$ kNm
Depth to tension reinforcement; $d = h - c_{nom} - \phi_{x.bot} / 2 = 312$ mm

$K = M_{Ed.x.max} / (L_y \times d^2 \times f_{ck}) = 0.009$

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

$K' = 0.207$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = 296$ mm
Depth of neutral axis; $x = 2.5 \times (d - z) = 39$ mm
Area of tension reinforcement required; $A_{sx.bot.req} = M_{Ed.x.max} / (f_{yd} \times z) = 346$ mm^2
Tension reinforcement provided; 8 No.16 dia.bars bottom (230 c/c)
Area of tension reinforcement provided; $A_{sx.bot.prov} = 1608$ mm^2
Minimum area of reinforcement (exp.9.1N); $A_{s.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times L_y \times d = 799$ mm^2
Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s.max} = 0.04 \times L_y \times d = 21216$ mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

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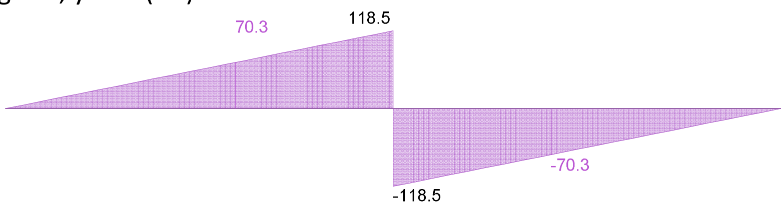
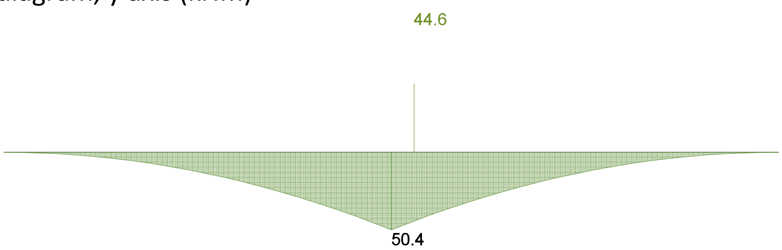
Crack control (Section 7.3)
Limiting crack width; w_{max} = 0.3 mm
Variable load factor (EN1990 – Table A1.1); ψ_2 = 0.3
Serviceability bending moment; $M_{sls.x.max}$ = 25.4 kNm
Tensile stress in reinforcement; $\sigma_s = M_{sls.x.max} / (A_{sx.bot.prov} \times z)$ = 53.3 N/mm^2
Load duration factor; k_t = 0.4
Effective depth of concrete in tension; $h_{c.ef} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = 95 mm
Effective area of concrete in tension; $A_{c.eff} = h_{c.ef} \times L_y$ = 161500 mm^2
Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm}$ = 2.9 N/mm^2
Reinforcement ratio; $\rho_{p.eff} = A_{sx.bot.prov} / A_{c.eff}$ = 0.010
Modular ratio; $\alpha_e = E_s / E_{cm}$ = 6.395
Bond property coefficient; k_1 = 0.8
Strain distribution coefficient; k_2 = 0.5
k_3 = 3.4 =3.4
k_4 = 0.425
Maximum crack spacing (exp.7.11); $s_{r.max} = k_3 \times c_{nom} + k_1 \times k_2 \times k_4 \times \phi_{x.bot} / \rho_{p.eff}$ = 375 mm
Maximum crack width (exp.7.8); $w_k = s_{r.max} \times \max([\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff})] / E_s, 0.6 \times \sigma_s / E_s)$ = 0.057 mm
PASS - Maximum crack width is less than limiting crack width
Library item: Crack width output
Rectangular section in shear (Section 6.2)
Design shear force; $V_{Ed.x.max}$ = 70.3 kN
$C_{Rd,c} = 0.18 / \gamma_C$ = 0.120
$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = 1.822
Longitudinal reinforcement ratio; $\rho_l = \min(A_{sx.bot.prov} / (L_y \times d), 0.02)$ = 0.003
$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = 0.471 N/mm^2
Design shear resistance (exp.6.2a & 6.2b); $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times L_y \times d$
$V_{Rd.c}$ = 237.2 kN
PASS - Design shear resistance exceeds design shear force
Shear diagram, y axis (kN)



Moment diagram, y axis (kNm)



Rectangular section in flexure (Section 6.1)

Design bending moment; $M_{Ed.y.max}$ = 44.6 kNm
Depth to tension reinforcement; $d = h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2$ = 296 mm

$K = M_{Ed.y.max} / (L_x \times d^2 \times f_{ck}) = 0.010$

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

$K' = 0.207$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = 281 mm
Depth of neutral axis; $x = 2.5 \times (d - z)$ = 37 mm
Area of tension reinforcement required; $A_{sy.bot.req} = M_{Ed.y.max} / (f_{yd} \times z)$ = 365 mm^2
Tension reinforcement provided; 8 No.16 dia.bars bottom (230 c/c)
Area of tension reinforcement provided; $A_{sy.bot.prov}$ = 1608 mm^2
Minimum area of reinforcement (exp.9.1N); $A_{s.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times L_x \times d$ = 758 mm^2
Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s.max} = 0.04 \times L_x \times d$ = 20128 mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control (Section 7.3)
Limiting crack width; w_{max} = 0.3 mm
Variable load factor (EN1990 – Table A1.1); $\psi_2 = 0.3$
Serviceability bending moment; $M_{sls.y.max}$ = 25.4 kNm
Tensile stress in reinforcement; $\sigma_s = M_{sls.y.max} / (A_{sy.bot.prov} \times z)$ = 56.2 N/mm^2
Load duration factor; $k_t = 0.4$
Effective depth of concrete in tension; $h_{c.ef} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = 104 mm
Effective area of concrete in tension; $A_{c.eff} = h_{c.ef} \times L_x$ = 177367 mm^2
Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm}$ = 2.9 N/mm^2
Reinforcement ratio; $\rho_{p.eff} = A_{sy.bot.prov} / A_{c.eff} = 0.009$
Modular ratio; $\alpha_e = E_s / E_{cm} = 6.395$
Bond property coefficient; $k_1 = 0.8$
Strain distribution coefficient; $k_2 = 0.5$

$k_3 = 3.4 = 3.4$

$k_4 = 0.425$

Maximum crack spacing (exp.7.11); $s_{r.max} = k_3 \times (c_{nom} + \phi_{x.bot}) + k_1 \times k_2 \times k_4 \times \phi_{y.bot} / \rho_{p.eff}$ = 456 mm
Maximum crack width (exp.7.8); $w_k = s_{r.max} \times \max([\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff})] / E_s, 0.6 \times \sigma_s / E_s)$ = 0.073 mm

PASS - Maximum crack width is less than limiting crack width

Library item: Crack width output

Rectangular section in shear (Section 6.2)
Design shear force; $V_{Ed.y.max}$ = 70.3 kN

$C_{Rd,c} = 0.18 / \gamma_C = 0.120$

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = 1.822$

Longitudinal reinforcement ratio; $\rho_l = \min(A_{sy.bot.prov} / (L_x \times d), 0.02) = 0.003$

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = 0.471 N/mm^2

Design shear resistance (exp.6.2a & 6.2b); $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times L_x \times d$

$V_{Rd.c}$ = 237.2 kN

PASS - Design shear resistance exceeds design shear force

Punching shear (Section 6.4)
Strength reduction factor (exp 6.6N); $v = 0.6 \times [1 - f_{ck} / 250 \text{ N/mm}^2] = 0.528$

Average depth to reinforcement; $d = 304$ mm
Maximum punching shear resistance (cl.6.4.5(3)); $v_{Rd.max} = 0.5 \times v \times f_{cd} = 4.488$ N/mm^2
$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = 1.811$
Longitudinal reinforcement ratio (cl.6.4.4(1)); $\rho_{lx} = A_{sx.bot.prov} / (L_y \times d) = 0.003$
$\rho_{ly} = A_{sy.bot.prov} / (L_x \times d) = 0.003$
$\rho_l = \min(\sqrt{(\rho_{lx} \times \rho_{ly})}, 0.02) = 0.003$
$C_{Rd,c} = 0.18 / \gamma_C = 0.120$
$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = 0.467$ N/mm^2
Design punching shear resistance (exp.6.47); $v_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min})$
$= 0.467$ N/mm^2
Design punching shear resistance at 1d (exp. 6.50); $v_{Rd.c1} = (2 \times d / d) \times v_{Rd.c} = 0.934$ N/mm^2
Column No.1 - Punching shear perimeter at column face
Punching shear perimeter; $u_0 = 400$ mm
Area within punching shear perimeter; $A_0 = 0.010$ m^2
Maximum punching shear force; $V_{Ed.max} = 236.2$ kN
Punching shear stress factor (fig 6.21N); $\beta = 1.000$
Maximum punching shear stress (exp 6.38); $v_{Ed.max} = \beta \times V_{Ed.max} / (u_0 \times d) = 1.942$ N/mm^2
PASS - Maximum punching shear resistance exceeds maximum punching shear stress
Column No.1 - Punching shear perimeter at 1d from column face
Punching shear perimeter; $u_1 = 2310$ mm
Area within punching shear perimeter; $A_1 = 0.422$ m^2
Design punching shear force; $V_{Ed.1} = 202.4$ kN
Punching shear stress factor (fig 6.21N); $\beta = 1.000$
Design punching shear stress (exp 6.38); $v_{Ed.1} = \beta \times V_{Ed.1} / (u_1 \times d) = 0.288$ N/mm^2
PASS - Design punching shear resistance exceeds increased design punching shear stress
Column No.1 - Punching shear perimeter at 2d from column face
Punching shear perimeter; $u_2 = 4220$ mm
Area within punching shear perimeter; $A_2 = 1.415$ m^2
Design punching shear force; $V_{Ed.2} = 121$ kN
Punching shear stress factor (fig 6.21N); $\beta = 1.000$
Design punching shear stress (exp 6.38); $v_{Ed.2} = \beta \times V_{Ed.2} / (u_2 \times d) = 0.094$ N/mm^2
PASS - Design punching shear resistance exceeds design punching shear stress

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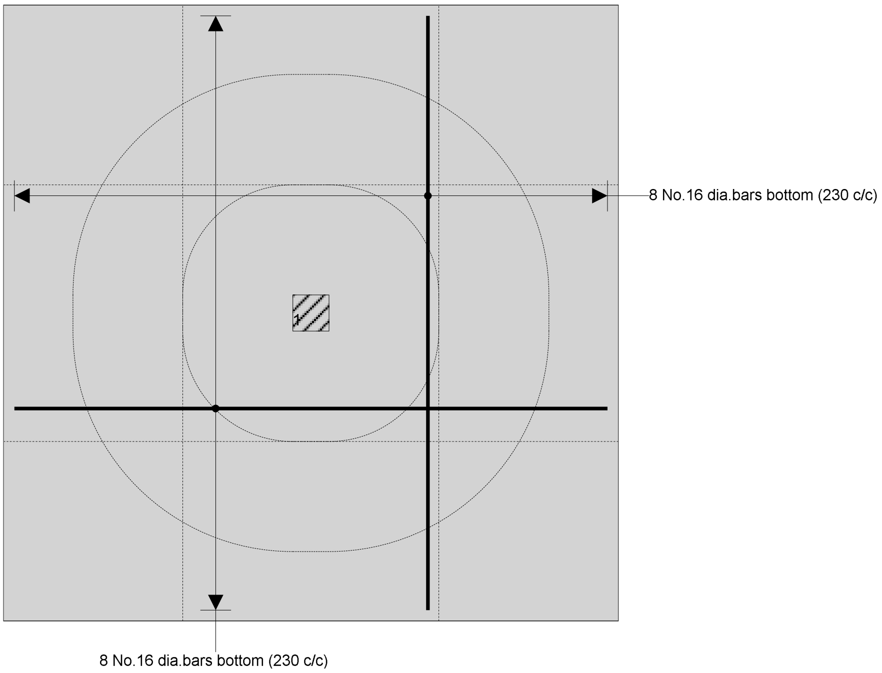

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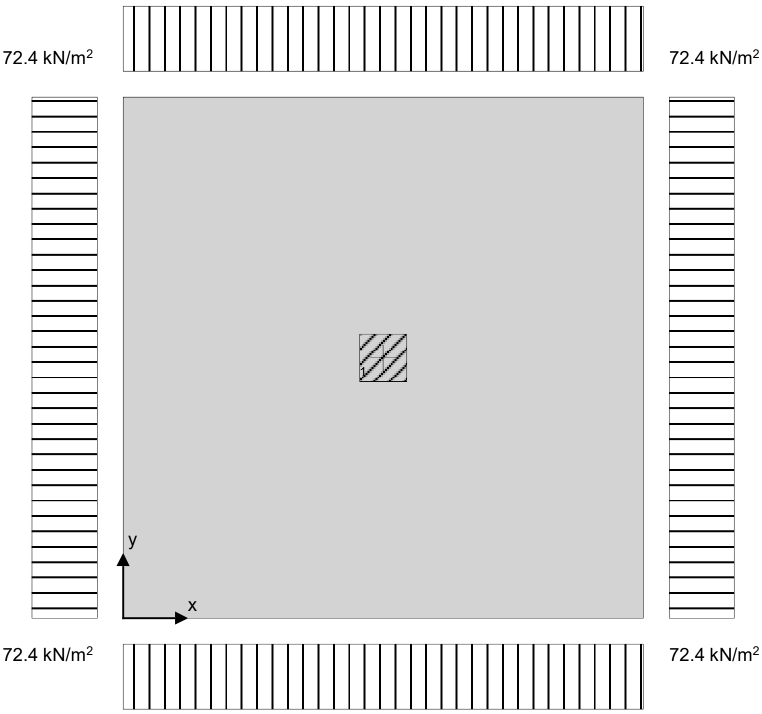
P5 pad

Foundation analysis (EN1997-1:2004)
In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1
TEDDS calculation version 3.2.15
Pad foundation details
Length of foundation; $L_x = 1100$ mm
Width of foundation; $L_y = 1100$ mm
Foundation area; $A = L_x \times L_y = 1.210$ m^2
Depth of foundation; h = 250 mm
Depth of soil over foundation; $h_{soil} = 0$ mm
Level of water; $h_{water} = 0$ mm
Density of water; $\gamma_{water} = 9.8$ kN/m^3
Density of concrete; $\gamma_{conc} = 25.0$ kN/m^3



Column no.1 details
Length of column; $l_{x1} = 100$ mm
Width of column; $l_{y1} = 100$ mm
position in x-axis; $x_1 = 550$ mm
position in y-axis; $y_1 = 550$ mm
Soil properties
Density of soil; $\gamma_{soil} = 18.0$ kN/m^3
Characteristic cohesion; $c'_k = 0$ kN/m^2
Characteristic effective shear resistance angle; $\phi'_k = 30$ deg
Characteristic friction angle; $\delta_k = 20$ deg
Foundation loads

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Self weight; $F_{swt} = h \times \gamma_{conc} = 6.3$ kN/m^2
Column no.1 loads
Permanent load in z; $F_{Gz1} = 55.0$ kN
Variable load in z; $F_{Qz1} = 25.0$ kN
Bearing resistance (Section 6.5.2)
Forces on foundation
Force in z-axis; $F_{dz} = A \times F_{swt} + F_{Gz1} + F_{Qz1} = 87.6$ kN
Moments on foundation
Moment in x-axis; $M_{dx} = A \times F_{swt} \times L_x / 2 + F_{Gz1} \times x_1 + F_{Qz1} \times x_1 = 48.2$ kNm
Moment in y-axis; $M_{dy} = A \times F_{swt} \times L_y / 2 + F_{Gz1} \times y_1 + F_{Qz1} \times y_1 = 48.2$ kNm
Eccentricity of base reaction
Eccentricity of base reaction in x-axis; $e_x = M_{dx} / F_{dz} - L_x / 2 = 0$ mm
Eccentricity of base reaction in y-axis; $e_y = M_{dy} / F_{dz} - L_y / 2 = 0$ mm
Pad base pressures
$q_1 = F_{dz} \times (1 - 6 \times e_x / L_x - 6 \times e_y / L_y) / (L_x \times L_y) = 72.4$ kN/m^2
$q_2 = F_{dz} \times (1 - 6 \times e_x / L_x + 6 \times e_y / L_y) / (L_x \times L_y) = 72.4$ kN/m^2
$q_3 = F_{dz} \times (1 + 6 \times e_x / L_x - 6 \times e_y / L_y) / (L_x \times L_y) = 72.4$ kN/m^2
$q_4 = F_{dz} \times (1 + 6 \times e_x / L_x + 6 \times e_y / L_y) / (L_x \times L_y) = 72.4$ kN/m^2
Minimum base pressure; $q_{min} = \min(q_1, q_2, q_3, q_4) = 72.4$ kN/m^2
Maximum base pressure; $q_{max} = \max(q_1, q_2, q_3, q_4) = 72.4$ kN/m^2
Presumed bearing capacity
Presumed bearing capacity; $P_{bearing} = 75.0$ kN/m^2
PASS - Presumed bearing capacity exceeds design base pressure
Design approach 1
Partial factors on actions - Combination1
Partial factor set; A1
Permanent unfavourable action - Table A.3; $\gamma_G = 1.35$
Permanent favourable action - Table A.3; $\gamma_{Gf} = 1.00$
Variable unfavourable action - Table A.3; $\gamma_Q = 1.50$
Variable favourable action - Table A.3; $\gamma_{Qf} = 0.00$
Partial factors for spread foundations - Combination1
Resistance factor set; R1
Bearing - Table A.5; $\gamma_{R.v} = 1.00$
Sliding - Table A.5; $\gamma_{R.h} = 1.00$
Forces on foundation
Force in z-axis; $F_{dz} = \gamma_G \times (A \times F_{swt} + F_{Gz1}) + \gamma_Q \times F_{Qz1} = 122.0$ kN
Moments on foundation
Moment in x-axis; $M_{dx} = \gamma_G \times (A \times F_{swt} \times L_x / 2 + F_{Gz1} \times x_1) + \gamma_Q \times F_{Qz1} \times x_1 = 67.1$ kNm
Moment in y-axis; $M_{dy} = \gamma_G \times (A \times F_{swt} \times L_y / 2 + F_{Gz1} \times y_1) + \gamma_Q \times F_{Qz1} \times y_1 = 67.1$ kNm
Eccentricity of base reaction
Eccentricity of base reaction in x-axis; $e_x = M_{dx} / F_{dz} - L_x / 2 = 0$ mm
Eccentricity of base reaction in y-axis; $e_y = M_{dy} / F_{dz} - L_y / 2 = 0$ mm
Effective area of base
Effective length; $L'_x = L_x - 2 \times e_x = 1100$ mm
Effective width; $L'_y = L_y - 2 \times e_y = 1100$ mm
Effective area; $A' = L'_x \times L'_y = 1.210$ m^2

Pad base pressure
Design base pressure; $f_{dz} = F_{dz} / A' = 100.8$ kN/m^2
Design approach 1
Partial factors on actions - Combination2
Partial factor set; A2
Permanent unfavourable action - Table A.3; $\gamma_G = 1.00$
Permanent favourable action - Table A.3; $\gamma_{Gf} = 1.00$
Variable unfavourable action - Table A.3; $\gamma_Q = 1.30$
Variable favourable action - Table A.3; $\gamma_{Qf} = 0.00$
Partial factors for spread foundations - Combination2
Resistance factor set; R1
Bearing - Table A.5; $\gamma_{R.v} = 1.00$
Sliding - Table A.5; $\gamma_{R.h} = 1.00$
Forces on foundation
Force in z-axis; $F_{dz} = \gamma_G \times (A \times F_{swt} + F_{Gz1}) + \gamma_Q \times F_{Qz1} = 95.1$ kN
Moments on foundation
Moment in x-axis; $M_{dx} = \gamma_G \times (A \times F_{swt} \times L_x / 2 + F_{Gz1} \times x_1) + \gamma_Q \times F_{Qz1} \times x_1 = 52.3$ kNm
Moment in y-axis; $M_{dy} = \gamma_G \times (A \times F_{swt} \times L_y / 2 + F_{Gz1} \times y_1) + \gamma_Q \times F_{Qz1} \times y_1 = 52.3$ kNm
Eccentricity of base reaction
Eccentricity of base reaction in x-axis; $e_x = M_{dx} / F_{dz} - L_x / 2 = 0$ mm
Eccentricity of base reaction in y-axis; $e_y = M_{dy} / F_{dz} - L_y / 2 = 0$ mm
Effective area of base
Effective length; $L'_x = L_x - 2 \times e_x = 1100$ mm
Effective width; $L'_y = L_y - 2 \times e_y = 1100$ mm
Effective area; $A' = L'_x \times L'_y = 1.210$ m^2
Pad base pressure
Design base pressure; $f_{dz} = F_{dz} / A' = 78.6$ kN/m^2
Foundation design (EN1992-1-1:2004)
In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1
TEDDS calculation version 3.2.15
Concrete details (Table 3.1 - Strength and deformation characteristics for concrete)
Concrete strength class;C30/37
Characteristic compressive cylinder strength; $f_{ck} = 30$ N/mm^2
Characteristic compressive cube strength; $f_{ck,cube} = 37$ N/mm^2
Mean value of compressive cylinder strength; $f_{cm} = f_{ck} + 8$ N/mm^2 = 38 N/mm^2
Mean value of axial tensile strength; $f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck}/ 1$ N/mm$^2)^{2/3}$ = 2.9 N/mm^2
5% fractile of axial tensile strength; $f_{ctk,0.05} = 0.7 \times f_{ctm} = 2.0$ N/mm^2
Secant modulus of elasticity of concrete; $E_{cm} = 22$ kN/mm$^2 \times [f_{cm}/10$ N/mm$^2]^{0.3}$ = 32837 N/mm^2
Partial factor for concrete (Table 2.1N); $\gamma_C = 1.50$
Compressive strength coefficient (cl.3.1.6(1)); $\alpha_{cc} = 0.85$
Design compressive concrete strength (exp.3.15); $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = 17.0$ N/mm^2
Tens.strength coeff.for plain concrete (cl.12.3.1(1)); $\alpha_{ct,pl} = 0.80$
Des.tens.strength for plain concrete (exp.12.1); $f_{ctd,pl} = \alpha_{ct,pl} \times f_{ctk,0.05} / \gamma_C = 1.1$ N/mm^2
Maximum aggregate size; $h_{agg} = 20$ mm
Ultimate strain - Table 3.1; $\varepsilon_{cu2} = 0.0035$

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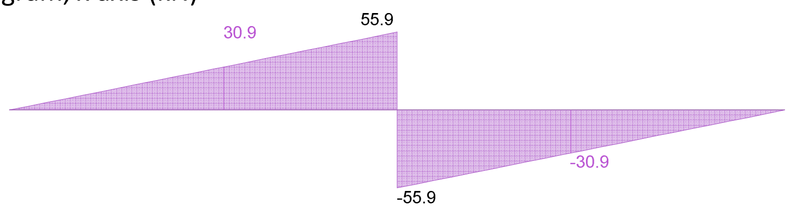
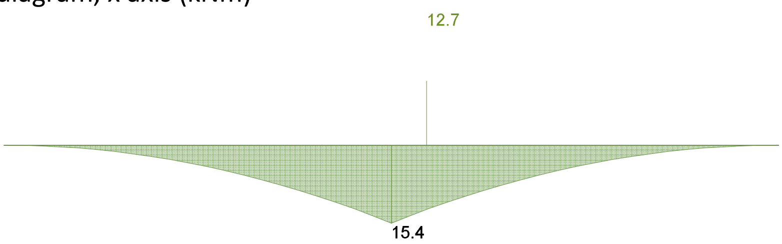
Shortening strain - Table 3.1; ε_{cu3} = 0.0035
Effective compression zone height factor; λ = 0.80
Effective strength factor; η = 1.00
Bending coefficient k_1; K_1 = 0.40
Bending coefficient k_2; $K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ =1.00
Bending coefficient k_3; K_3 =0.40
Bending coefficient k_4; $K_4 =1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ = 1.00
Reinforcement details
Characteristic yield strength of reinforcement; f_{yk} = 500 N/mm^2
Modulus of elasticity of reinforcement; E_s = 210000 N/mm^2
Partial factor for reinforcing steel (Table 2.1N); γ_S = 1.15
Design yield strength of reinforcement; $f_{yd} = f_{yk} / \gamma_S$ = 435 N/mm^2
Nominal cover to reinforcement; c_{nom} = 30 mm
Shear diagram, x axis (kN)



Moment diagram, x axis (kNm)



Rectangular section in flexure (Section 6.1)
Design bending moment; $M_{Ed.x.max}$ = 12.7 kNm
Depth to tension reinforcement; $d = h - c_{nom} - \phi_{x.bot} / 2$ = 212 mm

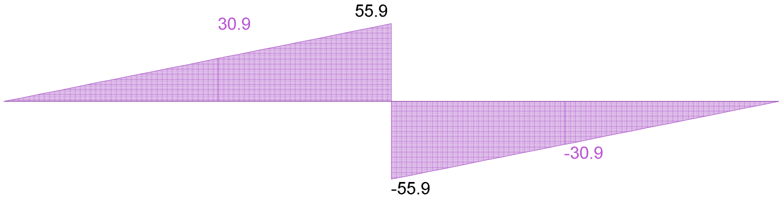
$K = M_{Ed.x.max} / (L_y \times d^2 \times f_{ck})$ = 0.009

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

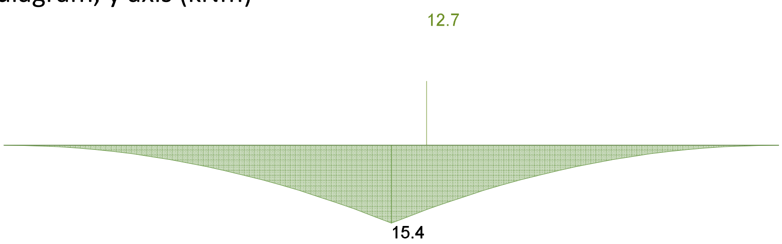
K' = 0.207

K' > K - No compression reinforcement is required
Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = 201 mm
Depth of neutral axis; $x = 2.5 \times (d - z)$ = 26 mm
Area of tension reinforcement required; $A_{sx.bot.req} = M_{Ed.x.max} / (f_{yd} \times z)$ = 145 mm^2
Tension reinforcement provided; 5 No.16 dia.bars bottom (260 c/c)
Area of tension reinforcement provided; $A_{sx.bot.prov}$ = 1005 mm^2
Minimum area of reinforcement (exp.9.1N); $A_{s.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times L_y \times d$ = 351 mm^2
Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s.max} = 0.04 \times L_y \times d$ = 9328 mm^2
PASS - Area of reinforcement provided is greater than area of reinforcement required
Crack control (Section 7.3)
Limiting crack width; w_{max} = 0.3 mm

Variable load factor (EN1990 – Table A1.1); $\psi_2 = 0.3$
Serviceability bending moment; $M_{sls.x.max} = 7.1$ kNm
Tensile stress in reinforcement; $\sigma_s = M_{sls.x.max} / (A_{sx.bot.prov} \times z) = 35.1$ N/mm^2
Load duration factor; $k_t = 0.4$
Effective depth of concrete in tension; $h_{c.ef} = \min(2.5 \times (h - d), (h - x) / 3, h / 2) = 75$ mm
Effective area of concrete in tension; $A_{c.eff} = h_{c.ef} \times L_y = 81950$ mm^2
Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} = 2.9$ N/mm^2
Reinforcement ratio; $\rho_{p.eff} = A_{sx.bot.prov} / A_{c.eff} = 0.012$
Modular ratio; $\alpha_e = E_s / E_{cm} = 6.395$
Bond property coefficient; $k_1 = 0.8$
Strain distribution coefficient; $k_2 = 0.5$
$k_3 = 3.4 = 3.4$
$k_4 = 0.425$
Maximum crack spacing (exp.7.11); $s_{r.max} = k_3 \times c_{nom} + k_1 \times k_2 \times k_4 \times \phi_{x.bot} / \rho_{p.eff} = 324$ mm
Maximum crack width (exp.7.8); $w_k = s_{r.max} \times \max([\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff})] / E_s, 0.6 \times \sigma_s / E_s) = 0.032$ mm
PASS - Maximum crack width is less than limiting crack width
Library item: Crack width output
Rectangular section in shear (Section 6.2)
Design shear force; $V_{Ed.x.max} = 30.9$ kN
$C_{Rd,c} = 0.18 / \gamma_C = 0.120$
$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = 2.000$
Longitudinal reinforcement ratio; $\rho_l = \min(A_{sx.bot.prov} / (L_y \times d), 0.02) = 0.005$
$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = 0.542$ N/mm^2
Design shear resistance (exp.6.2a & 6.2b); $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times L_y \times d$
$V_{Rd.c} = 124.7$ kN
PASS - Design shear resistance exceeds design shear force
Shear diagram, y axis (kN)



Moment diagram, y axis (kNm)



Rectangular section in flexure (Section 6.1)
Design bending moment; $M_{Ed.y.max} = 12.7$ kNm
Depth to tension reinforcement; $d = h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2 = 196$ mm

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$K = M_{Ed.y.max} / (L_x \times d^2 \times f_{ck}) = 0.010$

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

$K' = 0.207$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = 186$ mm

Depth of neutral axis; $x = 2.5 \times (d - z) = 25$ mm

Area of tension reinforcement required; $A_{sy.bot.req} = M_{Ed.y.max} / (f_{yd} \times z) = 157$ mm^2

Tension reinforcement provided; 5 No.16 dia.bars bottom (260 c/c)

Area of tension reinforcement provided; $A_{sy.bot.prov} = 1005$ mm^2

Minimum area of reinforcement (exp.9.1N); $A_{s.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times L_x \times d = 325$ mm^2

Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s.max} = 0.04 \times L_x \times d = 8624$ mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control (Section 7.3)

Limiting crack width; $w_{max} = 0.3$ mm

Variable load factor (EN1990 – Table A1.1); $\psi_2 = 0.3$

Serviceability bending moment; $M_{sls.y.max} = 7.1$ kNm

Tensile stress in reinforcement; $\sigma_s = M_{sls.y.max} / (A_{sy.bot.prov} \times z) = 37.9$ N/mm^2

Load duration factor; $k_t = 0.4$

Effective depth of concrete in tension; $h_{c.ef} = \min(2.5 \times (h - d), (h - x) / 3, h / 2) = 75$ mm

Effective area of concrete in tension; $A_{c.eff} = h_{c.ef} \times L_x = 82683$ mm^2

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} = 2.9$ N/mm^2

Reinforcement ratio; $\rho_{p.eff} = A_{sy.bot.prov} / A_{c.eff} = 0.012$

Modular ratio; $\alpha_e = E_s / E_{cm} = 6.395$

Bond property coefficient; $k_1 = 0.8$

Strain distribution coefficient; $k_2 = 0.5$

$k_3 = 3.4 = 3.4$

$k_4 = 0.425$

Maximum crack spacing (exp.7.11); $s_{r.max} = k_3 \times (c_{nom} + \phi_{x.bot}) + k_1 \times k_2 \times k_4 \times \phi_{y.bot} / \rho_{p.eff} = 380$ mm

Maximum crack width (exp.7.8); $w_k = s_{r.max} \times \max([\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff})] / E_s, 0.6 \times \sigma_s / E_s) = 0.041$ mm

PASS - Maximum crack width is less than limiting crack width

Library item: Crack width output

Rectangular section in shear (Section 6.2)

Design shear force; $V_{Ed.y.max} = 30.9$ kN

$C_{Rd,c} = 0.18 / \gamma_C = 0.120$

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = 2.000$

Longitudinal reinforcement ratio; $\rho_l = \min(A_{sy.bot.prov} / (L_x \times d), 0.02) = 0.005$

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = 0.542$ N/mm^2

Design shear resistance (exp.6.2a & 6.2b); $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times L_x \times d$

$V_{Rd.c} = 124.7$ kN

PASS - Design shear resistance exceeds design shear force

Punching shear (Section 6.4)

Strength reduction factor (exp 6.6N); $v = 0.6 \times [1 - f_{ck} / 250 \text{ N/mm}^2] = 0.528$

Average depth to reinforcement; $d = 204$ mm

Maximum punching shear resistance (cl.6.4.5(3)); $v_{Rd.max} = 0.5 \times v \times f_{cd} = 4.488$ N/mm^2

$k = \min(1 + \sqrt{(200\ mm / d)}, 2) = 1.990$

Longitudinal reinforcement ratio (cl.6.4.4(1)); $\rho_{lx} = A_{sx.bot.prov} / (L_y \times d) = 0.004$

$\rho_{ly} = A_{sy.bot.prov} / (L_x \times d) = 0.004$

$\rho_l = \min(\sqrt{(\rho_{lx} \times \rho_{ly})}, 0.02) = 0.004$

$C_{Rd,c} = 0.18 / \gamma_C = 0.120$

$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} = 0.538\ N/mm^2$

Design punching shear resistance (exp.6.47); $v_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) = 0.568\ N/mm^2$

Design punching shear resistance at 1d (exp. 6.50); $v_{Rd.c1} = (2 \times d / d) \times v_{Rd.c} = 1.136\ N/mm^2$

Column No.1 - Punching shear perimeter at column face

Punching shear perimeter; $u_0 = 400$ mm

Area within punching shear perimeter; $A_0 = 0.010\ m^2$

Maximum punching shear force; $V_{Ed.max} = 110.8$ kN

Punching shear stress factor (fig 6.21N); $\beta = 1.000$

Maximum punching shear stress (exp 6.38); $v_{Ed.max} = \beta \times V_{Ed.max} / (u_0 \times d) = 1.358\ N/mm^2$

PASS - Maximum punching shear resistance exceeds maximum punching shear stress

Column No.1 - Punching shear perimeter at 1d from column face

Punching shear perimeter; $u_1 = 1682$ mm

Area within punching shear perimeter; $A_1 = 0.222\ m^2$

Design punching shear force; $V_{Ed.1} = 91.2$ kN

Punching shear stress factor (fig 6.21N); $\beta = 1.000$

Design punching shear stress (exp 6.38); $v_{Ed.1} = \beta \times V_{Ed.1} / (u_1 \times d) = 0.266\ N/mm^2$

PASS - Design punching shear resistance exceeds increased design punching shear stress

Column No.1 - Punching shear perimeter at 2d from column face

Punching shear perimeter; $u_2 = 2964$ mm

Area within punching shear perimeter; $A_2 = 0.696\ m^2$

Design punching shear force; $V_{Ed.2} = 47.5$ kN

Punching shear stress factor (fig 6.21N); $\beta = 1.000$

Design punching shear stress (exp 6.38); $v_{Ed.2} = \beta \times V_{Ed.2} / (u_2 \times d) = 0.078\ N/mm^2$

PASS - Design punching shear resistance exceeds design punching shear stress

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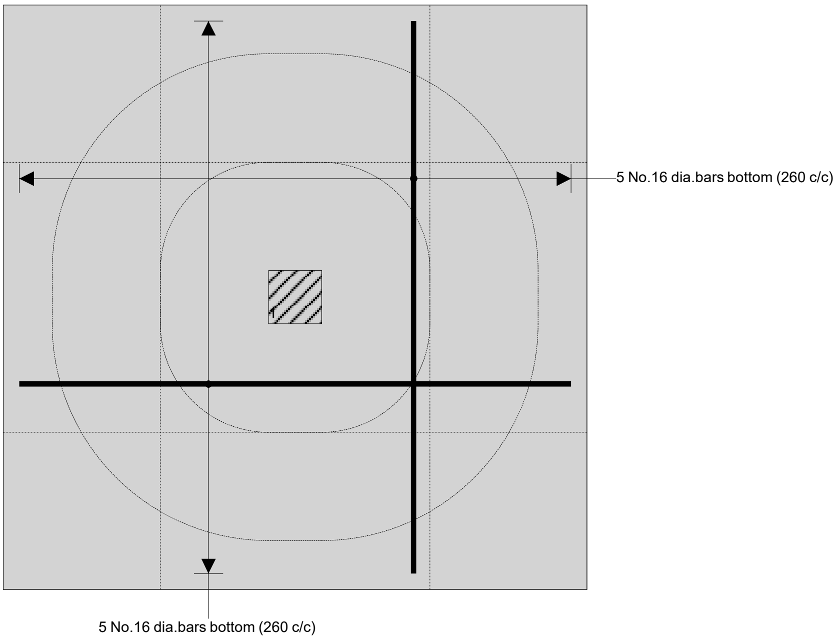
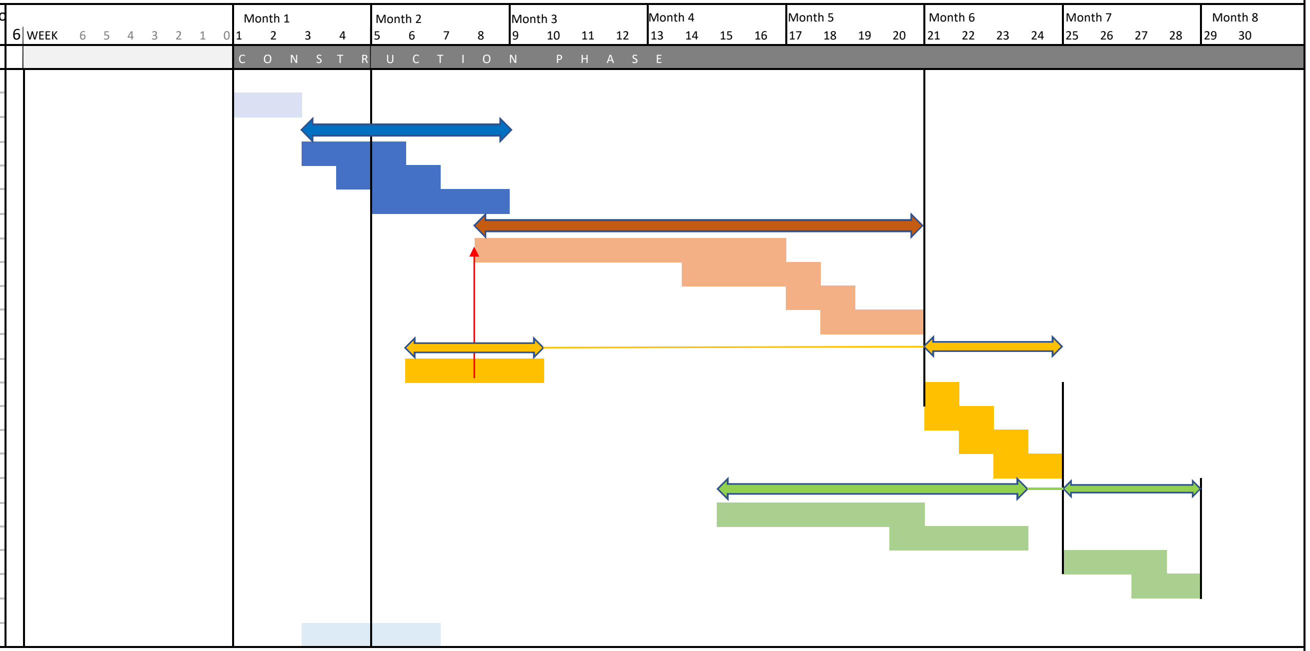


52 Eton Avenue, London NW3 3HN

ID	i	Task Mode	Task Name	Duration Working Days	Start	Finish	Predecessors
			Planning & Pre-Construction				
1			**Phase 1 Enabling Works: Shell & Core**				
2			Site setup and mobilization	2 wks	Mon Week 1	Fri Week 2	2
3			**Substructures**	**25 days**	**Mon Week 3**	**Fri Week 8**	
4			Soft strip	2.5 wks	Wed Week 3	Fri Week 5	2
5			Temporary Supports	2.5 wks	Wed Week 4	Fri Week 6	2
6			Demolition	3.5 wks	Wed Week 5	Fri Week 8	2
7			**Basement Works**	**60.5 days**	**Mon Week 5**	**Fri Week 6**	
8			Underpinning	9 wks	Mon Week 8	Fri Week 16	3
9			Excavation and disposal	4 wks	Mon Week 14	Fri Week 17	5
10			Below ground drainage	1.5 wks	Mon Week 17	Wed Week 18	5
11			Basement slab	2.5 wk	Wed Week 18	Fri Week 20	1
12			**Superstructures**	**37.5 days**	**Mon Week 6**	**Fri Week 24**	
13			Install Permanent Steelwork Above	3.5 wks	Mon Week 6	Wed Week 9	2
14			Basement Steel Columns	0.5 wks	Mon Week 21	Wed Week 21	2
15			Install Ground Floor Steelwork	1.5 wks	Wed Week 21	Fri Week 22	2
16			Shuttering of slab	1.5 wks	Mon Week 22	Wed Week 23	2
17			Install Ground Floor Slab	1.5 wks	Wed Week 23	Fri Week 24	1
18			**Sewer & Drainage**	**45 days**	**Mon Week 1**	**Fri Week 28**	
19			Redirection of Sewer	6 wks	Mon Week 15	Fri Week 20	3
			Installation of SUDS	3.5 wks	Mon Week 20	Wed Week 23	1
20			Cavity Drainage Waterproofing	3 wks	Mon Week 25	Fri Week 27	3
21			Commissioning of pumps	2 wks	Mon Week 27	Fri Week 28	5
22			**External envelope**	**20 days**	**Mon Week 3**	**Fri Week 6**	
23			Erect Scaffolding	4 wks	Mon Week 3	Fri Week 6	2

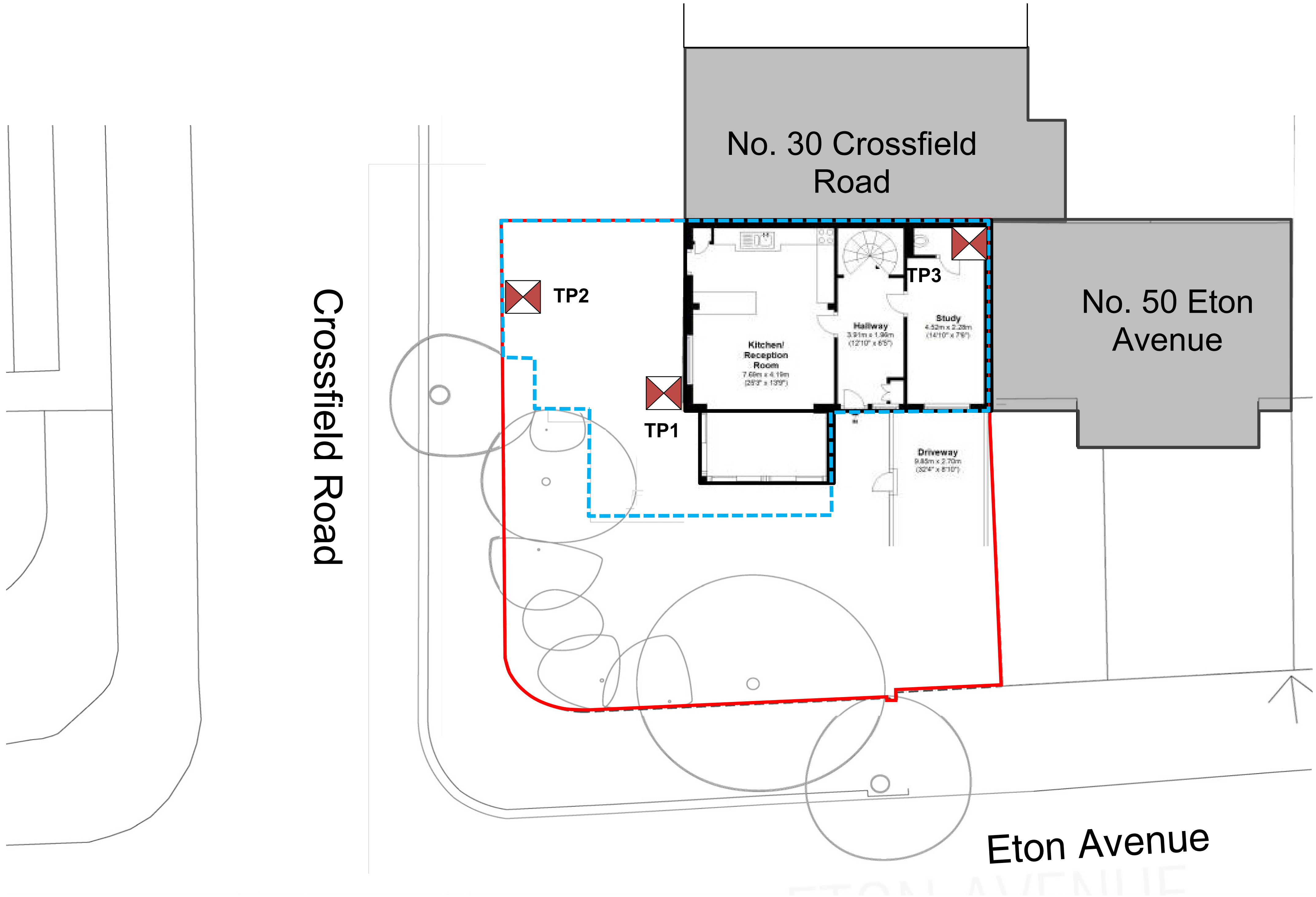


Project Programme Phase 1 - DRAFT
5 Eton Avenue

Ground Investigation

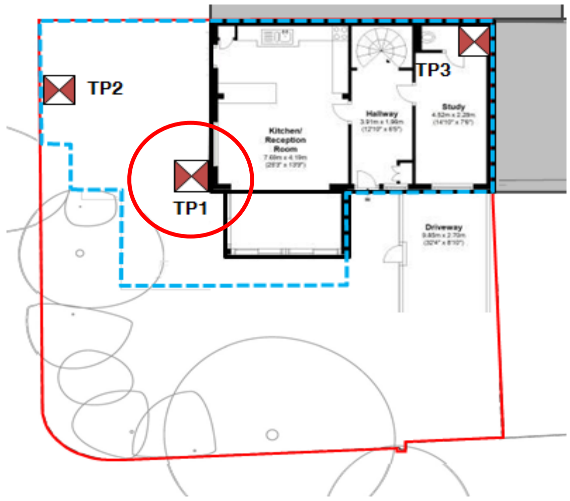
Issue Date: **April 2019**

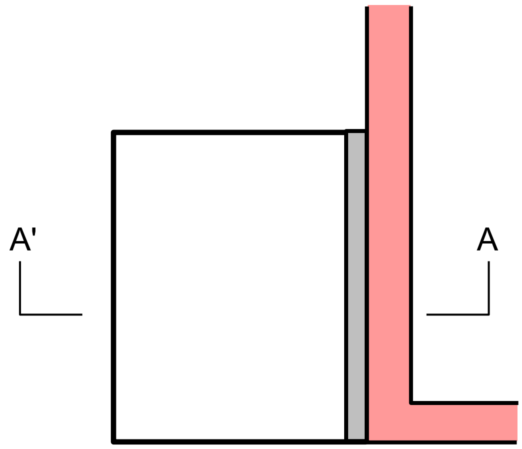
Address: **52 Eton Avenue**
London
NW3 3HN

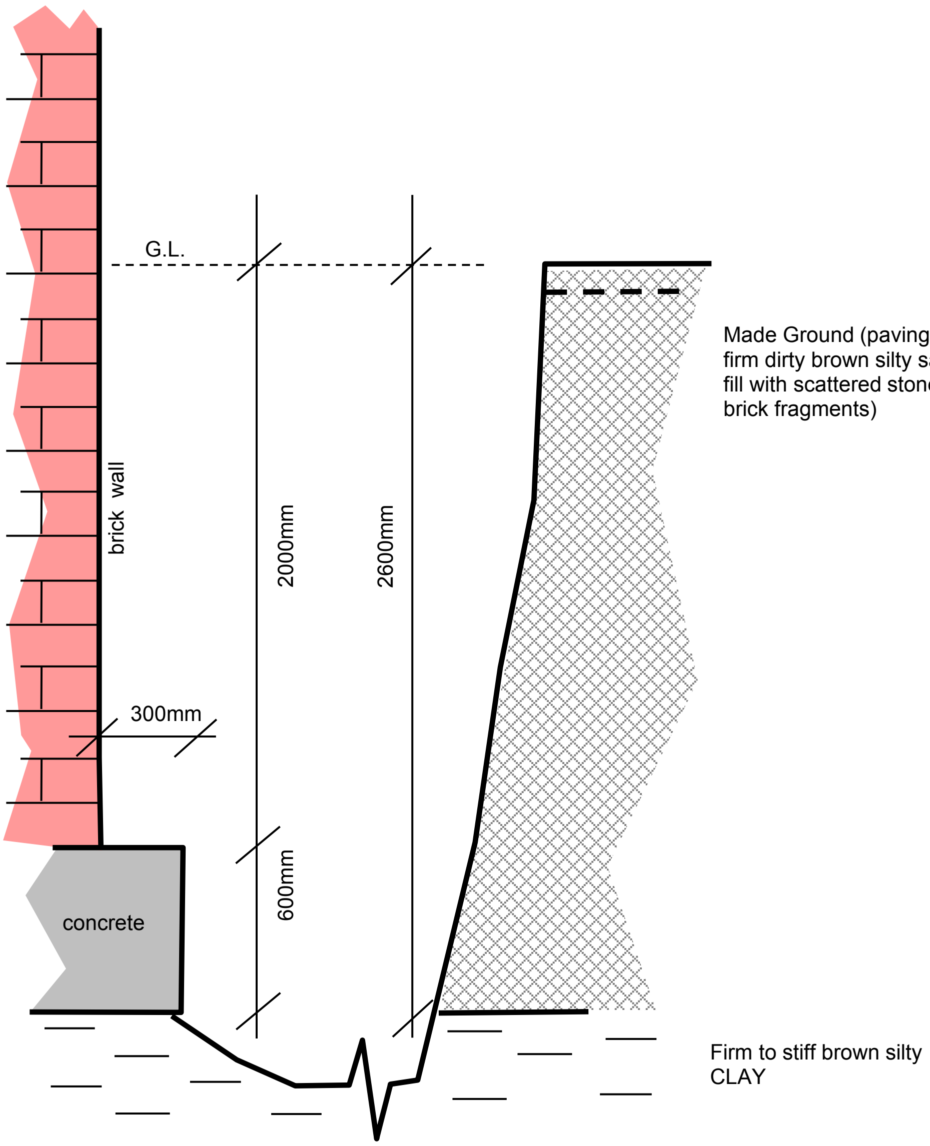

No. 30 Crossfield Road
No. 50 Eton Avenue
Crossfield Road
Eton Avenue
TP1
TP2
TP3
Kitchen/ Reception Room
Hallway
Study
Driveway
9.85m x 2.70m

SITE: 52 Eton Avenue

TRIAL PIT 1





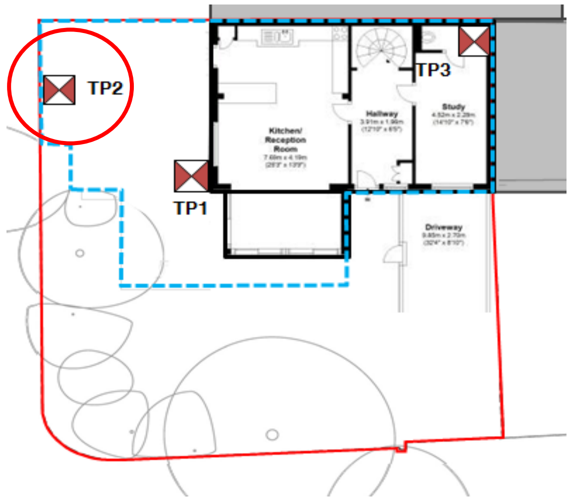


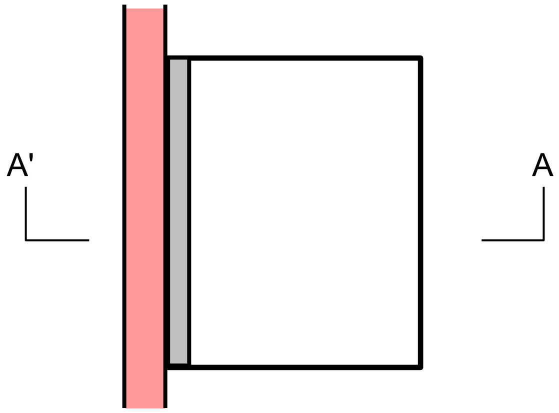
SECTION A - A'

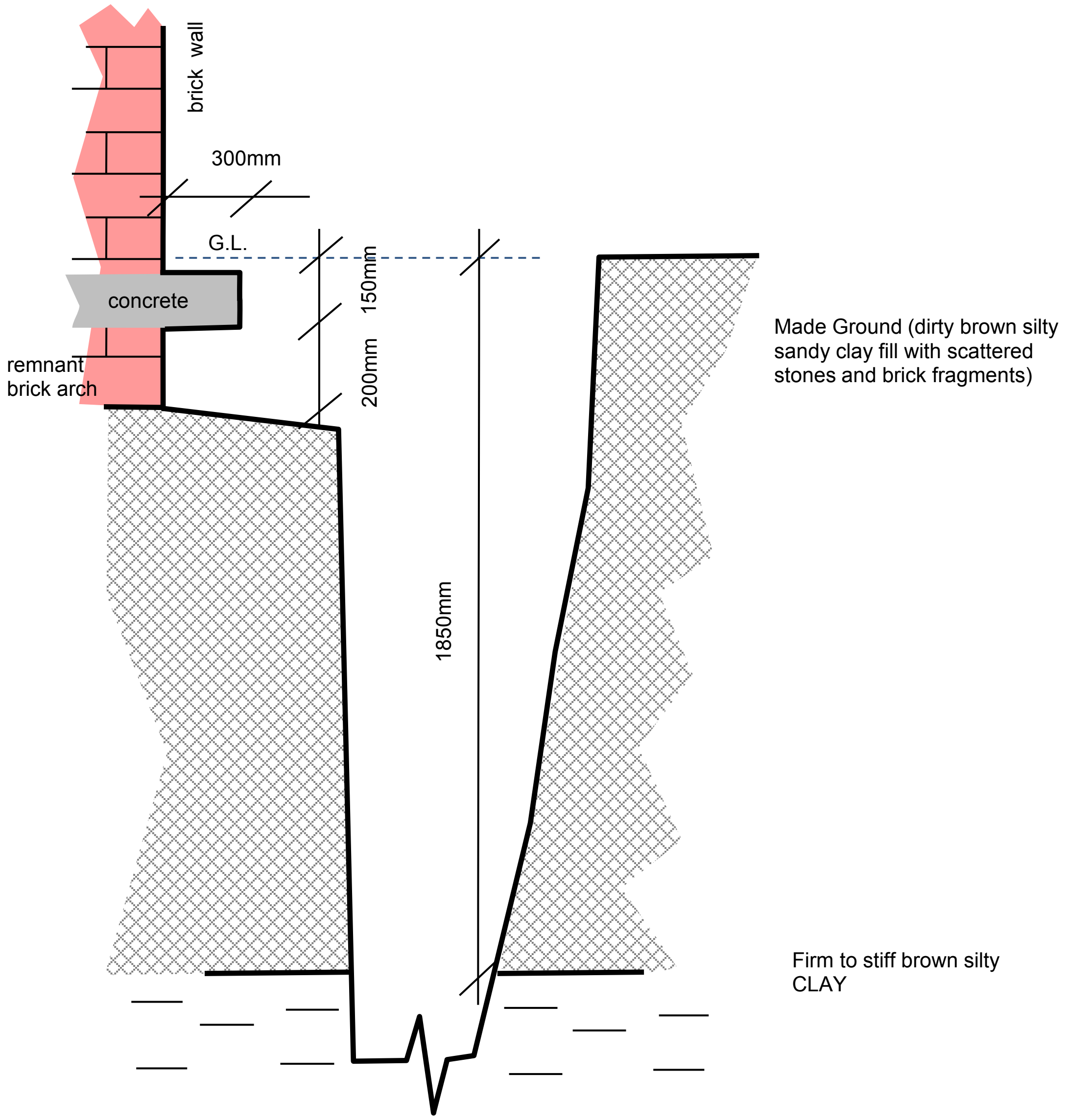
Pit Constructed 21/02/19
G.L Approx. +9.7m OD
All Dimensions in mm
Do Not Scale

PROJECT: 52 Eton Avenue

TRIAL PIT 2





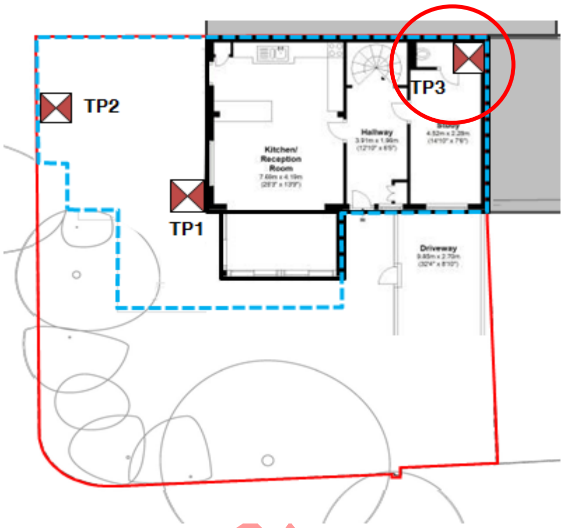


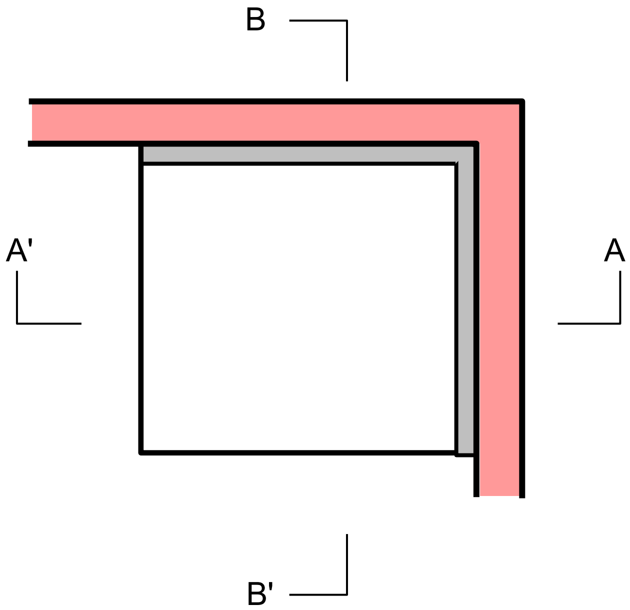
SECTION A - A'

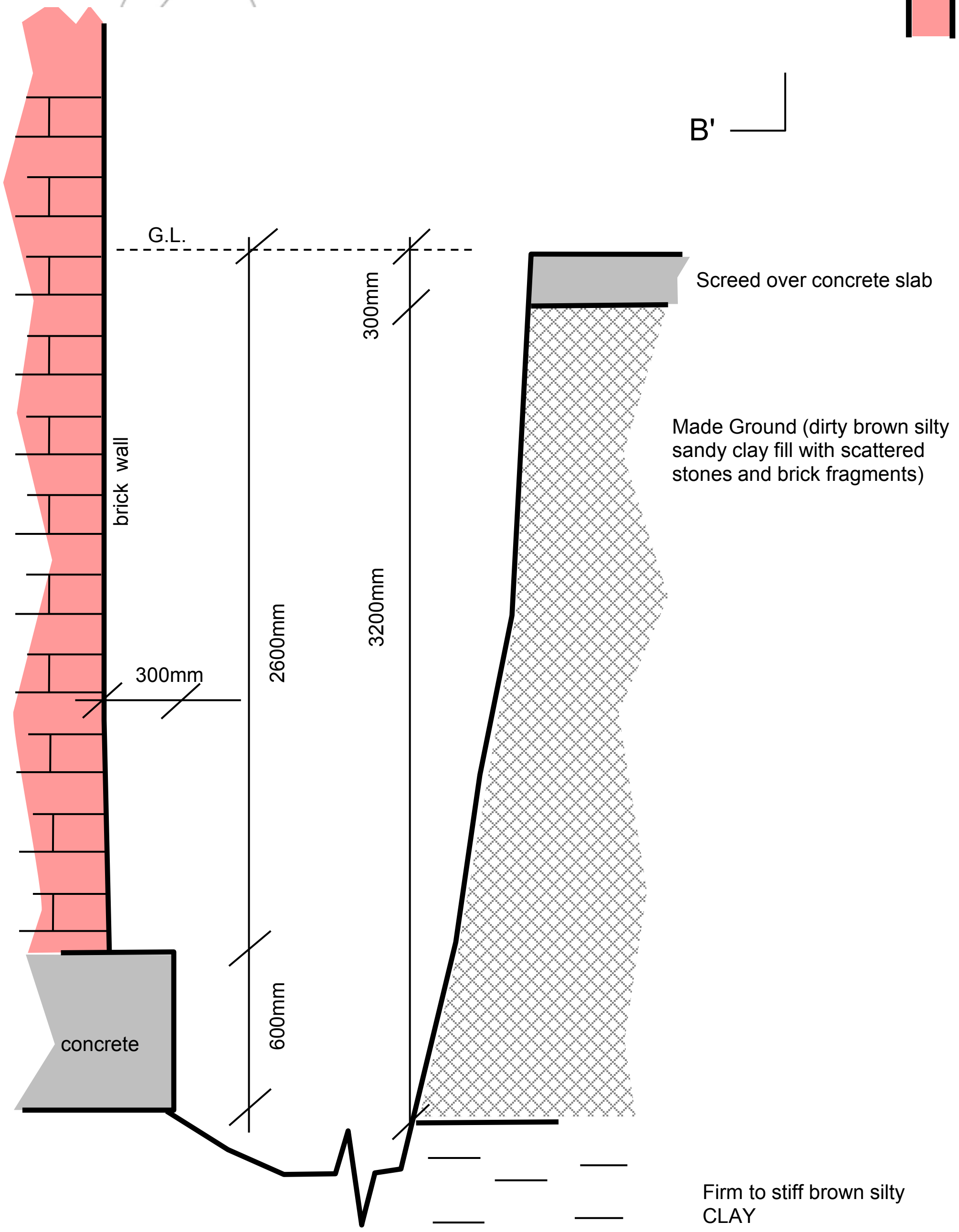
Pit constructed 03/05/19
G.L Approx. +10.1m OD
All Dimensions in mm
Do Not Scale

PROJECT: 52 Eton Avenue

TRIAL PIT 3





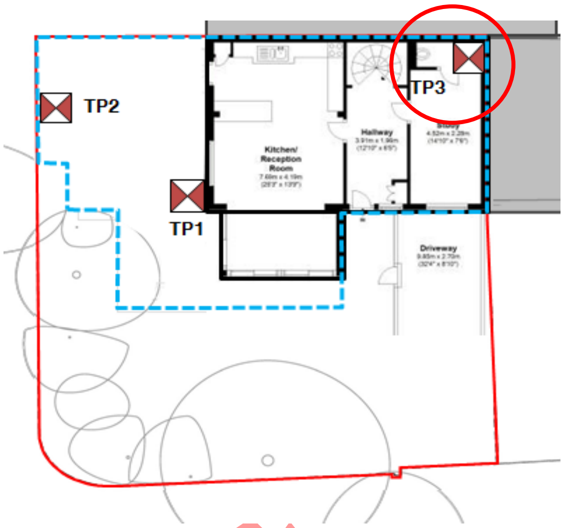


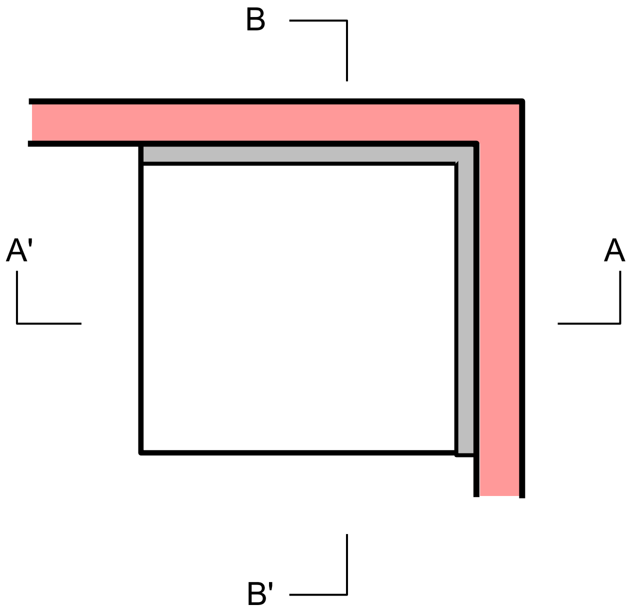
SECTION A - A'

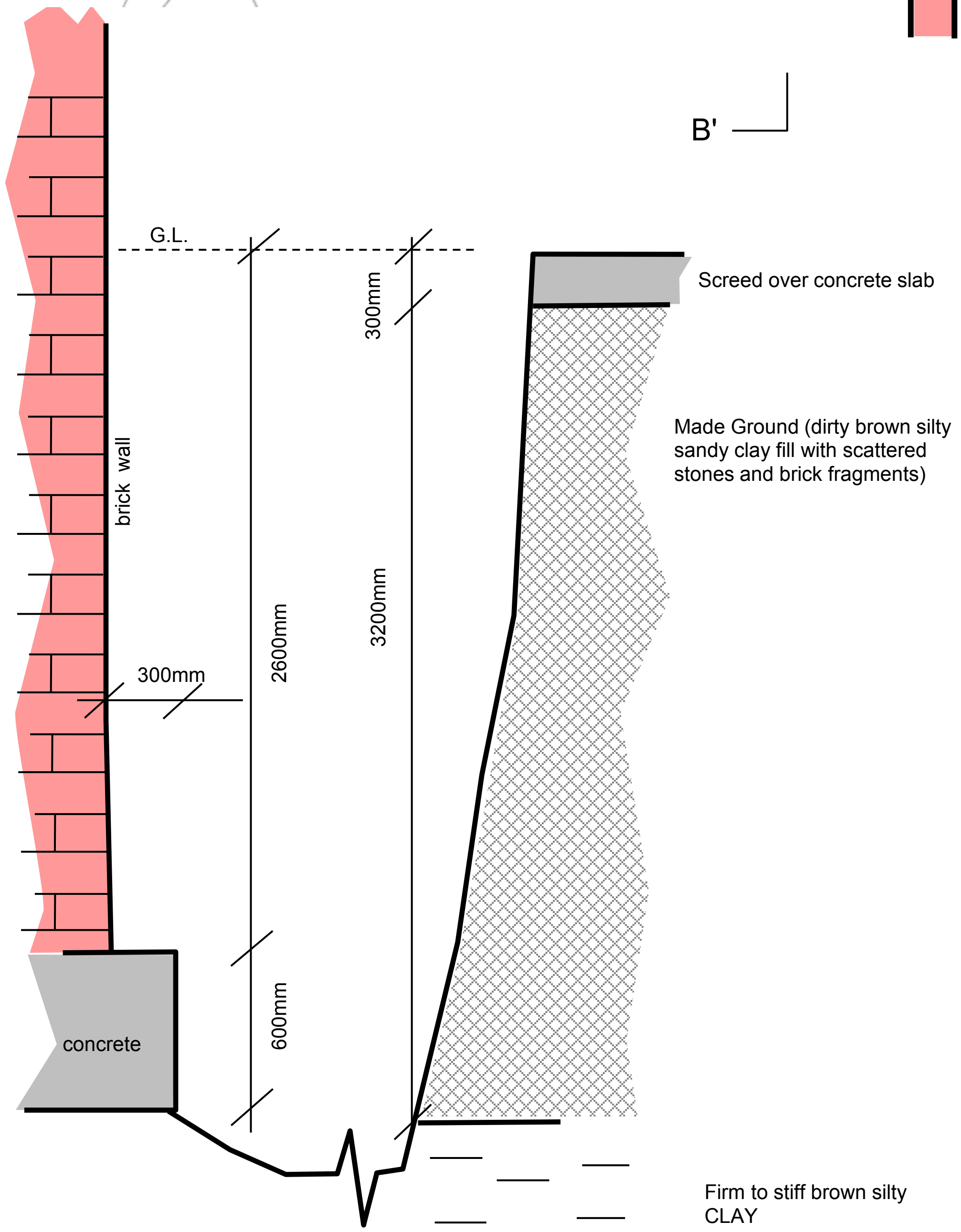
Pit constructed 03/05/19
G.L Approx. +9.9m OD
All Dimensions in mm
Do Not Scale

PROJECT: 52 Eton Avenue

TRIAL PIT 3







SECTION B - B'

Pit constructed 03/05/19
G.L Approx. +9.9m OD
All Dimensions in mm
Do Not Scale

Full Response to CRH Audit Checklist comments

Item	Yes / No	CRH Audit Comment (24th April 2019)	LBHW ENGINEERS Response (May 2019)
Are BIA Author(s) credentials satisfactory?	Yes	The qualifications satisfy the requirements of CPG. Refer to Page 2 of the BIA (Reference LBH4564, dated January 2019).	BIA audit author qualifications?
Is data required by Cl.233 of the GSD presented?	No	Insufficient information presented regarding the physical form of the development (layout, dimension etc.) since only architectural drawings are presented. Engineers' drawings are required. Further ground investigation and a work programme for construction is required. Refer to Section 4 of this report for more details.	Incorrect ? Structural engineer's drawings are not a requirement of the guidance. The architectural drawings provided include a scale and provide a good indication of the physical form, layout and dimensions of the proposed development. **A ground investigation has now been undertaken.** A work programme for construction is not a specific requirement of the guidance.
Does the description of the proposed development include all aspects of temporary and permanent works which might impact upon geology, hydrogeology and hydrology?	No	Since the ground investigation data presented is limited, it is not possible to assess the impact the proposed development can have on the existing geology, hydrogeology and hydrology.	Incorrect ? Worst credible conditions were adopted for the initial assessment of basement impact. CRH have acknowledged (see 1.17of the audit report) that the development will <u>not</u> impact on the hydrogeology of the area. It seems CRH are erroneously seeking to assess the impact of the proposed development on the existing geology. This would be an unusual action and is certainly not a requirement of the guidance. The hydrological impacts were covered the surface water assessment and SuDS strategy that accompanied the audited report

Are suitable plan/maps included?	No	The plans included are architectural in nature and do not have dimensions and elevations included. Engineers' drawings are requested. For maps, Refer to pages 10-14 and the Appendix to the BIA (Reference LBH4564, dated January 2019).	Incorrect ? On the contrary the architectural drawings do include a scale. Structural engineer's drawings are not a requirement of the guidance. Accepted
Do the plans/maps show the whole of the relevant area of study and do they show it in sufficient detail?	No	The plans included are architectural in nature and do not have dimensions and elevations included. Engineers' drawings are requested. The maps included are however satisfactory and have sufficient information.	Incorrect? On the contrary the architectural drawings do include a scale. Structural engineer's drawings are not a requirement of the guidance. Accepted
Land Stability Screening: Have appropriate data sources been consulted? Is justification provided for 'No' answers?	Yes	Refer to Page 15 and Appendix to the BIA (Reference LBH4564, dated January 2019).	Noted
Hydrogeology Screening: Have appropriate data sources been consulted? Is justification provided for 'No' answers?	Yes	Refer to Page 15 and Appendix to the BIA (Reference LBH4564, dated January 2019).	Noted
Hydrology Screening: Have appropriate data sources been consulted? Is justification provided for 'No' answers?	Yes	Refer to Page 15 and Appendix to the BIA (Reference LBH4564, dated January 2019).	Noted
Is a conceptual model presented?	No		Incorrect? The conceptual ground model was presented in section 5 of the audited report and was again summarised in section 7 of that document
Land Stability Scoping Provided? Is scoping consistent with screening outcome?	Yes	Refer to Page 17 of the BIA (Reference LBH4564, dated January 2019).	Noted

Hydrogeology Scoping Provided? Is scoping consistent with screening outcome?	No	As above.	Incorrect? (assumed typo?).
Hydrology Scoping Provided? Is scoping consistent with screening outcome?	Yes	As above.	Noted
Is factual ground investigation data provided?	Yes	However, only a section of a trial pit is provided and this is considered insufficient to allow all potential impacts to be fully assessed.	Note: This trial pit log was neither included nor referred to in the audited document. The initial impact assessment was satisfactorily concluded based upon **worst credible conditions.**
Is monitoring data presented?	No		Note: The investigation has confirmed that there is no groundwater at this site
Is the ground investigation informed by a desk study?	NA		
Has a site walkover been undertaken?	Yes		Incorrect? (the property was under different ownership at the time of the audited report)
Is the presence/absence of adjacent or nearby basements confirmed?	Yes	Refer to Page 23 of the BIA (Reference LBH4564, dated January 2019).	Noted
Is a geotechnical interpretation presented?	Yes	The soil parameters presented for London Clay are based on widely accepted archive and empirical information. However, the depth from ground level at which London Clay is present on site has to be proven. This is further discussed under Section 4 of this report.	Note: **Worst credible** assumptions have been made in regard to the depth to the London Clay.
Does the geotechnical interpretation include information on retaining wall design?	No	As above.	Incorrect? see section 6.3 of the audited report.

Are reports on other investigations required by screening and scoping presented?	No	A ground investigation report and a works programme for construction as per CPG is required.	Incorrect? A works programme for construction is not a specific requirement of the guidance.
Are the baseline conditions described, based on the GSD?	No	The ground investigation data is insufficient to confirm the ground conditions. The drawings presented lack sufficient information as they are architectural in nature and require more details for a thorough assessment.	Incorrect? No ground investigation data was submitted. The baseline ground conditions have been described and the drawings submitted do provide a good indication of the physical form, layout and dimensions of the existing development.
Do the base line conditions consider adjacent or nearby basements?	Yes		Noted
Is an Impact Assessment provided?	Yes	However, this is insufficient due to the lack of sufficient ground investigation.	Incorrect? The initial impact assessment has been undertaken on the basis of worst credible assumptions
Are estimates of ground movement and structural impact presented?	No	Although a ground movement assessment has been presented, its adequacy cannot be confirmed unless a ground investigation is carried to prove the depth at which London Clay is present and that it would indeed form the bearing stratum. Engineers' drawings with details of the existing and proposed development and indicative temporary and permanent works sequencing and calculations are also required.	Incorrect? The initial ground movement assessment has been undertaken on the basis of worst credible assumptions. Foundations would be indeed be extended into the London Clay as this is to be the bearing stratum Accepted. Further details will be supplied.
Is the Impact Assessment appropriate to the matters identified by screen and scoping?	No	The impact assessment is not valid until a detailed ground investigation is carried out and appropriate soil parameters are adopted	Incorrect? CRH have acknowledged above that the adopted soil parameters have been drawn from accepted sources. The initial impact assessment is entirely valid as it has been undertaken on the basis of worst credible assumptions. Which of these assumptions is disputed?

Has the need for mitigation been considered and are appropriate mitigation methods incorporated in the scheme?	NA		
Has the need for monitoring during construction been considered?	Yes	An outline structural monitoring plan is provided. Refer to Page 30 of the BIA (Reference LBH4564, dated January 2019).	Noted
Have the residual (after mitigation) impacts been clearly identified?	No	Cannot be confirmed until the ground conditions are confirmed using detailed ground investigation.	Incorrect? The initial assessment of residual impacts, has been undertaken on the basis of worst credible assumptions.
Has the scheme demonstrated that the structural stability of the building and neighbouring properties and infrastructure will be maintained?	No	As above.	Incorrect? The initial assessment of structural stability has been undertaken on the basis of worst credible assumptions and demonstrates that the structural stability of the building and neighbouring properties and infrastructure will be maintained.
Has the scheme avoided adversely affecting drainage and run-off or causing other damage to the water environment?	Yes	However, it is recommended that Engineers drawings be presented to verify that the proposed increase/decrease to impermeable area cannot be verified.	Note: Details of the changes in impermeable areas were contained in the surface water report. That accompanied and was referred to by the audited report.
Has the scheme avoided cumulative impacts upon structural stability or the water environment in the local area?	No	Unless documents requested in Appendix 2 of this report are furnished, the cumulative impact the proposal can have on the structural stability of the neighbouring properties and the water environment cannot be confirmed.	Incorrect? The initial assessment of cumulative impact on the structural stability of the neighbouring properties and the water environment has been undertaken on the basis of worst credible assumptions.
Does report state that damage to surrounding buildings will be no worse than Burland Category 1?	No	Unless documents requested in Appendix 2 of this report are furnished, the ground movement assessment cannot be verified.	Incorrect? The initial ground movement assessment undertaken on the basis of worst credible assumptions result in no worse than Burland Category 1 damage.

Are non-technical summaries provided?	Yes	Refer to Page 6 of the BIA (Reference LBH4564, dated January 2019). However, this should be updated in accordance with the comments presented in this audit report.	Noted