



Document Title: Planning Application Overview

A6 Off Licence 797, Stockport Road, M19 3DL

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Introduction

The Crime Impact Statement (CIS) is the report requested by local planning authorities, to be submitted with major planning applications. These are sometimes referred to as Crime Impact Assessments or Design Out Crime Assessments (DOCA). The CIS is not just a report however, but a consultation and evaluation process undertaken by Design for Security.

Introduced in 2006 by Greater Manchester Police & Manchester Planning Authorities, the CIS process involves identifying, predicting, evaluating and mitigating the crime and disorder effects of a development proposal early in the design process—prior to planning decisions being taken and commitments made. The purpose of the CIS is to ensure that design decision-makers consider crime, disorder and fear of crime before determining whether to proceed with new projects (see location plan in figure 1).

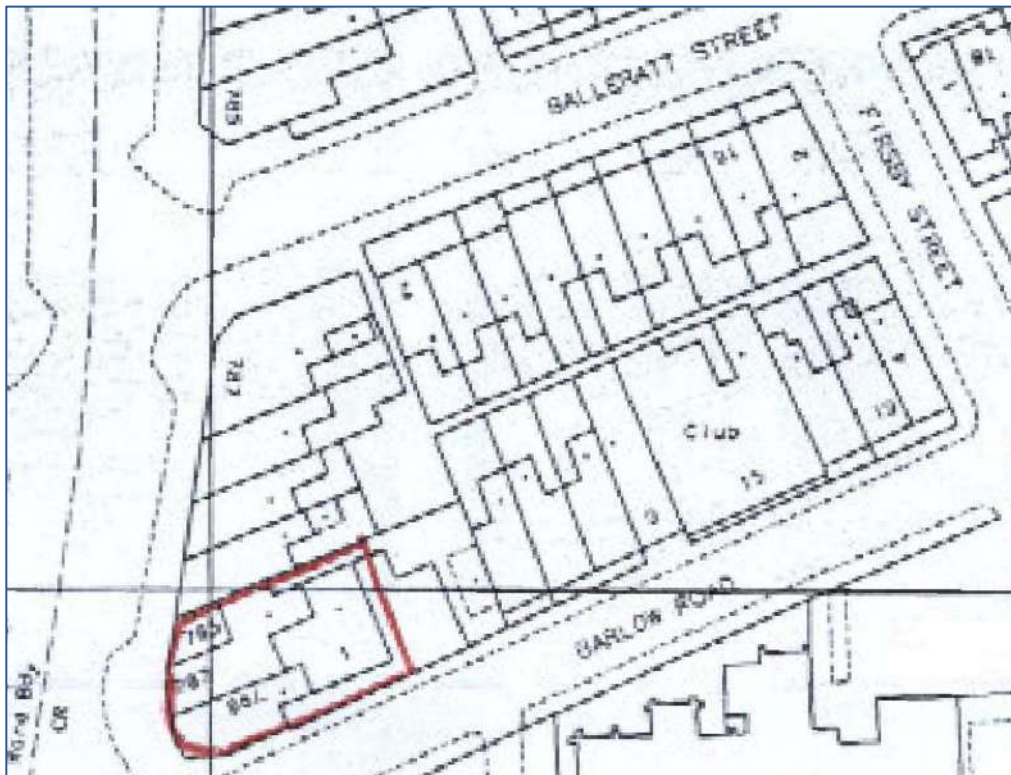


Figure 1 – Site Location Plan.



The Existing Dwelling

The existing structure consists of a two-storey commercial property with the ground floor used as a convenience store and the first floor as habitable accommodation.

The original floor plan of the existing dwelling is shown in figure 2 below (this figure is shown to scale on the submitted planning drawing named M19 3DL - A002 EXIST FLOOR PLANS).

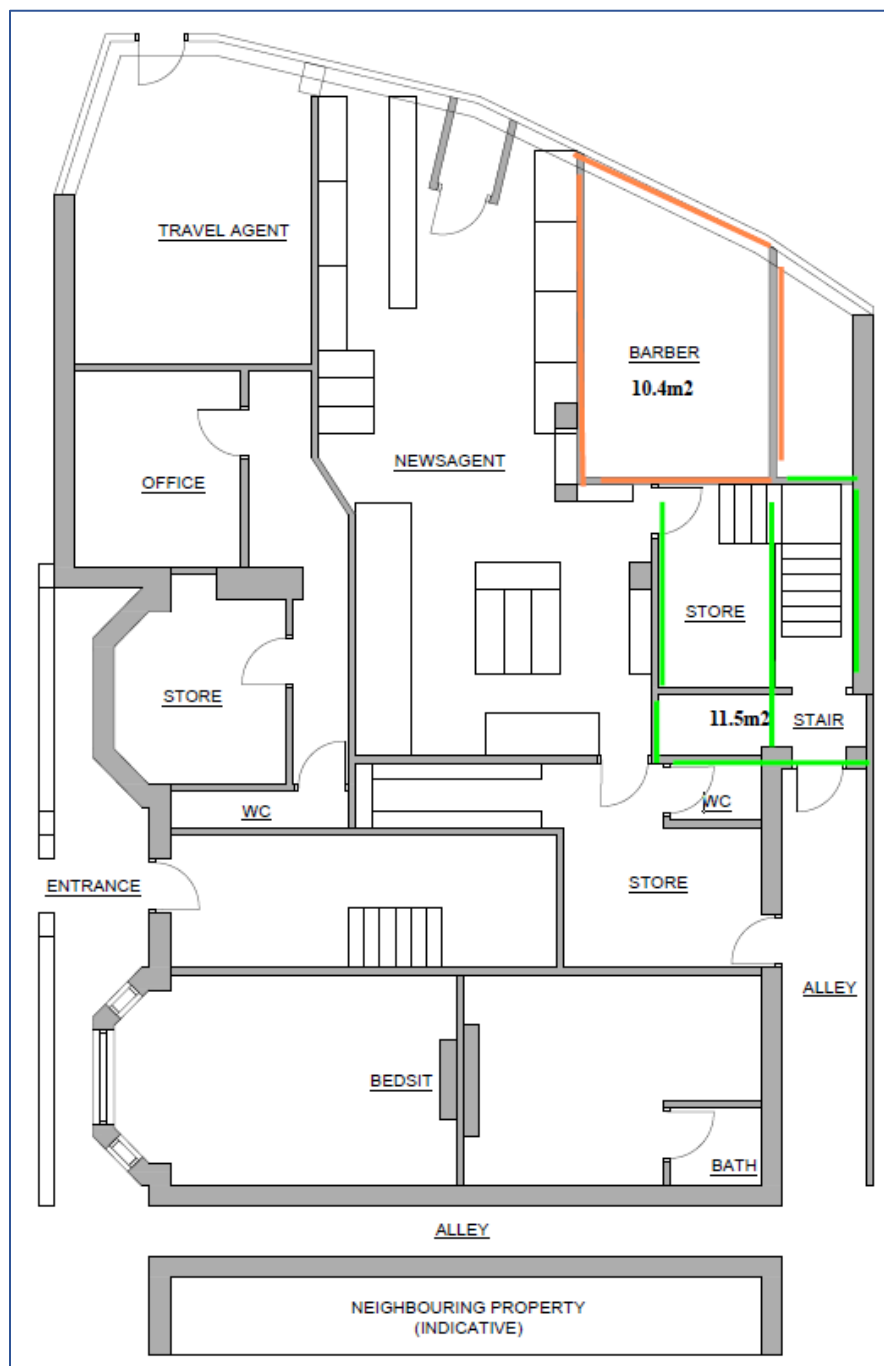


Figure 2 – Existing Floor Plan



The Proposed Dwelling

The proposed structure will consist of an open plan shopping area which will see the previous barbershop shown in figure 1 removed and form part of the newly expanded newsagent. There will be an aesthetic amendment to the external façade of the shop which is shown in Appendix A. the existing rear staircase will also be altered to accommodate extra floor space.

Apart from the amended staircase there will be no amendments to the first floor. The extension which is being proposed is approximately 52m². The exact measurements can be scaled from the planning drawing named M19 3DL - A003 PROP GA.

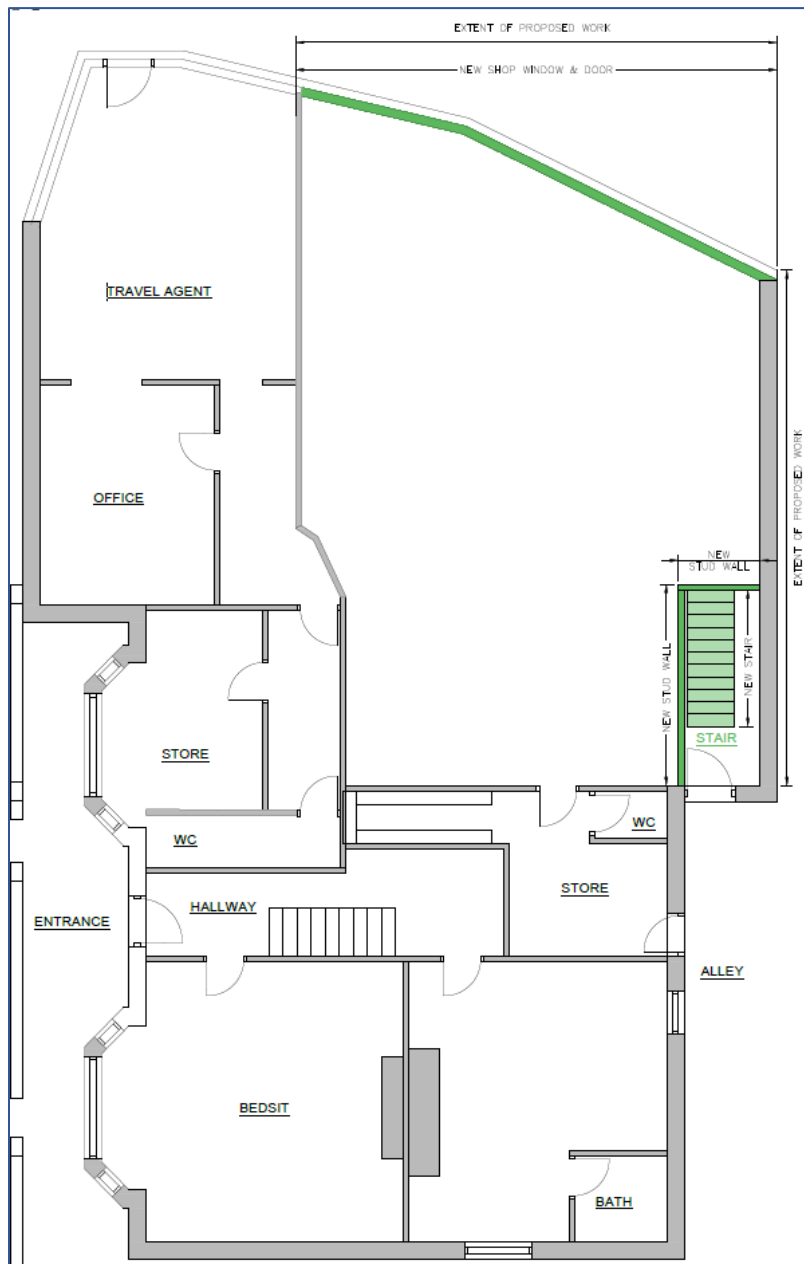
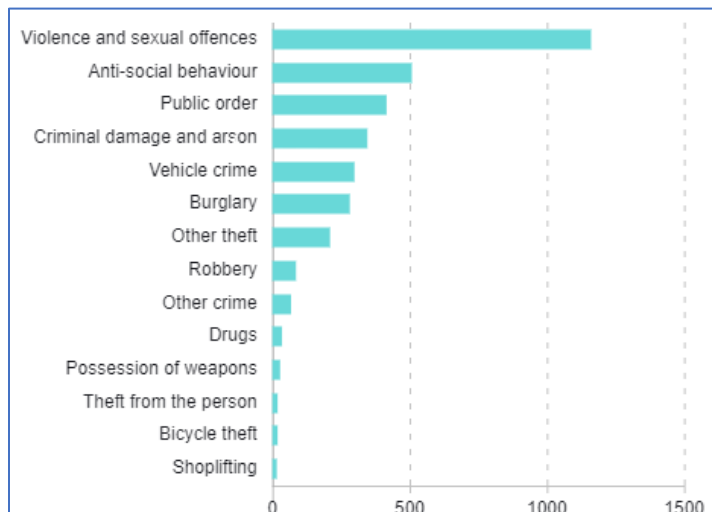


Figure 3 – Proposed Site Layout



Local Crime Statistics

The images below depict the crime statistics from 2017-2020 within the postcode M19 3DL. The most committed crime being violence and sexual offences and the least being Shoplifting.



Reference: <https://www.police.uk/pu/your-area/greater-manchester-police/gorton-south/?tab=Statistics>

Type	Total	Percentage
Other theft	213	6%
Violence and sexual offences	1164	32.9%
Drugs	38	1.1%
Criminal damage and arson	349	9.9%
Anti-social behaviour	511	14.5%
Burglary	285	8.1%
Possession of weapons	31	0.9%
Public order	419	11.9%
Robbery	89	2.5%
Shoplifting	19	0.5%
Theft from the person	22	0.6%
Vehicle crime	302	8.5%
Other crime	71	2%
Bicycle theft	22	0.6%

The images below depict the crime statistics from 2019-2020 within the postcode M19 3DL. The most committed crimes being violence and sexual offences and drugs.



Type	Total	Percentage
Other theft	1	33.3%
Violence and sexual offences	1	33.3%
Drugs	1	33.3%



Construction Design Management Residual Risk Register

ID	Risk	Detail	Rating (1-5)	Mitigation(s)
1	Working at height	When erecting the steelwork there will be a requirement for the contractors to work at height. There must be allowances in place to ensure the contractors are operating safely at all times.	3 - Med	- No lone working on site. - Manual support at the base of all ladders when in operation. - Appropriate Personal Protective Equipment (PPE to be worn at all times)
2	Potential encounter with buried services	It is not currently envisaged that any buried services are within the construction area. But, there is a possibility for a potential encounter when excavating for new strip footings.	2 – Low	- A Ground Penetrating Radar (GPR) Survey of the area recommended but the commission of this is at the Clients discretion.
3	Quality of existing foundations	There is a possibility that the existing foundations and ground bearing slabs have detreated and structural integrity has been compromised.	2 – Low	- If foundations appear to be compromised. Stop works and contact a chartered structural engineer.
4	Support to existing walls during construction	There will be the requirement for existing load bearing walls to be temporarily removed during construction.	4 – High	- The appointed competent Contractor to ensure all load bearing walls are properly supported before and during their removal.



Risk Factors

The typical security risks for a development of this nature are:

- Theft Shoplifting
- Criminal Damage
- Burglary
- Anti-Social Behaviour
- Theft of/from parked vehicles
- Neighbour Disputes

Design Layout & Appraisal

As mentioned in the introduction, this application has been submitted to allow the Client to refurbish the existing convenience store and extend it into the neighboring unit.

The Newsagent is located on the main road known as Stockport Road and is part of an existing row commercial units.

Positive Aspects of Proposal

The proposed Newsagent will all front onto the main road, resulting in good levels of surveillance of and from the existing units/dwellings and parked vehicles.

The large windows on the front elevation of the unit will maximize surveillance opportunities over the street and parked vehicles.

There will be a new CCTV system both inside and outside the Newsagents. To provide sophisticated surveillance of the area 24/7.



Physical Security

The following standards and features will be required for this development to demonstrate a level of physical security acceptable to design for security.

Doors

Front and rear doors to the wine bar must be compliant with and certified to BS PAS 24, WCL2, or LPS 1175 SR2.

Front door should have fixed external handles or split spindles meaning access is only possible with a key.

Roller shutters to be installed to cover the front elevation when the bar is not open to the public.

Windows

Windows must be compliant with and certified to BS 7950.

Glazing

Glazing to a height of 2.4m (or if otherwise accessible) must incorporate at least one pane of glass with a 0.8mm PVB interlayer (e.g 6.8mm laminated glass), or a glass rated as P2A under EN356 (or latest). The remaining pane in a double-glazed unit may be toughened glass.

Alarms

The unit should have an intruder alarm to comply with either:

- BS EN 50131 and PD 6662 (or latest) for wired systems
- BS 6799 (or latest) for wireless systems

Alarms should be lined to contacts on all external doors and PIR detectors should cover all ground floor rooms and windows.



External Features

Landscaping

In vulnerable locations any proposed planting should not exceed 1m in height and tree canopies should fall no lower than 2m from the ground. This is in order to allow people to see their surroundings better, make rational choice of routes and eliminate hiding places. A maintenance agreement should stipulate that these planting dimensions are adhered to.

Loose surface material in the publicly accessible areas should be avoided. These can be used as a potential missile against people or premises.

Planting must be avoided that will aid climbing over boundary treatments.

Lighting

Lighting to all roads/footpaths and communal parking areas should be adoptable standards to comply with BS5489

External lighting must be provided to the front and rear operated by photoelectric cell/daylight sensor.

Fittings should produce 'white light' as opposed to yellow or orange light. Metal halide should be used, as these offer superior color rendition.

All lighting will be positioned to avoid climbing aids.

Communal Area

Any public open space should be subject to an effective maintenance contract. This should ensure that all damage is rectified in a timely manner.

Management and Maintenance

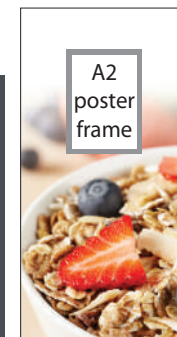
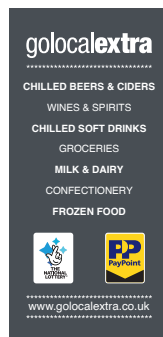
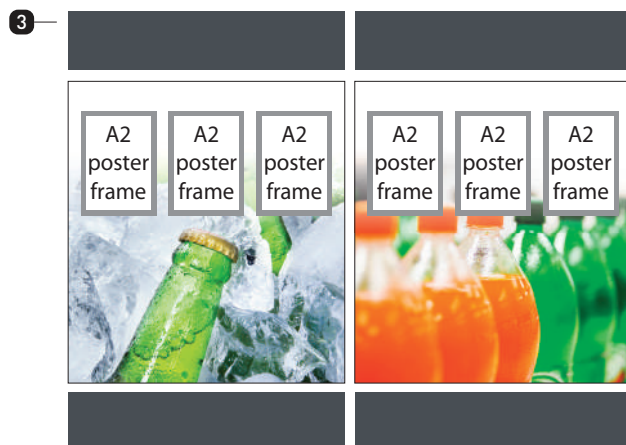
The upkeep and development over its lifetime can be crucial to the level of security and safety within. Aspects of a development, which are left to deteriorate, have the potential to attract further crime – a process known as the 'broken window theory'. A maintenance plan should be drawn up to address issues such as:

- Litter/graffiti removal
- Repair to communal areas
- Trimming and pruning to any future shrubs.



Appendix A: Façade Details

Proposed signage



1. ACM sign tray
CENTRAL SPLIT TO WRAP BUILDING
C/W vinyl graphics mounted to face.
feature logo pod c/w 3d metal stand off letters
DEEP LOGO RETURNS TO SPAN FASCIA ANGLE
2No led troughlight fitted to fascia
BESPOKE STEEL BRACKETS TO LEFT SIDE

2. ACM flat panels c/w vinyl website
fitted to existing shutter boxes

3. Window graphics
printed graphics fitted EXTERNALLY
opening hours fitted INTERNALLY
A2 external poster frames

Please check all details such as spellings, tel. no.
etc carefully. Sign UK cannot accept liability for
errors found after approval has been given

Existing signage



Proposed signage



Rev.	Item	Description	Date
002			00/00/19
003			
004			
005			

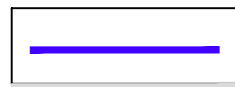


Appendix B: Planning Drawings



CLIENT TO CONFIRM OWNERSHIP BOUNDARY

0 50 100
SCALE 1 : 1250

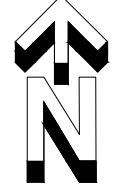


APPLICANTS LAND OWNERSHIP
BOUNDARY 0m² (0.000 Ha)
NO ADDITIONAL LAND OWNED

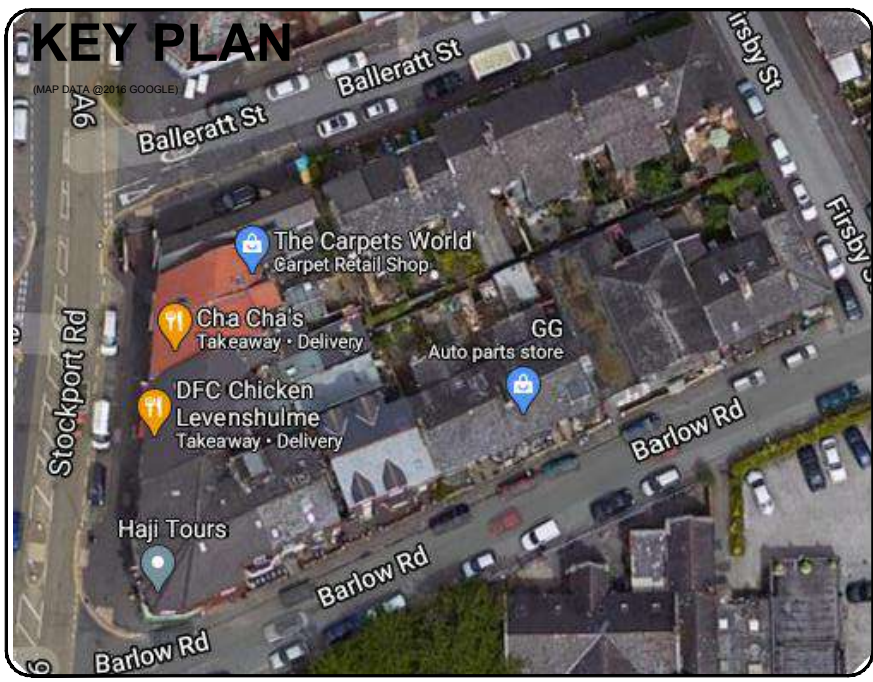


APPLICANTS PROPOSED SITE
TBCm²

P1	ISSUED FOR INFORMATION	15.10.20
REVISION	DESCRIPTION	DATE
M19 3DL / A001 SITE LOCATION PLAN (797 STOCKPORT ROAD)		
DRAWN RJL	DATE 15.10.20	SCALE 1:1250 AT A4

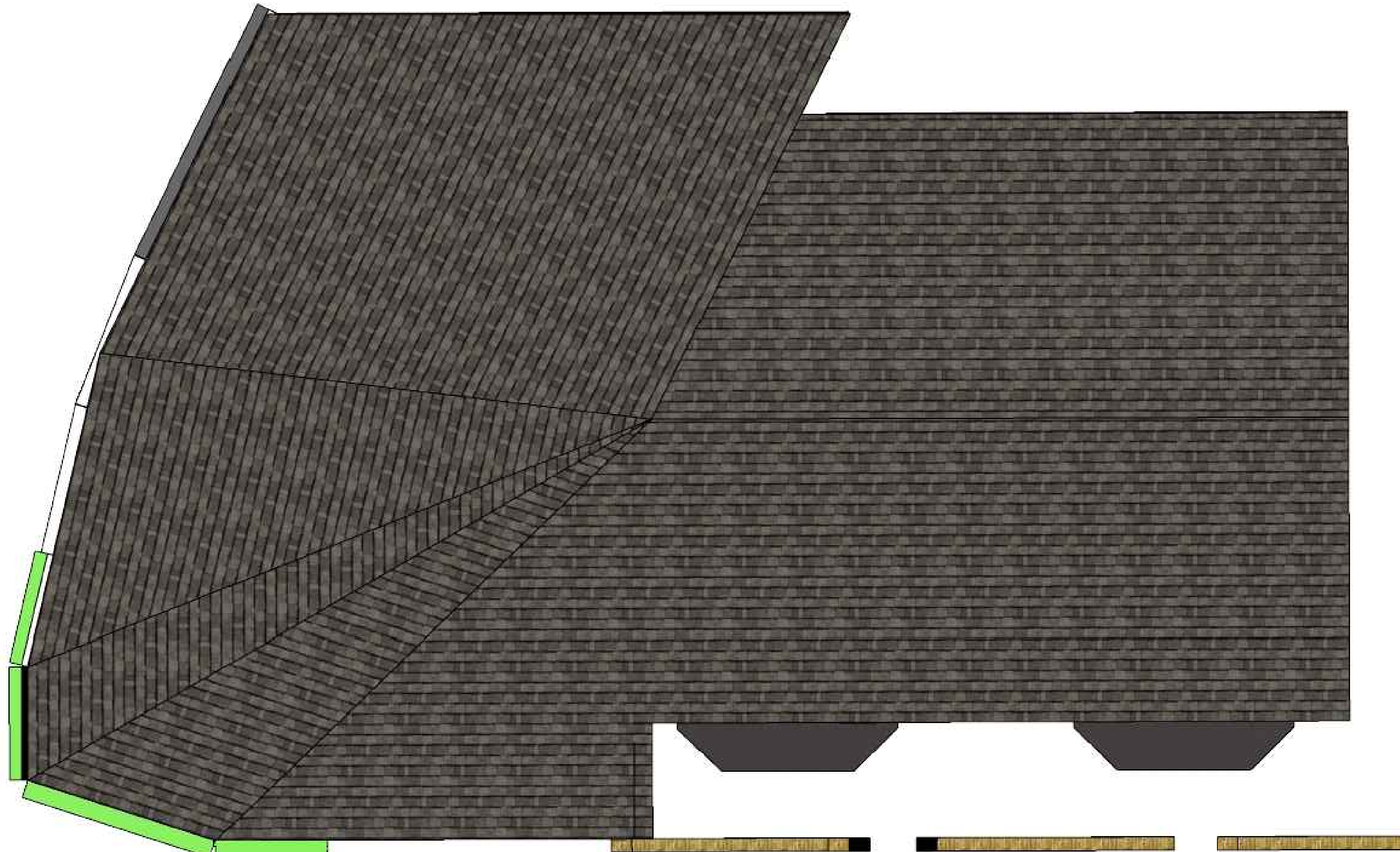


drawing to scale @ A0



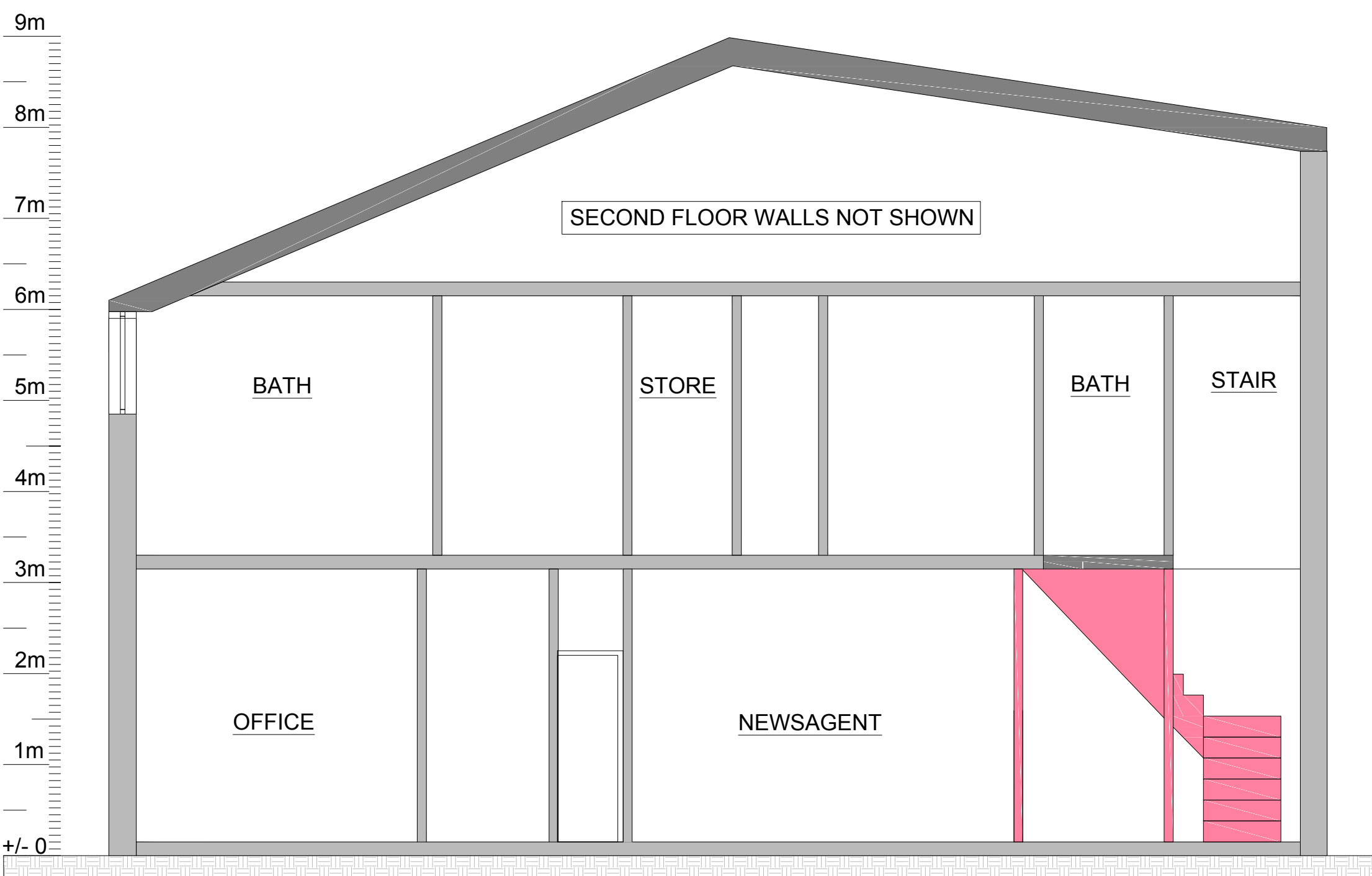
- GENERAL NOTES:
- DO NOT SCALE FROM DRAWINGS
 - DIMENSIONS TO BE CHECKED ON SITE
 - DIMENSIONS IN MILLIMETERS & LEVELS IN METERS UNLESS NOTED OTHERWISE
 - ALL WORKS MUST BE CARRIED OUT TO THE SATISFACTION OF LOCAL AUTHORITY BUILDING CONTROL OFFICER IN ACCORDANCE WITH CURRENT BUILDING REGULATIONS
 - DRAWINGS PRODUCED FOR THE PURPOSE BUILDING REGULATIONS APPROVALS AND DO NOT CONSTITUTE FULL WORKING DRAWINGS
 - THIS DRAWING MAY NOT BE COPIED WITHOUT WRITTEN CONSENT
- KEY (GENERAL)
- | | | | |
|---|--|--|--------------------------------|
| | EXTENT OF EXISTING | | SECTION / ELEVATION |
| | EXTENT OF NEW WORK AND/OR ALTERATION | | DETAIL (SEE DETAIL SHEET) |
| | EXTENT OF DEMOLITION AND/OR ALTERATION | | DENOTES DOOR WITH VISION PANEL |
| ALL DIMENSIONS WITH *** SUFFIX ARE APPROXIMATE AND NEED TO BE SITE MEASURED | | | |

NOTE: THE PROPOSED WORK SCOPE DOES NOT AFFECT THE ROOF PLAN



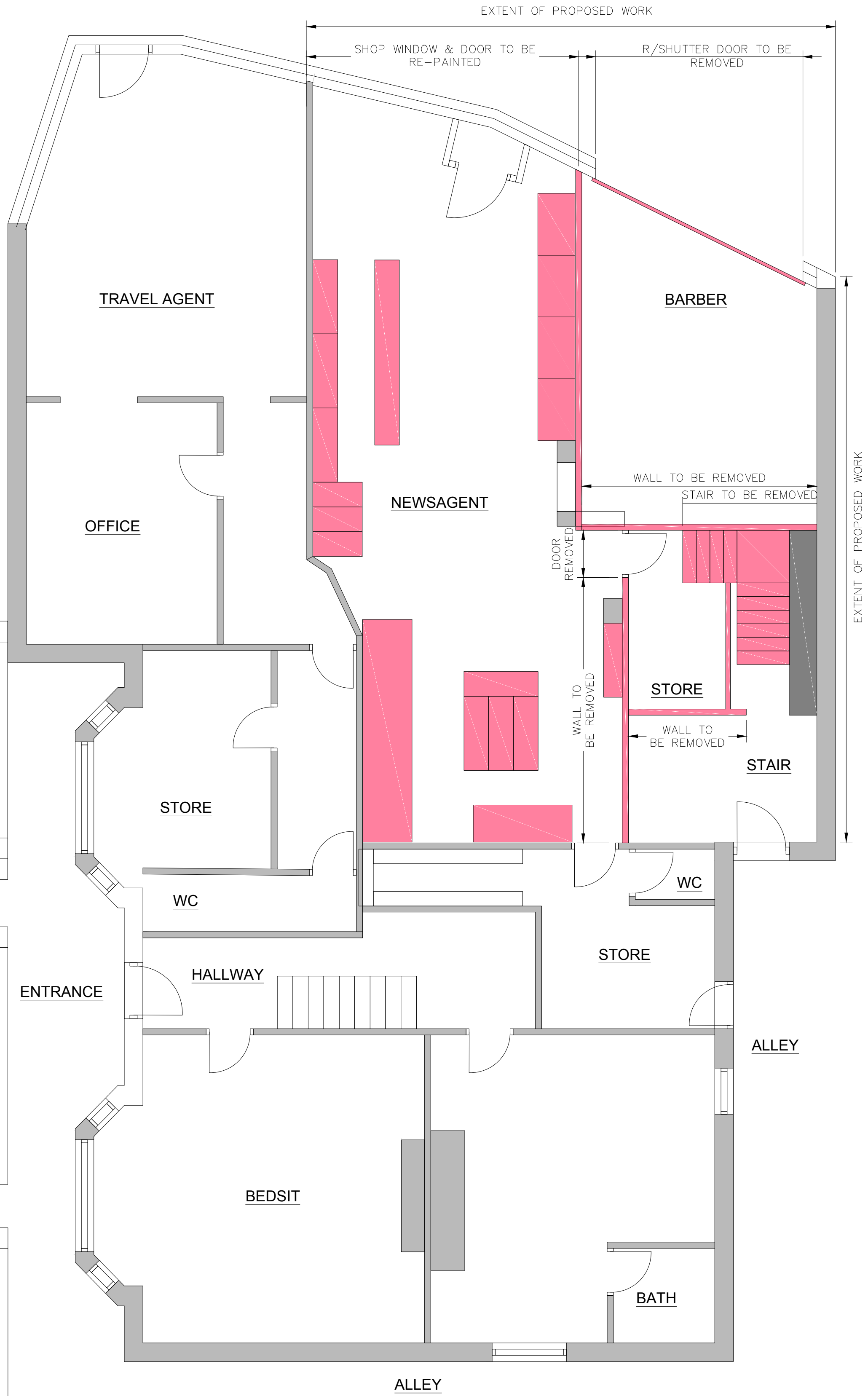
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SCALE 1:50



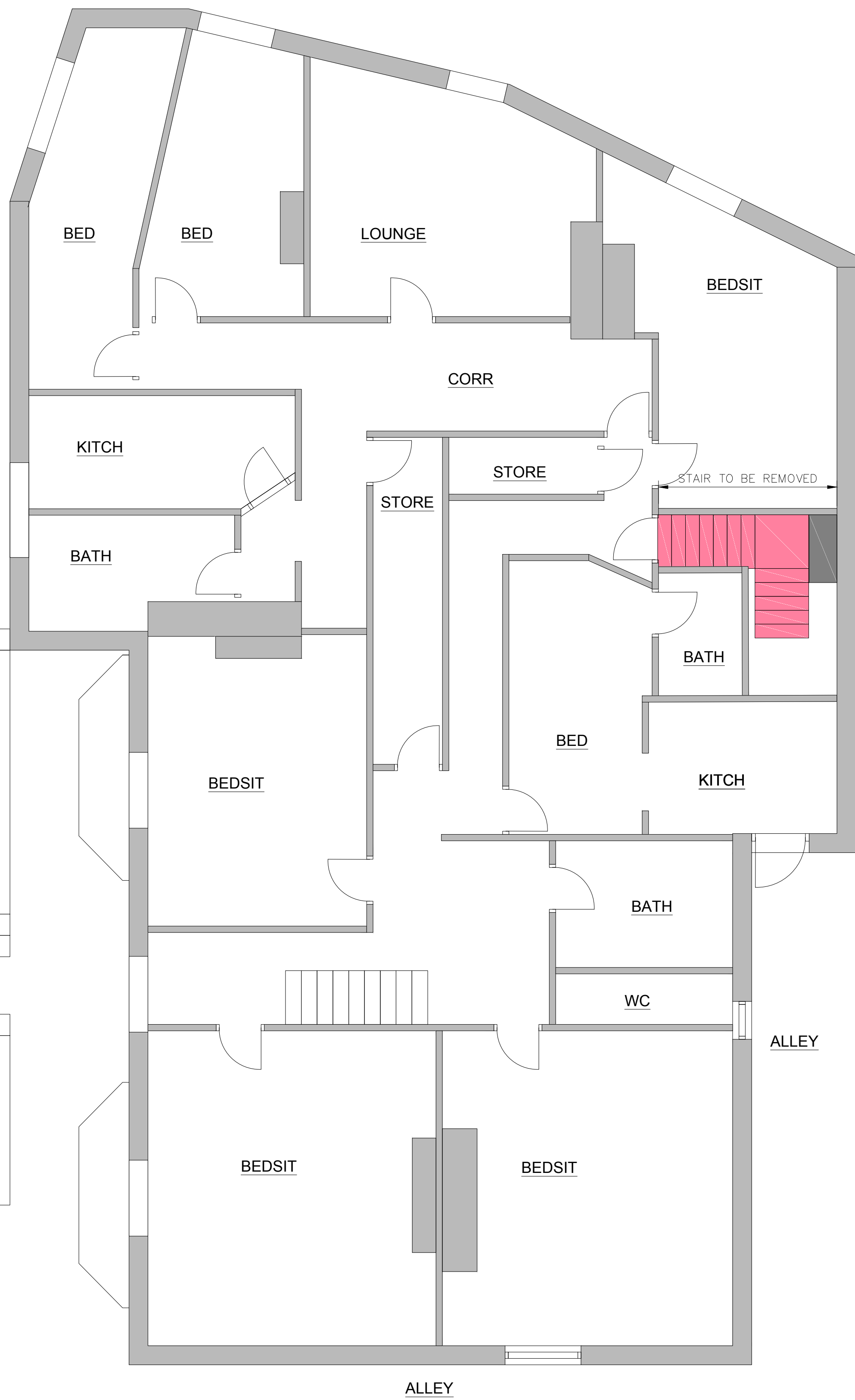
EXISTING: SECTION A-A

SCALE 1:50



EXISTING/ DEMOLITION: GROUND FLOOR PLAN

SCALE 1:50



EXISTING/ DEMOLITION: FIRST FLOOR PLAN

SCALE 1:50



EXISTING: FRONT ELEVATION

SCALE 1:50

NOTE: THE PROPOSED WORK SCOPE DOES NOT AFFECT THIS ELEVATION



EXISTING: BACK ELEVATION

SCALE 1:50

NOTE: THE PROPOSED WORK SCOPE DOES NOT AFFECT THIS ELEVATION



EXISTING: SIDE ELEVATION

SCALE 1:50

**ISSUED FOR
PLANNING**

ARCHITECTURAL REFERENCE DRAWINGS

M19 3DL/A001	- SITE LOCATION PLAN (A4)
M19 3DL/A002	- EXIST/ DEMO GENERAL ARRANGEMENT (A0)
M19 3DL/A003	- PROPOSED GENERAL ARRANGEMENT (A0)

0 1000 2000 3000 4000 5000 mm

SCALE 1:50

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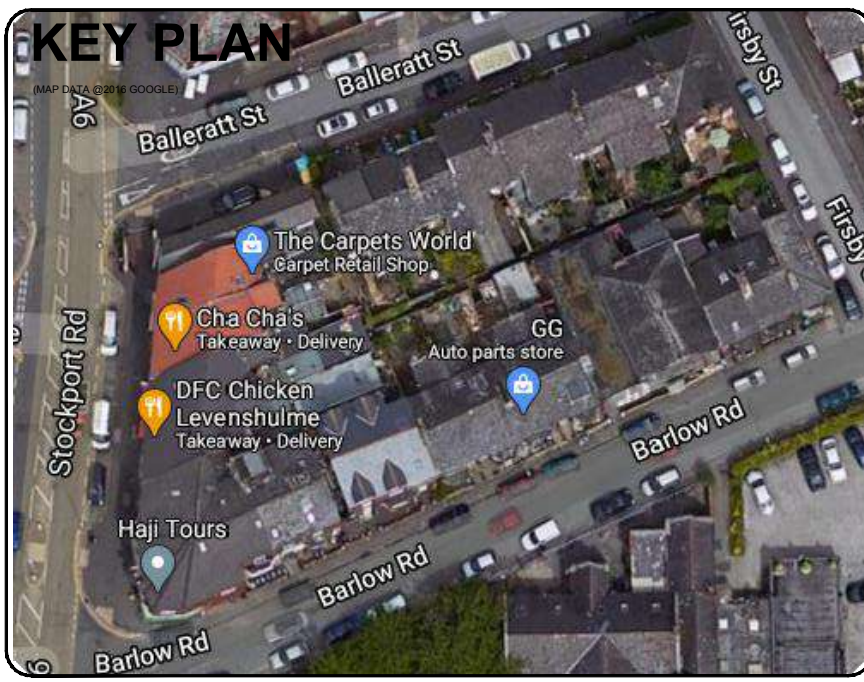
PROJECT TITLE:
NEWSAGENT EXTENSION & SIGNAGE

DRAWING TITLE:
EXIST/ DEMO GENERAL ARRANGEMENT

ADDRESS:
797 STOCKPORT ROAD, LEVENSHULME, MANCHESTER, M19 3DL

M19 3DL / A 002

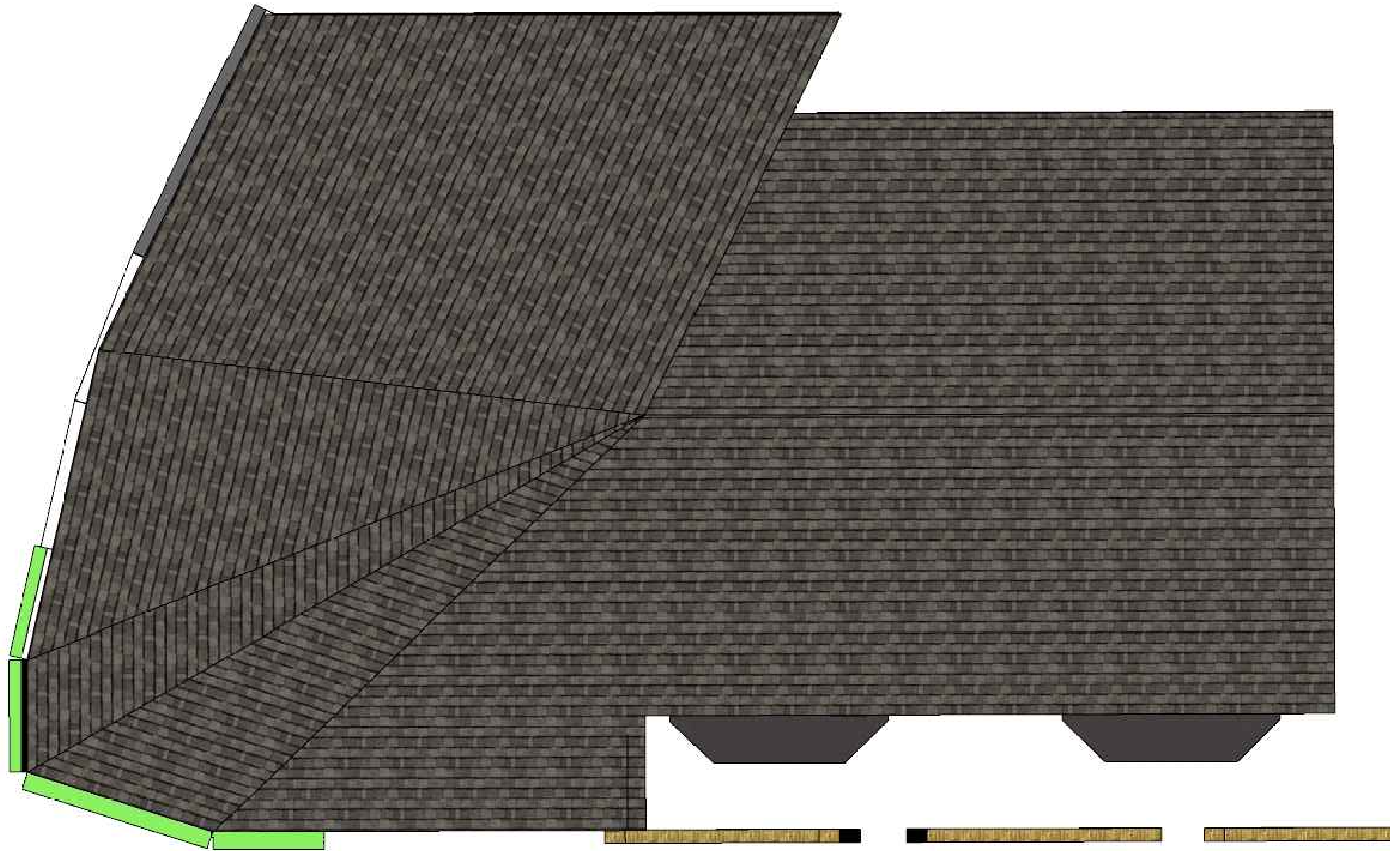
DATE: 02-10-2020 REV: A



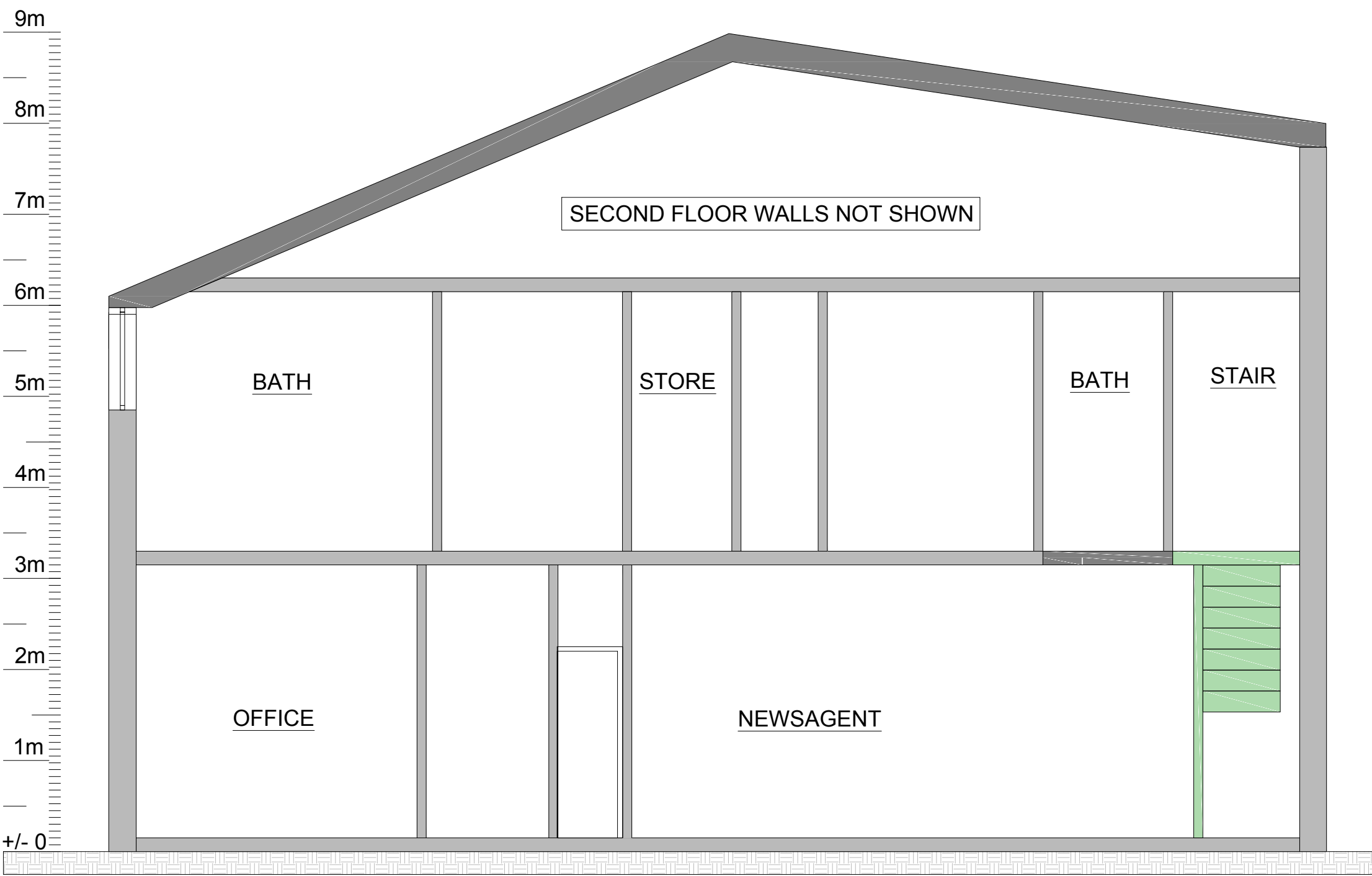
- GENERAL NOTES:
- DO NOT SCALE FROM DRAWINGS
 - DIMENSIONS TO BE CHECKED ON SITE
 - DIMENSIONS IN MILLIMETERS & LEVELS IN METERS UNLESS NOTED OTHERWISE
 - ALL WORKS MUST BE CARRIED OUT TO THE SATISFACTION OF LOCAL AUTHORITY BUILDING CONTROL OFFICER IN ACCORDANCE WITH CURRENT BUILDING REGULATIONS
 - DRAWINGS PRODUCED FOR THE PURPOSES OF BUILDING REGULATIONS APPROVALS AND DO NOT CONSTITUTE FULL WORKING DRAWINGS
 - THIS DRAWING MAY NOT BE COPIED WITHOUT WRITTEN CONSENT

- KEY (GENERAL)
- EXTENT OF EXISTING
 - EXTENT OF NEW WORK AND/OR ALTERATION
 - EXTENT OF DEMOLITION AND/OR ALTERATION
 - SECTION / ELEVATION
 - DETAIL (SEE DETAIL SHEET)
 - DENOTES DOOR WITH VISION PANEL
 - ALL DIMENSIONS WITH *** SUFFIX ARE APPROXIMATE AND NEED TO BE SITE MEASURED

NOTE: THE PROPOSED WORK SCOPE DOES NOT AFFECT THE ROOF PLAN



PROPOSED: ROOF PLAN
SCALE 1:50



PROPOSED SECTION A-A
SCALE 1:50

NOTE: THE PROPOSED WORK SCOPE DOES NOT AFFECT THIS ELEVATION

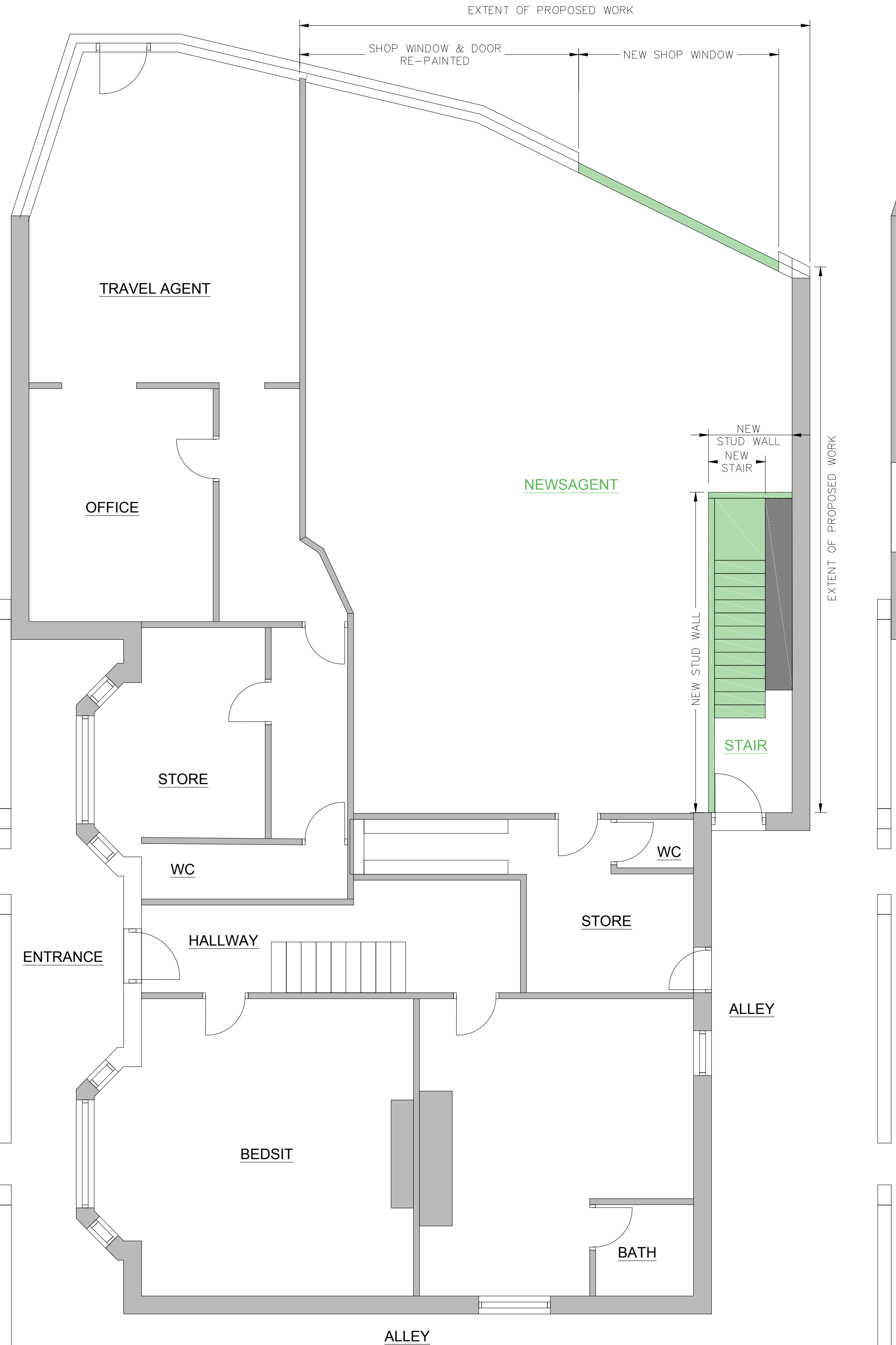


PROPOSED: BACK ELEVATION
SCALE 1:50

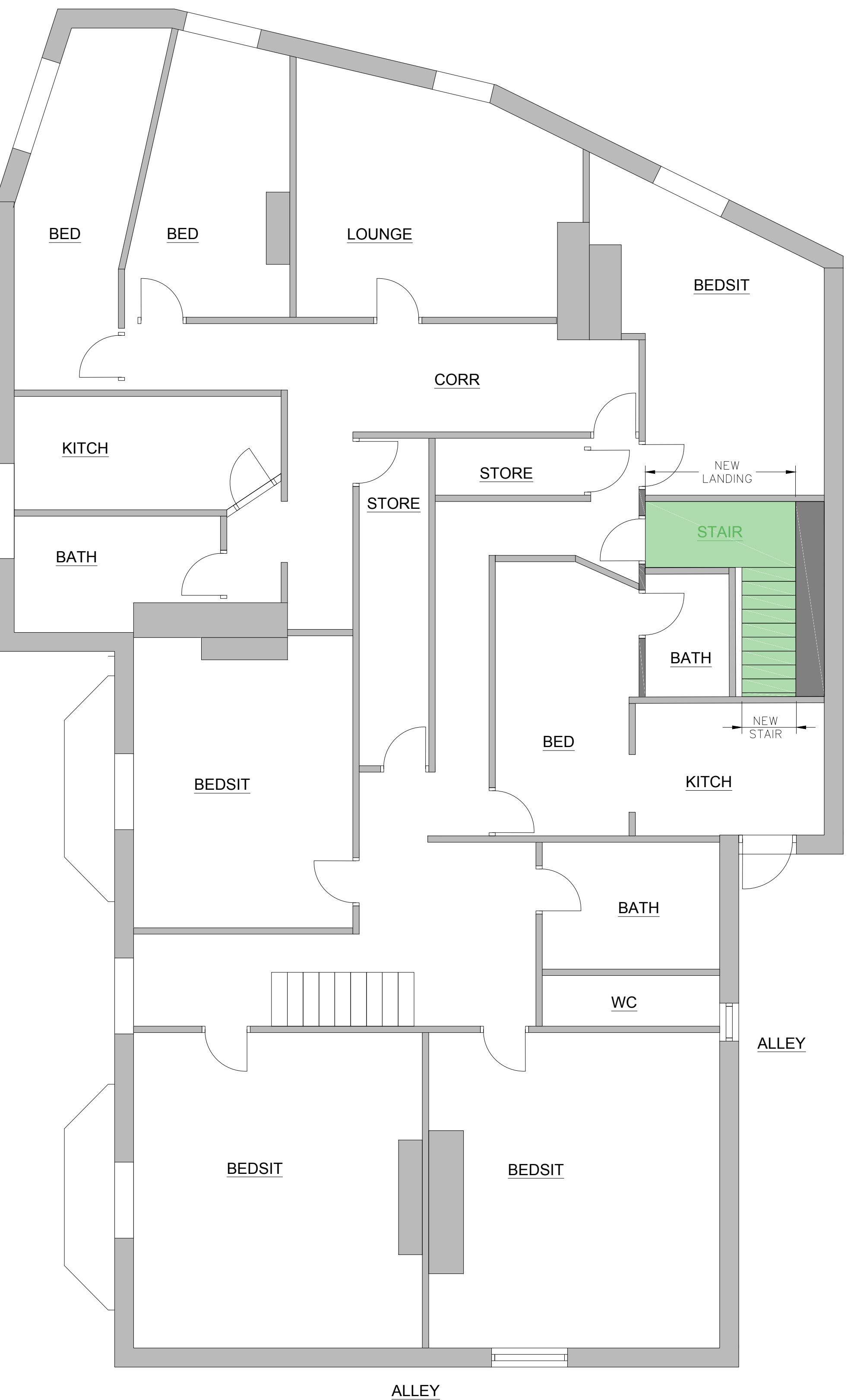
NOTE: THE PROPOSED WORK SCOPE DOES NOT AFFECT THIS ELEVATION



PROPOSED: SIDE ELEVATION
SCALE 1:50



PROPOSED: GROUND FLOOR PLAN
SCALE 1:50



PROPOSED: FIRST FLOOR PLAN
SCALE 1:50



PROPOSED: FRONT ELEVATION
SCALE 1:50

ISSUED FOR PLANNING

ARCHITECTURAL REFERENCE DRAWINGS

- M19 3DL/A001 - SITE LOCATION PLAN (A4)
- M19 3DL/A002 - EXIST/ DEMO GENERAL ARRANGEMENT (A0)
- M19 3DL/A003 - PROPOSED GENERAL ARRANGEMENT (A0)

0 1000 2000 3000 4000 5000 mm
SCALE 1:50

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PROJECT TITLE:
NEWSAGENT EXTENSION & SIGNAGE

DRAWING TITLE:
PROPOSED GENERAL ARRANGEMENT

ADDRESS:
797 STOCKPORT ROAD, LEVENSHULME, MANCHESTER, M19 3DL

M19 3DL / A 003

DATE: 02-10-2020 REV: A

Job No.	Sheet No.	Rev
	01	P01
Member Location		
Drg. Ref. MD19 3DL/A 003 - FORENA		
Made by	Date	Chd.
	15/12/2020	

Job title

M19 3DL - 797 Stockport Rd

STRUCTURAL CALCULATIONS FOR REFURBISHMENT AT M19 3DL 797 STOCKPORT RD. MANCHESTER

 Calculation sheet	Job No.	Sheet No.	Rev
		01	P01
	Member Location		
Job title	Drg. Ref. MD19 3DL/A 003 - FORENA		
M19 3DL - 797 Stockport Rd	Made by	Date 15/12/2020	Chd.

Introduction

The following calculations cover the required structural elements required to form the proposed building alteration as shown on architectural drawings MD19 3DL/A 003 and following. Many assumptions have been made to allow for an initial sizing of the supporting members and contractor to verify and confirm those when necessary.

Contents

Assumptions

Loadings

Design

Appendix

Loadings

All calculations are in accordance with the relevant Euro-codes and UK annexes and standards, inclusive of Approved Document A.

Eurocode 0 BS EN 1990: Basis of structural design
 Eurocode 1 BS EN 1991: Actions on structures
 Eurocode 2 BS EN 1992: Design of concrete structures
 Eurocode 3 BS EN 1993: Design of steel structures
 Eurocode 5 BS EN 1995: Design of timber structures
 Eurocode 6 BS EN 1996: Design of masonry structures

Vertical loads

Roof (Gk)	Roof tiles, battens and felt	0.55 kN/m ²
	Timber , insulation	0.30 kN/m ²
	Services (no ceiling)	0.15 kN/m ²

Tot. 1.2 kN/m²

Roof pitch = 23deg -> roof tot load = **1.32kN/m²**

Floor (Gk)	Timber joist + timber board	0.5 kN/m ²
Roof (Qk)	Snow	0.60 kN/m ²

Tot. **0.6 kN/m²**

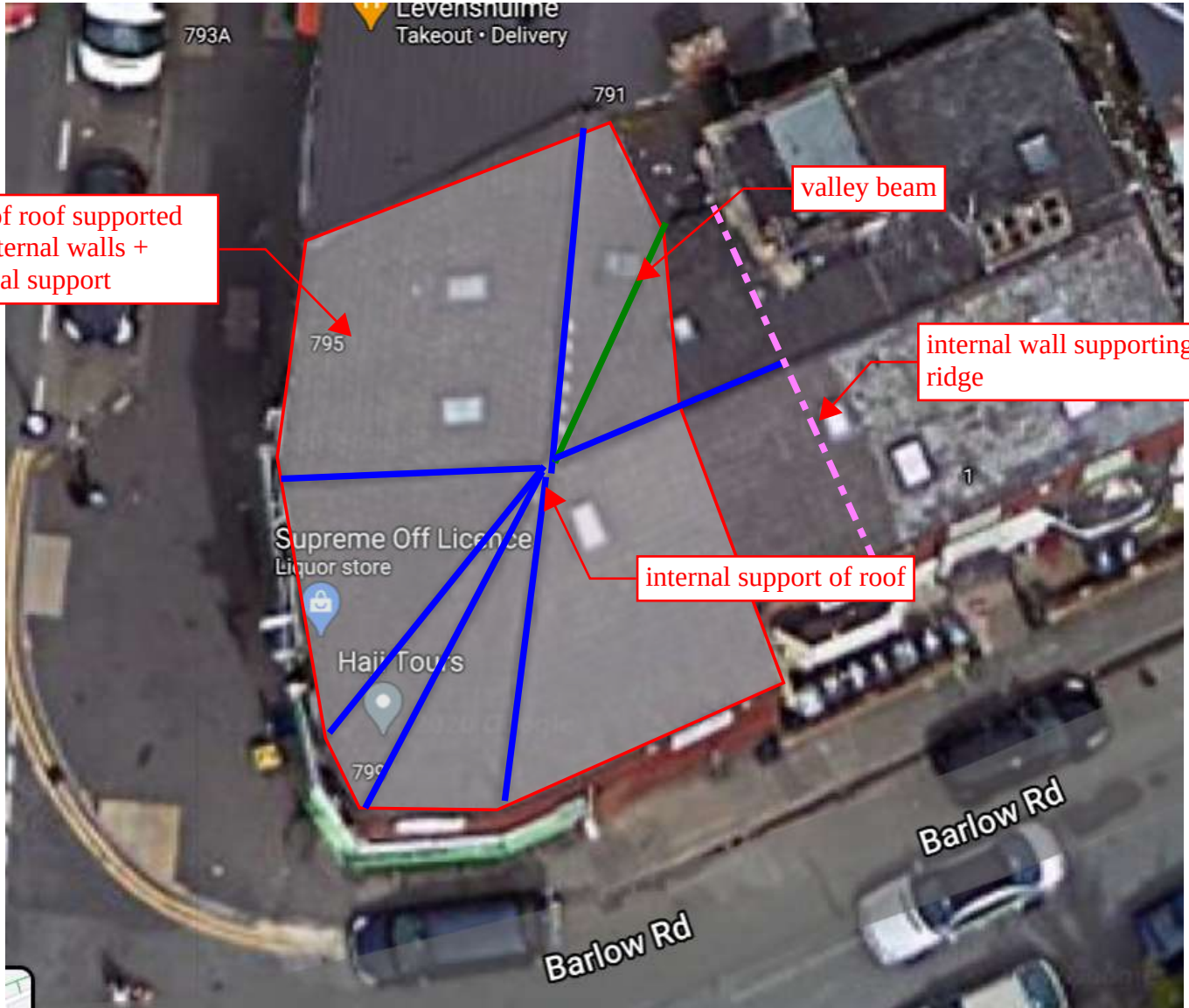
Floor (Qk)	Residential	1.5 kN/m ²
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Tot. **1.5 kN/m²**

Floor (Qk)	Storage(Ceil)	0.9 kN/m ²
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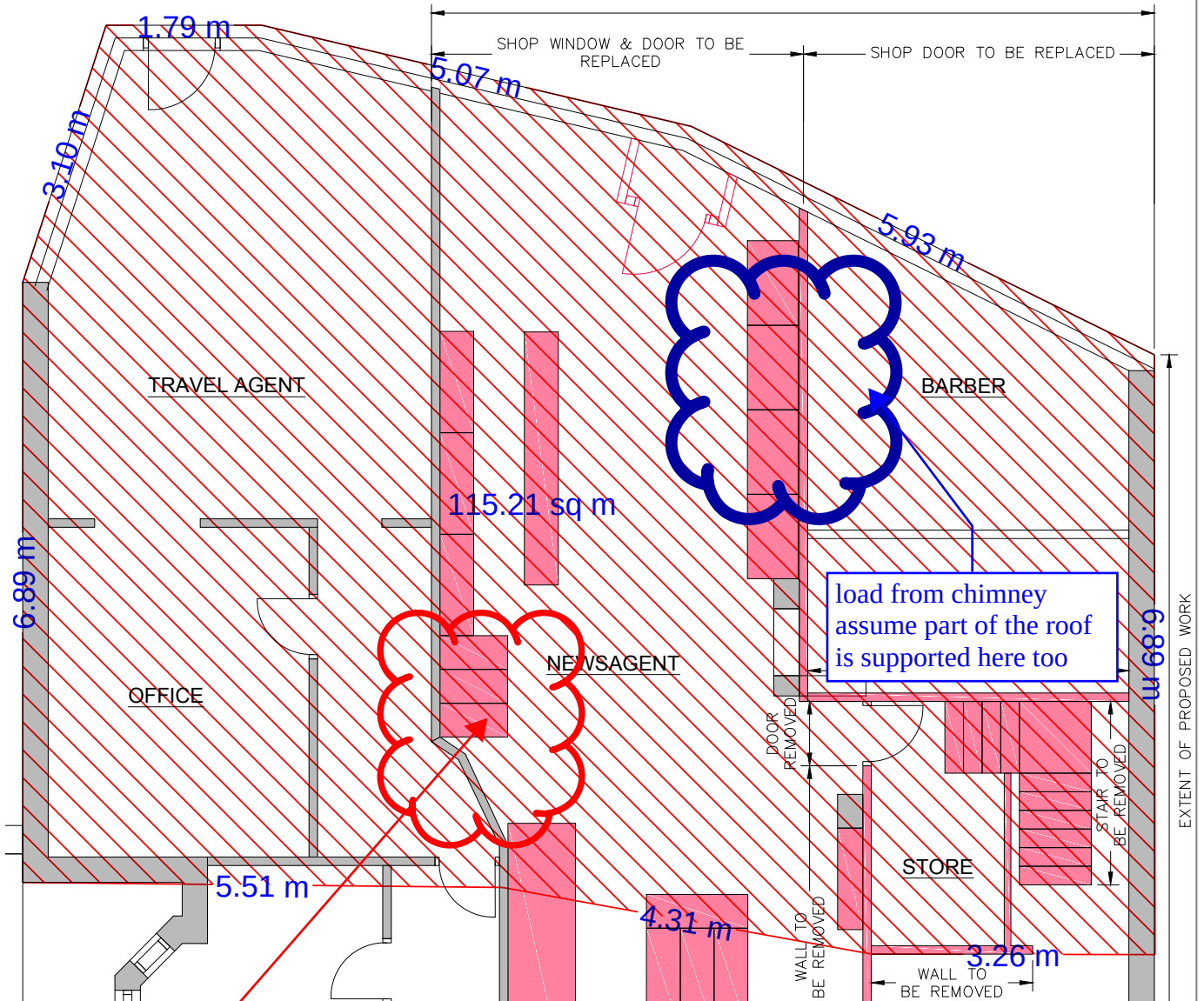
Tot. 0.9 kN/m²

Job No.	Sheet No.	Rev
	02	P01
Member Location		
Job title		
M19 3DL - 797 Stockport Rd		
Drg. Ref.	MD19 3DL/A 003 - FORENA	
Made by	Date	Chd.
	15/12/2020	



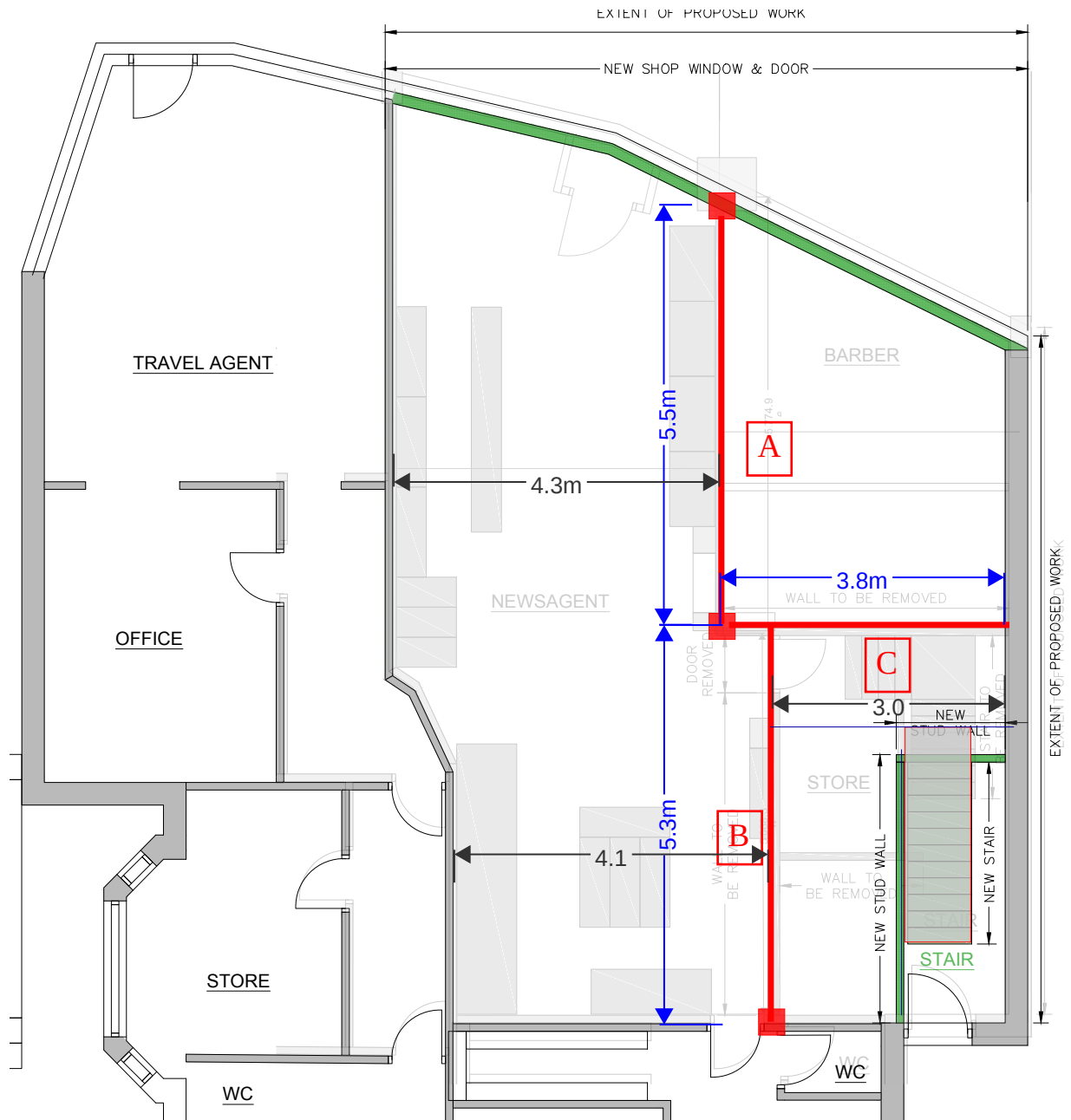
Important: no information are yet available for the roof - from the review of maps images and architectural drawings it is assumed that 60% of the roof load is carried on internal supports

Job No.	Sheet No.	03	Rev	P01
Member Location				
Job title				
M19 3DL - 797 Stockport Rd				
Drg. Ref. MD19 3DL/A 003 - FORENA				
Made by		Date	15/12/2020	Chd.



EXTENT OF PROPOSED WORK

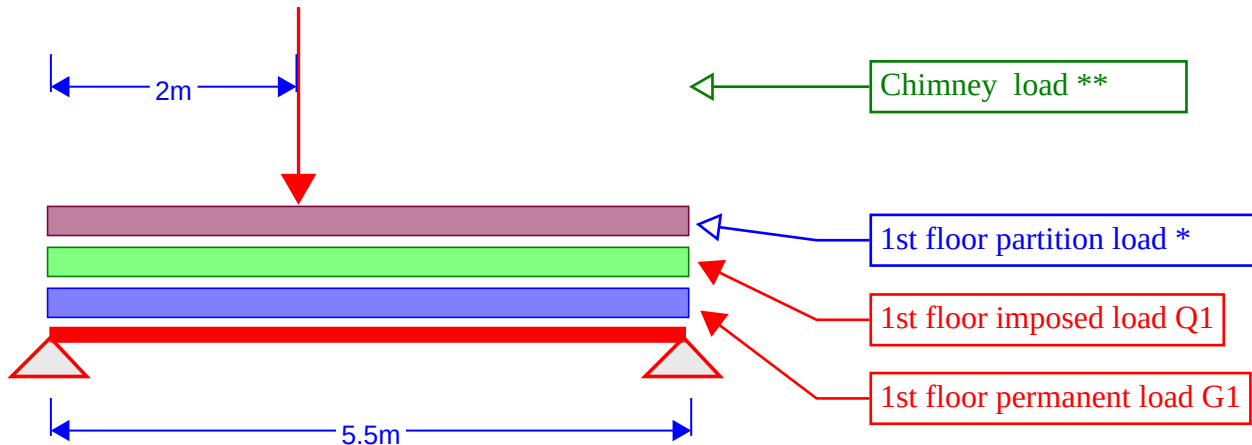
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Member Location		
Job title	Drg. Ref. MD19 3DL/A 003 - FORENA	
M19 3DL - 797 Stockport Rd	Made by	Date 15/12/2020 Chd.



Job title

M19 3DL - 797 Stockport Rd

Beam A



* Partition load comprises of:

S/W of masonry wall (excluded chimney) G2
2nd floor permanent load G3
2nd floor imposed load Q2

** Chimney load assumed:

S/W of masonry G4
Roof permanent load G5
Roof imposed load Q3

Loaded width 1st floor = 4.1m

Loaded area roof level = 15m²

assumed tbc

input to TEDDS the following

s/w of beam automatically calculated

G1 = 2.05 kN/m

G2 = 11.7 kN/m (consider 2.4 + 2.0m wall)

G3 = 2.05 kN/m

G4 = 50kN (consider 1.8 * 0.4 chimney stack 2 storey above beam)

G5 = 20kN

assumed tbc

assumed tbc

Q1 = 6.2 kN/m

Q2 = 6.2 kN/m

Q3 = 9 kN

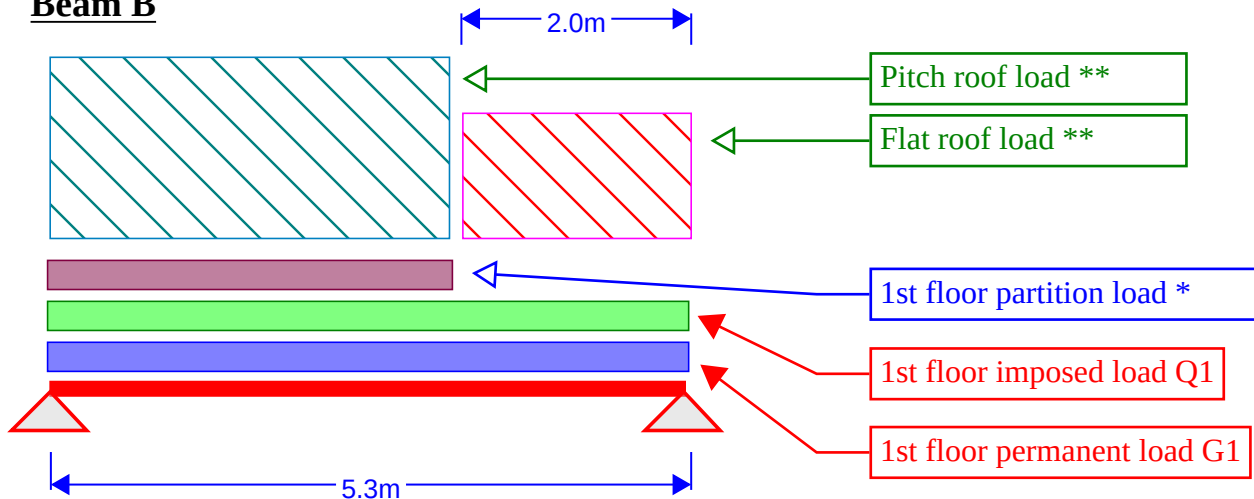
Depending on size of chimney (to be confirmed with a survey) use one of the followings:

a) min 2no. **305x165x46 UB** - side by side with spacers at 600crs

b) min 1 no. **356x171x67 UB**

Job title
M19 3DL - 797 Stockport Rd

Beam B



* Partition load comprises of:

S/W of masonry wall (UDL all beam) G2
2nd floor permanent load G3
2nd floor imposed load Q2

** Floor load assumed:

S/W of masonry G4
Roof permanent load G5
G6(flat)
Roof imposed load Q3
Snow Drift Q4

Loaded width 1st floor = 3.5m

Loaded width roof = 3.5

Loaded width flat roof = 1.5 m

_input to TEDDS the following

s/w of beam automatically calculated

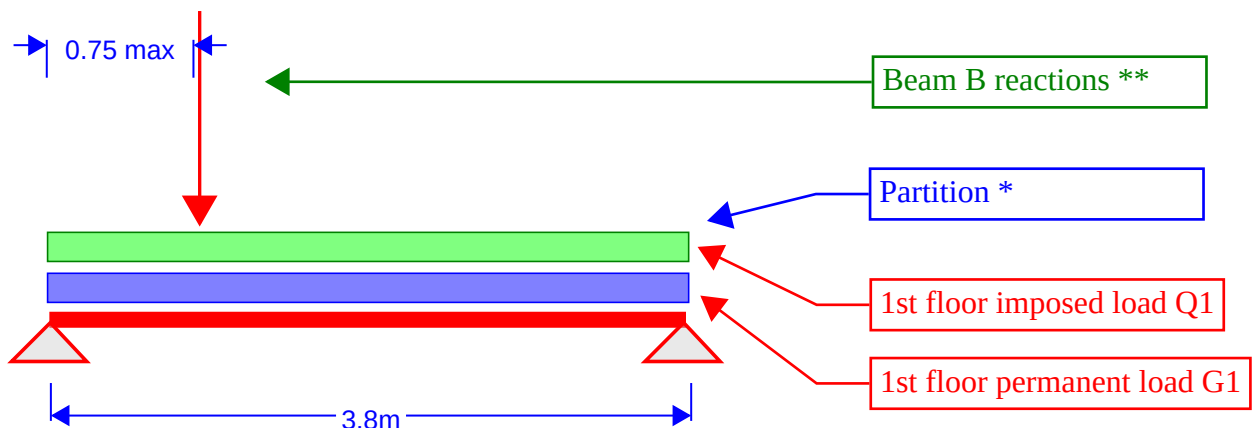
G1 = 1.75 kN/m
G2 = 6.4 kN/m (consider 2.4m wall)
G3 = 1.75 kN/m
G4 = 5.4 kN/m
G5 = 4.7kN/m
G6 = 4.5kN/m
Q1 = 5.25 kN/m
Q2 = 5.25 kN/m
Q3 = 2.5 kN/m

assumed tbc

Minimum size required

min 305x65x54 UB

Beam C



* Partition load comprises of:

S/W of masonry wall (UDL all beam) G2

2nd floor permanent load G3

2nd floor imposed load Q2

S/W of masonry to roof and part of roof (1.2m wide strip) G4

** Beam B:

61kN G5

28kN Q3

_input to TEDDS the following

s/w of beam automatically calculated

G1 = 0.6 kN/m

G2 = 6.4 kN/m (consider 2.4m wall)

G3 = 0.6 kN/m

G4 = 7.5 kN/m

G5 = 61 kN

Q1 = 1.8 kN/m

Q2 = 1.8 kN/m

Q3 = 28 kN/m

Minimum size required

min 305x65x46 UB

Job No.	Sheet No.	Rev
	08	P01
Member Location		
Drg. Ref. MD19 3DL/A 003 - FORENA		
Made by	Date	Chd.
	15/12/2020	

Job title

M19 3DL - 797 Stockport Rd

Column supporting 3 incoming beams

_input to TEDDS the following

Unrestrained length of beam (including basement) = 1.8m + 2.4m = 4.2m (assume full restraint at GF level)

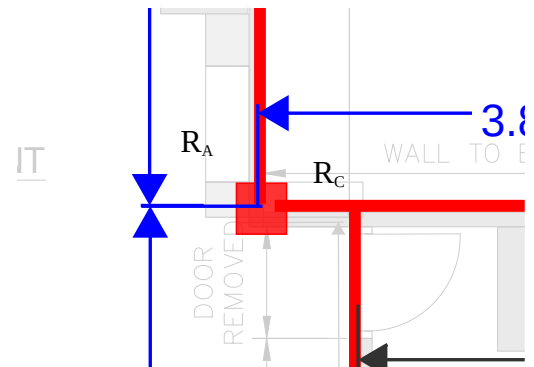
s/w of column automatically calculated

Reaction beam A (R_A)

Maximum reaction at support B $R_{B_max} = 151.9$ kN
 Unfactored permanent load reaction at support B $R_{B_Permanent} = 74.7$ kN
 Unfactored variable load reaction at support B $R_{B_Variable} = 34.1$ kN

Reaction beam C (R_C)

Maximum reaction at support A $R_{A_max} = 149.8$ kN
 Unfactored permanent load reaction at support A $R_{A_Permanent} = 78.4$ kN
 Unfactored variable load reaction at support A $R_{A_Variable} = 29.3$ kN



Moment Dir 1 = $151.9 * (0.1 + 0.1) = 30.5$ kNm

Moment Dir 2 = $149.8 * 0.1 = 15$ kNm

Minimum size required

min 203x203x46 UC

Foundation min requirement

2.0 x 2.0 x 1.0 deep mass concrete foundation

Assumed 50kPa ground bearing

Assumed foundation is at same level with existing foundations

ref to calc in appendix

Job No.	Sheet No.	Rev
	09	P01
Member Location		
Job title	Drg. Ref.	MD19 3DL/A 003 - FORENA
M19 3DL - 797 Stockport Rd	Made by	Date 15/12/2020 Chd.

Column on facade supporting Beam A end

_input to TEDDS the following

Unrestrained length of beam (including basement) = 1.8m + 2.4m = 4.2m (assume full restraint at GF level)

s/w of column automatically calculated

part of stability system - consider stability provided by masonry walls/piers at facade location top be replaced by column + beam

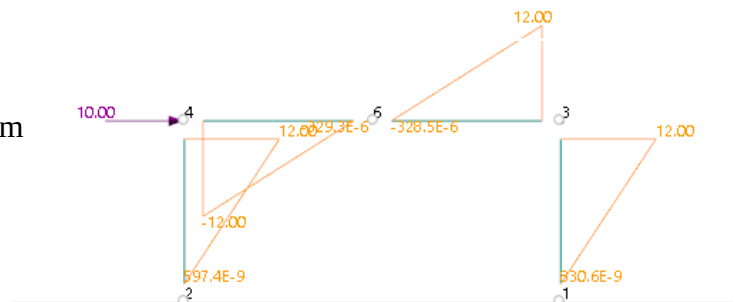
Reaction beam A (R_A)

Maximum reaction at support A $R_{A_max} = 181 \text{ kN}$
 Unfactored permanent load reaction at support A $R_{A_Permanent} = 96.2 \text{ kN}$
 Unfactored variable load reaction at support A $R_{A_Variable} = 34.1 \text{ kN}$

Moment Dir 1 = $181 * (0.1 + 0.1) = 36.2 \text{ kNm}$

Moment Dir 2 = $181 * 0.1 = 18.1 \text{ kNm}$

Additional moment - 1kPa wind on facade - 4 m wide - 2.4m tall = 12kNm



Minimum size required

min 203x203x46 UC or 152x152x44 UC (not always available)

Foundation min requirement

1.75x1.75x0.75 deep mass concrete foundation

Assumed 50kPa ground bearing

Assumed foundation is at same level with existing foundations

ref to calc in appendix

 Calculation sheet	Job No.	Sheet No.	Rev
		10	P01
	Member Location		
Job title	Drg. Ref.		
M19 3DL - 797 Stockport Rd	MD19 3DL/A 003 - FORENA		
	Made by	Date	Chd.
		15/12/2020	

Wall Bearing supporting beam A end

Not enough information are available on the front of the building masonry pier - therefore a calculation of this support will be provided in due course. Masonry pier or steel column is likely to be necessary due to the larger load attracted in this location

Wall Bearing supporting beam B end

The following assumptions have been made

wall to be 140 thick brick (clay) - good conditions and repointed as necessary.
2400mm high wall, no opening at 600mm from centre of beam

(ref to appendix calculation for more details)

Wall bearing support is acceptable with use of 600x215x140 concrete padstone.

If this is not achievable or assumptions are not valid a steel column/masonry pier need to be provided
- design tbc if necessary

Wall Bearing supporting beam C end

The following assumptions have been made

wall to be 140 thick brick (clay) - good conditions and repointed as necessary.
2400mm high wall, no opening at 600mm from centre of beam

Wall bearing support is acceptable with use of 440x215x140 concrete padstone.

If this is not achievable or assumptions are not valid a steel column/masonry pier need to be provided
- design tbc if necessary

Job No.	Sheet No.	Appendix	Rev	P01
Member Location				
Drg. Ref. MD19 3DL/A 003 - FORENA				
Made by		Date	15/12/2020	Chd.

Job title

M19 3DL - 797 Stockport Rd

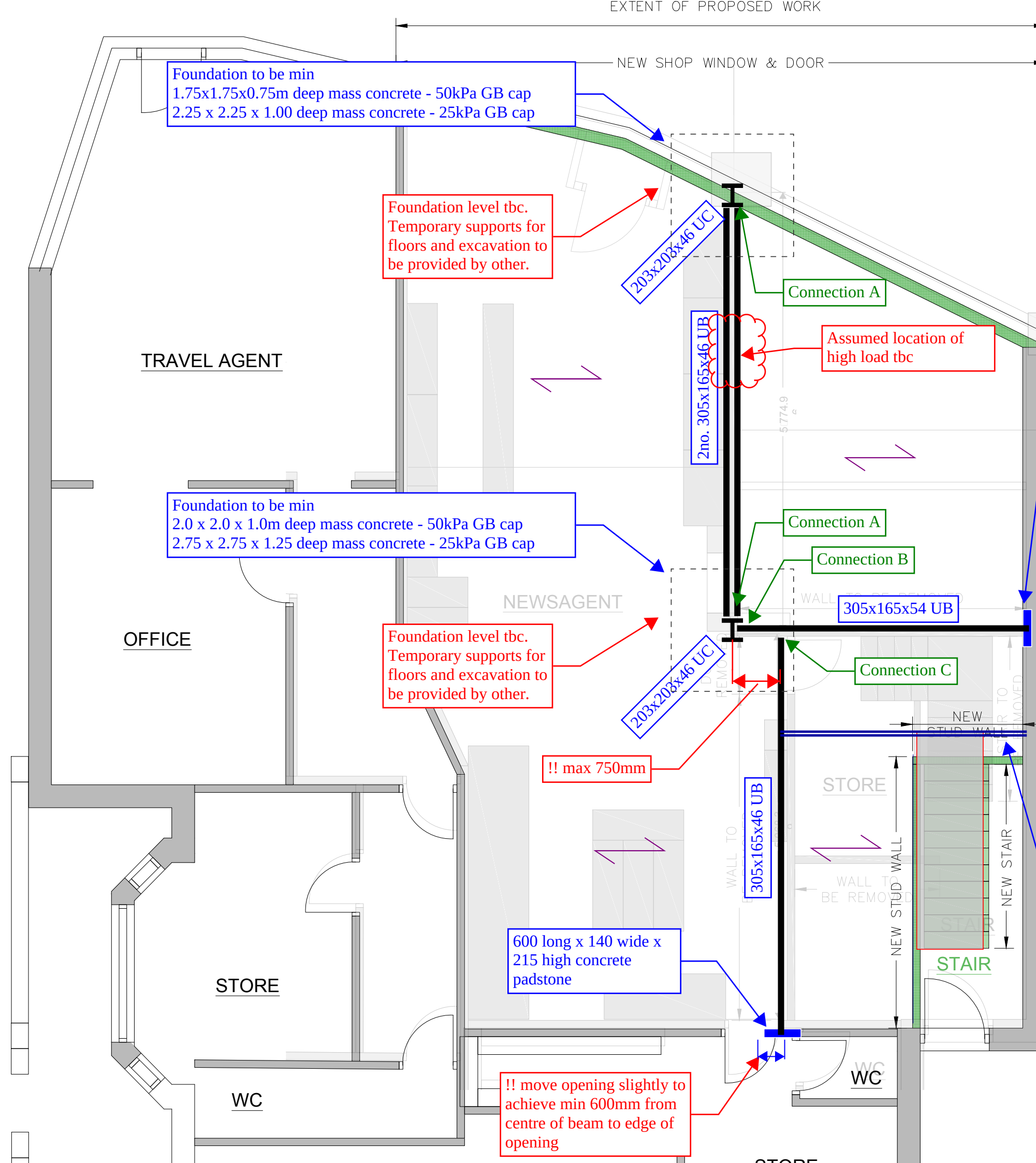
Appendix

Structural sketch

Structural Notes

Elements Design

- *Beam A option a*
- *Beam A option b*
- *Beam B*
- *Beam C*
- *Column*
- *Bearing wall*
- *Foundation*



***NOTE**
All elements not described on this drawing but part of the structural system are designed by others including:

- timber joists (to match existing)
- masonry panels
- joist hangers and fixings
- connections and baseplate (loading provided)
- foundations
- fire protection

Connection A
Permanent = 100kN
Imposed = 40kN
Moment = 25kNm

Connection B
Permanent = 85kN
Imposed = 35kN

Connection C
Permanent = 65kN
Imposed = 30kN

Assumed span of
floor joists (EX)

THE FOLLOWING REQUIRE CONFIRMATION PRIOR TO COMMENCE OF WORKS:

1. CONFIRM LOAD BEARING WALLS SIZE
2. CONFIRM EXISTING FLOOR JOIST (SPAN/ SIZE)
3. CONFIRM EXTENT/ SIZE OF BASEMENT AND EXISTING FOUNDATIONS
- 4 CONFIRM ROOF SUPPORTS AND GEOMETRY
- 5 CONFIRM GROUND BEARING CAPACITY (GB cap)
(assumed between 25kPa to 50 kPa at 1.5 m deep)

Ground floor general arrangement

M19 3DL - 797 Stockport Rd
04.01.2020 Drawing NTS

SK -001

P01

GENERAL CONSTRUCTION NOTES:

ALL SETTING OUT, LEVELS & GENERAL BUILDERS DETAILS TO BE CONFIRMED BY OTHERS.

ALL DRAINAGE, INSULATION, TANKING (WATERPROOFING) & SERVICE DUCT DETAILS TO BE CONFIRMED BY OTHERS.

ALL STEELWORK BELOW STRUCTURAL SLAB LEVEL (SSL) TO BE ENCASED IN MIN. 75mm THICK C28/35 CONCRETE, UNLESS NOTED OTHERWISE.

IN THE ABSENCE OF SITE INFORMATION, ALL FOUNDATIONS WILL BE DESIGNED ASSUMING A MINIMUM SAFE GROUND BEARING CAPACITY (SGBC) OF 50kN/m2. GROUND CONDITIONS TO BE CONFIRMED BY TRIAL PITTING PRIOR TO CONSTRUCTING FOUNDATIONS. USE C16/20 MASS CONCRETE FILL TO ACHIEVE REQUIRED BEARING LEVELS (IF REQUIRED).

ALL FOUNDATIONS TO BE STEPPED IN ACCORDANCE WITH DIAGRAM 21 OF APPROVED DOCUMENT BUILDING REGULATIONS 2010.

IN GRANULAR SOILS THERE IS TO BE A MINIMUM DEPTH TO THE UNDERSIDE OF FOOTINGS OF 550mm FROM EXTERNAL GROUND LEVEL. IN COHESIVE SOILS THIS IS TO BE INCREASED TO 750mm. WHERE TREES ARE CLOSE TO FOUNDATIONS, THE DEPTH OF THE FOUNDATION SHALL BE SET IN ACCORDANCE WITH THE DIRECTIONS GIVEN IN THE NHBC STANDARDS.

BLINDING, MASS CONCRETE TRENCH FILL & MASS CONCRETE FOUNDATIONS TO BE C16/20 (GEN 3, MIN 175kg/m³).

CONCRETE SLABS TO BE C28/35 (MIN 300kg/m³). REINFORCED CONCRETE FOUNDATIONS TO BE C28/35 (RC35, MIN 300kg/m³, MAX FREE WATER/CEMENT RATIO 0.60), ALL UNLESS NOTED OTHERWISE.

ALL STEEL BEAMS TO BE SUPPORTED ON MINIMUM 440x100x215mm DEEP MASS CONCRETE PADSTONES, UNLESS NOTED OTHERWISE. RAGBOLTS TO BE USED TO BOLT BOTTOM FLANGES TO CONCRETE WHERE SPECIFIED.

ALL INTERNAL LINTELS (SUPPORTING 100mm THICK LOAD BEARING BLOCK PARTITIONS) TO BE 100x140mm DEEP R15A PRESTRESSED CONCRETE (BY STRESSLINE, TEL. 0870 7503167, OR SIMILAR APPROVED), UNLESS NOTED OTHERWISE.

ALL EXTERNAL CAVITY WALL LINTELS TO BE STANDARD L1/S (BY IG, TEL. 01633 486460, OR SIMILAR APPROVED), UNLESS NOTED OTHERWISE.

MIN 100mm END BEARINGS FOR STEEL AND TIMBER BEAMS, UNLESS OTHERWISE NOTED.

MIN. 150mm END BEARINGS FOR CAVITY (IG AND CATNIC) AND PRESTRESSED CONCRETE LINTELS, UNLESS NOTED OTHERWISE.

INTERNAL STEELWORK PAINT PROTECTION SYSTEM FOR HEATED BUILDING TO BE AS FOLLOWS, UNLESS NOTED OTHERWISE.

STEELWORK TO BE BLAST CLEANED TO ISO SA. 2½ AFTER FABRICATION AND RECEIVE MINIMUM DFT OF 80 MICRONS OF A SUITABLE EPOXY ZINC PHOSPHATE PRIMER (E.G. INTERGARD 251, BY CROMADEx, TEL. 0161 848 7056). FINAL FINISH TO BE CONFIRMED BY ARCHITECT. ALL TO BS5493 U.N.O. ON DRAWINGS.

STEELWORK IN CAVITIES SHALL RECEIVE ADDITIONAL COAT OF BITUMEN PAINT (E.G. INTERTUF 16 BITUMEN, BY CROMADEx, TEL. 0161 848 7056).

EXTERNAL STEELWORK (INCLUDING STEELWORK WITHIN 25mm OF EXTERNAL LEAF) TO BE GALVANISED, UNLESS NOTED OTHERWISE. STEELWORK TO BE BLAST CLEANED TO ISO SA. 2½ WITH G24 ANGULAR IRON GRIT AFTER FABRICATION, AND GALVANISED IN ACCORDANCE WITH BS EN 1461 & BS EN 14713. STEEL MEMBERS WITH PLATE THICKNESS LESS THAN 6mm TO HAVE 100 MICRONS & STEEL MEMBERS WITH PLATE THICKNESS GREATER THAN 6mm TO HAVE 140 MICRONS.

ALL STEELWORK TO BE LINED, LEVELLED & GROUTED PRIOR TO CONSTRUCTING ADJACENT WALL PANELS, CLADDING ETC.

STEEL TO BE EXECUTION CLASS EXC2, IN ACCORDANCE WITH BS EN 1090-2. ALL STEELWORK TO BE GRADE S355JR, UNLESS NOTED OTHERWISE

ALL LOAD BEARING BLOCKWORK MASONRY TO BE MIN. 7N/mm², WITH MORTAR DESIGNATION III, UNLESS NOTED OTHERWISE.

ALL FIRE PROTECTION TO NEW STRUCTURE TO BE SPECIFIED BY OTHERS.

ALL SUSPENDED FLOORS AND ROOFS TO BE STRAPPED TO EXTERNAL WALL PANELS IN ACCORDANCE WITH THE CURRENT BUILDING REGULATIONS & BRITISH STANDARDS.

ALL TIMBER TO TIMBER CONNECTIONS TO BE FORMED USING SUITABLE STEEL BRACKETS, CLIPS, HANGERS ETC., INSTALLED IN ACCORDANCE WITH THE MANUFACTURERS SPECIFICATIONS (BY SIMPSON STRONG-TIE, TEL. 01827255600, OR SIMILAR APPROVED), UNLESS NOTED OTHERWISE

GROUT UNDER BASEPLATES & STEEL BEAMS TO BE FLOWSET 690 (BY EVERBUILD, TEL. 0113 2402424, OR SIMILAR APPROVED).

ALL STEEL TO STEEL CONNECTIONS TO CONTAIN MIN. 10mm THICK MS PLATES WITH MIN. 2/M20 (gr8.8) BOLTS. ALL WELDS TO BE 6mm CONTINUOUS FILLET WELDS, UNLESS NOTED OTHERWISE.

ALL SUSPENDED FLOORS AND ROOFS TO BE STRAPPED TO EXTERNAL WALL PANELS IN ACCORDANCE WITH THE CURRENT BUILDING REGULATIONS & BRITISH STANDARDS.

ALL TIMBER TO TIMBER CONNECTIONS TO BE FORMED USING SUITABLE STEEL BRACKETS, CLIPS, HANGERS ETC., INSTALLED IN ACCORDANCE WITH THE MANUFACTURERS SPECIFICATIONS (BY SIMPSON STRONG-TIE, TEL. 01827255600, OR SIMILAR APPROVED), UNLESS NOTED OTHERWISE

Standard Construction Notes

M19 3DL - 797 Stockport Rd
15 Dec 2020 Drawing NTS

SK -002

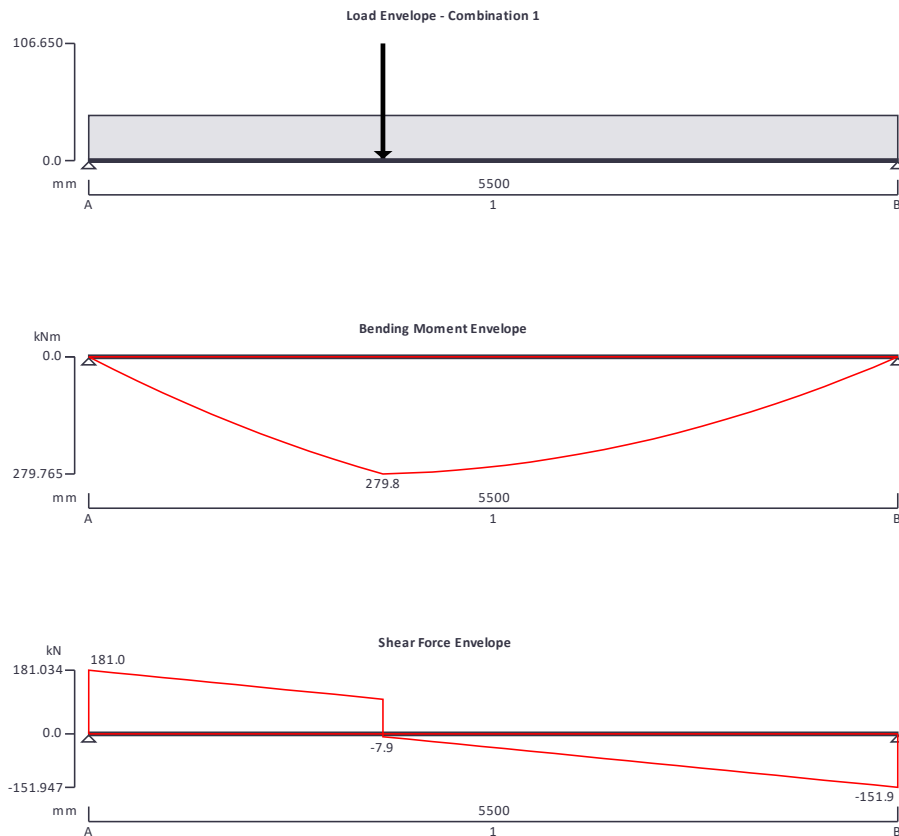
P01

Beam A option "a"

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

Q3 - Permanent point load 9 kN at 2000 mm

Q2 - Variable full UDL 6.2 kN/m

Q1 - Variable full UDL 6.2 kN/m

G5 - Permanent point load 20 kN at 2000 mm

G4 - Permanent point load 50 kN at 2000 mm

G3 - Permanent full UDL 2.05 kN/m

G2 - Permanent full UDL 11.7 kN/m

G1 - Permanent full UDL 2.05 kN/m

sw - Permanent self weight of beam $\times 1$

Load combinations

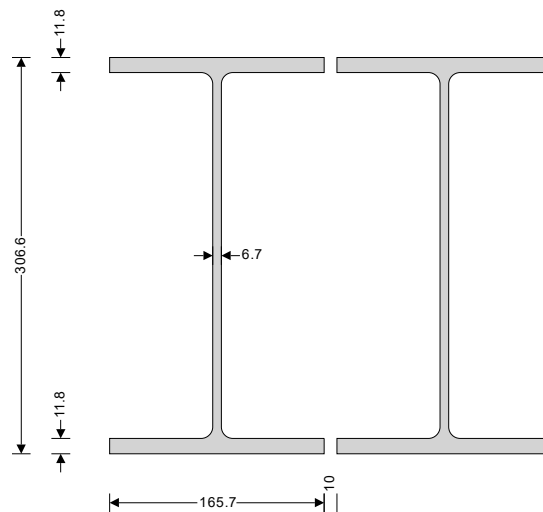
Load combination 1	Support A	Permanent $\times 1.35$ Variable $\times 1.50$
	Support B	Permanent $\times 1.35$ Variable $\times 1.50$

Analysis results

Maximum moment	$M_{\max} = 279.8 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum moment span 1 segment 1	$M_{s1_seg1_max} = 210 \text{ kNm}$	$M_{s1_seg1_min} = 0 \text{ kNm}$
Maximum moment span 1 segment 2	$M_{s1_seg2_max} = 279.8 \text{ kNm}$	$M_{s1_seg2_min} = 0 \text{ kNm}$
Maximum moment span 1 segment 3	$M_{s1_seg3_max} = 262.3 \text{ kNm}$	$M_{s1_seg3_min} = 0 \text{ kNm}$
Maximum moment span 1 segment 4	$M_{s1_seg4_max} = 170 \text{ kNm}$	$M_{s1_seg4_min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 181 \text{ kN}$	$V_{\min} = -151.9 \text{ kN}$
Maximum shear span 1 segment 1	$V_{s1_seg1_max} = 181 \text{ kN}$	$V_{s1_seg1_min} = 0 \text{ kN}$
Maximum shear span 1 segment 2	$V_{s1_seg2_max} = 124.5 \text{ kN}$	$V_{s1_seg2_min} = -38.8 \text{ kN}$
Maximum shear span 1 segment 3	$V_{s1_seg3_max} = 0 \text{ kN}$	$V_{s1_seg3_min} = -95.4 \text{ kN}$
Maximum shear span 1 segment 4	$V_{s1_seg4_max} = 0 \text{ kN}$	$V_{s1_seg4_min} = -151.9 \text{ kN}$
Deflection segment 5	$\delta_{\max} = 14.3 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 181 \text{ kN}$	$R_{A_min} = 181 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_Permanent} = 96.2 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_Variable} = 34.1 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 151.9 \text{ kN}$	$R_{B_min} = 151.9 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_Permanent} = 74.7 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_Variable} = 34.1 \text{ kN}$	

Section details

Section type	2 x UKB 305x165x46 (Tata Steel Advance)
Steel grade	S355
EN 10025-2:2004 - Hot rolled products of structural steels	
Nominal thickness of element	$t = \max(t_f, t_w) = 11.8 \text{ mm}$
Nominal yield strength	$f_y = 355 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 470 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



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 plus quarter 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	$K_{LT,A} = 1.000$
	$K_{LT,B} = 1.000$

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	$c = d = 265.2 \text{ mm}$	
	$c / t_w = 48.6 \times \varepsilon \leq 72 \times \varepsilon$	Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section	$c = (b - t_w - 2 \times r) / 2 = 70.6 \text{ mm}$	
	$c / t_f = 7.4 \times \varepsilon \leq 9 \times \varepsilon$	Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f = 283 \text{ mm}$
Shear area factor	$\eta = 1.000$
	$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force	$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 181 \text{ kN}$
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 2253 \text{ mm}^2$
Design shear resistance - cl 6.2.6(2)	$V_{c,Rd} = V_{pl,Rd} = N \times A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = 923.6 \text{ kN}$

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 $M_{Ed} = \max(\text{abs}(M_{s1_seg2_max}), \text{abs}(M_{s1_seg2_min})) = 279.8 \text{ kNm}$

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = N \times W_{pl,y} \times f_y / \gamma_{M0} = 511.2 \text{ kNm}$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = 0.98$$

$$C_1 = 1 / k_c^2 = 1.041$$

Curvature factor

$$g = \sqrt{1 - (I_z / I_y)} = 0.954$$

Poissons ratio

$$\nu = 0.3$$

Shear modulus

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

Unrestrained length

$$L = 1.0 \times L_{s1_seg2} = 1375 \text{ mm}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)} = 1645.1 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 0.394$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = 0.4$$

$$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0} - \text{Lateral torsional buckling can be ignored}$$

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

$$\delta_{lim} = L_{s1} / 360 = 15.3 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 14.284 \text{ mm}$$

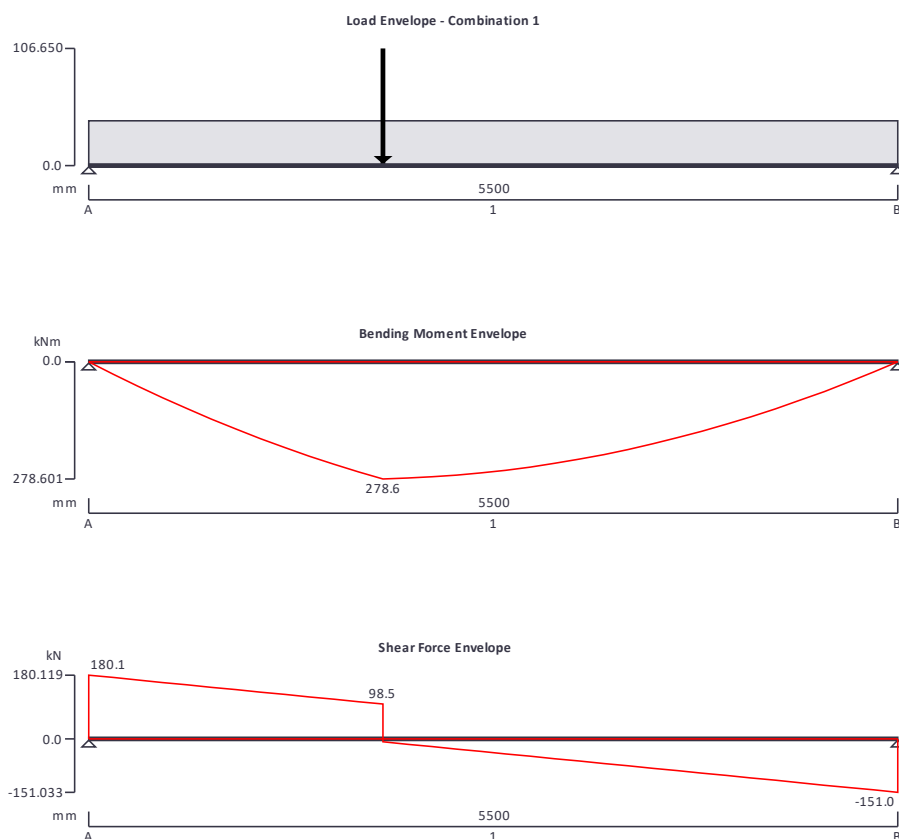
PASS - Maximum deflection does not exceed deflection limit

Beam A option "b"

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

Q3 - Permanent point load 9 kN at 2000 mm

Q2 - Variable full UDL 6.2 kN/m

Q1 - Variable full UDL 6.2 kN/m

G5 - Permanent point load 20 kN at 2000 mm

G4 - Permanent point load 50 kN at 2000 mm

G3 - Permanent full UDL 2.05 kN/m

G2 - Permanent full UDL 11.7 kN/m

G1 - Permanent full UDL 2.05 kN/m

sw - Permanent self weight of beam $\times 1$

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

Variable $\times 1.50$

Analysis results

Maximum moment

$M_{\max} = 278.6$ kNm

$M_{\min} = 0$ kNm

Maximum moment span 1 segment 1

$M_{s1_seg1_max} = 209.1$ kNm

$M_{s1_seg1_min} = 0$ kNm

Maximum moment span 1 segment 2

$M_{s1_seg2_max} = 278.6$ kNm

$M_{s1_seg2_min} = 0$ kNm

Maximum moment span 1 segment 3

$M_{s1_seg3_max} = 261$ kNm

$M_{s1_seg3_min} = 0$ kNm

Maximum moment span 1 segment 4

$M_{s1_seg4_max} = 169.1$ kNm

$M_{s1_seg4_min} = 0$ kNm

Maximum shear

$V_{\max} = 180.1$ kN

$V_{\min} = -151$ kN

Maximum shear span 1 segment 1

$V_{s1_seg1_max} = 180.1$ kN

$V_{s1_seg1_min} = 0$ kN

Maximum shear span 1 segment 2

$V_{s1_seg2_max} = 124$ kN

$V_{s1_seg2_min} = -38.8$ kN

Maximum shear span 1 segment 3

$V_{s1_seg3_max} = 0$ kN

$V_{s1_seg3_min} = -94.9$ kN

Maximum shear span 1 segment 4

$V_{s1_seg4_max} = 0$ kN

$V_{s1_seg4_min} = -151$ kN

Deflection segment 5

$\delta_{\max} = 14.5$ mm

$\delta_{\min} = 0$ mm

Maximum reaction at support A

$R_{A_max} = 180.1$ kN

$R_{A_min} = 180.1$ kN

Unfactored permanent load reaction at support A

$R_{A_Permanent} = 95.5$ kN

Unfactored variable load reaction at support A

$R_{A_Variable} = 34.1$ kN

Maximum reaction at support B

$R_{B_max} = 151$ kN

$R_{B_min} = 151$ kN

Unfactored permanent load reaction at support B

$R_{B_Permanent} = 74$ kN

Unfactored variable load reaction at support B

$R_{B_Variable} = 34.1$ kN

Section details

Section type

UKB 356x171x67

Steel grade

S355

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

Nominal thickness of element

$t = \max(t_f, t_w) = 15.7$ mm

Nominal yield strength

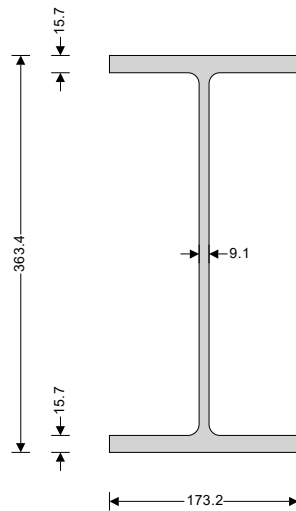
$f_y = 355$ N/mm²

Nominal ultimate tensile strength

$f_u = 470$ N/mm²

Modulus of elasticity

$E = 210000$ N/mm²



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 plus quarter 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

$$K_{LT,A} = 1.000$$

$$K_{LT,B} = 1.000$$

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

$$c = d = 311.6 \text{ mm}$$

$$c / t_w = 42.1 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = 71.9 \text{ mm}$$

$$c / t_f = 5.6 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = 332 \text{ mm}$$

Shear area factor

$$\eta = 1.000$$

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

Shear buckling resistance can be ignored

Design shear force

$$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 180.1 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 3574 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 732.5 \text{ kN}$$

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 $M_{Ed} = \max(\text{abs}(M_{s1_seg2_max}), \text{abs}(M_{s1_seg2_min})) = 278.6 \text{ kNm}$

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 429.9 \text{ kNm}$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = 0.98$$

$$C_1 = 1 / k_c^2 = 1.041$$

Curvature factor

$$g = \sqrt{1 - (I_z / I_y)} = 0.964$$

Poissons ratio

$$\nu = 0.3$$

Shear modulus

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

Unrestrained length

$$L = 1.0 \times L_{s1_seg2} = 1375 \text{ mm}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)} = 2938.8 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 0.382$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = 0.4$$

$$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0} - \text{Lateral torsional buckling can be ignored}$$

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

$$\delta_{lim} = L_{s1} / 360 = 15.3 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 14.458 \text{ mm}$$

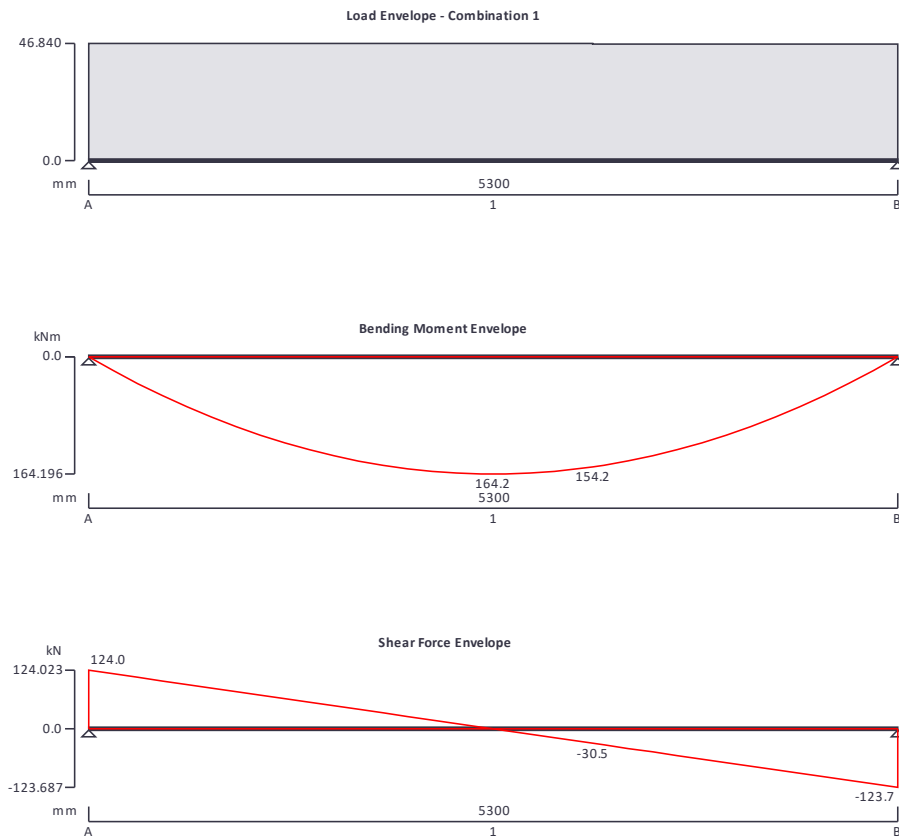
PASS - Maximum deflection does not exceed deflection limit

Beam B

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

Q3 - Permanent full UDL 2.5 kN/m

Q2 - Variable full UDL 5.25 kN/m

Q1 - Variable full UDL 5.25 kN/m

G6 - Permanent partial UDL 4.5 kN/m from 3300 mm to 5300 mm

G5 - Permanent partial UDL 4.7 kN/m from 0 mm to 3300 mm

G4 - Permanent full UDL 5.4 kN/m

G3 - Permanent full UDL 1.75 kN/m

G2 - Permanent full UDL 6.4 kN/m

G1 - Permanent full UDL 1.75 kN/m

sw - Permanent self weight of beam $\times 1$

Load combinations

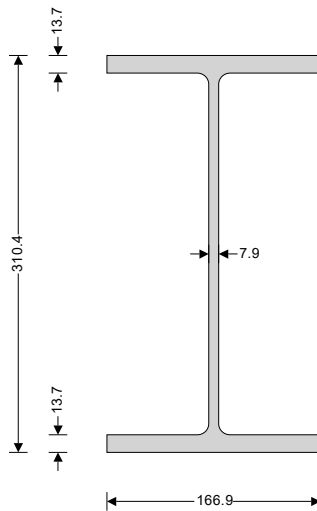
Load combination 1	Support A	Permanent $\times 1.35$
		Variable $\times 1.50$
	Support B	Permanent $\times 1.35$
		Variable $\times 1.50$

Analysis results

Maximum moment	$M_{\max} = 164.2 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum moment span 1 segment 1	$M_{s1_seg1_max} = 123.2 \text{ kNm}$	$M_{s1_seg1_min} = 0 \text{ kNm}$
Maximum moment span 1 segment 2	$M_{s1_seg2_max} = 164.2 \text{ kNm}$	$M_{s1_seg2_min} = 0 \text{ kNm}$
Maximum moment span 1 segment 3	$M_{s1_seg3_max} = 164.2 \text{ kNm}$	$M_{s1_seg3_min} = 0 \text{ kNm}$
Maximum moment span 1 segment 4	$M_{s1_seg4_max} = 123 \text{ kNm}$	$M_{s1_seg4_min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 124 \text{ kN}$	$V_{\min} = -123.7 \text{ kN}$
Maximum shear span 1 segment 1	$V_{s1_seg1_max} = 124 \text{ kN}$	$V_{s1_seg1_min} = 0 \text{ kN}$
Maximum shear span 1 segment 2	$V_{s1_seg2_max} = 62 \text{ kN}$	$V_{s1_seg2_min} = -0.1 \text{ kN}$
Maximum shear span 1 segment 3	$V_{s1_seg3_max} = 0 \text{ kN}$	$V_{s1_seg3_min} = -62 \text{ kN}$
Maximum shear span 1 segment 4	$V_{s1_seg4_max} = 0 \text{ kN}$	$V_{s1_seg4_min} = -123.7 \text{ kN}$
Deflection segment 5	$\delta_{\max} = 14 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 124 \text{ kN}$	$R_{A_min} = 124 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_Permanent} = 61 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_Variable} = 27.8 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 123.7 \text{ kN}$	$R_{B_min} = 123.7 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_Permanent} = 60.7 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_Variable} = 27.8 \text{ kN}$	

Section details

Section type	UKB 305x165x54 (Tata Steel Advance)
Steel grade	S355
EN 10025-2:2004 - Hot rolled products of structural steels	
Nominal thickness of element	$t = \max(t_f, t_w) = 13.7 \text{ mm}$
Nominal yield strength	$f_y = 355 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 470 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



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 plus quarter 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

$$K_{LT,A} = 1.000$$

$$K_{LT,B} = 1.000$$

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

$$c = d = 265.2 \text{ mm}$$

$$c / t_w = 41.3 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = 70.6 \text{ mm}$$

$$c / t_f = 6.3 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = 283 \text{ mm}$$

Shear area factor

$$\eta = 1.000$$

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

Shear buckling resistance can be ignored

Design shear force

$$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 124 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 2656 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 544.3 \text{ kN}$$

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 $M_{Ed} = \max(\text{abs}(M_{s1_seg2_max}), \text{abs}(M_{s1_seg2_min})) = 164.2 \text{ kNm}$

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 300.4 \text{ kNm}$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = 0.969$$

$$C_1 = 1 / k_c^2 = 1.065$$

Curvature factor

$$g = \sqrt{1 - (I_z / I_y)} = 0.953$$

Poissons ratio

$$\nu = 0.3$$

Shear modulus

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

Unrestrained length

$$L = 1.0 \times L_{s1_seg2} = 1325 \text{ mm}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)} = 2181.7 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 0.371$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = 0.4$$

$$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0} - \text{Lateral torsional buckling can be ignored}$$

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

$$\delta_{lim} = L_{s1} / 360 = 14.7 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 14 \text{ mm}$$

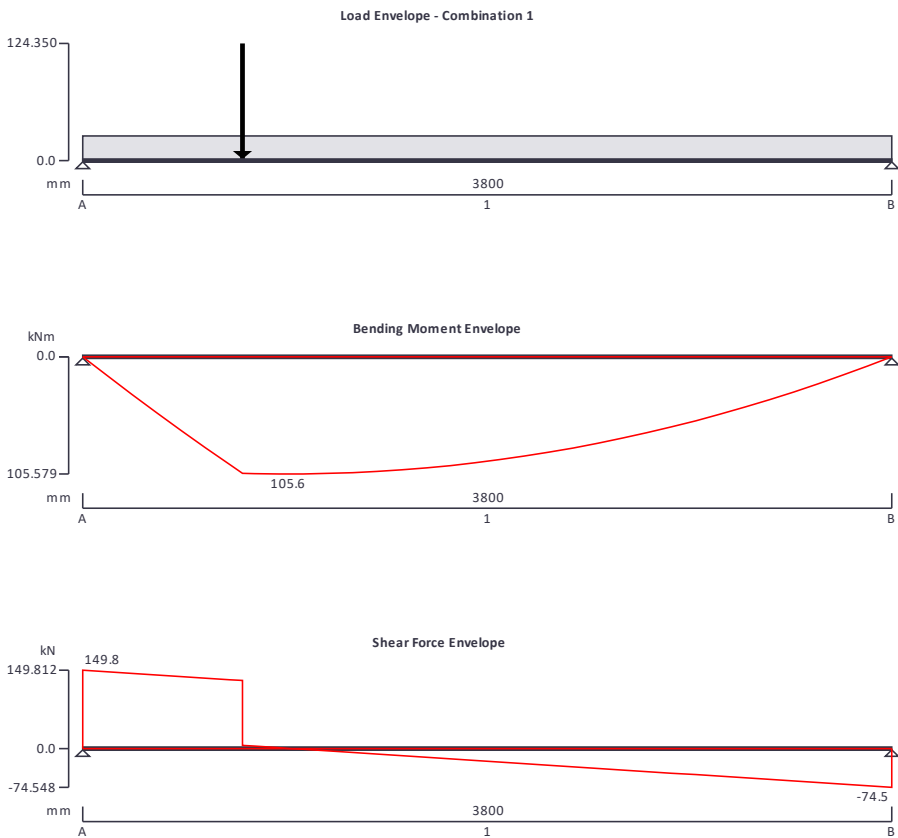
PASS - Maximum deflection does not exceed deflection limit

Beam C

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

Q3 - Variable point load 28 kN at 750 mm

Q2 - Variable full UDL 1.8 kN/m

Q1 - Variable full UDL 1.8 kN/m

G5 - Permanent point load 61 kN at 750 mm

G4 - Permanent full UDL 7.5 kN/m

G3 - Permanent full UDL 0.6 kN/m

G2 - Permanent full UDL 6.4 kN/m

G1 - Permanent full UDL 0.6 kN/m

sw - Permanent self weight of beam $\times 1$

Load combinations

Load combination 1	Support A	Permanent $\times 1.35$ Variable $\times 1.50$ Permanent $\times 1.35$ Variable $\times 1.50$
	Support B	Permanent $\times 1.35$ Variable $\times 1.50$

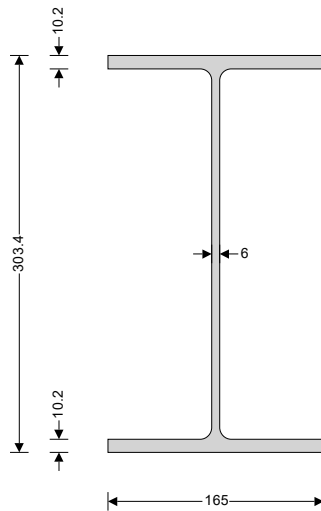
Analysis results

Maximum moment	$M_{\max} = 105.6 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum moment span 1 segment 1	$M_{s1_seg1_max} = 105.6 \text{ kNm}$	$M_{s1_seg1_min} = 0 \text{ kNm}$
Maximum moment span 1 segment 2	$M_{s1_seg2_max} = 105.6 \text{ kNm}$	$M_{s1_seg2_min} = 0 \text{ kNm}$
Maximum moment span 1 segment 3	$M_{s1_seg3_max} = 94.1 \text{ kNm}$	$M_{s1_seg3_min} = 0 \text{ kNm}$
Maximum moment span 1 segment 4	$M_{s1_seg4_max} = 58.9 \text{ kNm}$	$M_{s1_seg4_min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 149.8 \text{ kN}$	$V_{\min} = -74.5 \text{ kN}$
Maximum shear span 1 segment 1	$V_{s1_seg1_max} = 149.8 \text{ kN}$	$V_{s1_seg1_min} = 0 \text{ kN}$
Maximum shear span 1 segment 2	$V_{s1_seg2_max} = 0.5 \text{ kN}$	$V_{s1_seg2_min} = -24.5 \text{ kN}$
Maximum shear span 1 segment 3	$V_{s1_seg3_max} = 0 \text{ kN}$	$V_{s1_seg3_min} = -49.5 \text{ kN}$
Maximum shear span 1 segment 4	$V_{s1_seg4_max} = 0 \text{ kN}$	$V_{s1_seg4_min} = -74.5 \text{ kN}$
Deflection segment 5	$\delta_{\max} = 6.1 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 149.8 \text{ kN}$	$R_{A_min} = 149.8 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_Permanent} = 78.4 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_Variable} = 29.3 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 74.5 \text{ kN}$	$R_{B_min} = 74.5 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_Permanent} = 41.5 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_Variable} = 12.4 \text{ kN}$	

Section details

Section type	UKB 305x165x40 (Tata Steel Advance)
Steel grade	S355
EN 10025-2:2004 - Hot rolled products of structural steels	
Nominal thickness of element	$t = \max(t_f, t_w) = 10.2 \text{ mm}$
Nominal yield strength	$f_y = 355 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 470 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$

use 46 to match other beams size



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 plus quarter 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

$$K_{LT,A} = 1.000$$

$$K_{LT,B} = 1.000$$

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

$$c = d = 265.2 \text{ mm}$$

$$c / t_w = 54.3 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = 70.6 \text{ mm}$$

$$c / t_f = 8.5 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = 283 \text{ mm}$$

Shear area factor

$$\eta = 1.000$$

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

Shear buckling resistance can be ignored

Design shear force

$$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 149.8 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 2009 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 411.7 \text{ kN}$$

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 $M_{Ed} = \max(\text{abs}(M_{s1_seg2_max}), \text{abs}(M_{s1_seg2_min})) = 105.6 \text{ kNm}$

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 221.2 \text{ kNm}$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = 0.981$$

$$C_1 = 1 / k_c^2 = 1.04$$

Curvature factor

$$g = \sqrt{1 - (I_z / I_y)} = 0.954$$

Poissons ratio

$$\nu = 0.3$$

Shear modulus

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

Unrestrained length

$$L = 1.0 \times L_{s1_seg2} = 950 \text{ mm}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)} = 2849.2 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{W_{pl,y} \times f_y / M_{cr}} = 0.279$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = 0.4$$

$$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0} - \text{Lateral torsional buckling can be ignored}$$

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

$$\delta_{lim} = L_{s1} / 360 = 10.6 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 6.135 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

Column 203 UC

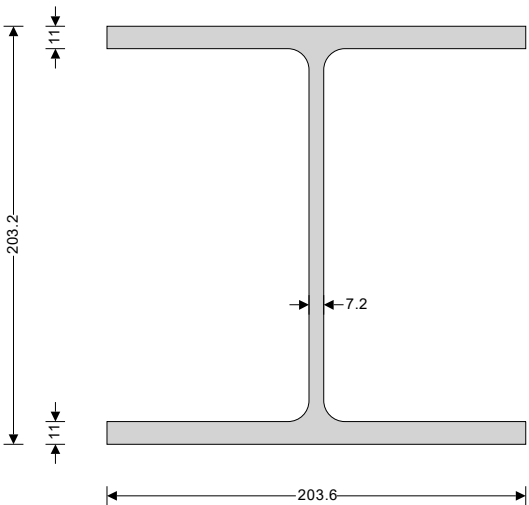
STEEL COLUMN DESIGN

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

Tedds calculation version 1.1.04

Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1$
Resistance of members to instability	$\gamma_{M1} = 1$
Resistance of cross-sections in tension to fracture	$\gamma_{M2} = 1.1$



UKC 203x203x46 (Tata Steel Advance)	
Section depth, h,	203.2 mm
Section breadth, b,	203.6 mm
Mass of section, Mass,	46.1 kg/m
Flange thickness, t_f ,	11 mm
Web thickness, t_w ,	7.2 mm
Root radius, r,	10.2 mm
Area of section, A,	5873 mm ²
Radius of gyration about y-axis, i_y ,	88.19 mm
Radius of gyration about z-axis, i_z ,	51.343 mm
Elastic section modulus about y-axis, $W_{el,y}$,	449588 mm ³
Elastic section modulus about z-axis, $W_{el,z}$,	152083 mm ³
Plastic section modulus about y-axis, $W_{pl,y}$,	497439 mm ³
Plastic section modulus about z-axis, $W_{pl,z}$,	230864 mm ³
Second moment of area about y-axis, I_y ,	45678168 mm ⁴
Second moment of area about z-axis, I_z ,	15482057 mm ⁴

Column details

Column section	UKC 203x203x46
Steel grade	S355
Yield strength	$f_y = 355 \text{ N/mm}^2$
Ultimate strength	$f_u = 470 \text{ N/mm}^2$
Modulus of elasticity	$E = 210 \text{ kN/mm}^2$
Poisson's ratio	$\nu = 0.3$
Shear modulus	$G = E / [2 \times (1 + \nu)] = 80.8 \text{ kN/mm}^2$

Column geometry

System length for buckling - Major axis	$L_y = 4200 \text{ mm}$
System length for buckling - Minor axis	$L_z = 4200 \text{ mm}$
The column is not part of a sway frame in the direction of the minor axis	
The column is not part of a sway frame in the direction of the major axis	

Column loading

Axial load	$N_{Ed} = 302 \text{ kN}$ (Compression)
Major axis moment at end 1 - Bottom	$M_{y,Ed1} = 0.0 \text{ kNm}$
Major axis moment at end 2 - Top	$M_{y,Ed2} = 30.5 \text{ kNm}$
Major axis bending is single curvature	
Minor axis moment at end 1 - Bottom	$M_{z,Ed1} = 0.0 \text{ kNm}$
Minor axis moment at end 2 - Top	$M_{z,Ed2} = 15.0 \text{ kNm}$
Minor axis bending is single curvature	

Major axis shear force $V_{y,Ed} = 0 \text{ kN}$

Minor axis shear force $V_{z,Ed} = 0 \text{ kN}$

Buckling length for flexural buckling - Major axis

Buckling length $L_{cr,y} = 2400 \text{ mm}$

Buckling length for flexural buckling - Minor axis

Buckling length $L_{cr,z} = 4200 \text{ mm}$

Web section classification (Table 5.2)

Coefficient depending on f_y $\varepsilon = \sqrt{(235 \text{ N/mm}^2 / f_y)} = 0.814$

Depth between fillets $c_w = h - 2 \times (t_f + r) = 160.8 \text{ mm}$

Ratio of c/t $ratio_w = c_w / t_w = 22.33$

Length of web taken by axial load $l_w = \min(N_{Ed} / (f_y \times t_w), c_w) = 118.2 \text{ mm}$

For class 1 & 2 proportion in compression $\alpha = (c_w/2 + l_w/2) / c_w = 0.867$

Limit for class 1 web $Limit_{1w} = (396 \times \varepsilon) / (13 \times \alpha - 1) = 31.35$

The web is class 1

Flange section classification (Table 5.2)

Outstand length $c_f = (b - t_w) / 2 - r = 88.0 \text{ mm}$

Ratio of c/t $ratio_f = c_f / t_f = 8.00$

Conservatively assume uniform compression in flange

Limit for class 1 flange $Limit_{1f} = 9 \times \varepsilon = 7.32$

Limit for class 2 flange $Limit_{2f} = 10 \times \varepsilon = 8.14$

Limit for class 3 flange $Limit_{3f} = 14 \times \varepsilon = 11.39$

The flange is class 2

Overall section classification

The section is class 2

Resistance of cross section (cl. 6.2)

Compression (cl. 6.2.4)

Design force $N_{Ed} = 302 \text{ kN}$

Design resistance $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0} = 2085 \text{ kN}$

$N_{Ed} / N_{c,Rd} = 0.145$

PASS - The compression design resistance exceeds the design force

Bending - Major axis (cl. 6.2.5)

Design bending moment $M_{y,Ed} = \max(\text{abs}(M_{y,Ed1}), \text{abs}(M_{y,Ed2})) = 30.5 \text{ kNm}$

Section modulus $W_y = W_{pl,y} = 497.4 \text{ cm}^3$

Design resistance $M_{c,y,Rd} = W_y \times f_y / \gamma_{M0} = 176.6 \text{ kNm}$

$M_{y,Ed} / M_{c,y,Rd} = 0.173$

PASS - The bending design resistance exceeds the design moment

Bending - Major axis (cl. 6.2.5)

Design bending moment $M_{z,Ed} = \max(\text{abs}(M_{z,Ed1}), \text{abs}(M_{z,Ed2})) = 15.0 \text{ kNm}$

Section modulus $W_z = W_{pl,z} = 230.9 \text{ cm}^3$

Design resistance $M_{c,z,Rd} = W_z \times f_y / \gamma_{M0} = 82.0 \text{ kNm}$

$M_{z,Ed} / M_{c,z,Rd} = 0.183$

PASS - The bending design resistance exceeds the design moment

Combined bending and axial force (cl. 6.2.9)

Ratio design axial to design plastic resistance $n = \text{abs}(N_{Ed}) / N_{pl,Rd} = 0.145$

Ratio web area to gross area

$$a = \min(0.5, (A - 2 \times b \times t_f) / A) = \mathbf{0.237}$$

Bending - Major axis (cl. 6.2.9.1)

Design bending moment

$$M_{y,Ed} = \max(\text{abs}(M_{y,Ed1}), \text{abs}(M_{y,Ed2})) = \mathbf{30.5 \text{ kNm}}$$

Plastic design resistance

$$M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = \mathbf{176.6 \text{ kNm}}$$

Modified design resistance

$$M_{N,y,Rd} = M_{pl,y,Rd} \times \min(1, (1 - n) / (1 - 0.5 \times a)) = \mathbf{171.3 \text{ kNm}}$$

$$M_{y,Ed} / M_{N,y,Rd} = \mathbf{0.178}$$

PASS - Bending resistance in presence of axial load exceeds design moment

Bending - Minor axis (cl. 6.2.9.1)

Design bending moment

$$M_{z,Ed} = \max(\text{abs}(M_{z,Ed1}), \text{abs}(M_{z,Ed2})) = \mathbf{15.0 \text{ kNm}}$$

Plastic design resistance

$$M_{pl,z,Rd} = W_{pl,z} \times f_y / \gamma_{M0} = \mathbf{82.0 \text{ kNm}}$$

Modified design resistance

$$M_{N,z,Rd} = M_{pl,z,Rd} = \mathbf{82.0 \text{ kNm}}$$

$$M_{z,Ed} / M_{N,z,Rd} = \mathbf{0.183}$$

PASS - Bending resistance in presence of axial load exceeds design moment

Biaxial bending

Exponent α

$$\alpha = \mathbf{2.00}$$

Exponent β

$$\beta = \max(1, 5 \times n) = \mathbf{1.00}$$

Section utilisation at end 1

$$UR_{CS_1} = [\text{abs}(M_{y,Ed1}) / M_{N,y,Rd}]^\alpha + [\text{abs}(M_{z,Ed1}) / M_{N,z,Rd}]^\beta = \mathbf{0.000}$$

Section utilisation at end 2

$$UR_{CS_2} = [\text{abs}(M_{y,Ed2}) / M_{N,y,Rd}]^\alpha + [\text{abs}(M_{z,Ed2}) / M_{N,z,Rd}]^\beta = \mathbf{0.215}$$

PASS - The cross-section resistance is adequate

Buckling resistance (cl. 6.3)

Yield strength for buckling resistance

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

Flexural buckling - Major axis

Elastic critical buckling force

$$N_{cr,y} = \pi^2 \times E \times I_y / L_{cr_y}^2 = \mathbf{16436 \text{ kN}}$$

Non-dimensional slenderness

$$\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} = \mathbf{0.356}$$

Buckling curve (Table 6.2)

b

Imperfection factor (Table 6.1)

$$\alpha_y = \mathbf{0.34}$$

Parameter Φ

$$\Phi_y = 0.5 \times [1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2] = \mathbf{0.590}$$

Reduction factor

$$\chi_y = \min(1.0, 1 / [\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}]) = \mathbf{0.943}$$

Design buckling resistance

$$N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} = \mathbf{1966.4 \text{ kN}}$$

$$N_{Ed} / N_{b,y,Rd} = \mathbf{0.154}$$

PASS - The flexural buckling resistance exceeds the design axial load

Flexural buckling - Minor axis

Elastic critical buckling force

$$N_{cr,z} = \pi^2 \times E \times I_z / L_{cr_z}^2 = \mathbf{1819 \text{ kN}}$$

Non-dimensional slenderness

$$\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})} = \mathbf{1.071}$$

Buckling curve (Table 6.2)

c

Imperfection factor (Table 6.1)

$$\alpha_z = \mathbf{0.49}$$

Parameter Φ

$$\Phi_z = 0.5 \times [1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2] = \mathbf{1.286}$$

Reduction factor

$$\chi_z = \min(1.0, 1 / [\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}]) = \mathbf{0.500}$$

Design buckling resistance

$$N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} = \mathbf{1042.7 \text{ kN}}$$

$$N_{Ed} / N_{b,z,Rd} = \mathbf{0.29}$$

PASS - The flexural buckling resistance exceeds the design axial load

Torsional and torsional-flexural buckling (cl. 6.3.1.4)

Torsional buckling length factor

$$K_T = \mathbf{1.00}$$

Effective buckling length

$$L_{cr_T} = K_T \times \max(L_y, L_z) = \mathbf{4200 \text{ mm}}$$

Distance from shear ctr to centroid along major axis

$$y_0 = 0.0 \text{ mm}$$

Distance from shear ctr to centroid along minor axis

$$z_0 = 0.0 \text{ mm}$$

$$i_0 = \sqrt{(i_y^2 + i_z^2 + y_0^2 + z_0^2)} = 102.0 \text{ mm}$$

$$\beta_T = 1 - (y_0 / i_0)^2 = 1.000$$

Elastic critical torsional buckling force

$$N_{cr,T} = 1 / i_0^2 \times (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) = 3332 \text{ kN}$$

Elastic critical torsional-flexural buckling force

$$N_{cr,TF} = N_{cr,y} / (2 \times \beta_T) \times [1 + N_{cr,T} / N_{cr,y} - \sqrt{[(1 - N_{cr,T} / N_{cr,y})^2 + 4 \times (y_0 / i_0)^2 \times N_{cr,T} / N_{cr,y}]]}$$
$$N_{cr,TF} = 3332 \text{ kN}$$

Non-dimensional slenderness

$$\bar{\lambda}_T = \sqrt{(A \times f_y / \min(N_{cr,T}, N_{cr,TF}))} = 0.791$$

Buckling curve (Table 6.2)

c

Imperfection factor (Table 6.1)

$$\alpha_T = 0.49$$

Parameter Φ

$$\Phi_T = 0.5 \times [1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2] = 0.958$$

Reduction factor

$$\chi_T = \min(1.0, 1 / [\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}]) = 0.668$$

Design buckling resistance

$$N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1} = 1392.2 \text{ kN}$$

$$N_{Ed} / N_{b,T,Rd} = 0.217$$

PASS - The torsional/torsional-flexural buckling resistance exceeds the design axial load

Minimum buckling resistance

Minimum buckling resistance

$$N_{b,Rd} = \min(N_{b,y,Rd}, N_{b,z,Rd}, N_{b,T,Rd}) = 1042.7 \text{ kN}$$

$$N_{Ed} / N_{b,Rd} = 0.29$$

PASS - The axial load buckling resistance exceeds the design axial load

Buckling resistance moment (cl.6.3.2.1)

Lateral torsional buckling length factor

$$K_{LT} = 1.00$$

Effective buckling length

$$L_{cr,LT} = K_{LT} \times L_z = 4200 \text{ mm}$$

End moment factor

$$\psi = M_{y,Ed1} / M_{y,Ed2} = 0.000$$

Moment distribution correction factor (Table 6.6)

$$k_c = 1 / (1.33 - 0.33 \times \psi) = 0.752$$

$$C_1 = 1 / k_c^2 = 1.769$$

Curvature factor

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

Poissons ratio

$$\nu = 0.3$$

Shear modulus

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

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z \times \sqrt{[I_w / I_z + L_{cr,LT}^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} / (L_{cr,LT}^2 \times g)$$
$$M_{cr} = 546.5 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{[W_y \times f_y / M_{cr}]} = 0.568$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = 0.40$$

Correction factor for rolled sections

$$\beta_r = 0.75$$

Buckling curve (Table 6.5)

b

Imperfection factor (Table 6.1)

$$\alpha_{LT} = 0.34$$

Parameter Φ_{LT}

$$\Phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta_r \times \bar{\lambda}_{LT}^2] = 0.650$$

Reduction factor

$$\chi_{LT} = \min(1.0, 1 / \bar{\lambda}_{LT}^2, 1 / [\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta_r \times \bar{\lambda}_{LT}^2)}]) = 0.931$$

Modification factor

$$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 0.889$$

Modified LTB reduction factor - eq 6.58

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2) = 1.000$$

Design buckling resistance moment

$$M_{b,Rd} = \chi_{LT,mod} \times W_y \times f_y / \gamma_{M1} = 176.6 \text{ kNm}$$

Design bending moment

$$M_{y,Ed} = \max(\text{abs}(M_{y,Ed1}), \text{abs}(M_{y,Ed2})) = 30.5 \text{ kNm}$$

$$M_{y,Ed} / M_{b,Rd} = 0.173$$

PASS - The design buckling resistance moment exceeds the maximum design moment

Combined bending and axial compression (cl. 6.3.3)

Characteristic resistance to normal force $N_{Rk} = A \times f_y = \mathbf{2085 \text{ kN}}$

Characteristic moment resistance - Major axis $M_{y,Rk} = W_{pl,y} \times f_y = \mathbf{176.6 \text{ kNm}}$

Characteristic moment resistance - Minor axis $M_{z,Rk} = W_{pl,z} \times f_y = \mathbf{82.0 \text{ kNm}}$

$\psi_y = \text{if}(\text{abs}(M_{y,Ed1}) \leq \text{abs}(M_{y,Ed2}), M_{y,Ed1} / \text{if}(M_{y,Ed2} > 0 \text{ kNm}, \max(M_{y,Ed2}, 0.0001 \text{ kNm}), M_{y,Ed2}), M_{y,Ed2} / \text{if}(M_{y,Ed1} > 0 \text{ kNm}, \max(M_{y,Ed1}, 0.0001 \text{ kNm}), M_{y,Ed1})) = \mathbf{0.000}$

Moment distribution factor - Major axis $\psi_y = M_{y,Ed1} / M_{y,Ed2} = \mathbf{0.000}$

Moment factor - Major axis $C_{my} = \max(0.4, 0.6 + 0.4 \times \psi_y) = \mathbf{0.600}$

Moment distribution factor - Minor axis $\psi_z = M_{z,Ed1} / M_{z,Ed2} = \mathbf{0.000}$

Moment factor - Minor axis $C_{mz} = \max(0.4, 0.6 + 0.4 \times \psi_z) = \mathbf{0.600}$

Moment distribution factor for LTB $\psi_{LT} = M_{y,Ed1} / M_{y,Ed2} = \mathbf{0.000}$

Moment factor for LTB $C_{mLT} = \max(0.4, 0.6 + 0.4 \times \psi_{LT}) = \mathbf{0.600}$

Interaction factor $k_{yy} = C_{my} \times [1 + \min(0.8, \bar{\lambda}_y - 0.2) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})] = \mathbf{0.614}$

Interaction factor $k_{zy} = 1 - \min(0.1, 0.1 \times \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times (\chi_z \times N_{Rk} / \gamma_{M1})) = \mathbf{0.917}$

Interaction factor $k_{zz} = C_{mz} \times [1 + \min(1.4, 2 \times \bar{\lambda}_z - 0.6) \times N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1})] = \mathbf{0.843}$

Interaction factor $k_{yz} = 0.6 \times k_{zz} = \mathbf{0.506}$

Section utilisation $UR_{B_1} = N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) + k_{yz} \times M_{z,Ed} / (M_{z,Rk} / \gamma_{M1})$
 $UR_{B_1} = \mathbf{0.360}$

$UR_{B_2} = N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) + k_{zz} \times M_{z,Ed} / (M_{z,Rk} / \gamma_{M1})$
 $UR_{B_2} = \mathbf{0.614}$

PASS - The buckling resistance is adequate

Column 152 UC

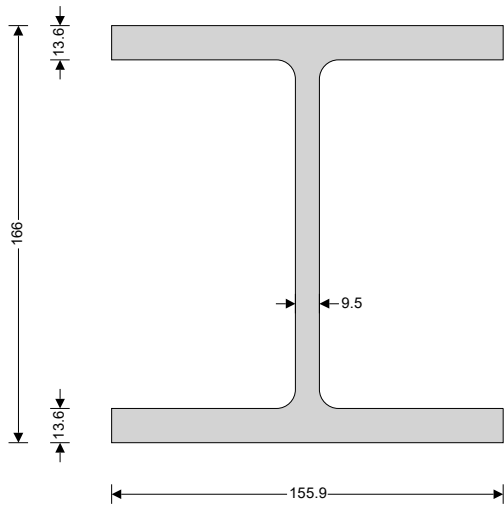
STEEL COLUMN DESIGN

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

Tedds calculation version 1.1.04

Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1$
Resistance of members to instability	$\gamma_{M1} = 1$
Resistance of cross-sections in tension to fracture	$\gamma_{M2} = 1.1$



UKC 152x152x44 (Tata Steel Advance)
Section depth, h, 166 mm
Section breadth, b, 155.9 mm
Mass of section, Mass, 44 kg/m
Flange thickness, t_f , 13.6 mm
Web thickness, t_w , 9.5 mm
Root radius, r, 7.6 mm
Area of section, A, 5609 mm ²
Radius of gyration about y-axis, i_y , 69.424 mm
Radius of gyration about z-axis, i_z , 39.16 mm
Elastic section modulus about y-axis, $W_{el,y}$, 325684 mm ³
Elastic section modulus about z-axis, $W_{el,z}$, 110337 mm ³
Plastic section modulus about y-axis, $W_{pl,y}$, 372237 mm ³
Plastic section modulus about z-axis, $W_{pl,z}$, 168724 mm ³
Second moment of area about y-axis, I_y , 27031767 mm ⁴
Second moment of area about z-axis, I_z , 8600751 mm ⁴

Column details

Column section	UKC 152x152x44
Steel grade	S355
Yield strength	$f_y = 355 \text{ N/mm}^2$
Ultimate strength	$f_u = 470 \text{ N/mm}^2$
Modulus of elasticity	$E = 210 \text{ kN/mm}^2$
Poisson's ratio	$\nu = 0.3$
Shear modulus	$G = E / [2 \times (1 + \nu)] = 80.8 \text{ kN/mm}^2$

Column geometry

System length for buckling - Major axis	$L_y = 4200 \text{ mm}$
System length for buckling - Minor axis	$L_z = 4200 \text{ mm}$
The column is not part of a sway frame in the direction of the minor axis	
The column is not part of a sway frame in the direction of the major axis	

Column loading

Axial load	$N_{Ed} = 181 \text{ kN}$ (Compression)
Major axis moment at end 1 - Bottom	$M_{y,Ed1} = 0.0 \text{ kNm}$
Major axis moment at end 2 - Top	$M_{y,Ed2} = 48.2 \text{ kNm}$
Major axis bending is single curvature	
Minor axis moment at end 1 - Bottom	$M_{z,Ed1} = 0.0 \text{ kNm}$
Minor axis moment at end 2 - Top	$M_{z,Ed2} = 30.1 \text{ kNm}$
Minor axis bending is single curvature	

Major axis shear force $V_{y,Ed} = 0 \text{ kN}$

Minor axis shear force $V_{z,Ed} = 0 \text{ kN}$

Buckling length for flexural buckling - Major axis

Buckling length $L_{cr,y} = 2400 \text{ mm}$

Buckling length for flexural buckling - Minor axis

Buckling length $L_{cr,z} = 2400 \text{ mm}$

Web section classification (Table 5.2)

Coefficient depending on f_y $\varepsilon = \sqrt{(235 \text{ N/mm}^2 / f_y)} = 0.814$

Depth between fillets $c_w = h - 2 \times (t_f + r) = 123.6 \text{ mm}$

Ratio of c/t $ratio_w = c_w / t_w = 13.01$

Length of web taken by axial load $l_w = \min(N_{Ed} / (f_y \times t_w), c_w) = 53.7 \text{ mm}$

For class 1 & 2 proportion in compression $\alpha = (c_w/2 + l_w/2) / c_w = 0.717$

Limit for class 1 web $Limit_{1w} = (396 \times \varepsilon) / (13 \times \alpha - 1) = 38.71$

The web is class 1

Flange section classification (Table 5.2)

Outstand length $c_f = (b - t_w) / 2 - r = 65.6 \text{ mm}$

Ratio of c/t $ratio_f = c_f / t_f = 4.82$

Conservatively assume uniform compression in flange

Limit for class 1 flange $Limit_{1f} = 9 \times \varepsilon = 7.32$

Limit for class 2 flange $Limit_{2f} = 10 \times \varepsilon = 8.14$

Limit for class 3 flange $Limit_{3f} = 14 \times \varepsilon = 11.39$

The flange is class 1

Overall section classification

The section is class 1

Resistance of cross section (cl. 6.2)

Compression (cl. 6.2.4)

Design force $N_{Ed} = 181 \text{ kN}$

Design resistance $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0} = 1991 \text{ kN}$

$N_{Ed} / N_{c,Rd} = 0.091$

PASS - The compression design resistance exceeds the design force

Bending - Major axis (cl. 6.2.5)

Design bending moment $M_{y,Ed} = \max(\text{abs}(M_{y,Ed1}), \text{abs}(M_{y,Ed2})) = 48.2 \text{ kNm}$

Section modulus $W_y = W_{pl,y} = 372.2 \text{ cm}^3$

Design resistance $M_{c,y,Rd} = W_y \times f_y / \gamma_{M0} = 132.1 \text{ kNm}$

$M_{y,Ed} / M_{c,y,Rd} = 0.365$

PASS - The bending design resistance exceeds the design moment

Bending - Major axis (cl. 6.2.5)

Design bending moment $M_{z,Ed} = \max(\text{abs}(M_{z,Ed1}), \text{abs}(M_{z,Ed2})) = 30.1 \text{ kNm}$

Section modulus $W_z = W_{pl,z} = 168.7 \text{ cm}^3$

Design resistance $M_{c,z,Rd} = W_z \times f_y / \gamma_{M0} = 59.9 \text{ kNm}$

$M_{z,Ed} / M_{c,z,Rd} = 0.503$

PASS - The bending design resistance exceeds the design moment

Combined bending and axial force (cl. 6.2.9)

Ratio design axial to design plastic resistance $n = \text{abs}(N_{Ed}) / N_{pl,Rd} = 0.091$

Ratio web area to gross area

$$a = \min(0.5, (A - 2 \times b \times t_f) / A) = \mathbf{0.244}$$

Bending - Major axis (cl. 6.2.9.1)

Design bending moment

$$M_{y,Ed} = \max(\text{abs}(M_{y,Ed1}), \text{abs}(M_{y,Ed2})) = \mathbf{48.2 \text{ kNm}}$$

Plastic design resistance

$$M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = \mathbf{132.1 \text{ kNm}}$$

Modified design resistance

$$M_{N,y,Rd} = M_{pl,y,Rd} \times \min(1, (1 - n) / (1 - 0.5 \times a)) = \mathbf{132.1 \text{ kNm}}$$

$$M_{y,Ed} / M_{N,y,Rd} = \mathbf{0.365}$$

PASS - Bending resistance in presence of axial load exceeds design moment

Bending - Minor axis (cl. 6.2.9.1)

Design bending moment

$$M_{z,Ed} = \max(\text{abs}(M_{z,Ed1}), \text{abs}(M_{z,Ed2})) = \mathbf{30.1 \text{ kNm}}$$

Plastic design resistance

$$M_{pl,z,Rd} = W_{pl,z} \times f_y / \gamma_{M0} = \mathbf{59.9 \text{ kNm}}$$

Modified design resistance

$$M_{N,z,Rd} = M_{pl,z,Rd} = \mathbf{59.9 \text{ kNm}}$$

$$M_{z,Ed} / M_{N,z,Rd} = \mathbf{0.503}$$

PASS - Bending resistance in presence of axial load exceeds design moment

Biaxial bending

Exponent α

$$\alpha = \mathbf{2.00}$$

Exponent β

$$\beta = \max(1, 5 \times n) = \mathbf{1.00}$$

Section utilisation at end 1

$$UR_{CS_1} = [\text{abs}(M_{y,Ed1}) / M_{N,y,Rd}]^\alpha + [\text{abs}(M_{z,Ed1}) / M_{N,z,Rd}]^\beta = \mathbf{0.000}$$

Section utilisation at end 2

$$UR_{CS_2} = [\text{abs}(M_{y,Ed2}) / M_{N,y,Rd}]^\alpha + [\text{abs}(M_{z,Ed2}) / M_{N,z,Rd}]^\beta = \mathbf{0.636}$$

PASS - The cross-section resistance is adequate

Buckling resistance (cl. 6.3)

Yield strength for buckling resistance

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

Flexural buckling - Major axis

Elastic critical buckling force

$$N_{cr,y} = \pi^2 \times E \times I_y / L_{cr_y}^2 = \mathbf{9727 \text{ kN}}$$

Non-dimensional slenderness

$$\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} = \mathbf{0.452}$$

Buckling curve (Table 6.2)

b

Imperfection factor (Table 6.1)

$$\alpha_y = \mathbf{0.34}$$

Parameter Φ

$$\Phi_y = 0.5 \times [1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2] = \mathbf{0.645}$$

Reduction factor

$$\chi_y = \min(1.0, 1 / [\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}]) = \mathbf{0.905}$$

Design buckling resistance

$$N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} = \mathbf{1801.3 \text{ kN}}$$

$$N_{Ed} / N_{b,y,Rd} = \mathbf{0.1}$$

PASS - The flexural buckling resistance exceeds the design axial load

Flexural buckling - Minor axis

Elastic critical buckling force

$$N_{cr,z} = \pi^2 \times E \times I_z / L_{cr_z}^2 = \mathbf{3095 \text{ kN}}$$

Non-dimensional slenderness

$$\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})} = \mathbf{0.802}$$

Buckling curve (Table 6.2)

c

Imperfection factor (Table 6.1)

$$\alpha_z = \mathbf{0.49}$$

Parameter Φ

$$\Phi_z = 0.5 \times [1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2] = \mathbf{0.969}$$

Reduction factor

$$\chi_z = \min(1.0, 1 / [\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}]) = \mathbf{0.661}$$

Design buckling resistance

$$N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} = \mathbf{1315.8 \text{ kN}}$$

$$N_{Ed} / N_{b,z,Rd} = \mathbf{0.138}$$

PASS - The flexural buckling resistance exceeds the design axial load

Torsional and torsional-flexural buckling (cl. 6.3.1.4)

Torsional buckling length factor

$$K_T = \mathbf{1.00}$$

Effective buckling length

$$L_{cr_T} = K_T \times \max(L_y, L_z) = \mathbf{4200 \text{ mm}}$$

Distance from shear ctr to centroid along major axis

$$y_0 = 0.0 \text{ mm}$$

Distance from shear ctr to centroid along minor axis

$$z_0 = 0.0 \text{ mm}$$

$$i_0 = \sqrt{(i_y^2 + i_z^2 + y_0^2 + z_0^2)} = 79.7 \text{ mm}$$

$$\beta_T = 1 - (y_0 / i_0)^2 = 1.000$$

Elastic critical torsional buckling force

$$N_{cr,T} = 1 / i_0^2 \times (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) = 4953 \text{ kN}$$

Elastic critical torsional-flexural buckling force

$$N_{cr,TF} = N_{cr,y} / (2 \times \beta_T) \times [1 + N_{cr,T} / N_{cr,y} - \sqrt{[(1 - N_{cr,T} / N_{cr,y})^2 + 4 \times (y_0 / i_0)^2 \times N_{cr,T} / N_{cr,y}]]}$$
$$N_{cr,TF} = 4953 \text{ kN}$$

Non-dimensional slenderness

$$\bar{\lambda}_T = \sqrt{(A \times f_y / \min(N_{cr,T}, N_{cr,TF}))} = 0.634$$

Buckling curve (Table 6.2)

c

Imperfection factor (Table 6.1)

$$\alpha_T = 0.49$$

Parameter Φ

$$\Phi_T = 0.5 \times [1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2] = 0.807$$

Reduction factor

$$\chi_T = \min(1.0, 1 / [\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}]) = 0.765$$

Design buckling resistance

$$N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1} = 1523.3 \text{ kN}$$

$$N_{Ed} / N_{b,T,Rd} = 0.119$$

PASS - The torsional/torsional-flexural buckling resistance exceeds the design axial load

Minimum buckling resistance

Minimum buckling resistance

$$N_{b,Rd} = \min(N_{b,y,Rd}, N_{b,z,Rd}, N_{b,T,Rd}) = 1315.8 \text{ kN}$$

$$N_{Ed} / N_{b,Rd} = 0.138$$

PASS - The axial load buckling resistance exceeds the design axial load

Buckling resistance moment (cl.6.3.2.1)

Lateral torsional buckling length factor

$$K_{LT} = 1.00$$

Effective buckling length

$$L_{cr,LT} = K_{LT} \times L_z = 4200 \text{ mm}$$

End moment factor

$$\psi = M_{y,Ed1} / M_{y,Ed2} = 0.000$$

Moment distribution correction factor (Table 6.6)

$$k_c = 1 / (1.33 - 0.33 \times \psi) = 0.752$$

$$C_1 = 1 / k_c^2 = 1.769$$

Curvature factor

$$g = \sqrt{[1 - (I_z / I_y)]} = 0.826$$

Poissons ratio

$$\nu = 0.3$$

Shear modulus

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

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z \times \sqrt{[I_w / I_z + L_{cr,LT}^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} / (L_{cr,LT}^2 \times g)$$
$$M_{cr} = 382.0 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{[W_y \times f_y / M_{cr}]} = 0.588$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = 0.40$$

Correction factor for rolled sections

$$\beta_r = 0.75$$

Buckling curve (Table 6.5)

b

Imperfection factor (Table 6.1)

$$\alpha_{LT} = 0.34$$

Parameter Φ_{LT}

$$\Phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta_r \times \bar{\lambda}_{LT}^2] = 0.662$$

Reduction factor

$$\chi_{LT} = \min(1.0, 1 / \bar{\lambda}_{LT}^2, 1 / [\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta_r \times \bar{\lambda}_{LT}^2)}]) = 0.922$$

Modification factor

$$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 0.887$$

Modified LTB reduction factor - eq 6.58

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2) = 1.000$$

Design buckling resistance moment

$$M_{b,Rd} = \chi_{LT,mod} \times W_y \times f_y / \gamma_{M1} = 132.1 \text{ kNm}$$

Design bending moment

$$M_{y,Ed} = \max(\text{abs}(M_{y,Ed1}), \text{abs}(M_{y,Ed2})) = 48.2 \text{ kNm}$$

$$M_{y,Ed} / M_{b,Rd} = 0.365$$

PASS - The design buckling resistance moment exceeds the maximum design moment

Combined bending and axial compression (cl. 6.3.3)

Characteristic resistance to normal force $N_{Rk} = A \times f_y = \mathbf{1991 \text{ kN}}$

Characteristic moment resistance - Major axis $M_{y,Rk} = W_{pl,y} \times f_y = \mathbf{132.1 \text{ kNm}}$

Characteristic moment resistance - Minor axis $M_{z,Rk} = W_{pl,z} \times f_y = \mathbf{59.9 \text{ kNm}}$

$\psi_y = \text{if}(\text{abs}(M_{y,Ed1}) \leq \text{abs}(M_{y,Ed2}), M_{y,Ed1} / \text{if}(M_{y,Ed2} > 0 \text{ kNm}, \max(M_{y,Ed2}, 0.0001 \text{ kNm}), M_{y,Ed2}), M_{y,Ed2} / \text{if}(M_{y,Ed1} > 0 \text{ kNm}, \max(M_{y,Ed1}, 0.0001 \text{ kNm}), M_{y,Ed1})) = \mathbf{0.000}$

Moment distribution factor - Major axis $\psi_y = M_{y,Ed1} / M_{y,Ed2} = \mathbf{0.000}$

Moment factor - Major axis $C_{my} = \max(0.4, 0.6 + 0.4 \times \psi_y) = \mathbf{0.600}$

Moment distribution factor - Minor axis $\psi_z = M_{z,Ed1} / M_{z,Ed2} = \mathbf{0.000}$

Moment factor - Minor axis $C_{mz} = \max(0.4, 0.6 + 0.4 \times \psi_z) = \mathbf{0.600}$

Moment distribution factor for LTB $\psi_{LT} = M_{y,Ed1} / M_{y,Ed2} = \mathbf{0.000}$

Moment factor for LTB $C_{mLT} = \max(0.4, 0.6 + 0.4 \times \psi_{LT}) = \mathbf{0.600}$

Interaction factor $k_{yy} = C_{my} \times [1 + \min(0.8, \bar{\lambda}_y - 0.2) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})] = \mathbf{0.615}$

Interaction factor $k_{zy} = 1 - \min(0.1, 0.1 \times \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times (\chi_z \times N_{Rk} / \gamma_{M1})) = \mathbf{0.968}$

Interaction factor $k_{zz} = C_{mz} \times [1 + \min(1.4, 2 \times \bar{\lambda}_z - 0.6) \times N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1})] = \mathbf{0.683}$

Interaction factor $k_{yz} = 0.6 \times k_{zz} = \mathbf{0.410}$

Section utilisation $UR_{B_1} = N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) + k_{yz} \times M_{z,Ed} / (M_{z,Rk} / \gamma_{M1})$
 $UR_{B_1} = \mathbf{0.550}$

$UR_{B_2} = N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) + k_{zz} \times M_{z,Ed} / (M_{z,Rk} / \gamma_{M1})$
 $UR_{B_2} = \mathbf{0.864}$

PASS - The buckling resistance is adequate

MASONRY BEARING DESIGN

In accordance with EN1996-1-1:2005 + A1:2012, incorporating Corrigenda February 2006 and July 2009 and the UK National Annex.

Tedds calculation version 1.0.10

Summary table

Load	Local concentration		Spreader		Utilisation	
	Design force	Resistance	Design stress	Resistance		
1	124.4 kN	26.1 kN	1.51 N/mm ²	1.83 N/mm ²	0.826	Pass

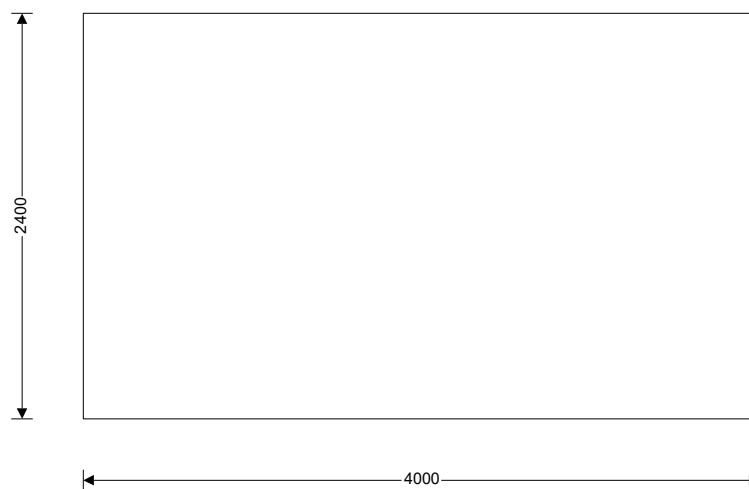
Masonry panel details

Panel length $L = 4000$ mm

Panel height $h = 2400$ mm

Panel support conditions

All edges supported



Effective height of masonry wall - Section 5.5.1.2

Vertical restraints

$L < 30 \times t_{ef}$ (Wall restrained at the top and bottom and stiffened on two vertical edges)

Horizontal restraints

Simply supported

Reduction factor - eq. 5.5

$\rho_2 = 1.00$

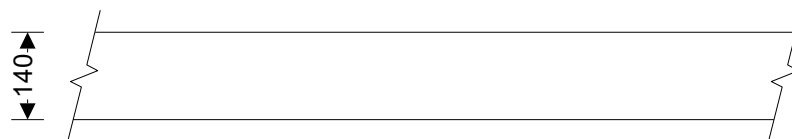
Reduction factor - eq. 5.8

$\rho_4 = 1/(1 + [(p_2 \times h) / L]^2) \times \rho_2 = 0.74$

Effective height of wall - eq. 5.2

$h_{ef} = \rho_4 \times h = 1765$ mm

Wall construction details



Wall type

Single leaf panel

Overall wall thickness

$t = 140.0$ mm

Effective thickness of masonry wall - Section 5.5.1.3

Effective thickness $t_{ef} = t = 140.0 \text{ mm}$

Masonry material details

Unit type

Clay - Group 1

Compressive strength of masonry unit

$f_c = 7.3 \text{ N/mm}^2$

Height of unit

$h_u = 215 \text{ mm}$

Width of unit

$w_u = 140 \text{ mm}$

Conditioning factor

$k = 1.0$

- Conditioning to the air dry condition in accordance with cl.7.3.2

Shape factor - Table A.1

$d_{sf} = 1.3$

Mean compressive strength of masonry unit

$f_b = f_c \times k \times d_{sf} = 9.49 \text{ N/mm}^2$

Specific weight of units

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

Mortar type

M4 - General Purpose

Compressive strength of mortar

$f_m = 4.0 \text{ N/mm}^2$

Compressive strength factor - Tbl. NA 4

$K = 0.50$

Characteristic compressive strength - eq. 3.1

$f_k = K \times f_b^{0.7} \times f_m^{0.3} = 3.66 \text{ N/mm}^2$

Short term secant modulus of elasticity factor

$K_E = 1000$

Modulus of elasticity - cl.3.7.2

$E_w = K_E \times f_k = 3662 \text{ N/mm}^2$

Design compressive strength of masonry

Category of manufacturing control

Category II

Class of execution control

Class 2

Partial factor for compressive strength

$\gamma_M = 3.00$

Cross-sectional area of wall

$A = L \times t = 0.56 \text{ m}^2$

Design compressive strength of masonry

$f_d = f_k / \gamma_M = 1.22 \text{ N/mm}^2$

Partial safety factors for design loads

Partial safety factor for permanent load

$\gamma_{FG} = 1.35$

Partial safety factor for variable load

$\gamma_{FQ} = 1.50$

Superimposed vertical loading details

Permanent UDL at top of wall

$g_k = 0.00 \text{ kN/m}$

Variable UDL at top of wall

$q_k = 0.00 \text{ kN/m}$

Eccentricity of permanent UDL load

$e_{gu} = 0 \text{ mm}$

Eccentricity of variable UDL load

$e_{qu} = 0 \text{ mm}$

Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit

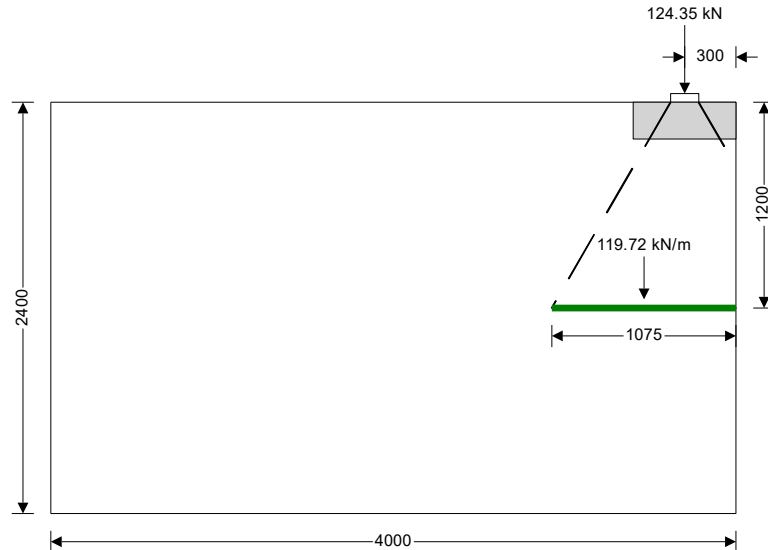
$\lambda_{lim} = 27$

Slenderness ratio

$\lambda = h_{ef} / t_{ef} = 12.6$

PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details



Permanent concentrated load	$G_{kc1} = 61.00 \text{ kN}$
Variable concentrated load	$Q_{kc1} = 28.00 \text{ kN}$
Eccentricity of concentrated load	$e_{c1} = 0 \text{ mm}$
Length of concentrated load	$L_{c1} = 165 \text{ mm}$
Width of concentrated load	$w_{c1} = 100 \text{ mm}$
Height of concentrated load	$h_{c1} = 2400 \text{ mm}$
Distance of load to right vertical edge	$r_{11} = 217 \text{ mm}$
Distance of load to nearest vertical edge	$a_{11} = 217 \text{ mm}$

Walls subjected to concentrated loads - Section 6.1.3

Eccentricity check	$e_{c1} \leq t / 4$
--------------------	---------------------

PASS - Eccentricity of load is less than $t/4$

Area of bearing	$A_{b1} = L_{c1} \times w_{c1} = 16500 \text{ mm}^2$
Effective length of bearing at mid-height	$l_{efm1} = L_{c1} + h_{c1} / 2 \times \tan(30) + r_{11} = 1075 \text{ mm}$
Effective bearing area	$A_{ef1} = l_{efm1} \times t = 150545 \text{ mm}^2$
Bearing area ratio check	$A_{ratio1} = \text{Min}(A_{b1} / A_{ef1}, 0.45) = 0.11$
Initial enhancement factor	$\beta_{init1} = \text{Max}((1 + 0.3 \times a_{11} / h_{c1}) \times (1.5 - 1.1 \times A_{ratio1}), 1.0) = 1.42$
Maximum enhancement factor	$\beta_{max1} = \text{Min}(1.25 + a_{11} / (2 \times h_{c1}), 1.5) = 1.30$
Enhancement factor for concentrated loads	$\beta_1 = \text{Min}(\beta_{init1}, \beta_{max1}) = 1.30$
Design value of the concentrated load	$N_{Edc1} = G_{kc1} \times \gamma_G + Q_{kc1} \times \gamma_Q = 124.35 \text{ kN}$
Design value concentrated load resistance	$N_{Rdc1} = \beta_1 \times A_{b1} \times f_d = 26.09 \text{ kN}$

Applied concentrated load exceeds design resistance, spreader required!

Design of spreader beam

Type of spreader	Concrete padstone
Type of bearing onto spreader	Uniformly distributed
Point load as a UDL	$N_{udl1} = N_{Edc1} / L_{c1} = 753.64 \text{ kN/m}$
Start of load from RHS of spreader	$P_{start1} = 382.50 \text{ mm}$
End of load from RHS of spreader	$P_{end1} = 217.50 \text{ mm}$
Length of spreader	$L_{sp1} = 600.00 \text{ mm}$

Height of spreader	$h_{sp1} = \mathbf{215.00 \text{ mm}}$
Width of spreader	$w_{sp1} = \mathbf{140.00 \text{ mm}}$
Eccentricity of load on spreader	$e_{sp1} = \mathbf{0 \text{ mm}}$
Modulus of elasticity	$E_{sp1} = \mathbf{29962 \text{ N/mm}^2}$
Second moment of area	$I_{sp1} = 1/12 \times w_{sp1} \times h_{sp1}^3 = \mathbf{115947708 \text{ mm}^4}$
Modulus of the wall	$k_0 = E_w / h = \mathbf{1.53 \text{ N/mm}^2/\text{mm}}$
Winkler's constant	$K_{c1} = k_0 \times w_{sp1} = \mathbf{213.60 \text{ N/mm/mm}}$
Characteristic of the system	$\alpha_1 = (K_{c1} / (4 \times E_{sp1} \times I_{sp1}))^{1/4} = \mathbf{0.00198 \text{ mm}^{-1}}$
Classification of spreader	$\alpha L_1 = \alpha_1 \times L_{sp1} = \mathbf{1.19 \text{ Medium}}$
Krilov's functions for the spreader length	$B_{\alpha 1} = 1/2 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) + \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi)) = \mathbf{1.11}$ $C_{\alpha 1} = 1/2 \times \sinh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) = \mathbf{0.69}$ $D_{\alpha 1} = 1/4 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) - \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi)) = \mathbf{0.28}$
Krilov's functions at the start of the load	$B_{\alpha P_{start1}} = 1/2 \times (\cosh(\alpha_1 \times P_{start1}) \times \sin(180 \times \alpha_1 \times P_{start1} / \pi) + \sinh(\alpha_1 \times P_{start1}) \times \cos(180 \times \alpha_1 \times P_{start1} / \pi)) = \mathbf{0.75}$ $C_{\alpha P_{start1}} = 1/2 \times \sinh(\alpha_1 \times P_{start1}) \times \sin(180 \times \alpha_1 \times P_{start1} / \pi) = \mathbf{0.29}$
Krilov's functions at the end of the load	$B_{\alpha P_{end1}} = 1/2 \times (\cosh(\alpha_1 \times P_{end1}) \times \sin(180 \times \alpha_1 \times P_{end1} / \pi) + \sinh(\alpha_1 \times P_{end1}) \times \cos(180 \times \alpha_1 \times P_{end1} / \pi)) = \mathbf{0.43}$ $C_{\alpha P_{end1}} = 1/2 \times \sinh(\alpha_1 \times P_{end1}) \times \sin(180 \times \alpha_1 \times P_{end1} / \pi) = \mathbf{0.09}$
Using method of initial conditions	
Initial moment of LH edge	$M_{01} = \mathbf{0 \text{ kNm}}$
Initial shear of LH edge	$V_{01} = \mathbf{0 \text{ kN}}$
Which gives	$(4 \times \alpha_1^2 \times C_{\alpha 1} \times \delta_{01} + 4 \times \alpha_1 \times D_{\alpha 1} \times \Phi_{01}) \times E_{sp1} \times I_{sp1} - N_{ud1} / \alpha_1^2 \times (C_{\alpha P_{start1}} - C_{\alpha P_{end1}}) = \mathbf{0.00 \text{ kNm}}$ $(4 \times \alpha_1^3 \times B_{\alpha 1} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{\alpha 1} \times \Phi_{01}) \times E_{sp1} \times I_{sp1} - N_{ud1} / \alpha_1 \times (B_{\alpha P_{start1}} - B_{\alpha P_{end1}}) = \mathbf{0.00 \text{ kN}}$
and	
Therefore,	
Initial deflection of LH edge	$\delta_{01} = \mathbf{0.93761 \text{ mm}}$
Initial rotation of LH edge	$\Phi_{01} = \mathbf{0.000244}$
Location of maximum deflection	$x_{def1} = \mathbf{300 \text{ mm}}$
Krilov's functions at the spreader length	$A_{\alpha x_{def1}} = \cosh(\alpha_1 \times x_{def1}) \times \cos(180 \times \alpha_1 \times x_{def1} / \pi) = \mathbf{0.98}$ $B_{\alpha x_{def1}} = 1/2 \times (\cosh(\alpha_1 \times x_{def1}) \times \sin(180 \times \alpha_1 \times x_{def1} / \pi) + \sinh(\alpha_1 \times x_{def1}) \times \cos(180 \times \alpha_1 \times x_{def1} / \pi)) = \mathbf{0.59}$ $p_{startdef1} = \mathbf{82.5 \text{ mm}}$ $A_{\alpha p_{startdef1}} = \cosh(\alpha_1 \times p_{startdef1}) \times \cos(180 \times \alpha_1 \times p_{startdef1} / \pi) = \mathbf{1.00}$ $p_{enddef1} = \mathbf{0 \text{ mm}}$ $A_{\alpha p_{enddef1}} = \cosh(\alpha_1 \times p_{enddef1}) \times \cos(180 \times \alpha_1 \times p_{enddef1} / \pi) = \mathbf{1.00}$ $\delta'_1 = (-N_{ud1} / (4 \times \alpha_1^4) \times (A_{\alpha p_{startdef1}} - A_{\alpha p_{enddef1}})) / (I_{sp1} \times E_{sp1}) = \mathbf{0.00042}$
Distance from start load right of location	
Krilov's functions at the spreader length	
Distance from end load right of location	
Krilov's functions at the spreader length	
Particular integral due to load	
mm	
Maximum deflection	$\delta_{max1} = A_{\alpha x_{def1}} \times \delta_{01} + B_{\alpha x_{def1}} \times \Phi_{01} / \alpha_1 + \delta'_1 = \mathbf{0.99148 \text{ mm}}$
Location of maximum moment	$x_{M1} = \mathbf{300 \text{ mm}}$
Krilov's functions at the spreader length	$C_{\alpha x_{M1}} = 1/2 \times \sinh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi) = \mathbf{0.18}$

Distance from start load right of location
Krilov's functions at the spreader length
Distance from end load right of location
Krilov's functions at the spreader length
Particular integral due to load
Maximum moment

$$D_{\alpha x M1} = 1/4 \times (\cosh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi) - \sinh(\alpha_1 \times x_{M1}) \times \cos(180 \times \alpha_1 \times x_{M1} / \pi)) = \mathbf{0.03}$$

$$p_{start M1} = \mathbf{82.5 \text{ mm}}$$

$$C_{\alpha p start M1} = 1/2 \times \sinh(\alpha_1 \times p_{start M1}) \times \sin(180 \times \alpha_1 \times p_{start M1} / \pi) = \mathbf{0.01}$$

$$p_{end M1} = \mathbf{0 \text{ mm}}$$

$$C_{\alpha p end M1} = 1/2 \times \sinh(\alpha_1 \times p_{end M1}) \times \sin(180 \times \alpha_1 \times p_{end M1} / \pi) = \mathbf{0.00}$$

$$M'_1 = -N_{udl1} / \alpha_1^2 \times (C_{\alpha p start M1} - C_{\alpha p end M1}) = \mathbf{-2.56 \text{ kNm}}$$

$$M_{Edsp1} = (4 \times \alpha_1^2 \times C_{\alpha x M1} \times \delta_{01} + 4 \times \alpha_1 \times D_{\alpha x M1} \times \Phi_{01}) \times (I_{sp1} \times E_{sp1}) + M'_1 = \mathbf{6.67 \text{ kNm}}$$

Location of maximum shear
Krilov's functions at the spreader length

$$x_{V1} = \mathbf{382.5 \text{ mm}}$$

$$B_{\alpha x V1} = 1/2 \times (\cosh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi) + \sinh(\alpha_1 \times x_{V1}) \times \cos(180 \times \alpha_1 \times x_{V1} / \pi)) = \mathbf{0.75}$$

Distance from start load right of location
Krilov's functions at the spreader length

$$C_{\alpha x V1} = 1/2 \times \sinh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi) = \mathbf{0.29}$$

Distance from end load right of location
Krilov's functions at the spreader length

$$p_{start V1} = \mathbf{165 \text{ mm}}$$

$$B_{\alpha p start V1} = 1/2 \times (\cosh(\alpha_1 \times p_{start V1}) \times \sin(180 \times \alpha_1 \times p_{start V1} / \pi) + \sinh(\alpha_1 \times p_{start V1}) \times \cos(180 \times \alpha_1 \times p_{start V1} / \pi)) = \mathbf{0.33}$$

$$p_{end V1} = \mathbf{0 \text{ mm}}$$

$$B_{\alpha p end V1} = 1/2 \times (\cosh(\alpha_1 \times p_{end V1}) \times \sin(180 \times \alpha_1 \times p_{end V1} / \pi) + \sinh(\alpha_1 \times p_{end V1}) \times \cos(180 \times \alpha_1 \times p_{end V1} / \pi)) = \mathbf{0.00}$$

Particular integral due to load
Maximum shear

$$V'_1 = -N_{udl1} / \alpha_1 \times (B_{\alpha p start V1} - B_{\alpha p end V1}) = \mathbf{-124.30 \text{ kN}}$$

$$V_{Edsp1} = (4 \times \alpha_1^3 \times B_{\alpha x V1} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{\alpha x V1} \times \Phi_{01}) \times (I_{sp1} \times E_{sp1}) + V'_1 = \mathbf{-44.74 \text{ kN}}$$

Maximum allowable stress under spreader
Maximum reaction
Design stress

$$\sigma_{Rdsp1} = 1.5 \times f_d = \mathbf{1.83 \text{ N/mm}^2}$$

$$N_{Edsp1} = K_{c1} \times \delta_{max1} = \mathbf{211.78 \text{ kN/m}}$$

$$\sigma_{Edsp1} = N_{Edsp1} / W_{sp1} = \mathbf{1.51 \text{ N/mm}^2}$$

PASS - Design stress under spreader is less than the allowable bearing stress

Walls subjected to mainly vertical loading - Section 6.1.2

Eccentricity of permanent UDL at mid-height below concentrated load

$$e_{gmu1} = e_{gu} \times h_{c1} / (2 \times h) = \mathbf{0.0 \text{ mm}}$$

Eccentricity of variable UDL at mid-height below concentrated load

$$e_{qmu1} = e_{qu} \times h_{c1} / (2 \times h) = \mathbf{0.0 \text{ mm}}$$

Eccentricity of concentrated load at mid-height

$$e_{mc1} = e_{c1} / 2 = \mathbf{0.0 \text{ mm}}$$

Initial eccentricity - cl.5.5.1.1(4)

$$e_{init} = h / 450 = \mathbf{5.3 \text{ mm}}$$

Concentrated load at mid-height as UDL

$$N_{mc1} = N_{Edc1} / l_{efm1} = \mathbf{115.64 \text{ kN/m}}$$

Vertical load at mid-height

$$N_{Ed1} = (g_k + \gamma \times t \times (h - h_{c1} / 2)) \times \gamma_{fG} + q_k \times \gamma_{fQ} + N_{mc1} = \mathbf{119.72 \text{ kN/m}}$$

Design moment at mid-height

$$M_{Ed1} = g_k \times \gamma_{fG} \times e_{gmu1} + q_k \times \gamma_{fQ} \times e_{qmu1} + N_{mc1} \times e_{mc1} = \mathbf{0.00 \text{ kNm/m}}$$

Eccentricities due to loads - eq. 6.7

$$e_{m1} = \text{Abs}(M_{Ed1}) / N_{Ed1} + e_{init} = \mathbf{5.3 \text{ mm}}$$

Slenderness ratio limit for creep eccentricity

$$\lambda_c = \mathbf{27}$$

Eccentricity due to creep

$$e_{k1} = \mathbf{0.0 \text{ mm}}$$

Eccentricity at mid-height - eq. 6.6

$$e_{mk1} = \text{Max}(e_{m1} + e_{k1}, 0.05 \times t) = \mathbf{7.0 \text{ mm}}$$

From eq. G2

$$A_{11} = 1 - 2 \times e_{mk1} / t = \mathbf{0.90}$$

From eq. G3

$$u_1 = (h_{ef} / t_{ef} \times (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 \times e_{mk1} / t) = \mathbf{0.50}$$

Capacity reduction factor - eq. G1

$$\Phi_{m1} = A_{11} \times \exp(-(u_1^2) / 2) = \mathbf{0.79}$$

Design vertical resistance of panel - eq.6.2

$$N_{Rd1} = \Phi_{m1} \times t \times f_d = \mathbf{135.73 \text{ kN/m}}$$

PASS - Design value of vertical resistance exceeds applied vertical load

Project

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Simple Foundation Design in Accordance with Eurocodes: Vertical Loads Only

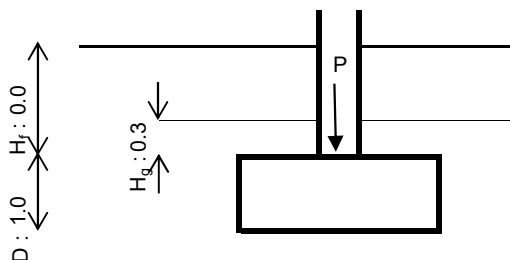
Description : 203 UC Foundation design
 Column Size : 203 203 46 ~UC
 Baseplate Length : 0.35 m
 Baseplate Width : 0.35 m
 Presumed allowable net bearing resistance : 50 kN/m²

ACTIONS

Permanent P_g : 155.0 kN
 Variable 1 $P_{q,1}$: 63.0 kN
 Variable 2 $P_{q,2}$: -10.0 kN (Uplift)
 s/w of fill γ_{fill} : 19.0 kN/m³
 s/w natural ground γ_{ground} : 24.0 kN/m⁴

SIZING

Length L : 2.00 m
 Width B : 2.00 m
 Depth D : 1.00 m
 Depth of Fill H_f : 0.00 m
 Original GL H_g : 0.25 m
 SLS Load P_{SLS} : 218.0 kN
 S/W Concrete P_{conc} : 100.0 kN
 S/W Fill P_{conc} : 0.0 kN
 Gross bearing pressure : 79.5 kN/m²
 Original ground pressure : 30.0 kN/m²



Nett Bearing Pressure : **49.5 kN/m² < 50 kN/m² : PASS**

Uplift Verification : 0.9 x 255.0 + 1.5 x -10.0 : **214.5 kN > 0 : PASS**

PLAIN CONCRETE CHECK

Concrete strength Grade : C28/35 f_{ctd} : 1.27 N/mm²
 Design Action : 1.35 x 255.0 + 1.5 x 63.0 : **438.75 kN**
 Design bearing pressure σ_{gd} : 438750 / 4E+06 : 0.1 N/mm²
 Outstand dimension a : 825 mm

Minimum foundation depth for plain concrete h_f : $\frac{825}{0.85} \times \frac{9 \times 0.1^{0.5}}{1.27}$: 856.85 mm

D > h_f therefore reinforcement is not required

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Simple Foundation Design in Accordance with Eurocodes: Vertical Loads Only

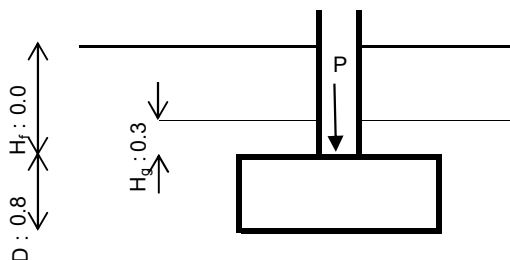
Description : 152 UC Foundation design
 Column Size : 152x152.UC
 Baseplate Length : 0.35 m
 Baseplate Width : 0.35 m
 Presumed allowable net bearing resistance : 50 kN/m²

ACTIONS

Permanent P_g : 96.2 kN
 Variable 1 $P_{q,1}$: 34.1 kN
 Variable 2 $P_{q,2}$: -0.1 kN (Uplift)
 s/w of fill γ_{fill} : 19.0 kN/m³
 s/w natural ground γ_{ground} : 24.0 kN/m⁴

SIZING

Length L : 1.75 m
 Width B : 1.75 m
 Depth D : 0.75 m
 Depth of Fill H_f : 0.00 m
 Original GL H_g : 0.25 m
 SLS Load P_{SLS} : 130.3 kN
 S/W Concrete P_{conc} : 57.4 kN
 S/W Fill P_{conc} : 0.0 kN
 Gross bearing pressure : 61.3 kN/m²
 Original ground pressure : 24.0 kN/m²



Nett Bearing Pressure : **37.3 kN/m²** < **50 kN/m²** : **PASS**

Uplift Verification : 0.9 x 153.6 + 1.5 x -0.1 : **138.11 kN** > **0** : **PASS**

PLAIN CONCRETE CHECK

Concrete strength Grade : C28/35 f_{ctd} : 1.27 N/mm²
 Design Action : 1.35 x 153.6 + 1.5 x 34.1 : **258.54 kN**
 Design bearing pressure σ_{gd} : 258540 / 3E+06 : 0.1 N/mm²
 Outstand dimension a : 700 mm

$$\text{Minimum foundation depth for plain concrete } h_f : \frac{700}{0.85} \times \frac{9 \times 0.1^{0.5}}{1.27} : 637.81 \text{ mm}$$

D > h_f therefore reinforcement is not required