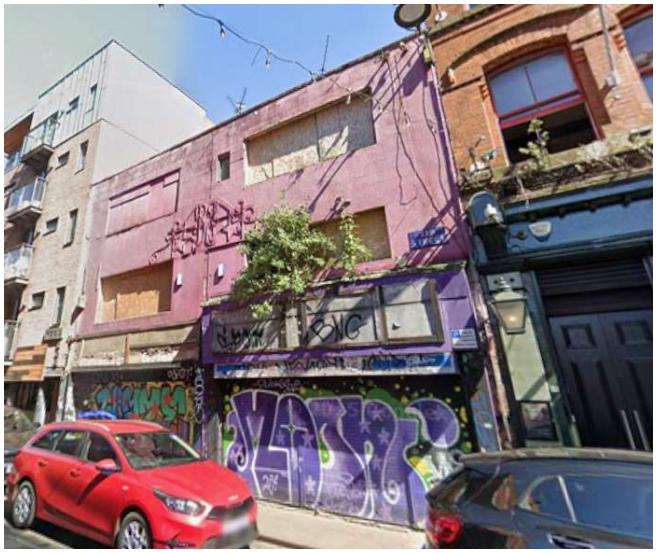
RD-ENGINEERING
Structural & Architectural Solutions

STRUCTURAL APPRAISAL FOR ADDITIONAL TWO STOREY EXTENSION



THE PROPERTY:

78-80 Tib Street Manchester M4 1LG	Lau Yuk Ltd.

PREPARED FOR:

SURVEY DATE:
20th October 2025

PREPARED BY:
Mr Chris Donnelly – Structural Engineer / Director
B.Sc (Hons), M.Sc, I.Eng, MICE

REPORT REFERENCE:
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STRUCTURAL REPORT at:
78-80 Tib Street, Manchester, M4 1LG

Prepared By: Mr Chris Donnelly
B.Sc (Hons), M.Sc, I.Eng, MICE

Approved by: Mr Thomas Rankin
B.Sc (Hons), M.Sc

Signed: C. Donnelly

Signed: T. Rankin

Date: 4th November 2025

Date: 5th November 2025

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2. SURVEY SUMMARY

The client appointed RD Engineering Associates Ltd. to undertake a Structural Condition Assessment of the mid terrace commercial premises at 78-80 Tib Street, Manchester, M4 1LG.

The client, who proposes to renovate the premises and install an additional two storeys as detailed in Appendix A

As part of our professional service, a full visual survey of the property was undertaken to investigate structural movement. The aim of the structural report is to undertake an investigation into the root cause of any identified structural issues, concluding with appropriate repair strategies and/or further investigation works to be undertaken.

The survey consisted of a non-intrusive survey of the property.

The surveying engineer was not able to inspect the foundations, internal masonry, (overlain with plasterboard/plaster finishes), or other parts of the building which were covered, unexposed or inaccessible and we are therefore unable to report that any such part of the building is free from defect from the visual inspection.

Furthermore, no investigations have been undertaken to determine whether asbestos products or other hazardous materials were used in the construction, and we are therefore unable to report that the building is free from risk in this respect.

3. DETAILS OF INSTRUCTION

Address of Property Surveyed	78-80 Tib Street Manchester M4 1LG.
Client Name	Lau Yuk Ltd.
Client Address if different from above	N/A
Terms of Reference	Structural Appraisal of Premises for Additional two Storeys.
Date of Inspection	20th October 2025
Other Information	None

4. ABOUT THE PROPERTY

4.1 DESCRIPTION OF PROPERTY

The premises is currently an abandoned three-story terraced building in Manchester city centre, located in the “Northern Quarter.”

The property was previously a commercial premises operating as a shop and is thought to have been constructed between 1900-1920.

The property is of traditional construction with masonry load bearing walls, suspended timber floors and a timber flat roof structure. The engineers observed that the majority of the lintels were timber and were in poor condition.

Structural steel beams are used to support the 1st floor joists, which are supported off the party walls and internal steel columns.

The ground floor was observed to be a ground bearing concrete slab.

4.2 LOCAL ENVIRONMENT

4.2.1 GEOLOGY

British Geological Survey (BGS) maps indicate the immediate superficial deposits, which are likely to be supporting the building foundations as “Till – Boulder Clay”.

Till is defined as unsorted material deposited directly by glacial ice and showing no stratification. Till is sometimes called boulder clay because it is composed of clay, boulders of intermediate sizes, or a mixture of these. The rock fragments are usually angular and sharp rather than rounded, because they are deposited from the ice and have undergone little water transport.

The bedrock below the superficial deposits is highlighted as the “Cannel Rock (South Lancashire) - comprising of sandstone.

4.2.2 GEOTECHNICAL DESKTOP STUDY

A review of the historical borehole information revealed several boreholes around the property as shown in Figure 1 below:

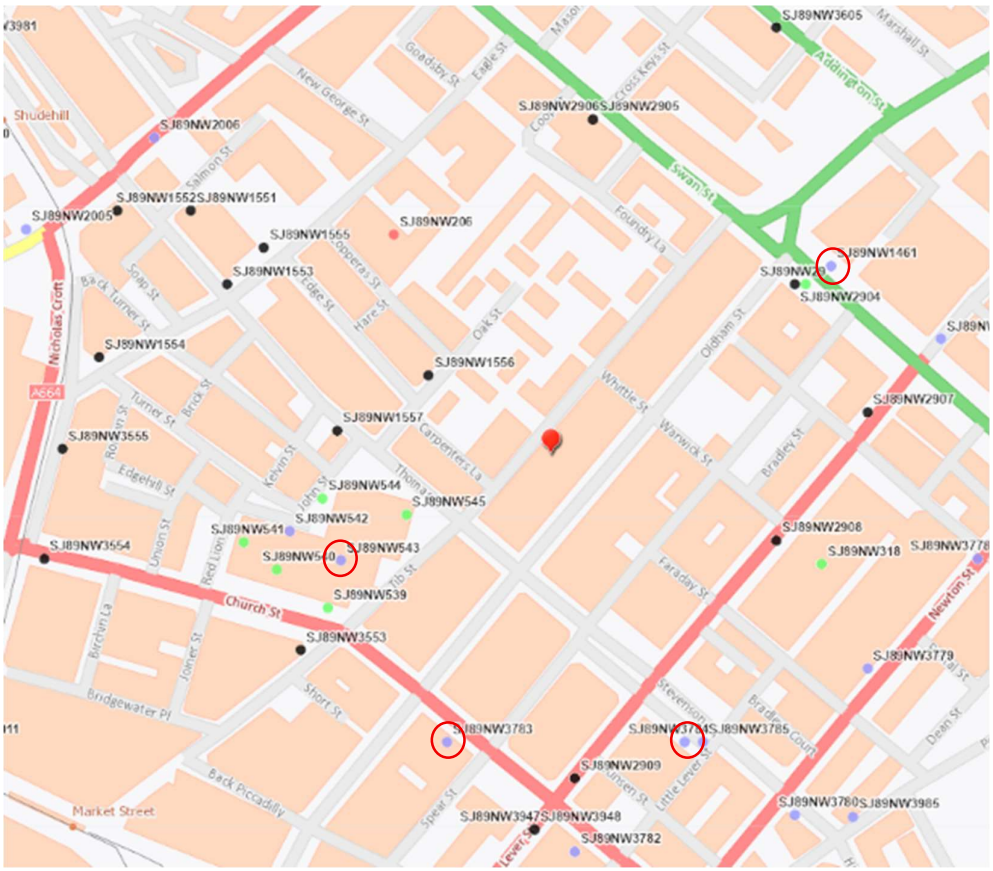


Figure 6.1 - British Geological Historical Borehole viewer results for postcode M4 1LG

Unfortunately, no historical site investigation information was freely available in close proximity to the premises and as such the client is advised to undertake a full site investigation to determine local ground conditions.

SJ 89 NW / 543

HOLST & CO. LTD.
SITE INVESTIGATION DEPT.
84412 98519
PARKSIDE LANE
LEEDS LS11 5SX.

Contract No. M4043/F1612 Borehole No. 5
Location Church Street, Ground Level 150.2
Client Manchester City Engineer Date 19th August, 1970

BOREHOLE LOG

STRATA	Legend	Depth below Ground Level	Thickness of Strata	Type of Sample	c lb/sq. ft.	ø deg.	m.c %	y lb/cu. ft.	N
ROAD SURFACING		3"	3"						
COBBLE STONES		1'0"	9"						
MADE GROUND: Soft CLAY		3'6"	2'6"	4'0" []	3200	6	19.3	126	
Stiff brown boulder CLAY				8'6" []	4800	0	18.5	129	
			21'6"						
				13'6" []	4400	0	18.9	136	
				18'6" []	1500	0	20.3	123	
		25'0"		23'6" []	2100	0	21.5	126	

Water Struck at None Encountered — Maximum Observed Water Level

Undisturbed Sample []
Disturbed Sample o
Water Sample ▲
Penetration Test I

c = Cohesion
ø = Angle of Internal Friction
m.c = Moisture Content
y = Bulk Density
N = Standard Penetration Value

Water levels are subject to seasonal or tidal variation and should not be taken as constant

British Geological Survey

Figure 6.2: Borehole Number SJ89NW543

MANCHESTER CITY COUNCIL

CITY ENGINEER AND SURVEYORS DEPARTMENT

SITE INVESTIGATION FOR Main Drainage — DATE COMMENCED 24. 4. 1983

LOCATION Newton Street REF: — BORING METHOD Cable Percussive

BOREHOLE No. 6 — BORING DIAMETER 150mm — 384478

GRID REFERENCE — SCALE: 1.50 (Approx) — 398423

DATE	CASING DEPTH (m)	WATER LEVEL MORNING & EVENING	DEPTH OF STRATA BELOW GL	Reduced Level	DESCRIPTION	No. and Type of Sample	SAMPLE DEPTH PENETRATION From	To	No. of Blows	Legend
24.4.83			0.15		Bituminous surfacing					
			0.35		Concrete					
			0.85		Clay, stone, ashes, brick pieces	D1	0.50			
					Stiff/very stiff brown silty and sandy CLAY with grey fissures, stone and coal inclusions	D2	1.0			
						U1	1.50	1.95		
						D3	2.0			
						U2	2.50	2.95		
						04	3.0			
						U3	3.50	3.95		
						D5	4.0			
						U4	4.5	4.95		
						D6	5.0			
						U5	5.50	5.95		
						D7	6.0			
						U6	6.50	6.95		
4.4.83	6.00	Dry	7.00							

WATER OBSERVATIONS DURING BORING

DEPTH OF WATER ENTRY (m)	ESTIMATED SEE PAGE	STANDING LEVEL (m)	DEPTH OF CASING WHEN SEALED (m)
None			

WATER LEVEL IN CASED/UNCASED BOREHOLE ON COMPLETION

DATE	WATER LEVEL	DEPTH OF CASING

REMARKS:–

SAMPLE TEST KEY

D – DISTURBED SAMPLE B – BULK SAMPLE U – UNDISTURBED SAMPLE
P – SPT SAMPLE W – WATER SAMPLE

NOTE: WHERE FULL 0.3 PENETRATION HAS NOT BEEN ACHIEVED, THE NUMBER OF BLOWS FOR THE QUOTED PENETRATION IS GIVEN (NOT N – VALUE)

ALL DEPTHS AND REDUCED LEVELS IN (m) UNLESS OTHERWISE STATED

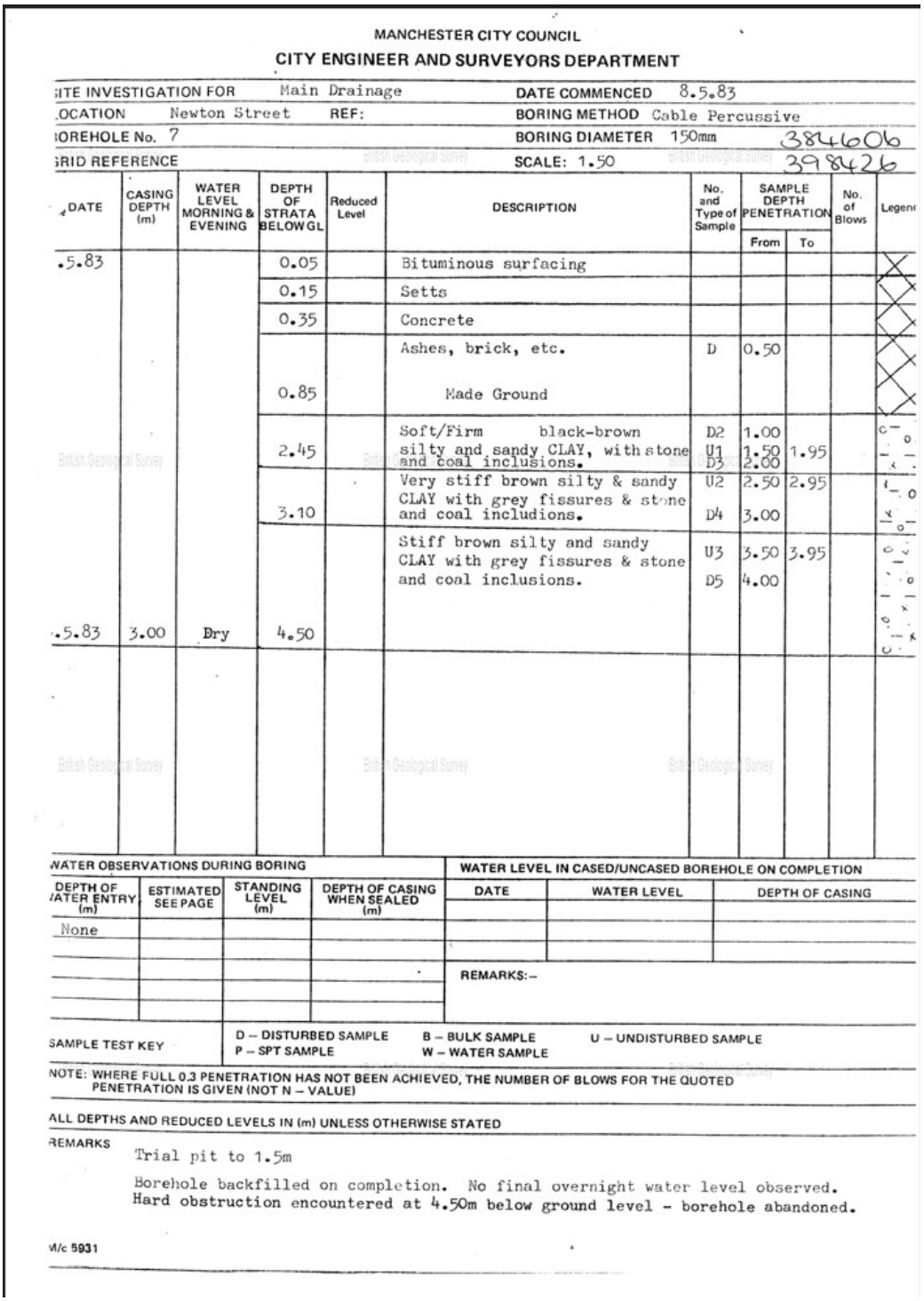
REMARKS

Trial hole excavated to 1.50 metres

Borehole backfilled on completion. No final overnight water level observed.

M/c 5931

Figure 6.3: Borehole Number SJ89NW3783



MANCHESTER CITY COUNCIL

CITY ENGINEER AND SURVEYORS DEPARTMENT

SITE INVESTIGATION FOR Main Drainage — DATE COMMENCED 8.5.83

LOCATION Newton Street REF: — BORING METHOD Cable Percussive

BOREHOLE No. 7 — BORING DIAMETER 150mm — 384606

GRID REFERENCE — SCALE: 1.50 — 398426

DATE	CASING DEPTH (m)	WATER LEVEL MORNING & EVENING	DEPTH OF STRATA BELOW GL	Reduced Level	DESCRIPTION	No. and Type of Sample	SAMPLE DEPTH PENETRATION From	To	No. of Blows	Legend
8.5.83			0.05		Bituminous surfacing					
			0.15		Setts					
			0.35		Concrete					
			0.85		Ashes, brick, etc. Made Ground	D	0.50			
			2.45		Soft/Firm black-brown silty and sandy CLAY, with stone and coal inclusions.	D2 U1 D3	1.00 1.50 2.00	1.95		
			3.10		Very stiff brown silty & sandy CLAY with grey fissures & stone and coal includions.	U2 D4	2.50 3.00	2.95		
8.5.83	3.00	Dry	4.50		Stiff brown silty and sandy CLAY with grey fissures & stone and coal inclusions.	U3 D5	3.50 4.00	3.95		

WATER OBSERVATIONS DURING BORING

DEPTH OF WATER ENTRY (m)	ESTIMATED SEE PAGE	STANDING LEVEL (m)	DEPTH OF CASING WHEN SEALED (m)
None			

WATER LEVEL IN CASED/UNCASED BOREHOLE ON COMPLETION

DATE	WATER LEVEL	DEPTH OF CASING

REMARKS:-

SAMPLE TEST KEY — D – DISTURBED SAMPLE; B – BULK SAMPLE; U – UNDISTURBED SAMPLE; P – SPT SAMPLE; W – WATER SAMPLE

NOTE: WHERE FULL 0.3 PENETRATION HAS NOT BEEN ACHIEVED, THE NUMBER OF BLOWS FOR THE QUOTED PENETRATION IS GIVEN (NOT N – VALUE)

ALL DEPTHS AND REDUCED LEVELS IN (m) UNLESS OTHERWISE STATED

REMARKS

Trial pit to 1.5m

Borehole backfilled on completion. No final overnight water level observed.
Hard obstruction encountered at 4.50m below ground level - borehole abandoned.

M/c 5931

Figure 6.4: Borehole Number SJ89NW3784

MANCHESTER CITY COUNCIL SJ89NW

CITY ENGINEER AND SURVEYORS DEPARTMENT 8468 9868 1461

INVESTIGATION FOR	Main Drainage	DATE COMMENCED	28.3.83
LOCATION	Great Ancoats Street REF: 83/6	BORING METHOD	Cable Percussive
BOREHOLE No.	1	BORING DIAMETER	150mm
B.D REFERENCE		SCALE:	1:50

DATE	CASING DEPTH (m)	WATER LEVEL MORNING & EVENING	DEPTH OF STRATA BELOW GL	Reduced Level	DESCRIPTION	No. and Type of Sample	SAMPLE DEPTH PENETRATION From	SAMPLE DEPTH PENETRATION To	No. of Blows	Legend
28.3.83			0.05		Bituminous Surfacing					
			0.15		Setts					
			0.30		Concrete					
					Bricks, ashes, stones etc. Made Ground.	D1	0.50			
						D2	1.00			
			1.25							
					Very stiff brown silty and sandy CLAY with grey fissures and stone inclusions.	U1	1.50	1.95		
						D3	2.00			
						U2	2.50	2.95		
						D4	3.00			
						U3	3.50	3.95		
			3.80							
						D5	4.00			
						U4	4.50	4.95		
					Stiff brown silty and sandy CLAY with stone inclusions, with sand lens	D6	5.00			
						U5	5.50			
						D7	6.00			

WATER OBSERVATIONS DURING BORING

DEPTH OF WATER ENTRY (m)	ESTIMATED SEE PAGE	STANDING LEVEL (m)	DEPTH OF CASING WHEN SEALED (m)
None			

WATER LEVEL IN CASED/UNCASED BOREHOLE ON COMPLETION

DATE	WATER LEVEL	DEPTH OF CASING

REMARKS:-

SAMPLE TEST KEY: D – DISTURBED SAMPLE; B – BULK SAMPLE; U – UNDISTURBED SAMPLE; P – SPT SAMPLE; W – WATER SAMPLE

NOTE: WHERE FULL 0.3 PENETRATION HAS NOT BEEN ACHIEVED, THE NUMBER OF BLOWS FOR THE QUOTED PENETRATION IS GIVEN (NOT N – VALUE)

ALL DEPTHS AND REDUCED LEVELS IN (m) UNLESS OTHERWISE STATED

REMARKS

Trial hole excavated to 1.50m
Borehole backfilled on completion, no final overnight standing water level observed

F/c 5931

Figure 6.5: Borehole Number SJ89NW1461

Summary of Borehole Logs

SJ89NW543 (approx. 80m southwest of property) – Stiff Boulder clay at 4ft/ 1.21m depth

SJ89NW3783 (approx. 170m southwest of property) - Stiff Boulder clay at 0.84m depth

SJ89NW3784 (approx. 180m south of property) – Made ground down to 2.35m, overlying a thin bank of soft clay down (100mm) before stiff clay observed at 2.45m depth.

SJ89NW1461 (approx. 180m south of property) – Made ground down to 1.5m overlying stiff clay.

The historical boreholes within the local area highlight good ground conditions within the local area (stiff clay/Very stiff clay) but the boreholes have revealed discrepancies between the depth of very stiff clay required to support the loads from the buildings.

It should also be noted that the nearest available borehole was >80m from the property and as such there is a risk that different ground conditions may be encountered at the property.

Unfortunately, the engineer was unable to confirm the existing foundations at the property due the foundations being buried and obscured from view.

A site investigation is required to confirm the existing foundations and what underlying strata they are bearing on.

4.2.3 MINING AND MINERAL EXTRACTION

A review of the Coal Authority Interactive Map indicates that this property **is** located within influencing distance of past, current, or proposed mine workings.

For full details the client is advised to obtain a coal mining report from the Coal Authority to determine the extent of any historical mining activities in the area.

4.2.4 TOPOGRAPHY

The ground level across the site is level at the front and side of the property, and there are no significant retaining structures within influencing distance of the property and there is no risk of slope instability of the site.

4.2.5 TREES

Tree which are growing within potential influencing distance of the property, which could, in certain ground conditions, adversely affect the stability of the foundations. Where the sub-soils are cohesive, clay soils, trees will draw moisture from the clay, resulting in shrinkage of the clay and associated subsidence. This can cause foundations to move downwards.

On this site, there are no trees in close proximity to the property and as such there is no risk of tree related ground movements at the property.

5. STRUCTURAL CONDITION ASSESSMENT

5.1 EXTERNAL OBSERVATIONS

A visual non-intrusive survey of the premises was undertaken, and the following was observed:

5.1.1 FRONT ELEVATION

The front elevation is in a state of disrepair with vegetation growing out of the 1st floor walls above No. 80 of the premises (78 and 80 are combined into one premises).

Excessive deflection of the structural openings above the windows is indicative of a) inadequate existing supports or b) existing supports are defective due to long term degradation.

Furthermore, the masonry at the interface with the adjoining apartment block was in poor condition at 1st floor level as shown in photograph 1.



Photograph 1



Photograph 2

Cracking and bulging of the front elevation render were observed adjacent to adjoining apartment block photograph 2.

The cracking is indicative of structural movement to the front elevation structural steel supports over the window/door openings at ground floor.

5.1.2 REAR ELEVATION

No access was obtained to the rear elevation of the premises and as such no survey was possible.

Due to a lack of access, we are unable to determine the condition of the rear elevation except from a visual survey from within the premises looking towards the rear.

5.2 INTERNAL OBSERVATIONS

During the non-intrusive survey, the surveying structural engineers observed the following:

5.2.1 GROUND BEARING CONCRETE SLAB

No observable issues with the ground bearing concrete slab were observed such as bowing, sloping or cracking.

As part of preliminary investigation works at the premises, 2 No trial pits were excavated with the property to a) obtain the construction of the ground floor slab, b) to determine local ground conditions and c) to confirm the existing foundations type/dimensions on the party wall.

Appendix A details the location of the proposed trail pits on the ground floor.

Trail pit 1 (Adjacent to Pub)

Trial pit 1 dimensions - 1.3m x 0.8m x 2.0m deep

The foundation comprises of corbelled(stepped) brickwork bearing directly to ground (common foundation detail for buildings of this age)

The widest part of the foundation is approximately 425mm wide (225mm thick wall + 100mm thickening each side of the party walls). The ground bearing slab was observed to vary between 100-150mm deep with no reinforcement observed.

Trail pit 2 (Adjacent to Residential Apartment Block)

Trial pit 2 dimensions - 1.4m x 1.4m x 1.4m deep

No corbelled masonry was identified as in trial pit 1. A masonry arch was observed at 1m depth, which the masonry wall sits directly on top. The purpose of the arch but may be historical access to a cellar/basement between buildings.
Based on this observation there is also the possibility that the basement area below the ground floor was infilled previously.

The ground bearing slab was observed to vary between 100-150mm deep with no reinforcement observed.

Due to the age of the premises, the current ground bearing slab does not conform to modern building regulations requirements and hence the existing slab will need to be fully broken out and removed.

5.2.2 EXTERNAL WALLS

Side Elevation / Party Walls

Observed to be solid 225mm brickwork walls and within verticality tolerances.

No structural movement or cracking was observed on any of the walls indicating they are in acceptable condition and that the existing foundations are adequate for the previous and existing applied loads.

The surveying engineer was able to observe the top of the masonry walls from roof level, which appeared to show that the walls are independent of neighbouring properties masonry wall.

However, due to obstructions, we are unable to confirm if the side elevation walls are tied in/bonded to the masonry walls adjacent to the property. Further investigations are required to determine the construction and interfaces of the side elevation walls.

Rear Elevation walls

The rear elevation walls are in very poor condition with defective masonry observed n the form of poor/defective mortar joints.

Evidence of previous infills to windows/doors were observed and these have been poorly installed.

5.2.3 INTERNAL MASONRY WALLS

The internal walls on ground floor are in very poor condition (particularly around lintel openings), with partial collapse of in a number of locations to the rear of the building.

The spine wall dividing No 78 & 80 is in moderate condition with no significant bowing/tilting, but there is local collapse of masonry around defective lintels.

Due to poor condition of the internal walls (not including spine wall), it is advised that these walls are demolished and rebuilt.

The internal walls are providing lateral restraint to the building structure and as such their removal will require a temporary works shoring design to ensure sufficient lateral stability is obtained during any remedial/ renovations works.



Photograph 3 – Defective masonry/lintel



Photograph 4 –Blockwork wall infill poorly constructed



Photograph 5 – Defective masonry/lintel

5.2.4 STRUCTURAL STEELWORK

A significant amount structural steelwork is installed within the premises, primarily for supporting the suspended timber floor and roof structures. Appendix B details the locations of the structural supports within the building.

Overall, the structural steelwork is severely corroded due to a lack of waterproofing in the building, with rainfall directly entering the building via a collapsed roof structure. During the survey the building had significant damp/water ingress issues.

A survey of the structural steelwork within the building raised significant concerns due to inadequate connection details.

The column baseplates were observed to be inadequate in size and incorrect bolt spacing, indicating that the base plates/fixings were installed ad-hoc on site and not to any structural design specification/standards

Structural steel connections and column base plate due to a lack of appropriate connection size and inadequate bolts.

The engineer advises that all structural steel connections and base plates would require replacing due to their inadequacy.

The client proposed to change the building change of use to a restaurant/bar which has much higher live loads due to the building use. RD Engineering Associates Ltd undertook a structural assessment of the existing structural steel beams at 1st floor level to determine if they are adequate for the increased live load – Appendix C.

The assessment confirmed that the existing structural steel beams are inadequate for the proposed building change of use and as such would require replacement or a new structural system to support the new floor construction (replacement of very poor condition suspended timber floor).



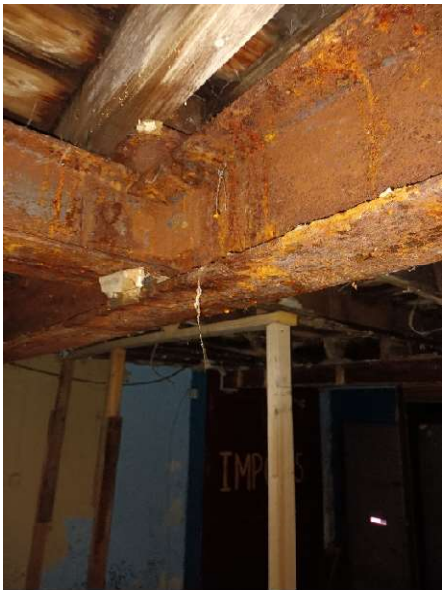
Photograph 6 – Severely corroded structural steel beam adjacent to front elevation on ground floor



Photograph 7 – Severely corroded structural steel beam adjacent to front elevation on ground floor



Photograph 8 – Severely corroded structural steel beam to beam connection



Photograph 9 – Severely corroded structural steel beam to beam connection



Photograph 10 – Severely corroded structural steel beam to party wall connection



Photograph 11 – Severely corroded column and steel beams



Photograph 12 – Severely corroded structural steel beams



Photograph 13 – Severely corroded structural steel column and baseplate. Note basement size and fixings are inadequate.



Photograph 14 – Severely corroded structural steel column and baseplate. Note basement size and fixings are inadequate.



Photograph 15 – Severely corroded structural steel beam on 1st floor.



Photograph 16 – Corroded structural steel beam on 2nd floor (collapse roof structure)

5.2.5 LINTELS (INTERNAL AND EXTERNAL)

External Lintels

The structural openings on the front elevation comprise of structural steel beams at ground floor level, whilst the windows on the 1st and 2nd floor comprise of an external lintel/support with timber beams supporting the internal leaf of masonry.
We were unable to confirm the condition/type of the external supports but there was no significant cracking/sagging of the masonry above indicating that the existing supports are adequate.

However, the timber beams supporting the internal leaf were in very poor condition and require replacing.

The rear elevation lintels/supports were in poor condition and require replacement

Internal lintels

The Internal lintels are a combination of timber joists and concrete lintels.

The timber lintels were in poor condition and there were issues with the masonry wall supports for the lintels.

Due to the poor condition of the lintels and supporting masonry, we advise that all the internal lintels and surrounding masonry is replaced.



Photograph 17 – Defective Masonry bearing and lintel



Photograph 18 – Significant deflection on timber beam supporting masonry wall



Photograph 19 – Defective Masonry and lintel



Photograph 20 – Defective Masonry and lintel



Photograph 21 – Defective Masonry and lintel

5.2.6 SUSPENDED TIMBER FLOORS

The suspended timber floor had partially collapsed in a number of areas on the 1st and 2nd floor due to long term water ingress due to the collapsed roof structure.

The timber floor joists are in very poor condition and deflect sag excessively as a result of long-term water damage which has reduced their structural capacity.

The upper floors are unsafe and there is a risk of collapse should persons walk on the 1st and 2nd floors.

All timber floor joists are in very poor condition are require removal.



Photograph 22 – Partially collapsed 1st floor suspended timber floor



Photograph 23 – Partially collapsed 1st floor suspended timber floor



Photograph 24 – Partially collapsed 1st floor suspended timber floor



Photograph 25 – Partially collapsed 1st floor suspended timber floor

5.2.7 ROOF STRUCTURE

Flat Roof Structure

The timber flat roof has collapsed and as such there is no roof structure present at the premises. The existing timber flat roof joists that remain are exposed and have been subjected to long term exposure to external conditions.

The flat roof joists are in poor condition and are unsuitable to be retained or reused.



Photograph 26 – Fully exposed roof structure due to collapse of flat roof (looking towards apartment building)



Photograph 27 – Fully exposed roof structure due to collapse of flat roof (looking towards apartment building)



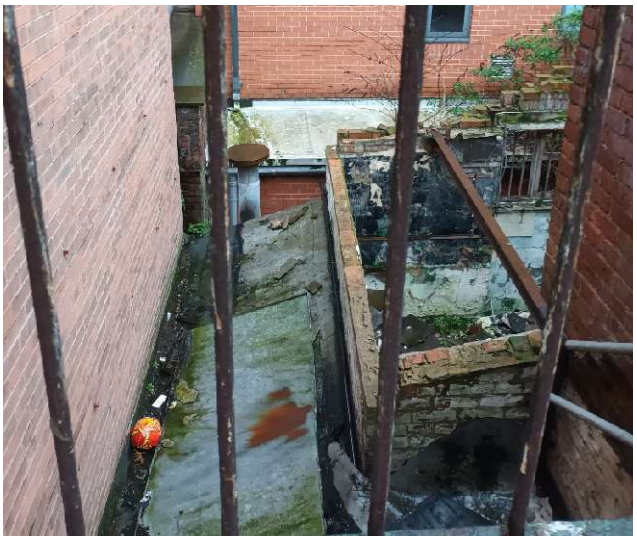
Photograph 28 – Fully exposed roof structure due to collapse of flat roof (looking towards pub building)

Pitched Roof Structures (Rear of premises).

The pitched roof structures to the rear of the premises (rear extensions) are in very poor condition.

When observed from the upper floors. a significant amount of debris was present and a number of trees were growing out of the gutter system, with significant water ingress into the building fabric.

The rear elevation roofs require removal as due to their poor condition they cannot be retained.



Photograph 29 – View of ground floor and 2nd floor roof structures on rear elevation extension.



Photograph 30 – View of 2nd floor roof structure on rear elevation extension.

6. STRUCTURAL APPRASIAL FOR PROPOSED RENOVATION WORKS

The client proposes to change the current building use to a multi-purpose premises comprising of a restaurant (Ground and 1st Floor), retail (2nd Floor) and residential apartments (3rd and 4th floor) as per Appendix A.

A review of the existing structure both in condition and the feasibility of retaining existing structural members to support the proposed redevelopment works was undertaken.

6.1 OVERVIEW OF EXISTING CONDITION OF THE PREMISES

Section 5 of this report (condition assessment) highlighted the following:

- Structural steelwork - A structural assessment of the existing structural steel beams was undertaken which confirmed that the existing beams do not have the capacity for the additional loading from the proposed development and hence require replacement with new members. The existing steelwork is in very poor condition due with severe corrosion which also supports removal and replacement.

- Suspended Timber Floors - The existing suspended timber floors are in poor condition and require full removal and replacement.

- Timber Roof structures - The existing flat and pitched roof structures are in very poor condition and require replacement.

- Lintels - The existing lintels (external and internal) are in poor condition and require replacement.

- Ground Bearing Slab - The existing slab is in adequate condition but due to the age of the premises will not meet the requirement for the

- Internal load bearing walls – The internal walls are in moderate condition, with local areas that require demolition and rebuilding due to their poor condition – i.e. significant cracking and loose masonry.

- Side Elevation/Party Masonry Walls - The side elevation/party walls are in moderate reasonable condition with no significant structural defects observed.

  However, due to the lack of information relating to the construction (i.e. interface with the neighbouring properties) there is significant risk of structural movement/damage to the walls if they are disturbed and subjected to additional loading during construction works.

- Masonry Spine wall between No 78/80 –The masonry spine walls are in moderate condition, with local areas that require remediation due to their poor condition – i.e. significant cracking and loose masonry.

The internal masonry spine wall could be retained with remedial works, however there are concerns regarding any increased loading on the assumed existing corbelled masonry foundations.

Furthermore, the proposed layouts confirm that the majority of the spine wall is to be removed to facilitate the proposed room layouts, and such a new internal wall structure is required.

The property is of traditional construction (load bearing masonry walls, suspended timber floors and roof) with structural steelwork to support the suspended floor construction. Appendix C details the existing structural steel layouts and locations.

Loads from the roof, floors and supporting steelwork are transferred to the side elevation and internal spine wall down to the building foundations via axial loads.

As such as proposal to modify the existing structure for the additional 2 storeys and building change of use on the existing floors, will require the additional loads to transfer down the existing load bearing walls and to the foundations.

The feasibility of reusing the existing structure is dependent on the existing load transfer systems – i.e load bearing walls.

Due to the uncertainty in the existing loading conditions by neighbouring buildings and the lack of information of the existing foundations, it is our professional opinion that the existing side elevation/party walls and foundations will be inadequate to support the new layout.

The structural engineer advises that the new development is designed to ensure that the existing side elevation walls are not subjected to any increase in applied loading. This may require a new structural steel frame installed off independent foundations to avoid any structural issues with the existing walls/foundations.

There are a number of issues associated with constructions works on the side elevation walls:

- New structural steel beams and padstones installed onto party wall could result in localised structural movement due to the poor quality of the brickwork and mortar joints. This can lead to damage to the neighbouring properties walls

- Structural movement to walls due to an increase in loading from additional storeys. The condition of the mortar joints behind the mortar joints observed internally is unknown.

- Increased loading on the existing foundations may result in immediate settlement leading to structural movement and cracking to the wall.

To minimise any potential damage to the existing wall and to avert any potential disputes due to damage to the adjacent/masonry party wall with the neighbouring

properties, we advise that the new development should be independent of the side elevation/party walls and constructed off new designed foundations.

Furthermore, party wall notices would need to be served to both neighbouring properties and subsequent structural movement monitoring set up to determine the impact of any construction activities on the walls.

6.2 EXISTING FOUNDATIONS

Based on the age of the building (pre-1900), the foundations are likely comprised of corbelled brickwork bearing directly onto firm/stiff clay based on the desktop geotechnical studies.

As part of the initial site investigations 2No trial pits were dug to determine the existing foundations and to confirm the local ground conditions, however both of the trial pits did not observe the foundations at a depth of 2m below existing slab level (continued masonry wall was observed down to 2m level).

A visual survey of the existing foundations was not possible as a 2m deep excavation did not reveal the exiting foundation and as such it is assumed that the foundations are corbelled masonry at 440mm width (assumed 4 brick widths wide plus mortar joints).

As such it was not considered safe to continue a deep excavation to determine the trial pits and hence foundation investigation works were halted.

The depth of the foundation also complicates any potential underpinning works of the existing foundations for the additional two storey and change of use loads.

Whilst the side elevation walls of the premises appear to be constructed offline from the neighbouring buildings side elevation walls, due to close proximity of the neighbouring pub building, it is assumed that the loads from the side elevation will be loading the existing building foundations.

The neighbouring properties construction and building types is unknown and as such an accurate foundation load assessment on the existing foundations cannot be undertaken.

As an accurate load assessment cannot be undertaken, we cannot provide confidence that the existing foundations can withstand the additional loading from the proposed development.

Furthermore, the proposed development requires the removal of the spine wall, and if the existing side elevation walls/foundations were retained, would result in a significant increase in load onto these components for which they are not designed for.

It is the authors professional opinion that the current premises is to be demolished and rebuilt with a new structural steel frame and independent foundations to achieve the proposed layouts and additional storeys as detailed in Appendix A.

7. CONCLUSION

The structural appraisal has concluded that the existing structure is in very poor condition with the following requiring removal and replacement to ensure the building is structurally sound in its current layout:

1) Suspended Timber floor construction
2) Internal and external lintels
3) Sections of the internal masonry walls
4) Structural steel which supports the floor construction
5) Timber roof construction

There is a lack of information relating to the existing foundations on the side elevation/party walls and as such the client is advised that any proposed structural design solution does not result in load increases onto these walls.

The internal spine wall (historically the party wall between 78-80) is in moderate condition and can be retained, however the proposed floor layout show a that over 50% is removed at ground floor and fully removed at 2nd floor. New load supporting structures are required to support the structures whilst ensuring the existing spine wall is not subjected to increased loads.

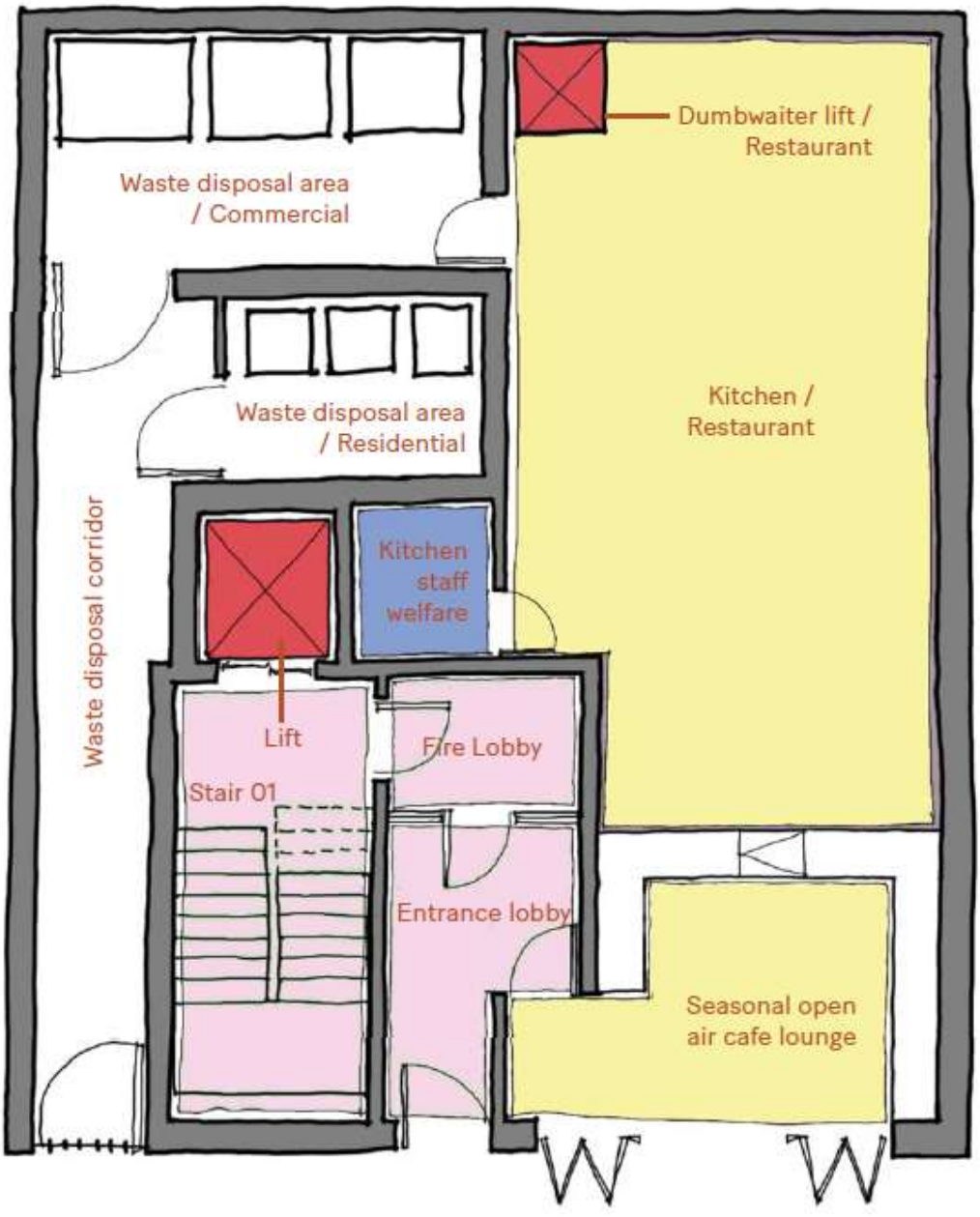
It is the authors professional opinion that the current premises is to be demolished and rebuilt to achieve the proposed layouts.

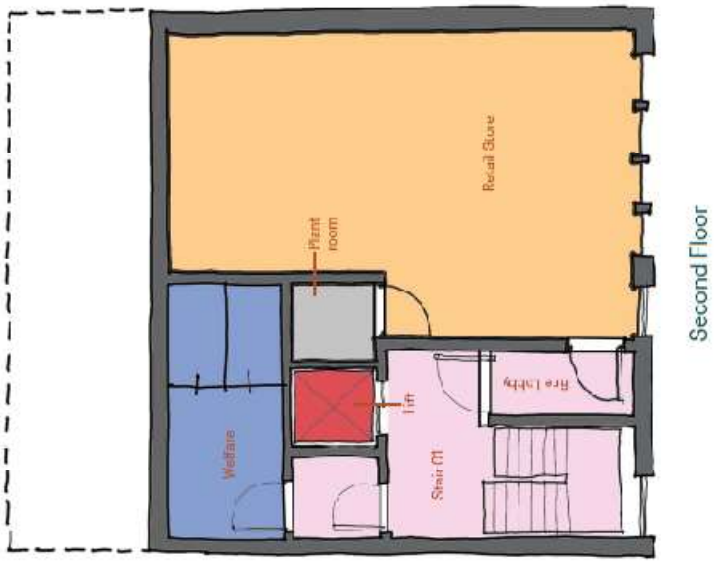
The existing side elevation walls and spine wall can be retained but are not to be subjected to increased loads due to the lack of information regarding the existing foundations and local ground conditions.

As such as new structural design is required to support the upper storeys and building change of use, whilst not adversely affecting the existing side/party elevation walls.

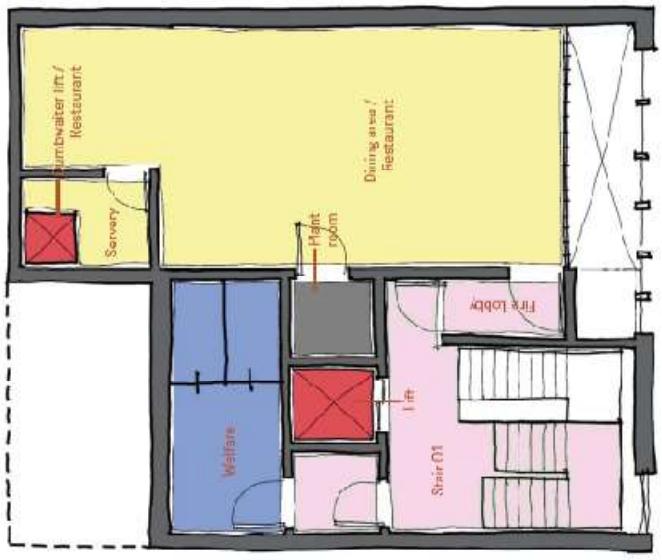
A new structural steel frame and independent foundations to achieve the proposed layouts and additional storeys as detailed in Appendix A should be considered.

APPENDIX A – PROPOSED GA AND ELEVATION DRAWINGS

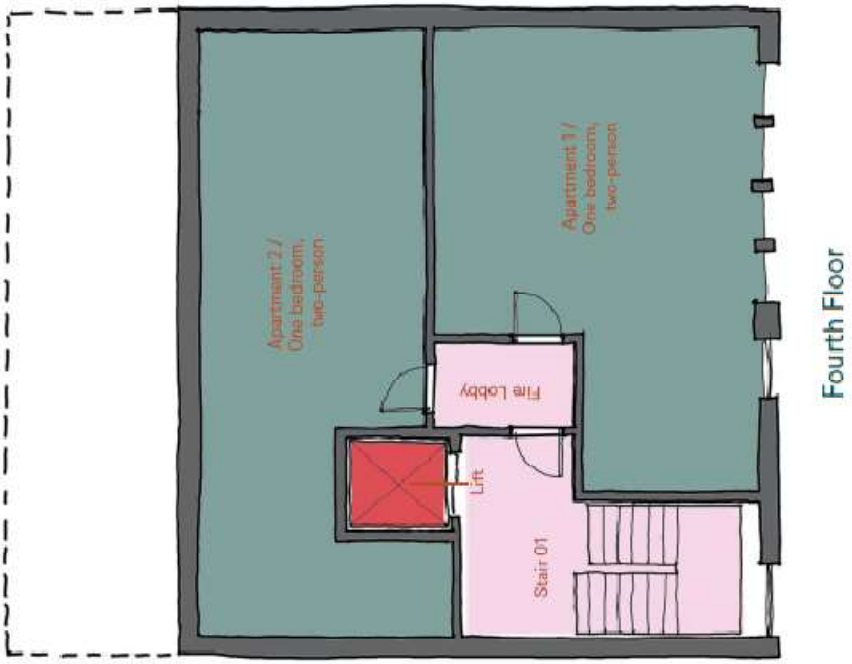




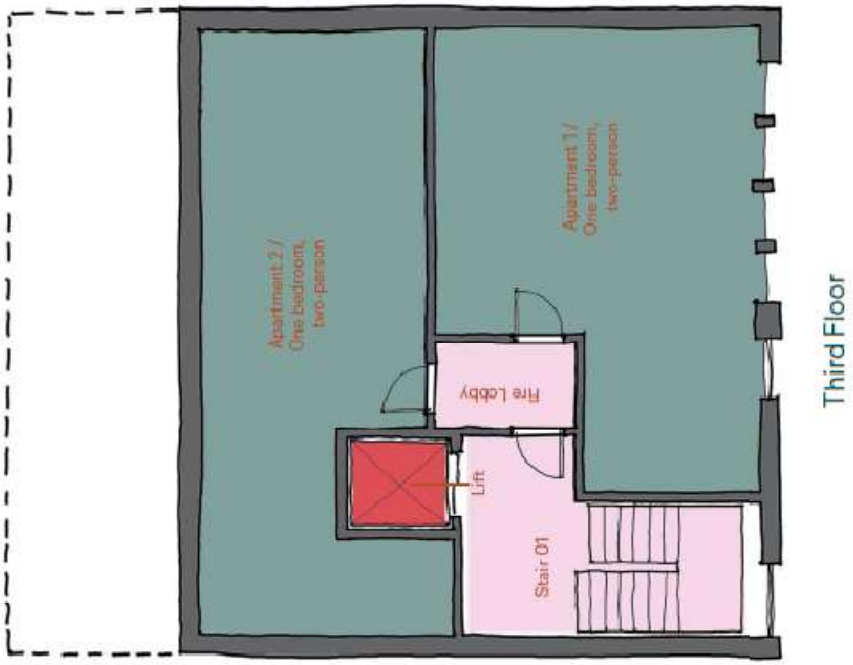
Second Floor



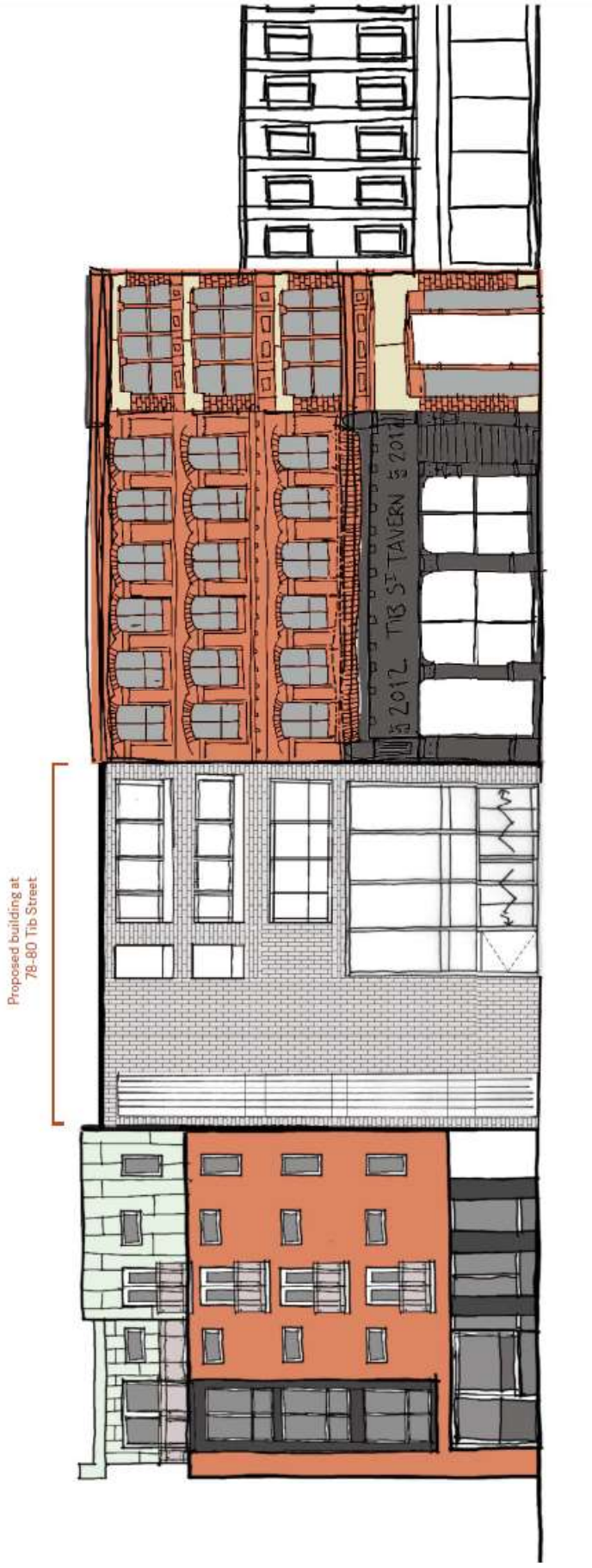
First Floor



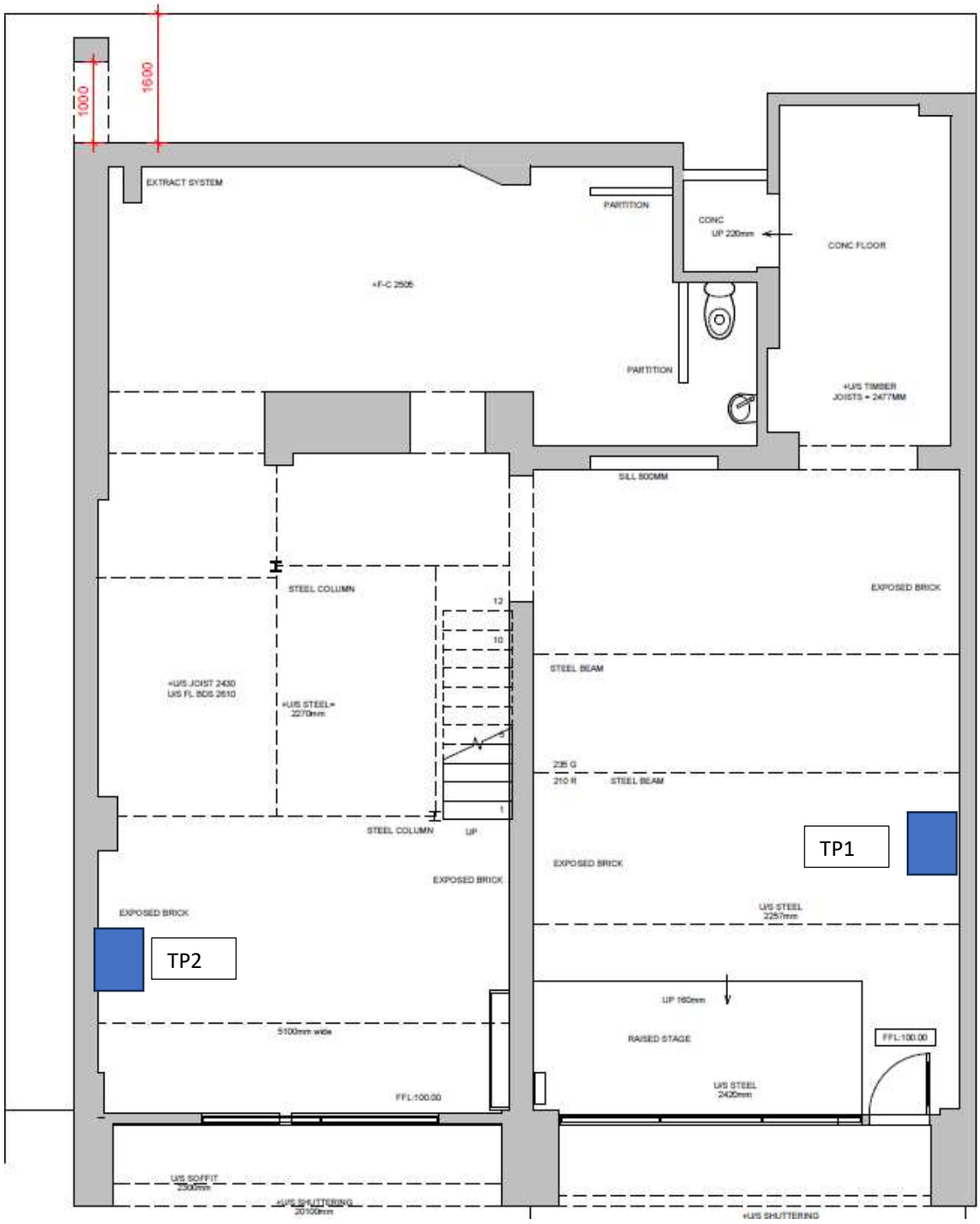
Fourth Floor



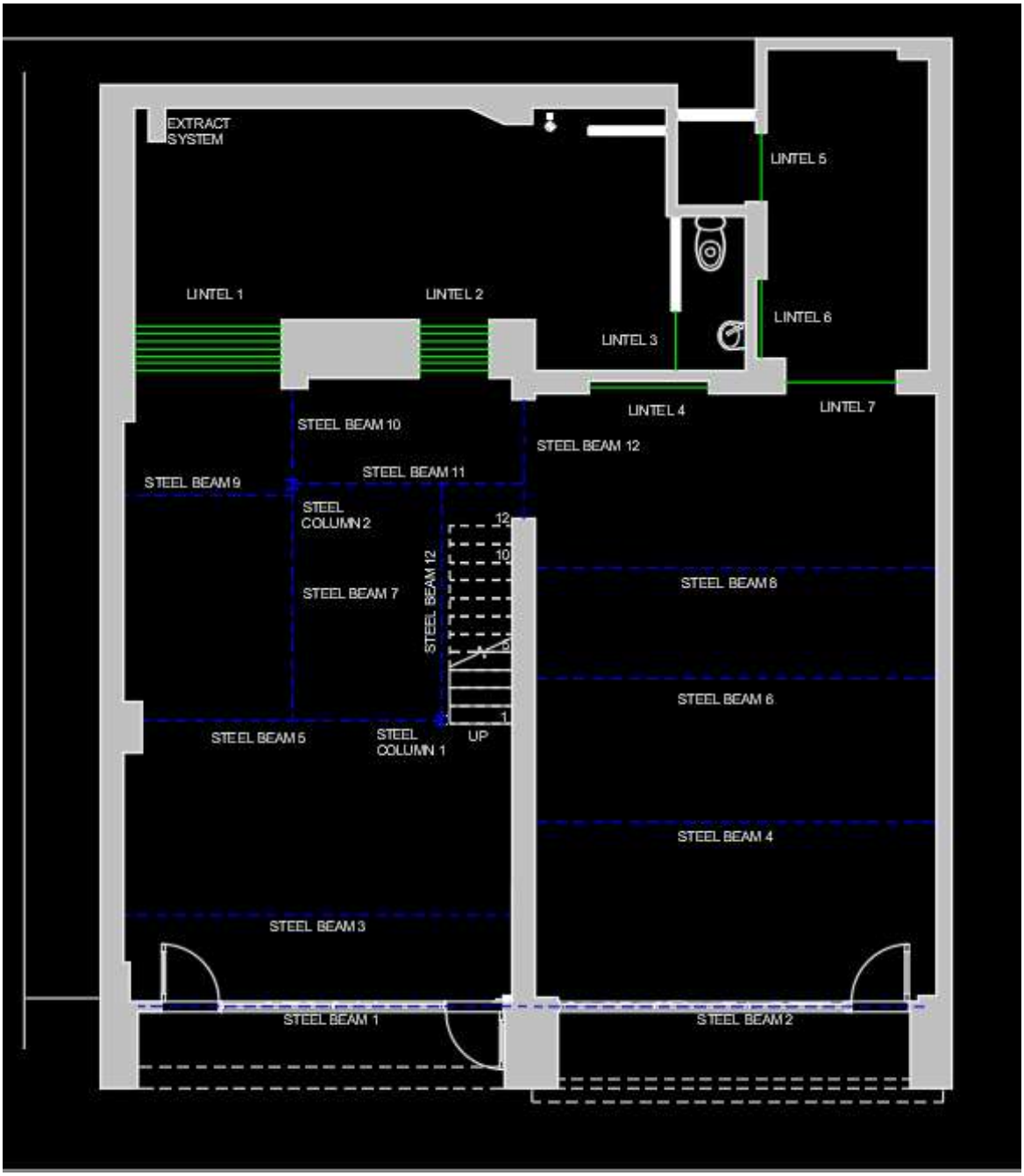
Third Floor


Proposed building at
78-80 Tib Street

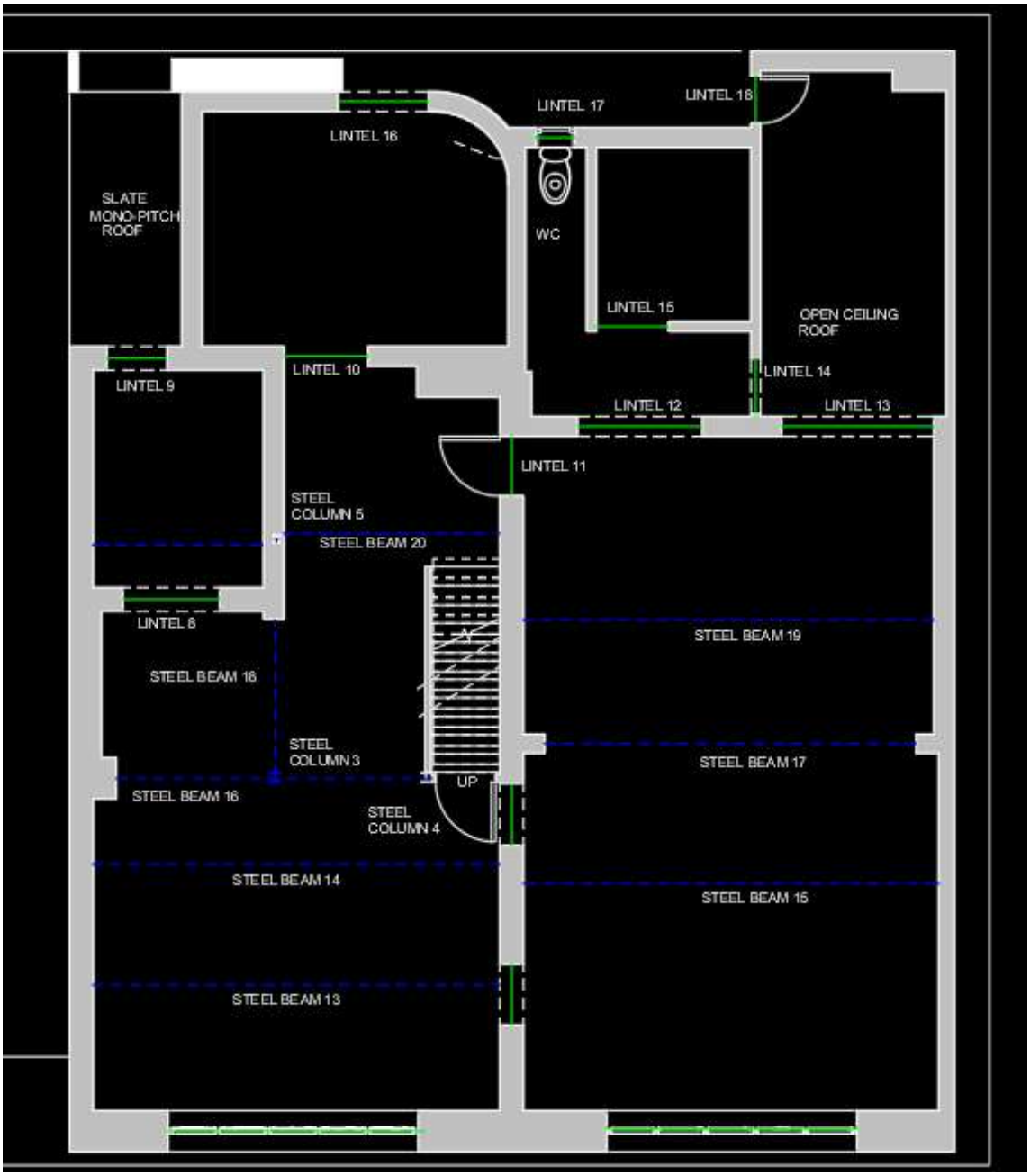
APPENDIX B – TRIAL PIT SURVEY LOCATION SKETCH

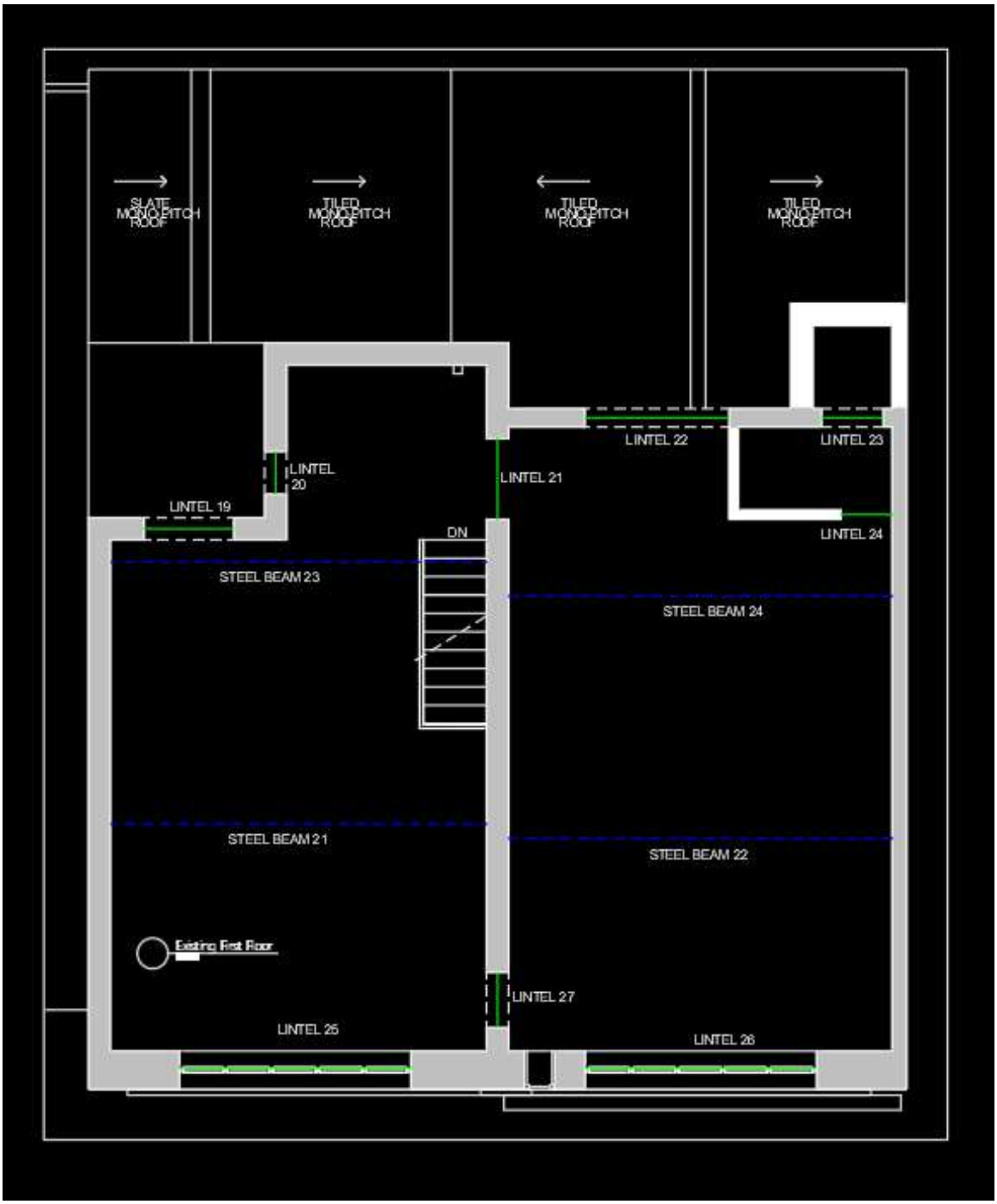


APPENDIX C – EXISTING GENERAL ARRANGEMENT DRAWINGS



Existing Ground Floor GA


LINTEL 17
LINTEL 18
LINTEL 16
SLATE MONO-PITCH ROOF
WC
LINTEL 15
OPEN CEILING ROOF
LINTEL 10
LINTEL 14
LINTEL 9
LINTEL 12
LINTEL 13
LINTEL 11
STEEL COLUMN 5
STEEL BEAM 20
LINTEL 8
STEEL BEAM 19
STEEL BEAM 18
STEEL COLUMN 3
STEEL BEAM 17
UP
STEEL BEAM 16
STEEL COLUMN 4
STEEL BEAM 14
STEEL BEAM 15
STEEL BEAM 13


SLATE MONO PITCH ROOF
TILED MONO PITCH ROOF
TILED MONO PITCH ROOF
TILED MONO PITCH ROOF
LINTEL 19
LINTEL 20
LINTEL 21
LINTEL 22
LINTEL 23
LINTEL 24
DN
STEEL BEAM 23
STEEL BEAM 24
STEEL BEAM 21
STEEL BEAM 22
Existing first floor
LINTEL 25
LINTEL 26
LINTEL 27

APPENDIX D – STRUCTURAL ASSESSMENT OF EXISTING STRUCTURAL STEEL BEAMS

Structural Design Codes:

The existing structural steel members were assessed using the following design codes:

- BS 6399-1:1996 Loading for buildings. Code of practice for dead and imposed loads
- BS 5950 Structural Steelwork Design

NOTE – no assessment has been undertaken as all timber lintels, floor joists and roof members are to be removed and replaced due to their poor condition.

Applied Loadings

Dead Loading

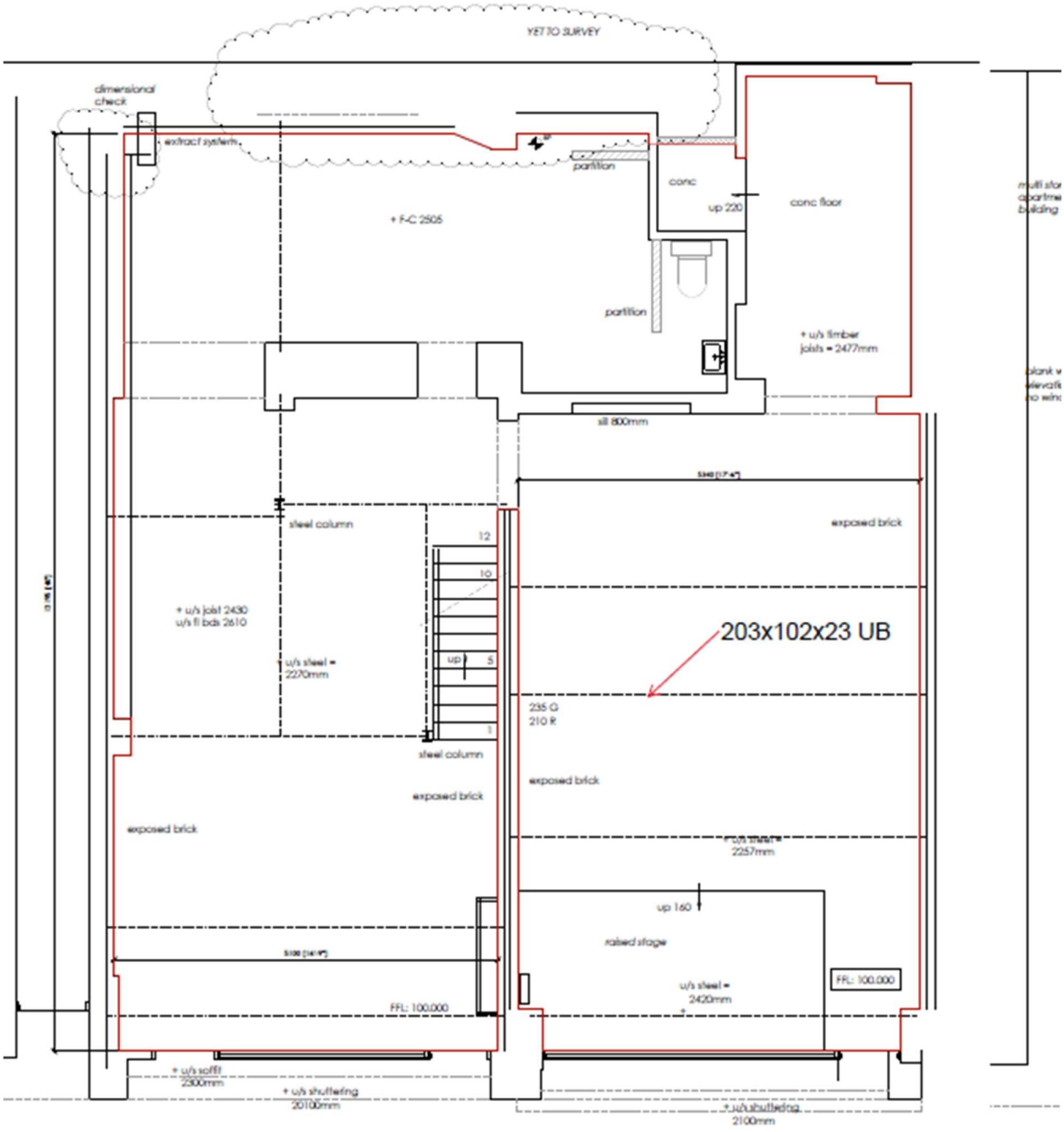
First Floor						
Dead Load	Finish	0.10		0.14		
	Boards	0.13		0.18		
	Joists	0.22		0.31		
	Ceiling	0.14		0.20		
	Partitions	0.55		0.77		
	Services	0.05		0.07		
		TOTAL	**1.19**	**kN/m^2**	**1.67**	**kN/m^2**

Imposed Loading

A review of the architectural proposals in Appendix`s B, C, D and E identifies a number of different uses of the floors in the premises.

BS 6399-1:1996 Loading for buildings. Code of practice for dead and imposed loads states the following for imposed loading:

- Shop floors for the sale and display of merchandise 4.0 kN/m^2 as per table 1 Minimum Imposed Floor Loads
- Kitchens, laundries, laboratories 3.0 kN/m^2 as per table 1 Minimum Imposed Floor Loads
- Public, institutional and communal dining rooms and lounges, cafes and restaurants are 2.0 kN/m^2 as per table 1 Minimum Imposed Floor Loads



Sketch 1 - Ground Floor Plan identifying location of Existing Beam (203 x 102 x 23 UB)

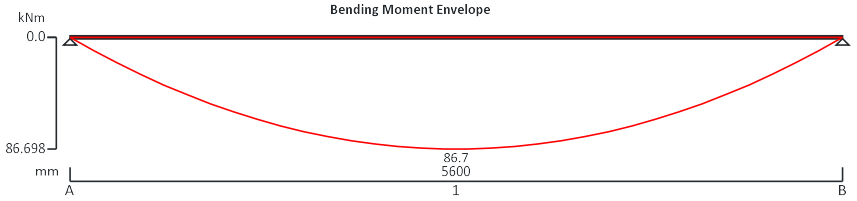
Steel beam 203x102x23 UB analysis & checking (BS5950)

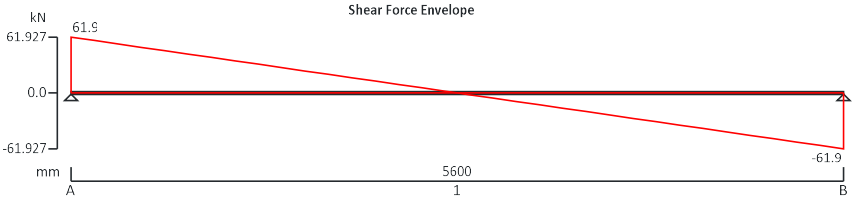
Steel beam analysis & design (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

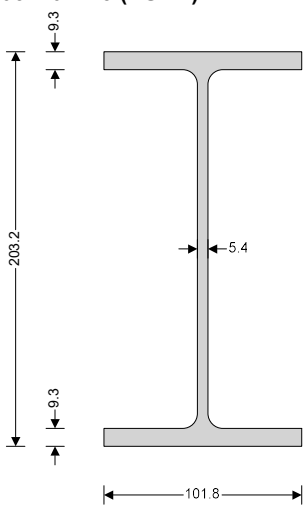
Beam loads	SW - Dead self weight of beam × 1 Floor 1.1x5.5/2 - Dead full UDL 3 kN/m Floor 4.0x5.5/2 - Imposed full UDL 11 kN/m

Analysis results

Maximum moment	M_{max} = **86.7** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **61.9** kN	V_{min} = **-61.9** kN
Deflection	δ_{max} = **32.6** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **61.9** kN	R_{A_min} = **61.9** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **9** kN	
Unfactored imposed load reaction at support A	$R_{A_Imposed}$ = **30.8** kN	
Maximum reaction at support B	R_{B_max} = **61.9** kN	R_{B_min} = **61.9** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **9** kN	
Unfactored imposed load reaction at support B	$R_{B_Imposed}$ = **30.8** kN	

Section details

Section type	**UB 203x102x23 (BS4-1)**	Steel grade	**S275**



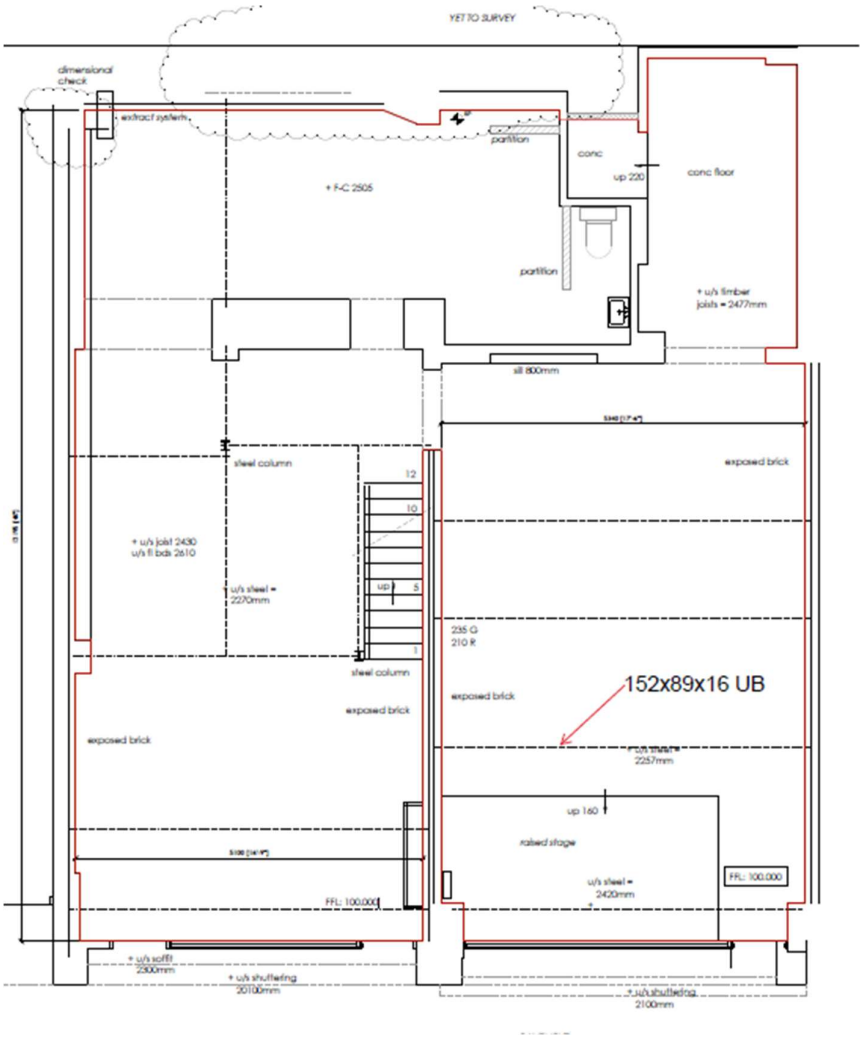
Classification of cross sections - Section 3.5

Tensile strain coefficient	ε = **1.00**	Section classification	**Plastic**

Moment capacity - Section 4.2.5

Design bending moment kNm	M = **86.7** kNm	Moment capacity low shear	M_c = **64.4**

FAIL - Design bending moment exceeds design bending moment capacity



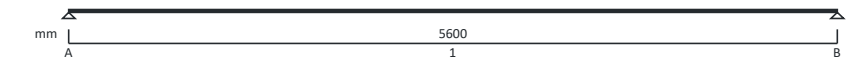
Sketch 2 - Ground Floor Plan identifying location of Existing Beam (152 x 89 x 16 UB)

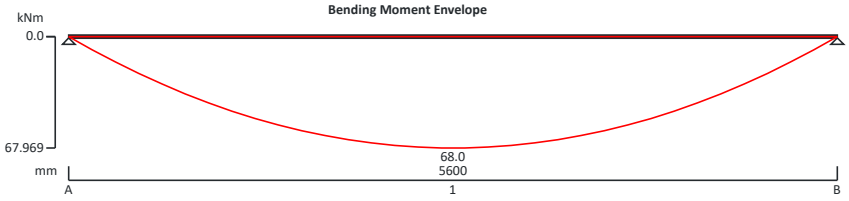
STEEL BEAM First Floor152x89x16 UB ANALYSIS & CHECKING (BS5950)

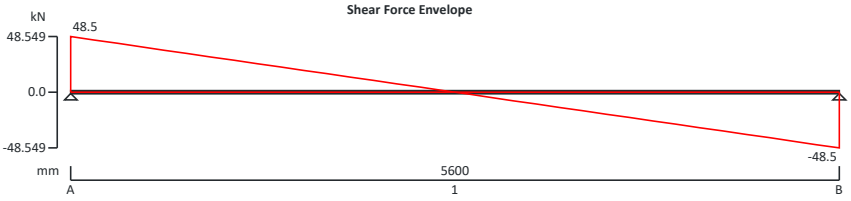
Steel beam analysis & design (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

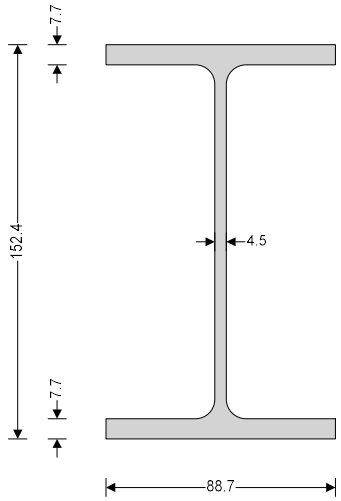
Beam loads	SW - Dead self weight of beam × 1 Floor 1.1x4.3/2 - Dead full UDL 2.4 kN/m Floor 4.0x4.3/2 - Imposed full UDL 8.6 kN/m

Analysis results

Maximum moment	M_{max} = **68** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **48.5** kN	V_{min} = **-48.5** kN
Deflection	δ_{max} = **64.4** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **48.5** kN	R_{A_min} = **48.5** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **7.2** kN	
Unfactored imposed load reaction at support A	$R_{A_Imposed}$ = **24.1** kN	
Maximum reaction at support B	R_{B_max} = **48.5** kN	R_{B_min} = **48.5** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **7.2** kN	
Unfactored imposed load reaction at support B	$R_{B_Imposed}$ = **24.1** kN	

Section details

Section type	**UB 152x89x16 (BS4-1)**	Steel grade	**S275**



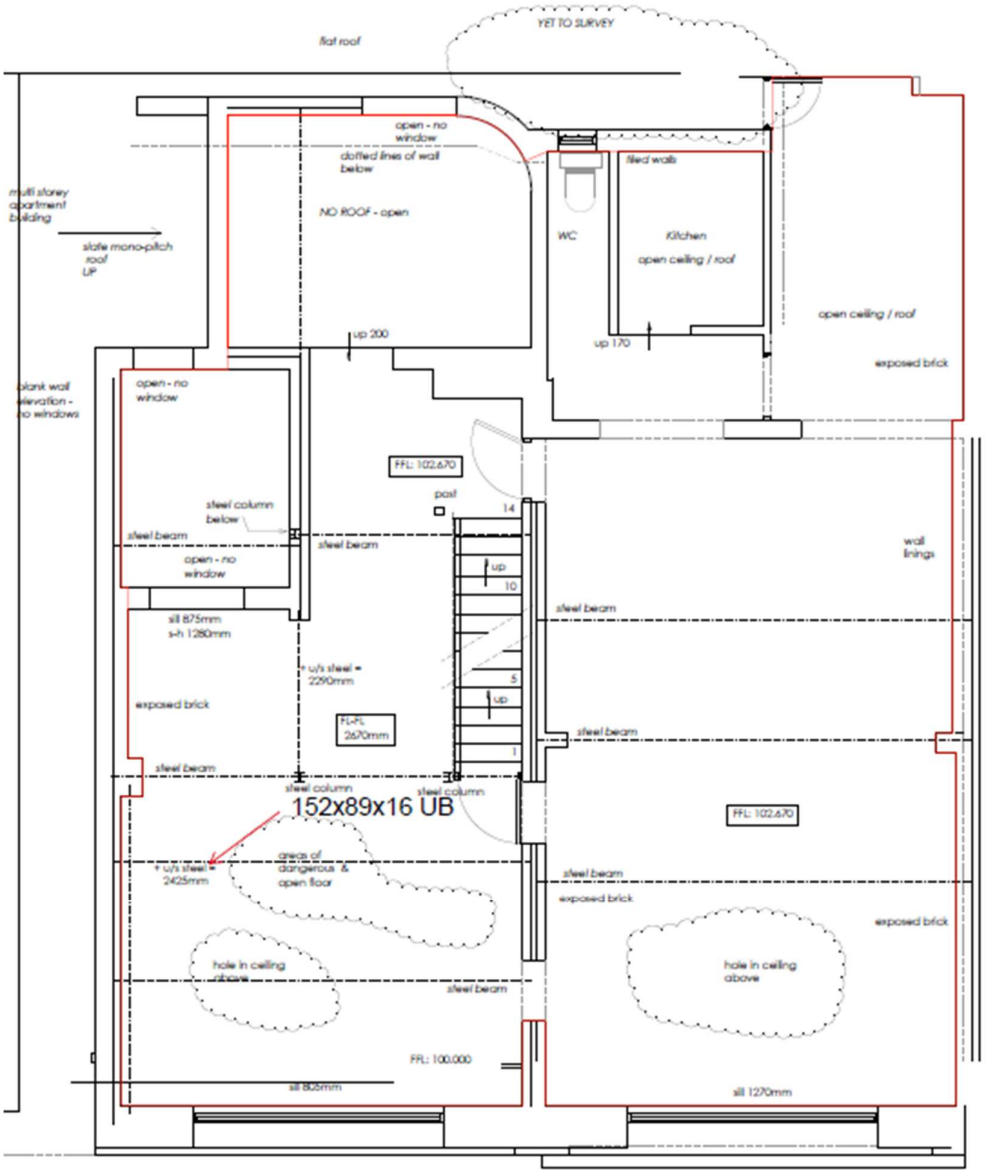
Classification of cross sections - Section 3.5

Tensile strain coefficient	ε = **1.00**	Section classification	**Plastic**

Moment capacity - Section 4.2.5

Design bending moment kNm	M = **68** kNm	Moment capacity low shear	M_c = **33.9**

FAIL - Design bending moment exceeds design bending moment capacity



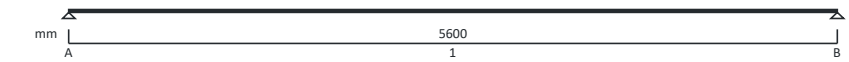
Sketch 3 - Ground Floor Plan identifying location of Existing Beam (152 x 89 x 16 UB)

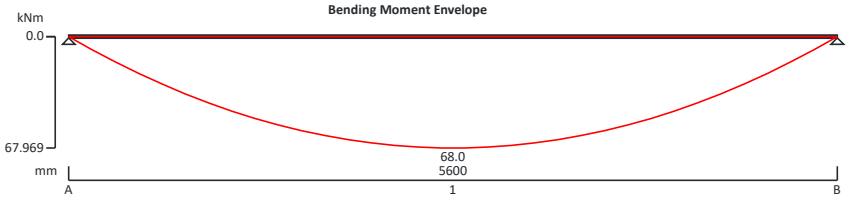
STEEL BEAM 152X89X16 UB ANALYSIS & CHECKING (BS5950)

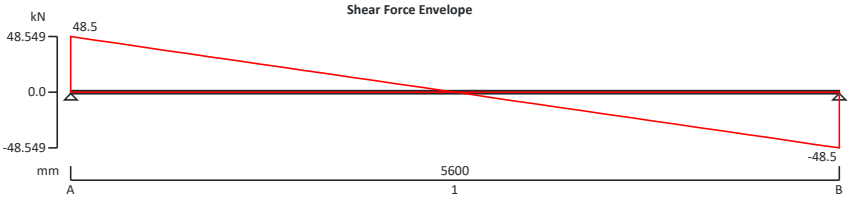
Steel beam analysis & design (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

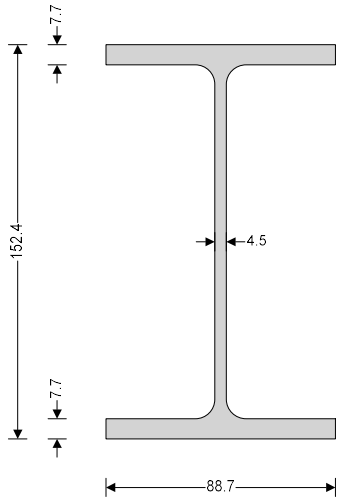
Beam loads	SW - Dead self weight of beam × 1 Floor 1.1x4.3/2 - Dead full UDL 2.4 kN/m Floor 4.0x4.3/2 - Imposed full UDL 8.6 kN/m

Analysis results

Maximum moment	M_{max} = **68** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **48.5** kN	V_{min} = **-48.5** kN
Deflection	δ_{max} = **64.4** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **48.5** kN	R_{A_min} = **48.5** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **7.2** kN	
Unfactored imposed load reaction at support A	$R_{A_Imposed}$ = **24.1** kN	
Maximum reaction at support B	R_{B_max} = **48.5** kN	R_{B_min} = **48.5** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **7.2** kN	
Unfactored imposed load reaction at support B	$R_{B_Imposed}$ = **24.1** kN	

Section details

Section type	**UB 152x89x16 (BS4-1)**	Steel grade	**S275**



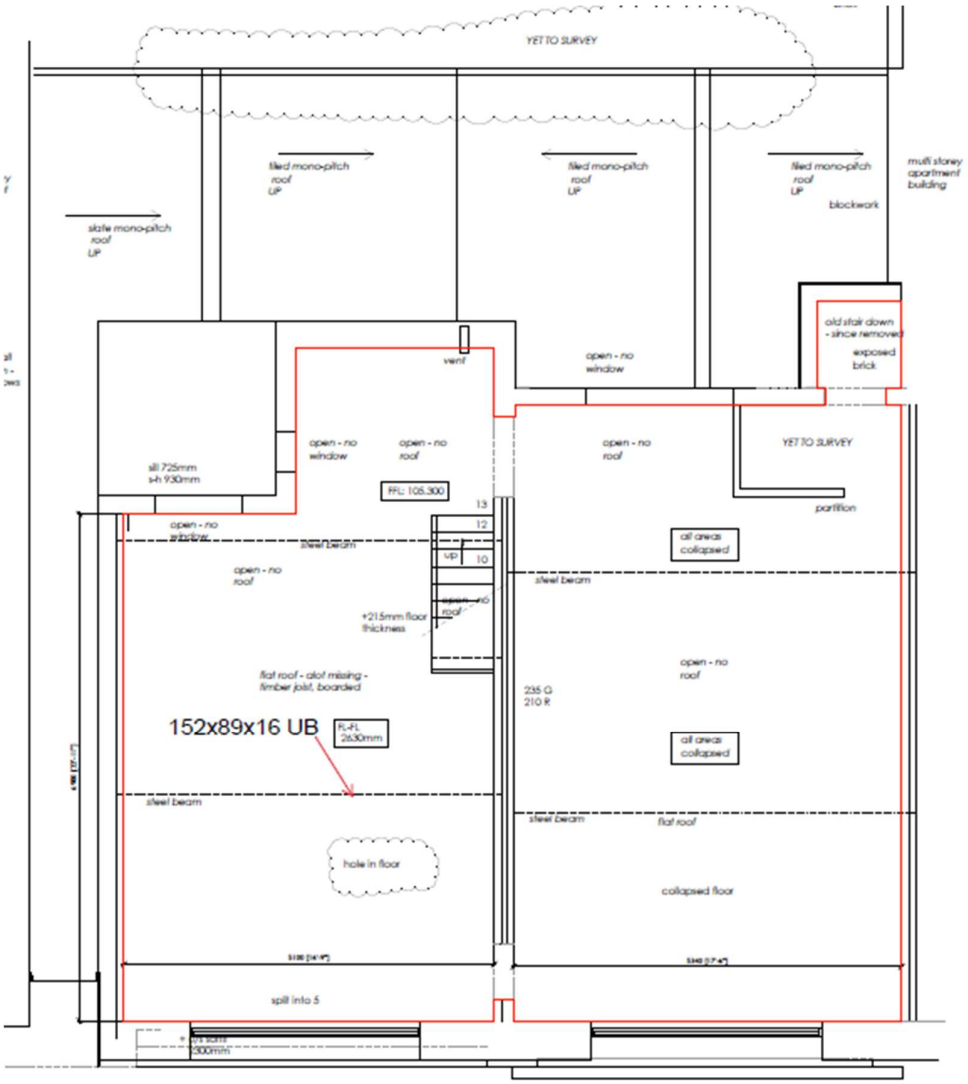
Classification of cross sections - Section 3.5

Tensile strain coefficient	ε = **1.00**	Section classification	**Plastic**

Moment capacity - Section 4.2.5

Design bending moment kNm	M = **68** kNm	Moment capacity low shear	M_c = **33.9**

FAIL - Design bending moment exceeds design bending moment capacity



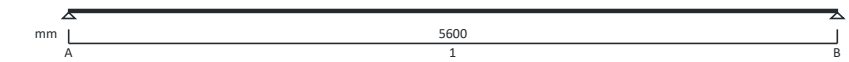
Sketch 4 - Ground Floor Plan identifying location of Existing Beam (152 x 89 x 16 UB)

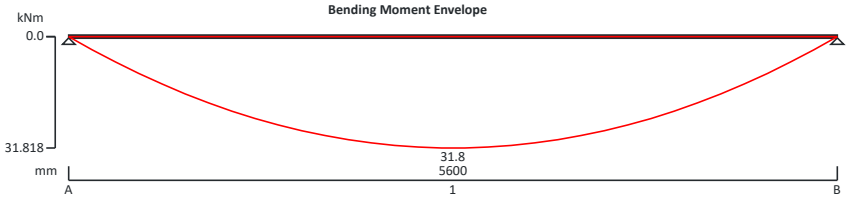
STEEL BEAM Roof 152X89X16 UB ANALYSIS & CHECKING (BS5950)

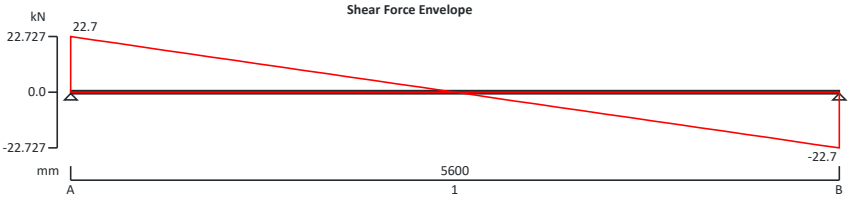
Steel beam analysis & design (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

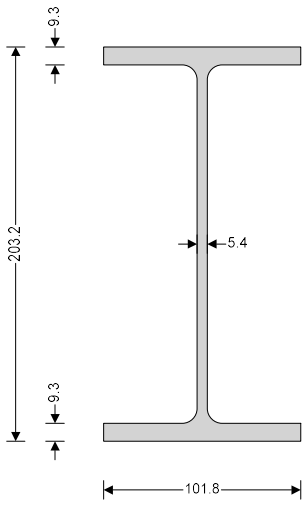
Beam loads	SW - Dead self weight of beam × 1 Floor 0.95x6.0/2 - Dead full UDL 3 kN/m Floor 0.75x6.0/2 - Imposed full UDL 2.25 kN/m

Analysis results

Maximum moment	M_{max} = **31.8** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **22.7** kN	V_{min} = **-22.7** kN
Deflection	δ_{max} = **6.7** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **22.7** kN	R_{A_min} = **22.7** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **9** kN	
Unfactored imposed load reaction at support A	$R_{A_Imposed}$ = **6.3** kN	
Maximum reaction at support B	R_{B_max} = **22.7** kN	R_{B_min} = **22.7** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **9** kN	
Unfactored imposed load reaction at support B	$R_{B_Imposed}$ = **6.3** kN	

Section details

Section type	**UB 203x102x23 (BS4-1)**	Steel grade	**S275**



Classification of cross sections - Section 3.5

Tensile strain coefficient	ε = **1.00**	Section classification	**Plastic**

Shear capacity - Section 4.2.3

Design shear force kN	F_v = **22.7** kN	Design shear resistance	P_v = **181.1**

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment kNm	M = **31.8** kNm	Moment capacity low shear	M_c = **64.4**

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment	M_b = **19.4** kNm	M_b / m_{LT} = **20.9** kNm

FAIL - Design bending moment exceeds buckling resistance moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection mm	δ_{lim} = **15.556** mm	Maximum deflection	δ = **6.677**

PASS - Maximum deflection does not exceed deflection limit

Limitations

a) Our survey is limited to the structural aspects of the building, such as the roof, floors, walls and foundations.

b) We have not inspected, nor reported on, non-structural elements such as windows, doors, rainwater goods and internal decorative finishes.

c). We have not inspected, nor reported on, the plumbing, heating system, electrical wiring, gas or water supplies to the building.

d). Unless explicitly stated to the contrary, we have not carried out a dampness survey, or a timber infestation survey, and therefore cannot comment on these aspects of the building.

e). We have not inspected woodwork which is covered or any other concealed or inaccessible parts of the structure and therefore cannot report that any such part of the building is free from defect.

f) No comment is made in this report as to the presence of new or old mine workings or tunnelling, heavy metals, chemical, biological, electromagnetic or radioactive contamination or pollution, or radon, methane or other gases, underground services, or underground structures, springs and watercourses, sink holes or the like, noise or vibration, pollution, mould, asbestos or asbestos products.

g) Unless explicitly stated to the contrary, the void below the ground floor has not been examined and therefore we cannot give any opinion on the condition of materials under the floor.

h) For the avoidance of doubt, The Contracts (Rights of Third Parties) Act, 1999, shall not apply to this Contract.

Our service does not include for confirming the presence or not, of any invasive plant species that may be present on the site such as Japanese Knotweed, Giant Hogweed, Himalayan Balsam etc. This is outside our area of expertise.

i) We are unable to confirm the presence of asbestos at the property as this is out of area of expertise.