

51-53 Richmond Street

Civil & Structural Engineering

Stage 2 Report

For

Eminence Development Ltd

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1. Introduction

Booth King Partnership Ltd have been appointed by Eminence Development Ltd to develop Civil and Structural Engineering design proposals for the conversion of an existing Grade II listed mill building in central Manchester. Proposals comprise refurbishment of the existing building into 4No. new residential demises and the construction of a 5-storey town house in the narrow parcel of land adjacent the site.

This report establishes the key civil & structural engineering design philosophy for the proposed refurbishment works and development of the brownfield site.



Figure 1: Architect's Render of the Existing Building and New Build Adjacent (SHR)

3 Site Location

The site is located in central Manchester at 51 to 53 Richmond Street, M1 3WB (E: 384318 , N:397809). The site is at approximately 41.0mAOD.



Figure 2: Site Location Plan

4 Existing Site Description

The site is currently occupied by a Grade II mill building originally constructed in c.1860. It is 4-storeys in height including a partial basement and is bound by Richmond Street to the front elevation and a narrow alley and low-medium rise buildings to the rear.

Either side of the building are more contemporary low-medium rise buildings. To the north the building shares a party wall with its neighbour whilst to the south a narrow parcel of brownfield land is occupied by a single parking space.

The existing building is currently occupied by a textile merchant “Albert Jones Textiles Ltd” and has, by all accounts, served a similar purpose throughout its history. The building is used generally for material storage with a small office space albeit the second floor and basement have been disused for some time.

5 Geo-Environmental Conditions

A Preliminary Desktop Study has been undertaken for the site. The study was carried out by NX Consulting in June 2022, report reference NX451. Please refer to NX Consulting’s report for full details.

Key findings of the report are as summarised in the following sub-sections.

5.1 Geotechnical

The report indicates that a thickness of made ground would be expected on site given its development history. Based on historical borehole logs, underlying the fill a superficial deposit of Till comprising stiff clay, sands and gravel is expected above a bedrock of Chester Pebble Beds largely comprising sandstone at depth.

Relic in ground structures relating to former development are anticipated within the fill which may result in traditional trench / strip footings becoming uneconomical. A piled foundation solution is suggested.

5.2 Hydrological

Historical borehole logs (from the BGS website) reviewed show that no groundwater was encountered albeit the risk of flooding from groundwater is considered moderate. The site does not lie within a groundwater abstraction zone.

The site is within Flood Zone 1, less than 1Ha and therefore does not require a flood risk assessment. There is negligible risk of flooding from surface water.

5.3 Contamination

Potential contamination sources affecting the site were identified as heavy metals, PAHs, sulphate, asbestos, petroleum hydrocarbons PCBs, solvents and gas (carbon dioxide, methane and depleted oxygen) arising from the historic industrial uses of the site and the surrounding area.

The proposed building footprint covers the entire site and as such migration pathways of potential contaminants to end users are restricted.

5.4 Unexploded Ordnance

The site is in an area of moderate risk of encountering UXO and therefore a UXO desk study is required for the site. Historical maps show building clearance in the vicinity of the site circa. 1948, indicative of structures damaged from bombing.

5.5 Tunnels

The Guardian Underground Telephone Exchange Tunnel is located 20m to the NE of the site. The Duke Street Tunnel is located 80m to the south of the site.

The impact on these known assets from the development is considered to be low. Wayleaves, load restrictions and ground vibration limitations are to be adhered to in the design and construction of the new development.

MCC have confirmed a large diameter sewer located 4m away from the site that runs beneath Richmond Street. Former works in the area resulted in damage to the asset whereby piled foundations penetrated the sewer wall and filled the asset with concrete, subsequent remedial work was undertaken. The asset is to be located within the general vicinity of the site and the piled foundation solution is to be developed to avoid impacting the asset.

6 Services

The presence of buried services that may be impacted by the new development is currently unknown.

Record maps showing the location of known buried services are to be obtained.

7 Refurbishment of Existing Structure

7.1 *Proposals*

The existing Grade II listed building is to be refurbished and provide 4No. apartments accessed via a shared staircase.

To accommodate the change in use the existing floors will need to be upgraded for acoustics, fire compartmentation and insulation. Similarly wall construction will need to be upgraded to provide better thermal efficiency and it is understood that roof finishes will need to be refurbished or replaced to provide adequate weather tightness.

The partial basement is currently disused, and proposals will need to incorporate a new waterproofing system to prevent the continued ingress of ground and surface water into the space.

In addition to the Architectural upgrades and upon visual inspection of the property there are several latent structural defects that would need to be addressed and a few structural interventions that would need to be realised to permit the proposed change of use.

Proposed structural scheme for the existing building refurbishment can be found in Appendix A on drawings “14551-BKP-V1-ZZ-DR-S-0300 and 0301.”

Site survey notes can be found in Appendix B “Structural Survey Report” and Appendix B.1 on drawings “14551-BKP-V1-ZZ-DR-S-0300 and 0301.”

7.2 *Previous Investigations*

A Structural Inspection Report was undertaken by Brennan Consult in February 2022 (2762).

A further Structural Inspection Report was undertaken by BKPL in November 2022. The findings largely correlated with the BKPL survey obtaining structural member sizes in addition to verifying the Brennan proposals.

A measured building survey was conducted by ND Oliver in March 2022.

A specialist timber survey was undertaken by Hutton + Rostron in September 2022. The report identified damp timber members but did not observe rot generally (exception being embedded timbers in the basement forming the staircase). The report identified an area of previous fire damage. The report noted that all structural timber elements are softwood species. No visual strength grading was made.

7.3 *Remedial Works and Alterations*

The following sections are to be read in conjunction with the observations, conclusions, and recommendations of the aforementioned previous investigations.

7.3.1 *General*

1. Generally, the building is in good observable structural condition. The risk of imminent collapse due to structural instability is considered unlikely albeit there is evidence of structural movement and

excessive deformation, particularly at roof level. These issues should be addressed to, at the very least, arrest the development of further structural movement which may well result in instability.

2. Prolonged and ongoing moisture ingress is observable in damp timber elements, corroded metal elements and spalling brickwork elements. The moisture ingress should be addressed immediately to prevent further / future decay in the interim to site mobilisation. This would appear to be addressed by maintenance and repair of high-level rainwater goods, particularly to the rear elevation. Roof finishes were observed to be in good condition by the timber survey report although defective verge mortar was observed at the gables. The neighbouring property owner should also be advised that defects in their rainwater goods are adversely impacting the property at 51-53.
3. The building was constructed at a time when there was little to no consideration of structural robustness and redundancy. Proposals would classify the building as Class 2A (medium consequence) as such refurbishment works should allow for meeting the robustness requirements of such a class of building. This will involve installing effective horizontal tying between the floors and walls.
4. Assessment of the capacity of existing structural members at all levels and comparison against future loading once refurbished is essential. An alternative approach would be to compare current and historic loading against that expected once refurbished, a shortfall in applied load would demonstrate structural adequacy. Assessment of the timber elements will need to be conservative and assume Grade C16 timber. This assessment will need to have due consideration for the structural capacity of a member under fire conditions if it is to be left exposed and unprotected. Depending on the outcome of the assessments, members will need to be adequately strengthened or fire protected to provide the requisite capacity.
5. Chimney stack, if not proposed for use should be capped at high level.

7.3.2 Basement Level

1. Generally, in good observable structural condition. See general recommendations.
 2. The basement requires significant remedial tanking to halt the ingress of water into the basement. This could take the form of an internally applied waterproofing slurry and the construction of a cavity drain system. The assistance of a waterproofing specialist should be sought to arrive at the most suitable solution.
 3. The existing basement floor should be removed wholesale and replaced with a new structural concrete floor slab with waterproofing integrated into the wall tanking solution. Existing foundations are to be located such that proposed flooring solution installation does not result in undermining the structure or its neighbour.
 4. The charring to the main internal floor beam identified in the timber survey report needs to be exposed and assessed in detail by the timber specialist. At this juncture it would be prudent to allow for repair of this section of the beam by introduction of steel flitch plates.
 5. Central timber beam bearing at the rear elevation is to be assessed by specialist for fungal rot and beetle infestation due to prolonged rainwater ingress. The bearing end could not be assessed at the time of survey due to obstruction by stored goods. At this juncture it would be prudent to allow for repair of this bearing end of the beam by introduction of steel flitch plates.
 6. Embedded timber elements forming the staircase structure above have been identified as exhibiting signs of rot and are to be removed and replaced in the reconstruction of the staircase.
 7. Embedded material and voids in the masonry are to be removed and infilled.
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8. The staircase to the basement is to be reconstructed. Materiality can be largely dictated by the Architect in this case.
9. Positive fixing between the column head and beam should be confirmed and provided if absent.
10. Noggins to be installed at joist midspan and at supports.

7.3.3 Ground Floor Level

1. Generally, in good observable structural condition. See general recommendations.
2. Embedded material and voids in the masonry are to be removed and infilled.
3. Central timber beam bearing at the rear elevation is to be assessed by specialist for fungal rot and beetle infestation due to prolonged rainwater ingress. The bearing end could not be assessed at the time of survey due to obstruction by stored goods. At this juncture it would be prudent to allow for repair of this bearing end of the beam by introduction of steel flitch plates.
4. Positive fixing between the column base/head and beam should be confirmed and provided if absent.
5. Noggins to be installed at joist midspan and at supports.
6. There is a region to the corner of the front of the building where the FFL is to be raised and allow for adequate head height to the entrance steps into the basement. Here the floor joists will need to be cut and trimmed to suit and a suitably raised timber floor construction installed.

7.3.4 First Floor Level

1. Generally, in good observable structural condition. See general recommendations.
2. Embedded material and voids in the masonry are to be removed and infilled.
3. The gap observed at ceiling level (at ground floor below) is a result of lack of tying between wall and floor. Retrospective tying of the front elevation to the wall should stop further movement, involving the installation of noggins and positively fixed helical steel bars through the wall and into the floor structure. Should the movement need to be corrected for aesthetic purposes, a (number of) tying rod(s) would need to be installed across the building and mechanically jack the masonry plumb and tight to the floor slab. Pattress plates would be visible externally, similar to those existing at second floor level.
4. Positive fixing between the column base/head and beam should be confirmed and provided if absent.
5. Split to the side face of the beam is to be investigated by specialist.
6. Noggins to be installed at joist midspan and at supports.

7.3.5 Second Floor Level and Roof Structure

1. Generally, in good observable structural condition. See general recommendations.
2. Embedded material and voids in the masonry are to be removed and infilled.
3. The masonry piers either side of the central truss bearing piers will need to be reconstructed plumb and cracking in the masonry will need to be addressed. This will likely require temporary propping of the rafters bearing onto the wall plate above the piers. Supplementary in plane bracing may also be required to prevent additional movement due to tension in the floor tying rods upon removal of vertical load from the masonry.
4. Spread of the truss has resulted in minor cracking at the bearing piers, the front pier more significant than the rear. These will need to be locally rebuilt, and the truss ends re-seated onto them. This will require temporary propping of the central truss. The main concern structurally here is that the spread

has occurred despite a cast iron tie suggesting some softening of the timber sections, likely a result of moisture ingress.

5. The racking in the ceiling structure will need further analysis to ascertain whether some degree of correction is necessary. If required, this is likely to involve the temporary propping and jacking of the purlins and roof truss. The position of the roof truss can be corrected plumb and the roof purlins then re-seated onto the truss top chord. Roof finishes will need to be removed whilst this work is carried out and at the same opportunity, the build-up improved or replaced as specified by the Architect.
6. In plane roof bracing will need to be installed to arrest further racking and spread of the truss. The lack of bracing in the roof structure is an inherent structural flaw that requires remedial action.

7.3.6 Front Elevation

1. Refer to "First Floor Level" Point 3.
2. Refer to "Second Floor Level and Roof Structure" Point 3.
3. Washed out mortar to be re-pointed with suitably specified lime mortar.

7.3.7 Gable Elevation

1. Render to be removed to reveal extent of masonry defects.
2. Render to be carefully removed to reveal underlying brickwork.
3. Structural cracking to be repaired via embedded steel helical ties within the mortar joints across the cracking.
4. Washed out mortar to be re-pointed with suitably specified lime mortar.

7.3.8 Rear Elevation

1. Inspection of the rear elevation was very limited due to access restrictions and health & safety concern.
2. Washed out mortar to be re-pointed with suitably specified lime mortar.
3. Gap between paving and building face to be repaired and mortar fillet to be provided.
4. Removal of the metal escape staircase affixed to the rear of the building should be considered. Significant shearing of the upper-level flights was observed in the timber survey report and this would be considered unsafe for in its current condition. Lower flights were not accessed or observable.

7.4 Assessment of Existing Structure

7.4.1 Load Balance

The historically applied loading is compared to proposed loading following refurbishment below.

The historically applied loading has been derived from observations made during the site walkover and anecdotal evidence of the floor use from the building tenant.

Despite a significant increase in the proposed dead load, arising from the addition of compartmentalising elements such as a 50mm concrete screed, there is to be a decrease in the applied live loading on the building whereby historic storage loading, taken to be 3.0kN/m² is reduced to domestic loading of 1.5kN/m² + 0.5kN/m² for lightweight timber stud partitions.

At roof level the addition of the 5-storey town house to the gable creates opportunity for the accumulation of snow drift against it where previously this was not the case. As such imposed snow load has been calculated to an increased 0.78kN/m^2 , albeit this is a local effect adjacent the new build.

Due to the increase in dead load, there is an increase in overall loading on the building structure of approximately 11%. As such the structural members as measured during the BKPL internal walkover have been assessed for the proposed loading.

Table 1: Load Balance of Historically Applied Load vs Proposed Applied Load

Level	Historic Floor Load (kN/m^2)		Proposed Floor Load (kN/m^2)		Change in Dead Load	Change in Live Load	Change in Total Load
	Dead Load	Imposed Load	Dead Load	Imposed Load			
Ground	0.20	3.00	1.60	2.00	700%	-33%	11%
First	0.20	3.00	1.60	2.00	700%	-33%	11%
Second	0.20	3.00	1.60	2.00	700%	-33%	11%
Roof	1.00	0.60	1.00	0.78	0%	30%	10%

7.4.2 Timber Floor Joists

Grade of timber has been conservatively assumed to be a softwood Grade C16.

As per the observations made during the site walk over, the bearing at the central beam end is taken to be notched from the bottom to 140mm depth and 80mm bearing. The bearing into the wall was not confirmed and has been assumed to be full depth and 80mm.

The following material properties have been adopted in assessment of the timber floor joists at each level.

Grade	C16
Service Class	2
Tensile Strength	10N/mm^2
Compressive Strength	17N/mm^2
Young's Modulus	8000N/mm^2

The timber joists have been assessed in accordance with:

- BS EN 1995-1-1 Design of Timber Structures - General rules, and rules for buildings
- BS EN 1995-1-2 Design of Timber Structures – Structural Fire Design

Normal Conditions

Under general working conditions and occupancy load the 280x80 timber floor joists at 400mm c/c are **adequate** for the proposed refurbishment.

As noted previously, the joists require the addition of noggins and restraint strapping to accord with Approved Document A and the timber survey report will provide recommendations on the condition of the damp joist ends.

Fire Conditions

Under fire conditions and occupancy load, the 280x80 timber floor joists at 400mm c/c are **inadequate** for the proposed refurbishment and required 60minute fire period.

It is recommended that the timber floor joists are protected by an underdrawn ceiling.

7.4.3 Timber Floor Beam

Grade of timber has been conservatively assumed to be a softwood Grade C16.

As per the observations made during the site walk over, the bearing at the central column has been taken to be 250mm long. The bearing into the wall was not confirmed and has been assumed to be full depth and 100mm.

The following material properties have been adopted in assessment of the timber floor joists at each level.

Grade	C16
Service Class	2
Tensile Strength	10N/mm ²
Compressive Strength	17N/mm ²
Young's Modulus	8000N/mm ²

The timber joists have been assessed in accordance with:

- BS EN 1995-1-1 Design of Timber Structures - General rules, and rules for buildings
- BS EN 1995-1-2 Design of Timber Structures – Structural Fire Design

Normal Conditions

Under general working conditions and occupancy load the 400x240 timber floor beams are **inadequate** for the proposed refurbishment.

The beam currently fails in compressive shear failure perpendicular to the grain at the wall bearings. It must be noted that this failure mode does not appear to have occurred in the current configuration and use. This suggests that the assessment is indeed too conservative and that an upgraded, perhaps hardwood, strength grade is adopted.

With the previous recommendation to board the timber joists, there may be alternative compartmentalisation measures that do not include the addition of a concrete screed. This would in turn reduce the applied dead load and likely result in adequacy.

The timber survey report will provide recommendations on the condition of the damp joist ends.

Fire Conditions

Under fire conditions and occupancy load, the 400x240 timber floor beams are **adequate** for the proposed refurbishment and required 60minute fire period.

Once justified under normal working load, the beam may be left exposed in the proposed condition.

7.4.4 Cast Iron Column

As per the recommendations of CIRIA Report 111, the following material properties have been adopted in assessment of the cast iron column located centrally through the building.

Tensile Strength	120N/mm ²
Compressive Strength	590N/mm ²
Young's Modulus	66000N/mm ²

Wall thickness of the section has conservatively been adopted as ½ inch (12mm). This should be verified by sampling at the top-most column.

Normal Conditions

Under general working conditions and occupancy load the 100mm diameter cast iron columns are **adequate** for the proposed refurbishment.

Fire Conditions

The column and head plates should be adequately protected through use of an applied intumescent paint.

Care should be taken when stripping the existing paint from the column. The era of the original construction saw common use of lead-based paint.

7.4.5 Roof Structure

With negligible increase in imposed load and overall greater certainty regarding the historic loading vs the proposed loading for the roof, it is by engineering judgement that the structural members present are adequate for the refurbishment.

The assessment has instead focussed on the suitability of the roof structure in its currently deformed geometry for adequacy. As per the structural inspection report, the roof appears to have been reconstructed some time after WWII with latent defects in its construction, namely the lack of in plane bracing and restraint.

The main truss has racked approximately 5 degrees out of plumb (to be accurately verified on site). At the apex of the roof this equates to a sideways deflection of approximately 210mm.

In its current configuration the roof structure is, theoretically unstable, with very little in the way of resistance to lateral force being offered by either gable wall. The lateral forces are a result of vertical load being applied to the out of plane truss, the truss tends to lean further to the side its already deflecting toward and in turn applies an axial force to the purlins which apply a lateral reaction into the head of the gable wall.

The intended intervention here would be to strap the purlin ends to the gable walls and provide new in plane timber bracing tying the purlin ends to the main timber truss. This should stabilise the roof structure by reducing the lateral reaction into the gable walls and limit any further sideways racking to an acceptable amount.

It should be noted that the restraint of the gable wall will be improved under the development proposals via the erection of the new build element tight to the gable in the vacant plot. The lateral resistance of the gable wall will not be zero as theoretically assumed.

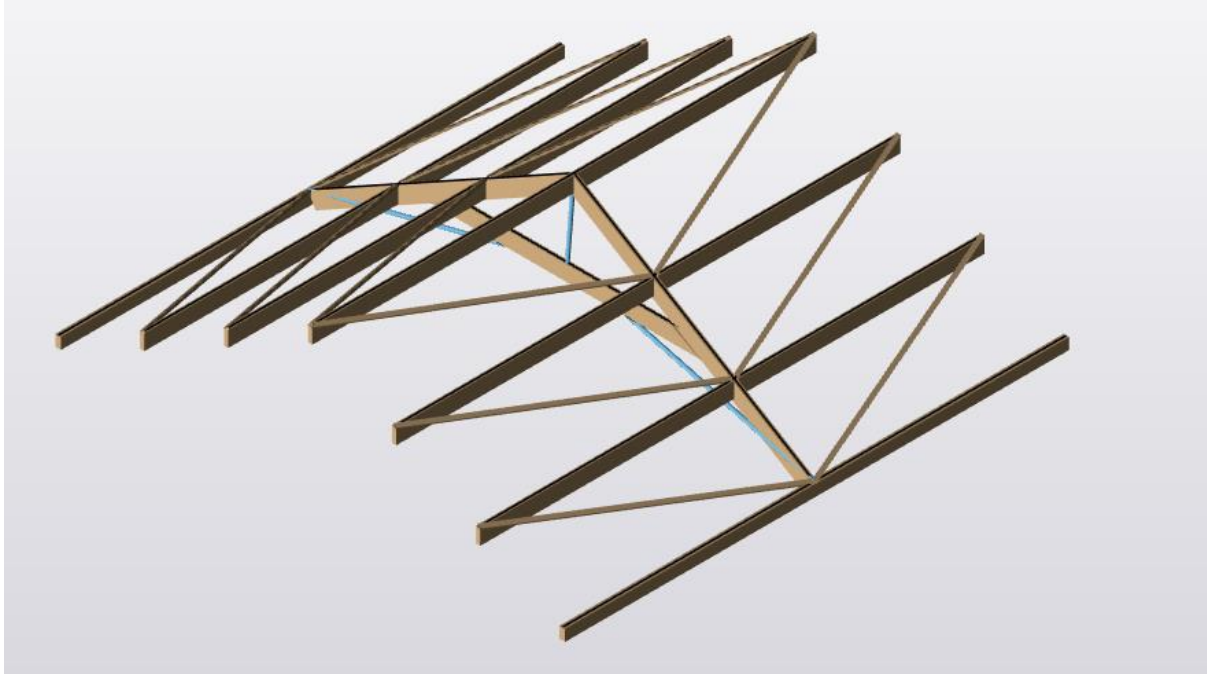


Figure 3: Tekla Model of the Braced Roof Structure (Isometric View) NB. Rafters omitted for Clarity

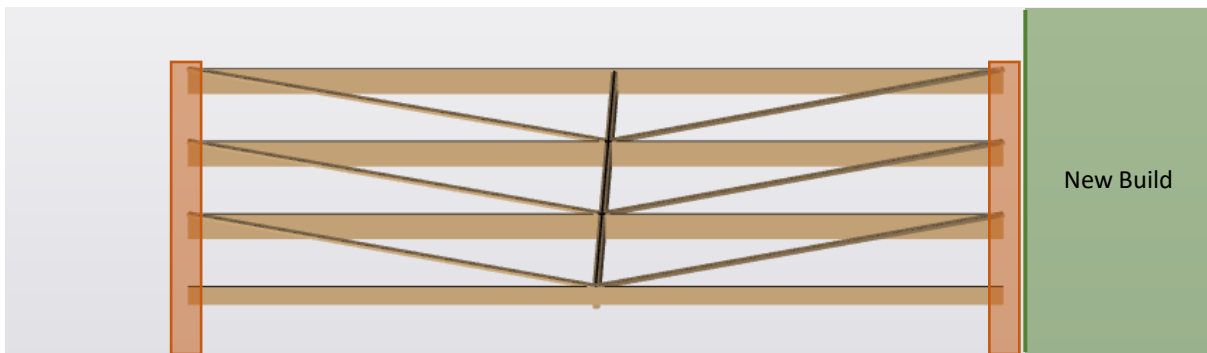


Figure 4: Tekla Model of the Braced Roof Structure (Front Elevation)

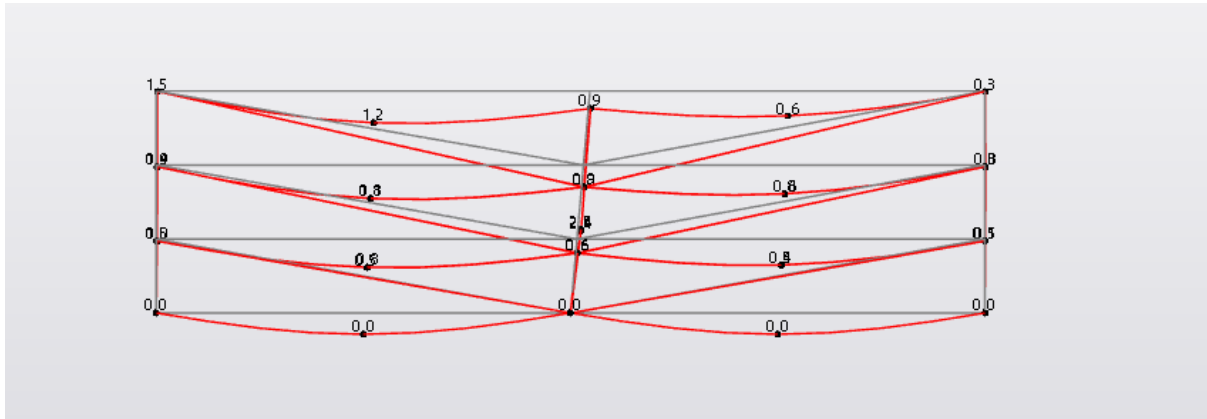


Figure 5: Predicted Further Lateral Deflection Under Proposed Loading once Braced

Should the Architect wish to realign the truss plumb for aesthetic purposes, then it is envisaged that through propping of the roof purlins would be needed to basement level to relieve the truss of applied load and temporarily stabilise it. There may also be need for splicing of new bearing ends to the purlins to achieve physical support onto the re-aligned central truss. Should roof rafters also need to be realigned, these would need to be detached and reattached in turn upon realignment of the central truss and augmentation of the purlins. This process would need to be co-ordinated with upgrade/replacement of roof finishes.

9 New Build Structure

The new build portion of the proposals comprises a five-storey town house to be constructed in the narrow gap between the refurbished mill building and the neighbouring property.



The plot is currently used as parking. There is a masonry wall to the rear and a galvanised steel full width gate to the front.

To the rear there is also a metal escape staircase encroaching slightly over the brick wall.

Proposed structural scheme for the new build can be found in Appendix A on drawings “14551-BKP-V2-ZZ-DR-S-0300, 0301 and 0310.”

9.1 Super-Structure

The current structural concept proposes a hot-rolled steel frame with a suspended CLT floor slabs at this stage. Alternative options would include a full CLT structure, employing CLT slabs and structural walls as well as a steel framed option with alternative flooring solution (ie. timber joists or concrete composite slabs).

Due to the tight site constraints the proposals include a centralised line of columns and a line of columns against to the neighbouring building. The floor structure cantilevers approximately 1.5m toward the existing building. This allows for the structural columns to be positioned on partition wall lines within the townhouse floor plan and allow for new foundations remote from the existing building.

Vertical action arising building self-weight and occupancy loading are to be supported from timber floor joists and steel beams which transfer the loads via bending and shear into the supporting columns. The supporting columns, in turn, transfer the loads via axial compression, bending and shear to the concrete substructure.

Vertical load is to be derived in accordance with BS EN 1991-1-1 to 3 (inc).

Any barrier loading is to be derived in accordance with BS EN 1991-1-1.

9.2 Sub-Structure

Due to ground conditions, presence of a known basement on one side and a potential basement on the other, a piled foundation solution is proposed. A deep foundation solution will bypass the anticipated thickness of made ground, relic in ground structures and avoid surcharging the adjacent building's basement and foundations.

Piles will transfer vertical loads via axial compression into the natural deposits beneath the made ground. Actual ground conditions on site are to be ascertained via intrusive investigation.

The foundation scheme will employ a RC ground beam suspended on a series of bored piles. The piles will need to be installed with clearance from the adjacent buildings and likely need to be installed using low-vibration techniques to avoid undue disruption to neighbours and disturbance of the underground tunnels identified in the Desktop Study. As such the columns against the neighbouring footings are to be supported from slightly cantilevered RC ground beams.

We anticipate a suspended RC ground floor slab.

9.3 Stability

The structure will be subject to lateral load arising wind and geometric frame imperfections. Stability against such loads is provided by diaphragm action of the floor structure and transfer of in plane forces to the steel frame and the braced bays in orthogonal directions. Braced bays will then transfer the loads via axial tension and compression in the bracing and columns, respectively, to the foundations.

The braced bays are located to the rear of the building around the full height staircase. Bracing is in the form of steel flat cross bracing.

Wind action is to be derived in accordance with BS EN 1991-1-4.

Equivalent Horizontal Forces (EHFs) are to be derived in accordance with BS EN 1993-1-1 and BS EN 1991-1-6.

9.4 Disproportionate Collapse

The building is a single dwelling over 5-storeys. In accordance with BS EN 1991-1-7 the building is classified as Class 2A (Lower Risk Group – Medium Consequence of failure).

The structure will be designed based on limiting the extent of localised failure due to unidentified accidental actions.

The robustness requirements for a Class 2A building includes designing the structure in accordance with the rules given in Approved Document A and providing effective horizontal ties.

Horizontal tying can be beneficial to a structure affected by an accidental action by enabling catenary action to develop and hold columns in position in the event of a localised failure due to accidental action.

Design tying forces are to be derived from equations A.1 and A.2 of BS EN 1991-1-7.

9.5 Movement Joints

No movement joints are anticipated within the new structure.

Full height movement joints are to be provided between the existing buildings at each side of the new structure.

Against the existing building's gable wall, it is intended to provide positive fixity between the new steel frame and the existing building for the reasons described prior in improving restraint to the wall.

9.6 Stairs

The staircase within the new building is expected to be of secondary steel construction. A timber solution would be equally viable.

9.7 Cladding

External wall build up to the new building is proposed to be of prefabricated insulated panel construction with an outer layer timber cladding.

At each floor level the perimeter beam to the front and rear elevation are exposed. This is expected to be detailed as a dummy member that is thermally divorced from the actual structure.

This gives a stripped back, industrial aesthetic, in keeping with the building's heritage and surroundings.

9.8 Party Wall

The new structure will be constructed tight to the neighbouring property. As such a party wall agreement will need to be entered into between the Client and the owner of the neighbouring property. A party wall surveyor will need to be appointed.

Low level air bricks are located on the neighbouring building and ventilation will need to be maintained by the proposed development.

Structural proposals will be considerate to this and as such the steelwork nor the substructure will take support from nor surcharge the party wall.

9.9 Construction Sequence

Tight site conditions and Architectural plans dictate the execution of a structural steel frame (an alternative timber / CLT frame would be equally feasible). The steel frame can be erected quickly and quietly in the existing gap without the need for space hungry temporary works or formwork.

1. The general construction sequence will be as follows:
2. Remove existing wall, gate, car park surfacing and buried drainage.
3. Identify and grub out any relic foundations.
4. Reduce level dig to formation level and install nominal piling mat (to specialist's design and detail).
5. Install piling (to specialist's design and detail).
6. Construct new RC ground beam and cast in holding down bolts.
7. Assemble structural frame from the rear of the site forward including staircase.
8. Install RC ground floor slab.
9. Install timber floor structure.
10. Install the roof level waterproofing and cladding.

9.10 Design Codes

As a minimum the building will comply with all relevant building design standards, particularly, but not limited to:

Building Regs Approved Document A: Structure

- | | |
|----------------|--|
| BS EN 1990-1 | Basis of Structural Design |
| BS EN 1991-1-1 | Densities, self-weight, imposed loads for buildings |
| BS EN 1991-1-2 | Actions on structures exposed to fire |
| BS EN 1991-1-3 | General actions - Snow loads |
| BS EN 1991-1-4 | General actions - Wind actions |
| BS EN 1991-1-6 | General actions - Actions during execution |
| BS EN 1991-1-7 | General actions – Accidental actions |
| BS EN 1993-1-1 | Design of Steel Structures - General rules, and rules for buildings |
| BS EN 1995-1-1 | Design of Timber Structures - General rules, and rules for buildings |
| BS EN 1995-1-2 | Design of Timber Structures – Structural Fire Design |
| BS EN 1997-1-1 | Geotechnical Design – General rules |

10 Drainage

Please refer to the Drainage Strategy Report.

10.1 Disclaimer

This report is for the use of Eminence Development Ltd in connection with this appointment and the benefit is not to be extended to any third party without the written permission of Booth King Partnership Ltd.

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