CIVIC



Marlsbro House, Manchester

Structural Appraisal Report

November 2025

PREPARED FOR

Aclo Assets Ltd

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Appendices

1. Introduction

Civic were instructed by Aclo Assets Limited in September 2025 to carry out structural appraisal of the existing building at No 52 Newton Street (known as Marlsbro House), to inform the design process for the proposed redevelopment works.

This report provides a description of the structural form and condition of Marlsbro House ahead of determining the scope of structural remedial and intervention works required to convert the building to commercial office and retail/leisure use. Detailed proposals for the redevelopment works are not known at the time of survey and writing, and an architect appointment has not yet been made.

The site comprises a four storey 19th century building with attic and basement, which was originally constructed in the early 19th century as a cotton mill. It was subsequently altered to become an iron manufacturers in 1830, and then a series of light industrial/commercial uses from 1850. It was extensively altered in 1960 which includes the addition of a cement based render to the existing external walls, new entrance foyer and other substantial internal and external alterations.

The site is bounded by Faraday Street to the north-east, Port Street to the south-east, Hilton Street to the south-west, and Newton Street to the north-west (see Fig 1.1). The building is Grade II Listed and lies within the Stevenson Square Conservation Area (see Fig. 1.2 and 1.3).

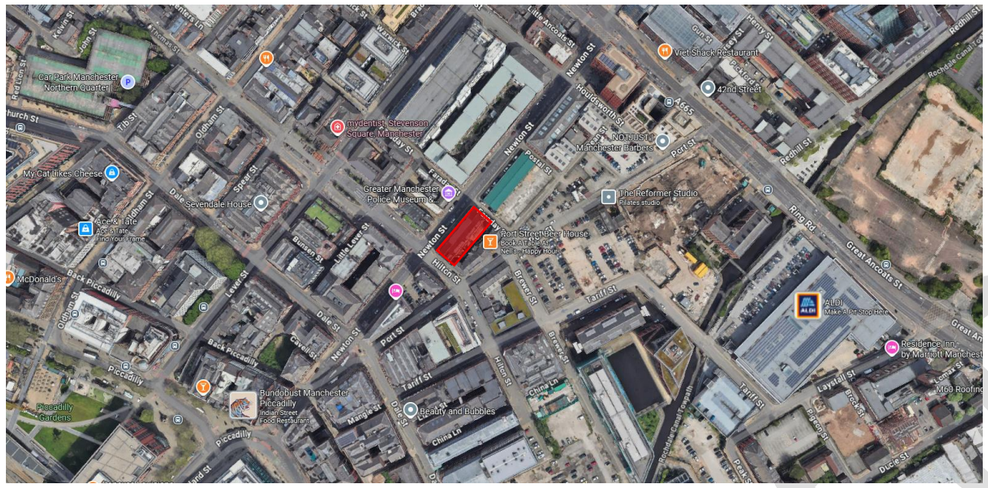


Figure 1.1: Site Location from Google Maps

The site was first developed between 1780 and 1794 with the construction of smaller square building in the centre of the plot, with yards to the front and back. This building is likely to have been an early cotton mill.

The current footprint of the building was first fully developed by 1809 as a cotton mill.

A total deconstruction of that cotton mill is recorded following a devastating fire in February 1823. The ruins of the mill were presumably demolished and a new structure constructed between 1823 and 1824 on the same footprint (the present Marlsbro House).

The building underwent substantial refurbishment in 1960, by Drury & Gomersall architects. The scheme was to modernise the building and created areas for a raincoat manufacturer, factory, warehouse and distribution.

The building is a long rectangular block occupying half of the plot bounded by Newton St, Faraday St, Port St and Hilton St (see Fig. 1.2).

The three street elevations are faced with a checkboard pattern of grey cementitious render. On the rear elevation a grey cementitious render covers the external masonry wall.

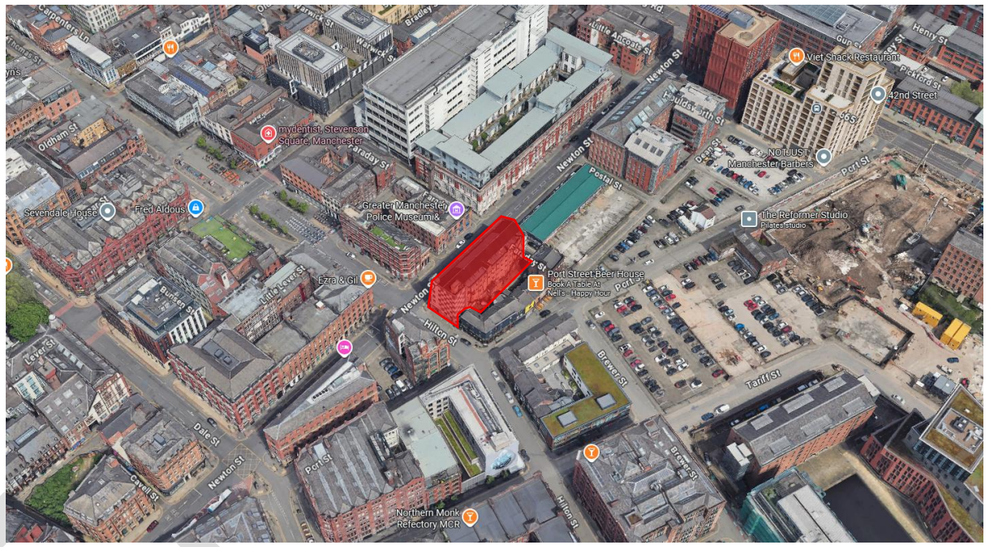


Figure 1.2: Google Maps 3D Imagery of Marlsbro House

The building is of loadbearing masonry (brick) wall construction with timber beam and millboard floors, supported by a central line of slender cast iron columns. External walls comprise solid brick piers linked by thinner brick spandrel panels beneath window openings in some locations. A more recently constructed lift and staircase core extension was added in the southern corner of the building.

The roof structure compromises a combination of both cast/wrought iron and timber members in a flawed adaptation of a ‘queen-post’ truss arrangement (TBC following further access for inspection).

A visual non-intrusive assessment was conducted on the 19th September 2025 with a second site visit on the 17th October 2025.

Prior to this, Civic Engineers were engaged by the former building owner, and completed a structural assessment and feasibility studies to inform a planning application submitted in 2017.

The building is understood to have been vacant since 2020.

The survey was limited to a general visual inspection of the property. Externally the building was viewed from ground level and internally from either ground or upper floor levels where safe access was possible. The existing floor beams and millboards and external walls had been stripped of finishes and are generally visible in their entirety throughout the floor plates at basement, ground and upper floor levels. Finishes remain in place within the attic floor level due to previous conservation restrictions during the prior strip out works. As a result, it is understood asbestos remains in situ within the attic and eaves areas, limiting visual inspection of the structure at this level and the roof construction over. No opening up works were carried during the inspection.

Some localised intrusive exploratory works were carried out to the elevations in June 2017 to assess the nature of the applied render.

Whilst all reasonable care has been exercised in identifying defects and structural problems, there may be underlying faults which are not ordinarily revealed during an inspection of this kind, which could affect the outcome and cost of the proposed remedial works.

1.1. Site History

This section of the report provides an outline of the historic development of the site and the notable structural alterations. For a fuller account, refer to *MARLSBRO HOUSE, 52-54 NEWTON STREET, MANCHESTER - Heritage Impact Assessment*, prepared by Donald Insall Associates.

Marlsbro House sits within the Stevenson Square Conservation Area (see Fig. 1.3), which lies on the north-eastern edge of the Manchester City Centre and was designated in February 1987. The building was Grade II Listed in November 2019.

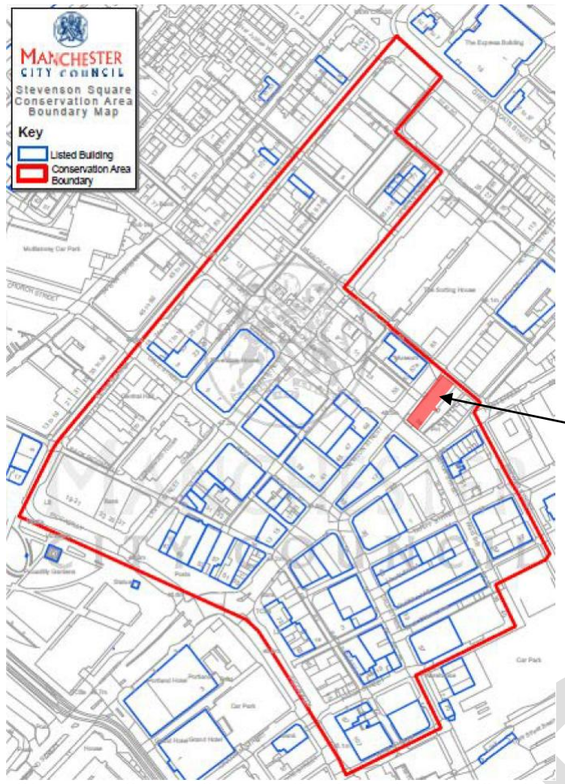


Figure 1.3: The extent of the Stevenson Square Conservation Area shown outlined in red showing with listed buildings outlined in blue.

The following is a brief description of the historical development of the site, highlighting the different structural alterations that affected the building, as far as we can reasonably ascertain:

1824

New building constructed on the same footprint as the present Marlsbro House following reconstruction of Samuel Barker's cotton mill due to fire. (see Fig 1.4). It maintained its previous use as a cotton mill.



Figure 1.4: 1824 map showing the site following the reconstruction after the fire in 1823.

1850 - 1920

The building was sold by William Bennett and it appears that the building subsequently became used by multiple tenants for different purposes.

It is understood that the building was subjected to minor alterations during this period:

- 1820s metal framed windows were replaced with Victorian sash windows;
- The main entrance was moved to the Hilton Street façade, where a new stone ground floor was inserted, creating a central doorway access;
- The gable wall facing Hilton Street was extended to the apex of the pitched roof to conceal the attic floor and roof from view (refer to Fig. 1.5).

By 1913 the building was used as a mantle factory on the ground floor and a costume factory at the upper floors. A hoist/lift serving each floor was later added in the north-west corner of the building (see Fig. 1.6).



Figure 1.5: 1932 – A view looking north-east down at the building showing the extended elevation on Hilton Street up to the roof apex.

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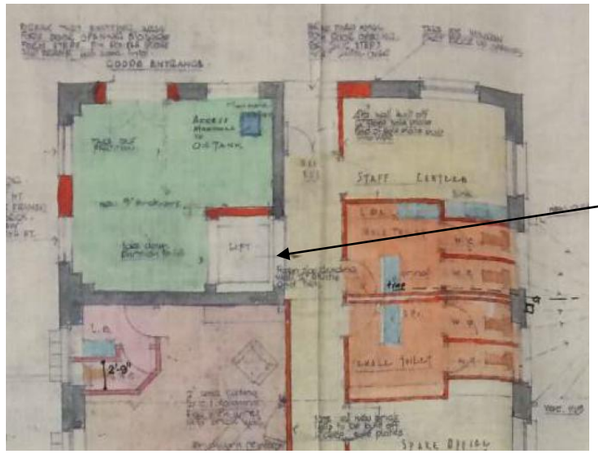


Figure 1.6: 1958 plan for the new hoist.

1960

The building underwent substantial refurbishment in 1960, designed by Drury & Gomersall architects.

Internally the building was altered with the extension of the stair and lift core in the southern corner (see Fig. 1.7), which extended from the basement to attic floors. The works also included the creation of a new level access entrance from the corner of Hilton Street and Newton Street and the formation of a new entrance foyer.

Externally the building was rendered with a checkboard pattern of grey render, assumed to be cement based render (see Fig. 1.8). On the rear elevation a homogeneous grey render covers the external masonry brickwork.

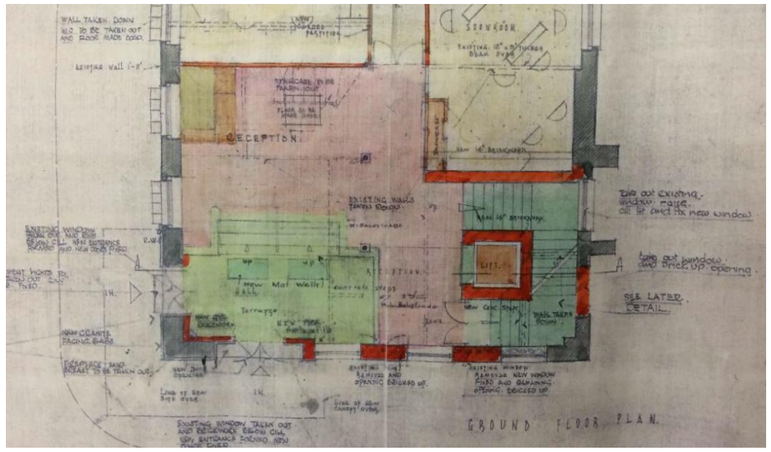


Figure 1.7: 1958 plan for the new stair and lift area.

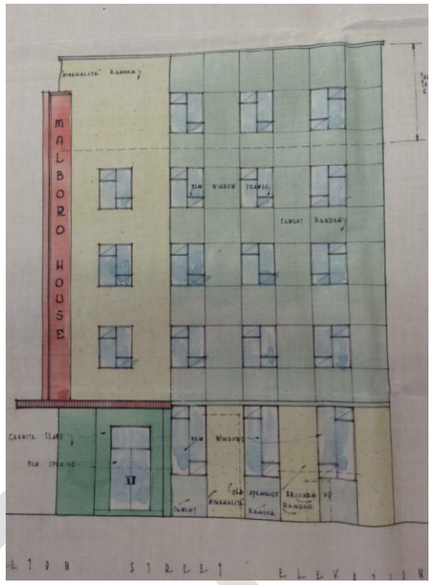


Figure 1.8: 1958 Hilton Street elevation.

1.2. Site Geology

At the time of writing this report, an intrusive site investigation (SI) has not been carried out to establish either the ground conditions, or the exact form and size of the existing foundations.

Other site investigations in the area have recorded made ground over layers of drift deposits comprising clays ranging from firm to very stiff, followed by stiff to very stiff glacial clays. Below these clays is Sherwood Sandstone (also known as “Bunter Sandstone”).

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2. Existing Structural Arrangement

The building is of loadbearing masonry (brick) wall construction with timber beam and millboard floors, supported by a central line of slender cast iron columns (see Fig. 2.1 and 2.2). External walls comprise solid brick piers linked by thinner brick spandrel panels beneath window openings in some locations. A more recently extended lift and staircase core was added in the southern corner of the building (see Fig. 1.7).

The roof structure compromises a combination of both cast/wrought iron and timber members in a flawed adaptation of a 'queen-post' truss arrangement (TBC following further access for inspection).

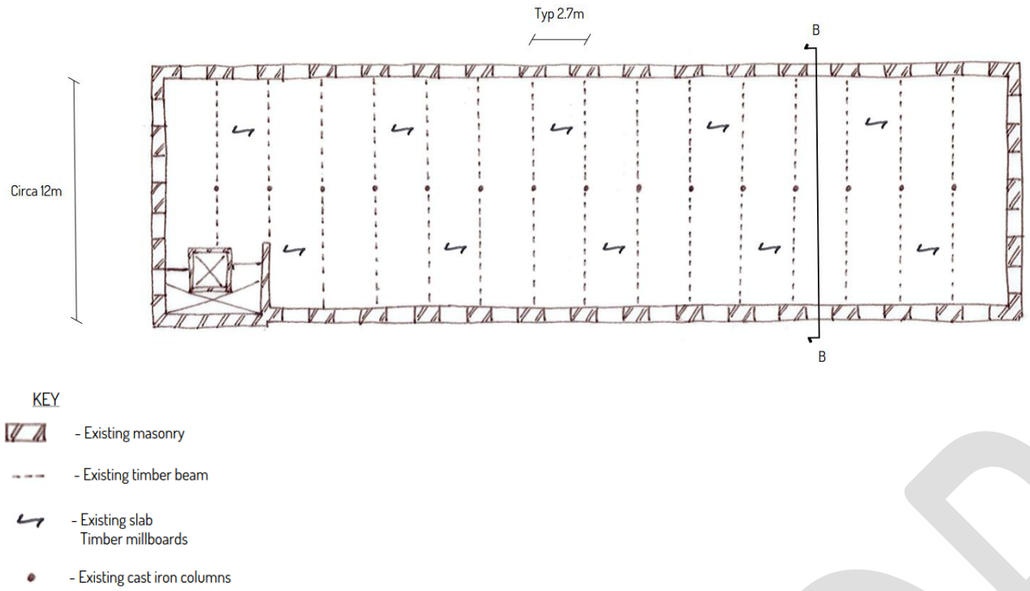


Figure 2.1: Typical floor structural arrangement

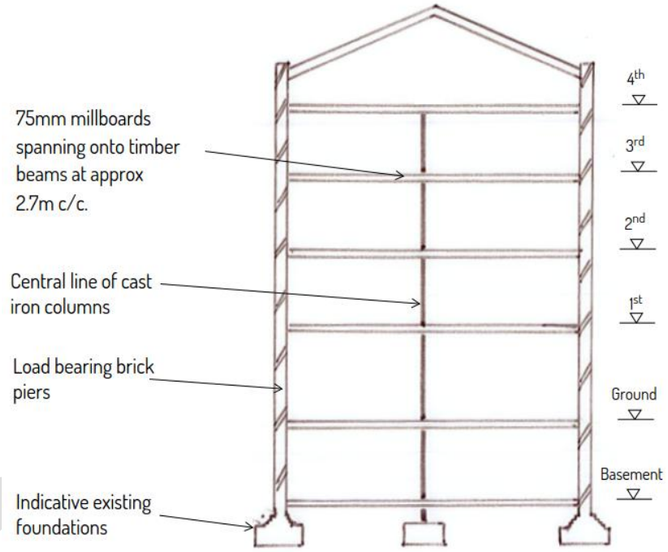


Figure 2.2: Section B-B – Typical building section.

2.1. Roof Structure

The roof structure comprises a combination of both cast/wrought (TBC) iron and timber members in an adaptation of a 'queen-post' truss in combination with a 'mansard truss' arrangement (refer to Fig. 2.3). Given the presence of finishes remaining insitu at this level, it is not possible to garner a clear, unobstructed view of the roof structure in its entirety.

It was designed at a time when the transition from all-timber to all-iron roof was underway, and it has a mixture of iron and timber members. It appears that the roof trusses and the iron frames for the "clerestory windows" are contemporary with each other. There are iron purlins whose positions and profiles suggest that they were designed for the present roof profile (refer to nodes 9,10,11 and 12 on Fig. 2.3). If the roof line followed the line of the strut it would cover the clerestory windows so they could not admit natural light. Despite these indications, however, it is not currently possible to determine absolutely that the current arrangement is original.

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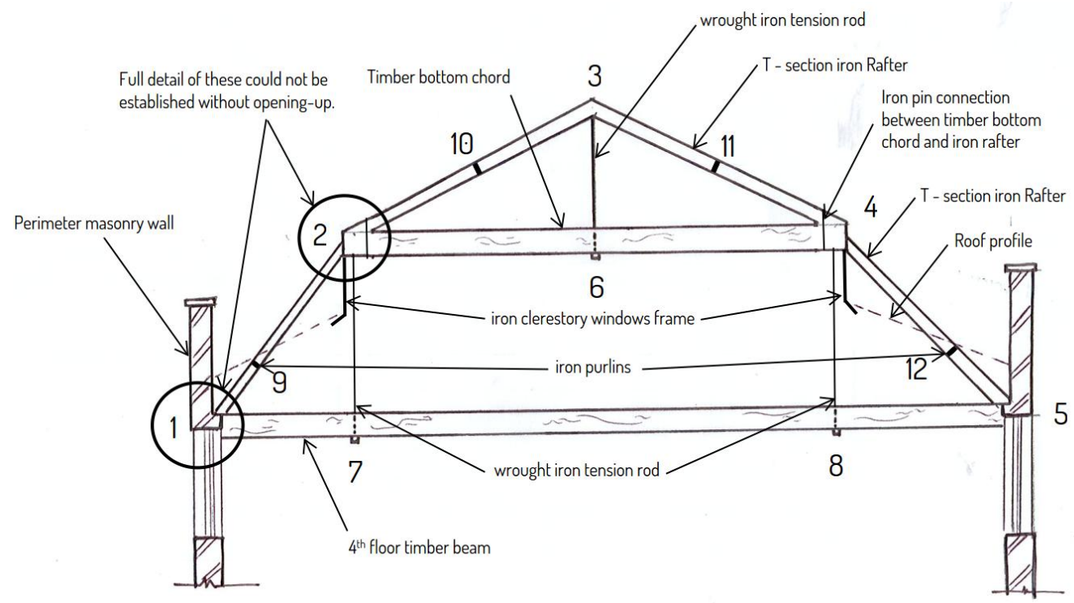


Figure 2.3 : Typical roof structure section.

The construction of the roof is slate coverings on timber battens supported off timber rafters spanning between iron purlins. The iron purlins span onto the iron rafter described above (refer to Fig. 2.4 and 2.5).

Typical clear headroom from floor to underside of timber beam is approx. 2.5 m.





Figure 2.4 and 2.5: Attic level showing iron rafters and purlins (images from 2017 due to current restricted access)

For simplicity in the description, the truss structure is split into the Attic and the 4th Floor section.

At attic level (refer to Fig. 2.6) the iron T-section rafters appear to be connected at the base by iron pin to the timber bottom chord (refer to Fig. 2.7), which supports the attic level timber floor. This timber member spans the width of the attic level (which is reduced from the overall building width due to the roof pitch). A wrought iron tie rod, hung from the apex of the iron rafter above, offers an additional support at the midspan of the bottom chord (see Fig. 2.8 and 2.9). Refer to section 3.1 for further notes on condition.





Figure 2.6 and 2.7: Attic level general view and pin connection between bottom chord and iron rafter (taken in 2017 due to current restricted access)





Figure 2.8 and 2.9: Attic level iron tension rod and mid-span pin connection to timber member (2025)

At the 4th level (refer to Fig. 2.10), additional iron (T section) rafters transfer the roof and the attic floor loads to the brick piers at the lower level (see Fig. 2.5). A pair of tension rods (wrought iron) run from the bottom of the timber bottom chord at the attic level to the timber beams of the fourth floor below (refer to Fig. 2.11, 2.12 and 2.13).





Figure 2.10 and 2.11: General view of 4th floor level and tension rod at eaves location





Figure 2.12 and 2.13: Wrought iron tension rod and clerestory window frame and pin connection between iron members

The structural issues relating to the roof trusses are best understood by comparing the trusses at Marlsbro House with a typical queen post truss from an earlier time, when all the members were timber, and they were all capable of carrying tensile or compression sizes (refer to Fig. 2.14).

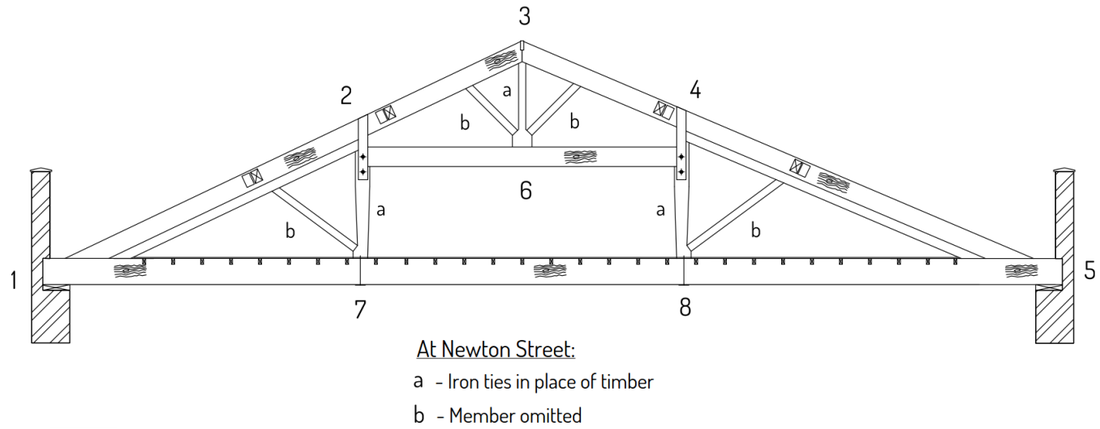


Figure 2.14: Typical all timber queen post truss.

The Marlsbro House trusses have some timber members omitted, and others have been replaced with iron tie rods. Under most conditions this would not be a problem, and the rods would remain in tension (refer to Fig. 2.15).

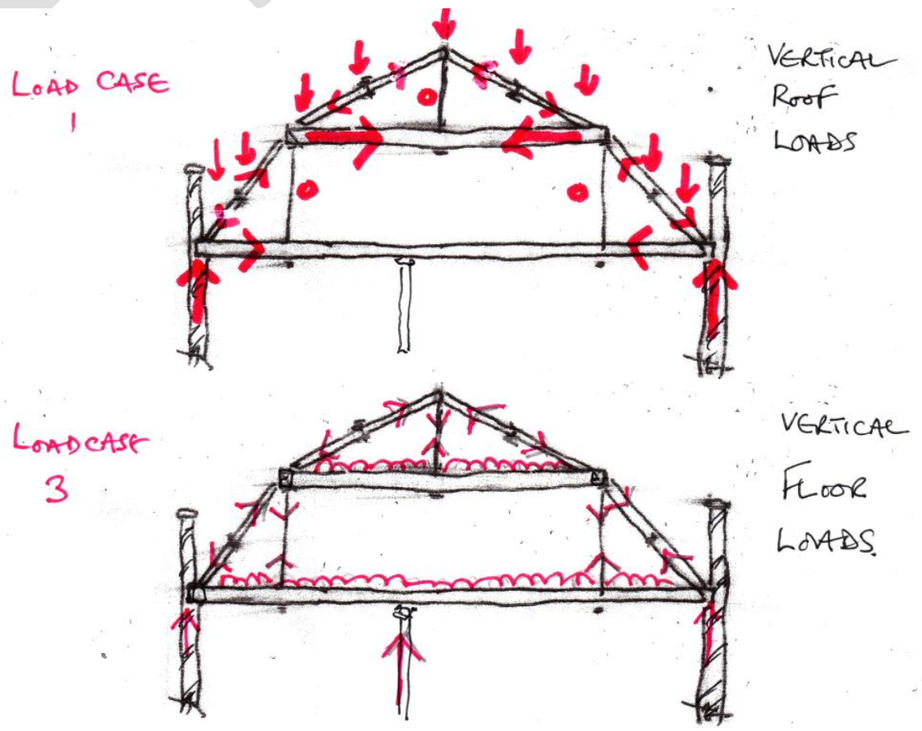


Figure 2.15: Roof truss analysis - vertical loads

However, there are conditions where the rods would go into compression which they cannot resist and therefore they buckle.

These conditions are:

- Where small changes in truss geometry occur at nodes 1,2,4 or 5. These could happen due to shrinkage or rot in the timber sitting in the iron shoes, or local crushing of timber around highly loaded bolts (refer to Fig. 2.16).
- Under some wind conditions involving lateral loads and uplift (refer to Fig. 2.17).

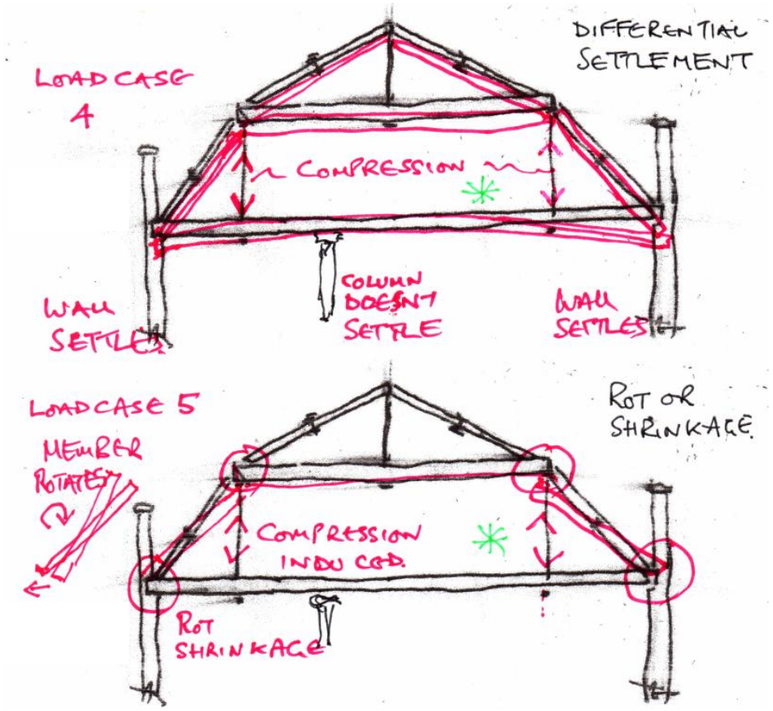


Figure 2.16: Roof truss analysis – Changes in truss geometry

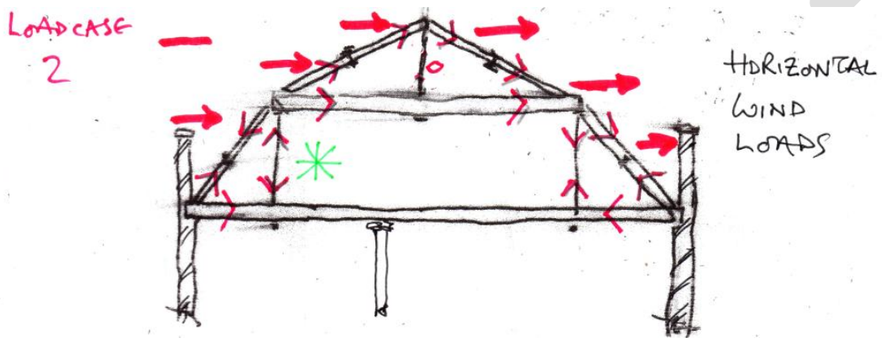


Figure 2.17: Roof truss analysis – Wind load

This is a classic example of a structural form which worked in one material being compromised by the partial substitution of another material. The full implications of the substitutions were not recognised, resulting in a flawed design. (TBC following further access for inspection).

2.2. 4th Floor Structure

The fourth floor structure is formed with 30mm deep softwood floorboards butted together spanning onto softwood timber beams, which are the bottom chord of the roof truss arrangement, at approx. 2.7m c/c. The butt-joints have been under-boarded on the soffit with a timber strip. A varying arrangement of timber joists split the span of the timber floorboards at mid- or third-points between the main timber floor beams, with three varying primary and secondary spacing arrangements in sections along the length of the building (refer to Fig 2.18 and 2.19)



Figure 2.18 and 2.19: Varying primary and secondary timber joist arrangements to underside of 4th floor

This floorboard and joist arrangement differs from the floor construction at the lower floor levels suggesting this arrangement was constructed at a different time to the main floors. It also appears that the joist arrangement has been installed from the underside of the floor at a further later date, potentially in response to excessive deflection of the floorboards following use of the floor above.

Timber floor beams, which are also the bottom chord of the roof truss arrangement mentioned above, span across the width of the building and are supported on load bearing brick piers on the front and rear elevation. Cast iron columns prop the timber beams at circa mid-span with a differing appearance to the columns at lower floors (refer to Fig. 2.20). The head detail of the column to the timber beam/bottom chord is a flat plate offered up to the underside of the beam indicating that the timber member is continuous over the support, with distinct head plate arrangements circular and rectangular plate (refer to Fig. 2.22 and 2.23). The base of the column is an adjustable screwed pin detail allowing the length of the columns to be adjusted (refer to Fig 2.21). These two connection point details suggest these columns are likely to have been installed at a later date, possibly in response to excessive deflection of the timber beams following use of the floor above.

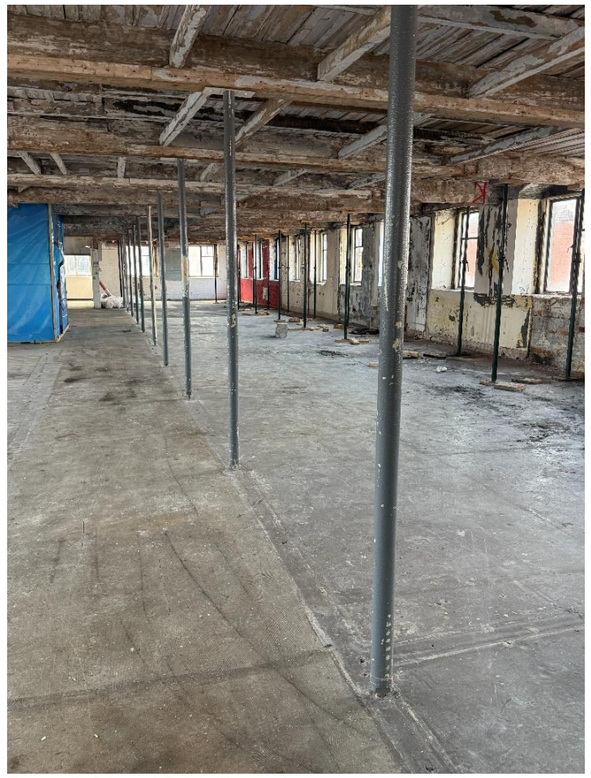




Figure 2.20 and 2.21: Central cast iron 'prop' column to underside of 4th floor level and adjustable screw base connection





Figure 2.22 and 2.23: Central cast iron 'prop' column varied head detail to underside of 4th floor level

Each of these variations within the 'floor' construction at this level in comparison to the lower floor levels suggest these are later interventions, and that the fourth floor potentially wasn't originally constructed as a useable floor level.

As described above, steel tension rods hung from the roof members offer additional support to the timber floor beams (See Fig 2.11, 2.12 and 2.13).

The structural member typical dimensions are:

- Brick Piers: 620mm deep x 1060mm wide piers;
- Timber Beam: 290mm deep x 200mm wide;

2.3. (1st to 3rd) Upper Floors Structure

Generally, timber floor beams span across the width of the building and bear into load bearing brick piers on the front and rear elevation (see Fig. 4.24 and 4.25). Cast iron columns support the timber beams at circa mid span. It is not clear whether the timber beams span continuously over the cast iron bearing at mid-span.





Figure 2.24 and 2.25: General view of 2nd and 1st floor levels

The floor construction is formed with 75mm (3") deep softwood millboards butted together spanning onto softwood timber beams at approx. 2.7m c/c and external masonry walls at the gable ends. It is likely that the floors beams are Scotch Pine, based on visual inspection only. The timber floor construction is typically overlaid with an asphalt screed of circa 15-25mm thick.

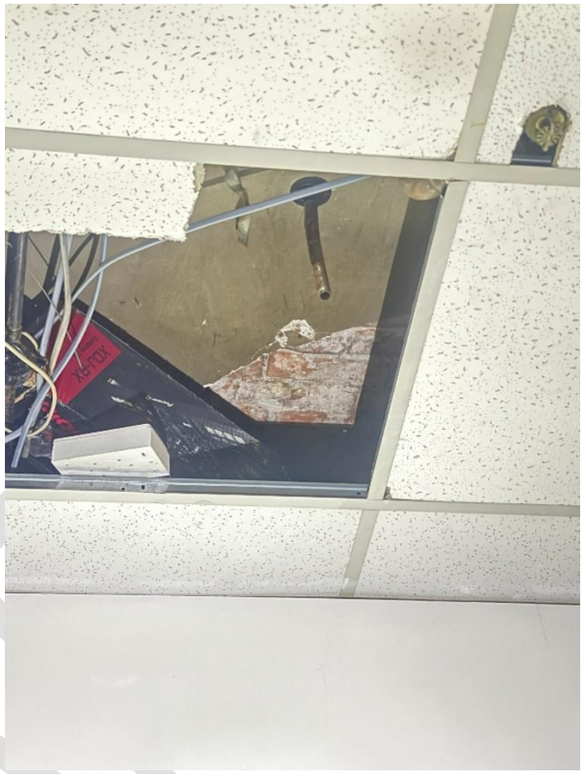
Load bearing brick piers are typically of similar geometry at upper floors of 620mm deep x 1060mm wide.

Timber beam and cast iron columns were measured on site and whilst similar at each level, there are minor differences in geometry between the upper floors in the building:

Floor Level	Timber Beams (mm)	Cast Iron Columns (mm)
3rd Floor	305dp x 240wd or 315 x 230wd	80⌀
2nd Floor	305dp x 225wd or 315 x 230wd	100⌀
1st Floor	300dp x 230wd or 295 x 225wd	115⌀

Typical floor to underside of timber beam is approx. 2.6 m.

At first floor level, there is a variation in the floor construction in that the final three structural bays at the Hilton Street end of the building where the floor structure is formed with brick jack-arches spanning between cast-iron beams, and the gable end wall of the building at the end-most structural bay. Finishes generally remain insitu in this location, however some locations of removed or broken ceiling tiles allow this to be observed (refer to Fig. 2.26 and 2.27).



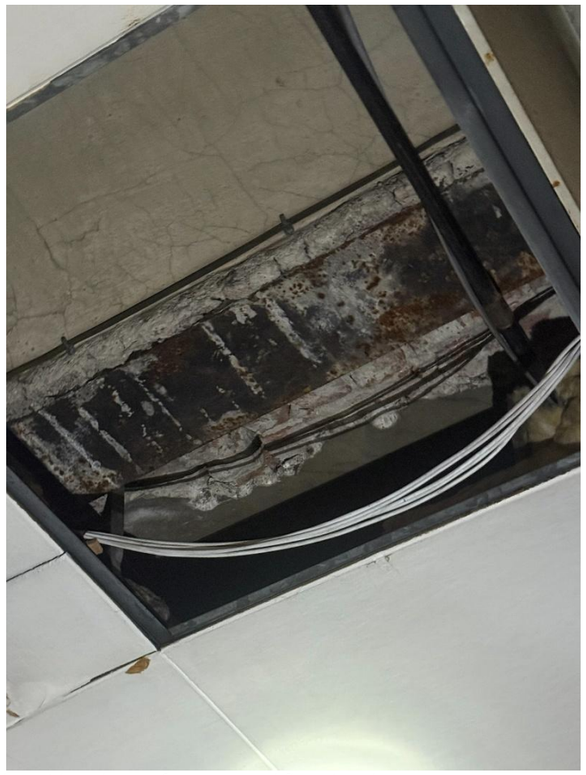


Figure 2.26 and 2.27: View of underside of jack arch floor and cast iron beam from ground floor level

Further to the above, there is a variation in the end support of the timber beam in one location on the Newton Street elevation where there had previously been a masonry arch forming a historic entrance to the building at ground floor level. The arch, now visible internally due to recent strip out works, spans two structural bays leaving the end of the adjacent first floor timber beam without a masonry pier to bear onto for support. The end support of the timber floor beam was achieved by a cast-iron hanger from the floor beam above at second floor level (refer to Fig. 2.28 and 2.29).

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Figure 2.28 and 2.29: Former arch in Newton Street façade and iron hanger supporting adjacent timber floor beam

More recent refurbishment works at the building subsequently removed the opening from this location in the elevation to Newton Street, and therefore a masonry pier was constructed at ground floor level providing a full-bearing to the adjacent timber beam removing the reliance on the cast-iron hanger support to the beam end. The cast-iron hanger remains in situ, albeit the load from the beam is taken down to foundation level via the more recently constructed masonry pier (refer to Fig. 2.28).

2.4. Ground Floor Structure

The ground floor structure is similar to the upper floors with the timber floor beams spanning the width of the building and supported on load bearing brick piers within the front and rear elevations. Cast iron columns support the timber beams at circa mid span. Typical timber beams and cast iron column spacing is 2.68 m.

In the majority of the floor footprint, the floor construction is timber mill boards supported by the timber floor beams and external masonry walls at the gable ends. The millboards are assumed to be a softwood species with a thickness of circa 75 mm. The timber floor construction is typically overlaid with an asphalt screed of circa 15-25mm thick.

There is a variation in the floor construction in that the lower level floor at the current building entrance at the Hilton Street end of the building is formed with concrete spanning onto masonry walls within the basement level, and the gable end wall of the building.

The minimum structural member typical dimensions are:

- Brick Piers: 620mm deep x 1060mm wide;
- Cast Iron column: 125mm diameter;
- Timber Beam: 375mm deep x 255mm wide, or 380mm deep x 255mm wide;

Typical clear headroom from floor to underside of timber beam is approx. 3 m.

As mentioned in the previous section, a former arch in the Newton Street elevation was infilled with a masonry pier (refer to Fig. 2.30) providing additional support to the beam end at first floor over. Support to the beam ends at ground floor level in two adjacent bays also appear to have been altered with the addition of parallel steel PFC sections extending from the end of a short timber floor beam into the external masonry wall (refer to Fig. 2.31). This detail is typical of a replaced decayed timber beam end, however the notching and end detail of the remaining timber beam at this location, as well as 'scars' within the finish of the basement floor level, suggest a masonry wall may have previously been present to support the end of the shorter timber beams. The soffit of the ground floor construction in this location also varies from the typical construction suggesting this has been infilled, potentially replacing a previously lower section of floor to provide a lower stepped access from street level (refer to Fig. 2.32).



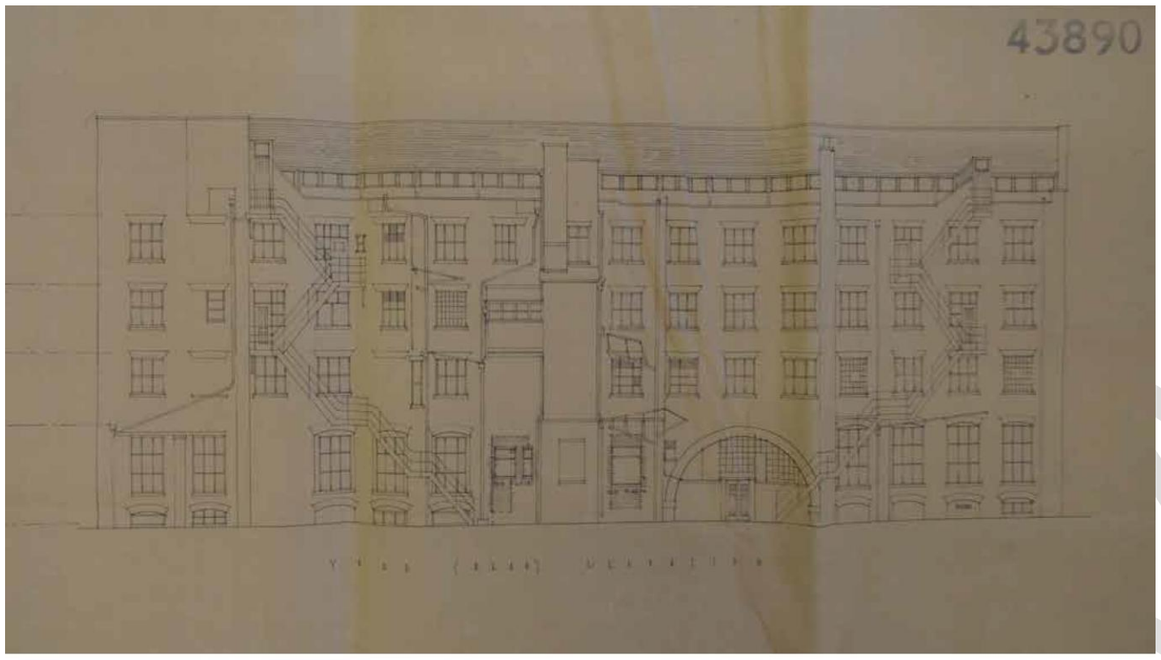
Figure 2.30 and 2.31: More recent masonry pier in arch location and adjacent timber floor beam end adjustments



Figure 2.32: Historic 1898 view of former arched entrance on Newton Street (Donald Insall Associates)

Further historic images and drawings of the building suggest there were further masonry arches at each end of the front Newton Street elevation. It is understood that the more recent refurbishment works to the Hilton Street end of the building removed the former arch, however recent internal strip out works suggest the arch at the Faraday Street end of the elevation may remain insitu behind the more recent render.

A historic elevation drawing of the rear elevation also suggests there was a former large arch spanning three structural bays into the rear courtyard area at ground floor level (refer to Fig. 2.33) , however this was previously removed and replaced with masonry piers as described on the 'Proposed' historic drawings of the refurbishment works dated 1958 (refer to Fig. 2.34).



2.24 *Existing rear elevation, July 1958, Manchester Archives*

Figure 2.33: Extract from Donald Insall Associates report of historic rear elevation drawing showing former arch



2.25 *Proposed rear elevation, July 1958, Manchester Archives*

Figure 2.34: Extract from Dondald Insall Associates report of historic 'proposed' works dated 1957 replacing former arch

This location can be seen within the building where flat lintels span over windows, where this would typically be a masonry arch lintel over an original window (refer to Fig. 2.35 and 2.36).



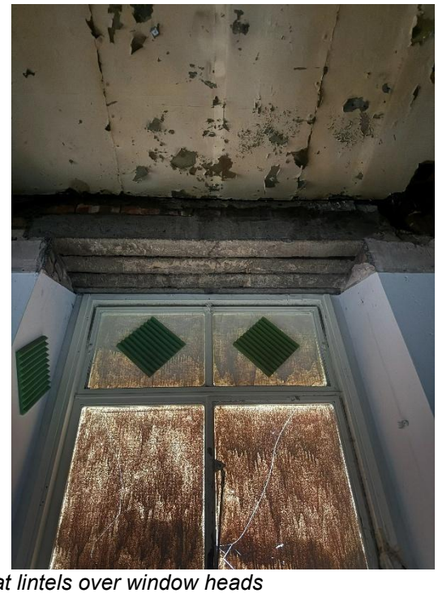


Figure 2.35 and 2.36: Location of former arch in rear façade replaced with flat lintels over window heads

Internally, a masonry enclosure houses the lift shaft at the Faraday Street and Newton Street corner of the building (refer to Figure 2.27) where internally the floor level is at the lower street level and accessible from a door opening on the

Faraday Street corner. The extent of the masonry enclosure coincides with the second structural bay column location and it is unclear whether the timber floor beams are bearing onto the masonry wall or whether there is an enclosed cast iron column. The timber beam appears to continue along the head of the wall as it returns towards the Newton Street elevation a the second structural bay line.



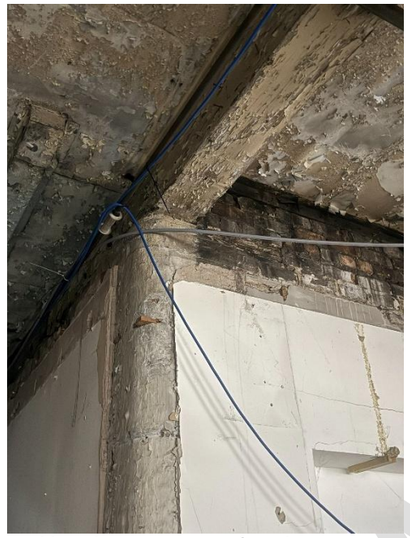


Figure 2.37 and 2.38: General view of lift shaft enclosure and timber beam end bearing on corner of enclosure

2.5. Basement Structure

The walls to the perimeter of the basement are earth-retaining, being partially below ground level, and are of solid brickwork construction (refer to Fig. 2.39).



Figure 2.39: General view of basement level with masonry enclosure to Faraday St and Newton St corner in background

The basement floor is of solid construction, assumed to be a simple unreinforced concrete ground bearing slab.

Brick piers and cast iron columns were measured at the basement level and typical structural member dimensions are:

- Brick Piers: 760mm deep x 1060mm wide piers;
- Cast Iron column: 152mm diameter;

Typical clear headroom from floor to underside of timber beam is approx. 3 m.

The masonry enclosure to the Faraday Street and Newton Street corner of the building continues through basement level where the floor level internally is as external street level.

2.6. Foundations + Substructures

At the time of writing this report, an intrusive ground investigation has not been carried out to establish either the ground conditions, or the exact form and size of the existing foundations.

Other site investigations in the area have recorded made ground over layers of drift deposits comprising clays ranging from firm to very stiff, followed by stiff to very stiff glacial clays. Below these clays is Sherwood Sandstone (also known as “Bunter Sandstone”).

Based on the anticipated ground conditions, it is likely that the building has spread foundations at the stiff clay depth. These would consist of corbelled brick, and, possibly, mass concrete.

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2.7. Building Stability System

A series of embedded cast-iron beams are evident within the gable walls at each floor, braced at the internal corners with an iron quadrant plate bracket (refer to Fig. 2.32). In one location to the underside of third floor (visible from second floor level), at the Hilton Street and Newton Street corner, there is a flat plate horizontal brace connected into the bracket and projecting back to the central column location at the second structural bay from the gable wall (refer to Fig. 2.33, 2.34 and 2.35), at which point it returns back towards the adjacent corner of the building but appears to have been cut off (refer to Fig. 2.36 and 2.37).

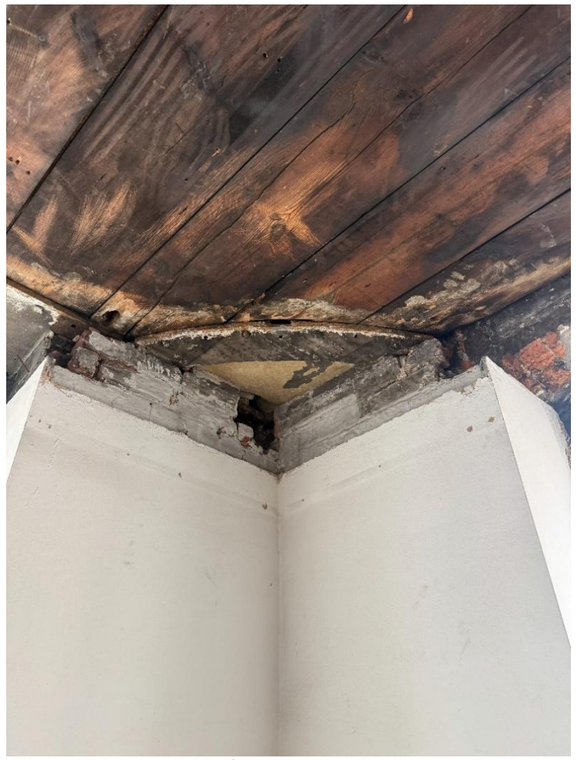




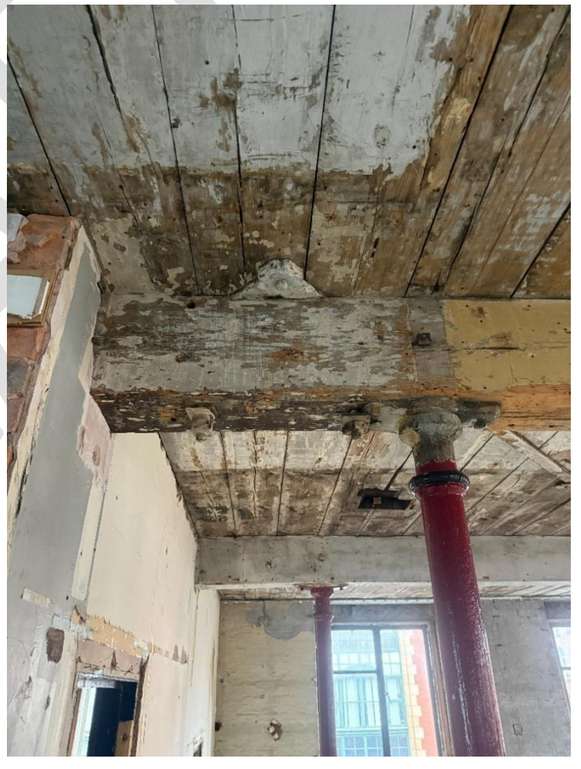
Figure 2.32 and 2.33: Cast iron quadrant bracket to gable wall corners without and with diagonal flat horizontal brace

In all other locations at all corners and floor levels, this brace appears to have been removed leaving a slot in the corner bracket where the braced was previously fixed. These are referred to in the Historic England listings as ring-beams to which the timber cross-beams and heavy floorboards are fixed, and are thought to be an early structural arrangement to achieve a diaphragm at each floor level.





Figure 2.34 and 2.35: Diagonal flat horizontal brace projecting to and from central column location



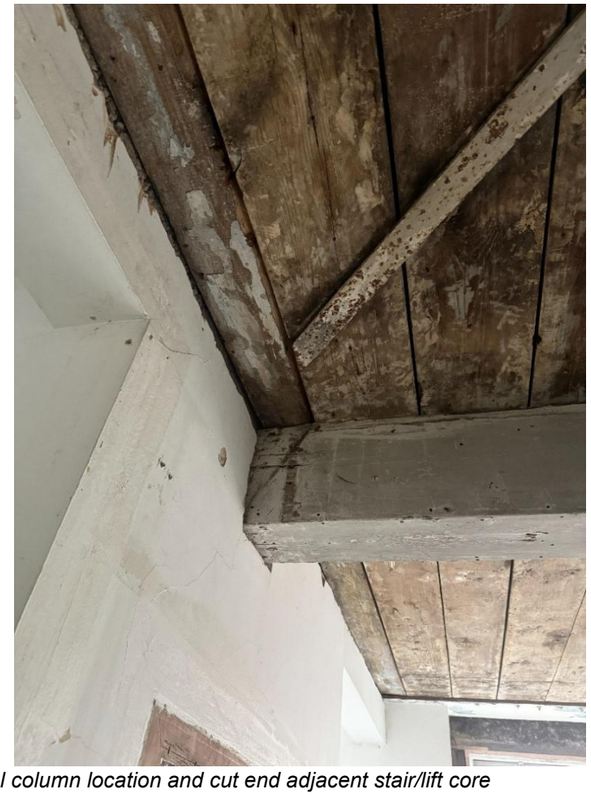


Figure 2.36 and 2.37: Diagonal flat horizontal brace at central column location and cut end adjacent stair/lift core

3. Structural Condition

The inspections carried out by Civic on 29th September 2025 and 17th October 2025 were limited to a general visual inspection of the property. The existing upper floor construction (timber beams and millboards) and external wall construction were generally visible due to previous strip out works removing most finishes. During the site visits, access was limited at attic level due to the presence of asbestos and therefore finishes remaining in place. The initial condition of the building structure on site has been determined through visual inspection only, no material samples have been taken for testing, and no internal in-situ tests have been conducted to date, however these surveys and tests

3.1. Roof Structure

The condition of the roof structure that Civic observed during the site visits on the 29th September and 17th October is poor.

Both the iron and the timber elements of the roof structure (refer to section 2.1 of this report) appear to be in poor condition as the result of exposure to damp conditions with little or no ongoing maintenance and repair. The condition of the roof covering is unclear, but likely to need renewal given the condition of the attic structure and visible extent of continued water ingress.

Rainwater goods have failed in several locations or are not present which has resulted in staining and local areas of vegetation growth.

An obvious defect and path for moisture ingress is created by the cast iron T-sections running between the 4th floor and the attic level (refer to section 2.1 of this report). These elements, in the majority of the roof area, are partially external of the envelope of the building and where they pass through the roof line, introduce a weak point where water ingress is more probable (refer to Fig. 3.1 and 3.2).



Figure 3.1 and 3.2: Cast iron T-section rafter passing through roof covering – internal and external view (2017)

This water ingress is most evident when viewed from the underside of the fourth floor where timber mill boards and beam ends are visibly decaying and deteriorating (refer to Fig. 3.3 and 3.4).



Figure 3.3 and 3.4: Water ingress and subsequent deterioration at underside of 4th floor level

A detailed survey has been undertaken on site by Hutton + Rostron during the writing of this report, and the extent of damp within the masonry walls and timber condition is described in more detail in Hutton + Rostron drawings in Appendix B and C respectively.

As described in section 2.1, Civic believe the roof structure is a flawed arrangement resulting in structural elements being exposed to structural loads for which they are not capable of resisting. This is manifesting in the wrought iron tension rods visibly buckling and deflecting horizontally (refer to Fig. 3.5 and 3.6). Wind loads applied to the unrestrained horizontal cast iron window members would also result in inward movement into the tension rods compounding the horizontal deflection.





Figure 3.5 and 3.6: Deflected iron tension rod and full section crack through iron clerestory window frame (2017)

3.2. External Masonry Walls

It is likely the proposed redevelopment of the building will prefer to see the removal of the 1960's render from the external elevations to reinstate the former building's appearance as a brick-faced mill for which Manchester is known.

Whilst no recent intrusive works have been undertaken yet, Civic Engineers have previously overseen some localised opening-up works on site on 30th of June 2017 to assess the render composition and its adhesion to the brickwork (refer to Fig. 3.7 for opening up works key plan).

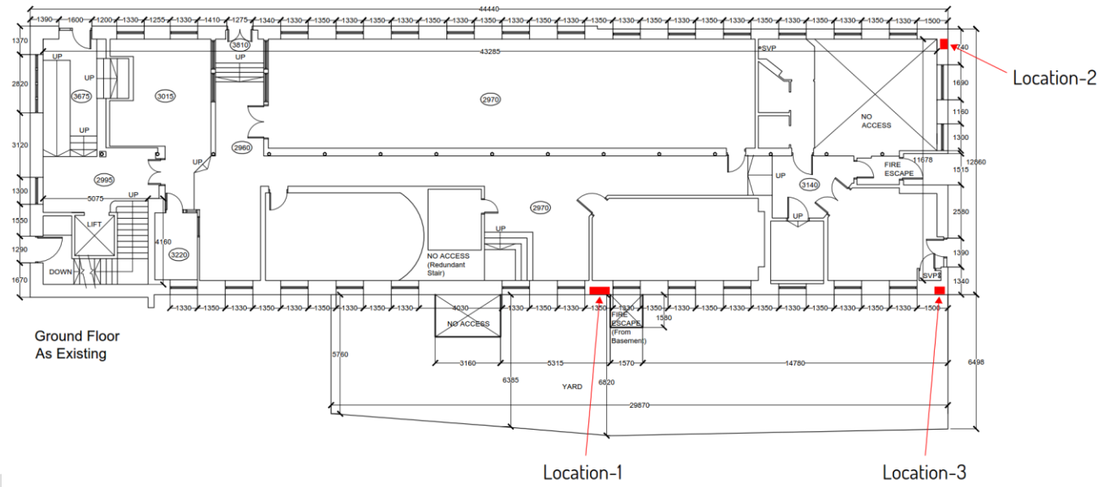


Figure 3.7: Render opening-up works – Key Plan

The works consisted of local breaking off of the render with hand tools at street level to the rear and Faraday Street elevations.

The following two different render compositions have been observed:

1. Rear elevation (location 1 in Fig. 3.7): Base coat of cementitious render approx. 10mm thick, and a top coat of approx. 4mm. There is no mineralite on the rear elevation (refer to Fig. 3.8). This could be chiselled off relatively easily, to reveal brickwork behind (refer to Fig. 3.9);



Figure 3.8 : Location 1 – Render Sample

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Figure 3.9: Location 1 – Brickwork and mortar joint condition following render removal

2. Faraday Street elevation (locations 2 and 3 in Fig.3.7): Similar two coats of render as the rear elevation (cementitious render 10mm, and top coat 4mm) with an additional approx. 8mm thick finishing coat of 'mineralite' (refer to Fig. 3.10, 3.11 and 3.13, 3.14). Removal of the render on this street elevation revealed the horizontal mortar joints had been raked out to provide a stronger bond onto the brickwork (unlike the rear elevation where the render cracks and peels away quite readily). From the relatively small areas explored, the render could be chiselled off leaving brickwork that is intact but 'pecked' on its surface. In the areas exposed, the mortar joints appeared to be in poor condition/badly eroded (refer to Fig. 3.12 and 3.15).



Figure 3.10: Location 2 – Render Sample

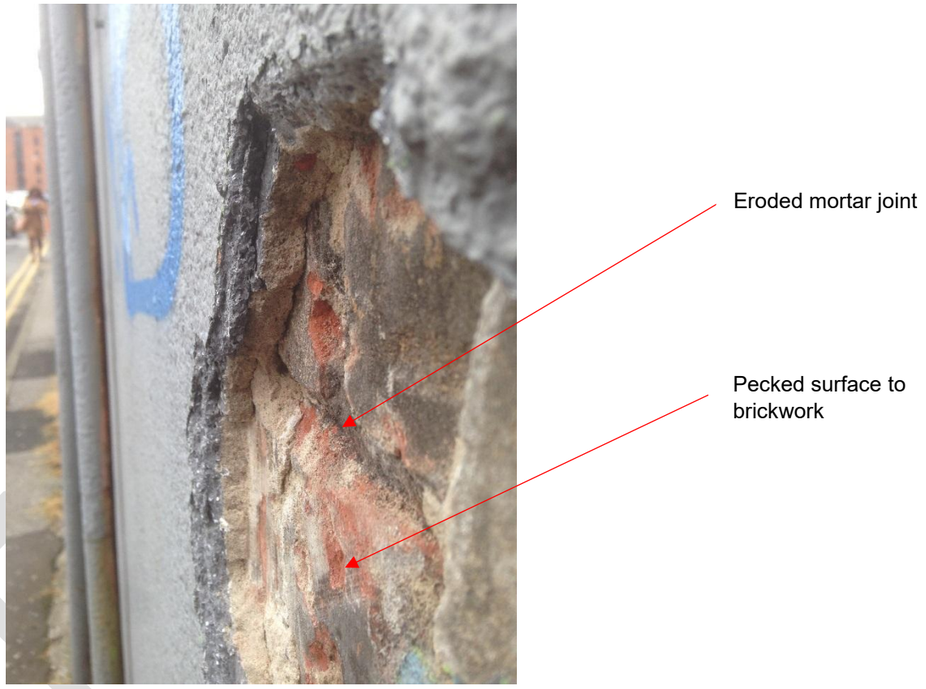


Figure 3.11: Location 2 – Render layers

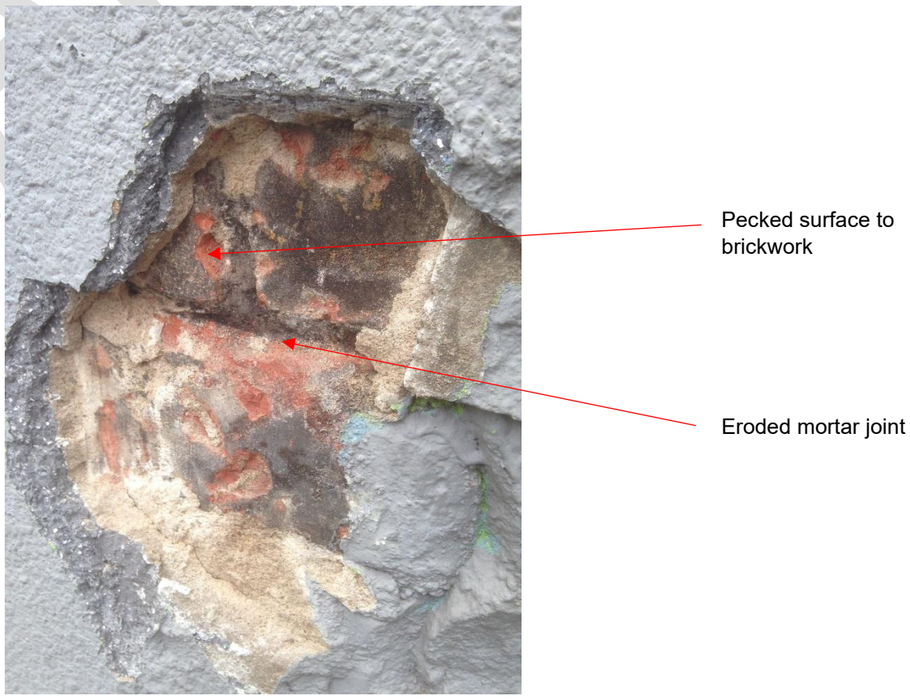


Figure 3.12: Location 2 – Brickwork and mortar joint condition following render removal

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Figure 3.13: Location 3 – Render Sample



Figure 3.14: Location 3 – Render layers

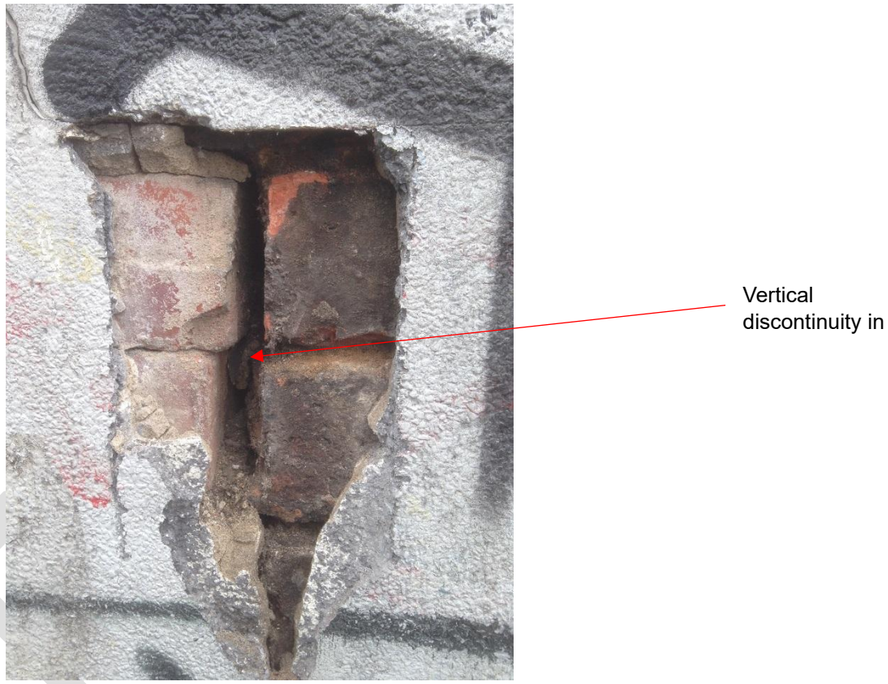


Figure 3.15: Location 3 - Brickwork and mortar joint condition following render removal

3.3. Internal Framing – (1st to 3rd) Upper Floors

The internal framing is generally visible throughout the floor plate at 1st to 3rd floor levels due to recent strip out works of the building finishes and partitions (Fig. 2.24 and 2.25 in above section).

The existing members are generally in reasonable condition given their age and material type, however there are localised areas of deterioration and decay primarily due to water ingress at the perimeter of the floor plates adjacent the masonry facades (refer to Fig. 3.16 and 3.17). The continued water ingress is more evident to the upper levels of the building, becoming less evident through the lower suspended floors (refer to section 3.4 for observations relating to ground floor level specifically).

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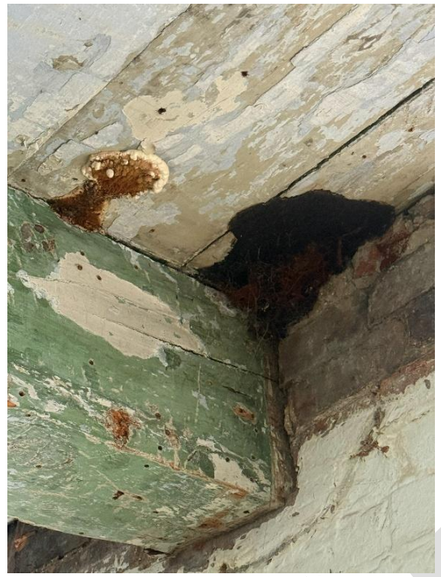


Figure 3.16 and 3.17: Evident continued water ingress (second floor) and fungal growth at beam end support (1st floor)

As a result, the timber beam ends are propped with acrow props in many locations at each level along both elevations (refer to Fig. 3.18 and 3.19), and there appears to have been previous structural intervention to the underside of third floor level with the installation of steel PFC sections in four locations to strengthen the existing timber beams. These interventions take two forms: PFCs fixed through the sides of the timber floor beams and spanning between central column and masonry pier (refer to Fig. 3.20); or a PFC offered up to the underside of the timber beam and propped with a small diameter iron post (refer to Fig. 3.21).



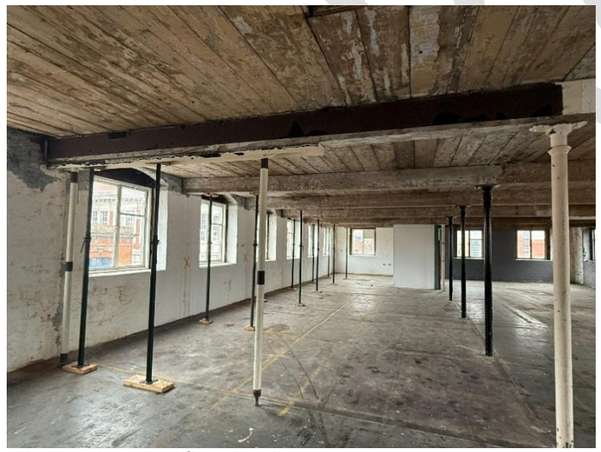


Figure 3.18 and 3.19: Acrow props to rotten timber beam ends at third and second floor levels

A detailed survey has been undertaken on site by Hutton + Rostron during the writing of this report, and the extent of damp within the masonry walls and timber condition is described in more detail in Hutton + Rostron drawings in Appendix B and C respectively.





Figure 3.20 and 3.21: Two forms of previous structural intervention at underside of third floor level

Further to the above, there are areas of the floor plates showing deflection of the timber floor beams in the mid-span between the perimeter wall and the central line of cast iron columns, which was measured to be 30-40mm at second floor level. This deflection is potentially symptomatic of historic over-stress due to floor loading exceeding the relatively low bending capacity of the timber floor beams.

The internal brick face to the external walls is mostly concealed behind paintwork, however there are areas at first floor level where sandblasting has been undertaken to reveal the brick finish (refer to Fig. 3.22). Whilst the walls are considered to be generally structurally sound, the majority of the brickwork is in reasonably poor condition with areas with missing bricks, poor pointing and localised cracking or damaged bricks (refer to Fig. 3.23). There is evidence of previous remedial works having been undertaken in areas such as window cills where different brick type and age is observed (refer to Fig. 3.23).





Figure 3.22 and 3.23: Area of sandblasted brickwork at first floor and example location of poor condition brickwork

3.4. Ground Floor

Recent strip out works have removed the majority of the partitions across the floor plate, however paint finishes remain insitu to the soffit of the timber floor and walls.

Water ingress is evident within the floor plate, likely migrating down the building from previously discussed weak points at roof and upper floor levels (refer to Fig. 3.25), with numerous areas of standing water on the surface of the ground floor (refer to Fig. 3.24).





Figure 3.24 and 3.25: Example of standing water at ground floor level and evidence of water ingress from upper levels

Whilst the level of the ground floor is typically raised above external ground level, the proximity of the floor to the external surface, apparent lack of damp proofing within the wall and encapsulation of the masonry walls with cementitious render externally and paint internally (refer to Fig. 3.26), has resulted in the deterioration and decay of the timber beam ends due to damp issues within the walls (refer to Fig. 3.27).

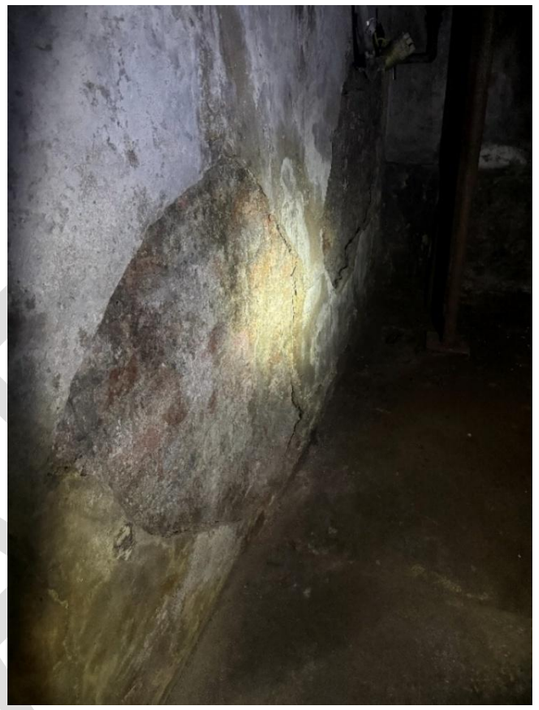




Figure 3.26 and 3.27: Internal paint/render trapping moisture within the external walls and timber beam end decay

The extent of damp within the masonry walls and timber condition is described in more detail in Hutton + Rostron drawings in Appendix B and C respectively.

3.5. Basement Level

As with the upper floors, the recent strip out works have removed the partitions across the floor plate, however paint finishes remain insitu to the soffit of the timber ground floor over and external masonry walls.

Water ingress is evident with areas of standing water on the surface of the basement floor, and spalling of the paint finishes to the external walls, particularly where these walls are below the adjacent external ground level. Where paint has been removed or deteriorated and fallen off, the brickwork behind appears to be in relatively poor condition with areas of spalling and mortar deterioration.

The application of a paint finish directly to the surface of the brickwork creates a barrier to the brickwork trapping moisture within preventing evaporation. This trapped moisture will continue to damage the brickwork and mortar until the paint finish is removed (refer to Fig. 3.26).

The extent of damp within the masonry walls and timber condition is described in more detail in Hutton + Rostron drawings in Appendix B and C respectively.

4. Further Building Assessment

Inspections by Civic to date were undertaken from street level externally and floor levels internally, with access to the fourth floor and attic space restricted due to the presence of hazardous materials, and remaining partitions and finishes.

To facilitate the completion of Civic's appraisal of the building, access is required within the attic space to allow a clear view of the geometry, arrangement and condition of the roof structure to provide a better understanding of whether the trusses are original, or have been altered, and whether a valid load path through the structural members of the trusses can be proven.

No detailed inspection of masonry parapet walls, coping stones, lift over runs and the extended masonry gable end walls has been completed to date, and it is likely defects will be present. A full survey will be undertaken once appropriate safe access can be provided.

Whilst some preliminary removal of the external render finishes has been completed previously, and is discussed in section 3.2 of this report, the locations and extent of these investigations were limited and further sample areas of render removal would be recommended to provide a better understanding of the ease of removal, and the quality of the brickwork and mortar joints once removed. This sample area should include areas of the two known compositions of render mentioned in this report, and expose features such as window surrounds, including cills and lintels.

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Appendix A

Existing Structural Plans

Appendix B

Hutton + Rostron – Masonry Damp Drawings

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Appendix C

Hutton + Rostron – Timber Condition Drawings

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