

1/20445



Stage 1 / 2
Structural
Inspection 32-34
Laystall Street

32-34 Laystall Street,
Manchester,

M1 2JZ

23 October 2020

1/20445/REP/001

Rev P1

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1.0 INTRODUCTION

Clancy Consulting has been appointed by McCauls Ltd to provide Civil and Structural Engineering services in relation to the redevelopment of their properties at 32 to 34 Laystall Street, Manchester.

The proposed redevelopment comprises three distinct elements; the careful renovation and refurbishment of the existing Grade II Listed Building fronting Laystall Street; the extension of this existing Grade II Listed Building to the southwest of the site; and the demolition of the existing factory and workshops to the southeast of the site, between Brock Street and Pigeon Street.

This report addresses first the existing condition of the structure of the retained Grade II Listed Building, taking a top-down approach to the observations described. This report identifies the condition and stability of the main structural elements, i.e. the roof, walls, floors and foundations and aims to identify any existing significant structural defects observed during the site visit, identifies likely causes, makes recommendations for appropriate further investigations and finally suggests an outline strategy to mitigate further degradation and structurally strengthen where necessary.

This report shall also address the proposed restoration proposals to be undertaken to the building as part of the wider redevelopment of the site. Utilising observations and measurements taken on site of the size and form of the existing structural layout and construction materials, the report shall determine the suitability of the building for the proposed future use. It shall also identify likely future alterations necessary to facilitate the new extension to the south west elevation.

As the building is of significant heritage value, understood to be Grade II listed, any intrusions into the existing fabric of the building or material alterations will be identified by the Architect as part of the Planning submission to allow the heritage impact to be reviewed.

At the time of our inspection, the property was unoccupied. However, there was evidence of recent unauthorised access having been gained into the building, along with guano deposits throughout due to infestation of wildlife through open doors and windows.

Andrew Eccles and Joe Partridge of Clancy Consulting visited the site on Thursday the 1st October 2020 to carry out a visual structural survey of the existing building. The survey was undertaken from floor level at basement, ground, first and second floors 1 to 3. We did not access the roof and limited inspection of the exterior of the building was undertaken from street level, aided by the use of binoculars. Limited opening up work was carried out internally to expose the existing structure. No trial holes were excavated to expose the existing foundations and bearing strata. These general limitations should be considered when reading the findings of our report

This report is prepared specifically for McCauls Ltd; it is not for transmission in whole or in part to any other third party and we bear no obligation to any such third party without our express permission in writing. Such permission will not be unreasonably withheld.

2.0 EXISTING STRUCTURAL ARRANGEMENT

The existing structure comprises three upper storeys with a basement and dates from approximately 1879. It is primarily load bearing masonry with timber joist floors and a timber flat roof construction, which we understand replaced the original pitched roof, which was damaged by fire in 1927. The main entrance to the building is the northwest elevation off Laystall Street, which is ornately decorated with an array of terracotta embellishments.

A central line of cast iron columns runs parallel to Laystall street. Four columns run the full height of the building, with an additional fifth column being present in the basement which supports and terminates at the ground floor over.

The cast iron columns support structural members spanning the shorter distance from the front to the back of the building and running between the column heads longitudinally. These members break the span of the timber joists and provide restraint to the columns at each floor level. The members have been set at oblique angles to the supporting external wall in some locations, to avoid loading over external window openings.

Inspections undertaken on these members through the limited opening up works undertaken on site to date have in one location identified a potential brittle fracture to the top flange of a member at second floor level. This would suggest that these members are most likely to be of wrought iron construction, which would correspond with construction techniques common of the time the property was built.



Photo 1 – Possible Brittle Fracture to Top Flange of Second Floor Beam

Throughout the building, where lintels were visible, these were found to be of timber construction. This is again typical of the age of the building.

Although the floors are typically of timber construction, a loading bay exists at ground floor in

the centre of the building, accessed via Laystall Street. In this area of the building the floor was found to comprise a brick arch construction with tie rods to accommodate the higher vehicle loading on this floor area.

3.0 EXTERNAL OBSERVATIONS

3.1 There is of self-seeded vegetation up the building, and particularly at roof level, which is rooted into the external fabric of the building.



Photo 2 – Vegetative Growth Up External Building Face

3.2 Displaced and missing masonry and stonework were visible at higher levels where vegetation was most likely once present.



Photo 3 – Missing Terracotta Parapet Moulding

3.3 Throughout the external façade, cracking to terracotta features was observed, principally at first floor cill and ground floor level. This is less evident or conspicuous at second floor level, although closer inspection may reveal this to be present



Photos 4 to 7 – Cracking to Terracotta Castings

3.4 This was less evident at second floor or roof level, although closer inspection may reveal it.

3.5 Throughout the external façade, redundant ferrous fixings were observed, some of which had corroded and caused fracturing of the external masonry or terracotta finishes. Whilst fixings further up the building were less evident from street level, it is likely some exist.



Photo 8 – Vertical Masonry Cracking Due to Corrosion of Ferrous Fixing

3.6 At lower levels of the building extensive spalling of the brickwork, as a result of freeze thaw action, was observed.



Photo 9 – Lower Level Freeze / Thaw Spawling of Brickwork

3.7 At first floor level, external pattress plates were visible in the northeast corner of the building, facing Laystall Street. Internal inspections revealed a large cast metal tank stand within the corner of the building, which the pattress plates had clearly been installed to fix back to the masonry corner of the building. Assuming removal of the tank stand, these plates would effectively be redundant.



Photo 10 – External Spreader Plates on Northeast Building Corner

3.8 In several locations the existing terracotta blocks were found to be damaged or missing at cornice level, most likely as a result of vehicle strikes.



Photo 11 – Vehicle Strike Damage to Terracotta on Northeast Elevation

3.9 Noticeably, different brickwork was identified below an existing window on the northeast elevation at second floor level and visual evidence of rope wear to the head of the window suggest that this once formed a larger loading bay, which has since been infilled to form the current window opening.



Photo 12 – Historic Loading Bay at Second Floor – Northeast Elevation

3.10 Almost the entire southwestern building elevation was over-clad in a profiled metal sheet or over-rendered at the lower levels. It was therefore not possible to fully observe the original fabric of the building in this location.



Photo 13 – Profiled Metal Cladding to Southwest Elevation

3.11 From visual observations around the building perimeter there was found to be no evidence of significant structural movement of the brickwork or supporting foundations.

4.1.3 Timber roof joists are 225mm deep x 45mm wide at approximately 400mm centres. Timber boarding was visible on top of the joists, but it was not possible to accurately measure the thickness of this from the second-floor level.



Photo 14 – Existing Splice Connection Detail to Roof

4.1.4 At the northeast corner of the building, a section of the timber joists had been subject to a splice repair in the past. Existing joist ends had been removed, most likely due to wet rot of the timber in the wall, and new timber members have been fixed. These new timbers have been built into the wall with a DPM wrap to the ends to prevent further damage and new brickwork installed around tight up to the new timber.

4.1.5 Observations of the connections between the original timber joists and the new spliced additions identified significant displacement of the bolts, which appear inadequate in number. There is a general lack of steel spreader plates (although some are present) which has allowed the bolts to locally crush the timber resulting in the joists slipping out of alignment with the other joists on the same building line. We would suggest that the roof in this area should be replaced as part of the wider works and to facilitate the proposed new roof coverings.

The structural floor plan illustrates the layout of a building, highlighting a collapsed ceiling area. Key features include:

- Structural Elements:**
 - Columns:** Cast iron columns are indicated by red dashed lines and labels. Wrought iron beams are shown as solid red lines.
 - Joists:** 275x75 joists are labeled in blue.
 - Beams:** Wrought iron beams are labeled in red.
- Rooms and Areas:**
 - Shower:** Located in the bottom left corner.
 - Collapsed Ceiling:** A red wavy line and the label "COLLAPSED CEILING" indicate the damaged area in the center.
 - Staircases:** Multiple staircases are shown with "UP" and "DN" arrows.
 - Other Rooms:** Various rooms are outlined, including a large central hall and several smaller rooms.
- Annotations and Dimensions:**
 - Dimensions like 52.28, 52.23, 54.45, 52.24, 54.45, 52.22, 54.64, 54.6, 54.23, 54.25, 54.21, 54.19, 54.18, 54.17, 54.16, 54.15, 54.14, 54.13, 54.12, 54.11, 54.10, 54.09, 54.08, 54.07, 54.06, 54.05, 54.04, 54.03, 54.02, 54.01, 54.00, 53.99, 53.98, 53.97, 53.96, 53.95, 53.94, 53.93, 53.92, 53.91, 53.90, 53.89, 53.88, 53.87, 53.86, 53.85, 53.84, 53.83, 53.82, 53.81, 53.80, 53.79, 53.78, 53.77, 53.76, 53.75, 53.74, 53.73, 53.72, 53.71, 53.70, 53.69, 53.68, 53.67, 53.66, 53.65, 53.64, 53.63, 53.62, 53.61, 53.60, 53.59, 53.58, 53.57, 53.56, 53.55, 53.54, 53.53, 53.52, 53.51, 53.50, 53.49, 53.48, 53.47, 53.46, 53.45, 53.44, 53.43, 53.42, 53.41, 53.40, 53.39, 53.38, 53.37, 53.36, 53.35, 53.34, 53.33, 53.32, 53.31, 53.30, 53.29, 53.28, 53.27, 53.26, 53.25, 53.24, 53.23, 53.22, 53.21, 53.20, 53.19, 53.18, 53.17, 53.16, 53.15, 53.14, 53.13, 53.12, 53.11, 53.10, 53.09, 53.08, 53.07, 53.06, 53.05, 53.04, 53.03, 53.02, 53.01, 53.00, 52.99, 52.98, 52.97, 52.96, 52.95, 52.94, 52.93, 52.92, 52.91, 52.90, 52.89, 52.88, 52.87, 52.86, 52.85, 52.84, 52.83, 52.82, 52.81, 52.80, 52.79, 52.78, 52.77, 52.76, 52.75, 52.74, 52.73, 52.72, 52.71, 52.70, 52.69, 52.68, 52.67, 52.66, 52.65, 52.64, 52.63, 52.62, 52.61, 52.60, 52.59, 52.58, 52.57, 52.56, 52.55, 52.54, 52.53, 52.52, 52.51, 52.50, 52.49, 52.48, 52.47, 52.46, 52.45, 52.44, 52.43, 52.42, 52.41, 52.40, 52.39, 52.38, 52.37, 52.36, 52.35, 52.34, 52.33, 52.32, 52.31, 52.30, 52.29, 52.28, 52.27, 52.26, 52.25, 52.24, 52.23, 52.22, 52.21, 52.20, 52.19, 52.18, 52.17, 52.16, 52.15, 52.14, 52.13, 52.12, 52.11, 52.10, 52.09, 52.08, 52.07, 52.06, 52.05, 52.04, 52.03, 52.02, 52.01, 52.00, 51.99, 51.98, 51.97, 51.96, 51.95, 51.94, 51.93, 51.92, 51.91, 51.90, 51.89, 51.88, 51.87, 51.86, 51.85, 51.84, 51.83, 51.82, 51.81, 51.80, 51.79, 51.78, 51.77, 51.76, 51.75, 51.74, 51.73, 51.72, 51.71, 51.70, 51.69, 51.68, 51.67, 51.66, 51.65, 51.64, 51.63, 51.62, 51.61, 51.60, 51.59, 51.58, 51.57, 51.56, 51.55, 51.54, 51.53, 51.52, 51.51, 51.50, 51.49, 51.48, 51.47, 51.46, 51.45, 51.44, 51.43, 51.42, 51.41, 51.40, 51.39, 51.38, 51.37, 51.36, 51.35, 51.34, 51.33, 51.32, 51.31, 51.30, 51.29, 51.28, 51.27, 51.26, 51.25, 51.24, 51.23, 51.22, 51.21, 51.20, 51.19, 51.18, 51.17, 51.16, 51.15, 51.14, 51.13, 51.12, 51.11, 51.10, 51.09, 51.08, 51.07, 51.06, 51.05, 51.04, 51.03, 51.02, 51.01, 51.00, 50.99, 50.98, 50.97, 50.96, 50.95, 50.94, 50.93, 50.92, 50.91, 50.90, 50.89, 50.88, 50.87, 50.86, 50.85, 50.84, 50.83, 50.82, 50.81, 50.80, 50.79, 50.78, 50.77, 50.76, 50.75, 50.74, 50.73, 50.72, 50.71, 50.70, 50.69, 50.68, 50.67, 50.66, 50.65, 50.64, 50.63, 50.62, 50.61, 50.60, 50.59, 50.58, 50.57, 50.56, 50.55, 50.54, 50.53, 50.52, 50.51, 50.50, 50.49, 50.48, 50.47, 50.46, 50.45, 50.44, 50.43, 50.42, 50.41, 50.40, 50.39, 50.38, 50.37, 50.36, 50.35, 50.34, 50.33, 50.32, 50.31, 50.30, 50.29, 50.28, 50.27, 50.26, 50.25, 50.24, 50.23, 50.22, 50.21, 50.20, 50.19, 50.18, 50.17, 50.16, 50.15, 50.14, 50.13, 50.12, 50.11, 50.10, 50.09, 50.08, 50.07, 50.06, 50.05, 50.04, 50.03, 50.02, 50.01, 50.00, 49.99, 49.98, 49.97, 49.96, 49.95, 49.94, 49.93, 49.92, 49.91, 49.90, 49.89, 49.88, 49.87, 49.86, 49.85, 49.84, 49.83, 49.82, 49.81, 49.80, 49.79, 49.78, 49.77, 49.76, 49.75, 49.74, 49.73, 49.72, 49.71, 49.70, 49.69, 49.68, 49.67, 49.66, 49.65, 49.64, 49.63, 49.62, 49.61, 49.60, 49.59, 49.58, 49.57, 49.56, 49.55, 49.54, 49.53, 49.52, 49.51, 49.50, 49.49, 49.48, 49.47, 49.46, 49.45, 49.44, 49.43, 49.42, 49.41, 49.40, 49.39, 49.38, 49.37, 49.36, 49.35, 49.34, 49.33, 49.32, 49.31, 49.30, 49.29, 49.28, 49.27, 49.26, 49.25, 49.24, 49.23, 49.22, 49.21, 49.20, 49.19, 49.18, 49.17, 49.16, 49.15, 49.14, 49.13, 49.12, 49.11, 49.10, 49.09, 49.08, 49.07, 49.06, 49.05, 49.04, 49.03, 49.02, 49.01, 49.00, 48.99, 48.98, 48.97, 48.96, 48.95, 48.94, 48.93, 48.92, 48

4.2.2 It was not possible to accurately measure the existing wrought iron members running down the centre of the building due to the presence of a possible asbestos containing material and guano fouling. From the second-floor vantage point it was possible to identify that the members were rivetted, with these rivets present in a staggered line down the centre of the member.

4.2.3 One lintel was exposed by removal of a fascia board and found to have been subject to fire damaged, with charring present on its exposed internal faces. This may have affected the structural integrity of this lintel however the charring affect will have afforded some protection to the loss of significant sectional area.



Photo 15 – Existing Fire-Damaged Timber Beam to Northeast Elevation

4.2.4 There is vertical cracking beneath window openings, which generally follows the line of the mortar joints.



Photo 16 – Vertical Cracking Below Window Opening to Northwest Elevation

4.2.5 Throughout the building there is evidence of damp having penetrating through from the roof, with damage to plasterwork and paintwork throughout. Some efforts have been made to arrest this water ingress and plastic sheeting was evidence at the second-floor level which appears to be conducive with this.



Photo 17 – Evidence of Water Ingress at Second Floor Level

4.2.6 Water ingress may pose a risk to the structural integrity of the timber members built within the external envelope of the building, notably the timber lintels and roof and floor joists throughout.

4.2.7 A small area of opening-up was undertaken close to an existing column on the second-floor area, exposing the second-floor structure for inspection. Timber tongue and groove floorboards, approximately 40mm deep were recorded bearing on 275mm deep x 75mm wide timber joists at 400mm centres running the longitudinal length of the building.

4.2.8 Second floor timber joists span southwest to northeast and were found to be supported on intermediate wrought iron members running from the central line of cast iron columns to the northwest and southeast external building elevations. Some of these members were observed from the floor below as being installed at oblique angles to the external walls in order not to bear over window openings.



Photo 18 – Oblique Wrought Iron Member Bearing on Cast Iron Column at Second Floor

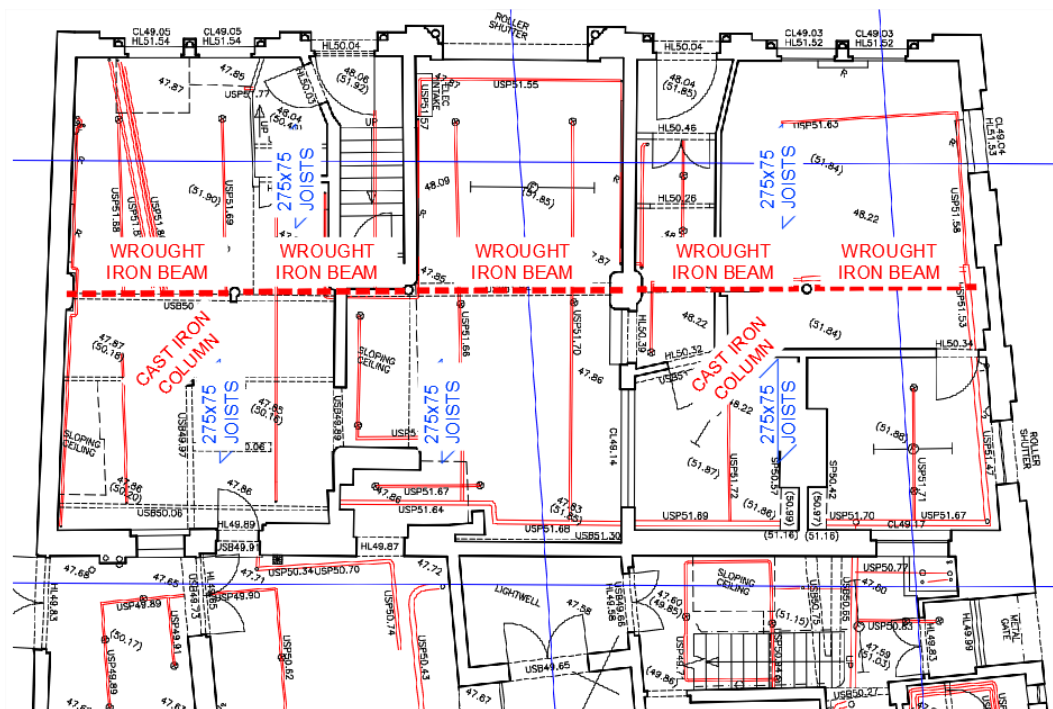
4.2.9 An exposed wrought iron member was measured and found to be approximately 250mm deep by 130mm wide. These wrought iron members were supported on brackets which appeared to be cast as one with the columns, however it was not possible to observe any bolts etc due to the limited extent of opening up works.



Photo 19 – Cast Support Brackets at Column Head Supporting Roof Members

4.2.10 Although not observed due to existing plasterwork finishes, it is assumed the second-floor timber joists are built into the external walls, in keeping with the construction methods of the time.

4.3 FIRST FLOOR



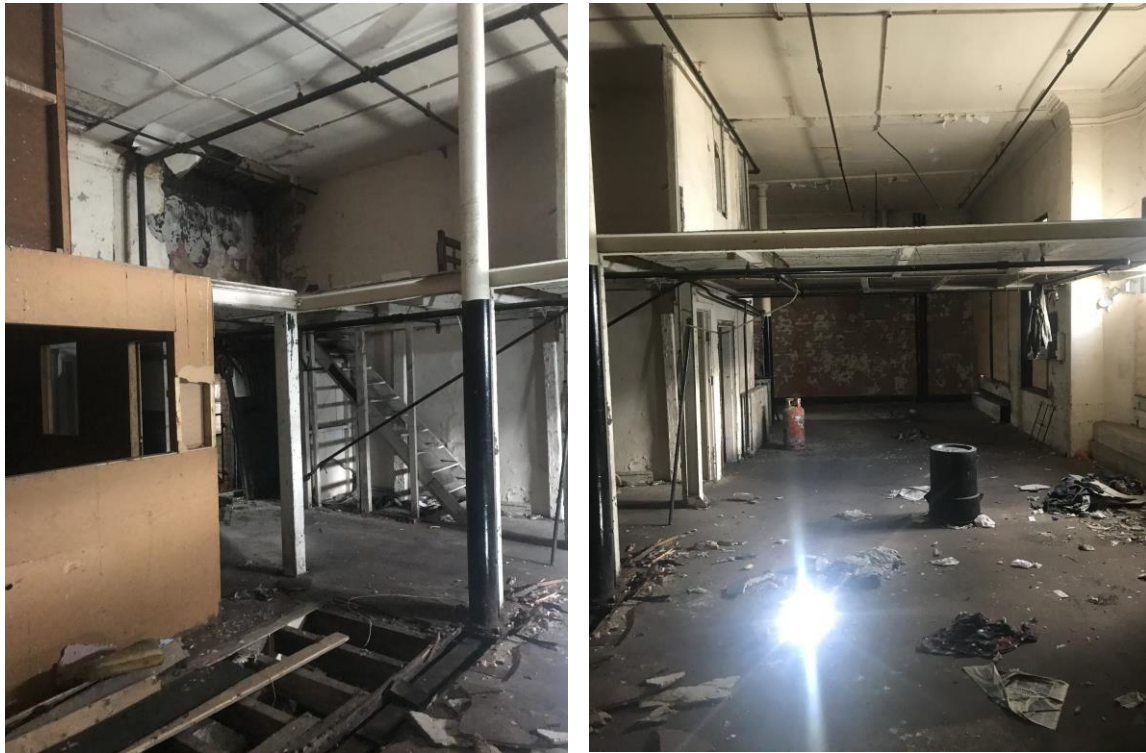
4.3.1 Access to the first floor was via an internal timber staircase which runs from the original front door to the right of the loading bay up and to the rear of the building. This is in reasonable condition, although we understand the proposal is for all existing internal staircases to be removed and access to this building to be made via a new lift and staircase within the proposed extension to the southwest building elevation.

4.3.2 First floor timber joists were found to measure 275mm deep by 75mm wide, installed at 440mm centres and spanning front to back (northeast to southwest). These timber joists supported tongue and groove timber floorboards, which were measured as 50mm thick. These floor joists are assumed to bear into the external walls and be supported on the central line of wrought iron beams, which run through the centre of the building.



Photo 20 – Timber Infill to First Floor Stair Void and Original Trimmer Beam

4.3.3 A partial mezzanine floor with restricted head height exists at first floor level. This is formed of timber construction and is accessed via a timber staircase within the main room at this level, which is otherwise largely open plan. Our understanding is that the mezzanine floor will be removed as part of the refurbishment works, in order to provide a larger open space for the new offices.



Photos 21 & 22 – Partial Mezzanine Floor at First Floor Level

4.3.4 Damp penetration through from the roof level is evident at the top of the mezzanine floor staircase. A large area of plaster and paintwork is damaged, and the ceiling collapsed.



Photo 23 – Collapsed Ceiling Above First Floor to Mezzanine

4.3.5 There is evidence of a previously existing ground to first floor staircase whose

opening at first floor level has been infilled with timber joists and partially boarded over.

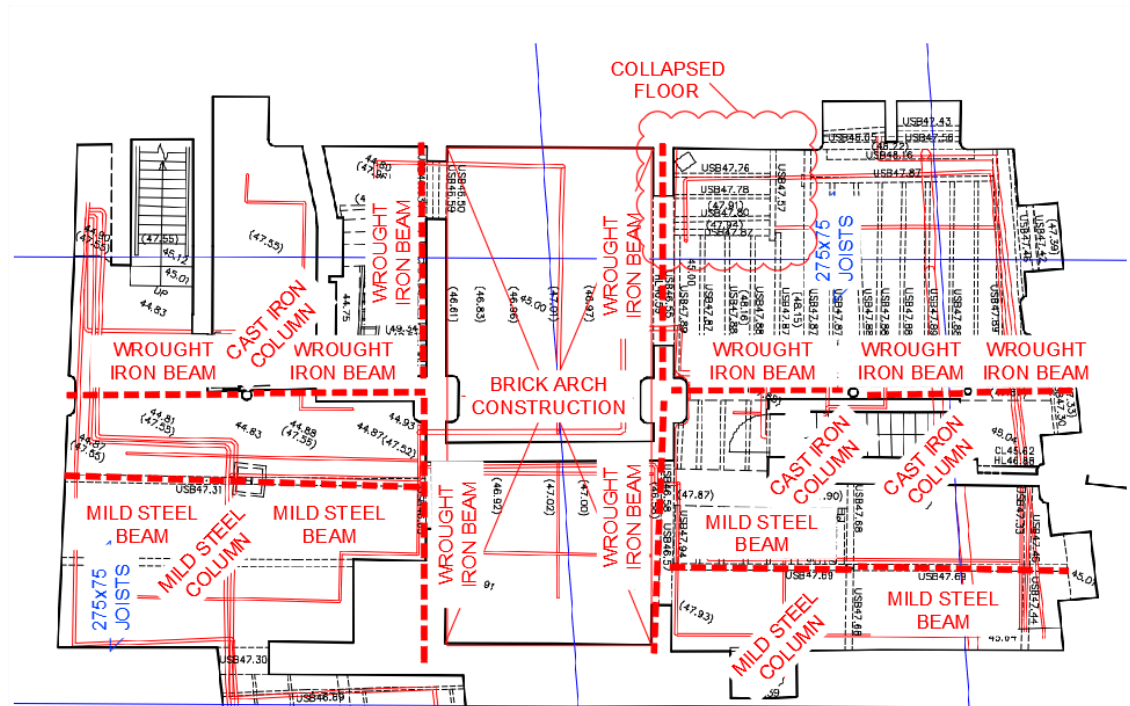
4.3.7 The tank stand, the spreader plates of which were visible from outside, was visible in the northeast corner of the building at this level.



Photo 24 – Former Tank Stand in Northeast Building Corner

4.3.8 A small existing window opening was observed on the southwest elevation in a shower room. This window is not visible externally due to the presence of the profiled metal over-cladding mentioned earlier in the report.

4.4 GROUND



4.4.1 Access to the ground floor level was via the rear entrance from the workshop area to the rear of the property. There were also two doors at the front of the property accessed via Laystall Street and a roller shutter door accessing the loading in the central section of the building.

4.4.2 At ground floor the building is essentially separated down the middle by a full height loading bay, which has a floor of concrete construction over brick arches, visible in the basement. The concrete floor is set at a lower level than the general ground floor construction and nominally falls towards the rear of the building.

4.4.3 Away from the loading bay, the ground floor construction is generally timber joists which were found to be 275mm deep by 75mm wide at approximately 440mm centres. Timber tongue and groove floor boarding, approximately 50mm deep was observed at the entrance to the basement, which is in the northwest corner of the building.

4.4.4 A partial mezzanine floor exists in the southwestern portion of building. This has been formed of timber with a timber staircase in southwest corner of building.



Photos 25 & 26 – Ground Floor Partial Mezzanine with Visible Riveted Beams

4.4.5 Above the front wall of the room at mezzanine floor level the bottom flange of a rivetted wrought iron structural member is visible. This member runs the longitudinal length of the building and supports the first-floor timber joists over.

4.4.6 Remnants of timber staircase from ground to first floor still evident in the northeast portion of the building, however these have been cut back previously and the first-floor opening infilled with timber.

4.4.7 To the northeast portion of the building there is evidence of structural damage through the ingress of water. Timber joists supporting a cast concrete slab and tiled entranceway at the front door has rotted at the bearings and resulted in an area of collapsed flooring at main entrance to left of loading bay.



Photo 27 – Ground Floor Partial Collapse at Laystall Street Entrance Door

4.5 BASEMENT

4.5.1 Access to the basement was made via a set of timber stairs in the northwest corner of the building. It is understood that this access point will be infilled as part of the main works and a new access created at the southwest end of the basement.

4.5.2 An informal floor covering of sand and rubble exists throughout the basement.

4.5.3 The brick arch construction of the loading bay area in the central part of the building was fully visible with tie rods restraining the supporting wrought iron members above the bottom flanges where the brick arches bear. This structure appears in relatively sound condition, however there was evidence that corrosion of the tie ends had resulted in fracturing of the surrounding brickwork.

4.5.4 Further rivetted wrought iron sections were visible from the basement supporting the original ground floor timber members. An additional cast iron column was identified to the northeast portion of building, bringing the total number of columns at basement to ground floor level to five. Above ground floor level, only four columns continue up to roof level.

4.5.5 Evidence of previous structural strengthening works to the ground floor were evident in the basement with an additional line of mild steel columns and beams to rear half of the basement. This new steelwork has effectively reduced the span of the existing timber members and will have assisted in its load-bearing capacity and helped reduce deflections. There was no evidence to suggest the joists in this area had been subject to any significant structural failure, so this is likely to be a proactive measure to improve the floors structural

characteristics.



Photos 28 & 29 – Ground Floor Strengthening Works In Basement

4.5.6 Corrosion to these newer mild steel sections was visibly and appeared more severe than the corrosion affecting the older wrought iron members. This adds further to the presumption that the original structural metal members are indeed wrought iron.

4.5.7 From the basement level the collapsed floor at the main entrance to the left of the loading bay can be observed fully. The existing timber ends have rotten and the bearings failed allowing the supported floor over to collapse into the basement.

4.5.8 Corrugated sheet metal has been applied to the underside of the original timber joists, perhaps as some form of fire protection, or perhaps just to formalise the ceiling. This metalwork has however rusted and is hanging loose in a number of locations and should be removed and replaced with a suitable fire protective layer, as necessary.



Photo 30 – Partial Collapse of Corrugated Sheet Metal in Basement

4.5.9 There is an existing masonry and stone staircase in northeast portion of building which has since been boarded off at ground floor level. This appear to be in reasonable structural condition.

5.0 DISCUSSION

As described in the previous text, the external façade has suffered from a general lack of maintenance and self-seeding vegetation is present. Along with its ability to retain moisture, which can then freeze and expand, is likely to have caused physical damage to the external envelope of the building, which will need to be addressed as part of the works.

In overall terms, the damage to the external terracotta embellishments, although not strictly of structural significance, is likely to be significant in terms of programme duration and reparation costs. Some of this damage is likely to have occurred as a result of moisture ingress causing corrosion and expansion of ferrous fixings, such as locator dowels, within the body of the terracotta castings. We understand the desire would be for these terracotta items to be repaired or replaced as part of the works. Any items which are too badly damaged to be repaired would likely need to be recast due to their bespoke nature and input should be sought from a specialist in this respect.

We would suggest the removal of any redundant ferrous fixings is undertaken during the restoration work to prevent further damage from the external façade from occurring.

Although the current condition of the masonry does not cause structural concern, the delamination of the brickwork over time will inhibit the structural capacity of the wall, as well as being unsightly. We would suggest that any delaminated brickwork at low level is removed and replace with similarly face masonry as part of the wider refurbishment works.

Local repointing of the existing mortar would also be required throughout and this would act to conceal local differences in brickwork finishes slightly, such as at the second floor loading bay, however replacement of the header bricks above the window in this location may be necessary following closer inspection of their condition.

The structural integrity of the existing timber lintels may have been compromised by the historic damage caused by the previous building fire and ingress of water into the building. Any loss of sectional capacity would no doubt have been mitigated by the arching affect the brickwork would have over the opening, preventing any form of local or global failure of walls. We would recommend a thorough review of the existing structural members as part of the refurbishment works, with structural defects addressed on a case-by-case basis as and when they are uncovered.

Ingress of water into the building is also found to have affected ceilings and floors in a number of locations and, although not strictly a structural issue, the floor boarding and joists should be checked in these locations and any damaged members should be removed and replaced.

Vertical cracking through the masonry mortar joints to the front building elevation are likely a result of contraction of the brickwork at the larger masonry panels, which has manifested itself as movement cracks at the location where the minimal brickwork exists. This is not of any structural significance and local repairs should be undertaken as part of the wider refurbishment works.

Given the age of the building, it would be doubtful if any form of restraint strapping were present in the floor, meaning compliance with disproportionate collapse tie requirements may not currently be achieved. This aspect of the existing structure should be further investigated as part of the refurbishment works and the additional of restraint ties within the floor zone considered further.

Closing up of the existing staircase openings once removed should be undertaken as part of the refurbishment works and in order to make safe the existing floor boarding throughout.

Existing failed timber joists at the main entrance supporting the original tiling will need to be carefully removed and replaced with new structural timber members, with appropriate protection against moisture, particularly where built into the external walls.

Existing steel or wrought ironwork in the basement should be brushed clean of corrosion and treated with a suitable rust preventive treatment to ensure further degradation does not occur.

We understand that the current proposal is to retain the existing masonry staircase in the basement as a feature, albeit not to re-utilise it as a stair and some local mortar repair may be required in order maintain it's present condition.

The existing infill brickwork between wrought iron beam flanges in the basement should be locally repaired and replaced, however this should not be a significant structural undertaking.

6.0 LOADING INFORMATION

The existing building has generally been used as offices and a show room for the terracotta business which previously occupied the site. The proposed refurbishment will see the building retained for use as an office, however due to current standards for fire and acoustics it is anticipated that the applied permanent load will increase overall.

Prior to 1909 there was no statutory basis for the calculation of design loads and so our analysis aims to identify what could be reasonably justifiable in terms of allowable applied loads, utilising modern variable load allowances for office buildings.

BS EN 1991-1 currently requires a design variable load of 3.0kN/m² (unfactored) for office areas at ground floor level or below and 2.5kN/m² (unfactored) for office areas above ground floor level. Factored loading requirements are for 4.5kN/m² and 3.75kN/m² for ground and upper floors respectively.

EXISTING LOADING CONDITION

Summary of Existing Roof Component Weights

Existing ≤50mm T&G Floor Boards	=	0.30 kN/m ²
Joists Typical 225x75 @ 400 crs	=	0.25 kN/m ²
Lath and Plaster Ceiling	=	0.25 kN/m ²
Services	=	0.10 kN/m ²
<u>Total Existing Permanent Loading</u>	=	<u>0.90 kN/m²</u>

Summary of Variable Load

Snow & Access	=	0.75 kN/m ²
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Note:- Wind & Snow assumed not to be dominant when considering global loading conditions, however local assessment will be required as part of detailed design process.

Summary of Existing First and Second Floor Component Weights

Asphalt Floor covering 12mm thick	=	0.29 kN/m ²
Existing ≤50mm T&G Floor Boards	=	0.30 kN/m ²
Joists Typical 275x75 @ ≥400 crs	=	0.25 kN/m ²
Lath and Plaster Ceiling	=	0.25 kN/m ²
Services	=	0.10 kN/m ²
<u>Total Existing Permanent Loading</u>	=	<u>1.19 kN/m²</u>

Summary of Variable Load

Partitions	=	1.50 kN/m ²
Offices (Category B1)	=	2.50 kN/m ²

Summary of Existing Ground Floor Component Weights

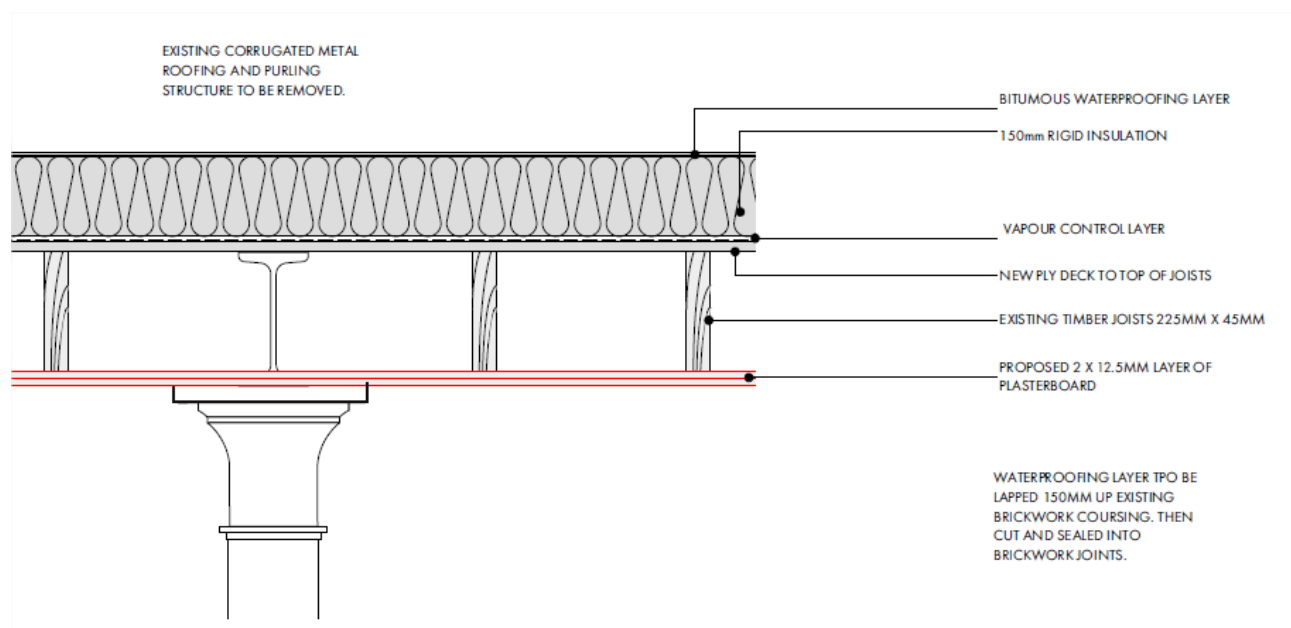
Asphalt Floor covering 12mm thick	=	0.29 kN/m ²
Existing 50mm T&G Floorboards	=	0.30 kN/m ²
Joists Typical 275x75 @ 440 crs	=	0.25 kN/m ²
Corrugated metal underlining	=	0.05 kN/m ²
Services	=	0.10 kN/m ²
Total Existing Permanent Loading	=	<u>0.99 kN/m²</u>

Summary of Variable Load

Partitions	=	1.50 kN/m ²
Offices (Category B2)	=	3.00 kN/m ²

PROPOSED LOADING CONDITION

Summary of Proposed Component Weights – Roof



Bituminous Waterproof Layer	=	0.45 kN/m ²
Rigid Insulation & Vapour Control Layer	=	0.10 kN/m ²
New Plywood Decking	=	0.15 kN/m ²
Timber Joists Typical 225x75 @ 400 crs	=	0.25 kN/m ²
Ceiling	=	0.15 kN/m ²
Services	=	0.10 kN/m ²
Total Proposed Permanent Loading	=	<u>1.20 kN/m²</u>

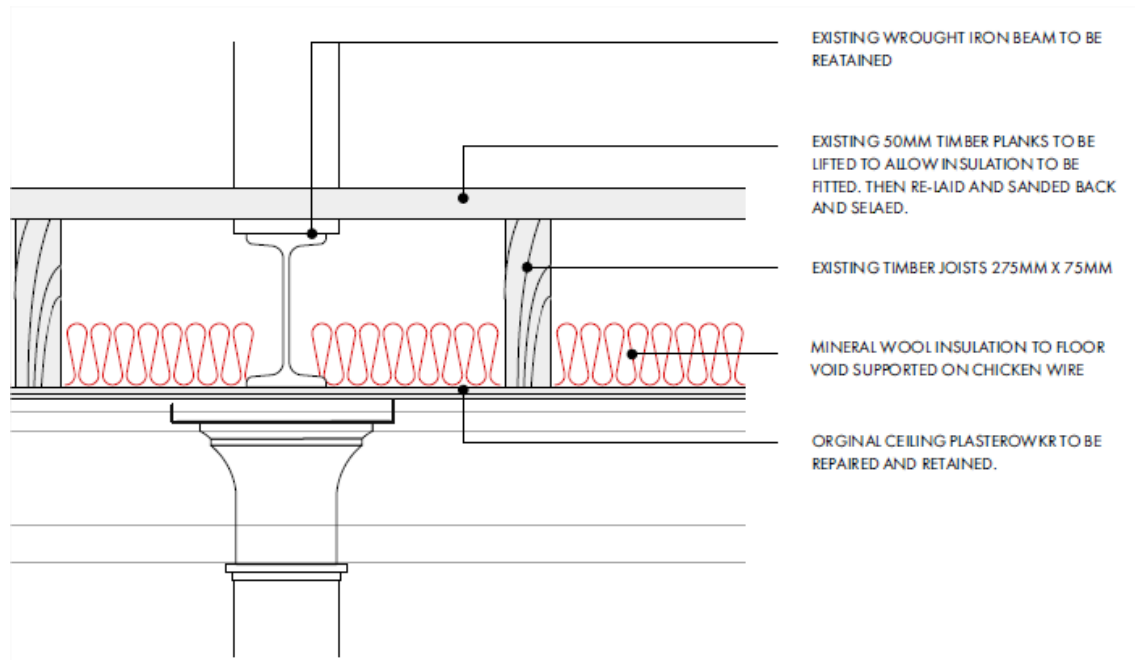
Summary of Proposed Variable Load

Snow & Access	=	0.75 kN/m ²
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Note:- Wind & Snow assumed not to be dominant when considering global loading

conditions, however local assessment will be required as part of detailed design process.

Summary of Proposed First and Second Floor Component Weights - Untiled Areas

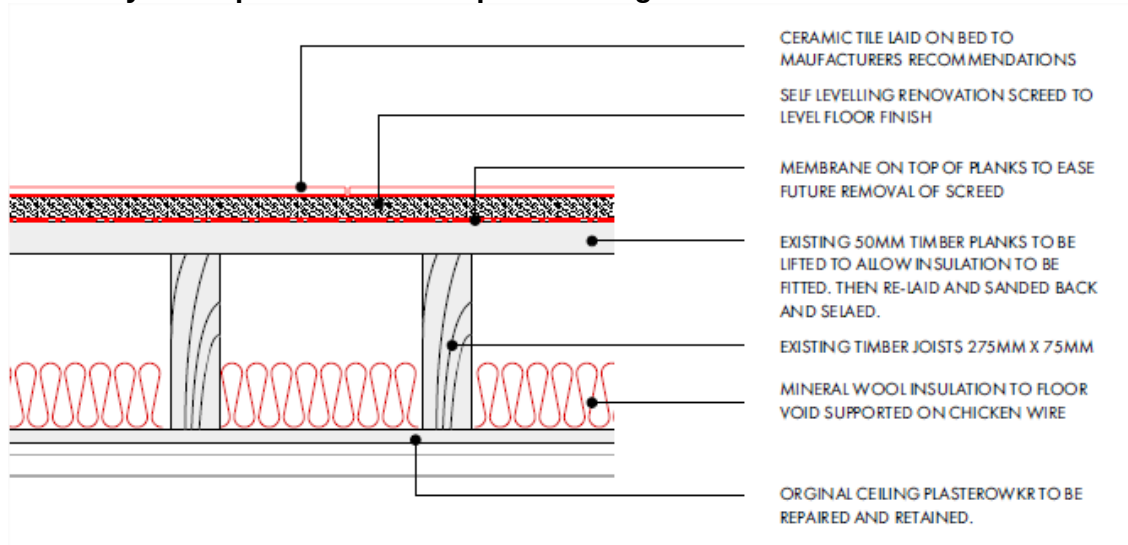


Existing $\leq 50\text{mm}$ T&G Floor Boards	=	0.30 kN/m ²
Joists Typical 275x75 @ ≥ 400 crs	=	0.25 kN/m ²
Insulation	=	0.10 kN/m ²
Lath and Plaster Ceiling	=	0.25 kN/m ²
Services	=	0.10 kN/m ²
Total Proposed Permanent Loading	=	<u>1.00 kN/m²</u>

Summary of Variable Load

Partitions	=	1.00 kN/m ²
Offices (Category B1)	=	2.50 kN/m ²

Summary of Proposed Floor Component Weights - Tiled Areas

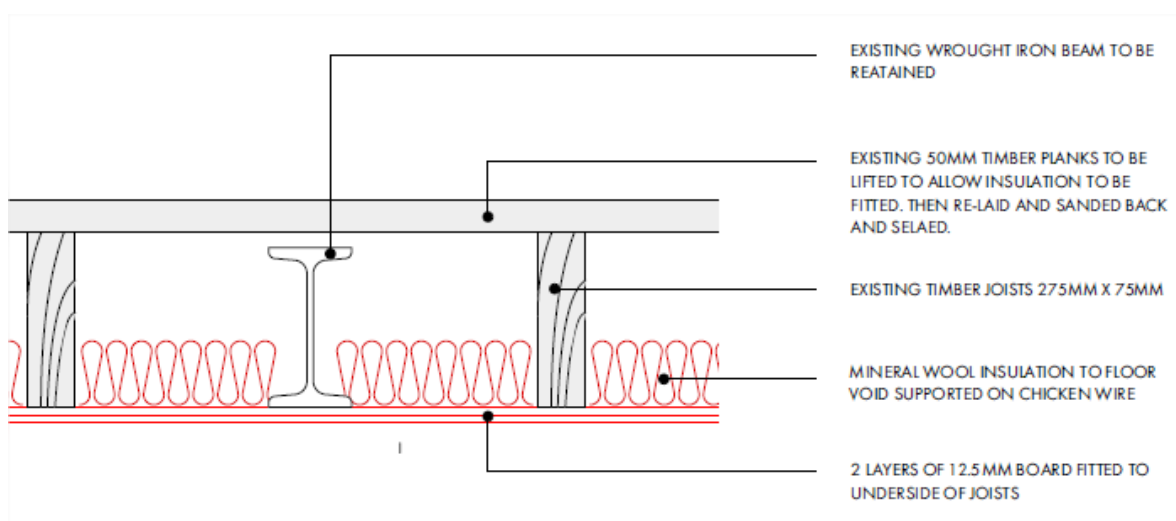


Ceramic Tiling and Bedding	=	0.32 KN/m ²
Self-levelling Screed c. 40mm	=	0.88 KN/m ²
Existing ≤50mm T&G Floorboards	=	0.30 KN/m ²
Joists Typical 275x75 @ ≥400 crs	=	0.25 KN/m ²
Insulation	=	0.10 KN/m ²
Lath and Plaster Ceiling (worst-case)	=	0.25 KN/m ²
Services	=	0.10 KN/m ²
Total Proposed Permanent Loading	=	<u>2.20 kN/m²</u>

Summary of Variable Load

Corridors (C33)	=	4.00 KN/m ²
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Summary of Proposed Ground Floor Component Weights



Existing ≤50mm T&G Floor Boards	=	0.30 KN/m ²
Joists Typical 275x75 @ ≥400 crs	=	0.25 KN/m ²

Insulation	=	0.10 kN/m ²
Double-Boarded Ceiling	=	0.30 kN/m ²
Services	=	0.10 kN/m ²
<u>Total Proposed Permanent Loading</u>	=	<u>1.05 kN/m²</u>

Summary of Variable Load

Partitions	=	1.00 kN/m ²
Offices (Category B2)	=	3.00 kN/m ²

Summary

Roof

Given the above calculations, it is anticipated that there will be an increase in the applied permanent roof loading as a result of the new bituminous waterproof layer, insulation and plywood boarding.

Calculated Existing Total Applied Permanent Loading	=	0.90 kN/m ²
Calculated Proposed Total Applied Permanent Loading	=	1.20 kN/m ²

Upper Floors

There shall be two distinct areas at the upper floor levels in terms of proposed new loadings; the area to be tiled and the area to remain untiled. It is anticipated that in the area to be tiled there shall be a net increase in the applied permanent loading as a result of the new build-up. However in the area to remain untiled, it is anticipated there will be a net reduction as the existing asphalt floor covering shall be removed as part of the works, with the proposed finish being exposed timber floor boarding. If the opportunity is taken to vacuum any debris from between the joists prior to the installation of the new installation, this should further reduce the load on the floor, but to an unquantifiable amount.

Calculated Existing Total Applied Permanent Loading	=	1.19 kN/m ²
Calculated Proposed Total Applied Permanent Loading (Tiled Area)	=	2.20 kN/m ²
Calculated Proposed Total Applied Permanent Loading (Untiled Area)	=	1.00 kN/m ²

Ground Floor

Similar to the upper floors, where the existing asphalt floor coverings are to be removed, a net reduction in the applied permanent floor loadings is anticipated. However, where the floor is proposed to be tiled, the applied permanent loadings is anticipated to increase.

Calculated Existing Total Applied Permanent Loading	=	0.99 kN/m ²
Calculated Proposed Total Applied Permanent Loading (Tiled Area)	=	2.20 kN/m ²
Calculated Proposed Total Applied Permanent Loading (Untiled Area)	=	1.00 kN/m ²

Existing structural components shall therefore be checked in the areas where an increase in applied permanent loading is anticipated, with all other areas being considered as acceptable, subject to a review of their existing condition.

7.0 EXISTING TIMBER FLOOR JOIST CHECKS

The worst-case total load at roof level has been considered based on the previously described proposed component weight, which is;

225mm deep x 45mm wide joists - maximum span = 4.7m

Permanent Load = 1.2 kN/m² = 0.48 kN/m / 0.40m crs
Variable Load = 0.75 kN/m² = 0.30 kN/m / 0.40m crs

Several influential factors and assumptions have to be considered;

- Load conditions are to be conservatively considered "long term"
- Timber grade for the joists is considered to be C16 which is considered conservative, base on the growth rate of trees used for structural members at the time
- Deflection limit of Span/250 is to be used

Results of this calculation process can be seen below:

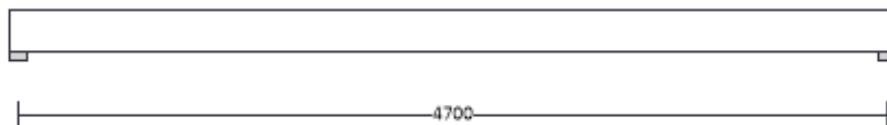
TIMBER JOIST ANALYSIS & DESIGN (EN1995-1-1:2004)

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Tedds calculation version 1.0.03

Joist details

Description 45 x 225 C16 timber joists
Joist spacing $s_{joist} = 400$ mm



Forces input on Joist

Vertical permanent load on joist $F_{G,joist} = 1.20$ kN/m²
Vertical imposed load on joist $F_{Q,joist} = 0.75$ kN/m²

Joist loading details

Distributed loads

Vertical permanent load on joist $p_G = F_{G,joist} \times s_{joist} = 0.48$ kN/m
Vertical imposed load on joist $p_Q = F_{Q,joist} \times s_{joist} = 0.30$ kN/m

ANALYSIS

Tedds calculation version 1.0.30

Loading

Self weight included (Permanent x 1)

Load combination factors

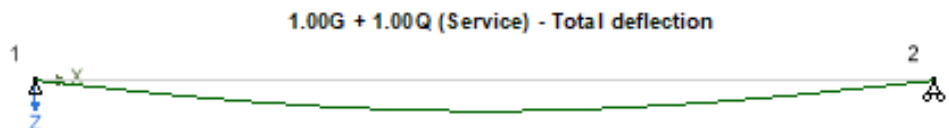
Load combination	Permanent	Imposed	Snow	Wind
1.35 G + 1.50 Q (Strength)	1.35	1.50	0.00	0.00
1.00 G + 1.00 Q (Service)	1.00	1.00	0.00	0.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.48 kN/m at 0 m to 4.7 m
Member	Imposed	UDL	GlobalZ	0.3 kN/m at 0 m to 4.7 m

Results

Total deflection



Node deflections

Load combination: 1.35G + 1.50Q (Strength)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.82655	
2	0	0	-0.82655	

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.58808	
2	0	0	-0.58808	

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.50Q (Strength)	0	5.4
1.00G + 1.00Q (Service)	0	3.8

Element end forces

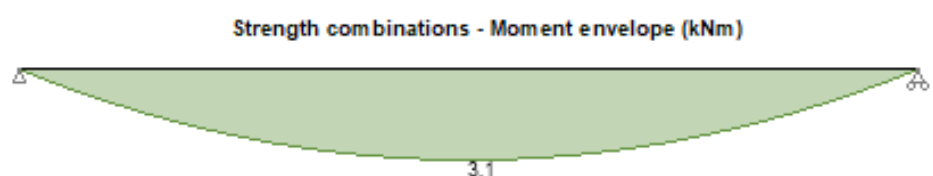
Load combination: 1.35G + 1.50Q (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	4.7	1	0	-2.7	0
		2	0	-2.7	0

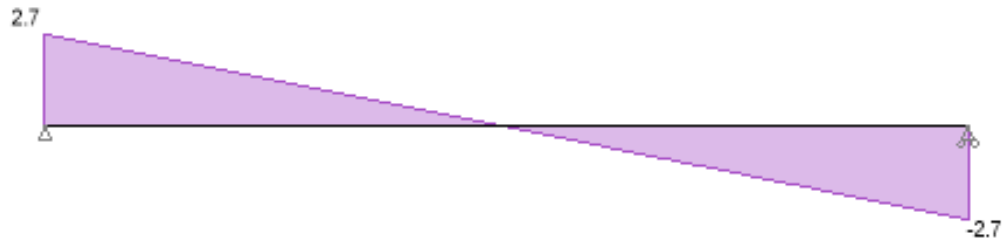
Load combination: 1.00G + 1.00Q (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	4.7	1	0	-1.9	0
		2	0	-1.9	0

Forces



Strength combinations - Shear envelope (kN)



Member results

Envelope - Strength combinations

Member	Position (m)	Shear force (kN)	Moment (kNm)
Member	0	2.7	0 (min)
	2.35	0	3.1 (max)
	4.7	-2.7 (max abs)	0 (min)

Tedds calculation version 2.2.03

Member - Span 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 $\gamma_M = 1.300$

Member details

Load duration - cl.2.3.1.2 Long-term

Service class - cl.2.3.1.3 1

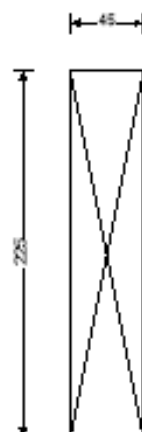
Timber section details

Number of timber sections in member N = 1

Breadth of sections b = 45 mm

Depth of sections h = 225 mm

Timber strength class - EN 338:2016 Table 1 C16



45x225 timber section

Cross-sectional area, A, 10125 mm²

Section modulus, W_y, 379687.5 mm³

Section modulus, W_z, 75937 mm³

Second moment of area, I_y, 42714844 mm⁴

Second moment of area, I_z, 1706594 mm⁴

Radius of gyration, I_y, 65 mm

Radius of gyration, I_z, 13 mm

Timber strength class C16

Characteristic bending strength, f_{b,k}, 16 N/mm²

Characteristic shear strength, f_{v,k}, 3.2 N/mm²

Characteristic compression strength parallel to grain, f_{c,k}, 17 N/mm²

Characteristic compression strength perpendicular to grain, f_{c90,k}, 2.2 N/mm²

Characteristic tension strength parallel to grain, f_{t,k}, 8.5 N/mm²

Mean modulus of elasticity, E_{mean}, 8000 N/mm²

Fifth percentile modulus of elasticity, E_{5,0}, 5400 N/mm²

Shear modulus of elasticity, G_{mean}, 500 N/mm²

Characteristic density, ρ_k, 310 kg/m³

Mean density, ρ_{mean}, 370 kg/m³

Span details

Bearing length $L_b = 100 \text{ mm}$

Consider Combination 1 - 1.35G + 1.50Q (Strength)

Modification factors

Duration of load and moisture content - Table 3.1 $k_{mod} = 0.7$

Deformation factor - Table 3.2 $k_{def} = 0.6$

Bending stress re-distribution factor - cl.6.1.6(2) $k_{m} = 0.7$

Crack factor for shear resistance - cl.6.1.7(2) $k_{cr} = 0.67$

System strength factor - cl.6.6 $k_{sys} = 1.1$

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d} = 2.678 \text{ kN}$

Effective contact length $L_{b,ef} = L_b = 100 \text{ mm}$

Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef}) = 0.595 \text{ N/mm}^2$

Design perpendicular compressive strength $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c,90,k} / \gamma_M = 1.303 \text{ N/mm}^2$
 $\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d}) = 0.457$

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d} = 2.678 \text{ kN}$

Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h) = 0.592 \text{ N/mm}^2$

Design shear strength $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 1.895 \text{ N/mm}^2$
 $\tau_{y,d} / f_{v,y,d} = 0.312$

PASS - Design shear strength exceeds design shear stress

Check design 2350 mm along span

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d} = 3.147 \text{ kNm}$

Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y = 8.287 \text{ N/mm}^2$

Design bending strength $f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m,k} / \gamma_M = 9.477 \text{ N/mm}^2$
 $\sigma_{m,y,d} / f_{m,y,d} = 0.874$

PASS - Design bending strength exceeds design bending stress

Consider Combination 2 - 1.00G + 1.00Q (Service)

Check design 2350 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection $\delta_y = 15.6 \text{ mm}$

Allowable deflection $\delta_{y,Allowable} = L_{m1,s1} / 250 = 18.8 \text{ mm}$

$\delta_y / \delta_{y,Allowable} = 0.83$

PASS - Allowable deflection exceeds final deflection

From the calculations above it can be seen that the roof joist can generally be considered to be adequate to support the proposed roof build-up and subsequent increase in loads, subject to condition.

The worst case total load at ground and upper floor level has been considered based on the previously described proposed component weight, which is;

275mm deep x 75mm wide joists - maximum span = 4.6m

Permanent Load = 2.2 kN/m² = 0.97 kN/m / 0.44m crs

Variable Load = 4.0 kN/m² = 1.76 kN/m / 0.44m crs

Given the very nominal increase in the proposed permanent load as a result of the addition of insulation across the rest of the building, by inspection that

Several influential factors and assumptions have to be considered;

- Load conditions are to be conservatively considered "long term"
- Timber grade for the joists is considered to be C16 which is considered conservative, base on the growth rate of trees used for structural members at the time
- Deflection limit of Span/250 is to be used

Results of this calculation process can be seen below:

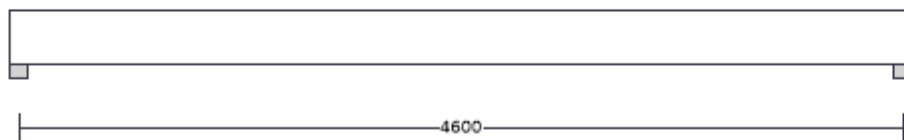
TIMBER JOIST ANALYSIS & DESIGN (EN1995-1-1:2004)

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Tedds calculation version 1.0.03

Joist details

Description 75 x 275 C16 timber joists
Joist spacing $s_{Joist} = 440$ mm



Forces input on Joist

Vertical permanent load on joist $F_{G,Joist} = 2.20$ kN/m²
Vertical imposed load on joist $F_{Q,Joist} = 4.00$ kN/m²

Joist loading details

Distributed loads

Vertical permanent load on joist $p_G = F_{G,Joist} \times s_{Joist} = 0.97$ kN/m
Vertical imposed load on joist $p_Q = F_{Q,Joist} \times s_{Joist} = 1.76$ kN/m

ANALYSIS

Tedds calculation version 1.0.30

Loading

Self weight included (Permanent x 1)

Load combination factors

Load combination	Permanent	Imposed	Snow	Wind
1.35G + 1.50Q (Strength)	1.35	1.50	0.00	0.00
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.97 kN/m at 0 m to 4.6 m
Member	Imposed	UDL	GlobalZ	1.76 kN/m at 0 m to 4.6 m

Results

Total deflection

1.35G + 1.50Q (Strength) - Total deflection



1.00G + 1.00Q (Service) - Total deflection



Node deflections

Load combination: 1.35G + 1.50Q (Strength)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.9009	
2	0	0	-0.9009	

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.62363	
2	0	0	-0.62363	

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.50Q (Strength)	0	18.5
1.00G + 1.00Q (Service)	0	12.8

Element end forces

Load combination: 1.35G + 1.50Q (Strength)

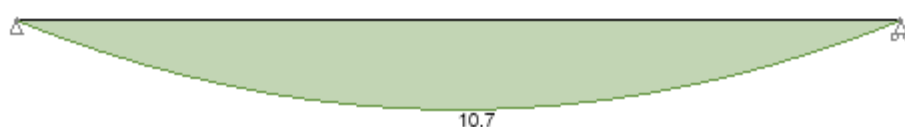
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	4.6	1	0	-9.3	0
		2	0	-9.3	0

Load combination: 1.00G + 1.00Q (Service)

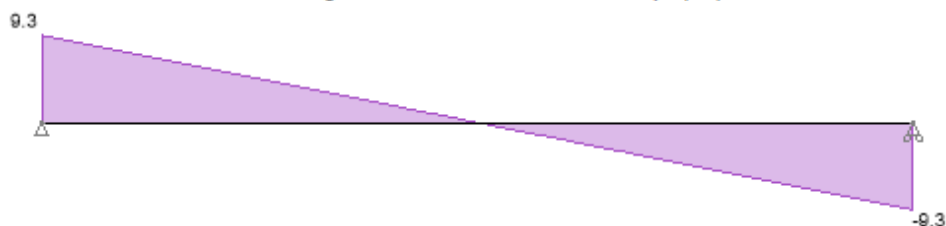
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	4.6	1	0	-6.4	0
		2	0	-6.4	0

Forces

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Member results

Envelope - Strength combinations

Member	Position (m)	Shear force (kN)	Moment (kNm)
Member	0	9.3	0 (min)
	2.3	0	10.7 (max)
	4.6	-9.3 (max abs)	0 (min)

Tedds calculation version 2.2.03

Member - Span 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 $\gamma_M = 1.300$

Member details

Load duration - cl.2.3.1.2 Long-term

Service class - cl.2.3.1.3 1

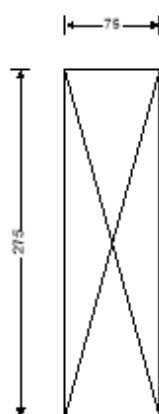
Timber section details

Number of timber sections in member $N = 1$

Breadth of sections $b = 75$ mm

Depth of sections $h = 275$ mm

Timber strength class - EN 338:2016 Table 1 C16



75x275 timber section

Cross-sectional area, A , 20625 mm²

Section modulus, W_y , 945312.5 mm³

Section modulus, W_z , 257813 mm³

Second moment of area, I_y , 129980469 mm⁴

Second moment of area, I_z , 9667969 mm⁴

Radius of gyration, i_y , 79.4 mm

Radius of gyration, i_z , 21.7 mm

Timber strength class C16

Characteristic bending strength, $f_{t,0,k}$, 16 N/mm²

Characteristic shear strength, $f_{v,k}$, 3.2 N/mm²

Characteristic compression strength parallel to grain, $f_{c,0,k}$, 17 N/mm²

Characteristic compression strength perpendicular to grain, $f_{c,90,k}$, 2.2 N/mm²

Characteristic tension strength parallel to grain, $f_{t,0,k}$, 8.5 N/mm²

Mean modulus of elasticity, $E_{0,mean}$, 8000 N/mm²

Fifth percentile modulus of elasticity, $E_{0,05}$, 5400 N/mm²

Shear modulus of elasticity, G_{mean} , 500 N/mm²

Characteristic density, ρ_k , 310 kg/m³

Mean density, ρ_{mean} , 370 kg/m³

Span details

Bearing length $L_b = 100 \text{ mm}$

Consider Combination 1 - 1.35G + 1.50Q (Strength)

Modification factors

Duration of load and moisture content - Table 3.1 $k_{mod} = 0.7$

Deformation factor - Table 3.2 $k_{def} = 0.6$

Bending stress re-distribution factor - cl.6.1.6(2) $k_m = 0.7$

Crack factor for shear resistance - cl.6.1.7(2) $k_{cr} = 0.67$

System strength factor - cl.6.8 $k_{sys} = 1.1$

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d} = 9.272 \text{ kN}$

Effective contact length $L_{b,ef} = L_b = 100 \text{ mm}$

Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef}) = 1.236 \text{ N/mm}^2$

Design perpendicular compressive strength $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c,90,k} / \gamma_M = 1.303 \text{ N/mm}^2$

$$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d}) = 0.949$$

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d} = 9.272 \text{ kN}$

Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h) = 1.006 \text{ N/mm}^2$

Design shear strength $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 1.895 \text{ N/mm}^2$

$$\tau_{y,d} / f_{v,y,d} = 0.531$$

PASS - Design shear strength exceeds design shear stress

Check design 2300 mm along span

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d} = 10.663 \text{ kNm}$

Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y = 11.28 \text{ N/mm}^2$

Design bending strength $f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m,k} / \gamma_M = 9.477 \text{ N/mm}^2$

$$\sigma_{m,y,d} / f_{m,y,d} = 1.19$$

FAIL - Design bending stress exceeds design bending strength

Consider Combination 2 - 1.00G + 1.00Q (Service)

Check design 2300 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection $\delta_y = 16.5 \text{ mm}$

Allowable deflection $\delta_{y,Allowable} = L_{m1,s1} / 250 = 18.4 \text{ mm}$

$$\delta_y / \delta_{y,Allowable} = 0.897$$

PASS - Allowable deflection exceeds final deflection

It should be noted that, although the bending stress is greater than the allowable, this is based on the worst-case load being applied to the full length of the beam. However, the proposal is for this load to be applied only to the 3.0m of the joists (approximately 2/3 of its total length) closest to the supporting wall on the southeast elevation. The remaining 1.6m of the joist will be subject to the lower untiled loading environment of 1.0kN/m² permanent and 3.5kN/m² variable, so this stress will not be realised.

Noting the above and considering the results of the calculations, it can be seen that the ground, first and second floor joists can generally be considered to be adequate to support the proposed new build-ups and subsequent increase in loads, subject to condition.

8.0 EXISTING FLOOR BEAM CHECKS

Opening up on site around the exiting wrought iron floor beams was limited and access to the underside of the beams not possible, the below calculations are therefore undertaken on the basis of the measurements which were achievable on site, namely a section depth being 250mm and top flange width being 130mm.

It is assumed that the bottom flange of the member would also be 130mm wide in the calculations, however on site it was noted that a riveted plate appears to the underside of the beams. The beams may therefore be acting compositely with the riveted bottom plate, which is most likely included to provide bearing to the timber floor joists. For the purposes of the calculation, any composite action with the bottom plate has been ignored, which is conservative.

Worst-case beam span (between columns) = 3.8m

Attributable load width between beams = $(4.4\text{m} + 4.4\text{m}) / 2 = 4.4\text{m}$

An untiled floor is proposed in this area, therefore design permanent loadcase = 1.00 kN/m²

Design variable load = 2.5 kN/m² + 1.0 kN/m² (partitions) = 3.5 kN/m²

Analysis for simply-supported single-span beam to BCSA 11/84

TEDDS calculation version 1.0.02

Span length & partial factors for loading

Span (mm)	Factors for moments & forces			Factors for deflection		
	γ_{fd}	γ_{fi}	γ_{fw}	γ_{dd}	γ_{di}	γ_{dw}
3800	1.00	1.00	1.00	1.00	1.00	1.00

Load descriptions

Loads are applied normal to the major principal axis (x-axis) of the member.

Ref.	Category	Description
1	"Dead"	"Floor Build-Up"
2	"Imposed"	"Imposed & Partitions"

Loading data (unfactored)

Ref.	Category	Type	Load kN/m	Position mm	Load kN/m	Position mm
1	"Dead"	UDL	4.4	0	-	3800
2	"Imposed"	UDL	15.4	0	-	3800

Analysis results - entire span

R_a kN (fac)	R_b kN (fac)	F_{vy} kN (fac)	M_x kNm (fac)	Sense	Deflection: δEI_x kNm ³	Direction
37.6	37.6	37.6	35.7	"Sagging"	53.76	"Down"

Unfactored support reactions

Support A;	Dead load; -8.4 kN;	Live load; -29.3 kN;	Wind load; 0.0 kN;
Support B;	Dead load; -8.4 kN;	Live load; -29.3 kN;	Wind load; 0.0 kN;

Member design checks for a simply-supported single-span beam to BC SA 11/84

HISTORICAL SECTION DESIGN

Try Dorman Long Rolled Beams 1887 10x5x38

For Year 1879

SHEAR CAPACITY

$$F_{vy} = 37.6 \text{ kN}$$

$$A_v = k_s \times t \times D = 3658 \text{ mm}^2$$

$$P_{vy} = p_v \times A_v = 282 \text{ kN}$$

$$\text{Utilisation ratio } \text{abs}(F_{vy})/P_{vy} = 0.133$$

PASS - Shear check

MOMENT CAPACITY - FULLY RESTRAINED

$$M_x = 35.7 \text{ kNm}$$

$$M_{cx} = \min(p_{bc} \times Z_{cx}, p_{bt} \times Z_{cx}) = 36.7 \text{ kNm}$$

PASS - Bending check

DEFLECTION

$$\text{Maximum bending deflection } \delta = \delta E I_x / (E_{55250} \times I_{xx}) = 3.5 \text{ mm}$$

$$\text{Allowable deflection } \text{Lesser of span} / 250 \text{ (ie 15.2 mm) and 16.4 mm}$$

Pass - Deflection within specified limit

From the above calculations and using the described conservative assumptions, it can be seen that the longest spanning wrought iron section shall be sufficient to support the proposed new floor loads, without taking account of any composite action provided by the addition of the riveted plate.

9.0 SUMMARY AND CONCLUSIONS

The building is in reasonable condition and free from evidence of significant structural movement. The new development may put additional load on these members locally which will require specific consideration, however overall there is anticipated to be a net reduction in the loads through the building as the existing asphalt floor covering shall be removed.

The capacity of the floor members are generally considered to be adequate for the proposed alterations, subject to verification of condition beyond the visible sample.

The existing floor joists have sufficient capacity for the live load and increased dead load associated with the proposed new floor finishes under normal loading. We would however recommend a further visual assessment of the existing timber more generally to confirm the grade of timber.

Primary supporting frame members are understood to be wrought iron. These beams are considered to have sufficient capacity for the live load and increased dead load associated with the proposed new floor finishes under normal loading. Subject to condition on a case by case basis and the introduction of additional ties.

We would suggest a more thorough examination of the existing wrought iron members is undertaken once the opening up works have been completed.

A further assessment will be required of those members that are currently not exposed and those that during the course of the works are found to be damaged or previously altered.