

structural planning report

Rylands Building, Manchester
CD9 Properties (Manchester) Limited

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1.0 executive summary

It is proposed to carry out design for the proposed restoration, refurbishment, and roof extension of the Rylands Building in Manchester. Woolgar Hunter has been commissioned by CD9 Properties (Manchester) Limited to provide civil and structural engineering advice on the project. The intention is that this report will summarise the engineering principles to support the planning and listed building application.

1.1 Introduction

This report provides a commentary on the structural engineering aspects for the proposed refurbishment and roof extension of the Rylands Building in Manchester.

The design proposal is to refurbish and redevelop the existing building to provide retail, leisure, and office space. The existing building structure would be retained with various structural alterations to the existing structure. Jeffrey Bell Architects (JBA) have developed a series of options which have led to the current proposed option. Please refer to Stephen Levrant Heritage Architecture (SLHA) for existing drawings and JBA for proposed drawings.

The current design has been developed in accordance with the IStructE Plan of Work to Stage 2 concept design.

1.2 The Project

It is proposed to refurbish and redevelop the Grade II listed existing eight storey – plus two basements – building in the heart of Manchester City Centre.

The architectural proposals involve an array of structural interventions, namely:

- Introduction of natural light via the inclusion of a central glass-covered atrium.
- Addition of four storeys of office along Bridgewater Place.
- New supporting structure for the inclusion of additional offices which includes columns on GL-12 as well as additional columns on GL-14 where there are existing loading bays.
- The introduction of new vertical servicing cores for people and services.
- Introduction of winter gardens at sixth floor level to compensate for the lack of natural light due to minimal fenestration.
- Removal of the existing mezzanine level.
- Partial demolition of the ground floor and reinstatement of the ground floor at various levels.
- Strengthening of existing columns and beams locally to support changes in loading.
- Infilling of various existing voids through the floor.
- Local foundation strengthening.



Figure 1.1: Rylands Building Within Emerging Context

2.0 superstructure

The superstructure section of the report details the design principles and optioneering for the Rylands Building in Manchester. This includes insight into the historic information reviewed at this stage.

2.1 Building Description

The Rylands Building was constructed in 1932 for the purpose of being a textile warehouse for the Rylands family. The property, currently in use as a department store after a conversion in the 1950s, is a Grade II listed building.

The ground floor was originally designed to act as a department store/shopping arcade based on the available archive drawings obtained from the Manchester Central Library archives. This provision also included a small area of mezzanine floor which was further extended by Debenhams around 1977. Further historic details can be found in the Heritage Statement, which has been prepared by SLHA and submitted as part of the applications.

Other structural interventions to the building include (but is not limited to) the introduction of large apertures to accommodate escalators between the shop floors of Debenhams of which no record information could be obtained.

The structure itself, which has large floor plates, is comprised of eight storeys plus two basements. The sub-basement has a reduced footprint area compared to the basement. There is a bank of loading bays at the rear of the ground floor, which can be accessed from Bridgewater Place.

The building is a steel frame that was originally constructed in two phases to ease the construction sequencing. Phase one is denoted as section one on the planning drawings, while phase two is denoted as section two and section three. Generally, the building is constructed of a steel frame, of which construction archive photographs and planning drawings exist, which show bespoke fabricated steel stanchions and beams with supplementary plates to create the primary grid. The majority of the stanchions on the internal floor plate appear to have been concrete encased as well as some of the floor beams. From the limited access to the building it would appear that the perimeter columns are not typically concrete encased and are instead encapsulated by brick on the internal façade and Portland stone on the external façade. There is evidence of corrosion on some of the perimeter columns although the extents and severity could not be determined and it is strongly recommended that the condition of the steel frame and the associated bracketry for the Portland stone façade is investigated as a priority.

The site is bounded by Metrolink on two sides (Market Street and High Street), with fixings to support the Metrolink cabling fixed into the building's façade.

It has been noted that the building has been poorly maintained with several areas of the rainwater goods having small trees growing from the hoppers and areas of water ingress and damage noted within the building particularly at the upper floors and around the loading bay area.

Several reports have been undertaken in the past three years - both externally and internally - including a survey of the façade which note that there is the potential for the embedded steel to need repairs, but to date no intrusive inspections have happened but there is evidence of corrosion internally for the upper floors. Intrusive inspections and the input of remediation specialists will be required to categorise the risk and the costs associated with the necessary remedial works.

The elevations of the building are clad in stone, thought to be Portland as noted by Sandberg in their Assessment of Water Ingress Damage report in 2017. The façade itself is Grade II and has been recently surveyed by Quadriga, however an intrusive survey to inspect the steelwork and the connection details within the facade is still required.

Due to the current use of the building as a department store, this has hindered inspection of the trading floor plates where the structure is concealed behind shop fittings as well as limited access to the overall building generally.

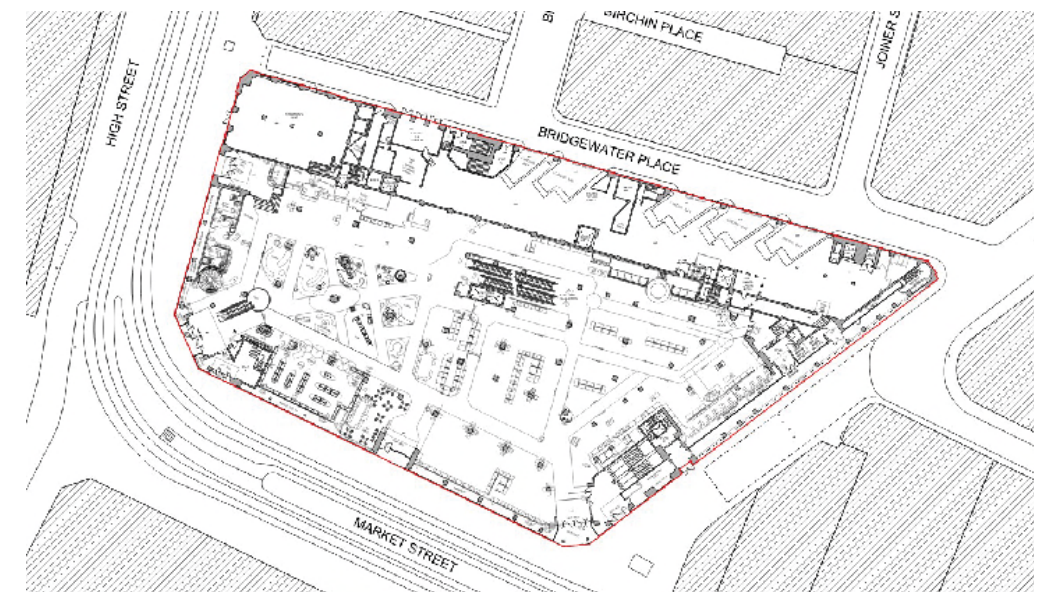


Figure 2.1: Site Plan

2.2 Historic Information

Historic information on the Rylands Building has been obtained from the Manchester Central Library, by reviewing archive information in the public domain. Records of the original planning drawings for floor plates showing reinforced concrete slab, beam, and column arrangements were found that greatly aided the understanding of the existing building. It must be noted that these original linen drawings appear to be only for the original planning application and are not the 'as built' drawings.

The sub-basement, basement, and ground floor heights are noted on the original drawings as 11 ft 9 in (3.58 m), 12 ft (3.67 m), and 20 ft 9 in (6.32 m), respectively. Other than this, the typical floor to floor heights are noted to be 13 ft (3.96 m). The basement floors were constructed with a staggered footprint, with the sub-basement level being slightly smaller on plan than the basement level.

It has already been noted by JBA and Woolgar Hunter that there are discrepancies between the planning drawings and the survey which was undertaken on the Rylands Building. These structural discrepancies include column and stair/ lift core locations, some of which have been relocated while Debenhams have been tenants. TGA notes that the structural alterations were undertaken around 1977, which include staircases, escalators, and mezzanines.



Figure 2.2: Historic Photograph of Rylands Building Under Construction

2.3 Design Stresses

Using known information about the age and design use of the Rylands Building, as well as informed engineering assumptions, an estimate of the design stresses has been generated.

As the frame was built after 1910 and all the sections specified on the steel drawings conform to member sizes from BS4, the beams and columns are assumed to be mild steel; and so mild steel stresses for the period have been used.

When designing to yield strength as per more modern codes such as BS5950 or EC3, an assumption can be made from BS15 in 1948 where yield strengths were first introduced.

In the 1948 edition of BS15 Structural Steel, the ultimate tensile strength was still taken as the 28-32 tons/sq.in as per the 1912 and 1930 BS15, allowing for the assumption that the yield strength would be the same or similar in 1932 when the Rylands Building constructed.

Coupon testing of the yield and ultimate strength of the existing beams should be performed in order to confirm the engineering assumptions made by Woolgar Hunter. These tests could be performed on existing steel beams that are to be demolished from the building to facilitate the proposed atrium. As the building was built in three phases, testing of the 'to be demolished' atrium steelwork will provide a good sample of the material properties for the existing frame between GL-A to GL-O. It would be prudent to also coupon test existing steel between GL-O and GL-Y.

2.4 Restraint

While the Institution of Structural Engineers Report on Steelwork for Buildings Part 1 Loads and Stresses printed in 1927 didn't specifically mention lateral instability, it did allow for a reduction to the permissible bending compression stress (pbc). This was later redefined as lateral restraint in the 1938 document under the same name. This indicates that lateral restraint was beginning to be understood during this period, as neither the 1909 L.C.C Act nor Andrews in Theory and Design of Structures mention lateral stability.

From the load assessment of the steel beams, as further described in section 2.6, it was determined that shallow, secondary beams at 2 metre centres were designed as though they were fully restrained, but the larger primary beams on the 12.2 metre spans along Bridgewater Place were designed as unrestrained. Other large beams were designed as either unrestrained or only restrained at incoming beam locations.

This indicates that the concrete encasement to the beams was used solely for fire protection and was not assumed to restrain the compression flange of the beams. This means that the existing primary beams likely have a higher capacity than the design originally intended. This additional capacity cannot be utilised however, as the beam end connections, the columns, nor the foundations will have this spare capacity without costly intervention.

2.5 Net Capacity

Although there is information for the floor plates of the Rylands Building, there is no information available for the foundations. The capacity of the existing structure has been determined by conducting a load equivalency assessment.

After the load equivalency assessment, scenarios for removing and adding structure can be reviewed in order to determine if the proposals exceed the loading of the existing foundations. Fundamentally, the spare capacity of a structure is the net of the original use of the building and the proposed use. If the proposed use exceeds the load of the original use, this solution would not be viable without structural intervention at foundation level.

While it is possible to strengthen foundations, this is difficult, disruptive, and costly. Limited investigation into the foundation extents has been undertaken by Henderson Thomas Associates (HTA) over the course of three days. The investigation was informative but inconclusive. See section 3.0 for further commentry on substructure and the HTA investigation.

The difference in load between the original use of the structure and proposed use of the structure is the thought process for considering additional storeys on the structure without considering the foundations. The more storeys that you can accrue the spare capacity, the greater the likelihood of adding storeys to the structure.

It must be noted that the additional height from the new storeys will increase the wind face that the structure must resist by utilising the stair and lift cores. This increased load must still be transferred down to the foundations.

As mentioned, the structure as it currently stands has been modified from its initial construction. This includes modifying the core layouts, presumably to make the building more compatible with the proposed use of a department store. From a structural engineering perspective, there may be benefit in introducing new vertical core elements to the structure on a storey-by-storey basis and integrating the temporary and permanent works as part of the design.

As there are disparities between the original drawings and survey drawings, opening up works are required to determine existing structure.

2.6 Load Assessment

Using known information, such as the previous use of the Rylands Building as well as informed engineering assumptions which will later have to be verified, a loading assessment on the structure has been undertaken.

As there are no original calculations available for the Rylands Building, the original design allowable floor loading must be determined in order to understand the net free capacity of the structure as described in section 2.5. The capacity of the existing frame has been determined in three ways, which all result in the same original design allowable floor loading:

- Historical live load classification from codes of practice.
- Calculation of the capacity of the slab and floor beams using modern codes (and historic assumptions relating to restraint and design stresses as indicated in section 2.3 and section 2.4).
- Assumed total SLS beam end reactions from archive planning drawings obtained from the Manchester Central Library.

Based on the reinforced concrete slab alone, a significantly higher floor capacity can be calculated. However, due to the short spans, the sizing of the concrete slab is likely to be due to the thinnest practical construction depth as opposed to being design driven. Therefore, determining the floor capacity is likely to be governed by the structural steel beams instead.

The assumptions of the steel mechanical properties must be confirmed as described in section 2.3.

2.7 Roof Capacity

The mastic asphalt surface on the roof is in poor condition, as evident from Figure 2.3.

There are several roof leaks which have caused damage to the internal finishes as well as to existing concrete encasement, exposing reinforcement and structural steelwork.

The existing rooflights appear to be traditional flat glass squares, which have since been covered with roof felt during a later re-roofing. The existing rooflights are sometimes known as pavement lights, as their usual use was to provide daylight into basements.

At the roof, the historic drawings obtained from the Manchester Central Library suggest that the roof beams are the same as the storeys below except where specific enhancement has been made locally to the plant rooms. This indicates that the roof may also have been designed to support the same loading allowance, with a larger allowance where plant rooms exist. Testing is required at this level also to confirm.



Figure 2.3: Asphalt Surface

2.8 Slab Capacity

Where access was available, Woolgar Hunter confirmed that there are reinforced concrete slabs which span, approximately two metres, onto a system of primary and secondary beams. From the original planning drawings, the reinforced concrete slab thicknesses vary across the floor plates, being typically thicker on the first bay back on the three front elevations; due to the larger three metre spans to fit in with the facade arrangement. While the thickness could not be confirmed from a site walkaround, a change in depth could be determined due to a change in height to underside of slab.

There are numerous different types of floor finishes which include tiles, parquet, and cork, which appear to be laid on a 1" (25 mm) concrete screed.

While the slab load capacity was determined to be much higher than the supporting secondary and primary steel beams, the short spans mean that the sizing of the concrete slab is likely to be due to the thinnest practical construction depth as opposed to being design driven. Therefore, the floor capacity is likely to be governed by the structural steel beams instead.

2.9 Beam Capacity

The capacity of the beams has been determined as described in section 2.4 where historic restraint conditions and rules have been considered. It was typically found that shallow secondary beams were considered fully restrained, while primary beams were typically only restrained by the secondaries, typically at third points. Beams spanning over two bays were typically restrained at midspan by incoming primary beams, and the long 12.2 metre spans to the back of the building on Bridgewater Place were considered unrestrained.

The secondary beam end connections are typically pinned, with historic photographs suggesting that two back-to-back angle sections in a similar fashion to a modern fin plate connection were used to provide a pinned end.

The primary beams were often without an end plate from historic photographs, and were instead sat on a ledge angle with the top flange restrained with another angle section at the top of the beam.

2.10 Column Capacity

The existing column capacities have been indirectly determined from the load assessment of incoming beams, due to no information being available on the columns without intrusive investigation.

Historic information on column sizes, connections, and changes in section are unavailable, and the concrete encasement of the columns has made non-intrusive measurement difficult to determine section sizes.

From historic photographs, it can be determined that upper storey columns may be rolled sections, while lower storey columns would appear to be fabricated bespoke sections formed from plates and rivets. Due to the limited historic photographs, the extents of fabricated sections is difficult to determine, but it does indicate that the columns were specifically sized for the load they were to carry, due to the apparent changes in column section throughout the building.

All the internal columns appear to be encased in concrete, presumably for fire protection. The exception to which are the columns on the elevations which appear to be contained within the Portland stone cladding. This is also evident from the original drawings accessed from Manchester Central Library archives.

The use of the concrete in concrete encased steel sections to enhance the structural capacity was not permitted in the design codes until 1948, meaning that the encasement is likely only for fire protection, and was not designed to provide an enhancement to capacity. Under current codes of practice this will still be the case, as there appears to be no structural reinforcement in the existing concrete encasement noted on the historic drawings.



Figure 2.4: Splayed Columns

The safe assumption that the columns were designed only for the incoming load of each storey has been made. As such, the columns have been marked for potential enhancement wherever the proposed load is greater than the load the column was originally designed for.

In the sub-basement, the primary columns have splayed bases which are shown below in Figure 2.5. The splayed bases may be encasing gusset plates and can be seen on historic construction photographs.

The secondary columns at sub-basement level appear to be supporting only two storeys of loading, which subsequently require smaller base plates and no gusset. It would appear that the secondary columns are intended to provide an increased loading allowance at basement and ground floor. However, this is subject to surveys.



Figure 2.5: Splayed Columns

2.11 Stability

The Rylands Building in Manchester is stabilised by three existing masonry cores with steel columns in the corners, which have been labelled as shown in Figure 2.6. From visits to site, it is known that the masonry in the cores was constructed tight to the columns and beams to prevent the primary columns and beams from racking. This provides lateral stability and transfers the wind loading to the foundations beneath the cores.

It is known from construction photographs that the building was constructed in two phases. Phase one (noted as section on planning drawings) was constructed first, with phase two (noted on the planning drawings as section two and three) being constructed thereafter. Along GL-7, GL-8, GL-9, and GL-14 (at least) it was noted that the base of the columns at basement and sub-basement level are flared. In a couple of instances, concrete was missing from these flared bases and steel was visible. After reviewing the original construction photographs, it appears as though the steel visible from the site visits are the gusset plates which have been affixed at the bases of the columns. As such, it is believed that these columns with flared bases may have been designed to provide some lateral stability during the construction of section one.

The additional proposed four storeys and one plant storey increase the wind face of the building, and thus increase the loads on the existing stability cores. The existing core at the Core 2 location is to be maintained with no structural intervention due to the original bird cage lift being retained and being of particular heritage merit.

To enable the bird cage core to be unmodified, and as the existing Core 1 is to be demolished and a new concrete core introduced further south, the newly introduced core has been sized to limit the load applied to Core 2. Core 3 will also require enhancement to deal with the additional load and prevent additional load from being applied to Core 2.

These modifications to Core 1 and 3 will limit any structural enhancement required to the more heritage-sensitive Core 2.

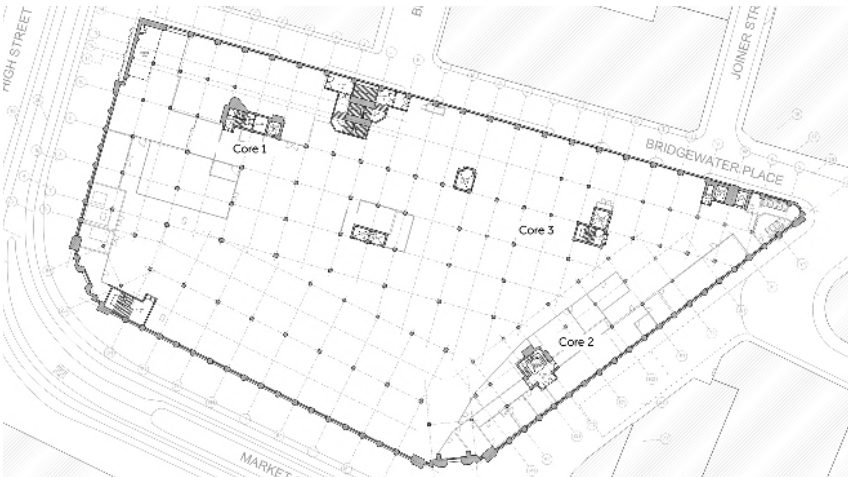


Figure 2.6: Existing Cores

2.12 Architectural Proposals

Large portions of the building are currently left empty or grossly underutilised and the building has a sizeable portion of its floors unoccupied.

The building has been surveyed and faces a multi-million pound repair bill to rectify the years of neglect from the previous owner and, in order to realise the restoration of the building and to bring it back into useful service, a series of interventions are required to make the deep floor plates appropriate for office use and commercially lettable.

These interventions broadly include:

- Introduction of natural light via the inclusion of a central glass-covered atrium.
- Addition of four storeys of office along Bridgewater Place.
- New supporting structure for the inclusion of additional offices which includes columns on GL-12 as well as additional columns on GL-14 where there are existing loading bays.
- The introduction of new vertical servicing cores for people and services.
- Introduction of winter gardens at sixth floor level to compensate for the lack of natural light due to the lack of windows.
- Removal of the existing mezzanine level.
- Partial demolition of the ground floor and reinstatement of the ground floor at various levels.

The proposal for the Rylands Building includes the partial removal of floor plates from first to seventh floor to create an open atrium through the building. To allow light into the structure at seventh, there is a proposed hipped roof which covers the void. At first to sixth floor, there are terrace areas shown to the right hand side of the atrium void. Additionally, there are proposed terrace areas around the external perimeter at sixth floor level described as winter gardens. Due to the limited light penetration at sixth floor level, the architectural proposals appear to be removing existing covered up pavement roof lights to achieve this light infiltration. To maintain restraint of the façade at this level, the existing beams which pass under the roof lights must not be removed.

The proposed roof extension is four storeys of office with one storey of plant loading atop the office, which will be disguised by a sawtooth roof. The current proposals for the office include being clad with a top hung unitised glazing system.

To accommodate for the new proposed uses, lift and stair cores throughout the building are proposed to be relocated. This will reconfigure the frame into distinct regions. The proposed modification of cores is described in section 2.13.

The existing mezzanine level is to be removed in its entirety, which is indicated on the JBA demolition drawings. Part of the mezzanine is believed to be original, from the 1916 and the 1929 archive drawings for the building.

At the ground floor on Bridgewater Place, there are existing loading bays and dock levellers. It is proposed that the dock levellers be removed and the levels for the ground floor will be modified to suit the current proposals.

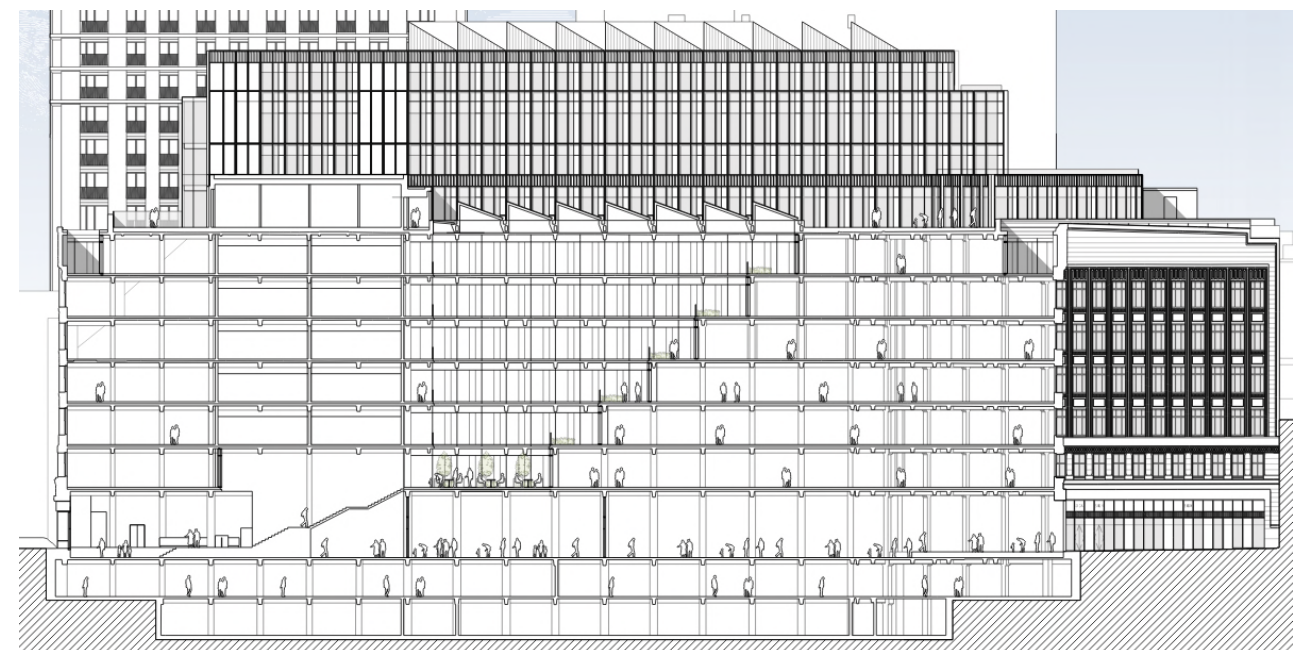


Figure 2.7: Rylands Building Proposals

2.13 Cores

In order to distribute modern services up through to the upper floors and also to integrate a dedicated series of lifts, there will be a need to introduce new apertures through the floor plates and associated supporting structure.

These new cores will also provide lateral resistance to resist the increase in lateral loads from the additional floors and will be sized suitably.

Where possible a minimal intervention will be adopted but this may not always be the most practicable, cost-effective, carbon efficient, or safe method of construction.

From the archive information available from the Manchester Central Library achieves it is understood that the lateral stability of the building is provided by three existing masonry cores with steel columns to the corners of the cores.

Through the extension of the building with an additional four storeys and a plant level, the lateral wind loading is almost doubled. Initial analysis shows that the existing masonry shear walls will likely be overstressed without enhancement.

Core 1

The design philosophy proposes that Core 1 integrates most of the new services for the building creating a vertical corridor linking all of the floor plates together for modern specification lifts and to integrate an

efficient M&E servicing strategy. In order to accommodate the increase in services required for a modern building as opposed to the original low demand storage floor plates originally constructed a series of new apertures are proposed.

There are two possible methods for constructing Core 1, both methods require the demolition of the existing masonry walls in order to form the new core arrangements.

Method 1 – partially remove a bay comprising of two columns, the local supporting floor beams, slab, and foundation. Then construct a new piled foundation, including lift pits, with the core formed using a reinforced concrete core wall and reinstate the small sections of floor around the perimeter and tie into the existing floor plates through the use of pull out bars. This is the preferred engineering method given that this is the primary core and there is a requirement to integrate four new lift pits at the lowest level and constructing the core in this manner will be safer and more predictable with regards the required lift shaft sizes.

Method 2 – on each floor plate, break out a slot through the existing floor slab at every floor level to enable a reinforcement cage to be threaded between floors and cast each portion on a floor by floor basis, tying the vertical reinforcement into the existing floor reinforcement at every level. Each lift would have to be poured in two sections to prevent axial shortening through shrinkage exerting additional forces onto the existing floor plates. This method requires a detailed survey of the existing structural beams and is the slowest form of construction and will require the modification of the existing foundations in order

to form four new lift pits. This will be complex and if this method is selected then additional opening up works will be required to confirm the feasibility within the foundation levels. Foundation arrangements are likely to be complex, slower to construct and may compromise the space for this option due to the existing large foundations beneath the existing columns and the need to form four lift pits. This method has significant risk, cost and a high likelihood of not enabling a functioning core to be integrated within the building and is included within this report for completeness to demonstrate that a less radical intervention has been considered.

Core 2

The are no major structural interventions proposed for Core 2. While the new roof extension will significantly increase the wind face of the building, the interventions at Core 1 and Core 3 have been developed to absorb this additional lateral load, which will leave Core 2 largely unaffected given the significance of this core.



Figure 2.8: Proposed Cores

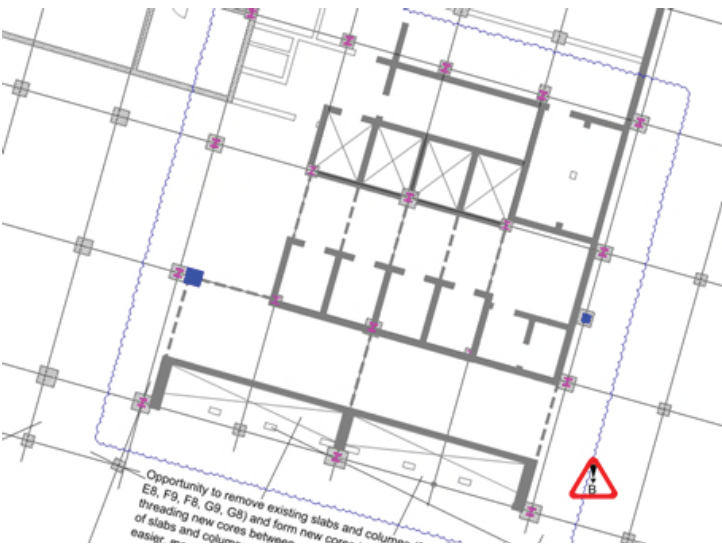


Figure 2.9: Core 1

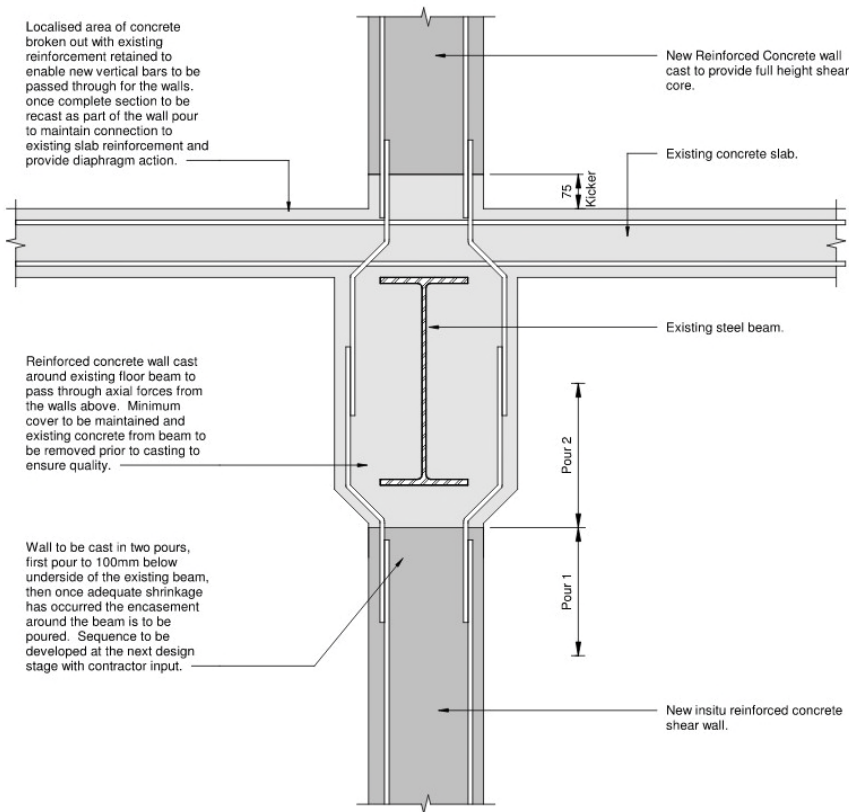


Figure 2.10: Core Detail

2.13 Cores (cont.)

Core 3

Core 3 is proposed to be extended on plan through the introduction of new concrete shear walls threaded through the existing structure. As there are no new lift pits required, it is proposed that additional walls are cast adjacent to the existing masonry walls to create a concrete box that encapsulates the existing shear walls. New foundations are proposed to be mini-piled down to suitable strata.

Core 1 is likely to be constructed from slip or jump form. In locations where the steel beams connect to the core, the core walls need to be 300 mm thick to accommodate the steel beam connection via a cast in steel plate. Core 3 will be cast on a floor by floor basis and may require additional propping where cast against existing brickwork.

The steel beams will be connected to the core with a 50 mm plate cast into the wall. The steel plate will be oversized to allow for tolerance of erection. To the rear of the steel plate reinforcement will be welded to tie into with the wall reinforcement.

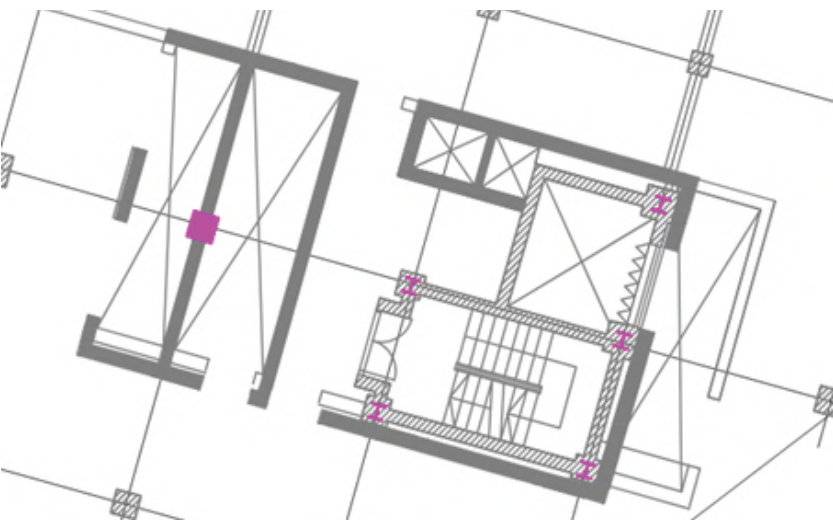


Figure 2.11: Core 3

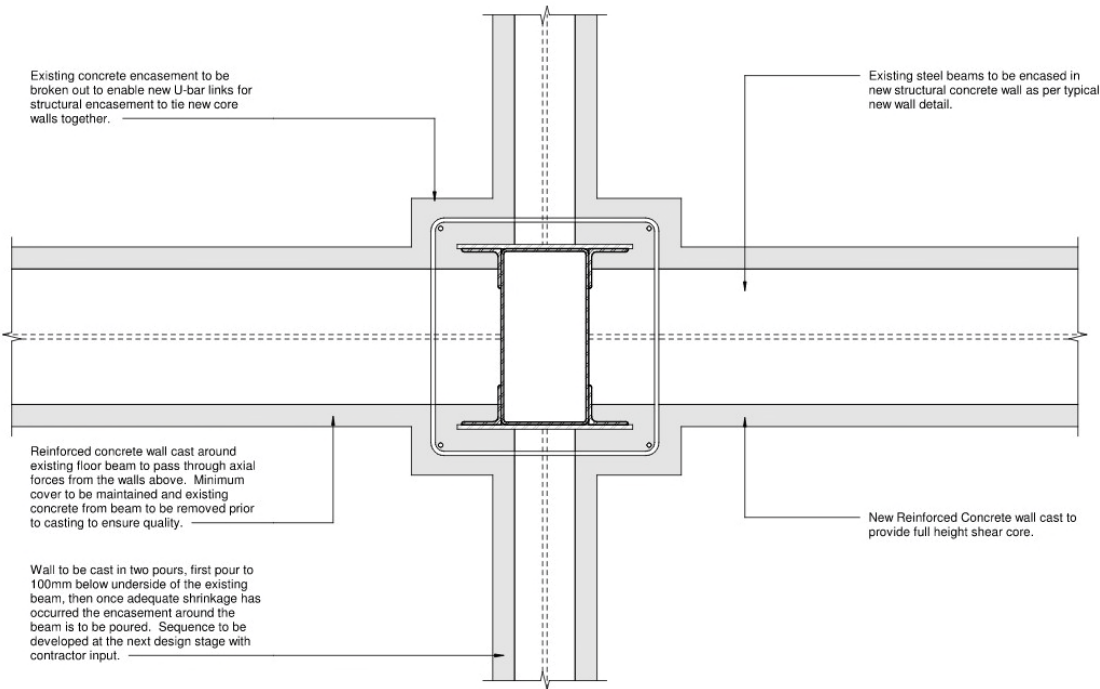


Figure 2.12: Core Detail

2.14 Atrium

The floor plates for the existing building are deep and not originally designed to integrate large amounts of natural light as their intended use was understood to be for storage of cloth. To adapt the building to a proposed office use, there will be a need for the introduction of more natural light through the introduction of a central atrium that varies in depth throughout the height of the building.

Following feedback from agents and to increase floor presence from the proposed core locations the new aperture, shown in Figure 2.13, falls along the middle of an existing beam line. This is further complicated in that the floor plates have numerous grid configurations and the central run of columns span two grids.

With the atrium edge being positioned approximately midspan of existing simply supported beams, there is a need for temporary propping to support the beams during demolition. It is proposed that twin columns will be threaded between these beams for the full height of the building to support the aperture in both the temporary and permanent state. These twin columns will also support the new roof extension facade line and feature atrium roof.

Twin columns have been proposed to allow for variances in the position of the steel beams to avoid making single columns appear crooked as they run through the building. The specific distance between the twin columns will be determined following a detailed positional survey of the existing beams over the full height.

The twin columns carry through the building and are transferred at first floor level so that half of them carry down to the sub-basement level where a new foundation is proposed to support the columns.



Figure 2.14: Atrium Column Foundations

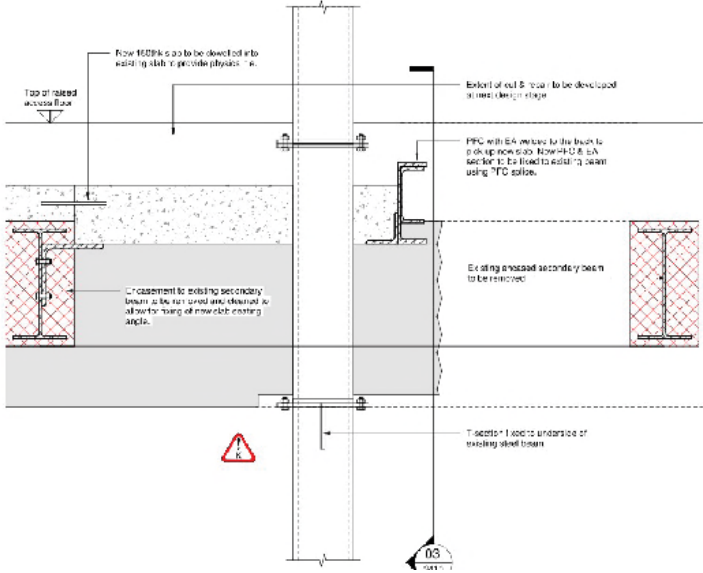


Figure 2.16: Atrium Edge Detail

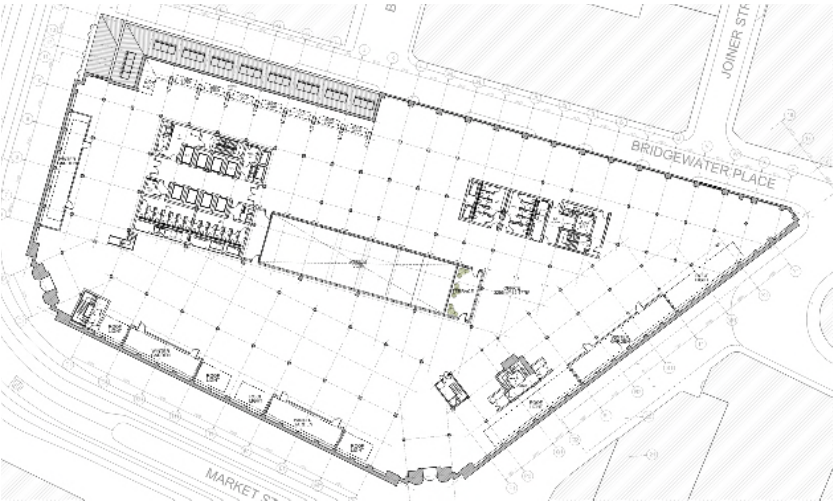


Figure 2.13: JBA Sixth Floor

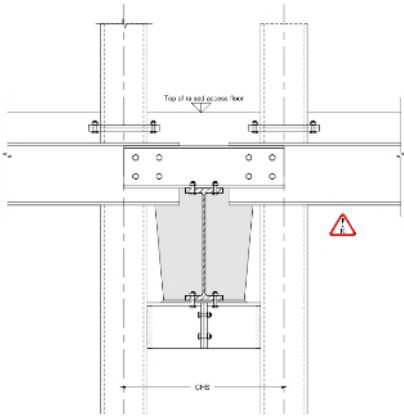


Figure 2.15: Atrium Edge Detail

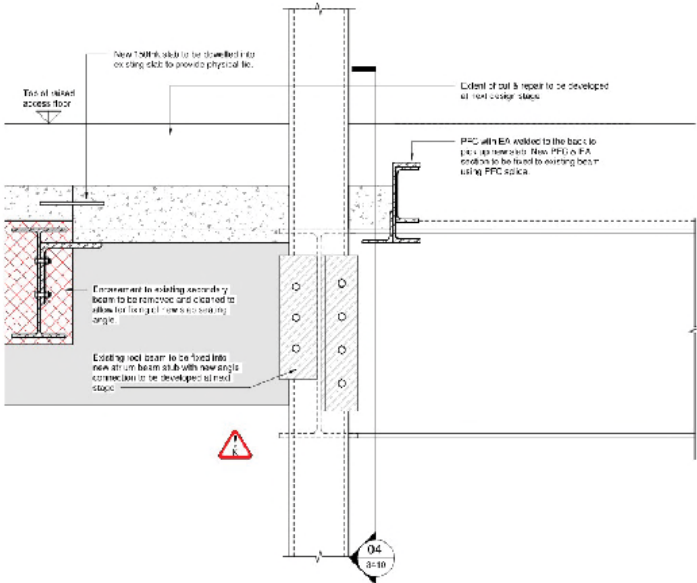


Figure 2.17: Atrium Edge Detail

2.14 Atrium

A walkway is required immediately after exiting Core 1, linking two sides of the atrium. This portion of slab is currently supported by simply supported beams. Two options for supporting this edge using a new beam are proposed in Figure 2.18 and Figure 2.19.

Option one would involve the temporary propping of the existing structure while a new beam is installed. This option will maintain the existing slab and beam, which involves less demolition of the existing structure.

Option two involves cutting the portion of slab back to existing beam lines while forming the rest of the atrium. A new beam and concrete slab can be installed without the need to temporarily prop the existing structure. Moreover, the structural slab will be designed in accordance with modern design codes and is likely to be thicker as a result. The top of slab level will be maintained to assist the installation of the raised access floor.

Option two is the preferred method for forming the atrium walkway.

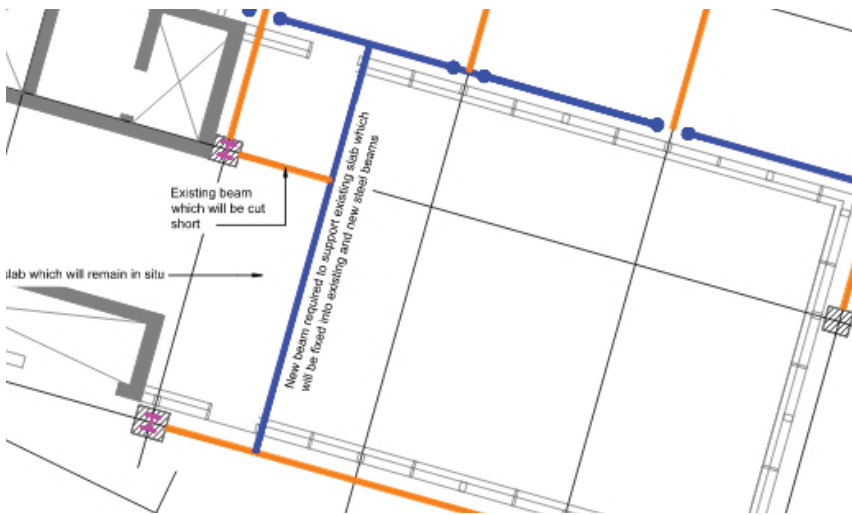


Figure 2.18: Atrium Option One



Figure 2.19: Atrium Option Two

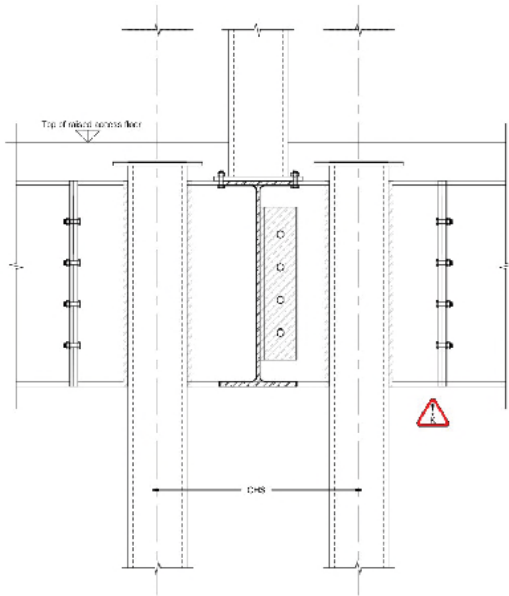


Figure 2.20: Atrium Detail

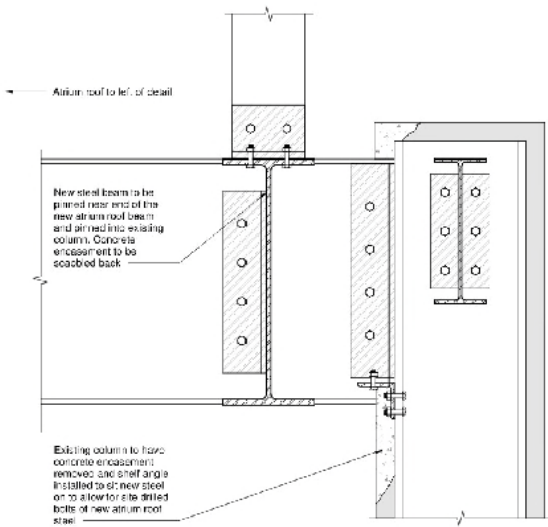


Figure 2.21: Atrium Detail

2.15 Roof Extension

The roof extension is proposed as a four storey frame with a saw tooth roof over that conceals a high level plant space. The additional floors are proposed to be clad in a top hung unitised cladding system and thus deflections are critical.

A floor zone for the roof extension has been identified of 1,100 mm including a 200 mm raised access floor, structure, ceiling, and services.

There are a variety of servicing strategies identified for the development, including either 400 mm diameter circular ducts or 250 mm deep rectangular ducts (both noted as external diameter including any insulation).

To comply with the available depth both of the duct types will need to be utilised. Where possible, shallower (and thus lighter) beams will be adopted to allow rectangular services to pass underneath. However, there will be instances where the grillage is dictated through transfer point loads from columns above, driven by deflection criteria, and driven by the existing column layout below, thus deeper beams will be required which will not allow for rectangular ducts to pass underneath. In these areas a minimum depth of a 610 UKB structural steel section will be adopted with 450 mm cells cut through to enable the services to pass through as circular ducts.

The proposed reinforced concrete cores that are integrated within the existing building carry through up to roof level and provide the lateral stability for the roof extension too.

The steelwork at the upper floors will be painted with intumescent paint to the fire engineer’s details and specifications.

Column positions are largely dictated by the existing structural grid and are aligned to ensure efficient load transfer and limiting the need for transfer structure. A nominal 500 mm zone above the existing roof

slab constrains the ability to overbeam the roof structure and several assumptions have been made and still require confirmation as part of the opening up works for the building once access can be successfully negotiated. Existing column heads will need to be exposed to form a new connection detail with the floor above, this may take the form of a site welded baseplate and will be subject to opening up works.

- The historic drawing show that the existing steel beam grillage is the same as the lower floors indicating that the roof structure has a similar load carrying capacity.
- The thickness of the roof slab is also noted as being the same as the lower floors.
- The condition of the structure is assumed to be reasonable.

If the roof slab and the existing beams are as anticipated it is likely that the roof slab and beams can be reused. Although where columns are unable to be passed through directly to the column beneath there will likely be a need for strengthening to the beam to resist the additional forces.

From the planning drawings viewed at Manchester Central Library archives, the title block indicates the reinforcement intent is generally the same from first through seventh floors, with local variations. Additionally, the steel beams specified are largely the same with exceptions at seventh where there are plant rooms etc. above.

If the slab is found to be inadequate then there remains an option that the beam can be overlain with a cold formed cassette based flooring system onto the existing beams and this will be able to transfer the

loads back to the primary beams within the allowable 500 mm structural zone.

If the beams are found to be inadequate, then strengthening will be required or the available 500 mm zone will need to be reviewed to enable an over beaming strategy to be developed.

A line of new columns has been introduced along GL-12 for the full height of the building. This is proposed to be cast in situ reinforced concrete around the existing beams. It is proposed that these are cast as two distinct pours for each storey to prevent shrinkage introducing secondary forces to the existing frame. A detail explaining the integration of columns on GL-12 is shown in Figure 2.23. This will place a constraint on the programme and options involving precast columns could be investigated with a contractor at the next stage.

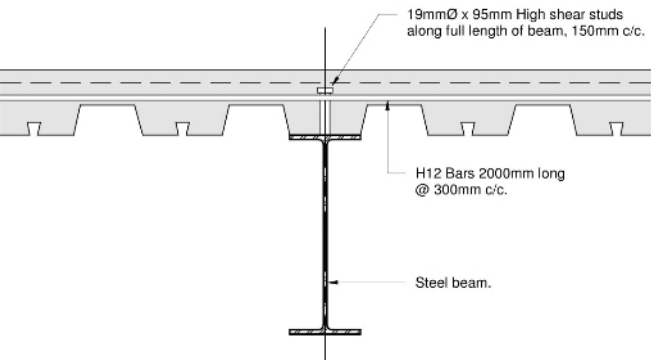


Figure 2.22: Typical Floor

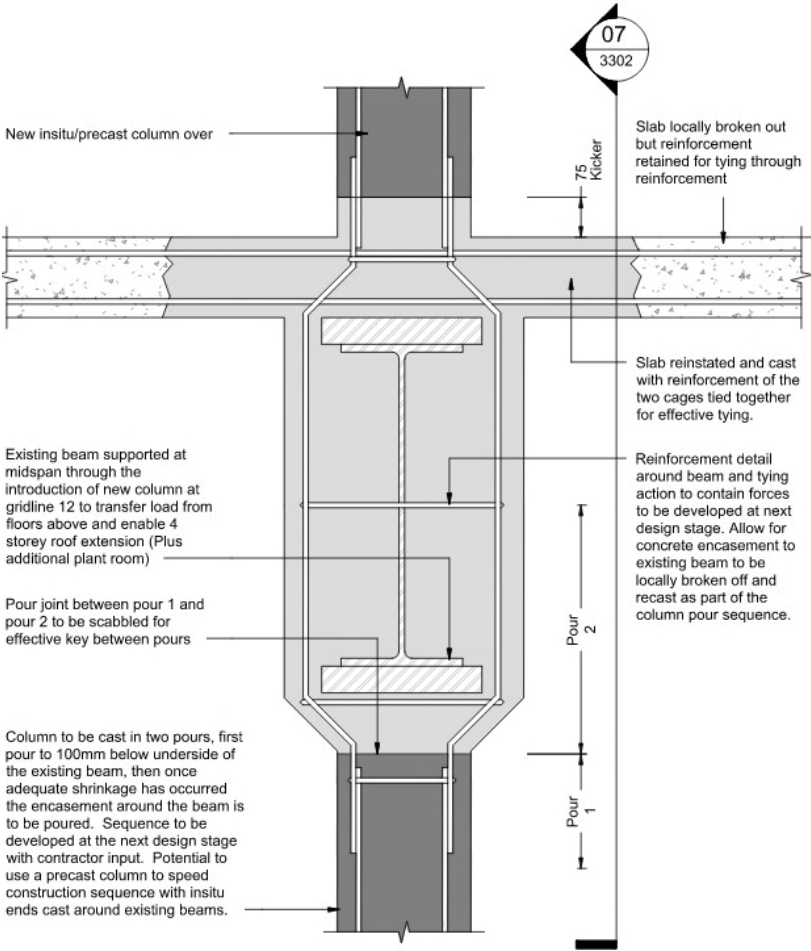


Figure 2.23: Gridline 12 Column Detail

2.15 Roof Extension (cont.)

Visual inspection of the mansard roof shows signs of visual distress and commentary from Quadriga following their inspection on the 26th and 27th May 2020 would indicate that the stonework at the same corner is moving slightly. The mansard area is likely to require significant rebuilding and/or repair and strengthening works. With the location of the additional floors above it has been assumed that the additional column at GL-12 will have a flared head to coincide with the column position for the new roof extension to control deflections for the mansard roof.

The information provided at Feasibility Stage with BIM is far in advance of the tonnage estimates process and arguably provides too much information. The issue is that the original estimates allowed for a certain amount of design development, typically to cover architectural details which were not developed at the Feasibility Stage, for example, upstands, parapets, etc. This is a high volume of steel but represents a relatively low tonnage. Therefore, it will not adversely affect the steel tonnage but will affect the piece count that the tender returns are based on. It is recommended that the contractor allows a contingency to allow for an increase to the piece count as well as some design development and takes into consideration their own expertise in making sensible allowances for both cost and programme.

All steelwork will be required to be to provide two hours fire protection.

Typical Office Floor Plates

The office floor plate will be constructed adopting standard Universal Beam sections. The steel beams will have a number of service holes cut through the beams. The holes will typically be 450 mm in diameter to allow the transfer of the services. In some locations in order to limit the movement of the floor plate, we have installed some fabricated plated steel beams to maintain a minimum floor zone.

In locations where unknown services are required, additional steel will be required to form the openings in the slab. An assessment for the number of holes should be advised by the M&E engineer and an assessment should be taken by the QS.

Office Roof

The office roof has been designed to support specific Air Handling Units concealed within the saw tooth roof with each weighing 12 tonnes. Any additional equipment or increase in loading will unlikely be accommodated.

It is assumed that the flat roofs will have their falls achieved through the finishes and will not require additional steelwork.

Plant space at the roof is also identified over the top of the proposed new concrete cores and these will have the potential to be more flexible if constructed as a new standalone core. No allowance has been made for any associated plant screens for these areas.

All exposed steelwork should be assumed to be galvanized.

Vibration

The recommended dynamic performance criteria for an office floor slab given in the relevant British Standard and SCI documentation is that office floor plates should be designed for an $R=8$, based on a dampening factor of 3%. This is also stated in the BCO guidance 2019.

The BCO Guide states "the values proposed by the SCI have been accepted as best practice for a number of years, giving a good balance between floor performance and structural economy. This has led to some designers to move towards providing a greater floor performance – or lower the multiplying factor- for a typical office; this has dropped to 6 in some cases."

With the above guidance and respecting the tenant's desire for a "premium quality open plan office" generally a $R\leq 6.0$ will be targeted for open floor plates.

2.16 Enhancement of Existing Structural Frame

Desktop assessments of the structural frame based on the historic archive information have established working estimates of what the anticipated loading regime would have been for the original storage building.

However, whilst the introduction of additional floors at the roof naturally changes the loading regime, the anticipated foundation loads are expected to remain within the existing net loading envelope. But with the introduction of additional floors at the roof level, the accumulation of the loads within the columns will be generated differently from the existing building and consequently column strengthening will be required in areas beneath the roof top extension.

Perimeter Columns

For the perimeter columns, typically along Bridgewater Place, it is anticipated from the archive drawings that these columns are largely open steel sections which corresponds with visual inspections in a small number of positions at sixth floor. These columns are showing evidence that they may have been subjected to water ingress through the façade and will likely require some remedial works in addition to the strengthening. The preferred method would be to clean and repair the existing steel and supplement it with additional steelwork sections and anodic/cathodic protection, subject to intrusive survey. It is also recommended that the perimeter columns have the introduction of an anodic/cathodic protection system or similar specialist designed protection system to extend the longevity of the structural frame.

There are two potential column strengthening details that could be proposed for the existing steel stanchions which include either:

- Concrete encasement with suitable reinforcement to create a composite column section, although this may prove challenging with the retention of the pressure of the wet concrete on the existing stone.
- The provision of additional steel to create a compound steel section to increase the overall axial capacity.

There are signs of corrosion on the steelwork within the façade and there is a strong likelihood that repair and remedial details will be needed to the steelwork (including bracketry) within the façade and a long term strategy such as anodic or cathodic protection should be considered with specialist input for the design. If the corrosion is extensive then a more radical intervention may be required to save the parent steel sections before the additional storeys are introduced to the building.

Internal Columns

The majority of the internal columns are believed to be concrete encased steel stanchions but given the period of the building the concrete encasement is likely to have no reinforcement and thus the columns cannot be justified as being composite sections. The concrete encasement was likely being provided solely as fire protection and may offer no justifiable axial capacity.

The options for strengthening these internal columns are to either:

- Remove the original concrete to expose the steel stanchion and then recast them with additional reinforcement in them to create a column benefitting from composite action.
- Providing the quality of the concrete encasement is appropriate following investigations the columns may be able to be wrapped with an advanced composite to contain the bursting forces and thus potentially increase the capacity. This would be a specialist CDP element of work, typically undertaken by a 'jacketing' contractor.
- Through bolting steel sections through the existing columns to create an enhanced cross section area, this would require a high strength grout cuff detail at the head and base of the column cast through the slab to ensure an effective load transfer path. The upstand detail could be contained within the proposed raised access floor.

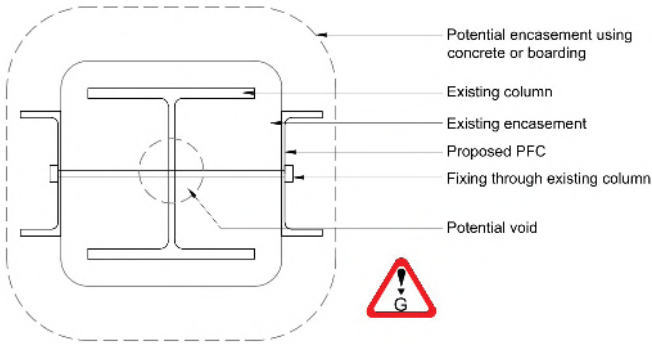


Figure 2.24: Column Enhancement

Column Extension

Where possible at the roof level columns will be extended through with the introduction of a stub connection detail fixed to the head of the existing column.

Along GL-14 and the edge of the single storey extension the columns cannot be located directly over the existing columns underneath without affecting the line of the existing stone parapet and consequently altering the line of the stonework.

In these positions the columns over are to be located as close as practicable to the ends of the existing beams and it is expected that this will influence the shear connection force at the ends of the beam. This will require strengthening and it is proposed that this is undertaken as sympathetically as possible. Historic photographs show that angle supports are used extensively for the steelwork connections and the proposed remediation details would aim to adopt this strategy where practicable with the replacement of the existing angle with a larger angle.

The positioning of the column part way along the existing beam may require the introduction of further strengthening to the ends of the beams. The introduction of the additional vertical line of structure along GL-12 will help reduce the bending moment within this beam but the columns along the rear elevation are also governed by deflection. Following investigation works it may be that these beams also require additional strengthening, and this would be proposed as additional steel sections fixed to the beams and re-encasing in concrete.

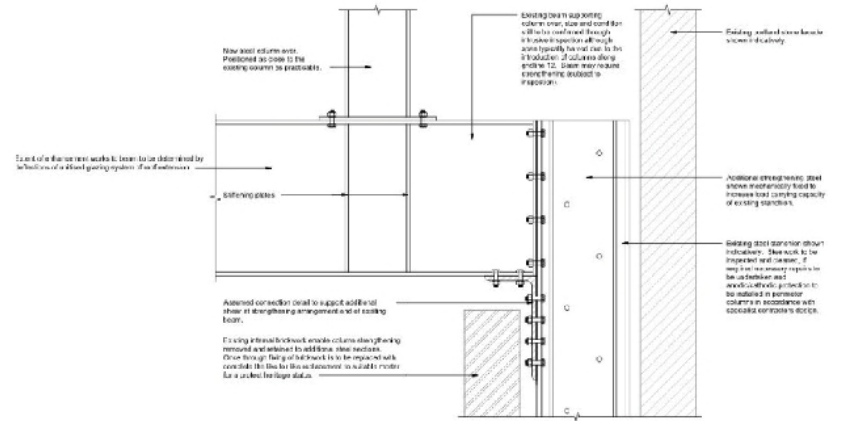


Figure 2.25: Anticipated GL-14 Enhancement

2.16 Enhancement of Existing Structural Frame (cont.)

Fire

All interventions to the frame will require co-ordination with the fire strategy, providing 2 hour fire resistance. The existing frame's fire resistance is still as yet undetermined, and the fire engineer has requested a bespoke assessment is undertaken by a specialist to determine this.

Next Steps

Access to the building has been challenging with the tenant and various surveys (non-intrusive and intrusive) were unable to be completed with a limited visual survey undertaken for the structure.

What has been presented within this report is a general high level strategy for the remediation, repair, and strengthening of the existing structure based on the available desktop information.

No access is available to the building during certain months as part of the tenancy agreement. It is proposed that access is gained as soon as practicable whereby a more detailed investigation will be able to undertaken which will confirm the arrangement of the sections, the condition, and the material properties through testing such as coupon testing of the steelwork which enable the base condition to be more firmly established and specific remediation details to be developed on a case by case basis.

2.17 Existing Façade Issues

As noted previously the façade is typically constructed as a Portland stone façade clad around a steel frame.

Various reports and surveys have been undertaken by the current owner of the building that each identify there are issues associated with the external façade. The purpose of this section is not to add further commentary to the surveys already undertaken by façade specialists. The tenant instructed a survey of the facade to be undertaken by Quadriga in May 2020, to review and ensure the immediate safety of the facade.

It is clear that many of the signs of movement in the façade are in part a consequence of the level of maintenance for the building. This has led to water ingress into the building which would appear to have caused some of the steelwork contained within the façade to have corroded and as a result expanded.

Notably these are the lack of maintenance of the rainwater goods which has resulted in the ingress of water into the building and into the façade.

Reports from Quadriga (May 2020), Stephen Levrant (April 2020), and Sandberg (October 2017) each note that there are signs of movement within the façade and evidence that repairs will be required to the façade. It is noted across the reports that whilst the steel within the

façade has not been accessible for a detailed façade inspection that there is a strong likelihood that there will be corrosion occurring within the steelwork contained within the façade.

This is a common defect commonly known as Regents Street Disease or more locally as Deansgate Disease, where moisture is able to enter the façade and cause the steelwork within the façade, including bracketry and steel sections, to corrode. This can be particularly challenging in areas where the steelwork is inaccessible.

An allowance should be made for the remedial works required for the façade and an initial budget has been allowed for in the report by TGA dated October 2017 although a significant amount of time has passed since this report was commissioned and it would appear that no further maintenance work has progressed. Concerns that further movement has occurred in the steelwork from the assessment undertaken by SLHA has been raised that has resulted in additional inspections by Quadriga.

It is imperative that access to the building is obtained and a combination of camera surveys and opening up works is undertaken by suitably specialists to investigate the condition of the embedded steelwork within the façade to ensure that (if present) the corrosion is halted and any repair works undertaken. The introduction and design of a specialist protection system such as an anodic or cathodic protection system should be considered to continue to extend the life of the building and the primary structural frame.

Any delay in the prevention of this corrosion is likely to continue to escalate the ongoing costs, expediency in rectifying the defects is highly advisable.

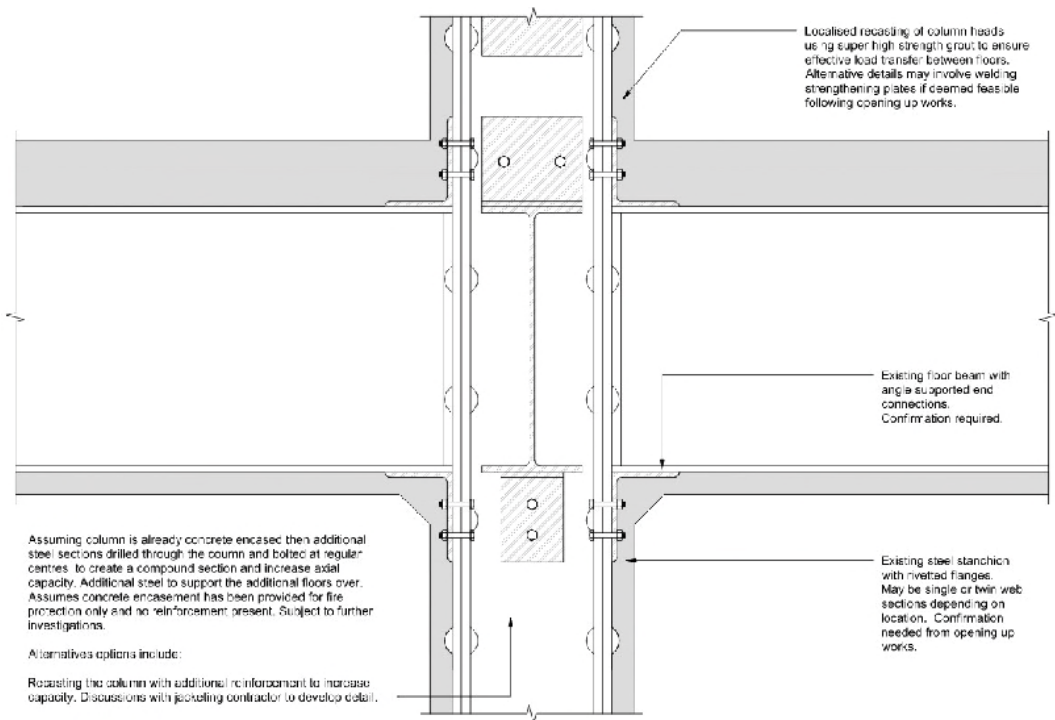


Figure 2.26: Enhancement Detail

2.18 Mezzanine Removal

From the original drawings available for the Rylands Building, mezzanine floor areas have been indicated on multiple drawings. The extents of the mezzanine is different on the 1916 and 1929 drawings.

It is currently unknown whether one, both, or neither of the mezzanine plans were constructed in 1932. The lack of access to the structure has hindered Woolgar Hunter’s knowledge of the building. However, it is currently known that a mezzanine level does span most of the Market Street and Tib Street elevations, as known from the spatial survey undertaken on the structure.

As part of the proposals for the Rylands Building, the extents of the mezzanine will be demolished, as indicated in Figure 2.27. There is no information on the extents of the mezzanine installed by Debenhams. Reinstatement works are potentially required for the extents of the existing mezzanine depending on how it was formed.

After the demolition of the existing mezzanine, any columns which were not initially designed to have restraint from the mezzanine, will no longer be restrained. The extents of these columns will be determined through opening up works in the form of a soft strip. It is believed a soft strip of the existing mezzanine area will inform which portion of the mezzanine, if any, is original or has been added post construction. Columns which originally relied on the mezzanine level for restraint, will have increased buckling lengths in the current proposals for the structure. Enhancement of the columns with an increased buckling length will be required, though this will only be determined once opening up works and inspections have been undertaken.



Figure 2.27: Mezzanine Demolition

2.19 Ground Floor

As noted in the list of interventions, the loading bays at ground floor – which are accessed via Bridgewater Place – are to be demolished to create new levels which are suitable with the current proposals from JBA. Figure 2.28 indicatively shows the proposals for demolishing and reinstating floor which will be at more compatible levels, which are depicted in Figure 2.29.

To achieve the new levels across the floor plate, it is intended that the deconstructed steelwork could be reintroduced at the new required levels to minimise the implementation of new steelwork. The reuse of existing steelwork will be subject to inspection and testing. Where it is not possible to reuse steelwork, a new flooring system will be installed to achieve these levels. For the deconstruction and reinstatement of

structure at the ground floor, this will have to be undertaken sequentially to minimise the requirements of temporary support due to the change in buckling lengths of the columns.

All solutions to rebuilding levels at ground floor, where structure is to be demolished, will require utilising slabs which conform to modern fire regulations. For portions of the floor plate which are required to be altered to achieve new levels, this could be achieved by implementing a lightweight flooring solution such as a raised access floor or expanded polystyrene (EPS).

On Bridgewater Place, it would appear that the external levels fall towards the loading bays and into the building. From inspection of basement level underneath the loading bays, there is evidence of significant water ingress.

Above the existing loading bays on GL-14 there are triple web girders which transfer out facade columns from the above structure. It is not possible to introduce columns under these girders under the current proposals. As such, the girders will have to be suitably strengthened or replaced in order to control the deflections of the proposed roof extension above.

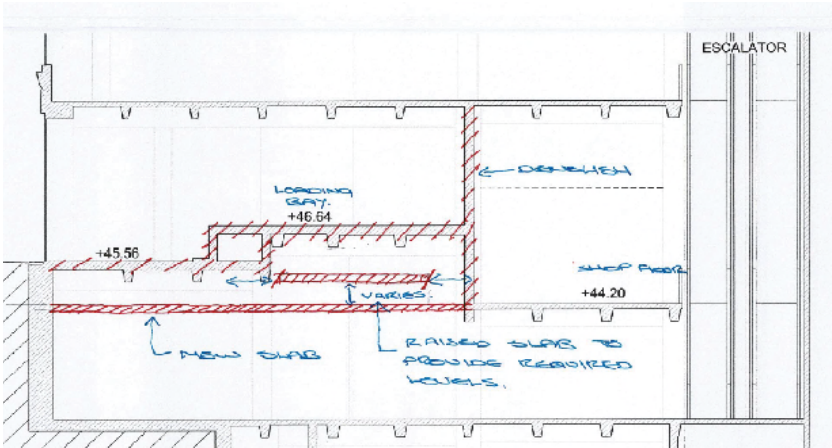


Figure 2.28: Loading Bay Demolition

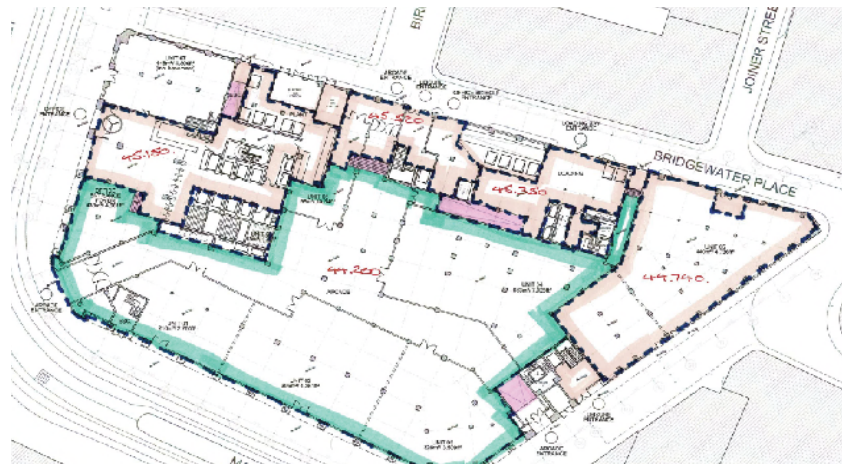


Figure 2.29: Ground Floor Levels



Figure 2.30: Historic Girders

2.20 Infill Floor

Naturally, the proposals for the Rylands Building involve the removal of structure to reconfigure the internal arrangement to create suitable lettable floor plates. Removal of structure, for example the escalators and loading bays, will result in apertures through the floor plates which must be infilled. These apertures through the floor plate will be irregular and will be reviewed on a case-by-case basis to determine the most suitable infill floor solution for each aperture.

It is the intention that demolished structure, once inspected and tested, may be suitable for reuse for the infilling of floor apertures to help with reducing the embedded carbon. Where this is not possible, new steelwork will be integrated into the infill floor solution. The new slabs will be in accordance with modern design codes and are likely to be thicker for the proposed two hour fire protection. A constant top of slab level will be maintained. To achieve continuity between the existing slabs and new slabs, reinforcement should be carried through wherever possible.

Figure 2.31 shows anticipated details which may be used for the infill of apertures through the floor plates. Though, as noted, this will have to be on a case-by-case basis to achieve a suitable solution. The infill flooring details will not be the same across all apertures across all floor plates.

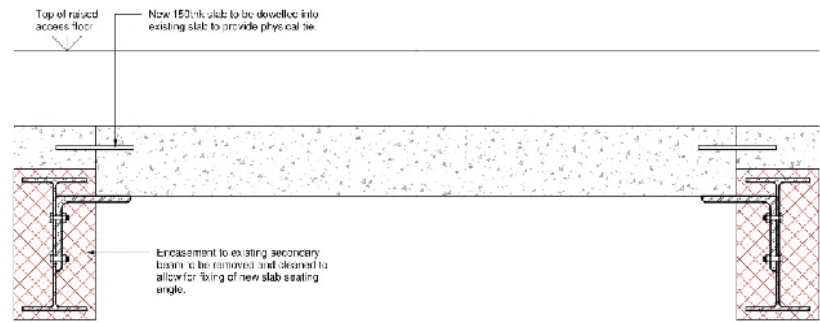


Figure 2.31: Anticipated Infill Floor Detail

2.21 Corrosion Protection

The steel columns which are contained within the Portland stone cladding are assumed to have no corrosion protection. As the masonry is permeable, the steelwork can slowly corrode which produces rust that occupies a larger volume than the original steelwork. The larger volume causes lateral tension in the masonry which causes it to crack. This is a fairly common problem in steel frame buildings of this era and is referred to as Regent Street Disease in London or Deansgate Disease in Manchester.

Typically, it is not economic nor practical to expose all steelwork to sand blast the steelwork and apply a modern method of protecting the steelwork. Remedial anti-corrosion methods, such as anodic/cathodic protection have been mentioned.

In order to determine the extent of corrosion protection to the existing frame, opening up works to inspect the existing primary structural frame will have to be undertaken. This has not been yet possible due to site visitation and works constraints imposed by Debenhams.

3.0 substructure

While substructure information was available from Manchester Central Library archives, there was no information regarding the substructure. A limited investigation was conducted into the existing substructure of the Rylands Building by HTA over three days. The investigation was informative but inconclusive.

3.1 Substructure

Whilst most of the existing columns are within the anticipated net loading criteria based on the historic archive information at the foundation level, there will inevitably be a requirement for additional foundations due to the changing load paths within the frame, introduction of additional floors, introduction of new cores and the associated increase in wind loadings.

A desktop study has been undertaken by IGE that has identified that stiff clay is anticipated beneath the lowest level of the basement and that rock is located within the capabilities of small piling rig. Groundwater level is indicated as being approximately 15 m BGL in the IGE desktop study report.

It is proposed that any new foundations make use of a restricted access piling rig to minimise any excavations and to enable the threading of piles between existing foundations where possible. As there is the formation of a new cycling ramp into the basement it is proposed that the modifications to this ramp may potentially provide access for a limited headroom rig. The use of a mini rig will be the least disruptive with regards the existing building from a heritage and conservation perspective, but this will need co-ordinating and developing with contractor input to align with the programme.

Where possible the new foundations will be threaded in between the existing foundations to minimise disruption to the existing frame. However, areas such as Core 3 will potentially require stepping details at the lowest floor to enable piles to be formed safely around existing lift pits and bases.

For Core 1 an entirely new base is proposed to be constructed as part of the core intervention which will require the removal of the existing foundations as part of the localised demolition. This will enable the new concrete core to be cast up and through the building and then tied effectively into the existing building whilst enabling it to attract the additional wind loading from the extension with the intention to avoid strengthening the 'birdcage' lift in Core 2.

New foundations will be required beneath the new atrium columns, the line of columns along gridline 12 and the stability cores in order to form the lift pits. Core 1 as outlined elsewhere in this report will require the removal of the some of the existing foundations in order to accommodate the new base beneath the cores, in particular the deep lift pits for four of the cores that carry through to the lowest basement level.

HTA have conducted a limited investigation into the existing foundations over the course of three days, due to the limited access imposed on the design team by the current tenants. GPR, drilling, and coring was undertaken on a limited number of foundations due to the time constraints of three days. HTA's investigation into the substructure over the limited three days was informative but inconclusive.

The GPR, drilling, and coring showed there is a variety of foundation

extents for columns. The extents of foundations beyond the base of the column vary, with some being as little as 160 mm and others being as great as 1,855 mm. Moreover, HTA determined a trench between column J-8 and J-9, where there is the proposed line of foundations for the columns supporting the atrium. The drilling to determine the foundation edges shows there is a trench of 4,550 mm between the extents of the foundations, which may allow new piled foundations to be threaded between the existing foundations.

3.2 Basement Issues

There is little desktop information is available with regards the basement and its construction, what is noted within this section of the report is based largely on the evidence of limited walk rounds within the building. No opening up works or drilling could be undertaken due to restricted access by the tenant.

There are areas of water ingress within the basement, although the route the water is taking to enter the basement is unclear. It may be through weak points in the building's fabric or even at higher levels. Depending on the proposed use and subsequent classification of the basement areas it may be worth either creating a buffer zone or providing internal tanking. See architect's details for insulation of the basement.

There is an urgent need to undertake a detailed survey of the basement walls, their reinforcement, thickness, composition, drainage arrangements, further investigation regarding the detailing of the structure and also an intrusive site investigation.

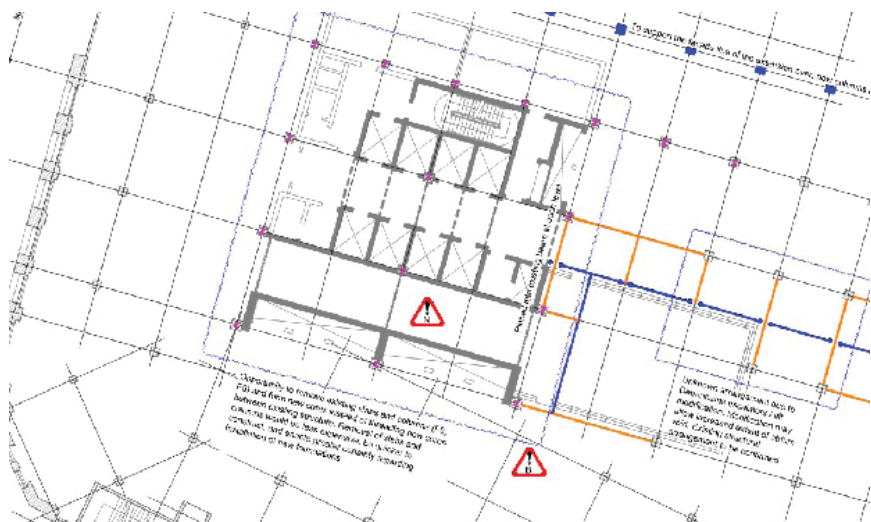


Figure 3.1: Core 1

3.3 Foundation Capacity

The existing foundations to the Rylands building were not found in any historic archival information, and as such, the capacity of the foundations has been limited to match the original design load of the building. Any new additional load over the original has been assumed to require foundation enhancement.

An intrusive survey of foundations was undertaken by Henderson Thomas Associates, which due to limited access was informative but not conclusive due to time pressures. If the foundations can be assumed to be square on plan, and with knowledge of the extents of a few of the foundations, then a ground bearing solution would have likely been feasible and the original building wouldn't have needed to be piled. A second, deeper, sub-basement would also suggest the foundations are likely to be in a more competent stratum.

Digging a second basement would have been challenging and stepping in of the floor plate for the sub-basement may have offered benefits with regards simplification of construction through the earth to be battered back during construction.

A desktop survey of the site by IGE Consulting has indicated that rock is ~15 metres below ground level. Which further lends to the idea that the foundations are ground bearing below the sub-basement.

As the foundation arrangement cannot be adequately concluded, an intrusive survey to determine where the sandstone is located, the extents and founding strata of the existing foundations, and composition is required to develop the foundation strategy further and to enable effective sequencing and co-ordination for subsequent design stages.

4.0 site history and geo-environmental issues

IGE Consulting have provided the geological and environmental insight for the Rylands Building.

4.1 Site History

The environmental and geological insight for the Rylands Building has been undertaken by IGE Consulting and is not within Woolgar Hunter’s current scope of work.

The desktop study indicates that the bearing pressure at the depth of the sub-basement is in the region of 250 kN/m² to 300 kN/m², with rock being approximately 15 metres below ground level.

Refer to the desktop study information from IGE Consulting for the full environmental and geological information.

An intrusive geotechnical investigation as well as extensive intrusive works were originally scheduled as part of the current design stage but were unable to be completed. These are still required for the development of the structural design, particularly the foundation strategy and design.

5.0 sustainability

All Woolgar Hunters designs are considered with sustainability and the environment in mind. We will aim to ensure that our designs work in harmony with the surrounding environment and provide dynamic, considered and efficient designs for future generations to enjoy. Our early use of BIM and 3D design tools allows accurate assessments of structural and civil engineering components.

5.1 Sustainability

The majority of the total developable area for the development is retained from the original building, with the additional floors adding a modest increase in lettable area to improve the commercial viability of the scheme.

Returning a largely abandoned 90 year old building to efficient and economical use whilst retaining a heritage asset will have a positive impact on the operational carbon when compared to a full replacement scheme and this should be seen as a positive.

Whilst the strategy of re-using the already embodied carbon for the existing building and extending the life of the development for many years is sound, there may also be opportunity to look at reducing the embedded carbon on the roof extension.

Whilst the design for the extension is still at an early stage, opportunities for reducing the embodied carbon will naturally mature through the design process. This is particularly relevant given the fluid nature of the economy at the moment with regards free movement of people as a result of Covid-19 and also of materials with the potential impact of Brexit.

Currently the following opportunities have been identified which will positively reduce the embodied carbon within the development.

- Cement replacement (GGBS up to 40%, potential for recycled aggregate but subject to demand).
- Retention and reintegration of existing structure where feasible (technically and economically).
- Procurement of steel from an electric arc furnace which makes use of recycled steel, although typically brought in from mainland Europe.
- Minimising excavation and disruption through the use of micro piled foundations.
- Combining the temporary and permanent works into a single philosophy where possible through careful consideration of the sequence of construction.
- Proposal to use Kingspan Multideck, with over 40% of 'new' steel being made from recycled steel. Kingspan's suppliers encourage, promote, and assist in the return of steel for recycling. Kingspan's EPD for metal decking references the LCA for the decking.

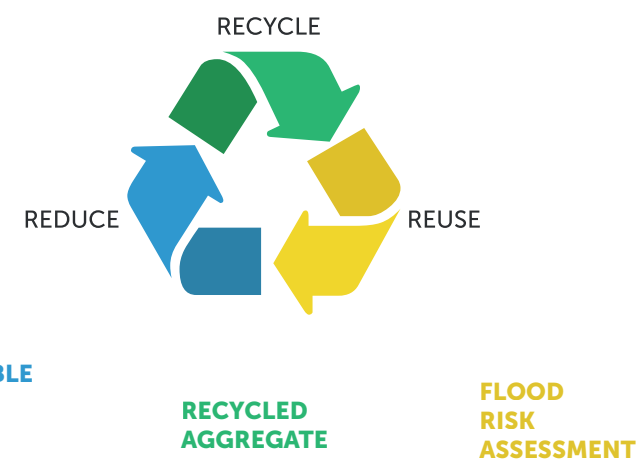


Figure 6.1: Sustainability

6.0 contractor design portion

The following items are assumed to be Contractor Design Portions. The list is generally considered industry standard, and will include items that may currently not be required on the project however it is Woolgar Hunters intention to establish the position from the beginning.

The following items are assumed to be Contractor Design Portions. The list is generally considered industry standard and will include items that may currently not be required on the project however it is Woolgar Hunters intention to establish the position from the beginning.

Woolgar Hunter will provide structural performance specifications and review design proposals to ensure compatibility with overall structural design for the following:

- Bulk earthworks including necessary soil stabilisation/ ground improvement works
- Piling
- Soil stabilisation/ ground improvement works
- Gas protection system (membrane and venting), including independent inspection, testing, and validation
- Steelwork connections
- Timber connections including timber to steel
- Engineered timber roof cassettes
- Precast walls, floors, and stair units
- SFS inner leaf of external walls
- Cladding including all fixings and framework required to connect it to the primary structural frame
- Architectural metalwork including stair, balustrades, handrails, and external link bridges etc.
- General metalwork including access ladders and walkways etc.
- All support systems and fixings plus wind posts to masonry walls
- Pre-stressed/ proprietary lintels
- Proprietary fire protection systems
- Corrosion protection systems
- Secondary timber elements within the cladding system and associated fixings onto the primary structure
- Glazing including all fixings and framework required to connect it to the primary structural frame including curtain walling and rooflights
- The performance of any waterproof additives to concrete mixes
- Pumping stations and rising mains
- Proprietary fencing
- Anodic and cathodic protection and remediation of the existing steel frame and supporting bracketry for the stone façade
- Proprietary cycle shelters
- Thermal/ vibration breaks
- Temporary works

appendix a

woolgar hunter reports and notes

file note

Subject: Site Visit Note
Project: Rylands Building, Manchester
Reference: FN001
Author: N Currie
Reviewer: W Neilson

1.0 Introduction

This file note captures the site walkover visit to the Rylands building, with Colin who is the building manager for Debenhams and Morgan Grennan from Jeffrey Bell Architects.

2.0 Summary.

The building is understood to have been constructed approximately 1930 as a concrete encased steel frame, clad in stone.

Another design team had undertaken a site walkover in the morning and made reference to the building having potential issues with Deansgate Disease, which is a local term for corroded steelwork in the façade displacing the stone.

The building is in close proximity to the Manchester tram network and this will complicate the access to the façade for at least two of the elevations.

2.1 Basement.

Colin noted that there have been issues associated with water ingress in the basement. This is believed to be from several pathways either through ingress from the building further up collecting in the basement, the drains had been damaged by the new development to the rear of the building and this had caused about 300mm worth of water to collect in the basement, or through sections of the basement wall leaking infrequently.

There are internal manholes covers within the basement, one of which was badly damaged and cracked. If the building is re-purposed it would be prudent to survey the drainage and undertake capacity checks for the network given the damage from the site to the rear and the increase in load from new toilets in the offices, residential, and hotel.

The proposal for the basement is car parking, which will require the introduction of a car lift as well as large openings for ventilation and smoke extraction.

2.2 Typical Framing.

The majority of the frame is believed to be concrete encased steelwork, with much of the frame being on an irregular grid. Sections of additional frame have been added to the building to frame new apertures such as the escalators through the building. It was noted in the lowest level of the basement additional columns at half grid positions have been added, presumably to increase the load carrying capacity of the floors above.

Some of the new steelwork that has been threaded through the building has not concrete encased and has instead been clad in a cementitious board.

The columns at the basement level have flared bases to the columns, presumably to help with the punching shear to the basement levels. The new columns within the basements lowest level did not have flared heads.

The formwork timber 'chocks' were still in place at the base of some of the concrete columns and were roughly 2" square, this will impede the coverage of the concrete to the steelwork.

The grillage of beams is highly irregular, both in terms of plan arrangements but also with regards the section size changing throughout the floor plates.

Equally, there was no regular column layout for the columns. The architect has noted that the original as built drawings do not always match with the survey drawings.

The walls around the lift shafts appeared to be formed with red brickwork.

At roof level there are glass block infill sections of roof that have been painted over black for some reason.

There are several large storage tanks scattered throughout the building, including two 60,000 litre tanks for the sprinklers in the basement. There are also large drinking water tanks at roof level in the plant rooms.

2.3 Perimeter Service Trench.



The building is understood to have some form of service trench around the perimeter, this was an issue with the cherry picker that collapsed a section and fell into the hole. Determining the extents of this service trench will be important for developing scaffolding and perimeter details.

2.4 Water Ingress Upper Floors.

At the upper levels, there was evidence of water ingress with some of the perimeter columns in particular. Certain columns had been exposed and there was severe lamination in parts of the steelwork. This may be a concern with the age of the building given that steelwork within the façade will not have been maintained and will not be easily inspected.

There was water damage to the underside of the roof slab in parts, with the board room showing clear signs of water ingress. The roof membrane was heavily bubbled and was noted as being a weak point in the fabric by the building maintenance team.

The concrete slab over the top of the board room had sections of the ceiling exposed and the reinforcement was clearly visible to the underside of the slab.

3.0 Proposals.

There is a substantial package of information available from the architect in summary these include:

Cut an atrium to introduce light to the floor plate as it is currently quite deep. Understood that the client is not in favour of this at the moment.

Equivalent loading will be required to enable the introduction of potentially an extra 5 storeys at the roof level.

The current proposal drawings would indicate that some of the staircores are being shown as being omitted and relocated as part of the proposals.

There is a garden at roof level.

4.0 Potential Surveys Required Moving Forward.

- Drains.
- Existing steelwork.
- Façade inspection.
- Foundations.
- Moisture content.
- Establish the trench extents.

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Ref: NGRC/DN001/19080-6007/09.07.2019

file note

Subject: Site Visit 31.06.20 – 01.07.20
Project: Rylands Building, Manchester
Reference: 196002-FN002
Author: EAJC

1.0 introduction

This file note captures the broad extent of the information gathered from the site visits undertaken by Woolgar Hunter on Tuesday 31 June and Wednesday 1 July. The attendees on site from Woolgar Hunter are Callum Sidley (Senior Engineer) and Edward Cook (Graduate Engineer).

2.0 sub-basement

At basement level, a number of columns are noted to have flared bases and heads in perpendicular directions. The last line of columns with flared bases appears to be along the perimeter of the initial construction phase, noted as Section 1 on the original planning drawings. It is thought that these columns may be providing lateral stability in the temporary state of the building being constructed. It is known from degraded columns at basement level that the flared bases encompass gusset plates. The gusset plate measured at basement level, shown in Figure 1, was approximately 110 mm in width.



Figure 1: Gusset Plate - Basement

The columns accessible at basement level, which are known to have flared bases, are detailed on the appended drawing. The columns with flared bases are denoted with an asterisk.

A total of three manholes were located at sub-basement level. Mitch, TSA staff, was unaware of any further manholes. A drain at high level was also located. However, it is thought this is a low level drain from basement, as opposed to a high level sub-basement drain.

3.0 basement through third

While visiting the site, staff from Debenhams noted there was an ongoing water ingress issue which was believed to be related to the flat roof. The location of this column is B9 – grid reference is in relation to those supplied by Jeffrey Bell Architects (JBA).

At basement level, six manholes were located by Woolgar Hunter – aided by Mitch, who is part of the TSA staff. Three drains were also located at basement level. One of which is adjacent to the sprinkler pump, while the other two are inset in the perimeter wall. The drawing of manhole and drain locations is appended to this file note. From labels found at basement level, it is believed that there are at least 14 manholes at that level. It is possible that there is an asset register, which is perhaps why the manholes are labelled. Though it should be noted that the manholes at sub-basement level were not labelled.

A structural intervention by Debenhams around 1977 saw additional holes cut through the floor plates to make way for escalators. To introduce the escalators, an additional column was introduced which passes through the centre of the escalators. This column appears to be from basement through third, though it is not mapped on the survey drawings. The column is restrained at every floor level parallel to the escalators but is only restrained at third floor and ground floor level perpendicular to this. The perpendicular restraint is shown in Figure 2 and Figure 3.



Figure 2: Escalators - Third Floor

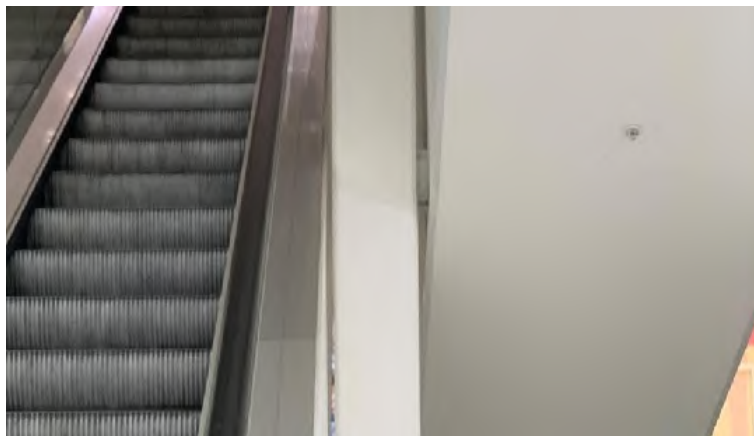


Figure 3: Escalators - Ground Floor

The beam arrangement is hidden at most levels where the floor plates are occupied by Debenhams. However, the beam arrangement is visible on a couple of the floors. Where possible, Woolgar Hunter were able to confirm that the existing beam and column arrangement matches the planning drawings from Manchester Central Library archive.

4.0 fourth

Fourth floor of the Rylands Building is currently in use by Debenhams as back of house to keep storage – currently holding stock for Debenhams in Trafford.

In general, the beams and columns appear to be in reasonable condition except for in some noted locations such as in Figure 4.



Figure 4: Beam Encasement - Fourth Floor

It is unclear whether the missing encasement from Figure 4 is related to previous opening up works, the water ingress above the beam, or another cause.

The most notable exception to the state of the fourth floor is in the northwest corner of the floor plate, where there is significant water ingress. Previous surveys of the structure in 2017 have noted water ingress across the structure, though the extent of the water ingress at the time is unknown. The current extent of the water ingress damage is evident from Figure 5 and Figure 6



Figure 5: Water Ingress - Fourth Floor



Figure 6: Water Ingress - Fourth Floor

The source of the water ingress in the northwest corner of the fourth floor appears to be stemming from the rainwater goods. It has been noted from previous reports that poor maintenance of the rainwater goods has led to plant growth and backing up of the respective pipework. The smell of damp is easily noted in the room.

5.0 fifth

Fifth floor in the Rylands Building is not currently inhabited by Debenhams, though it appears to be the case that remnants of previous stock has been strewn across the floor plate. In comparison to the fourth floor, which is also not public facing, the fifth floor has deteriorated notably.

There appears to be a range of beams and columns with encasement missing in parts, and previous opening up works adjacent to the façade as shown in Figure 7.



Figure 7: Opening Up Works - Fifth Floor

It should be noted that it is unknown if the opening up shown in Figure 7 is related to determining the condition of the structure or the investigation of other aspects, for example water ingress.

There is evident of further lack of upkeep regarding windows, whereby more than one window is propped up using timber and ballast. An example of this is shown in Figure 8.



Figure 8: Windows - Fifth Floor

Additionally, windows which form the lower portion of the mansard roof have been boarded up, presumably to prevent further water ingress in this corner of the building.



Figure 9: Mansard Roof - Fifth Floor

On the internal elevation on High Street, there are a number of boards – some of which are no longer standing – which restrict access to the elevation. On the boards, shown in Figure 10, is the notice of the area being quarantined.

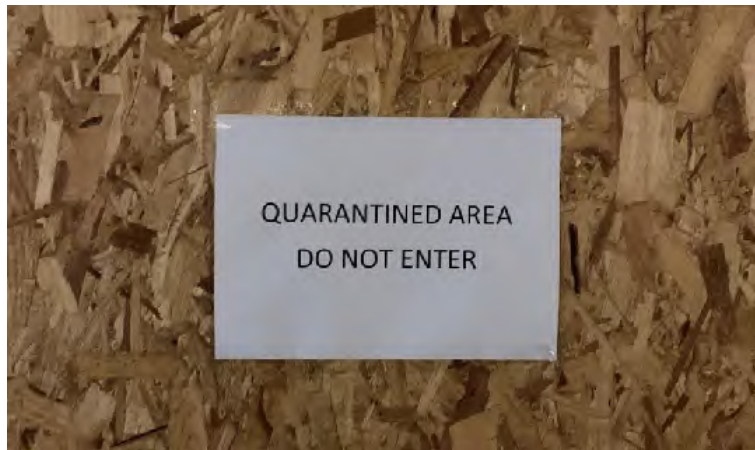


Figure 10: Quarantine: Fifth Floor

Woolgar Hunter do not believe this specifically relates to asbestos being present, as other areas of the building have specific signage to inform personnel of asbestos containing materials being present.

6.0 sixth

Sixth floor is in worse state of repair compared to the lower floors. The smell of damp is present across much of the floor plate, due to extensive water ingress across the floor plate – primarily around the perimeter.

Adjacent to the mansard roof, a column – shown in Figure 11 – has suffered extensive cracking over more than half the column stack.

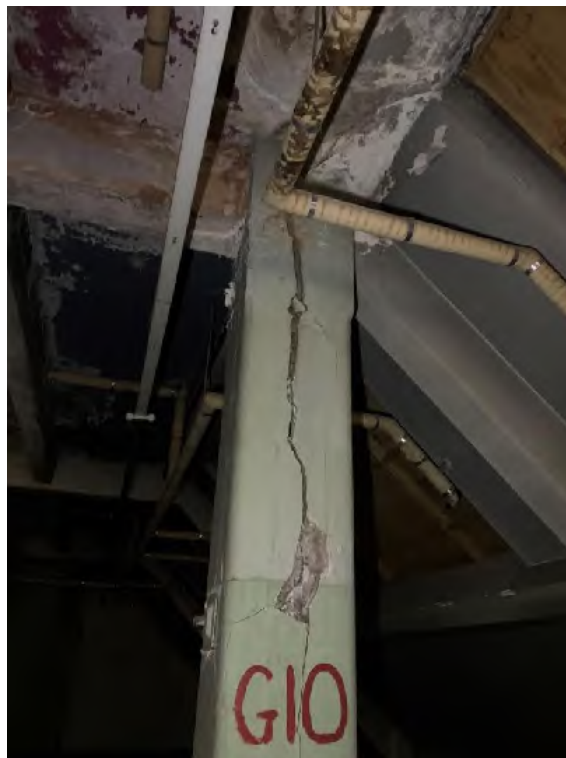


Figure 11: Column Cracking - Sixth Floor

It is possible that the encasement may have partially fallen away from the steel column inside, so it is unknown if the extend of the corrosion is exaggerated by the width of the crack.

The past water ingress is evident across much of the underside of seventh, with numerous locations whereby the slab reinforcement is fully exposed. Two examples of this instance are shown in Figure 12 and Figure 13.



Figure 12: Exposed Reinforcement - Sixth Floor



Figure 13: Exposed Reinforcement - Sixth Floor

There is further water ingress and corrosion in the toilet areas, which has been noted in the previous reports available to the design team.

7.0 seventh

In a similar manner to sixth floor, exposed reinforcement is also evident at seventh. One such example is in the office with the parquet flooring, which is shown in Figure 14.



Figure 14: Exposed Reinforcement - Seventh Floor

In the kitchen area, it is noted that there are raised tiles which form a 90 degree angle. It is thought this might be due to two structural elements converging or the expansion of a submerged element causing the ridged tile lines.



Figure 15: Tiled Floor - Seventh

Regent Street Disease (or Deansgate Disorder) is evident along the internal elevation on Market Street on multiple columns. Figure 16 displays an example of one of these columns whereby the cracking is the full height of the column stack.



Figure 16: Deansgate Disorder - Seventh Floor

Around the perimeter of seventh floor, it is evident that the waterproofing is no longer providing a barrier around the full perimeter. An example of the failing waterproofing is above the mansard roof. One portion of the failed waterproofing is shown in Figure 17.



Figure 17: Mansard Roof – Seventh

The extents of the failed waterproofing continues beyond the photograph in Figure 17.

As noted in a previous structural inspection by Sandy Porter, there is extensive cracking in the bitumen covering which has been laid across seventh.



Figure 18: Bitumen - Seventh Floor

There are several instances where the roofing appears to be harbouring water, particularly around the heating, ventilation and air conditioning units which are situated at several locations across seventh.

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Ref: EAJC / 196002-FN002 / Rylands Building, Manchester / 02 July 2020

structural report

The Rylands Building, Manchester
CD9 Properties (Manchester) Limited

Woolgar**Hunter**
engineers

Project Title: The Rylands Building, Manchester
Client: CD9 Properties (Manchester) Limited
Project No: 196002

version	date	description	author	reviewed by
V1.10	04.09.19	first issue	Alexander Porter	Neil Currie

Note: Version Status:
V1.00 series – draft issues
V1.10 series – formal issues

1.0 introduction

- 1.1 Woolgar Hunter were instructed by CD9 Properties (Manchester) Limited to inspect the property and to prepare a structural report.
- 1.2 The property is currently in use as a department store.
- 1.3 The building is Grade II Listed and dates from 1932. It was originally constructed as a textile warehouse with retail at ground floor level.
- 1.4 Unusually for a property of this age, there is a full set of construction photographs. These seem to indicate that the building was constructed in two consecutive phases. There is also a set of original floor plans which unfortunately do not give any structural information.
- 1.5 The present use of the building hindered the inspection of the structure at basement to third floors.

2.0 general description

- 2.1. The property is a large floorplate multi-storey building comprising seven storeys plus a double basement. The top storey, facing Market Street, is set back from the main elevation and of reduced width with feature domed towers at each end. There are isolated plant room structures behind this top storey. The lower sub-basement is of a reduced area, possibly to facilitate the original construction. (See figures). At the corner of Bridgewater Place and High Street the elevations of the top two storeys slope inwards resulting in a reduction in floor area. The sloping elevations are covered in roofing felt, probably due to historic water penetration.
- 2.2. The typical floor to floor height is noted on original architects' drawings as 13 ft (3.96m). The GF storey height is noted as 20 ft 9 inches (6.32m) and has a part mezzanine adjacent to the Market and Tib Street elevations. The basement and sub-basement storeys are given as 12 ft (3.67m) and 11 ft 9 inches (3.58m) respectively.
- 2.3. The building has a framed structure and lateral stability is probably provided by the various lift and stair cores, aided to some extent by the external elevations.
- 2.4. The elevations are clad with Portland stone and mass brickwork backing. It is unlikely that there is any cavity in the external walls.
- 2.5. There is a bank of loading bays at the rear of the ground floor, accessed from Bridgewater Place.
- 2.6. Given the age and use of the property, it is likely that the design imposed floor load was at least 1 cwt/sq. ft (5.3 kN/m²) and probably greater.
- 2.7. It is apparent, from the historic photographs, that the building frame is hot rolled steelwork with reinforced concrete floor slabs.

3.0 observations

- 3.1 The floors, where access was available, comprise 150mm reinforced concrete slabs, spanning 2m or thereby onto a system of primary and secondary beams (photographs 1 and 2). In the basement storeys, there are lines of secondary columns, presumably to cater for a higher ground floor imposed load (photographs 3 and plans). There is a larger grid on the seventh storey (photograph 6). Slab thicknesses may vary on different floors.
- 3.2 The steel frame is concrete encased (photograph 4). The exception being the columns on the elevation which appear to be incorporated into the masonry (photograph 5)
- 3.3 There is some cracking on the feature columns of the main elevations (photograph 5 and 7 to 9) along with areas of weather staining (photographs 10 to 12).
- 3.4 The downpipes on the Bridgewater Place elevation are leaking and are likely to be blocked where they turn back into the building at first floor level (photograph 13). One or two loading access doors have been cut into the rear elevation (photograph 14).
- 3.5 The mastic asphalt surface of the roof is in poor condition and is badly cracked with numerous rooflight areas (photographs 15 to 17).
- 3.6 There are numerous rooflights all of which are covered with roofing felt. The rooflights are of the traditional flat type with small glass squares (photograph 18 and Roof Plan).
- 3.7 Within the top storey, there is a vertical cracking on the line of some of the columns on the Market Street elevation. At one location this can be seen to be due to severe corrosion of the steel column (photograph 19).
- 3.8 Within the sub-basement level, the primary columns have splayed bases, probably being the encasement of steel gusseted baseplates (photograph 3). The secondary columns appear to support only a single floor of loading and subsequently require a smaller baseplate with no gusseting evident in comparison to the primary columns.
- 3.9 Excluding the retail areas, there are a number of different floor finishes such as tiles, parquet and cork, which appear to be laid on a 25mm concrete screed.
- 3.10 There are a number of roof leaks which have caused damage to internal finishes including concrete to reinforcement and structural steelwork (photograph 20 and 21)
- 3.11 Windows are single glazed and steel framed.
- 3.12 There are modern structural interventions such as the installation of escalators which have additional subframes to create the necessary voids. It would also appear that much of the mezzanine floor has been a modern addition.

4.0 discussion

- 4.1 The property consists of a substantial concrete encased steel frame with reinforced concrete floor slabs. However, it is apparent that maintenance of the building envelope has been neglected for a number of years. This should be rectified and will involve cleaning and repairing the stonework as well as cleaning and repairing the rainwater goods. Any retained roof finishes, after alterations, will also need replaced.
- 4.2 The perimeter steel columns are built into the external masonry and have no anti-corrosion protection. As the masonry is permeable, the steelwork can slowly corrode. The corrosion product, rust, occupies a considerably larger volume than the base metal which causes lateral tension in the masonry with consequential cracking.
- 4.3 This is quite a common defect in steel framed buildings of this era and is referred to as Regent Street disease after the area in London where it is most common (also parochially as Deansgate Disease in Manchester).
- 4.4 In terms of remedial works, it is not usually economic or practical to expose the steelwork in order to grit blast and apply a modern anti-corrosion coating. The usual approach is to employ cathodic protection, whereby sacrificial anodes are affixed to the steelwork and a small electrical current is passed through the steelwork. Further intrusive investigation will be necessary, from inside the building, in order to identify the extent of the problem and the solution.
- 4.5 The concrete encasement of steel frames was a historic method of providing fire protection, however, in buildings of this era the encasement does not meet modern standards for fire protection. Likewise, the concrete cover to reinforcement in slabs may not meet modern standards.
- 4.6 Where a change of use of the building is proposed and for a building of a height in excess of 30m from ground level to the top floor, a likely period of fire resistance of 120 minutes is likely to be required. The confirmation of the fire design period and suitability of the existing frame will require confirmation by the Fire Engineer.
- 4.7 The steel and reinforcement in buildings of this era are susceptible to water penetration as the concrete protection is affected by carbonation which reduces the alkaline protection of the cement paste thereby allowing corrosion in the presence of moisture, with the consequences noted in 4.2 above.
- 4.8 It is understood from the proposed architectural drawings that parts of the vertical structure and areas of floor are to be removed for the development of . This should be relatively straightforward providing the lines of existing structure are used as guides for proposed removal.
- 4.9 It should be noted that removal of substantial parts of the structure should ideally be done after insertion of new structural elements, which will require careful co-ordination with the temporary works design.
- 4.10 The introduction and/or removal of substantial parts of the structure will incur a significant additional cost if tenants are to remain in place compared to the modifications being undertaken in an unoccupied building.
- 4.11 The cracked stonework on the elevations will require close inspection in order to establish proposals for necessary remedial works. It is, however, likely that stainless steel pinning and resin repairs will be required.

5.0 conclusions and recommendations

- 5.1 The building has a substantial structural frame which should be suitable for loads resulting from the proposed change of use.
- 5.2 Intrusive investigations will be necessary in order to determine the extent of corrosion of the steel columns in the building elevations and to establish the need for remedial works. This will have to be undertaken internally due to difficulties and restrictions of external access.
- 5.3 Remedial works will be necessary to the building envelope with respect to the following:
 - Weathering of stonework and removal of dirt.
 - Cracking of stonework.
 - Roof leaks and deterioration of surfaces.
 - Blocked drainage.
 - Defective roof lights.
 - Inclined facades.
- 5.4 A review of the fire protection to the frame and floors consequential to the change of use should be undertaken. A fire engineering consultant should be appointed in this respect.
- 5.5 Depending on the proposed use of the upper floors, improvements to glazing and insulation may be required.
- 5.6 It is recommended that a specialist inspection of the windows be undertaken.

limitations and constraints

- a) This report is intended for the use of the named party. It is not intended for use by third-parties in terms upon which it can be relied.
- b) No intrusive investigations have been undertaken to identify the nature or condition of hidden structural elements.
- c) No boreholes or trial pits have been excavated in order to identify the nature or condition of foundations or bearing strata.
- d) This report does not constitute a specialist report on the presence of asbestos containing materials or timber decay.

figures - courtesy of Jeffrey Bell Architects

sub-basement plan
basement plan
ground floor plan
mezzanine plan
first floor plan
second floor plan
third floor plan
fourth floor plan
fifth floor plan
sixth floor plan
seventh floor plan
roof plan
long section
cross section

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