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Structural Appraisal Report for 1830 Warehouse – Internal Inspection

Museum of Science & Industry, Manchester

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1.0 BRIEF

1.1 Conisbee were appointed by our client, the Science Museum Group to undertake a detailed visual inspection of the internal structure of the 1830 Warehouse in order to record the condition of the building fabric and structure and to propose appropriate remedial solutions to address any notable defects or deterioration.

1.2 Scott Brookes and Simon Wilkinson of Conisbee visited site on 2nd November 2018 and carried out the visual inspection of the structure.

1.3 No fixtures or fittings were removed or intrusive/destructive testing undertaken, however comprehensive photographs were taken for record purposes.

1.4 Whilst our investigation work has been taken far enough to satisfy the requirements of the brief, it has, of necessity, not been exhaustive. The findings cannot therefore be warranted to apply to areas of the building not inspected or investigated.

1.5 The building fabric has been assessed as observed at the time of inspection and the condition may be subject to change between the time of recording and commencement of works on site.

1.6 This report is intended for the use of the client, the Science Museum Group, and no liability can be accepted for use by any third party.

1.7 In summary, the requirements of our involvement are two-fold –

- Establish that these structures are, and will remain, suitable to support the floor, roof and walls both in its current arrangement and following the upcoming re-roofing works.
- Appraise the material of the structure to understand what maintenance or remedial works are likely to be required in the short and long term.

1.8 This report should be read in conjunction with Hutton & Rostron's timber condition survey report.

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2.0 PROPERTY TYPE, CONSTRUCTION & CONTEXT

2.1 The 1830 Warehouse is Grade I listed for its special architectural and historic interest. The identity of the architectural designer remains speculative with both George Stephenson and Thomas Haigh being suggested as candidates however it is known to have been constructed by contractor David Bellhouse Jr in 1830. It is thought to be the world's first railway warehouse building and also forms a contiguous part of the world's first railway station, located on the opposite side of the viaduct to the south.

2.2 The building is slightly curved on plan, presumably to follow the line of the railway viaduct to south to which it abuts. For the purposes of this report, the elevations are assumed to be orthogonal about the compass axis. The building has 4 main storeys (inc. basement) but with an intermediate 'mezzanine' floor between Ground and 1st at bays 1 and 2, only. Working top down, the floors will be identified as follows –

- Roof
- 2nd Floor
- 1st Floor
- Mezzanine Floor
- Ground Floor
- Basement

2.3 At roof level, the building presents as 10 individual gable-ended bays each with dual-pitched roofs with ridgelines running approximately north-south. The building description presented in this report however follows the internal structural layout which is formed of five double bays each separated by load-bearing brickwork walls. The bays are numbered from 1 to 5 running west to east for ease of reference.

2.4 The structure generally comprises load bearing red brick walls laid in lime mortar in Flemish bond which form the outer envelope with brickwork internal cross walls running north-south and separating each bay. The internal structure within each bay (including the roofs) comprises traditional cut timber framing. Queen post trusses span between bay cross walls and beams and columns providing support at the central valley of each bay. The trusses support 2 no. purlins to each pitch supported on cleats and embedded in the gable wall masonry. The queen post trusses have a small king post from the straining beam up to the ridge plate.

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2.5 Dimensions of timber roof elements as follows (depth x width) –

- Queen-post trusses
  - Principal rafters 210mm x 120mm
  - Tie beam 350mm x 180mm
  - Straining beam 220mm x 130mm
  - Queen posts 190mm x 120mm
  - Struts 80mm x 80mm
  - King post 200mm x 80mm
- Principal members
  - Upper purlin 240 x 150mm
  - Lower purlin 240 x 150mm
- Valley gutter
  - Upper gutter beams 250 x 100mm x2 (460mm total)
  - Lower beam 200 x 300mm
  - Propping posts 220 x 220mm
- Secondary timbers
  - Rafters 75 x 65mm @360mm ctrs.
  - Battens 50 x 25mm

2.6 The vertical load from the roof trusses is transferred to the cross walls, gutter beams and regularly spaced timber posts down to 'yard level', then into cast iron columns at basement level and ultimately the building's foundations.

2.7 Externally the bays on the north-east elevation generally follow a pattern of having either one or two full height loading-bay openings in the brickwork with either single or paired window openings at first and second floor level. Hoisting equipment to lift goods into the warehouse at each level was installed at the top of each slot opening and although no longer operational, remnant elements are present in most bays.

2.8 Hoisting equipment is also assumed to have been present historically on the south elevation but was largely removed at some stage. The date of these works cannot be definitively confirmed but appraisal of archive photographs confirms that they were pre-1955.

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2.9 Hutton & Rostron undertook an inspection of internal structural timbers at the south elevation of Bay 4 in November 2015. This report addresses distortion and cracking in replacement timber lintels, beams and joists (presumed installed with the previous 25- 30 years) and also notes that cracking has occurred in surrounding masonry following drying shrinkage of unseasoned timber. Recommended remedial actions include moderating heating levels within the building to reduce excessive drying, and thereby permit slow partial rehydration and expansion of the affected timber. Following achievement of a more normal moisture content for timber in this use, re-pointing of cracked masonry to re-establish contact was recommended. It is not known whether action to adjust the internal environmental conditions has been taken, however there was no visual evidence of crack repairs or re-pointing was found during our recent inspection.

2.10 Hutton+Rostron carried out a further damp and timber decay survey at the roof frame (only) on 6-7 November 2018 in accordance with instructions received from Naomi Atherton of SIM. The findings of this survey have been taken into consideration in this report, but reference should still be made to its findings, in conjunction with Conisbee's.

2.11 Extensive repair works are understood to have been undertaken in around 1992 as part of a project to convert the building into a gallery space. The exact nature of these works cannot be fully confirmed through reference to archive materials but internally appears through visual inspection to have included extensive timber repair and replacement, including to the bearing ends of tie beams, principal rafters and purlins together with replacement of roof finishes and leadwork externally. Internal frame repairs include fabricated stainless steel flitches with through-bolted connections and scarf joints. Whilst the repairs are widespread, a number of compromised timbers still remain.

3.0 INSPECTIONS & REPORTING

3.1 Our inspection methodology involved a visual inspection of the internal building structure, gaining hands-on access wherever possible using a ladder to record and photograph each individual defective or deteriorated item or area. This provided a comprehensive record of the condition of the internal frame.

3.2 No safe access was possible to roof level for inspection of structure or finishes.

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4.0 OBSERVATIONS

Annotated floor plans of the below observations will be provided following receipt of measured survey drawings.

4.1 BRICKWORK ENVELOPE

4.1.1 Refer to Photos 1 to 13 in Appendix A.

4.1.2 Various 'fine' to 'moderate' vertical and diagonal cracks were noted at the cross walls of the building, particularly beneath lintel bearings at door penetrations. Some of these cracks have been filled with a cementitious grout mix.

4.1.3 Macro distortions were observed at the north elevation where brickwork was noted as bulging outwards at first floor level either side of the tall loading bay penetrations. The bulge is in the vertical plane between the top and bottom of the building, with the wall locally bending from yard level upwards. Rot of a length of timber 'quoin' exposed the sectional build-up of this North wall and revealed that the 2 skins of brickwork were very poorly bonded.

4.1.4 The floor joists at each level largely run longitudinally, parallel and clear of the north and south long elevations, bearing onto cross walls between each bay and perpendicular primary beams (ridge and valley). These north-south spanning primary beams bear into pockets in the wall and along with the cross walls, provide intermittent restraint to the long elevations. Shrinkage of the timber appears to have reduced the bearing length of these embedded beams, but none to a concerning degree.

4.1.5 Vertical and diagonal cracking was noted at various locations up the inside face of the piers either side of the tall doors at the North elevation, which possibly suggests high localised stresses. Historic crack filling was evident suggesting that the issue may be ongoing.

4.1.6 The inside face of the West end elevation is a patchwork of recovered, possibly reused stone and bricks, with significant variations in the keying and coursing of the masonry over its length. No cracking or distortion was evident and whilst representative of an historic poor repair, the panel seems to be performing adequately.

4.1.7 A substantial panel of infill brickwork at Bay 3 on the North elevation appeared poorly tied into the surrounding structure and to have moved in the past, allowing a gap to form through which clear daylight could be seen.

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4.1.8 Inspection around the former pulley openings on the South elevation shows that later infill brickwork is not fully bonded into the main thickness of the wall and therefore appears to be supported mainly on the timber lintels which have been subject to creep and deflection under long-term loading resulting in cracking to the brickwork above. This is structurally unsatisfactory and now requires remedial action.

4.1.9 Distortion and cracking was observed in replacement timber lintels, beams and joists (presumed installed with the previous 25- 30 years) and cracking in surrounding masonry following drying shrinkage of unseasoned timber.

4.2 MOISTURE INGRESS

4.2.1 Refer to Photos 14 to 16 in Appendix A.

4.2.2 Moderate to severe staining at top floor level was noted, localised to the top and sides of the valley beams and to the plywood and tongue and groove gutter timbers, suggestive of historic valley gutter blockage, rather than liner failure. Areas were not wet to the touch at the time of inspection but the integrity of the gutter and/or rainwater goods was not assessed.

4.2.3 There appears to have been a history of water penetration via the gable ends and many of the purlin bearings have been repaired as a result of timber decay. Salt crystallisation on the internal surface of the gable brickwork is indicative of a history of wetting and evaporation. These repeated cycles of dissolution/crystallisation have caused stresses within the brickwork, causing some minor spalling of brick faces in these areas.

4.2.4 Moisture staining at lower levels of the timber frame was not observed.

4.2.5 South elevation at track level - moisture and severe salt leaching evident at the inside face of the wall directly abutting the track. The extent of permeation has allowed a salt crust to form. We know from documentary evidence that a more recently installed concrete gulley runs directly adjacent to this, at this very level, between the warehouse building and the track. Should this gulley be undersized or damaged, water would be able to frequently saturate the brickwork.

4.3 TIMBER CONDITION

4.3.1 Refer to Photos 17 and 18 in Appendix A.

4.3.2 Reference should be made to Hutton & Rostron's timber condition report on the roof frame. Conisbee's observations extend to notable defects that were considered structural in nature.

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4.3.3 The valley gutter in Bay 4 is partially decayed through its upper 50mm for approximately 1000mm at its northern end. The eastern tie beam at this location was also decayed at its bearing. Repair required.

4.3.4 The southern end of the valley gutter in bay 4 was decayed for approximately 500mm and the bearing connection of the principal rafter was also decayed. There was also partial decay in the valley beam directly beneath. Repair is required.

4.3.5 Substantial section loss at the corners of an original post within Bay 1 at mezzanine floor level was observed. This is not seen elsewhere in the building and the frayed surface of the timber suggests that contractors may have hacked away timber which they thought defective. This is not deemed to be structurally significant and no structural repair works are required.

4.3.6 Severe shrinkage of ground floor joists in Bay 2 has led to significant splitting along the side and underside faces, rendering the joists unfit for purpose. The joists are currently supported on acrow-props at basement level and will need to be replaced.

4.3.7 The tie beam (bottom chord) of a primary truss in bay 2 has failed, and has since been propped with SHS posts. The failure is at the position of a knot, which is a natural weak point.

4.4 METALWORK

4.4.1 Refer to Photos 19 to 24 in Appendix A.

4.4.2 Minor surface corrosion was apparent on the horizontal bearing face of the basement level cast iron column heads. No significant section loss was evident and no remedial works are required, although the shoe is without weep holes and moisture could get trapped.

4.4.3 The lifting of a loose basement floor cobble at the base of a cast iron column column allowed sight of surface corrosion to an unpainted length beneath floor level. This may suggest that corrosion is typical to similar unpainted length at other columns, however the level of exposure to moisture and hence corrosion could be hugely variable. Further investigation is required.

4.4.4 Historic wrought iron bolted strapping which is tying corbel timbers to primary beams close to the bearing point was often seen to be loose and in need of tightening to varying degrees.

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4.4.5 A now redundant crane at track level on the South side of the building passes through the floor plate and is sat atop the timber floor structure at the mezzanine floor beneath. The crane looks to be in adequate condition although hands-on inspection of the upper levels could not be made. No distress in the floor carrying the weight of the crane was noted.

4.4.6 A heavily corroded downpipe penetrates through the inside face of the South wall at basement level. There is no evidence of salt deposits or tide marks around the opening which may suggest the penetration doesn't pass through the full thickness of the wall, but the pipe itself is exhibiting moderate to high levels of corrosion and further investigation is warranted to determine its functionality.

4.5 TIMBER FRAME

4.5.1 Refer to Photos 25 to 39 in Appendix A.

4.5.2 Significant deflection of the bottom chord of the primary longitudinal roof trusses was recorded throughout the building, suggestive of under-sizing and/or overloading. The deflection is most pronounced at Bay 2, where there has been historic overloading of the trusses during goods movement into and out the warehouse. Significantly sized lifting beams are secured to the side of these tie beams and running north-south between trusses. In1No. location, the bottom chord has actually snapped (see 4.3.7).

Square Hollow Section steel columns (4No) have been installed in recent times to prop this failed truss and the adjacent truss to the North, which receives the other end of the old lifting beam. Tracking the location of these SHS props through the floor, we noted that the columns are sat on the boards between joists, and away from a primary North-South beams. Whilst no failure is evident in the boarding, minor stress fractures were seen in the joist directly below the prop that is beneath the failed truss, and this arrangement should be reviewed to transfer the roof loads back to adequately sized structure.

It is evident from the number of lifting beams and the amount of historic propping to trusses that Bay 2 was, at least in the latter days of the functioning warehouse, being used to bring the heaviest goods from the tracks.

A large hoist again utilises the bottom chord of these trusses and spans perpendicularly between them before running out over the tracks at the South elevation.

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4.5.3 The floor levels vary hugely within each bay, with distortions in the plane of the floor easily visible to the naked eye and able to be felt when walking around each floor. This local change in floor joist level is dictated by the level of their bearings at cross beams and ultimately, the level of the column supports. The magnitude of this variation appears consistent across the floors but this is to be confirmed upon receipt of the measured survey drawings and spot levels.

4.5.4 There was a noticeable southward lean of the timber frame at the top floor level, with rotation of columns seemingly about floor level. Whilst seen to a small degree in the primary columns running down the spine of each bay, it was most clearly evident in the small mezzanine frames at the north and south sides of every bay.

4.5.5 Due to the curvature of the building, both sets of roof rafters meet the ridge plate at different angles. In a perpendicular building, the rafters would typically apply an equal and opposite thrust, but this arrangement could be creating a net horizontal force down the line of the ridge in the southwards direction, exacerbating the frame lean.

4.5.6 The timber floor framing at the 1st floor typically, visible from the mezzanine floor under, utilises a side fixed corbel onto which the 1st floor primary beam dead-bears. Shrinkage and frame racking has pulled the timber off this bearing and now very little contact remains. 'Telltale' movement monitoring gauges have previously been installed at this beam support during previous assessment, but no movement has been recorded.

4.6 FOUNDATIONS

4.6.1 Inspection at basement level showed considerable variation in the cast iron column's foundations, with some on monolithic stone bases, some on what appeared to be cast concrete pads (possibly underpinning) and some with the column continuing below the level of the cobbles. Lifting of a loose cobble close to the base of the column did not expose the column's termination point.

4.7 WINCHES

4.7.1 Bay 3 north side top floor; a pulley beam has significantly rotated such that the back span was lifted from the mezzanine. A single holding down nail was still in the beam.

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5.0 DISCUSSION

5.1 Following Conisbee and Hutton & Rostron's on-site inspection work, the assessment of the structure adhered to the following process –

- Review historical and chronological context
- Understand the performance of the structural system (construction, load transfer, stability).
- Review the condition of structural members and investigate the causes of any defects discovered.

5.2 We have structurally modelled elements to aid our understanding of the structural behaviour of the frame and have undertaken a design check to assess its capacity.

5.3 Hutton & Rostron's 2018 timber species assessment provided the basis for our structural assessment of the frame –

- Roof framing - Softwood Scots pine (Pinus sylvestris)
- Posts/Columns - Hardwood Oak (*Quercus spp) OR* Hardwood Sweet Chestnut (*Castanea sativa)*

5.4 The broad approach of the repairs and remedial work proposed is to attempt to prevent future deterioration based on a minimum intervention approach in line with sound conservation principles. The aim is to retain as much of the existing fabric as possible and to incorporate good conservation maintenance, for example by minimising water ingress into the facade and to repair currently defective elements using compatible techniques and materials that will not compromise the long-term health of the building structure and decorative features.

5.5 The extension of our involvement to assess the internal frame was to allow consideration of the building structure as a whole and determine if the configuration or behaviour of the internal frame had any bearing on the defects observed at the external brickwork façade.

5.6 The vertical loads from the roof such as finishes, snow and roof access are transferred to the roof trusses and in turn are passed through the cross walls and through regularly spaced timber posts down to 'yard level' and into cast iron columns at basement level. The lateral stability of the structure is maintained by the stiffness of the envelope walls in the longitudinal direction and the regularity of the cross walls, transversely.

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5.7 Generally, the timber frame is sound and not unduly concerning from a structural perspective. The speed with which this building was erected is evident in the form of rudimentary jointing and the magnitude of unchecked drying shrinkage that in some cases has caused knock-on issues such as reduced bearing lengths. Given the speed of the build, it is also no surprise to see that the timber size and species selection was quite varied, with the roof timbers generally slightly undersized and likely selected on availability, rather than suitability. The lack of construction coordination between the erection of the brick skin and the internal frame has also caused issues.

The curvature of the building on plan is typically rectified in the timber frame by the breaks provided by regular brick cross walls. By bearing onto each of these walls between bays, the timber frame was able to re-orientate and run straight, longitudinally along the building. In effect, whilst the external envelope is curved, the internal bays are 5 straight lengths. At the end of each bay then, it can be deduced that the longitudinal members will be quite separate from the masonry envelope, and this has shown to be the case on site. This isolation between the bracing frame and the brickwork has led to excessively slender brickwork panels and piers having to span full height of the building with little in the way of intermediate restraint.

5.8 Localised areas of brickwork and stonework joints at the inside face have been re-pointed using a modern conventional sand and cement mortar. The age of this work could not be confirmed however it is unlikely to post-date the known previous internal repair works undertaken in 1992 due to increased awareness of appropriate mortar choice and planning controls. The pointing is relatively impermeable to water vapour and the resultant increased moisture levels within the body of the masonry have had a detrimental effect on the brick and stonework resulting in spalling from frost damage and salt deposition within the pore structures. In internal environments, this is typically not as significant as the fact that cementitious pointing is also stiffer than the surrounding brickwork which means that the bricks become sacrificial during minor structural movement which may occur both on a seasonal and diurnal basis due to hydrothermal effects as well as periodic minor ground movements.

5.9 The vertical bulging of brickwork at the North Elevation either side of the tall door penetrations is likely due to a lack of horizontal wall restraint and an excessive slenderness of the walls. The floor joists at each floor were noted to run parallel to the north elevation, having little interaction with the wall. The only restraint then at each floor comes from the embedded primary beams running north/south. Cracking above and below these embedment points is suggestive of high stresses at restraint points.

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Rot of a length of timber 'quoin' showed the North elevation wall in section and it was apparent that the two skins of brickwork that make up the thickness of the masonry are poorly bonded. If representative of the full brick envelope, this lack of tying could be a contributory factor in reduced wall rigidity and vertical bowing. It is possible that the bulge seen from the outside of the building is of the outer skin only, and this can only be confirmed through relative internal/external verticality surveys or intrusive inspection.

Nutted bolt shanks vertically spaced at the inside face of the wall either side of the tall door openings suggest the original timber quoins were secured to the full thickness of the wall, providing a stiff column which would have stiffened this area between floors. Historic removal and replacement of these timbers with blue engineering bricks may have reduced the vertical stiffness of the piers either side of the openings and promoting bowing

5.10 As noted in our external fabric report, the gable ends of the north-east elevation have lost verticality and are leaning southwards. Internal inspection of this corresponding area at the uppermost floor shows a corresponding lean of the small raised mezzanine floors at both North and South sides of the top floor space. A typical gable brickwork panel supports the timber frame through the pocketed bearing of underpurlins and ridge plates. It is considered that the timber frame's jointing provides little in the way of racking resistance and so in turn has allowed the brick gables to lean inwards (possibly through wind loading or relative thermal expansion around the curve of the building) which has pushed these mezzanine frames over. The magnitude of lean is only moderate, but still noticeable to the naked eye.

5.11 The typical distortion of the roof trusses can be attributed to long term softwood creep deflection, partly due to the slight under sizing of roof timbers, but also due to the age of the building. Creep deflection of the bottom chord tie beam has in turn allowed the queen posts to drop and rotate and in turn the level of the ridge plate and rafters to lower.

5.12 It is plainly evident that historic local overloading of the truss bottom chords in bay 2 is the cause of the additional, more pronounced deflection and tie beam failure. Semi-permanent propping in the form of SHS posts have been installed (likely in 1992) and serve to transfer the roof load to the floor boards and joists of the 2nd floor, and away from primary structure. The failure of the tie beam at this Bay 2 truss means that the post at this location will be receiving the majority of the roof load and it is our opinion that the floor boards and joists under are not suitable to receive such loading in the long term. A repair should be implemented to re-establish the structural integrity of this tie beam thereby re-instating the original load path down the primary timber posts.

5.13 Our on-site observations showed that the floor plates at each floor level and within each bay were at hugely variable levels. Whilst axial shrinkage of the timber columns will undoubtedly have affected the length of each column and consequently the bearing level of each primary beam, on-site review showed the magnitude of drop to be largely consistent down the building. This is suggestive of settlement of the structure at basement level, most likely at the building's foundations. The variation in base detail at each cast iron column may itself have allowed variable settlement shortly following or during construction, although the concrete pads seen may be an historic retrofit. Further investigation is required.

5.14 The significant sag of the roof ridge, plainly visible during our external inspection, as well as the opening of timber frame joints can likely be attributed to one, other or both of the above two mechanisms; deflection of the primary longitudinal trusses with variable foundation settlement.

5.15 The performance of the foundations is critical to the ongoing integrity of the building and our specified repairs. The form of timber framing has little in the way of innate lateral rigidity and a static base will allow repairs to be carried out without cyclical or progressive movement causing defect recurrence.

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6.0 CONCLUSIONS, RECOMMENDATIONS AND REPAIRS

6.1 Whilst the majority of the masonry envelope defects affecting the durability of the structure can be addressed with relatively superficial repairs that can be integrated into ongoing maintenance, we have some concerns regarding more advanced local deterioration and the macro-stability of some areas. This requires specific consideration.

6.2 The structural stability of the internal timber frame is adequate, with the early 1990's repairs addressing the majority of latent concerns. There are outstanding issues relating to local rot from moisture ingress, the lean of the frame to the South, the deflection of the primary transverse trusses and the acceptability of the historic propping of such trusses in Bay 2.

6.3 Our outline remedial recommendations are as follows:

6.3.1 Bowing of North Elevation -

- Install Helifix Cemties or similar approved through the South elevation local to door penetrations to tie inner and outer brickwork skins together and locally improve wall integrity. Cemties are to be positioned in 2 vertical rows 900mm apart, vertically spaced at 450mm.
- Helifix 'Bowties' are to be installed through the external skin into the ends of primary beams.

6.3.2 Unrestrained brick panel, Bay 3 North elevation –

- Tie panel into brickwork reveals either side with angled Cemties through the external face (plugged to match external brickwork) and internal Helibar running continuously through bed joints across the full width and cranking into each side reveal. Fill gaps/cracks with heritage grout and point surface with hydraulic lime mortar, colour matched.

6.3.3 Historic cementitious re-pointing –

- Excessively intrusive repairs to rectify the use of this mortar is to be avoided, as any removal, however careful, will likely damage the arris of the surrounding brickwork, to the detriment of the building. Any loose cement mortar should be removed at the surface and the joint raked back to allow deep re-pointing with a colour matched suitable hydraulic lime mortar (NHL2.5).

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6.3.4 Poorly restrained infill at pulley openings, South elevation

- Tie panel into brickwork reveals either side with angled cemties through the external face (plugged to match external brickwork). Fill gaps/cracks with heritage grout and point surface with hydraulic lime mortar, colour matched.

6.3.5 Variable levels of timber frame -

- Further investigations are required to determine whether the movement is static or progressive and its root cause. Trial pits and suitable additional testing (window sampling) at basement level are recommended to determine the foundation details, bearing strata and geotechnical properties of the earth or rock beneath, that will allow both assessment and specification of further monitoring or intrusive underpinning, should it be required.

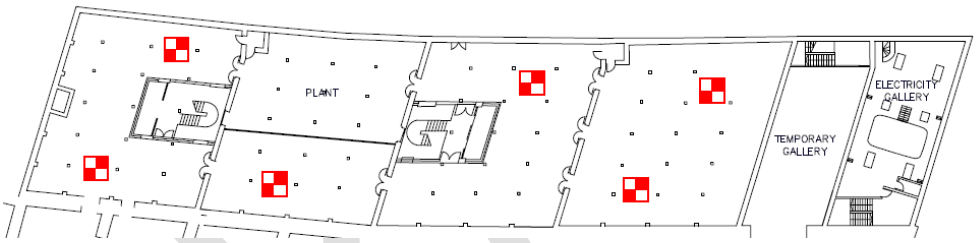

Figure 1: proposed trial pit locations across basement

6.3.6 Miscellaneous internal cracking -

- There are a small number of cracks through brickwork at cross walls that are not suggestive of a larger issue, likely local thermally induced high stress points. It is proposed that these are stitched using helical bed joint reinforcement at 3 course vertical spacing, extending a minimum of 500mm either side of the crack. The crack is to be filled with a thixotropic grout such as Helibond and faced with a bead of NHL2.5 mortar.
- Cracking at joist bearings is to be filled with a thixotropic crack repair resin. No stitching required.

6.3.7 Failed trusses, Bay 2 –

- In the short term, re-position existing props directly onto joists or preferably primary beams directly below.

- Discuss with the Science Museum Group, Studio Downie Architects and Conservation Officer regarding their aspirations for removing the propping.
- Devise bottom chord tie beam strengthening detail. Likely external steel beams either side of the bottom chord to span from primary valley beam to wall. Failure and significant bowing of existing bottom chords makes member recovery difficult.

6.3.8 Cracking/mortar loss around embedded timbers –

- We Recommend moderating heating levels within the building to reduce excessive drying, and thereby permit slow partial rehydration and expansion of the affected timber.
- Re-point in colour matched hydraulic lime mortar around timbers.

6.3.9 Heavily split mezzanine floor joists -

- Severe shrinkage of floor joists visible from basement level in Bay 2 has led to significant splitting. These are currently propped and should these be removed, the joists should be replaced in the long term, like for like in both materials and dimensions.

6.3.10 Moisture ingress -

- Maintenance of the valley gutters and hoppers serving the rainwater goods should be undertaken at the earliest convenience in order to remove all current blockages and reduce overtopping.
- Improvements in the maintenance of the roof drainage system required, as part of upcoming re-roofing works. Localised accelerated drying of damp structural materials should follow and this should be formally specified and monitored.

6.3.11 Minimal bearing –

- Bay 1 basement. A primary beam splice is positioned at the edge of a column head plate (see Photograph 36). Install tell tale monitoring and keep formal record at quarterly intervals.
- At mezzanine floor (see Photographs 34 & 35). Continue to monitor existing tell tales.
- A reduced bearing length of an upper beam at a primary roof truss in Bay 2. Instal formal monitoring devices.

6.3.12 Bay 2 northern end of valley gutter -

- 1250mm long bolted table scarf in Scots Pine *(Pinus sylvestris)* to end of valley beam.
- Splice in Scots Pine *(Pinus sylvestris)* at bearing of eastern tie beam at valley beam.

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6.3.13 Bay 2 southern end of valley gutter –

- 750 long bolted table scarf in Scots Pine *(Pinus sylvestris)*to end of valley beam.
- Splice in Scots Pine *(Pinus sylvestris)* at end of principal rafter 500mm at bearing.

6.3.14 Lack of holding down restrain at the back span of a winch beam, Bay 3 north side top floor –

- Re-secure back end of winch beam to structure beneath with minimum 2No. M12 bolts.

7.0 PENDING ACTIONS

7.1 The Science Museum Group is to engage a suitable contractor to carry out excavation of trial pits at Basement level to expose the base of the column shafts or foundations. The cobbles should be removed by hand, preserved and retained for reinstatement. Conisbee then to carry out an inspection of the foundation detail and general geotechnical conditions, with the contractor in attendance for further excavation as required.

7.2 Should it be required, Conisbee to write a geotechnical investigation brief for assessment of the bearing strata's technical performance, to be sent out by the Client to suitable consultants.

7.3 Latest measured survey drawings at each floor level to be delivered to Conisbee to allow markup of report observations for issue to the Client.

7.4 Once received, the floor spot levels at each bay on the survey drawings are to be reviewed to confirm floor level variation theories and define critical areas.

7.5 Conisbee to re-issue updated report inclusive of latest specialist information, with remedial recommendations for costing.

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8.0 APPENDIX A - PHOTOGRAPHS



Photograph 1: Looking vertical from yard level. Vertical bowing of the brickwork at the location of full height door penetrations.



Photograph 2: Rot of a length of timber 'quoin' revealed the North elevation wall in section. Note that the 2 skins are poorly bonded, and If representative this lack of tying could be a contributory factor in a diminished wall rigidity and the vertical bowing local the full height penetrations.







Photographs 3, 4 & 5 - Stepped cracking, mortar loss and historic re-pointing with a cement based mix at the face and return of the piers either side of the full height doors was common.



Photograph 6 – A full height photo of a 1st floor pier at the inside of the Northern elevation, showing the extent of vertical cracking evident through bricks, that has since been repaired with cementitious mortar. Little subsequent cracking evident.



Photograph 7 – Cracking at the inside vertical face of a timber lintel could be suggestive of movement of the pier relative to the timberwork. Given extensive re-pointing has been carried out in the vicinity, it is possible that this crack has occurred more recently as a result of further wall movement, as opposed to initial timber shrinkage.



Photograph 8 – A substantial panel of infill brickwork at Bay 3 on the north-east elevation appeared not to be fully tied into the surrounding structure and to have moved in the past, allowing a gap to form through which clear daylight could be seen.



Photograph 9 - Note the roof level valley beam beams on the North wall, providing lateral restraint to the head of the wall. Shrinkage of the timber appears to have reduced the bearing length of the valley beam.



Photograph 10 – Mortar loss around embedded timber elements relatively typical as a result of relative initial drying shrinkage.



Photograph 11 – Vertical cracking through a primary beam support pier at the West end wall at basement level.



Photograph 12 – Re-laid brickwork around a door opening through a basement level cross wall not coursed or tied into original brickwork. New ties retrofit wall ties and epoxy grout filling of gap required, with lime mortar pointing.



Photograph 13 – Cracking around a 1st Floor door through a cross wall towards the East end of the building. Likely as a result of door lintel settlement.





Photographs 14 & 15 – Moderate to severe staining at top floor level at the Valley beams, suggestive of historic valley gutter failure or blockages causing overtopping. Areas were not wet to the touch at the time of inspection but the integrity of the gutter and/or rainwater goods to be assessed.



Photograph 16 – South elevation at track level. Moisture and severe salt leeching evident at the inside face of the wall directly abutting the track. A later installed concrete gulley runs adjacent to this area, which may be undersized allowing water to saturate the brickwork. Little brick surface spalling but the outside face at the same location may be more susceptible to deterioration.



Photograph 17 – Substantial section loss of a column within Bay 1 at mezzanine floor level. This is not seen elsewhere in the building and the surface of the timber suggests that contractors may have hacked away timber, possibly they thought to be defective.



Photograph 18 – Severe shrinkage of floor joists visible from basement level in Bay 2 has led to significant splitting.



Figure 19 - Minor corrosion was apparent at basement level cast iron column heads. No significant section loss and no remedial works required. Columns otherwise satisfactory however we have no understanding of the condition of the metalwork that is below cobbled floor level.



Photograph 20 - The lifting of a loose cobble around a column allowed sight of surface corrosion to an unpainted length of column. This may suggest that corrosion is more advanced below floor level, and the level of exposure to moisture and hence corrosion could be hugely variable.



Photograph 21 – Historic strapping tying corbel timbers to primary beams was seen to be loose and in need of tightening throughout.



Photograph 22 – An original crane at track level presented as part of the exhibition. No structural issues evident.



Photograph 23 – The vertical stem of the crane continuous through the floor at track level to the mezzanine level below, where it is packed down to the timber joisted floor with timber cross bearers. No overstressing evident at floor structure.



Photograph 24 – A heavily corroded downpipe is penetrating through the inside face of the South wall at basement level. No evidence of moisture around the opening which may suggest the penetration is full depth, but the pipe is exhibiting moderate to high levels of corrosion and further investigation is warranted to determine its functionality.



Photograph 25 – A general view of the top floor of the building, showing the typical arrangement of regularly spaced square timber columns, timber block corbel heads and timber primary valley beams onto which the longitudinal spanning timber Queen post trusses sit.



Photograph 26 –Significant deflection of the bottom chord of the primary longitudinal roof trusses was seen throughout the building, but it is most pronounced at Bay 2.



Photograph 27 – Square Hollow Section columns (4No), serve to prop failed trusses at the East side of Bay 2.



Photograph 28 – The column head plates are pressed firmly to the underside of the trusses using a threaded plate seperation system.



Photograph 29 – Due to the curve of the building, both sets of rafters meet at the ridge plate at different horizontal angles. This could be creating a net horizontal force vector down the line of the ridge in the Southwards direction, exacerbating the frame lean.



Photograph 30 - The lean is most evident at the top floor mezzanines. These small upper floor areas bear into the North and South walls, but are not braced and so are susceptible to movements in the building envelope.



Photograph 31 – The head of a South mezzanine post at the top floor had significantly shifted southwards, possibly dragged by the mezzanine flooring that bears into the South wall.



Photograph 32 - Further evidence of lean in the mezzanine posts. The corbel remains square on column under that has rotated to the South (right in the photo), and the stiffness of the primary valley beam over allows it to stay flat, creating the gap.



Photograph 33 – A reduced bearing length of an upper beam at a primary roof truss in Bay 2 was noted. This could be as a result of drying shrinkage in the Queen post beam, dropping of the column bearing, poor carpentry or possibly deflection in the bottom chord destabilising the queen post.





Photographs 34 & 35– The timber framing at the mezzanine floor typically utilises a side fixed corbel onto which the beam dead-bears. Shrinkage and frame racking has pulled the timber off this bearing and it now very little contact remains. Telltales have previously been installed at this beam support, but not movement has been recorded.



Photograph 36 – Bay 1 basement. A primary beam splice is positioned at the edge of a column head plate. Minimal bearing.



Photograph 37 – The historic propping of trusses in Bay 2 with timber posts onto timber sole plates is suggestive of a long standing issue with excessive deflection in this area.



Photograph 38 – This photo represents the extent to which the floor drops between cross walls, in both directions.





Photograph 39 & 40 - It was noted that some basement columns have monolithic stone bases, other cast concrete pads and some are continuous into the ground with an unknown base detail.



Photograph 41 – There is evidence of heave locally to certain columns, which warrants further geotechnical investigation.



Photograph 42 – Lack of holding down restrain at the back span of a winch beam.