Manhattan Loft Corporation

St Pancras Booking Hall

Ticket Office Mezzanine – Phase 2 of Structural Feasibility Study

Phase 2

Final | 24 June 2016

Job number 24421900

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ARUP

Contents

1 Executive Summary

The structural feasibility of installing a mezzanine over the existing ticket office in the former booking hall was further evaluated in this report. This consisted of a more refined calculation of the structural integrity of the existing cast iron steel columns and masonry floor arches. Data that became available for this assessment included an intrusive investigation of the column diameters and thicknesses (see Appendix A), processing of laser survey data to estimate global dimensions, and intrusive investigations to determine the approximate depth of the beams and thickness of the masonry arch voussoirs.

Revised calculations indicate that the existing cast iron columns can adequately carry the new mezzanine level. However, the existing masonry arch flooring is unable to carry concentrated, non-uniform loadings imposed from the mezzanine level with adequate factors of safety. Thus, we have devised a special steel grillage detail (see Figure A) which concentrates loads directly above the columns, and thereby avoids loading the masonry arch. The detailing requires excavation, locally, into the fill of the existing floor so that the steel grillage does not penetrate the existing wooden walls of the Booking hall.

With this novel detailing, and bearing in mind the risks identified in Section 9, we believe we have a workable structural solution. Following planning, the next steps include developing a scheme and detailed design in conjunction with architect.

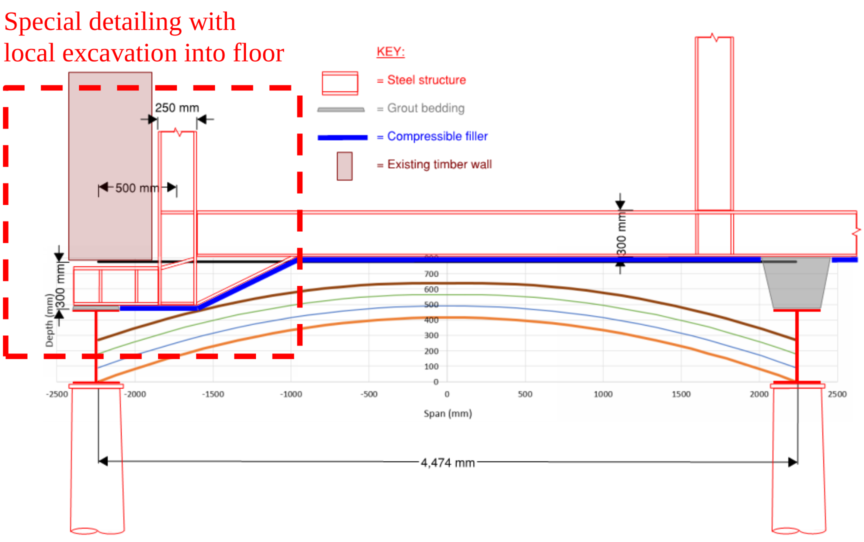

Figure A: Schematic of proposed structure to avoid loading the masonry arch flooring

2 Introduction

Arup has been engaged by Manhattan Loft Corporation (MLC) to assess the structural feasibility of installing a mezzanine over the existing ticket office in the former booking hall. This report is Phase 2 of the study into whether it is feasible for the existing structure to support a new mezzanine level.

Phase 1 of the assessment gave a description of the existing structure and proposed new structural layout and was issued on 22nd July 2015. Investigative works were carried out on the 1st and 23rd March 2016 to determine key structural dimensions and the results were summarised in a report dated 3rd May 2016.

Phase 2 of the assessment is a more refined calculation of the structural integrity of the existing cast iron steel columns and masonry floor arches. Data that became available for this assessment include an intrusive investigation of the column diameters and thicknesses (see Appendix A), processing of laser survey data to estimate global dimensions, and intrusive investigations to determine the approximate depth of the beams and thickness of the masonry arch voussoirs.

3 Existing Structure

The feasibility of installing a new mezzanine depends on the ability of the existing structure beneath to support this new structure with additional imposed loadings.

The existing structure beneath the Booking Hall consists of masonry arches spanning between wrought iron beams. The beams bear on cast iron columns. Based on existing drawings the foundations appear to be square concrete footings. The key dimensions of the existing structure are given in Fig. 1.

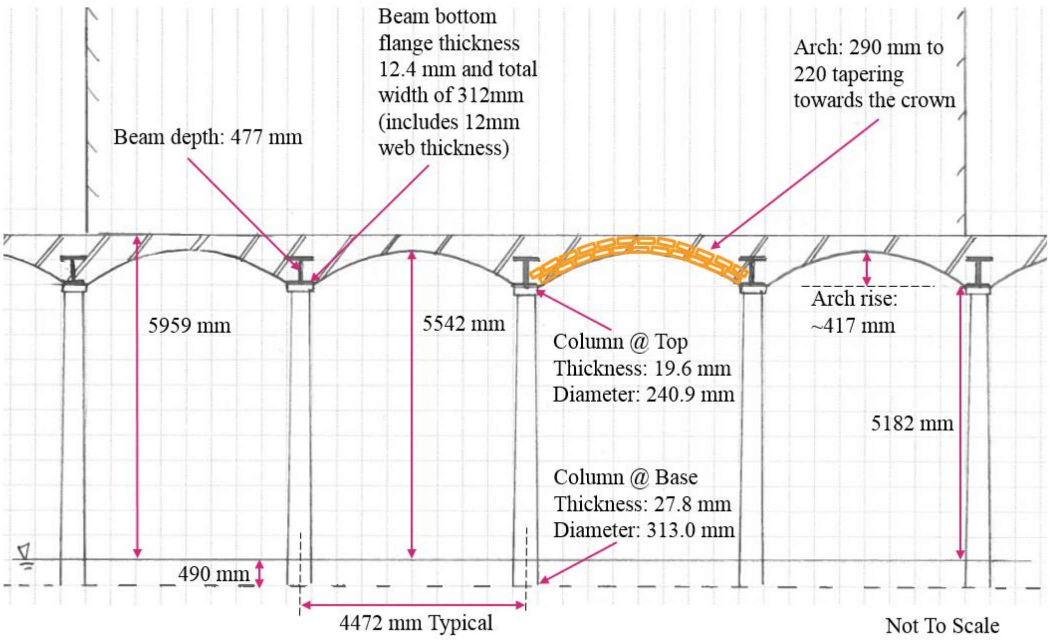

Figure 1: Dimensions of existing structure assumed for feasibility study

The flooring consists of non-structural fill and a brick voussoir resting on wrought iron beams. The existing drawings and laser survey confirm that the span of this arch is about 4472mm between beam centrelines. The intrusive survey data suggests that the rise of the arch masonry barrel is 471mm (May 3rd 2016 report).

The tapered columns were investigated using ultrasonic measurement techniques on 3 columns. The measurements were calibrated against physical thickness measurements obtained by drilling into each column at one location along its height. A linear fit was applied to the measurements over the height of the column to give an estimated thickness and diameter at the top and bottom of the columns.

4 Proposed New Structure & Loadings

The proposed structural solution from the 22nd July 2015 report is reproduced in Fig. 2. Approximate dimensions, valid for this feasibility study are included and used in calculating the dead and imposed loadings. The loadings assumed over the mezzanine and lower floor areas are:

Mezzanine:	Services	=	1 kPa
	New steel structure	=	2 kPa
	Live loading	=	4 kPa
Lower floor:	Services & Partitions	=	1 kPa
	New steel structure	=	2 kPa
	Live loading	=	4 kPa

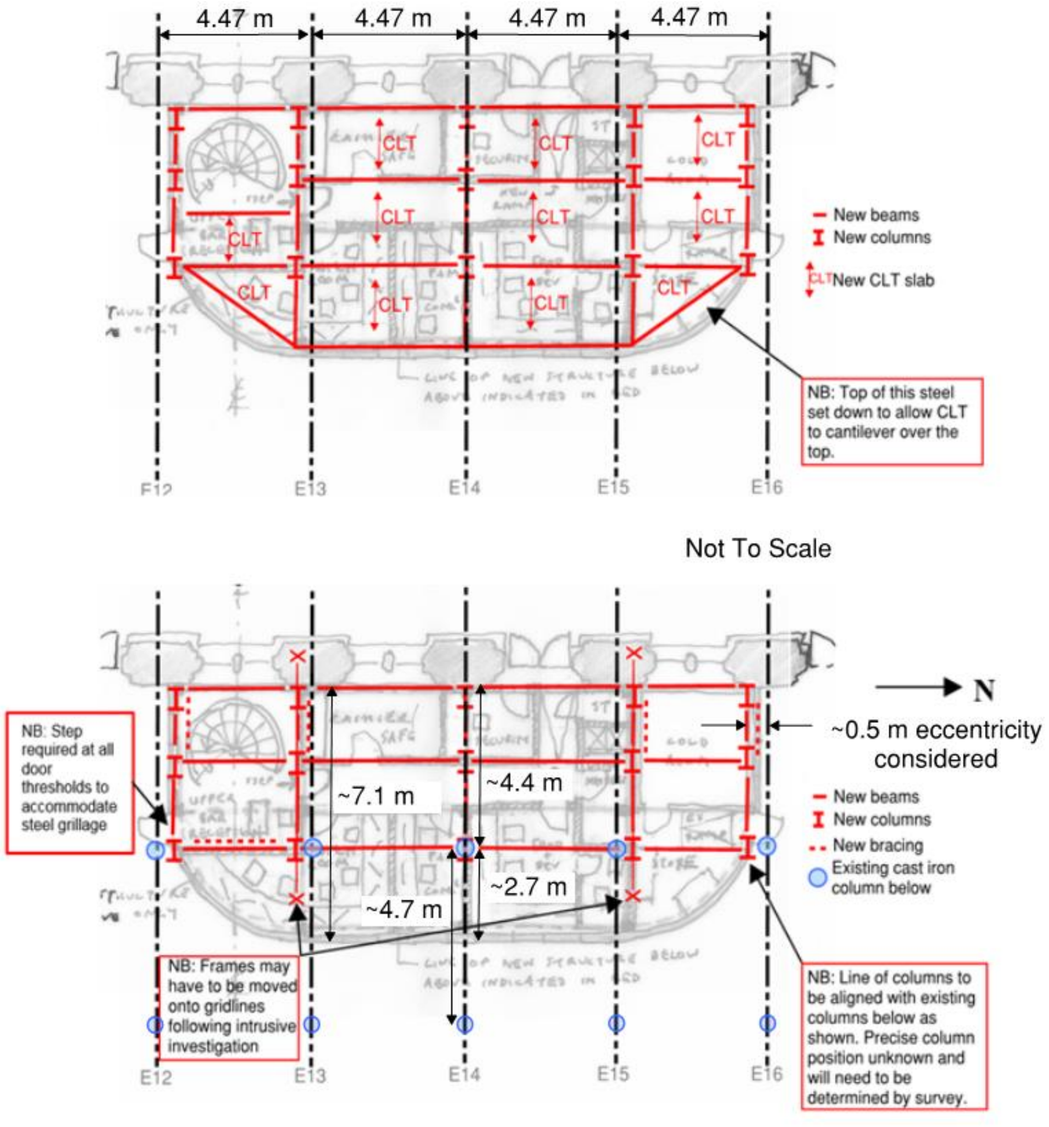

Figure 2: Dimensions of mezzanine and lower floor assumed for feasibility study

5 Masonry Arch Flooring

The basis of our proposed new structure is to bring the new loads onto the column lines as far as practical, thus avoid loading the existing beams and masonry floor. Since the booking office stops short of the final column lines, we have had to put some of load on the masonry arches. The eccentricity of the new columns lines from gridlines E12 and E16 are shown in Figure 2. Architectural requirements have also meant that the new columns are eccentric from gridlines E12 and E15 (so that the new columns can be placed within walls). Thus it is necessary to assess the ability of the existing masonry arch floor to carry these new loads.

The brick voussoir of the floor arch acts in compression to carry loads imposed on the arch. The arch is inherently good at resisting uniformly distributed loads. For this feasibility study, a line-of-thrust analysis was conducted to investigate the arches' ability to carry non-uniform loading, specifically, an eccentric point load from the columns supporting the mezzanine. The steel grillage on the floor below the mezzanine level is eccentric from the line of the wrought iron beams as shown in Fig 2. A maximum eccentricity of 500mm was considered in this analysis.

The line-of-thrust analysis is a simple geometric method to assess the arches stability. There are three main assumptions 1) masonry has no tensile strength, 2) stresses are low in that the masonry has infinite compressive strength and 3) sliding failure does not occur. The stability of the arch is assessed for unfactored loadings, with a geometrical factor of safety used. If the thrust line remains within the middle third of the voussoir depth (220mm assumed) then there are no tension stresses developed. This corresponds to a geometrical factor of safety of 3.

An analysis using the dimensions in Figs. 1 and 2 and unfactored loadings in Sect. 3 was conducted. Various loading cases were considered to give the worst case. A large point load with little uniformly distributed load gave the most critical condition. Fig 3 illustrates the analysis for the case where there is point loading at each end of the beam with a 500mm eccentricity from the support of the arch. It shows that the compression thrust line is within the voussoir, hence is stable, but only has a geometric factor of safety of 1.4 (within the middle 70%). Since this line of thrust goes outside the middle third of the voussoir depth, cracking will occur in the arch. Furthermore, to achieve this line of thrust requires a large horizontal thrust which may make the neighbouring arch unstable if that arch has little vertical load acting on it. Thus our study indicates that the existing masonry arch is not suitable to carry the large eccentric point loads from the new mezzanine columns.

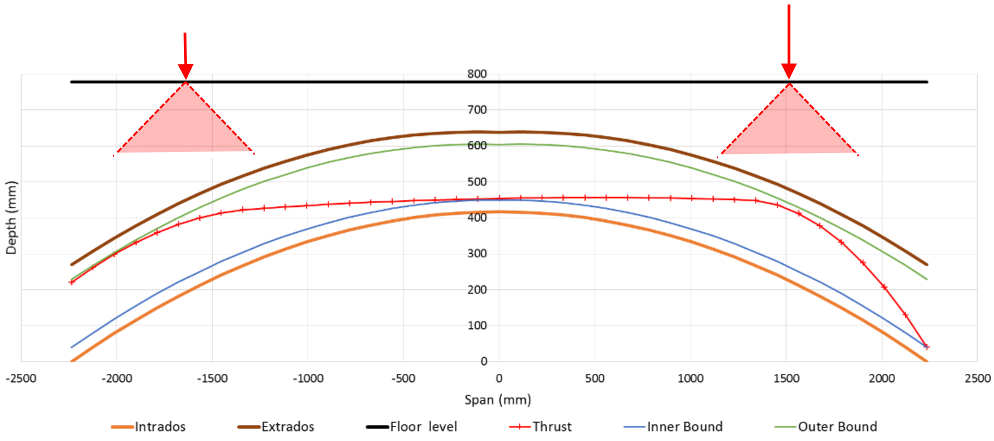

Figure 3: Line of thrust analysis with eccentric point loading

6 Beams

It is assumed that the beams are made of wrought iron (instead of cast iron) as the beams are a built-up section made of plates and angles- a common characteristic for wrought iron. The beams are embedded in the brick flooring, with only the bottom flanges able to be observed. By drilling from above we were able to determine the depth of the beam. This drilling also indicated that there is a top flange to the beam. The dimension assumed for this study are given in Fig. 4

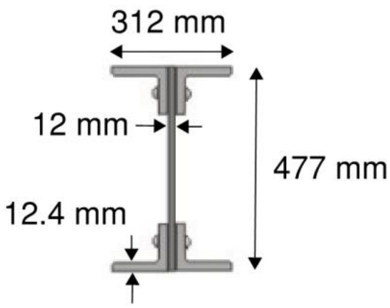

Figure 4: Assumed dimensions of the built-up wrought iron beam

The proposed new structure in Fig. 2 was devised so that the loadings from the new mezzanine level are not imposed on the existing beams. Thus the existing beams are not envisaged to be loaded significantly more than their current state.

7 Columns

The column analysis has been refined based on intrusive investigations by Sandberg Consulting Engineers (see Report in Appendix A). Non-intrusive testing carried out in Phase 1 was distorted by the thickness of the intumescent paint which we did not have permission to remove at that stage. The intrusive investigations now show that the diameter of the columns is smaller than previously assumed, but that the cast iron wall thickness is thicker.

The column wall thickness vs. height and column diameter vs. height are shown in Fig 5. The average measured thickness/diameter is plotted as blue points and the linear interpolation is shown as an orange line. The cast iron columns are tapered and have a wall thickness and diameter of 19.6 mm and 241 mm, respectively at the top and 27.8 mm and 313 mm, respectively at the bottom.

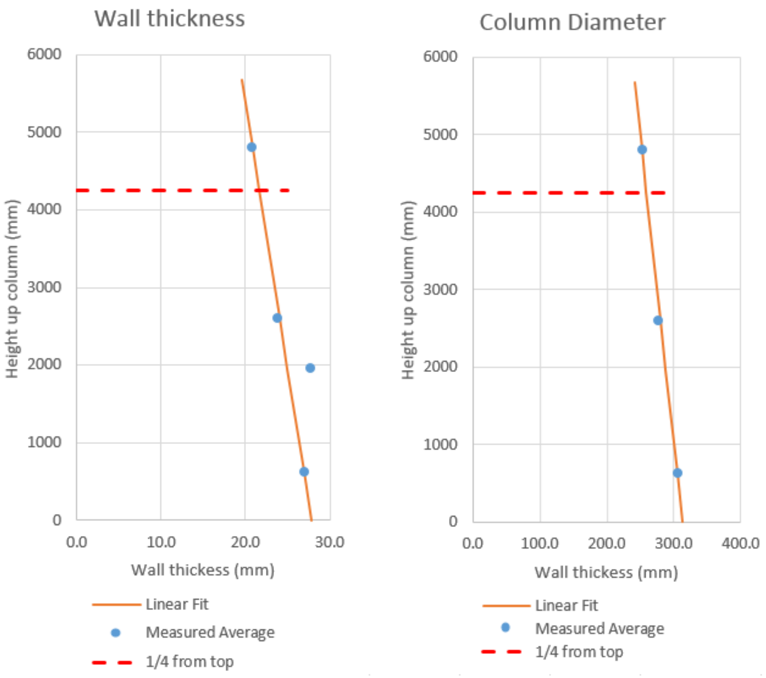

Figure 5: Tapered cast iron columns wall thickness and diameter

The Rankine-Gordon formula was used to calculate the buckling capacity of the column assuming a pin-end column top and bottom. This ultimate capacity was divided by a factor of safety of 5.0 to get the allowable compressive stress. This allowable stress for different column slenderness ratios (L/r) is plotted in Fig. 6.

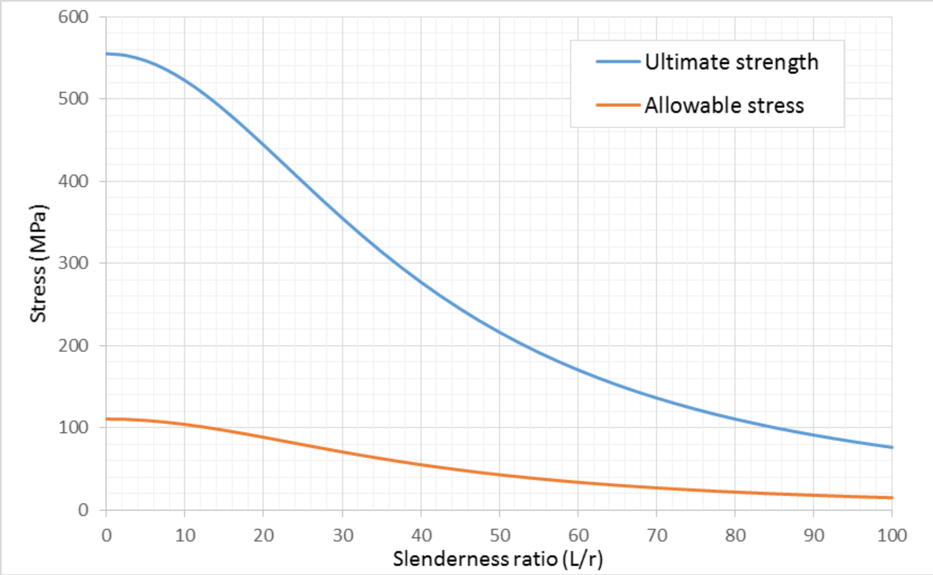

Figure 6: Buckling capacity of cast iron columns.

The slenderness ratio of the column changes over the height of the column. For this feasibility study, the slenderness and column area at a quarter of the total height (from the top) was used to calculate the capacity. This gave an allowable capacity of about 468kN. An estimate of the unfactored loads using a conservative estimate of the live loads gave a usage ratio of 1.0, which is just adequate. As the design evolves, and the imposed loadings become better understood this usage ratio may improve. Nonetheless, the column capacity can also be increased by providing better fixity/restraint at the bottom (and/or top) of the column, which has currently been conservatively been taken as pinned. Thus this study indicates that the existing columns are adequate for concentric loading.

The existing foundations were not assessed in this study. The percentage change in the dead load on the foundation is likely to be less than 40%.

8 Proposed Detailing of New Structure

Due to the existing masonry arch flooring being unable to carry eccentric, concentrated loading, we have devised a detail to avoid loading this floor arch. A schematic of this detail is shown in Fig. 7. The detail involves excavating locally into the fill of the floor (see Fig. 8), beneath the existing wooden walls and floor finishes, and having the new steel structure (which supports the new mezzanine) bear directly above the columns. Fig. 7 is drawn approximately to scale and indicates that there is sufficient space to install the new structure. Although this proposed structure will involve complicated detailing and tight tolerances, we believe it is a workable solution.

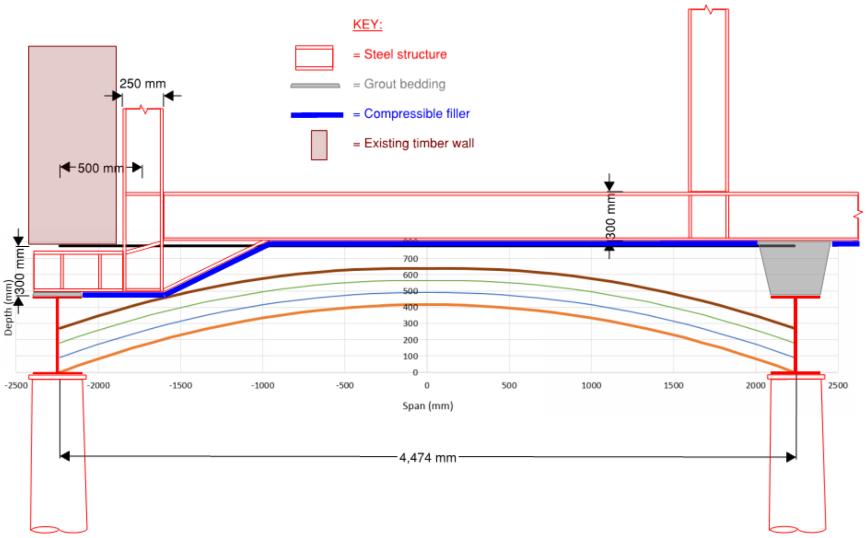

Figure 7: Schematic of proposed structure to avoid loading the masonry arch flooring

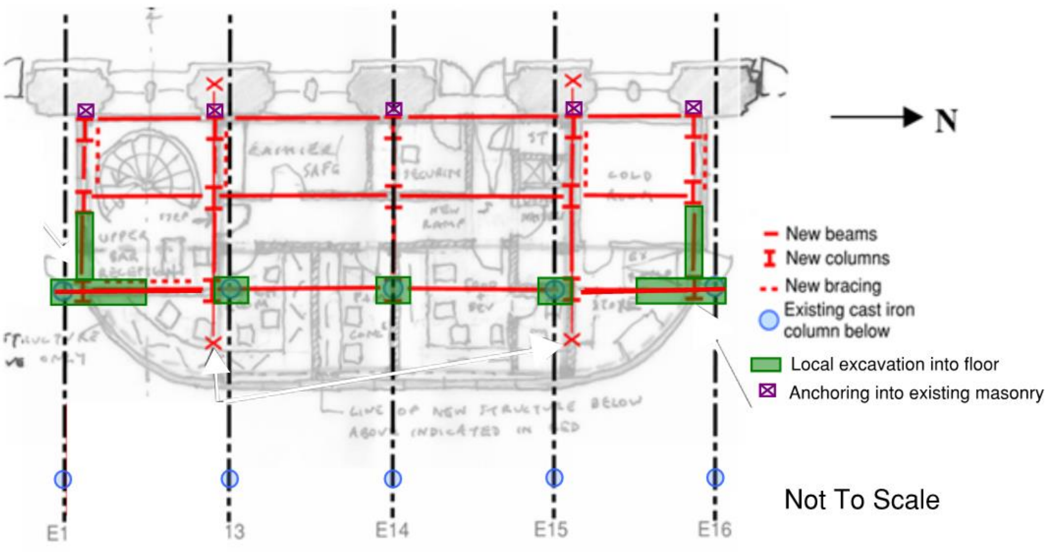

Figure 8: Plan of new structure with excavation and anchoring locations

Fig. 8 shows the approximate locations where the cutting into the existing masonry floor is proposed to accommodate the detail in Fig. 7. The excavation shall only be into the fill of the floor and should not penetrate into the masonry voussoir part of the arch. The detailing is intended to avoid the existing wooden walls by going beneath them and, as such, assumes that the walls do not extend below the floor level.

Also shown in Fig. 8 are the locations where the new structure will be anchored into the existing masonry columns. To engage enough weight of the existing masonry may involve drilling for up to 0.5 or 1 meter. More investigation is required to determine details of this connection.

9 Updated Risks and Conclusions

This study has addressed some of the risks highlighted in our 22nd July 2015 report. Specific items addressed include:

- Intrusive investigations have revealed that the wall thickness of the columns is thicker than previously assumed, however the diameter of the columns is slightly less. Revised calculations indicate that the existing cast iron columns can adequately carry the new mezzanine level.

- The existing masonry arch flooring is unable to carry concentrated, non-uniform loadings imposed from the mezzanine level with adequate factors of safety. A new detail is devised which concentrates loads directly above the columns, and thereby avoids loading the masonry arch.

A number of risks for the next stage of design include:

- Complex detailing and tight tolerances of the proposed new structural detail that is used to avoid loading the arch flooring. Also more noisy works within the hotel; unexpected details when we dig into floor and walls.
- Architectural detailing is still to be finalised. Issues requiring consideration include the raising of the floor level within the booking office back of house areas (using a false floor) and how that is detailed.
- The imposed loadings on the structure have been crudely estimated and may change based on architectural requirements, and more detailed load map calculations. This study has used a conservative estimate of loads to minimize this risk.
- Practical and heritage difficulties in forming pocket into existing brickwork to support ends of new beams, difficulties in drilling anchors into structure, and we also need heritage approval for digging into the floor: Early discussions required with English heritage and potential contractors.
- The current loading in the storage room is unclear. Arup would like to set a loading limit for the storage room in the next stage of the works.

Following planning, the next steps include developing a scheme and detailed design in conjunction with architect

Appendix A: Intrusive Column Survey Report

SANDBERG
CONSULTING ENGINEERS
INVESTIGATION INSPECTION
MATERIALS TESTING

SURVEY REPORT

Report ref. and computer file ref.				
55509J	SR	01	TP	0
Job No.	Type	Rept.No.	Insptr.	Rev.

Client:	Manhattan Loft St Pancras Hotel Ltd
Contract:	St Pancras Booking Hall
Works/Site:	Hatchards Retail Unit, St Pancras Station
Telephone No.:	07583 063 478
Person contacted:	Brian Duffus
Name, Grade, Reg. No.:	Tony Pitman, Project Manager Ian Tomlinson, Surveyor
Visit Date(s):	1st March 2016 – Nightshift
Unconfirmed report sent:	No
Attachments (list):	None

NDT disciplines undertaken by Sandberg and covered by this report:

Ultrasonic Testing ☐ Surface Testing (MPI or DPI) ☐ Eddy Current Testing ☐ Other ☒

Controlling documents		
	Specification (s):	ARUP - File Note / Job Number 2442190 / dated 2nd October 2015.
	ITP:	
	QMP:	
	Other:	

Reviewer's Comments:

Rev.00

NOTE – This report should not be taken as conferring compliance with any European Regulation or Directive but may be used in support of a statement of compliance.

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		Document	Project specific for:	Date
Date: 8th March 2016	Date: 09/03/2016	SD SR R03	N/A	Jan 2013

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1 INTRODUCTION

It was requested that Sandberg undertake a dimensional survey on three cast iron columns and the soffit of one wrought iron beam located in the Hatchards retail unit of St Pancras Station. The columns and beams within the retail unit are part of a series that support the vaulted arches beneath the Booking Hall.

2 INSPECTION PROCEDURES AND TEST EQUIPMENT

2.1 Visual Inspections

The visual inspections were completed in florescent and torch lit conditions. Areas of interest were photographed with a digital camera.

2.2 Dry Film Thickness (DFT) Survey

DFT readings of the protective coating system were recorded using an electromagnet coating thickness meter:- Defelsko PosiTector 6000 (Sandberg Ref. SI 0195), with calibrated shim set (Sandberg Ref. SI 0196). Testing was undertaken in accordance with BS EN ISO 2808: 2007 with the gauge being checked against a zero plate and a minimum of two shims.

2.3 General Dimensions

General dimensions were recorded using a combination of Vernier caliper, tape measure, steel rule and wire.

2.4 Cast Iron Column Wall Thicknesses

The restricted access for Columns 1 and 2 meant that it was only possible to survey at two heights during the attendance (towards base and mid height). Column 3 had good access and was surveyed at three heights (towards base, mid height and towards top).

At each of the accessible heights survey locations were spaced at 90° intervals around the circumference of column.

Each survey location had a 50mm^2 area of protective coating removed and was cleaned back to bright metal using a hand held abrasive.

Columns 1 and 3 each had a Ø6mm hole drilled into them in order to physically measure the wall thicknesses and allow calibration of the ultrasonic thickness gauge against the cast iron material. Column 2 had two Ø6mm holes drilled into it.

Wall thickness readings were recorded with an ultrasonic Krautkramer DMS 2 (Sandberg ref SI 0037) with a 2MHz probe.

3 RESULTS

3.1 Visual Inspection

The cast iron columns and wrought iron beams had what appeared to be an intumescent coating system present (see Photographs 1 to 3).

The cast iron columns were tapered.

The retail unit had a raised floor which was approximately 500m above the concrete that encased the base of the column (see Photograph 4).

Only the soffit of the wrought iron beams was accessible during the attendance. It appeared that the beams are a composite using what is assumed to be back to back angles (see Photograph 5).

3.2 Dry Film Thickness (DFT) Survey

Location	Dry film thicknesses (mm)			
	No. readings	Min	Max	Mean
Column 1	10	2.4	4.3	3.3
Column 2	10	2.5	4.8	3.3
Column 3	10	2.2	6.5	3.8

3.3 General Dimensions

3.3.1 Cast Iron Columns

From the top of the middle column to the concrete encasing the base of the column an approximate height of 5130mm was recorded.

3.3.2 Wrought Iron Beam

The following dimensions were recorded on the wrought iron beam:-

Soffit width of each angle section - 150mm

Total width of both angle sections and thickness of web plate - 312mm

Toe thickness of angle section - 12.4mm

3.4 Column Wall Thicknesses

The wall thicknesses recorded on the columns can be found in the table below:-

Column Identification	Height on column from the concrete (mm)	Estimated diameter of column minus coating (mm)	Wall Thickness Measurement (mm)			
			Location Around Circumference of Column			
			0° front of retail unit	90°	180°	270°
1 - towards front of retail unit	620	304	26.0 (Note 1)	26.2	26.7	25.5
	2600	278	17.7	23.7	29.4	25.7
2 - middle of retail unit	620	304	27.7	27.5 (Note 1)	30.0 (Note 1)	22.7
	2600	278	23.3	27.0	25.6	18.5
	4800	253	20.8	21.2	24.3	17.0
3 - towards rear of retail unit	620	N/D	(Note 2)	(Note 2)	28.7	27.0 (Note 1)
	1950	N/D	(Note 2)	(Note 2)	30.1	25.4
Note 1 - Drilled location Note 2 - No access due to proximity of book shelves and false walls						

After drilling the holes it was noted that the columns were hollow.

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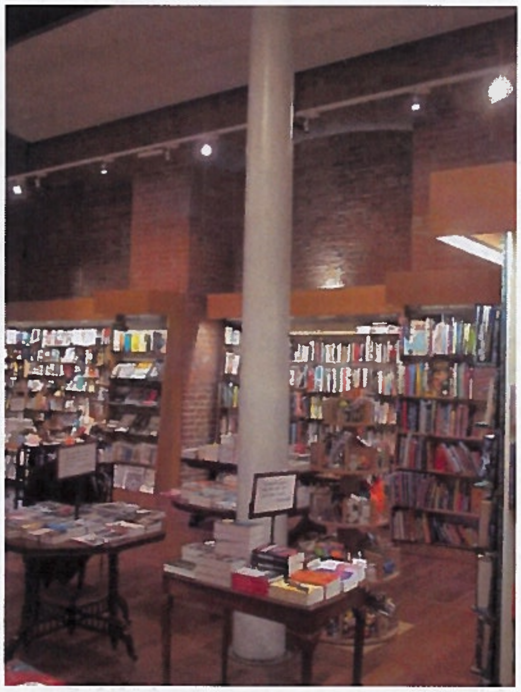
Photograph 1. **Showing Column 1**



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Photograph 2. **Showing Column 2**

Photograph 3. **Showing Column 3**

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Photograph 4. **Showing the concrete base around Column 2**

Photograph 5. **Showing a length of the wrought iron beam**

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