Kingsway House

Structural Engineering
Stage 3 Report

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1 Introduction

Elliott Wood has been appointed by GMS Estates to provide structural and civil engineering design services for the proposed redevelopment of Kingsway House, London, WC2B 6QX.

Kingsway House is an existing 9 storey (including lower ground floor) building located on the west side of the Kingsway between High Holborn and the Strand. It was constructed in approximately 1906-07.

The proposed redevelopment consists of removing the internal floor plates and supporting internal structure to allow for the installation of a RC frame with open floor plates and a new servicing core. The existing masonry façade and party wall will be retained with the introduction of a further storey at roof level as part of a newly constructed mansard.

This document is to be read in conjunction with the structural drawings and design information produced by the rest of the design team.

2 Executive Summary

2.1 Existing building

Kingsway House was originally constructed in 1906 extending 8 storeys to the 6th floor mansard with an additional 7th floor added at a later date. The existing structure comprises of an irregular load bearing masonry façade/core with interior steel or cast iron columns, supported on shallow concrete footings. The floor structure comprises of steel beams at each level supporting clinker concrete and filler joist floor plates.

2.2 Options Appraisal

During the period between the first issue of the stage 3 report in June 2018 and this report issue, a full options appraisal has been undertaken for the entire scheme. This was driven by a review of the fit-out brief by the client which requires a more contemporary aesthetic with exposed structural soffits. The options considered were CLT, Precast Panels, Metal Deck and an RC frame.

2.3 Proposed scheme

Reinforced concrete frame

The proposed scheme retains the existing masonry façade and removes all interior cores, stairwells, floor plates and columns to create functional open plan commercial space. This is achieved through a new structural RC frame with services integrated within the raised floor allow for a refined, exposed soffit.

Façade Retention

The existing masonry façade & party wall, as well as the majority of their existing foundations, shall be retained. The masonry facade will support only its own self weight and, along with the party wall, be tied back into the new structural steel frame for lateral restraint.

Rooftop extension

The existing two storey mansard is to be replaced with a new RC framed mansard including an additional storey and roof terrace.

Stability

Lateral stability is provided through: rigid floor plates and a new RC core. The moment frames at the front in the previous scheme have been removed.

Servicing

Services are distributed to each floor through vertical risers incorporated into the proposed core. To achieve a refined soffit the bulk of the horizontal services will be located within the raised floor zone with only sprinkler and lighting systems suspended from the soffit of the slab.

Foundations

The proposed structural RC frame shall be founded on a new raft foundation set back from the existing façade line on 3 sides so as not to conflict with the existing foundations. Along the front elevation the existing pad footings are to be removed and replaced by the new raft to avoid internal columns and transfer structures. This will require the façade to be supported on temporary works during the course of construction.

Pavement vaults

The existing pavement vault structure and waterproofing is currently in poor condition due to infiltration of tree roots from ground level and the resultant water ingress. The pavement vault structure is to be strengthened and retained to avoid unnecessary risk to the underlying services in the pavement.

Below ground drainage

The three existing sewer connections will be retained as part of the new proposals. The existing below ground drainage will be removed and replaced with a new drainage network. The new drainage network at lower ground floor will installed in cast iron pipework and will be cast within the new raft slab.

Site & utilities

The Kingsway House site conflicts with LUL assets as the building foot print overlies two Piccadilly underground line tunnels. Consultation with LUL and a specialist geotechnical consultant is underway.

A multitude of service assets run through the pavements adjacent to the front façade (in the case of the front façade, above the building pavement vaults). Any works in proximity to these services must be coordinated with the relevant service providers.

Figure 2.1: Existing Building

Figure 2.2: Existing Facade and Proposed Structural Frame

3 Site Information and Constraints

3.1 Usage

The ground floor of Kingsway House is currently used for retail purposes and the higher levels for office space

The London Met Archives hold the Building Act Case File for Kingsway House, according to these documents the original application made to the London County Council [LCC] was for Offices only. A further application was made to LCC for Offices and Shops

Figure 3.1 : External elevation

3.2 Historical maps

Historical maps confirm that Kingsway House, in its current form, was constructed in approximately 1906-1907 (*Figure 3.2* and

Figure 3.3). The LCC bomb damage maps show that Kingsway House experienced no bomb damage in the Second World War

Figure 3.2: 1896 town plan: Kingsway House site prior to current form.

Figure 3.3: 1910 town plan: extent of Kingsway House

Figure 3.4: LCC bomb damage maps 1939-1945: Kingsway House sit

3.3 LUL assets

After consultation with the relevant LUL authorities, it has been found that the Kingsway House site overlies LUL assets, with two Piccadilly underground line tunnels running underneath the site (Figure 3.5).

Therefore the London Underground Infrastructure Protection must be provided with the following details of the proposed works involved with the redevelopment at the Kingsway House site; demolition, structural works, excavation, boreholes or piling, highway works. In addition, any changes in load over the building footprint will need to be agreed with LUL as the resultant forces on the roof of the LUL tunnels will need to be verified.

Consultation with both LUL and a specialist geotechnical consultant are underway. A line and level survey of the tunnels has been carried out by Survey Associates LTD (SAL). SAL have also been appointed to carry out a conditions survey of the tunnel prior to any commencement of the works.

During the next design phase (Stage 4) it is intended to liaise with Geotechnical Consulting Group (GCG) to carry out a ground movement assessment of the site which will determine the interaction and associated movements between the ground and new foundations based on the proposed loadings. This will be critical to finalising the design of the raft foundation (thicknesses and reinforcement quantities) along with gaining approval for the works from LUL.

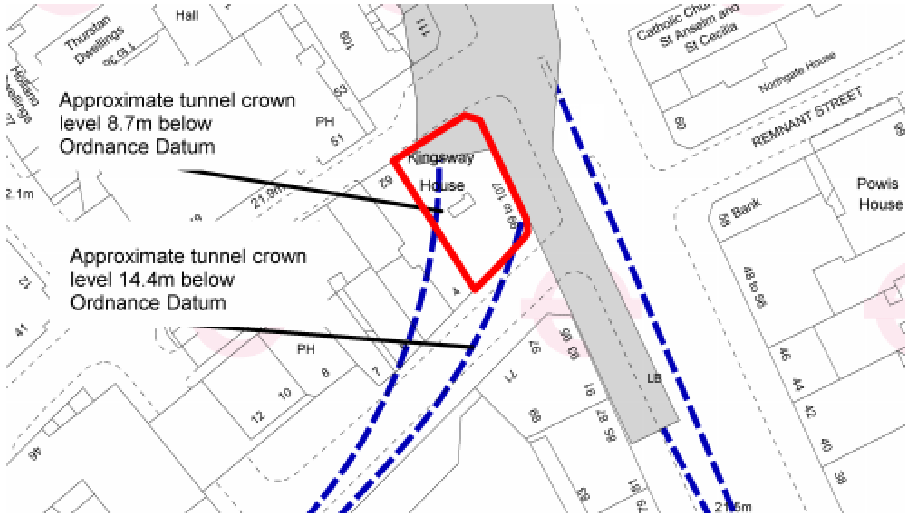


Figure 3.5: Current plan: TFL assets (Piccadilly line shown in blue)

3.4 Utilities

Gas, water, sewage, communications, environmental and transport assets were consulted. Many of these assets run adjacent to the site along the neighbouring public highways therefore extensive coordination is required between the contractor and the service providers where the pavement vaults are to be redeveloped or broken through to accommodate the external scaffolding and façade retention temporary works.

3.5 Historic documents

Historic drawings show that Kingsway House originally extended 8 storeys (including basement) to the 6th floor mansard (Figure 3.6). An additional 7th floor has since been added although it is unknown when this occurred.

A party wall running parallel with Kingsway shown in drawings by A. Sykes architects 1905 (Figure 3.7) for the introduction of oriel windows at the 3rd and 4th floors, suggests the building may have been two separate properties. An alternative arrangement in the centre of the building was then later adopted, as seen in subsequent drawings. The building was built with a central core with lift and stairs and a second set of stairs to the rear of the property.

Drawings submitted to the LCC in 1938 (Figure 3.8 and Figure 3.9) show proposed alterations to the structure at basement and ground floor respectively. Internal columns are shown at various points, many in line with the assumed former party wall line. Vaults are shown extending out beyond the ground floor perimeter on two sides.

Drawings by R.A. Bingham architects from 1954 show the steel transfer system to accommodate the current ground floor shop frontage (3.11).

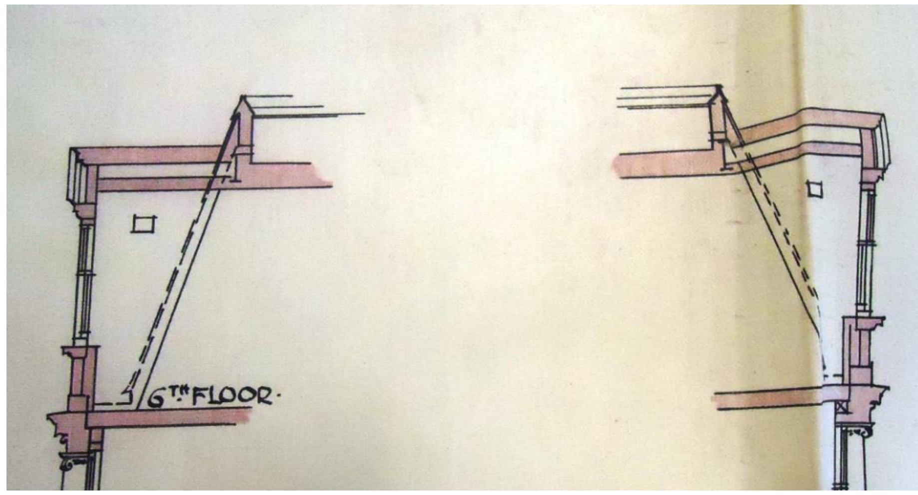


Figure 3.6 : A. Sykes 1905: 6th floor mansard section

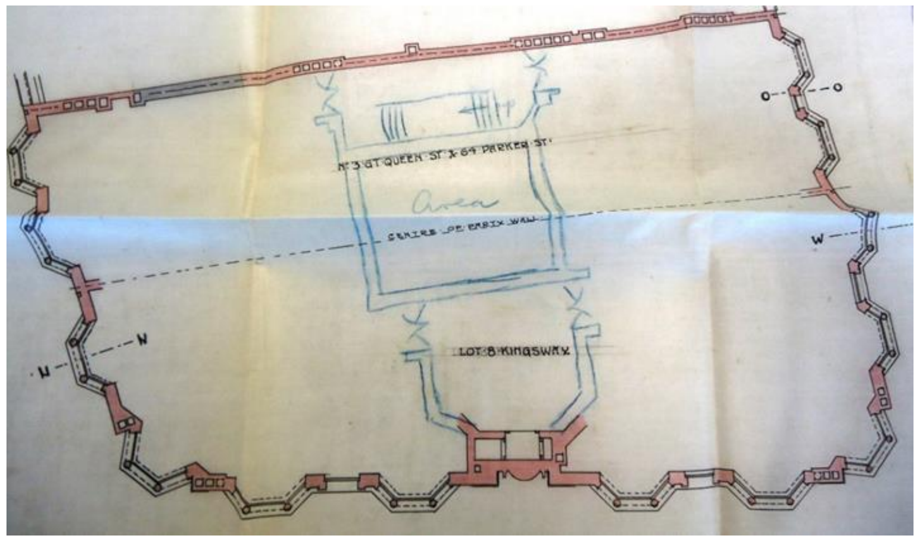
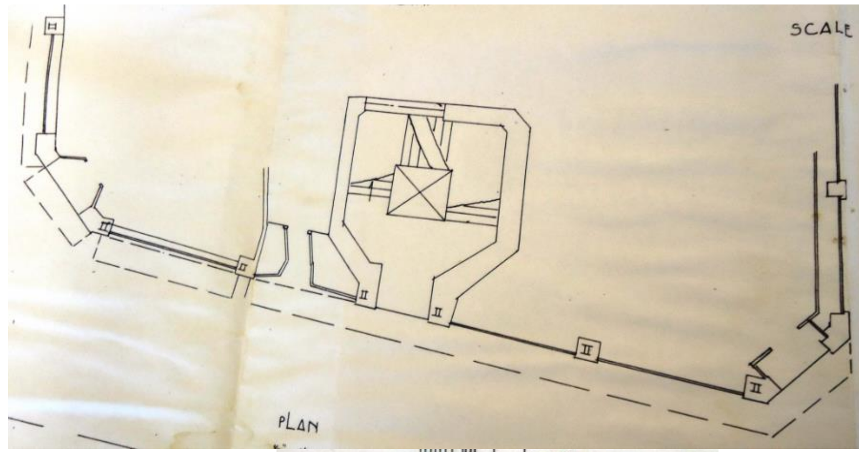
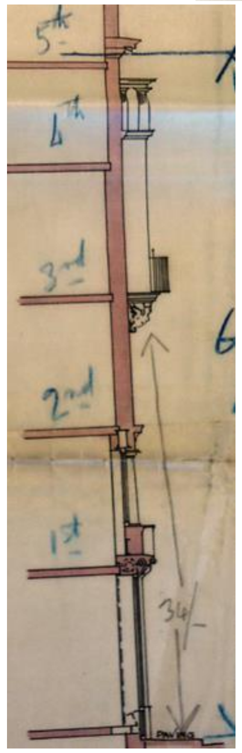
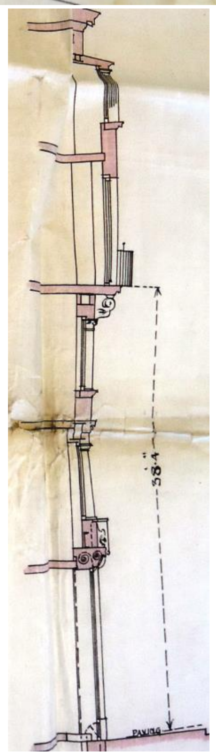


Figure 3.7 : A. Sykes 1905: 4th floor plan







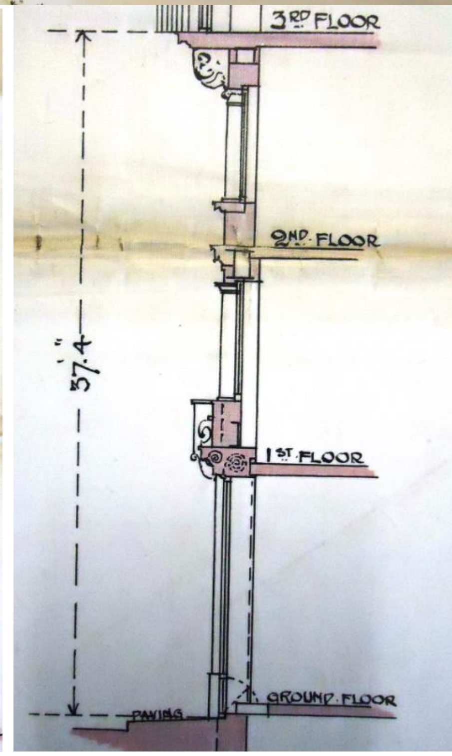


Figure 3.8 – section O-O; Figure 3.9 – section W-W; Figure 3.10 – section N-N

Figure 3.11: LCC 1938: lower ground floor plan

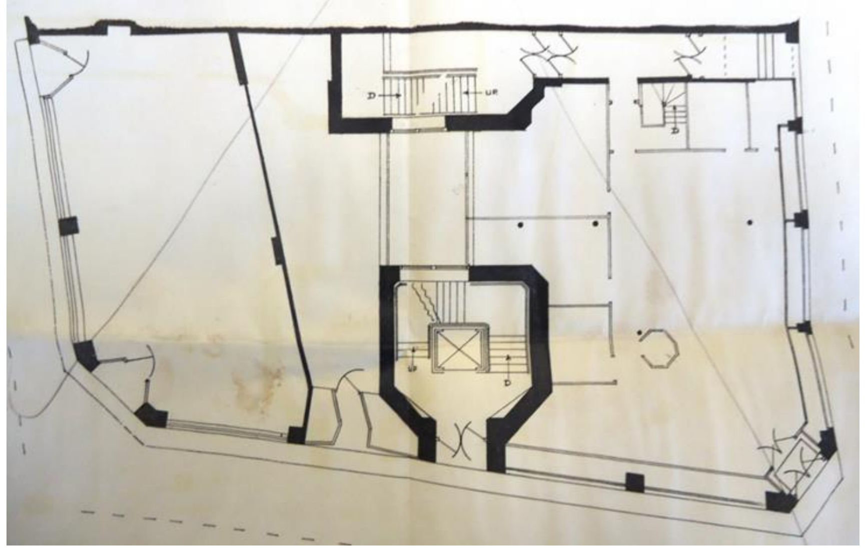


Figure 3.12: LCC 1938: ground floor plan

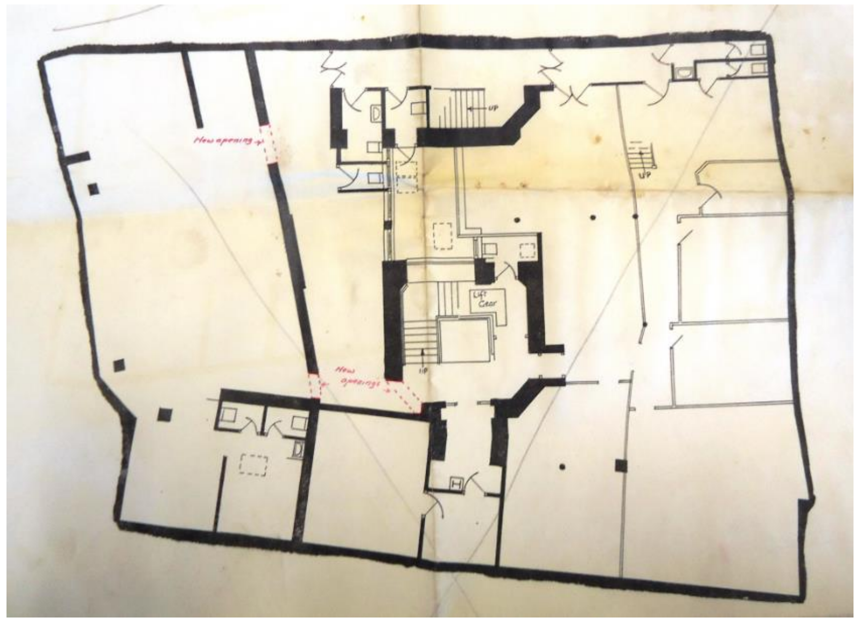


Figure 3.13: R.A. Bingham 1954: ground floor plan

3.6 Existing Structure

The existing floor plate is constructed around a centrally located masonry core. The perimeter walls forming the building facades are of masonry and stone construction and have been confirmed to be loadbearing at the upper floors. The position of openings in the masonry façade change at each level and consequently it is possible that the full façade is load bearing including the decorative 'columns'. Steel transfer beams are assumed to occur intermittently within the masonry façade at various levels to accommodate façade detailing and past alterations.

As seen from the historic planning drawing and documents, to accommodate the current shop frontage, steel columns and beam transfers were installed at ground floor and lower ground floor level.

The floor plate consists of clinker/steel filler joist slabs 200mm deep and spanning bays of approximately 3.5m. The clinker/filler joists are supported by a system of steel floor beams (assumed), often partially encased within the slab. The beams span a maximum of 7m.

The steel beams bear onto the loadbearing masonry core; party wall; perimeter piers and an arrangement of internal columns. The internal columns found along the line of the former party wall comprise of steel I sections with steel plates bolted to each flange. The bolted plate dimensions increase towards the lower floors. Each column is either encased or set into masonry. The columns offset from this line are circular steel sections.

At ground floor level the steel beams and columns transfer the façade loads over the shop front openings. The sizes and detailing of this structure is currently unknown and needs to be verified by investigative works.

Lateral loads are currently transferred through the rigid floor plates. The loads are subsequently transferred to the masonry perimeter and core which act as shear walls. The shear walls then transfer these loads to the basement and foundations.

At basement level all loads are transferred to assumed mass concrete pad and strip footings via an arrangement of steel columns or masonry piers. The masonry walls are supported on brick corbel footings on shallow MC strip footings approximately 600Wd x 300Dp. The existing façade and perimeter columns are supported on MC footings approximately 2.95x2.65x1.2m Dp. The core is founded on a large mass concrete footing approx. 1.2m deep. Figure 3.14 shows the expected basement plan and Figure 3.15 the typical existing floor plate.

3.7 Party Walls

The existing building shares a party wall with 62 Parker Street and 4 Great Queen Street (occupied by The Red Rooms but owned by GMS estates). Archive drawings show that this is a 4 storey mixed use building, with a bar at 1st and ground floor and offices and studios above. The structure has a partial basement which is currently used as changing rooms. The building is likely to be founded on mass concrete strip footings under the load bearing walls and a ground bearing slab. These buildings were included in the original survey by Green Hatch Group Ltd.

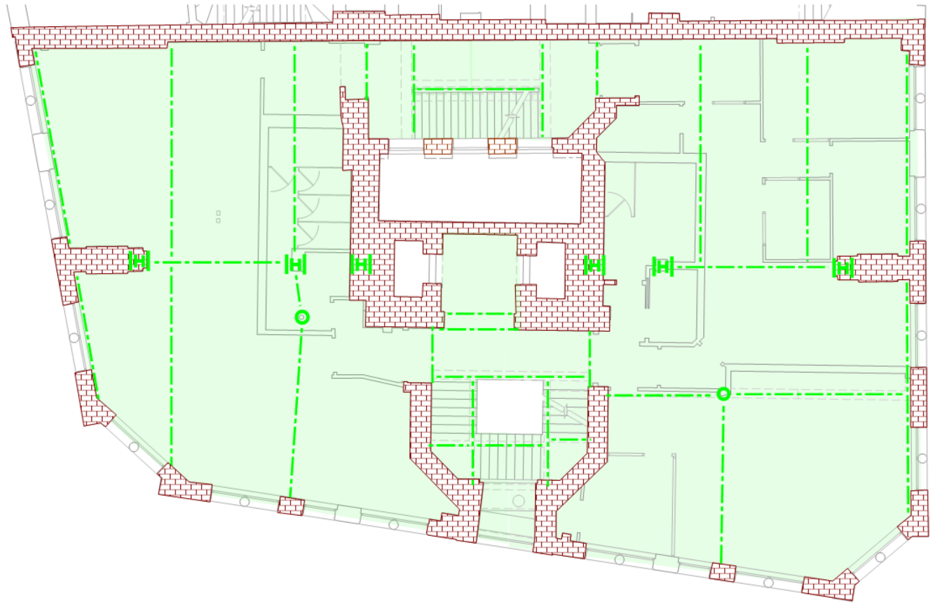


Figure 3.15: Existing Typical Floor Plan

GBG completed pilot holes to confirm the Party wall thickness. The party wall thickness varies up the building; from basement to ground it is 690mm thick, from ground to second it is 565mm thick and above 460mm thk. Figure 3.16 shows the build up plan from the GBG survey.

Figure 3.14: Existing Basement Plan

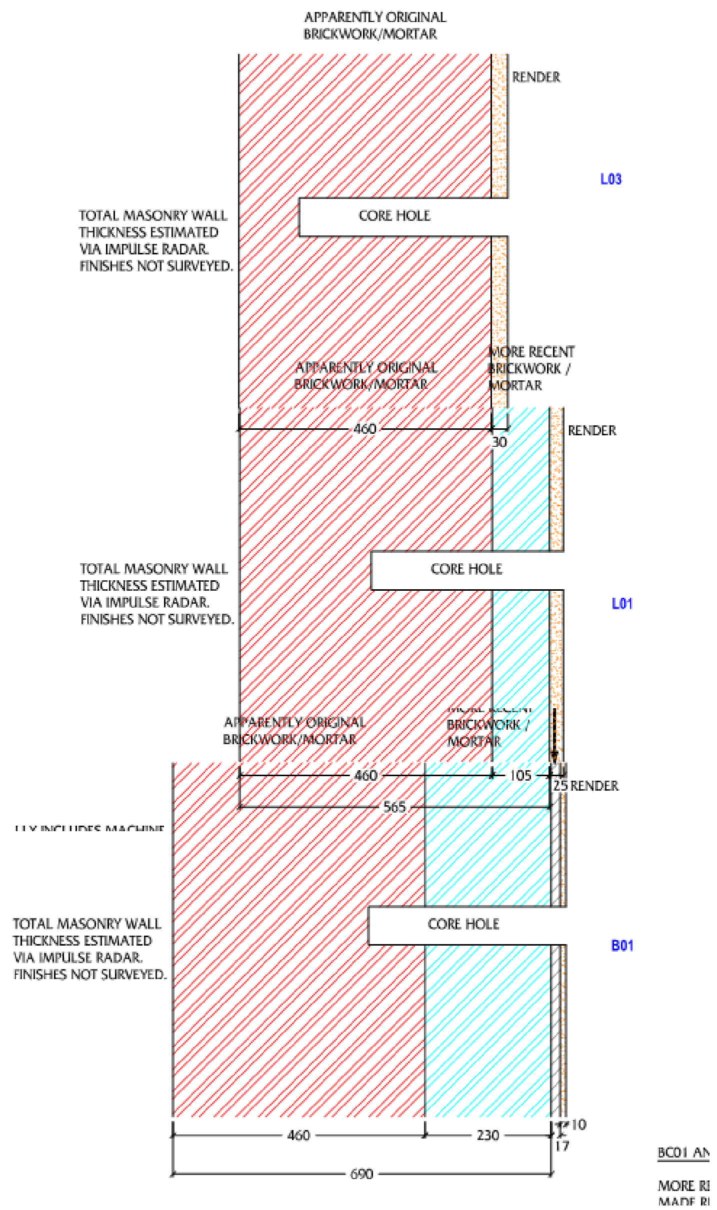


Figure 3.16: Party Wall Thickness

3.8 Existing Below Ground Drainage

Sewer records

Thames Water sewer records have been obtained which confirm that the sewers in the roads surrounding the site are combined water sewers. There is a 1219x813mm sewer in Great Queen Street, a 1676x914mm sewer in Parker Street and a 1397x813mm sewer in Kingsway.

CCTV

A CCTV survey was carried out by Clearview Surveys to confirm the condition and configuration of the below ground drainage network. The survey has confirmed that the existing below ground drainage is combined (foul and surface water together) and that there are three outlets to the Thames Water public sewers in Kingsway, Great Queen Street and Parker Street. The existing drainage outlets are all 150mm diameter and the outlet pipe invert levels vary between 0.6 – 1.0m below finish floor level. Further CCTV surveys are required to confirm the size and condition of the existing outlets.

Flood Risk

A site specific Flood Risk Assessment (FRA) was undertaken to accompany the BREEAM submission for Kingsway House. This FRA confirmed that the site was at low risk of flooding from all sources.

3.9 Ground Conditions

The results from the site investigation works undertaken by GEA show that below a moderate thickness of made ground (0.3 -1.2m Thick), Lynch Hill Gravel (2.5-2.6m Thick) was encountered over the London Clay Formation, which was proved to the maximum depth excavated. From available geological information Upper Chalk is likely to be found beneath the Clay at depths. Groundwater was encountered at depths 2.00m and 2.50m into the Lynch Hill Gravel.

The SI recommends that due to the tunnels beneath the site piled foundations are not suitable and consideration should be given to spread foundations like a Raft. The results of the SI confirm an allowable safe bearing pressure of 250kN/m^2 at a depth of 0.75m below basement level. For more information refer to GEA report “J17049 Rep Issue 1” and GBG report “GBG Final Report 4431”.

Figure 3.17: Superficial Deposits Plan

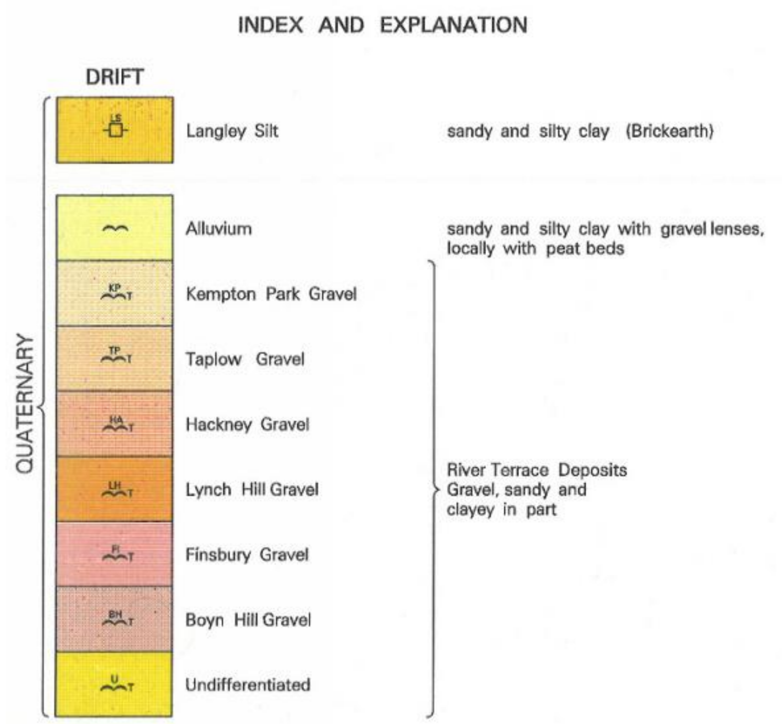


Figure 3.18: Superficial Deposits Key

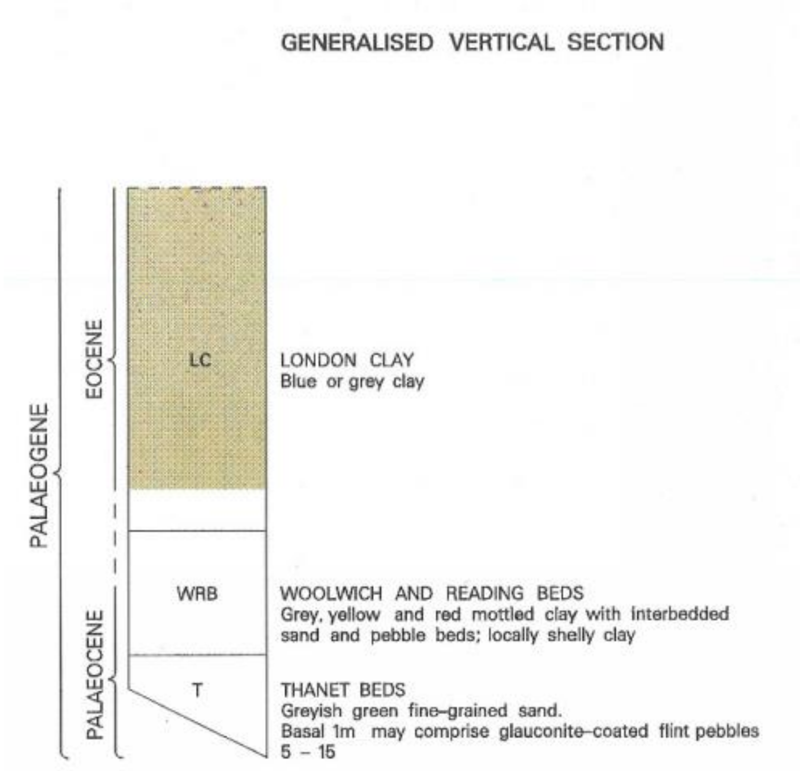


Figure 3.19: Generalised Vertical Section

4 Site Investigations and Surveys

4.1 Site investigation & recent survey

A number of surveys and investigation works were identified at the start of Stage 1 as being required in order to provide additional information on the site and existing buildings. The purpose of the surveys is to reduce the number of unknowns and to mitigate the risks on site as far as possible during the early design stages.

A number of these surveys have been carried out to date and the main structural and civil surveys are summarised in Table 4.1 below.

Table 4.1: Surveys Carried Out

Survey	Results
Green Hatch Ltd Survey	A survey by Green Hatch Ltd in August 2014 confirmed internal layout of the building including the form and thickness of the masonry cores and the presence of internal columns up to the 6th floor. The survey also suggested that the structural floor depth was approximately 200mm.
GEA Site Investigation	GEA completed site investigation works in June 2017. The scope of these works included 12 No. Trial Pits and 3 No. Boreholes. The results of these are discussed in Section 4.5.
GBG Structural Investigation	GBG completed additional investigation works in May 2018. The scope of these works included No additional trial pits and pilot holes into the existing façade columns. 200mm. The internal columns along the former party wall were masonry encased and took the form of plated steel members which increased in dimension towards the basement. The columns offset from this line were found to be steel CHS. The floor was found to comprise of clinker and filler joists to an approximate depth of 200mm, supported by an arrangement of clinker concrete encased steel beams
Line and Level Survey SAL	SAL completed line and level surveys to get an accurate location and level of the TfL Piccadilly line tunnels running under the site. A ground movement assessment is required to assess the stresses on the tunnels due to the development.
CCTV Survey	A CCTV survey was carried out by Clearview Surveys to confirm the condition and configuration of the below ground drainage network. The survey has confirmed that the existing below ground drainage is combined (foul and surface water together) and that there are three outlets to the Thames Water public sewers in Kingsway, Great Queen Street and Parker Street.

4.2 Further Surveys Required

A number of further surveys will be required during the early part of the Stage 4 design and during the demolition and construction works on site. These are summarised in Table 4.2.

Table 4.2: Surveys to be completed

Survey	Required Information/Reason
Further Structural Investigations and Material Testing	Further testing is required for the party wall, the quality of the restraint from the existing floor joists in the neighbouring structure need to be investigated.
LUL tunnel rail gauge surveys, condition surveys and monitoring	A condition survey of section of LUL tunnel beneath site is required prior to demolition and construction works. Rail gauge survey of tunnel section to be undertaken to guide monitoring criteria. LUL tunnels beneath site to then be monitored prior to, and during, demolition works and main construction works.
Movement monitoring of the retained façade during demolition and construction works	Movement monitoring must be provided for existing façade structures to be retained prior to, and during, demolition and construction. Requirements of monitoring, including tolerance limits, to be outlined in Elliott Wood structural specification.
CCTV Surveys	Further CCTV surveys are required to confirm the size and condition of the existing outlets. Refer to drawing C1000.

5 Basis of design

5.1 Codes of practice

Where appropriate, the structure will be designed in accordance with the design codes and reference documents listed in Table 5.1.

5.2 Design Loads

Permanent Loads

Permanent loads used in the analysis of the structure are shown in Table 5.3.

Variable Loads

Variable loads used in the analysis of the structure are shown in Table 5.4.

Wind/Effective Horizontal Loads

Wind loads to be calculated in accordance with the standard simplified method of BS EN 1991-1-4:2010 based on a 50-year return period. The peak velocity pressure was found to be 0.66kN/m^2.

Effective horizontal loads to be calculated in accordance with BS EN 1993-1-1:2005.

The lateral system is to be designed for the worst case of the above loading, or combination thereof, in accordance with the design standards.

5.3 Materials

All concrete is designed in accordance with BS EN 1992- 2004 and BS EN 8500. All construction is in accordance with the National Structural Concrete Specification (NSCS). See Table 5.6 for details.

All hot-rolled steel designed in accordance with BSEN 1993. All construction is in accordance with the National Structural Steelwork Specification (NSSS). All internal steelwork is assumed to be Corrosivity Category C1 (Very Low) according to BS EN ISO 12944-2-1998. All external steelwork is assumed to be Corrosivity Category C3 (Medium) according to BS EN ISO 12944-2-1998. See Table 5.5 for details.

5.4 Deflection criteria

The loading conditions applied for deflection checks are in all cases un-factored. Deflection limits given in this section are in accordance with the governing design codes and established best practice. See **Error! Reference source not found.** for details.

Deflections of concrete elements rely on underlying assumptions of the construction sequence. The contractor is to notify Elliott Wood if any of the below assumptions are not valid with their programme:

- Formwork is struck after minimum 7 No. days from casting of slab
- Cladding is installed minimum 40 days after casting of slab
- Maximum 2 No. storeys of back propping
- Outside relative humidity of 55% and inside relative humidity of 80%
- Construction live load of 0.75kN/m2
- Quasi-permanent load case used for deflection calculations

5.5 Vibration

The new structure will be designed to the following criteria as set out in BS:6472 - to ensure that vibration of the floor structure does not have any adverse effects on the building's users:

- Natural Frequency of floor slabs – 4.0Hz
- Response Factor – 8 (quiet office environment)

Table 5.1: Design codes of practice

Reference	Publication Date	Title
BS EN 1990 UK National Annex	2002 2005	Eurocode 0: Basis of structural design
BS EN 1991-1-1 UK National Annex	2002 2002	Eurocode 1: Actions on structures – General actions - Densities, Self-weight, imposed loads for buildings
BS EN 1991-1-3 UK National Annex	2003 2003	Eurocode 1: Actions on structures – General actions – Snow Loads
BS EN 1991-1-4 UK National Annex	2005 2010	Eurocode 1: Actions on structures – General actions – Wind Actions
BS EN 1992 UK National Annex	2004 2004	Eurocode 2: Design of concrete structures
BS EN 1993 UK National Annex	2005 2007	Eurocode 3: Design of steel structures
BS EN 1995 UK National Annex	2004 2004	Eurocode 5: Design of timber structures
BS EN 1996 UK National Annex	2005 2005	Eurocode 6: Design of masonry structures
NSSS	2007	National Structural Steelwork Specification 5th edition
NSCS	2010	National Structural Concrete Specification 4th edition
SCI Report P354	2009	Design of Floors for Vibration

Table 5.2: Permanent façade loads

Description	Gk (kN/m)	Notes
Typical glazed facade Curtain walling system @ 1.0kN/m^2	3.5	Based on a Floor to Floor Height of 3.5m
Typical masonry facade Metsec +100mm cavity/insulation + 100mm outer brickwork @ 2.55kN/m^2	9.0	Based on a Floor to Floor Height of 3.5m (rear façade only)
Internal wall 150mm block + 2 sides plaster @ 2.55kN/m^2	8.9	Based on a Floor to Floor Height of 3.5m

Table 5.3: Permanent loads

Description	Gk (kN/m^2)	Notes
Office floors		
Raised access floor	0.50	
300mm RC Flat Slab	7.50	
Ceiling and services	0.50	
Total	**8.50**	
Retail floors		
Screed & floor finishes	3.00	
300mm RC Flat Slab	7.50	
Ceiling and services	0.50	
Total	**8.50**	
Roof		
Finishes, waterproofing, insulation	0.50	
200mm RC Slab	7.50	
Ceiling and services	0.50	
Total	**8.50**	
PV allowance	2.00	Localised areas only

Description	Gk (kN/m²)	Notes
Ground bearing RC slab		
Finishes, waterproofing, insulation	0.5	
50mm screed	1.20	
150mm RC slab	3.75	
Insulation	0.10	
Total	5.55	

Table 5.4: Variable loads

Description	Qk (kN/m²)	Notes
Office	2.5 +1.0	Allows for movable partitions
Retail	4.0 + 1.0	Allows for movable partitions
Circulation	4.0	
Storage	7.5	Locations TBC
Roof (maintenance only)	0.75	

5.6 Fire protection

The fire ratings of all areas are to be confirmed by the appointed Fire Consultant.

Our current design is based on a minimum fire rating of 90 minute with the potential for 120 minutes for firefighting lobbies and core.

Steel elements are to be protected to the architect's details. Solutions include intumescent paint or boxing out with fire rated boards.

Reinforced concrete elements, such as RC metal deck, will be specified with the appropriate concrete cover and minimum section sizes to suit the required rating and exposure conditions.

See

Table 5.7 Vertical deflection for concrete

Element	Load Condition	Deflection Limit	Notes
Beams / Slab	Gk + Qk + Creep	L/250	+/- 25% to be applied to deflection value stated to allow for shrinkage and construction sequence
Beams / Slab	Qk + Creep	L/360 or 25mm* (whichever is less)	Normal Finish *Only applies for spans up to 10m +/- 25% to be applied to deflection value stated to allow for shrinkage and construction sequence
Beams / Slab	Qk + Creep	L/500	Brittle Finish +/- 25% to be applied to deflection value stated to allow for shrinkage and construction sequence Difference between long term deflection and deflection at time of installation of cladding and brittle partitions
Cantilever Beams	Gk + Qk	L/125	+/- 25% to be applied to deflection value stated to allow for shrinkage and construction sequence
Cantilever Beams	Qk + Creep	L/180	+/- 25% to be applied to deflection value stated to allow for shrinkage and construction sequence

Table 5.7 for details

5.7 Tolerances

As a minimum, the structure is to be built to tolerances stated in the National Structural Concrete Specification and the National Structural Steelwork Specification. Specified tolerances may differ from the NSSS or NSCS as required to suit any specific building requirements.

5.8 Disproportionate collapse

In accordance with Approved Document A – Structure, the new building at Kingsway House, 103 Kingsway will be class 2B for disproportionate collapse since it is an office structure which exceeds 4 storeys. The new structure will be designed to meet the requirements of this classification through the provision of effective horizontal and vertical ties. This will be achieved through the appropriate detailing of steelwork connections.

5.9 *Design life*

The structural frame will be designed in accordance with BS EN 1992-1-1:2004, BS EN 1995-1-1:2004 and BS EN 1993-1-1:2005 which provides a design life of 50 years.

Appropriate concrete cover for concrete elements (taking into consideration sulphates, fire, carbonation, chlorides, and freeze/thaw attack) and paint/galvanising systems for steel elements will be specified as required to provide adequate protection.

Periodic inspection and maintenance will be required throughout the life of the building to ensure protection measures are performing adequately. External structures will require more frequent inspection and maintenance than internal structures due to more severe exposure conditions.

The pavement vault structure and waterproofing is to be demolished and reinstated as part of the proposed scheme. This will increase the longevity of the vaults which form part of the public pavement at ground level.

Table 5.5: Properties of steel

Element	Grade	Section Limit	Notes
Internal Steel	S355 JR	<30mm	
Internal Steel	S355 JO	>30mm	
External Steel	S355 J2	N.A.	

Table 5.6: Properties of concrete

Element	Grade	Sulphate Class	Cover to reinforcement*	Notes
RC Raft	C35/45	TBC following SI	75mm bottom 40 all sides	Exposed to groundwater
Typical Slab	C35/45	N/A	25mm	Not Exposed
Columns / Walls	C35/45	N/A	25mm	Not Exposed

** cover to also meet requirements for fire resistance & bond, see relevant drawings and calculations*

Table 5.7 Vertical deflection for concrete

Element	Load Condition	Deflection Limit	Notes
Beams / Slab	$G_k + Q_{k}$ + Creep	L/250	+/- 25% to be applied to deflection value stated to allow for shrinkage and construction sequence
Beams / Slab	Q_{k} + Creep	L/360 or 25mm* (whichever is less)	Normal Finish *Only applies for spans up to 10m +/- 25% to be applied to deflection value stated to allow for shrinkage and construction sequence
Beams / Slab	Q_{k} + Creep	L/500	Brittle Finish +/- 25% to be applied to deflection value stated to allow for shrinkage and construction sequence Difference between long term deflection and deflection at time of installation of cladding and brittle partitions
Cantilever Beams	$G_k + Q_k$	L/125	+/- 25% to be applied to deflection value stated to allow for shrinkage and construction sequence
Cantilever Beams	Q_{k} + Creep	L/180	+/- 25% to be applied to deflection value stated to allow for shrinkage and construction sequence

Table 5.7: Fire protection

Element	Duration of Resistance (min)	Form of Resistance	Notes
RC Slab	90	Cover to reinforcement	
RC Columns	90	Cover to reinforcement and minimum column dimensions	To architects details
RC Walls	90	Cover to reinforcement and minimum thicknesses	To architects details
Core & Fire Fighting Lobbies	120	Intumescent Paint or fire boarding	To architects details

6 Options Appraisal

During the period between the first issue of the stage 3 report in June 2018 and this report issue, a full options appraisal has been undertaken for the scheme. This was driven by a review of the fit-out brief by the client which now requires a more contemporary aesthetic with exposed structural soffits. The floor plate options compared were; composite metal deck on steelwork, CLT on steelwork, RC flat slab and Hollowcore planks on steelwork. Figure 6.1 shows typical details of the new proposed options.

Table 6.1 summarises the advantages and disadvantages of each option. The client's preferred option was the RC frame due to the flat soffit, aesthetic qualities, and relatively lower risk in terms of procurement compared with the CLT solution. (Figure 6.2).

Table 6.1: Advantages and Disadvantages

Floor Type	Advantages	Disadvantages
130 Thk RC Slab on 1.2mm Gauge Kingspan MD-60 V2 Deck	-Decks can be re-bundled and lifted together into place and distributed by hand -Reduced cranage requirements. -Speed of construction and detailing. -Thinnest deck maximising space between steels More downstands restricting overall height.	-Trimming out of builders work holes required depending on their size -Less aesthetically pleasing soffit -Less flexibility for services due to downstands -More cast insitu concrete required relative to precast planks
250mm Thk Precast Planks on Steelwork	-Can achieve large spans with less frequent secondary beams. -Less cast in situ concrete required -Similar weight to current scheme -Units provide a flat soffit.	-Early coordination of BWHs and services required before fabrication of planks. -Deeper steel sections required as heavier and longer spans. Less flexibility with service coordination. -Transport and handling costs are high -Finish of planks can be less aesthetically pleasing.
300m Thk RC Slab	-No or limited downstands, flexibility of horizontal services distribution. -Flexibility on site - Limited requirement for site surveys. -Short lead time -Thermal mass, fire and acoustic advantages.	-40% heavier than current scheme will have implications on raft design and slightly higher risk to ground movement & LUL tunnels -Provision of large holes can prove difficult -Quality control of surfaces dictated by site workmanship - less control. -High embodied carbon
220mm Thk CLT on Steelwork	-20% lighter and reduced depth of steel downstand -Quick erection time, no curing or formwork required and only option with no wet works -Negative embodied carbon -Aesthetically pleasing and high marketability –suits target tenants? -High quality factory finish and flat soffit	-Accurate site surveys required post strip out to coordinate CLT panels. -CLT has long lead time (8-10 weeks) and requires early coordination of services BWHs -Limited suppliers (although prices are becoming increasingly competitive based on other projects). - Additional acoustic treatment possibly required over other options.

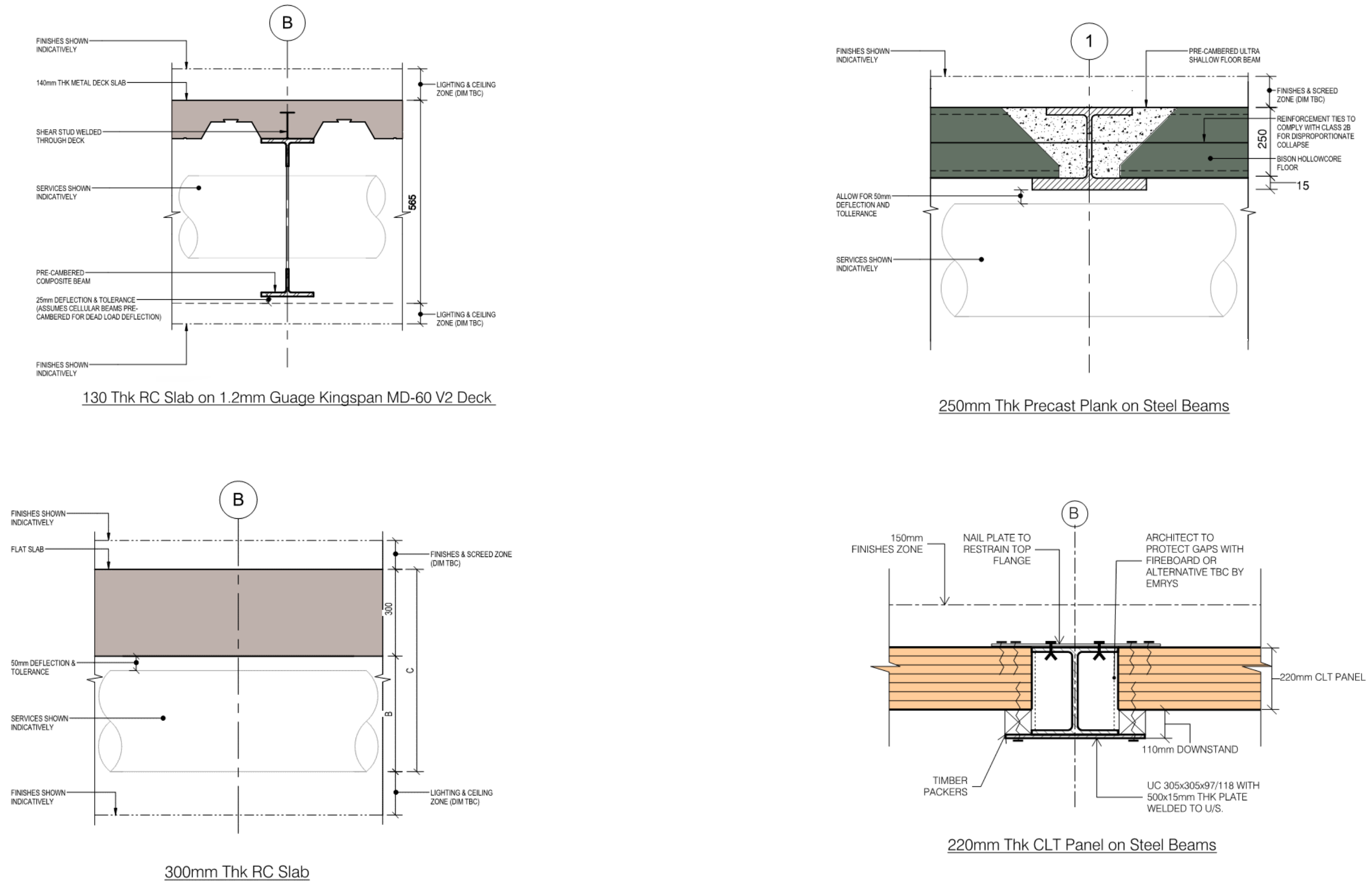


Figure 6.1: Proposed details for different options

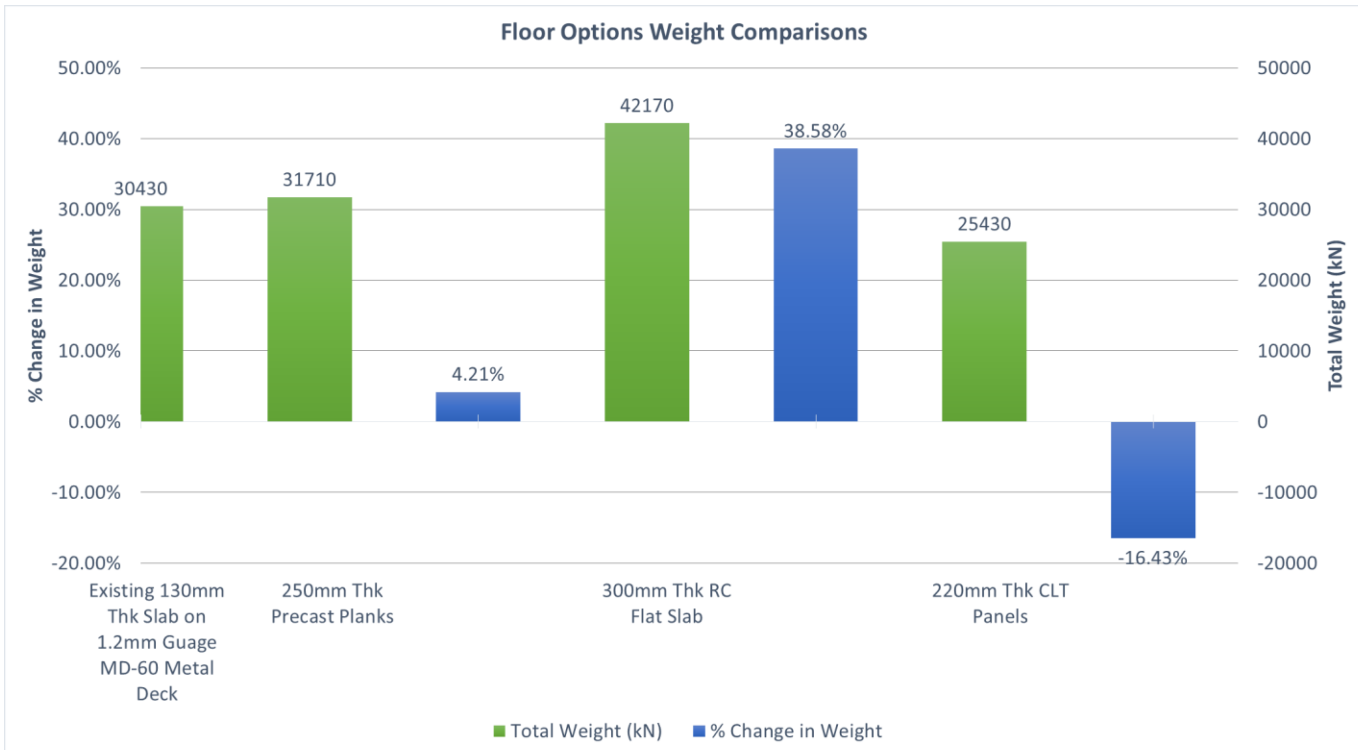


Figure 6.2: Options Load Comparison

7 Proposed Scheme

The proposed scheme retains the existing masonry façade and party wall, and removes all interior cores, stairwells, floor plates and columns to create functional open plan commercial space. This is achieved through a new structural RC frame and core, which supports a new composite floor deck at each level. Figure 7.1 shows the typical floor plate scheme. Due to the subterranean LUL tunnels running below the site, the proposed foundation is to be a shallow reinforced concrete raft, founded at the same level as the strip footings and pad foundations found in the existing condition.

8 Proposed Superstructure Design

8.1 Core

The new core is to be located along the rear party wall centrally within the building footprint, and to be founded on the new raft. The walls of the core are to be constructed in cast in-situ reinforced concrete in a slip-form sequence, allowing for a reduced construction programme in comparison to traditionally staged formwork.

Except for the rear and south wall, the core walls form continuous vertical elements from raft foundation to roof level.

To maximise the internal space, the core slightly overlaps the existing party wall where retained from basement to third floor. To support the rear core wall, which extends above third floor in line with the existing masonry wall below, three reinforced concrete columns are cast into the existing brickwork which then transfer the loads to the raft foundation. These columns are tied into the party wall so that the horizontal shear forces are transferred into the existing masonry.

To accommodate a shift in riser position at 6th floor, the south core wall steps in along GL 1. The wall at the higher level will act as a transfer beam between the party wall line and the front of the core.

The external core walls are to be 220mm thick and the internal core walls 180mm thick. Figure 8.1 shows the arrangement of walls from Basement to 3rd Floor; Figure 8.2 shows the core arrangement from 3rd to 6th Floor; and Figure 8.3 shows the arrangement from 6th to Roof.

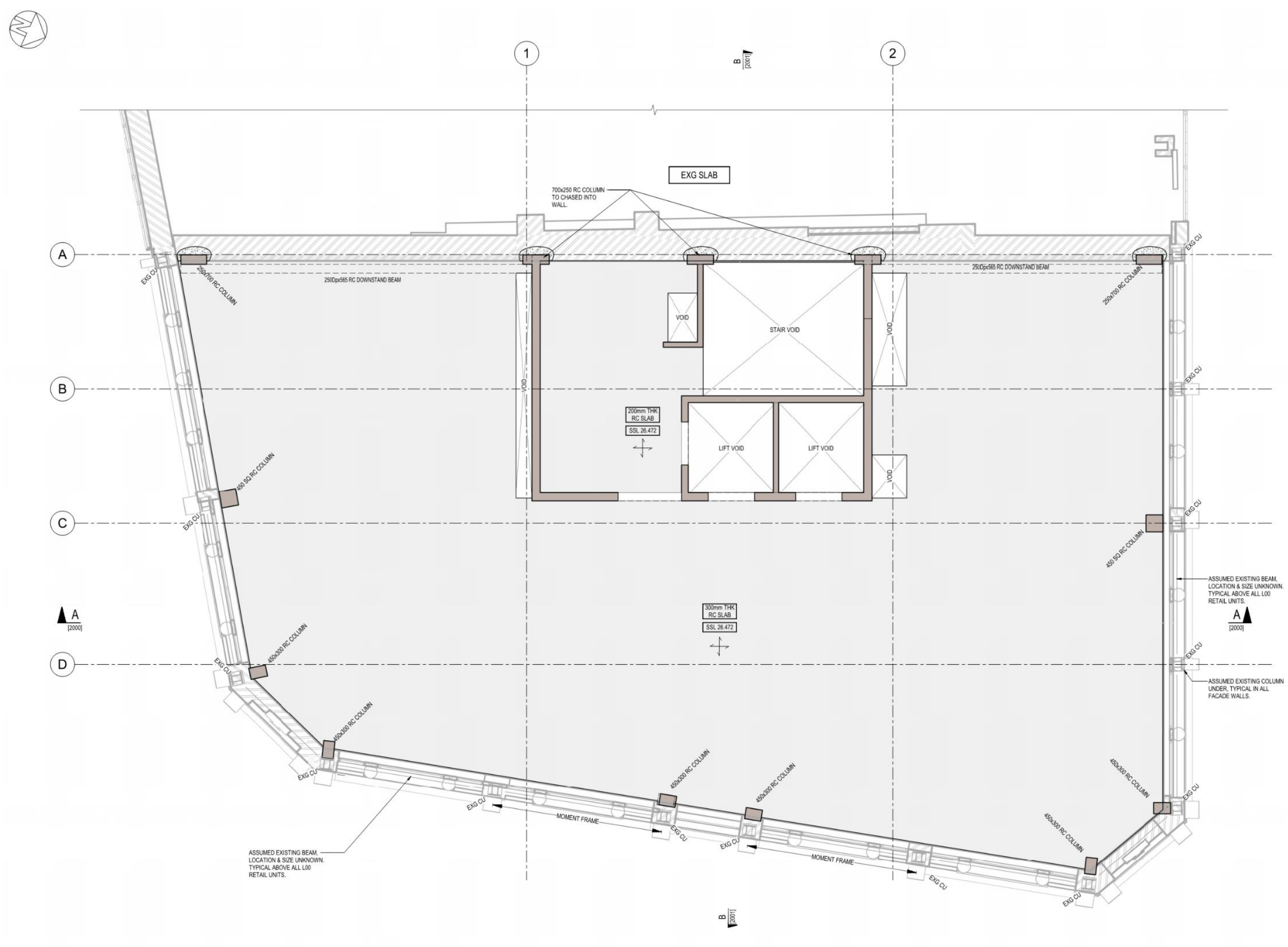


Figure 7.1: Typical Floor Plate

8.2 Stability

The new RC frame has been designed to be laterally stable without utilising any inherent stability of the façade walls. However the party wall is required for some stability at lower levels. The new RC slab will form a structural diaphragm at each level, which will distribute lateral loads to the RC core. The main stability of the structure comes from the proposed RC core. The additional moment frames in the steel scheme have been removed for the RC scheme. The core is shown in Figure 8.4.

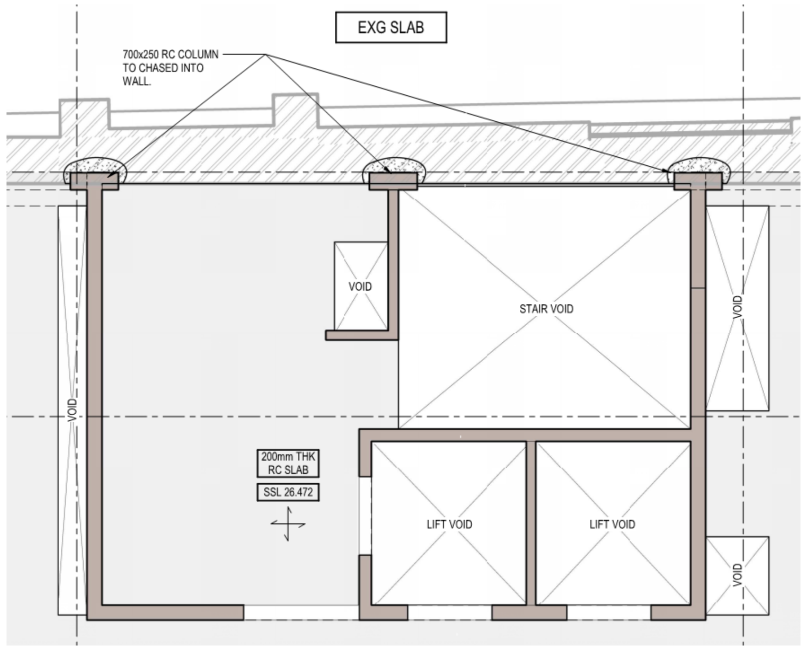


Figure 8.1: Core - Basement to 3rd Floor

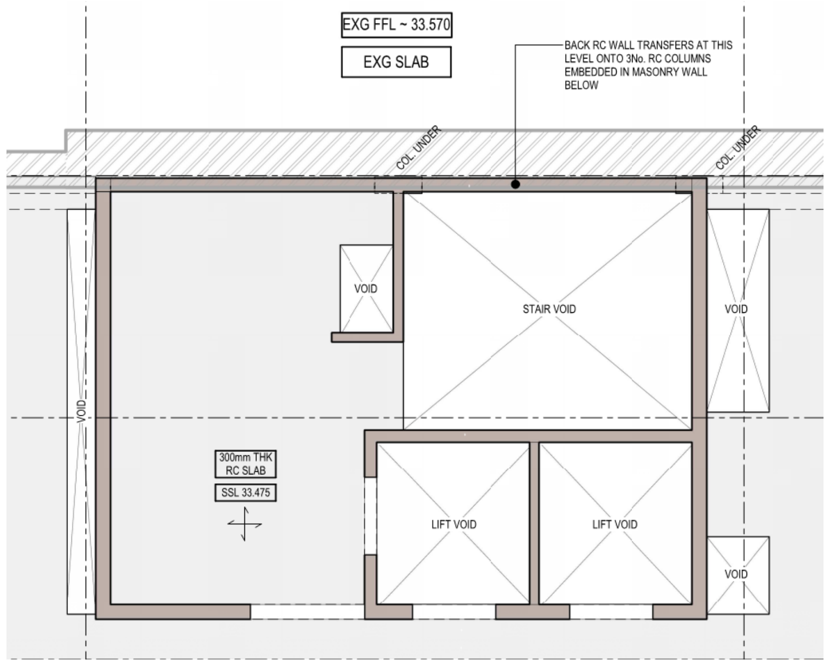


Figure 8.2: Core - 3rd Floor to 6th Floor

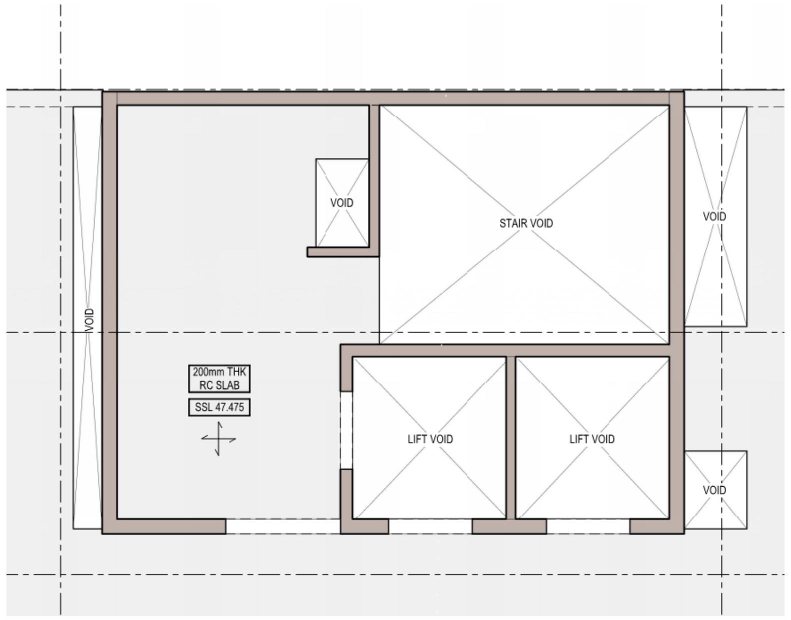


Figure 8.3: Core - 6th Floor to Roof

Figure 8.4: Stability Structure

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8.3 RC Frame

The proposed floor system spans from the core to new columns located along the building perimeter, providing an interior column free floor plate. An RC structure has been selected due to its aesthetic qualities, flat soffit and relatively low risk profile with regards to procurement. Confirmation has been received in principle from the geotechnical consultant that due to their large depth below ground the tunnels the site would not be significantly impacted by an RC frame.

The typical floor plate consists of a 300mm thick slab with a raised floor build-up supported as a flat slab on columns with varying sizes to fit within the façade constraints. The long term deflection of the RC slabs is within the limits defined in section 5.4.

The columns have been located tight against the façade to limit their intrusion into the floor plate. Due to the existing steps in façade thickness, this has resulted in some columns being chased into the façade space to ensure they are aligned vertically between floors. The chases have typically been limited to the inner half of the façade to limit the risk of damaging the external stonework.

It is proposed for the mechanical and electrical services to run through the raised floor so that the floor will have an exposed flat soffit.

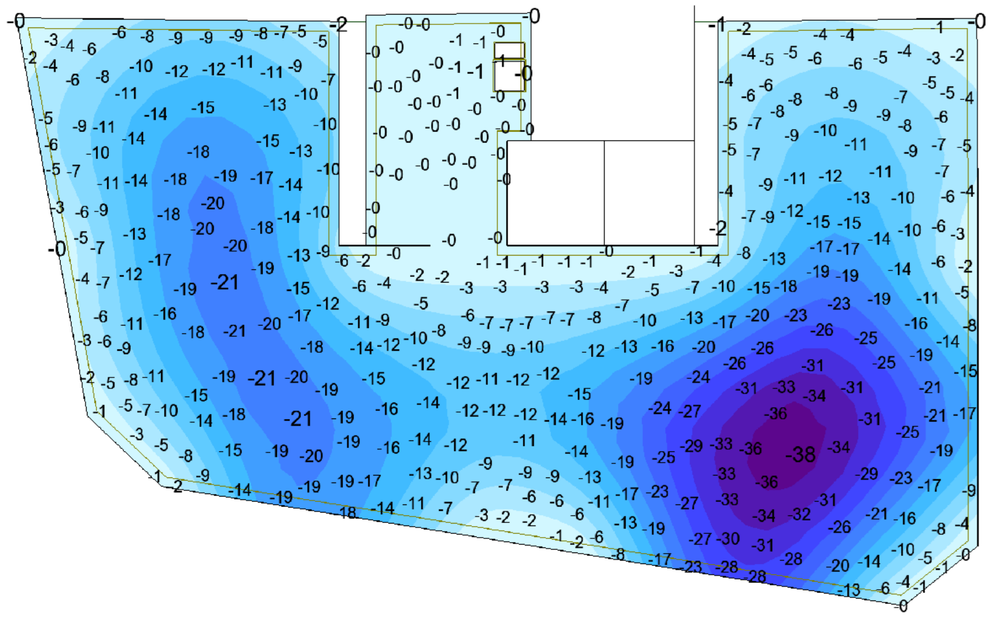

Figure 8.5: Long Term Deflections

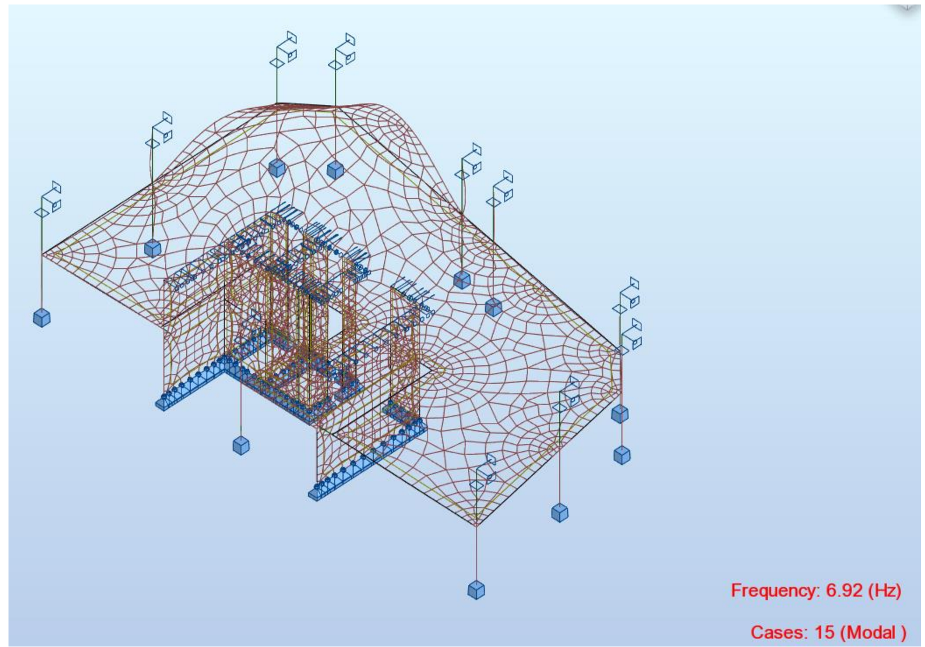

Figure 8.7: 1st Frequency of the Structure (6.92 Hz)

8.4 Vibration

The floor plate has been analysed in Autodesk Robot to check the response factors and natural frequencies. The structure has been fully fixed and no offset on slabs have been applied conservatively. Figure 7.5 shows the first mode of frequency of the structure is 6.92 Hz which is greater than 4 Hz so the structure is acceptable. Figure 7.6 shows the response factor of the structure which is around 2.62 and acceptable for office structures in accordance with Concrete Centre, A Design Guide for Footfall Induced Vibration of Structures (2007).

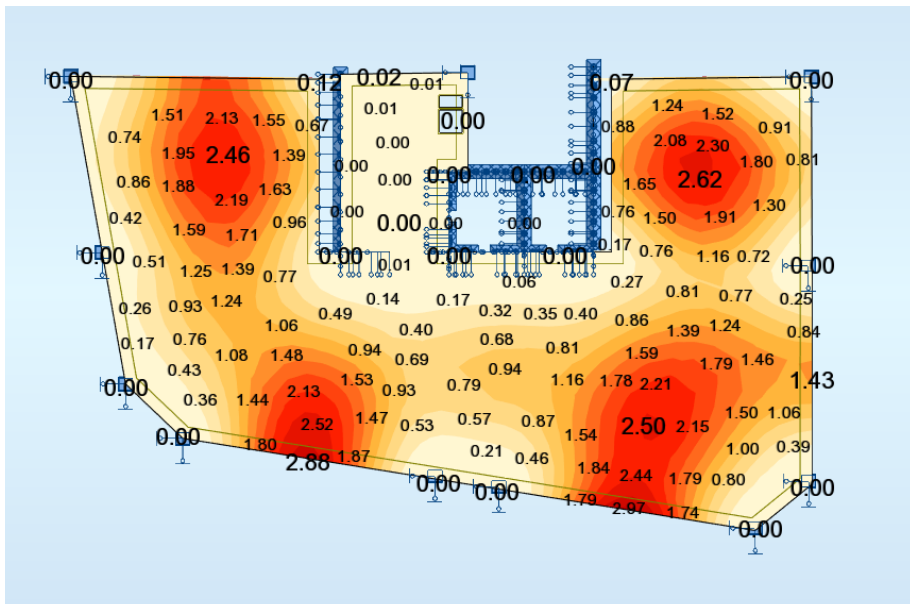

Figure 8.6: Footfall Response of Structure maximum 2.62

8.5 Façade Retention

The existing masonry façade & party wall, as well as their existing foundations, shall be retained. However the steel columns supporting the façade from basement to ground floor will be replaced. In the temporary case, a new steel structure is proposed to restrain the façade walls. The temporary steel frame will be located on all elevations, and founded on the party wall at the rear. The party wall is assumed to be stabilised by the adjacent property up to 4th floor. Although the condition of the fixity of the party wall needs to be investigated. Refer to Table 5.2.

The frame will form a ring beam every second storey with lateral loads transferred to the ground via vertical cross bracing. The retention frame is to be founded on the existing retaining walls around the perimeter of the building and along the pavement vaults. The benefit of locating the frame externally is that it keeps the internal floor plate relatively free from obstructions and it can be installed with minimal requirement for demolition works. However, the principles would need to be agreed with Camden Highways prior to installation.

The temporary steel frame can be formed from standard steel sections or lower grade RMD S275 MegaShor and SuperSlim Soldier sections. The final design will depend on performance criteria and contractor preference.

The façade retention has been analysed in Tekla structural design with RMD sections modelled. The frame is designed to resist only the wind loads. Figure 7.7 shows a 3D image of the proposed façade retention scheme showing the sections working and Figure 7.8 shows a floor plan with the trusses.

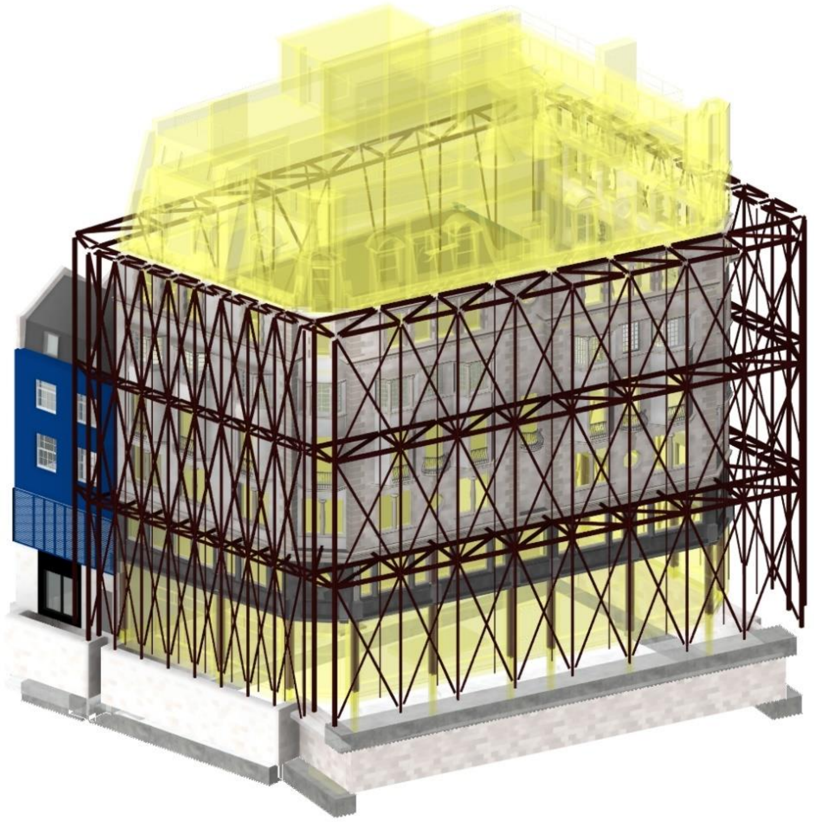

Figure 8.2: 3D render of Temporary Works

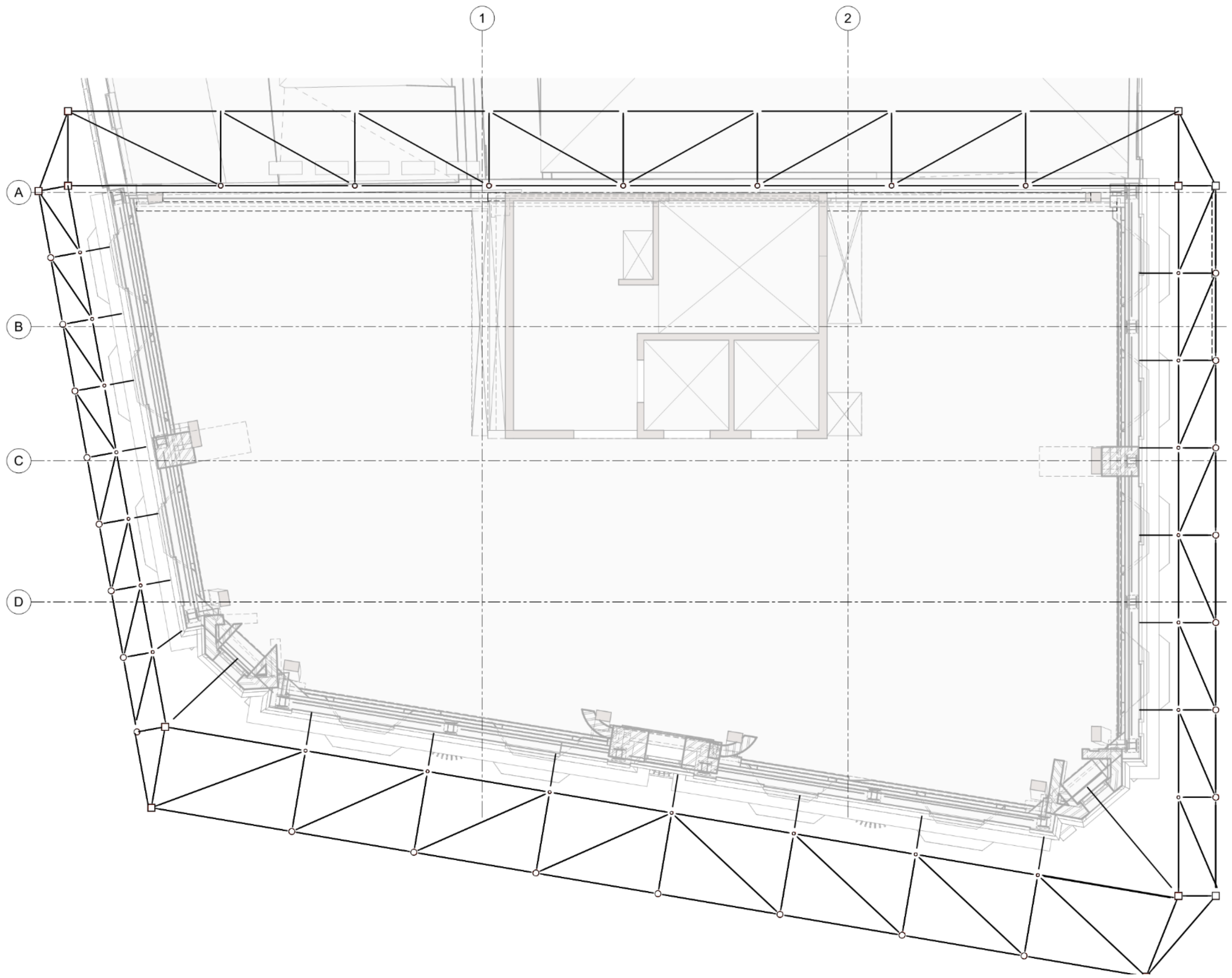

Figure 8.3: Suggested Temporary Works Plan at 2nd Floor

8.6 Rooftop extension

The proposals allow for the removal of the existing 6th floor mansard and 7th floor. These floors will then be replaced with a modern mansard over two floors framed with RC inclined columns. A final 8th floor will then be added but set back from the proposed mansard supported on CHS steel columns to minimise obstruction to the glazed curtain walling. The proposed 300mm thick RC slab will transfer the loads. The structural framing at the core will remain consistent at all floors but will be scaled back at high level to accommodate the smaller floor plate area.

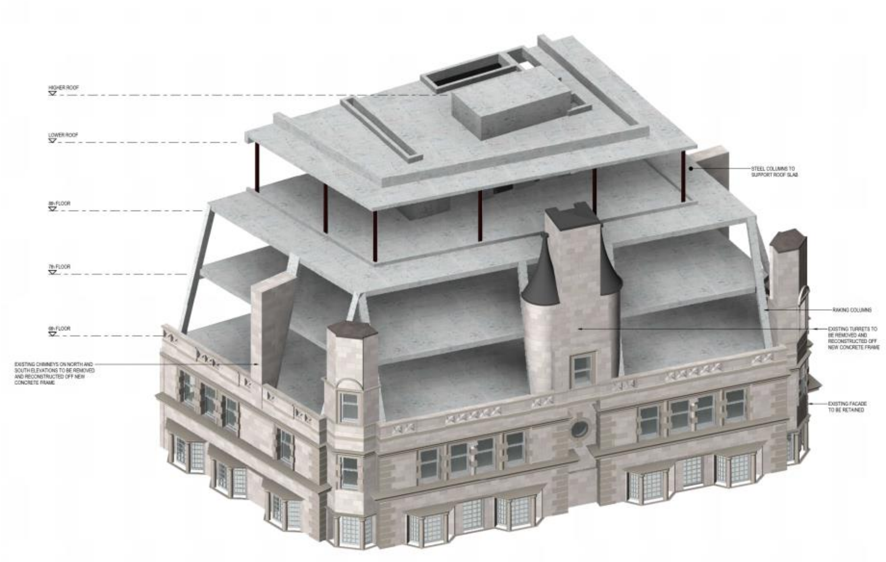

Figure 8.4: Proposed Rooftop Extension

8.7 Pavement Vaults

The existing pavement vault structure and waterproofing is currently in poor condition due to infiltration of tree roots from ground level and the resultant water ingress. The existing pavements contain a large amount of services and disconnecting or diverting these is likely to be costly and will carry a high level of risk. Therefore, it is proposed to repair the vaults from the underside.

The proposals are for new beams are to be installed to create a secondary deck below the existing structure. The existing filler joist and clinker concrete slab will become a sacrificial layer which is allowed to degrade. A new 250mm thk RC stab is to be installed spanning between the beams and this is to be cast using sprayed shotcrete by Cemex or others. The advantage of this is that it can be sprayed from the underside. Figure 7.10 shows a section through the RC Vaults.

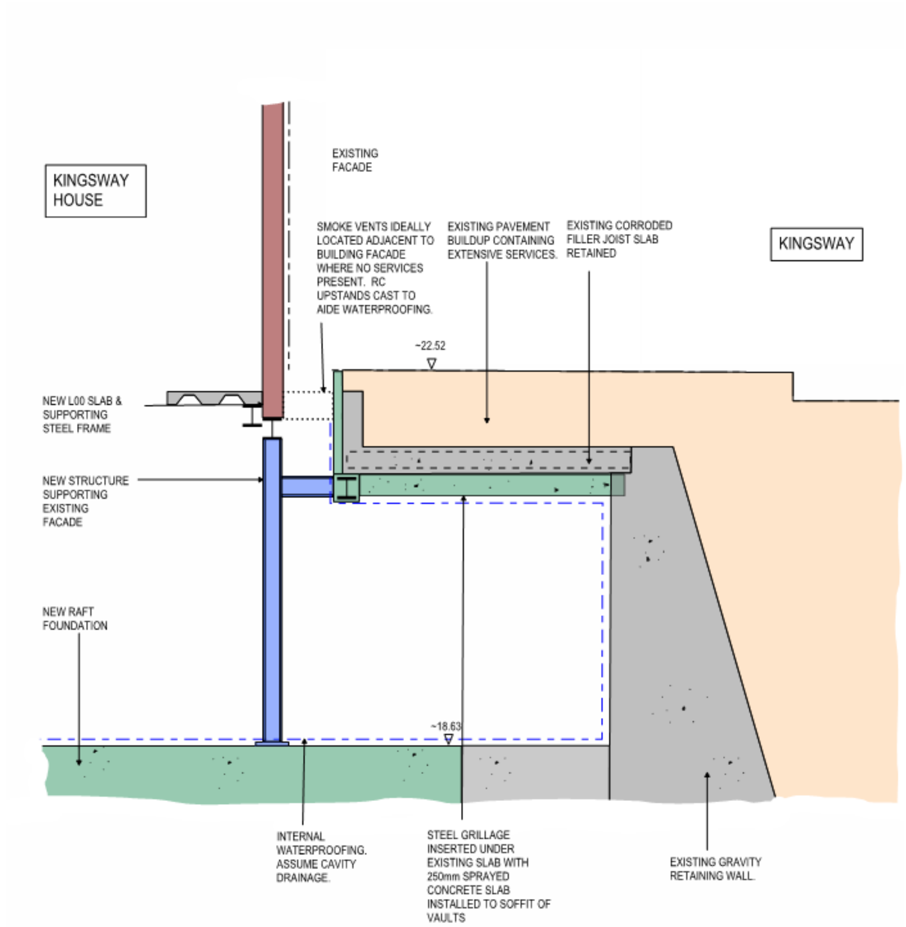

Figure 8.5: Proposed Pavement Vaults Repair Works

8.8 Servicing

Two risers at either side of the core will distribute services from source to all floors. Services can then be distributed at each floor through the raised floor zone achieving a flat soffit below the slab.

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9 Proposed Substructure Design

9.1 Scheme

Deep foundations are likely to be unfeasible due to the proximity of the underlying Piccadilly underground line. Because of this, only a shallow foundation system has been considered.

The removal of the existing masonry core and internal columns means that the existing foundations for these elements can be removed. This allows for the introduction of a new raft foundation across the internal area of the building and an evenly distributed load across the whole building footprint. This in turn will minimise the surcharge on the crown of the underground tunnels beneath the site.

The foundations for the retained masonry piers, party wall, and perimeter retaining walls, will remain. As previously outlined, the perimeter columns for the proposed scheme run, and are founded, immediately adjacent to the inside of the foundations for the existing facade. Along the party wall & north and south elevations, 1.0m wide RC 'toothing' will be pocketed into the existing masonry to tie the slab into the existing raft. Along the west façade the raft slab is to be chamfered by 150mm so the existing wall footings are not undermined. Additionally along the party wall it has been verified that the existing foundations can be justified for these limited alterations. Figure 8.2 shows the proposed basement plan.

Along the front façade the site investigation has proven pad footings in a number of locations which support the existing steel columns under the front elevation. The size of these pad footings is large enough that significant transfer structures would be required to transfer the load from the new perimeter columns to the raft foundation.

To avoid the introduction of these transfers into the scheme and to prevent the requirement for internal columns it is proposed to remove and replace the existing foundations. These will require significant temporary works which are outlined further in Section 9.3

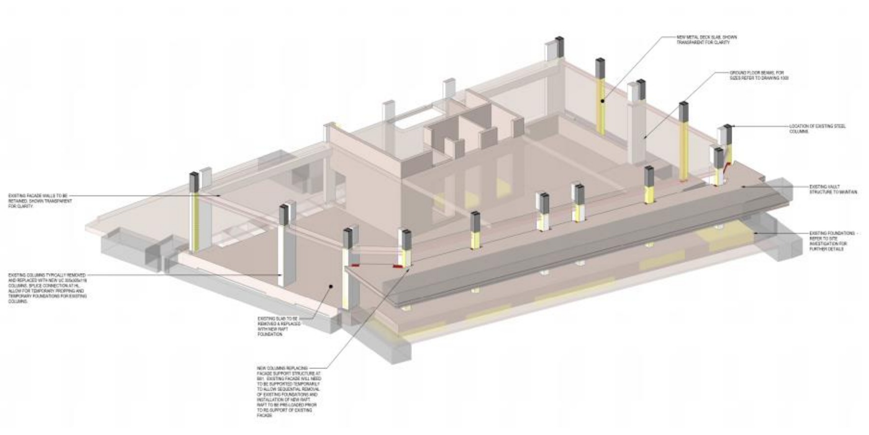

Figure 9.1: Basement 3D View

9.2 Raft Analysis

To analyse the feasibility of a raft foundation an analysis exercise has been undertaken with Geotechnical Consulting Group Ltd (GCG). The process involved constructing a Robot model of the raft and applying all of the loads from the Tekla superstructure model. The raft was then meshed into finite elements. The initial spring stiffness for the raft was assumed to be equal to 8000kN/m2, this was calculated using the allowable bearing pressure 200kN/m2 and dividing by the allowable deflection (25mm). This spring stiffness was then applied to every node on the mesh.

The nibs were modelled as cantilever beams with no support nodes underneath them. The 200kPa bearing pressure was applied upwards at the nib locations.

The analysis was undertaken and the reactions, node numbers and locations at the spring supports were exported and sent to GCG. GCG then applied these loads to their ground model and calculated the deformation of the soil strata at the node locations and sent us the calculated displacements. This process was then repeated three times to get accurate spring stiffness values.

The results of the updated raft analysis will be finalised during stage 4.

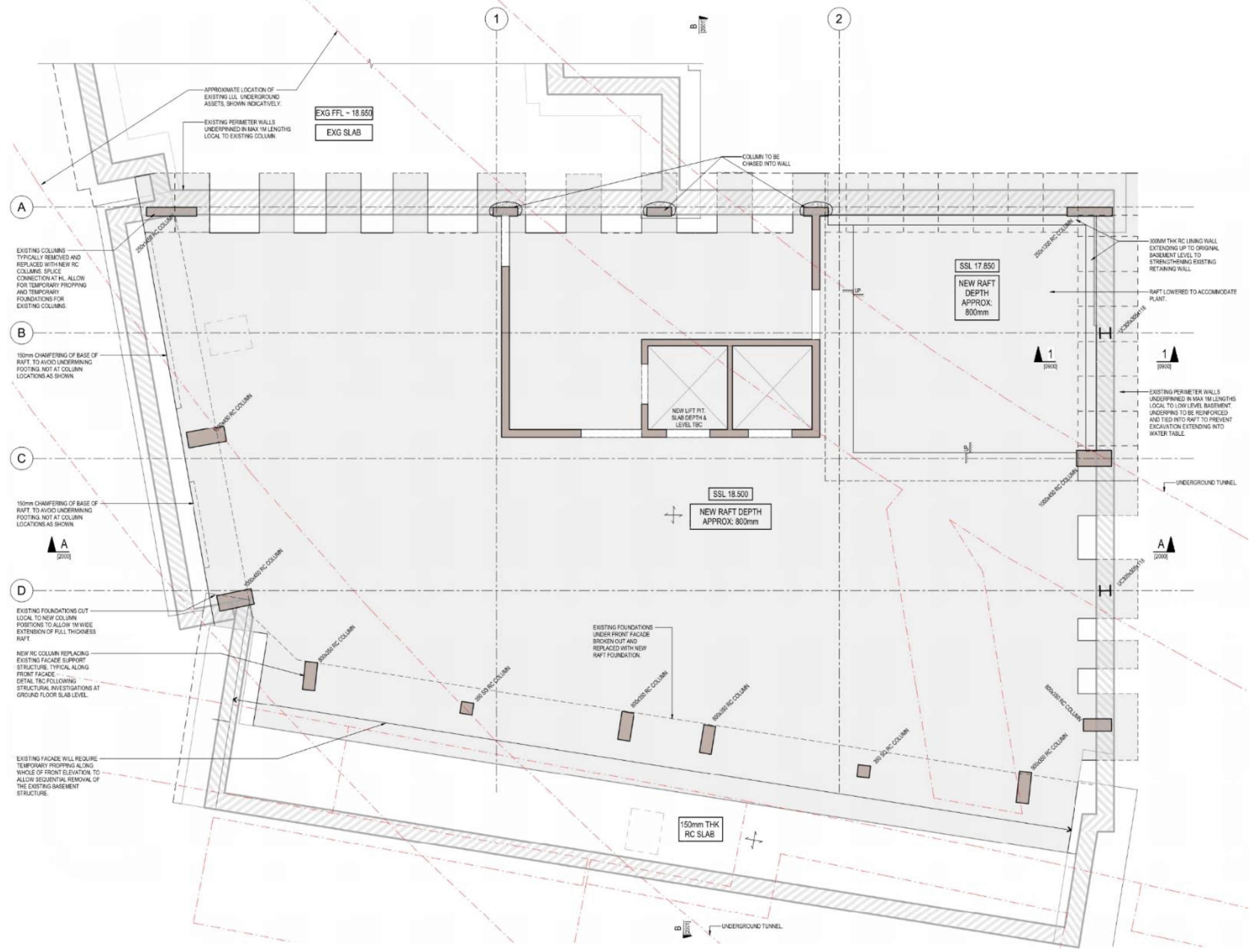

Figure 9.2: Basement GA

9.3 Ground Movement Assessment

In addition the Ground Movement Assessment will also be undertaken by GCG during Stage 4. This will verify the stresses on the existing TfL assets bellow the site and differential settlements between the existing and proposed foundations.

9.4 Basement Temporary Works

In order to install the proposed raft to the full extents of the existing basement, the existing masonry façade at the south, east and north must be supported in the temporary case. Note that on the north elevation the existing steel column extents are to be confirmed through further investigation works. It is proposed to re-support the existing façade on temporary steelwork at basement level. The temporary steelwork will to be founded on RC footings which can eventually form part of the raft foundation to limit the amount of demolition required.

From recent testing of the existing steel frame, it has been determined that the existing steel found at Kingsway House is predominantly of mild steel composition and suitable for welding. Therefore, a suggested connection between existing steel frame and temporary steelwork has been detailed to ascertain the scale of the temporary works required. The details proposes to comprise of two PFC sections welded to the flanges of the existing column at ground floor level. These PFCs are then supported on a series of transfer beams which are in turn supported on Columns offset from the existing footings. This arrangement allows for the loads from the existing façade to be transferred to ground away from the existing foundations. Figure 8.2 shows the proposed arrangement of temporary works along the east elevation and Figure 8.3 is section A-A.

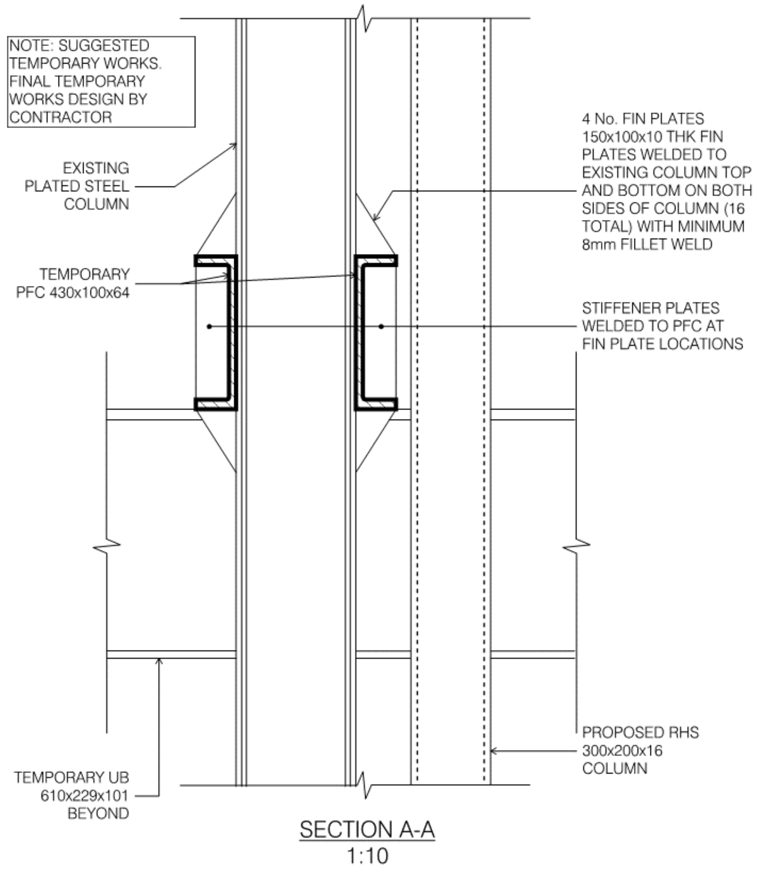

Figure 9.4: Section A-A

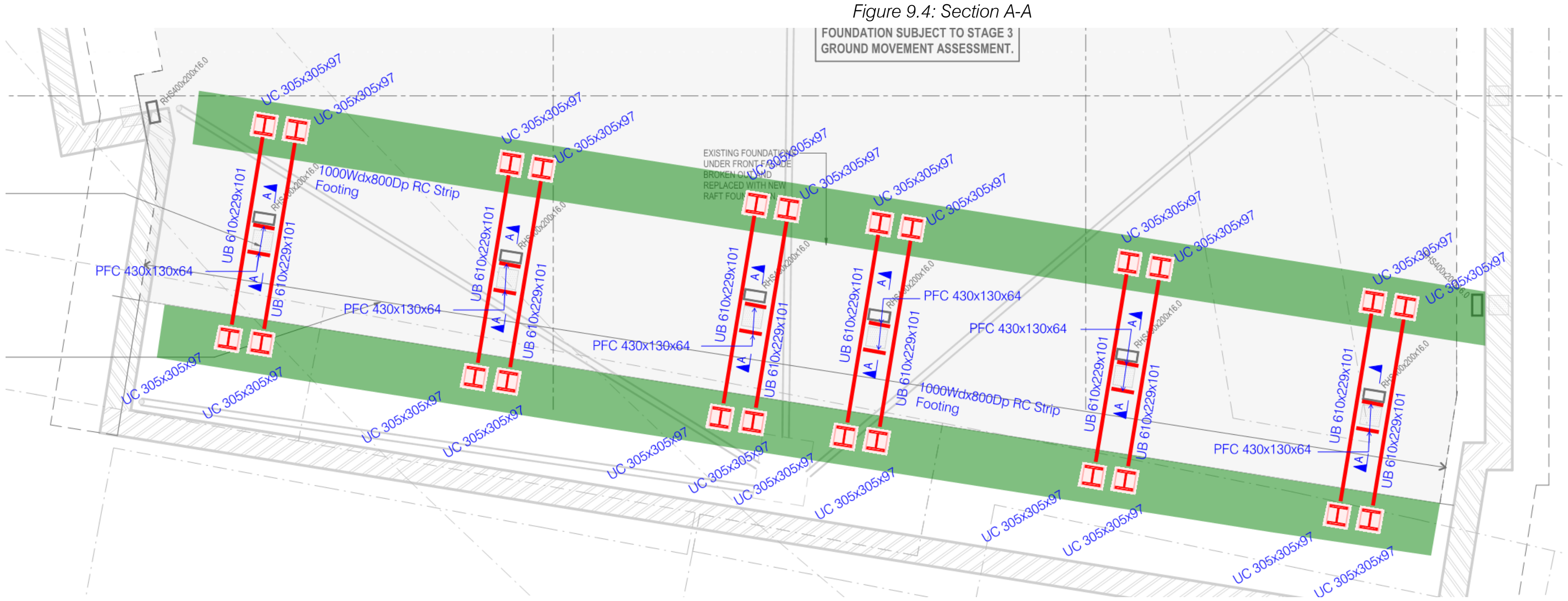

Figure 9.3: Temporary Works to Support Facade Columns

9.5 Proposed Below Ground Drainage

The proposed below ground drainage layout can be found in Appendix XXX. The proposals include the reuse of the three existing outlets to the Thames Water combined sewers (subject to condition). The below ground drainage will drain via gravity except for in the lowered plant room where a packaged pumping station will be required to lift the discharge from the floor gulley's into the outlet to the Thames Water sewer in Parker Street. The connections will be subject to a Section 106 Agreement with Thames Water. All other below ground drainage at the lower ground floor level will be removed and replaced with cast iron pipework and PPIC inspection chambers. Due to the shallow outfall levels, the new drainage network will be installed within the proposed raft slab.

There will be no increase in surface water run-off as the existing site is completely hard-standing. The proposed green roof will help to reduce both the peak and total surface water discharge from the site, whilst also bringing ecological benefits to the area. Other SuDS measures have been discounted due the lack of external space and limitations on excavation at lower ground floor level.

The drainage for the development will remain private and will be maintained by the property owners/ management company.

10 Construction methodology

An example of an indicative construction methodology for the Kingsway House redevelopment is outlined below and highlights key temporary works phases. The final methodology will be subject to input from the appointed contractor and their preferred sequenced of works.

Temporary Works Philosophy

The proposal to demolish the existing internal structure will require a facade retention structure to be installed to ensure stability of the facade during construction.

The presence of wide pavements with underlying vaults and retaining walls along the front elevation of Kingsway House provides an opportunity (subject to obtaining the relevant permissions) to install the facade retention structure along the external face of the facade. This frees up valuable space internally and minimises the potential coordination and safety issues associated with installing permanent structure between extensive temporary works.

Stage #1: Erection of the façade retention scheme

Along the front façade the retention structure shall be placed to the exterior of the building and be founded over the existing pavements. Due to the multitude of services running parallel to the pavement, all works will need to be carefully coordinated between the contractor and the service providers, however the intention is to minimise the excavations in this area and limit high column loadings to areas directly above retaining walls. As the retention structure shall encroach on public walkways and roads, permissions from the relevant bodies must be granted. Considerations must be made regarding provision of safe pedestrian flow e.g. via protected walkways through the retention structure.

Loads will be transferred back to the main external retention structure which will be designed as a series of linked, vertical cantilevers.

Further information on temporary works proposals is provided in the drawings within this package.

Stage #2: Demolition of mansard, internal structure and foundations

Demolition of mansard, internal structure and foundations occurs in a carefully managed sequence. The new temporary columns and braced bay along the party wall line are to be installed once the bracing an to propping beam is installed the party wall can be removed down to 4th floor.

Stage #3: Preparation and installation of tower crane

A tower crane for use in the construction process shall be installed within the building footprint on temporary foundations located in the basement or within the proposed lift core. The new RC core is then constructed using a slip form system

Stage #4: Construction of new structural frame/floor plates

The construction of core and floor plates shall commence from basement and continue in sequence to roof level.

At each floor level the new structure will be mechanically tied into the existing facade to provide restraint. The temporary facade retention structure can be removed on completion of the structure up to 5th floor level.

Erection of the façade retention scheme

Demolition of mansard, internal structure and foundations

Completion of superstructure & removal of temporary works

11 Risks and opportunities

11.1 Enabling Works Package

An early works or enabling works package may be beneficial on this project in order to reduce some of the risks on site prior to construction and to reduce the overall length of the construction programme. This package might include the following items:

- Soft strip, hard strip and demolition of non-load bearing internal walls and structure on vacant floors.
- Ground probing and removal of existing foundations and obstructions at the rear of the site
- Temporary works to existing façade columns.
- Underpinning, basement excavation and construction works

1 Design Risk Register

Ref	Design Risk	Risk Details	Assessment of Risk (H/M/L)	Actions/Control Measures	Risk Owner	Status/Comments
DEMOLITION, ENABLING WORKS & SUBSTRUCTURE						
1	Raft Analysis	The findings of the initial Ground Movement Assessment by GCG confirmed the viability of the proposed raft foundations and provided sufficient information to approximate the raft thickness and reinforcement quantities. Further iterations will take place in Stage 4 to determine if there are any areas of the raft which need to be stiffened or refined.	Low	Further iterations of analysis between EW & GCG to continue in the beginning of Stage 4.	Elliott Wood/GCG	Ongoing – Stage 4
2	LUL Approvals	LUL must grant approval for the works prior to commencement on site	Medium	GCG to complete Ground Movement Analysis early in Stage 4 and EW to incorporate any required adaptions to the structure prior to the report being issued to LUL for comment and approval	GCG/Elliott Wood	Ongoing – Stage 4
3	Accuracy of Internal Surveys	The accuracy of internal surveys has been limited by existing finishes. Assumptions have been made regarding exact positions of existing columns and perimeter walls which could have an effect on the setting out of the new structure and the complexity of proposed underpinning & temporary works.	High	Where possible EW/Emrys to build flexibility into the design. HC to organise additional measured surveys once soft strip of the building has occurred.	EW/HC/Emrys	Ongoing – Stage 5
4	Existing Footing Depths	Existing footings across the site have been found to extend below the formation level of the proposed raft (See markup of findings in Appendix A). The exact extent of existing footings cannot be defined but there may be an amount of graded backfill required to reach the proposed formation level.	High	EW to develop most cost effective strategy for removal of the foundations and installation of the new raft with support from the design team and consultation with contractors	EW/Design Team	Ongoing – Stage 4
5	Groundwater	The groundwater level, as approximated in the ground investigation report, has been taken into account in the design of the proposed foundations. However, the ground water level may vary in location and over time. Ground water levels in excess of those approximated in the ground investigation report may affect the proposed method of construction of the lowered basement plant zone.	High	Further ground water level monitoring required prior to and during construction of substructure. Contractor to review at commencement of Enabling Works. If groundwater levels are found to vary then the foundation details are to be reviewed to mitigate any risks.	Elliott Wood/Contractor	Ongoing – Stage 4
6	Temporary works & Approvals from Highways	Proposed external temporary works solution is subject to agreement with Local Authorities and requires an AIP from highways. The contractor is required to submit this.	Medium	Tender prelims to specify that the contractor review the existing temporary works and to take responsibility for their design or carry out any required alterations.	Contractor	Ongoing – Stage 4

7	Connection of Party Wall to floor Plates of neighbouring Building	It must be confirmed that the floor plates or internal structure within the neighbouring building provide enough restraint to the party wall to negate the requirement for temporary lateral support to be installed following demolition of the existing structure.	High	Further opening up works from within the neighbouring property required. EW to develop detail which ties the party wall back to the neighbouring structure and can be installed from the Kingsway side of the building.		Ongoing – Stage 4
8	Party wall notices	Are party wall notices required with the long term tenant?	Medium	GMS/client to advise	Client	Ongoing – Stage 4
9	Demolition & Reconstruction of Turrets	Turrets and chimneys will require considerable alterations when reconstructed to maximise internal NIA and tie in with the new steel framed mansard.	Medium	Emrys & EW to develop details further in Stage 4	EW/Emrys	Ongoing – Stage 4
10	Condition of existing retained structure	Defects within the existing structure may require alternative construction methodologies to be adopted and may affect the scope of structural works.	Medium	EW provided a schedule of investigative works based on the latest design (and which were not covered by the works carried out as part of the previous project) in the enabling works tender package. Enabling works contractor to undertake investigative works as soon as possible once given access to site.	ElliottWood/Client	Ongoing – Stage 4
11	Condition & Level of existing below Ground Drainage outfalls	The primary Kingsway sewer outfall has been surveyed and cracks discovered due to tree root ingress. The Great Queen Street outfall is also very shallow which will require the	High	EW has flagged the issue to Thames Water who have the responsibility to repair the outfall. Updates to follow in Stage 4. EW to coordinate the drainage through the raft. Cast iron pipework is to be specified for robustness and the size of the raft may need to be varied locally to accommodate the pipe runs.	EW/Thames Water	Ongoing – Stage 4

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