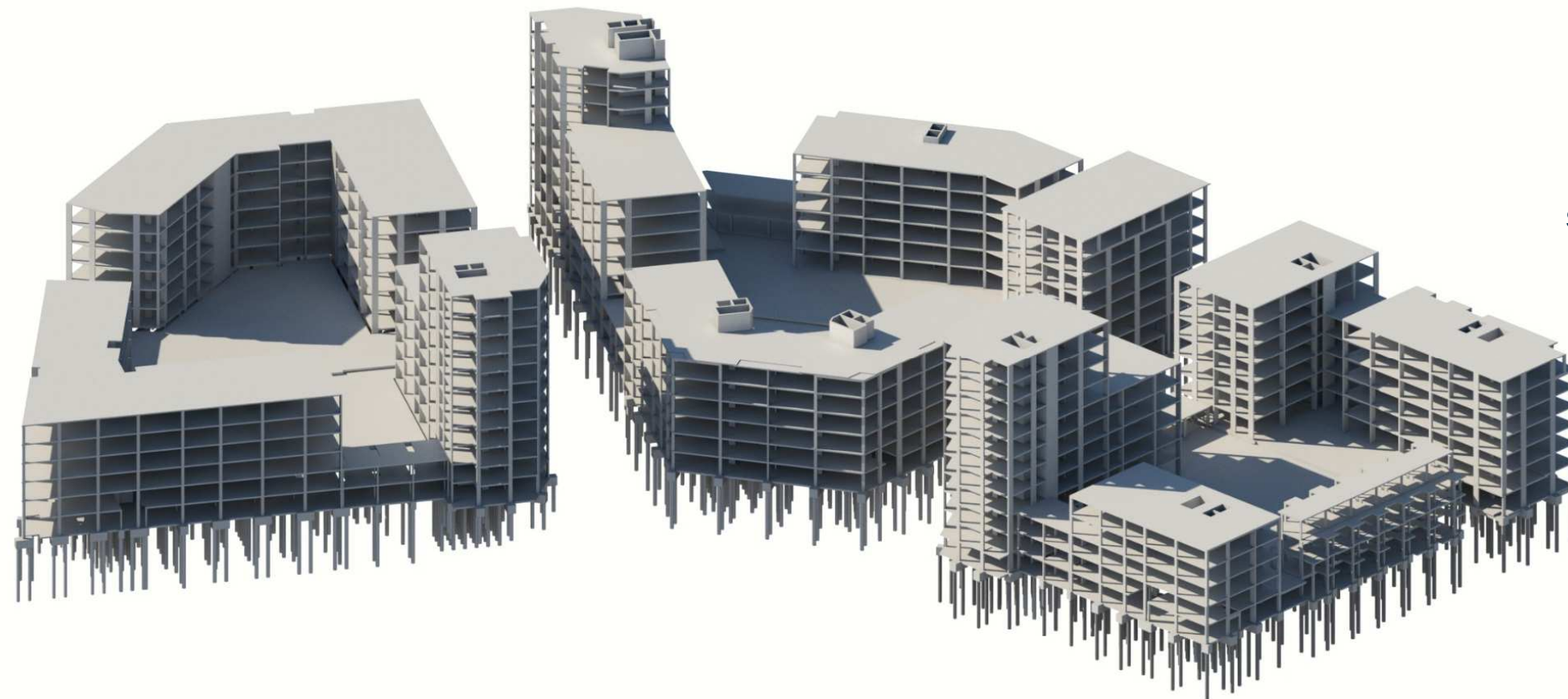




Kenavon Drive

Structural Engineering Design

RIBA Stage 3 Report

On behalf of Berkeley Homes

Berkeley
Designed for life



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1 Introduction

1.1 General

This document reports on the structural engineering aspects of the proposed development at Kenavon Drive, Reading. It describes the development since the inception of the project and explains the forms of construction which are to be adopted, with options where appropriate, and records the parameters influencing the development of the structural design.

Specific topics covered include:

- Description of the architecture in terms of the structural engineering;
- Description of the structure including substructure;
- The design loads and actions;
- The performance criteria;
- The Codes and Standards to be used.

Information is also given on construction issues, likely temporary works requirements and programme implications. In instances where investigations have not been carried out or where information from other parties is outstanding, these are highlighted.

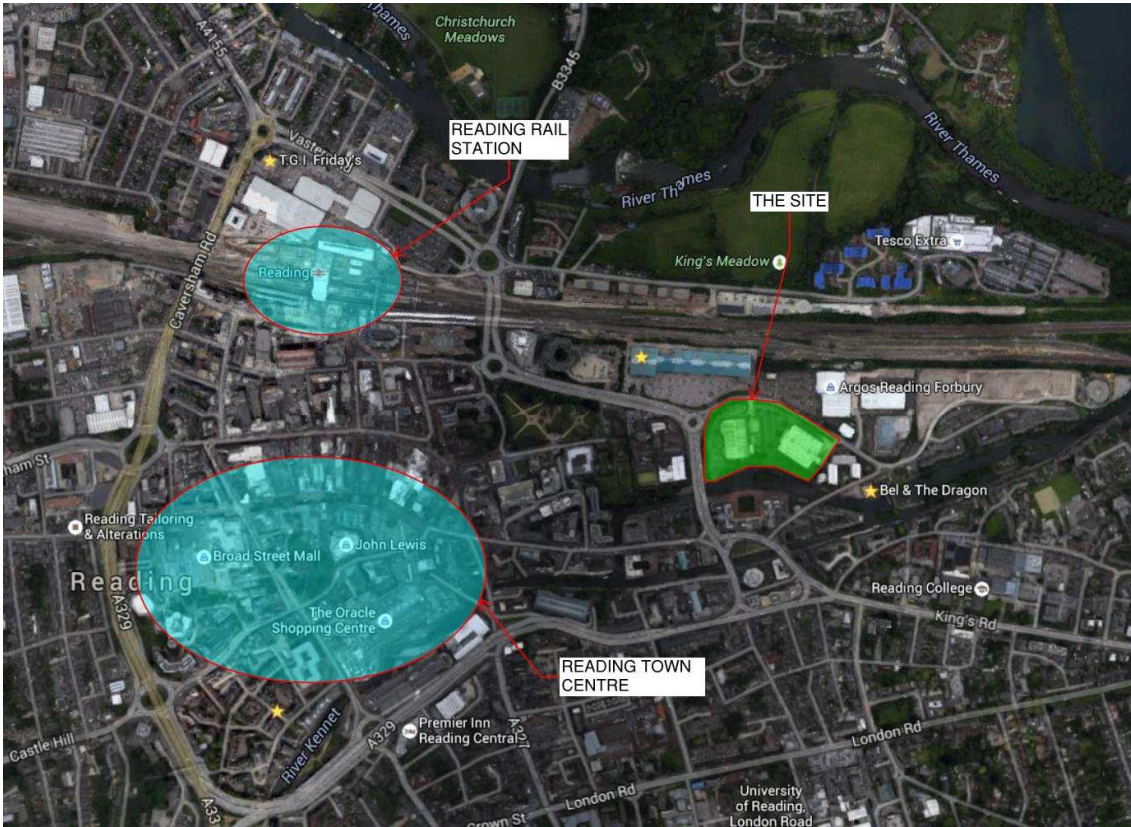


Figure 1.1: Aerial view of central Reading showing site location.

1.2 The Site

The site is located in central Reading, at National Grid Reference 472270, 173570. The Site comprises an irregular shaped plot of approximately 2.74 ha, situated to the south of Kenavon Drive. It is occupied by two large commercial retail stores, a Toy's R Us in the east of the Site and a Homebase in the west with associated asphalt car parks forming the remainder of the Site.

The Site is bound to the north by Kenavon Drive with further retail units beyond. To the west by Forbury Road and HM Prison Reading and to the south by a line of mature trees and the River Kennet. Residential apartments are located to the east of the Site.

Historic maps from 1879 indicate that the site was occupied by a Biscuit Factory and this continued until 1979. By 1990 the site had been partially redeveloped with a single rectangular retail building in the east of the site. By 1995 a second large rectangular building was present in the western area of the site. No further significant changes are noted.

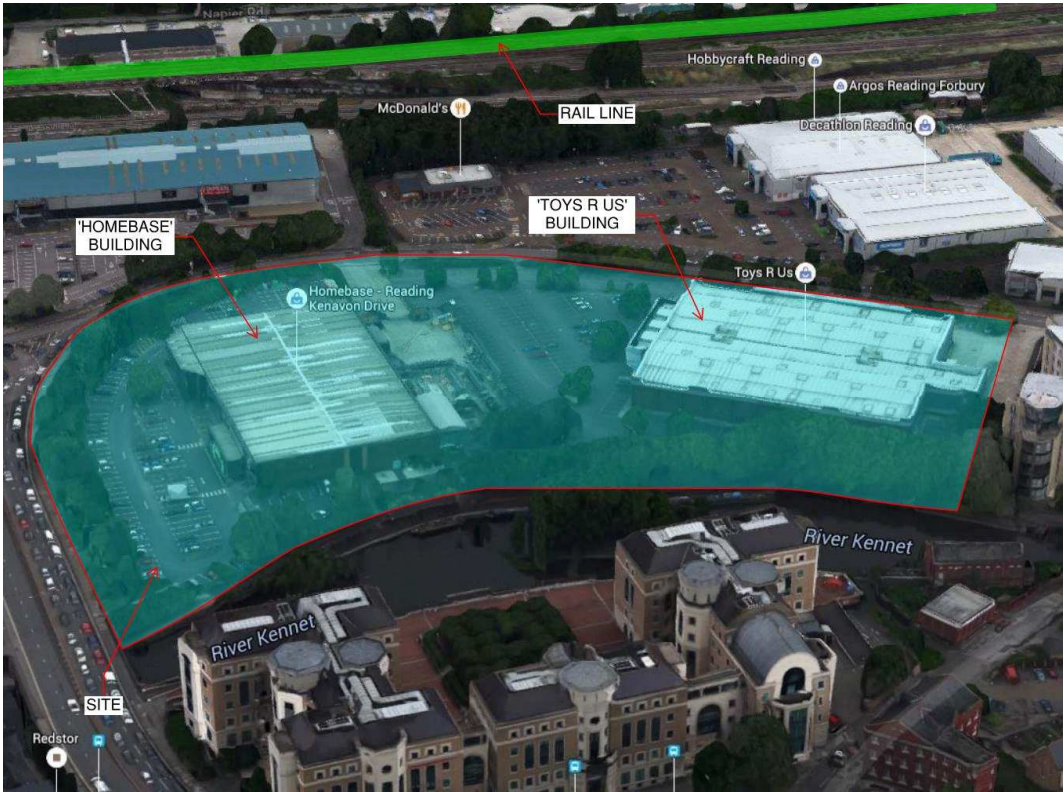


Figure 1.2: Aerial view showing the site and existing buildings.

1.3 Existing Buildings

The site was previously occupied by two large commercial retail stores, which are currently being demolished: the Toy's R Us building to the east of the Site of an approx. plan area of 4000m² and the Homebase building to the west of approx. 3500m². While the low rise nature of the buildings would suggest that shallow footings would have been utilised, knowledge of the ground conditions and the made ground thickness suggest deep foundation in the form of piles, is likely to have been used, transferring the building loads deep into the Chalk.

Once the existing buildings are completely demolished and the existing foundations removed, additional geotechnical investigation works will take place to better understand the soil conditions under the buildings.

1.4 Subterranean features and services

Utility Services

Utility services in and around the Site are present and are described in the Civil Engineering section.

Drainage

The existing drainage system is discussed in the Civil Engineering section.

Underground Obstructions

Obstructions in the ground, including boulders of concrete and a reinforced concrete slab were encountered in the south-west corner of the site.

Further underground obstructions and potential contamination due to historic usage of the site and the presence of the biscuit factory, is likely to be encountered.

Unexploded Ordnance

A previous report from Delta-Simons, dated 2013, indicates a low potential for UXO at the site, but recommends a preliminary UXO threat assessment is undertaken to confirm.

1.5 Proposed Building Development

The proposed development is to comprise a number of residential buildings of varying height, providing a total in the range of 750 units. The buildings are arranged in three separate blocks namely West, Middle and East. The buildings comprise the outer perimeter of each block with an enclosed amenity podium space at first floor level. This space will be accessible by the residents and will consist of hard and soft landscaping. The podium houses a car park on ground floor for the residents, which in instances extends under the buildings. There is a localised semi-basement area where the Energy Centre is located in the Middle block which is lowered circa 800mm into the ground to provide sufficient clear height for the required plant. On ground floor there is also provision of plant areas including a substation in each of the blocks.

The building height ranges typically from 6 to 11 storeys with 2/3 storeys link buildings between them; building E4 in the East block consists of low rise 4 storey terraced houses.

The buildings with their respective identification set by PRP are shown in the figure below.

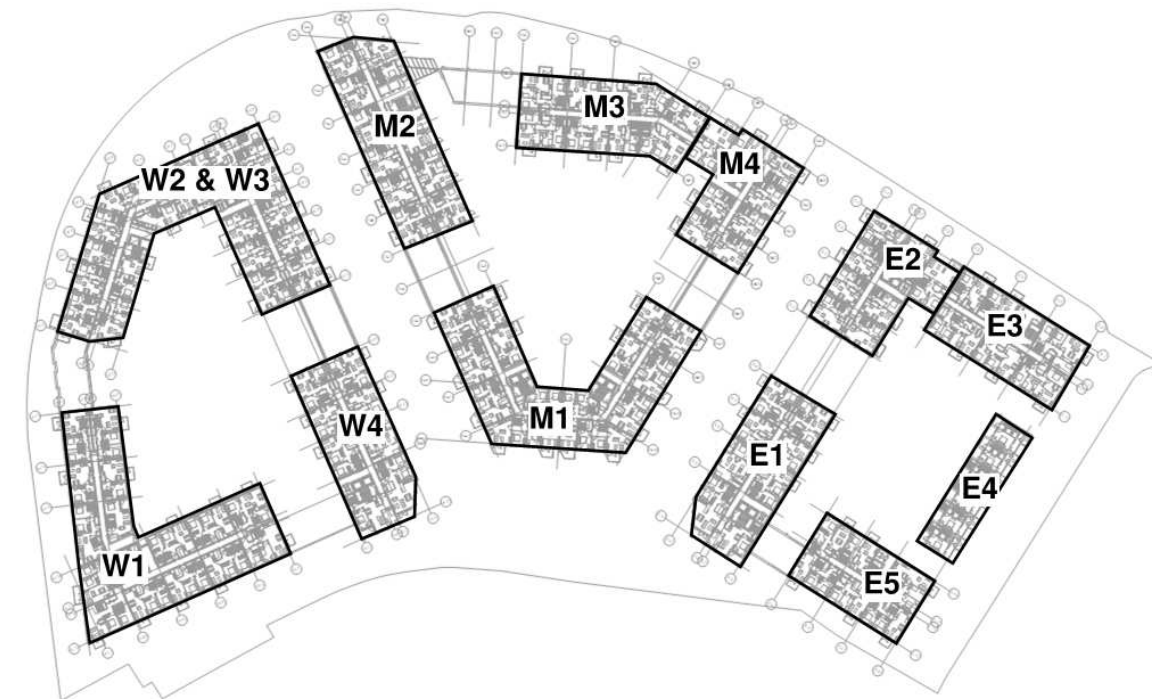


Figure 1.3: Buildings' identification plan

2 Ground Conditions

2.1 Introduction

A site specific geotechnical investigation (GI) has been carried out by Card Geotechnics Ltd (CGL) who are the appointed geotechnical consultants. The site specific GI proposed by CGL is further discussed in Section 2.4.

Reference to the Geotechnical and Geoenvironmental Interpretative Report dated November 2016 should be made. The following sections are based on information extracted from this report.

2.2 Geotechnical

Based on the historical and current ground investigations the ground conditions comprise the following:

Majority of the site

Made Ground recorded to between about 2 and 4 m depth, underlain by Alluvium (typically to 4 to 5 m depth) underlain locally River Terrace Deposits (typically to 6 m depth) and then Chalk investigated to a maximum depth of 38 m depth.

South-west part of the site

Made Ground about 1 to 2 m thick lies directly on the Chalk.

Olfactory and visual signs of contamination were not noted generally during the investigations with the exception of a hydrocarbon odour recorded in DS108 between 2.7 and 3.0 m depth.

Groundwater was recorded to be between about 2.5 and 3.5 m depth with a groundwater flow direction in the chalk from the south-west to the north-east.

Stratum	Depth to top of stratum (m bgl) [mOD]	Typical thickness (m)
[MADE GROUND] Tarmac overlying very loose to very dense dark brown to dark grey sandy gravel. Sand is fine to coarse, Gravel is angular to rounded, fine to coarse of flint, brick, concrete, chalk and clinker (boreholes WS5, WS6, WS8a, WS9, WS11 and CP01). Rare to occasional subangular to subrounded cobbles of brick, concrete and flint. Or Very soft to firm dark brown to light grey sandy gravelly clay to slightly sandy slightly gravelly clay.	0 [38.08 to 38.97]	0.3 to 5.3
Very soft to firm dark grey clay to slightly silty CLAY, with infrequent sandy lenses and occasional rootlets Light grey clayey SAND to clayey sandy GRAVEL encountered boreholes WS05 and WS08a. Sand is fine to coarse. Gravel is subangular to subrounded fine to medium of flint. <i>Encountered in boreholes WS8a, WS9, WS10 and WS11, CP01, CP02, CP04 only</i>	1.8 to 3.8 [36.28 to 36.99]	0.6 to 3.5
[ALLUVIUM] Loose to medium dense light to dark brown slightly gravelly clayey SAND to silty sandy GRAVEL. Sand is fine to coarse. Gravel is subangular to rounded, fine to coarse of flint. Or Firm dark brown slightly sandy slightly gravelly CLAY to slightly gravelly sandy CLAY encountered in boreholes WS1 and WS11. <i>Encountered in boreholes WS1, WS5, WS6, WS8, WS8a and WS9 and CP03 only</i>	1.8 to 5.3 [34.49 to 35.53]	0.9 to 2.9
[TAPLOW GRAVEL FORMATION] Chalk recovered as structureless white sandy clayey GRAVEL and COBBLES to slightly gravelly sandy SILT. Clasts are weak to moderately strong, medium dense, white with no to occasional black specks. Occasional fine to coarse flint gravel. Recovered as Grade Dm to Dc due to sampling techniques. [SEAFORD CHALK FORMATION AND NEWHAVEN CHALK FORMATION (Undifferentiated) – recovered as Grade Dm to Dc]	0.3 to 6.5 [31.82 to 38.55]	Confirmed up to 36.25m bgl

Table 2.1: Summary of Ground Conditions (extracted from CGL's report)

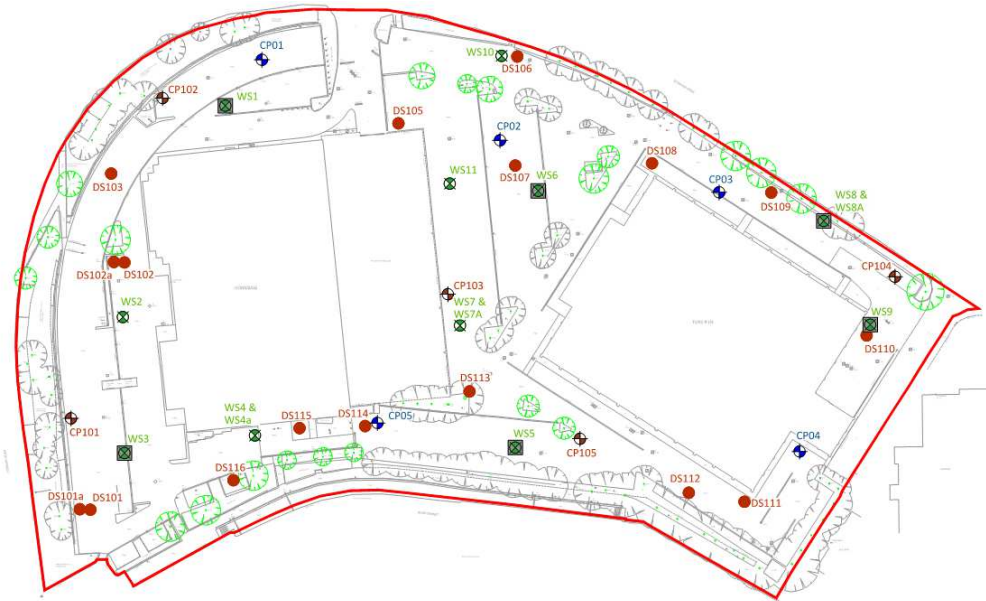


Figure 2.1: Exploratory hole location plan

SPT 'N' data						
Strata	Range		Equivalent c_u^1 (kPa)	Relative consistency/density		
Made Ground - cohesive	1 to 28 ²		4.5 to 126 ²	Very soft to firm		
Made Ground - granular	1 to 50		NA	Very loose to very dense		
Alluvium - cohesive	0 to 8		0 to 36	Very soft to firm		
Alluvium - granular	No results		NA			
Taplow Gravel Formation - cohesive	2 to 30		9.0 to 135	Very soft to firm		
Taplow Gravel Formation - granular	8 to 25		NA	Loose to medium dense		
Chalk	0 to 50		NA	NA		
Atterberg Limits						
Strata	Moisture content (%)	Liquid Limit (%)	Plastic Limit (%)	% material <425 μ m	I _p ³	Volume change potential (%) ⁴
Alluvium	33.2 to 81.3	42 to 64	20 to 38	98 to 100	17 to 40	Low to high
Shear strength tests (triaxial tests)						
Strata	Undrained shear strength - c_u (kPa)					
Alluvium - cohesive	9 to 23					
Saturation moisture content						
Strata	Water Content (%)	Bulk Density (Mg/m ³)	Dry Density (Mg/m ³)	Saturation Moisture content (%)	Porosity (%)	
Chalk	19.2 to 26.1	1.93 to 2.02	1.58 to 1.62	25 to 26	40 to 41	
Particle size distributions						
Strata	Silt & Clay (%)		Sand (%)	Gravel (%)	Cobbles (%)	
Alluvium - granular	4 to 43		10 to 34	23 to 81	0 to 7	
Consolidation tests						
Strata	Load Increment (kPa)		Coefficient of volume compressibility - m_v (m ² /MN)		Coefficient of consolidation - C_v (m ² /year)	
Alluvium	0-10		0.2 to 1.9		0.57 to 1.2	
	480-640		0.13 to 0.25		0.5 to 2.6	

Notes

- Based on $f_1 = 4.5$
- High SPT 'N' values and their equivalent c_u values in the Made Ground are due to the presence of gravel and cobbles within the cohesive strata and presence of obstructions. These results have been discounted from the undrained shear strength calculation.
- Modified Plasticity Index.
- Based on volume change potentials given in NHBC Standards 2016, Chapter 4.2 – Building Near Trees

Table 2.2: Geotechnical test data summary (extracted from CGL's report)

2.3 Geoenvironmental

Soils

The measured concentrations of potential contaminants are in general below the selected assessment values appropriate for land uses as residential land use. The exceptions comprise slightly elevated concentrations of metals (lead), locally DS106 elevated concentrations of Total Petroleum Hydrocarbons (TPH in DS106) and individual Polycyclic Aromatic Hydrocarbons (PAHs). Low level asbestos concentration (between <0.001 to 0.004 %) was detected in 7 samples.

Groundwater

The measured concentrations of potential contaminants are in general below the selected assessment criteria for assessing potential groundwater impacts on surface waters and generally below the UK drinking water quality standards. The exceptions comprise slightly elevated concentrations of lead, mercury, cyanide and individual PAHs (anthracene and fluoranthene). When comparing the concentrations of up and down gradient wells there is no obvious trend for the water samples tested.

Ground Gases

The measured concentrations of ground gases indicate slightly elevated concentrations of methane (up to 3.1%), elevated concentrations of carbon dioxide (12.2% to 14.3%) and corresponding depleted oxygen concentrations. Gas flow was not detected in the majority of the monitoring wells, however gas flow of 3.7 l/hr and 11.4 l/hr was locally detected.

2.4 Mitigation/Remediation Measures

Soils and Ground Gas

Basic mitigation measures will be required for redevelopment of the site for residential use including: health and safety for construction workers, clean capping in areas of landscaping/gardens, and ground gas protection measures. Protected water supply pipes may be required by the Water Supplier. It is considered that these actions and mitigation measures are typical for similar brownfield redevelopments and should not be considered particularly onerous or unexpected.

Groundwater

It is recommended that at least two additional rounds of groundwater sampling and testing are carried out. The groundwater testing suite should include free cyanide. Based on the additional testing a further assessment will be required to be carried out in order to establish if the elevated concentrations are related to a specific reason, or whether it is likely to reflect the background quality of the groundwater in the vicinity of the site owing to the former industrial use and general urban environment in the area, as any contamination actually arising from the site itself. *At this stage based on the available information we would recommend to remove any reference in the CGL report with regards to groundwater remediation.*

2.5 Foundations

For the ground conditions present on the Site the proposed structures will need to be supported on pile foundations due to the anticipated structural loads from the proposed development and the presence of low strength superficial deposits.

While shallow foundations could be considered for the low rise buildings in the southwest where the Chalk has been encountered close to the ground surface and the foundations would be above the water table, it is considered unlikely to be a suitable solution due to differential settlement issues at the interfaces with adjacent building and the podium structure.

Bored and cast-in-place piles formed using conventional rotary auger or continuous flight auger techniques should be appropriate although the presence of any existing foundations may form obstructions to piling works. If conventional rotary auger techniques are adopted, temporary casing may be required in the Made Ground and the River Terrace Deposits to support the pile bore and to exclude groundwater.

Stratum	Bulk Unit Weight γ_b (kN/m ³)	Undrained Cohesion c_u (kPa) [c']	Friction Angle ϕ' (°)	Young's Modulus E_u (MPa) ^c [E'] ^d
Made Ground (cohesive)	18 ^a	20 [0]	25	12 [9]
Made Ground (granular)	18 ^a	-	28 ^b	[20] ^a
Alluvium (cohesive)	19 ^a	20 [0]	22 ^e	12 [9]
Alluvium (granular)	18 ^a	-	28 ^b	[20] ^a
Taplow Gravel Formation (cohesive)	19 ^a	20 [0]	26 ^e	12 [9]
Taplow Gravel Formation (granular)	19 ^a	-	32 ^b	[30] ^a
Chalk (structureless)	19 ^e	- [5] ^e	33 ^f	- [100] ^f
Chalk (Medium dense structured)	20 ^e	- [20] ^e	39 ^f	- [500] ^f

Table 2.3: Preliminary geotechnical design parameters (extracted from CGL's report)

Preliminary pile safe working loads (SWLs) have been derived, by CGL, for the site in accordance with CIRIA C574. Separate assessments have been undertaken for the southwest of the site where the Chalk is shallow and for the rest of the site where superficial deposits are present and the structured Chalk is encountered at greater depths. Plots of the SWLs against level for the southwest and rest of the site are shown in Figures 2.2 and 2.3 respectively. The following assumptions and considerations have been used in the calculations:

1. Factor of safety on skin friction of 1.5;
2. Factor of safety on base resistance of 1.0;
3. Assume CFA piling method;
4. Design based on limiting pile settlement to approx. 10mm;

5. Skin resistance based on 0.45 x vertical effective stress;

6. Limiting skin friction of 100kPa;

7. Structureless chalk assumed to have a Qall of 600kPa based on SPT <25;

8. Structured chalk assumed to have a Qall of 1500kPa based on SPT <25;

9. Contribution from Made Ground and superficial deposits (where present) assumed to be 0; and

10. Groundwater level assumed to be 3.3mbgl in southwest of site and 2.8mbgl in rest of the site.

The proposed foundation solution is described in detail in Section 4.

Buried concrete assessment indicated soil classifications in accordance with BRE SD1:

- Made Ground: DS-3/AC-2s
- Alluvium: DS-3/AC-3s
- Taplow Gravel: DS-1/AC-1
- Chalk: DS -1, AC -1

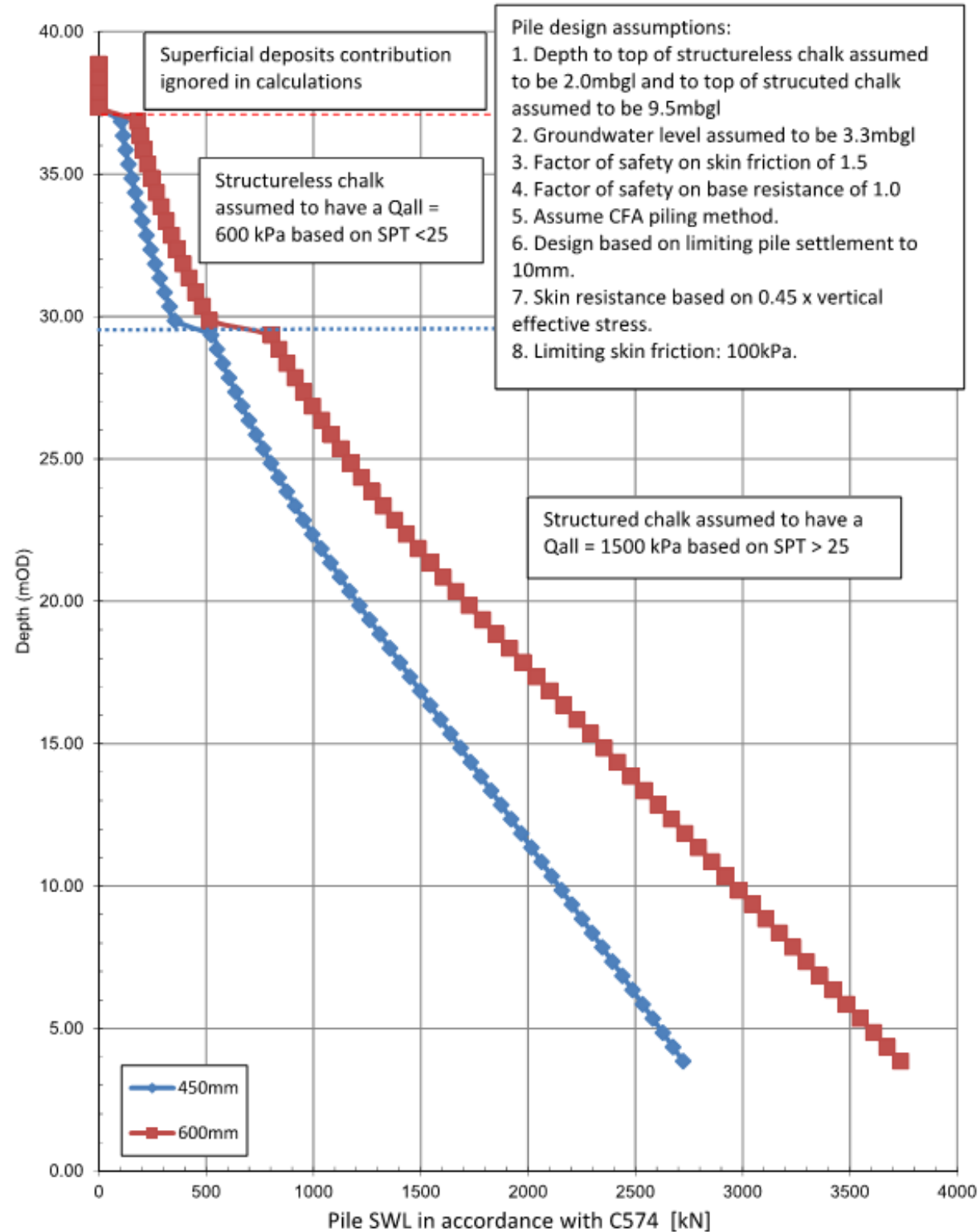


Figure 2.2: Pile safe working loads (kN) vs level (mOD) – SW of site

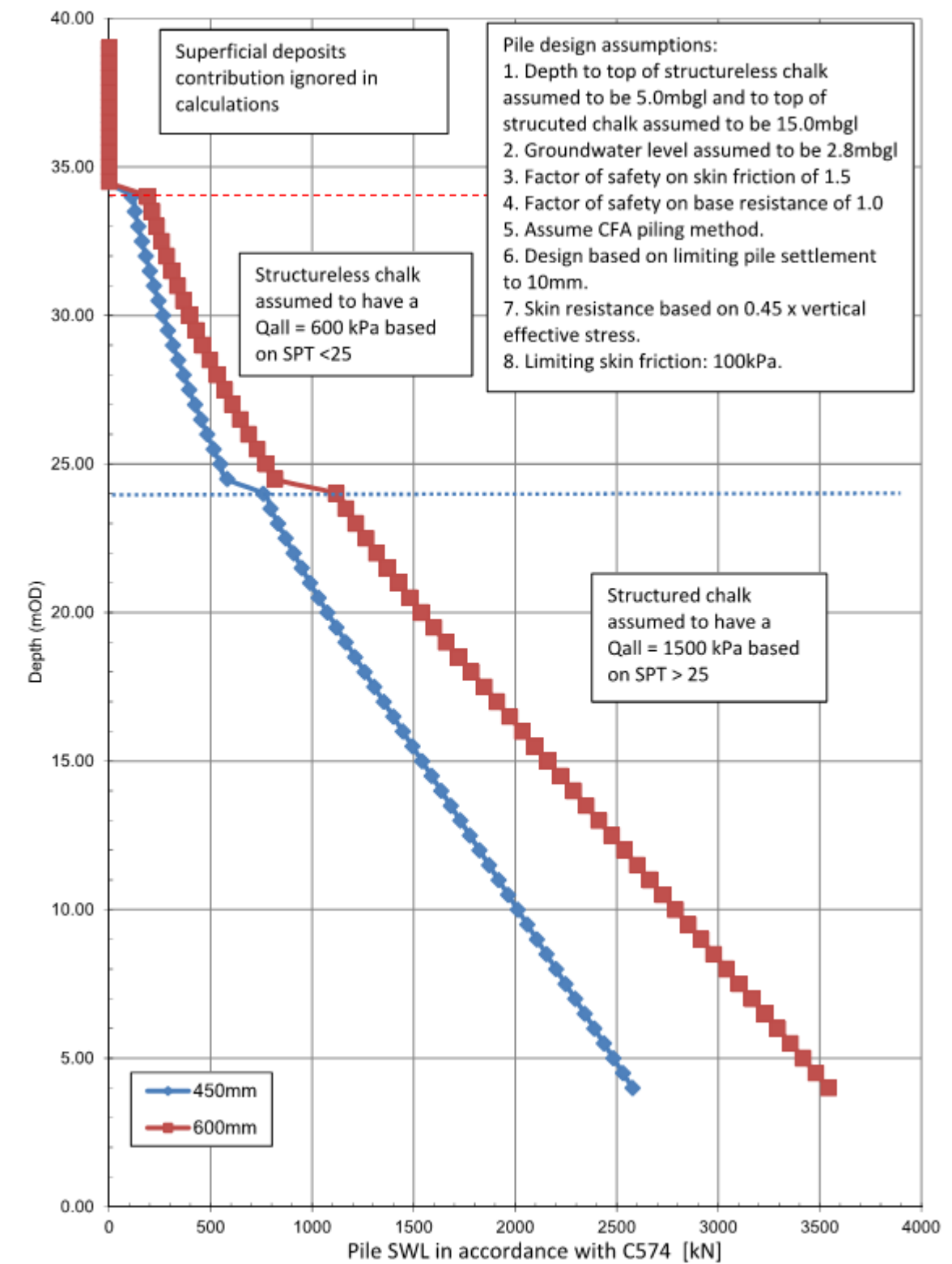


Figure 2.3: Pile safe working loads (kN) vs level (mOD) – Rest of site

3 Superstructure

3.1 Design Criteria and Constraints

The specification for the project has been developed in-line with Building Regulations, British and European Standards where applicable, and current best practice guidance with specific consideration to the following design criteria:

- Standard design criteria – such as economy, structural performance, durability, acoustics and fire resistance, etc, are all required to be met.
- Services integration – The buildings require the structure to integrate efficiently with M&E services, producing economy for both structure and services, and minimising the overall floor zone.
- Ease of construction – Consideration has been given to issues such as transportation of these materials to site, ease and speed of erection and construction programme.
- Sustainability – achieving the criteria above whilst meeting the aspirations of minimising the overall environmental impact of the construction and the operational building. Consideration has also been given to the concrete form in order to minimise material use

3.2 Floor Slabs

Floor slabs to residential buildings

The proposed upper floors are reinforced concrete flat slabs supported by a grid of reinforced concrete columns. Flat slab construction enables fairly rapid construction times coupled with flat soffits for ease of services distribution.

The residential slabs are supported on columns at a spacing not exceeding 7m typically. This results in a cost-effective solution of 225mm thick RC flat slabs. The proposed layout for the flat slabs is shown on the structural drawings.

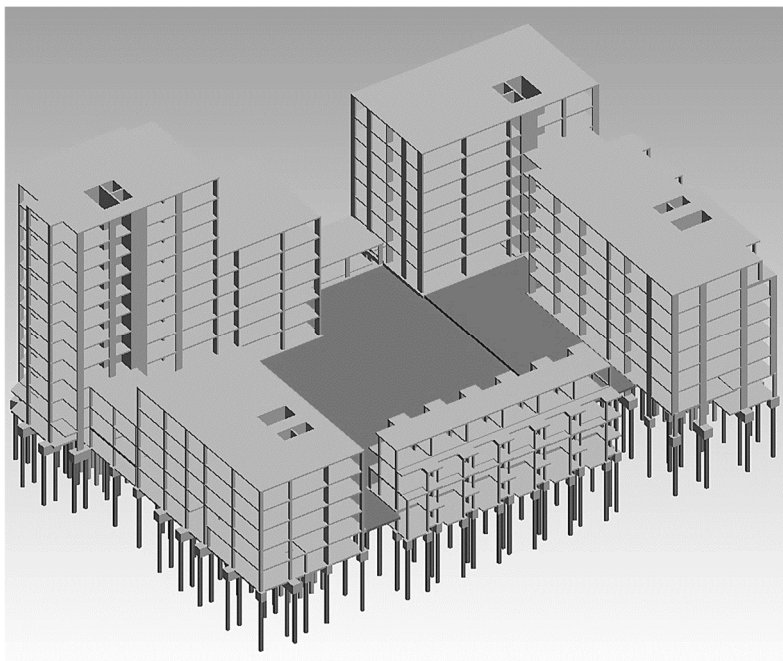


Figure 3.1: 3d view of the East block concrete frame structure

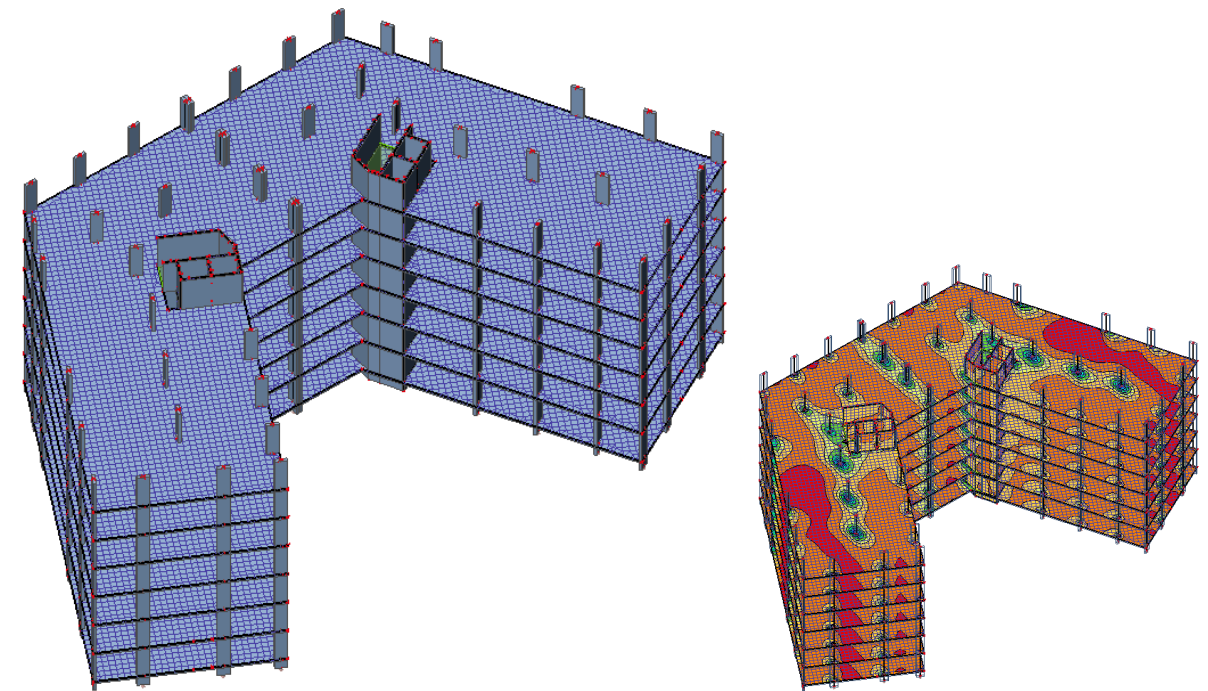


Figure 3.2: Finite element structural model.

Locally, due to the presence in some of the blocks of duplex units at ground and 1st floor with different layout from the floors above, the columns have been moved and set 7.5m apart in one direction. PBA have proposed this solution in order to avoid costly transfer elements above the duplex units, and associated implications to headroom and building height.

The slab thickness in these areas has been maintained at 225mm thick, making the slab punching design around the columns critical with the need to control this by properly sizing the columns. This could, in instances, lead to larger size than would normally be required for strength. Floor penetrations in the vicinity of the columns become critical, significantly affecting the design and should be avoided where possible.

Subject to confirmation, the slab finishes for the typical upper floor are likely to include a 75mm non-structural screed with under-floor heating.

The slab thickness and density of the slabs is expected to satisfy Part E of the building regulations. The overall proposed build-up by PRP Architects, is however to be considered and adequacy of the proposals is to be confirmed by the Acoustics consultant.

Top floor RC slab to W1 and E5 buildings

In the case of buildings W1 and E5, the top floor slab is to be 350mm thick as it will support the two-storey penthouses sitting on top. The penthouses are not aligned with the structure below and it is proposed to be built using hot rolled steel construction. Such a steel frame solution would result in a 350mm thick transfer RC slab supporting the penthouses, but would avoid significant size transfer structure that would otherwise be required if a heavier solution is selected, such as reinforced concrete construction.

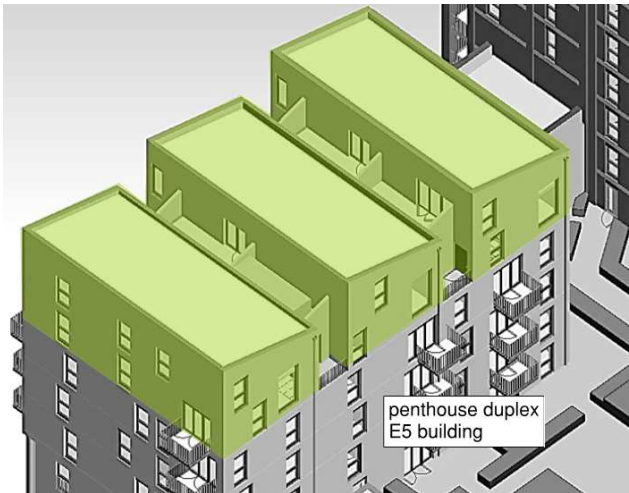


Figure 3.3: Two-storey penthouses to be built in steel on top of RC frame (building E5).

Penthouse steel deck slab to W1 and E5 buildings

As previously noted the two-storey penthouses will be a hot rolled steel frame supported on the top RC floor slab. The intermediate slab will be a composite slab with a steel deck supported on steel beams. Indicative beam layout is shown in Figure 3.4. A timber floor was also considered but discarded because the fire and acoustic requirements would lead to an excessive depth build-up (circa 550mm).

Due to depth constraint, the typical structural depth of the floor structure has been reduced to 250mm by using UC profiled beams and supporting the steel deck on angles welded to the web of the beam. This results in a heavier structure but reduces significantly the overall depth. It is foreseen that locally some UB beams may be required and their depth and location will need to be coordinated with M&E services.

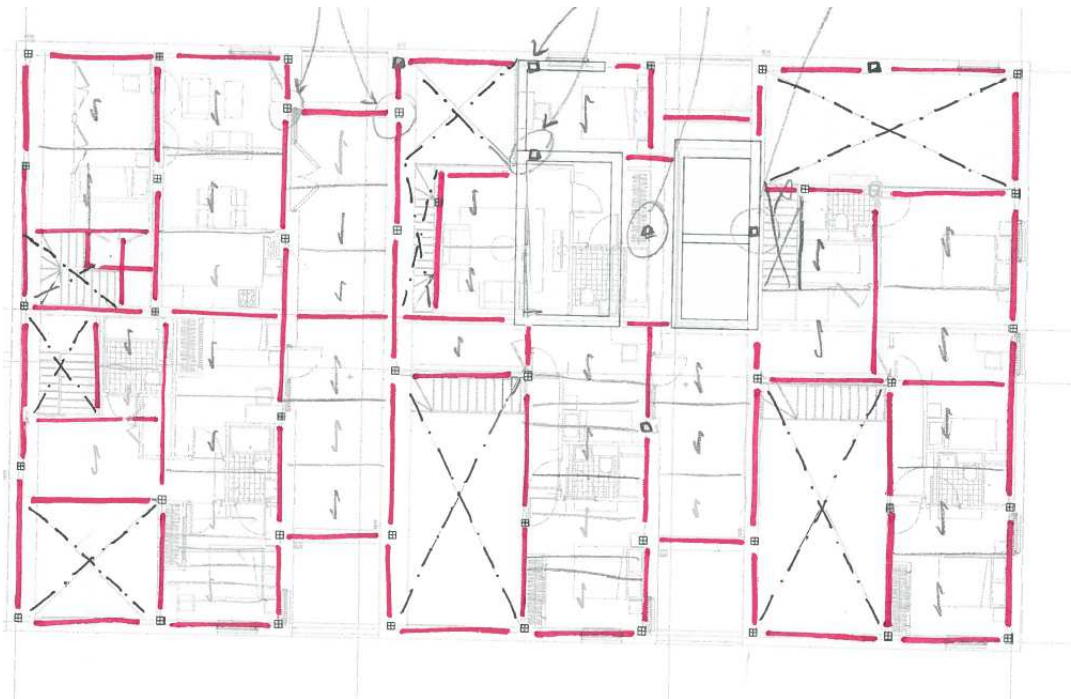


Figure 3.4: Indicative composite steel deck layout for intermediate floor slab at the two-storey penthouses (building E5).

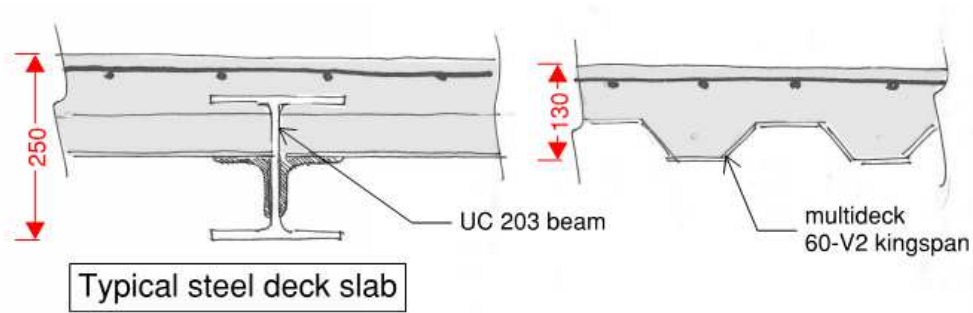


Figure 3.5: Composite floor section

Top floor slab to M2 building

In the case of M2 building, the roof is recessed which results in vertical discontinuity of the façade columns and the need for a transfer structure. The transfer structure can be achieved by increasing the RC slab thickness to 300mm and using a lightweight steel frame structure above for the top floor construction.



Figure 3.6: Recessed top floor in M2 building to be built in steel on top of the RC frame.

Floor slabs to terraced houses – Building E4

Building E4 consists of 4-storey terraced houses which are to be constructed with reinforced concrete. A traditional masonry construction was considered at previous stages but was dismissed in favour of the RC solution due to the discontinuity of the walls and the recessed facades.

The proposed solution consists of a 225mm thick slab for all floors and roof with downstand beams required at 1st floor and 3rd floor. The slab will need to step down in areas where the balconies are set back from the façade line but occupied below due to the thermal insulation requirements between external/internal areas.

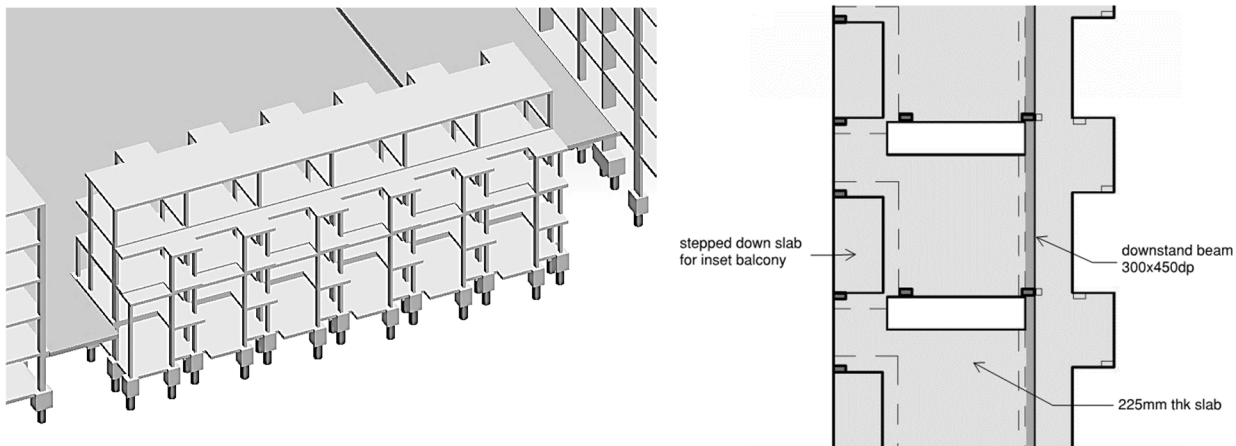


Figure 3.7: Terraced houses RC frame - Building E4

3.3 Roof structure

Roof slabs to residential buildings

All RC buildings are generally to have flat roofs which will comprise architecturally specified roof finishes supported on reinforced concrete flat slabs, typically 225mm thick. The proposed typical roof, ballast over insulation, consist of an inverted roof with liquid applied waterproofing over the slab and parapet upstands. The superimposed load considered is approximately 1.5 kN/m²

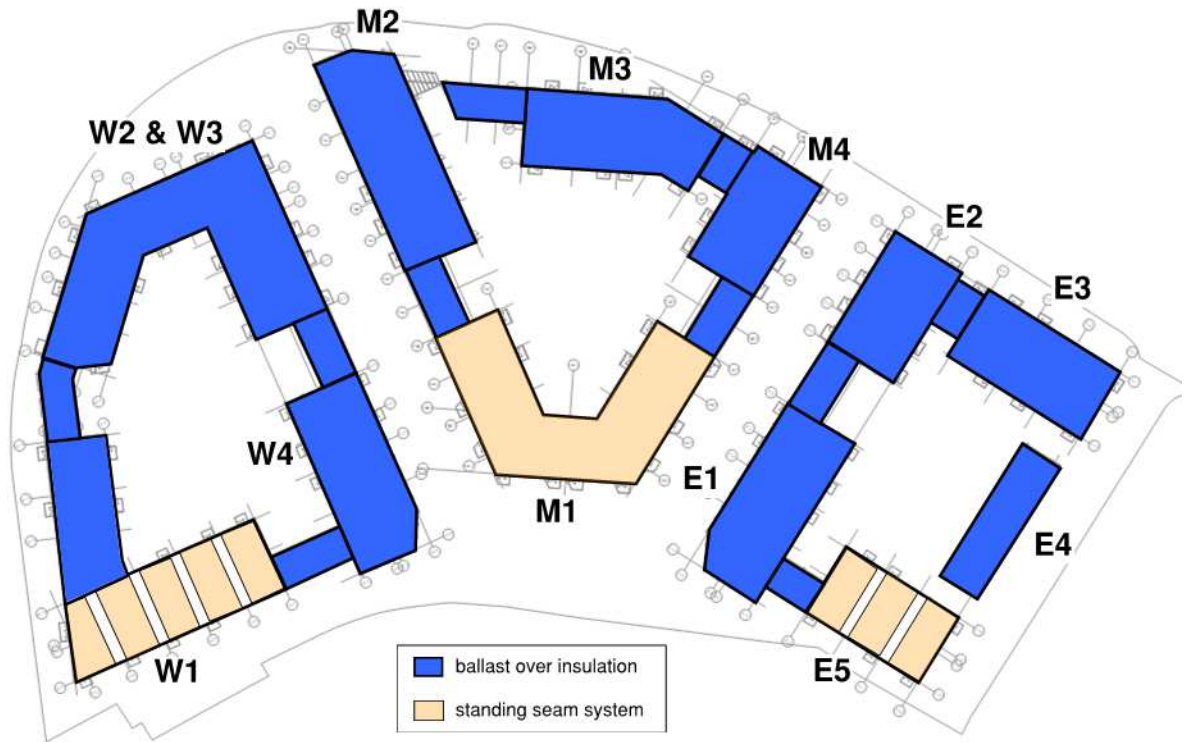


Figure 3.8: Proposed roof types.

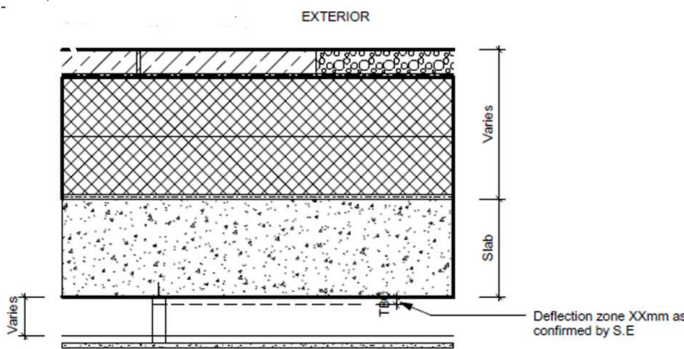


Figure 3.9: Typical inverted roof.

Mono-pitched Roof to buildings W1 and E5

The proposed build-up for the mono-pitched roof of the penthouses on top of blocks W1 and E5 is a standing seam roof system. As previously noted, the two-storey penthouses are to be built in hot-rolled steel. The secondary beams will be timber joists (245x72mm @ 400 c/c).

Beams are generally to be UB 254, with the top of the beam flush with the timber joists. Locally some longer spans may require a heavier UC profiled beam, in order to maintain the overall depth to 250mm. Refer to Figure 3.12 for the roof's beam layout.



Figure 3.10: Typical standing seam roof system

The Figure below shows the proposed beam layout

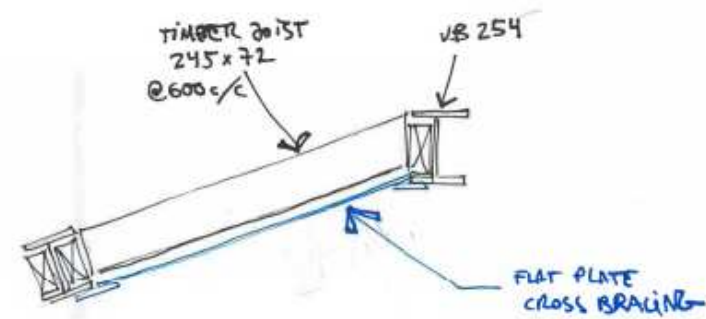


Figure 3.11: Steel beam and timber joists for pitched roofs

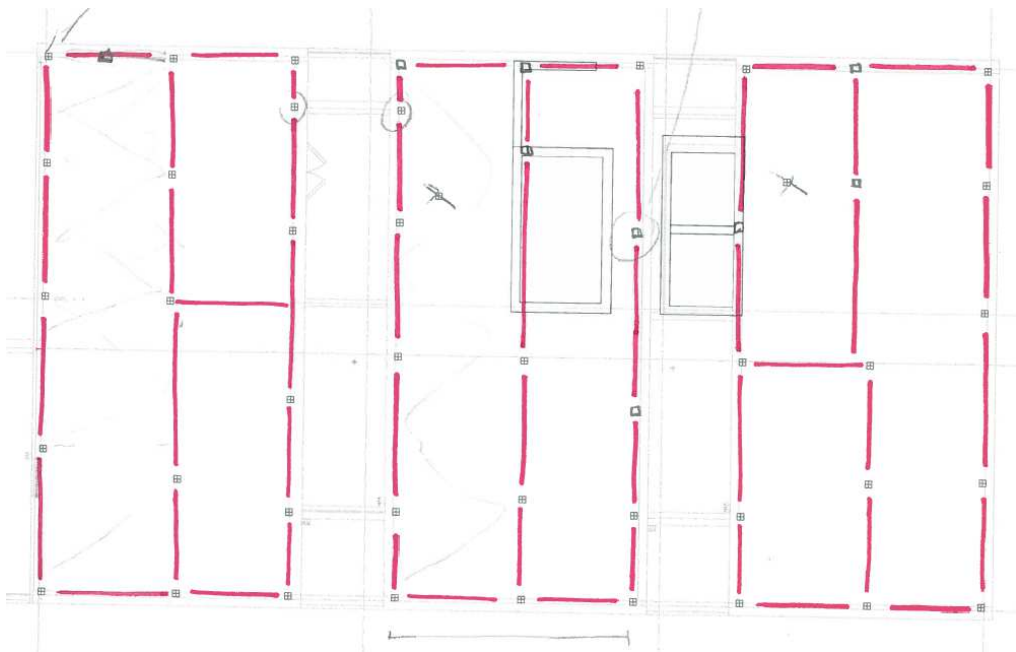


Figure 3.12: Indicative layout for steel beams at roof (building E5)

It is assumed that access to the roof would be limited to maintenance only and no further loading allowance has been made at this stage.

The maintenance strategy is yet to be defined by PRP. At this stage a suitable allowance should be made for mansafe systems and façade maintenance equipment that will need to be confirmed by the architect at the next design stage to inform the structural design.

Roof to M1 building

The proposed build-up for the duo-pitched roof in building M1 is also a standing seam system. The top floor columns and roof will be a steel frame supported on the 6th floor level. In this case, no additional thickness is required for the top floor RC slab as the steel columns are generally aligned with the RC columns below, with few exceptions to sit on transfer beams.

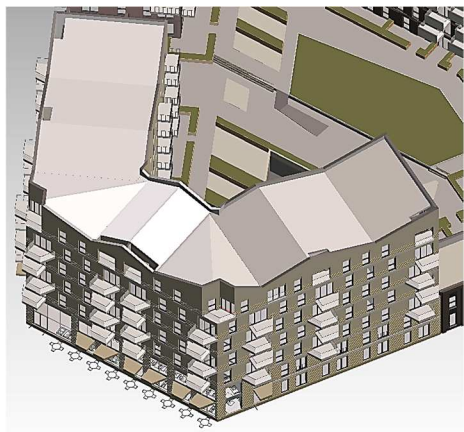


Figure 3.13: Pitched roof – M1 building

The build-up will be the same as for the penthouses’ roof, with timber joists embedded in the steel beams. See Figure 3.14 for an indicative beam layout. Note some of the beams are to be cranked.

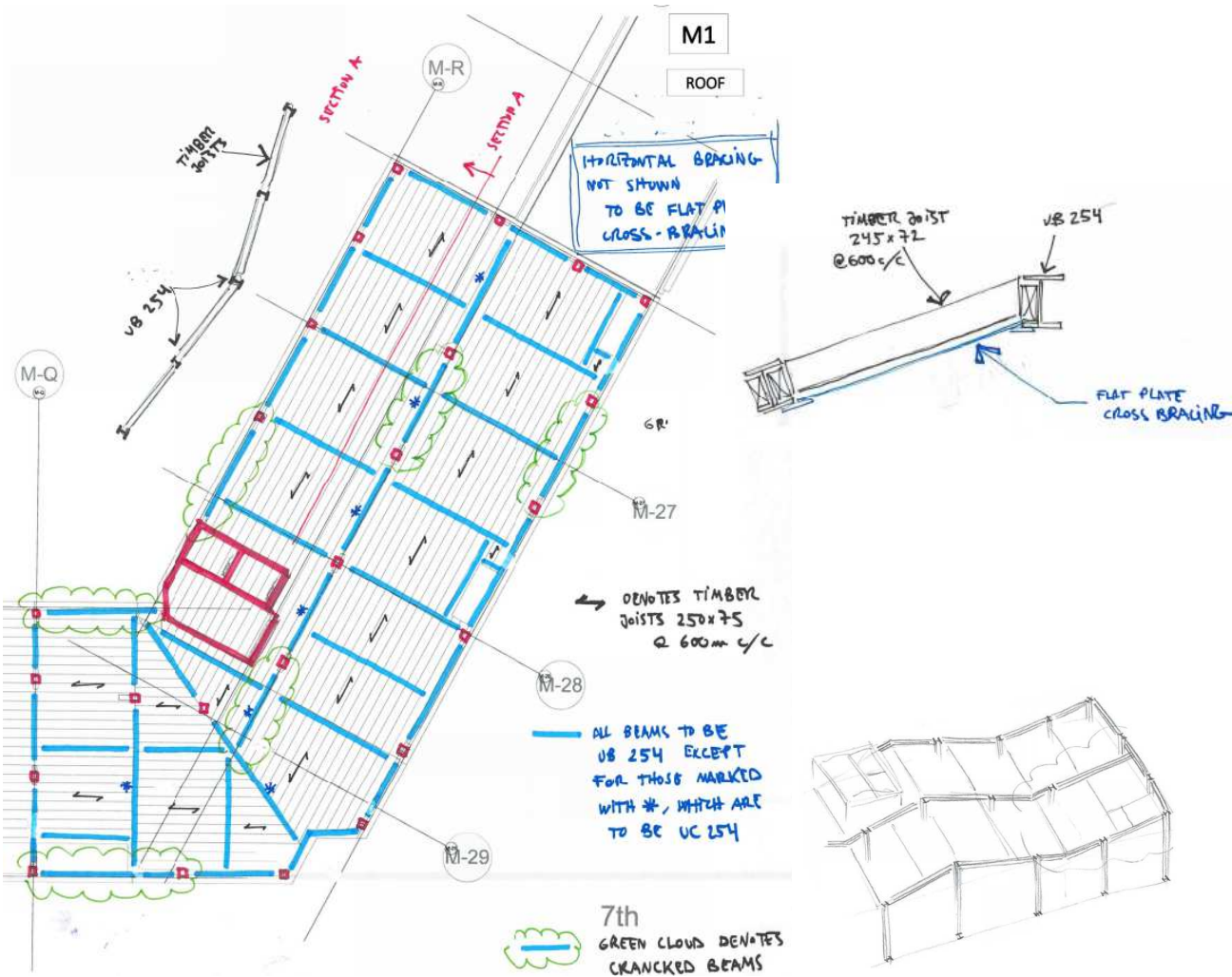


Figure 3.14: Steel beam layout for pitched roof – M1 building

3.4 Ground Floor

Ground floor slabs to residential buildings

Due to the depth of Made Ground present on the Site, it is proposed that suspended flat slabs are used for the lowest level floors for the medium-rise and high-rise blocks. These will typically be 275mm thick cast in situ flat slabs. These slabs will also have the role of providing stability to the pilecaps, thus avoiding the need for stabilising beams, with the top of pilecaps flush with the underside of the slab.

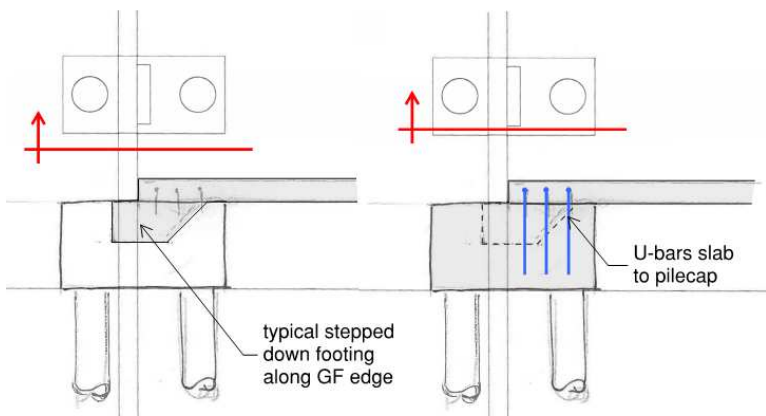


Figure 3.15: GF edge detail & slab to pilecap typical detail

The connection between the pilecaps and the GF slab will consist of U-bars anchored in the pilecap as shown in Figure 3.15.

Figure 3.16 shows indicatively the extent of the GF slab for the East block.

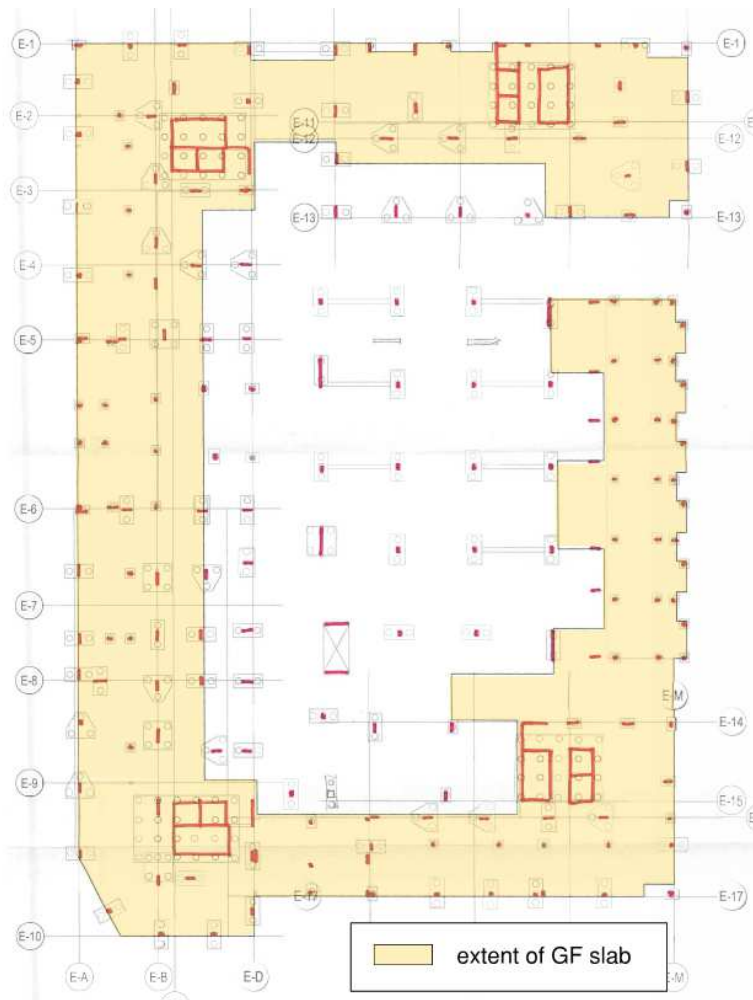


Figure 3.16: Extent of ground floor RC slab

Subject to confirmation by the architect and services engineer, the slab finishes for the ground floor are likely to include a 150mm total zone comprising non-structural screed above a layer of rigid insulation.

Ground floor to car park areas

A traditional form of road construction is proposed to be used in the car parking areas under the central podia of the three blocks.

It would comprise a granular sub-base foundation and a bituminous engineered base (refer to the civil engineering report)

The pilecaps in this area will require tie beams, typically 400mm WD X 600mm DP.

3.5 Car Park and Podium Structure

The buildings comprise the outer perimeter of each block with an enclosed amenity podium space at first floor level. The podium houses a car park on ground floor for the residents, which in instances extends under the buildings.

The podium construction, consistent with the proposed building construction form, is proposed to be reinforced concrete. The extent of the podium is shown in Figure 3.17.

Each of the podium slabs have a step so as to provide greater clear height to the disabled carpark spaces.

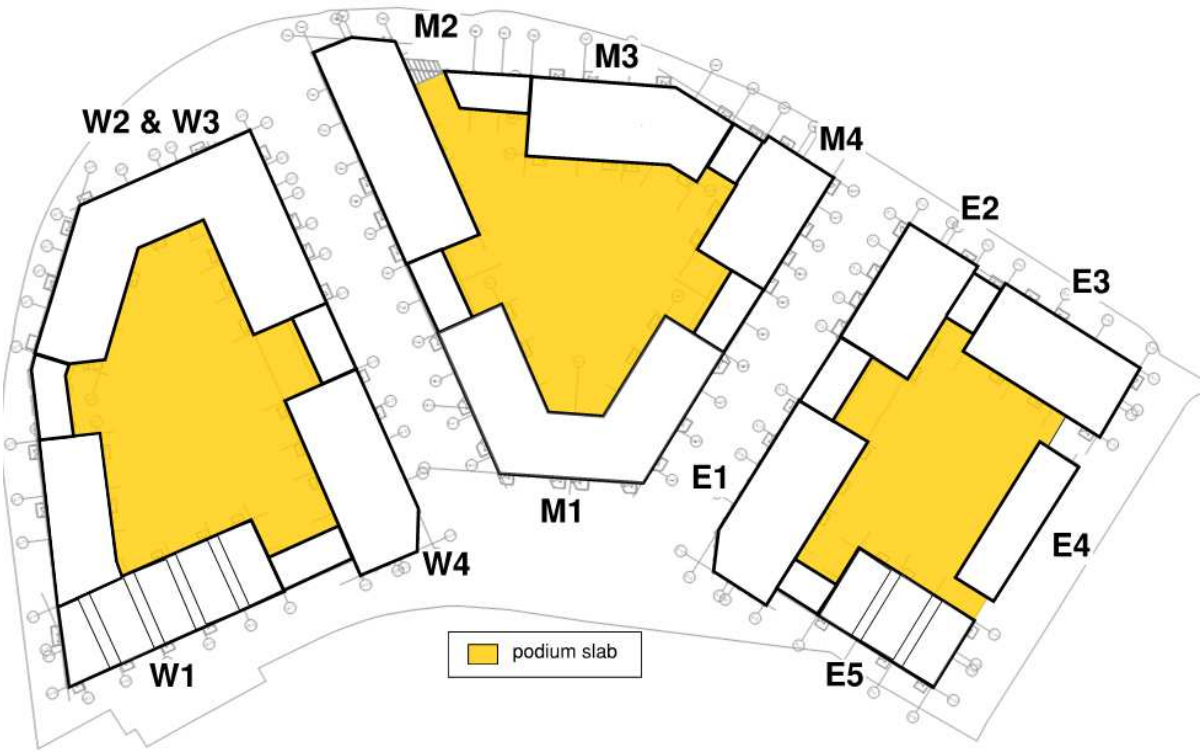


Figure 3.17: Extent of suspended podium slabs.

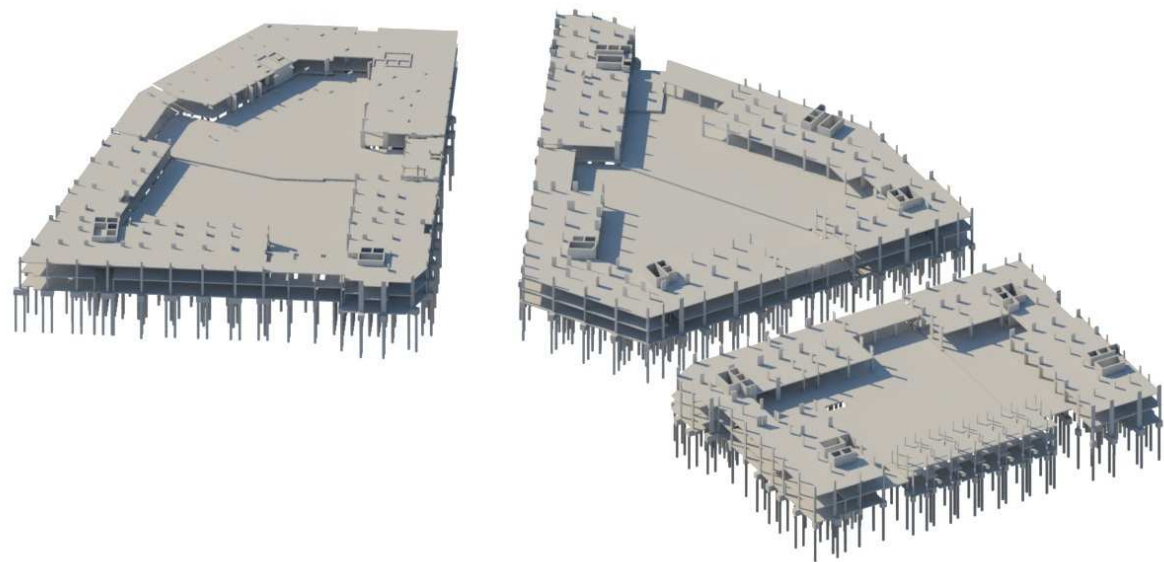


Figure 3.18: 3d view of podium

The typical column layout responds to a 7.85m x 7.2m grid, in accordance with the 'Design recommendations for multi-storey and underground car parks' publication by the Institution of Structural Engineers (IStructE). Obviously, the column layout has been set out to suit and optimise the carpark spacing.

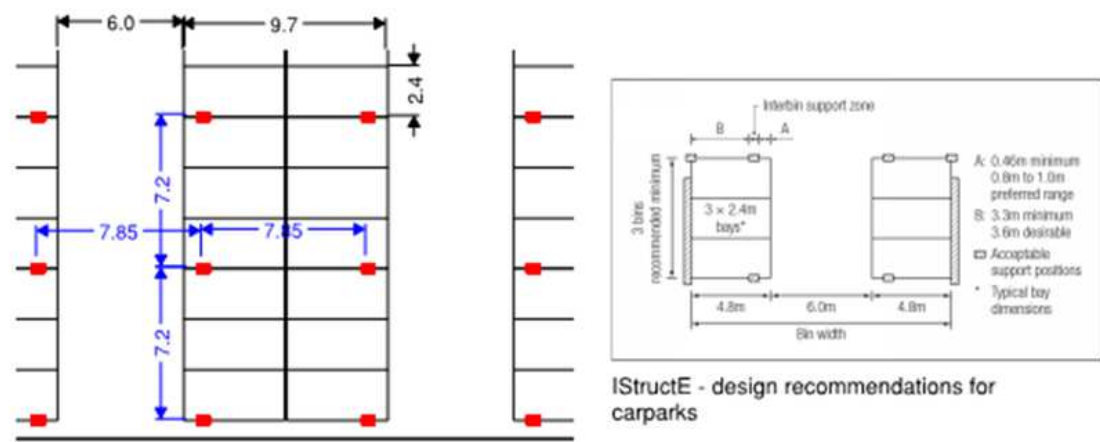


Figure 3.19: Typical car park column grid

A number of options for the podium slab construction had been investigated in previous stages leading to the conclusion that a 350mm thick flat slab is the most cost effective option, considering the ventilation strategy and potential implications to the air flow under the slab.

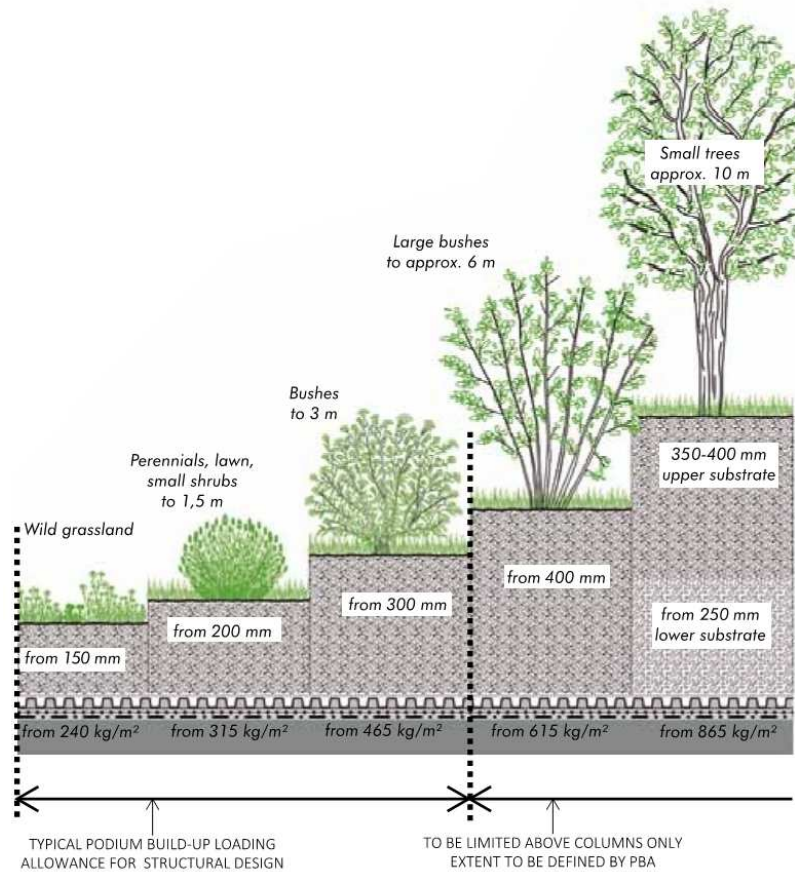


Figure 3.20: Range of green roof depths and associated loading

The proposed architectural finishes for the podium slab are an intensive green roof with approximately 300mm thick growing medium and an estimated superimposed load of 4.65 kN/m².

It has been agreed that proposed trees, with increased soil depth requirement and subsequently increased loading, would be positioned over the columns resulting in direct load transfer to the columns so as not to affect the slab design.

A drainage system is proposed to be used over a hot-melt system waterproofed slab which would be used for shallow attenuation and conveyance of surface water on the podium deck. The system should be designed as a combined drainage component and sub-base replacement system with high compressive strength and bending resistance to create a horizontal structural 'raft' as required.

Special care should be given in coordinating the podium levels with the surrounding blocks. Areas of different use have different requirements of minimum floor to ceiling headroom, services, structural zone and finishes. The diagram in Figure 3.21 shows the assumed minimum required heights and zones for the carpark, retail and residential areas. The structural zones have been advised by PBA, and PRP have been coordinating the different levels between the buildings and the podium.

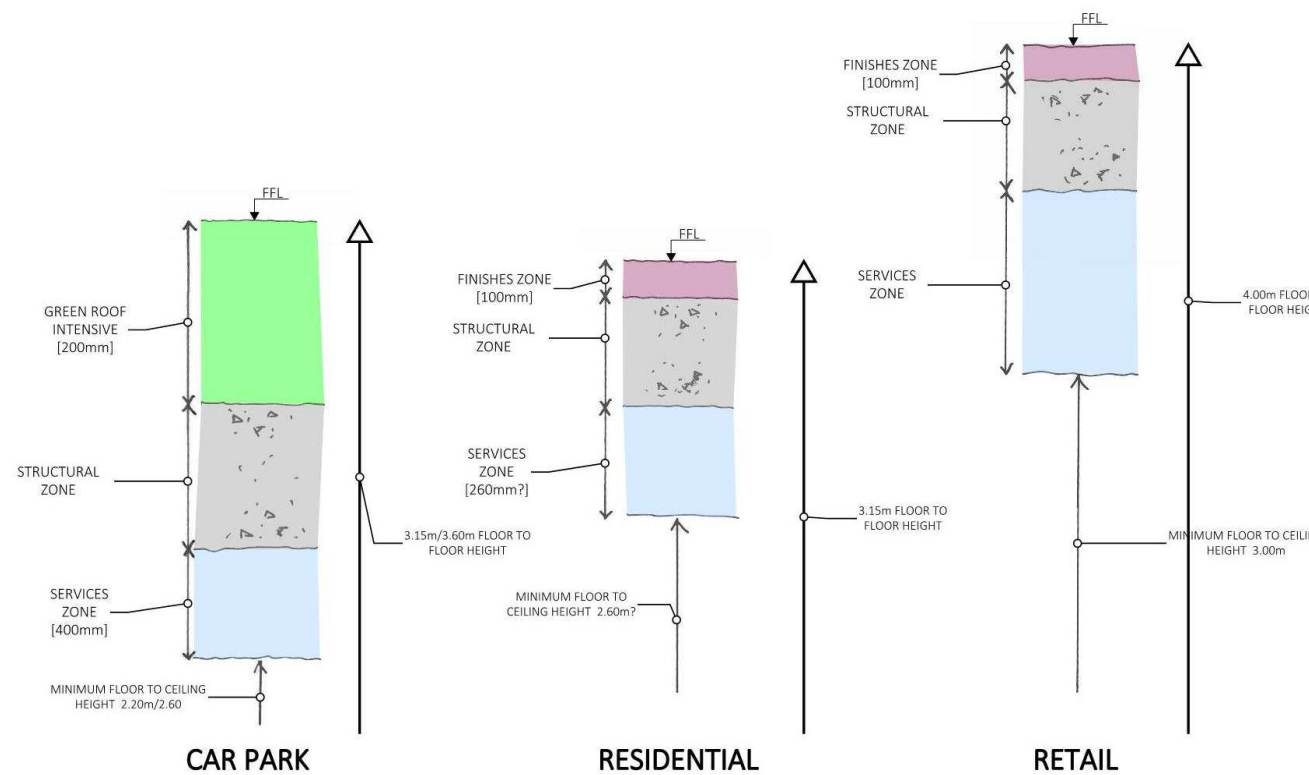


Figure 3.21: Height requirements at podium and adjacent 1st floor slabs.

The podium slab is supported along its perimeter by the columns of the adjacent blocks by means of shear connectors. The buildings slab will have a typical downstand edge beam 600mm WD X 400mm DP that will run along the perimeter to support the podium slab. Where a transfer beam is required (see section 3.10) the transfer beam will itself support the podium slab and will accommodate the shear connectors.

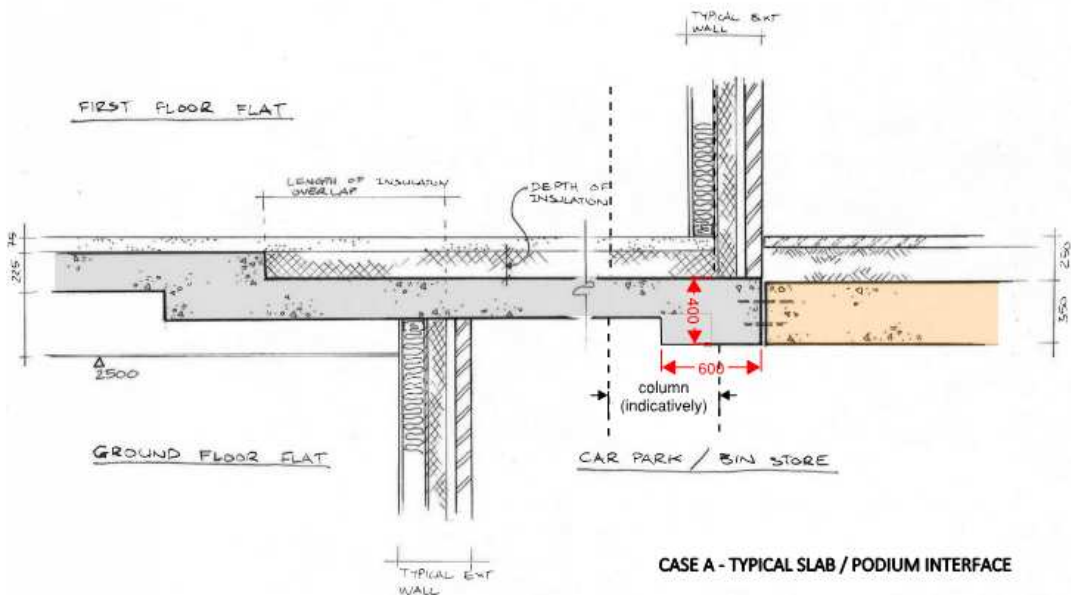


Figure 3.22: Podium to Buildings interface detail

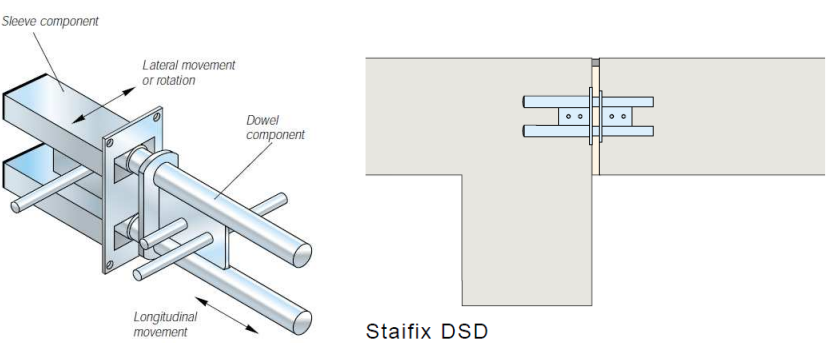


Figure 3.23: Proprietary shear connector

This support detail does allow relative horizontal movement between the podium and the building in line with the stability and expansion joints strategy. A proprietary dowel shear connector is proposed to be used which will be able to accommodate the expected relative movements. For costing purposes assume ANCON DSDQ100 connectors at 1.2m centres.

At previous stages other support options, described below, were considered and discarded:

- A downstand beam with a corbel under the podium slab with an elastomeric bearing strip, which would clash with the services zone.
- A double-column arrangement which was not compatible with the carpark layout.

The construction sequence should be defined at the detailed design stage in some detail to better understand potential implications of relative vertical movements between the podium and the buildings. The relative movements may be more pronounced during construction, when the buildings could settle relative to the podium while being built assuming they are built concurrently, imposing stresses to the podium's slab and columns. Mitigation measures should be considered at the detailed design stage, if deemed necessary such as constructing the podium following completion of the building's structure or using infill strips to be infilled at a later date.

3.6 Energy Centre

The energy centre is located in the central block, between buildings M2 and M3. It is sunk into the ground circa 1m below GF level with perimeter retaining walls and a 350mm thick raft.

Generally an imposed load of 7.5 kN/m² has been considered throughout the plant area. Water storage tanks are likely to be heavier than this allowance and the structure has been designed for 20kN/m² in that area. The M&E engineer should provide a plan layout of the plant equipment, size and loading to enable PBA to define the load allowance and carry out the structural design accordingly. The strategy for the installation and removal of the plant equipment is still to be confirmed.

The energy centre is located at GF between the M2 and the M3 building. A two-storey duplex building sits partially above the energy centre with the energy centre extending into M2 building at GF. The column layout for the duplex and M2 building results in the need for transfer beams to avoid clashes with the plant equipment.

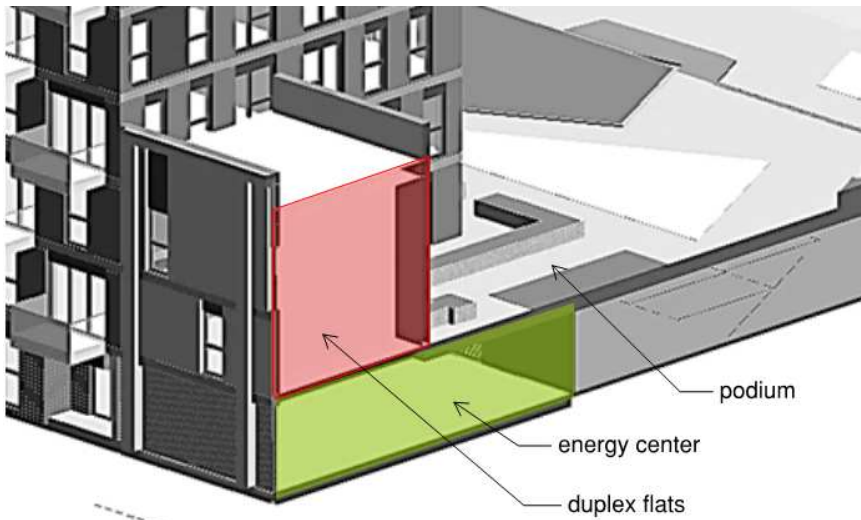


Figure 3.24: Energy centre section

The internal columns are proposed to be spaced at a maximum spacing of 7m to suit the plant layout. The slab above the energy centre will be 225mm thick below the duplex and M2 buildings, and 350mm thick below the podium.

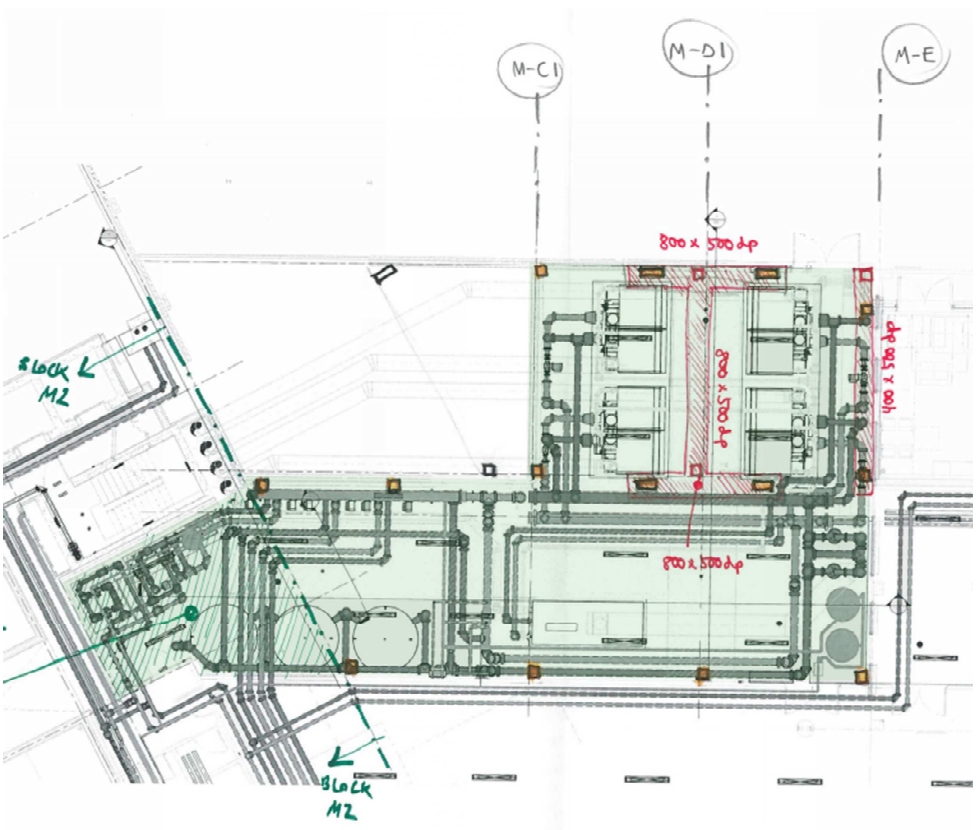


Figure 3.25: Energy centre plan showing columns layout and transfer structure requirements.

3.7 Columns

The vertical elements of the structure consist of fin columns located at internal wall locations not exceeding a 7m spacing typically to maintain the slab thickness at 225mm as discussed in section 3.2.

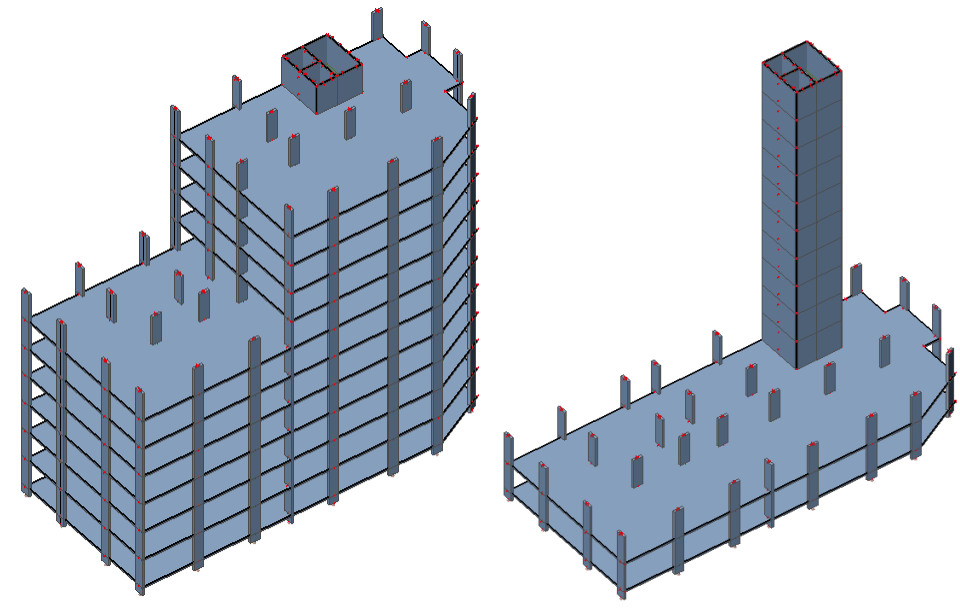


Figure 3.26: Structural model showing columns layout and lift/stair core for block W2-W3.

The sizing of the columns depends on a number of variables such as fire protection requirements, strength requirements and slab shear punching design. The available sizing options have been advised to PRP. The Architect's aspirations have led to the use of blade columns either to fit within internal partition walls if possible or minimise the impact of protrusion within the internal spaces.

These are typically 220mm Wide X 1000mm Long. For fire resistance purposes these columns are classified as 'walls', due to the length:width ratio exceeding 4:1, and thus provide the required fire resistance without the need of additional measures.

In addition to the blade columns a number of different sizes has been used in some instances due to architectural constraints. These will either need to comply with minimum thickness requirements or be fire protected by alternative means (i.e. fire rated boarding).

The columns are subject to axial shortening in the long term due to creep and shrinkage effects. This is a function of the construction methodology and characteristics of the elastic performance of the vertical elements. This phenomenon can normally be ignored in low-rise building but it's importance increases with the height of the building.

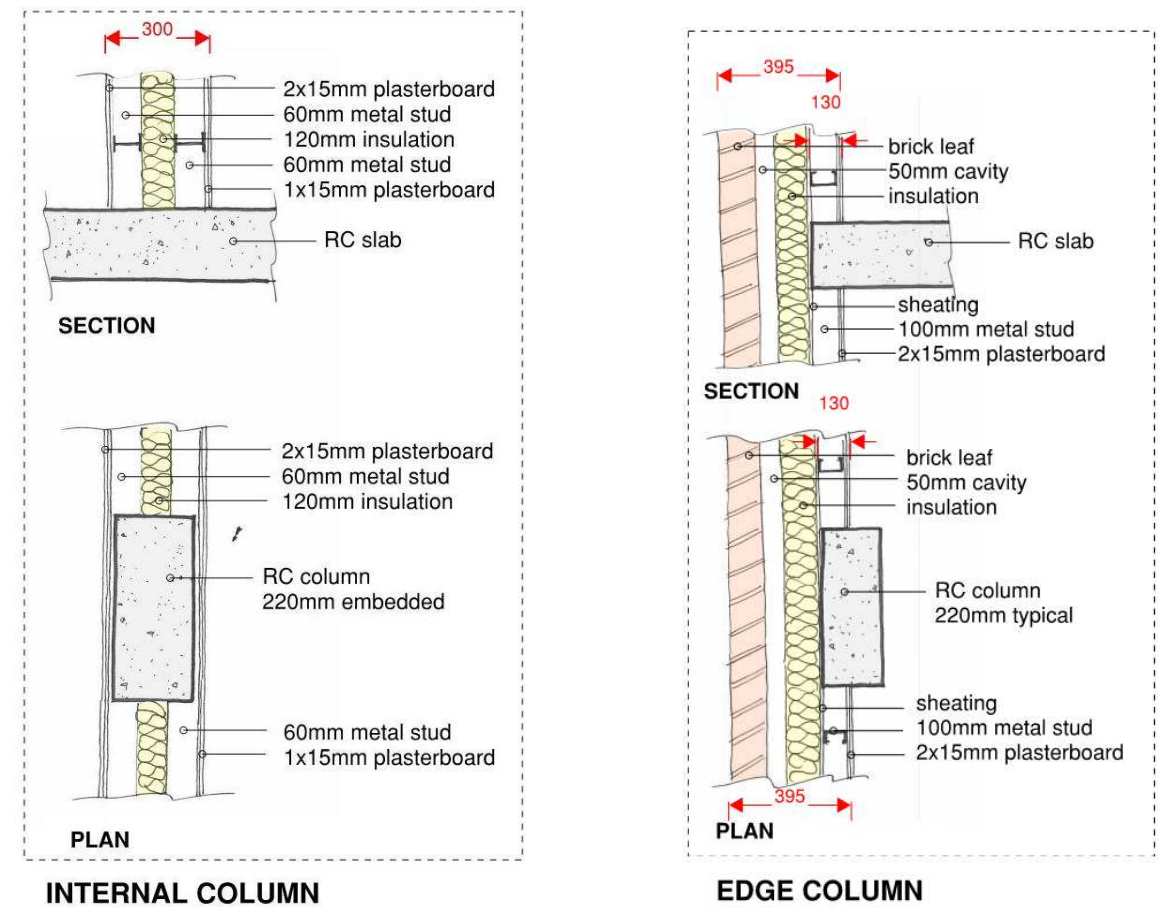


Figure 3.27: Typical column arrangement

3.8 Vertical Load Path

The structure to all blocks generally has a direct load-path, with columns and core walls continuing from roof level down to the piled foundation. Transfer beams have been used to divert columns when necessary, which is described in section 3.10.

3.9 Lateral Stability

Lateral stability in both directions will be achieved by the reinforced concrete lift/stairs core walls. Transfer of lateral forces into the lift core and columns assumes a simple distribution based on wall stiffness. Horizontal loading on the elevation is transferred through floor slabs into the walls which continue down the building from roof to ground floor. The resulting overturning effect transmitted by the reinforced concrete walls will be counteracted by the group of piles below the lift/stairs core.

The central lift/stairs core walls are arranged in a simplistic uniform and consistent arrangement throughout each floor to enable effective use of formwork. A pull-out bar system or mechanical couplers are to be used at each floor level.

The core shear walls have been sized as 250mm thick for all the blocks. The thickness of the walls could potentially be reduced on the upper levels as the formwork is raised and the forces to the walls are reduced.

The fire protection and resistance requirements are discussed in section 5.13.

The stair/lift shaft score incorporates some vertical services throughout the height of the building: water, electrical, heating and smoke vent risers. Further design development and coordination with the architect and M&E consultant should be carried out for the sizing and positioning of the openings in all buildings.

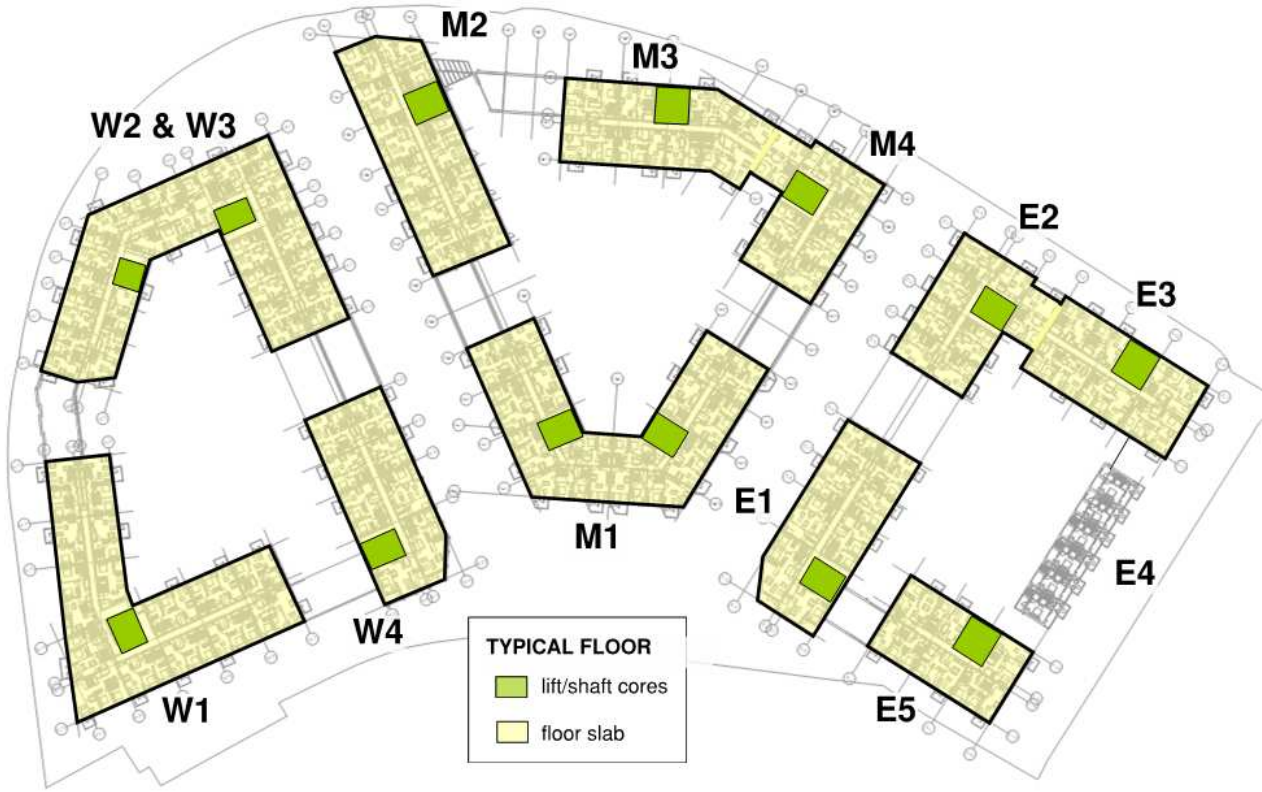


Figure 3.28: Diagram indicating the stability cores in green for each building.

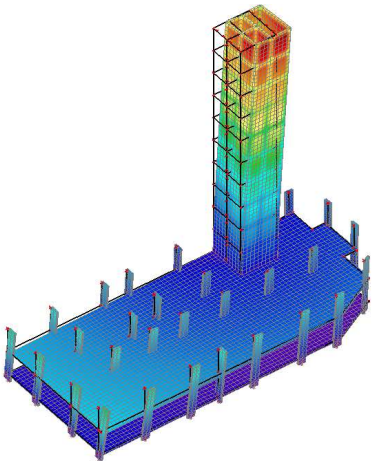


Figure 3.29: FE stability model – Deflection coloured contours

The podium structure is laterally independent from the buildings as there is an expansion joint provided at the interface. Thus, the podium should have its own stability system and thus shear walls are provided in both directions.

The link buildings are laterally independent of the continuous main buildings as there is a movement joint between them. Walls have been provided in all link buildings in the direction perpendicular to the podium. The connection between the link buildings and the podium slab would provide restraint in the other direction.

The steel frames have also been designed with its own bracing systems, typically cross bracing flat plates, both horizontal at roof level and vertical down to the concrete frame.

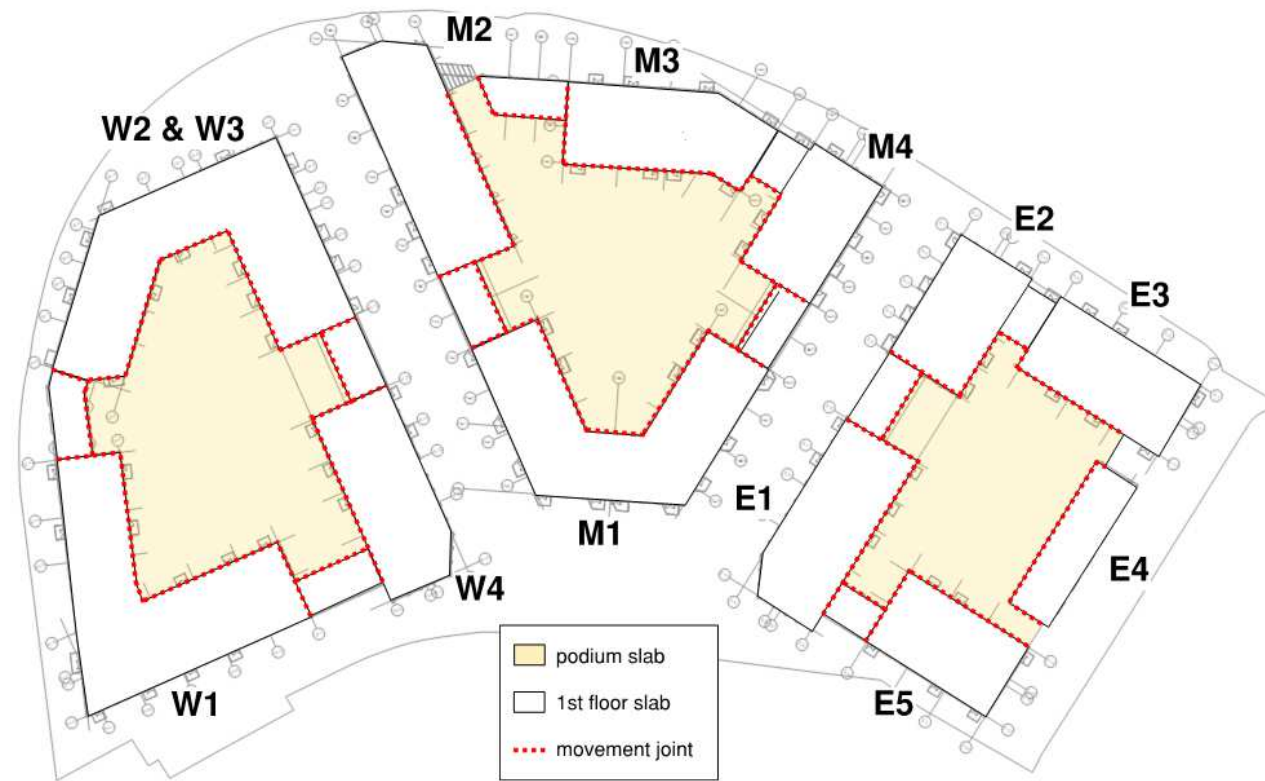


Figure 3.30: Movement joints

The figure above shows the location of movement joints at first floor level. There is generally a movement joint all along the perimeter of the podium slab to allow for thermal and differential movements to the adjacent blocks.

Thermal actions and expansion joints are further discussed in section 5.5.

3.10 Transfer Structures

The current structural arrangement has been adopted to minimise transfer structures where possible. The structure of the building generally has a direct load path, with columns and core walls continuing from roof level down to raft foundation. However, two main types of transfer structures have been incorporated:

- transfer beams at 1st floor level to divert some of the columns in order to maximise the carpark layout at GF.
- transfer beams at other levels as a consequence of column diversions.

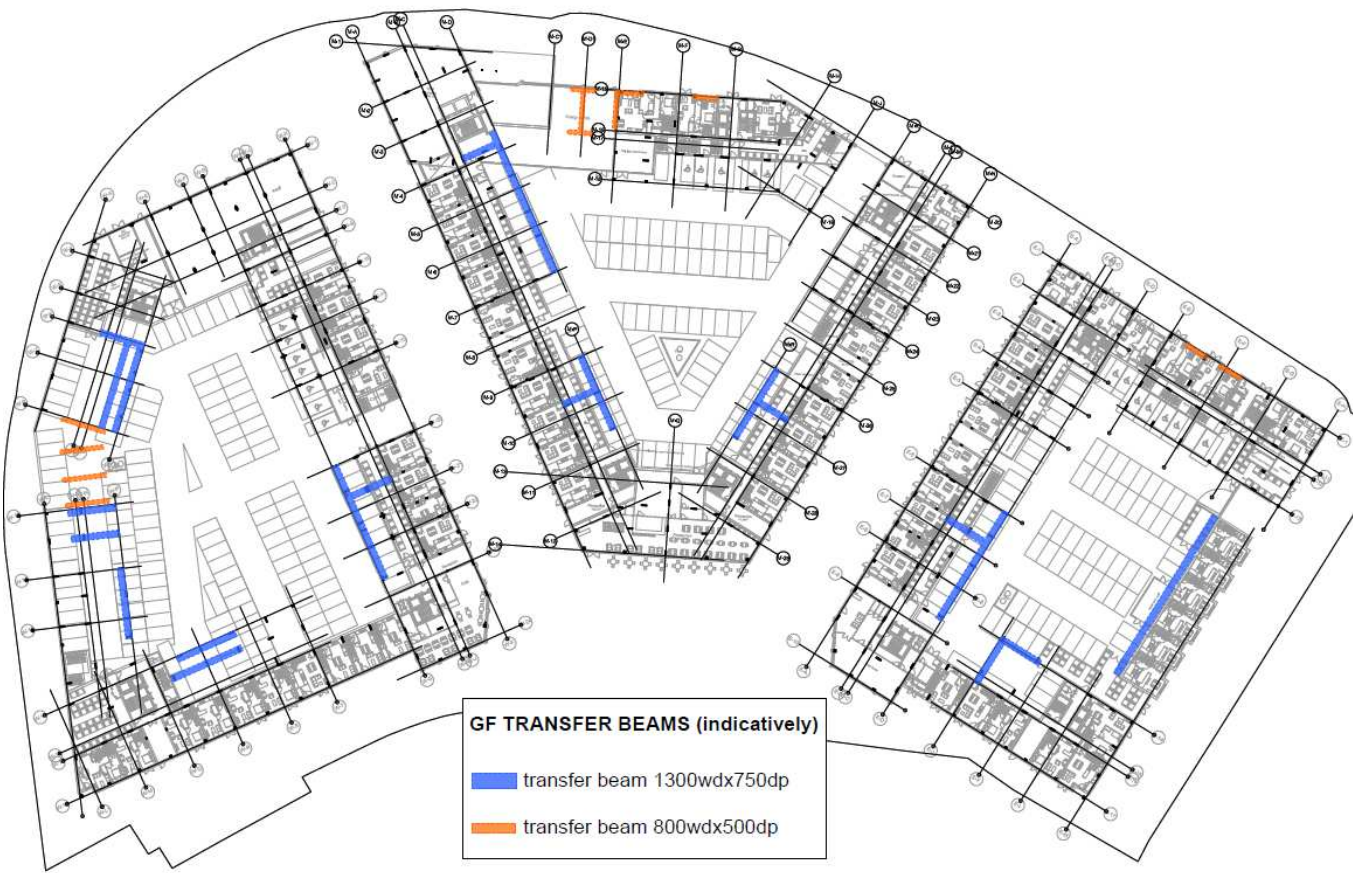


Figure 3.31: Downstand transfer beams at first floor level.

The typical transfer beam at 1st floor level, adjacent to the podium, to divert some façade columns to match with carpark layout consist of a 1300mm WD X 750mm DP beam (see Figure 3.32) spanning a maximum of 7.2m (3 car park spaces).

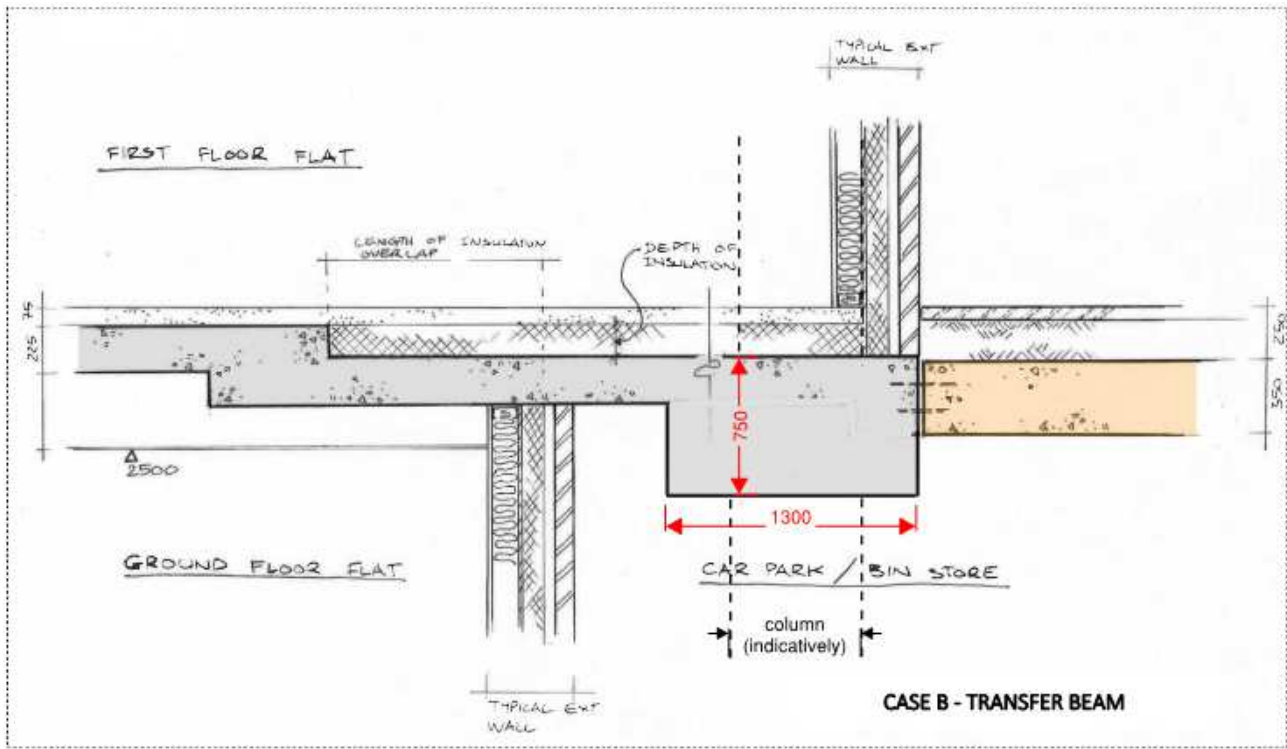


Figure 3.32: Downstand transfer beam at first floor level – Podium/Building interface detail.

There are other column diversions which have required transfer beams, especially along the façade. An example is the column relocation at top floors as a result of different layouts as shown in Figure 3.33.

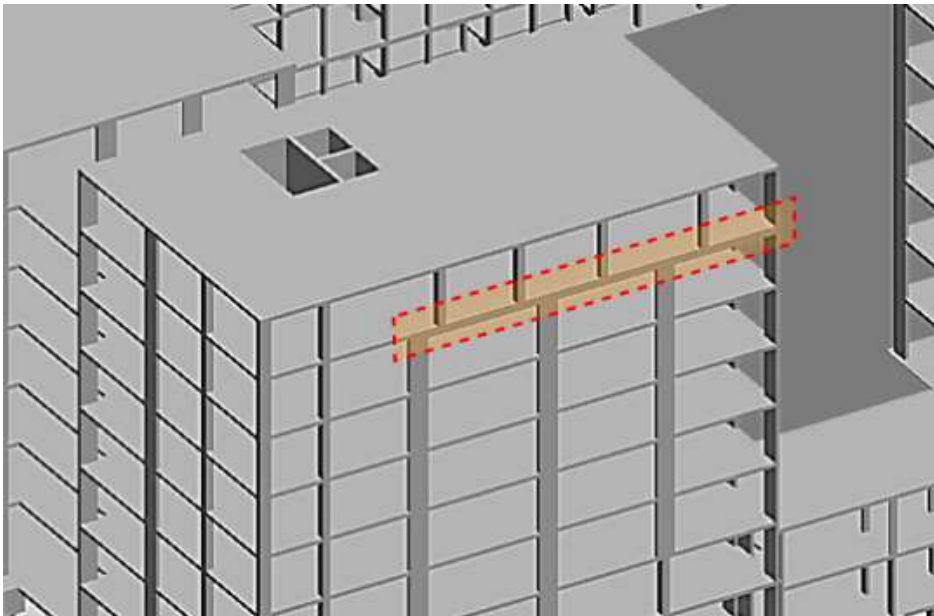


Figure 3.33: Top floor edge transfer beam.

3.11 Cladding and External Envelope

The external envelope build up is formed by an external leaf of brickwork, a cavity, insulation and a lightweight stud wall internally. The build-up is listed below:

- 102.5mm Brick
- 50mm cavity
- 100mm Insulation
- 12.5mm Sheathing
- 100mm Metal stud full fill batt insulation
- 30mm (2x15) plasterboard.

Figure 3.34 below shows the proposed build up.

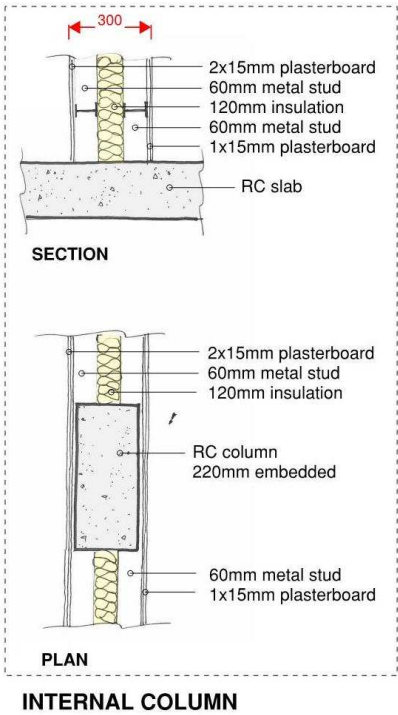


Figure 3.34: Cladding build-up

The external brickwork will be supported on proprietary shelf angles (Ancon/Halfen or similar) fixed to cast-in the concrete slab channels.

Non-standard brick details proposed by PRP will require bespoke elements to support the projecting brickwork. A preliminary proposal is shown in Fig. 3.35.

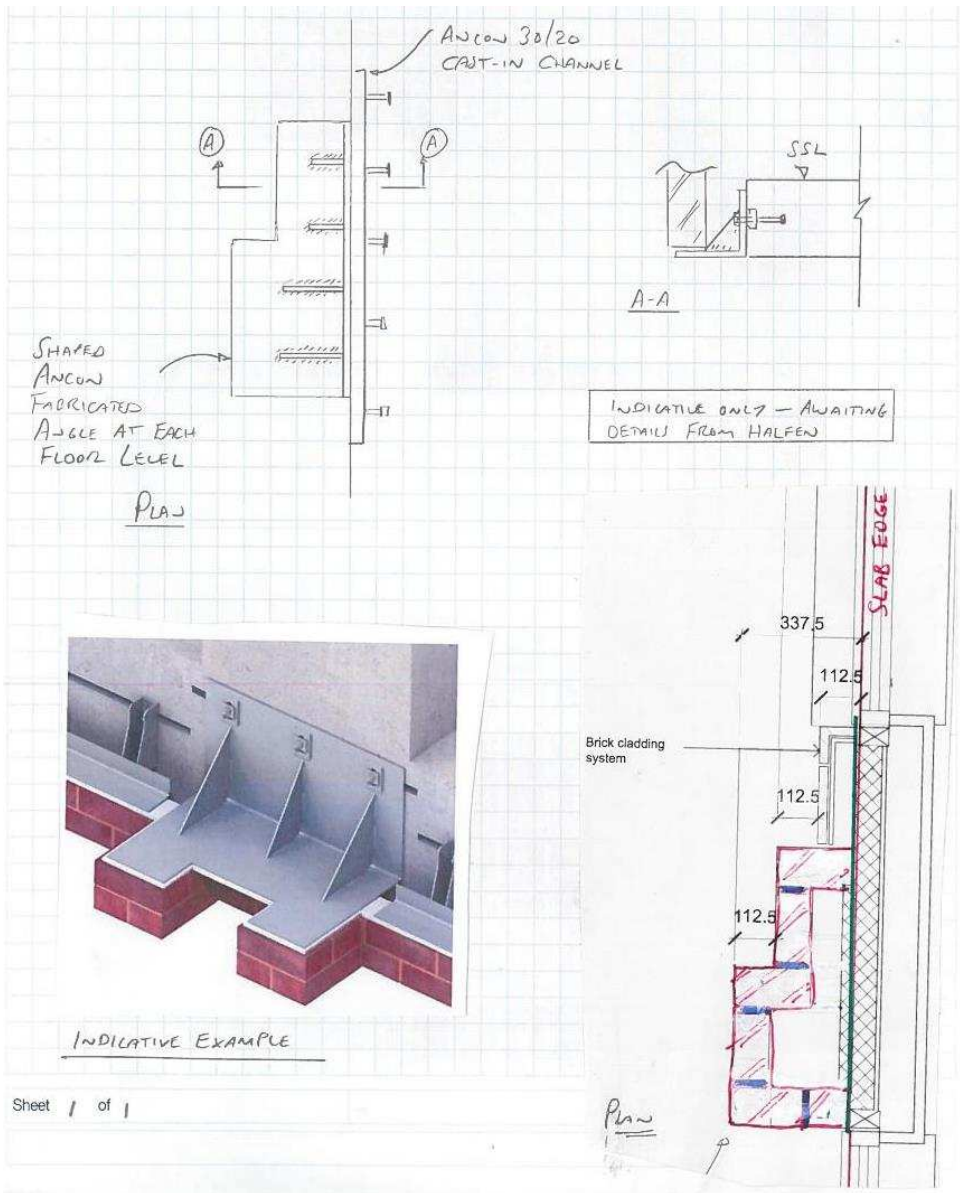


Figure 3.35: Preliminary non-standard brickwork support details.

3.12 Stairs

The primary core stairs are likely to be constructed in pre-cast concrete flights and landings supported on angles anchored to the core walls. However, contractors may prefer to use steel framed stairs and additional steelwork should be allowed for in the cost plan to cover this possibility.

Allowance should also be made for internal stairs such as in the duplex units and the terraced house, anticipated to be lightweight timber construction or, steel part of the architectural metalwork package. A suitable cost allowance should be made at this stage until the architect's aspirations are known.

3.13 Balconies

The balconies are proposed to be reinforced concrete 200mm thick, thermally isolated from the internal concrete structure to avoid cold bridging using Schoeck type thermal breaks. The balconies can be split structurally between cantilever balconies and recessed balconies.

Figure 3.37 below show the balustrade design intent by the architect, with 3 different types of balustrade considered: glass balustrade, PPC steel and perforated metal panel balustrade.

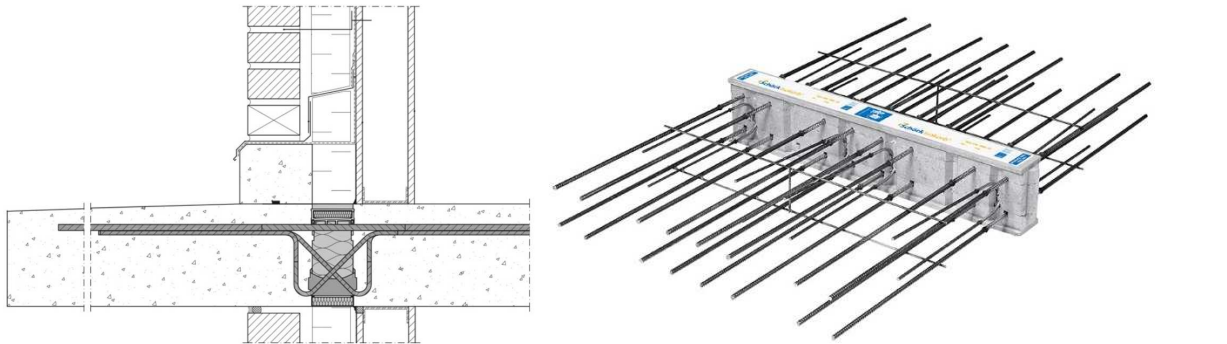


Figure 3.36: Cantilever balcony options

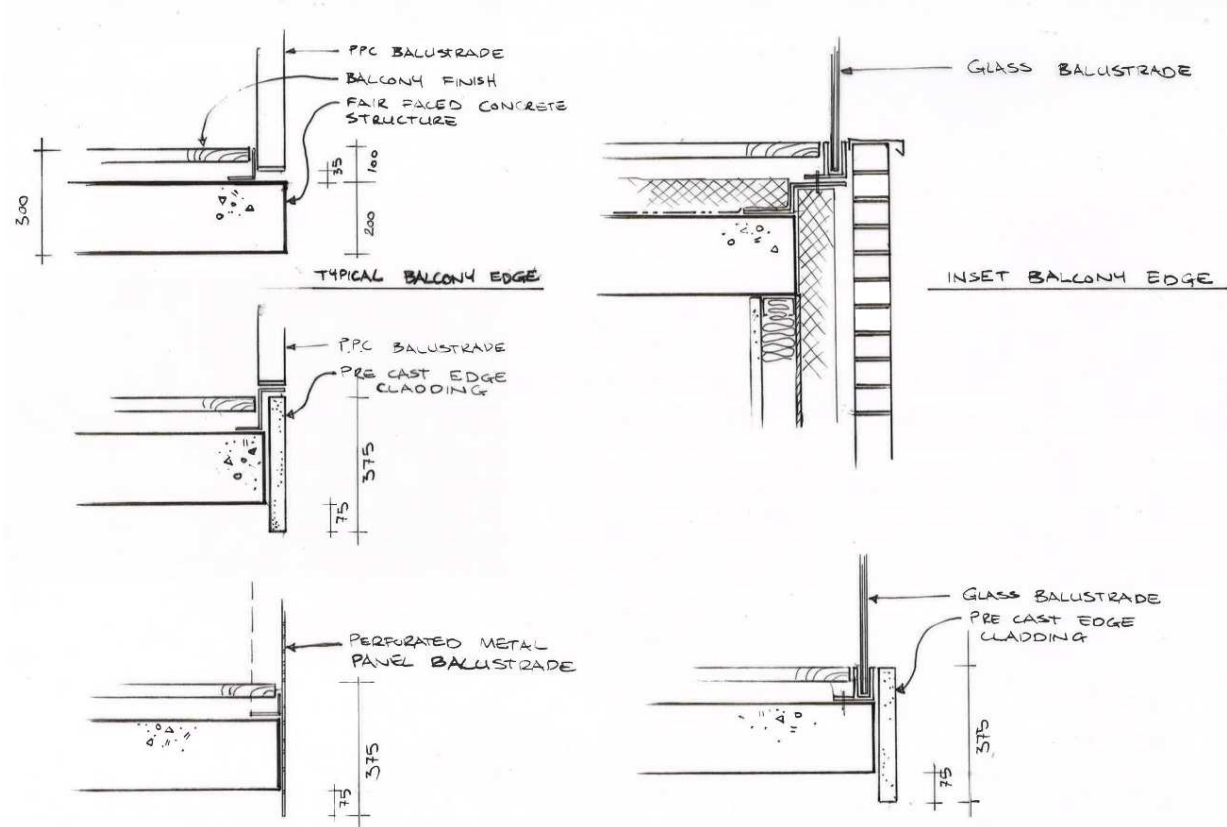


Figure 3.37: Typical balcony edge details from PRP

4 Substructure

4.1 Proposed Foundations

For the ground conditions present on the Site the proposed structures will need to be supported on pile foundations due to the anticipated structural loads from the proposed development and the presence of low strength superficial deposits.

While shallow foundations could be considered for the low rise buildings in the southwest where the Chalk has been encountered close to the ground surface and the foundations would be above the water table, it is considered unlikely to be a suitable solution due to differential settlement issues at the interfaces with adjacent building and the podium structure.

Bored and cast-in-place piles formed using conventional rotary auger or continuous flight auger techniques should be appropriate although the presence of any existing foundations may form obstructions to piling works. If conventional rotary auger techniques are adopted, temporary casing may be required in the Made Ground and the River Terrace Deposits to support the pile bore and to exclude groundwater.

A preliminary foundations layout has been developed to be used for costing purposes based on available geotechnical information to date.

A pile diameter of 600mm has been used consistently across the site, with pile capacities used in accordance with Figures 2.2 and 2.3 assuming 22m deep CFA piles. The concrete specification will need to be for DS-3, AC-3s with a minimum C25/30 grade concrete.

Load transfer to the piles from the superstructure columns varies depending on the height of the buildings. For the majority of the 6-11 storey buildings the typical pile arrangement consists of 2 piles under the perimeter columns and 3-4 No piles under the internal columns which carry a larger proportion of the slab loading.

The building cores which provide the required structural stability to the buildings, are also supported on a group of piles as shown on the structural drawings.

Under the residential buildings the ground floor slab is assumed to be at the same level as the top of the pile caps, effectively restraining the piles and resisting the overturning moment. This arrangement may result in cast-in the caps items like RWPs coming down next to columns if they can not be diverted away. The alternative to drop the pile caps lower down has also been investigated but advice from the cost consultant indicates that the currently adopted solution is the preferred cost-effective solution.

Pile and pilecaps under the podium where there is no RC slab proposed, would need lateral restraint in one direction in the form of tie beams which will be designed to resist any overturning moment due to geometrical imperfection/construction tolerances resulting in eccentric columns relative to the piles as-built position accounting for tolerances.

Alternatively the piles could be designed to resist the overturning moment without any tie beams, but this is expected to adversely affect the sizing of the piles as they will be unrestrained/unpropped at head level. This would increase their slenderness considerably, relying on the pile/soil interaction to provide lateral resistance and given the thick made ground layer it is anticipated that the piles will need to be designed as cantilevered columns, a not cost effective solution.

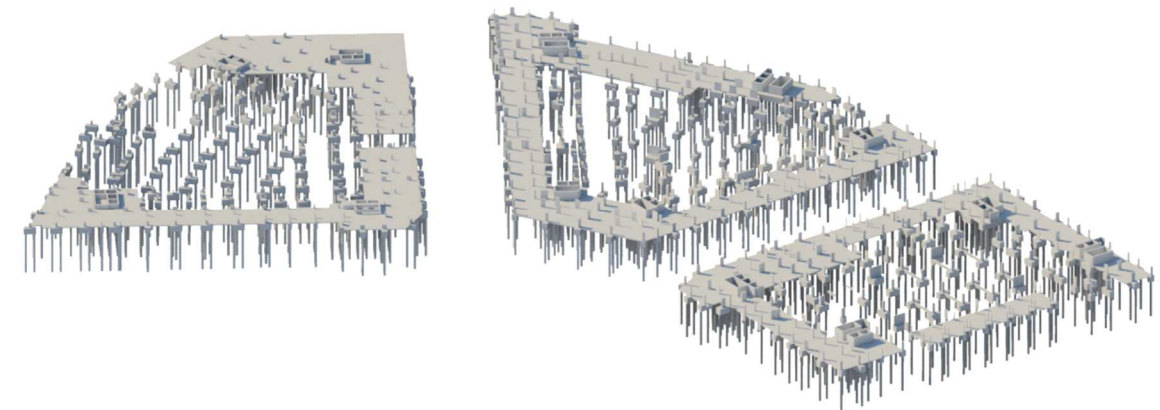


Figure 4.1: 3d view showing piled foundations.

4.2 Retaining Walls

Energy Centre

The energy centre is located between the M2 and M3 building, 11 and 7 storeys high respectively.

Recent proposals from MLM indicate that the plant equipment can all fit within a single storey space. This would require lowering the ground floor slab in order to achieve the minimum headroom requirements given the associated constraints including the superstructure proposals and transfer beam requirements.

Due to the timing this was advised and timescales to complete the current report, the design team are yet to confirm the exact level requirements for the energy centre basement.

It is anticipated that the ground floor lowering will allow the use of battered excavation in order to construct the foundations, ground floor slab and a permanent retaining wall, with temporary trench sheet piles used in the north due to close proximity to the sewer.

4.3 Hydrostatic Pressure & Tension Piles

There is no basements present on the Site and thus no hydrostatic pressures will be applied to structural elements. Tension piles will not be required to resist hydrostatic pressures.

4.4 Piling Working Platforms

It is generally necessary that piling rigs operate from a granular platform of over-site fill that is compacted into place in order to provide a safe working area. The platform design is carried out by the specialist subcontractor using the recommendations of the Federation of Piling Specialists (FPS) and Building Research Establishment (BRE), Working platforms for tracked plant (2004).

The geotechnical consultant should provide preliminary guidance for costing purposes until a specialist contractor is engaged in the process and provide detailed guidance.

4.5 Principal aquifer / Contamination - Piling aspects

Special requirements relate to the boring/drilling of boreholes where contamination of underlying aquifers can occur. The Environment Agency will be consulted, and special precautions may need to be employed, such as boring through a bentonite seal and backfilling with cement/bentonite.

4.6 Existing Trees & Heave Protection

A number of existing trees in close proximity to the proposed buildings are to be retained and heave protection to the foundations is likely to be required. A suitable cost allowance should be made at this stage until the extent and details are confirmed.

4.7 Waterproofing

The waterproofing strategy will be the Architect's responsibility. Once a decision on the appropriate waterproofing system has been made PBA will design and take responsibility for that part of the system which is an integral part of the main structural elements, this would cover concrete water retaining structures and any designed joints within the concrete substructure. The design of any water proofing additives to concrete will be the responsibility of the Supplier.

The design of any water proofing system that is not an integral part of the main structure will be to the design and under the responsibility of others.

Requirements of the building regulations

The internal environment must at least satisfy the performance requirements of the Building Regulations. Where an external wall or floor is subject to ground water pressure the Building Regulations refer to BS 8102 (1990) 'Protection of Structures against Water from Ground' for recommendations. This document has been referred to in consideration of the structural aspects of this report.

The remaining requirements of the Building Regulations are considered to fall within the brief of the architect and services engineer.

Selection of the most appropriate form of passive water and vapour protection is the preliminary step prior to considering the active precautions required to achieve the desired internal environment and whether the basement would comply with ventilation, thermal transmission and resistance to moisture penetration requirements

It is assumed that a grade 2 'basement' in accordance with Table 1 of BS 8102:2009 is satisfactory for the Energy Centre 'lower ground'. This will need to be confirmed by the Client / Architect / M&E engineer.

Grade	Example of use of structure ^{A)}	Performance level
1	Car parking; plant rooms (excluding electrical equipment); workshops	Some seepage and damp areas tolerable, dependent on the intended use ^{B)} Local drainage might be necessary to deal with seepage
2	Plant rooms and workshops requiring a drier environment (than Grade 1); storage areas	No water penetration acceptable Damp areas tolerable; ventilation might be required
3	Ventilated residential and commercial areas, including offices, restaurants etc.; leisure centres	No water penetration acceptable Ventilation, dehumidification or air conditioning necessary, appropriate to the intended use
^{A)} The previous edition of this standard referred to Grade 4 environments. However, this grade has not been retained as its only difference from Grade 3 is the performance level related to ventilation, dehumidification or air conditioning (see BS 5454 for recommendations for the storage and exhibition of archival documents). The structural form for Grade 4 could be the same or similar to Grade 3.		
^{B)} Seepage and damp areas for some forms of construction can be quantified by reference to industry standards, such as the ICE's <i>Specification for piling and embedded retaining walls</i> [1].		

Table 4.1: Grades of waterproofing protection to BS 8102:2009.

The 'lower-ground' floor construction should provide the same level of water protection required to achieve the grade described previously.

The need for continuity in the waterproofing protection should also be considered when selecting a type of protection. In most circumstances, the protection should be continuous. In certain situations, e.g. where a drained cavity is combined with an underslab membrane, discontinuity with respect to waterproofing can be acceptable subject to careful detailing and an appropriate assessment of risk.

A reinforced concrete slab will form the basement slab. The water resistance could be provided by external or internal tanking with additional external land drainage if required, to Architect's specification.

5 Structural Design Parameters

5.1 Vertical Actions

The loading allowances made at this stage are shown on the table below. These are based on preliminary information from PRP and MLM.

Case Description	Load [kN/m ²]
Self-weight	-
Permanent (kN/m ²) – General	2.05
Permanent (kN/m ²) – Circulation	2.05
Permanent (kN/m ²) – Podium Level	4.5
Permanent (kN/m ²) – Brown Roof	3.75
Permanent (kN/m ²) – Inverted with ballast	2.00
Permanent (kN/m ²) – Standing Seam Roof	1.00
Cladding (kN/m ² in elevation)	3.00
Live – Residential (kN/m ²)	1.50
Live – Internal Partitions (lightweight movable) (kN/m ²)	1.00
Live – Café/Bar/Shop Area (kN/m ²)	5.00
Live – Circulation (kN/m ²)	3.00
Live – Car Park (kN/m ²)	2.50
Live - Plant (kN/m ²)	7.50
Live - Maintenance (kN/m ²)	0.60

5.2 Snow Loads

The derivation of snow load requires the designer to make judgements on the environment the structure is placed in as well as the form of the structure itself. The potential for the build-up of snow must be allowed for when determining the magnitude of the resulting persistent snow load onto a structure. Eurocode 1-1-3 addresses these issues by establishing a base load and then applying factors to that load that represent snowdrift. The loading and drift potential requirements to all areas of the building including the set-back features at roof levels are being considered in accordance with BS EN 1991-1-3 General Actions – Snow Loads for buildings.

The basic snow load (s_k) is defined as the amount of snow on the ground based on an altitude of 100m above mean sea level. Clause 1.6.1 of Eurocode 1-1-3 sets the probability of exceeding this value at 1 in 50 per year.

5.3 Abnormal Loads

N/A

5.4 Construction Loads

The permanent structure is to be designed for the loads outlined within the previous sections and as detailed on the loading diagrams.

Where the Contractor proposes to support construction loads on the permanent structure he will be responsible for verifying that such loads do not exceed the capacity of the structure. Where additional load capacity is required to support construction loads, the Contractor will be required to design and install temporary works to support these loads. In the design of composite slabs on profiled metal decks, where the deck is required to act as formwork to the slab in the temporary case, a standard construction load of 1.5kN/m² has been considered.

5.5 Thermal Actions – Expansion Joints

Where a building structure is not designed to tolerate the strains generated by moisture changes, temperature variation or settlement, movement joints must be provided in order to reduce the potential for damage to the structural fabric. Technical guidance documents vary in their recommendations regarding the typical spacing for such major movement joints:

- BS6093:2006 Design of joints and jointing in building structures states that major movement joints are typically provided at 30-40m centres
- CIRIA Technical Note 107 recommends major movement joints in non-exposed structures may typically be located at 60-70m centres.

The maximum unjointed structural dimension in the current scheme exceeds the above recommendations. As a result, it will be necessary to design the structure so that it accommodates all movements. In reality the most significant movements will be induced by thermal expansions and contractions. The average annual ambient temperature range is quoted as -10°C to +30°C in the CIRIA guide. For internal structure the temperature range quoted for a building in normal use is +10°C to +30°C and -5°C to +35°C for an empty building, or a building out of use.

In practice these ranges could be adjusted to reflect the beneficial effects of modern insulation but for the purposes of scheme design, a design range of -5°C to +35°C seems reasonable. This design range of 40°C would occur if concrete was cast at -5°C and warmed to +35°C. Over 65m this would equate to an elongation of concrete of circa 30mm.

Given the variation in guidance as detailed above, a technical approach will be favoured in subsequent design stages. Thermal effects will be considered where relevant via the application of temperature load cases in combination with other loads as prescribed by the relevant design codes. Moisture change effects will be similarly addressed by providing adequate minimum reinforcement to limit crack formation in accordance with the code requirements.

5.6 Wind Loading

Wind loads will be derived using BS EN 1991-1-4:2005::

Basic wind speed = 21.3 m/s

Site Altitude = 40m

Symmetric and asymmetric wind load cases are calculated for all directions normal to the building facade. Asymmetric wind load is calculated in accordance with BS EN1991-1-4 clause 7.1.2 'Asymmetric and Counteracting Pressures' to model maximum torsion effect.

5.7 Equivalent Horizontal Force (EHF)

Equivalent (Notional) horizontal forces, which account for lack of fit and eccentricities caused by construction tolerances, are calculated in accordance with BS EN 1992-1-1.

5.8 Load/Action Combinations

The loading combinations defined in BS EN 1990 will be used. The basic loading combinations are listed below.

Combination	Description*
ULS	1.0(SW) + 1.5(WL Total) + 1.0(EHF)
ULS	1.0(SW) + 1.5(WL Asymmetric) + 1.0(EHF)
SLS	1.0(SW) + 1.0(WL Total)
SLS	1.0(SW) + 1.0(WL Asymmetric)

5.9 Movements

For thermal actions and movements refer to section 5.5.

Architectural movement joints to cladding systems and internal partitions will be required subject to sub-contractor design details.

5.10 Deflections and tolerances

Construction tolerances require careful consideration during detailed design and in particular the ‘long’ slab spans. The main issues to address are the cladding to slab edge zones & slab soffits, lift shaft and structural slab levels. Concrete tolerances will be as specified in NSCS Section 7(i) and close control on workmanship will be required to ensure that these are achieved.

Typical tolerances which are likely to be achieved on site are given in BS 5606(i), Accuracy of Building. It should be acknowledged in a survey of completed buildings carried out for BS 5606, that 95% of the slab surfaces surveyed were found to be up to ± 25 mm out of level, with 5% outside the range. This range of tolerance should be considered as likely when designing any trunking or services cast into screeds even though specified ‘target’ values will be lower. Elsewhere, “loose fit” interface zones should be provided in locations where it might be too optimistic to expect specified tolerances to be achieved on site. For further guidance refer to BS 5606.

Deflection limits for the reinforced concrete flat slabs will be given in a PBA Performance Specification and are typically as recommended in App. D of IStructE publication ‘Manual for the design of building structures to Eurocode 1 and basis of structural design’.

Horizontal deflection of the building under wind action will be restricted to less than height/500 (to be agreed subject to cladding requirements).

Vertical Deflections

Design Criteria	Limit
Deflection - Total Long Term	L/250
Deflection - General (following installation of finishes, internal partitions)	L/360 or 20.0 mm
Deflection – Slab edges (following installation of cladding)	L/500 or 12.0 mm
Crack Control – for internally controlled environment	0.4 mm
Crack Control – for external environment	0.3 mm

5.11 Robustness – Disproportionate Collapse

In the United Kingdom buildings are classified according to the number of storeys, occupancy and floor area, which all contribute to the relative severity of a structural failure or collapse. The classification is formalised in Approved Document A of the UK building regulations into Consequence Classes 1, 2a, 2b and 3, increasing in risk from low to high. The revised document incorporating 2013 amendments then makes reference to Eurocodes for requirements for each class.

The proposed development is classified as Consequence Class 2b. The Eurocode recommendations for horizontal and vertical tying, or alternatively, notional removal of elements should be followed.

5.12 Durability

The primary structure will be designed in accordance with the relevant Codes of Practice including relevant British Standards and Eurocodes. This implies that a design life of ‘Normal Life, minimum period 60 years’, as defined in BS 7543, Table 1 will be met. It is assumed that consideration will be given by the architect to design life for other elements of the building such as finishes or waterproofing which have a shorter design life.

The long term durability of the reinforced concrete structure will be achieved by providing adequate cover to the reinforcement as recommended in BS EN 1992-1-1 and BS8500. The choice of protection type will be influenced by location and aesthetics.

Concrete structure within the building has a ‘mild’ exposure condition. All concrete in contact with the ground will be chosen to ensure that it is appropriate for the soil chemistry.

Corrosion protection for the steelwork will be achieved by a suitable paint system which provides a 10-15 year life to first major maintenance. Where steelwork is exposed to the external environment, a galvanized coating will improve durability and reduce maintenance periods. An allowance should be made in the cost plan for an over-painted galvanized finish to all external steelwork, in particular.

The long term durability of timber and glulam elements will be achieved by providing the required preservative treatments in accordance with BS5268-5.

It has been assumed that structure which is located within the cladding zone, such as perimeter columns are located in board of the weatherproof line and can therefore be treated as for internal structure. However, given the columns are inaccessible after construction and to cater for a potential failure in the waterproof membrane, we would recommend that an enhanced level of corrosion protection be considered. It would be prudent to allow in the cost plan for long life high performance paint to be used for all perimeter steel columns which cannot be accessed.

Care is required in the selection of materials for structural components in the cladding zone outside the dry envelope. Fixings are required to address both general corrosion and electrolytic corrosion induced by dissimilar metals.

5.13 Fire Protection

As part of the fire safety strategy, new construction within the building will be required to have a minimum fire resistance. The conceptual requirements for structural fire protection defined by the Fire Consultant Hoare Lea are as follows:

- Residential buildings structural fire protection as shown on table below.
- Car park 30 minute fire rated construction
- Fire-fighting shafts should be afforded 120 minutes fire resistance in terms of loadbearing capacity, integrity and insulation measured from side remote to shaft.

Building Height	No of Storeys	Fire Resistance requirement	Building Block
m	No	min	
0-18	< 6	60	E4
18-30	6 - 9	90	E2, E3, E5, M1, M3, M4, W1, W2-3
30+	≥ 10	120	E1, M2, W4

Inherent fire protection to the concrete elements will be provided by ensuring adequate section size and sufficient cover to reinforcement as recommended in BS EN 1992-1-2:2004, Eurocode 2: Design of concrete structures, Part 1-2: General rules — Structural fire design.

The proposed 225mm thick flat slabs have been designed using the minimum cover allowance resulting in the most efficient design. This nominal cover is 25mm required for durability. With this arrangement the flat slab has inherent fire resistance up to and including 90min. This would cover all building up to 30m high. However, buildings E1, M2 and W4 exceeding 30m in height would require a 120min fire resistance. This could be achieved by either of the following:

- Increasing the concrete cover to 30mm. This would affect the slab design and is anticipated to result in increased reinforcement, or

- Use of fire rated ceiling to provide the required protection. It is assumed at this stage that the ceiling is to provide the full fire resistance required.

The proposed core walls are 250mm thick typically for all buildings up to 11 storeys with 300mm thick walls proposed for the 18 storey M2 building. The required fire resistance of 90min and 120min respectively can be achieved with the proposed wall thicknesses.

Structural steel elements will generally be protected by means of an appropriate intumescent coating. Board-type protection may be preferable in the case of columns. All such fire protection systems will be specified by the Architect. An allowance will need to be made for on-site repair to intumescent coatings

5.14 Design Life

The primary structure will be designed in accordance with the relevant Codes of Practice including relevant British Standards and Eurocodes. This implies that a design life of 'Normal Life, minimum period 60 years', as defined in BS 7543, Table 1 will be met. It is assumed that consideration will be given by the architect to design life for other elements of the building such as finishes or waterproofing which have a shorter design life.

5.15 Design Standards

Eurocodes and their associated National Annexes (providing country-specific design parameters), have superseded British Standards as the principle design codes for structural elements in the United Kingdom. Reference will be made to British Standards and other technical guidance where topics are not adequately addressed in the Eurocodes. It is of note that while no longer current, the superseded British Standards generally remain cited within UK Building Regulations.

The following codes and design guides will be used principally in preparing the structural design for the project. For the sake of brevity National Annexes are not listed:

Eurocode 0: Basis of structural design

BS EN EN1990:2002

Eurocode 1: Actions on structures

BS EN 1991-1-1, BS EN 1991-1-2, BS EN 1991-1-3, BS EN 1991-1-4, BS EN 1991-1-5, BS EN 1991-1-6, BS EN 1991-1-7, BS EN 1991-2, BS EN 1991-3, BS EN 1991-4

Eurocode 2: Design of concrete structures

BS EN 1992-1-1, BS EN 1992-1-2, BS EN 1992-2, BS EN 1992-3

Eurocode 3: Design of steel structures

BS EN 1993-1-1, BS EN 1993-1-2, BS EN 1993-1-3, BS EN 1993-1-4, BS EN 1993-1-5, BS EN 1993-1-6, BS EN 1993-1-7, BS EN 1993-1-8, BS EN 1993-1-9, BS EN 1993-1-10, BS EN 1993-1-11, BS EN 1993-1-12, BS EN 1993-2, BS EN 1993-3-1, BS EN 1993-3-2, BS EN 1993-4-1, BS EN 1993-4-2, BS EN 1993-4-3, BS EN 1993-5, BS EN 1993-6

Eurocode 4: Design of composite steel and concrete structures

BS EN 1994-1-1, BS EN 1994-1-2, BS EN 1994-2

Eurocode 5: Design of timber structures

BS EN 1995-1-1, BS EN 1995-1-2, BS EN 1995-2

Eurocode 6: Design of masonry structures

BS EN 1996-1-1, BS EN 1996-1-2, BS EN 1996-2, BS EN 1996-3

Eurocode 7: Geotechnical design

BS EN 1997-1, BS EN 1997-2

Eurocode 8: Design of structures for earthquake resistance

BS EN 1998-1, BS EN 1998-2, BS EN 1998-3, BS EN 1998-4, BS EN 1998-5, BS EN 1998-6

Eurocode 9: Design of aluminium structures

BS EN 1999-1-1, BS EN 1999-1-2, BS EN 1999-1-3, BS EN 1999-1-4, BS EN 1999-1-5

BS8102 2009 Protection of below ground structures against water from the ground

SCI P354 Design of floors for vibration: A new approach

BS6472-1:2008-:Evaluation of human exposure to vibration in buildings

The Concrete Centre: A design guide for footfall induced vibration of structures

Building Regulations: all relevant sections, including Approved documents A & B concerning structure and fire safety

Steelwork Design Guide to BS 5950, Vol 1, 2nd Edition - SCI

Movement in Buildings - CIRIA Technical Note107

National Structural Concrete Specification (NSCS) – 3rd Edition

National Structural Steelwork Specification – 5th Edition

5.16 Reinforcement quantities

Preliminary reinforcement quantity estimates for the concrete elements are shown in the table below.

RC element	Reinforcement Grade	Element Size	Reinforcement Estimate	Additional Requirements
CFA piles	C32/40 - B500B	D 600	50kg/m3 (assuming 10m long cage, 6H20, H10 Helical with 200mm Pitch, Piles 22m Long)	DS-3, AC-3s
Pile caps	C40/50 - B500B		150 kg/m3	DS-3, AC-3s
Ground slab	C40/50 - B500B	275mm	130 kg/m3	DS-3, AC-3s
RC retaining walls	C40/50 - B500B	250/300thk	150 kg/m3	DS-3, AC-3s
Ground beams	C40/50 - B500B	400 WD x 600 DP	250 kg/m3	DS-3, AC-3s
RC columns	C40/50 - B500B	variable	320 kg/m3	
RC core walls	C40/50 - B500B	250 thick	130 kg/m3	
RC flat slabs	C40/50 - B500B	typical 225mm	160 kg/m3	

Table 5.1: Reinforcement Estimates

6 Design Interfaces

6.1 Introduction

Structural components need to be considered in conjunction with finishes and the requirements of other disciplines which usually follow in construction programming terms. This section of the report serves to highlight the various interfaces which need to be resolved at the final proposals stage.

6.2 Building Services within the Building

A flat slab soffit to all floor slabs has been provided to ensure maximum flexibility for the distribution of services. Where clashes are present i.e. builders work openings in core walls, careful coordination of the services will be required to ensure minimal intrusion into the clear headroom space below.

All builders work will be coordinated with the M&E design.

6.3 Building Services/Utilities serving the Site

To facilitate co-ordination an integrated underground services layout should be produced for construction incorporating incoming utilities, underground drainage and substructure. Information on incoming utilities will need to be provided by the services consultant.

Underground drainage will be fully coordinated with the substructure.

Underground drainage will be fully coordinated with the substructure. Careful co-ordination will be required where services are required to pass through the foundation raft.

6.4 Cladding/Roofing

Detailed co-ordination will be required with the cladding consultant/manufacture. Options are being investigated in order to avoid the need for scaffolding for the erection of the external cladding envelope and maximise the use of prefabrication/panelisation.

At this stage an external brickwork leaf has been assumed to be supported on proprietary shelf angles which would be fixed to the slab edge using cast-in channels.

The floor slabs will need to be designed to perform within specific tolerances and deflections, based on recommendations given by the cladding consultant/manufacture.

6.5 Acoustics

Acoustic detailing needs to be addressed for compliance with Approved Document E of the Building Regulations

7 Constraints and Approvals

- 7.1 Thames Water**
Refer to the Civil Engineering Design Stage 3 Report
- 7.2 Environment Agency**
Refer to the Civil Engineering Design Stage 3 Report
- 7.3 River Wall**
Refer to the Civil Engineering Design Stage 3 Report
- 7.4 Reading Borough Council Highways**
Refer to the Civil Engineering Design Stage 3 Report

8 Health and Safety – Design Risks

8.1 Construction (Design and Management) Regulations 2015

- 8.1.1 This construction project falls under the Construction (Design and Management) Regulations 2015. These regulations impose legal responsibilities on various parties involved in the procurement, design and construction of building projects, including the client

PBA are classified as designers under the regulations. We confirm that we have the relevant experience to complete design services for projects of this type, size and nature and that adequate, competent resources have been and will be allocated to the project at the appropriate design stages.

8.2 Design Risks

It is good practice, and a requirement under the CDM regulations, that competent contractors are appointed to carry out the works. It can be assumed therefore that a competent contractor will be aware of all the normal design risks involved in the construction process for projects of a similar type, size and nature.

Design risks will be considered by PBA at each stage of the design progress. Where possible these risks will be designed out of the project. Where this is not possible any residual risks will be noted on the project drawings.

The 'abnormal' design risks at this stage of the project are listed below.

- The existing buildings are likely to be founded on piled foundations. Existing piles' position should be recorded during demolition of the existing buildings and the proposed foundations design amended accordingly to avoid clash with existing piles where possible.
- Thames water sewer to the north in close proximity to the proposed building foundations. Instances with piling within easement zone but outside 1.5m exclusion zone will need a 'build near' the sewer agreement with TW.
- The existence of the river wall ties propping the river wall at high level is to be confirmed. Further investigation works may be required to confirm this.
- Tow path changes
- Soil contamination risk. A site specific geotechnical investigation has been carried out. The geotechnical consultant should advise of any specific risks and requirements.
- Underground obstructions during excavations are likely to be found and an allowance should be made at this stage for their removal.

8.3 Temporary Works

Information about the responsibilities associated with temporary works can be found in the Institution of Civil Engineers Conditions of Contract and BS 5975:2008 temporary works procedures and the permissible stress design of falsework. This requires a careful approach to temporary works design and a review undertaken by competent persons and the appointment of a Temporary Works Co-ordinator to oversee the process and falsework for slabs and beams

Specific temporary works items (not exhaustive) as follows:

- Demolition
- Temporary propping existing retaining walls
- Foundation excavation, propping and sequencing
- Piling mat
- Propping and falsework for slabs and beams

9 Further Investigations

The following lists further recommended investigations to establish necessary engineering parameters prior to construction to enable economy of design.

9.1 Existing building

Existing pile location should be recorded as part of the demolition works to avoid clashing with new piles.

9.2 River Wall

The existence and extent of river wall ties will need to be confirmed. No record information of the river wall construction has been found, and thus intrusive ground investigation will be required in agreement with the EA. A Ground Penetrating Radar (GPR) investigation could also be considered prior to any intrusive investigation.