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Report on
Geotechnical Desktop Investigation

Proposed Commercial Development
383 Kent Street, Sydney

Prepared for
Charter Hall Holdings Pty Ltd

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Report on Geotechnical Desktop Investigation
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1. Introduction

This report presents the results of a geotechnical desktop investigation undertaken for a proposed commercial development at 383 Kent Street, Sydney. The desktop study was carried out for the Planning Proposal stage, with the project seeking approval for a building envelope, only. The report has been prepared on behalf of Charter Hall Holdings (the proponent) and it was undertaken in accordance with Douglas Partners' proposal 217267.00.P.001.Rev1 dated 2 September 2022.

It is understood the proposed development will comprise a >75,000sqm GFA office tower (with a single basement level car park (below Sussex Street) with premium grade services.

The aim of the desktop study was to provide preliminary geotechnical advice comprising the following:

- Geology, including groundwater.
- Excavatability of materials likely to be encountered.
- Shoring/retention systems.
- Foundations.
- Impact on Transport for New South Wales (TfNSW) assets, the CBD Rail Link tunnels (Up and Down tracks).
- Further geotechnical work.

2. Site Description

The proposed development covers 383 Kent Street, Sydney (DP778342), as shown in Figure 1. The site is approximately rectangular shaped, bounded by Kent Street to the east, Sussex Street to the west, 397-411 Kent Street to the south, 379-381 Kent Street to the north-east and 160-166 Sussex Street to the north-west. The site has an area of approximately 3,600 m^2 and has a street frontage of approximately 43 m on Sussex Street and 52 m on Kent Street (refer Detail Survey Plan Drawing: Detail 3D-A, prepared by Beveridge Williams, Appendix B). The ground surface level along Kent Street boundary and the Sussex Street boundary is approximately RL 19.1 m and RL 9.5 m, respectively.

The site is currently occupied by a mixed-use building consisting of 10 levels of public car park and 11 levels of commercial space above. The site has three existing basement levels below Kent Street, which extends horizontally towards Sussex Street ground level. There is vehicle access from both Kent Street and Sussex Street. It is further understood the site is constrained by the future CBD Rail Link rail reserve with proposed tunnels positioned under both Kent Street and Sussex Street (refer Sketch No: SK-001- 1, prepared by Robert Bird Group, Appendix D).

Figure 1: Aerial photograph of site (red boundary line).

3. Regional Geology

Reference to the Sydney 1:100 000 Geological Series Sheet 9130 indicates that the site is underlain by the Hawkesbury Sandstone of Triassic age, comprising medium to coarse-grained quartz sandstone with minor shale lenses (refer Figure 2). The Hawkesbury Sandstone typically is pale to mid grey in colour, when fresh, and has both massive and cross bedded units with strength properties mainly in the medium to high strength range. The rock is prone to weathering with red brown or brown iron staining common in the upper beds.

Geological mapping carried out in the Sydney region identified two main joint sets which will most likely be present on this site:

- **Set 1** - NNE striking joints dipping 75° to 90° to the east and west, generally spaced between about 1 to 10 m and persistent over many metres.
- **Set 2** - ESE striking joints dipping 75° to 90° to the north and south, generally widely spaced but can be as close a 500 mm apart. These joints are generally strata bound.

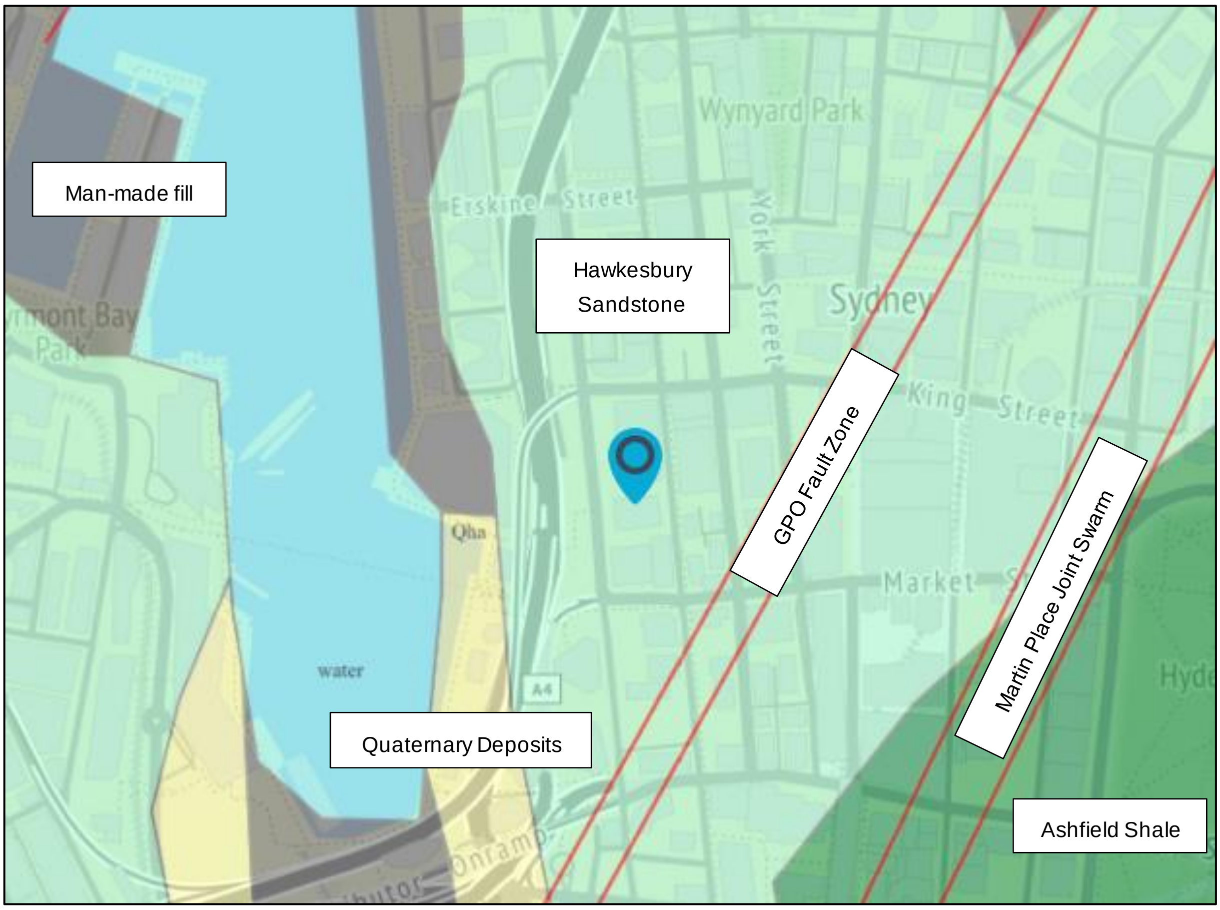


Figure 2: Extract from Sydney 1: 100,000 Geological Series Sheet (Site Shown by Blue Pin)

The extent of the north-north-east trending GPO Fault Zone and Martin Place Joint Swarm are indicated by the red lines.

4. Previous DP Investigations

DP has previously carried out a number of geotechnical and environmental investigations and provided advice/services during construction at a number of nearby sites (see Figure 3). Some notable geotechnical projects in the immediate vicinity include the following:

- **350 Kent Street (DP Ref 10990):** Geotechnical Investigation carried out in 1988, consisting of three test cores to bedrock at depths between 3.3 m and 3.5 m.
- **355 Kent Street and 361-363 Kent Street (DP Ref: 29695):** Geotechnical investigation carried out in 2001, consisting of test pit and rock face inspections.
- **365-377 Kent Street (DP Ref 24515A):** Geotechnical investigation carried out in 1998, consisting of two boreholes drilled to depths of up to 27 m below ground level.

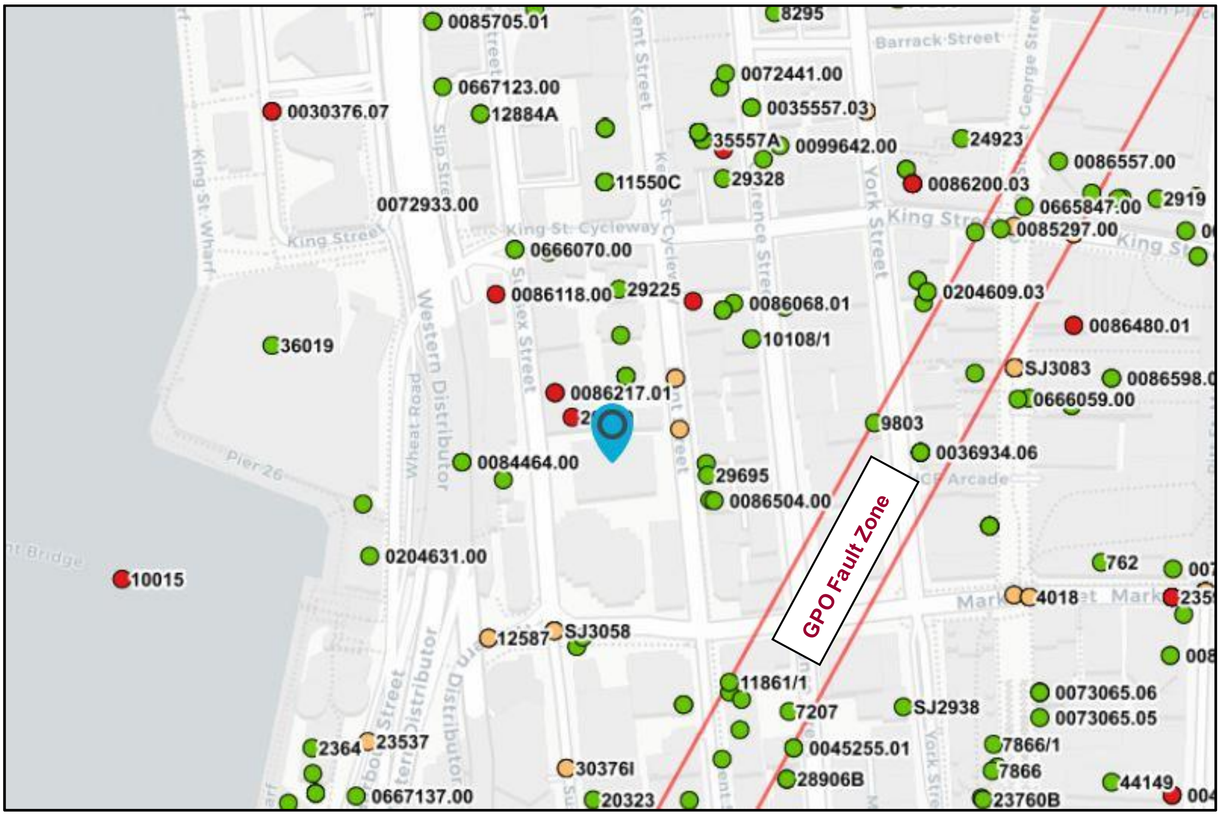


Figure 3: DP projects in the Vicinity of 383 Kent Street, Sydney, and proximity to GPO Fault Zone.

5. Preliminary Geotechnical Model

Based on the findings from these geotechnical investigations, the information from the Geological Sheet and DP's knowledge from other projects involving excavations nearby, the expected subsurface profile from original ground level at the site can be summarised in Table 1:

Table 1: Expected Subsurface Profile from Original Ground Level

Units		Description
1	Fill	Local fill between 0.8 m and 1.2 m.
2	Residual Soils	Stiff to very stiff sandy clays to depths of 2.0 m, with ironstone bands present.
3	Weathered Rock	Extremely low to low strength weathered sandstone to depths of up to 3.2 m.
4	Sandstone	Medium strength and stronger sandstone below depths of 2.2 m and 3.2 m.

Based on available information, the thickness of soil and weathered rock increases towards the south.

The permanent groundwater level is likely to be at depth (i.e., below the neighbouring basement levels), however, it is likely that groundwater seepage will occur along the soil/rock interface and bedding planes, joints, and faults, particularly after wet weather.

The ground profile presented above is preliminary, only and will need to be confirmed by subsurface investigation including diamond coring of rock at several locations across the site and the installation and monitoring of water levels in temporary groundwater monitoring wells.

6. Proposed Development

The proposed development at the site is in its planning stage. It is understood the proposed development is to create a >75,000sqm GFA office tower (including a single level basement) with premium grade services (refer "For Information" drawings prepared by fjmt, Appendix C). It is understood that there will be a one level basement below Sussex Street level to be utilised as a car park which DP have been advised by Touchstone Partners to be between 3 m and 3.5 m below the existing ground level (at approximately RL 9.8 m at Sussex Street).

The site is constrained at both Kent Street and Sussex Street ends with the 2 and 3 protection zones crossing over the property boundary (refer drawings Appendix C and Appendix D). The Type 1 protection zone is shown to cross the property boundary along Sussex Street, but strikes parallel outside the property boundary along Kent Street.

7. Comments

The following comments have been prepared for planning and preliminary design purposes, only. The geotechnical model and advice will need to be reviewed following completion of a detailed geotechnical investigation.

7.1 Existing Retention Structures and Adjacent Buildings

Prior to below ground demolition and excavation, it will be necessary to determine the type, thickness and founding conditions of the existing basement retaining structures along the north, eastern and southern boundaries. Determining the details of the existing basement retaining structures will require investigation by careful and controlled exposure of the ground behind and at the base of the existing walls, as required to assess the current lateral earth pressures on the walls and founding conditions. This process is critical as demolition and excavation could potentially destabilise the walls and footings.

Information of the neighbouring footings along the boundaries of the site will also be required if they will be affected by the development. These footings may be founded adjacent to or within the zone of influence (taken as a 45° line drawn up from the base of the proposed bulk excavation level) of the proposed excavation down to RL 6.8 m. Excavation adjacent to these footings may remove confinement (especially higher-level footings) which may induce additional settlement and reduce the allowable bearing pressure of the material beneath the footings. An assessment of the bearing capacity beneath these neighbouring footings should be undertaken to ensure the foundations remain within their serviceability design limits. Depending on their founding level and foundation material, the neighbour's footings may require underpinning.

If reliable and accurate records of the existing structure and adjacent footing types and foundation conditions are available, these records would assist in the preliminary assessment. DP can provide an appropriate investigation and underpinning methodology once the position of the neighbouring footings relative to the zone of influence of the proposed bulk excavation has been established.

7.2 Excavation Conditions

Excavation is currently planned to RL 6.8 m. Excavation is expected to encounter fill, residual soil and sandstone of up medium strength or better.

7.2.1 Excavatability

Fill, residual soils and extremely low to very low strength rock should be readily excavated by hydraulic excavators. Excavation of the underlying bedrock will largely be dependent on the rock strength and discontinuity spacing encountered and may require rock hammers, rock saws and ripping.

Detailed excavation for footings and service trenches / pits should be achievable using rock hammers, hydraulic rock saws or milling heads. Rock saws may also be required to reduce the risk of vibration affecting adjacent structures. Piling may be required within the Type 2 and 3 protection reserves. It is recommended the piling/earthworks contractor carry out an independent excavatability assessment (once geotechnical investigation has been completed) prior to tendering for excavation.

7.2.2 Trafficability

During construction, problems may be experienced with site trafficability during wet weather in areas where residual soils are found at surface or at excavation level. For general construction machinery, tracked vehicles should be used.

If larger plant such as piling rigs, heavy mobile cranes etc are to be used on fill, residual soils or very low strength rock, a working platform is likely to be required. A working platform assessment should be carried out based on the detailed applied track loads provided by the piling contractor or earthworks contractor for the different rigs/cranes.

7.2.3 Ground-borne Vibration

During demolition and excavation, it will be necessary to use appropriate methods and equipment to keep ground vibration at adjacent buildings and structures within acceptable limits. For buildings, the level of acceptable vibration is dependent on various factors including the type of building structure (e.g., reinforced concrete, brick, etc.), its structural condition, founding conditions, the frequency range of vibration produced by the construction equipment, the natural frequency of the building and the vibration transmitting medium.

Ground vibration can be strongly perceptible to humans at levels above 3 mm/s component peak particle velocity (PPVi). This is generally much lower than the vibration levels required to cause structural damage to most buildings. The Standard AS / ISO 2631.2 – 2014 "Mechanical vibration and shock –

Evaluation of human exposure to whole-body vibration – Vibration in buildings (1 Hz to 80 Hz)" suggests an acceptable daytime limit of 8 mm/s component PPVi for human comfort.

Based on DP's experience and with reference to AS/ISO 2631.2, it is suggested that a maximum component PPVi of 8 mm/s (measured at the first occupied level of neighbouring buildings) be provisionally adopted at this site for both architectural and human comfort considerations for 'modern buildings.

DP maintains an extensive construction vibration database. As a preliminary estimate, Table 2 provides approximate minimum buffer distances for selected equipment for excavation, based on a set vibration limit of 8 mm/s (assuming that plant is appropriately sized for the ground conditions).

Table 2: Approximate buffer distances for selected Plant (PPVi 8 mm/s)

Excavation Plant		Distance from plant at which vibration attenuates to 8 mm/s	
Type	**Operating Weight**	**From DP Trial Maxima[1]**	**From DP Trial Average**
Rock saw on excavator[2]	-	1 m	0.5 m
Ripper on 20 t excavator	-	3 m	0.7 m
Rock Hammer	<500 kg	7 m	3 m
	501 – 1000 kg	8 m	3 m
	1001 – 2000 kg	13 m	5 m

Notes:

1. *Smaller distances can generally be determined from individual trials, as indicated by those from trial averages.*
2. *Buffer distances for rock hammers may be slightly reduced by prior saw cutting along, or parallel to, excavation boundaries.*
3. *Loading effects from adjacent buildings may reduce vibration levels, to enable boundary saw cuts with few exceedances.*

As the magnitude of vibration transmission is site specific, it is recommended that a vibration trial is carried out at the commencement of demolition and rock excavation. These trials may indicate that smaller or different types of excavation equipment are required to reduce vibration to acceptable levels.

7.2.4 Dilapidation Surveys and Monitoring

Dilapidation surveys should be carried out on adjacent buildings, structures, pavements, services and sensitive structures that may be affected by the excavation works. A baseline (reference) survey should be carried out before the commencement of any demolition or excavation work in order to document existing defects so that any claims for damage due to construction related activities can be accurately assessed.

Follow on dilapidation surveys may be required during construction. Final dilapidation surveys should be carried out on completion of the project to check for any impact from the works.

7.2.5 Disposal of Excavated Material

All surplus excavated materials will need to be disposed of in accordance with the provisions of the current legislation and guidelines including the *Waste Classification Guidelines* (EPA, 2014) and the Protection of the Environment Operations Act 1997 (POEO Act). All materials removed from the site are defined as waste under the POEO Act and must be disposed of in accordance with one of the following:

- Virgin excavated natural materials (VENM) as defined under the POEO Act, permitting beneficial reuse.
- A waste category meeting the criteria set out in the NSW EPA Waste Classification Guidelines (2014), with the materials disposed to a landfill licenced to receive the waste under the assigned classification or taken to a recycling facility licenced to receive the waste.
- Material complying with a Resource Recovery Order (RRO) as defined under the Protection of the Environment Operations (Waste) Regulation 2014, with complying materials able to be reused under certain conditions.

Accordingly, environmental testing will need to be carried out to determine the most appropriate off-site destination(s) for the surplus excavated material.

7.3 Excavation Support

7.3.1 General

Careful consideration must be given to the planning and design of excavations and excavation retention system(s), especially along boundaries where excessive deformation or failure can cause damage to nearby buildings, road infrastructure, footpaths, services, etc.

The proposed additional basement level is shown not to extend the full length between Sussex Street and Kent Street. No additional excavation is planned along the Kent Street boundary, whereas the Sussex Street side of the development will be deepened by 3.0 m below current ground level. It may be possible to use the existing retaining walls to temporarily shore the upper excavation, depending on the founding depth, founding material and position of these walls. Careful consideration should therefore be given to the design of the excavation sidewall retention systems. Whether existing or new, all walls are to be temporarily supported with anchors. New shoring will be required where the excavation faces do not align with the existing walls or where the existing walls are in the way. Particular care should be taken where installation of the new wall is obstructed by the existing wall. A special approach will be required in such a case, where the old wall is systematically removed as the new wall is installed. This will require a design and construct approach and is generally carried out under close supervision of the geotechnical and structural engineers. Horizontal drilling and slot investigations will be required where existing basement walls are used.

Shoring should be designed to support the soil, weak rock and any surcharge loads, taking into account the allowable deformation limits of any affected services and structures as well as Transport for New South Wales (TfNSW) requirements (see Section 7.7). The levels and types of footings beneath the adjacent buildings are not known. It is assumed, until confirmed otherwise, that they are founded at a higher level than the existing bulk excavation level at the site. As-built drawings of the neighbouring buildings should be requested. Investigation will need to be carried out to determine the founding level and founding conditions where these drawings are not available. Underpinning and additional support of the neighbouring buildings may be required.

7.3.2 Battering/Excavation Faces

Battering of the excavation sides at safe angles may be possible where there is sufficient distance to the boundary (likely along Kent Street). Temporary batters and permanent batters of <3 m in height in fill/soils should be cut at no steeper than 1.5:1 (H:V) and 2:1 (H:V), respectively. Temporary batters and permanent batters of <3 m in height in low strength sandstone, should be cut no steeper than 0.75:1 (H:V).

All batters will have to be inspected with every 1.5 m drop in level to confirm that the rock is not adversely affected by discontinuities. Permanent batters over 3 m in height should be designed individually. These recommended safe batter angles are expected to remain stable provided all surcharge loads, including construction loads and stockpiles, are kept well clear (at least 3 m) of the crest of the batters.

Where there is insufficient space for battering, excavations will require temporary and permanent retention. The retention system (shoring) should be designed to support the soil, low strength rock and all surcharge loads, taking into account the allowable deformation limits for adjacent buildings and surrounding services.

Excavation for the single level basement is planned, excavation may cause stress relief within the rock mass depending on depth and rock strength. From numerical modelling and site monitoring at similar sites within the Sydney, the stress relief movements vary from 0.5 to 2 mm/m depth of rock excavation, measured at the crest, midpoint of the face, reducing to near zero in the corners of the excavation. Stress relief movement decreases horizontally with distance away from the excavation. Horizontal stress relief movement can be expected to occur (albeit very minor) to distances back from the excavation of up to the equivalent of 2 times the length of the excavated face. Careful consideration of the effects of stress relief will be needed when considering the existing neighbouring buildings and surrounding services.

7.3.3 Earth Pressures for Shoring Design

It is suggested that preliminary design of shoring with one row of anchors or propping, should be based on a triangular earth pressure distribution using the earth pressure coefficients provided in Table 3. 'Active' earth pressure coefficient (K_a) values may be used where some wall movement is acceptable. 'At rest' earth pressure coefficient (K_o) values should be used where the wall movement needs to be limited.

Table 3: Earth Pressure Coefficients for Preliminary Design Purposes

Material	Unit Weight (kN/m^3)	Earth Pressure Coefficient	
		Active (K_a)	At Rest (K_o)
Fill	20	0.35	0.5
Residual soil	20	0.35	0.5
Very low/low strength sandstone	22	0.2	0.3
Medium strength or stronger sandstone	24	0*[1]	0*[1]

Note: *[1] Assuming no adverse dipping joints are present

The triangular earth pressure distribution on the wall can be calculated as follows:

$$H_z = K(\gamma z + p)$$

Where:	H_z	=	horizontal pressure at depth z (kPa)
	γ	=	unit weight of soil or rock (kN/m^3)
	K	=	earth pressure coefficient
	z	=	depth (m)
	p	=	vertical surcharge pressure (kPa)

For braced walls or where two or more rows of anchors/propping are used, the shoring can be designed using a rectangular or trapezoidal earth pressure distribution.

An alternative approach could also be used (commonly used for braced shoring systems or where multiple rows of anchors are installed), where the support pressure is related to the height of soil/weathered rock retained. Where there are no movement-sensitive structures nearby, an earth pressure distribution equal to 4H kPa can be used (where H, in metres, equals the depth to the top of self-supporting medium strength or stronger rock). Where the wall movement is to be minimised (i.e., close to adjacent buildings or services) the lateral earth pressure can be calculated using 6H kPa. For movement-sensitive structures, where it is critical that deformation is controlled, it may be necessary to calculate the pressure using 8H kPa.

These pressures can be applied as either rectangular or trapezoidal earth pressure distributions. Note these earth pressure distributions are "pressure envelopes", selected to ensure that no row of anchors are overloaded during the temporary support phase. The actual magnitude and distribution of lateral earth pressures for the building in its final (long term) condition may differ from the uniform distributions given above. The final condition earth pressures can be assessed using numerical methods

In all cases, additional surcharge loads such as new and existing footings, construction loads, etc., must be allowed for in the design, applied as a rectangular earth pressure distribution over the depth of influence.

The earth pressure loading described above does not include either earthquake loads or hydrostatic pressures. Unless positive drainage measures are incorporated to prevent water pressure build-up behind the walls, full hydrostatic head should be allowed for in design, while at the same time reducing the unit weight to account for the buoyant condition.

Passive resistance for piles or structures founded below bulk excavation level may be based on a 'working' passive bearing pressure of 3500 kPa, provided that the rock comprises medium strength or stronger sandstone which is not adversely affected by discontinuities. The first 0.5 m of rock socket or excavation below the bulk excavation level should not be considered for the purpose of passive restraint. The minimum socket depth should be equal to the greater of one pile diameter or 1.0 m below the lowest level of any nearby excavation (including any detailed excavations), subject to analysis. This is also relevant where toe anchors are installed, just prior to fully exposing the toe of the pile. All other cases should be assessed individually.

Staged excavation and inspection by a suitably qualified geotechnical engineer will be required to confirm that the rock in front of the wall/pile is not adversely affected by discontinuities where passive resistance is relied upon.

7.3.4 Self-Supporting Rock Faces and Rock Discontinuities

As discussed in Section 3, the two major joint sets (NNE and ESE) in Hawkesbury Sandstone are very prominent and can dip up to 15° (off the vertical) to the east or west. The ESE joints are typically strata bound, are generally not as persistent and are more widely spaced than the NNE joint set. Bedding planes and soft weathered seams are common in the Hawkesbury Sandstone, even if the rock is of high strength. These joints, bedding planes and seams (discontinuities) can adversely affect the rock mass and form unstable feather edges, rock slivers, blocks and wedges.

Excavation for the additional basement level on the Sussex Street side is expected to encounter medium strength or stronger sandstone. Excavated faces in medium strength or stronger sandstone are only considered self-supporting if they are not adversely affected by discontinuities. Rock mass support can only be finalised once the actual joint location, dip and dip direction have been determined during excavation. Excavation should therefore be carried out in a controlled manner, with inspections by a suitably experienced engineering geologist / geotechnical professional every 1.5 m drop to determine if such wedges are present and whether support is required. The requirement for regular geotechnical inspections every 1.5 m drop should be explicitly stated on the drawings and the earthworks contractor should be made fully aware of this requirement.

Allowance should be made for ground anchors, rockbolting and shotcrete support. All clay seams and shale layers (>50 mm thick) will require shotcrete protection to prevent future weathering and fretting/regression. All thick shale / laminate seams will also require, in addition to the shotcrete face protection, rockbolting or anchor support.

7.3.5 Ground Anchors and Rockbolts

It is anticipated that the building will support the shoring wall in the long term and therefore any ground anchors are expected to be temporary only. The use of permanent anchors, if required, would need careful attention to corrosion protection for which further geotechnical advice should be sought.

Post-stressed ground anchors, rockbolts and dowels (support elements) can be used to laterally support existing walls, new shoring, underpinning works or unstable rock blocks and wedges. Anchors could also be used vertically as hold-down anchors to resist temporary or long term uplift of the core / walls and should be designed as per AS 4678. The designer should check the cone-pull-out failure mechanism by assuming a 90° cone in medium to high strength, slightly fractured sandstone (or better). Note that the buoyant weight of the rock should be used below the water table.

Support elements used for lateral support should be bonded in the stronger rock, inclined as required, but preferably not steeper than 30° below the horizontal. Ground anchors should be designed to have a free length equal to their height above the base of the shoring with a minimum free length of 3 m. Table 4 provides ultimate and allowable bond stresses for preliminary design and estimating purposes.

Table 4: Bond Stresses

Material	Allowable Bond Stress (kPa)	Ultimate Bond Stress (kPa)
Very low to low strength sandstone	150	350
Medium strength sandstone	350	800
Medium to high strength sandstone	600	1500

These values should be confirmed by pull-out tests prior to installation of support elements. Ultimately, it is the contractor's responsibility to ensure that the correct design values (specific to the support system and method of installation) are used and that the support element holes are carefully cleaned prior to grouting.

After temporary support elements have been installed, it is recommended that they are tested to 125% of their nominal Working load. Where stress relief or further unavoidable movement of the shoring is expected, it is recommended that the support elements are locked-off at a lower value, as required to accommodate the additional movement and subsequent increase in stress in the support elements. Checks (lift off-tests) should be carried out to confirm that the load in the support elements has been maintained and that losses due to creep or other causes do not occur.

Shorter support elements (i.e., rockbolts, dowels and pins) may be required to support unstable rock wedges, slivers, blocks or feather edges formed where sub-parallel joints intersect the face. Shotcrete with mesh (or fibrecrete) may be required where beds / seams of extremely low or very low strength rock are encountered within higher strength sandstone, secured with anchors, rockbolts, dowels or pins, as required.

Care should be exercised to ensure that anchors are installed progressively during excavation and stressed prior to excavation of the next drop to ensure that stability is maintained at all times. All shoring support elements should be installed prior to demolition of the existing basement floor slabs.

It should be noted that permission will be required from authorities and adjacent property owners prior to installing rockbolts / ground anchors below their land. Due consideration should also be given to below-ground excavations, services, etc.

7.4 Groundwater

It is expected that the regional groundwater table will be near the planned bulk excavation level of the basement. Seepage should therefore be expected along the top of the rock (particularly after periods of wet weather) and through the rock mass, joints and bedding planes in the rock face.

If the groundwater level is found to be above the bulk excavation level, yearly seepage could exceed 3 megalitres. During construction and in the long term, however, it is anticipated that seepage into the excavation could be controlled by perimeter drains connected to a "sump-and-pump" system. Approval from Water NSW, however, will be required prior to designing and construction of a drained basement. A drained basement, if approved by Water NSW, will require permanent subfloor drainage to direct seepage to the stormwater drainage system.

It is not possible to provide a reliable estimate of the seepage quantity that may be expected within the basement based on the available data. Rock mass permeability testing will therefore be required during the geotechnical investigation to provide the necessary parameters for seepage analysis.

Previous experience in Sydney is that seepage will likely contain relatively high levels of soluble iron that will form a precipitate in the form of a gelatinous 'sludge' when exposed to oxygen. This 'sludge' has the potential to block-up subsoil (gravel) drains and 'seize-up' pumps. Therefore, detailing of subfloor drains, sumps and pumps should incorporate provision for regular maintenance such as flushing and 'rodding' of drains and/or "baffle" pits.

7.5 Foundations

The preliminary geotechnical model suggests medium or medium to high strength sandstone is expected at bulk level.

Typical parameters for the design of foundations on sandstone, based on the classification methods of Pells et al. (1998) are shown in Table 5, subject to spoon testing/proof coring, where required. Shaft adhesion values for uplift (tension) in piles or hold down anchors may be taken as being equal to 70% of the values for compression, provided that adequate socket roughness is achieved. Note that hold down anchors will also require a cone of sufficient rock mass to resist uplift, as per 4678-2002(+A2).

Table 5: Preliminary Design Parameters for Foundation Design

Foundation Stratum	Allowable Bearing Pressure (Serviceability)		Ultimate Bearing Pressure		Typical Field Modulus (MPa)
	End Bearing (kPa)	Shaft Adhesion (Compression) (kPa)	End Bearing (kPa)	Shaft Adhesion (Compression) (kPa)	
Medium strength sandstone	3,500	350	20,000	800	700
Medium to high strength sandstone	6,000	600	60,000	1,500	1200

Note: Shaft adhesion applicable to the design of bored piles, uncased over the rock socket length, where adequate sidewall cleanliness and roughness are achieved.

Foundations proportioned on the basis of the allowable bearing pressures in Table 5 would be expected to experience total settlements of less than 1% of the pile diameter or footing width under the applied working load, with differential settlements between adjacent columns expected to be less than half of this value.

To use a bearing pressure value for design of greater than 3.5 MPa, a minimum of six cored bores are required with spoon testing carried out in a third of footings across the site during construction. If bearing pressures greater than 6 MPa are used in design, then cored bores at a maximum 10 m grid spacing or cored bores for 50% of footings and spoon testing carried out on the remaining footings are required.

For spoon testing, a 50 mm diameter hole is drilled below the base of the footing to a depth of 1.5 times the footing width, followed by testing to check for the presence of weak layers or clay bands.

For design using the ultimate values provided in Table 5, a geotechnical strength reduction factor ($Ø_g$) should be determined by the designer. The serviceability assessment should be based on using geotechnical parameters that are appropriately selected and to which no reduction factor is applied.

Footings from neighbouring buildings may be founded within the zone of influence of proposed excavation. The zone of influence may be taken as a 45° line drawn up from the base of the proposed bulk excavation level. The allowable bearing pressure beneath neighbouring footings located within this zone of influence is generally reduced, down to 60% of the original value. An assessment of the bearing capacity beneath these neighbouring footings should be undertaken to ensure the foundation remain within their serviceability design limits. Progressive inspections of excavated faces below neighbouring footings will be necessary in 1.0 m drops to check the ground profile including any defects or adversely dipping joints that may affect the neighbouring foundation performance.

All foundations should be inspected by a geotechnical engineer to confirm that foundation conditions are suitable for the design parameters, and proof drilled, or spoon tested as appropriate. If weak seams or defects are encountered, footings may need to be deepened until suitable foundation material is reached. Alternatively, the footing can be enlarged (bearing in mind differential settlement and structural tolerance), or redesigned for a lower bearing pressure.

Additional geotechnical advice for pile design can be provided if deep piles are required.

7.6 Design for Earthquake Loading

A Hazard Factor (Z) of 0.08 would be appropriate for preliminary design in accordance with Australian Standard AS 1170.4 – 2007 *Structural design actions – Part 4: Earthquake actions in Australia*. The site sub-soil class is considered to be Class B_e.

7.7 Geotechnical Considerations Relating to the CBDRL Corridors

Based on data available from the surrounding area, the geotechnical conditions of the site can be predicted with a reasonable level of confidence. However, site-specific conditions will need to be investigated.

It is understood from the information provided that the future CBD Rail Link (CBDRL), with a Up and Down tunnel proposed below Kent Street and Sussex Street, respectively. The ASA Standard (Developments near Rail Tunnels T HR CI 12051 ST V2 - Developments Near Rail Tunnels, dated November 2018), sets out TfNSW requirements for proposed developments near existing underground rail tunnels and infrastructure. All excavations exceeding 2 m in depth closer than 25 m from the rail corridor requires assessment of the potential impact of the proposed excavation on the tunnels or vice versa. New foundation loads, including a change in load from existing, conditions are also required to be assessed when within 25 m of the rail corridor (such as this case).

It is noted that the CBDRL is currently an easement and, therefore, any building will need to take into account he future construction of the tunnel and not impede the construction of the tunnel. Based on current plans of the CBDRL, the proposed bulk excavation is likely to be approximately 10 m above the "First Reserve" of the proposed down track and the up track is offset from the boundary. As the bulk excavation is predicted to be in medium strength rock or better, the impact from the construction of the tunnels is predicted to be small and manageable (to be confirmed by numerical modelling).

8. Recommended Additional Geotechnical Work

The above advice is based on a desktop assessment of predicted subsurface conditions at 383 Kent Street, Sydney. It is suitable for planning purposes only. Confirmation of ground conditions will therefore be required.

The following additional work is recommended at a later stage:

1) Geotechnical investigation of the site comprising diamond core drilling to at least 4 m below the bulk excavation level at four (4) locations across the proposed basement footprint, with two of the cored boreholes extending below the invert level of the proposed CBDRL tunnels along Sussex Street and Kent Street.
2) Installation and monitoring of water levels across the basement footprint. Minimum three temporary groundwater monitoring wells required to triangulate groundwater flow.
3) Slot inspections in the existing basement walls to determine shoring requirements.
4) Footing investigation of any adjacent buildings to determine footing type(s), founding depths and conditions.
5) Waste Classification Assessment of material proposed to be transported off site, in accordance with the appropriate guidelines.
6) Full details of the proposed CBDRL tunnels should be obtained from Sydney Trains so that their location can be plotted (plan and section) in relation to the basement excavation. A registered surveyor will be required to prepare/certify a cross section showing the tunnel positions at the closest point to the excavation.

Other works may be requested by TfNSW includes:

- A deep borehole down to the proposed invert level of the CBDRL, including permeability testing and determination of the standing water table.
- Numerical analysis using Finite Element or Finite Difference software for predictions of the effects of the proposed development on the adjacent rail infrastructure.
- Risk assessment in accordance with TfNSW framework.

9. Limitations

Douglas Partners (DP) has prepared this report for this project at 383 Kent Street, Sydney in accordance with DP's proposal dated 2 September 2022 and acceptance received from Sharan Saini of Touchstone Partners Pty Ltd. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of Charter Hall Holdings Pty Ltd for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

DP's advice is based upon the published data and DPs experience with similar developments. The accuracy of the advice provided by DP in this report may be affected by undetected variations in ground conditions across the site between and beyond the sampling and/or testing locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

The assessment of atypical safety hazards arising from this advice is restricted to the (geotechnical / environmental / groundwater) components set out in this report and based on known project conditions and stated design advice and assumptions. While some recommendations for safe controls may be provided, detailed 'safety in design' assessment is outside the current scope of this report and requires additional project data and assessment.

This report must be read in conjunction with all of the attached and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome, or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by DP. This is because this report has been written as advice and opinion rather than instructions for construction.

Douglas Partners Pty Ltd

Appendix A

About This Report

Introduction

These notes have been provided to amplify DP's report in regard to classification methods, field procedures and the comments section. Not all are necessarily relevant to all reports.

DP's reports are based on information gained from limited subsurface excavations and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Copyright

This report is the property of Douglas Partners Pty Ltd. The report may only be used for the purpose for which it was commissioned and in accordance with the Conditions of Engagement for the commission supplied at the time of proposal. Unauthorised use of this report in any form whatsoever is prohibited.

Borehole and Test Pit Logs

The borehole and test pit logs presented in this report are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable or possible to justify on economic grounds. In any case the boreholes and test pits represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes or pits, the frequency of sampling, and the possibility of other than 'straight line' variations between the test locations.

Groundwater

Where groundwater levels are measured in boreholes there are several potential problems, namely:

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole is left open;
- A localised, perched water table may lead to an erroneous indication of the true water table;
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report; and
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water measurements are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical and environmental aspects, and recommendations or suggestions for design and construction. However, DP cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

If these occur, DP will be pleased to assist with investigations or advice to resolve the matter.

About this Report

Site Anomalies

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, DP requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

Information for Contractual Purposes

Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. DP would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection

The company will always be pleased to provide engineering inspection services for geotechnical and environmental aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

Appendix B

Survey Site Plan

NOTES:

- THESE NOTES AND LEGEND (IF SHOWN) FORM PART OF THE PLAN AND SURVEY AND MUST REMAIN WITH THE PLAN IN ANY REPRODUCTION IN WHOLE OR PART.
- THE CAD FILE USES METRES AS ITS BASE UNIT AND IS IN A "GROUND" COORDINATE SYSTEM. IF THE SURVEY IS STATED AS MGA, ANY POINT IN THE FILE WILL BE AN APPROXIMATE MGA COORDINATE.
- SOME SYMBOLS REPRESENTING PHYSICAL STRUCTURES SUCH AS POWER POLES AND PITS ARE DIAGRAMMATIC ONLY AND DO NOT NECESSARILY REPRESENT THE ACTUAL SIZE AND EXTENT OF THESE FEATURES.
- THE SURVEY INFORMATION SHOWN HERE WAS PREPARED FOR A SPECIFIC PURPOSE FOR THE CLIENT SHOWN. THIS INFORMATION IS NOT INTENDED TO BE USED FOR ANY OTHER PURPOSE OR BY ANYONE NOT AUTHORISED BY THIS CLIENT.
- BOUNDARY DIMENSIONS AND AREAS HAVE BEEN DETERMINED BY CURRENT CADASTRAL SURVEY AND THE BOUNDARY AND EASEMENT LINES IN THE ELECTRONIC FILE HAVE BEEN INCLUDED USING THOSE SURVEYED DIMENSIONS. THE TITLE DIMENSIONS SHOWN ON THE HARD COPY PLAN TAKE PRECEDENCE OVER THE LINES IN THE ELECTRONIC FILE.
- THE TITLE/S TO THE SUBJECT LAND HAS BEEN REVIEWED AND THE POSITION OF ALL EASEMENTS AFFECTING THE LAND ARE SHOWN. THE TERMS OF ANY EASEMENT, RESTRICTION ON THE USE OF LAND OR COVENANT AFFECTING THE LAND HAVE NOT BEEN INVESTIGATED. LEASES AND OTHER NOTATIONS MAY ALSO EXIST WHICH AFFECT THE LAND.
- UNDERGROUND SERVICES OTHER THAN THOSE SHOWN HAVE NOT BEEN INVESTIGATED.
  PRIOR TO DEMOLITION, EXCAVATION OR CONSTRUCTION WORK ON THE SITE, THE RELEVANT SERVICE AUTHORITY SHOULD BE CONTACTED TO ESTABLISH DETAILED LOCATION AND DEPTH.
- THIS SURVEY IS LIMITED TO IMPROVEMENTS AND OTHER DETAIL WHICH WERE VISIBLE AND ACCESSIBLE AT THE TIME OF SURVEY. THE LOCATION OF DETAIL SUCH AS PRIVATE UNDERGROUND SERVICE LINES AND BUILDING FOUNDATIONS WITHIN THE SITE IS UNKNOWN.
- THE COORDINATES WITHIN THIS DRAWING RELATE TO THE DATUM SHOWN IN THE TITLE BLOCK. REFER TO A REGISTERED LAND SURVEYOR FOR FURTHER CLARIFICATION. CAUTION SHOULD BE TAKEN WHEN IMPORTING INFORMATION OBTAINED FROM OTHER SUB-CONSULTANTS OR SOURCES TO ENSURE THAT THE DATA IS ON A MATCHING COORDINATE SYSTEM.
- CONTOURS SHOWN HEREON DEPICT THE GENERAL TOPOGRAPHY ONLY.
  EXCEPT AT SPOT LEVELS SHOWN, THEY DO NOT NECESSARILY REPRESENT THE EXACT LEVEL AT ANY PARTICULAR POINT.
- CONTOUR INTERVAL 0.5m
- ANY GUTTER, RIDGE, ROOF AND WINDOW DETAILS AND LEVELS SHOWN HAVE BEEN OBTAINED VIA INDIRECT SURVEY METHODS WHERE VISIBLE FROM GROUND LEVEL AND ARE SHOWN ON THIS PLAN IN THEIR APPROXIMATE LOCATION FOR THE PURPOSE OF GENERAL SITE ANALYSIS ONLY.
- ANY TREE CANOPIES, TRUNK DIAMETERS AND HEIGHTS SHOWN ARE APPROXIMATE ONLY AND SHOULD BE VERIFIED BY FURTHER SURVEY WORKS IF CRITICAL TO DESIGN OR SITE ANALYSIS.
- SMALL TREES, SHRUBS, GARDEN FEATURES, PATHWAYS AND OTHER MINOR DETAIL MAY NOT BE SHOWN ON THIS PLAN, FOR THE PURPOSES OF THIS SURVEY.

ORIGIN OF LEVELS: PM 43307
R.L. 16.752 (AHD)

ORIGIN OF MGA 2020 CO-ORDINATES
E.333955.491
N.6250722.208

LEGEND:

FL	FLOOR LEVEL
DCK	DECK
DS	DOOR SILL
PAR	PARAPET
PP	POWER POLE
RF	ROOF
RDG	RIDGE
SGN	ROAD SIGN
SIC	SEWER INSPECTION COVER
TG	TOP OF GUTTER
TP	TOP OF PERGOLA
TK	TOP OF KERB
TOG	TOP OF GRATE
TRW	TOP OF WALL
TF	TOP OF FENCE (GLASS)
VER	VERANDAH
US	UNDERSIDE OF BUILDING
COL	COLUMN
SL	DRAINAGE PIT SURFACE LEVEL
INV	DRAINAGE PIT INVERT LEVEL
VER	VERANDAH
AWN	AWNING
GRW	DENOTES GROUND LEVEL WALL
L5W	DENOTES LEVEL 5 WALL
L6W	DENOTES LEVEL 6 WALL

Comms Pit/Manhole
Drainage Manhole
Electrical Power Pole
Gas Meter
Miscellaneous Manhole
Sewer Vent
Sewer Lamphole
Sign Post
Tree (Canopy spread, trunk diameter, Height)
Water Meter
Water Tap

NOTATIONS SHOWN ON CERTIFICATE OF TITLE
LOT 1 D.P.778342

AFFECTING THE LAND:

(A) EASEMENT FOR SUPPORT VARIABLE WIDTH BENEFITTING THE SUBJECT LAND (VIDE DP778342)

CREATED BY DEALING AA977313
SUBSTATION PREMISES No 7901 IS LOCATED AT BASEMENT 3 & 4
EASEMENTS FOR ELECTRICITY PURPOSES EXIST AT BASEMENT 1 & 4.
RIGHTS OF WAY EXIST AT BASEMENT 1, 2 & 3
EASEMENT FOR ELECTRICITY CABLE RISER EXISTS AT BASEMENT 1 & 2
SITE OF SUBSTATION PREMISES, RIGHTS OF WAY AND EASEMENTS FOR ELECTRICITY HAVE NOT BEEN SHOWN FOR CLARITY PURPOSES

RESTRICTIONS AS312062, 7246097

THE TERMS AND LOCATIONS OF COVENANTS & RESTRICTIONS HAVE NOT BEEN INVESTIGATED.

DANIEL GERARD WILLIAMS
REGISTERED SURVEYOR
BEVERIDGE WILLIAMS
447 KENT STREET, SYDNEY 2000

VER	BY	AMENDMENTS	DATE
A	V.M.	INITIAL ISSUE	30.09.22
B	V.M.	TRUE NORTH ADDED	07.10.22
C	D.W.	CO-ORDINATES SHIFTED FROM GDA94 TO MGA2020	11.10.22
D			
E			
F			
G			

- SERVICE AUTHORITY PITS, MANHOLES, POLES, MARKER POSTS, ETC., WHERE SIGHTED AT TIME OF SURVEY, HAVE BEEN LOCATED. THE SURVEY DOES NOT INCLUDE INVESTIGATION OR LOCATION OF UNDERGROUND INFRASTRUCTURE.
- PRIOR TO ANY DEMOLITION, EXCAVATION OR CONSTRUCTION ON OR ADJACENT TO THE SITE IT IS THE RESPONSIBILITY OF THE DEVELOPER AND CONTRACTORS TO APPLY FOR AND OBTAIN UP TO DATE PLANS THROUGH A NEW DIAL BEFORE YOU DIG SEARCH AND TO CONTACT ALL THE RELEVANT AUTHORITIES TO ESTABLISH AND CONFIRM THE DETAILED LOCATION AND DEPTH OF ALL UNDERGROUND SERVICES.

www.dialbeforeyoudig.com.au DIAL 1100 BEFORE YOU DIG

CLIENT:
Charter Hall Holdings Pty Ltd

Beveridge Williams
Land Development Consultants
Registered Surveyors
Sydney (02) 9283 6677
www.beveridgewilliams.com.au

DETAILS:
DETAIL SURVEY PLAN FOR
DEVELOPMENT APPLICATION PURPOSES
LOT 1 DP 778342
383 KENT STREET SYDNEY

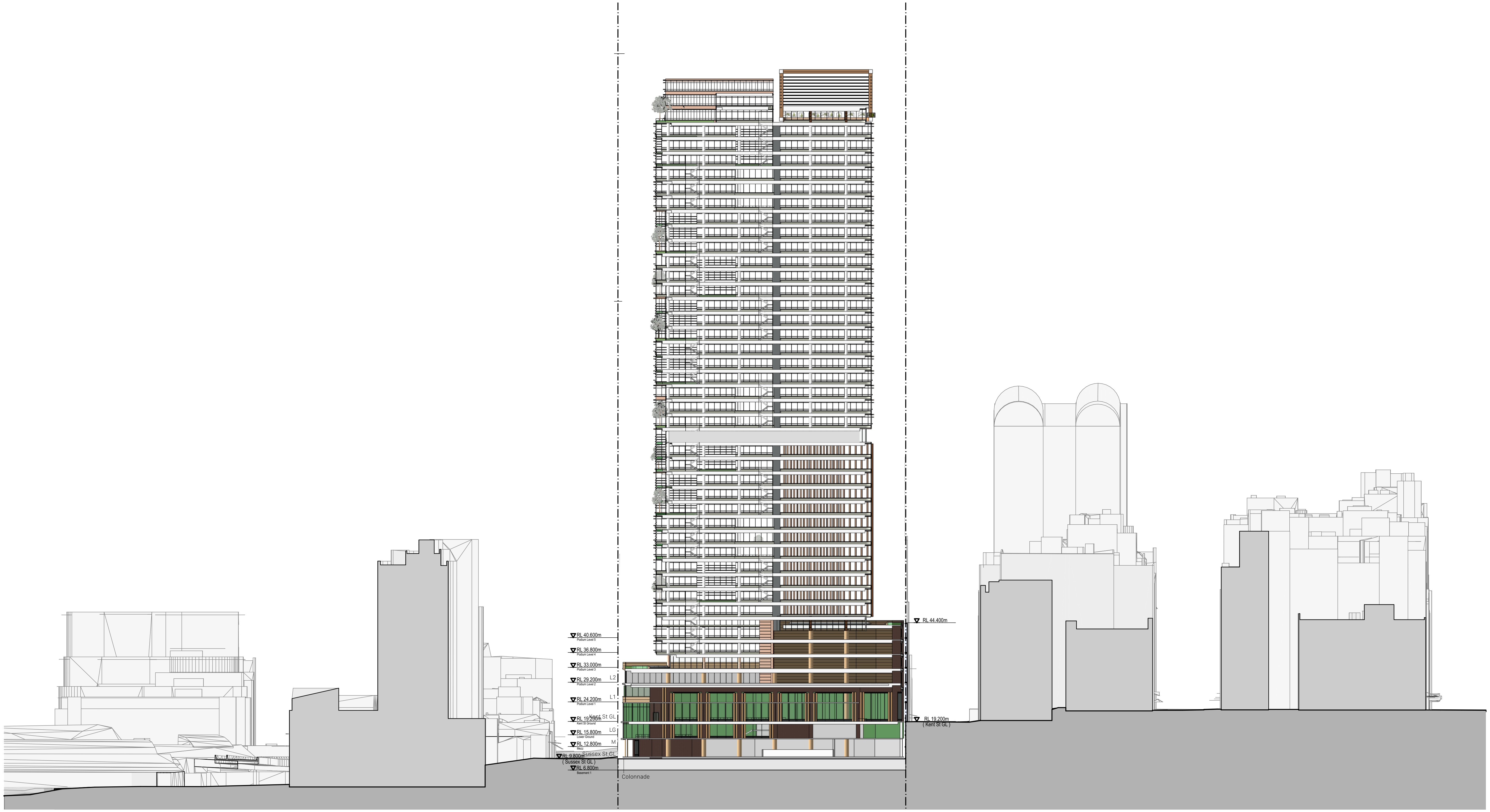
ORIGINAL
SCALE 1:300 SHEET SIZE A1
CAD REFERENCE: 2201978_DET3D_221011
0 6 12 18
SCALE ON ORIGINAL DRAWING AT 1:300

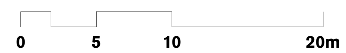
SURVEYOR:	V.M.
DRAWN:	J.T.
CHECKED:	V.M.
SURVEY DATE:	07.09.2022
VERTICAL DATUM:	AHD
HORIZONTAL DATUM:	MGA 2020

PROJECT No.
2201978
DRAWING REF.
DETAIL 3D
VERSION A
SHEET 1 OF 7

Appendix C

Proposed Development Drawings





fjmt

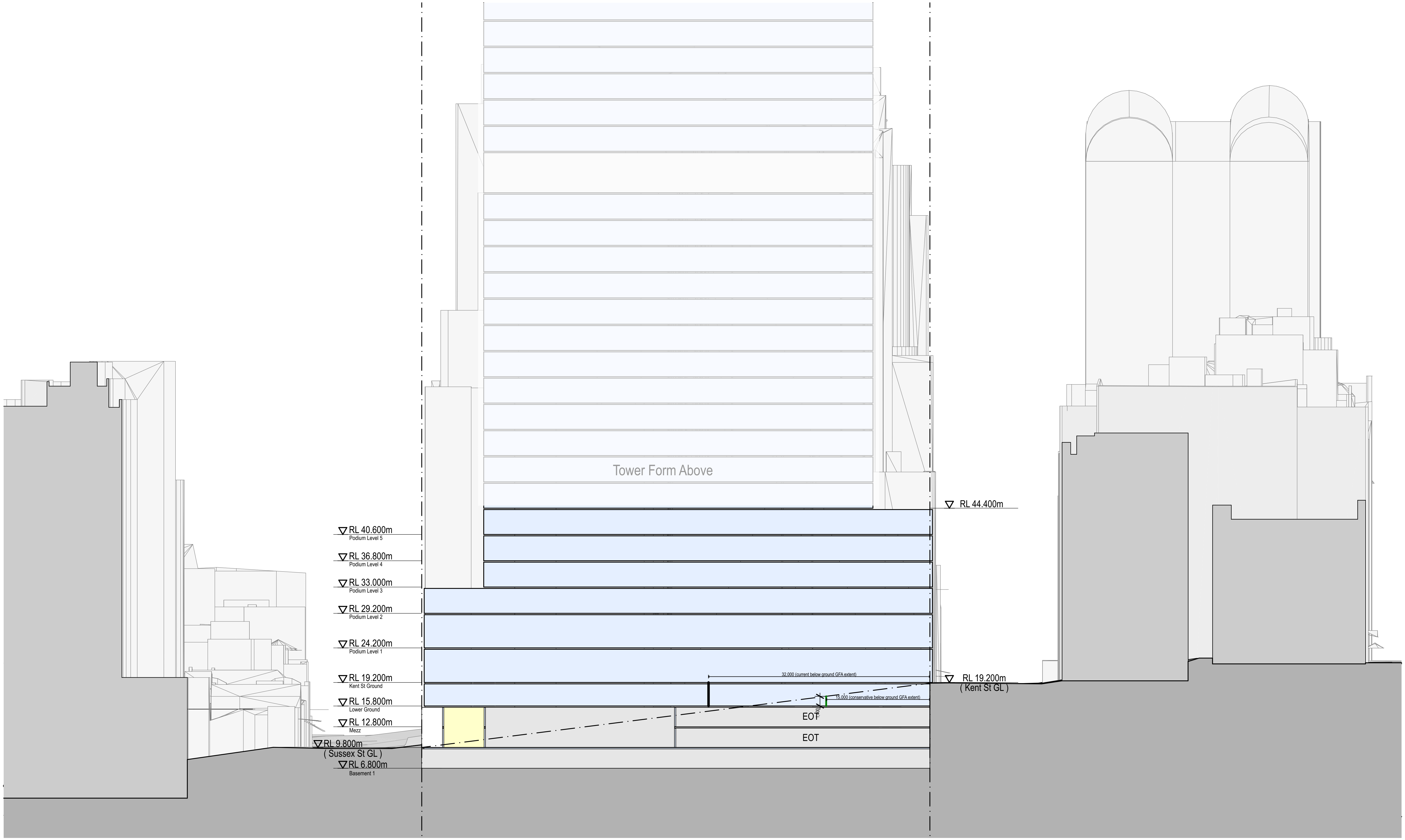
West-East Section

1:500 @ A3

4/5/23

Charter Hall - 383 Kent Street

For Information



fjmt

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West-East Podium Section

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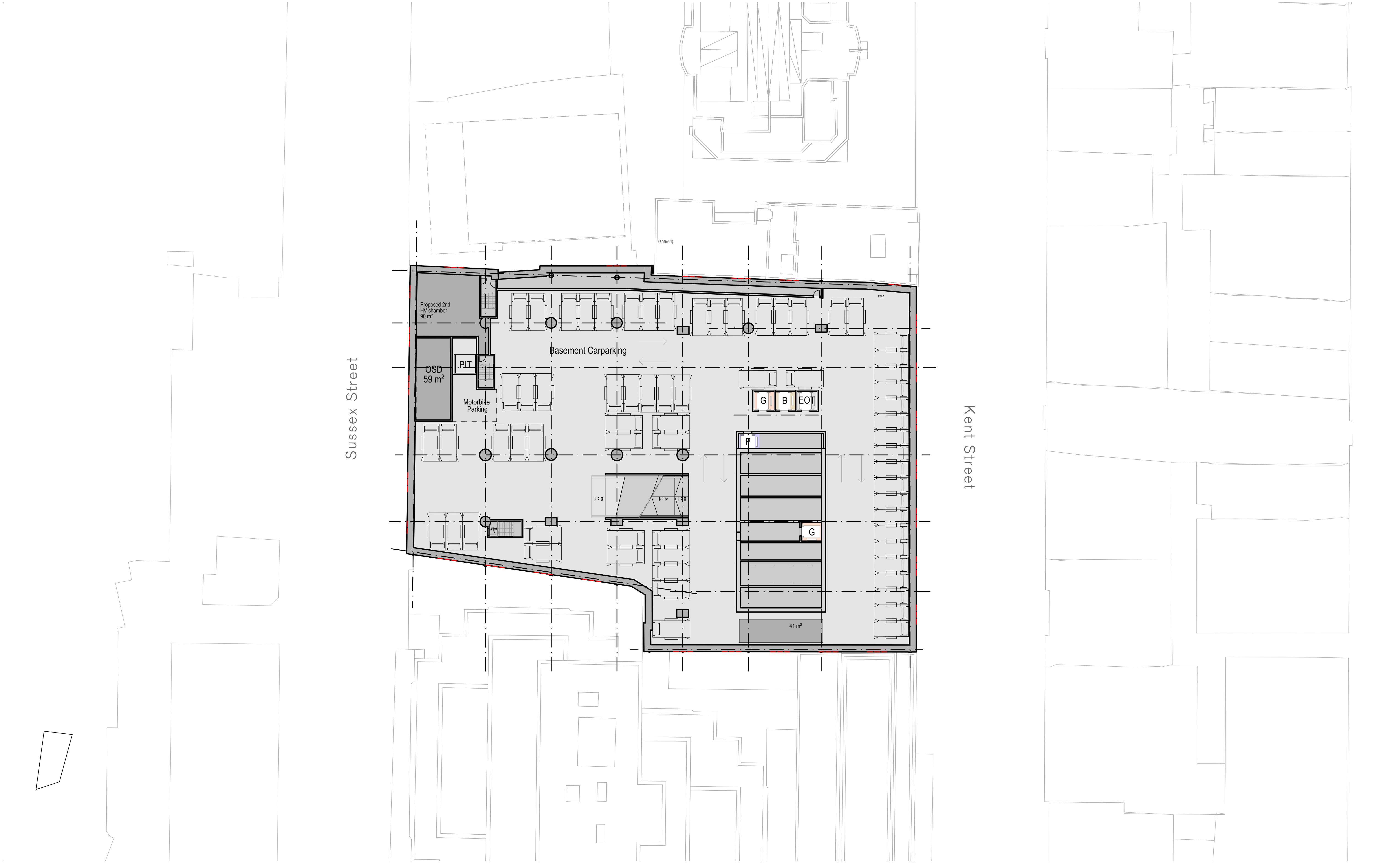
4/5/23

Charter Hall - 383 Kent Street

For Information



fjmt

Basement 1 - Carparking

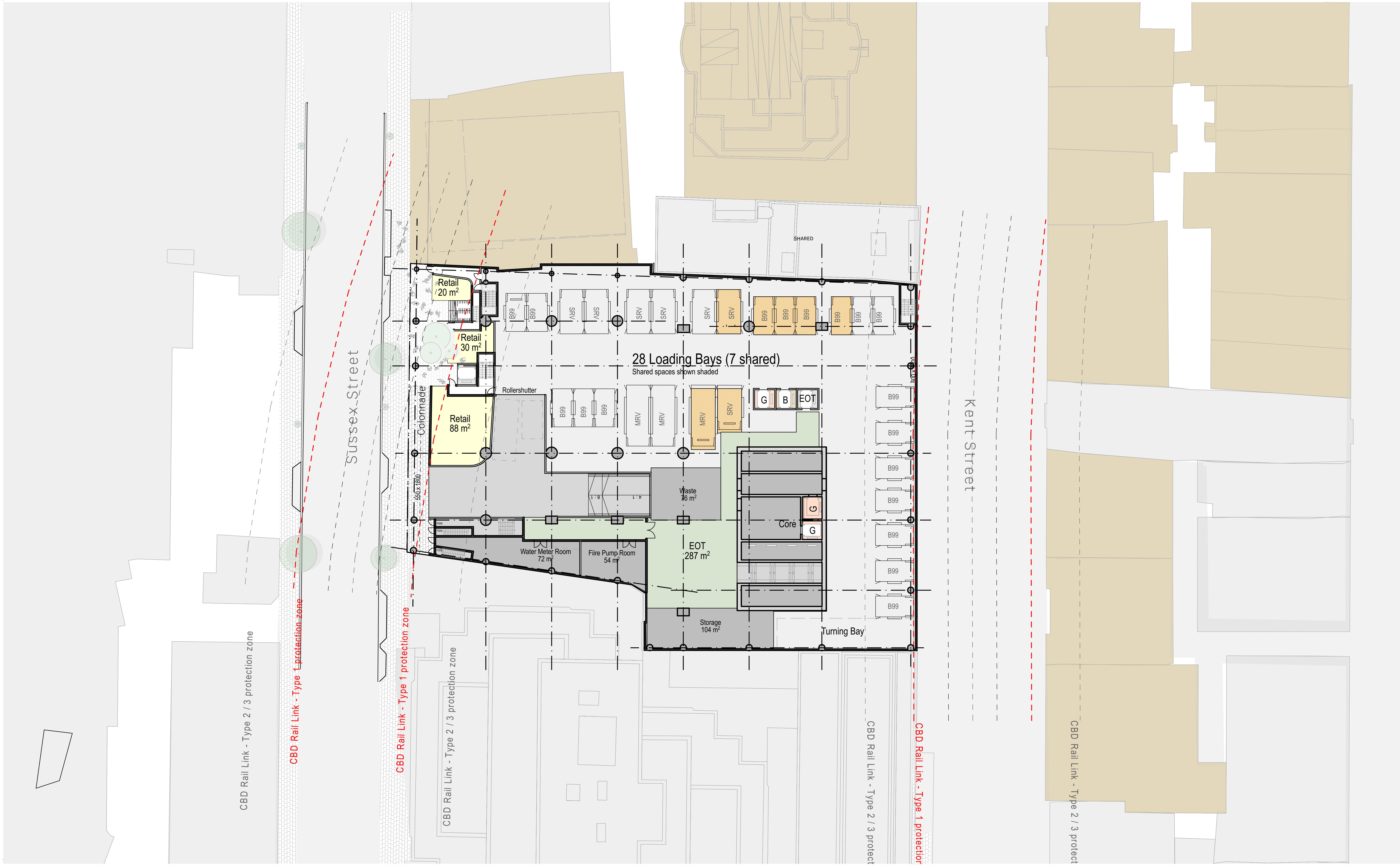
Charter Hall - 383 Kent Street

0 5 10 20m

1:500 @ A3

<16/02/2023>

Issued for Approval



fjmt

Sussex St GL - Shared Loading/ retail

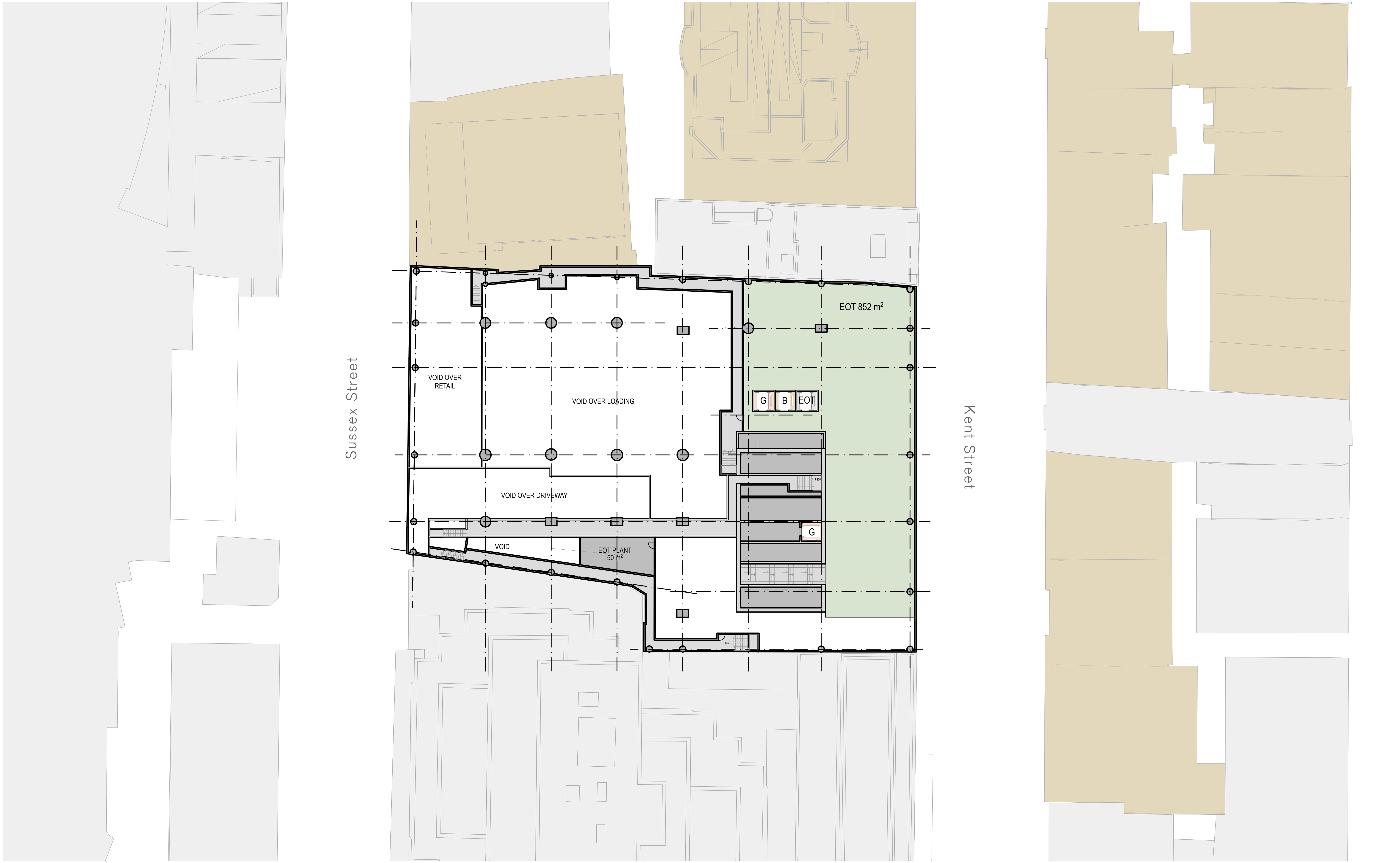
Charter Hall - 383 Kent Street

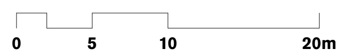
0 5 10 20m

1:500 @ A3

1/12/23

Issued for Approval





fjmt

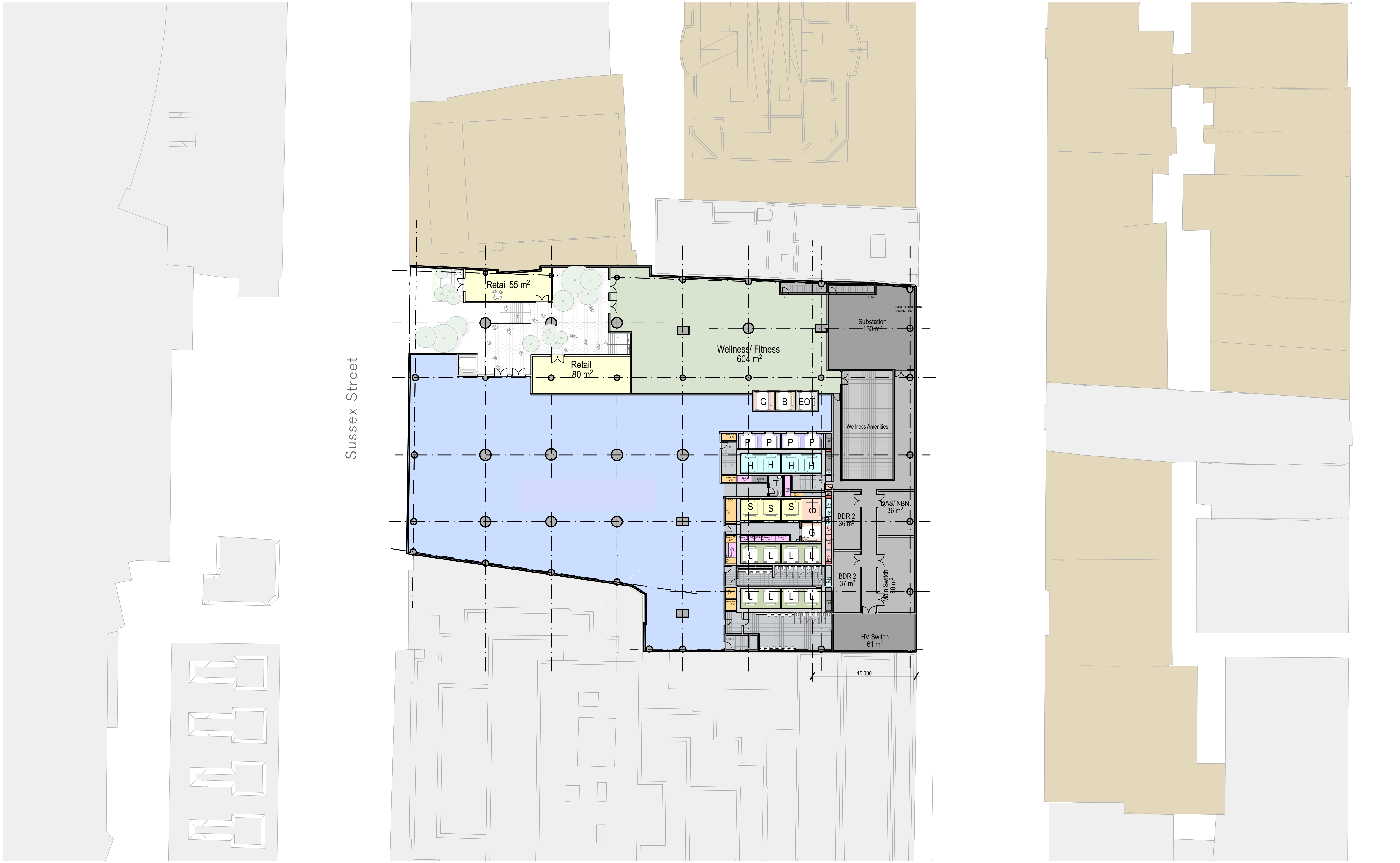
Sussex St Mezzanine - EOT

1:500 @ A3

1/12/23

Charter Hall - 383 Kent Street

Issued for Approval



fjmt

0 5 10 20m

Lower GL - Commercial/ EOT/ Plant

1:500 @ A3

1/12/23

Charter Hall - 383 Kent Street

Issued for Approval

fjmt

Kent St GL - Lobby/ Retail/ Commercial

1:500 @ A3

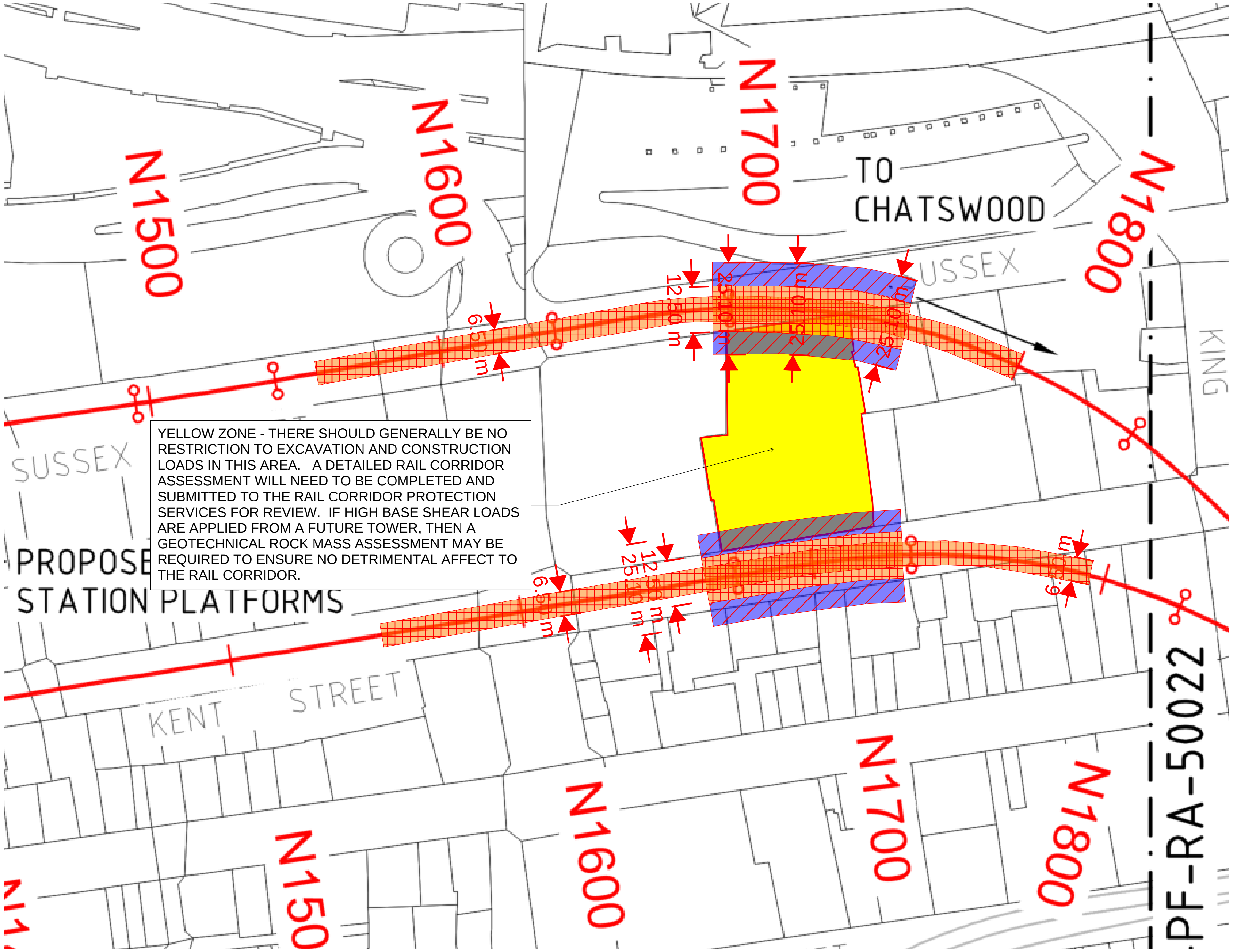
1/12/23

Charter Hall - 383 Kent Street

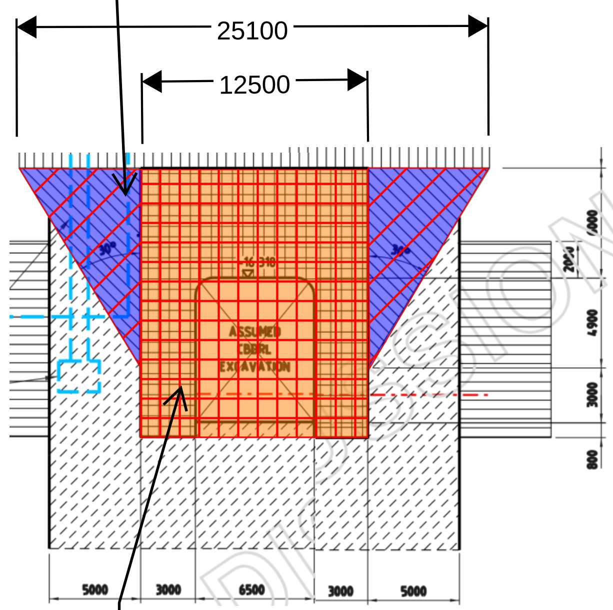
Issued for Approval

Appendix D

CBD Rail Link Drawings



TYPE 2 ZONE - REFER TO DETRMINATION DRAWING. EXCAVATION IS POSSIBLE, BUT YOU CANNOT APPLY SHALLOW FOOTING PRESSURE TO TOP OF THE WEDGE.



TYPE 1 ZONE - NO EXCAVATION OR PILING

RAIL CORRIDOR ASSESSMENT

THIS IS A HIGH LEVEL REVIEW ONLY USING INFORMATION PROVIDED AND ATTACHED TO THIS DOCUMENT. THE DOCUMENTS RECIEVED ARE ASSUMED TO SCALE. ALL ANNOTATIONS HAVE BEEN DONE USING APPROXIMATE SCALES PROVIDED AND SHOULD ONLY BE USED FOR HIGH LEVEL OBSERVATIONS ONLY. IN ORDER TO COMPLETE A DETAILED ASSESSMENT, SURVEYED AND SCALED CAD INFORMATION IS REQUIRED OF THE SITE AND ALIGNMENT

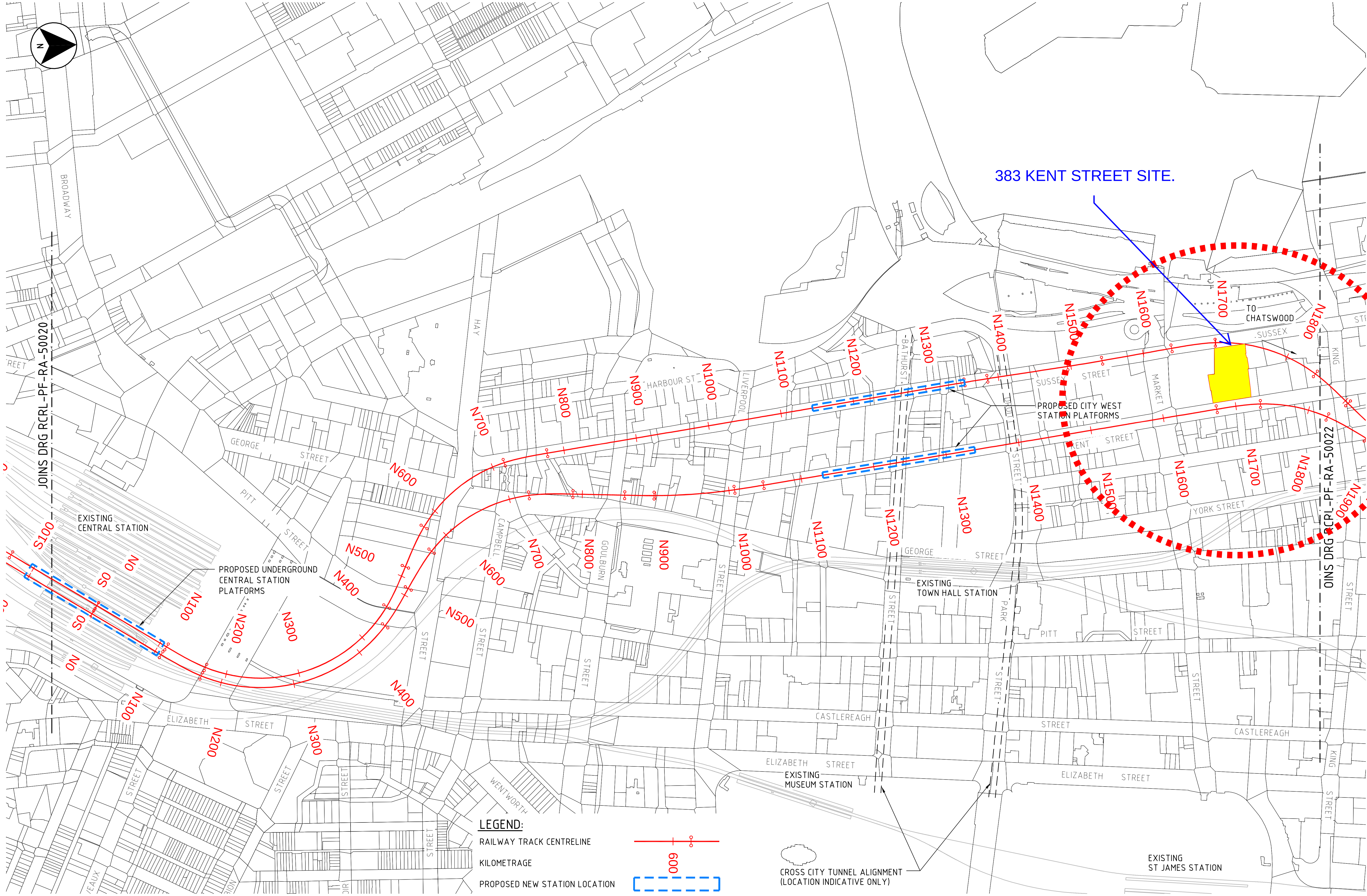
Structural, Civil & Construction Engineering Consultant
RobertBirdGroup
Member of the Surbana Jurong Group
SYDNEY OFFICE
Robert Bird Group Pty Ltd
PO Box A2309
Sydney South, NSW 1235
Level 11, 151 Castlereagh St
Sydney NSW 2000
Ph: (02) 8246 3200
Fax: (02) 8246 3201
Email: sydney@robertbird.com.au
Web: www.robertbird.com
ACN 010 580 248

Rev.	Revision Description	App	Date
A	FOR INFORMATION	MH	28.8.20

Client:	CHARTER HALL
Project:	383 KENT STREET
Title:	RAIL CORRIDOR ASSESSMENT
Designer:	
Checker:	
Approved:	MH
Status:	PRELIMINARY - NOT FOR CONSTRUCTION
Sketch No:	SK-001
Rev:	1

Cad ref: K:\20012506_00_MRE\CAD\Dwg\03_RCRL\RCRL-PF-RA-50021.dwg   Last modified: 06 Jun 06 - 08:38

CABINET IN CONFIDENCE



NOT FOR CONSTRUCTION

No.	BY	DATE	DESCRIPTION	APPD
C	BK	6.06.06	REFERENCE SCHEME - THIRD DRAFT	
B	BK	3.04.06	REFERENCE SCHEME - SECOND DRAFT	
A	BK	29.03.06	REFERENCE SCHEME - FIRST DRAFT	

THE SIGNING OF THIS TITLE BLOCK CONFIRMS THE DESIGN AND DRAFTING OF THIS PROJECT HAVE BEEN PREPARED AND CHECKED IN ACCORDANCE WITH THE MAUNSELL QUALITY ASSURANCE SYSTEM TO ISO 9001-2000

DESIGNED		CHECKED	
DRAWN		CHECKED	
APPROVED		DATE	

SCALES: 25 0 50 100m 1 : 5000

DESIGNER: MAUNSELL | AECOM
Maunsell Australia Pty Ltd A.B.N. 20 093 846 925

DESIGNER: Halcrow

CLIENT: Transport Infrastructure Development Corporation

CBD RAIL LINK

METROWEST UNDER HARBOUR OPTION PLAN

SHEET 2 OF 7

STATUS: PRELIMINARY

DRAWING No: RCRL-PF-RA-50021

REV: C

Cad ref: K:\20012506_00_MRE\CAD\Dwg\03_RCRL\RCRL-PF-RA-50033.dwg Last modified: 06 Jun 06 - 08:45

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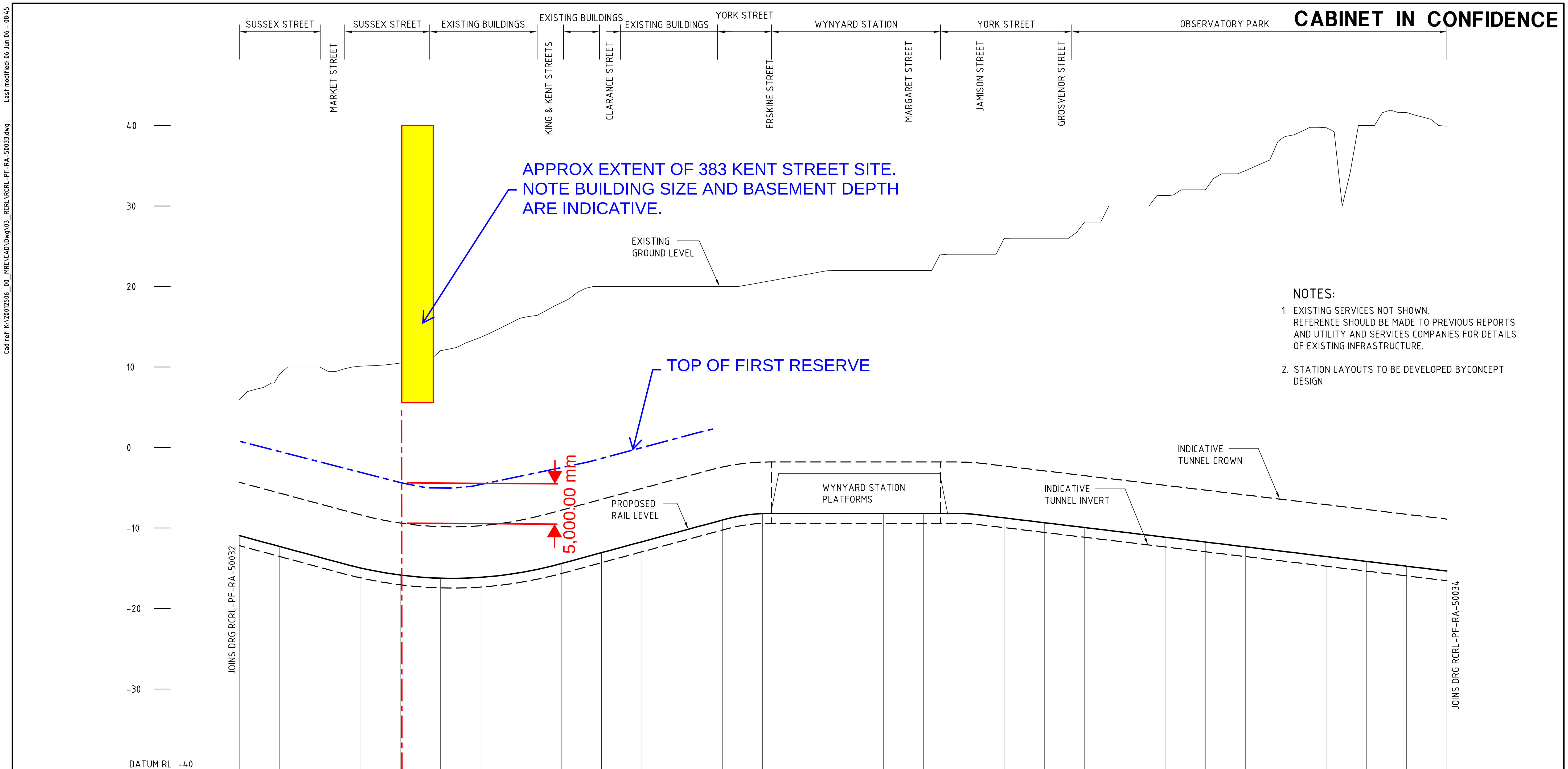


NOTES:

1. EXISTING SERVICES NOT SHOWN. REFERENCE SHOULD BE MADE TO PREVIOUS REPORTS AND UTILITY AND SERVICES COMPANIES FOR DETAILS OF EXISTING INFRASTRUCTURE.
2. STATION LAYOUTS TO BE DEVELOPED BYCONCEPT DESIGN.

VERTICAL ALIGNMENT: P=-2.700% L=212.259 | R=5000.000 L=270.000 | P=2.700% L=203.741 | R | P=-0.000% L=236.000 | R | P=-1.200% L=667.667

HORIZONTAL ALIGNMENT: R | D=96.154 | CL | R=215.000 L=145.780 | CL=60.000 | CL | R=-200.000 L=128.344 | CL=60.000 | D=643.197 | CL | R=415.400 L=233.852

KILOMETRAGE	PROPOSED RAIL LEVELS	EXISTING LEVELS
N1500	-10.940	5.909
N1543.139 N1550	-12.290	9.141
N1600	-13.640	10.000
N1639.293 N1650	-14.947	10.096
N1699.293 N1700	-15.840	10.503
N1750	-16.232	12.041
N1800	-16.125	13.710
N1845.073 N1850	-15.517	16.087
N1900 N1905.073	-14.410	17.990
N1950 N1965.073	-13.060	20.000
N2000	-11.710	20.000
N2050	-10.360	20.000
N2093.417 N2100	-9.010	20.000
N2150 N2153.417	-8.212	20.534
N2200	-8.200	21.425
N2250	-8.200	22.000
N2300	-8.200	22.000
N2350	-8.200	22.000
N2400	-8.212	24.000
N2450	-8.740	25.979
N2500	-9.340	26.000
N2550	-9.940	28.000
N2600	-10.540	30.000
N2650	-11.140	31.301
N2700	-11.740	32.000
N2750	-12.340	34.410
N2796.614 N2800	-12.940	38.653
N2850 N2856.614	-13.540	39.725
N2900	-14.140	40.000
N2950	-14.740	41.628
N3000	-15.340	39.917

CROSS SECTION CONCEPT (UP AND DOWN TRACKS): TWIN ROADHEADER TUNNELS REFER DRAWING RCRL-PF-CI-50101 AND 50102 | WYNYARD STATION | TWIN ROADHEADER TUNNELS REFER DRAWING RCRL-PF-CI-50101 AND 50102

LONGITUDINAL SECTION - DOWN TRACK ALIGNMENT

NOT FOR CONSTRUCTION

No.	BY	DATE	DESCRIPTION	APPD
B	BK	6.06.06	REFERENCE SCHEME - SECOND DRAFT	
A	BK	3.04.06	REFERENCE SCHEME - FIRST DRAFT	

THE SIGNING OF THIS TITLE BLOCK CONFIRMS THE DESIGN AND DRAFTING OF THIS PROJECT HAVE BEEN PREPARED AND CHECKED IN ACCORDANCE WITH THE MAUNSELL QUALITY ASSURANCE SYSTEM TO ISO 9001-2000

DESIGNED | CHECKED
DRAWN | CHECKED
APPROVED | DATE

SCALES: 2.5 0 5 10m VERTICAL 1:500; 25 0 50 100m HORIZONTAL 1:5000

DESIGNER: MAUNSELL | AECOM
Maunsell Australia Pty Ltd A.B.N. 20 093 846 925

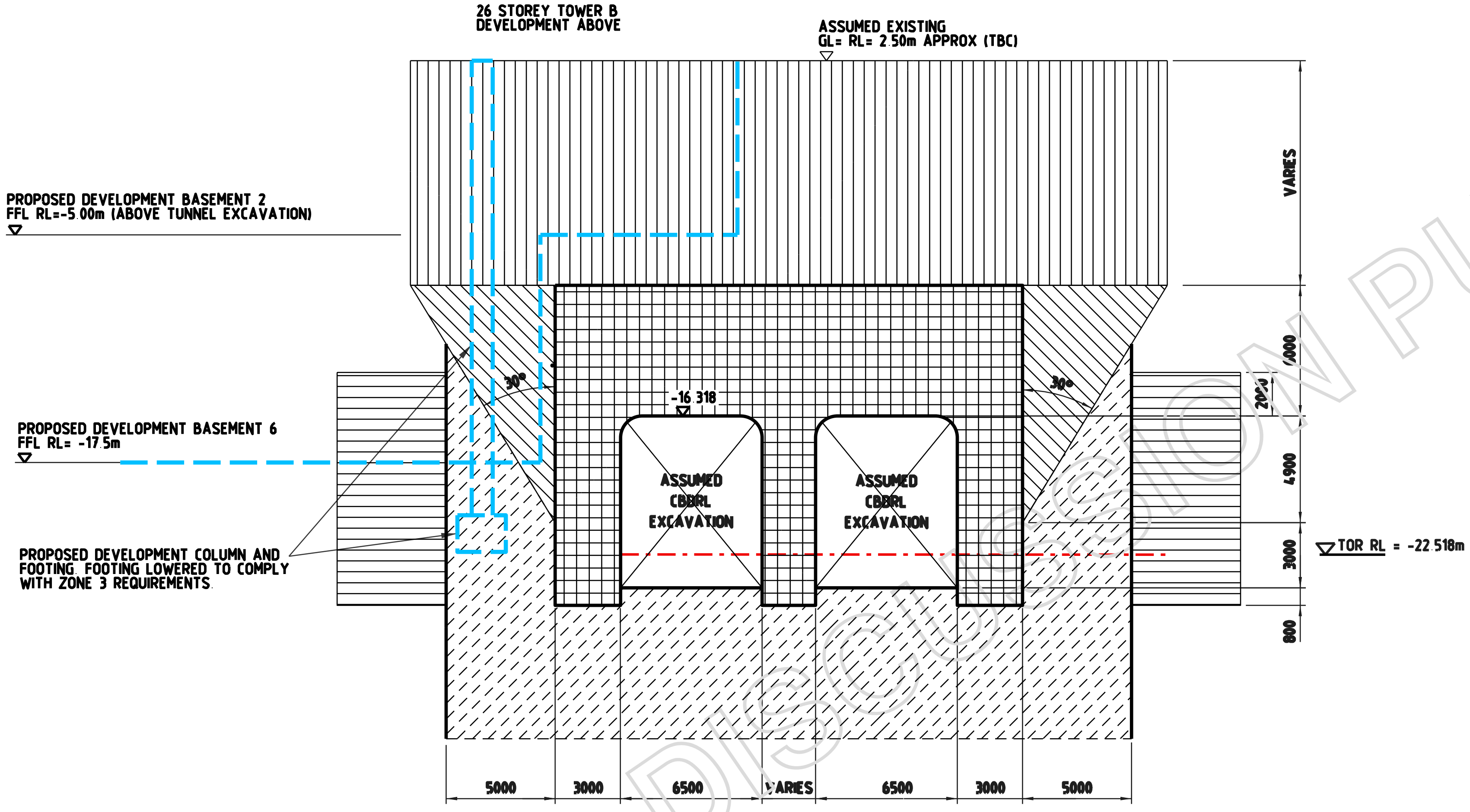
DESIGNER: Halcrow

CLIENT: Transport Infrastructure Development Corporation

CBD RAIL LINK
METROWEST UNDER HARBOUR OPTION
LONGITUDINAL SECTION
DOWN TRACK ALIGNMENT

SHEET 4 OF 8

STATUS: PRELIMINARY | DRAWING NO: RCRL-PF-RA-50033 | REV: B



SECTION A-A

KEY:

STRUCTURE EXCLUSION ZONES:

ASSUMED CBDRL TUNNEL EXCAVATION ZONE

TYPE 1 ZONE — EXCAVATION, STRUCTURES, OR STRUCTURES APPLYING LOADING DIRECTLY TO ROCK MASS ARE NOT PERMITTED IN THIS ZONE. HORIZONTAL LOADING EFFECTS EITHER FROM STRUCTURES AND FOUNDATIONS BEARING DIRECTLY ON ZONE 1 OR FOUNDATIONS OUTSIDE ZONE 1 AND ACTING THROUGH THE GROUND SHALL NOT CAUSE SIGNIFICANT HORIZONTAL LOADING TO ACT ON ZONE 1 BELOW OR WITHIN 2m OF THE TUNNEL CROWN LEVEL.

TYPE 2 ZONE — FOUNDATIONS APPLYING VERTICAL LOADING DIRECTLY TO ROCK MASS ARE NOT PERMITTED IN THIS ZONE EXCEPT FOR TRANSFER STRUCTURES AND GROUND ANCHORS APPLYING LOADS GENERALLY AWAY FROM THE TUNNEL. EXCAVATION AND CONSTRUCTION OF STRUCTURES PERMITTED BUT MUST AVOID DETERIORATION OF ROCK STRUCTURE IN ADJACENT ROCK MASS.

TYPE 3 ZONE — VERTICAL DOWNWARDS LOADS FROM STRUCTURES PERMITTED. VERTICAL UPWARDS LOADS (AND COMPONENTS THEREOF) TO BE DESIGNED TO ALLOW FOR REMOVAL OF ROCK MASS IN ASSUMED TUNNEL ZONE. HORIZONTAL LOADS PERMITTED BELOW LEVEL OF TUNNEL INVERT. LOADING TO BE VERIFIED AND SHALL ALLOW FOR ASSUMED TUNNEL EXCAVATION.

TYPE 4 ZONE — SUM OF VERTICAL DOWNWARDS LOADING TO BE LIMITED TO 3000kN/m² ON 2.0mX2.0m AREAS AND AN AVERAGE OF 250 kN/m² ON ANY 10mX10m AREA (WHICH EVER IS THE MOST ONEROUS CONDITION). FOUNDATIONS AND SUPPORTING STRUCTURE TO BE DESIGNED TO ACCOMMODATE MOVEMENTS DUE TO FUTURE CBDRL WORKS. NOTE 10 APPLIES.

SIGNIFICANT HORIZONTAL LOADING TOWARDS THE PROPOSED CBDRL TUNNEL IS NOT PERMITTED. (REFER TO ZONE 1 NOTES)

GENERAL NOTES:

G1. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
G2. ALL LEVELS ARE IN METRES TO A.H.D UNLESS OTHERWISE NOTED.
G3. DRAWINGS TTSRCP030, 031 & 032 TO BE READ TOGETHER.
G4. FOR DETAILS OF ASSUMED RAIL TUNNEL EXCAVATION REFER TO DRAWINGS TTSRCP030 & 031.

NOTES:

1. INDICATIVE LOADS ARE UNFACTORED WORKING LOADS. (TYPE 4 ZONE)
2. SUPPORT OF LOADS FROM PROPERTY FOUNDATIONS TO ALLOW FOR ASSUMED TUNNEL EXCAVATION. LOADS OF A DYNAMIC NATURE (SUCH AS FROM WIND) ARE NOT TO HAVE A NEGATIVE IMPACT ON THE TUNNEL SUPPORT INCLUDING SANDSTONE BEDDING PLANES AND ROCK BOLT DESIGN.
3. LOADING REQUIREMENTS FOR ZONE 4 ARE INDICATIVE ONLY AND ARE TO BE DETERMINED.
4. EXCAVATION FACE TO BE INSPECTED AND MAPPED PRIOR TO COVERING BY PROPERTY DEVELOPER.
5. SITE INVESTIGATION OF GROUND CONDITIONS SHALL BE UNDERTAKEN AND RECORDED.
6. REQUIREMENTS ARE BASED ON DEVELOPMENT APPLICATION No ????.
7. LOADING REQUIREMENTS ARE BASED UPON ASSUMPTION THAT ZONE 1 IS FAVOURABLE ROCK CONDITIONS WHICH IS DEFINED AS CLASS 1 OR 2 SYDNEY SANDSTONE.
8. CONSTRUCTION METHOD SHALL AVOID DETERIORATION OF ROCK STRUCTURE IN ZONE 1.
9. ALLOWANCE SHALL BE MADE IN THE ASSUMED TUNNEL EXCAVATION FOR POSSIBLE OVERBREAK DURING CONSTRUCTION OF THE TUNNEL. THE PROPERTY DEVELOPER SHALL MAKE IT'S OWN DETERMINATION IN THIS REGARD BUT SHALL ALLOW FOR POSSIBLE OVERBREAK OF BLOCKS AT LEAST 1m DEEP IN THE TUNNEL CROWN OR 1m WIDE IN THE TUNNEL SIDE WALL.
10. FOUNDATION LOADS IN ZONE 4 ARE ONLY PERMITTED IF GOOD QUALITY ROCK EXISTS BETWEEN FOUNDATIONS AND THE ASSUMED TUNNEL AND IT IS DEMONSTRATED THAT A NEGLIGBLE OR LOWER RISK EXISTS OF PERSISTENT SUB-VERTICAL JOINTING BEING PRESENT WHICH WOULD POTENTIALLY CAUSE INSTABILITY OF THE TUNNEL EXTRADOS.
11. MINIMUM DIMENSION OF ROCK BETWEEN ASSUMED STATION EXCAVATION AND PROPERTY BASEMENT TO BE DETERMINED BY ASSESSMENT OF HYDROGEOLOGY AND ANY MITIGATION MEASURES PROPOSED BY THE DEVELOPER TO ENSURE WATER INGRESS INTO THE STATION CAVERN CAN BE CONTROLLED WITHIN REASONABLE LIMITS.

NOT FOR CONSTRUCTION

Issue	Revision - Revise on CAD do not amend by hand	Chk'd	App'd	Date
8	FOR TfNSW DEED ISSUE	IM		19.10.16
7	FOR DISCUSSION PURPOSES	RH		02.06.15
6	DRAFT	RH		12.05.11

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Project: FUTURE RAIL CORRIDOR PROTECTION

NSW GOVERNMENT Transport for NSW

Title: 1 ALFRED STREET SYDNEY
CBD RAIL LINK (CBDRL)
LOADING REQUIREMENTS

Drg No: TTSRCP-032

Version: 8