338 Pitt Street, Sydney

Geotechnical Desktop Report

PSM3102-005R 22 January 2020

Table of Contents

List of Tables

List of Figures

List of Photos

Preamble

This report presents the geotechnical desktop study for the Site at 338 Pitt Street, Sydney to support a Stage Significant Development Application (SSDA) for the mixed use redevelopment, which is submitted to the City of Sydney pursuant to Part 4 of the Environmental Planning and Assessment Act 1979 (EP&A Act). China Centre Development Pty Ltd is the proponent of the SSDA.

The Site is located at the corner of Pitt Street and Liverpool Street, within the 'Mid Town' precinct of Sydney's Central Business District (CBD). The site is approximately 150m west of Museum Station and Hyde Park, and approximately 350m east from Town Hall Station. The site includes several allotments and constitutes nearly one third of the city block between Bathurst Street, Pitt Street and Liverpool Street. The site is an irregular shape and has a combined area of approximately 5,900m².

The proposed development comprises hotels, residential, commercial and retail space. The development will include the following:

- demolition of all existing structures
- excavation and site preparation, including any required remediation
- construction and use of a mixed-use development, with an iconic 258m two-tower built form above a podium and internal courtyard
- four (4) basement levels and a lower ground level accommodating residential, retail and hotel car parking, motorcycle parking, bicycle parking, loading dock, storage and relevant building services
- improvements to the public domain, including landscaping, pedestrian thoroughfares/connections, and landscaping; and
- augmentation and extension of utilities and services.

A detailed description of development is provided by Ethos Urban within the EIS.

The Site plan is shown below in Inset 1.

Inset 1: Aerial photo of the site

1. Introduction

This report presents the results of a geotechnical desktop study completed by PSM for the proposed development at 338 Pitt Street, Sydney (the Site). This work has been undertaken in accordance with our proposal PSM3102-004L dated 13 August 2019.

The purpose of this study is to consider all the currently available geotechnical information to inform the preliminary design of the proposed basement excavation at the Site. Specifically, the following scope has been completed and reported:

1. Collation of available geotechnical information relevant to the Site.
2. Development of a preliminary geotechnical model based on the available information.
3. Identifying information gaps, project risks, and provision of recommendations on further geotechnical investigations and testing.
4. Preliminary geotechnical advice.

Based on recent communications and drawings from Touchstone Partners and FJMT Studio (FJMT), we understand the following about the development:

- The proposed development is situated at 338 Pitt Street, Sydney and comprises mixed-use high-rise development.
- Demolition of existing low-rise buildings and basements within the site boundary will be required.
- The current drawings from FJMT show four basement levels extending down approximately 20 m below Pitt Street to approximately RL 0 m (Ref: H338-2000 to 2003 "General Arrangement Plans Basement 1 to 4", dated 2 December 2019).
- Column loads are up to 60 MN (Ref: 338 Pitt St, Sydney Geotechnical Site Investigation & Design Brief, dated 25 July 2019).
- Uplift loads are up to 10 MN (Ref: 338 Pitt St, Sydney Geotechnical Site Investigation & Design Brief, dated 25 July 2019).

PSM has previously issued the following documents with regards to this proposed development:

- Initial geotechnical advice – Hans 338 development (Ref: PSM3102-002L dated 3 August 2016)
- Geometrical desktop report (Ref: PSM3102-006R dated 28 October 2019).

2. Adjacent Buildings and Infrastructure

2.1 Adjacent Buildings

The Site is currently occupied by the following:

- One (1) high-rise building at 324-330 Pitt Street with four (4) levels of basement below ground;
- One (1) low-rise building at 332-336 Pitt Street with one (1) level of basement below ground;
- One (1) high-rise building at 338 Pitt Street with two (2) levels of basements below ground;
- Three (3) low-rise buildings at 126-130 Liverpool Street;
- One (1) high-rise building at 233 Castlereagh Street with four (4) levels of basement below ground;
- Three (3) low-rise buildings at 241-249 Castlereagh Street;
- One (1) low rise building at 219-223 Castlereagh Street;

The 2016 LTS survey of the draft basement (Ref. LTS Surveyor, Plan of Detail and Levels Over Lot 3 In DP1044304 at 233 Castlereagh St Rev C, Sydney, dated 4 July 2016) indicates that:

- The surface level at Castlereagh Street is approximately at RL 22.7 m AHD;
- The surface level at Pitt Street is approximately at RL 19.5 m AHD.

The following existing buildings are adjacent to the Site:

- South East boundary: 255 Castlereagh Street building with one (1) basement level to approximately RL 17.5 m AHD (basement and footing plan to be confirmed).
- North boundary: 310 – 322 Pitt Street, 30-storey building with one (1) basement level to RL 16.87 m AHD.
- North boundary: 225 - 227 Castlereagh Street, 8-storey building with one (1) basement level to RL 17.5 m AHD.

The building at 225 - 227 Castlereagh Street is a heritage listed building as listed by the NSW government – Office of Environment & Heritage.

Other buildings in the vicinity of the Site with significant basements include:

- Beyond the southwest corner: World Square building at 644 George St with basement levels extending to approximately RL -6 m AHD.
- Beyond the northeast boundary: 255-269 Elizabeth Street with up to three (3) basement levels to RL 14.51 m AHD.
- Beyond the northwest corner: 343-357 Pitt Street with six (6) basement levels to RL 3.37 m AHD.

The geometrical desktop report (Ref: PSM3102-006R dated 28 October 2019) has outlined the known existing basement geometries.

2.2 Adjacent Infrastructure

The following subsurface infrastructure are known to be adjacent to the Site:

1. Sydney Metro City & South West: The project is still under construction with 2 single track running tunnels traversing under Pitt Street and Castlereagh Street. There is also a cross passage tunnel connecting the 2 running tunnels under Liverpool Street. These tunnels are closely adjacent to the proposed basement excavation;
2. Telstra Tunnel: the tunnel running along Pitt Street and Liverpool Street;
3. Existing City Circle Rail tunnels (4 single track tunnels) to the South West and East of the Site;
4. Underground services (locations unknown at this time);
5. CBD Rail Link (CBDRL) protection corridor (i.e. not yet physical infrastructure). We do not currently have any information on this;
6. Roads.

Our geometrical desktop study report (Ref: PSM3102-006R dated 28 October 2019) has outlined the existing adjacent infrastructure in plan and cross section.

2.3 Interaction with Proposed Development

We note the following adjacent infrastructure and buildings would have the most significant interaction with the shoring and excavation design.

- Both mainline tunnels and a cross passage of Sydney Metro City & South West;
- Telstra Tunnel along Pitt Street and its chamber at 320 Pitt Street with unknown geometry;
- Building at 310 – 322 Pitt Street;
- Building at 225 – 227 Castlereagh Street;
- Building at 255 - 269 Castlereagh Street;
- Existing Roads including Pitt Street, Castlereagh Street and Liverpool Street (Council / RMS has particular requirements relating to ground anchors beneath roads).

We discuss this interaction further in our geometrical desktop study report (Ref: PSM3102-006R dated 28 October 2019).

3. Preliminary Geotechnical Model

3.1 Available Geotechnical Information

This geotechnical desktop study was based on published data and PSM's database. No site investigation has been undertaken yet for this project. These available data include the following which are relevant to the Site:

- Geotechnical information (including mapping, borehole logs and excavation photographs) for the following sites:
  - Boreholes from the Museum Station project;
  - Face mappings of adjacent building basements from the PSM database;
  - Geotechnical data from other projects from the PSM database.

Figure 1 presents a locality plan of the Site and available geotechnical information in the vicinity of the Site that has been considered in development of the preliminary geotechnical model.

3.2 Geological Setting

The 1:100,000 Sydney Geological Map (Sheet 9130, Ed 1 1983) indicates the Site is underlain by a relatively thin layer of Ashfield Shale (*Rwa*), overlying Hawkesbury Sandstone (*Rh*). The actual presence of Shale at the Site is currently unknown.

3.2.1 Ashfield Shale

The Ashfield Shale unit is up to 60 m thick comprising four siltstone and laminate sub-groups. The lower part of the Ashfield Shale is likely to have been deposited in a freshwater lake paleoenvironment on the Hawkesbury Sandstone / Mittagong Formation alluvial plain. The upper part is likely to have been formed in a shallow marine paleo-environment.

Ashfield Shale is of lower abrasiveness than Hawkesbury Sandstone. It typically contains 20% to 40% quartz and 40% to 60% clay minerals, which are predominantly kaolinite and illite.

3.2.2 Hawkesbury Sandstone

Hawkesbury Sandstone is described as a medium to coarse grained, quartzose sandstone deposited in generally 1 m to 3 m thick layers. The top and bottom of these layers form primary bedding planes. Layers of shale breccia or clasts comprising fragments of siltstone between 50 mm and 4000 mm wide occur within the Hawkesbury Sandstone. They commonly occur as layers and often accumulate along primary bedding planes.

Sandstone between the primary beds is described as either massive or cross bedded, the latter being referred to as 'sheet facies'. Sheet facies make up approximately 70% of the Hawkesbury Sandstone.

Siltstone interbeds termed 'laminites', or 'mudstone facies', form a minor part of the unit (around 5%). Laminites typically range in thickness from 0.5 m to 3 m but generally occur less than 1 m and rarely up to around 12 m. The lateral extent of these units is highly variable and can occur laterally from tens of meters to hundreds of meters. These laminate beds may be associated with increased occurring of shearing of bedding.

3.3 Subsurface Conditions

Based on the available geotechnical data in the vicinity of the site, a preliminary geotechnical model was developed. The inferred subsurface conditions are presented below.

3.3.1 Soil and Rock Mass

The inferred subsurface profile for the Site is presented in Table 1. The inferred subsurface conditions comprise a layer of soil (fill and residual) overlying "poorer quality rock" comprising of weathered shale/siltstone/sandstone bedrock, overlying "better quality rock" comprising of slightly weathered to fresh sandstone bedrock (Hawkesbury Sandstone).

We note the data relied upon for the geotechnical subsurface profile included in Table 1 has been interpolated from the available data and may contain gaps and uncertainties. It is expected that the geological boundaries have variation due to natural variability or in the case of filled areas, previous construction. The uncertainties with the subsurface profile can be improved by further investigation within the Site.

Table 1 - Inferred Subsurface Profile

Inferred Unit	Typical Description	Inferred Depth to Top of Unit (m)	Estimated Thickness of Unit (m)
Soil	Variable Fill and Residual Soil (e.g. Sandy Clay / Sandy Gravel / Clay)	0	1 to 6
"Poorer quality rock"	Sandstone / Siltstone / Shale, highly to moderately weathered, very low to medium strength (approximately corresponding to Class III, IV and V Shale and / or Sandstone)	1 to 6	3 to 15
"Better quality rock"	Sandstone, moderately weathered to fresh, medium to high strength (approximately corresponding to Class I and II Sandstone)	7 to 15	> 20

Figures 4 to 6 presents cross sections with inferred geotechnical units across the Site. Whilst these can be used for preliminary shoring design, the designer should note the likelihood that the profiles will change following the site investigation.

3.3.2 Filling and Near Surface Modifications to Natural Ground

Near the surface, the natural soil conditions are expected to have been significantly modified by previous construction activities. Some of these modifications are discussed below.

The Telstra utility tunnels comprise of network of tunnels carrying communication cables located in the Sydney CBD. The construction of the Telstra tunnels started in the late 1800s with most of the tunnels built using the cut and cover method, with backfill placed around the tunnels for drainage. The construction method of the Telstra tunnels was predominantly cut and cover which has resulted in the presence of fill material on top of (and likely around the sides of) the tunnel structures. The fill material around the excavation is understood in some locations to have been excavated to form 1H:1V batters in order to enable compaction during backfilling. Due to the close proximity of Telstra tunnels near the site, deeper fill could therefore be expected around the site boundaries along Pitt Street and Liverpool Street.

Engineered and non-engineered fill is expected to underlie the street road pavements adjacent to the Site. The available geotechnical data indicates fill thickness to be approximately 2 m deep on the south boundary of the proposed development and approximately 3 m deep in the northwest area of the proposed development. We note historical excavations in the Sydney CBD may result in fill depths greater than those noted.

We note that there may be some underground services (which are unknown at this stage) adjacent to or within the Site. The construction of these underground services has likely resulted in the presence of fill material along the alignments. It is expected that a layer fill material could also be encountered around the existing building foundations (e.g. shallow footings) and adjacent to existing basement structures, depending on how these were constructed.

3.3.3 Geological Structures

A brief discussion on the major geological structures that could be encountered follows:

<u>Bedding</u>

The sedimentary bedding planes in the Sydney Basin are generally sub-horizontal. Bedding is expected to dip very gently between 0° and 10°. However, dips up to 30° with variable orientation have been observed in the Hawkesbury Sandstone in association with cross bedding. It is also common for bedding to be locally steeper adjacent to major geological structures such as faults and dykes. Generally cross bedding is closed and tight and does not necessarily form a defect; these are termed bedding fabric. Bedding that does form a defect is termed bedding partings. The average bedding parting spacing in the Hawkesbury Sandstone (massive and sheet facies) is between 1 m and 3 m. Bedding parting spacing of massive Hawkesbury Sandstone facies increases to approximately 6 m on average.

Joints / Faults

Joints in the Hawkesbury Sandstone includes an orthogonal pair of sub-vertical joint sets striking approximately north-north east and a less developed set commonly observed striking east-southeast; these are considered to be ubiquitous. Spacing ranges from between 1 m to 10 m with closer spaced (0.1 m to 0.5 m) sub-vertical joints often locally occurring in association with faulting or “joint swarms” (as discussed below).

As shown on the Pells et al (2004)[1] map, the Site is in moderate proximity to the Martin Place Joint Swarm which has been mapped about 100 m to the west of the Site.

The Martin Place Joint Swarm can be described as a sub-vertical north-northeast striking zone with sub-vertical closely spaced joints, zones of crushing and shearing and low angled thrust faults. Some of these closely spaced joints were observed during basement excavations in the Sydney CBD. The rock mass class is anticipated to be locally reduced in association with this geological structure. Exposures of this geological structure are shown in Photo 1. Some of this type of conditions might be encountered at the Site although given that the Site is not immediately adjacent to the mapped Martin Place Joint Swarm, such conditions might not be present.


Photo 1: Martin Place Joint Swarm Exposed in A Basement Excavation in The Sydney CBD

Dykes

Dykes (igneous intrusions) in the Sydney CBD area are typically sub-vertical and strike ENE to WSW. A small dyke was observed in the 157 Liverpool Street excavation, and the Oxford Street dyke is projected further to the north. Similar features may be encountered in the 338 Pitt Street excavation. Dykes may be associated with more weathered and weaker rock.

Geological Contacts

Within the “poorer quality rock” and at interfaces between geological units, geological contacts between rock units may be locally weaker and there may be relatively more jointing.

Figure 2 presents a locality plan showing available geotechnical mapping and the associated major defects from PSM database.

[1] Pells PJN, Braybrook JC & Och DJ (2004), Map and Selected Details of Near Vertical Structural Features in the Sydney CBD.

3.3.4 Groundwater

Based on the available piezometric data, the groundwater table has previously been encountered at:

- RL 6.0 m AHD near the intersection of Central Street and Pitt Street;
- Higher than RL 2.0 m AHD along Liverpool Street.

The 'regional' groundwater table at the Site is expected to be within the "better quality rock" between RL 0 m AHD and RL 10 m AHD. It can be expected that the proposed excavation will encounter the 'regional' groundwater table, 'perched' groundwater above this (e.g. within soils) and potentially water from adjacent leaky services. Therefore, the shoring designer should consider groundwater pressure and drainage in the basement design.

3.4 Site Investigation and Testing Recommendations

Based on the FJMT plans (Ref: H338-2000 to 2003 "General Arrangement Plans Basement 1 to 4", dated 2 December 2019), we understand that the base of basement excavation is at about RL 0 m AHD. We assume the uplift anchors / piles will be installed to about 15 m below base of the basement excavation.

As per the Section 3.3 in the previous proposal (Ref: PSM3102-004L dated 13 August 2019), we recommend undertaking six (6) boreholes as per Table 2 below:

Table 2 – Proposed Boreholes

Borehole ID	Location[1]	Base of Borehole (m AHD)	Declination (°)	Others	Purpose
BH01	Within Dugate Lane on ground, adjacent to 255 Castlereagh Street	-20	90	Standpipe piezometer to be installed	Investigate the ground conditions at the base of uplift anchors; Monitor the groundwater level on site.
BH02	Within building basement at 338 Pitt Street	-5	70	N/A	Investigate the sub-vertical defects in rock (e.g. joints, dyke and faults).
BH03	Within Dugate Lane on ground, adjacent to 255 Castlereagh Street	-5	90	N/A	Investigate ground profile for shoring design.
BH04	Within building basement at 233 Castlereagh Street	-5	90	N/A	Investigate ground profile for shoring design.
BH05	Within building basement at 324 Pitt Street	-5	70	N/A	Investigate the sub-vertical defects in rock (e.g. joints, dykes and faults).
BH06	Adjacent to Telstra Chamber Room	-20	90	N/A	Investigate the ground conditions at the base of uplift anchors; Easy to access on ground, adjacent to Telstra Chamber Room

Notes:

[1] Locations depend on the access, to be confirmed at later stage.

Based on the PSM database, we note that the building at 241 – 249 Castlereagh Street has no basement level. It may not be feasible to undertake any boreholes within this lot prior to demolition of the existing building. We expect that the borehole BH01, BH03 and BH06 will be able to be accessed on Dugate Lane from outside of the building (assumed no basement beneath).

Some test pits and / or core holes may be required in close proximity to the north boundary of 338 Pitt street, adjacent to the 310-320 Pitt St and 225-227 Castlereagh St buildings to confirm the footing foundation conditions prior to exposure of the footings.

Regarding Telstra Tunnel, the ground conditions are uncertain along the tunnel alignment. Some core holes may be required through the existing basement wall at 338 Pitt Street to confirm the ground condition behind the wall.

Figure 7 presents the proposed site investigation plan.

Details of the geotechnical site investigation work will be prepared at later stage in conjunction with the designer.

4. Preliminary Geotechnical Inputs to Design

The geotechnical advice and recommendations provided herein are suitable for preliminary design and are to be confirmed following detailed design incorporating the results of additional site investigation.

4.1 Shoring

4.1.1 Discussion

We understand that the shoring concept includes reinforced concrete piles, or steel sections with a concrete base plug, with lagging or shotcrete in between the piles, founded on a ledge at the top of "better quality rock". The piles would need to be supported laterally e.g. by ground anchors and / or by internal propping. Pile spacing and lateral support would depend on how much deflections need to be controlled.

The minimum clearance between proposed ground anchors and the existing roads / adjacent infrastructure / buildings shall be considered in the design. Restrictions on anchor locations can have a significant effect on support design.

Based on the Arup Temporary Shoring Plan with mark-up (Ref: 191105 338 Pitt shoring mark-up rev2, dated 7 November 2019), we understand that where the excavation wall is directly adjacent to the first protection reserve for the Sydney Metro infrastructure with no allowance for temporary rock bolts, shoring piles are proposed to be extended below the base of excavation to support potentially unstable rock blocks.

The rock conditions below the toe of piles (founded on a ledge) are important to the stability of the structure. The provided bearing capacity assume the rock ledge beneath the toe of piles has no adverse defects. The designer should consider the following to manage the geotechnical risk associated with pile toe support:

Design

- The inspection criteria should be included in the design, such as the pile toes and rock mass condition (including defects) beneath the toe of each pile to be inspected immediately after being exposed by excavation to confirm the assumed allowable end bearing.
- Some rock face support (e.g. rock bolting) may be required at the rock ledge beneath pile toes, if any defects are encountered.

Construction

- Minimise the disturbance of the rock ledge beneath pile toe, such as using saw cutting on the rock face.
- Avoid overbreak and undercutting at the rock ledge during the construction.

4.1.2 Retaining Wall Parameters

Table 3 provides soil and rock mass strength and stiffness parameters for the preliminary design of the shoring structure described above. The preliminary shoring design should consider the variability in ground profile as discussed in Section 3.

Table 3 – Geotechnical Parameters for Preliminary Design of Shoring Structures [1]

Material	Unit Weight γ (kN/m³)	Effective Cohesion[2] c' (kPa)	Effective Friction Angle[2] ϕ' (degrees)	Young's Modulus E' (MPa)	Poisson's Ratio ν	Allowable End Bearing on A Ledge[3] (kPa)	Rock Anchor Bond Stress[4]	
							Allowable (kPa)	Ultimate (kPa)
Fill	18	0	25	8	0.3	N/A	N/A	N/A
Residual Soil	20	0	32	30	0.3	N/A	N/A	N/A
"Poorer quality rock"	22	10	30	75	0.3	N/A	70	150
"Better quality rock"	24	500	45	2,000	0.2	1,000-2,000	1,000	2,000

Notes:

1. Parameters in this table are not to be used for design of vertically loaded building foundation piles. For vertically loaded building foundation piles, the parameters presented in Table 7 should be adopted.
2. c' and ϕ' can be used to calculate lateral earth pressure coefficients (k_0, k_a). Does not imply a maximum possible passive pressure, for scenarios where passive pressure is a load.
3. Assumes no adverse defects beneath pile toe and that excavation does not undercut pile toe. Refer to above discussion.
4. Anchorage testing should be undertaken after the installation of anchor to verify bond stress. During ground anchor installation, selected anchor holes should be logged by a geotechnical engineer to confirm the assumed material.
5. Shoring piles and / or excavation should be inspected to confirm the assumed ground profile.

4.2 Temporary Rock Face Support

Based on the inferred conditions, the temporary rock face support presented in Table 4 should be considered for the preliminary design purposes.

Table 4 – Potential Temporary Rock Face Support [1]

Inferred Unit	Potential Excavation Support
"Poorer quality rock"	1. Shoring structure continues down through the "poorer quality rock"; 2. If no shoring, an allowance for pattern rock bolts at 1.5 m x 1.5 m spacing with shotcrete.
"Better quality rock" (e.g. Sandstone Class II or better)	1. Shoring structure continues down through the "better quality rock"; 2. An allowance for spot rock bolts (installed as required based on field observations during construction) to support any rock blocks or wedges that may be formed by intersecting rock defects.
Fault zone / other adverse jointing / dykes etc. within "better quality rock"	1. An allowance for pattern rock bolts at 1.0 m x 1.0 m spacing with 200 mm thick shotcrete. However, actual support is highly dependent on the nature of the adverse condition.

Notes:

1 Rock face support to be confirmed during detailed design and construction.

Anchor / rock bolt locations need to consider underground infrastructure like adjacent building foundations, Telstra Tunnel and rail protection reserves.

4.3 Permanent Rock Face Support

Regarding permanent rock face support, if the excavation face is set in from the boundary with sufficient allowance for installing anchors and rock bolts, the temporary rock face support could be adapted to be permanent (e.g. by incorporation of appropriate corrosion protection to achieve durability). If the horizontal offset between the site boundary and excavation face is insufficient for installing permanent anchors and rock bolts, temporary anchors need to be destressed at the completion of basement excavation, and a permanent basement structure (e.g. concrete wall) would be required to provide permanent retention where required.

4.4 Underpinning

Some underpinning may be required where the proposed basement is excavated below and adjacent to footings of neighbouring buildings. The underpinning support will depend on whether the shoring will provide adequate support on its own, the level and loads of the existing foundations for both the neighbouring buildings and existing buildings within the Site, and the geological structure at the Site. PSM has not been provided with specific foundation data for the neighbouring buildings except at 255 Castlereagh Street where an as-built plan is available.

A range of potential underpinning solutions is presented in Table 5, the exact requirement for underpinning will be assessed at a later design stage when more details are known.

Table 5 – Potential Underpinning Solutions

Underpinning Solutions	Applicable Scenarios
Use of shoring (e.g. pile walls)	Footings founded in soil; Rock ledge strength not enough for considered loads.
Anchors, rock bolts, shotcrete	Footings founded on rock, that can be stabilised for the loads; The underpinning solutions depending mostly on the rock mass strength and the presence of defects.
Structural	Rock ledge strength not enough given the loads, and anchors or shoring solutions insufficient.

4.5 Ground Movements

This section provides a preliminary estimate of ground movements due to shoring deflection, rock mass relaxation and construction induced movements.

This report does not address ground movements in detail, nor effects on adjacent infrastructure. That will be assessed and reported in a separate package that follows on from the geometrical desktop report (Ref: PSM3102-006R dated 28 October 2019).

4.5.1 Shoring Deflection

The shoring system should be designed with consideration of the effects on neighbouring buildings and other infrastructure. We note typically this type of shoring design with appropriate construction sequencing and anchorages / propping, results in deflections of the shoring of 5 mm to 30 mm for soil depths less than 6 m.

4.5.2 Rock Mass Relaxation

The Hawkesbury Sandstone can have relatively high in-situ horizontal stresses; these can exist even in close proximity to the surface. An excavation in this environment causes the stresses to redistribute around and beneath the excavation and elastic relaxation of the excavated face. This process results in inward movement of the excavation walls as a result of relief of stress concentrations in the rock mass, known as rock mass relaxation. In Sydney, the lateral movements in rock at the surface around the excavation perimeter have usually ranged between 0.5 mm and 1 mm per metre depth of rock excavation. For a long (2-dimensional) 20 m deep excavation in rock such as this one, horizontal movement at the top of the rock face could therefore be about 10 mm to 20 mm, with some associated downwards movement too. This is additional to shoring deflection.

We note that there are neighbouring buildings close to the proposed excavation footprints. The movement due to rock mass relaxation is essentially unavoidable and therefore its effects on adjacent infrastructure need to be considered.

4.5.3 Construction Induced Movement

The construction induced movement could be due to the following:

- Ground loss from hole collapse during pile excavation (particularly within fill and cohesionless soil units)
- Ground loss from anchor drilling (particularly within fill and cohesionless soil units)
- Decommissioning of existing buildings and infrastructure (e.g. removal of retaining walls).

The designer should consider this issue, as it has the potential to cause additional ground deformation.

4.6 Building Foundations

4.6.1 Shallow Footings

For preliminary design, shallow footings may be proportioned on the basis of an allowable bearing pressure (ABP) for centric vertical loads on the sandstone units as provided in Table 6. Higher bearing capacities are often available subject to specific investigation and advice at detailed design.

Table 6 – Preliminary Shallow Footings Parameters of Inferred Geotechnical Units

Inferred Unit	Bulk Unit Weight (kN/m^3)	Ultimate Bearing Pressure (kPa)[1,2,4]	Allowable Bearing Pressure (kPa)[1,3,4]
"Better quality rock" (Sandstone Class II or better)	24	60,000	6,000

Notes:

1 Shallow footings should have a minimum plan dimension of 1.0 m and a minimum embedment depth of 0.5 m.

2 Ultimate values occur at large settlement (>5% of minimum footing dimension).

3 End bearing pressure associated with a settlement of <1% of minimum footing dimension (assuming a clean rock surface).

4 Under vertical centric loading in compression only.

We expect shallow footings will be founded on "better quality rock" due to the depth of the proposed basement excavation.

A higher allowable end bearing pressure for the "better quality rock" unit (Class II or better Sandstone) may be adopted subject to site investigation and possibly undertaking spoon testing during construction.

The exposed foundation construction after excavation should be inspected by a suitably qualified geotechnical engineer to confirm the assumed design parameters.

4.6.2 Piles (Compression)

Where piles are required, they should be designed in accordance with the requirements of an appropriate standard, such as AS 2159-2009, *Piling – Design and Installation*. The parameters provided in Table 7 may assist in the preliminary design of piles within the "better quality rock" (Sandstone Class II or better) units.

Table 7 - Foundation Engineering Parameters of Inferred Geotechnical Units

Inferred Unit	Bulk Unit Weight (kN/m^3)	Ultimate Shaft Adhesion (kPa)[1, 4]	Allowable Bearing Pressure[2, 3]	Typical Long-Term Young's Modulus (MPa)	Poisson's Ratio
"Better quality rock" (Sandstone Class II or better)	24	2,000	6,000	900-2,000	0.2

Notes:

1 Assumes clean socket with roughness category R2 or better, to be verified during construction.

2 End bearing pressure associated with a settlement of <1% of minimum pile dimension (assuming a clean pile base, to be verified during construction).

3 Under vertical centric loading in compression only.

4 Coincidence of shaft adhesion and end bearing to be considered in design.

We understand that higher bearing capacities may be required for the lift shaft/core. These could be available following further review of geotechnical data. Specific analyses to predict the settlements resulting from the higher loads may be required.

For settlement of bored piles founded in rock, the following should be noted:

- Where the pile is sized using the serviceable end bearing pressure in Table 7 (i.e. assuming all the serviceability load is carried by the base), the settlement would be expected to be less than 1% of the pile diameter, and

- Where the design utilises the shaft resistance of socketed piles in rock, Pells (1999)[2] provides guidance on methods to assess settlements for such piles.

Pile inspections to confirm the foundation conditions and verify the assumed design parameters are recommended. The inspection should be performed by a suitably qualified geotechnical engineer prior to pouring concrete. Details of the inspection regime are to be finalised once loading and construction details are finalised by the designer.

Where adjacent foundation details differ (e.g. pile and pad, differing loads or ground conditions) differential settlement should be assessed.

Where piles or footings are to be founded in close proximity to a vertical face, a reduced bearing capacity should be adopted. Further advice should be sought if this is applicable.

4.6.3 Tension Piles / Anchors

Where anchors are required, they should be designed in accordance with appropriate standards such as in Appendix B of AS4678-2002, *Earth-retaining Structures* or AS5100.3-2017, *Foundation and Soil Supporting Structures*. Where piles are required, they should also be designed in accordance with the requirements of an appropriate standard, such as AS 2159-2009, *Piling – Design and Installation*. The parameters presented in Table 8 may assist in the preliminary design.

Table 8 – Geotechnical Parameters for Preliminary Design of Tension Piles / Anchors

Material	Bulk Unit Weight γ (kN/m^3)	Buoyant Unit Weight [2] γ_{β} (kN/m^3)	Typical Long-Term Young's Modulus E' (MPa)	Poisson's Ratio ν	Grout (concrete) / Rock Bond Stress [1,3]	
					Allowable (kPa)	Ultimate (kPa)
"Better quality rock"	24	14	900-2,000	0.2	1,000	2,000

Notes:

1 Assumes clean pile socket with roughness category R2 or better.

2 Buoyant unit weight shall be adopted when the rock is below groundwater table.

3 Anchorage testing shall be undertaken after the installation of anchor. During ground anchor installation, selected anchor holes shall be logged by a geotechnical engineer to confirm the assumed material.

The design of the tension piles / anchors should take into account the following mechanisms of geotechnical failure:

- Piston pull-out (failure at interface with rock)
- Cone pull-out (uplift of a mass of rock)
- Group effect (reduction of the above two capacities due to closely spaced piles locations).

We note that neighbouring structures could be impacted by the area of influence of the tension elements. As such a detailed analysis should be undertaken when details and locations of those piles are known.

The design of uplift anchors / tension piles shall consider effects due to any rock mass structure or localized lower rock mass strength.

4.7 Excavatability

It is our experience that excavatability is heavily dependent on both the operator and the plant used. The contractor should satisfy itself with regards to suitable construction plant to achieve excavatability of the rock units, especially the "better quality rock" unit.

---

[2] P.J.N. Pells (1999), State of Practice For the Design of Socketed Piles in Rock

4.8 Sandstone Harvesting

In the Planning Secretary's Environmental Assessment Requirements (SEARs) dated 19 August 2019, it is noted that:

"An investigation and analysis of the quality of sandstone to be removed during the excavation, including consideration of contamination and an assessment of the suitability of the rock for removal by cutting into quarry blocks for use as high-quality building construction material."

At this stage, we are not able to assess the suitability of the rock for removal by cutting into quarry blocks for use as building construction materials, as no site-specific data is available, and an environmental assessment has not been completed. Further assessment will be undertaken following the site investigation. Suitability will depend on factors such as weathering, defects and lithology.

4.9 Drainage and Inflow

Where excavation below the water table is proposed for any basements, construction stage and permanent dewatering may be required. It is likely that this could be undertaken by a pump and sump methods within the excavation.

We note that recent experience indicates that the New South Wales Office of Water (NoW) has been conditioning approval of basement excavations on the basis of the following:

- Temporary dewatering allowed during excavation. Permits will need to be sought for both extraction of the water and disposal.
- No inflows into the basement allowed in the permanent condition. That is the final basement needs to be watertight, i.e. tanked. Such a requirement results in the basement floor slab, and the walls needing to be designed for the full hydrostatic load below a maximum foreseeable water table. This requirement, if enforced by the regulatory authorities, may have a significant effect on the design of the permanent structure. It is our experience that NoW may relax the requirement for tanking where monitoring of groundwater levels in combination with analysis of inflows in the permanent condition indicate yearly inflow into the excavation of less than 3 ML/year and that the extraction of groundwater has no adverse impact on other groundwater users.
- The developments are usually conditioned on monitoring of groundwater levels, assessment and estimation of temporary inflows during construction, and assessment of effect on neighbouring structures.

Based on the available geotechnical data, we have undertaken a preliminary assessment of the inflow of groundwater due to the basement excavation. It indicates that the total steady state inflow rate for a drained basement may be around 3 ML/year. Given that the natural variability and unknown groundwater table, we are unable to conclude at this time that the total water inflow would be less than the nominal 3 ML/year limit. We intend to undertake a detailed assessment after site investigation.

4.10 Excavation Induced Vibration

We note that the site is surrounded by buildings. To reduce the likelihood of damage to these buildings, we recommend that a general peak particle velocity limit of 10 mm/s is adopted at the boundary (consistent with limits provided in AS2187.2 (2006) Appendix J), with tighter limits adopted if required for particular buildings. We recommend monitoring at the boundary of the site. A vibration monitoring / management plan may be required.

The contractor should make their own assessment on appropriate excavation equipment to achieve the limits. The contractor should recognise that there is a potential for damage to adjacent buildings and consider this in planning and executing their work.

4.11 Site Subsoil Category

We have classified the Site in accordance with Australian Standard AS1170.4 – 2007 *"Structural Design Actions: Part 4: Earthquake Actions in Australia"*. Based on the available geotechnical data, we consider that the Site is classified as Class C – Shallow soil site.

4.12 Monitoring

It is expected that the designer (in conjunction with the geotechnical engineer) will develop an appropriate monitoring plan and system to confirm the adequacy and behaviour of the support design (e.g. shoring and rock face) and the impact on adjacent buildings during construction. The designer should confirm the tolerance of the adjacent infrastructure and buildings to any ground movements and incorporate this in the monitoring plan. It is expected that monitoring may include survey prisms, inclinometers, and potentially extensometers.

Should there be any queries, please contact the undersigned.

For and on behalf of
PELLS SULLIVAN MEYNINK

JUNO LIANG
GEOTECHNICAL ENGINEER

JEREMY TOH
PRINCIPAL

Brisbane

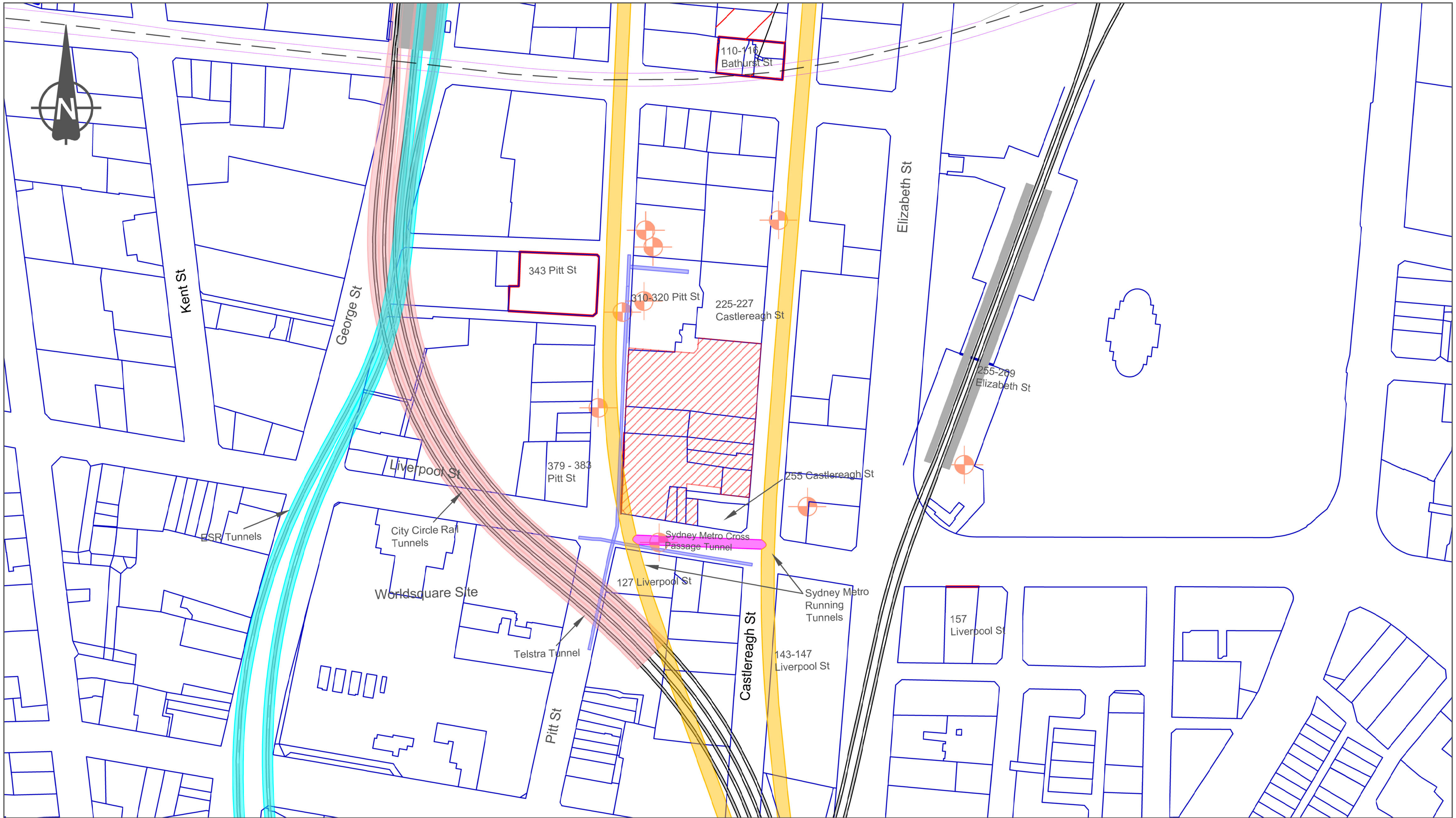
Level 6, 500 Queen Street
GPO Box 3244
Brisbane QLD 4000
+61 7 3220 8300

Sydney

G3-56 Delhi Road
North Ryde NSW 2113
+61 2 9812 5000

Perth

Level 3 22 Delhi Street
West Perth WA 6005
+61 8 9462 8400



LEGEND

Proposed extent of basement at The Site (338 Pitt Street)

Boreholes from PSM database

Geotechnical Mapping Data

Notes:

1. Proposed site basement extent is taken from FMJT Drawing "General Arrangement Plans - Basement 1" dated 2 December 2019
2. Existing rail tunnels geometry is preliminary and to be confirmed by surveying and / or exact data
3. Refer to geometrical study report (PSM3102-006R, dated 28 October 2019) for a full list of referenced documents

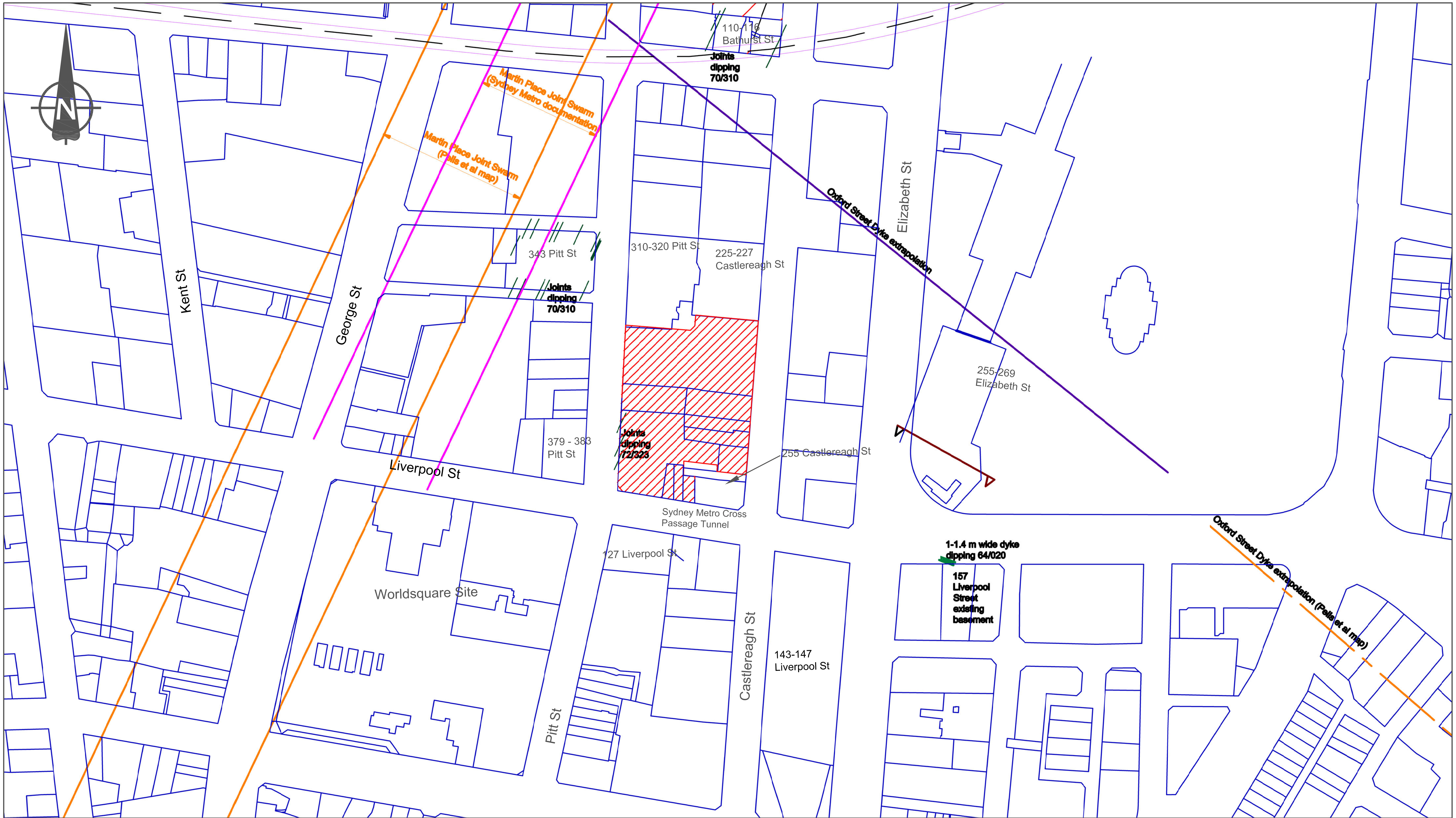
0 40 80m
1 : 2000 at A3

PSM

Hans Sydney Pty Ltd
Hans 338 Pitt Street
Sydney

GEOTECHNICAL DESKTOP STUDY
SUMMARY OF AVAILABLE GEOTECHNICAL DATA

PSM3102-005R	FIGURE 1



LEGEND

Proposed extent of basement at The Site (338 Pitt Street)

Mapped Joint

Notes:

1. Proposed site basement extent is taken from FMJT Drawing "General Arrangement Plans - Basement 1" dated 2 December 2019
2. Refer to geometrical study report (PSM3102-006R) for a full list of referenced documents
3. Location of joints is based on PSM database

0 40 80m

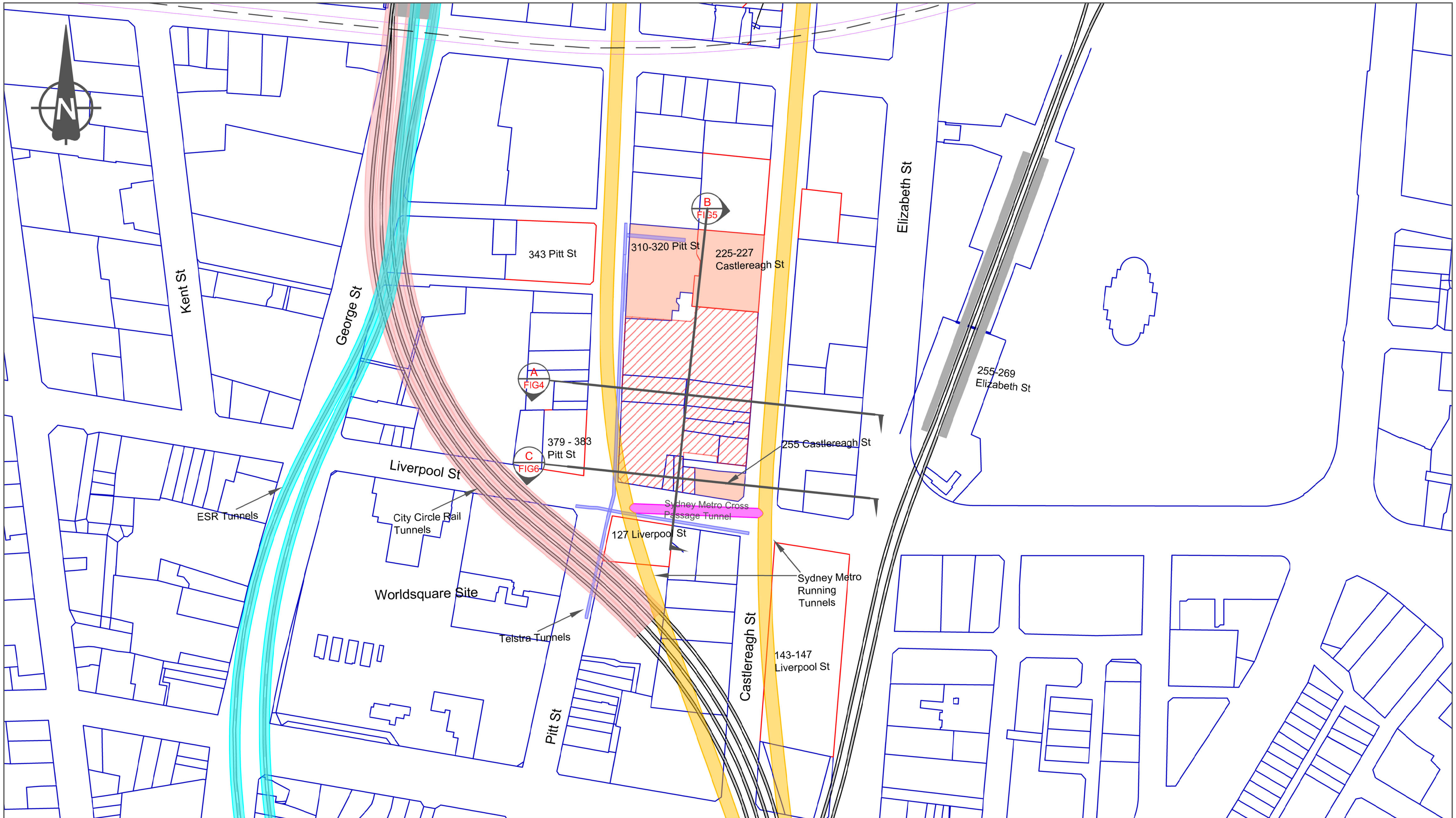
1 : 2000 at A3

PSM

Hans Sydney Pty Ltd
Hans 338 Pitt Street
Sydney

GEOTECHNICAL DESKTOP STUDY
SUMMARY OF MAJOR GEOTECHNICAL STRUCTURES

PSM3102-005R	FIGURE 2



LEGEND

- Proposed extent of basement at The Site (338 Pitt Street)
- Adjacent Buildings
- Heritage Listed

Notes:

1. Proposed site basement extent is taken from FMJT Drawing "General Arrangement Plans - Basement 1" dated 2 December 2019
2. Refer to geometrical study report (PSM3102-006R) for a full list of referenced documents
3. Heritage listed buildings are sourced from PSM previous projects and to be confirmed with relevant authority
4. Existing rail tunnels geometry is preliminary and to be confirmed by surveying and / or exact data

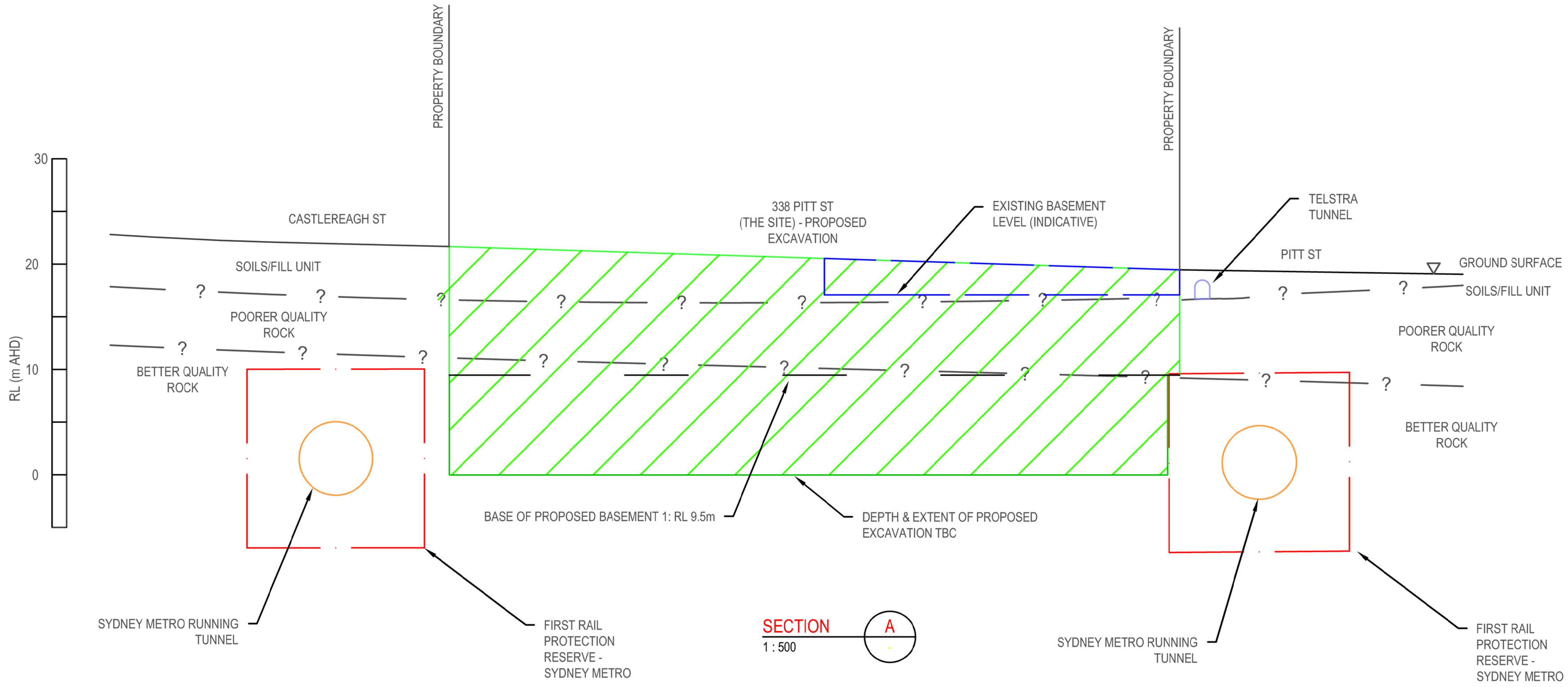
0 40 80m
1 : 2000 at A3

PSM

Hans Sydney Pty Ltd
Hans 338 Pitt Street
Sydney

GEOTECHNICAL DESKTOP STUDY
SITE LOCALITY PLAN

PSM3102-005R | FIGURE 3



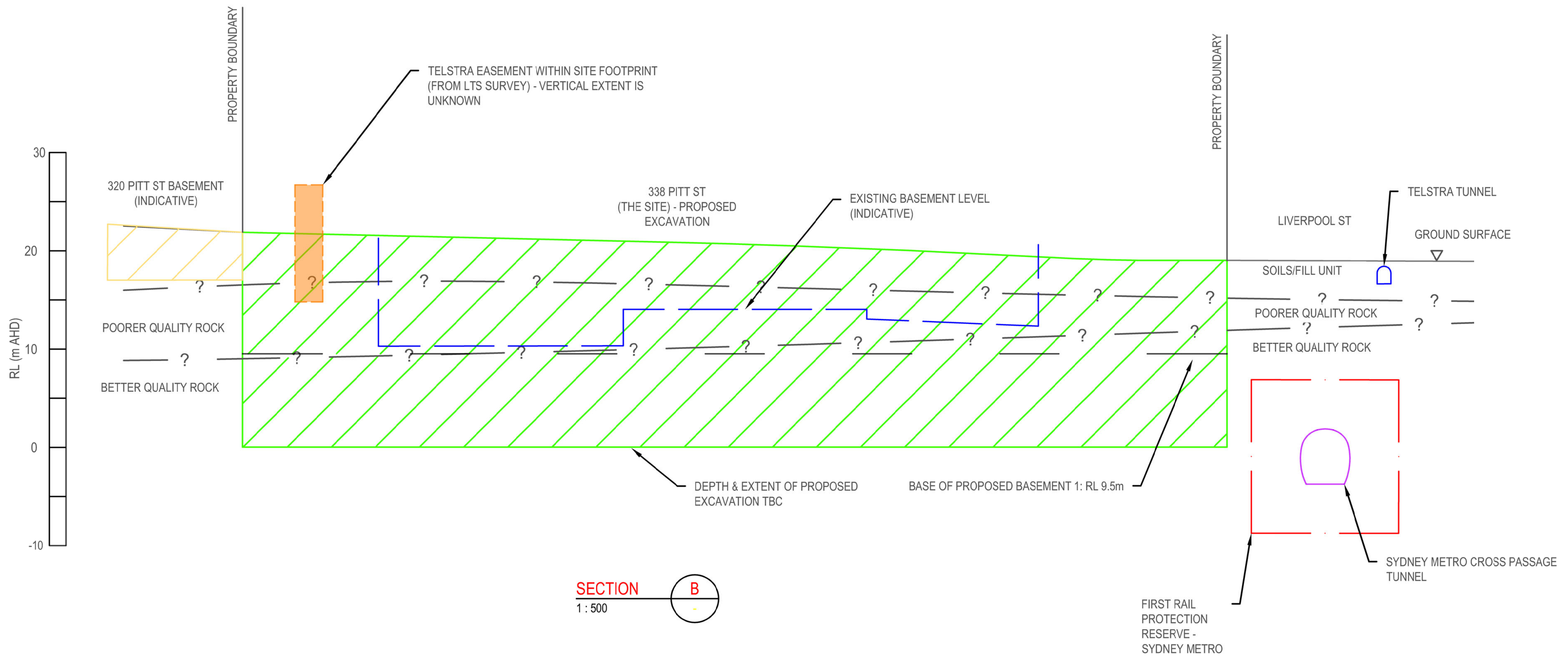
NOTES:
1. Extent of Rail protection reserves are assessed based on the TfNSW standard - Sydney Metro Underground Corridor Protection - Technical Guidelines dated 16 October 2017
2. Drawings are preliminary and subject to verification by survey
3. The basement footprint is indicative based on the FJMT drawing "Basement 1, 2, 3 and 4" dated 2 December 2019
4. The ground model is indicative based on available geotechnical data from PSM database and will be revised by further investigation. It does not explicitly consider local scale excavations and backfilling at this time.

0 10 20m
1 : 500

Hans Sydney Pty Ltd
Hans 338 Pitt Street
Sydney

GEOTECHNICAL DESKTOP STUDY
SECTION A

PSM3102-005R	FIGURE 4



SECTION B
1 : 500

NOTES:
1. Extent of Rail protection reserves are assessed based on the TfNSW standard - Sydney Metro Underground Corridor Protection - Technical Guidelines dated 16 October 2017
2. Drawings are preliminary and subject to verification by survey
3. The basement footprint is indicative based on the FJMT drawing "Basement 1, 2, 3 and 4" dated 2 December 2019
4. The ground model is indicative based on available geotechnical data from PSM database and will be revised by further investigation. It does not explicitly consider local scale excavations and backfilling at this time.

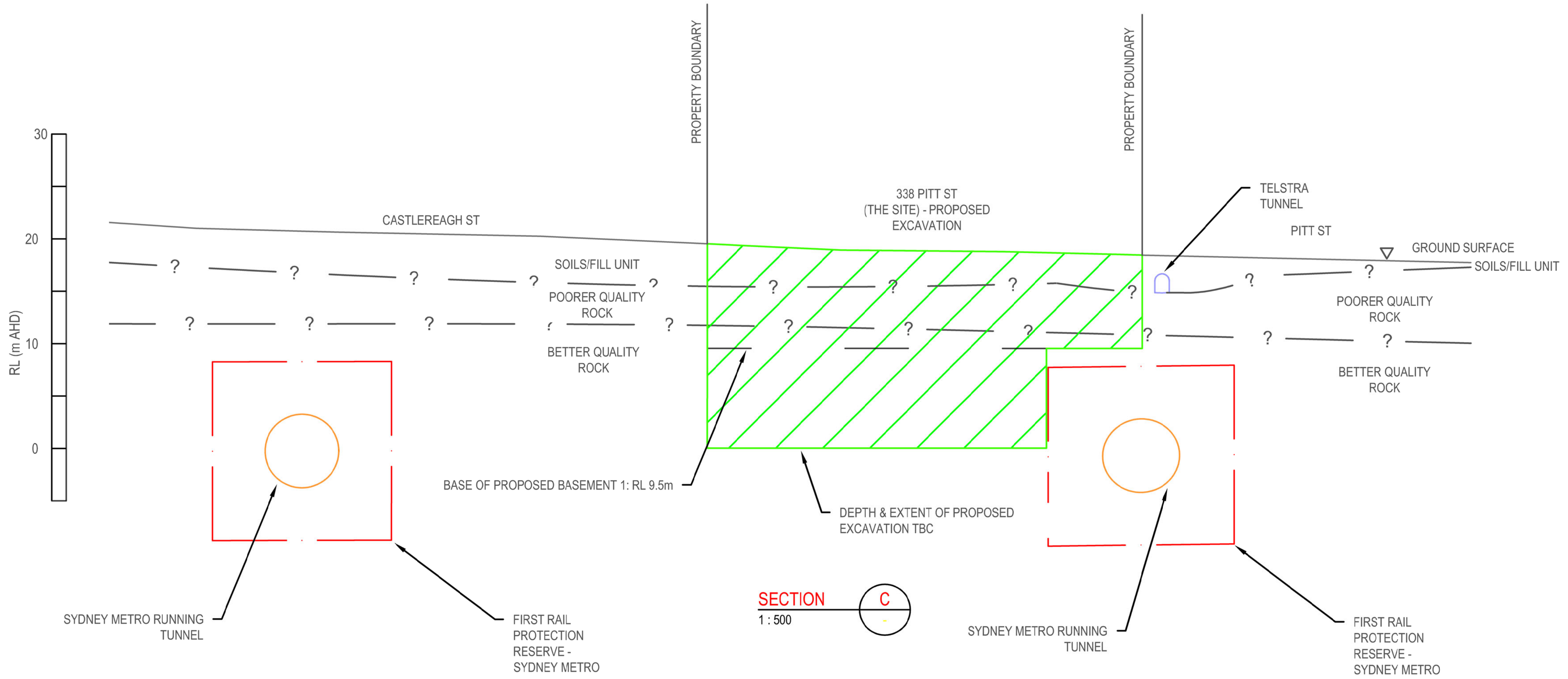
0 10 20m
1 : 500

PSM

Hans Sydney Pty Ltd
Hans 338 Pitt Street
Sydney

GEOTECHNICAL DESKTOP STUDY
SECTION B

PSM3102-005R	FIGURE 5



NOTES:

1. Extent of Rail protection reserves are assessed based on the TfNSW standard - Sydney Metro Underground Corridor Protection - Technical Guidelines dated 16 October 2017
2. Drawings are preliminary and subject to verification by survey
3. The basement footprint is indicative based on the FJMT drawing "Basement 1, 2, 3 and 4" dated 2 December 2019
4. The ground model is indicative based on available geotechnical data from PSM database and will be revised by further investigation. It does not explicitly consider local scale excavations and backfilling at this time.

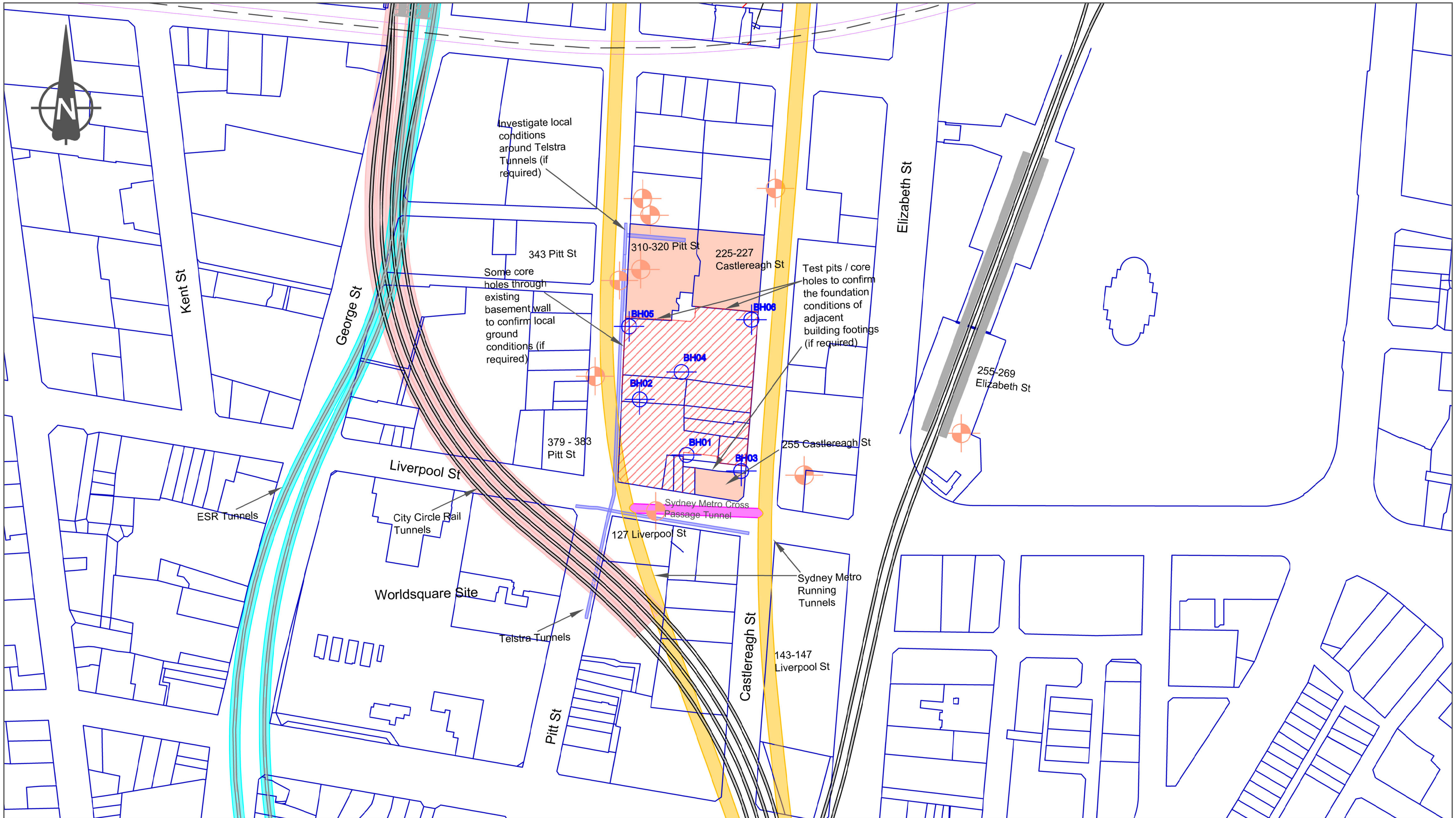
0 10 20m
1 : 500

Hans Sydney Pty Ltd
Hans 338 Pitt Street
Sydney

GEOTECHNICAL DESKTOP STUDY
SECTION C

PSM3102-005R

FIGURE 6



LEGEND

- Proposed extent of basement at The Site (338 Pitt Street)
- Adjacent Buildings
- Existing Boreholes from PSM database
- Proposed Boreholes (Approximate)

Notes:

1. Proposed site basement extent is taken from FMJT Drawing "General Arrangement Plans - Basement 1" dated 2 December 2019
2. Refer to geometrical study report (PSM3102-006R) for a full list of referenced documents
3. Borehole details (e.g. depths and inclinations) refer to this report.

PSM

Hans Sydney Pty Ltd
Hans 338 Pitt Street
Sydney

GEOTECHNICAL DESKTOP STUDY
SITE INVESTIGATION PLAN

PSM3102-005R	FIGURE 7