Attachment A10

Geotechnical Desktop Investigation

Douglas Partners
Geotechnics / Environment / Groundwater

Report on
Geotechnical Desktop Investigation

Proposed Commercial Building
232-240 Elizabeth Street, Surry Hills

Prepared for
Stasia Holdings Pty. Limited

Project 218198.00
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Integrated Practical Solutions

Douglas Partners
Geotechnics / Environment / Groundwater

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The undersigned, on behalf of Douglas Partners Pty Ltd, confirm that this document and all attached drawings, logs and test results have been checked and reviewed for errors, omissions and inaccuracies.

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1. Introduction

This report presents the results of a geotechnical desktop investigation undertaken for a proposed commercial building at 232-240 Elizabeth Street, Surry Hills. The investigation was commissioned in an email dated 13 September 2022 by Peter Kouvelas of Candalepas Associates on behalf of Stasia Holdings Pty. Limited and was undertaken in accordance with Douglas Partners' proposal 218198.00.P.001.Rev0 dated 19/09/2022.

We understand that the project involves demolition of the five existing buildings and construction of a new nine-storey commercial building with ground floor retail space and a minimum of three basement levels (see Appendix B).

The aim of the desktop study was to review Douglas Partners (DP) records and other available published information to provide:

- a general description of the geological profile for the site, including subsurface conditions and groundwater;
- comments on shoring wall system and excavation methodology;
- comments on groundwater;
- high level comments of the effect excavation, dewatering (if required) and retaining wall construction may have on neighbouring structures (no analysis); and
- foundation options and preliminary bearing capacities.

DP has carried out a review of available published data as well as DP's the previous investigation at the site in 2015.

2. Site Description

The roughly square site comprises three Lots and a strata plan encompassing 232 to 240 Elizabeth Street, Surry Hills (see Figure 1). The approximately 900 m^2 site has Elizabeth and Reservoir Streets on its western and southern sides and is bound to the north by a nine storey brick building and on the east by a six storey brick building. Foster Lane provides access to the northeast corner of the site. The site gently slopes to the southwest with a surface elevation on Elizabeth Street at about RL 10.9 m and RL 11.6 m at the southern end of Foster Lane.

Figure 1: Aerial photograph of the site (outlined in red).

The site is currently occupied by 1 to 3 storey brick built buildings, which appear to have no basements. The building to the east (50 Reservoir Street) appears to have no basements. It is uncertain whether the building to the north (230 Elizabeth Street) has any basement levels, however there is a roller shutter door on Foster Street which might indicate vehicle access.

The supplied architectural drawing indicates that an existing underground sewer pipe approximately 3 m depth enters the site from the north-east, just south of Foster Lane, at the upper-most basement level and that this pipeline will remain in place during and after construction of the proposed structure. The sewer pipe runs southwest across the site.

We do not believe the site is within 25 m of Sydney Trains assets, and therefore does not trigger additional assessments required by Sydney Trains under the ASA Standard T-HR-CI-12051-ST-v2.0 Developments near Rail Tunnels v2, during the on-going project. The surveyor should check the distance from the site to Sydney Trains assets.

3. Geology

Reference to the Sydney 1:100 000 Geological Series Sheet (see Figure 2) indicates that the site is underlain by Quaternary aged alluvial and estuarine sediments which typically comprise silty to peaty quartz sand, silt, and clay with ferruginous and humic cementation in places and common shell layers.

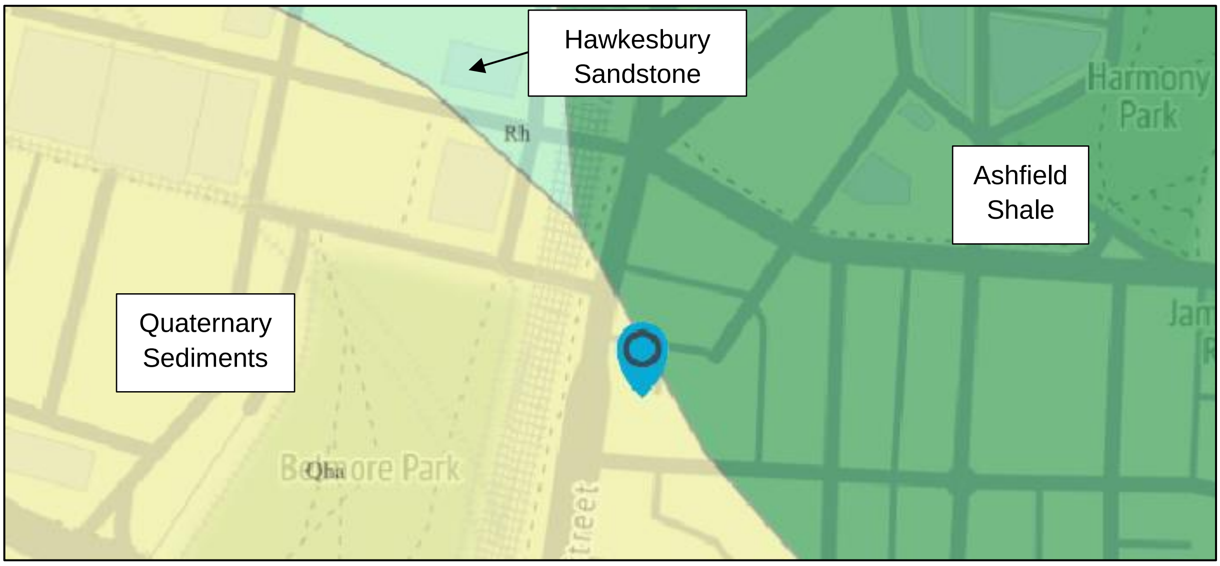

Figure 2: Excerpt of Sydney 1:100 000 Geological Series Sheet with Site (Blue Marker).

The site is located close to the boundary between the Hawkesbury Sandstone and Ashfield Shale units, both of Triassic Age but the overlying sediments mask the boundary. The Mittagong Formation, which typically contains interbedded shale, laminite and medium-grained quartz sandstone, is a transitional unit between the Ashfield Shale and the Hawkesbury Sandstone. The Hawkesbury Sandstone comprises flat or gently dipping medium and coarse grained, quartzose sandstone with minor shale and siltstone interbeds, while the Ashfield Shale comprises shale, laminite and carbonaceous shale.

The field investigation at the site in 2015 confirmed the presence of clayey residual soil overlying a relatively thin fine grained sandstone layer, inferred to belong to the Mittagong Formation, underlain by medium and coarse grained sandstone of the Hawkesbury Sandstone. No fine grained sediments of Quaternary Age were encountered during the field investigation.

4. Previous DP Investigations

DP has carried out an investigation at the site and a number of geotechnical and environmental investigations in the surrounding area (see Figure 3).

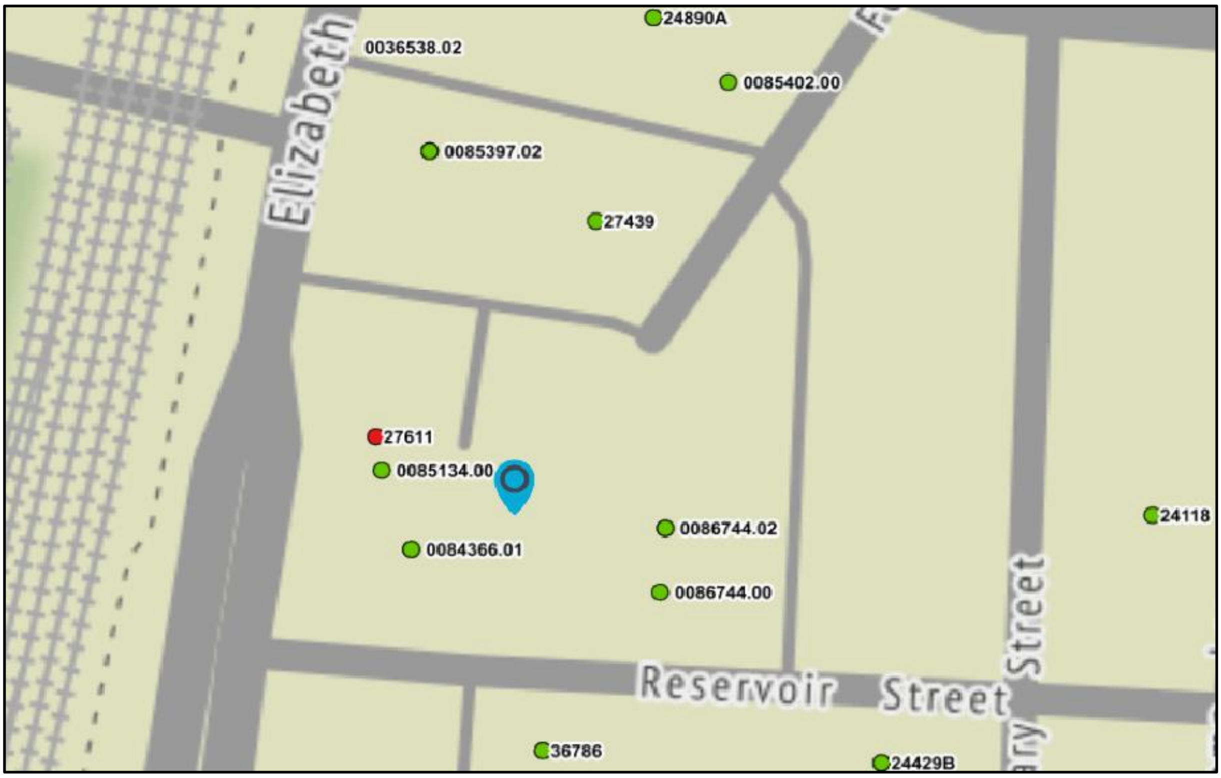

Figure 3: Location of Previous DP Projects (site indicated by blue marker).

4.1 Previous Site Investigation by DP in 2015

DP carried out a previous investigation at the site in 2015 (DP Report 85134.00.R.001.Rev0). The 2015 investigation comprised a site walkover by a senior engineering geologist and the drilling of one (1) borehole (see for borehole location and borehole log).

The subsurface profile at the borehole location comprised:

- Concrete and pavers - concrete slab over pavers to about 200 mm.
- Filling – clayey filling with some sand and sandstone fragments to about 0.75 m.
- Silty Clay - grey mottled brown, medium to high plasticity, stiff to very stiff silty clay with a trace of ironstone gravel (inferred residual soil) to a depth of about 5.3 m. Ironstone gravel bands were noted below 2.5 m.
- Sandstone – fine to medium grained sandstone, extremely to highly weathered and very low to very low strength encountered to about 6.35 m, with medium to coarse sandstone moderately weathered to fresh, medium to high and high strength, unbroken to borehole termination at 11.52 m (RL 0.01).
- Groundwater – no free groundwater was encountered while augering to 4.76 m and the use of water during NMLC coring precluded water observations while drilling in the rock to 11.52 m depth. A standpipe with a filter zone from 7 m to 10 m in the sandstone allowed water level monitoring and permeability testing to be carried out. The groundwater level measured 8 days after the completion of the drilling was at a depth of 1.2 m (RL 10.4 m) and recovered from a depth of 9.6 m (bailed out) to 3.5 m (RL 8.1 m) in 120 minutes during the permeability test. The estimated hydraulic conductivity of the sandstone (10^{-7} m/s) was typical for Hawkesbury Sandstone.

4.2 Nearby Investigations

DP have also undertaken investigations near to the site, as shown in Figure 3. The relevant findings of these previous reports are summarised below:

- Geotechnical Report, 'Proposed Multi-Storey Development', 52-58 Reservoir Street, Surry Hills (DP Ref: 86744.00:R.001.Rev0 dated April 2019).
    - Fill to depths of up to 2.05 m underlain by residual soil up to 5.8 m depth, underlain by sandstone bedrock which was high strength below 6.14 m and 6.6 m depth. Groundwater was recorded at RL 8.9 m, 11 days after well development.
- Geotechnical Report, 'Mixed Use Development', 216 – 228a Elizabeth Street, Surry Hills (DP Ref: 85397.00.R.001.Rev1 dated April 2016).
    - Fill to depths of up to 1.3 m underlain by residual soil up to 3.2 m to 5.0 m deep underlain by sandstone bedrock which was high strength below 4.9 m to 6.9 m depth. Groundwater was recorded at 5.07 m (approx. RL 5.93 m) and 4.63 m (RL 6.37 m) in BH1 and BH3 respectively.

5. Preliminary Geotechnical Model

Based on a review of the previous investigation at the site and nearby, the expected ground profile at the site is likely to comprise:

Filling	Encountered to approximately 0.75 m
Residual Silty Clay	Encountered to approximately 5.3 m
Very Low to very low strength Sandstone	Encountered to approximately 6.35 m.
Medium to High strength Sandstone	Encountered to base of borehole at 11.5 m.

Groundwater was encountered in BH1 at approximately RL 10.4 m 8 days after drilling and prior to carrying out the permeability test. This value should be treated with caution as it may vary across the site and may have changed in the meantime due to seasonal fluctuations, construction of adjacent basements and climate change.

The geotechnical model should be confirmed by additional boreholes and water level standpipes at the site.

6. Proposed Development

The supplied information indicates that the development will comprise construction of a ten level commercial use building with a three level basement, requiring bulk excavation to a depth of approximately 9.7 m (RL 1.6) and detailed excavation a further 700 mm to approximately RL 0.9. It is understood that the existing sewer line which crosses the site diagonally must be maintained as operational. The existing 1.45 m x 1.78 m brick sewer main will be left in place and will need to be temporarily supported while the basement is excavated and supported by the structure in the permanent condition.

7. Comments

Note that the comments in this section are of a preliminary nature only and are based on limited geotechnical information mainly one borehole at the site and from neighbouring sites which, in some cases, dates back to 2014. For design purposes, a full geotechnical investigation involving several cored bores with standpipe piezometer (wells) for groundwater monitoring (and sampling) will be required.

7.1 Excavation, Batter Slopes, Shoring Design, Ground Anchors & Stress Relief

Careful consideration will be required in planning and executing the bulk excavation given the physical constraints of the site, the sewer left in place and the adjacent building foundations/basement(s).

7.1.1 Excavation Conditions

It is anticipated that excavation to depths of about 9.7 m (RL 1.6 m) will be through filling, stiff to very stiff clay with some ironstone bands, very low to low strength sandstone and below about 6.5 m, medium to high and high strength sandstone.

Excavation of the material above the top of medium strength sandstone should be readily achieved using conventional earthmoving equipment such as tracked hydraulic excavators with bucket attachments. Excavation of medium and high strength, slightly fractured sandstone, as encountered in the borehole, can be achieved by heavy ripping and excavator mounted hydraulic rock hammers. Excavator mounted hydraulic rock saws may be used to control overbreak along boundary lines, cut breakage lines in the massive rock sections and for detailed footing excavation.

The use of such equipment will generally cause dust, noise and vibration that has the potential to affect adjacent below ground infrastructure, heritage buildings and occupants of nearby buildings. Where rock hammers are required in the vicinity of adjacent structures (closer than 20 m) it would be prudent to monitor and limit vibration on these structures. Based on DP’s experience and with reference to AS2670, a maximum peak particle velocity of 8 mm/sec (in any component direction) at the foundation level of adjacent structures is suggested for human comfort considerations. Vibration trials are suggested during initial excavation of the rock to verify vibration levels.

It should be noted that any off-site disposal of spoil will generally require assessment for use or classification in accordance with the current Waste Classification Guidelines (NSW EPA 2014). 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; or
- 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; or
- 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.

It is recommended that dilapidation surveys are carried out on surrounding buildings, pavements and the existing heritage brick sewer line. Dilapidation surveys of adjacent buildings and tunnels should be carried out before the commencement of any work (i.e., prior to demolition) to document existing defects so that any claims for damage due to construction related activities can be accurately assessed. It is recommended that a detailed assessment of structural characteristics of the sewer is carried out, to allow design of adequate support.

7.1.2 Excavations Adjacent to Existing Buildings

Prior to commencing bulk excavation, it will be necessary to obtain all records of the adjacent existing footings and any information on the founding conditions. Further investigation of all adjacent foundations and founding conditions may be required if the information is not reliable or not available. It will also be necessary to determine the extent and depth of any adjacent basements. This process is critical as excavation of the proposed new basement could destabilise existing structures and leave potentially unstable slivers of soil or rock in place. Affected footings may require underpinning.

7.1.3 Batter Slopes

Where room permits, temporary batters up to 3 m in height in soil and rock can be cut at batter slopes shown in Table 1 below, subject to detailed assessment of rock conditions by a suitably qualified geotechnical engineer/engineering geologist as the excavation progresses. Taller batters will require detailed analysis.

Table 1: Suggested Safe Batter Slopes (<3m in Height)

Material type	Maximum Temporary Batter Slope (H:V)	Maximum Permanent Batter Slope (H:V)
Stiff to very stiff clay	1.5:1	2:1
Very low strength laminite/sandstone	0.75:1	1:1
Low strength laminite/sandstone	0.5:1	1:1
Medium strength laminite/sandstone	Vertical*	Vertical*
High strength laminite/sandstone	Vertical*	Vertical*

Note: * Subject to discontinuity assessment by experienced Geotechnical Engineer/Engineering Geologist (spot bolting may be required).

Where insufficient room exists for the suggested batter slopes, such as along the site boundaries and adjacent to existing services, support of the excavation face can be provided by anchored soldier piles with shotcrete infill panels, by contiguous piles tied back with anchors / bolts or by soil nailed walls in the material above the medium strength rock.

7.1.4 Shoring Design

Design pressures for retaining walls should take into account the requirement to limit movement of the surrounding ground and adjacent structures and to ensure an adequate factor of safety is maintained against failure (for temporary and permanent retaining walls).

It is suggested that the design of cantilevered shoring systems (or shoring systems with one row of anchors) be based on a triangular earth pressure distribution using the earth pressure coefficients provided in Table 2. ‘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 2: Design Parameters for Shoring and Retaining Systems

Material Type	Unit Weight (kN/m^3)	Earth Pressure Coefficient	
		Active (K_a)	At Rest (K_o)
Very stiff to hard clay	20	0.25 (0.30)	(0.5)
Very low strength laminite/shale/sandstone	22	0.25 (0.30)	(0.40)
Low strength laminite/sandstone	22	0.10 (0.15)	(0.15)
Medium and High strength sandstone	24	0.0*	0.0*

Notes: * Subject to discontinuity assessment by experienced Geotechnical Engineer/Engineering Geologist (Spot bolting may be required);

( ) Permanent earth pressure coefficients shown in brackets.

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

An alternative approach, where the support pressure is related to the height of soil/weathered rock retained, could also be used. Where there are no movement-sensitive structures within the influence zone behind retaining walls, an earth pressure distribution equal to 4H kPa (where H, in metres, equals the depth to the top of self-supporting medium strength or stronger rock) can be used. 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 is 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.

Where shoring comprises soldier piles, the passive resistance for piles founded in rock below the base of the excavation may be based on an ultimate passive bearing capacity of 3500 kPa, provided that the sandstone is of at least medium strength and not adversely affected by discontinuities. Higher values may be possible but will depend on the strength and quality of the rock.

The first 0.5 m of rock socket below the bulk excavation level should not be taken into account 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), but subject to analysis. This is also relevant where anchors are installed (or toe anchors, just prior to fully exposing the toe of the pile). Staged excavation and inspection by a suitably qualified geotechnical engineer will therefore be required to confirm that the rock in front of the piles is not adversely affected by discontinuities, especially where passive resistance is relied upon.

A factor of safety must be applied to this ultimate value, while considering the displacement that is required to mobilise the passive resistance. Additional support will be required if allowable displacements are exceeded or if the rock is adversely affected by faults, bedding or jointing. Piles may be socketed into the top of free-standing medium strength or stronger sandstone provided adequate retention and toe support are provided.

Additional surcharge loads, such as new and existing footings, hoardings, façade retention systems, pavements and construction related activities must also be allowed for in the design.

The passive earth pressure loading described above does not include either earthquake loads or hydrostatic pressure due to the build-up of groundwater behind impermeable walls, which must also be considered in the design.

Medium strength and stronger rock is considered to be self-supporting, and their faces can be excavated vertically, subject to regular inspection every 1.5 m drop in excavation by a suitably qualified and experienced engineering geologist/geotechnical engineer, to confirm that the rock mass is not adversely affected by discontinuities or soft seams.

7.1.5 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.

It should be noted that permission from adjacent property owners will be required prior to installing bolts/anchors below their land. Due consideration should also be given to buried services and any excavations, basements or tunnels nearby. TfNSW, Sydney Water and other service providers may require assessment on the effects that rockbolts and anchors may have on their assets.

Pre-stressed ground anchors, rockbolts and dowels (support elements) can be used to laterally support existing walls, new shoring, underpinning works or unstable rock masses. These support elements should be bonded into the stronger rock, inclined as required, but preferably not steeper than

30° below the horizontal. Table 3 provides allowable bond stresses for estimating purposes. The parameters given in Table 3 assume that the drill holes are clean and adequately flushed.

Table 3: Preliminary Bond Stresses for Anchor Design

Material Description	Allowable Bond Stress (kPa)
Very low strength rock	50
Low strength rock	100
Medium strength rock	350
High strength rock	1000

These values should be confirmed by pull-out tests, carried out prior to installation of support elements. Ultimately, it is the anchoring 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 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 should be carried out to confirm that the load in the support elements is maintained and that losses due to creep or other causes do not occur.

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

7.1.6 Stress Relief

Locked-in stresses are present within the rock. During excavation, these stresses are released which generally results in lateral movement of the rock mass face towards the excavation, dragging the soil (and any shoring) with it as movement occurs. Generally, units of stiffer rock (medium strength or stronger rock) will have higher horizontal locked-in stresses and experience more displacement. The degree of displacement is also dependent on rock excavation depth, bedding planes and jointing in the rock mass, excavation face length and face orientation. As the maximum principal stress in Sydney is in the north-south direction, the north and south faces can be expected to experience the most stress relief deformation. Although the east-west locked-in stress is less, the east and west faces will still experience substantial stress relief displacement.

From monitoring (and supported by numerical modelling within the Sydney CBD), horizontal stress relief movements are typically between 0.5 and 2 mm per metre depth of rock excavation. Maximum movement typically occurs at the top of the midpoint of the face and reduces to near zero in the corners of the excavation. Back from the crest of the excavation, movement can occur over a distance of up to three times the excavated rock depth with an initial reduction of approximately 1 to 1.5 mm per metre, reducing with distance from the face. This differential movement will give rise to strain in both the rock mass and the soil beyond the excavation. Most of the movement would be expected to occur progressively during the excavation. Heave may occur where relatively thin beds of competent rock is left in the base (bed separation due to buckling).

Stress relief movement may be less in areas that have already been partly de-stressed (stress relief may have already been caused by existing basements or tunnels).

Stress relief movements can crack adjacent buildings and tunnels close to the excavation and may also increase loads on any ground support anchors installed. The effects of this movement on the various buildings, tunnels and infrastructure should therefore be assessed by a structural engineer. Appropriate allowance should be made for the potential repair of these structures, should it be required. It is also recommended that dilapidation surveys of adjacent buildings and the sewer be carried out at various stages of excavation to carefully record the condition of the structures.

Consideration should be given to the locations of columns, connections with perimeter walls and other structural elements to ensure that future stress relief movements do not affect the building.

In-situ virgin stress conditions have not been measured on the site and the following stresses are suggested:

$\sigma 1 = \sigma_{NS} = 0.5$ MPa $+ 2.0\ \sigma V$

$\sigma 2 = \sigma_{EW} = 0.5$ MPa $+ 1.1 \sigma 1$

$\sigma 3 = \sigma_{V} = 0.024\ H$ MPa

where H = height of excavated medium strength or stronger rock face (m)

Notes:

- These stress correlations do not take into consideration the effect of nearby excavations and hence separate allowances should be made if this is the case.
- These stress correlations do not allow for stress relief or stress concentration due to faults, or intrusions, or the presence of high strength beds. Hence site conditions may vary significantly from the above correlations, depending on the specific features and their proximity to the site.

7.2 Groundwater

Given the limited groundwater data available from the investigation at the site in 2015, the bulk excavation is likely to intercept groundwater. Seepage during construction and in the long term should therefore be expected along the top of the rock (particularly after periods of wet weather) and through joints and bedding planes in the rock mass.

It is not possible to provide a reliable estimate of the seepage quantity that may be expected based on the available data. Additional boreholes to triangulate water levels and flow direction together with permeability testing will therefore be required during the geotechnical investigation to provide the necessary parameters for seepage analysis. Information about nearby basement and whether drained or tanked would also be required as drained basements can locally reduce the water levels locally.

Typically, seepage into basements in Sydney during construction and in the long term, are controlled by perimeter drains connected to a "sump-and-pump" system. Approval from WaterNSW, however, will be required prior to designing and construction of a drained basement. A drained basement, if approved by WaterNSW, will require permanent subfloor drainage to direct seepage to the stormwater drainage system for which Council approval will be required.

The need for ongoing dewatering after construction will depend on whether the basement is designed as a drained or tanked basement as described below:

- a drained basement will require permanent subfloor drainage below the basement floor slab connected to a sump which regularly pumps out the water. The disposal requirements of water collected on-site will be dependent on the chemical consumption of the water. Normally, water is disposed to a stormwater or sewer system subject to approval from the Council. However, a drained basement will act as a low point to which groundwater will flow. Therefore, if present, any contamination within the surrounding groundwater system could flow into the basement and adversely affect the quality of the water collected on site.
- a tanked basement would avoid the need for dewatering but is likely to be more expensive than a drained basement. A tanked basement would need to be designed to resist uplift forces associated with groundwater pressure, for which preliminary design should be based on a groundwater level determined by water level monitoring carried out in at least 3 standpipes installed as part of a future geotechnical investigation.

The amount of water seeping into the excavation during construction should be monitored as this will give an indication of likely inflows for the long term condition.

The geological map indicated that the site is underlain by alluvium, which can contain high permeability zones. Hence, there is a need to conduct a detailed investigation before design is completed and this should include long term pump out tests to determine the hydraulic properties of the alluvium and the sandstone below.

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.

7.3 Foundations

Based on the geotechnical model in Section 4, at least medium or even medium to high strength sandstone is expected at bulk level. A suitable foundation system would therefore comprise pad and strip footings, suitably sized for the typical parameters for the design of foundations on sandstone, based on the classification methods of Pells et al. (1998) in Table 4.

Table 4: Preliminary Design Parameters for Pad/Strip Footings

Material	Maximum Allowable End Bearing Pressure (kPa)	Ultimate End Bearing Pressure (kPa)	Field Young's Modulus, E (MPa)
Class III - medium strength sandstone	3,500	20,000	600
Class II - medium to high strength sandstone	6,000	60,000	1,200
Class I - high strength sandstone	10,000	120,000	2,000

Note that classification of the material in Table 4 is subject to the required number of boreholes, cored boreholes and spoon testing being carried out. In 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 by a geotechnical engineer to check for the presence of weak layers or clay bands.

Foundations proportioned on the basis of the allowable parameters would be expected to experience total settlements of less than 1% of the minimum footing's width under the applied working load, with differential settlement between adjacent columns expected to be less than half this value.

For design using the ultimate values provided in Table 4, a geotechnical strength reduction factor ($Ø_g$) should be determined by the designer in accordance with the piling code AS 2159-2009. Serviceability criteria will also need to be met when using ultimate design parameters.

All footing excavations should be inspected, and spoon tested (as required) by a geotechnical engineer to confirm that foundation conditions are suitable for the design parameters. Spoon testing will be required for all foundations with allowable bearing pressures of, or in excess, of 3,500 kPa.

7.4 Seismic Design

In accordance with AS1170-2007 "Structural Design Actions, Part 4: Earthquake Actions in Australia" a hazard factor (Z) of 0.08 and a site subsoil Class C_e is considered to be appropriate for the site provided all structural elements are supported by piles bearing on rock.

7.5 Geotechnical Considerations Relating to the Rail Corridors

The TfNSW Standard[1] sets out guidelines specifically for the tunnels. The Standard outlines 'protection reserves', construction restrictions and other aspects relating to developments in the vicinity of the rail infrastructure e.g., load limits on tunnels, tunnel displacement and tunnel monitoring criteria. The NSW Department of Planning also have guidelines for development near rail corridors

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[1] T HR CI 12051 ST V2 - Developments Near Rail Tunnels, November 2018

and busy roads[2]. The guidelines outline aspects relating to developments that are specified in Section 2 of the SEPP that have additional requirements to be considered before seeking approval.

We recommend that accurate drawings are obtained from TfNSW and a TfNSW registered surveyor checks the distance from the site boundary to the closest TfNSW infrastructure to determine whether it is within the protection reserves set out in the TfNSW guidelines. Further advice can be provided if the site is within 25m of the infrastructure.

8. Further Investigation

Further geotechnical investigation will be required to determine the site ground profile and in particular the depth to groundwater and to bedrock across the site and the strength of the rock at or below design foundation level. Coring of the bedrock is recommended to determine the appropriate parameters for economic foundation design. The investigation should include:

- a minimum of three (3) additional boreholes drilled to a depth of 3 m below the bulk excavation level with permeability testing, with at least three (3) monitoring wells to triangulate water levels and flow direction and assess the inflow rate.
- footing investigation of any adjacent buildings to determine footing type(s), founding depths and conditions if reliable information is not available. This may have to occur after demolition of the existing structure(s) and before commencement of excavation. This will typically involve test pits or coring.
- waste classification assessment of material proposed to be transported off site, in accordance with the appropriate guidelines.

9. Limitations

Douglas Partners (DP) has prepared this report (or services) for this project at 232-240 Elizabeth Street, Surry Hills in accordance with DP's proposal dated 19/09/2022 and acceptance received from Peter Kouvelas dated 12 October 2022. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of Stasia Holdings Pty. Limited 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.

The results provided in the report are indicative of the sub-surface conditions on the site only at the specific sampling and/or testing locations, and then only to the depths investigated and at the time the work was carried out. Sub-surface conditions can change abruptly due to variable geological processes and also as a result of human influences. Such changes may occur after DP's field testing has been completed.

---

[2] NSW Government: Department of Planning

DP's advice is based upon the conditions encountered during this investigation. 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.

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

Architectural Drawings

1

SECTION A
1:100

This drawing is to be read in conjunction with all relevant project documentation (incl. written architectural specifications) and all specialist consultant documentation incl. but not limited to structural, mechanical, electrical, and hydraulic engineering documentation.

Do not scale from this drawing. Only figured dimensions shall be used.

Report any discrepancy between this drawing and other project documentation immediately to the architect for clarification prior to commencement of related work on site.

All sizes of structural components that are shown on these architectural documents are for reference only. Refer to structural engineer's drawings for structural dimensions and architectural drawings for concrete set out dimensions.

Shop drawings are to be completed for all metalwork, joinery and specified trade items and reviewed by the architect (and structural engineer where required) prior to fabrication.

Refer to the 'Architectural Drawing Notes Page' for further notation.

DRAWING ORIGINAL SIZE: A1

0MM (SIZE ON ORGINAL)

Issue	Date	Description
P6 - WIP	Work in Progress	Consultant Issue
P5	2/11/2022	Client Issue
P4	19/10/2022	Consultant Issue
03	17/10/2022	Consultant Issue

Project:
232-240 Elizabeth Street, Surry Hills

Site:
232-240 Elizabeth Street, Sydney NSW

Client:
STASIA PTY LIMITED

Scale:	Drawn By:	Checked 1:	Checked 2:	Approved:
1:100 @ A1	IN, RB			

Architect:
Candalepas Associates
Registration No. NSW 5773
Registration No. VIC 17978
Registration No. WA 2405
Registration No. ACT 2603
Registration No. QLD 5463

309 Sussex Street, Sydney NSW 2000
info@candalepas.com.au
T: 02 9283 7755 F: 02 9283 7477

PRELIMINARY
NOT FOR CONSTRUCTION - DESIGN INTENT ONLY

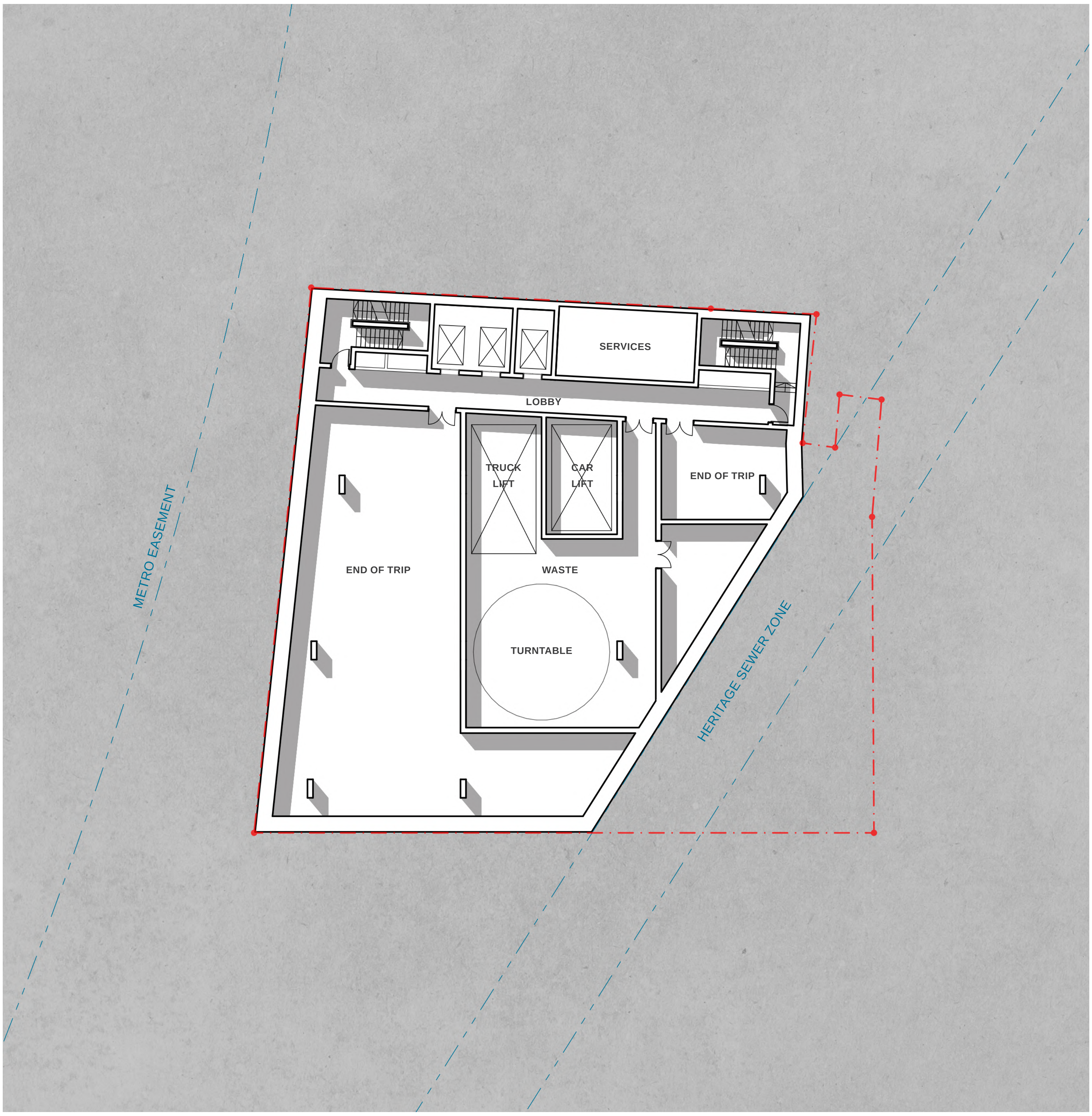
Drawing:
SECTION A

Job Number:
5968

Drawing Number:
PP 1101

Issue:
P6 - WIP

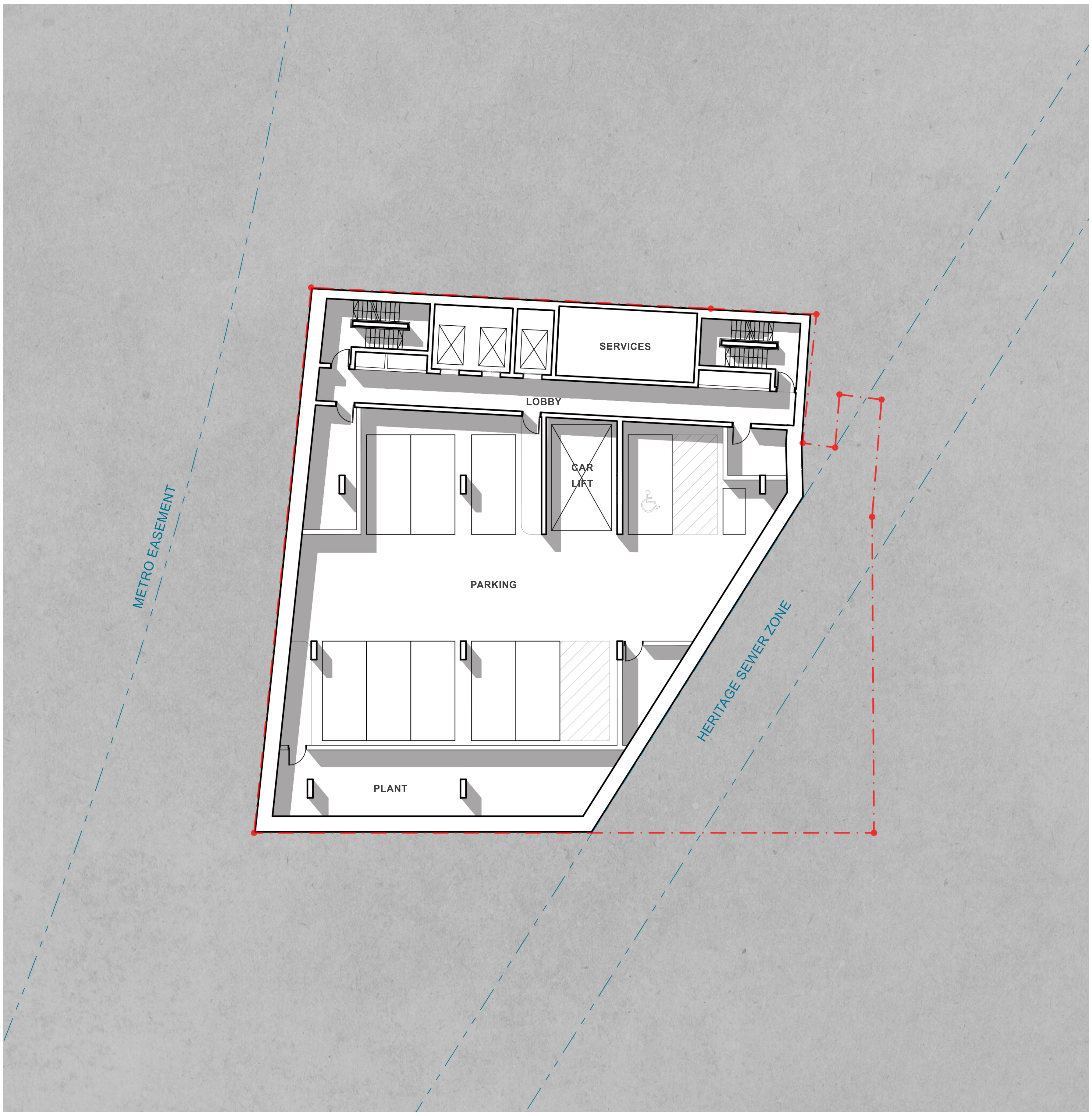
Print Date: Tuesday, 8 November 2022 3:43 PM



BASEMENT 1 PLAN

END OF TRIP GFA 270 m²

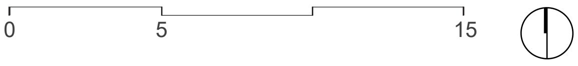
SCALE 1:250

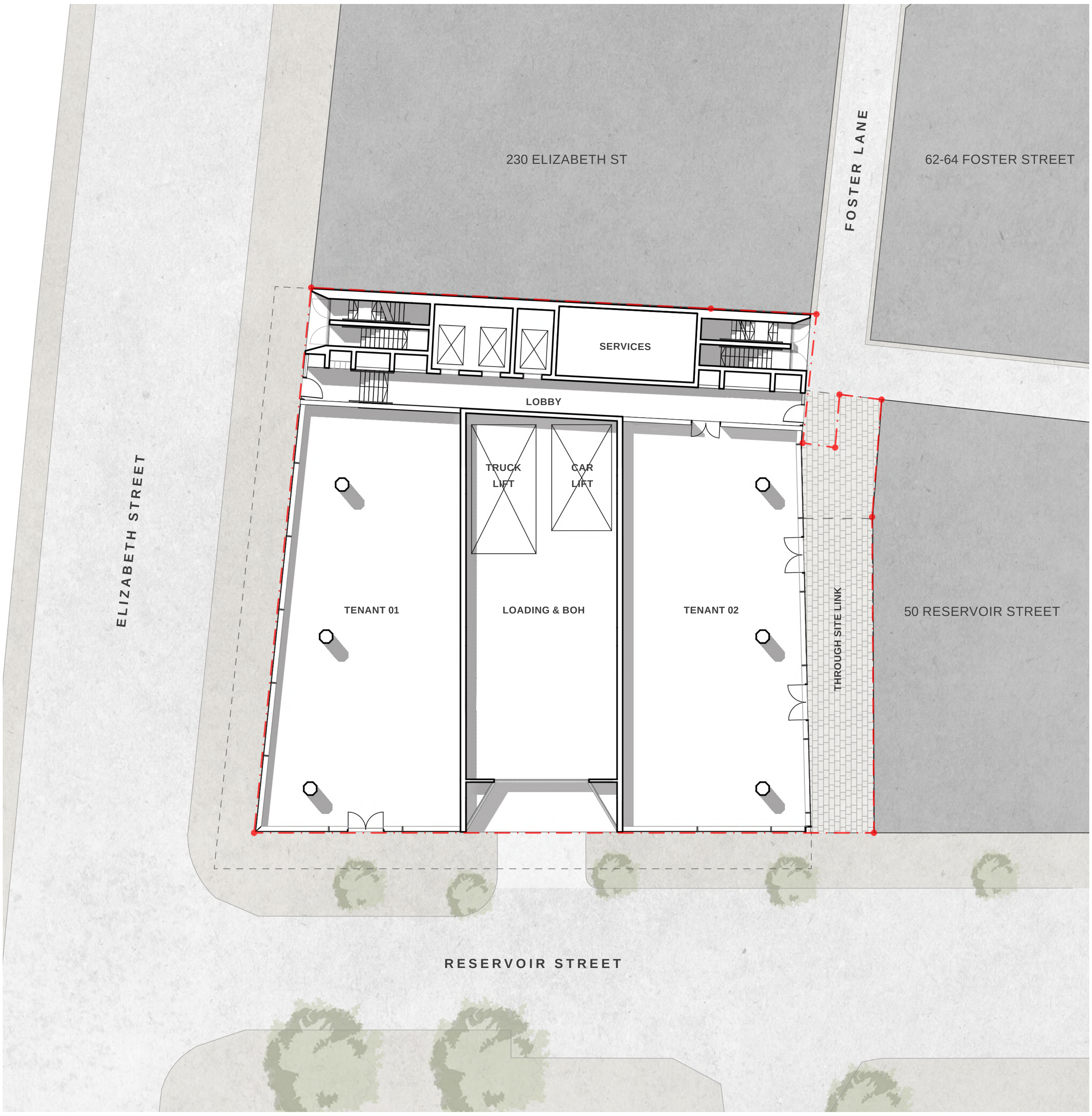


BASEMENT 2-3 PLAN

CAR PARKS PER LVL	9
MOTORCYCLE PARKS PER LVL	1

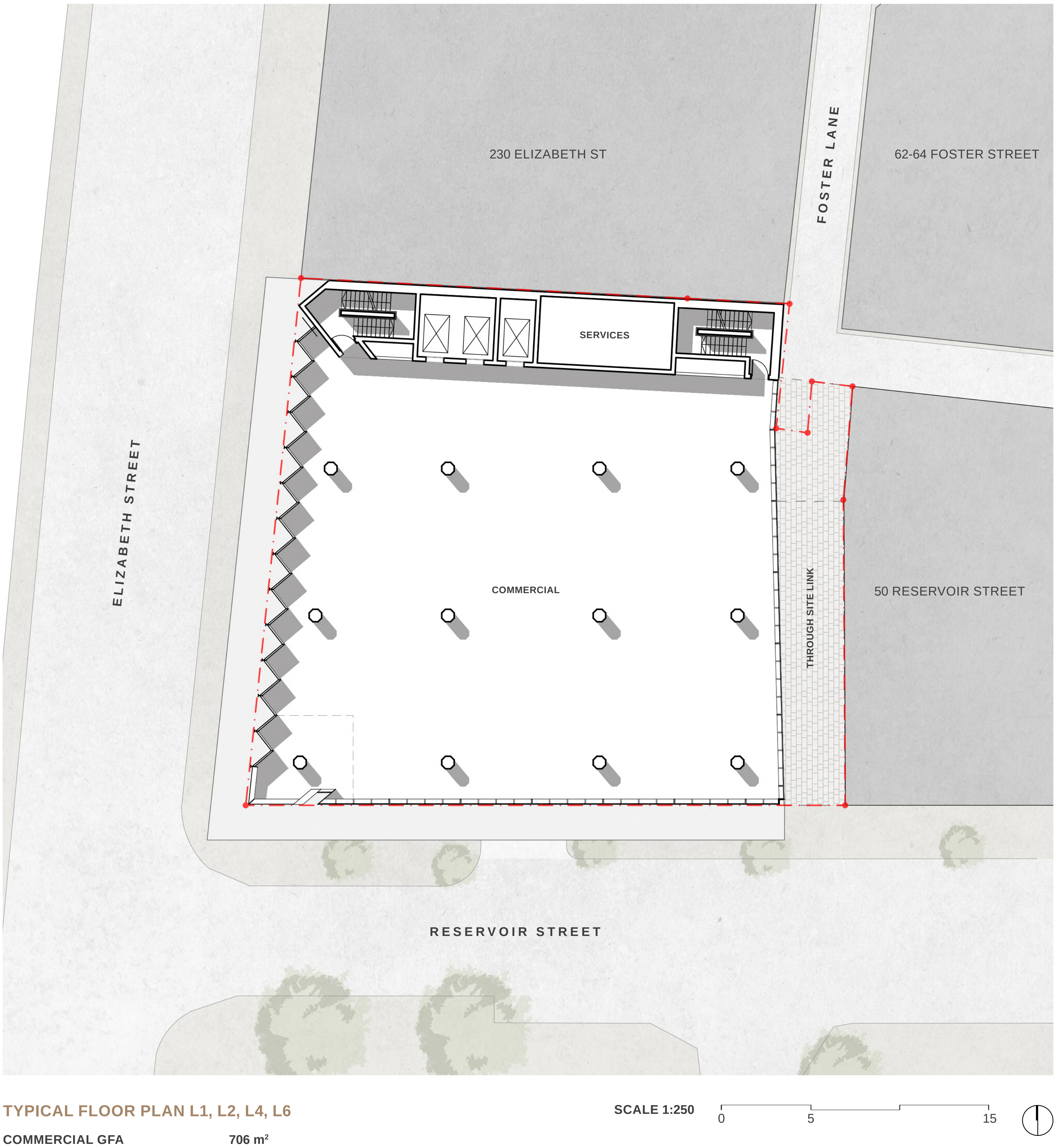
SCALE 1:250





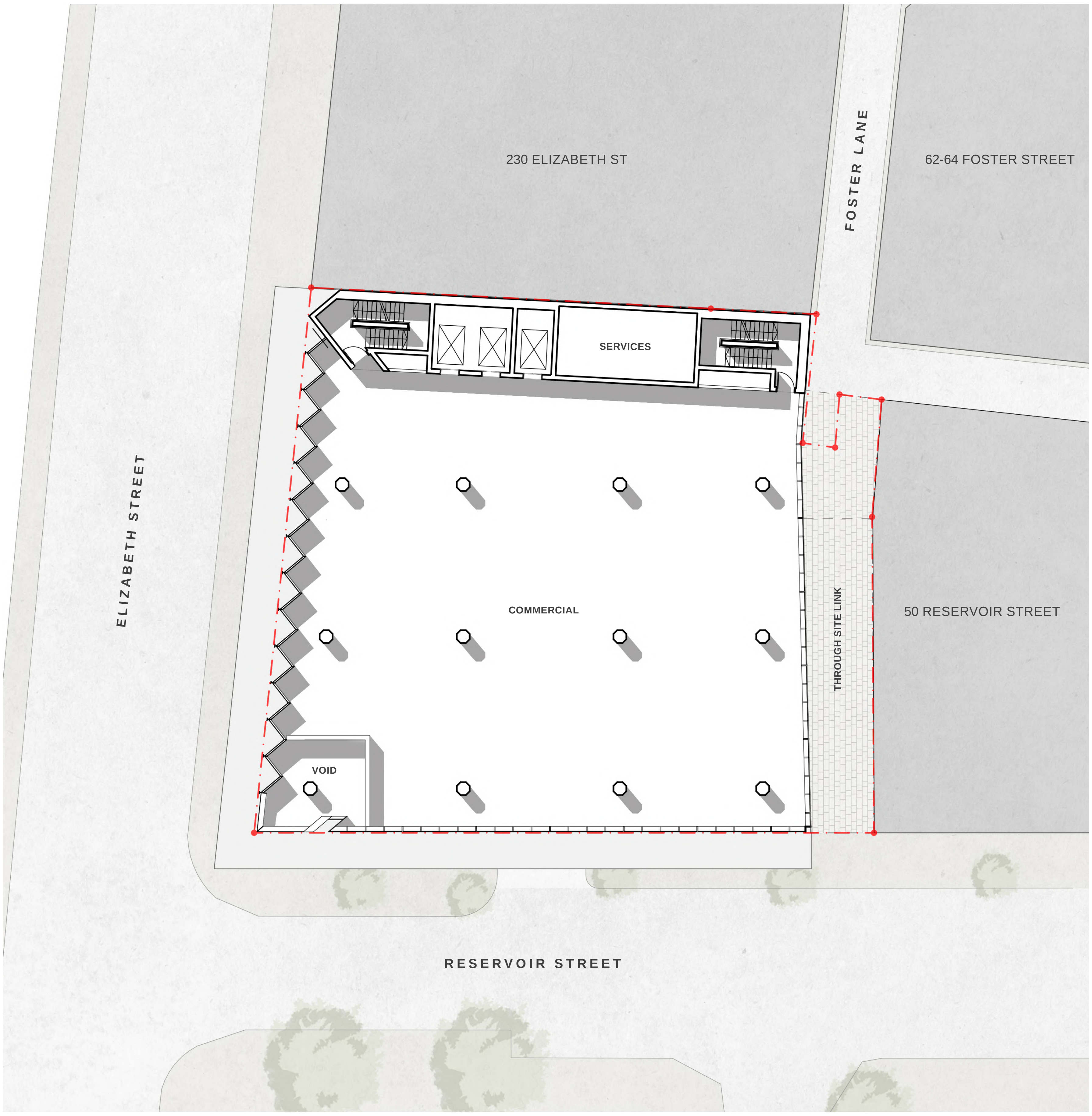
GROUND FLOOR PLAN

COMMERCIAL GFA	**79 m^2**
RETAIL GFA	**452 m^2**



TYPICAL FLOOR PLAN L1, L2, L4, L6

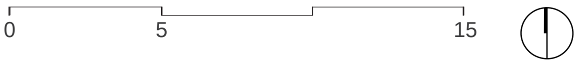
COMMERCIAL GFA	706 m²

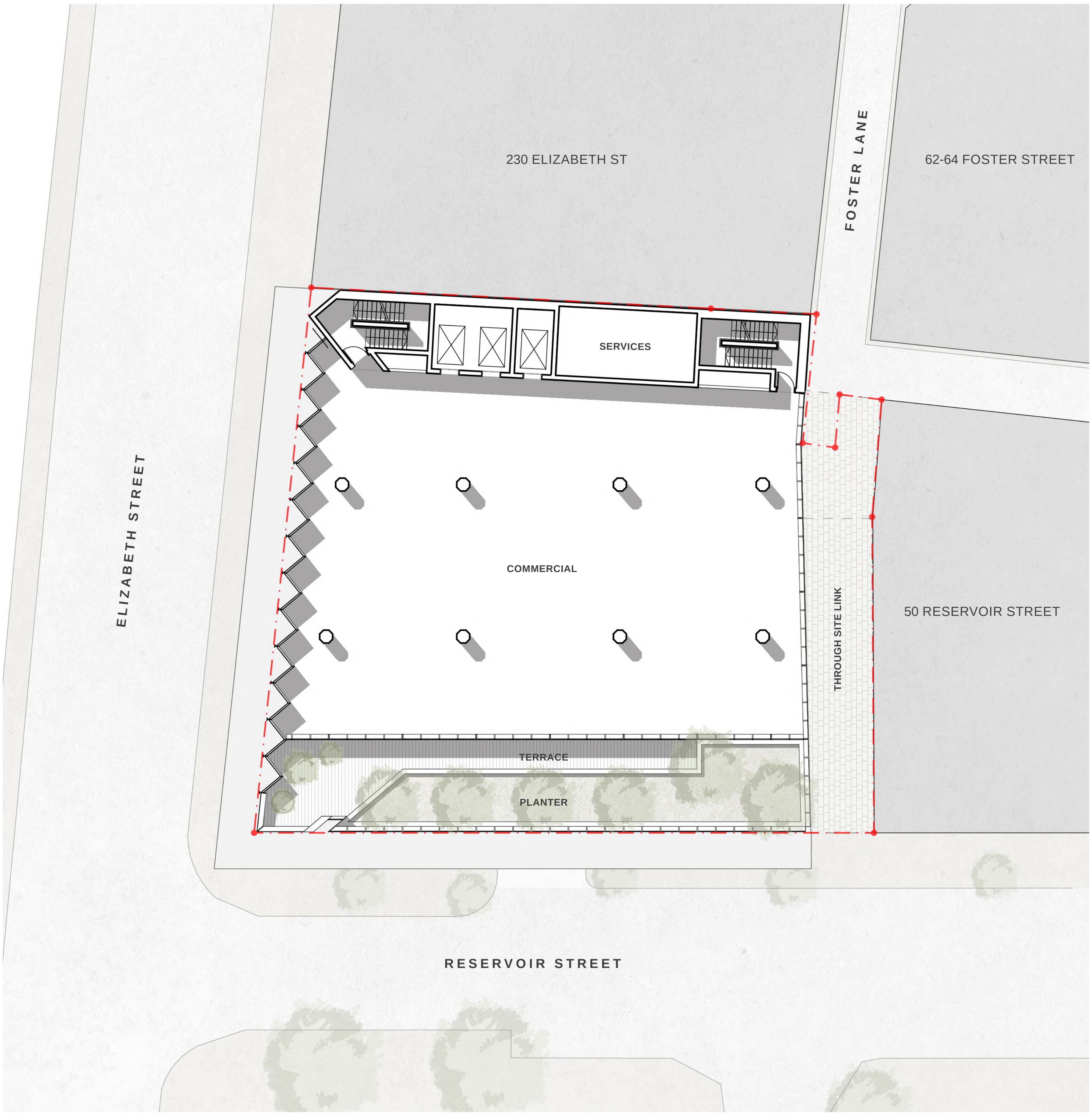


TYPICAL FLOOR PLAN L3, L5, L7

COMMERCIAL GFA	**681 m²**

SCALE 1:250

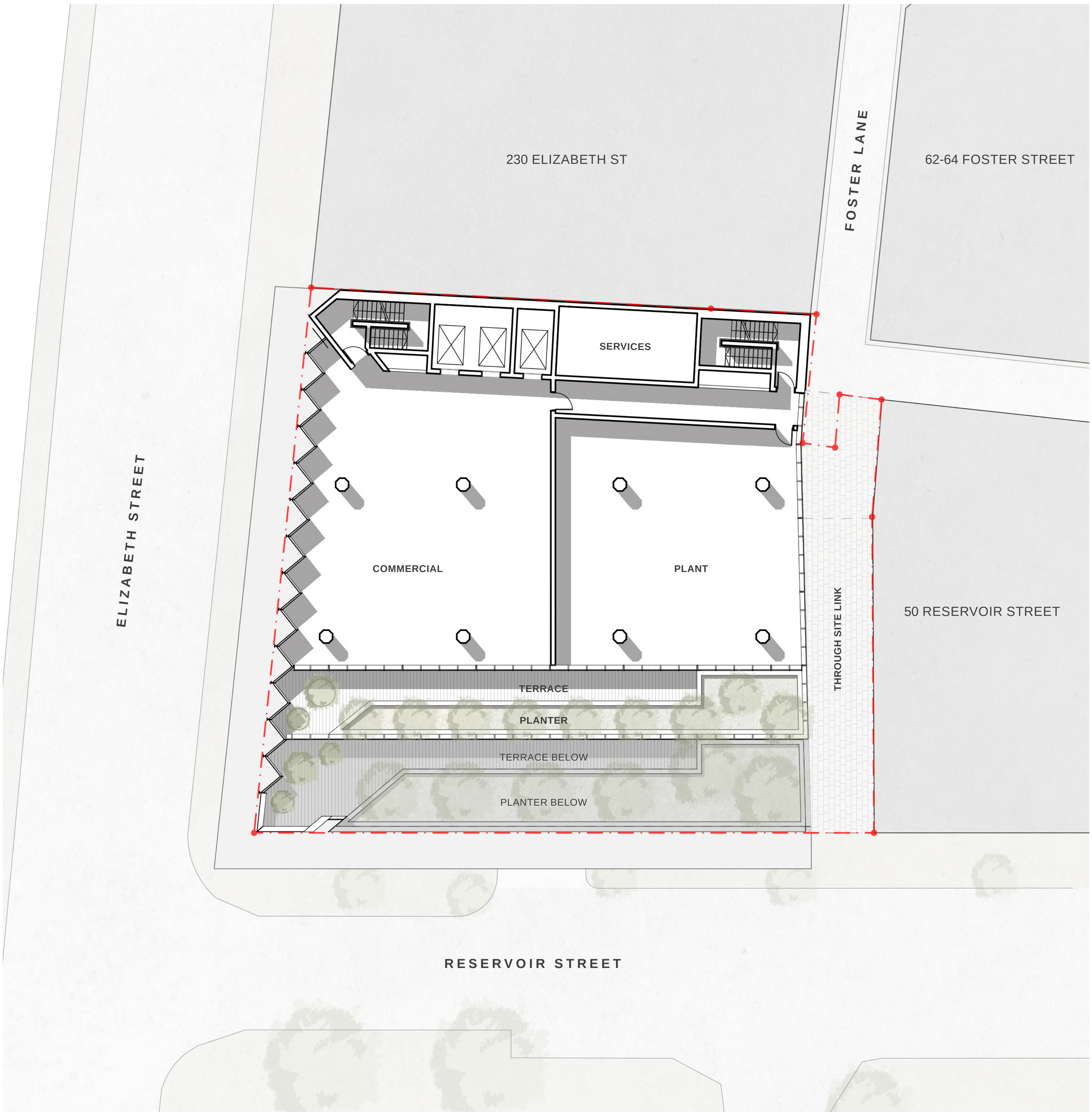




FLOOR PLAN - L8

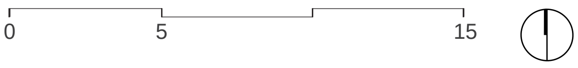
SCALE 1:250

COMMERCIAL GFA	561 m²
TERRACE	61 m²
PLANTER	78 m²

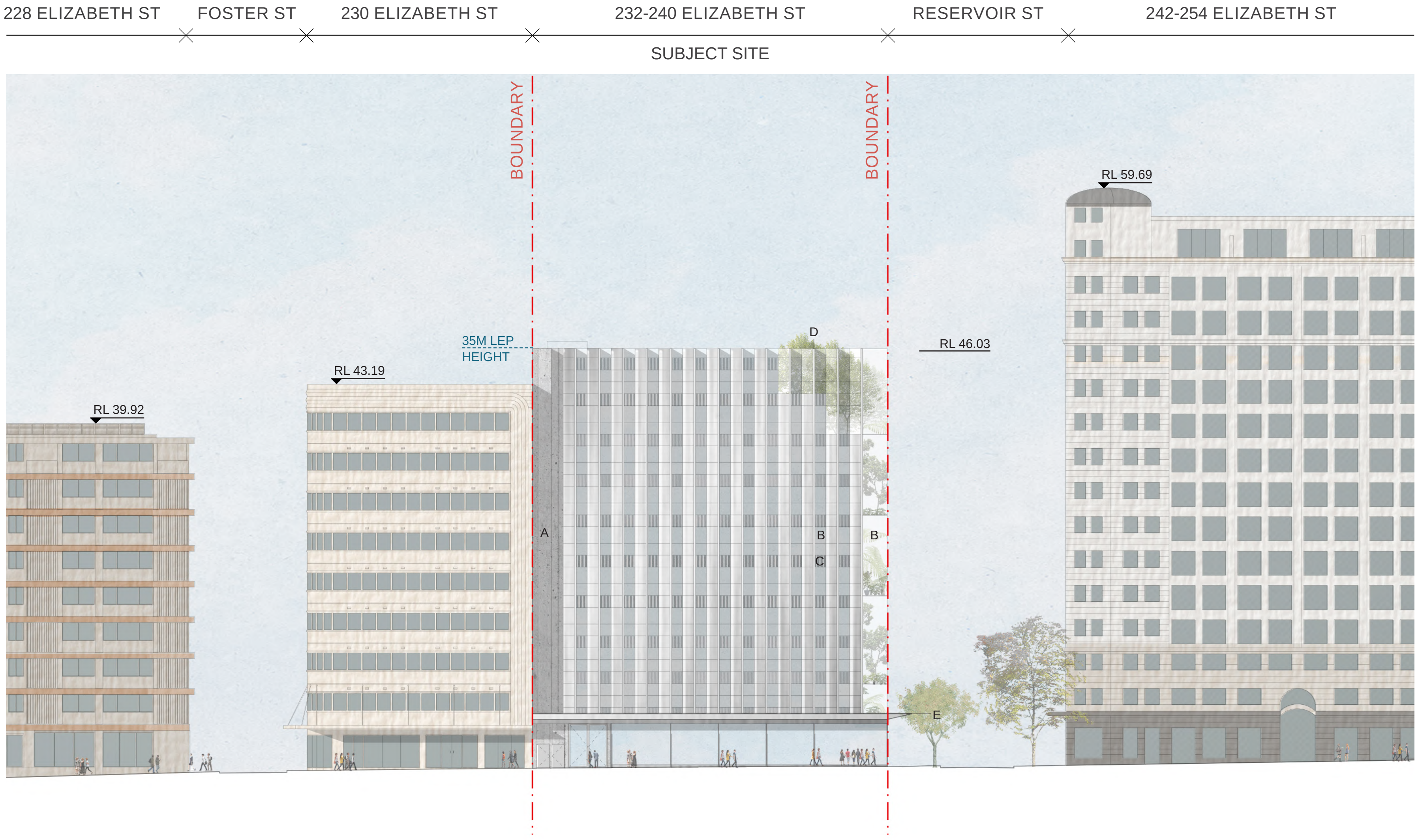


FLOOR PLAN - L9

SCALE 1:250

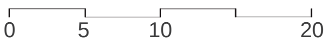


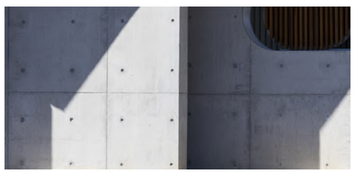
COMMERCIAL GFA	**380 m²**
TERRACE	**44 m²**
PLANTER	**58 m²**
PLANT	**66 m²**



WEST ELEVATION - ELIZABETH STREET

SCALE 1:500





A

Off-form Concrete

President Avenue Apartments,
Candalepas Associates

B

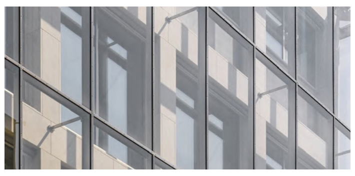
Glass

Cullen Aalhuitzen House,
Candalepas Associates

C

Reeded Glass

Plain English Design



D

Stainless Steel

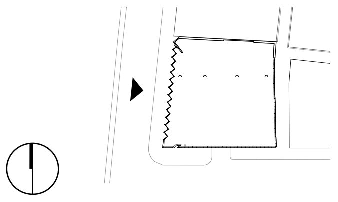
Virchow 6,
Álvaro Siza Vieira

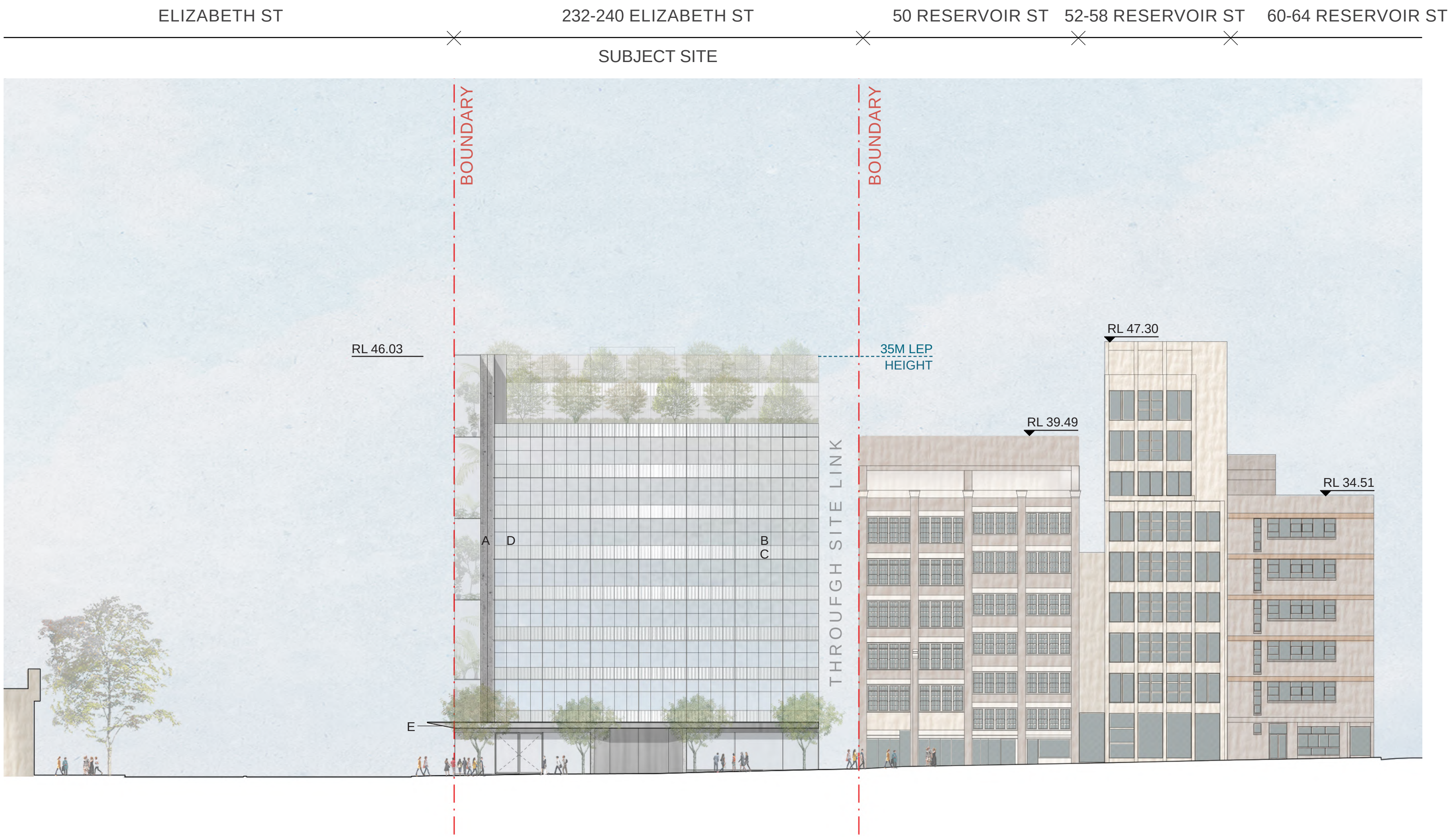
E

Dark Painted Steel

QT Hotel, Melbourne,
Candalepas Associates

MATERIALS





SOUTH ELEVATION - RESERVOIR STREET

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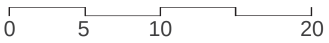

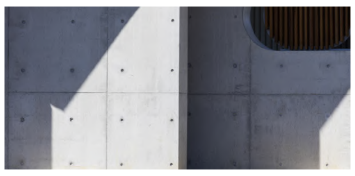

A

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President Avenue Apartments,
Candalepas Associates

B

Glass

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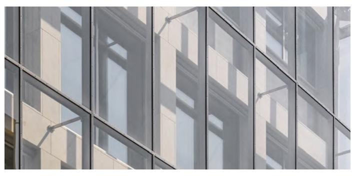
Candalepas Associates

C

Reeded Glass

Plain English Design



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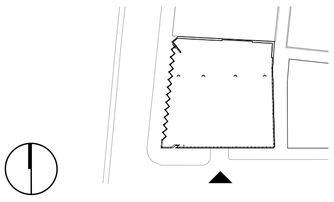
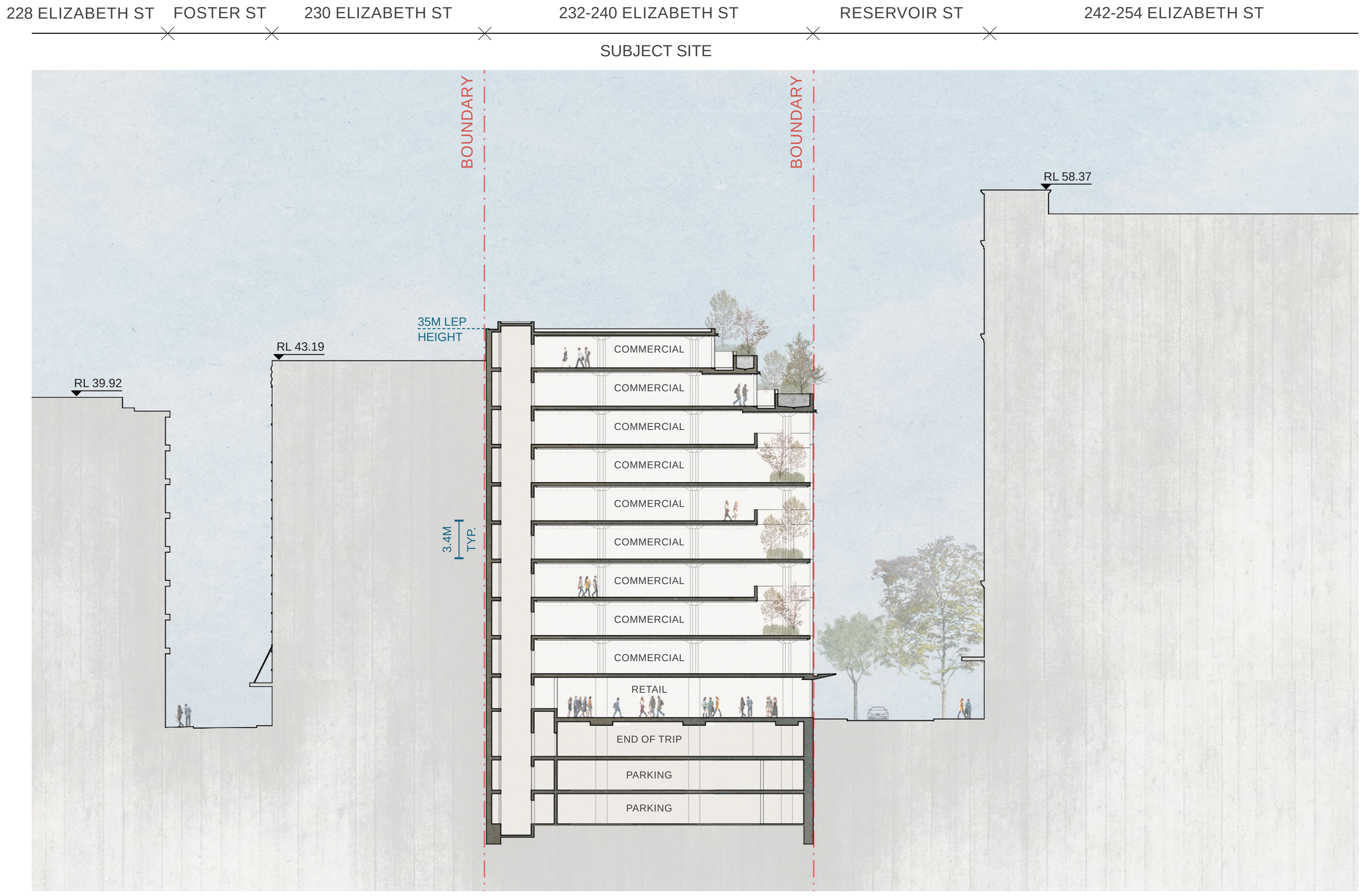
QT Hotel, Melbourne,
Candalepas Associates

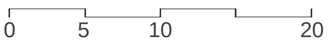
MATERIALS

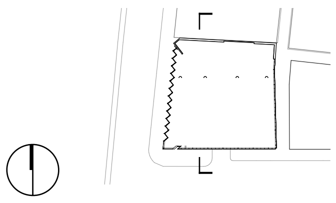


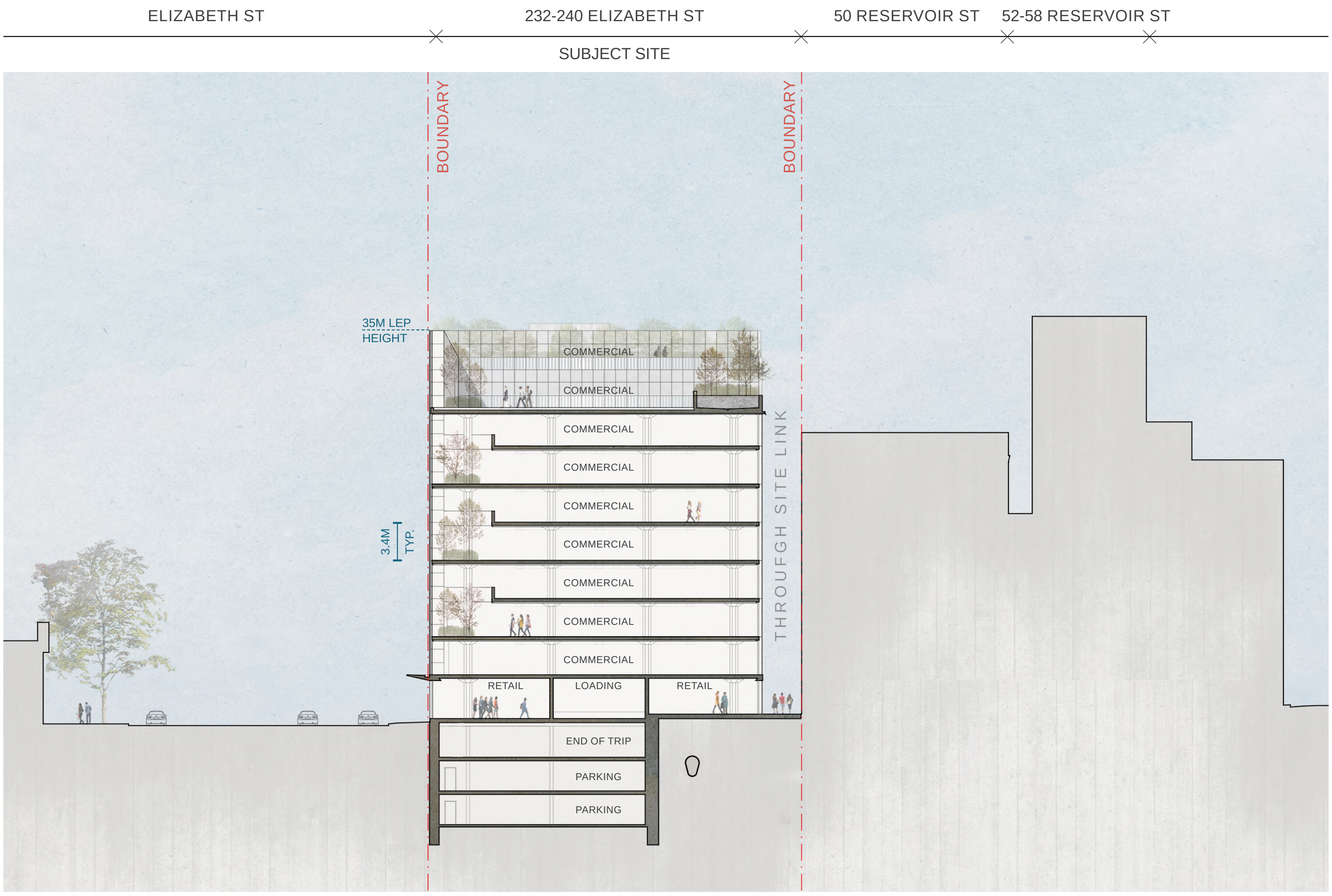


SECTION AA

SCALE 1:500







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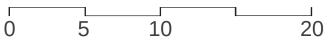
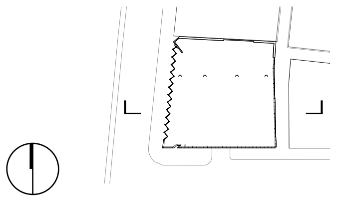
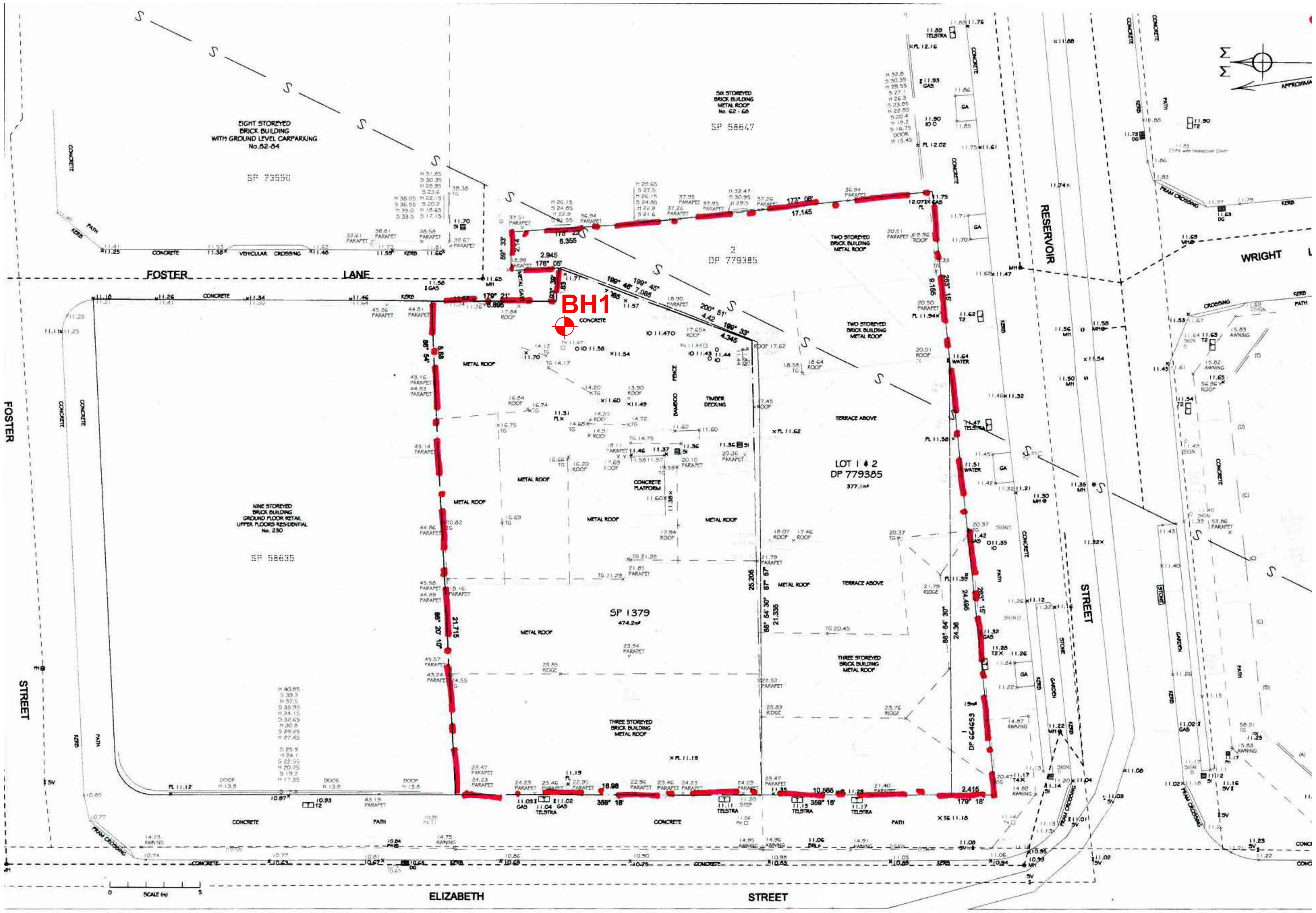
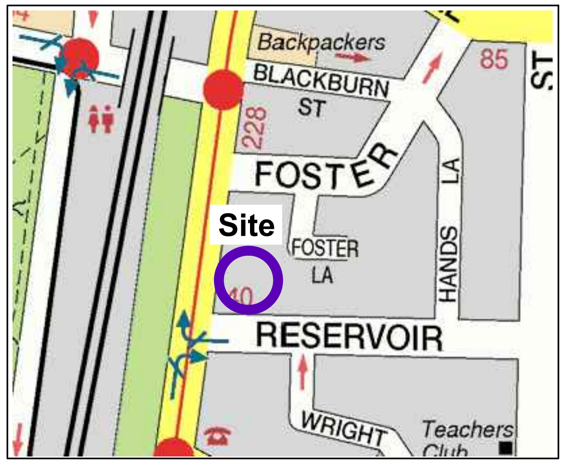




Appendix C

Previous Borehole Investigation





Locality Plan

NOTE: 1. Base drawing from Eric Scerri & Associates Pty Ltd
(Dwg 2332/14, dated 18.6.2015)
2. Test locations are approximate only and are shown with reference to existing site features.

LEGEND

⊕ Borehole location

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Patglen Pty Ltd
OFFICE: Sydney
DRAWN BY: PSCH
SCALE: 1:250 @ A3 approx.
DATE: 12.11.2015

TITLE: **Site Layout and Borehole Location**
Proposed Residential/ Commercial Development
232-240 Elizabeth Street, Surry Hills

PROJECT No: 85134.00
DRAWING No: 1
REVISION: 0

549

P:\85134.00 - SURRY HILLS - 232 - 240 Elizabeth Street\7.0 Drawings\7.2 Out\85134.00-1.dwg, 20/11/2015 12:07:43 PM

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are generally based on Australian Standard AS1726:2017, Geotechnical Site Investigations. In general, the descriptions include strength or density, colour, structure, soil or rock type and inclusions.

Soil Types

Soil types are described according to the predominant particle size, qualified by the grading of other particles present:

Type	Particle size (mm)
Boulder	>200
Cobble	63 - 200
Gravel	2.36 - 63
Sand	0.075 - 2.36
Silt	0.002 - 0.075
Clay	<0.002

The sand and gravel sizes can be further subdivided as follows:

Type	Particle size (mm)
Coarse gravel	19 - 63
Medium gravel	6.7 - 19
Fine gravel	2.36 – 6.7
Coarse sand	0.6 - 2.36
Medium sand	0.21 - 0.6
Fine sand	0.075 - 0.21

Definitions of grading terms used are:

- Well graded - a good representation of all particle sizes
- Poorly graded - an excess or deficiency of particular sizes within the specified range
- Uniformly graded - an excess of a particular particle size
- Gap graded - a deficiency of a particular particle size with the range

The proportions of secondary constituents of soils are described as follows:

In fine grained soils (>35% fines)

Term	Proportion of sand or gravel	Example
And	Specify	Clay (60%) and Sand (40%)
Adjective	>30%	Sandy Clay
With	15 – 30%	Clay with sand
Trace	0 - 15%	Clay with trace sand

In coarse grained soils (>65% coarse)
- with clays or silts

Term	Proportion of fines	Example
And	Specify	Sand (70%) and Clay (30%)
Adjective	>12%	Clayey Sand
With	5 - 12%	Sand with clay
Trace	0 - 5%	Sand with trace clay

In coarse grained soils (>65% coarse)
- with coarser fraction

Term	Proportion of coarser fraction	Example
And	Specify	Sand (60%) and Gravel (40%)
Adjective	>30%	Gravelly Sand
With	15 - 30%	Sand with gravel
Trace	0 - 15%	Sand with trace gravel

The presence of cobbles and boulders shall be specifically noted by beginning the description with ‘Mix of Soil and Cobbles/Boulders’ with the word order indicating the dominant first and the proportion of cobbles and boulders described together.

Soil Descriptions

Cohesive Soils

Cohesive soils, such as clays, are classified on the basis of undrained shear strength. The strength may be measured by laboratory testing, or estimated by field tests or engineering examination. The strength terms are defined as follows:

Description	Abbreviation	Undrained shear strength (kPa)
Very soft	VS	<12
Soft	S	12 - 25
Firm	F	25 - 50
Stiff	St	50 - 100
Very stiff	VSt	100 - 200
Hard	H	>200
Friable	Fr	-

Cohesionless Soils

Cohesionless soils, such as clean sands, are classified on the basis of relative density, generally from the results of standard penetration tests (SPT), cone penetration tests (CPT) or dynamic penetrometers (PSP). The relative density terms are given below:

Relative Density	Abbreviation	Density Index (%)
Very loose	VL	<15
Loose	L	15-35
Medium dense	MD	35-65
Dense	D	65-85
Very dense	VD	>85

Soil Origin

It is often difficult to accurately determine the origin of a soil. Soils can generally be classified as:

- Residual soil - derived from in-situ weathering of the underlying rock;
- Extremely weathered material – formed from in-situ weathering of geological formations. Has soil strength but retains the structure or fabric of the parent rock;
- Alluvial soil – deposited by streams and rivers;
- Estuarine soil – deposited in coastal estuaries;
- Marine soil – deposited in a marine environment;
- Lacustrine soil – deposited in freshwater lakes;
- Aeolian soil – carried and deposited by wind;
- Colluvial soil – soil and rock debris transported down slopes by gravity;
- Topsoil – mantle of surface soil, often with high levels of organic material.
- Fill – any material which has been moved by man.

Moisture Condition – Coarse Grained Soils

For coarse grained soils the moisture condition should be described by appearance and feel using the following terms:

Dry (D)	Non-cohesive and free-running.
Moist (M)	Soil feels cool, darkened in colour. Soil tends to stick together. Sand forms weak ball but breaks easily.
Wet (W)	Soil feels cool, darkened in colour. Soil tends to stick together, free water forms when handling.

Moisture Condition – Fine Grained Soils

For fine grained soils the assessment of moisture content is relative to their plastic limit or liquid limit, as follows:

- ‘Moist, dry of plastic limit’ or ‘w <PL’ (i.e. hard and friable or powdery).
- ‘Moist, near plastic limit’ or ‘w ≈ PL (i.e. soil can be moulded at moisture content approximately equal to the plastic limit).
- ‘Moist, wet of plastic limit’ or ‘w >PL’ (i.e. soils usually weakened and free water forms on the hands when handling).
- ‘Wet’ or ‘w ≈LL’ (i.e. near the liquid limit).
- ‘Wet’ or ‘w >LL’ (i.e. wet of the liquid limit).

Rock Descriptions

Rock Strength

Rock strength is defined by the Unconfined Compressive Strength and it refers to the strength of the rock substance and not the strength of the overall rock mass, which may be considerably weaker due to defects.

The Point Load Strength Index Is$_{(50)}$ is commonly used to provide an estimate of the rock strength and site specific correlations should be developed to allow UCS values to be determined. The point load strength test procedure is described by Australian Standard AS4133.4.1-2007. The terms used to describe rock strength are as follows:

Strength Term	Abbreviation	Unconfined Compressive Strength MPa	Point Load Index * Is$_{(50)}$ MPa
Very low	VL	0.6 - 2	0.03 - 0.1
Low	L	2 - 6	0.1 - 0.3
Medium	M	6 - 20	0.3 - 1.0
High	H	20 - 60	1 - 3
Very high	VH	60 - 200	3 - 10
Extremely high	EH	>200	>10

* Assumes a ratio of 20:1 for UCS to Is$_{(50)}$. It should be noted that the UCS to Is$_{(50)}$ ratio varies significantly for different rock types and specific ratios should be determined for each site.

Degree of Weathering

The degree of weathering of rock is classified as follows:

Term	Abbreviation	Description
Residual Soil	RS	Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.
Extremely weathered	XW	Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible
Highly weathered	HW	The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Moderately weathered	MW	The whole of the rock material is discoloured , usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.
Slightly weathered	SW	Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.
Fresh	FR	No signs of decomposition or staining.
Note: If HW and MW cannot be differentiated use DW (see below)		
Distinctly weathered	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching or may be decreased due to deposition of weathered products in pores.

Rock Descriptions

Degree of Fracturing

The following classification applies to the spacing of natural fractures in diamond drill cores. It includes bedding plane partings, joints and other defects, but excludes drilling breaks.

Term	Description
Fragmented	Fragments of <20 mm
Highly Fractured	Core lengths of 20-40 mm with occasional fragments
Fractured	Core lengths of 30-100 mm with occasional shorter and longer sections
Slightly Fractured	Core lengths of 300 mm or longer with occasional sections of 100-300 mm
Unbroken	Core contains very few fractures

Rock Quality Designation

The quality of the cored rock can be measured using the Rock Quality Designation (RQD) index, defined as:

$$\text{RQD \%} = \frac{\text{cumulative length of 'sound' core sections} \geq 100 \text{ mm long}}{\text{total drilled length of section being assessed}}$$

where 'sound' rock is assessed to be rock of low strength or stronger. The RQD applies only to natural fractures. If the core is broken by drilling or handling (i.e. drilling breaks) then the broken pieces are fitted back together and are not included in the calculation of RQD.

Stratification Spacing

For sedimentary rocks the following terms may be used to describe the spacing of bedding partings:

Term	Separation of Stratification Planes
Thinly laminated	< 6 mm
Laminated	6 mm to 20 mm
Very thinly bedded	20 mm to 60 mm
Thinly bedded	60 mm to 0.2 m
Medium bedded	0.2 m to 0.6 m
Thickly bedded	0.6 m to 2 m
Very thickly bedded	> 2 m

Symbols & Abbreviations

Douglas Partners

Introduction

These notes summarise abbreviations commonly used on borehole logs and test pit reports.

Drilling or Excavation Methods

C	Core drilling
R	Rotary drilling
SFA	Spiral flight augers
NMLC	Diamond core - 52 mm dia
NQ	Diamond core - 47 mm dia
HQ	Diamond core - 63 mm dia
PQ	Diamond core - 81 mm dia

Water

▷	Water seep
▽	Water level

Sampling and Testing

A	Auger sample
B	Bulk sample
D	Disturbed sample
E	Environmental sample
U_{50}	Undisturbed tube sample (50mm)
W	Water sample
pp	Pocket penetrometer (kPa)
PID	Photo ionisation detector
PL	Point load strength Is(50) MPa
S	Standard Penetration Test
V	Shear vane (kPa)

Description of Defects in Rock

The abbreviated descriptions of the defects should be in the following order: Depth, Type, Orientation, Coating, Shape, Roughness and Other. Drilling and handling breaks are not usually included on the logs.

Defect Type

B	Bedding plane
Cs	Clay seam
Cv	Cleavage
Cz	Crushed zone
Ds	Decomposed seam
F	Fault
J	Joint
Lam	Lamination
Pt	Parting
Sz	Sheared Zone
V	Vein

Orientation

The inclination of defects is always measured from the perpendicular to the core axis.

h	horizontal
v	vertical
sh	sub-horizontal
sv	sub-vertical

Coating or Infilling Term

cln	clean
co	coating
he	healed
inf	infilled
stn	stained
ti	tight
vn	veneer

Coating Descriptor

ca	calcite
cbs	carbonaceous
cly	clay
fe	iron oxide
mn	manganese
slt	silty

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

po	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

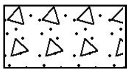
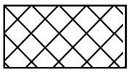
Other

fg	fragmented
bnd	band
qtz	quartz

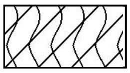
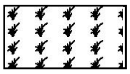
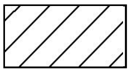
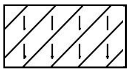
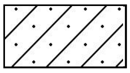
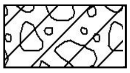
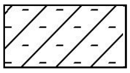
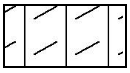
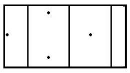
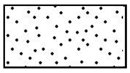
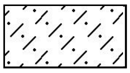
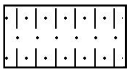
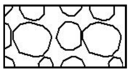
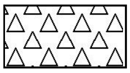
Symbols & Abbreviations

Graphic Symbols for Soil and Rock

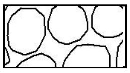
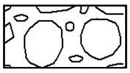
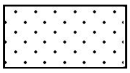
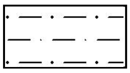
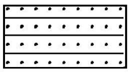
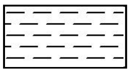
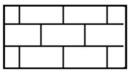
General

	Asphalt
	Road base
	Concrete
	Filling

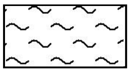
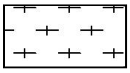
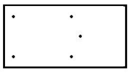
Soils

	Topsoil
	Peat
	Clay
	Silty clay
	Sandy clay
	Gravelly clay
	Shaly clay
	Silt
	Clayey silt
	Sandy silt
	Sand
	Clayey sand
	Silty sand
	Gravel
	Sandy gravel
	Cobbles, boulders
	Talus

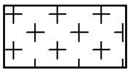
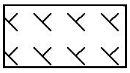
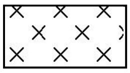
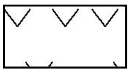
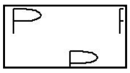
Sedimentary Rocks

	Boulder conglomerate
	Conglomerate
	Conglomeratic sandstone
	Sandstone
	Siltstone
	Laminite
	Mudstone, claystone, shale
	Coal
	Limestone

Metamorphic Rocks

	Slate, phyllite, schist
	Gneiss
	Quartzite

Igneous Rocks

	Granite
	Dolerite, basalt, andesite
	Dacite, epidote
	Tuff, breccia
	Porphyry

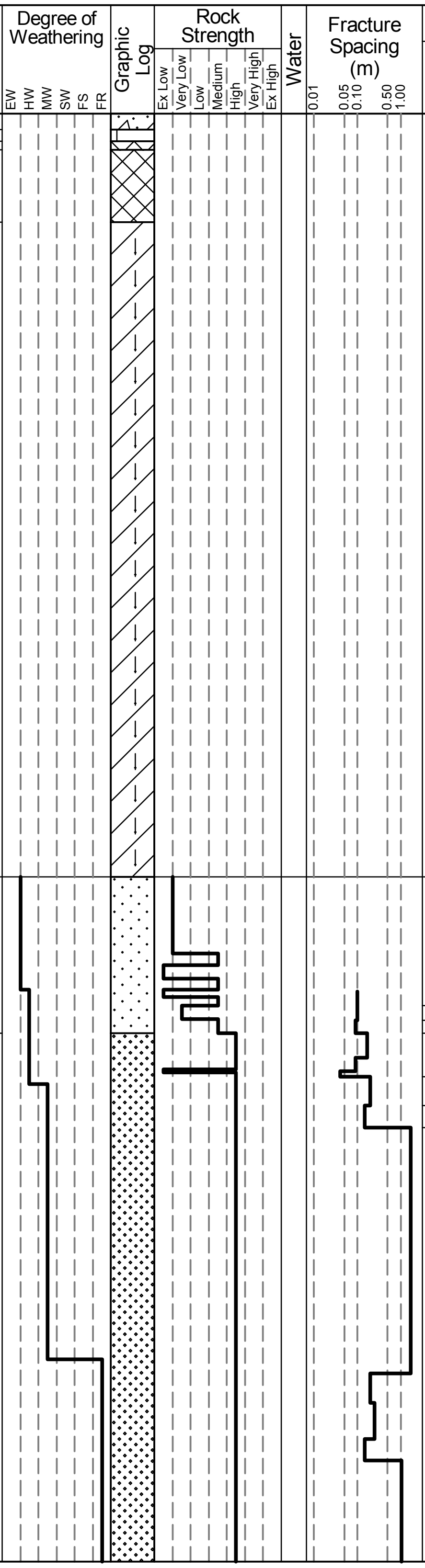
BOREHOLE LOG

CLIENT: Potglen Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 232-240 Elizabeth St & 2-8 Campbell Pde
Surry Hills

SURFACE LEVEL: 11.6 AHD*
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 85134
DATE: 22/10/2015
SHEET 1 OF 2

Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
0.11	CONCRETE					
0.19	CERAMIC PAVING		A			
0.25	FILLING - yellow-brown, fine to medium sand filling		A			
0.75	FILLING - brown-grey and red-brown, silty clay filling with some sand and sandstone fragments					
	SILTY CLAY - stiff, grey mottled brown and red-brown, silty clay with a trace of ironstone gravel, MC>>apparently		S			3,4,6 N = 10
	- ironstone gravel bands from 2.5m		S			5,5,8 N = 13
		Note: Unless otherwise stated, rock is fractured along rough planar bedding dipping 0°- 10°	S			5,9,11 N = 20
5.27	SANDSTONE - extremely low to very low strength, extremely to highly weathered, pale grey-brown to red-brown, fine to medium grained sandstone with some medium strength iron-cemented bands		C	100	0	PL(A) = 0.1
6.35	SANDSTONE - high and medium to high strength, moderately weathered then fresh, slightly fractured and unbroken, brown then pale grey, medium to coarse grained sandstone	6.06-6.1m: Cs 6.16-6.2m: Cs 6.26m: B10°, cly vn/ti 6.35 & 6.52m:B15°, fe 6.61m: B15°, fe, cly, 10mm 6.65-6.76m: J80°, un, ro, fe 6.85m: B10°, fe 7m: B0°, cly, 5mm	C	100	97	PL(A) = 1.4 PL(A) = 2.1 PL(A) = 2.5
		8.7 & 8.9m: B5°, cly vn				PL(A) = 2.3
		9.15 & 9.3m: B5°, cly vn, ti	C	100	100	PL(A) = 1.8



RIG: Tightsite **DRILLER:** ID **LOGGED:** JS/SI **CASING:** HW to 4.0m
TYPE OF BORING: Solid flight auger to 4.76m; Rotary to 6.27m; NMLC-Coring to 11.52m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: Standpipe installed to 11.52m. *Surface RL determined using standard levelling techniques

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

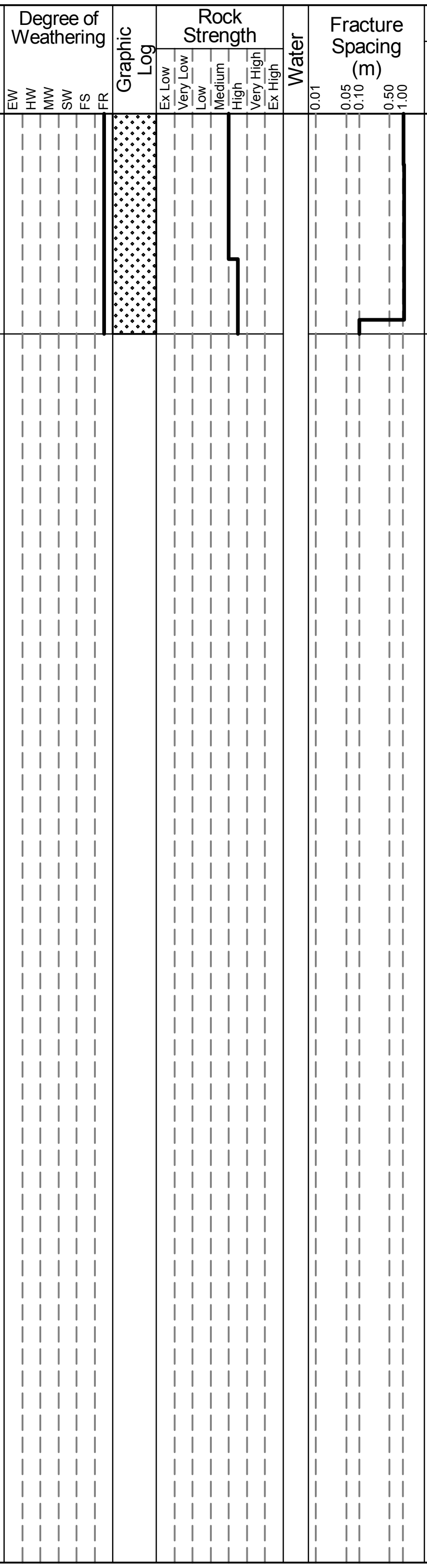
Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Potglen Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 232-240 Elizabeth St & 2-8 Campbell Pde
Surry Hills

SURFACE LEVEL: 11.6 AHD*
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 85134
DATE: 22/10/2015
SHEET 2 OF 2



RL	Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
1	11	SANDSTONE - high and medium to high strength, moderately weathered then fresh, slightly fractured and unbroken, brown then pale grey, medium to coarse grained sandstone *(continued)*	10.35 & 11.42m: B0°, cly vn	C	100	100	PL(A) = 1 PL(A) = 1.3
0	11.52	Bore discontinued at 11.52m					
	12						
-1	13						
-2	14						
-3	15						
-4	16						
-5	17						
-6	18						
-7	19						
-8							

RIG: Tightsite **DRILLER:** ID **LOGGED:** JS/SI **CASING:** HW to 4.0m

TYPE OF BORING: Solid flight auger to 4.76m; Rotary to 6.27m; NMLC-Coring to 11.52m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Standpipe installed to 11.52m. *Surface RL determined using standard levelling techniques

SAMPLING & IN SITU TESTING LEGEND

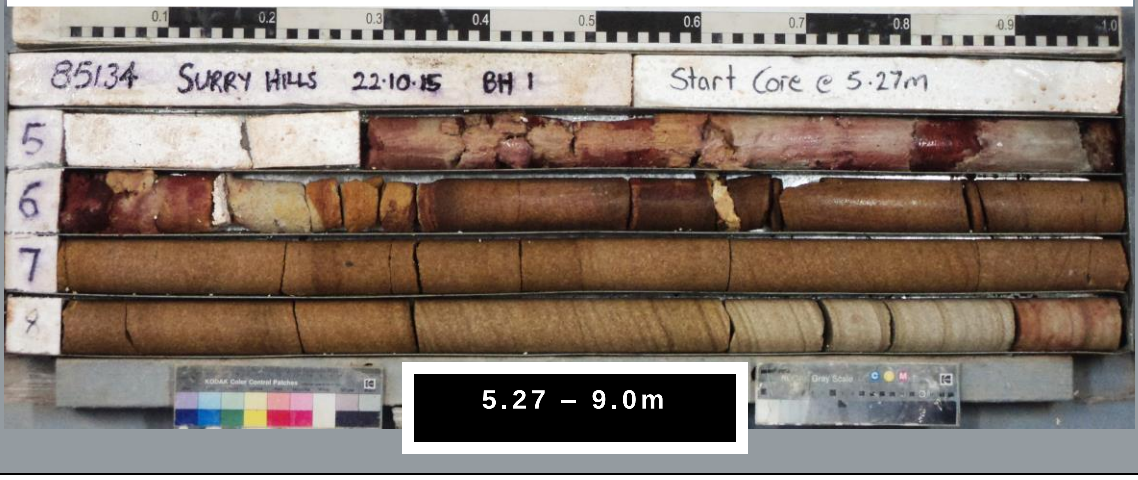
A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

Douglas Partners
Geotechnics / Environment / Groundwater

DOUGLAS PARTNERS PTY LTD

PROPOSED RESIDENTIAL DEVELOPMENT – SURRY HILL

BORE 1 PROJECT 85134 OCT 2015



5.27 – 9.0m

DOUGLAS PARTNERS PTY LTD

PROPOSED RESIDENTIAL DEVELOPMENT – SURRY HILL

BORE 1 PROJECT 85134 OCT 2015

9.0 – 11.52m

Appendix D

Permeability Test Result from Previous Investigation

Douglas Partners
Geotechnics / Environment / Groundwater

Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au
96 Hermitage Road
West Ryde NSW 2114
PO Box 472
West Ryde NSW 1685
Phone (02) 9809 0666
Fax (02) 9809 4095

Permeability Testing - Simple Slug Test Report

Client:	Patglen Pty Ltd and The Summit Hotel Bondi Beach	Project No:	85134
Project:		Date:	30-Oct-15
Location:	232-240 Elizabeth Street, Surry Hills	Tested by:	JS

Test Location		**Test No.**	1	
Description:	Basement	Easting:		m
Material type:	clay over sandstone	Northing		m
		Surface Level:	11.6	m AHD

Details of Well Installation

Well casing diameter	60	mm	Depth to water before test	1.2	m
Well screen diameter	60	mm	Depth to water at start of test	9.6	m
Length of well screen	3	m			

Test Results

Time (min)	Depth (m)	Change in Head δH (m)	δH/Ho
0.0	0.00	8.4	1.000
1.0	0.2	8.2	0.976
2	0.4	8	0.952
3	0.6	7.8	0.929
5	0.9	7.5	0.893
10	1.6	6.8	0.810
20	2.8	5.6	0.667
35	4.2	4.2	0.500
50	4.9	3.5	0.417
60	5.3	3.1	0.369
90	5.9	2.5	0.298
120	6.1	2.3	0.274
			0.000
			0.000
			0.000
			0.000
			0.000
			0.000
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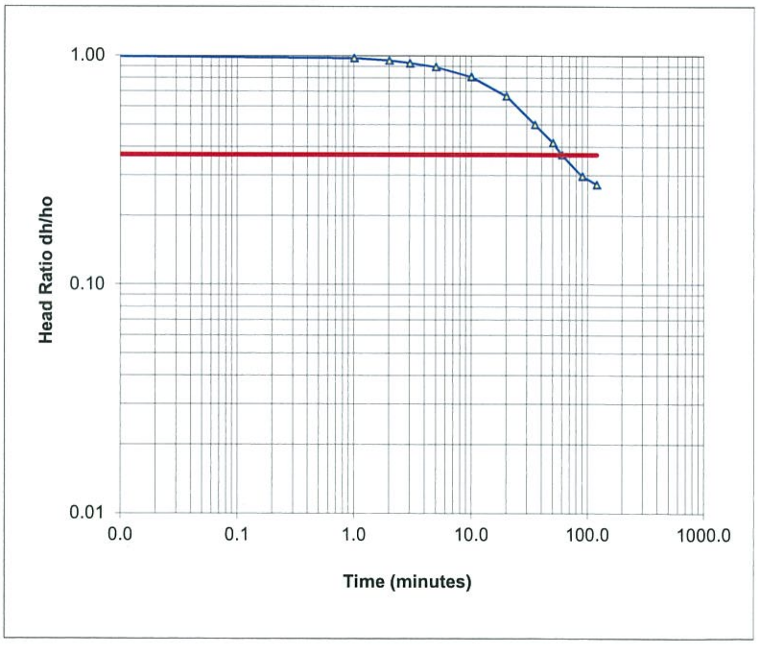
			0.000
			0.000
			0.000



To = 50 mins
3000 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity k = **2.3E-07** m/sec
= **0.083** cm/hour