ENGEO Limited
8 Greydene Place, Takapuna, Auckland 0622
PO Box 33-1527, Takapuna, Auckland 0740
T: +64 9 972 2205
www.engeo.co.nz

Project Number #20034.000.001

Geotechnical Investigation

617 New North Road, Kingsland, Auckland

Submitted to:
Pink Beluga Civil Limited
76 Gladstone Road
Parnell
Auckland 1052

Contents

Tables

Figures

Appendices

ENGEO Document Control:

Report Title	Geotechnical Investigation - 617 New North Road, Kingsland		
Project No.	20034.000.001	**Doc ID**	02
Client	Pink Beluga Civil Limited	**Client Contact**	Rory O'Connor
Distribution (PDF)	Rory O'Connor (Pink Beluga Civil Limited)		

Date	**Revision Details / Status**	**Author**	**Reviewer**	**WP**
09/05/2022	Issued to Client	HF	PF	VB

1 Introduction

ENGEO Limited was requested by Pink Beluga Civil Limited to undertake a geotechnical investigation of the property at 617 New North Road, Kingsland, Auckland (herein referred to as 'the site'). This work has been carried out in accordance with our signed agreement dated 16 February 2022.

1.1 Scope of work

The purpose of this report is to characterise underlying ground conditions to inform design of the proposed apartment and retail building, and also to support an application for resource consent.

The scope of work for this report included:

- Review of published geological and geotechnical information relevant to the site.
- A site walkover and geomorphological assessment by an experienced geotechnical professional.
- Completion of three hand auger boreholes across the site to establish a geological model, including assessment of geotechnical parameters for the soils encountered.
- Coordination of a machine borehole drilling contractor to drill two machine boreholes within the proposed building platform.
    - Installation of a piezometer and continuous groundwater data logger in one of the machine boreholes. Recording of groundwater levels over a minimum monitoring period of two weeks to support the ground model and to inform the assessment against the Auckland Unitary Plan (AUP).
- Preparation of a conceptual geological ground site model including one geological cross section through the site.
- Preparation of this report presenting the findings of the investigation, including geotechnical recommendations to inform design of foundation, earthworks and retaining wall design for the proposed development, and to support an application for resource consent.

Our scope of work does not include the following items:

- Specific geotechnical design of foundation or retaining solutions (including PS1);
- Production of any technical specifications or design drawings;
- Attendance at any project / construction meetings; and / or
- Geotechnical testing and observation during construction phase (including PS4).

These services may be required as the project progresses and if requested, we are happy to provide these services as part of a separate agreement.

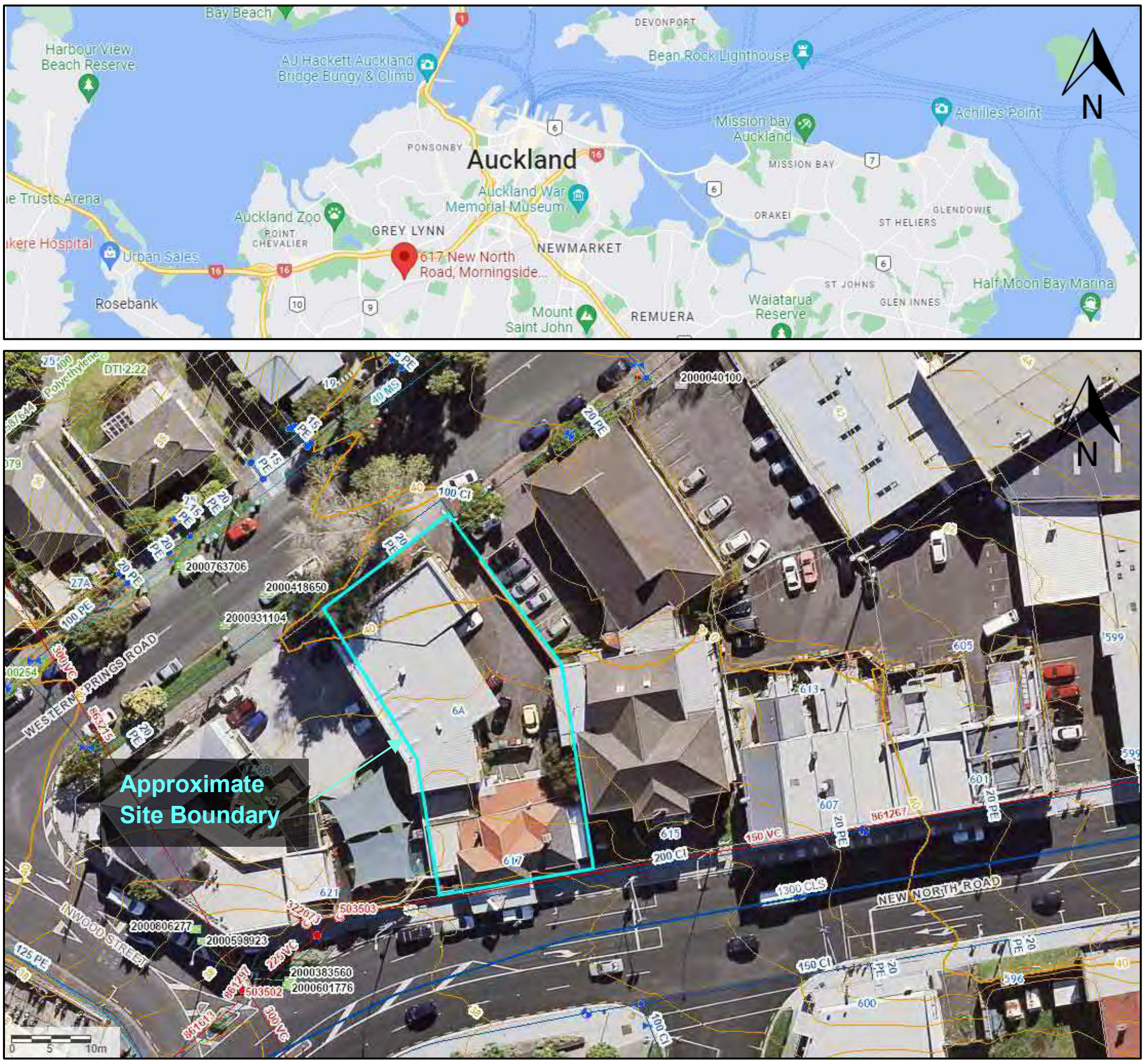
2 Site Description

The site at 617 New North Road, Kingsland covers approximately 943 m² (legal description Lot 2 DP 72255), on a generally flat to gently sloping south-facing site. The site is currently developed with a single level dwelling which has been converted into a café, currently accessed from New North Road, and a single-story office building and carpark accessed from Western Springs Road to the north. The carpark level and Western Springs Road is formed at approximately RL 40 m. The café entrance on the south side of the site is formed at approximately RL 38 m.

Commercial property bounds the site to the east and west, Western Springs Road to the north, and New North Road to the south.

On the Auckland Council GeoMaps portal, an existing gas line, a 150 mm diameter vitreous clay sanitary sewer and a 200 mm diameter water line run parallel to the southern boundary, and a 100 mm diameter water line runs parallel to the northern boundary. No public service lines are shown crossing the site.

Figure 1: Site Location Plan



Note: Images sourced from Google Maps and Auckland GIS. Not to scale.

3 Proposed Development

We have been provided with the Matz Architects Limited Conceptual plans for the site dated 6 April 2022 (ref. 1309). These plans depict a proposal to construct a new eight-storey building, including a retail area on the ground floor on the southern side of the site, and a single level basement with car stacking pits covering the remainder of the ground floor / basement footprint (Figure 2). The basement carpark will be accessed via a ramp from Western Springs Road.

These plans indicate that the basement will require cuts of up to 2.8 m on the southern side and 3.8 m on the northern side of the site. No structural, foundation, basement or earthworks plans were provided at this stage of the development.

Complete Matz Architects Limited Conceptual Plans are presented in Appendix 1.

Figure 2: Proposed Development

Image sourced from Matz Architects Limited Conceptual Plans dated 6 April 2022.

4 Area Wide Geotechnical Data

4.1 Regional Geology

Regional geological mapping by the Institute of Geological and Nuclear Sciences (GNS) indicates that the project site is underlain by the East Coast Bays Formation (Waitemata Group). The East Coast Bays Formation (ECBF) is described by GNS as *alternating sandstone and mudstone with variable volcanic content and interbedded volcaniclastic grits.*

GNS Science mapping indicates a boundary between the ECBF and Auckland basalt (Qb) volcanic tuff (Auckland volcanic Field), is approximately 150 m southeast of the site (Figure 3). The volcanic tuff is described by GNS as *lithic tuff, comprising comminuted pre-volcanic materials with basaltic fragments, and unconsolidated ash and lapilli deposits*.

Figure 3: Mapped Geology

Note: Images sourced from GNS 1:250,000 Zealand Geology Map overlay.

4.2 Seismicity

The Auckland area is one of the lowest earthquake activity regions in New Zealand. Over the last 150 years, only two earthquakes with magnitudes greater than M5 have been recorded in the region.

We have reviewed the GNS Science New Zealand Active Fault Database, which indicates there are no known active faults on-site. The nearest active fault is the Waikopua Fault located approximately 34 km southeast of the site. The Waikopua Fault dips southwest and is a normal (extensional) type fault. GNS Science have not established a dip angle, vertical slip rate, recurrence interval or date for the last event at the Waikopua Fault.

4.3 Volcanic Activity

Volcanic activity presents a significant risk in Auckland. However, the location and timing of eruptions are difficult to predict due to the primarily monogenetic nature of the volcanic field.

The eruption history of the Auckland Volcanic Field (AVF) is known to date back over the last 150,000 years. Nineteen eruptions are known to have occurred within the last 20,000 years with 18 of the most recent eruptions occurring between 20,000 and 10,000 years ago. Rangitoto was the last known eruption event which was estimated to be 550 years before present.

Hazards proximal to an eruption include pyroclastic surge, block fall and lava flows. Ash fall at a greater distance can cause large disturbance with remobilisation of ash deposits possible, particularly during rainfall events.

Although the AVF is thought to have a high risk of eruption, it is generally considered to have a low occurrence. Based on the number and frequency of past eruptions it is estimated there is approximately a 1 in 1000 (0.1%) chance an eruption could occur in any one year.

4.4 Historic Aerial Photography

We have reviewed historical aerial photographs of the site sourced from Auckland Council Geomaps, Retrolens and Nearmaps. These photographs were viewed under the context of underlying areas of potential instability and significant changes to landform. We have summarised our key findings in Table 1.

Table 1: Historic Aerial Review

Date	Summary
1940 to 1959	The site contains two dwellings at the time of this photo. One fronting Western Springs Road and one fronting New North Road. The wider area had been developed for industrial and residential use prior to the earliest aerial photography.
1959 to 1975	No change.
1975 to 1985	The dwelling within the southern part of the site remains. The previous dwelling within the northern part of the site has been demolished and the office building (which is currently on-site), has been built.
1985 to 2022	The dwelling, office building and the site appears to remain unchanged through this period.

4.5 Historical Significance Overlay

No historic overlay maps or extents were mapped on-site. The site directly east of the site has been identified as an area of historic significance and is mapped with a '*Historic Heritage Extent of place Overlay*'.

5 Site Investigation

5.1 Site Walkover

ENGEO visited the site on 25 March 2022 and made the following observations:

- The dwelling accessed from New North Road is currently used as a café. The building fronting Western Springs Road is utilised as office space (Photo 1).
- The site currently slopes gently toward the south from Western Springs Road (RL 40 m) toward New North Road (RL 38 m). A small landscape and partially integrated retaining wall is adjacent to the existing café and office building.
- 615 New North Road (directly east of the site) is an historically listed, heavy clad and concrete two storey building currently being renovated (Photo 2). The building footprint for this site is directly adjacent the eastern property boundary (within 2 m of the boundary).
- 621 New North Road (directly west of the site) is currently a two-storey heavy clad childcare facility. The carpark and children's play area currently boarders the western building perimeter (Photo 4).

Figure 4: Site Photographs

Photo 1: View of the dwelling which has been converted into a café at the southern side of the site (New North Road side), photo facing northwest.

Photo 2: Eastern boundary toward the rear or the existing dwelling / café. Note: proximity of adjacent building to eastern property boundary, photo facing southeast.

Photo 3: Existing office building on the northern side of the site, photo facing south.

Photo 4: Carpark and play area of adjacent site looking toward the existing office building onsite, photo facing southeast.

5.2 Site Investigation

ENGEO completed the site investigation on-site between 28 March and 29 March 2022. Testing locations are presented in Appendix 2 and full logs are presented in Appendix 3 and are written in general accordance with the New Zealand Geotechnical Society field classification guidelines (NZGS, 2005).

Details of the investigation are summarised in the following sections.

5.2.1 Hand Auger Boreholes

ENGEO completed three hand auger boreholes with associated strength tests (shear vane and Scala penetrometer), to 4.0 m bgl on 28 and 29 March 2022. Standing water was encountered between 2.2 m and 2.3 m depending on the borehole location.

5.2.2 Machine Borehole

ENGEO completed two machine boreholes with SPT testing conducted at 1.5 m intervals to between 19.6 m and 21.1 m depth on 28 and 29 March 2022.

A standpipe piezometer (P-01) was installed in machine borehole 02 (MBH02) location to measure groundwater levels within the upper 6.0 m of the soil profile to inform the groundwater drawdown requirements for the proposed basement excavations.

5.3 Groundwater

Standing water levels were measured by ENGEO by dip testing the hand auger boreholes following drilling, and in machine borehole MBH02 immediately following installation of the standpipe piezometer and then again approximately 3 to 4 weeks following installation of the standpipe piezometer.

Piezometer P-01 was constructed as detailed on the borehole log. Maximum screen depth was 6.0 m below ground level. A groundwater data logger was installed within the piezometer installed in MB02 (P-01) to allow continuous monitoring of groundwater data. Continuous groundwater data was recorded hourly in P-01 and a summary of the outputs are presented in Appendix 4.

The results of the groundwater monitoring in the standpipe piezometer are summarised in Table 2.

Table 2: Measured Standing Water Readings

MBH / Piezometer ID	Measured Groundwater Level (m)	
	29/03/2022 (one day after drilling)	22/04/2022
MBH02 / P-01	2.1 m / RL[1] 37.4	2.0 m / RL[1] 37.8 m

[1]Note: Surface RL is approximately RL 39.5 m.

It should be noted that the location of MB02 is within the proposed basement footprint and will likely be destroyed during earthworks. An additional monitoring location may be required to be established beyond the extent of the construction excavation to allow for continuous monitoring of the groundwater level at the site throughout construction.

Excavations are proposed as part of this development to a depth of up to 4.3 m bgl / RL 35.7 m (base of the car stacker pits plus 0.5 m for slab and slab preparation works). As shown on the groundwater monitoring data included in Appendix 4, this excavation will extend below the groundwater level measured on-site.

5.4 Geotechnical Ground Model

The material encountered in our subsurface investigations is broadly consistent with published mapping. Table 3 provides a generalised summary of the subsurface conditions compiled from our site specific testing; the test locations should be consulted for specific subsurface conditions at each location.

One interpreted geological section, referenced as A-A’ is presented in Appendix 5. This geological section is based on our site observations, available contour data and site subsurface data inferred from the hand auger and machine boreholes.

Table 3: Engineering Geology Model

Depth Range (m bgl)		Material / Unit	Typical Soil Density / Consistency or Rock Strength
MBH01	MBH02		
0 to 0.6	0 to 0.6	Asphalt / Hardfill / Silty Clay with minor fine sand (FILL)	N/A / Loose to Dense / Firm
0.6 to 12.5	0.6 to 17.2	Silty Clay / Clayey Silt / Sandy Silt / Silty Sand – Residual soil, East Coast Bays Formation (ECBF)	Stiff to Hard / Loose to Dense
12.5 to 15.5	17.2 to 17.9	Completely to Extremely Weathered Siltstone / Sandstone (recovered as Clayey Silt, trace gravel / Silty Sand) – East Coast Bays Formation (ECBF)	Stiff to Hard / Dense to Very Dense

Depth Range (m bgl)		Material / Unit	Typical Soil Density / Consistency or Rock Strength
MBH01	MBH02		
15.5 to 19.6	17.9 to 21.1	Moderately Weathered Siltstone / Sandstone - East Coast Bays Formation (ECBF)	Very Weak

6 Geohazards and Geotechnical Assessment

6.1 Soil Classification

For the purpose of seismic design, we anticipate the NZS 1170.5:2004 soil classification for this site to be 'Class C – Shallow Soil'.

6.2 Consolidation Settlement

It is considered that the proposed building structure will impose significant loading to the bearing strata. As such it is expected that this eight-storey development will be supported on piled foundations to avoid over loading shallow soils. Provided the building is piled, we do not have any concerns that loads from the proposed building will induce unacceptable consolidation settlement on the underlying soils.

However, excavations proposed as part of the works carry the potential to induce consolidation settlement of underlying soils due to drawdown of the groundwater table resulting in increased pressure on soils underlying neighbouring properties. In this instance the proposed excavation extends below the measured groundwater table and thus consolidation is expected to occur as a result of groundwater drawdown induced by the proposed development.

6.3 Expansive Soils

Expansive clay and silt soils are common in the Auckland area, and they have a tendency to shrink and swell, particularly with seasonal fluctuations of soil water content. This behaviour has implications for shallow foundation design and surface structures. We note that silt and clay rich soils were encountered beneath uncontrolled fill across the majority of the site.

Any ancillary structures associated with this development that are proposed to be supported upon shallow foundations should be designed for an expansive site soils classification of "H" Highly Expansive.

6.4 Seismic Hazards

Potential seismic hazards resulting from nearby moderate to major earthquakes can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, ground lurching, regional subsidence or uplift, soil liquefaction, lateral spreading, landslides, tsunamis, flooding, or seiches. Based on topographic and lithologic data, risk from earthquake-induced regional subsidence / uplift, landslides, ground lurching, flooding, and tsunamis and seiches are considered negligible at the site. The following sections present a discussion of other seismic hazards, and liquefaction risk as they apply to the site.

6.5 Ground Rupture

As previously discussed, there are no known active faults located within the site. Based on regional mapping, and the results of our field observations, it is our opinion that fault-related ground rupture is unlikely at the subject property.

6.6 Ground Shaking

According to NZS 1170.5:2004, Importance Level 2 buildings are required to be designed to resist earthquake shaking with an annual probability of exceedance of 1 / 500 (i.e. a 500 year return period). This is the ultimate limit state (ULS) design seismic loading. Structures are expected to retain their structural integrity during the ULS earthquake, and not collapse or endanger life. Furthermore, Importance Level 2 buildings should sustain little or no structural damage under a serviceability limit state (SLS) design load case, which is based on earthquake shaking with a 25 year return period.

Peak horizontal ground accelerations (a_{max}), have been calculated in accordance with the Ministry of Business, Innovation and Employment (MBIE) and New Zealand geotechnical Society (NZGS) Earthquake Geotechnical Engineering Practice Module 1, Appendix A (2021), using the following:

- ULS (1/500 year event): 0.19 g
- SLS (1/25 year event): 0.05 g
- SLS 2 (1/150 year event): 0.11 g

6.6.1 Liquefaction and Lateral Spreading Potential

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soils most susceptible to liquefaction are clean, loose, saturated, uniformly graded, fine-grained, cohesionless materials. Empirical evidence indicates that loose to medium dense gravels, silty sands, low-plasticity silts, and some low-plasticity clays are also potentially liquefiable.

Based on the regional geological setting and the presence of high plasticity clays observed in hand auger and machine boreholes, it is considered that the potential for liquefaction at this site is low, however a site-specific liquefaction assessment has not been undertaken at this stage.

6.7 Flooding

The Auckland GIS website shows that no mapped overland flow paths, flood plains, flood prone areas or flood sensitive areas are mapped within this site.

6.7.1 Landslides

It is our opinion that the subject site is unlikely to be subject to slope instability due to the gentle slope angle (<10°), water table depth, and inferred strength of the ECBF soil.

7 Permitted Activity Assessment - Auckland Unitary Plan

Auckland Council require an assessment against the Auckland Unitary Plan (AUP): Operative in Part (Table E7), where proposed excavations may extend below the groundwater table within the site. The tables in Appendix 5 present an assessment against Standards E7.6.1.6 and E7.6.1.10. Based on the current Resource Consent Plan Set for the proposed development, and the groundwater monitoring data, it is evident that the proposed excavations are expected to extend below groundwater level. As such and based on the assessment against the Auckland Unitary Plan, a consent for active dewatering, impeding groundwater and groundwater drawdown will be required for the proposed development.

8 Geotechnical Recommendations

Based on our site investigation, assessment and observations, we consider the site at 617 New North Road to be geotechnically suitable for the proposed commercial development, subject to the following recommendations.

Foundation design should be undertaken by a Chartered Engineering Professional familiar with the contents of this report and any supplementary reporting completed at the detailed design stage.

8.1 Shallow Foundations

Due to the scale of assumed loading from the proposed development and the lower strength of the soils exposed at basement level, shallow foundations are not considered to be suitable for the proposed development.

Accordingly, with the information provided and the deep investigations completed at the time of writing this report, we recommend the proposed building be supported on deep foundations that achieve bearing on the underlying very weak ECBF moderately weathered rock at approximately 15.5 m to 19 m depth, as summarised in the following sections.

However, the following shallow soil parameters are considered appropriate for design of shallow foundations for any ancillary structures proposed on-site.

A geotechnical ultimate bearing capacity of 300 kPa is recommended for shallow foundations constructed on identified competent natural ground beneath any topsoil and existing non-engineered fill or on engineer certified fill. This bearing capacity is considered appropriate for conventional shallow strip and pad foundations up to 1.5 m wide or a conventional waffle / rib raft foundation solution.

8.2 Deep Foundations

We anticipate that appropriate deep foundations will be required for the proposed development and solutions may include bored and concreted, screw or Continuous Flight Auger (CFA) piles.

8.2.1 Bored and Concreted Piles

Capacities for bored reinforced concrete piles are presented in Table 4. If it is intended to use side friction in conjunction with end bearing capacity, it should be noted that the frictional capacity may mobilise before the end bearing capacity and accordingly, it is considered prudent to factor the geotechnical ultimate value by 0.45 prior to applying the appropriate strength reduction factor to allow for the development of residual side adhesion.

The structural designer should attend to all details of pile type, spacing, diameter, load capacity and uplift capacity and must also ensure that the design allows for any differential movement that may occur between piled and unpiled portions of the structure. Piles should be spaced at least three pile diameters apart (center to center) to minimise axial group effects.

Very weak ECBF rock was encountered at 15.5 m to 17.9 m depth below current ground level in the machine boreholes drilled on-site. We recommend that piles are embedded a minimum of 3D into the proposed bearing materials.

Table 4: Ultimate Side Adhesion and Ultimate End Bearing Capacities for Bored Reinforced Concrete Piles

Geologic Material	Ultimate Side Friction (kPa)	Ultimate End Bearing Capacity (kPa)
ECBF Residual Soils	30[1]	750[2]
ECBF Transition Zone (Hard) Soils	100[1]	1000
ECBF Rock (Very Weak)	250[3]	5500[3]

Notes:

1. Skin friction should be ignored in the upper 1 m of the ground surface, and within the zone of influence of any service line.
2. Where piles are embedded <5D an end bearing of 500 kPa should be used.
3. Capacity for a smooth pile shaft. If the pile hole is grooved, then 500 kPa shaft friction may be adopted however casing requirements may prohibit grooving.

If increased pile capacities are required, we recommend that UCS testing is completed on the very weak rock recovered in our machine boreholes.

Pile uplift may use the ultimate side friction values summarised in Table 4 but disregard the upper 1 m of side friction.

CFA piles present a potential option with relatively low vibration and noise that will not require casing. The parameters in Table 4 may be adopted for design of these piling options.

8.2.2 Screw Piles

Screw piles may be suitable for this site – depending upon the scale of lateral loads required to be carried by these piles. Anticipated load carrying capacities need to be assessed in conjunction with the specialist contractors who promote this product.

8.2.3 Soil Subgrade Modulus

Soil subgrade moduli, both vertical and horizontal, are expressions of soil stiffness or resistance to dynamic loading, e.g. resistance to lateral pile deformation or resistance to vertical pad footing deformation under seismic loading. Other building components, such as basement walls and shear walls can also provide lateral or vertical resistance to deformation by means of passive pressure mobilisation.

At strains of less than 0.01%, the lateral soil modulus, i.e. the resistance to pile deformation, may be taken as:

Ks = 390,000 kN/m^3 for competent Waitemata Group deposits, for a nominal pile diameter of 0.9 metres.

8.2.4 Bridging Services

Bridging piles will be required where building foundations fall within the 45-degree zone of influence of buried service lines. No service lines are shown within the proposed building footprint on the Auckland Council GIS maps.

Auckland Council and Watercare have specific requirements regarding bridging pile foundation design. Foundations should be designed so that they meet the relevant requirements.

Skin friction should be ignored where the pile is within the 45-degree zone of influence of a point 500 mm below the pipe invert. The unfactored values provided in Table 4 should be factored by appropriate strength reduction factors to determine structural capacity of the bridging piles.

8.3 Pavement Design

Based on our site investigation and the proposed development levels, we consider that a preliminary subgrade design CBR value of 3% may be adopted for pavement design across the site in native stiff to very stiff ECBF material. This is likely to be a conservative value, but it should be noted that actual CBR values can be highly affected by moisture content (i.e. exposure to the elements), and trafficking. We therefore recommend that the subgrade is only trimmed to final level immediately prior to placing basecourse. A programme of CBR testing should be carried out during construction to confirm actual values.

8.4 Differential Settlement

The building should be designed to tolerate differential settlements of up to 1 in 240 (approximately 25 mm over a 6 m length of building) as required by the New Zealand Building Code Handbook, Appendix B Section B1/VM4, clause B1.0.2, under the serviceability limit state load combinations of NZS 1170.0, unless the structure is specifically designed to limit damage under a greater settlement.

8.5 Strength Reduction Factor

As required by Section B1/VM4 of the New Zealand Building Code Handbook, a strength reduction factor of 0.50 must be applied to the geotechnical ultimate soil capacity when using factored design load cases for static calculations.

9 Basement Recommendations

Based on the groundwater monitoring outlined herein, the groundwater level on-site is approximately 2.0 m to 2.3 m below existing ground level. Considering the proposed basement design requires an excavation of up to approximately 4.3 m / RL 35.7 m (base of the car stacker pits plus 0.5 m for slab and slab preparation works), the basement excavation will extend below the groundwater table. Accordingly, dewatering will be required to facilitate construction of the basement.

Consideration should be given to the proximity of the neighbouring structures when designing for construction of the basement walls. The preliminary groundwater implications in relation to the proposed development for both drained and fully tanked basement designs are presented in Appendix 5.

9.1 Excavation Near Property Boundaries

Excavation and retention of up to 4.3 m is proposed adjacent to property boundaries to facilitate the proposed basement. An assessment of the mechanical settlements resulting from wall deflections will be required as part of the consent for active dewatering, impeding groundwater and groundwater drawdown including a draft groundwater and settlement monitoring and contingency plan.

Temporary construction cases will need to be assessed as part of the retaining wall design as there is unlikely to be space for batters given the proximity of the wall to the boundary and adjacent properties. This may require either top-down construction of the permanent wall prior to excavation or installation of temporary retention to allow for construction of the permanent solution to take place (in the event that the structure is intended to retain the ground long term).

9.1.1 Temporary Batters

- Temporary unsupported (and not surcharged by adjacent buildings) cut slopes up to a height of 1.5 m should not exceed a batter of 1H:1V (45° from horizontal) and should not be left unsupported at this batter angle for longer than 48 hours.

- All temporary cuts and batters proximate to boundaries should take into account the potential surcharge and risk of undermining neighbouring properties.

- Excavated materials should not be placed or stockpiled above unsupported cuts, to avoid surcharging and triggering potential collapse or instability of the cut face.

- Cuts should not be exposed to adverse weather conditions and should be covered (with polythene sheeting or similar) and have appropriate methods of water diversion to minimise potential environmental runoff effects.

- Suitable drainage channels must be put in place to divert surface water from unsupported cut faces. Subsurface drains should also be considered for the toe of long-term slopes.

- If any permanent cuts are to be higher than 1.5 m, they should be supported with a specifically designed retaining wall and will need to be approved by a Chartered Professional Engineer practicing in Geotechnical Engineering.

- Where vertical and sub-vertical cut faces higher than 0.5 m are required for the construction of retaining walls, in addition to the above recommendations, we recommend that this is done in shortened sections, where possible (< 5 m) and the faces are left unsupported for a minimal time period (i.e. one week) or temporarily shored, particularly in close proximity to site boundaries and structures.

- All cuts and batters should be in line with the WorkSafe Good Practice Guidelines for Excavation Safety (July 2016).

10 Preliminary Retaining Wall Design Parameters

The soil parameters presented in Table 5 may be assumed for preliminary design of basement retaining walls.

Table 5: Geotechnical Soil Parameters for Retaining Wall Design

Material Type	Unit Weight (kN/m^3)	Friction Angle ϕ (degrees)	Effective Cohesion (c') (kPa)	Undrained Shear Strength (kPa)
Uncontrolled Fill	17	26	2	
ECBF Residual Soil	17.5	30	5	50
ECBF Transitional Soils	18	32	7	200
ECBF Very Weak to Weak Rock	19	40	30	N/A

These values are considered to be appropriate for the existing soils identified in our shallow and deep soil testing. However, if significant variation or zones of soft material are encountered during the site works, then the matter should be referred back to ENGEO for review and comment, as necessary.

The retaining wall designer should consider all appropriate surcharge loadings, back and toe slope angles. If the walls are flexible, the soil may be assumed to be in the active state and the soil pressure may be calculated using active conditions (K_a). If no significant movement is acceptable at the SLS level, or if the wall can deflect less than 0.3% of its height, then the at-rest condition should be used (K_o). The designer should also determine whether deflections of the wall are acceptable and therefore whether 'active' (Ka) or 'at rest' (Ko) lateral earth pressure design should be used.

All retaining walls, including foundation walls, should be back drained to prevent the build-up of hydrostatic pressures. The back drain should discharge to an approved outlet. Additionally, all basement walls should be suitably waterproofed to current industry standards.

11 Site Works and Construction Recommendations

11.1 Demolition

It is essential that all foundations and building debris from demolition of the existing building is completely removed prior to earthworks commencing. We anticipate that most of the demolition debris on-site will be removed as a consequence of excavating the proposed building footprint. Where foundations are removed below final ground level, they will need to be backfilled with approved hardfill (e.g. GAP65 or similar approved product) compacted in maximum 200 mm thick layers to ensure a consistent subgrade.

If any existing services are to be decommissioned, the abandoned lines should be fully removed or backfilled with a grout / bentonite slurry to avoid creating preferential groundwater flow paths. All trench backfill will also need to be removed and replaced with engineer certified fill in the vicinity of the proposed building in order to avoid the need for pipe bridging piles.

Any existing uncontrolled fill uncovered by site clearing work should be inspected by a suitably qualified geotechnical professional to confirm its suitability to remain on-site. A provisional allowance should be included in the construction scope for undercut and removal of existing fill associated with the existing structures and landscaping.

11.2 General Earthworks

Topsoil and uncontrolled fill should be stripped from all cut and fill areas prior to earthworks commencing. Stockpiles of topsoil and unsuitable materials should be sited well clear of the works on suitable, approved areas of natural ground.

Fill should comprise clean clay or hardfill and should be approved by the geotechnical engineer prior to use. Compaction should be carried out to certified standards (NZS 4431) with conventional plant and under engineering control. The geotechnical engineer should be given every opportunity to observe materials prior to placement and during compaction to carry out QA testing as required. Should soft materials be exposed they may require undercutting and replacement with engineered fill (i.e. SPR or similar approved material).

11.3 Piling

Based on our encountered ground and groundwater conditions on-site, we consider that risk of hole collapse (for bored piles) should be allowed for. Casing (and potentially tremmie pouring) should be allowed for.

The piling contractor should allow for piling into very weak ECBF sandstone and siltstone (UCS range of 1.0 to 5.0 MPa).

11.4 Groundwater

Groundwater is expected to be encountered during the proposed bulk excavations and car stacker excavations. The contractor should allow for groundwater control measures during the excavation. We note that the proposed car stacker excavations will likely extend into the ECBF residual soils. Tanked excavations may be required for the car stacker pits to mitigate against groundwater ingress.

11.5 Sediment Erosion Control

During construction, measures should be undertaken to control and treat stormwater runoff, with silt and erosion controls complying with Auckland Council Guidance for Erosion & Sediment Control (GD05).

12 Further Work

12.1 Basement Retaining Wall Design and Assessment of Effects

As the proposed basement will extend below measured groundwater levels, an assessment of effects report and a draft Groundwater and Settlement Monitoring and Contingency Plan (GSMCP), will be required to support your application for Resource Consent for this development.

This report will include an assessment of mechanical settlements induced by deflection of basement retention systems and an assessment of consolidation caused by groundwater drawdown. The draft GSMCP will include monitoring requirements for this and affected neighbouring sites.

12.2 Plan Review and Site Observations

If the final development concept varies significantly from the concept assessed by this report, we should be given the opportunity to review the updated working drawings (plan review) to ensure our recommendations have been interpreted as intended.

It is also essential that we are given every opportunity to attend a pre-start meeting on-site prior to works commencing and then to observe site works, including site stripping, earthworks operations and ground conditions in subgrades and retaining wall excavations (prior to pouring concrete) to confirm works are carried out in accordance with the recommendations of this report and that ground conditions are as assumed.

13 Limitations

i. We have prepared this report in accordance with the brief as provided. This report has been prepared for the use of our client, Pink Beluga Civil Limited, their professional advisers and the relevant Territorial Authorities in relation to the specified project brief described in this report. No liability is accepted for the use of any part of the report for any other purpose or by any other person or entity.

ii. The recommendations in this report are based on the ground conditions indicated from published sources, site assessments and subsurface investigations described in this report based on accepted normal methods of site investigations. Only a limited amount of information has been collected to meet the specific financial and technical requirements of the client's brief and this report does not purport to completely describe all the site characteristics and properties. The nature and continuity of the ground between test locations has been inferred using experience and judgement and it should be appreciated that actual conditions could vary from the assumed model.

iii. Subsurface conditions relevant to construction works should be assessed by contractors who can make their own interpretation of the factual data provided. They should perform any additional tests as necessary for their own purposes.

iv. This Limitation should be read in conjunction with the Engineering NZ / ACENZ Standard Terms of Engagement.

v. This report is not to be reproduced either wholly or in part without our prior written permission.

We trust that this information meets your current requirements. Please do not hesitate to contact the undersigned on (09) 972 2205 if you require any further information.

Report prepared by	Report reviewed by
Hamish Foy	**Paul Fletcher, CMEngNZ (CPEng)**
Senior Engineering Geologist	Associate Geotechnical Engineer

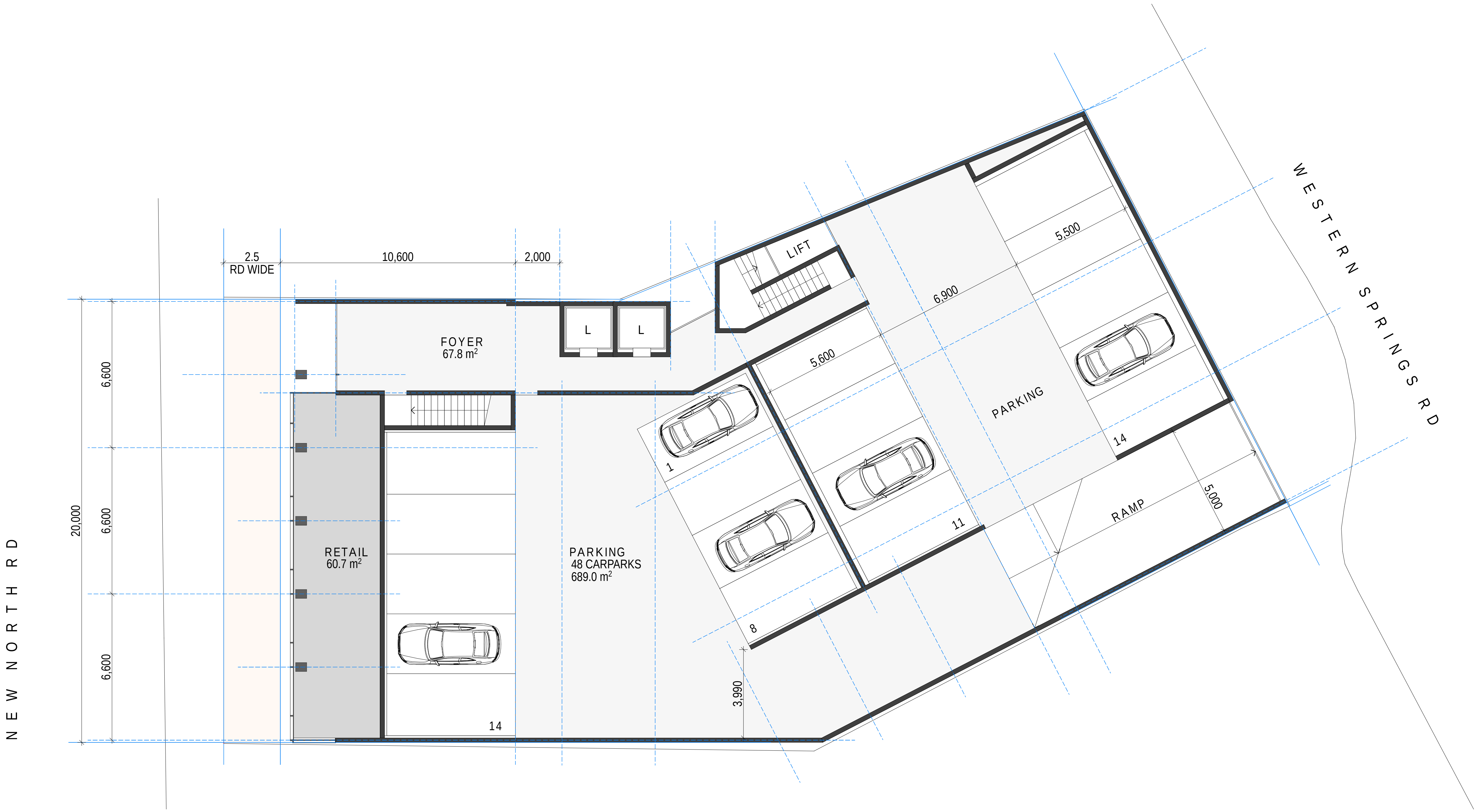
APPENDIX 1:

Proposed Development Plans

SKETCH 02
BUILDING MASS

SKETCH 01
BUILDING MASS

MASSING ELEVATION
1:250 @ A3



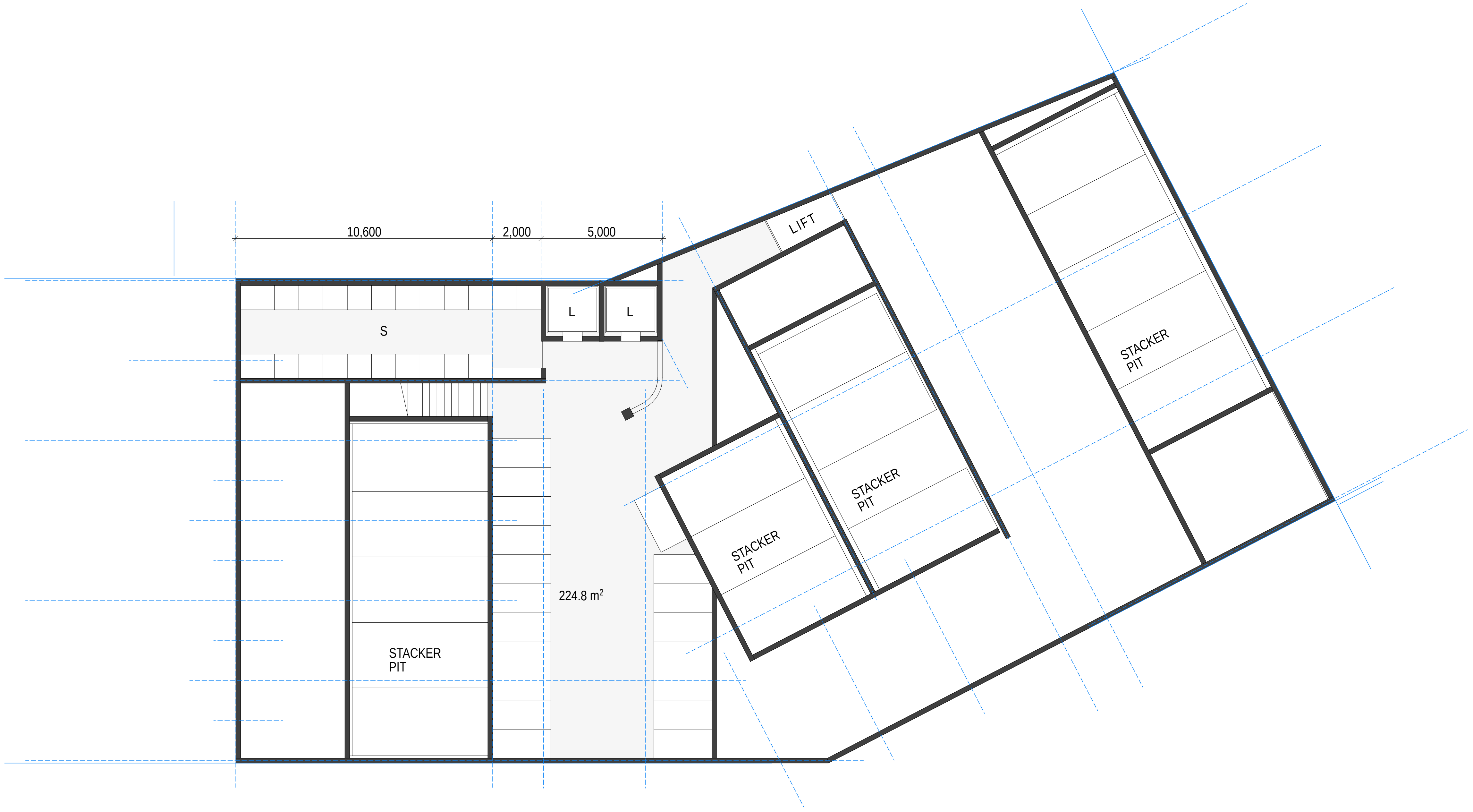
GROUND

1:200 @ A3

AREA SUMMARY

RETAIL	60 m^2
CIRCULATION	56 m^2 EX STAIR + LIFT
PARKING	707 m^2
TOTAL GFA	823 m^2

PRELIMINARY

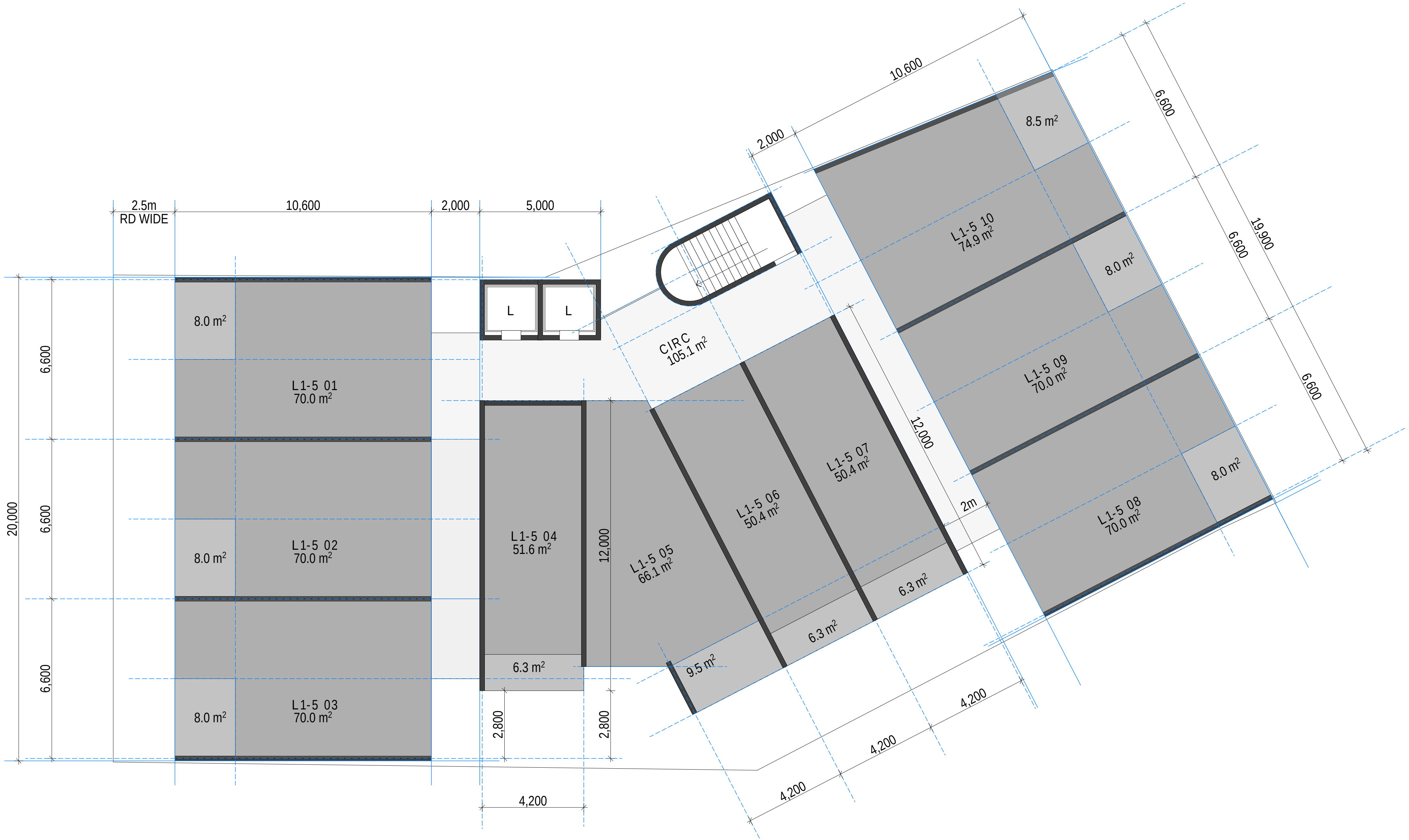


BASEMENT

1:200 @ A3

AREA SUMMARY
TOTAL GFA 214 m²

PRELIMINARY



LEVEL L01-L05

1:200 @ A3

AREA SUMMARY

APARTMENTS	640 m²
CIRCULATION	106 m² EX STAIR + LIFT
TOTAL GFA	746 m²

PRELIMINARY

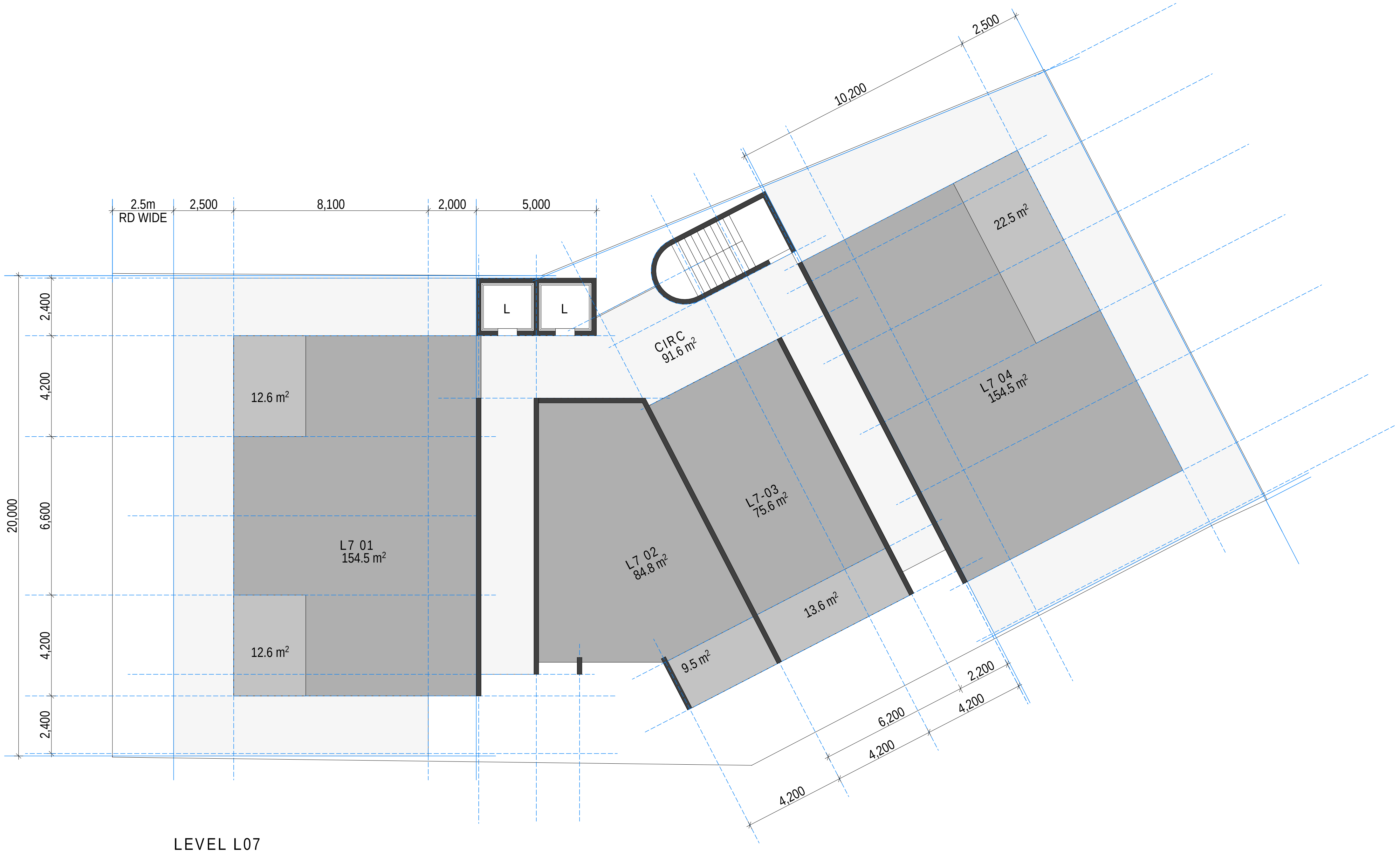
LEVEL L06

1:200 @ A3

AREA SUMMARY

APARTMENTS	640 m²
CIRCULATION	105 m² EX STAIR + LIFT
TOTAL GFA	746 m²

PRELIMINARY



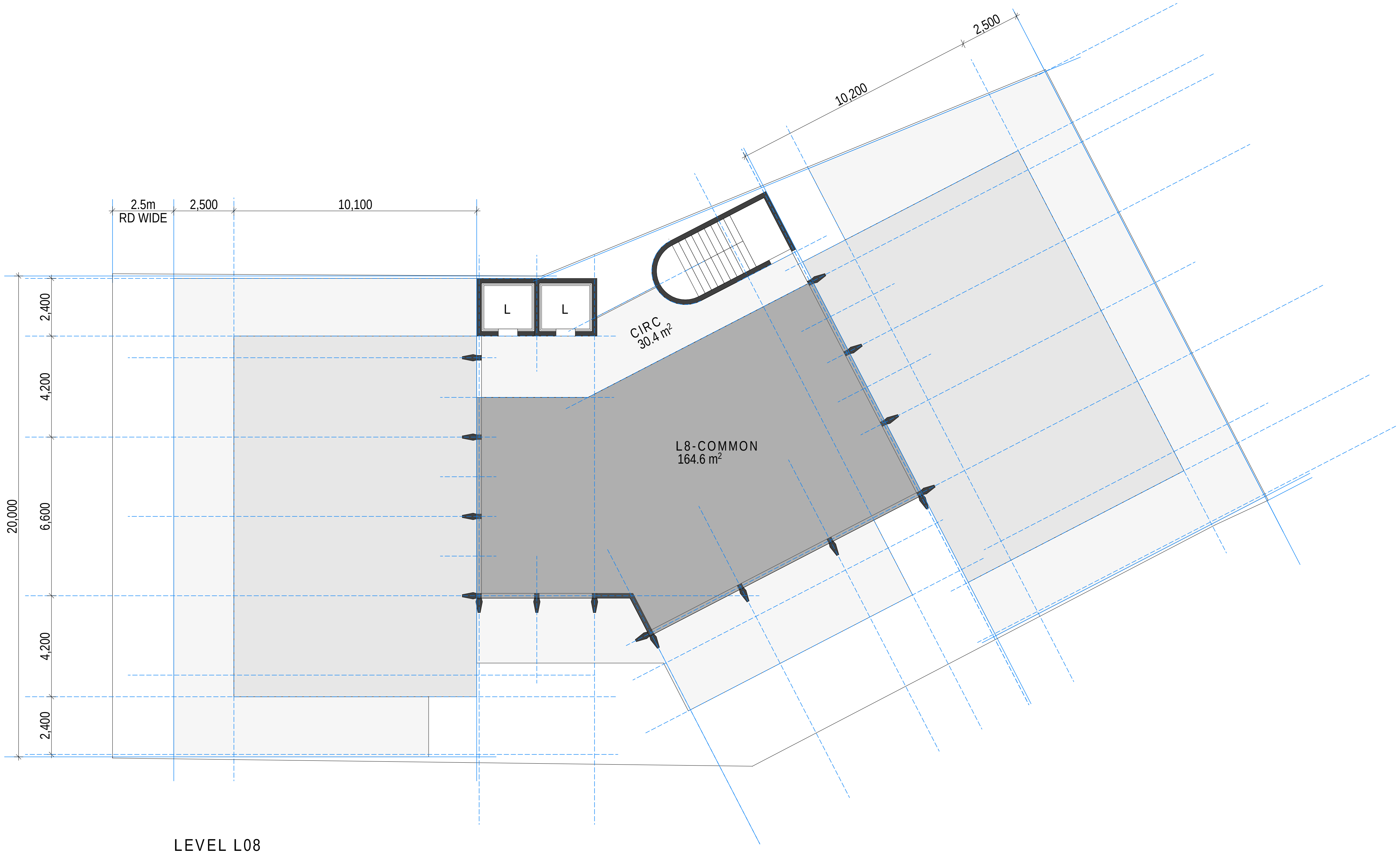
LEVEL L07

1:200 @ A3

AREA SUMMARY

APARTMENTS	470 m²
CIRCULATION	92 m² EX STAIR + LIFT
TOTAL GFA	562 m²

PRELIMINARY

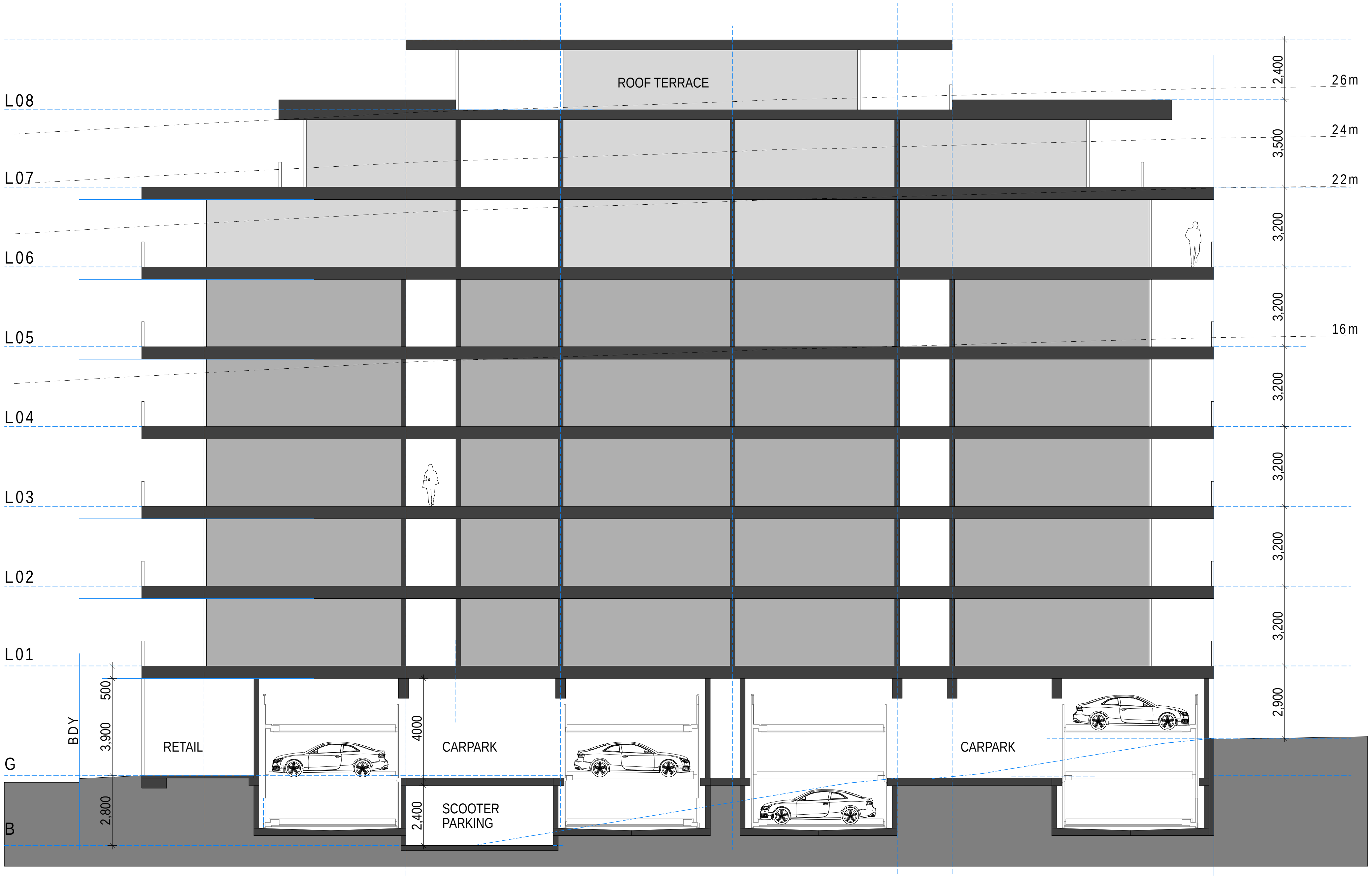


LEVEL L08

1:200 @ A3

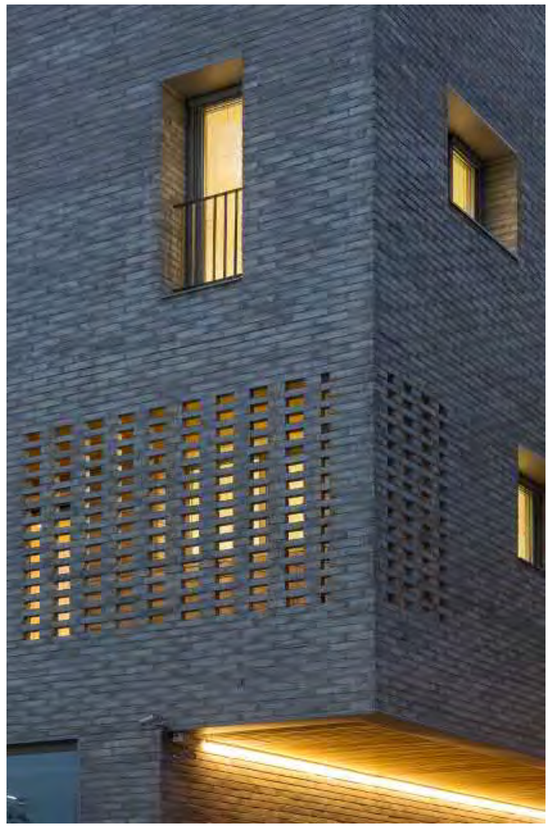
AREA SUMMARY

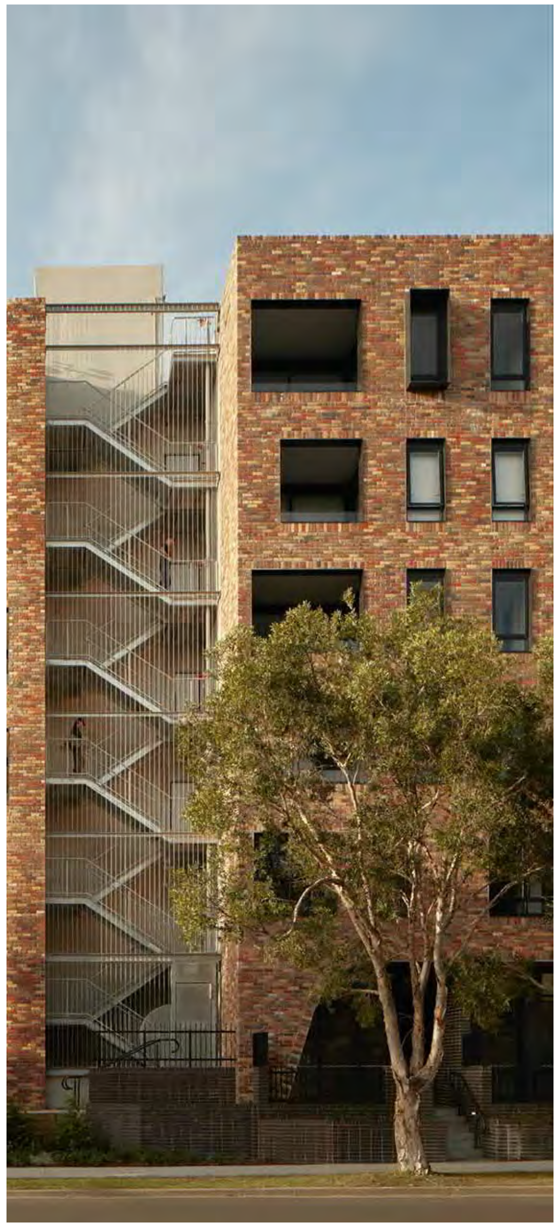
COMMON	165 m^2
CIRCULATION	59 m^2 EX STAIR + LIFT
TOTAL GFA	224 m^2



SECTION
1:200 @ A3

PRELIMINARY





SCALE & BRICK

APPENDIX 2:

Investigation Location Plan

Legend

- ⊕ Base Group
- ☐ Approximate Site Boundary
- Machine Boreholes
- Hand Augers

Services

- ● Wastewater Fitting (Local)
- Wastewater Manhole GIS ID Label (Local)
- ◎ Wastewater Manhole (Local)
- Wastewater Pipe GIS ID Label (Local)

Wastewater Pipe (Local)

- — Operational
- - - Operational Not Vested
- — Abandoned / Not Operational
- ☐ Wastewater Other Structure (Local)
- 🟪 Wastewater Pump Station (Local)
- ● Wastewater Fitting (Transmission)
- ◎ Wastewater Manhole (Transmission)

Wastewater Pipe (Transmission)

- — Operational
- — Not Operational
- - - Proposed
- ☐ Wastewater Chamber (Transmission)
- 🟪 Wastewater Pump Station (Transmission)

Stormwater Treatment Device

- Public
- Private

Stormwater Soakage System

0 5 m 10 m

© Nearmaps, , public

ENGEO

Produced by **Datanest.earth**

Title: Geotechnical Investigation Location plan

Client: Pink Beluga Civil Limited

Project: 617 New North Road, Kingsland	Drawn: JH	Figure No: 1 Size: A4
Date: 04-04-2022	Checked: LL	
Proj No: 20034.000.001	Scale: 1:300	Version: draft

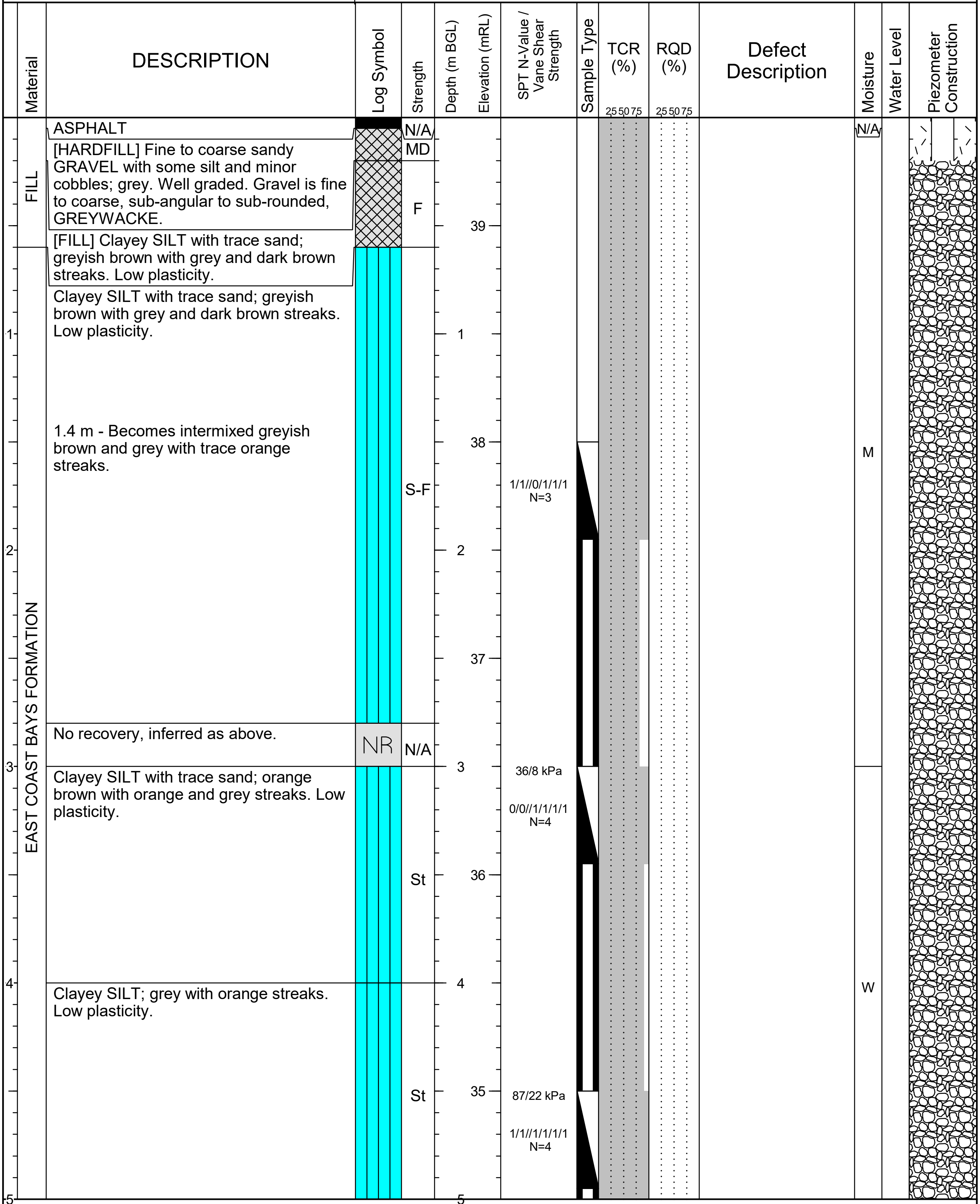
APPENDIX 3:

Geotechnical Logs

BOREHOLE LOG MBH01

Geotechnical Investigation
617 New North Road
Kingsland, Auckland
20034.000.001

Client : Pink Beluga Civil Limited
Date : 28/03/2022
Hole Depth : 19.6 m
Drilling Method : Mud Rotary
Drilling Contractor : Prodrill Ltd
Core Diameter : 83 mm
Energy Transfer Ratio : 75.8
Logged By/Reviewed By : JT / HF
Latitude : -36.873608
Longitude : 174.736256



Material	DESCRIPTION	Strength	SPT N-Value / Vane Shear Strength	Moisture
FILL	ASPHALT	N/A		N/A
	[HARDFILL] Fine to coarse sandy GRAVEL with some silt and minor cobbles; grey. Well graded. Gravel is fine to coarse, sub-angular to sub-rounded, GREYWACKE.	MD F		M
	[FILL] Clayey SILT with trace sand; greyish brown with grey and dark brown streaks. Low plasticity.			
EAST COAST BAYS FORMATION	Clayey SILT with trace sand; greyish brown with grey and dark brown streaks. Low plasticity. 1.4 m - Becomes intermixed greyish brown and grey with trace orange streaks.	S-F	1/1//0/1/1/1 N=3	
	No recovery, inferred as above.	N/A		
	Clayey SILT with trace sand; orange brown with orange and grey streaks. Low plasticity.	St	36/8 kPa 0/0//1/1/1/1 N=4	W
	Clayey SILT; grey with orange streaks. Low plasticity.	St	87/22 kPa 1/1//1/1/1/1 N=4	

Machine Borehole met target depth at 22.95 m.
N/A = Not Assessed; NR = No Recovery; UTP = Unable to Penetrate. Coordinates obtained from Google Earth Pro.
Elevation data obtained from Auckland Council GeoMaps.

BOREHOLE LOG MBH01

Geotechnical Investigation
617 New North Road
Kingsland, Auckland
20034.000.001

Client : Pink Beluga Civil Limited
Date : 28/03/2022
Hole Depth : 19.6 m
Drilling Method : Mud Rotary
Drilling Contractor : Prodrill Ltd
Core Diameter : 83 mm
Energy Transfer Ratio : 75.8
Logged By/Reviewed By : JT / HF
Latitude : -36.873608
Longitude : 174.736256

Material	DESCRIPTION	Strength	Moisture
EAST COAST BAYS FORMATION	Clayey SILT; grey with orange streaks. Low plasticity.	St	W
	Clayey SILT with trace sand; greyish brown with orange streaks. Low plasticity.	St	
	Clayey SILT with minor fine to medium sand; grey. Low plasticity. 6.45 m - Encountered trace sand. 7.5 m - Encountered minor fine to medium sand.	St-H	

Depth (m BGL)	Elevation (mRL)	SPT N-Value / Vane Shear Strength
	34	
6		70/16 kPa 1/1//1/1/1/1 N=4
	33	
7		
	32	UTP kPa 1/2//2/2/3/2 N=9
8		
	31	
9		59/16 kPa 1/2//3/3/3/4 N=13
	30	
10		

Sample Type	TCR (%)	RQD (%)	Defect Description	Water Level	Piezometer Construction
	25 50 75	25 50 75			

Machine Borehole met target depth at 22.95 m.
N/A = Not Assessed; NR = No Recovery; UTP = Unable to Penetrate. Coordinates obtained from Google Earth Pro.
Elevation data obtained from Auckland Council GeoMaps.

BOREHOLE LOG MBH01

Geotechnical Investigation
617 New North Road
Kingsland, Auckland
20034.000.001

Client : Pink Beluga Civil Limited
Date : 28/03/2022
Hole Depth : 19.6 m
Drilling Method : Mud Rotary
Drilling Contractor : Prodrill Ltd
Core Diameter : 83 mm
Energy Transfer Ratio : 75.8
Logged By/Reviewed By : JT / HF
Latitude : -36.873608
Longitude : 174.736256

Material	DESCRIPTION	Log Symbol	Strength	SPT N-Value / Vane Shear Strength	Moisture
EAST COAST BAYS FORMATION	Clayey SILT with minor fine to medium sand; grey. Low plasticity.		St-H		W
	No recovery; inferred as above.	NR	N/A		
	Clayey SILT; grey. Low plasticity.		VSt-H	UTP kPa 3/3//4/5/6/6 N=21	
	No recovery; inferred as above.	NR	N/A		
	Clayey SILT with minor fine to medium sand; grey. Low plasticity.		H	8/10//13/22/15 for 50 mm N=50+	M
	Moderately weathered, grey SILTSTONE. Very weak.		SLST		N/A
	Clayey SILT with minor fine to medium sand; dark grey. Low plasticity. [Highly weathered, extremely weak SILTSTONE].		H		M
	Silty fine to medium SAND; dark grey. Poorly graded. [Highly weathered, extremely weak SANDSTONE].		VD N/A VD	10/12//14/16/13/7 for 45 mm N=50+	
	Clayey SILT with minor fine to medium sand; dark grey with black streaks. Low plasticity. [Highly weathered, extremely weak SILTSTONE].		H		
	Silty fine to medium SAND; dark grey. Poorly graded. [Highly weathered, extremely weak SANDSTONE].		VD		
	Clayey SILT with minor fine to medium		H		

Depth (m BGL): 11, 12, 13, 14, 15

Elevation (mRL): 29, 28, 27, 26, 25

Sample Type | TCR (%) 25 50 75 | RQD (%) 25 50 75 | Defect Description | Water Level | Piezometer Construction

Machine Borehole met target depth at 22.95 m.
N/A = Not Assessed; NR = No Recovery; UTP = Unable to Penetrate. Coordinates obtained from Google Earth Pro.
Elevation data obtained from Auckland Council GeoMaps.

ENGEO

BOREHOLE LOG MBH01

Geotechnical Investigation
617 New North Road
Kingsland, Auckland
20034.000.001

Client : Pink Beluga Civil Limited
Date : 28/03/2022
Hole Depth : 19.6 m
Drilling Method : Mud Rotary
Drilling Contractor : Prodrill Ltd
Core Diameter : 83 mm
Energy Transfer Ratio : 75.8
Logged By/Reviewed By : JT / HF
Latitude : -36.873608
Longitude : 174.736256

Material	DESCRIPTION	Strength	Depth (m BGL)	Elevation (mRL)	SPT N-Value / Vane Shear Strength	Moisture
EAST COAST BAYS FORMATION	sand; dark grey. Low plasticity. [Highly weathered, extremely weak SILTSTONE].	VD				M
	Silty fine to medium SAND; dark grey. Poorly graded. [Highly weathered, extremely weak SANDSTONE].	N/A		24	14/16//14/14/12/1 for 55 mm N=50+	
	Moderately weathered, dark grey SILTSTONE. Very weak. 15.65 m - Joint; 60°, undulating, smooth, moderately narrow. 15.85 m - Joint; 38°, planar, slickensided, very narrow.	VW	16			N/A
	Moderately weathered, dark grey, fine to medium SANDSTONE. Very weak.	VW NR VW N/A	17 18	23 22	9/3//15/18/17 for 70 mm N=50+ 10/13//17/11/12/1 for 45 mm N=50+	
	Moderately weathered, dark grey, fine to medium SANDSTONE. Very weak. 18.6 m - Joint; 62°, undulating, smooth, narrow. 18.7 m - Joint; 34°, planar, rough, narrow.	VW	19	21		
	Moderately weathered, dark grey SILTSTONE. Very weak.	VW				
	Moderately weathered, dark grey, fine to medium SANDSTONE. Very weak.	VW		20		
	No recovery; solid cone SPT.	N/A			37/13 for 25 mm N=50+	

Log Symbol | Sample Type | TCR (%) 25 50 75 | RQD (%) 25 50 75 | Defect Description | Water Level | Piezometer Construction

End of Hole Depth: 19.6 m
Termination: Target depth

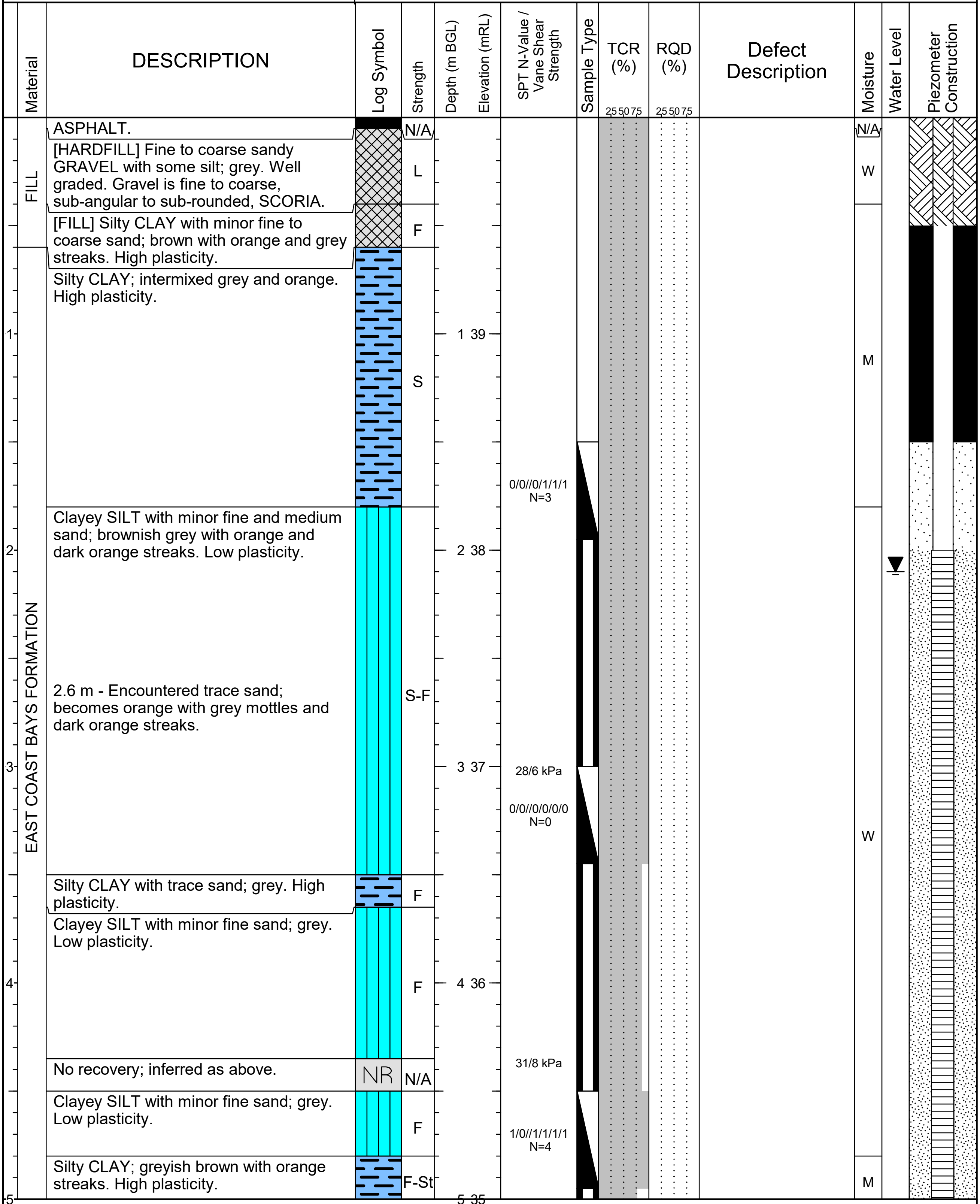
Machine Borehole met target depth at 22.95 m.
N/A = Not Assessed; NR = No Recovery; UTP = Unable to Penetrate. Coordinates obtained from Google Earth Pro.
Elevation data obtained from Auckland Council GeoMaps.

ENGEO

BOREHOLE LOG MBH02

Geotechnical Investigation
617 New North Road
Kingsland, Auckland
20034.000.001

Client : Pink Beluga Civil Limited
Date : 29/03/2022
Hole Depth : 21.1 m
Drilling Method : Mud Rotary
Drilling Contractor : Prodrill Ltd
Core Diameter : 83 mm
Energy Transfer Ratio : 75.8
Logged By/Reviewed By : JT / HF
Latitude : -36.87344
Longitude : 174.736161



Material	DESCRIPTION	Strength	Depth (m BGL)	Elevation (mRL)	SPT N-Value / Vane Shear Strength	Moisture
FILL	ASPHALT.	N/A				N/A
	[HARDFILL] Fine to coarse sandy GRAVEL with some silt; grey. Well graded. Gravel is fine to coarse, sub-angular to sub-rounded, SCORIA.	L				W
	[FILL] Silty CLAY with minor fine to coarse sand; brown with orange and grey streaks. High plasticity.	F				M
EAST COAST BAYS FORMATION	Silty CLAY; intermixed grey and orange. High plasticity.	S	1	39	0/0//0/1/1/1 N=3	M
	Clayey SILT with minor fine and medium sand; brownish grey with orange and dark orange streaks. Low plasticity. 2.6 m - Encountered trace sand; becomes orange with grey mottles and dark orange streaks.	S-F	2 3	38 37	28/6 kPa 0/0//0/0/0/0 N=0	W
	Silty CLAY with trace sand; grey. High plasticity.	F				W
	Clayey SILT with minor fine sand; grey. Low plasticity.	F	4	36		W
	No recovery; inferred as above.	N/A			31/8 kPa	W
	Clayey SILT with minor fine sand; grey. Low plasticity.	F			1/0//1/1/1/1 N=4	W
	Silty CLAY; greyish brown with orange streaks. High plasticity.	F-St	5	35		M

Machine Borehole met target depth at 22.95 m.
Standing groundwater encountered at 2.1 m depth at 7:30 am day of drilling.
N/A = Not Assessed; NR = No Recovery; UTP = Unable to Penetrate.

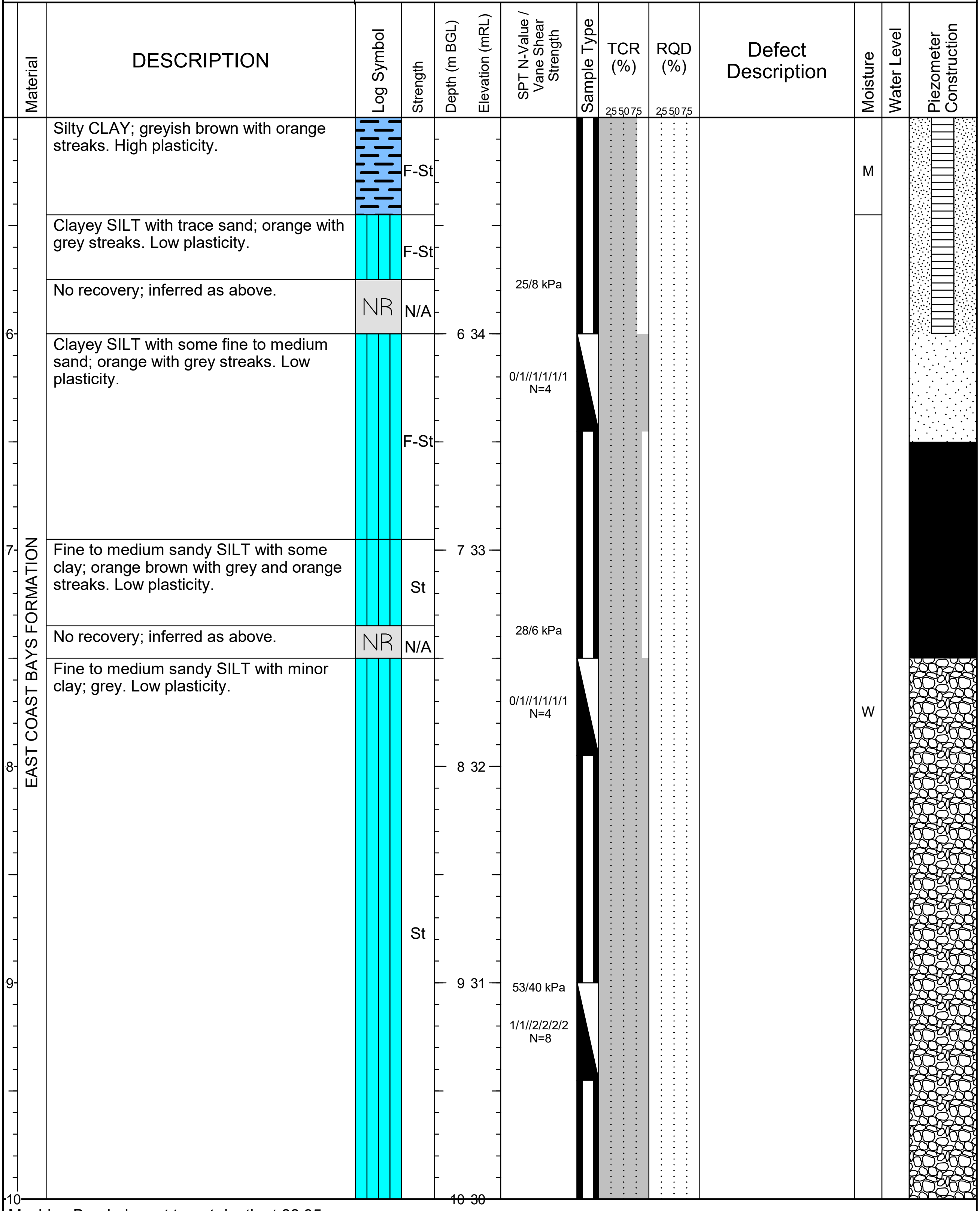
Elevation data obtained from Auckland Council GeoMaps.
Coordinates obtained from Google Earth Pro.

BOREHOLE LOG MBH02

Geotechnical Investigation
617 New North Road
Kingsland, Auckland
20034.000.001

Client : Pink Beluga Civil Limited
Date : 29/03/2022
Hole Depth : 21.1 m
Drilling Method : Mud Rotary
Drilling Contractor : Prodrill Ltd
Core Diameter : 83 mm
Energy Transfer Ratio : 75.8
Logged By/Reviewed By : JT / HF
Latitude : -36.87344
Longitude : 174.736161

Material	DESCRIPTION	Log Symbol	Strength	Moisture
EAST COAST BAYS FORMATION	Silty CLAY; greyish brown with orange streaks. High plasticity.		F-St	M
	Clayey SILT with trace sand; orange with grey streaks. Low plasticity.		F-St	W
	No recovery; inferred as above.	NR	N/A	
	Clayey SILT with some fine to medium sand; orange with grey streaks. Low plasticity.		F-St	
	Fine to medium sandy SILT with some clay; orange brown with grey and orange streaks. Low plasticity.		St	
	No recovery; inferred as above.	NR	N/A	
	Fine to medium sandy SILT with minor clay; grey. Low plasticity.		St	



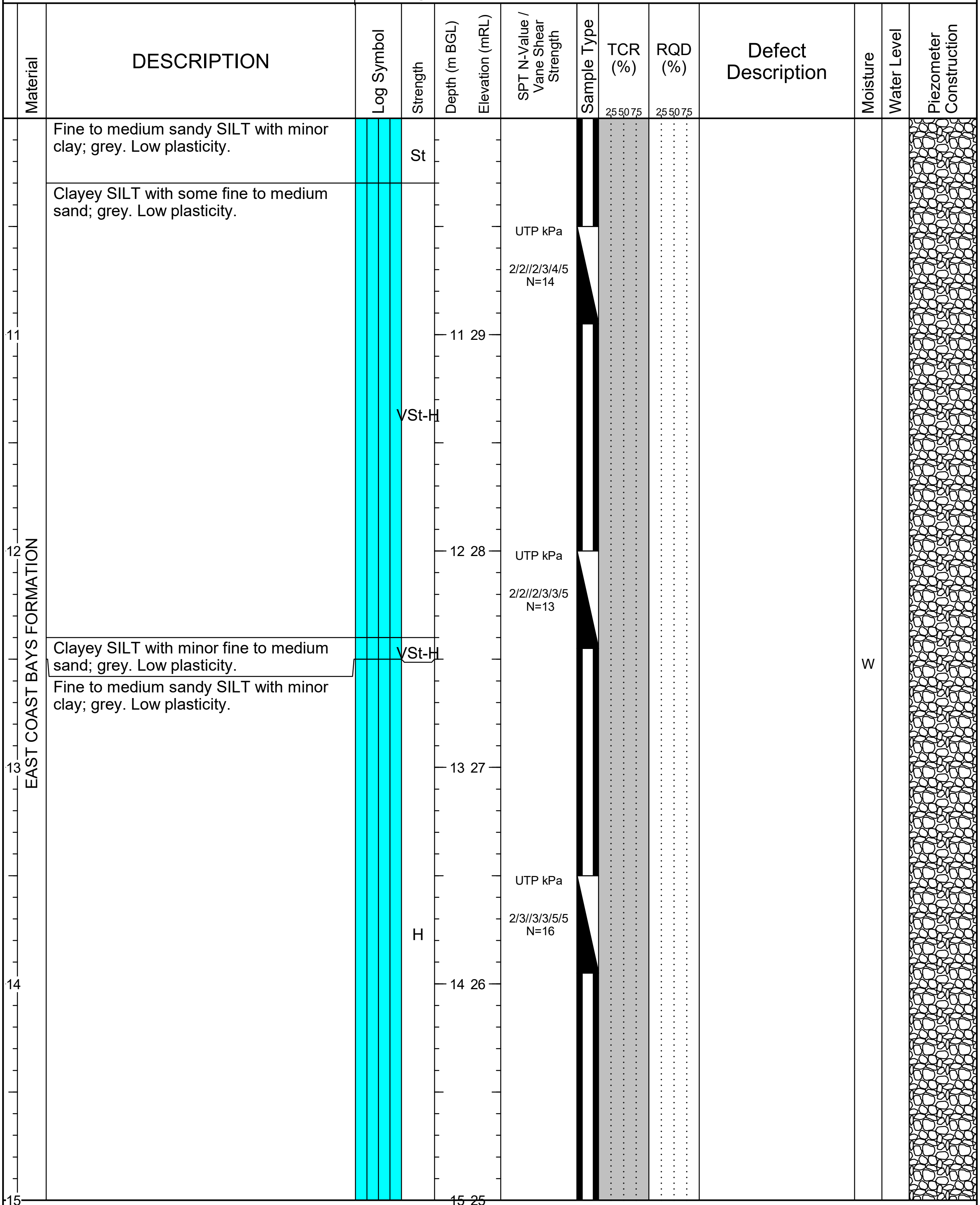
Machine Borehole met target depth at 22.95 m.
Standing groundwater encountered at 2.1 m depth at 7:30 am day of drilling.
N/A = Not Assessed; NR = No Recovery; UTP = Unable to Penetrate.

Elevation data obtained from Auckland Council GeoMaps.
Coordinates obtained from Google Earth Pro.

BOREHOLE LOG MBH02

Geotechnical Investigation
617 New North Road
Kingsland, Auckland
20034.000.001

Client : Pink Beluga Civil Limited
Date : 29/03/2022
Hole Depth : 21.1 m
Drilling Method : Mud Rotary
Drilling Contractor : Prodrill Ltd
Core Diameter : 83 mm
Energy Transfer Ratio : 75.8
Logged By/Reviewed By : JT / HF
Latitude : -36.87344
Longitude : 174.736161



Fine to medium sandy SILT with minor clay; grey. Low plasticity.

Clayey SILT with some fine to medium sand; grey. Low plasticity.

Clayey SILT with minor fine to medium sand; grey. Low plasticity.

Fine to medium sandy SILT with minor clay; grey. Low plasticity.

Machine Borehole met target depth at 22.95 m.
Standing groundwater encountered at 2.1 m depth at 7:30 am day of drilling.
N/A = Not Assessed; NR = No Recovery; UTP = Unable to Penetrate.

Elevation data obtained from Auckland Council GeoMaps.
Coordinates obtained from Google Earth Pro.

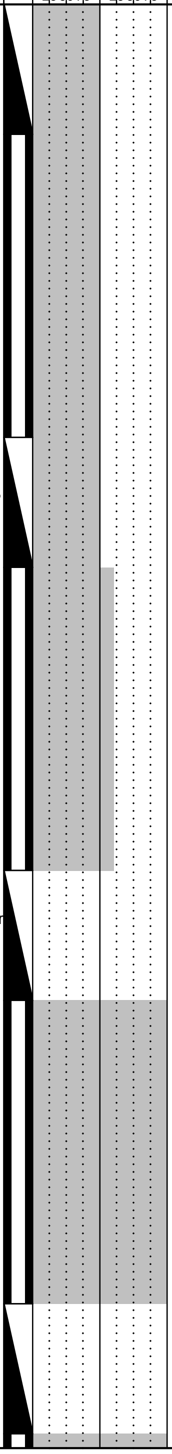
BOREHOLE LOG MBH02

Geotechnical Investigation
617 New North Road
Kingsland, Auckland
20034.000.001

Client : Pink Beluga Civil Limited
Date : 29/03/2022
Hole Depth : 21.1 m
Drilling Method : Mud Rotary
Drilling Contractor : Prodrill Ltd
Core Diameter : 83 mm
Energy Transfer Ratio : 75.8
Logged By/Reviewed By : JT / HF
Latitude : -36.87344
Longitude : 174.736161

Material	DESCRIPTION	Strength	SPT N-Value / Vane Shear Strength	Moisture
EAST COAST BAYS FORMATION	Fine to medium sandy SILT with minor clay; grey. Low plasticity.	H	UTP kPa 3/4//4/5/8/9 N=26	W
	Silty fine to medium SAND; grey. Poorly graded.	D	UTP kPa 4/6//6/9/12/13 N=40	M
	Silty fine to medium SAND; dark grey. Poorly graded. [Highly weathered, extremely weak SANDSTONE].	D		
	Clayey SILT; dark grey. Low plasticity. [Highly weathered, extremely weak SILTSTONE].	H		
	Moderately weathered, dark grey, fine to medium SANDSTONE. Very weak.	VW N/A VW N/A	3/17//24/26 for 75 mm N=50+	N/A
	Moderately weathered, dark grey, fine to medium SANDSTONE. Very weak. Widely spaced, moderately thickly interbedded SILTSTONE.	VW	18/32 for 75 mm N=50+	

Depth (m BGL)	Elevation (mRL)
16	24
17	23
18	22
19	21
20	20



Defect Description

Water Level

Piezometer Construction

Machine Borehole met target depth at 22.95 m.
Standing groundwater encountered at 2.1 m depth at 7:30 am day of drilling.
N/A = Not Assessed; NR = No Recovery; UTP = Unable to Penetrate.

Elevation data obtained from Auckland Council GeoMaps.
Coordinates obtained from Google Earth Pro.

BOREHOLE LOG MBH02

Geotechnical Investigation
617 New North Road
Kingsland, Auckland
20034.000.001

Client : Pink Beluga Civil Limited
Date : 29/03/2022
Hole Depth : 21.1 m
Drilling Method : Mud Rotary
Drilling Contractor : Prodrill Ltd
Core Diameter : 83 mm
Energy Transfer Ratio : 75.8
Logged By/Reviewed By : JT / HF
Latitude : -36.87344
Longitude : 174.736161

Material	DESCRIPTION	Log Symbol	Strength	Depth (m BGL)	Elevation (mRL)	SPT N-Value / Vane Shear Strength	Sample Type	TCR (%) 25 50 75	RQD (%) 25 50 75	Defect Description	Moisture	Water Level	Piezometer Construction
	Moderately weathered, dark grey, fine to medium SANDSTONE. Very weak. Widely spaced, moderately thickly interbedded SILTSTONE.		VW								N/A		
21			N/A	21	19	20/30 for 45 mm N=50+							

End of Hole Depth: 21.1 m
Termination: Target depth

Machine Borehole met target depth at 22.95 m.
Standing groundwater encountered at 2.1 m depth at 7:30 am day of drilling.
N/A = Not Assessed; NR = No Recovery; UTP = Unable to Penetrate.

Elevation data obtained from Auckland Council GeoMaps.
Coordinates obtained from Google Earth Pro.

LOG OF AUGER HA01

Geotechnical Investgation
617 New North Road
Kingsland, Auckland

Client : Pink Beluga Civil Limited
Client Ref. : 20034.000.001
Date : 28/03/2022
Hole Depth : 4 m
Hole Diameter : 50 mm

Shear Vane No : 3333
Logged By : JT
Reviewed By : HF
Latitude : -36.873491
Longitude : 174.736277

Material	USCS Symbol	DESCRIPTION	Moisture Cond.	Consistency/ Density Index
TOPSOIL	OL	TOPSOIL.	M	N/A St
EAST COAST BAYS FORMATION	CH	Silty CLAY; grey with orange mottles. High plasticity.	M	St
EAST COAST BAYS FORMATION	ML	Clayey SILT; brownish grey with orange streaks. Low plasticity. 2.1 m - Becomes brownish grey and saturated. 3.7 m - Becomes brownish grey with orange streaks.	W S	St-VSt
		End of Hole Depth: 4 m Termination Condition: Target depth		

Depth (m BGL): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0

Elevation (mRL): 39, 38, 37, 36

Water Level: ▼

Shear Vane Undrained Shear Strength (kPa) Peak/Remoled
62/9
65/25
84/34
96/25
99/23
102/25
90/33
118/31
113/40
101/36
124/42
136/68
123/53

Scala Penetrometer
Blows per 100mm
2 4 6 8 10 12

Hand auger met target depth at 4 m.
Dip test showed groundwater at 2.22 m depth at 1:30 pm, 28/03/22.
N/A = Not Assessed; UTP = Unable to Penetrate.
Elevation data obtained from Auckland Council GeoMaps.

Coordinates obtained from Datanest.

ENGEO

LOG OF AUGER HA02

Geotechnical Investgation
617 New North Road
Kingsland, Auckland

Client : Pink Beluga Civil Limited
Client Ref. : 20034.000.001
Date : 29/03/2022
Hole Depth : 4 m
Hole Diameter : 50 mm

Shear Vane No : 3333
Logged By : JT
Reviewed By : HF
Latitude : -36.873366
Longitude : 174.736102

Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Graphic Symbol	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded	Scala Penetrometer Blows per 100mm 2 4 6 8 10 12
	TOPSOIL	OL	TOPSOIL.	▒			M	N/A	UTP	
0.5	FILL	ML	[FILL] Clayey SILT with minor fine to coarse sand and trace fine gravel; dark brown. Low plasticity. Gravel is sub-angular.	▩				H	UTP	
1.0	EAST COAST BAYS FORMATION	CH	Silty CLAY; brown with orange, grey and dark brown streaks. High plasticity.	▤	39			VSt-H	200+	
1.5			1.2 m - Becomes intermixed orange and grey.	▤					164/37 189/47 141/62	
2.0		ML	Clayey SILT with trace sand; intermixed grey and orange. Low plasticity.	▥	38	▼		VSt	149/53 102/42	
2.5			2.6 m - Becomes brownish grey with orange streaks and wet.	▥			W	St	99/40	
3.0 3.5 4.0			3.0 m - Becomes saturated.	▥	37 36		S	VSt	143/56 152/59 116/50 143/56	

End of Hole Depth: 4 m
Termination Condition: Target depth

Hand auger met target depth at 4 m.
Dip test showed groundwater at 2.30 m depth at 10:30 am, 29/03/22.
N/A = Not Assessed; UTP = Unable to Penetrate.
Elevation data obtained from Auckland Council GeoMaps.

Coordinates obtained from Datanest.

ENGEO

LOG OF AUGER HA03

Geotechnical Investgation
617 New North Road
Kingsland, Auckland

Client : Pink Beluga Civil Limited
Client Ref. : 20034.000.001
Date : 29/03/2022
Hole Depth : 4 m
Hole Diameter : 50 mm

Shear Vane No : 3333
Logged By : JT
Reviewed By : HF
Latitude : -36.873425
Longitude : 174.736013

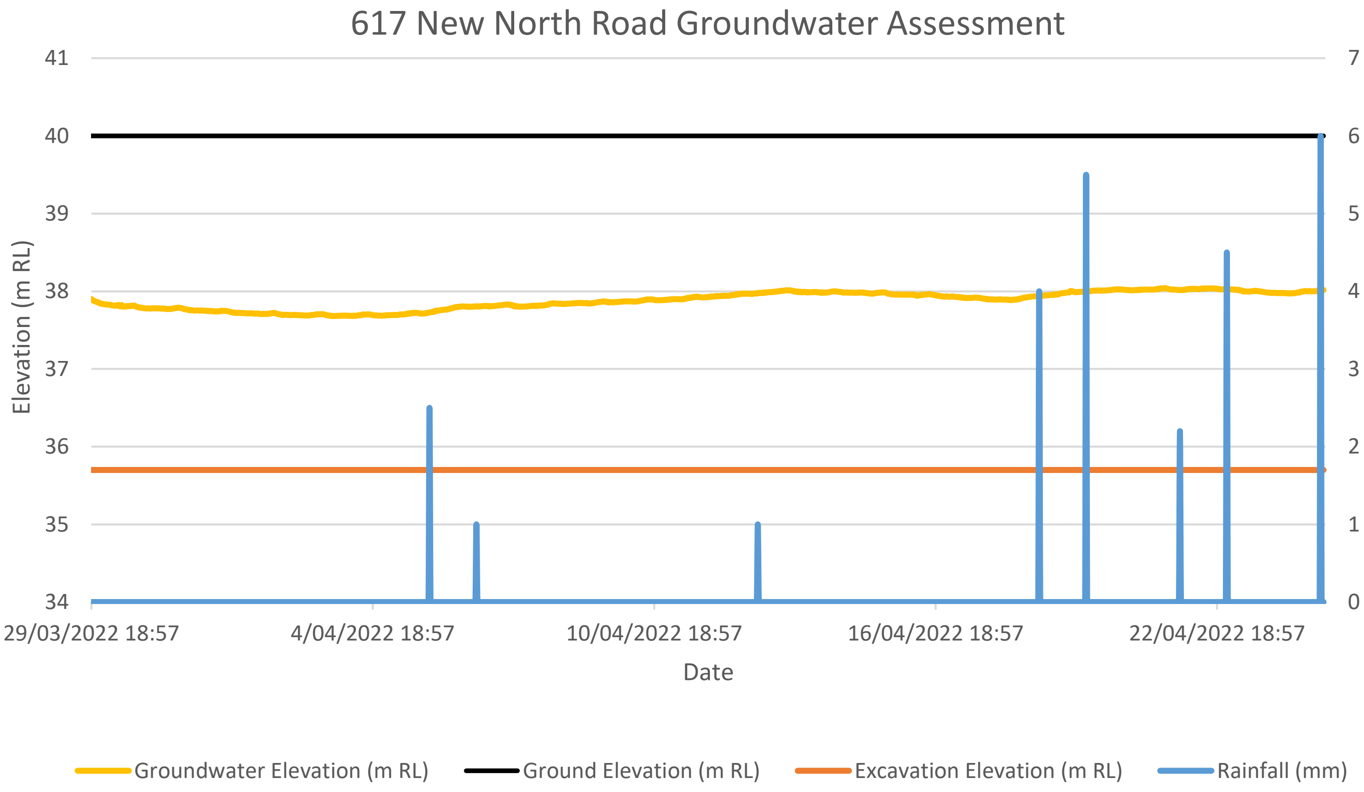
Depth (m BGL)	Material	USCS Symbol	DESCRIPTION	Graphic Symbol	Elevation (mRL)	Water Level	Moisture Cond.	Consistency/ Density Index	Shear Vane Undrained Shear Strength (kPa) Peak/Remolded	Scala Penetrometer Blows per 100mm 2 4 6 8 10 12
	TOPSOIL	OL	TOPSOIL.	▓				N/A	74/22	
	FILL	ML	[FILL] Clayey SILT with minor fine to coarse sand; blackish brown. Low plasticity.	▨				St		
0.5									71/16	
								H	UTP	
1.0	EAST COAST BAYS FORMATION	ML	Clayey SILT with trace sand; brown with orange and grey streaks. Low plasticity.	▥	39		M	VSt-H		
									UTP	
1.5			1.5 m - Becomes grey with orange streaks.						UTP	
									200+	
2.0					38				158/37	
									152/40	
2.5									168/53	
3.0					37				136/56	
			3.2 m - Becomes light grey and wet.				W		143/53	
3.5									141/51	
									161/65	
4.0			End of Hole Depth: 4 m Termination Condition: Target depth		36					

Hand auger met target depth at 4 m.
Standing groundwater was not encountered.
N/A = Not Assessed; UTP = Unable to Penetrate.
Elevation data obtained from Auckland Council GeoMaps.

Coordinates obtained from Datanest.

APPENDIX 4:

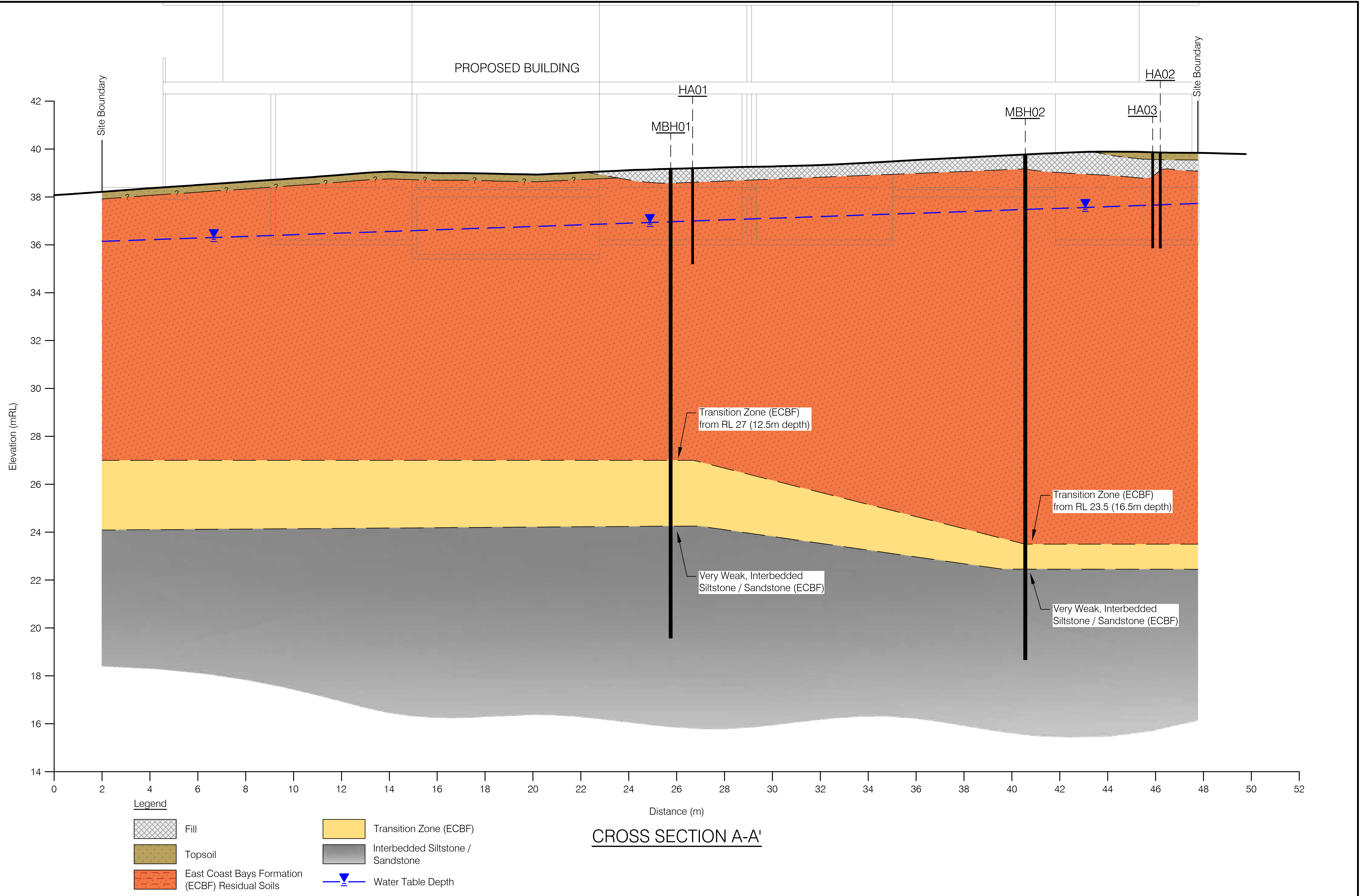
Ground Water Monitoring


617 New North Road Groundwater Assessment
Elevation (m RL)
41
40
39
38
37
36
35
34
Rainfall (mm / day)
7
6
5
4
3
2
1
0
29/03/2022 18:57
4/04/2022 18:57
10/04/2022 18:57
16/04/2022 18:57
22/04/2022 18:57
Date
Groundwater Elevation (m RL)
Ground Elevation (m RL)
Excavation Elevation (m RL)
Rainfall (mm)

APPENDIX 5:

Geotechnical Cross Section

DATE PLOTTED: 9 May 2022 4:34 pm BY : DOUG FINLAY



CROSS SECTION A-A'

		DRAFT		
A	9.5.22	Draft for review	DF	-
Rev	Date	Description	Drwn	Chkd

ENGEO

Auckland Office
8 Greydene Place
Takapuna 0622
Auckland
Tel: 09 972 2205
www.engeo.co.nz

Title:

GEOLOGICAL CROSS SECTION

Client: Pink Beluga Limited

Project:
617 New North Road
Kingsland
Auckland

Proj No: 20034.000.001

Designed: HF
Drawn: DF
Checked: -
Date: 9.5.22
Scale: 1:150

Figure No:
1
Size: A3
Rev: A

CAD File: Z:\Drafting\20001 to 21000\20034 - 617 New North Road\20034-GEX-617NewNthRd-0522.dwg
XREF's:

ORIGINAL FIGURE PRINTED IN COLOUR

APPENDIX 6:

Assessment Against the Auckland Unitary Plan

Table A1: E7.6.1.6 Dewatering or Groundwater Level Control Assessment Summary

<table>
<tr><th colspan="3">E7.6.1.6. Dewatering or groundwater level control associated with a groundwater diversion permitted under Standard E7.6.1.10</th></tr>
<tr><th>Criteria</th><th>Discussion</th><th>Assessment Findings</th></tr>
<tr><td>(1) The water take must not be geothermal water.</td><td>Under D1. High-use Aquifer Management Areas Overlay (Auckland Unitary Plan Operative in part), the site is not located in an area marked as a geothermal aquifer.</td><td>Criteria Met
✓</td></tr>
<tr><td>(2) The water take must not be for a period of more than ten days where it occurs in peat soils, or 30 days in other types of soil or rock.</td><td>Proposed excavations of up to 3.8 m are likely to extend below groundwater level. The piezometer on-site encountered groundwater at 2.1 m depth. The hand auger boreholes encountered groundwater between 2.2 m and 2.3 m depth. Therefore long-term drainage is likely.</td><td>Criteria Not Met
✗</td></tr>
<tr><td>(3) The water take must only occur during construction.</td><td>Proposed excavations up to 3.8 m depth are expected to extend below encountered groundwater level. Groundwater was encountered during our investigation between 2.1 m and 2.3 m. At this stage the basement is expected to be drained and therefore water take will be ongoing.</td><td>Criteria Not Met
✗</td></tr>
</table>

Table A2: E7.6.1.10. Diversion of Groundwater Assessment Summary

E7.6.1.10. Diversion of groundwater caused by any excavation (including trench) or tunnel		
Criteria	**Discussion**	**Assessment Findings**
(1) All of the following activities are exempt from the Standards E7.6.1.10 (2) – (6): (a) Pipes cables or tunnels including associated structures which are drilled or thrust and are less than 1.2 m in external diameter. (b) Pipes including associated structures up to 1.5 m in external diameter where a closed faced or earth pressure balanced machine is used. (c) Piles up to 1.5 m in external diameter are exempt from these standards. (d) Diversions for no longer than ten days. (e) Diversions for network utilities and road network linear trenching activities that are progressively opened, closed and stabilised where the part of the trench that is open at any given time is no longer than ten days.	Not Applicable.	Not Applicable – Proposed development does not include tunnelling, pipes / trenching where the groundwater is intersected or diverted for a period of time.
(2) Any excavation that extends below natural groundwater level, must not exceed: a) 1 ha in total area, and b) 6 m depth below the natural ground level.	Excavations will be less than 1 ha in total area and will not extend 6 m below natural ground level.	Criteria Met ✓
(3) The natural groundwater level must not be reduced by more than 2 m on the boundary of any adjoining site.	Proposed excavations are not expected to reduce groundwater level at the boundary by more than 2 m.	Criteria Met ✓

E7.6.1.10. Diversion of groundwater caused by any excavation (including trench) or tunnel		
(4) Any structure, excluding sheet piling, that remains in place for more than 30 days, that physically impedes the flow of groundwater through the site must not: (a) Impede the flow of groundwater over a length of more than 20 m. (b) Extend more than 2 m below the natural groundwater level.	Proposed excavations are expected to impede groundwater over a length greater than 20 m but will not extend more than 2 m below natural groundwater level.	Criteria Not Met ✗
(5) The distance to any existing building or structure (excluding timber fences and small structures on the boundary) on an adjoining site from the edge of any: (a) Trench or open excavation that extends below natural groundwater level must be at least equal to the depth of the excavation. (b) Tunnel or pipe with an external diameter of 0.2 - 1.5 m that extends below natural groundwater level must be 2 m or greater. (c) A tunnel or pipe with an external diameter of up to 0.2 m that extends below natural groundwater level has no separation requirement.	Proposed basement excavations are within 3.8 m of existing structures beyond the boundary.	Criteria Not Met ✗
(6) The distance from the edge of any excavation that extends below natural groundwater level, must not be less than: (a) 50 m from the Wetland Management Areas Overlay. (b) 10 m from a scheduled Historic Heritage Overlay. (c) 10 m from a lawful groundwater take.	(a) Under D8 Wetland Management Areas Overlay, Auckland Unitary Plan Operative in part, there are no Wetland Management Areas within 50 m of the site. (b) Under D17. Historic Heritage Overlay, Auckland Unitary Plan Operative in part, there is a Historic Overlay within 10 m of the site. (c) There are no known groundwater takes within 10 m of the site.	Criteria Not Met ✗