Stargate Property

Proposed Residential Development
194-214 Oxford Street & 2 Nelson Street, Bondi Junction NSW

Preliminary Geotechnical Assessment

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28 February 2025

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For and on behalf of
AssetGeoEnviro

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Executive Summary

This Preliminary Geotechnical Assessment report has been prepared by AssetGeoEnviro (Asset) to accompany a State Significant Development Application (SSDA) for a shop top housing development at 194-214 Oxford Street, 2 Nelson Street and part of Osmund Lane, Bondi Junction. The site is made up of nine (9) lots. The legal description of the site is outlined in Table 1.

Table 1 – Legal Description

Property Address	Title Description
194 Oxford Street Bondi Junction	Lot 10 in DP260116
196 Oxford Street Bondi Junction	Lot 11 in DP260116
198 Oxford Street Bondi Junction	Lot 12 in DP 260116
200 Oxford Street Bondi Junction	Lot 13 in DP260116
204 Oxford Street Bondi Junction	Lot 16 in DP68010 Lot 1 in DP79947
214 Oxford Street Bondi Junction	Lot 1 in DP708295
2 Nelson Street Bondi Junction	Lot 1 in DP583228
Part of Osmund Lane	Lot 1 in DP1300781

This report has been prepared to address the Secretary's Environmental Assessment Requirements (**SEARs**) issued for the project (SSD-77175998).

This report concludes that the proposed development is suitable and warrants approval subject to the implementation of the following mitigation measures:

- Design and construction to be carried out in accordance with the recommendations in this report.

Following the implementation of the above mitigation measures, the remaining impacts are appropriate.

Assessment

The building above Level 9 will have no additional impact on geotechnical conditions to the Site and surrounding area including ground movements and adjacent infrastructure, vibrations, groundwater, surface water, salinity management, and acid sulfate soil management.

Statement

We confirm that the proposed development is suitable for the site regarding the geotechnical impacts outlined in this report.

1. Introduction

1.1 General

This report presents the results of a Preliminary Geotechnical Assessment for a proposed residential development at 194-214 Oxford Street and 2 Nelson Street in Bondi Junction NSW (the Site). The assessment was commissioned on 17 March 2021 by Mr Igal Leis of Stargate Property. The work was carried out in accordance with the email proposal by AssetGeoEnviro (Asset) dated 12 March 2021.

Drawings supplied to us for this investigation comprised:

- Architectural Plans (by SJB Architects, reference 6289, revision 2, drawing nos DA-1000 to 1003, 1011 to 1029, 1401 to 1404, 1501, 1502, 6020, 6101,6102,6103, 6110 dated 14 February 2025).

Following a design excellence competition, development consent was granted to DA-400/2021 (herein, referred to as the parent development consent) which authorised demolition of existing buildings and the construction of a shop top housing development compromising ground floor retail and 10 storeys of residential apartments above the retail podium, across two tower buildings (herein referred to as Building A and Building B). Subsequently, an amending DA (DA-360/2023) was approved on 28 August 2024 which amended the Basement Levels 4, 3, 2 and 1 and the Ground Floor Level of the approved development under the parent development consent.

The proposed SSDA generally seeks approval for the redevelopment of 194-214 Oxford Street, 2 Nelson Street and part of Osmund Lane, Bondi Junction, proposing to retain key design principles in accordance with the parent consent. The proposal will provide additional residential dwellings, in accordance with the in-fill affordable housing provisions under the State Environmental Planning Policy (Housing) 2021 and incorporate a 30% increase in Gross Floor Area (GFA) and building height.

The development of the site has physically commenced pursuant to the development consent, with demolition and excavation completed. Construction Certification has been obtained, and construction is intended to continue for the lower portion of the building (up to Level 8).

Simultaneously with the construction of the lower parts of the building, the proponent seeks approval for new works to the remaining levels of the building (above level 9) as well as the internal fit out and servicing for the whole of the building (Basement to Level 16).

It is intended that the relationship between the approval of the SSDA and the existing consents be managed through the imposition of a condition pursuant to s 4.17(1)(b) of the EP&A Act and lodgement of a Notice of Modification pursuant to cl. 67 of the EP&A Regulation to ensure consistency across all development consents.

Specifically, this SSDA seeks development consent for:

<u>Proposed New Works Subject of this SSDA:</u>

- Construction of Levels 9 – 16 of the residential towers including Buildings A (Western Tower) and Building B (Eastern Tower) comprising:
    - Building A (Western Tower, Residential Levels 9 -13) – with a maximum height of 42.5m
    - Building B (Eastern Tower, Residential Levels 9 -16) – with a maximum height of 54.0m
    - Communal open space on Level 11(Building A)
    - Plant and lift overrun.
    - Public Domain Works
- Internal fit out of Level 09 – 16

Proposed Amendments to Existing Parent Development Consent

- Internal fit out from Basement Levels 01 – 04.
- Internal fit out from Ground Level to Level 08.
- The allocation of 1,708m^2 of affordable housing on Levels 1,2 and 3 of Building A and Building B.
- Additional services to overall development including an additional plant area at ground floor and an addition of a second substation.
- Basement services, including additional parking spaces and updated storage and waste storage areas.
- Awning over the ground retail along Oxford St and addition of a glazing window to create visual continuation from the neighbouring retail.

Cumulative Development (Existing Parent Development Consent and Subject SSDA)

- Construction of a shop-top housing development, comprising a podium with ground floor retail, two residential towers (Building A and Building B) as well as four levels of basement parking and associated public domain works.
  - The delivery of a total of 11,288m^2 of GFA.
  - 467m^2 of retail GFA.
  - 85 apartments, equating to a total residential GFA of 10,792m^2 including 1,708m^2 (17 apartments) of affordable housing GFA.
  - 29m2 GFA for communal amenities, incl. WC, steam room and sauna
  - The apartments will comprise the following mix:
    - 1 bedroom 2 (2%)
    - 2 bedroom 35 (42%)
    - 3+ bedroom 48(56%)
  - 4 levels of basement for 138 car parking spaces and 45 motorbike parking spaces, with vehicular access from Osmund Lane.
  - Storage areas and services.
  - Communal open space and associated landscaping.

1.2 Scope of Work

The main objectives of the Preliminary Geotechnical Assessment were to assess the surface conditions and likely subsurface conditions and to provide comments and preliminary recommendations relating to:

- Key geotechnical constraints to the development.
- Excavation conditions and methodology.
- Subgrade preparation and earthworks.
- Suitable foundation options.
- Allowable bearing pressure for shallow foundation / piles and shaft adhesion
- Groundwater levels and dewatering requirements.
- Underpinning.

The following scope of work was carried out to achieve the project objectives:

- A review of existing regional maps and reports relevant to the site held within our files.
- Visual observations of surface features by a Senior Principal Geotechnical Engineer (carried out on 22 March 2021). Selected site photos are included in this report.
- Engineering assessment and reporting.

This report must be read in conjunction with the attached "Important Information about your Geotechnical Report" in Appendix A. Attention is drawn to the limitations inherent in site investigations and the importance of verifying the subsurface conditions inferred herein.

2. Site Description

The site is located at 194 to 214 Oxford Street and 2 Nelson Street in Bondi Junction. It is irregularly shaped with a total site area of approximately 2,480m^2 (2,599.1m^2 including the land beneath Osmund Lane) with a northern frontage to Sydney Enfield Drive, an eastern frontage to Nelson Street, a southern frontage to Oxford Street and western frontage to York Road, as shown in Figure 1. The eastern suburbs railway tunnel is located about 75m to the northeast.

Topographically, the site is located in gently undulating terrain with ground surface slopes less than about 5° to 10°. Locally, the ground surface slopes down to the northeast.

At the time of the investigation, the site was occupied by residential garages and dwelling at 2 Nelson Street (Photos 1 to 3), residential terraces at 194 to 200 Oxford Street (Photos 6 and 9), a car rental premises at 202 to 212 Oxford Street (Photo 8), a commercial restaurant at 214 Oxford Street (Photo 8), and the western end of Osmund Lane separating the property at 2 Nelson Street from the rest of the development (Photos 5 and 7). Figure 2 shows an air photo of the current site development.

Overall, the existing structures were in moderate to poor condition, with signs of cracking in the garages on Nelson Street, and significant cracking and wall movement of the terrace at 194 Oxford Street.

Site drainage is generally to the northeast and east.

The site is mostly developed with only minor areas of landscaping and vegetation (grasses, some trees).

An historical air photo of the site from 1943 is shown in Figure 3. The residential development at 2 Nelson Street appears to have been present then, as well as the terraces along Oxford Street, but not the commercial restaurant at 214 Oxford Street. Subsequently, adjoining developments to the north and west were demolished to allow for construction of Syd Enfield Drive and York Street.

3. Subsurface Conditions

3.1 Geology

The 1:100,000 Sydney Geological Map indicates the site is underlain by Hawkesbury Sandstone to the west and Quaternary sand dune deposits to the east. The approximate delineation is shown in Figure 1. The sandstone usually weathers to form residual clay soils of medium to high plasticity. However, previous investigations in the area typically indicates minimal weathering profile with reasonably competent bedrock overlain by sand deposits, and bedrock increasing in quality with depth.

3.2 Subsurface Conditions

A generalised geotechnical model for the site based on nearby site investigations has been developed is shown in Table 2. This model is to be used only for preliminary planning only and must not be used for detailed design or construction.

Table 2 - Generalised Site Geotechnical Model

Unit	Origin	Description	Depth to Top of Unit [1] (m)	Unit Thickness [1] (m)
1	Fill	CONCRETE overlying: SAND with trace gravel, loose	Ground surface	Up to 1
2	Dune Deposit	SAND, fine to medium grained, density generally increasing with depth from very loose to loose at the surface, to dense and very dense at the base	Up to 1	Up to 7
3a	Bedrock[2]	SANDSTONE, fine to medium grained, highly weathered, low strength, assessed Class 4 Sandstone	1 to 7	1 to 1.5
3b	Bedrock[2]	SANDSTONE, fine to medium grained, slightly weathered to fresh, medium to high strength, assessed Class 3 to 2 Sandstone	1 to 7	5 to 10
3c	Bedrock[2]	SANDSTONE, fine to medium grained, slightly weathered to fresh, high strength, assessed Class 2 to 1 Sandstone	10 to 15	– –

Notes:

1. The depths and unit thicknesses are based on the information from the test locations only and do not necessarily represent the maximum and minimum values across the site.
2. Rock classification to Pells, P.J.N., Mostyn, G. & Walker, B.F., Foundations on Sandstone and Shale in the Sydney Region, Australian Geomechanics Journal, December 1998.

3.3 Groundwater

Groundwater is anticipated as an intermittent perched water table within the soils over bedrock, and within the fractures and defects of the underlying bedrock.

4. Discussions & Recommendations

The proposed development area is indicated in the Concept Plan shown in Figure 4. The proposed basement extents are located within the south-western and north-eastern parts of the site.

Based on a basement finished floor level approximately 9m and 12m below existing ground level, it is assessed that the excavation will be through variable depths of sandy soils and into sandstone bedrock.

Key geotechnical constraints to the development include vibrations during excavation, groundwater control (during construction and long-term), temporary shoring, permanent retaining, foundation conditions. The railway tunnel located to the northeast is assessed to be too far from the site to be impacted by, or be a constraint to, the proposed development.

Recommendations for design and construction of the development are provided in the following sections.

4.1 Construction Sequence

The following construction sequence is suggested for the basement level for the development:

1. Demolish existing buildings.
2. Remove existing pavements / concrete slabs.
3. Install temporary shoring around the basement perimeter socketed nominally into rock.

4. Excavate to bulk excavation level, installing temporary rock anchors progressively at multiple levels including a toe bolt to provide toe stability.
5. Carry out detail excavations for pad footings and lift pits.
6. Construct the lower basement ground floor.
7. Pour lower basement roof and continue up to existing ground surface level to provide permanent support to the excavation.

4.2 Temporary Shoring

It is understood that permanent batter slopes are not proposed for the development. The proposed depth of excavation, the presence of groundwater, and the lack of clearance between the basement and boundary would preclude temporary batters, and therefore temporary shoring will be required. Depending on the design of the shoring, it could also be incorporated into the permanent foundation and retaining works.

Several possible shoring systems could be considered for the site. These are summarised in Table 3 together with a brief description of the advantages and disadvantages of each.

Table 3 – Summary of Shoring Options

Option	Method	Advantages	Disadvantages
1	Conventional shoring with soldier piles and steel walers, or soldier piles and shotcrete infill panels	Relatively low cost	Risk of instability and loss of ground. Forms a poor seal against groundwater.
2	Steel sheet pile (driven or hydraulically installed)	Rapid installation. Lower cost than Option 3. Low permeability water barrier. Amenable to joint caulking.	Vibration may not be acceptable for adjoining developments. Permanent wall required.
3a or 3b	Contiguous or Secant bored piles	Can form part of the permanent structure. Minimum noise and vibration. Can maximise site building space as no temporary wall is required. Permanent waterproofing can be incorporated. Low permeability water barrier (secant piling very low permeability compared to contiguous piling).	For secant piles, ensuring complete contact of all piles over full pile length may be difficult. Additional finishing may be required following excavation if a 'smooth' internal wall is required. Relatively high cost.

Based on the advantages and disadvantages listed in Table 3, we recommend a contiguous (Option 3a) or secant (Option 3b) pile wall retention system for the basement excavation. We consider the geotechnical risks associated with Option 1 (predominantly excavation support) to be unacceptably high. Option 2 is not likely to be suitable due to the depth of excavation support and adjacent structures.

The founding depth of the retaining wall piles is a function of: -

- the required socket depth to achieve adequate embedment to resist overturning; and
- the required load carrying capacity if the piles are to be incorporated into the permanent works.

The higher quality sandstone anticipated to be encountered in the bulk excavation is likely to be suitable to stand unsupported and shoring piles could be terminated within this unit without the need to extend below bulk excavation level. This will be subject to adopting of a reduced bearing pressure and inspection of the rock face with remedial measures as required. Overturning resistance and toe kickout resistance would be provided by multiple rows of rock anchors.

Design of temporary shoring for carrying vertical loading should be in accordance with Section 4.3, and for lateral pressures, it should be in accordance with Section 4.6.

Detailed construction supervision, monitoring and inspections will be required during the piling and subsequent bulk excavation to ensure an adequate standard of workmanship and to minimise potential problems.

4.3 Footings

Suitable footings might comprise a slab on ground with pad footings under columns and strip footings under walls at the basement level, with shoring piles providing perimeter footings.

Edge beams for slabs, pad footings, and rock-socketed piles may be designed for the parameters in Table 4. Shoring piles must be founded on assessed Class 3 or better sandstone.

Table 4 – Footing Design Parameters

Founding Stratum	Maximum Allowable (Serviceability) Values (kPa)			Ultimate Strength Limit State Values (kPa)			Typical E_{field} MPa
	End Bearing†	Shaft Friction: Compression #	Shaft Friction: Tension	End Bearing†	Shaft Friction: Compression #	Shaft Friction: Tension*	
Class 4 Sandstone **(not for shoring piles)**	Max. 3,000	350	175	10,500	1,050	525	100-700
Class 3/2 Sandstone [shoring piles]	Max. 6,000 [4,000]	600 [n/a]	300 [n/a]	18,000 [12,000]	1,800 [n/a]	900 [n/a]	350-1,200

Note: Parameters for Class 4/5 Shale provided for strip and pad footings and bored piles only – these should not be used for CFA, CIS, or Steel Screw piles.

* Uplift capacity of piles in tension loading should also be checked for inverted cone pull out mechanism.

clean socket of roughness category R2 or better is assumed

† [ ] end bearing for shoring piles reduced to allow for edge effects

[n/a] not applicable for shoring piles

In accordance with AS2159-2009 "Piling–Design and Installation", for limit state design, the ultimate geotechnical pile capacity shall be multiplied by a geotechnical reduction factor (Φg). This factor is derived from an Average Risk Rating (ARR) which considers geotechnical uncertainties, redundancy of the foundation system, construction supervision, and the quantity and type of pile testing (if any). Where testing is undertaken, or more comprehensive ground investigation is carried out, it may be possible to adopt a larger Φg value that results in a more economical pile design. Further geotechnical advice will be required in consultation with the pile designer and piling contractor, to develop an appropriate Φg value.

Settlements for footings on rock are anticipated to be about 1% of the minimum footing dimension, based on serviceability parameters as per Table 4. Settlements for pad footings on clay are anticipated to be up to about 15mm where loading does not exceed the maximum allowable values.

Options for piles include:

Bored Piles. It assessed that the construction of sockets would require the use of a truck-mounted drilling rig. It is also assessed that the bored pile holes would require liners to support the overburden soils, although some over break and minor fretting should be allowed for. Groundwater may be expected within bored pile holes and dewatering by a down-hole pump may be required to limit softening of the bases prior to concreting.

Continuous Flight Auger (CFA) Piles. CFA piles are constructed by drilling a hollow-stemmed continuous flight auger to the required founding depth. Concrete is then injected under pressure through the auger stem as the auger is extracted from the soil. The reinforcing cage is then inserted upon completion of the concreting process. Pile diameters vary from 300mm to 1200mm. Drilled spoil is produced during CFA piling and must subsequently be removed from the site. CFA piles are considered non-displacement piles as defined in AS2159. Examples of CFA piles are Frankipile “Atlas” type piles or Vibropile “Omega” type piles. A high-capacity piling rig will be required to socket into Class 3 Sandstone but may not be able to penetrate through Class 2 or 1 Sandstone.

An experienced Geotechnical Engineer should review footing designs to check that the recommendations of the geotechnical report have been included and should assess footing excavations to confirm the design assumptions.

4.4 Earthworks

4.4.1 Excavation

The excavation for the proposed development is anticipated to be partially within soils, and partially within sandstone bedrock. Excavation within the soils and extremely weathered bedrock would be achievable using conventional earthmoving equipment (i.e. hydraulic excavator bucket).

Excavation within the less weathered bedrock will likely require the use of ripper tooth fitted to a hydraulic excavator bucket, a dozer fitted with ripper tooth, or a hydraulic hammer fitted to an excavator, possibly supplemented by rock saw and rock splitting techniques.

4.4.2 Vibration Management

Australian Standard AS 2187: Part 2-2006 recommends the frequency dependent guideline values and assessment methods given in BS 7385 Part 2-1993 “Evaluation and measurement for vibration in buildings Part 2” as they “are applicable to Australian conditions”. The standard sets guide values for building vibration based on the lowest vibration levels above which damage has been credibly demonstrated. These levels are judged to give a minimum risk of vibration-induced damage, where the minimal risk for a named effect is usually taken as a 95% probability of no effect.

Sources of vibration that are considered in the standard include demolition, blasting (carried out during mineral extraction or construction excavation), piling, ground treatments (e.g. compaction), construction equipment, tunnelling, road and rail traffic and industrial machinery.

For residential structures, BS 7385 recommends vibration criteria of 7.5 mm/s to 10 mm/s for frequencies between 4 Hz and 15 Hz, and 10 mm/s to 25 mm/s for frequencies between 15 Hz to 40 Hz and above. These values would normally be applicable for new residential structures or residential structures in good condition. Higher values would normally apply to commercial structures, and more conservative criteria would normally apply to heritage structures.

However, structures can withstand vibration levels significantly higher than those required to maintain comfort for their occupants. Human comfort is therefore likely to be the critical factor in vibration management.

Excavation methods should be adopted which limit ground vibrations at the adjoining developments to not more than 10mm/sec. Vibration monitoring is recommended to verify that this is achieved. However, if the contractor adopts methods and/or equipment in accordance with the recommendations in Table 5 for a ground vibration limit of 5mm/sec, vibration monitoring may not be required.

The limits of 5mm/sec and 10mm/sec are expected to be achievable if rock breaker equipment or other excavation methods are restricted as indicated in Table 5.

Table 5 – Recommendations for Rock Breaking Equipment

Distance from adjoining structure (m)	Maximum Peak Particle Velocity 5mm/sec		Maximum Peak Particle Velocity 10mm/sec*	
	Equipment	Operating Limit (% of Maximum Capacity)	Equipment	Operating Limit (% of Maximum Capacity)
1.5 to 2.5	Hand operated jackhammer only	100	300 kg rock hammer	50
2.5 to 5.0	300 kg rock hammer	50	300 kg rock hammer or 600 kg rock hammer	100 50
5.0 to 10.0	300 kg rock hammer or 600 kg rock hammer	100 50	600 kg rock hammer or 900 kg rock hammer	100 50

* Vibration monitoring is recommended for 10mm/sec vibration limit.

At all times, the excavation equipment must be operated by experienced personnel, per the manufacturer's instructions, and in a manner, consistent with minimising vibration effects.

Use of other techniques (e.g. chemical rock splitting, rock sawing), although less productive, would reduce or possibly eliminate risks of damage to adjoining property through vibration effects transmitted via the ground. Such techniques may be considered if an alternative to rock breaking is necessary. If rock sawing is carried out around excavation boundaries in not less than 1m deep lifts, a 900kg rock hammer could be used at up to 100% maximum operating capacity with an assessed peak particle velocity not exceeding 5 mm/sec, subject to observation and confirmation by a Geotechnical Engineer at the commencement of excavation.

It is pointed out that the rock classification system used in Table 2 is intended primarily for use in the design of foundations and is not intended to be used to directly assess rock excavation characteristics. Excavation contractors should refer to the detailed engineering logs, core photographs, laboratory strength tests, and inspection of rock core, and should not rely solely on the rock classifications presented in geotechnical engineering reports when assessing the suitability of their excavation equipment for the proposed development. Further geotechnical advice must be sought if rock excavation characteristics are critical to the proposed development.

It should be noted that vibrations that are below threshold levels for building damage may be experienced at adjoining developments. Rock excavation methodology should also consider acceptable noise limits as per the "Interim Construction Noise Guideline" (NSW EPA).

4.4.3 Subgrade Preparation

The following general recommendations are provided for subgrade preparation for earthworks, pavements, slab-on-ground construction, and minor structures:

- Strip existing fill and topsoil. Remove unsuitable materials from the site (e.g. material containing deleterious matter). Stockpile remainder for re-use as landscaping material or remove from site.

- Excavate residual clayey soils and rock where required to design subgrade level, stockpiling for re-use as engineered fill or remove to spoil. Rock could be stockpiled separately from clayey soils, for select use beneath pavements.
- Where rock is exposed in bulk excavation level beneath pavements, rip a further 150mm.
- Where rock is exposed at footing invert level, it should be free of loose, “drummy" and softened material before concrete is poured.
- Where soil is exposed at bulk excavation level (e.g. for ancillary structures beyond the basement), compact the upper 150mm depth to a dry density ratio (AS1289.5.4.1–2007) not less than 100% Standard.
- Areas which show visible heave under compaction equipment should be over-excavated a further 0.3m and replaced with approved fill compacted to a dry density ratio not less than 100%.

Further advice should be sought where filling is required to support major structures.

Any waste soils being removed from the site must be classified in accordance with current regulatory authority requirements to enable appropriate disposal to an appropriately licensed landfill facility.

4.4.4 Filling

Where filing is required, place in horizontal layers over prepared subgrade and compact as per Table 6.

Table 6 – Compaction Specifications

Parameter	Cohesive Fill	Non Cohesive Fill
Fill layer thickness (loose measurement): • Within 1.5m of the rear of retaining walls • Elsewhere	 0.2m 0.3m	 0.2m 0.3m
Density: • Beneath Pavements • Beneath Structures • Upper 150mm of subgrade	 ≥ 95% Std ≥ 98% Std ≥ 100% Std	 ≥ 70% ID ≥ 80% ID ≥ 80% ID
Moisture content during compaction	± 2% of optimum	Moist but not wet

Filling within 1.5m of the rear of any retaining walls should be compacted using lightweight equipment (e.g. hand-operated plate compactor or ride-on compactor not more than 3 tonnes static weight) to limit compaction-induced lateral pressures.

Any soils to be imported onto the site for backfilling and reinstatement of excavated areas should be free of contamination and deleterious material and should include appropriate validation documentation in accordance with current regulatory authority requirements which confirms its suitability for the proposed land use.

4.4.5 Batter Slopes

Recommended maximum slopes for permanent and temporary batters are presented in Table 7.

Table 7 – Recommended Maximum Dry Batter Slopes

Unit	Maximum Batter Slope (H : V)	
	Permanent	Temporary
Medium Dense Sand (or denser)	3 : 1	2 : 1
Class 4 (or better) Sandstone	vertical *	vertical *

* subject to inspection by a Geotechnical Engineer and carrying out remedial works as recommended (e.g. shotcrete, rock bolting).

4.5 Groundwater Control

Limited groundwater observations made for this investigation are described in Section 3.3. The observations indicate that groundwater is unlikely to be a constraint to the proposed development. However, good practice should be followed to cater for potential groundwater, such as designing retaining walls with adequate subsoil drainage. Further geotechnical advice must be sought if significant groundwater is encountered during construction.

4.6 Excavation Support

Excavation of soil and rock results in stress changes in the remaining material and some ground movement is inevitable. The magnitude and extent of lateral and vertical ground movements will depend on the design and construction of the excavation support system. Experience and published data suggest that lateral movements of an adequately designed and installed retention system in soil and weathered rock will typically be in the range of 0.2% to 0.5% of the retained height. The extent of the horizontal movement behind the excavation face typically varies from 1.5 to 3 times the excavated height.

4.6.1 Excavation Support Construction Methodology

Where temporary or permanent batter slopes as per Section 4.4.5 cannot be accommodated in the development or are not desired, temporary shoring and/or permanent retaining will be required.

Design of retaining walls will need to consider both long-term (i.e. permanent) and short-term (i.e. during construction) loading conditions, as well as the possible impact on adjoining developments.

In the long term, the ground floor slab will provide bracing at the top of the wall and the basement floor slab will provide bracing at the bottom of the wall. Therefore, basement retaining walls should be designed as braced walls for the long-term loading condition.

In the short term (i.e. during construction), the design of the basement retaining wall will depend on the method of construction adopted. Two common construction techniques include top–down and bottom–up construction. It is likely that bottom-up construction will be adopted for this site given the size and complexity.

Bottom-up construction typically involves:

- constructing the perimeter wall as contiguous or secant wall;
- excavating to basement subgrade level installing rock anchors as per design;
- pouring the basement floor slab and proceeding upwards;

- cutting the temporary anchors.

4.6.2 Excavation Support Design Parameters

Excavation support design can be relatively complex as it involves soil-structure interaction. Also, the pressures acting on the support will depend on a range of factors including the stiffness of the support, the construction sequence, external forces (e.g. surcharge loading), and varying groundwater conditions.

For relatively simple support systems (e.g. cantilever walls or anchored/propped walls with only one row of anchors/props, the design may be based on an Earth Pressure Approach and using closed-form solutions or simple analytical programs such as WALLAP.

For more complex support systems (e.g. multiple anchors/props), or where it is desired to optimise the design, more advanced numerical analysis tools are recommended (e.g. 2D Finite Element Method), which include more complex soil models that allow for stress re-adjustment to occur with wall movements. The use of 3D FEM software may also be appropriate depending on the excavation geometry and potential cost-savings by optimising the support design.

Earth Pressure Approach

Support systems designed using the Earth Pressure Approach may be based on the parameters given in Table 8.

Cantilever walls or walls within only a single row of anchors/props may be designed for a triangular earth pressure distribution with the lateral pressure being determined as follows:

$\sigma_z = K_{o,a,p}\ z\ \gamma$ where

σ_z = lateral earth pressure (kPa) at depth z

$K_{o,a,p}$ = earth pressure coefficient
o = 'at rest', a = 'active', p = 'passive'

z = depth (m)

γ = unit weight of soil / rock (kN/m^3)

Table 8 – Excavation Support Design Parameters (Earth Pressure Approach)

Material	Moist Unit Weight (γ_m) kN/m^3	'Active' Lateral Earth Pressure Coefficient (1) (K_a)	'At Rest' Coefficient (1) (K_o)	'Passive' Coefficient (2) (K_p)
Medium Dense Sand	19.0	0.3	0.5	N/A
Class 4 Sandstone (3)	22.0	0.1	0.3	15
Class 3 or better Sandstone (3)	24.0	0.0	0.0	30

Notes to table:

1. These values assume that some wall movement and relaxation of horizontal stress will occur due to the excavation. Actual in-situ K_0 values may be higher, particularly in the rock units.
2. Includes a reduction factor to the ultimate value of K_p to consider strain incompatibility between active and passive pressure conditions. Parameters assume horizontal backfill and no back of wall friction. **Passive pressures only applicable where shoring piles extend below the basement level.**
3. The values for rock assume no adversely dipping joints or other defects are present in the bedrock. All excavation rock faces should be inspected regularly by an experienced Geotechnical Engineer / Engineering Geologist as excavation proceeds.

The parameters for the 'at rest' condition (Ko) should be used for the design of lateral earth pressures where adjacent footings/structures are located within the 'zone of influence' of the wall. The 'zone of influence' may be taken as a line extending upwards and outwards at 45° above horizontal from the base of the wall. Piles for cantilever walls should be socketed below bulk excavation level by a depth at least equal to the retained height. For assessment of passive restraint embedded below excavation level, we recommend a triangular pressure distribution.

Walls supported by multiple rows of anchors/props may be designed for a uniform lateral earth pressure of 0.65 γ H Ka where γ = unit weight of the retained material, H = height of the wall, and Ka = earth pressure coefficient (Table 8). Piles for braced walls should be socketed at least 0.75m below basement subgrade level to provide toe "kick-in" resistance until the slab can be poured.

4.6.3 Surcharge

Allowance must also be made for surcharge loadings and footing loads from adjacent structures.

4.6.4 Hydrostatic Pressure

Where an adequate subsoil drainage system designed by an appropriately qualified and experienced Hydraulic / Stormwater Engineer is provided behind non-tanked retaining walls, no allowance for hydrostatic pressure would be necessary.

Where tanked retaining walls are to be adopted, they should be designed for a hydrostatic pressure based on an appropriate design groundwater level (refer to Section 4.5).

4.6.5 Underpinning

The proposed shoring should be designed to provide adequate lateral support to adjoining property, in which case underpinning is not required.

4.6.6 Ground Anchors

Prestressed anchoring of shoring / retaining walls can be adopted for the development, subject to obtaining permission from adjacent property owners/authorities where anchors extend outside the site boundaries.

Anchors could be inclined up to a maximum of 30° below horizontal if required to intercept bedrock / higher strength bedrock. Design of excavation support must be carried out by a suitably experienced and qualified structural/civil engineer. Requirements for rock support must be nominated or approved by the Geotechnical Engineer during excavation. Rock bolts may be designed for the parameters in Table 9.

Table 9 – Rock Bolting Design Parameters

Layer	Allowable Bond Stress
Class 4 Sandstone	250 kPa
Class 3 Sandstone	400 kPa
Class 2 Sandstone	500 kPa

The following should be noted during anchor design and construction:

- The contractor should adopt design values including an appropriate factor of safety relevant to the installation methodology and anchor type adopted.
- Anchor holes must be cleaned prior to grouting.
- Anchors should be check stressed to 125% of the nominal working load and then locked off at 60% to 80% of the working load.

4.7 Potential Impacts on Adjacent Developments

Potential geotechnical risks of construction on adjoining developments could include; vibration effects due to rock excavation, settlement/deflection of adjacent footings due to the basement excavation, and induced settlement due to groundwater drawdown. These risks have been discussed in the relevant sections of this report. We assess that if the development is designed and constructed in accordance with the recommendations given in this report, these effects are anticipated to have negligible impact and be within acceptable limits.

5. Geotechnical & Hydrogeological Monitoring Program

5.1 Acceptable Vibration & Deflection Limits

The contractor shall carry out excavation and construction activities so that the limits in Table 10 are not exceeded.

Table 10 – Vibration and Deflection Limits

Parameter	Limit		
Vertical settlement of ground surface at adjoining boundaries	5 mm		
Lateral deflection of temporary or permanent retaining works (measured at the top or any point of the retaining works)	5 mm		
Peak particle velocity[1] at foundations of any sensitive adjoining structure, at defined frequency ranges:	**1 to 10 Hz**	**10 to 50 Hz**	**50 to 100 Hz[2]**
Commercial, industrial buildings	20	20 to 40	40 to 50
Residential buildings	5	5 to 10	15 to 20
Sensitive structures (e.g., buildings under preservation order)	3	3 to 8	8 to 10

Notes:

1. The vibration limits noted above is within the 'safe' limits as defined in the German Standard DIN 4150-3, dated 2016: Structural vibration – Part 3: Effects of vibration on structures.
2. At frequencies above 100 Hz, the guideline values for 100 Hz can be applied as minimum values.

5.2 Monitoring System

5.2.1 Deflections / Settlement

Monitoring of deflections and settlements shall be carried out by a registered surveyor.

Survey points shall be established along the site boundaries where excavation is proposed, and adjoining property or movement-sensitive buried services are present within the depth-of-influence of the excavation. The depth-of-influence is defined as a line extending upwards and outwards at 45° above horizontal from the top of Class 4 or better sandstone.

Survey points shall be installed at a spacing of not more 10m. Survey measurements shall be taken:

- prior to the commencement of excavation
- immediately after installation of temporary retaining works
- immediately after bulk excavation
- immediately after basement floors constructed and before anchors are de-stressed
- immediately after anchors are de-stressed.

5.2.2 Vibration

Where excavation is carried out in accordance with Section 4.4.1, adopting a methodology for a maximum peak particle velocity of 5 mm/s, a permanent vibration monitoring system should not be required during the excavation works. However, we recommend that vibration levels at critical adjoining developments be measured at the commencement of rock excavation to confirm that vibrations being generated are below the target values and to provide guidance on modifying excavation techniques if the target values are exceeded.

5.3 Hold Points

Hold points shall be provided at the following stages to allow for inspection by a Geotechnical Engineer:

- At the commencement of shoring/pile installation.
- At the commencement of ground anchor installation.
- At the commencement of rock excavation.
- At the commencement of dewatering (if required)
- At the completion of bulk excavation.
- At the completion of detail footing excavation.

5.4 Contingency Plan

If the above listed acceptable limits are exceeded, the following works shall be carried out:

- The Project Geotechnical Engineer shall be notified immediately.
- Excavations adjacent to areas that have settled shall be backfilled with spoil or other suitable material.
- Additional bracing shall be installed adjacent to areas of temporary or permanent shoring.
- Excavation equipment shall cease work immediately, and vibration monitoring equipment shall be installed at locations selected by the Geotechnical Engineer to measure vibrations. If the vibration limit exceeds 10 mm/second, alternative equipment and/or methodology shall be used.

6. Limitations

In addition to the limitations inherent in site investigations (refer to the attached Information Sheets), it must be pointed out that the recommendations in this report are based on assessed subsurface conditions from limited investigations. To confirm the assessed soil and rock properties in this report, further investigation would be required such as coring and strength testing of rock and should be carried out if the scale of the development warrants, or if any of the properties are critical to the design, construction or performance of the development.

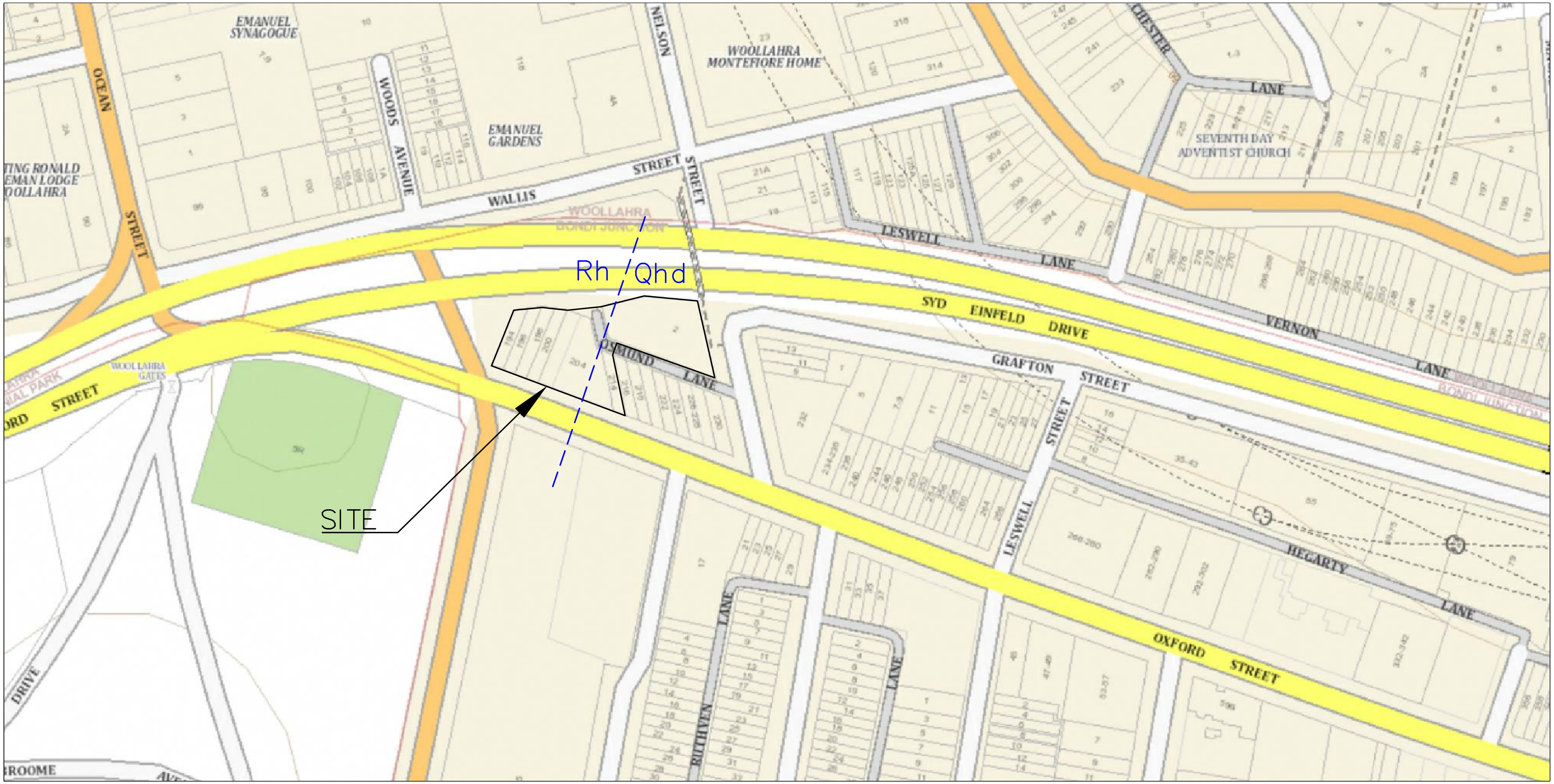
It is recommended that a qualified and experienced Geotechnical Engineer be engaged to provide further input and review during the design development; including site visits during construction to verify the site conditions and provide advice where conditions vary from those assumed in this report. Development of an appropriate inspection and testing plan should be carried out in consultation with the Geotechnical Engineer.

This report may have included geotechnical recommendations for design and construction of temporary works (e.g. temporary batter slopes or temporary shoring of excavations). Such temporary works are expected to perform adequately for a relatively short period only, which could range from a few days (for temporary batter slopes) up to six months (for temporary shoring). This period depends on a range of factors including but not limited to: site geology; groundwater conditions; weather conditions; design criteria; and level of care taken during construction. If there are factors which prevent temporary works from being completed and/or which require temporary works to function for periods longer than originally designed, further advice must be sought from the Geotechnical Engineer and Structural Engineer.

This report and details for the proposed development should be submitted to relevant regulatory authorities that have an interest in the property (e.g. Council) or are responsible for services that may be within or adjacent to the site (e.g. Sydney Water, Sydney Trains, Roads & Maritime Services), for their review.

Asset accepts no liability where our recommendations are not followed or are only partially followed. The document “Important Information about your Geotechnical Report” in Appendix A provides additional information about the uses and limitations of this report.

Figures



Approximate only – subject to detail survey.
Source: SixMaps.
This drawing is used to illustrate geotechnical features only and must not be used for any other purpose. Copyright of source drawing remains with SixMaps.

From Sydney 1:100,000 Geological Map
Rh = Hawkesbury Sandstone
Qhd = Quaternary Alluvium (sand dune deposit)

0 1:2,500 A4 100m

issue	date	description
A	22.3.21	Initial issue

assetgeoenviro
2.06/56 Delhi Rd
North Ryde NSW 2113
t: 02 9878 6005
e: info@assetgeoenviro.com.au

Proposed Residential Development
194–214 Oxford Street & 2 Nelson Street, Bondi Junction NSW
for
Stargate Property

Site Locality & Regional Geology

drawn: MAB
date: 22.3.2021
checked: MAB
scale: 1:2,500 A4
job no.: 6419
fig: 1
issue: A

Approximate only – subject to detail survey.
Source: Google Maps.
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0 1:1,000 A3 50m

issue	date	description
A	22.3.21	Initial issue

assetgeoenviro
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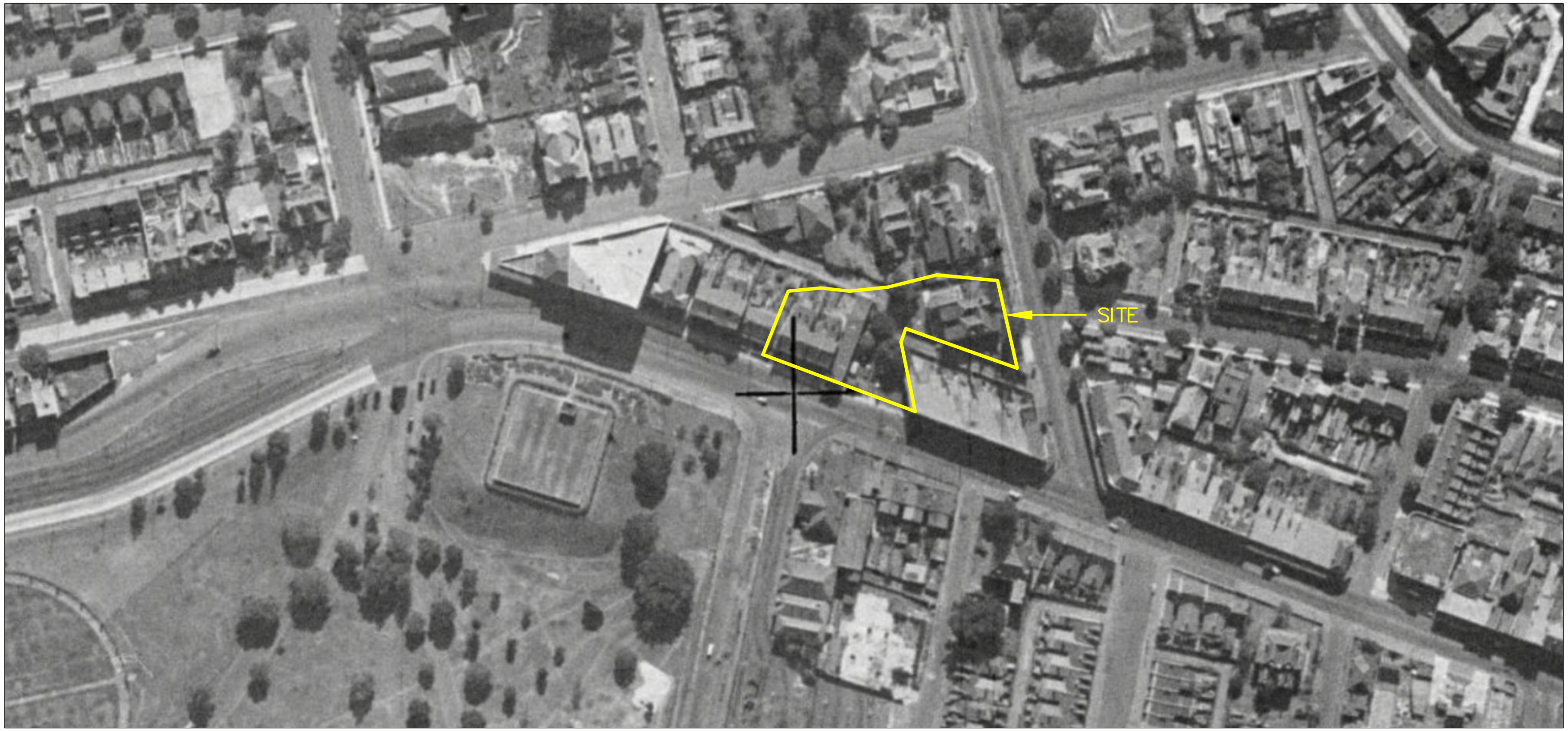
Proposed Residential Development
194–214 Oxford Street & 2 Nelson Street, Bondi Junction NSW
for
Stargate Property

Air Photo – Current

drawn: MAB
date: 22.3.2021
checked: MAB
scale: 1:1,000 A4

job no.: 6419
fig: 2
issue: A



Approximate only – subject to detail survey.
Source: Google Maps.
This drawing is used to illustrate geotechnical features only and must not be used for any other purpose. Copyright of source drawing remains with Google Maps.

0 1:2,500 A4 100m

issue	date	description
A	22.3.21	Initial issue

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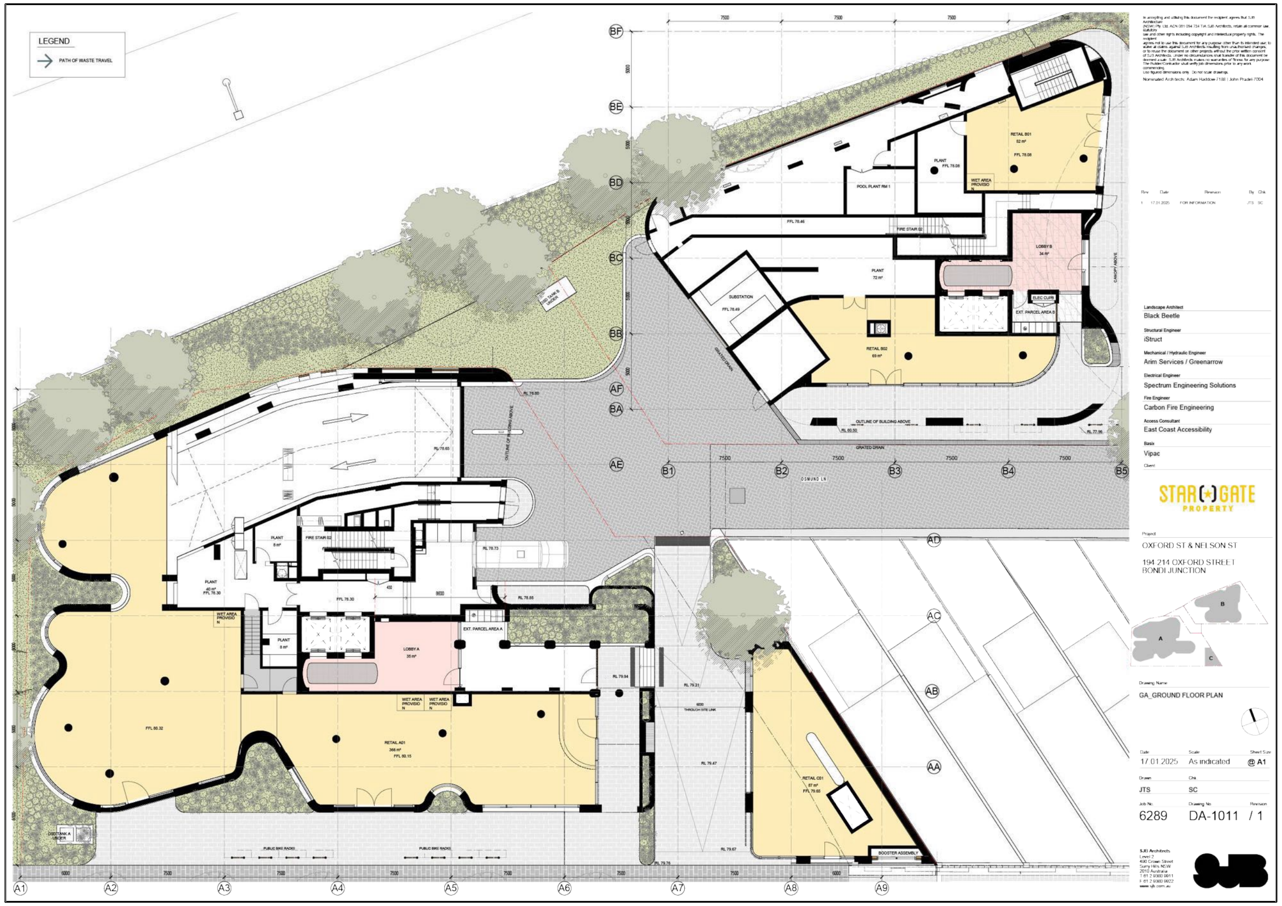
Proposed Residential Development
194–214 Oxford Street & 2 Nelson Street, Bondi Junction NSW
for
Stargate Property

Air Photo – 1943

drawn: MAB
date: 22.3.2021
checked: MAB
scale: 1:2,500 A4
job no.: 6419
fig: 3
issue: A



Approximate only – subject to detail survey.
Source: SJB Architects, Dwg DA–1011. This drawing is used to illustrate geotechnical features only and must not be used for any other purpose. Copyright of source drawing remains with SJB Architects.

0 1:500 A4 25m

issue	date	description
C	14.2.25	Updated plans
B	6.4.21	Updated plans
A	22.3.21	Initial issue

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Proposed Residential Development
194–214 Oxford Street & 2 Nelson Street, Bondi Junction NSW
for
Stargate Property

Concept Plan

drawn: MAB
date: 14.2.2025
checked: MAB
scale: 1:400 A4

job no.: 6419
fig: 4
issue: C

Appendix A

Important Information about your Geotechnical Report
Soil & Rock Explanation Sheets

Important Information about your Geotechnical Report

Scope of Services

The geotechnical report ("the report") was prepared in accordance with the contractual scope of services between the Client and AssetGeoEnviro ("Asset") for the specific site investigated. The scope of work may have been limited by factors like time, budget, access, or site disturbance.

Consult Asset before using the report if the project has changed. Asset won't be responsible for problems caused by project changes if not consulted.

Reliance on Data

Asset prepared the report using data provided by the Client and other individuals and organizations, including surveys, analyses, designs, maps, and plans. Asset has not verified the accuracy or completeness of the data except as stated in the report. Asset won't be liable for incorrect conclusions based on incorrect data, information, or conditions if they're concealed, withheld, misrepresented, or not fully disclosed.

Geotechnical Engineering

Geotechnical engineering heavily relies on judgment and opinion, making it less precise than other engineering disciplines. Reports are tailored to specific clients, projects, and needs, and may not be suitable for other clients or purposes. The report should only be used for its intended purpose unless additional geotechnical advice is obtained. If further geotechnical advice isn't obtained, the report can't be used if the proposed development's nature or details change.

Limitations of Site Investigation

The investigation program undertaken is a professional estimate of the scope of investigation required to provide a general profile of subsurface conditions. The data derived from the site investigation program and subsequent laboratory testing are extrapolated across the site to form an inferred geological model, and an engineering opinion is rendered about overall subsurface conditions and their likely behavior regarding the proposed development. Despite investigation, the actual conditions at the site might differ from those inferred to exist, since no subsurface exploration program, no matter how comprehensive, can reveal all subsurface details and anomalies.

The engineering logs are the subjective interpretation of subsurface conditions at a particular location and time, made by trained personnel. The actual interface between materials may be more gradual or abrupt than a report indicates.

Therefore, the recommendations in the report can only be regarded as preliminary. Asset should be retained during the project implementation to assess if the report's recommendations are valid and whether changes should be considered as the project proceeds.

Subsurface Conditions are Time Dependent

Subsurface conditions can be modified by changing natural forces or man-made influences. The report is based on conditions that existed at the time of subsurface exploration. Construction operations adjacent to the site, and natural events such as floods, or ground water fluctuations, may also affect subsurface conditions, and thus the continuing adequacy of a geotechnical report. Asset should be kept appraised of any such events and should be consulted to determine if any additional tests are necessary.

Verification of Site Conditions

Where ground conditions encountered at the site differ significantly from those anticipated in the report, either due to natural variability of subsurface conditions or construction activities, it is a condition of the report that Asset be notified of any variations and be provided with an opportunity to review the recommendations of this report. Recognition of change of soil and rock conditions requires experience, and it is recommended that a suitably experienced geotechnical engineer be engaged to visit the site with sufficient frequency to detect if conditions have changed significantly.

Reproduction of Reports

This report is the subject of copyright and shall not be reproduced either totally or in part without the express permission of this Company. Where information from the accompanying report is to be included in contract documents or engineering specification for the project, the entire report should be included to minimize the likelihood of misinterpretation from logs.

Report for Benefit of Client

The report has been prepared for the benefit of the Client and no other party. Asset assumes no responsibility and will not be liable to any other person or organization for or in relation to any matter dealt with or conclusions expressed in the report, or for any loss or damage suffered by any other person or organization arising from matters dealt with or conclusions expressed in the report (including without limitation matters arising from any negligent act or omission of Asset or for any loss or damage suffered by any other party relying upon the matters dealt with or conclusions expressed in the report). Other parties should not rely upon the report or the accuracy or completeness of any conclusions and should make their own inquiries and obtain independent advice in relation to such matters.

Data Must Not Be Separated from The Report

The report presents the site assessment and must not be copied in part or altered in any way.

Logs, figures, drawings, test results etc. included in our reports are developed by professionals based on their interpretation of field logs (assembled by field personnel) and laboratory evaluation of field samples. These data should not under any circumstances be redrawn for inclusion in other documents or separated from the report in any way.

Report Recommendations not Followed

Where the report's recommendations are not followed, there may be significant implications for the project (e.g., commercial, property, personal, or life loss). Consult Asset if you don't intend to follow all the report recommendations. Asset won't accept responsibility if all the report recommendations aren't followed.

Other Limitations

Asset will not be liable to update or revise the report to consider any events or emergent circumstances or fact occurring or becoming apparent after the date of the report.

Log Abbreviations & Notes

METHOD

borehole logs		excavation logs	
AS	auger screw *	NE	natural excavation
AD	auger drill *	HE	hand excavation
RR	roller / tricone	BH	backhoe bucket
W	washbore	EX	excavator bucket
CT	cable tool	DZ	dozer blade
HA	hand auger	R	ripper tooth
D	diatube		
B	blade / blank bit		
V	V-bit		
T	TC-bit		

* bit shown by suffix e.g. ADV

coring
NMLC, NQ, PQ, HQ

SUPPORT

borehole logs		excavation logs	
N	nil	N	nil
M	mud	S	shoring
C	casing	B	benched
NQ	NQ rods		

CORE—LIFT

¦	casing installed
⊢⊣	barrel withdrawn

NOTES, SAMPLES, TESTS

D	disturbed
B	bulk disturbed
U50	thin-walled sample, 50mm diameter
HP	hand penetrometer (kPa)
SV	shear vane test (kPa)
DCP	dynamic cone penetrometer (blows per 100mm penetration)
SPT	standard penetration test
N*	SPT value (blows per 300mm) * denotes sample taken
Nc	SPT with solid cone
R	refusal of DCP or SPT

USCS SYMBOLS

GW	Gravel and gravel-sand mixtures, little or no fines.
GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels
GM	Gravel-silt mixtures and gravel-sand-silt mixtures.
GC	Gravel-clay mixtures and gravel-sand-clay mixtures.
SW	Sand and gravel-sand mixtures, little or no fines.
SP	Sand and gravel sand mixtures, little or no fines.
SM	Sand-silt mixtures.
SC	Sand-clay mixtures.
ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity.
CL, CI	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays.
OL	Organic silts
MH	Inorganic silts
CH	Inorganic clays of high plasticity.
OH	Organic clays of medium to high plasticity, organic silt
PT	Peat, highly organic soils.

MOISTURE CONDITION

D	dry
M	moist
W	wet
Wp	plastic limit
Wl	liquid limit

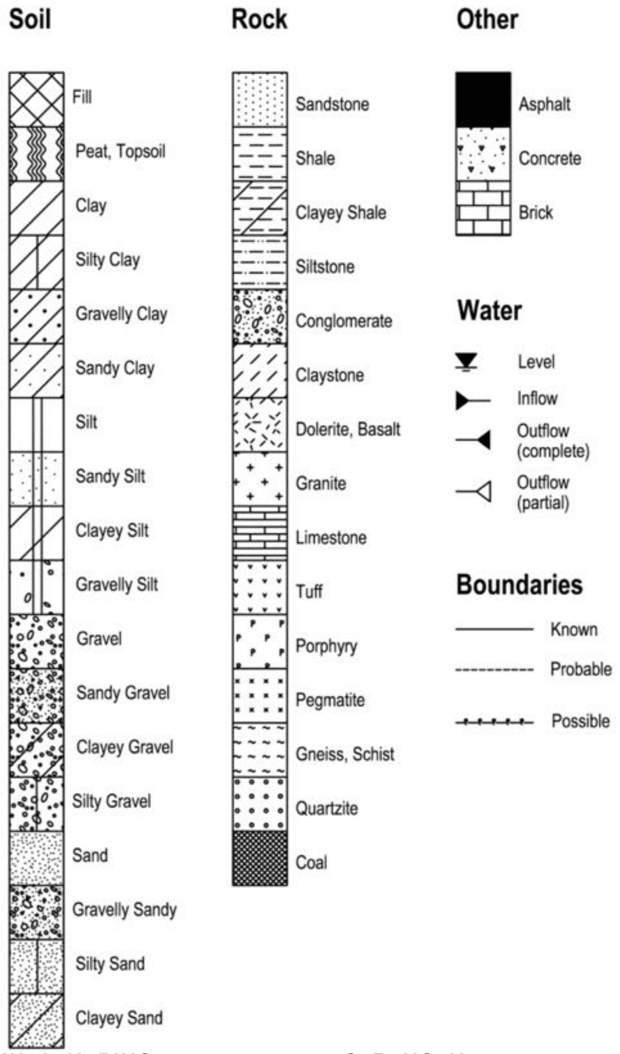
CONSISTENCY

VS	very soft
S	soft
F	firm
St	stiff
VSt	very stiff
H	hard
Fb	friable

DENSITY INDEX

VL	very loose
L	loose
MD	medium dense
D	dense
VD	very dense

Graphic Log



WEATHERING

XW	extremely weathered
HW	highly weathered
MW	moderately weathered
SW	slightly weathered
FR	fresh

STRENGTH

VL	very low
L	low
M	medium
H	high
VH	very high
EH	extremely high

RQD (%)

$$= \frac{\text{sum of intact core pieces} > 2 \times \text{diameter}}{\text{total length of core run drilled}} \times 100$$

DEFECTS:

type		coating	
JT	joint	cl	clean
PT	parting	st	stained
SZ	shear zone	ve	veneer
SM	seam	co	coating

shape		roughness	
pl	planar	po	polished
cu	curved	sl	slickensided
un	undulating	sm	smooth
st	stepped	ro	rough
ir	irregular	vr	very rough

inclination
measured above axis and perpendicular to core

AS1726–2017

Soils and rock are described in the following terms, which are broadly in accordance with AS1726–2017.

Soil

MOISTURE CONDITION

Term	Description
Dry	Looks and feels dry. Fine grained and cemented soils are hard, friable or powdery. Uncemented coarse grained soils run freely through hand.
Moist	Soil feels cool and darkened in colour. Fine grained soils can be moulded. Coarse soils tend to cohere.
Wet	As for moist, but with free water forming on hand.

Moisture content of cohesive soils may also be described in relation to plastic limit (W_P) or liquid limit (W_L) [>> much greater than, > greater than, < less than, << much less than].

CONSISTENCY OF FINE-GRAINED SOILS

Term	Su (kPa)	Term	Su (kPa)
Very soft	< 12	Very Stiff	>100 – ≤200
Soft	>12 – ≤25	Hard	> 200
Firm	>25 – ≤50	Friable	–
Stiff	>50 – ≤100		

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Term	Density Index (%)	Term	Density Index (%)
Very Loose	< 15	Dense	65 – 85
Loose	15 – 35	Very Dense	>85
Medium Dense	35 – 65		

PARTICLE SIZE

Name	Subdivision	Size (mm)
Boulders		> 200
Cobbles		63 – 200
Gravel	coarse	19 – 63
	medium	6.7 – 19
	fine	2.36 – 6.7
Sand	coarse	0.6 – 2.36
	medium	0.21 – 0.6
	fine	0.075 – 0.21
Silt		0.002 – 0.075
Clay		< 0.075

MATERIAL DELINEATION

Sand or gravel	>65% above 0.075mm
Clay or silt	>35% below 0.075mm

MINOR COMPONENTS

Term	Proportion by Mass:	
	coarse grained	***fine grained***
Trace	≤ 5%	≤ 5%
With	>15% ≤ 30%	>5% – ≤12%

SOIL ZONING

Layers	Continuous across exposures or sample.
Lenses	Discontinuous, lenticular shaped zones.
Pockets	Irregular shape zones of different material.

SOIL CEMENTING

Weakly	Easily broken up by hand pressure in water or air.
Moderately	Effort is required to break up by hand in water or in air.

USCS SYMBOLS

Symbol	Description
GW	Gravel and gravel-sand mixtures, little or no fines.
GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels.
GM	Gravel-silt mixtures and gravel-sand-silt mixtures.
GC	Gravel-clay mixtures and gravel-sand-clay mixtures.
SW	Sand and gravel-sand mixtures, little or no fines.
SP	Sand and gravel sand mixtures, little or no fines.
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SC	Sand-clay mixtures.
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CL, CI	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays.
OL	Organic silts
MH	Inorganic silts
CH	Inorganic clays of high plasticity.
OH	Organic clays of medium to high plasticity, organic silt
PT	Peat, highly organic soils.

Rock

SEDIMENTARY ROCK TYPE DEFINITIONS

Rock Type	Definition (more than 50% of rock consists of .....)
Conglomerate	... gravel sized (>2mm) fragments.
Sandstone	... sand sized (0.06 to 2mm) grains.
Siltstone	... silt sized (<0.06mm) particles, rock is not laminated.
Claystone	... clay, rock is not laminated.
Shale	... silt or clay sized particles, rock is laminated.

LAYERING

Term	Description
Massive	No layering apparent.
Poorly Developed	Layering just visible. Little effect on properties.
Well Developed	Layering distinct. Rock breaks more easily parallel to layering.

STRUCTURE

Term	Spacing (mm)	Term	Spacing
Thinly laminated	<6	Medium bedded	200 – 600
Laminated	6 – 20	Thickly bedded	600 – 2,000
Very thinly bedded	20 – 60	Very thickly bedded	> 2,000
Thinly bedded	60 – 200		

STRENGTH (NOTE: Is50 = Point Load Strength Index)

Term	Is50 (MPa)	Term	Is50 (MPa)
Very Low	0.03 – 0.1	High	1.0 – 3.0
Low	0.1 – 0.3	Very High	3.0 – 10.0
Medium	0.3 – 1.0	Extremely High	>10.0

WEATHERING

Term	Description
Residual Soil	Material is weathered to an extent that it has soil properties. Rock structures are no longer visible, but the soil has not been significantly transported.
Extremely	Material is weathered to the extent that it has soil properties. Mass structures, material texture & fabric of original rock is still visible.
Highly	Rock strength is significantly changed by weathering; rock is discolored, usually by iron staining or bleaching. Some primary minerals have weathered to clay minerals.
Moderately	Rock strength shows little or no change of strength from fresh rock; rock may be discolored.
Slightly	Rock is partially discolored but shows little or no change of strength from fresh rock.
Fresh	Rock shows no signs of decomposition or staining.

DEFECT DESCRIPTION

Type	
Joint	A surface or crack across which the rock has little or no tensile strength. May be open or closed.
Parting	A surface or crack across which the rock has little or no tensile strength. Parallel or sub-parallel to layering/bedding. May be open or closed.
Sheared Zone	Zone of rock substance with roughly parallel, near planar, curved or undulating boundaries cut by closely spaced joints, sheared surfaces or other defects.
Seam	Seam with deposited soil (infill), extremely weathered insitu rock (XW), or disoriented usually angular fragments of the host rock (crushed).
Shape	
Planar	Consistent orientation.
Curved	Gradual change in orientation.
Undulating	Wavy surface.
Stepped	One or more well defined steps.
Irregular	Many sharp changes in orientation.
Roughness	
Polished	Shiny smooth surface.
Slickensided	Grooved or striated surface, usually polished.
Smooth	Smooth to touch. Few or no surface irregularities.
Rough	Many small surface irregularities (amplitude generally <1mm). Feels like fine to coarse sandpaper.
Very Rough	Many large surface irregularities, amplitude generally >1mm. Feels like very coarse sandpaper.
Coating	
Clean	No visible coating or discolouring.
Stained	No visible coating but surfaces are discolored.
Veneer	A visible coating of soil or mineral, too thin to measure; may be patchy
Coating	Visible coating =1mm thick. Thicker soil material described as seam.

Appendix B

Site Photos

Photo 1

View of garages and residential development on Grafton Street

Photo 2

View of residential development on Nelson Street

Photo 3

Cracking of garage wall

Photo 4

View of Syd Enfield Drive to north of site viewed from pedestrian overbridge (development on left hand side)

Photo 5

View along Osmund Lane looking to rear of car rental property on Oxford Street

Photo 6

View of western brick wall of residential terrace building at western end of site, showing significant cracking and movement of the wall panel

Photo 7

View of slope down from end of Osmund Lane to Syd Enfield Drive

Photo 8

View of car rental premises on Oxford Street, and western side of restaurant at 214 Oxford Street.

Photo 9
View of residential terraces at western end of site.