Geotechnical Investigation: Proposed Multi Level Development , 351 St Kilda Road, St Kilda

Geotechnical Investigation
Proposed Multi Level Development
351 St Kilda Road
ST KILDA

Report Prepared for:

City of Port Phillip

Report Prepared by A.S. James Pty Ltd

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1. INTRODUCTION

1.01 **Investigation Requested By:** The geotechnical investigation was requested by Tom Temay of City of Port Phillip by their Purchase order number PO 20008698 dated Pty Ltd Geotech in an 16th June 2021. This is in response to our quotation CO 18231 Rev 1 dated 11 June 2021.

1.02 **Purpose of Investigation:** It is proposed by the City of Port Phillip to sell 351 St Kilda Road, St Kilda. This report assumes it will be developed with a single basement and up to three (3) levels. Herein, it was required to forecast foundation conditions and recommend design parameters for this type of structure.

Development proposals however are not available and may be subject to change.

The goals of the geotechnical investigation are outlined as follows:

- Establish the subsurface profile including ground water conditions from the boreholes.
- Provide recommendations for appropriate footing arrangements for the proposed development, including a hazard factor for earthquake loading in accordance with Australian Standard 1170.4, 2007.
- Provide minimum founding depths and allowable bearing pressures for the recommended footing arrangements, together with predictions of short and long-term settlements.
- Provide recommendations for appropriate retention systems for the basement walls, together with lateral earth pressures relevant to the site conditions.
- Provide advice in relation to the anticipated excavation conditions, including advice on site dewatering if required.
- Provide piezometer installation details.

At the time of preparing this report the structural details of the proposed development were not known. It has therefore been assumed for the purpose of this report that no unusual loads or performance specifications apply.

We have carried out geotechnical investigations at 78-82 Carlisle Street, St Kilda in March 2016, (report reference 117297) approximately 100m southwest of subject site. Knowledge of this report was assumed, however we, have carried out separate investigations at the subject site.

1.03 **Geology:** The Geological Survey of Victoria, 1:63,360 Series, Melbourne Sheet, indicates the site to be underlain by sedimentary deposits, which are of the Tertiary age and form part of the Brighton Group. Typically, these deposits comprise shallow surface sands and silt underlain by moderate to high strength clays. At depth the clays, which are generally of a moderate plasticity, grade to medium dense to dense silty/clayey sand. The Brighton Group often contains high strength cemented layers, which comprise ferruginous sandstone. Silurian siltstone and/or sandstones are expected at depth.

1.04 **Field Methods:** As part of the geotechnical investigation the following field methods were incorporated:

i) **Auger Drilling:** The boreholes were drilled using truck mounted Gemco HP 7 rotary drilling rig equipped with continuous flight 110-millimetre diameter augers fitted with a tungsten carbide drill bit.

ii) **In-situ Vane Shear Strength Testing:** In-situ vane shear strength testing was carried out within the cohesive soils at shallow depths using a Pilcon hand vane tester. The tests were conducted in accordance with the test procedure outlined in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes". Test Method 6.2.1.

iii) **Standard Penetration Testing:** Standard Penetration testing was conducted at regular intervals within the boreholes in accordance with the test procedure outlined in Australian Standard 1289, "Methods of Testing Soils For Engineering Purposes," Test Method 6.3.1.

iv) **Logging of Soil Profile:** The soil profile encountered in each of the borehole was logged in accordance with Australian Standard AS 1726 - 2017, "Geotechnical Site Investigations."

1.05 **Laboratory Test Methods:** All soil samples were transferred to A.S. James' National Association of Testing Authorities registered Clayton South laboratory, where testing was undertaken by trained laboratory technicians. All laboratory testing was performed in strict accordance with the test methods outlined in Australian Standard AS 1289, "Method of Testing Soils for Engineering Purposes".

	AS Test Method
• Atterberg Limits	1289 3.1.1, 3.2.1, 3.4.1

2. RESULTS

2.1 FIELD TESTING

2.1.1 Site Description: At the time of the site investigation, the following site features were noted:

- The site is essentially flat and vacant.
- The surface drainage of the site is moderate.
- The site is a normal garden with numerous medium to large trees are scattered particularly at the north, south and west boundaries.
- No boulders or rock visible on ground surface.

2.1.2 Borehole Drilling: Two (2) boreholes were drilled on the subject site at the approximate location indicated on Figure 1. The logs of the boreholes together with the results of in situ vane shear and standard penetration tests are given on Figures 2-3.

2.1.3 Sub-surface Profile: The boreholes have indicated that the topsoil at the site comprises fill/sand layers to an approximate depth of 0.8-0.9m. The fill/sand layers are underlain by grey brown/orange firm to stiff, silty clay with sand. The clay encountered is of moderately to high plasticity and assumed to be of quaternary alluvium origin. This clay layer continued down to a depth of 3.0-3.1m and graded to a layer of stiff, silty/very silty /sandy clay of moderate to low plasticity and is of Tertiary sedimentary age. In borehole 1, the tertiary clay layer encountered comprises orange brown with traces of pale grey silty /sandy clay with ironstone gravels. In borehole 2, orange brown/dark brown/black silty/sandy clay with ironstone gravels and some quartz exits down to an approximate depth of 5.5m and then graded to a layer of orange brown/pale grey silty/sandy clay. Both boreholes were terminated at programmed depth of 8.0m within the stiff silty/sandy clay.

Though not encountered, the clay layers could be interbedded with clayey/ silty sand layers, which are common in tertiary sedimentary deposits.

2.1.4 **Ground Water:** The ground water table was encountered at an approximate depth of 6.0-6.1m below the existing ground level at the time of the site investigation. Previous investigations carried out at 78-82 Carlisle Street, St Kilda by us in March 2016 indicated ground water level at 5.7-6.2m.

A standpipe piezometer was installed in borehole 2 to allow for monitoring of ground water levels leading up to construction.

The piezometer water level measurement made 22 days after drilling was recorded as 5.4m below the existing ground level. However, a minor water level fluctuations are anticipated.

It is recommended that monitoring of the ground water depths be conducted leading up to the commencement of the construction of the proposed building.

It should be noted that the ground water level underlying the subject site will possibly vary slightly with the seasons and should be checked prior to construction.

It should be appreciated that, following prolonged periods of rainfall the surface fill is susceptible to moisture ingress, thereby significantly reducing the workability and strengths of both the surface soils and the underlying clays at shallow depths. If a perched water table develops at the fill – clay interface, temporary drainage may need to be utilised.

2.2 LABORATORY TESTING

2.2.1 **Test Results:** Upon receipt in the laboratory, a clay sample was tested for Atterberg Limits. The test results are given on Appendix I.

3. RECOMMENDATIONS

3.1.1 **Pad and Strip Footings:** The use of pad and strip footings may be considered for the proposed structure, although it should be noted that differential settlements between isolated footing elements can potentially be a problem in some instances where adjacent footings are subjected to extremes in loading conditions or variable founding. As such, this aspect needs to be considered in detail once the precise loads of the proposed structure are available.

For pad and strip footings founded on stiff silty/sandy clay or medium dense clayey sand (if encountered) below the basement excavation, the following maximum bearing pressures should be adopted for footing design.

Pad Footings	-	350 kPa
Strip Footings	-	290 kPa

Due to the depth of the proposed basement excavation, the stress relief is likely to be partially balanced by the total load applied by the proposed structure. It is therefore anticipated that deep-seated settlements are not likely to be significant. Obviously however, localised settlements will take place directly beneath the individual footing elements, with these being as a consequence of recompression of the founding soils during the construction of the proposed structure.

In view of the properties of the proposed founding stratum, it is anticipated that the majority of the settlements will take place relatively quickly upon the application of the load. As a guide it is estimated that approximately 70% of the total settlement will have taken place by the completion of construction of the proposed structure. The remainder is anticipated to occur over the following years at a diminishing rate.

It is estimated that total settlements under pad footings with a width of 3.0 metres will not exceed 12-14mm*, which should be acceptable for the proposed structure. Where there is a significant variation in the applied loading and/or the width of the footing element, the level of angular distortion may possibly be critical.

Note: * For the settlement calculations, the net increase in pressure (considering the single (1) level bulk excavation) was assumed.

3.1.2 **<u>Proportioning of Strip Footings:</u>** Any proposed footings which are to be founded on stiff silty/sandy clay or medium dense clayey sand (if encountered) layer should be proportioned with minimum dimensions and reinforcement corresponding to the details given for a class "M "classification as outlined in A. S 2870-2011. It is emphasised this is provided as a guide, in that design will be based on engineering principles.

3.1.3 **Bored Piers:** Alternatively, the proposed structure could be founded on bored piers. Bored piers should be founded on stiff silty/sandy clay or medium dense clayey sand (if encountered) **subject to a minimum founding depth of three times pile diameter.** Subject to confirmation by a qualified engineer at the time of excavation, a **maximum allowable end bearing pressure of 450kPa** may be adopted for bored piers at this level. Where necessary a **side resistance of 40kPa** may be adopted for the portion of the bored pile extending into the stiff silty/sandy clay or medium dense clayey sand (if encountered) below 1.0m from finished ground level. The side resistance, however, can only be adopted if the sides of the bored piles are rough and free of any smearing.

Alternatively, if smearing of the sides of the pile excavations cannot be avoided it will be necessary to roughen or groove the sides of the pile excavations using a series of tungsten cutters fitted to the side of the auger drill.

Bored piers should have a minimum length to diameter ratio of 3.0.

A safety factor of 3.0 has been adopted for the allowable bearing capacity calculation.

The use of driven piles is not recommended due to the likelihood of vibration induced damage or settlement of any buildings with high level footings founded adjacent to the site.

3.1.4 If deeper piles are required ie for piles to be installed below the ground water level (if ground water is encountered), preference should be given to the use of CFA piles or an allowance for dewatering and casing should be made. Alternatively dewatering will be required.

3.1.5 **Strength Reduction Factor:** Considering the geological complexity of the site, extent of ground investigations and the amount and quality of data available, the subject site can be assigned with following risk factors (IRR and weighting factors (Wi)) based on the piling code. However, to determine the overall design risk rating (ARR), risk factors for design and installation of piles are also to be considered; however, these are dependent upon the contractor and his quality assurance systems.

TABLE 4.3.2 (A) of Piling Code (Reproduced) with Comment about the Rating of the Subject Site Weighting Factors and Individual Risk Factors for Risk Factors

Risk Factor	Weighting Factor (Wi)	Typical description of risk circumstances for individual risk rating (IRR)			IRR for the subject site
		1 Very Low Risk	3 Moderate	5 Very high Risk	
Site					
Geological Complexity of site	**2**	Horizontal strata, well-defined soil and rock characteristics	Some variability over site, but without abrupt changes in stratigraphy	Highly variable profile or presence of karstic features or steeply dipping rock levels or faults present on site, or combinations of these	**Moderate Risk (IRR 3)**
Extent of ground investigation	**2**	Extensive drilling investigation covering whole site to an adequate depth	Some boreholes extending at least 5 pile diameters below the base of the proposed pile foundation level	Very limited investigation with few shallow boreholes	**Moderate Risk (IRR 3)**
Amount and quality of geotechnical data	**2**	Detailed information on strength compressibility of the main strata	CPT probes over full depth of proposed piles or boreholes confirming rock as proposed founding level for piles	Limited amount of simple in-situ testing (ex SPT) or index tests only)	**Moderate Risk** **(IRR 3)** **SPT considered good data (homogeneous strength profile)**

3.1.6 **Earthquake Loading:** In accordance with Australian Standard 1170.4-2007, Part 4, "Earthquake Actions in Australia", site sub-soil class of – C_e –Shallow soil site and Hazard Factor (Z) of 0.08 should be adopted for the design of the proposed structure at the subject site.

3.2 SITE EXCAVATIONS AND RETAINING STRUCTURES.

3.2.1 **Temporary or Short Term Batters:** Assuming significant flows of seepage or ground water do not exist at the time of excavation, short term batters for a maximum excavation depth of 3.5 metres where no adjacent structural loads are present should not exceed the following values which are measured from the horizontal:

Surface Fill	-	30º
Stiff silty/sandy Clay	-	45º

Even at this angle, minor erosion and slumping could be anticipated for excavation faces.

It is observed that short term batters may not possible over most of the site due to adjoining building/properties.

3.2.2 **Long Term Batters:** Long term batters should not exceed 30 degrees in clays unless a retaining structure is incorporated.

Protection of long term batters against erosion/weathering may be required.

3.2.3 **Lateral Earth Pressures:** The distribution of lateral earth pressures for any proposed retaining walls associated with the proposed single basement excavation is dependent on several factors, including construction methods, types of restraint, surcharge loads and time periods all of which influence the horizontal stress distribution. Consequently, for minimal wall deflection, and for construction methods where restraint is applied via struts, bracing or anchors (prestressed to 100% of the design load), a temporary or short-term lateral earth pressure distribution should approximate a **trapezoidal distribution** in which a maximum pressure of **6H** is obtained at a depth of **0.25H** is where H is the total depth of excavation to be retained.

For the completed structure, the restraints imposed by the structure should not markedly alter the above short-term lateral earth pressure.

The above parameters assume that the drained situation exists and that any adjacent surcharge loading be superimposed using an 'at rest' earth pressure coefficient (K_0) of 0.58 in the **clay.**

For retaining walls where the active earth pressure condition is permitted to be mobilised, design should be based general basis of Civil Engineering Code of Practice No 2, 1951, "Earth Retaining Structures", using the following design parameters:

Stiff silty/sandy clay

- Coefficient of active pressure (Ka) for stiff clay / fill/silt/sand - 0.40
- Coefficient of passive pressure (Kp) for stiff clay - 2.5
- Bulk Density of Retained Soil - 1.85 tonne/metre^3

Surcharges should be superimposed where necessary

It is emphasised that where adjoining footings exist, the "at rest" pressures must be maintained and the active design condition is not appropriate.

The following soil parameters should be adopted for the retention pile design.

Short Term Parameters

Fill /surface sand	-	Neglect
Stiff silty/sandy clay (avg)	-	c_u = 120 kPa, $\phi_u = 0$

Long Term Parameters

Stiff silty/sandy clay	-	c' = 15 kPa, $\phi' = 25°$

Based on correlations and experience, lateral deflections of the piles can be calculated using the following estimated parameters:

Fill/surface Sand	-	Elastic Modulus = 10 MPa
Stiff silty/sandy clay	-	Elastic Modulus = 60 MPa

Retention Arrangements

3.2.4 **Pre-cast Concrete Retaining Wall Panels:** The use of suitably propped or anchored pre-cast concrete retaining wall panels, installed in a staged approach, may be considered for the retention of soil within the proposed basement level. In considering such a retention system, the following aspects should be taken into account in the design and construction of the retaining wall system:

- Assuming that the line loads exerted by any adjoining structures are not within the zone of influence, the bulk excavation batter for the proposed basement should be as outlined in Section 3.2.1. In addition, the top of the excavation batters should be at least 0.75 metres out from any existing boundary walls.

- For the purpose of determining a safe excavation batter adjacent to the adjoining structures it has been assumed that the depth of the bulk excavation will not exceed 3.5 metres below the existing ground surface level. It has also been assumed that the proposed basement and excavation will be suitably dewatered, in the event of seepage or ground water being intercepted.

- The proposed pre-cast concrete panel retaining wall sections should be installed in a 'hit one, miss two' sequence, with the maximum panel width not exceeding approximately 2.4 metres. This should be reduced to 1.5m where adjoining structures exist, although we are of the view this approach should not be used where adjoining structures exist.

- Upon installation of each of the concrete panel retaining wall sections, the void to the rear of the panels should be completely backfilled with either cement-stabilised sand or no fines concrete.

- Prior to the commencement of the next stage of the retaining wall construction sequence, the props and/or ground anchors providing temporary support to the concrete panel retaining wall sections should be stressed to ensure that the full "at rest" lateral earth pressure is taken by the props and/or ground anchors if "at rest" pressures are required.

- In cases where adjoining structures exist, tensioned anchors or props must be adopted, stressed to at rest pressures, whereas, in areas where adjoining structures do not exist and some minor yield possible, passive (not tensioned) anchors or nails could be used or cantilever piles adopted.

- Appropriate shoring must be provided at all times when workers are either working within confined excavations or adjacent to vertical excavations associated with the construction of the proposed retaining walls.

3.2.5 **Soldier Pile Retention Systems:** The use of anchored or cantilevered soldier piles is considered to represent a possible retention arrangement for the proposed excavation. In considering such a retention system, the following aspects should be taken into account in the design and construction of the proposed retaining walls:

- The soldier piles should be installed at maximum 2.4 metre centres prior to the commencement of the bulk excavation for the basement levels. Adjacent to any existing buildings the soldier piles should be installed at maximum 1.5 metre centres. Anchors should be provided to maintain at rest pressures where this is the design criteria.

- For soldier piles which are founded on silty/sandy clay or medium dense silty/clayey sand (if encountered) can be designed in accordance with section 3.1.3 of this report. If excavations extend below the ground water level, C.F.A piles should be preferred. Where ironstone lenses are encountered, rock coring bucket will be required and excavation will be slow.

- For soldier piles which are founded on stiff silty/sandy clay or medium dense silty/clayey sand (if encountered), at least 1.0 metres below the bulk excavation level for the proposed basement, a **design allowable end bearing pressure of 450 kPa** may be adopted for pile design.

- It is estimated that total settlements for soldier piles with a diameter not greater than 0.75 metres, subjected to a maximum end bearing pressure of 450 kPa, will not exceed 8 millimetres.

- In order to limit movements at the toe of the soldier piles it is recommended that the passive resistance be calculated using the passive earth pressure coefficient (Kp) of 2.5 within the stiff clay.

- Reinforced Shotcrete should be applied to all exposed faces of the basement excavation prior to the next level of excavation. Shotcrete should be applied before the bulk excavation exceeds a depth of approximately 0.8 -1.0 metres. However, this may require review once the levels of adjoining footings are known.

If however, a diaphragm wall (section 3.2.7) is adopted this would negate the uncertainty in relation to the application of the shotcrete.

- Excavation for the basement level should not extend more than 0.5 metres below the level of the ground anchors before the anchors are installed and fully prestressed, if anchors are used to maintain “at rest” pressures.
- The soldier piles should be positioned such that the adjoining footings are continuous between soldier piles.
- For soldier piles extending below the ground water level, continuous flight augers (C.F.A.) should be preferred.

3.2.6 **Ground Anchors:** (If required) Ground anchors used in connection with the temporary support of any retention structures should extend into the stiff silty/sandy clay, with the design being based on a grout/ground bond strength of 60 kPa (drilled using air flush or auger methods) above the ground water table and 40 kPa (water flush drilling) below the regional ground water table.

Whilst greater bond strengths may be available, it is recommended that the above values not be exceeded unless a field "pull out" test is first completed, given that the soils in which the ground anchors will be constructed are potentially susceptible to softening in the presence of water.

The free length of the ground anchors should be sufficient to ensure that failure cannot occur on a sliding wedge behind the retention wall structures. As a guide it is therefore recommended that the free length of the ground anchors should extend at least 1.5 metres beyond the 45° line extending from the base of the basement excavation.

Generally, ground anchors should be installed at an angle of approximately 15° to 20° below the horizontal and where possible the ground anchor bond length should not exceed 12 metres to ensure adequate load transfer characteristics.

3.2.7 **Diaphragm walls:** A proprietary “D –Wall” arrangement is available with specialist piling contractors and this may be suitable on this site. Herein details of the logs should be given to specialist contractors and firm proposals requested.

3.2.8 **Estimated Wall Deflections and Ground Settlements:** Minor lateral deflections within the proposed anchored soldier pile retaining wall structure are considered to be inevitable. The extent of the movements will, however, to some extent depend upon the level of prestress applied to the ground anchors and the position of the anchors.

It is therefore recommended that the proposed ground anchors if used be given sufficient capacity such that additional stress can be applied throughout the construction sequence to limit wall deflections, as required, based on regular monitoring of wall deflections. In addition, the depth to the top row of anchors should be not greater than 1.5 metres below the ground surface level.

Based on empirical methods, the maximum wall deflection is estimated to lie in the range between 0.25% and 0.35% of the excavation depth. Corresponding vertical settlements of between 0.20% and 0.25% of the excavation depth can be anticipated directly behind the wall, with settlements reducing to zero at a lateral distance approximately corresponding to the depth of the basement excavation.

When considering the influence of the anticipated settlements on the existing adjoining structures, the founding depths of the existing footings should be considered.

In addition to the inherent deformations which will take place within the proposed basement excavation, there may be some minor delays between excavation and the establishment of a suitable anchoring arrangement, during which time additional minor lateral deflections may take place. **A full dilapidation survey of any adjoining structures is therefore recommended prior to the commencement of the basement excavation.**

3.2.9 **Drainage of Retention Systems:** As seepage infiltration and ground water is quite likely to be present in the zones of influence, it is recommended that a suitable drainage system be installed and maintained behind all retaining wall structures to ensure the dissipation of any hydrostatic forces which may result from the accumulation of any seepage water behind the wall structures.

3.2.10 **Excavation Conditions:** Excavation within the fill, and clay should be straightforward, with high capacity plant assuming that the excavation is adequately dewatered at all times during construction.

Though not encountered significantly, it should be noted that ferruginous and cemented sand layers / lenses are common within the Brighton Group Formation. If encountered a rock coring bucket and / or hydraulic breaker may be required.

Depending upon the time of year in which the construction of the basement is carried out, the relatively permeable surface fill may contain moderate to significant seepage water flows. Such seepage water flows should readily be able to be intercepted by the construction of a suitable sub-surface cut-off drain on the uphill side of the subject site.

Assuming a single level basement and bulk excavation level no more than 3.0m, dewatering may not be necessary.

Information regarding site excavation conditions is given as a broad guide only and it is advised that excavation contractors must make their own assessment of possible difficulties.

3.2.11 **Excavation Support for Trenches:** Where trenches extend to a depth greater than **1.0 metre**, temporary benches and/or batters are not possible. If open-cut methods are adopted, a shoring system such as an 'internal propped steel shoring box' will be required for the excavated trenches as the clays and surface soils are not anticipated to be self-supporting under vertical excavation for any length of time.

Additional information is given in the WorkSafe Compliance Code – Excavation Edition 1, May 2018.

3.3 BASEMENT FLOOR SLAB CONSTRUCTION

3.3.1 **Basement Floor Construction:** Provided that the subgrade exists at a moisture condition whereby any disturbance may be made good and the basement is designed and constructed as a drained structure, such that negligible hydrostatic pressures will be generated on the underside of the basement floor, the use of a conventional concrete ground slab should perform satisfactorily in relation to the proposed utilisation. Such floor slabs should be constructed on stiff clay or dense silty/clayey sand (if encountered) subgrade at the proposed basement level and may be designed using a **Modulus of Subgrade reaction of 30kPa/mm (CBR 3.5%).** Under-slab drainage should be provided to the basement to prevent hydrostatic build-up.

If this is not possible, the slab should be checked for uplift.

3.3.2 **Subgrade Preparation:** Preparation of the basement floor subgrade should consist of stripping to grade and proof rolling the subgrade, ensuring that any localised soft or spongy areas are removed and made good with clean granular filling compacted to a dry density not less than 98% of the maximum dry density value determined by the Standard Compaction test in accordance with current Australian Standard AS1289.

If work is carried out following prolonged rain periods or the basement excavation is not adequately dewatered, it is quite possible that the subgrade may exist in a condition significantly wet of optimum moisture content. Under these conditions it is not possible to proof roll the subgrade and it will be necessary to review the situation at the time of construction.

3.3.3 **Pavement or Slab on Surface Clay (if required) :** Preparation of pavements subgrades should consist of stripping to grade and compacting the clay.

The moisture content of the subgrade should be within 85-115% of the Standard optimum moisture content at the time of compaction.

Upon completion of compaction the subgrade should be thoroughly proof rolled with an appropriate roller, ensuring that any localised soft or spongy areas are removed and made good with clean granular filling, which should be compacted to a minimum density ratio of 98% with current Australian Standard AS1289, 5.1.1.

If additional fill is to be placed, this should also be to the specification in section 3.3.4.

Given the above preparation the structural capacity of the pavement/basement ramp may be designed on the basis of a **CBR value of 3%,** which corresponds to a **Modulus of Subgrade Reaction value of 26 kPa/mm.**

Long term Young's Modulus	-	14 MPa
Short term Young's Modulus	-	20 MPa

3.3.4 **Earthworks:** It is pointed out that the clays are notoriously difficult to work as fill and if not compacted at or very close to the optimum moisture content, can exhibit measurable volume change with time. As such, the use of these clays will require close supervision.

Any imported structural fill should essentially be of a granular nature. Suitable material types are considered to include nondescript crushed rock, ripped siltstone or equivalent. All fill material should have a nominal particle size of 50 millimetres or less and, if required, a guide for selecting an appropriate material would be as follows:

- Plasticity Index. X Percentage Passing 0.425 millimetre (AS Sieve) less than or equal to 600
- Locally available clay or clayey sand could also be considered subject to approval

Structural fill should be compacted in layers not greater than 200 millimetres when loose and should be compacted to a dry density not less than 98% of the maximum density ratio determined by the Standard Compaction Test in accordance with current Australian Standard AS 1289 5.1.1., using an appropriate heavy weight vibrating roller.

During compaction, the fill material should have moisture content within the range 85% to 115% of the optimum moisture content as determined by the Standard Compaction Test in accordance with current AS 1289.

Should an increased subgrade strength be required for the proposed floor slab or pavements in any areas, or additional fill proposed to be imported, a design parameter for the improved subgrade could be calculated using the formula proposed by the Japan Road Association, and outlined as follows:

$$CBR_M = [\Sigma(h_n \text{ x } CBR_n^{0.33})]^3$$

Where n = layer number and Σh_n must be one metre
h_n = height or thickness of layer n
CBR_M = composite CBR of the multi-layered system, and
CBR_n = CBR of layer n

4. CONSTRUCTION AND MAINTENANCE OF FOOTING SYSTEMS

4.01 **Articulation of Masonry Walls:** All masonry walls should be adequately articulated in accordance with the recommendations outlined by the Cement and Concrete Association of Australia in Technical Note 61, "Articulated Walling."

Articulation spacings should not exceed the spacings given in Table 2 of Technical Note 61 unless structural design of the proposed structures is able to accommodate increased spacings. Articulation joints should also be provided at the transition points where more than one footing type is being used.

An adequate articulation joint may comprise of either a full height opening or a full height vertical joint in the brickwork, extending from the footing up to the eaves. A combination of the two is also considered to be adequate.

4.02 **General Site Drainage:** It is essential that no water be allowed to pond against footings once they have been constructed. The ground adjacent to the footings should be graded as soon as footing construction has been completed so as to provide a grade of at least 1 in 20 over the first 2.0 metres. Alternatively, all water run-off should be collected and permanently channelled away from the proposed structures.

Water should not be permitted to pond in footing excavations for any length of time during construction.

Service trench excavations located adjacent to footings should be avoided. However, where this cannot be avoided the service trench excavations should be backfilled in such a manner so as to prevent water from seeping beneath the footings.

All service pipes, drains, sewers, downpipes and guttering should be installed and maintained in such a manner that no leakages occur.

4.03 **Planting of Trees and Shrubs:** The recommendations of this report take into account the presence of trees, however, higher level footings are proposed the following should be adopted.

- Deepen all footings located within 0.75 times the mature height of any tree to a minimum founding depth of 2.5 metres below the existing ground surface level, or to rock, whichever is shallower. The use of bored piers may prove to be the most economical for such an arrangement.

- Construct a suitable moisture barrier between the proposed footings and the offending tree. The moisture barrier should extend to a depth of at least 2.5 metres, or to rock, whichever is shallower. In addition the moisture barrier should extend a distance equivalent to the mature height of the tree in either direction.

4.04 **Inspection of Footing Excavations:** All footing excavations must be carefully examined to ensure that the required founding soil has been exposed throughout. Any unusual features must be reported to this office immediately in order to ensure that the recommendations outlined in this report remain relevant.

4.05 **General:** The above recommendations are based on the bore and test results, together with experience of similar conditions and are expected to be typical of the area or areas being considered. Nevertheless, all excavations should be examined carefully and any unusual feature reported to us in order to determine whether any changes might be advisable.

Conditions may change with the seasons. In particular, the surface fill and near surface clays underlying the site at shallow depths may become saturated and unworkable following prolonged periods of rainfall.

Under no circumstance shall this report be reproduced unless in full.

Deepthika Herath

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Geotechnical Engineer
A.S. JAMES PTY LTD

Reviewed by:
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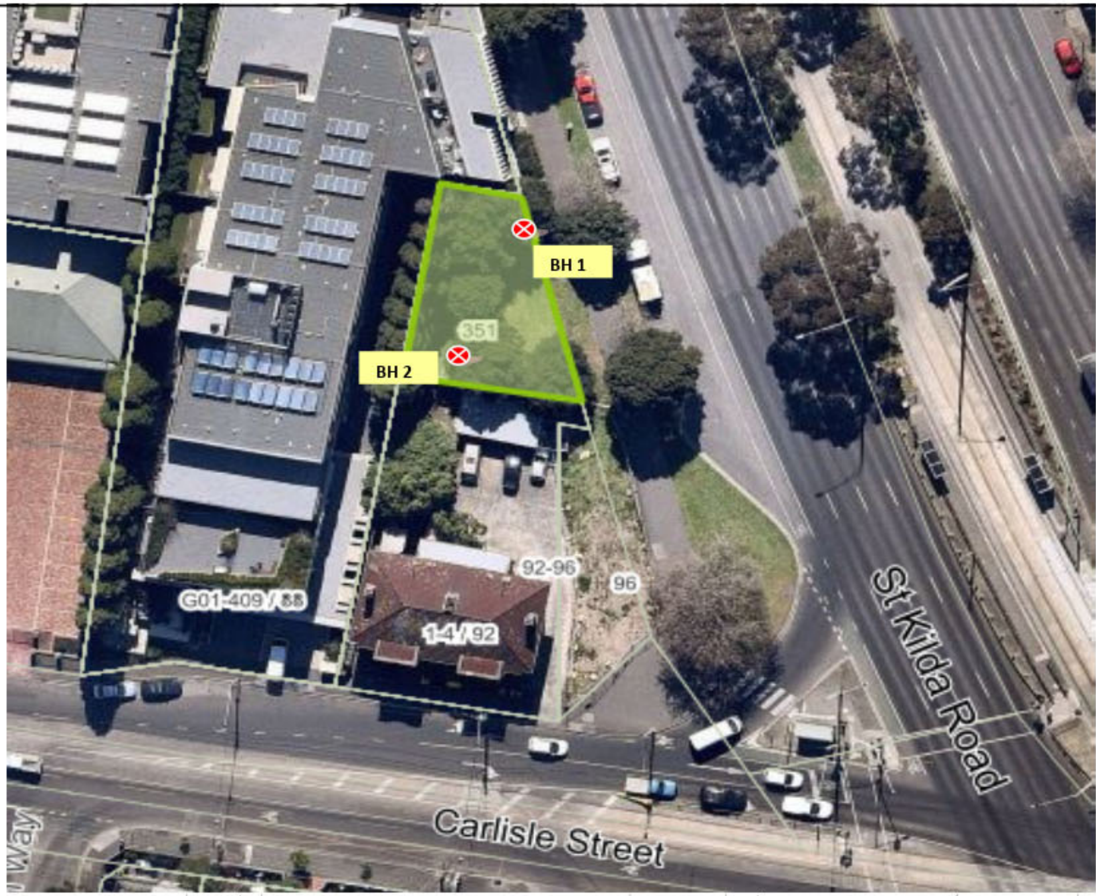

A.S. JAMES PTY LTD
Geotechnical Engineers

JOB: 351 St Kilda Road
St Kilda

JOB No: 121120 Date: July 2021

LEGEND

Denotes approximate borehole location

Borehole Plan

CHECKED: Tim Holt
DRAWN: Shyamalee

SOURCE:

FIGURE
1

A.S.JAMES PTY. LTD.
Geotechnical Engineers

Location: 351 St Kilda Road, St Kilda
Borehole: 1
Job No. 121120
Date: July 2021
Ground Water: 6.0m (at the time of drilling)

Soil Type	Description	Depth	Tests	Results
FILL	- Sand, silt, gravels and crushed rock and thin traces of bricks -Dark brown, slightly moist - Medium dense	0.00		
		0.80		
			.	s =110 kPa
CLAY (CH/CI)	- Silty with sand - Grey brown mottled orange tending orange mottled grey - Firm to stiff -some tree roots around 1.0m depth		.	s =110 kPa
			+	N=3/13/16 N=29
		3.10		
CLAY (CL/CI)	- Silty/sandy - With Ironstone gravels - Orange brown with traces of pale grey - Moist tending very moist from 5.0-6.0m - Stiff		+	N=3/5/8 N=13
			+	N=5/6/9 N=15
CLAY (CL/CI)	- Silty/sandy - Orange brown - Moist - Stiff	7.60	+	N=4/9/10 N=19
		8.00		
	BOREHOLE TERMINATED			

+ Standard Penetration Test - N blows/150mm. incr.	c Apparent Cohesion	L.L. Liquid Limit
I Undisturbed Sample - Diameter Stated	Ø Friction Angle	P.L. Plastic Limit
s Vane Shear Strength	P Wet Density	P.I. Plasticity Index
p Pocket Penetrometer Resistance	w Moisture Content	L.S. Linear Shrinkage

Figure 2

A.S.JAMES PTY. LTD.
Geotechnical Engineers

Location: 351 St Kilda Road, St Kilda
Borehole: 2
Job No. 121120
Date: July 2021
Ground Water: 5.4m Piezometer level (measured 22 days after)

Soil Type	Description	Depth	Test	Results
SILT/SAND (ML/SM)	- Silt /silty sand -Dark brown - Slightly moist - Loose to medium dense	0.00		
		0.90	.	s =120 kPa
CLAY (CH/CI)	- Silty with sand - Grey brown mottled orange brown tending orange brown mottled pale grey - Stiff		.	s =110 kPa
CLAY (CL/CI)	- Silty/sandy - With Ironstone gravels - Orange brown, dark brown and black with traces of grey - Moist - Stiff	3.00	+	N=9/19/15 N=24
CLAY (CL/CI)	- Silty/sandy with sand and quartz - Dark brown with traces of pale grey - Moist - Stiff	4.00	+	N=7/10/9 N=19
		5.50	+	N=4/5/8 N=13
CLAY (CL/CI)	- Silty/sandy - Orange brown mottled pale grey - Moist - Stiff		+	N=3/6/7 N=13
	BOREHOLE TERMINATED	8.00		

+ Standard Penetration Test - N blows/150mm. incr.	c Apparent Cohesion	L.L. Liquid Limit	**Figure 3**
I Undisturbed Sample - Diameter Stated	Ø Friction Angle	P.L. Plastic Limit	
s Vane Shear Strength	P Wet Density	P.I. Plasticity Index	
p Pocket Penetrometer Resistance	w Moisture Content	L.S. Linear Shrinkage	