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Report on
Preliminary Geotechnical Assessment

Proposed Residential Development
61 North Steyne, Manly

Prepared for
Manly Property Developments Pty Ltd

Project 216903.01
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The undersigned, on behalf of Douglas Partners Pty Ltd, confirm that this document and all attached drawings, logs and test results have been checked and reviewed for errors, omissions and inaccuracies.

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Report on Preliminary Geotechnical Assessment
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1. Introduction

This report presents the results of a preliminary geotechnical assessment for a proposed residential development at 61 North Steyne, Manly (the site). The assessment was commissioned in an email dated 28 July 2022 by Leigh Manser of Lindsay Bennelong Developments Pty Ltd acting on behalf of Manly Property Developments Pty Ltd. The assessment was commissioned in accordance with Douglas Partners Pty Ltd proposal (216903.00.P.001.Rev0) dated 28 July 2022.

It is understood that the proposed development will likely include the demolition of the existing structures and the construction of a five-storey residential building over two basement levels. Given the relatively flat ground surface levels within the site, the basement excavation is likely to extend to depths of about 6 m to 7 m below existing surface levels. Locally deeper excavation may be required for lift pits, service trenches and foundation excavations.

The desktop assessment was carried out to provide preliminary information on the likely subsurface conditions underlying the site for preliminary design and planning purposes and to provide information for a development application (DA) submission to the Northern Beaches Council (Council).

DP has carried out a number of geotechnical investigations in the general area and the information from these investigations has been used to prepare this assessment. Intrusive geotechnical investigations for the proposed development has not been carried out for the present report but is considered necessary to determine actual subsurface conditions for future detailed design purposes.

DP also completed a Preliminary Site Investigation (PSI) for contamination for this site. Reference should be made to the PSI report in project planning (Ref: 216903.00.R.001.Rev0, dated 30 August 2022)

2. Site Description

The property of 61 North Steyne, Manly also known as strata plan (SP) 2492, is located at the corner of North Steyne and Denison Street and forms a trapezoidal-shaped lot of approximately 410 m^2 in area. Surface levels are observed to be relatively flat across the site and approximately between reduced levels (RL) 5.3 m and 5.7 m with reference to the Australian height datum (AHD).

Currently, the property is occupied by a two-storey residential brick building and a small concrete pavement along the western site boundary for parking. These structures will be demolished for the proposed development.

The site is situated within an area for medium-density residential use. A summary of land uses adjacent to the proposed development site is provided in Table 1.

Table 1: Summary of Adjacent Land Use

Direction from the proposed development site	Description
North	Denison Street followed by a residential building of over 10 stories.
South	A residential building of 5 stories with at least 1 basement level.
East	North Steyne followed by Manly Beach
West	A two storey residential building, followed by Francis Lane and St Mary's Catholic School

3. Regional Mapping

3.1 Geology

Reference to the Sydney 1:100,000 Geological Series Sheet indicated that the site is likely underlain by fine to coarse quartz marine sands with varying amounts of shell fragments of Quaternary Age.

3.2 Hydrogeology

A search of the NSW Department of Primary Industries groundwater bore records for registered groundwater bores indicated that the nearest groundwater bore (GW116138) is located approximately 250 m to the south west of the site. The bore details state that standing water level within the installed bore was encountered at a depth of 4.5 m below existing surface levels at the time of reporting in 2017.

3.3 Acid Sulfate Potential

Reference should be made to DP's PSI report ref: 216903.01.R.001.Rev0 dated August 2022.

4. Previous Investigations

DP Assessment: 55 North Steyne, Manly (2018)
Cone penetration testing (CPTs) were undertaken to assess the existing ground conditions of the site. The CPTs encountered fill to 1.0 m overlying over variably dense sands. Generally the sands were very loose to loose with some medium dense and dense sands present between 3 m and 5 m and between 11 m and 13 m.

DP Investigation: Corner of Whistler and Denison Street, Manly (2007)
A geotechnical investigation comprising the drilling of boreholes and CPT testing. Results indicate the ground profile comprised of shallow fill overlying sand and silty sand to 20 m depth. Generally the sand was observed to be loose to medium dense to 10.5 m before grading to medium dense to very dense sand/silty sand with some loose bands.

Groundwater was encountered at an RL of between 0.6 m and 1.1 m AHD within groundwater monitoring wells installed during the investigation works.

DP Investigation: 69 North Steyne, Manly (2001)
A geotechnical investigation comprising the drilling of hand augered boreholes and CPT testing. Results indicate the ground profile comprised shallow fill to approximately 1.0 m overlying sand with some clay bands encountered at depth. The sand was observed to vary between loose to medium dense to 7.0 m depth.

Groundwater was encountered at RL 0.5 m AHD within groundwater monitor wells installed during the investigation works.

5. Proposed Development

Based on the preliminary information supplied by the client, it is understood that the proposed development will include the demolition of existing structures and construction of a five storey residential building over two basement levels. The preliminary architectural drawings indicate a basement finished floor level (FFL) of RL 0.1 m AHD. It is assumed that bulk excavation to at least RL-0.6 m may be required for a tanked slab but this will depend on structural design. Excavation for the basement is anticipated to extend to depths of between 6 m and 7 m below existing ground surface levels. Locally, deeper excavation may be required for service trenches, lift pits and crane pads.

6. Preliminary Geotechnical Model

Based on DP's experience within the immediate area it is anticipated that the ground profile within the site likely comprises:

- Fill and Topsoil – minor surface filling possibly comprising topsoil and rootlet affected soils at shallow depths with some areas of deeper fill.
- Marine sands –interbedded fine to coarse sands with thin lenses of clay to depths of at least 20 m. The sands are likely to vary in density between loose to medium dense with some dense bands to a depth of approximately 10 m before grading to dense sands with some loose bands with depth.
- Bedrock – bedrock is likely to be encountered at significant depths, below 20 m, and comprise Hawkesbury Sandstone.

The groundwater table is likely to be present at about RL0.5 m to RL1.0 m and is likely to exhibit a subdued fluctuation reflecting the coastal/tidal conditions present at Manly Beach. It should be noted that groundwater levels will fluctuate with climatic conditions and to lesser extent due to tidal influences, and are likely to increase following periods of extended wet weather.

7. Comments

7.1 Anticipated Geotechnical and Hydrogeological Issues

The anticipated geotechnical issues that need to be considered for the proposed development are:

- Groundwater is likely at a relatively shallow depth and dewatering may be required for the construction of the basement levels;
- A 'tanked (i.e. fully water-tight) basement will be required and should be designed for hydraulic uplift pressures;
- Control of temporary groundwater inflow during construction requiring dewatering and the construction of a shoring wall extending to sufficient depth beneath the excavation;
- Dewatering in sands leading to possible localised temporary draw down of the groundwater table which may cause settlement of adjacent buildings being supported on high level footings and similar effect of services, if not properly controlled;
- If anchors are used to temporarily support the shoring, it will be necessary to obtain permission from affected parties prior to installation.

7.2 Groundwater

Based on the observed groundwater levels in the area (typically RL0.5 to RL1.0 m AHD), it is anticipated that bulk excavation level to about RL -0.6 m maybe up to 1.6 m below the typical groundwater table. Localised deeper excavations for lift pits will extend further below the water table.

7.3 Dilapidation Survey

Dilapidation surveys should be undertaken on surrounding structures, buildings and pavements prior to commencing work on the site to document any existing defects so that any claims for damage due to construction related activities can be accurately assessed. The appropriate extent of dilapidation surveys may be better assessed once details of the proposed development and construction methods have been confirmed.

7.4 Dewatering and Tanking

7.4.1 General

In constructing the basement, it is assumed that a shoring wall will be installed to below the bulk excavation level, the site will then be dewatered, progressively excavated and finally a tanked basement will be constructed.

Generally the groundwater level should be lowered to at least 1 m below the bulk excavation level to allow machinery to operate and traverse the site. On this basis, the normal groundwater level may need to be temporarily lowered by about 2.1 m to 2.6 m to approximately RL -1.6 m AHD (but locally deeper for lift pits).

In order to reduce groundwater flows into the basement excavation and thereby reduce potential impacts to the surrounding groundwater, potential acid sulphate soils, and neighbouring buildings, a water-tight "cut-off" wall should be formed around the perimeter of the basement that intersects a lower permeability stratum or is sufficiently deep to allow a rate of dewatering and construction of a tanked (i.e. fully watertight) basement so there is no long term pumping requirements or drawdown of the water table surrounding the site.

It is recommended that the permanent basement structure be tanked and designed for hydrostatic uplift. In the absence of long term monitoring and for preliminary purposes, it is suggested that typical loads due to a groundwater table rising to at least RL3.0 m AHD should be considered in the basement design. It is not possible to guarantee (accurately predict) future groundwater levels and response to extreme rainfall events, climate change, sea level rise and other factors. If it is necessary to eliminate the risk of buoyancy issues for the tanked basement then the design could incorporate hydrostatic relief 'valves or portals' at the adopted design water level to prevent structural damage to the building, in the event that water levels rise to above the design level.

In the long term, the downward force to resist uplift is typically provided by the weight of the building itself, and the detailing of the slab and foundations should be designed accordingly. It is anticipated that the uplift pressures will be counteracted by the dead load of the building once it is completed (subject to confirmation by the structural engineer).

7.4.2 Groundwater Disposal

Groundwater that is removed from the site will require disposal. Generally, and subject to its environmental status, water resulting from dewatering operations should be suitable for disposal by pumping to stormwater drains or re-injection of the groundwater, subject to confirmation testing and approval from Council and the Office of Water NSW, as necessary

7.4.3 Further Groundwater Investigation and Assessment

Further investigation of the groundwater conditions within the site will be required to:

- Assess the groundwater level and fluctuations across the site and provide a detailed groundwater assessment to predict soil permeability, inflow rates, drawdown and its effect in the short and long term for the site and surrounding properties;
- Satisfy the relevant authorities and Council requirements for monitoring groundwater levels inside and outside of the site before and during dewatering.

7.5 Excavation Conditions and Batter Slopes

Excavations are expected to be carried out through fill and natural sands, which should be readily excavated using conventional earthmoving equipment such as tracked excavators. Soils excavated below the water table will remain wet even after dewatering operations. Due consideration should be given to handling such 'water logged' material and whether progressive spreading and drying of the soil prior to removal is feasible.

Trafficability on the sandy soils during construction will generally require the use of tracked plant and machinery. Trafficability after bulk excavation could be improved by placement of a layer of compacted crushed concrete or similar hard and durable rockfill, which may subsequently be used as subfloor drainage medium and slab fill layer.

During the bulk excavation phase, it is recommended that temporary batter slopes within the perimeter shoring walls and above the groundwater table, do not exceed 1.5H:1V (Horizontal : Vertical) in both fill and sandy soils. Excavations below the groundwater surface will collapse immediately on removal of soil.

All excavated materials will need to be disposed of in accordance with the provisions of the current legislation and guidelines including the Waste Classification Guidelines (EPA 2014). Reference should be made to the preliminary contamination assessment report prepared by DP (Ref: 216903.00.R.001).

7.6 Excavation Support

Vertical excavations within the sandy soils will require retaining structures both during construction and as part of the final structure. It is anticipated that one to two rows of temporary anchors or bracing will be required to provide lateral restraint and limit wall movements. Alternatively, top-down construction may be adopted, particularly if anchors cannot be used or if it is necessary to reduce wall movements.

7.6.1 Retaining Wall Design

It is suggested that the preliminary design of shoring with multiple rows of anchors should be based on a rectangular earth pressure of 6H kPa (H equals the retained height in metres) adjacent to buildings and 4H where there are no adjacent buildings in close proximity to the excavation.

A coefficient of passive earth pressure (K_p) equal to 3.5 may be assumed within at least medium dense sand below the bulk excavation level, to which a factor of safety must be applied in order to limit the movement which is required to mobilise the full passive resistance.

The pressure distributions given above do not include hydrostatic pressure due to groundwater behind retaining walls. It is suggested that a potential groundwater level to at least RL 2.5 m AHD behind the wall should be adopted in the preliminary design of retaining walls.

In the design of the retaining walls due allowance should be made for surcharge loads including adjacent footings (in particular the buildings to the north and east) and plant operating above the excavation during construction.

Detailed design of shoring should be carried out using WALLAP, FLAC or other accepted computer analysis programs capable of modelling progressive excavation and anchoring and predicting potential lateral movements, stresses and bending moments.

7.6.2 Retaining Wall Systems

A secant pile wall may be suitable for the site, comprising interlocking Continuous Flight Auger (CFA) piles or CFA piles with jet grouted columns between the piles. This shoring system can generally provide an effective seal to minimise sand loss and water inflow from behind the wall, and if adequately supported, minimise lateral deflections. The 'hard' (reinforced concrete) piles can be incorporated into the vertical load carrying footing system and can generally form part of the basement structure.

Soil mixed wall systems have been used as an alternative to the more conventional secant pile wall. These walls are constructed using specialised equipment to blend cement with the in-situ soils to create a soil-cement mix. There are several different systems available and further advice should be obtained from the specialist piling contractor regarding the suitability of the wall system to this site. In particular, confirmation should be sought in relation to the consistency/strength of the soil mixed wall, the long term durability, permeability, potential issues with blending cement with sometimes clayey soils and joining the soil mixed wall with the tanked basement slab.

Sheet piles are generally suitable for shallower excavations above the water table and where there are no movement sensitive structures adjacent to the excavation. For these reasons they are not recommended for use on this site.

A contiguous pile wall comprising closely spaced/touching CFA piles is also not recommended for this site due to risks associated with seepage and sand loss in between the piles, particularly below the groundwater table.

For CFA piles, care will be required to avoid decompression of the sandy soils during augering, which can lead to loosening of the foundations and damage to adjacent structures. It may be necessary to adopt temporary segmental casing to reduce the risk of decompression near structures and services. Further advice should be sought from an experienced piling contractor in this regard.

As a guide, well designed shoring walls in sand supported by anchors may experience lateral wall movements in the order of 1 mm to 2 mm for each metre of excavation height. The extent of movement will depend on the final design and construction methods used. A programme of precise survey monitoring should be adopted to assess shoring wall and adjacent building movement progressively during the excavation to ensure that tolerable limits are not exceeded and to provide an early indication of whether additional support is required.

7.6.3 Ground Anchors

It is presumed that temporary anchors or stiff propping will be used to restrict wall movements during the construction phase, with permanent support of walls provided by the final structure. Further advice can be provided following detailed investigations.

The anchors will need to be carefully positioned and possibly inclined at steeper angles to avoid adjacent services and footings for adjacent buildings. It is noted that permission from adjacent property owners (and RMS/council) will be required prior to installing soil anchors beneath their land.

It is recommended that only reputable, specialist anchor contractors be engaged to design and/or install temporary anchors on this site.

7.7 Subgrade Preparation

It is expected that at bulk excavation level, the subgrade will comprise loose to medium dense sand. This will need to be confirmed by investigation, preferably using CPTs.

For subgrade preparation, it is suggested that following excavation to achieve design levels, the exposed soil surface should be thoroughly rolled with a minimum of eight passes using an appropriately sized smooth drum roller (say 8 tonne static weight). The final pass (proof roll) should be inspected by a geotechnical engineer to help identify any soft or heaving areas.

If heavy plant (e.g. piling rigs) are required to operate within the excavation, it is expected that a working platform will need to be constructed. The platform should be constructed from good quality granular material with low fines, such as recycled concrete or high strength crushed rock. The thickness of the platform should be assessed once specific details of the heavy plant that will operate within the basement are known.

7.8 Foundations

7.8.1 Shallow Foundations

The foundation material below the proposed basement level is anticipated to comprise loose to medium dense sands. It is expected that shallow or high-level (e.g. pad or strip) footings could be founded on medium dense or better sand although footings may be relatively large and settlements would need to be considered and may be relatively high. Shallow foundations on loose sand are not recommended due to excessive settlement. Raft slabs are usually more appropriate for shallow foundations.

For shallow foundations, the allowable end bearing pressure in sands will depend on the density/strength of the foundations, depth of embedment and size of the footing and depth to groundwater. As a guide, allowable end bearing pressures and elastic modulus values for the typical soil strata are provided in Table 2. The ultimate and allowable end bearing pressures shown in Table 2 are based on a pad footing with a plan area of 2 m by 2 m, embedment of 0.5 m, groundwater at the base of the footing and a factor of safety equal to 2.5.

Table 2: Summary of Typical Design Parameters for Shallow Foundations

Unit	Foundation Material	Ultimate Bearing Capacity (kPa)	Allowable Bearing Capacity (kPa)	Young's Modulus (MPa)
2	L Sand	not recommended		
3	MD to D Sand	250	100	35

Notes: L = Loose, MD = Medium Dense, D = Dense

The settlement of shallow footings founded on sand may be estimated based on Young's Modulus values given in Table 2.

7.8.2 Raft Slabs

Consideration may be given to the use of a raft slab foundation. However, this will be subject to detailed review and analysis of bearing pressures and settlements once more specific details of the founding level, column layout and slab loadings have been confirmed. Further investigation using CPTs will be required to confirm the strength and consistency of the soil profile across the site once access is available.

Specific details of structural loads are not available at the time of preparing this report. Based on similar sized projects it is anticipated that a distributed slab load in the order of 70kPa may be applicable for the five storey building over two basement levels. As a guide, for raft slab foundations, preliminary settlement analyses have been carried out assuming a distributed slab load of 70 kPa, with a loaded area of 20 m by 20 m with a with a modulus of subgrade reaction (k) value in the order of 1 kPa/mm to 2 kPa/mm for the broadly loaded area may experience settlements of between 35 mm to 70 mm. It is noted that the ‘k’ value (which is not strictly a soil parameter) is very dependent on the size of the loaded area and the rigidity of the raft system.

Construction of the raft slab should incorporate subgrade preparation as outlined in Section 7.7 It is also suggested that a minimum 150 mm thick layer of good quality, durable rockfill such as recycled concrete or fine crushed rock should be placed and compacted over the prepared surface, particularly at the more heavily loaded areas. The granular layer will help to confine the sandy soils and improve the compaction and density of the surface soils.

A piled raft foundation may also be considered to reduce differential settlements, if required.

Further geotechnical investigation, analysis and advice will be required in relation to the detailed design and construction of both raft slabs and piled raft slabs, if these are to be considered.

7.8.3 Pile Foundations

The alternative to raft foundations is to support the structural loads on piles founded within at least dense sand or rock. The depth to rock has not been confirmed on the site and would require further investigation using cored boreholes if this option is to be considered (unlikely to be necessary). The previous investigations indicate that rock is deeper than 30 m.

Continuous Flight Auger (CFA), concrete injected piles could be considered for this site.

It is expected that noise and vibration constraints at this site will preclude the use of driven pile types. Open bored piles will not be appropriate due to the potential for soil collapse and groundwater inflow and the relatively small site will preclude the use of bored piles being drilled under bentonite due to the size of the equipment required.

7.9 Seismic Loading

In accordance with AS1170 – 2007 “Structural Design Actions, Part 4 : Earthquake Actions in Australia” a hazard factor (Z) of 0.08 and a site subsoil Class C_e is likely to be appropriate for the site.

7.10 Recommended Further Geotechnical Investigation

During the design development stage and following demolition of the onsite structures, it is recommended that intrusive geotechnical investigations are undertaken to obtain specific information on the site. The investigations could include (but not necessarily limited to):

- CPTs across the site to determine the soil profile and consistency;
- Boreholes for soil identification and collection of laboratory samples;
- Installation of groundwater monitoring wells with data loggers to measure groundwater levels before and during construction;
- Permeability testing in wells;
- Groundwater modelling to assess inflows and drawdown; and
- Shoring wall analyses.

8. Limitations

Douglas Partners (DP) has prepared this report for this project at 61 North Steyne, Manly in accordance with DP's proposal 216903.00.P.001.Rev0 dated 28 July 2022 and acceptance received from Leigh Manser of Lindsay Bennelong Developments Pty Ltd on 28 July 2022 acting on behalf of Manly Property Developments Pty Ltd. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of Manly Property Developments Pty Ltd for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and their agents.

The assessment of atypical safety hazards arising from this advice is restricted to the geotechnical components set out in this report and based on known project conditions and stated design advice and assumptions. While some recommendations for safe controls may be provided, detailed 'safety in design' assessment is outside the current scope of this report and requires additional project data and assessment.

This report must be read in conjunction with all of the attached and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by DP. This is because this report has been written as advice and opinion rather than instructions for construction.

Douglas Partners Pty Ltd

Appendix A

About This Report

About this Report

Douglas Partners

Introduction

These notes have been provided to amplify DP's report in regard to classification methods, field procedures and the comments section. Not all are necessarily relevant to all reports.

DP's reports are based on information gained from limited subsurface excavations and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

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This report is the property of Douglas Partners Pty Ltd. The report may only be used for the purpose for which it was commissioned and in accordance with the Conditions of Engagement for the commission supplied at the time of proposal. Unauthorised use of this report in any form whatsoever is prohibited.

Borehole and Test Pit Logs

The borehole and test pit logs presented in this report are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable or possible to justify on economic grounds. In any case the boreholes and test pits represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes or pits, the frequency of sampling, and the possibility of other than 'straight line' variations between the test locations.

Groundwater

Where groundwater levels are measured in boreholes there are several potential problems, namely:

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole is left open;
- A localised, perched water table may lead to an erroneous indication of the true water table;
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report; and
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water measurements are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical and environmental aspects, and recommendations or suggestions for design and construction. However, DP cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

If these occur, DP will be pleased to assist with investigations or advice to resolve the matter.

About this Report

Site Anomalies
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, DP requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

Information for Contractual Purposes
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. DP would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection
The company will always be pleased to provide engineering inspection services for geotechnical and environmental aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.