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Geotechnical Investigation Report

Proposed Development at Lot 10 in DP878167
50 Wylie Road, Kembla Grange NSW

prepared for

Bicorp Pty Ltd

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REFERENCES

1. Australian Standard AS1726-1993 ‘Geotechnical Site Investigation’; and
2. Australian Standard AS2870-2011 ‘Residential Slabs and Footings’;
3. Australian Standards – Guidelines on Earthworks for Commercial and Residential Developments, AS3798-2007.
4. Australian Geomechanics Society, Landslide Risk Management Sub-Committee Guidelines: *Landslip Risk Management Concepts and Guidelines,* March 2000.

Table of Contents

1.0 INTRODUCTION

Benviron Group was engaged to undertake a geotechnical investigation at the subject site located at 50 Wylie Road, Kembla Grange (Figure 1-Site Location) The purpose of this investigation is to assess the existing site and subsurface conditions in order to provide recommendations from a geotechnical viewpoint on the proposed scheme comprising buildings, roads and storage areas in the proposed Resource Recovery Facilities at Kembla Grange. It also addresses the stability assessment as required by Chapter E12 of the Wollongong DCP 2009.

This report presents and interprets the findings of the geotechnical investigation carried out on 19th December 2012 at the subject site, known as number 50 Wylie Road, Kembla Grange, NSW, and presents the followings:

- Method of investigation,
- Site description, including surface and sub-surface conditions,
- Site plan indicating borehole locations and footprint of the proposed roads and buildings in the development,
- Groundwater conditions and management, if encountered,
- Recommendations on the excavation conditions,
- Provision of earth pressure parameters for design of retaining structures if required,
- Recommendations on footings and serviceability bearing pressures,
- Recommendations on pavement and design parameters.
- Risk assessment of the site.

2.0 AVAILABLE INFORMATION

At the time of writing this report, information available for the proposed scheme from the client is summarized as follows:

1. Drawing prepared by KFW infrastructure Professionals on Proposed Industrial Development Lot 10 DP878167 Wyllie Road Kembla Grange - Access Road and Pavement Drawing Sheet 1&2.
2. Letter from NSW Government on planning and infrastructure refer 12/0812.
3. Letter from Wollongong City Council regarding application DE-2012/84.

3.0 PROPOSED DEVELOPMENT

3.1 Site Description

The site is located approximately 1Km northwest of the Kembla Grange Racecourse as shown in Figure 2 – Site Plan. It is currently a temporary Resource Recovery Facility with access from Wyllie Road from the southeastern corner of the site and bounded by a forest park on the northernmost and easternmost site boundary. There is a creek that dissects the site in a northeasterly to southwesterly direction at the southeastern section of the site. The ground is steeply sloping from the southeastern entrance from Wyllie Road at approximately + RL 44.0 mAHD to a level platform located at the western part of the site at +RL 21.0 mAHD. A site plan is shown in Figure 3 with the investigation locations.

3.2 Regional Geology

Reference to the Wollongong – Port Hacking 1: 1:100,000 Geological Series Sheet 9029-9129 Edition 1, 1985, (indicates the site to be underlain by Budgong Sandstone. Budgong Sandstone typically consists of red, brown and grey lithic sandstone.

4.0 FIELDWORK

Fieldwork for the geotechnical investigation was carried on the 19th December 2012 and comprised the following works:

- A detailed walk-over inspection of the site and surrounding environment to capture any significant geological features.
- Drilling of six (6) boreholes, BH1 to BH6 using a 1.5 tonne drilling rig mounted with tungsten carbide (TC) bit with solid flight augers to TC bit refusal.
- Excavation of one (1) test pit to collect samples for laboratory testing
- Standard Penetration Tests (SPT) was undertaken at regular intervals within the borehole to assess the in-situ strength of subsoil properties.

The approximate locations of the 6 boreholes and 1 test pit are shown in Figure 3-Borehole Locations and the Engineering Logs are presented in Appendix A.

5.0 FIELD WORK RESULTS

5.1 Subsoil Conditions

Based on information gathered and observations made from the site inspection, it can be inferred that it is likely the subsoil profile comprises topsoil, which overlies the residual material of varying degree of weathering ranging between a Clayey SAND to a more predominant sandy/CLAY matrix that sits on top of the Sandstone Bedrock.

The subsurface soil profile in the eastern section and within BH1 indicated predominantly a brownish yellow very stiff to stiff sandy CLAY from ground surface (Approximately at +29mAHD) to a depth of 3 meters. Underlying this layer is a medium dense SAND .

The subsurface soil profile along the westernmost platform edge as observed from BH2 is a predominant brown medium plastic CLAY for the top 1 meter below existing ground surface at approximately +24mAHD. Subsoil observations from BH3 and BH4 indicated a Clayey silty SAND for the first 1 meter below existing ground surface at approximately +21mAHD. Further southwards in the vicinity of BH5 and BH6, the subsoil material is a dark brownish CLAY of medium plasticity.

A test pit (TP1) was excavated immediately adjacent to BH 3 and BH4 to collect samples for laboratory testings.

5.2 Ground Water

Groundwater or seepage was not encountered in the boreholes/testpit during drilling. However, it should be noted groundwater levels may be subject to seasonal fluctuations, rainfall, prevailing weather conditions and also future developments of the areas and land forms.

6.0 LABORATORY TEST RESULTS

Samples have been collected from test pit (TP1) as shown in Figure 3-Borehole Locations. The sample collected were in the sandy clay layer underlying topsoil for the determination of the Atterberg limits (Liquid and Plastic limits) and Plasticity Index. The test results are as Table 1.

Table 1: Atterberg Limits Tests Result

Sample ID	Location	Depth	Plastic Limit	Liquid Limit	Plasticity Index
1	Test Pit 1	0.6-1.0 m	34	60	26

7.0 RISK AREAS

7.1 Instability Risk Assessment

The stability of a site is generally governed by site factors such as slope angles, water movements and drainage, depth of in-situ soils, and strength of sub-surface material. Based on the proposed development with regards to the site topography, the site is divided into two parts as shown in Figure 3: the relatively flat and level ground comprising the western section of the site at approximately +21mAHD as one part, and the more eastern hilly section with the ground rising to meet Wylie Road at the southeastern end at approximately +44mAHD as the other part.

7.2 Western Flat Section

For the majority of the western platform with the proposed designated storage areas and building platform, the assessment on the subsurface condition and the overall site slope angle of less than 10°, the site would be in low hazard area in terms of landslide instability.

The Australian Geomechanics Society (Reference 4) recommends the landslide risk of a site be assessed on the basis of the likelihood of a landslide event and the consequences of that event. The guidelines on qualitative measures for the likelihood and consequence of landslides and assumed level of risk are provided in Reference 4.

In this section the stability of the site at present is assessed based on AGS guidelines.

Applying the AGS guidelines, slope instability risk associated with the site before development works is assessed as follows:

- **Qualitative Measures of Likelihood** - It is our assessment that the *event might occur under very adverse circumstances* ($\approx 10^{-4}$), i.e.: it is **"Unlikely"** to occur.
- **Qualitative Measures of Consequences to Property** - It is our assessment that the consequences to the property would be **"Minor"**, Limited damage to part of structure, or part of site requiring some reinstatement/stabilisation works.
- **Qualitative Risk Analysis – Level of Risk to Property** - Based on the above Qualitative Measures, the site is assessed to have a **"Very Low to Low "** risk level of slope instability resulting from down slope soil creep and or landslide mass movement.

The definitions of the risk levels are provided in Reference 4 and an abstract is presented below:

Risk Level		***Implication***
VH	*Very High Risk*	*Extensive detailed investigation and research, planning and implementation of treatment options, essential to reduce risk to acceptable levels; may be too expensive and not practical.*
H	*High Risk*	*Detailed investigation, planning and implementation of treatment options required to reduce risk to acceptable levels.*
M	*Moderate Risk*	*Tolerable, provided treatment plan is implemented to maintain or reduce risks. May be accepted. May require investigation and planning of treatment options.*
L	*Low Risk*	*Usually accepted. Treatment requirements and responsibility to be defined to maintain or reduce risk.*
VL	*Very Low Risk*	*Acceptable. Manage by normal slope maintenance procedures.*

Based on **"Very low to Low"** risk level of slope instability, the proposed development have minimal impact on the slope instability, and can be addressed with maintenance procedures and well defined accountability and maintenance program to minimize risk.

7.3 Eastern Hilly Section

For the eastern hilly section western platform with the proposed designated storage areas and building platform, the assessment on the subsurface condition and the overall site slope angle of approximately 35°, the site would be in medium to High hazard area in terms of landslide instability.

The Australian Geomechanics Society (Reference 4) recommends the landslide risk of a site be assessed on the basis of the likelihood of a landslide event and the consequences of that event. The guidelines on qualitative measures for the likelihood and consequence of landslides and assumed level of risk are provided in Reference 4.

In this section the stability of the site at present is assessed based on AGS guidelines.

Applying the AGS guidelines, slope instability risk associated with the site before development works is assessed as follows:

- **Qualitative Measures of Likelihood** - It is our assessment that the *event might occur under very adverse circumstances* ($\approx 10^{-4}$), i.e.: it is **"Possible"** to occur.
- **Qualitative Measures of Consequences to Property** - It is our assessment that the consequences to the property would be **"Medium"**, Moderate damage to some of structure, or significant part of site requiring large stabilisation works.
- **Qualitative Risk Analysis – Level of Risk to Property** - Based on the above Qualitative Measures, the site is assessed to have a **"Moderate"** risk level of slope instability resulting from down slope soil creep and or landslide mass movement.

The definitions of the risk levels are provided in Reference 4 and an abstract is presented below:

Risk Level		***Implication***
VH	*Very High Risk*	*Extensive detailed investigation and research, planning and implementation of treatment options, essential to reduce risk to acceptable levels; may be too expensive and not practical.*
H	*High Risk*	*Detailed investigation, planning and implementation of treatment options required to reduce risk to acceptable levels.*
M	*Moderate Risk*	*Tolerable, provided treatment plan is implemented to maintain or reduce risks. May be accepted. May require investigation and planning of treatment options.*
L	*Low Risk*	*Usually accepted. Treatment requirements and responsibility to be defined to maintain or reduce risk.*
VL	*Very Low Risk*	*Acceptable. Manage by normal slope maintenance procedures.*

Based on **"Moderate"** risk level of slope instability for the eastern hilly section of the site, the proposed development have to consider the foundation loadings that need to have minimal impact on the slope stability (e.g. pile foundation) or can be addressed with the provision of retaining structures design to withstand the imposed loadings tin order to minimize the impact on the stability of the surrounding slope configurations.

8.0 RETAINING STRUCTURES AND FOUNDATIONS

8.1 Excavation Conditions

Based on currently information provided, excavations may be required for the construction of retaining structures within the development compound. However, should any excavation be considered or planned in the final stages of the development design, the following should be considered.

It is expected materials encountered during excavation are likely to comprise stiff to hard clays. Excavation of soil-based materials and extremely to highly weathered sandstone may be achieved using conventional earthmoving equipment such as backhoes or tracked excavators. Heavy ripping and/or vibratory rock breaking techniques are not likely to be required.

Site earthworks should be properly drained to minimise the effects of wetting up and softening of exposed, natural subgrade soils, which may be caused by extraneous water sources and climatic variations. Trafficability across the site may be restricted to tracked plant during and following periods of wet weather and the trafficking of wet subgrades with any plant would be expected to result in significant subgrade damage. Should possible bulk excavation be terminated within the silty clay or clay layers, it is considered the natural materials at the base of such excavations may be trafficable under favourable climatic conditions and lack of groundwater presence. However, similar trafficability problems, as outlined for site subgrades, may be anticipated where “wetting” may occur.

It is therefore suggested that consideration be given to the placement of a granular layers to provide convenient working platforms and improve site trafficability. Such a layer would also significantly assist in reducing potential drying out of reactive soil subgrades. Where such platforms are to be utilised for the support of heavy machinery or plant, it may be appropriate to design these platforms to such loads and if necessary have these confirmed and inspected by a geotechnical engineer.

8.2 Groundwater Management

Ground water was not observed within the investigated depths at the time of the investigation. However, it should be noted groundwater levels may vary subject to seasonal fluctuations, rainfall, prevailing weather conditions and also future development of the surrounding lands.

It is recommended possible groundwater presence or levels be confirmed if construction is undertaken during or following adverse weather or if a significant time period elapses between this investigation and construction.

Should groundwater or surface seepage be encountered during excavation, it is possible foundations excavations may be dewatered using appropriate drains and sump pits with a suitable pumping system.

A groundwater monitoring programme may be adopted prior to construction to confirm the groundwater regime and determine the design of appropriate drainage measures should groundwater presence be identified as problematic to construction or ongoing performance of structures.

8.3 Temporary Batter Slopes

Temporary batter slopes may be appropriate for possible excavations or cut slopes provided excavations or cut slopes are set back sufficiently from common site boundaries to facilitate the formation of the recommended safe temporary batters outlined in Table 2.

Table 2 - Minimum Temporary Batter Slopes

Materials	Temporary (Horizontal: Vertical)
Stiff CLAY	3.0:1.0
Very Stiff/ Hard Silty Clay	2.0:1.0
Distinctly Weathered Sandstone	1.0:1.0

Temporary surface protection against erosion may be provided by covering the batter with plastic sheets or other applicable methods. It is considered that plastic sheeting, if adopted, should extend at least 1.5m behind the crest of the cut face or at least up to the common site boundaries. Plastic sheeting should be positioned and fastened to prevent water infiltration into or onto the batter which may lead to softening and possible instability. All stormwater run-offs should be directed away from all temporary and permanent slopes.

8.4 Retaining Structures

In the long term, the excavation faces must be retained by engineered retaining structure in particularly along the eastern hilly section of the site. These structures should be designed to withstand the applied lateral pressures of the soil/rock layers, the existing surcharges in their zone of influence; including existing structures, and construction related activities, and also hydrostatic pressures (if it is appropriate).

The pressure distribution on cantilever retaining structures, only due to the earth pressures and surcharges behind the wall, may be assumed to be triangular and estimated as follows (ignoring cohesion effect):

$$p_h = gkH + qk$$

Where,

p_h = Horizontal pressure (kN/m^2)
g = Wet density (kN/m^3)
k = Coefficient of earth pressure (k_a or k_o)
H = Retained height (m)
q = Surcharge pressure behind retaining wall (kN/m^2)

For the design of flexible retaining structures, where some lateral movement is acceptable, an active earth pressure coefficient is recommended. Should it be critical to limit the horizontal deformation of a retaining structure, use of an earth pressure coefficient at rest should be considered. Recommended parameters for the design of retaining structures are presented in the following Table 3.

Table 3 : Geotechnical Design Parameters

Materials	Unit Weight (kN/m^3)	Active Earth Pressure coefficient (K_a)	At Rest Earth Pressure Coefficient (K_o)	Passive Earth Pressure coefficient (K_p)
Stiff/very stiff silty clay and clay	18	0.40	0.57	2.46
Hard silty clay	20	0.33	0.50	3
Extremely weathered sandstone (Class V or IV)	20	0.25	0.4	200kPa*
Distinctly weathered sandstone (Class IV or III)	22	0.15	0.25	400 kPa
Slightly weathered to fresh sandstone (Class III/II)	23	NA	NA	750kPa

* Passive lateral earth pressure.

The above coefficients assume that ground level behind the retaining structures is horizontal and the retained material is effectively drained.

8.5 Foundation System

Depending on proposed structures, associated structural loadings, tolerable settlements and cost-benefit considerations, foundation systems founded on very stiff to hard clays or silty clay may be applicable. Possible foundation systems for various structures founded within the soil profile may consist of shallow pad and strip footings and piled rafts.

Shallow foundation systems or piles, with minimum length of 3.0m, founded within the very stiff clay may be designed adopting an allowable end bearing pressure of 200 kPa with this value being increased to 500 kPa for systems founded within the hard clay-based materials.
End bearing piles founded within low strength, Class IV sandstone may be designed with a maximum allowable end bearing pressure of 1000 kPa. A minimum socket length of 0.5m is considered appropriate.

In case piles are to be founded on clay layers, potential total and differential settlements should be evaluated under service loadings and be considered in the structural design. Long-term creep/consolidation settlements should also be taken into account.

Ground slabs founded on stiff clays or medium dense sands may be designed using an allowable bearing capacity of 150 kPa.

Foundation systems associated to independent structures should be founded on similar foundation materials to minimise possible differential settlements.

Should groundwater flow or surface runoff be encountered within excavated footings, footing excavations should be dewatered and be clean and free of loose debris and wet soils prior to concrete placement or correct underwater placement techniques should be adopted. An experienced geotechnical engineer or engineering geologist should inspect foundation excavations at the time of excavation and prior to reinforcement placement and construction to ensure suitable bearing materials satisfying design criteria have been achieved.

"Geotechnical Strength Reduction Factor" of piled foundations can be determined in accordance with AS2159-2009 Cl.4.3.1. In absence of loading test of the piles, the factor can be determined based on risk ratings associated to Site, Design and Pile Installation; based on available information it could vary between ϕ_g=0.45-0.60 for low redundancy systems and ϕ_g=0.53-0.70 for high redundancy systems.

8.6 Site Classification

This site is classified as Class M (moderately reactive) in accordance with the requirements of AS 2870-2011 (Ref 2). This is based on observations made at the investigation holes and test pit as well as from results obtained from the laboratory testing. It should be noted that the classification must be reassessed should the ground profile changes significantly in areas by adding fill or removing soil.

8.7 Preliminary Pavement Design

Subgrade CBR

Observation at the boreholes indicate that subgrade materials for the proposed pavements across the site are likely to be stiff to hard silty sandy clay/ clayey sand with SPT numbers ranging from 13 to over 20. This is equivalent of a DCP Index of 50 mm/blow or lower. The CBR of subgrade material may be approximated from Figure 5.3 of Austroads Pavement Design Guide (2012) based on DCP Index for such fine grained soils. Reference to this figure suggests a minimum CBR of 3.5% for these materials, which is chosen as design CBR for the preliminary pavement design.

Design Traffic Load

A design traffic load of $5x10^5$ ESA is assumed for this preliminary pavement design considering a design life of 25 years for pavements. This is to be confirmed before construction considering the vehicle types accessing the site and frequency of their access.

Pavement Thickness

We have carried out a preliminary pavement design for a flexible pavement option adopting the above assumptions in accordance with Figure 8.4 of the Austroad Design Guide (2012). The pavement should comprise the following layers:

- 40 mm thick dense grade asphalt AC14 on 7-10mm primer seal coat,
- 120 mm thick DGB20 Base Course compacted to 98% Standard Compaction Ratio, and
- 330 mm thick DGS40 Sub-base Course in two equal layers compacted to 98% Standard Compaction Ratio.

8.8 Others Comments

The following are to be considered in the design and construction of proposed structures and pavements:

- Some variability in subsurface condition must be anticipated within the site;
- It is recommended that additional site investigations (confirmatory holes or pits) may be required at critical locations (e.g. on steeply sloping ground) to ensure the local and regional stability are assessed with respect to the proposed engineering elements and design performances;
- As part of site preparation prior to the construction works, all vegetation, topsoil and any uncontrolled fill shall be removed;

- All footings are recommended to be found on same bearing stratum;
- The base of all footing excavations are to be inspected by a qualified geotechnical to ensure footing will found on competent materials as designed for;
- Should variation in descriptions in soil types, colour or depths be discovered during construction, a geotechnical engineer should be notified so that potential influence on the footing as it may affect surrounding engineering elements may be assessed; and
- It is recommended to consider the comments provided in the CSIRO Sheet BFT-18 'foundation Maintenance and Footing Performance' (Appendix C).

9.0 CONCLUSIONS

This report presents the results of a preliminary geotechnical investigation for the proposed development of a Resource Recovery Facility located at 50 Wylie Road Kembla Grange NSW. The site is classified as Class M (Moderately reactive) at the time of the field works, and a stability assessment has been made with reference to Chapter E12 of the Wollongong DCP 2009. Geotechnical recommendations have been provided to address the issues as requested.

We consider that the proposed development is feasible in this site subjected to the recommendations presented in this report.

For and on behalf of
Benviron Group

Noriman Mak
Geotechnical Engineer
MIEAust., RPE (Civ, Geo), NPER (Civ, Geo)

LIMITATIONS

The assessment of the sub-surface profile within the proposed development area and the recommendations presented in this report are based on limited information available to date.

The recommendations and advice presented in this report on soil and rock condition is considered to be indicative only as only very limited areas were assessed on site to date. Site inspection by a consulting Geotechnical Engineer or Engineering Geologist are to be undertake when further investigation works are to be carried out to confirm the condition of founding materials in which this geotechnical assessment recommends.

Anecdotal evidence and Information provided by client is assumed to be relevant and to the best of knowledge be appropriate for its interpretation.

There is a possibility that the actual geotechnical and groundwater conditions across the site could differ from the inferred geotechnical assumptions and derivations on which our recommendations are presented in this report.