JOHN HOLLAND PTY LTD

# SOUTHWOOD PROJECT – GEOTECHNICAL & HYDROGEOLOGICAL INVESTIGATIONS

HUON DISTRICT, TASMANIA

HO154/1-AG

30 August 2002

HO154/1-AG AG
30 August 2002

John Holland Pty Limited,
70 Trenerry Crescent,
ABBOTSFORD VIC 3067

Attention: Mr Trevor Webster

Dear Sir,

RE: Southwood Project –Geotechnical & Hydrogeological Investigation

Please find attached our report on the geotechnical and hydrogeological assessment of the proposed Southwood project development at a site near the confluence of the Arve and Huon rivers.

Please feel free to contact the undersigned if you have any queries on this report.

For and on behalf of

COFFEY GEOSCIENCES PTY LTD

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# 1. INTRODUCTION

## 1.1 General

This report describes the geotechnical investigations and hydrogeological investigations carried out for John Holland Pty Ltd (JH) on the proposed project located at the Southwood site, Huon, Tasmania.

Mr. Trevor Webster of JH commissioned Coffey Geosciences Pty Ltd (Coffey) to carry out the geotechnical investigation works for this project following acceptance of Coffey's proposal ref Ho154/1-AB of 1 August 2002, based on the brief supplied by John Holland Development Investment Pty Ltd (JHDI).

It is understood that the project comprises the construction of the following main features:

- Power Station;
- Mechanising Yard;
- Veneer Mill and Saw Mill;
- Wood fibre Mill and Fuel processing; and
- Stromwater Pond 1

Coffey has undertaken a preliminary geotechnical assessment for this site and the information is presented in the report referenced HO53/1-AB, dated 23 August 2000. The findings of that report have been addressed as part of the evaluation carried out in this report.

An A1 sized contour plan of the site and proposed layout was provided by Forestry Tasmania for field use. Preliminary earthworks site plans and sections produced by GHD, were provided by JHDI in PDF format to assist in planing the field work and provide a base for geotechnical plans. Recent air photos (24/2/02) were sourced from Services Tasmania.

The development is expected to cover some 40ha area. Preliminary earthworks sections for the merchandising yard (GHD drawings 3210596-04 to –07) indicate cuts and fills up to 9m high, but typically 2 to 4m high.

## 1.2 Scope of Work

The purpose of this investigation, as described in the JHDI brief are to:

- Meet the relevant conditions set out in the development approval (Geo1, Geo2, GW1, part GW3, and GW4);
- Provide information to enable the economic design of the various structures on site.

The scope of work was limited by JH due to time constraints and site access issues prior to construction commencing. The scope is, however consistent with a staged approach appropriate to investigation and development of such a large site where final decisions on the exact location and nature of development have yet to be finalised. The scope of work adopted was as follows:

- 1 borehole to assess foundation conditions for potential pile or large scale footings at the power station site. To be converted to a temporary groundwater monitoring well (destroyed during construction of the power station) to satisfy GW4 and GEO1 assessment;
- 1 'deep' groundwater monitoring well (to standard environmental protocols) to assess and monitor

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groundwater conditions before and during construction and operation. Location to be confirmed with GHD and MRT;

- 1 borehole completed as a groundwater monitoring well (replaces the original probe hole in JH brief) to be located immediately down gradient of the proposed stormwater storage dam (as shown on current drawings) to assess groundwater conditions on the up slope part of the site and allow monitoring down gradient of the storage dam during operations;
- 4 test pits across the proposed mil sites to assess foundation bearing capacities;
- 3 pits to obtain CBR samples from the sub grade depth (to be confirmed) for proposed hardstand areas (pavement design can be provided by Coffey if required);
- Clay borrow assessment by test pitting in areas identified by the walkover;
- Laboratory testing of clay borrow materials to assess suitability in terms of DPIWE guidelines and relevant national standards;
- Laboratory testing of subgrade samples to assess CBR characteristics;
- Development, permeability testing and sampling of groundwater monitoring wells;
- Reporting of geotechnical findings and recommendations for bearing capacities, cut and fill batter angles, foundation concepts, pond concepts and any further work required to achieve detailed design;
- Assessment of hydrogeological conditions on the site, development of a conceptual model for input to an impact risk assessment by GHD (water quality data will not be available in time to incorporate in a draft report).

## 2. FIELD WORK

### 2.1 General

The field work consisted of:

- Drilling three boreholes (SMB1, SMB2 and SMB3) using a truck-mounted drill rig at the locations shown on Figure 1, to a maximum depth of 31.5 metres (BH1). Locations were limited by the inability to access areas away from established roads and formed tracks. Logging of the materials penetrated was undertaken during drilling, followed by installation of groundwater monitoring wells in the three boreholes for water sampling. Development, purging, sampling and permeability testing of the monitoring wells was also conducted (see Section 2.2);
- Excavation of 13 test pits using a 20t excavator at the locations shown on Figure 1, to a maximum depth of 6.7 metres (TP1). Logging, soil sampling and observations of groundwater conditions were were carried out d uring excavation. Some of these test pits were excavated to recover samples for CBR and associated testing (STP2, STP3 & STP13), while STP1, STP4, STP5, STP7 and STP8 are "foundation" pits for the assessment of the soils at the formation levels of proposed building. Test pits STP6, and STP9 to STP12 were excavated at the north western corner of the development site investigating the presence of clayey materials for construction of a dam, intended to be located in that vicinity.

- Geological and geomorphological observations from site walkovers, and office based assessment of air photos.

The three boreholes were planned to intersect varying underlying geological conditions, due to proposed drill depths and location within the Southwood site. Unforseen conditions present in SMB1 resulted in extension of the borehole some 20m beyond the proposed target depth to establish bedrock. Easy washbore drilling in silt, sand and clay layers, with only 2 recognised gravel/ cobble horizons, persisted to a depth of 27.2 metres, before any rock core was recoverable. The conditions encountered in this borehole extended the total drill metres for the project, and the total drilling program by 1.5 days.

The other two boreholes (SMB2 and SMB3) did not encounter hard rock thatcould deliver a core sample. SPT tests were carried out within clay layers in these boreholes.

The borehole drilling and excavation of the test pits were carried out in the period 12 to 15 August 2002. Site walkovers were undertaken on 8 August and 27 August 2002, with bore permeability testing conducted on 20 August 2002.

The engineering logs and photos of boreholes and test pits are presented in Appendix A, together with explanation sheets defining the terms and symbols used in their preparation. Monitoring well details and falling head permeability testing records are presented in Appendix B.

The test pits have been located by hand held GPS and map estimates. Forestry Tasmania personnel surveyed the locations of the boreholes .

### 2.2 Monitoring Well Installation

Three ground water monitoring wells were installed in boreholes drilled between the 12$^{th}$ and 15$^{th}$ August. The wells are designated SMB1, SMB2 and SMB3 (see Figure 1).

All wells were constructed using machine slotted 3m PVC screen at the base of the hole and screw-jointed PVC casing to surface. Graded sand was used to backfill the boreholes around each monitoring well, with 0.5m of bentonite and 0.5m concrete comprising the final 1m below surface. All wells were completed with 100mm PVC casing covers and caps, standing approximately 1m above ground surface.

Soil and rock types encountered during drilling of the wells were described in the field and logs of boreholes are provided in Appendix A.

#### 2.2.1 Well Development

Wells were developed on completion of each borehole. SMB1 failed to record a standing water level once installation was complete. Development of SMB2 and SMB3 continued until the bulk of drilling fines had been removed, or the water level was reduced to the base of the well.

A summary of conditions for each groundwater well is provided in Table 1.

#### 2.2.2 Bore Survey

The completed monitoring wells (SMB1, SMB2 and SMB3) were surveyed to establish their respective locations and elevation. Survey locations were conducted by Forestry Tasmania and are presented on the logs in Appendix B.

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### 2.2.3 Groundwater Sampling

Groundwater sampling was conducted by a Coffey scientist on 15th August 2002. Samples were taken to establish baseline information for the long-term monitoring of water quality. Prior to sampling, the water level at each bore was recorded and then each well was purged to standard environmental protocols, i.e. between 3 and 5 bore volumes were removed and field water quality parameters (pH, EC and temperature) were measured prior to sampling.

Groundwater samples were recovered from two of the three wells SMB2 and SMB3; with samples retaining the borehole nomenclature. Samples were decanted immediately into appropriate sample containers, then placed in eskies with ice, and dispatched to the designated laboratory (WSL) using standard environmental sample handling and chain of custody protocols.

All work was conducted in general accordance with standard Coffey environmental protocols with respect to sampling procedures.

Table 1: Summary of Monitoring Bore

| Bore No. | Construction & Monitoring Date | Drill Depth | Screen Interval | Geology of the screened interval | Measured depth from water to top of PVC | Stick up of PVC | RL of natural surface | pH | EC (dS/m) |
|---|---|---|---|---|---|---|---|---|---|
| SMB1 | 14/08/02 | 31.5m | 5.11-8.11m | Silty Sand | Dry well* | 0.6m | 103.55 | NA | NA |
| SMB2 | 14/08/02 | 10.8m | 6.85-9.85m | Silty Clay | 3.42m | 0.95m | 93.68 | 3.89 | 0.08 |
| SMB3 | 15/08/02 | 6.5m | 3.7-6.7m | Sandy Clay | 2.99m | 0.55m | | 5.15 | 0.21 |

* Borehole water level measured at 16.5m when borehole depth at 31.5m. Well installed as temporary monitoring point to assess perched groundwater within influence of the development.

## 3. LABORATORY WORK

### 3.1 Geotechnical

For the test pits located at the footprints of the proposed structures, a set of CBR, Standard Compaction and Atterberg Limits testing was undertaken on three samples recovered from test pits STP2, STP3 and STP13, to assess the soil subgrade for design of pavements in the merchandising yard and access roads. In addition, two samples were selected from test pits STP6 and STP11 for laboratory permeability, Standard Compaction and Atterberg Limits testing to assess the properties of the soils and weathered rock for possible use as clay liner for proposed dams on the site.

Samples collected during previous fieldwork by GHD (Sample 1, MY1 and MY2) and tested by Coffey, are included in this report for completeness. The results of the laboratory testing are summarised in Table 2 and presented in Appendix C.

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Table 2 Summary of Geotechnical Laboratory Test Results

| Test Pit Location | Sample Depth | Atterberg Limits | | | Std Compaction | | CBR | Permeability |
|---|---|---|---|---|---|---|---|---|
| | | PI (%) | LL (%) | Linear Shrink (%) | Max Dry Density (t/m³) | Optimum Moisture Cont (%) | (%) | (m/sec) * |
| STP2 | 2.0-2.5m | 10 | 35 | 6.5 | 1.64 | 19.0 | 13 | |
| STP3 | 2.0-2.5m | 18 | 36 | 7.5 | 1.90 | 13.0 | 11 | |
| STP13 | 1.0-1.5m | 6 | 27 | 3 | 1.73 | 18.0 | 20 | |
| STP6 | 0.4-1.2m | 26 | 55 | 10.5 | 1.44 | 30.5 | | 3.5E – 9 |
| STP11 | 0.4-1.2m | 15 | 37 | 7.5 | 1.44 | 21.5 | | 1.2E - 8 |
| Sample#1 (existing gravel pit) | Surface | N.P | N.P | 0 | 1.97 | 9.5 | 90 | |
| MPY1 | 0.5-1.0m | 9 | 28 | 6 | 1.71 | 19.5 | 20 | |
| MPY2 | >0.5m | 11 | 37 | 8 | | | | |

*compacted sample tested at MDD and OMC.

### 3.2 Water Quality

Water testing was undertaken on samples recovered from two of the three monitoring boreholes and sent to WSL laboratories in Melbourne, where the following tests were performed:

- Aromatic hydrocarbons (2 tests)
- TPH (2 tests)
- Total N (2 tests)
- Total P (2 tests)
- Anions content (Cl, PO4, SO4, CO3, HCO3) (2 tests)
- Cations content (Ca, Mg, Na, K, Al) (2 tests), and
- Faecal Coliforms & Streptococci (2 tests)

#### 3.2.1 Groundwater Analytical Results

The analytical results are summarised in Table 3 and presented in Appendix D.

Table 3 Summary of Groundwater Analytical Results

| Analysis | Monitoring Bore SMB2 | Monitoring Bore SMB3 |
|---|---|---|
| Faecal Coliforms | Absent | Detected |
| Faecal Strep. | Low levels present | Low levels present |
| Total N (mg/L) | 2.5 | 3.9 |
| Total P (mg/L) | 0.21 | 0.52 |
| Aromatic Hydrocarbons (mg/L) | <0.001 | <0.001 |
| TPH (mg/L) | <0.1 | <0.1 |

Note: details of anions and cations are provided in the laboratory reports.

# 4. SITE CONDITIONS

## 4.1 Location & Existing Developments

The proposed development site occupies an area of about 40ha and is situated on a broad ridge, trending roughly north-south, bordered on 3 sides (west, south, east) by an incised meander of the Huon River, where the Arve River flows into the Huon.

Existing site development consists of the recently constructed Geeveston/Judbury Road, bridge over the Huon River, forestry tracks, as well as other excavated flat areas.

## 4.2 Geology

The Geological Atlas 1:250,000 digital series Geology of Southeast Tasmania produced by Mineral Resources Tasmania (1999 edition) indicates that the proposed development site is underlain by Permian Age siltstone and sandstone deposits.

Further geology and geomorphology information can be found in the report titled "Geomorphic Values Survey: Barnbaback Block, Huon Forest District prepared by Mr. C Sharples for Forestry Tasmania (Huon Forest District). This report indicates that the area is underlain by Permian marine siltstone and sandstone bedrock, with an erosion remnant of Quaternary glacio-fluvial material forming a 'cap' on the top of the high level terrace at the site.

Observations for this study include the following:

- Bedrock comprising Permian siltstone, mudstone and some sandstone beds exposed in road cuttings (see Figure 2) and sub crop around the site. Highly weathered, low to medium strength, bedding dip approximately 5° towards the SW (with potential for minor variations), sub vertical joint sets striking NW-SE and NE-SW, and sub vertical shear/fault zones (containing altered and weathered material) striking approximately N-S;
- Quaternary glacio-fluvial alluvium, exposed in road cuts across the site, comprising Silty Sand,

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Sandy Clay and Silt with varying proportions of generally matrix supported, rounded gravel and cobbles (10 to 40%), and some Sandy Gravel lenses (clast supported). Sand and silt are generally fine grained, with low to medium plasticity clay and silt.

## 4.3 Geomorphology

### 4.3.1 General

The site is located on the crest and side slopes of a broad ridge with gentle upper slopes of approximately 5 to 10°, side slopes in the range of 10 to 20°, and locally 25 to 30° at the toe beside the Huon River (see Figure 2). The morphology is that of a remnant alluvial aggradation terrace at approximately RL100 to 106m that has been dissected by erosion of the Huon River (current approximate RL 40m) to produce lower degradation terraces and isolated remnants of alluvium (see Figures 2 to 4). Gullies and stream courses dissect the side slopes of the ridge, with head slopes in the region of 30°, discharge to the relatively narrow (<20m) modern floodplain of the Huon River. The Weld Plains to the west of the site represent a lower level (approximately RL 50m) alluvial terrace system that has also been mapped further downstream of this location.

Active natural erosion was not evident on any of the side slopes, which are blanketed with thick vegetation comprising button grass and heath on the terrace remnants and eucalypt forest on the slopes. The vegetation distribution reflects surface soil development, with a generally poorly drained peat sward occurring on the terrace remnants, and clay/silt colluvial soils developed over bedrock on side slopes.

Specific zones of groundwater seepage on side slopes were not observed during site walkovers due to the masking effects of persistent wet weather in recent months. However seepage is indicated towards the base of the alluvial terrace remnants and on the steeper lower bedrock slopes above the Weld Plains and the recent Huon River flood plain.

### 4.3.2 Landslides and Slope Stability

The previous work undertaken by Coffey (report ref HO53/1-AB, dated August 2000) had described a possible ancient landslide in bedrock on the side slopes at the SW end of the site (below the proposed power station and water pond). The geomorphology of this feature and 2 other possible large landslide features were confirmed by an overview of topographic maps and air photos for this study. The 3 possible landslide features are marked A, B and C on Figure 2.

A site walkover of the features has not revealed any field evidence to support the existence of landslides. If these features are landslides they are ancient failures that occurred in bedrock at a critical point in the down cutting of the Huon River, possibly several thousand years ago during rapid climate change at the end of the last glacial period.

Due to the age of features A, B and C, the degree of erosion and degradation, the absence of any evidence for recent or ongoing instability and the proposed location of development at least 50 to 100m from the crest of these features it is assessed that they do not pose a threat to site development.

Evidence of small scale recent and active slope instability, including in the heads of gullies, is absent. Soil creep is occurring on the bedrock and colluvium slopes along the Huon River, and there is potential for undercutting and local failures on these slopes. It is assessed that such failures would be very unlikely to undercut or other wise affect the main upper terrace area.

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### 4.4 Sub Surface Conditions

The information on sub surface conditions inferred from the test pits, boreholes and mapping observations that are to be encountered in the 2 main development areas is summarised in Tables 4 and 5 below. Natural scale cross sections presented in Figures 3 and 4 depict the main geological units, observed groundwater levels and extrapolated indicative bedrock structure.

Table 4: Southern Area, Main High terrace, Power station, Fuel storage, Merchandising Yard

| Geotechnical Unit | Thickness, m | Typical depth, m | Comments |
|---|---|---|---|
| Topsoil – Peat sward | 0.2-0.5 | 0.3 | Root zone & peat, generally wet, restricted drainage due to underlying coffee rock layer. |
| White Sand/Silt, or | 0.3-0.4 | 0.6 | STP1, STP3 &STP13; typically non-plastic silts and in a very loose state |
| Natural Clays | 0.3-0.5 | 0.5 | Occurs occasionally on the high level terrace in place of the white sand and coffee rock ( e.g. STP1), typical pocket penetrometer tests range 170-300 kPa |
| Organic/Fe layer, partially cemented ('coffee rock') | 0.05-0.2 | 0.7 | Underlying the white sand silt layer (e.g. STP3 & STP13) |
| Alluvium (Glacio-fluvial) | 5 to 25 | 6 to 26<br>Approx base RL 80m in the east to 90-100m in the west | Sand, silt and clay, fine grained, low to medium plasticity, with some gravel to gravelly. Some zones or lenses of sandy gravel (e.g. gravel pit in the NE of the terrace), medium dense to dense; soil matrix generally returned pocket penetrometer values of 300 to 600 kPa. Excavated in all test pits on the main terrace, except TP10 which encountered bedrock at 1m depth on the terrace margin. |
| Residual soils | 0.5 to 2 | 6 to 28 | 1.5m of black high plasticity clay intercepted in SMB1, possible weathered carbonaceous mudstone or organic soil at the base of the alluvium? |
| Bedrock | | | Siltstone, mudstone and sandstone, extremely to highly weathered and very low to low strength for in the upper 1 to 5m (depending on lithology and presence of steep shear zones). The 50% core recovery from 27.2 to 31.5m in SMB1 was extremely weathered, and fractured. |

Table 5 North West Area, Bedrock Slopes, Veneer Mill, Saw Mill, Woodfibre Mill & Dry Mill

| Ground Unit | Thickness, m | Typical depth, m | Comments |
|---|---|---|---|
| Topsoil | 0.05-0.5 | 0.3 | Silt/Clay soil with root zone & peat, generally wet |
| Residual soils | 0.4-0.7 | 0.9 | Typically clayey silts and sandy silts (uncovered at STP4 to STP12 inclusive). The residual cohesive soils were typically stiff, but occasionally firm, while the non plastic soils ranged from loose to dense |
| Bedrock | Extremely weathered approx 0.5-2 | Extremely weathered 2-3 | Siltstone and sandstone, extremely to highly weathered, very low to low strength (rock ripper used from 0.5m depth at STP4, 2.2m at STP5 & from 2.6m at STP7). Steeply joints dipping to wards the west noted in some test pits. |

### 4.5 Groundwater

Surface water and saturated topsoil conditions were prevalent at the time of the field work for this study. Ponded water in the soils on the main terrace and other remnants suggest that the drainage of the soil is impeded by the development of a thin partly cemented ‘coffee rock’ layer at about 1m depth..

The test pits generally did not encounter ground water, with the exception of the north-western corner of the site were some seepage was encountered in three of the test pits, as follows: STP6 (seepage from the southern side at 1.8m depth), STP10 (seepage at the northern corner at 0.9m depth), and STP12 (seepage at the southern side at 1.6m depth). This is the location of the proposed stormwater dam.

Standing water levels within two monitoring wells SMB2 and SMB3 were measured at 2.43m and 2.44m below ground level respectively. These water levels represent shallow groundwater within sandy aquifers overlying clayey alluvium. The monitoring well SMB1 is dry indicating that there is no perched aquifer at this location beneath the topsoil on the main terrace. The standing water level measured in the borehole before installing the well was 16.5m(with the hole collapsed back to 25m from 31m), suggesting that the silty sand and sandy clay materials are vertically well drained.

The results of hydraulic tests conducted in the monitoring wells (presented in Appendix B) indicate permeabilities in the order of $10^{1}$ to $10^{-2}$ m/day for the perched sandy aquifers and approximately $10^{-3}$ m/day for the unsaturated silty sand materials in SMB1.

A conceptual model for the groundwater system based on site observations and investigations is presented in Figure 5. This model is considered appropriate for developing a groundwater impact risk assessment and using as the basis for establishing a long term groundwater monitoring network at the site, once development locations are finalised and access is available to suitable monitoring sites.

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The major groundwater implication for development is that site earthworks that strip the upper layers of alluvium will be exposing the deeper alluvial aquifer to direct infiltration from site activities.

## 5. DISCUSSION AND RECOMMENDATIONS

### 5.1 Slope Stability

Natural slope stability is not considered to be a significant risk to the development provided standoffs of 100m from the potential ancient landslide features (Figure 2) and the crests of side slopes steeper than 20° are maintained. Any structures that are required inside the stand off zone will need a specific investigation to assess local subsurface conditions and risk of slope instability.

### 5.2 Earthworks for the Merchandising Yard

Site development will involve cuts up to 8 metres (as shown on Drawing 3210596-04, CH 100 at the location of the power station), and fills of up to about 7 metres (see CH 200 through the Fuel processing unit).

Excavation using ordinary earthworks machinery should be capable of excavating to the required foundation level, although ripping may be required on the western limit of the yard if siltstone and sandstone bedrock is encountered (will depend on final subgrade levels). The excavated materials should be suitable for usage to build up the formation levels for the foundations of the structures.

The general procedure recommended for engineered fill under roads and structures are as follows:

- Strip all existing fill, topsoil to spoil or stockpile for re-use as landscaping material only.
- Compact the exposed surface (upper 150mm) to a dry density ratio of 98% Standard Compaction (AS1289 54.1-1993). In any area where the above level of compaction cannot be achieved, excavate to a depth of 0.3 meters and fill with clean granular fill and compact as specified above.
- Place and compact clean fill in 150mm thick (compacted) layers to 95% Standard under roads and to 98% Standard under buildings and hard stand areas.
- The upper 150mm beneath road pavements should be compacted to 98% Standard Compaction.

Clay fill should be placed within +/- 2% of Standard Optimum Moisture Content. Where fill consists of clean sand and gravel, then it shall be compacted to a minimum 80% Density Index (AS1289 56.1-1993).

Where fill is placed on slopes in excess of 1V:8H (7°), a prepared surface should be benched/stepped into the natural slope. Proper sub-surface drainage should be provided beneath the new fill, as appropriate.

Based on the laboratory test results a subgrade CBR of 10 is recommended for the merchandising yard area.

### 5.3 Cut and Fill Batter Stability

Excavated slopes, not exceeding 8 metres height in natural ground areas should be constructed no steeper than the following:

- 1 V: 2H in alluvium, colluvium and residual soil;
- 1V:1.5H in extremely weathered rock;
- 1V:0.5H in highly weathered to fresh, sub-horizontally bedded rock.

Fill slopes constructed under engineering control should not be steeper than 1V:2H.

For all slopes:

- slope drainage should consist of catch drains
- erosion protection should be provided.

Support by retaining wall or specific investigation and design of slopes will be required if steeper slopes than those indicated above are required.

## 5.4 Foundations Of Structures

### 5.4.1 Shallow Foundations

Provided that the earthworks are undertaken in accordance with AS AS3798 "Guidelines on Earthworks for Commercial and Residential Developments", and the recommendations in Section 5.3, an allowable bearing capacity of 100 kPa should be available for the design of the foundations of new structures on fill.

Consideration should be given to the potential for differential settlement of foundations constructed partially on natural ground and partially on fill. Piled foundations for footings and slabs on fill may be appropriate for settlement sensitive structures.

Subject to inspection of all footings by a suitably qualified and experienced geotechnical engineer at the time of the foundation excavations, an allowable bearing capacity of 200 kPa should be available on alluvium and residual soils.

Shallow footings on weathered rock may have an allowable bearing capacity of 1000kPa, subject to inspection of all footings by a suitably qualified and experienced geotechnical engineer.

### 5.4.2 Deep Foundations

Piled foundations for heavily loaded structures on alluvium could include driven or bored piles. Preliminary estimates of allowable capacity are as follows:

- Driven or bored piles in very stiff clay, shaft resistance 20 to 25kPa;
- Driven piles in granular soils, shaft resistance 20kPa;
- Bored piles in granular soils, shaft resistance 10kPa;
- Driven piles end bearing capacity on extremely to highly weathered rock, 1000kPa;
- Bored piles end bearing capacity on extremely to highly weathered rock, 500kPa.

Note that large diameter bored piles may have advantages in terms of capacity achieved through large shaft surface area and stiffness of the pile for sensitive structures. Depending on the sensitivity of structures to settlement piles may have to be taken to rock. Further investigation will be required to assess site conditions sufficiently for detailed design and costing of foundations.

## 5.5 'Clay' Liner for Dams

Samples collected from residual soil and weathered rock in the NW area of the site are considered suitable for use as low permeability liner materials. The low plasticity, observed field moisture contents and compacted permeability values of $1x10^{-8}$ to $3x10^{-9}$ m/s indicate that design requirements required in the development approval can be achieved. Borrow would involve stripping of approximately 0.3m of top soil followed by excavation of 1 to 2m of suitable material. Further assessment and testing will be required when a clay borrow area is finalised to ensure that the overall (bulk) natural properties will be suitable.

### 5.6 Construction Monitoring of Merchandising Yard

Variations in ground conditions may occur between test locations. If conditions other than those described are encountered, further advice should be sought without delay.

During excavation, site visits should be made by a Geotechnical Engineer or Engineering Geologist to ensure that the uncovered ground conditions are as those anticipated during the site investigation stage.

During the field compaction of the fill materials, layer thickness and coverages by the compactor should be carefully controlled and density in places and compaction tests taken as required for the relevant level of fill construction.

### 5.7 Further Investigations

Additional investigations will be required at the location of all structures to assess specific footing and foundation conditions. This work should be conducted in conjunction with detailed design studies when the location of structures has been finalised. The power station site in particular will require investigation to establish the parameters for the design of piled foundations. We recommend that at least 3 boreholes will need to be drilled to rock, with Standard Penetration Tests undertaken at 1.5 metre intervals. Coring of the rock should be conducted for at least 3m in at least 2 of the boreholes.

Further investigation and laboratory testing is also recommended on the foundations of and the materials to be used for the dam construction.

Establishment of a long term groundwater monitoring well system will be required once the location of structures is finalised and access is available to suitable long term monitoring sites. The long term monitoring system will need to be designed to allow monitoring of the major aquifer systems and associated contamination pathways identified in the conceptual groundwater model. At this stage we would envisage an additional 3 or 4 wells.

For and on behalf of

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## APPENDIX A

Engineering Logs of Boreholes and Test Pits

## APPENDIX B

Monitoring Bore Permeability Test Results

## APPENDIX C

Laboratory Test Results – Soil

## APPENDIX D

Laboratory Test Results – Groundwater