

# **Block 51, Majura Road, ACT Proposed Dragway Geotechnical Investigation Report**

June 2006

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ACT Chief Minister's Department

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# 1. Introduction

## 1.1 General

This report covers the geotechnical investigation carried out by Parsons Brinckerhoff Australia Pty Ltd (PB) on behalf of the ACT Chief Minister's Department (CMD) for a geotechnical investigation of a potential dragway located on Block 51, Majura Road, ACT. The work was commissioned by Mr Philip Mitchell of the CMD on 19 April 2006.

The works were performed in general accordance with our proposal reference A352-2006-GP7/LT\_5661/DG/mg dated 31 March 2006. Additional rock testing was undertaken in accordance with a variation sent and approved by email on 30 May 2006.

## 1.2 Purpose and Scope of Investigation

The purpose of the investigation was to assess the suitability of the site for the construction of a dragway complex. This included the assessment of:

- the excavation conditions of soil and rock on site
- foundation conditions
- consolidation of engineered fill embankments
- groundwater conditions.

It is understood that the design is in a preliminary state. Option 1 – International Facility (Reduced Scope) as outlined in GHD's Report for Dragway Facility, Block 51, Majura, was used as a guideline. This report included a long-section (GHD – Option 1 Plan & Profile International Reduced Scope, Drwg No 23-11385). This option assumes:

- A maximum dragway cut depth of approximately 3m.
- Spectator seating (for 5,000) and noise mitigation to either require earth mounding of approximately 7.5m height above track level to provide for grassed embankment levels, or alternatively a 3m high mound with the additional height required for noise mitigation provided by a noise wall.
- The dragway to consist of flexible pavement with an asphalt surfacing 60mm thick.
- The burnout area to be concrete.
- Internal access roads to be gravel, rather than sealed.
- Parking areas to be grass, rather than gravel.
- No amenity buildings, but portable structures.
- Lighting of dragway, the site entrance and the major intersection of site will be required.



- A box culvert to allow for runoff along the existing drainage line below the dragway.
- Some pipe drainage system along the dragway.
- Swale drains and culverts for drainage to parking areas and access roads.
- Water (and sewerage) pipes.

## 2. The Site

### 2.1.1 Site Location

Block 51, Majura Road is located approximately 2 km north of Canberra International Airport and immediately south-west of the Defence firing range (*Figure 1*). The site is approximately 1.1 x 0.7km in size and is located within the Majura Valley.

### 2.1.2 Site Description

The site is bounded by a dirt road running sub-parallel to Majura Road to the west. Woolshed Creek flows west of the site between the gravel road and Majura Road. Most of the south-western side of the site is relatively flat, due to alluvial deposits along Woolshed Creek. Toward the north and east low hills are encountered. Boulders of volcanic rocks can be found at the surface of the hills. A gravel road and overhead power lines run through the site to a farming property at the eastern edge of the site. A creek or drainage line runs through the south-eastern corner of the site, consisting of creek beds and ponds. Some localised earthwork can be assumed to have taken place around parts of the road and some ponds.

The site is vegetated with grass and occasional clusters of eucalyptus trees and is currently being used as grazing land for cattle. Several barbed wire fences divide the land into plots, especially in the south-eastern part of the site.

### 2.1.3 Regional Geology

Reference to the Canberra (1:50, 000) geological sheet indicates that the site is likely to be underlain by Silurian Mount Ainslie Volcanics and Quaternary alluvium deposited on the Woolshed Creek floodplain (*Figure 2*). The volcanics generally include dacite and andesite, in part foliated, agglomerate, minor tuff and shale. A near vertical north-south striking foliation is indicated for this foliation. The alluvial sediments comprise sands, gravels and clays. A fault line is suspected to cross the site.

## 3. Method of Investigation

### 3.1 Site Plan and Test Pit Locations

During a pre-work site visit on 15 May 2006 test pit locations were determined and fixed using handheld GPS. The locations were plotted on an aerial photograph of the site, (*Figure 3*). GPS coordinates for the test pits are given in Table 3.1 below. The coordinate system used was Australian Map Grid Zone 55, MGA 94\_Z55. Contour lines were plotted based on information provided by CMD's 'Canberra Dragway Discussion Paper – Figure 2'. There are several known cultural heritage sites on the site and care was taken to not work in their vicinity.

**Table 3.1: Positons of Test Pits**

<b>Test Pit Number</b>	<b>Northing MGA94_Z5 5</b>	<b>Easting MGA94_Z5 5</b>	<b>Test Pit Number</b>	<b>Northing MGA94_Z5 5</b>	<b>Easting MGA94_Z5 5</b>
TP1	6095718	699987	TP7	6095493	700435
TP2	6095435	700042	TP8	6095386	700391
TP3	6095559	700195	TP9	6095289	700503
TP3B <sup>#</sup>	6095567	700201	TP10	6095185	700447
TP4	6095482	700281	TP11	6095175	700626
TP5	6095355	700241	TP12	6095269	700797
TP6	6095203	700223	TP13	6095085	700733

<sup>#</sup> TP3B is an additional test-pit excavated to confirm very shallow refusal in TP3.

The locations are approximate and assessed to be accurate to about  $\pm 2\text{m}$ , due to limitations of the handheld GPS.

### 3.2 Underground Services and Unexploded Ordinance

The test pit locations were scanned for unexploded ordinance prior to excavation. This work was undertaken on 17 and 18 May 2006 by a sub-consultant engaged by CMD.

Dial-before-you-dig service plans were ordered prior to conducting the work and a Telstra accredited underground services locator from National Cable Locators traced and marked known services on site on 18 May 2006, prior to excavation.

### 3.3 Test Pit Excavation

Fieldwork comprised the excavation of fourteen (14) test pits at GPS fixed positions. Thirteen (13) test pits were planned and one additional test pit was excavated in rock on

the hillside. A 20 ton ZX 200 excavator with a tiger-tooth 800mm bucket and a smooth ripping tyne on a quick-hatch system was used. Eight (8) test pits were placed along the dragway alignment, four (4) to the south of it, in areas of proposed parking lots and ponds, and two (2) test pits were placed to the north of the proposed dragway to target areas for other infrastructure.

The test pits were excavated to bucket refusal or to maximum reach of about six meters. When bucket refusal was reached the test pits were continued by using a smooth ripping tyne. The subsurface profile was logged and sampling was undertaken for representative soil and rock samples for laboratory testing as described in *Section 3.5*.

### **3.4 Test Pit Logging**

All fieldwork was overseen by an engineering geologist from Parsons Brinckerhoff, experienced in site investigation work. Whilst on site, the geologist logged the encountered subsurface profile and photographed the test pits. Where the ripping tyne was deployed the ease of ripping as well as the block size of ripped rock was recorded.

Hand penetrometer tests were carried out on cohesive soil samples at regular intervals and /or change of soil type to assess the strength of the subsurface soil.

Groundwater inflow levels were recorded where encountered. Due to the presence of livestock on the site test pits could not be left open for water levels to reach equilibrium.

Engineering test pit logs are presented in *Appendix A* along with sheets defining the terms and symbols with which they were prepared, as well as photographs of the test pits.

### **3.5 Soil and Rock Testing**

#### **3.5.1 Soil Testing**

Selected soil samples collected during the fieldwork were sent to Coffey Geosciences Pty Ltd, a NATA registered soil testing laboratory in Canberra. The following laboratory tests were carried out:

- 4 CBR and standard compaction tests
- 4 Atterberg Limits and moisture content tests
- 4 Emerson crumb tests

Laboratory reports are presented in *Appendix B*.

#### **3.5.2 Rock Testing**

Irregular lump Point Load Strength Index Testing was undertaken on selected ripped rock samples. The tests were undertaken at Parson Brinckerhoff's Sydney office. The results of the Point Load Strength Index Testing are presented in *Appendix C*.



Testing for acid producing potential of rock-samples from the dragway alignment was undertaken by NATA accredited SGC Laboratories in Cairns.

## **4. Results of Investigation**

### **4.1 Ground Conditions**

Subsurface conditions encountered in the test pits are summarised below. References should be made to the test pit logs in *Appendix A* for full details.

#### **4.1.1 Topsoil**

Topsoil was encountered in most test pits, and ranged in thickness from 0.1m to 0.4m. It can generally be described as light brown to brown, occasionally red-brown, clayey sand or gravelly clayey silt / silty clay with abundant rootlets in the top 0.1m. It has a loose (L) to medium dense (MD) relative density.

#### **4.1.2 Fill**

No fill was clearly identified on site. Possible fill was identified in test pit TP10 from surface to 0.8m.

#### **4.1.3 Alluvial Soil**

Alluvial soils were encountered in all test pits with the exception of three test pits on or at the foot of a hill that is made up of volcanic rocks. The alluvial soils are made up of flood plain and creek bed deposits and include red, red-brown and orange-brown mottled grey silty clay, sandy clay and gravels. The silty clay and sandy clay generally has a medium to high plasticity and very stiff (VST) to hard (H) consistency. The gravels are generally fine to coarse grained, subangular to (sub-) rounded in shape and are usually mixed with sands and fines.

#### **4.1.4 Residual Soil**

Residual soils were encountered in most test pits with the exception of two test pits (TP6 and TP8) where alluvial deposit were encountered to maximum reach of 6m. These soils generally consist of orange mottled grey, occasionally grey-green, or orange red sandy clays and clayey sands, often with gravels and/or cobbles of underlying bedrock. The sandy clay generally has a low to medium plasticity and stiff (ST) to hard (H) consistency.

#### **4.1.5 Rock**

Rock (to bucket refusal) was encountered at shallow depths (0.4-0.8m) on the hill, at 1.8m at the foot of the hill and between 3.0 and 4.7m in five of the eleven test pits on the valley floor. Three types of rock were identified on site. The majority of the site is underlain by dacite, with occasional flows of andesite. However, red mudstone was identified in test pits TP9 and TP8, where it was excavatable to the refusal depth of 6.0m. Ripping tyne refusal with a 20 ton excavator was generally reached in moderately to

slightly weathered, medium to very high strength dacite and andesite. The usually well developed and relatively smooth foliations within the rock appeared to facilitate ripping to some degree. In mudstone tyne refusal was reached in moderately weathered, medium to high strength rock. The smoother surfaces, resulting from finer grain-size, and seemingly less well developed defects likely led to an earlier refusal.

Depth to bucket refusal and to tyne refusal are presented in *Table 4.1*.

**Table 4.1: Depth to Bucket and Ripping Tyne Refusal**

Test Pit Number	Bucket Refusal (m)	Tyne Refusal (m)	Test Pit Number	Bucket Refusal (m)	Tyne Refusal (m)
TP1	>6.0 <sup>#</sup>	n/a	TP7	3.0	4.5
TP2	3.8	>5.9 <sup>##</sup>	TP8	>6.0 <sup>#</sup>	n/a
TP3	0.65	0.9	TP9	3.3	5.0
TP3B <sup>#</sup>	1.5	1.6	TP10	>6.0 <sup>#</sup>	n/a
TP4	2.2	>5.5	TP11	>6.0 <sup>#</sup>	n/a
TP5	>6.0 <sup>#</sup>	n/a	TP12	3.4	3.8
TP6	>6.0 <sup>#</sup>	n/a	TP13	4.7	4.9

<sup>#</sup> no bucket refusal was encountered to maximum reach of 6.0m.

<sup>##</sup> no ripping tyne refusal was encountered to maximum reach of 5.9m.

**Bold:** Test pits along dragway alignment

## 4.2 Groundwater

Free groundwater was encountered in four test pits. Depth to observed inflow of groundwater rather than standing water levels were measured, as test pits could not be left open due to livestock being present on site.

Groundwater inflow was observed in four test pits (TP1, TP2 (minor), TP6 and TP8). In test pit TP1 groundwater inflow was observed at 3.0m. In test pit TP2, a small amount of water inflow was observed within the highly weathered dacite rockhead. Massive water inflow was observed within a sandy gravel layer in test pit TP6 at 2.3m. Strong water inflow was observed in test pit TP8 observed within a sandy gravel layer at 3.1m. The measured levels are as shown in *Table 4.2*.

**Table 4.2: Groundwater Inflow Levels**

Test Pit	Groundwater Inflow Level (m)	Lithology
TP1	3.0	Residual Soil – Gravelly Sandy Clay
TP2	4.1 (small amount)	Bedrock – Highly weathered dacite
TP6	2.3	Alluvial – Sandy Gravel
TP8	3.1	Alluvial – Sandy Gravel

## 4.3 Laboratory Testing Results - Soils

### 4.3.1 CBR and Standard Compaction Test

Standard compaction and 4-days soaked CBR tests were carried out on four disturbed samples. Two samples came from test pits along the dragway alignment (TP8 and TP11), and two (TP2 and TP10) from proposed parking lot areas. Test results are summarised in *Table 4.3*.

**Table 4.3: CBR and Standard Compaction Test**

Test Pit	Sample Depth (m)	Soil Description	Field Moisture Content (%)	MDD <sup>1</sup> (t/m <sup>3</sup> )	OMC <sup>2</sup> (%)	Soaked CBR (%)
TP8	0.4-1.0	Alluvial – Sandy Clay	12.4	1.69	16.5	1.5
TP11	0.3-0.7	Alluvial – Gravelly Sandy Clay	8.1	1.82	13.0	9.0
TP2	0.5-1.0	Alluvial – Clay	19.0	1.52	24.5	7.0
TP10	0.4-0.8	Alluvial – Sandy Clay	12.8	1.74	16.5	4.0

<sup>1</sup>MDD – Maximum Dry Density

<sup>2</sup>OMC – Optimum Moisture Content

One sample from along the dragway (TP8 0.4-1.0m) returned a CBR value of less than 2%, while all other samples have CBR of 4-9%.

### 4.3.2 Atterberg Limits and Moisture Content

Four soil samples were tested for Atterberg limits and moisture content. The test results are summarised in *Table 4.4* and the plasticity chart based on the test results is presented in *Figure 4*.

**Table 4.4: Atterberg Limits and Moisture Content test**

Test Pit	Sample Depth (m)	Soil Description	Liquid Limit (%)	Plastic Limit (%)	Plastic Index (%)	Linear Shrinkage (%)	Moisture Content (%)
TP5	0.8-1.2	Alluvial – Sandy Clay	62	25	37	15	17.3
TP7	1.8-2.2	Residual – Sandy Clay	33	17	16	10	13.9
TP12	0.8-1.2	Residual – Silty Clay	38	18	20	12	12.7
TP13	0.5-0.8	Alluvial – Sandy Clay	32	16	16	10	11.1

The test results indicate the tested residual clays are of low to medium plasticity. The alluvial samples' test results indicate clays of low to medium to high plasticity. The plasticity chart indicates that the soils are likely to have low to high degree of expansion and swelling potential.

### 4.3.3 Emerson Crumb Test

Emerson Crumb tests were carried out on four soil samples. The test results are summarised in *Table 4.4*. The tests indicate that the clay samples are marginally dispersive (*Fell, MacGregor, Stapleton and Bell, 2005*).

**Table 4.4: Emerson Crumb Test**

Test Pit	Sample Depth (m)	Soil Description	Dispersion Class	Dispersivity
TP6	1.0-1.2	Alluvial – Silty Clay	5	Marginally dispersive
TP9	0.5-0.8	Alluvial – Sandy Clay	5	Marginally dispersive
TP10	0.4-0.8	Alluvial/(FIII?) – Sandy Clay	5	Marginally dispersive
TP12	0.8-1.2	Residual – Silty Clay	5	Marginally dispersive

## 4.4 Rock Testing

### 4.4.1 Point Load Strength Index Testing

Irregular Lump Point Load Strength Index Testing was conducted on the ripped rock samples from test pits. Results of the Point Load Strength Index Test results are summarised on the test pit logs in *Appendix A*. The Point Load Strength Index test results sheets are presented in *Appendix C*.

### 4.4.2 Acid Sulfate Rock Testing

Results of acid producing potential of rocks encountered on site are summarised in *Table 4.5*. The laboratory test report are attached in *Appendix B*.

**Table 4.5: Acid Sulfate Rock**

Test Pit	Depth Range (m)	Rock Type <sup>(1)</sup>	Acid Sulfate Test <sup>(2)</sup>						Interpretation <sup>(3)</sup>
			s-Net Acidity	a-Net Acidity	TOS	ANC	NAPP	NAG	
			%w/w S	Moles H <sup>+</sup> /tonne	w/w %	CaCO <sub>3</sub> %	H <sub>2</sub> SO <sub>4</sub> kg/tonne	H <sub>2</sub> SO <sub>4</sub> kg/tonne	
TP3	0.8m	Dacite (SW)	<0.02	<10	<0.01	7	-7	<0.5	NAF
TP4	5.2m	Dacite (MW)	<0.02	<10	0.09	15	-12	<0.5	NAF
TP9	4.5m	Mud-stone	<0.02	<10	1.05	280	-250	<0.5	NAF
TP13	4.8m	Andesite	<0.02	<10	0.87	33.0	-6.0	2.5	NAF

1. SW = slightly weathered, MW=moderately weathered
2. TOS = Total Oxidisable Sulfate, ANC = Acid Neutralising Capacity, NAPP = Net Acid Producing Potential, NAG = Net Acid Generation
3. NAF = Non Acid forming

## 5. Comments and Recommendations

On the basis of our experience and the subsurface conditions as revealed by investigations, comments and recommendations associated with the construction of the proposed development are discussed in this section.

### 5.1 Excavation Conditions

Test pits TP13 and TP11 are located in a planned cut area from approximate chainage 75 to 390 and test pits TP4, TP3/TP3B and TP1 in a planned cut from approximate chainage 800 to 1295 (*Figure 6*). Besides cuts to accommodate the dragway alignment, trenching for underground cables and provision for stormwater, water and sewerage are likely to be required.

There are numerous factors influencing productivity when ripping rock, including the unconfined compressive strength of the rock, the degree of weathering, seismic velocity, joint roughness and strength, spacing of joints and bedding in the unripped rock, and bulldozer mass (MacGregor et al., 1994). Furthermore all methods used to estimate rock rippability are considered to be approximate. The earthworks contractor's expectations and capabilities also play a role.

Test pits TP3 / TP3B on the hill near the maximum elevation of the proposed cut from chainage 800 to 1050 (approximately) revealed shallow tyne refusal at 0.8-1.5m in very high strength dacite. Based on this finding the rock on the hill top and slope, from approximate chainage 875 to 1010, can be expected to reach very high strength at a similarly shallow level. Conventional earthmoving plant, or hydraulic hammer, is not expected to be able to excavate to much greater depths. Our preliminary assessment using qualitative methods that consider rock strength, rock weathering and defects (after Pettifer and Fookes, 1994) suggests that the very high strength dacite may be rippable by a large bulldozer with a heavy ripper (D9 or heavier) to some depth greater than 1.0 – 1.5m, especially given the generally well developed two sets of foliations (*see Figure 5*). The additional use of a hydraulic hammer is likely beneficial.

However, experience by local contractors with similar rock ('bluerock') in other parts of Canberra and anecdotal evidence suggests pre-blasting may be a viable option to achieve more effective excavation to the proposed excavation depths of 3m or more. That is, paddock blasting on the relatively remote site, with subsequent deployment of possibly smaller ripping equipment may be the preferred option of a hired contractor to achieve greater productivity. We recommend that the test pit and rippability information be provided to the excavation contractor(s) to ensure that the most appropriate excavation method is utilised. Blasting operations will be required to comply with the relevant Australian Standard (AS2187.2-2006) and statutory requirements. Trimming of faces should be achievable by means of large excavators with hydraulic rock breakers.

Conditions along the foot of the hill may be represented by test pit TP4 (*compare Figure 5*). Here bucket refusal with the 20 ton excavator was reached at 2.2 m. No ripping tyne refusal was reached within highly weathered dacite to 5.5m. In similar conditions, planned excavations of 0.5 to 1.0m should be achievable with conventional excavating

machinery. However, volcanic flows can be irregular and conditions encountered in test pit TP 4 may not be generally representative of foothill conditions.

The excavation of alluvial and residual soils was easy with the ZX200 20 ton excavator. We expect that these soils can be excavated with conventional earthmoving plant with little difficulty.

## 5.2 Batter Stability

Temporary excavation batters in the alluvial soils, residual clay and low to medium strength bedrock should be graded no steeper than 1Horizontal (H) to 1Vertical (V) where excavation depths are less than 1m. For excavation deeper than 1m, the maximum temporary excavation batters in Table 5 should be adopted. Localised flattening of slopes may be required if excessive seepage occur from the batters.

Permanent excavation batters should be graded no steeper than 2H: 1V and protected from erosion by drainage and topsoiling. At locations where low to medium strength bedrock is encountered, a 1.5H: 1V batter is possible. If steeper batters are required they should be supported by engineer-designed retaining walls. Due to the presence of two well developed joint sets within the dacite, a 1.5H: 1V batter is also recommended for stronger rock. Additional safety measures such as shotcreting would be recommended for any cut faces immediately adjacent to the dragway alignment.

In gravel temporary batters would require shoring or a batter of 2H: 1V. Permanent excavations require a batter of no more than 2H:1V and application of topsoil vegetation. Adequate drainage needs to be provided to avoid erosion.

Table 5.1 provides a summary for temporary and permanent batters excavation that are deeper than 1m.

**Table 5.1: Maximum Temporary and Permanent Excavation Batters**

Materials	Max. Temporary Excavation Batter	Max. Permanent Excavation Batter
Topsoil, existing fill material	2Horizontal (H) : 1 Vertical (V)	2Horizontal (H) : 1 Vertical (V)
Soil, firm to stiff silty clay	1.5H : 1V	2H : 1V
Soil very stiff to hard silty clay and extremely low strength rock	1.5H : 1V	2H : 1V
Soil, loose to dense gravel	2H : 1V	2H : 1V
Low to medium strength rock	1H : 1V	1.5H : 1V
Medium to very high strength rock	1H : 1V	1.5H : 1V <sup>#</sup>

<sup>#</sup>: recommended due to the presence of two sets of well developed foliations. Additional protection such as shotcreting may be required adjacent to dragway.

The Emerson Crumb test results indicated the residual and alluvial sandy clays with low to high plasticity, are marginally dispersive. Therefore it is considered that piping failure within the soil and underneath foundations is unlikely to occur.

### **5.3 Potential Acid Production of Rock**

The testing for acid producing potential of the rocks (POCAS and NAPP/NAG) showed that the tested samples were non acid forming. However, the volcanic nature of the rock may mean variable rock chemistry across the site which has not been picked up by the limited number of samples analysed. It is recommended to analyse (NAPP/NAG) excavated rock prior to re-use or construction work taking place to ensure no adverse effects at a later stage.

### **5.4 Suitability of excavated materials for engineered fill**

Cut and fill operations (up to approximately 3m depth / height) are expected to take place. Stripped topsoil due to its generally silty nature is not recommended to be reused in areas other than landscaping areas. Stockpiling of this material is likely susceptible to wind and water erosion. Generation of dust is likely an issue in dry weather and comprehensive dust control measures would be required in these conditions. Any stockpiling in general should be well away from surface drainage areas, and silt trap should be used in accordance with standard practices and applicable regulations.

The alluvial and residual silty, gravelly and sandy clays encountered on site can be re-used in the construction of fill embankment as specified below in section 5.6.2 'Fill Areas / Consolidation'.

The alluvial gravels may be reused but removal of occasional oversize rock-lumps may be required to satisfy the grading requirements as specified in AS 3798-1996. The standard specifies that the maximum particle size of any rock or other lumps within a layer, after compaction, should not exceed two-thirds of the compacted layer thickness.

The quality of the excavated rock for embankment construction will depend on the extent of weathering. Fresh to slightly/moderately weathered dacite may be reused after crushing as select or general fill material to AS 3798-1996. However, care should be taken during excavation as to the mineralogical composition of the rock. Presence of the mineral pyrite should be noted, and, if observed, excavated material should be tested for acid producing potential to avoid any adverse chemical reactions at a later stage. Extremely to highly weathered rock is likely to be readily broken down by construction plant and should provide a fair quality fill.

### **5.5 Groundwater Conditions**

Groundwater occurrences indicate variable conditions. Generally large quantities of groundwater were only encountered near the drainage line in the south-eastern corner of the site. Groundwater inflow levels were observed at depths greater than 3m.

However it has to be considered that there have been extreme drought conditions for some time. Groundwater levels within the alluvial gravels may rise to within 1.5m of the ground surface. This has to be taken into account when designing foundations and drainage.

## **5.6 Foundation Conditions**

For the dragway alignment a design sub-grade CBR of 2% is recommended. Based on the preliminary nature of the design at this stage and the variability in CBR results, it is recommended to undertake further testing once the dragway design has been finalised. Shrink-swell testing would also be justified and further advice should be sought once the design has been finalised.

### **5.6.1 Cut areas**

Bearing capacities of >700kPa can be expected for the highly weathered to fresh dacite-rock on the hill (approximate chainage 850 to 1050). The widespread very stiff to hard alluvial and residual clays can be expected to have a bearing capacity of approximately 200kPa. Between chainage 100 and 390 stiff clays have been observed which have a bearing capacity of approximately 100kPa. A summary of bearing capacities for cohesive soils and rock is given in *Table 5.2*.

The bearing capacity of gravel for various footings can be calculated as required using standard bearing capacity equations, such as Terzaghi and using a friction angle of 35 degrees.

During site preparation any soft, wet or heaving spots should be excavated and backfilled with engineered fill.

**Table 5.2: Allowable (serviceability) Bearing Capacity for Shallow Foundations**

Founding Stratum	Allowable (serviceability) Bearing Pressures <sup>(1)</sup> (kPa)
Engineered fill	100
Residual Soil, firm to stiff silty clay	100
Residual Soil, very stiff to hard silty clay and extremely low strength bedrock	200
Low to medium strength Bedrock	700
Medium strength Bedrock	1500

<sup>(1)</sup> Assumes a minimum embedment of 0.5m for shallow foundation

### 5.6.2 Fill areas / Consolidation

In test pits TP 8 and TP9 (Ch 390 to 800 and within the proposed fill area) hard sandy clays of medium to high plasticity were encountered to depths of 1.5 -1.7m. These were underlain by sandy gravel. After stripping of topsoil to depths up to 0.4m fill up to approximately 2.5m is proposed in the preliminary design. With the relatively thin layers of clay which are often underlain by well drained layers any consolidation within the clays is expected to be relatively fast and unlikely to affect the long-term performance of the dragway pavement.

For subgrade compaction in general it is recommended that the exposed natural material, or the subgrade, should be proof rolled prior to placement of fill. Based on the site investigation, we recommend that exposed subgrade should be proof rolled using a 12 tonne smooth drum roller with at least 5 passes on the subgrade surface. Any soft or heaving areas observed should be excavated and replaced with a clean well graded material, such as crushed sandstone, and re-compacted.

Fill material excavated during the stripping works, with any organic and unsuitable materials removed, can be re-used. The fill material should be compacted to at least 98% of Standard Maximum Dry Density (SMDD) and within  $\pm 2\%$  of Standard Optimum Moisture Content (SOMC). In areas where fill is not required to provide structural support, the SMDD required can be reduced to 95% SMDD. Compaction and moisture content tests indicated that at present the natural moisture content is generally below the SOMC.

Frequent density tests should be completed to check that the compaction specification is achieved.

If site levels are to be raised by placement of imported fill, the fill material should be clean, well graded material of similar or preferably better quality to the in-situ soils, such as ripped or crushed sandstone. All compaction methods and the choice and placement of fill material should comply with AS3798-1996.

Long term or creep consolidation within engineered fill can be expected to occur at a rate of 0.2% of fill height over one log-cycle of time, e.g. with 3m fill a creep settlement of 6mm can be expected over each log-cycle of time.

### 5.6.3 Box Culvert over Drainage Line

Test pit TP 9, which was positioned nearest to the drainage line, encountered hard sandy clay of medium to high plasticity over sandy gravel at 1.5m. No groundwater was encountered, however, given the proximity to the drainage line, the gravel can be expected to be saturated under wet conditions.

### 5.6.4 Proposed Buildings / Lighting

Should any buildings be erected at the site, these are expected to be lightweight and could therefore be supported by shallow foundations.

Lighting may require deep foundations, depending on design specifications. For allowable bearing capacities of shallow foundations refer to Table 5.2 above. Deep foundations such as bored or screw piles may be used for heavily loaded structures. These foundation options are also suitable for structures that are subjected to uplift. For piles to be founded in bedrock, the socket depth should be a minimum of 500mm. Soil parameters for footing design are provided in *Table 5.3*. *Table 5.4* provides the estimated end bearing capacity and shaft adhesion for deep foundation design.

**Table 5.3: Soil Parameters for Piles and Footings Design**

Stratum	Bulk Density (kN/m <sup>3</sup> )	C <sub>u</sub> (kPa)	c' (kPa)	ϕ (deg)	v'
Silty Clay, firm to stiff	18	35	0	20	0.35
Silty Clay, very stiff to hard	18	100	5	25	0.35
Bedrock, extremely low to low strength	20	NA	100	35	0.30
Bedrock, low to medium strength	22	NA	200	40	0.30

#### **Legend**

C<sub>u</sub> – Undrained Shear Strength

v' – Poisson's ratio

NA - Not applicable

ϕ' – friction angle

c' – Cohesion drained

**Table 5.4: End Bearing Capacity for Deep Foundation Design**

Founding Stratum	Ultimate Bearing Pressures <sup>(1)</sup> (kPa)	Allowable Bearing Pressures <sup>(2)</sup> (kPa)	Allowable Shaft Adhesion (kPa)
Engineered fill	300	100	10
Residual Soil, firm to stiff silty clay	300	100	10
Residual Soil, very stiff to hard silty clay and extremely low strength bedrock	800	400	40
Low to medium strength Bedrock	1500	700	70
Medium strength Bedrock	3500	1500	150

<sup>(1)</sup> Assume depth of pile is  $\geq 4 \times$  Pile diameter. The ultimate values occur at settlement  $\geq 5\%$  of pile dimension.

<sup>(2)</sup> The allowable end bearing pressure to cause settlement  $\leq 1\%$  of pile dimension

<sup>(3)</sup> The shaft adhesion values assume that the pier shaft is cleaned of smear and have a surface roughness of at least R2 (see Explanatory notes on bored piles in Appendix A).

Based on findings from the site investigation, deep foundation excavation within thick alluvial soils should be achievable using conventional piling equipment. The recommended serviceability end bearing pressures assume that a pier base is essentially clean of spoil. Hence, footings should be excavated, cleaned out and poured with minimum delay. If footing excavations are to be left opened for an extended period of time, a concrete blinding layer should be provided to protect the foundation material. Should any uncompacted fill or locally deep topsoil be encountered during footing excavation, these materials should be penetrated and the footing founded in suitable materials.

Lighting foundations within high strength bedrock are expected to be shallow and may be achievable by conventional earthmoving plant such as excavator with pneumatic hammer.

The founding conditions should be verified by an engineering geologist/ geotechnical engineer prior to pouring concrete.



## **5.7 Drainage Maintenance**

Adequate site drainage should be installed to prevent ponding of surface water adjacent to structures and embankment. Surface flows should be directed away from foundation soils and discharged a suitable distance from the structure. Any roof run off should also be collected and discharged at a remote location.

Subsoil drains from any retaining walls should be connected to the drainage system. Surface dish drains should be provided at the crest of all cut or fill batters and retaining walls.



## 6. Limitations

This report should be read in conjunction with the “Limitations of Geotechnical regarding geotechnical investigation and assessment (*Appendix D*). Significant variation in subsurface conditions from those anticipated should be reported to this firm for reassessment.

## 7. References

- Australian standard AS3798-1996: Guidelines on Earthworks for commercial and Residential Developments.
- Australian Standard AS2159-1995: Piling – Design and Installation.
- Australian Standard AS3600-2001: Concrete Structures.
- Australian Standard AS2187.2-2006: Explosives – Storage and use – Use of explosives.
- Fell, MacGregor, Stapledon and Bell, 2005, Geotechnical Engineering Of Dams, Taylor & Francis Group plc, London, UK.
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- Henderson, GAM, Geology of Canberra, Queanbeyan and Environs 1:50,000 Geological Map, Bureau of Mineral Resources, Canberra, 1980
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