Appendix E – Maritime Lake Wall Assessment

Technical Note

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Project title	City to the Lake West Basin	Job number	240073
cc	Nathan Greig (LDA), Ian Wood-Bradley (LDA)	File reference	MAR-MEMO-001
Prepared by	Sam Isaacs	Date	20 February 2015
Subject	Condition of the existing West Basin lake wall structure and integration within the Stage 1A works		

1 Introduction

1.1 General

The City to the Lake Stage 1A works involve the construction of an area of land reclamation and a boardwalk structure within Lake Burley Griffin West Basin to create new public realm areas and enhance Canberra’s connection with the lake.

There is an approximately 300m length of existing mass gravity lake wall along the eastern end of the West Basin lake edge within the footprint of the Stage 1A works - the wall continues around under Commonwealth Bridge. Based on limited historic information the wall is assumed to have been constructed in the 1960’s.

The existing lake wall is expected to affect and be effected by the City to the Lake West Basin scheme arrangement as follows:

- The wall will ideally remain in place as a retaining structure for the design life of the scheme (assumed 50 years). To meet the proposed finished boardwalk and landscape level requirements it may be necessary to locally lower the height of the wall and retained fill behind the wall along certain sections of the wall alignment;
- There is the potential to use the existing lake wall as a structural support to the boardwalk structure where this intersects the lake wall.
- The wall (particularly the toe) could be a constraint on piling for the boardwalk which these interface.

This Technical Note discusses the opportunities and risks associated with the wall.

1.2 Location

Figure 1 shows the location and extent of the existing lake wall that is affected by the project and has been included within the scope of this investigation.

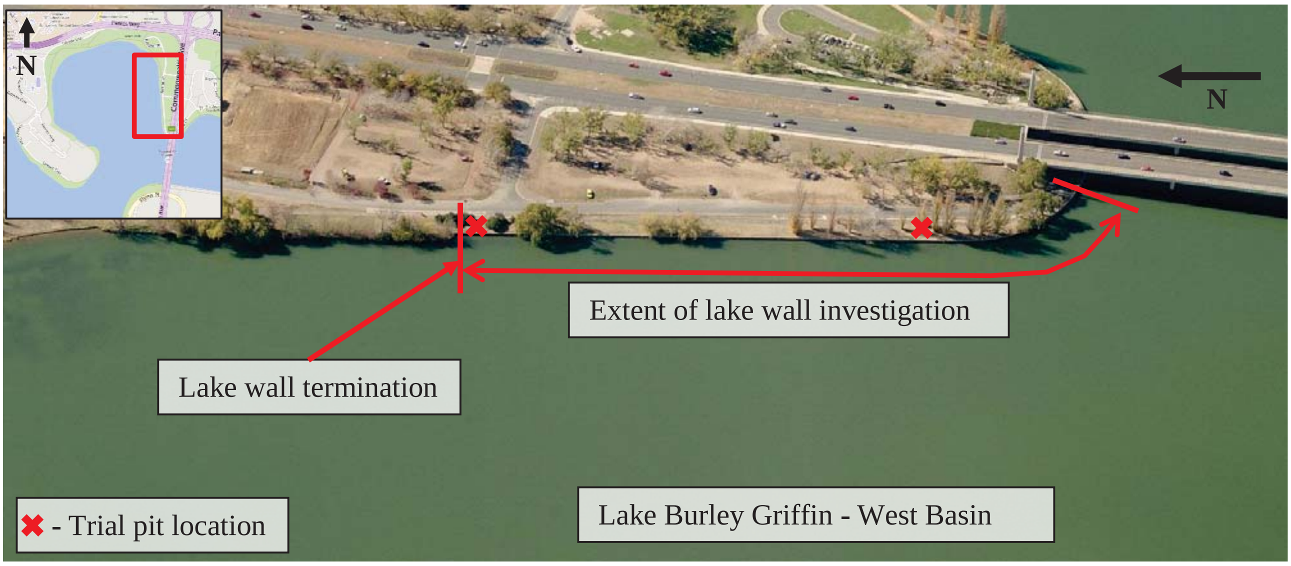


Figure 1 – Locality plan & extent of lake wall investigation plan and trial pit location

2 Available Historical Information

The NCA provided Arup with historic drawings relating to the different lake wall construction types adopted along Lake Burley Griffin, including West Basin. This included a rock lake wall type that is closest in type to the existing lake wall structure under investigation and has used as the basis for this investigation.

An extract from a 1964 drawing shows a typical section through this wall type and is reproduced as Figure 2.

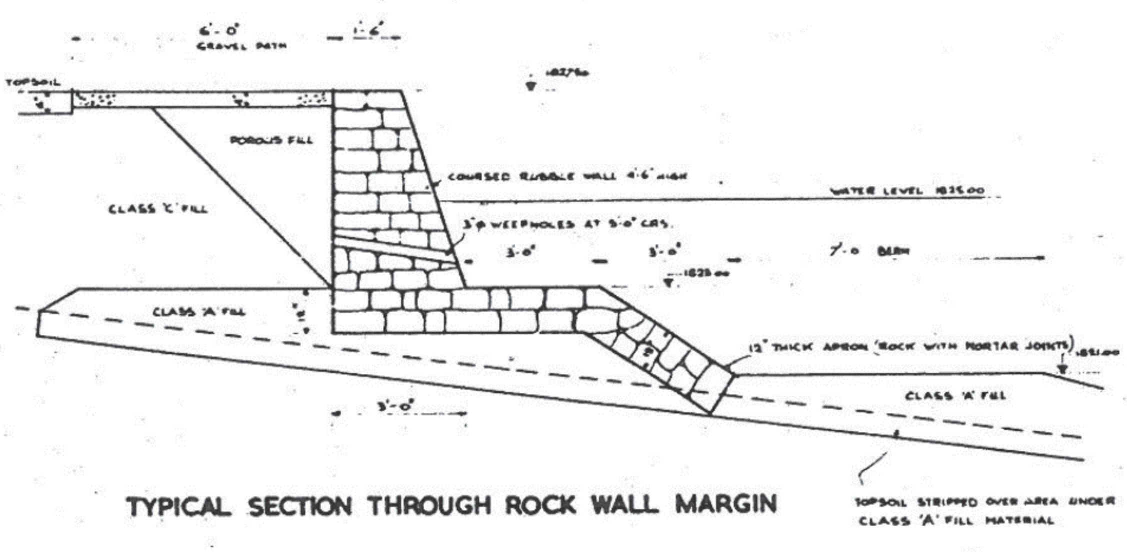


Figure 2 - Historic drawing of typical rock lake wall (provided by the NCA)

3 Site Investigation

3.1 General

To supplement the desk study, a site investigation was undertaken on 5th February 2015 comprising two trial pits located immediately behind the lake wall undertaken by Douglas Partners and a visual inspection undertaken by Arup.

Douglas Partners produced a factual report following the on-site investigation which is included within **Appendix A**.

3.2 Trial Pit Investigation

3.2.1 Locations and Purpose

Douglas Partners undertook two trial pits behind the existing lake wall at locations shown on Figure 3. The purpose of the trial pits was to identify the following:

- Confirmation of historic drawing wall cross section geometry; and
- Geotechnical conditions behind the wall.

Trial Pit 1 **Trial Pit 2**

Figure 3 - Trial pit location plan

3.2.2 Results

The results of the trial pit are broadly summarised in the cross sections drafted by Douglas Partners for each trial pit and reproduced as Figure 5.

The trial pit investigation identified the existing lake wall front face as angled 73 to 75 degrees from the horizontal, with an assumed vertical rear face. No steps or other footings to the rear face of the wall were identified during the trial pit and the base of the wall was not reached. The crest width of the wall was measured and varied between 0.45m and 0.5m across the two trial pits.

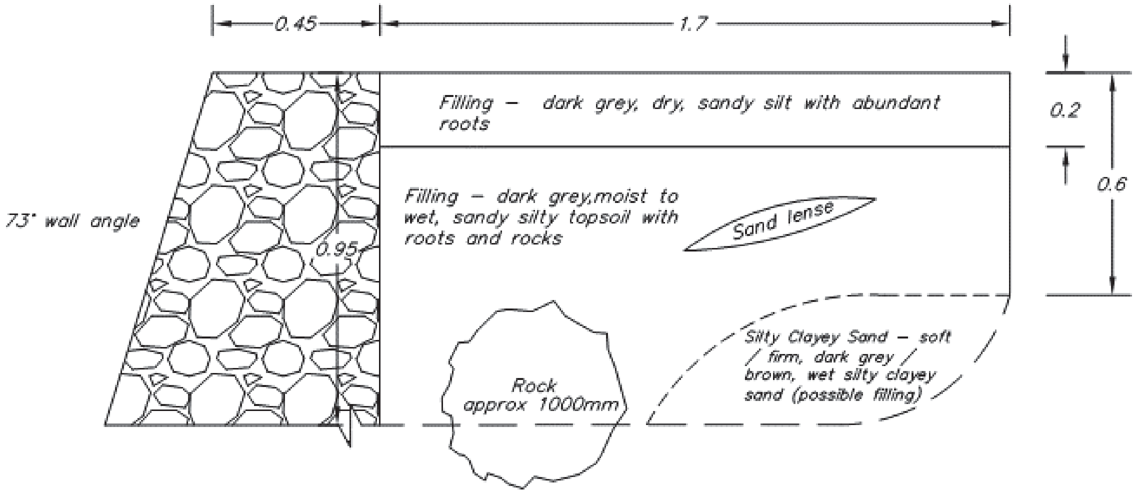
Groundwater was encountered in both test pits as soon as the pit depth reached the approximate lake level. No weep holes were identified in the trial pit, however groundwater freely entered the pit.

The test pits did not encounter any porous back fill material behind the wall as identified in the historic drawings, with the exception of a few large rocks. There was a considerable difference in the insitu strength and moisture content between the two test pits. The insitu strength of the soil at Pit 1 was firm to stiff whilst at Pit 2 it was of hard consistency.

As shown in Figure 4 trial pit two revealed an extensive root network and a large void under the wall.

Figure 4 - Root network and void within Trial Pit 2

Trial Pit 1



Trial Pit 2

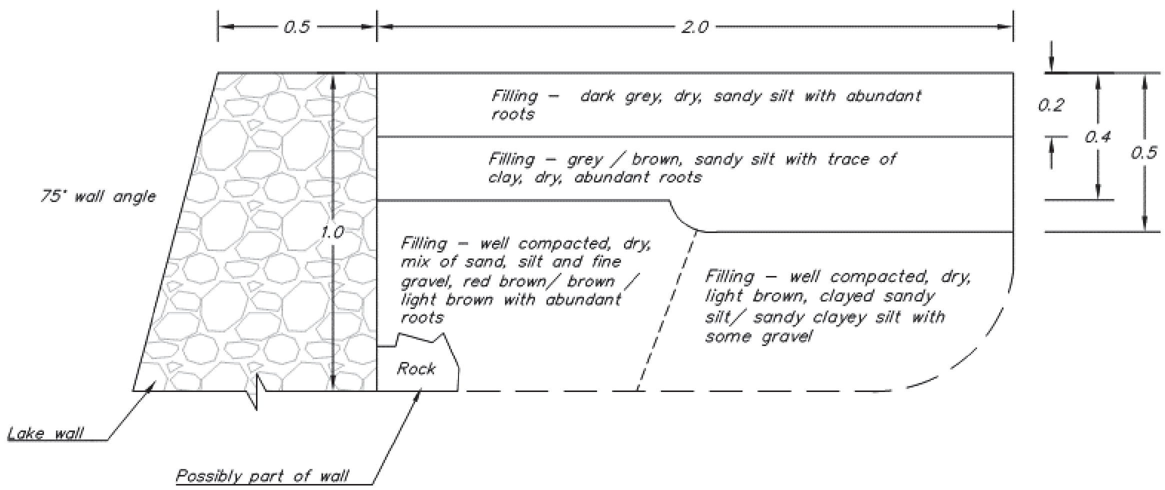


Figure 5 - Test Pit Cross Sections (source: Douglas Partners)

3.3 Visual Inspection

Arup undertook a visual inspection of the existing lake wall during a site walkover. The scope of this inspection included the following:

- Identify the wall construction;
- A high level assessment of the visual condition of the wall from land;
- Measure the freeboard of the wall (the distance from the lake level to the crest of the wall).

3.3.1 Results

As indicated on the historical drawings, the existing lake wall appears to consist of rock blocks bonded together with concrete. There appears to be a toe structure at the base of the wall which may consist of further concrete bonded rock, or plain mass concrete. The toe structure could was not readily visible during the site visit but could be identified by feeling the ground profile of the lake bed and base of the wall with a stick.

The freeboard of the wall was measured at several locations along the length of the wall between Commonwealth Bridge and the termination of the wall within West Basin. The freeboard was found to vary between 0.8m to 1.0m from the Commonwealth Bridge to the terminal respectively.

In many locations along the wall tree roots from flora growing on the landside of the wall penetrate through the wall, displacing the original wall materials (shown in Figure 6).

Aside from localised damage resulting from tree root growth the wall appears to be in a generally fair condition with minor concrete cracking and no visible displacement or distortion of the wall identified.

Figure 6 - Examples of root growth through the lake wall

4 Interpretation of Site Investigation

4.1 Remaining life for the design life

The wall is intended to remain on its existing alignment as a retaining soil structure within the Stage 1A scheme. It may be necessary in some locations to locally cut back the wall to a lower level to allow the boardwalk to straddle the wall.

The nature of the wall construction is such that local cut backs to the wall should not adversely affect the wall's overall integrity or durability protection as there is no identified internal steel reinforcement.

Aside from local damage resulting from tree roots penetrating through the lake wall it is in a reasonable condition. Based on an original construction date in the 1960's the wall is likely to be around 50 years old. It is unlikely the wall was originally designed with an intended functional design life, however 50 years would typically be adopted now for new structures of this type.

Whilst the wall is approaching the end of a notional 50 year design life the wall condition is such that it should continue to meet its functional requirements beyond this time frame. Without undertaking more detailed investigation and materials testing it is difficult to identify a precise ongoing service life. However this additional work is not considered necessary as the risk of failure of the wall is considered low, both with respect to the probability and consequence of failure.

Any failure of the lake wall would likely be a slow progressive deterioration of local sections of the wall, manifesting as visible local deformation and loss of retained material. This type of failure is typically slow to eventuate and unlikely to be catastrophic, therefore there is time to plan and implement engineering intervention to remediate damage.

4.2 Potential for New Boardwalk Support

There may be an option to use the existing lake wall to provide structural support to the boardwalk along the length of boardwalk that overlaps with the lake wall as shown in Figure 7.

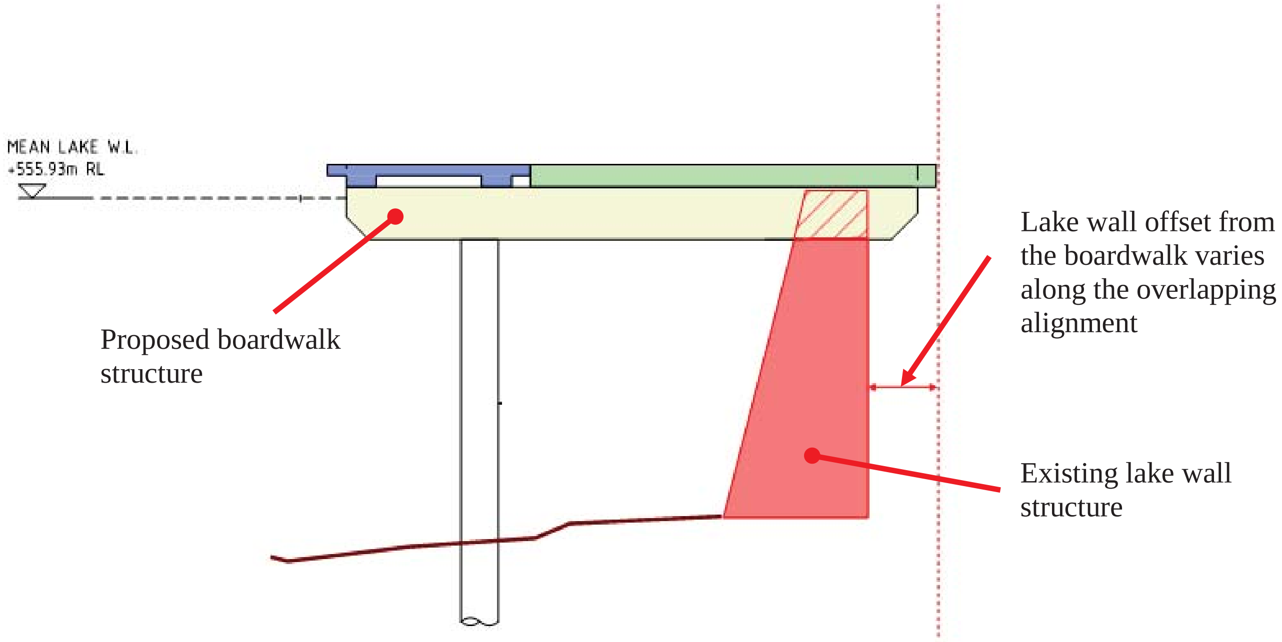


Figure 7 - Boardwalk support over the lake wall structure

The existing lake wall structure may not be suitable to provide structural support to the boardwalk. This is due to both uncertainty around the wall's remaining service life, as discussed in Section 4.1, and the unknown residual structural capacity to resist additional loads from the boardwalk.

Whilst the wall is assumed to have sufficient capacity to continue to retain fill until such a time that may require intervention any structures used to support the new boardwalk structure should have a defined design life equal to or greater than the boardwalk.

Further investigation could be pursued to provide more clarity over the remaining service life and structural capacity of the existing lake wall, however in the likely event that this would either provide inconclusive or negative results we propose to adopt a support system for the boardwalk that is independent to the existing lake wall structures for the concept design. Options for this support could include a piled structure or reinforced concrete footing behind the lake wall.

5 Summary & Recommendations

5.1 Summary

Douglas Partners and Arup undertook a trial pit investigation and visual inspection respectively of the existing lake wall structure in West Basin.

The trial pits revealed the wall construction differed to some extent from the historic records. The material uncovered behind the wall was found to be non-porous with highly varying strengths in place of the homogenous porous drainage material indicated on historic records. The geometry of the wall structure itself was broadly consistent with the drawings, consisting of a sub vertical front face, vertical rear face and a crest width of approx. 0.45 to 0.5m.

A visual inspection identified that the wall is generally in a reasonable condition with no obvious degradation to the structure except from localised damage resulting from the penetration of tree roots through the wall.

The wall is not considered at risk of failure in the short/medium term, however this cannot be conclusively determined without further investigation (likely to include a structural load rating, confirmation of wall toe and footing geometry and wall construction materials properties). Any long term wall failure mechanism will likely be a localised slow progressive translation of the wall and loss of material as opposed to catastrophic collapse. With ongoing monitoring and surveying of the wall the commencement of failure can be identified and remedied before significant damage occurs.

The existing lake wall is not proposed to be used to provide support to the boardwalk structures along the alignment where they intersect. This is due to an uncertainty that the wall has a remaining service life equal to that which would be adopted for the new-build boardwalk and further structural assessment would be required to confirm the wall's residual load carrying capacity is sufficient to resist the additional boardwalk load.

5.2 Recommendations

Arup propose the following recommendations:

- It is not proposed to include within the project scope any remedial works to the existing wall. Instead, we recommend that the wall is left in place (with lowering to accommodate the boardwalk where applicable) and subject to a scheduled monitoring programme (say as a minimum visual inspection every 3-5 years by a structural engineer) to identify potential changes in the condition and/or movement of the wall and necessary future maintenance/replacement over time (this is not unique to the West Basin and applies to many other stretches of wall around the lake originally built about the same time).
- For the Concept Design, it will be assumed that the existing lake wall is not suitable to use as support for the new boardwalk structure. Should the D&C contractor wish to incorporate the wall into their detailed design as structural support to the boardwalk they should be required to undertake further investigation and analysis to the existing lake wall structures – ***to be included as a requirement in the Technical Documentation***;
- The detailed design of the Stage 1A works should make allowances for the on-going safe access for monitoring and potential repairs works to the existing lake wall – ***to be included as a requirement in the Technical Documentation***;

- The D&C contractor should be required to monitor the existing wall during the construction works for damage caused as a result of the works, for which they would be responsible for rectifying – ***to be included as a requirement in the Technical Documentation;***
- Consider the extent of the lake wall footing and whether it presents an obstruction to piling works that may require local rock removal and/or concrete breakout to facilitate piling works – ***to be included on the PSP drawings.***

DOCUMENT CHECKING (not mandatory for File Note)

	Prepared by	Checked by	Approved by
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Signature			

Appendix A

Douglas Partners
Trial Pit Factual Report

Douglas Partners
Geotechnics / Environment / Groundwater

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Project 77417.05
10 February 2015
MJJ

Land Development Agency
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Dickson ACT 2602

Attention: Mr Nathan Greig

Email: nathan.greig@act.gov.au

Dear Sirs

Factual Report on Geotechnical Investigation
Existing Lake Wall
West Basin, Canberra City

1. Introduction

This letter report by Douglas Partners Pty Ltd (DP) summarises the factual results of a geotechnical investigation undertaken behind a section of existing lake wall in West Basin, Canberra City. The investigation was requested by the Land Development Agency, owners of the site.

It is understood that the investigation was requested to assist ARUP with their work on the City to Lake Project.

The investigation was undertaken to determine the dimensions of the existing wall and the backfill materials behind the wall.

The investigation comprised test pit excavation with in-situ testing and sampling of the subsurface strata, followed by factual reporting. Details of the work undertaken are given in the report.

This report must be read in conjunction with the attached notes About This Report.

2. Site Description and Regional Geology

The two test pit locations are shown on attached Drawing 1. They are located towards the south eastern extent of West Basin, close to Commonwealth Avenue Bridge.

Both test pit sites were grassed and located immediately adjacent to the existing lake wall which was constructed of grouted rock. The rock wall was estimated to be around 1 – 1.5 m in height.

The ground surface adjacent to the wall was near-level. Large shrubs and trees were present adjacent to the test pit locations.

Reference to the Central Canberra 1:10 000 Geological Series Sheet (Ref 1) indicates that the site is underlain by fill materials associated with the construction of Commonwealth Avenue Bridge. The geology sheet describes the filling as soil and waste rock. The filling material is mapped as being underlain by sedimentary rock of the Canberra Formation.

3. Field Work Methods

The field investigation comprised the excavation of two test pits (Pits 1 and 2) to depths of 0.95 m and 1.0 m using a Kubota KX057-4 mini excavator fitted with a 450 mm wide bucket working under the direction of a senior geotechnical engineer. The long side of the test pits were orientated perpendicular to the line of the lake wall to minimise the risk of damage to the wall.

Disturbed samples of the soils encountered in the test pits were collected for possible laboratory testing and to assist in strata identification. Dynamic cone penetrometer tests (AS1289 6.3.2) were also undertaken to provide an indication of the strength profile of the site soils at each test location.

4. Field Work Results

Details of the subsurface conditions encountered are summarised as cross sections in Drawings 2 and 3 which are attached. They must be read in conjunction with the attached explanatory notes that define classification methods and terms used to describe the soils.

In summary, the test pits did not encounter any porous back fill material behind the wall with the exception of a few large rocks. There was a considerable difference in the insitu strength and moisture content between the two test pits. The insitu strength of the soil at Pit 1 was firm to stiff whilst at Pit 2 it was of hard consistency.

Groundwater was encountered in both test pits as soon as the pit depth reached the approximate lake level. The test pits were discontinued at this depth due to safety concerns of potential collapsing pit walls.

Photographs of the test pit walls are attached in Photo Plates 1 – 4.

5. References

1. Geology of Central Canberra 1:10 000 Geological Series Sheet 208-600, Bureau of Mineral Resources, (1995).

6. Limitations

Douglas Partners (DP) has prepared this factual report for the work as described within. The report is provided for the exclusive use the Land Development Agency and ARUP for this project only and for the purpose(s) described in the report. It should not be used for other projects or by a third party. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

The results provided in the report are indicative of the sub-surface conditions only at the specific sampling or testing locations, and then only to the depths investigated and at the time the work was carried out. Sub-surface conditions can change abruptly due to variable geological processes and also as a result of anthropogenic influences. Such changes may occur after DP's field testing has been completed.

DP's advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by DP in this report may be limited by undetected variations in ground conditions between sampling locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

This report must be read in conjunction with all of the attached notes 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 given 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.

The contents of this report do not constitute formal design components such as are required, by the Health and Safety Legislation and Regulations, to be included in a Safety Report specifying the hazards likely to be encountered during construction and the controls required to mitigate risk. This design process requires risk assessment to be undertaken, with such assessment being dependent upon factors relating to likelihood of occurrence and consequences of damage to property and to life. This, in turn, requires project data and analysis presently beyond the knowledge and project role respectively of DP.

DP may be able, however, to assist the client in carrying out a risk assessment of potential hazards contained in the Comments section of this report, as an extension to the current scope of works, if so requested, and provided that suitable additional information is made available to DP. Any such risk assessment would, however, be necessarily restricted to the geotechnical components set out in this report and to their application by the project designers to project design, construction, maintenance and demolition.

We trust the above is in accordance with your present requirements. Should you have any questions at this stage please contact the undersigned.

Yours faithfully
Douglas Partners Pty Ltd

for **Anthony Hooper**
Geotechnical Engineer

Reviewed by

Michael Jones
Senior Associate

Attachments:
- About this Report
- Explanatory Notes
- Photo Plates (4 pages)
- Dynamic cone penetrometer report sheet (1 page)
- Drawings 1 – 3

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.

Copyright

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.

Sampling

Sampling is carried out during drilling or test pitting to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing it to obtain a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Test Pits

Test pits are usually excavated with a backhoe or an excavator, allowing close examination of the in-situ soil if it is safe to enter into the pit. The depth of excavation is limited to about 3 m for a backhoe and up to 6 m for a large excavator. A potential disadvantage of this investigation method is the larger area of disturbance to the site.

Large Diameter Augers

Boreholes can be drilled using a rotating plate or short spiral auger, generally 300 mm or larger in diameter commonly mounted on a standard piling rig. The cuttings are returned to the surface at intervals (generally not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube samples.

Continuous Spiral Flight Augers

The borehole is advanced using 90-115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are disturbed and may be mixed with soils from the sides of the hole. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively low reliability, due to the remoulding, possible mixing or softening of samples by groundwater.

Non-core Rotary Drilling

The borehole is advanced using a rotary bit, with water or drilling mud being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from the rate of penetration. Where drilling mud is used this can mask the cuttings and reliable identification is only possible from separate sampling such as SPTs.

Continuous Core Drilling

A continuous core sample can be obtained using a diamond tipped core barrel, usually with a 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in weak rocks and granular soils), this technique provides a very reliable method of investigation.

Standard Penetration Tests

Standard penetration tests (SPT) are used as a means of estimating the density or strength of soils and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, Methods of Testing Soils for Engineering Purposes - Test 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150 mm of, say, 4, 6 and 7 as:

  4,6,7
  N=13

- In the case where the test is discontinued before the full penetration depth, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm as:

  15, 30/40 mm

The results of the SPT tests can be related empirically to the engineering properties of the soils.

Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

Dynamic penetrometer tests (DCP or PSP) are carried out by driving a steel rod into the ground using a standard weight of hammer falling a specified distance. As the rod penetrates the soil the number of blows required to penetrate each successive 150 mm depth are recorded. Normally there is a depth limitation of 1.2 m, but this may be extended in certain conditions by the use of extension rods. Two types of penetrometer are commonly used.

- Perth sand penetrometer - a 16 mm diameter flat ended rod is driven using a 9 kg hammer dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands and is mainly used in granular soils and filling.
- Cone penetrometer - a 16 mm diameter rod with a 20 mm diameter cone end is driven using a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). This test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various road authorities.

Soil Descriptions

Douglas Partners

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard AS 1726, Geotechnical Site Investigations Code. In general, the descriptions include strength or density, colour, structure, soil or rock type and inclusions.

Soil Types

Soil types are described according to the predominant particle size, qualified by the grading of other particles present:

Type	Particle size (mm)
Boulder	>200
Cobble	63 - 200
Gravel	2.36 - 63
Sand	0.075 - 2.36
Silt	0.002 - 0.075
Clay	<0.002

The sand and gravel sizes can be further subdivided as follows:

Type	Particle size (mm)
Coarse gravel	20 - 63
Medium gravel	6 - 20
Fine gravel	2.36 - 6
Coarse sand	0.6 - 2.36
Medium sand	0.2 - 0.6
Fine sand	0.075 - 0.2

The proportions of secondary constituents of soils are described as:

Term	Proportion	Example
And	Specify	Clay (60%) and Sand (40%)
Adjective	20 - 35%	Sandy Clay
Slightly	12 - 20%	Slightly Sandy Clay
With some	5 - 12%	Clay with some sand
With a trace of	0 - 5%	Clay with a trace of sand

Definitions of grading terms used are:

- Well graded - a good representation of all particle sizes
- Poorly graded - an excess or deficiency of particular sizes within the specified range
- Uniformly graded - an excess of a particular particle size
- Gap graded - a deficiency of a particular particle size with the range

Cohesive Soils

Cohesive soils, such as clays, are classified on the basis of undrained shear strength. The strength may be measured by laboratory testing, or estimated by field tests or engineering examination. The strength terms are defined as follows:

Description	Abbreviation	Undrained shear strength (kPa)
Very soft	vs	<12
Soft	s	12 - 25
Firm	f	25 - 50
Stiff	st	50 - 100
Very stiff	vst	100 - 200
Hard	h	>200

Cohesionless Soils

Cohesionless soils, such as clean sands, are classified on the basis of relative density, generally from the results of standard penetration tests (SPT), cone penetration tests (CPT) or dynamic penetrometers (PSP). The relative density terms are given below:

Relative Density	Abbreviation	SPT N value	CPT qc value (MPa)
Very loose	vl	<4	<2
Loose	l	4 - 10	2 -5
Medium dense	md	10 - 30	5 - 15
Dense	d	30 - 50	15 - 25
Very dense	vd	>50	>25

Soil Descriptions

Soil Origin

It is often difficult to accurately determine the origin of a soil. Soils can generally be classified as:

- Residual soil - derived from in-situ weathering of the underlying rock;
- Transported soils - formed somewhere else and transported by nature to the site; or
- Filling - moved by man.

Transported soils may be further subdivided into:

- Alluvium - river deposits
- Lacustrine - lake deposits
- Aeolian - wind deposits
- Littoral - beach deposits
- Estuarine - tidal river deposits
- Talus - scree or coarse colluvium
- Slopewash or Colluvium - transported downslope by gravity assisted by water. Often includes angular rock fragments and boulders.

Rock Strength

Rock strength is defined by the Point Load Strength Index ($Is_{(50)}$) and refers to the strength of the rock substance and not the strength of the overall rock mass, which may be considerably weaker due to defects. The test procedure is described by Australian Standard 4133.4.1 - 1993. The terms used to describe rock strength are as follows:

Term	Abbreviation	Point Load Index $Is_{(50)}$ MPa	Approx Unconfined Compressive Strength MPa*
Extremely low	EL	<0.03	<0.6
Very low	VL	0.03 - 0.1	0.6 - 2
Low	L	0.1 - 0.3	2 - 6
Medium	M	0.3 - 1.0	6 - 20
High	H	1 - 3	20 - 60
Very high	VH	3 - 10	60 - 200
Extremely high	EH	>10	>200

* Assumes a ratio of 20:1 for UCS to $Is_{(50)}$

Degree of Weathering

The degree of weathering of rock is classified as follows:

Term	Abbreviation	Description
Extremely weathered	EW	Rock substance has soil properties, i.e. it can be remoulded and classified as a soil but the texture of the original rock is still evident.
Highly weathered	HW	Limonite staining or bleaching affects whole of rock substance and other signs of decomposition are evident. Porosity and strength may be altered as a result of iron leaching or deposition. Colour and strength of original fresh rock is not recognisable
Moderately weathered	MW	Staining and discolouration of rock substance has taken place
Slightly weathered	SW	Rock substance is slightly discoloured but shows little or no change of strength from fresh rock
Fresh stained	Fs	Rock substance unaffected by weathering but staining visible along defects
Fresh	Fr	No signs of decomposition or staining

Degree of Fracturing

The following classification applies to the spacing of natural fractures in diamond drill cores. It includes bedding plane partings, joints and other defects, but excludes drilling breaks.

Term	Description
Fragmented	Fragments of <20 mm
Highly Fractured	Core lengths of 20-40 mm with some fragments
Fractured	Core lengths of 40-200 mm with some shorter and longer sections
Slightly Fractured	Core lengths of 200-1000 mm with some shorter and loner sections
Unbroken	Core lengths mostly > 1000 mm

Rock Descriptions

Rock Quality Designation

The quality of the cored rock can be measured using the Rock Quality Designation (RQD) index, defined as:

$$\text{RQD \%} = \frac{\text{cumulative length of 'sound' core sections} \geq 100 \text{ mm long}}{\text{total drilled length of section being assessed}}$$

where 'sound' rock is assessed to be rock of low strength or better. The RQD applies only to natural fractures. If the core is broken by drilling or handling (i.e. drilling breaks) then the broken pieces are fitted back together and are not included in the calculation of RQD.

Stratification Spacing

For sedimentary rocks the following terms may be used to describe the spacing of bedding partings:

Term	Separation of Stratification Planes
Thinly laminated	< 6 mm
Laminated	6 mm to 20 mm
Very thinly bedded	20 mm to 60 mm
Thinly bedded	60 mm to 0.2 m
Medium bedded	0.2 m to 0.6 m
Thickly bedded	0.6 m to 2 m
Very thickly bedded	> 2 m

Symbols & Abbreviations

Douglas Partners

Introduction

These notes summarise abbreviations commonly used on borehole logs and test pit reports.

Drilling or Excavation Methods

C	Core Drilling
R	Rotary drilling
SFA	Spiral flight augers
NMLC	Diamond core - 52 mm dia
NQ	Diamond core - 47 mm dia
HQ	Diamond core - 63 mm dia
PQ	Diamond core - 81 mm dia

Water

▷	Water seep
▽	Water level

Sampling and Testing

A	Auger sample
B	Bulk sample
D	Disturbed sample
E	Environmental sample
U_{50}	Undisturbed tube sample (50mm)
W	Water sample
pp	pocket penetrometer (kPa)
PID	Photo ionisation detector
PL	Point load strength Is(50) MPa
S	Standard Penetration Test
V	Shear vane (kPa)

Description of Defects in Rock

The abbreviated descriptions of the defects should be in the following order: Depth, Type, Orientation, Coating, Shape, Roughness and Other. Drilling and handling breaks are not usually included on the logs.

Defect Type

B	Bedding plane
Cs	Clay seam
Cv	Cleavage
Cz	Crushed zone
Ds	Decomposed seam
F	Fault
J	Joint
Lam	lamination
Pt	Parting
Sz	Sheared Zone
V	Vein

Orientation

The inclination of defects is always measured from the perpendicular to the core axis.

h	horizontal
v	vertical
sh	sub-horizontal
sv	sub-vertical

Coating or Infilling Term

cln	clean
co	coating
he	healed
inf	infilled
stn	stained
ti	tight
vn	veneer

Coating Descriptor

ca	calcite
cbs	carbonaceous
cly	clay
fe	iron oxide
mn	manganese
slt	silty

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

po	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

Other

fg	fragmented
bnd	band
qtz	quartz

Symbols & Abbreviations

Graphic Symbols for Soil and Rock

General

- Asphalt
- Road base
- Concrete
- Filling

Soils

- Topsoil
- Peat
- Clay
- Silty clay
- Sandy clay
- Gravelly clay
- Shaly clay
- Silt
- Clayey silt
- Sandy silt
- Sand
- Clayey sand
- Silty sand
- Gravel
- Sandy gravel
- Cobbles, boulders
- Talus

Sedimentary Rocks

- Boulder conglomerate
- Conglomerate
- Conglomeratic sandstone
- Sandstone
- Siltstone
- Laminite
- Mudstone, claystone, shale
- Coal
- Limestone

Metamorphic Rocks

- Slate, phyllite, schist
- Gneiss
- Quartzite

Igneous Rocks

- Granite
- Dolerite, basalt, andesite
- Dacite, epidote
- Tuff, breccia
- Porphyry

Photo 1 – Test Pit 1 location

Photo 2 – View of front of Test Pit 1 adjacent to the lake wall

Douglas Partners Geotechnics / Environment / Groundwater	Site Photographs	PROJECT: 77417.05
	Existing Lake Wall	PLATE No: 1
	West Basin, Canberra City	REV: 0
	CLIENT: LDA	DATE: 8-Feb-15

Photo 3 – View of northern pit wall of Pit 1

Photo 4 – View of southern pit wall of Pit 1

Douglas Partners Geotechnics / Environment / Groundwater	**Site Photographs** **Existing Lake Wall** **West Basin, Canberra City**	PROJECT: 77417.05 PLATE No: 2 REV: 0
	CLIENT: LDA	DATE: 8-Feb-15

Photo 5 – Test Pit 2 location

Photo 6 – View of front of Test Pit 2 adjacent to the lake wall

Douglas Partners Geotechnics / Environment / Groundwater	**Site Photographs**	PROJECT:	77417.05
	Existing Lake Wall	PLATE No:	3
	West Basin, Canberra City	REV:	0
	CLIENT: LDA	DATE:	8-Feb-15

Photo 7 – View of northern pit wall of Pit 2

Photo 8 – View of southern pit wall of Pit 2

Douglas Partners Geotechnics / Environment / Groundwater	**Site Photographs**	PROJECT:	77417.05
	Existing Lake Wall	PLATE No:	4
	West Basin, Canberra City	REV:	0
	CLIENT: LDA	DATE:	8-Feb-15

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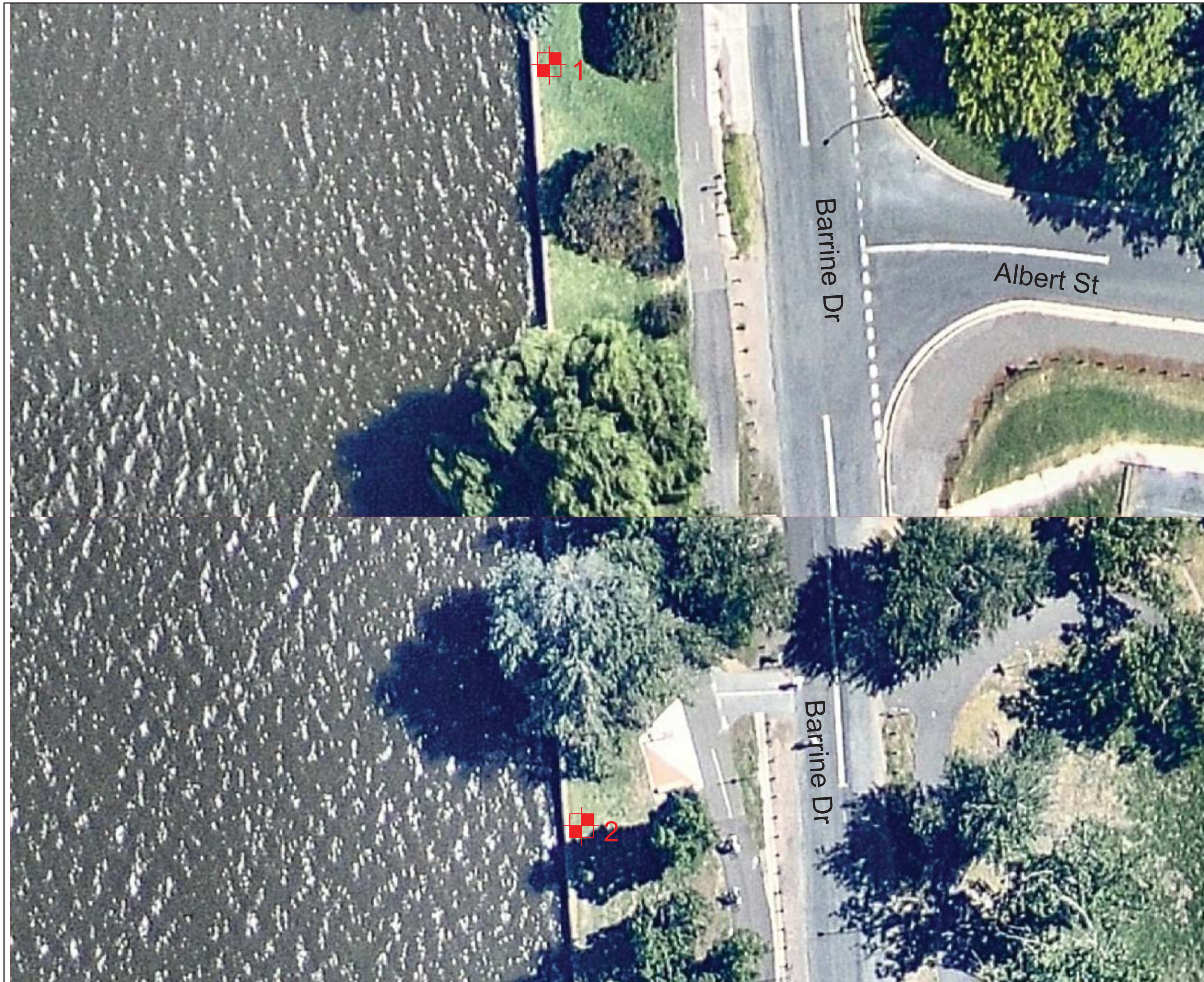
Results of Dynamic Penetrometer Tests

Client	Land Development Agency	**Project No.**	77417.05
Project	Existing Lake Wall	**Date**	05/02/15
Location	West Basin, Canberra City	**Page No.**	1 of 1

Test Location	1	2	3	4	5	6	7	8	9	10
RL of Test (AHD)										
Depth (m)	**Penetration Resistance** Blows/150 mm									
0 - 0.15	4	5								
0.15 - 0.30	4	10								
0.30 - 0.45	4	22								
0.45 - 0.60	3	21								
0.60 - 0.75	3	17								
0.75 - 0.90	5	25								
0.90 - 1.05	4	13								
1.05 - 1.20	6	18								
1.20 - 1.35	10	15								
1.35 - 1.50	13	15								
1.50 - 1.65	7	21								
1.65 - 1.80	7	19								
1.80 - 1.95		15								
1.95 - 2.10										
2.10 - 2.25										
2.25 - 2.40										
2.40 - 2.55										
2.55 - 2.70										
2.70 - 2.85										
2.85 - 3.00										
3.00 - 3.15										
3.15 - 3.30										
3.30 - 3.45										
3.45 - 3.60										

Test Method	AS 1289.6.3.2, Cone Penetrometer	☑	**Tested By**	MJJ
	AS 1289.6.3.3, Flat End Penetrometer	☐	**Checked By**	MJJ

Remarks



Locality Plan

LEGEND

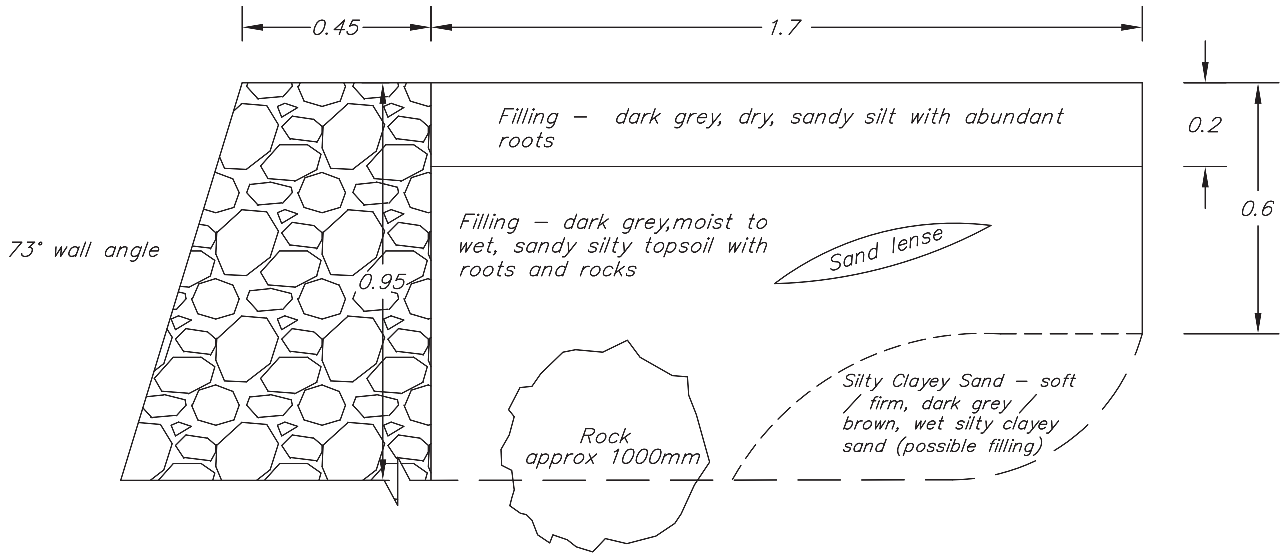
Approximate Site Boundary

Approximate Location of Test Pit

Note: Base Drawing from GoogleMaps

Douglas Partners *Geotechnics / Environment / Groundwater*	CLIENT: Land Development Agency	TITLE: **Test Pit Locality Plan** **Existing Lake Wall** **West Basin, Canberra City**	PROJECT No: 77417.05
	OFFICE: Canberra / DRAWN BY: APH		DRAWING No: 1
	SCALE: NTS / DATE: 09.02.2015		REVISION: 0

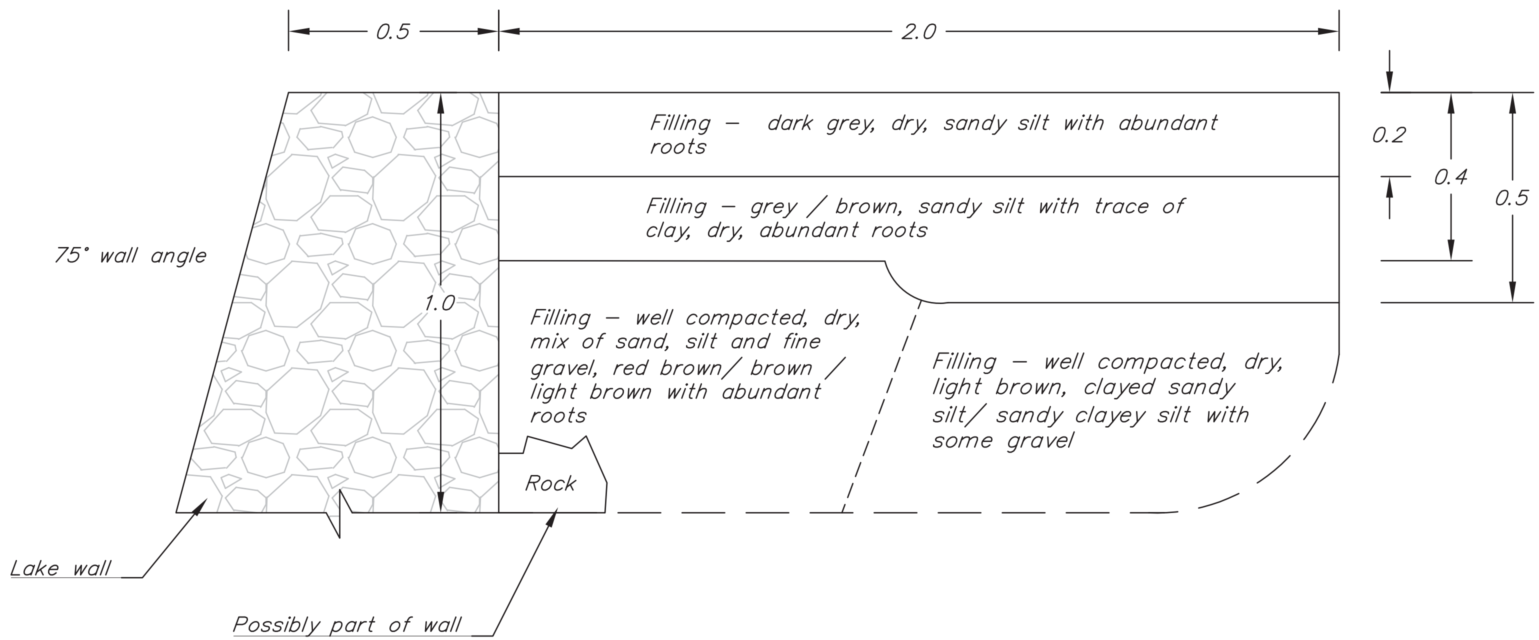
Test Pit 1



Note: All dimensions in meters

Douglas Partners Geotechnics / Environment / Groundwater	CLIENT: Land Development Agency	TITLE: **Cross Section - Test Pit 1**	PROJECT No: 77417.05
	OFFICE: Canberra / DRAWN BY: APH	**Existing Lake Wall**	DRAWING No: 2
	SCALE: NTS / DATE: 09.02.2015	**West Basin, Canberra City**	REVISION: 0

Test Pit 2



Note: All dimensions in meters

Douglas Partners Geotechnics / Environment / Groundwater	CLIENT: Land Development Agency		TITLE: Cross Section - Test Pit 2 Existing Lake Wall West Basin, Canberra City
	OFFICE: Canberra	DRAWN BY: APH	
	SCALE: NTS	DATE: 09.02.2015	

PROJECT No:	77417.05
DRAWING No:	3
REVISION:	0