INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING

The paper was published in the proceedings of the 8th Australia New Zealand Conference on Geomechanics and was edited by Nihal Vitharana and Randal Colman. The conference was held in Hobart, Tasmania, Australia, 15 - 17 February 1999.

Performance of Lake Kabbamup Causeway Founded on Variable Residual Soils

S.P. Masterson
B.Eng., MIEAust.
Senior Engineer, M.P.A. Williams and Associates Pty Ltd, Perth, Western Australia

Summary A geotechnical investigation carried out for the Lake Kabbamup Causeway has revealed a weathered laterite profile with an upper discontinuous ferricrete layer overlying generally stiff clays or silts. However, a metamorphosed zone about 120 m long was encountered in one area (northern tributary) in which the materials were soft to firm clays or silts, with low dry densities and high moisture contents. Laboratory testing demonstrated the presence of halloysites. Triaxial testing showed that the material had quite high peak and residual shear strengths, and consolidation testing indicated low to moderate compressibility. Site monitoring of the magnitude and rate of embankment settlement was close to that predicted from laboratory testing. The soft clays in the northern tributary were about twice as compressible as the stiff clays elsewhere.

1. INTRODUCTION

Alcoa of Australia Limited (Alcoa) has carried out construction of an access from the Orion area to the Arundel facilities as part of a project to mine bauxite ore in the Willowdale North area. The access is presently used for ore haulage traffic and in the future will be used for an overland conveyor.

All potential access routes passed through environmentally sensitive areas. The selected route cuts across the upper reaches of Lake Kabbamup and avoids large cuttings west of Samson Dam, and Lane Poole Reserve east of Samson Brook Reservoir (Lake Kabbamup).

The access route includes a causeway across the upstream end of Lake Kabbamup. The causeway comprises a 600 m long earthfill embankment with a volume of 150,000 m^3 and is up to 10 m high at tributary crossings. Twin culverts were constructed through the embankment at the crossings to balance water levels on either side of the causeway. Sumps were constructed to manage sediment, hydrocarbons and stormwater runoff.

A geotechnical investigation was undertaken along the causeway alignment. The subsurface conditions were deeply weathered rock, with thin discontinuous laterite caprock, overlying residual soils (RS) comprising sandy clays and sandy silts. The RS materials were stiff, except for a zone about 120 m long where materials were soft to firm. The soft materials exhibited low dry densities and high in-situ moisture content. Laboratory testing indicated the presence of mineral halloysite. Triaxial compression strength testing showed that the material was reasonably strong despite the low strength inferred from CPT results.

The selection of parameters for shear strength and permeability of foundation materials was vital to the design. The approach taken in this selection is discussed.

Construction performance is reviewed and compared to that predicted from field and laboratory testing.

2. SITE DESCRIPTION AND GEOLOGY

The regional geology of the area is described by the Geological Survey of WA (1980) on the Pinjarra 1:250,000 Geological Map Sheet and comprises porphyritic granite and laterite including pisolithic gravel and lateritised sand, as well as areas of alluvium and minor colluvium developed on laterite of the Darling Range.

At the time of the investigation (mid-1995) the causeway and borrow areas were partially cleared in the areas around the tributaries of Samson Brook. The confluence of the tributaries was about 150 m downstream from the centreline of the proposed causeway. The site contained numerous large tree stumps, which caused some difficulty in access. A site plan is presented on Figure 1.

3. DESCRIPTION OF FIELDWORK AND LABORATORY TESTING

Fieldwork was carried out in three phases as follows:

Phase 1 12 CPT's to depths from 0.8 m to 18.2 m, and 3 test pits to depths 5.0 m to 5.2 m, in the northern and southern tributary areas.

Phase 2 34 CPT's (20 m × 20 m grid) to depths from 4.5 m to 15 m in the northern tributary where soft materials were identified from Phase 1.

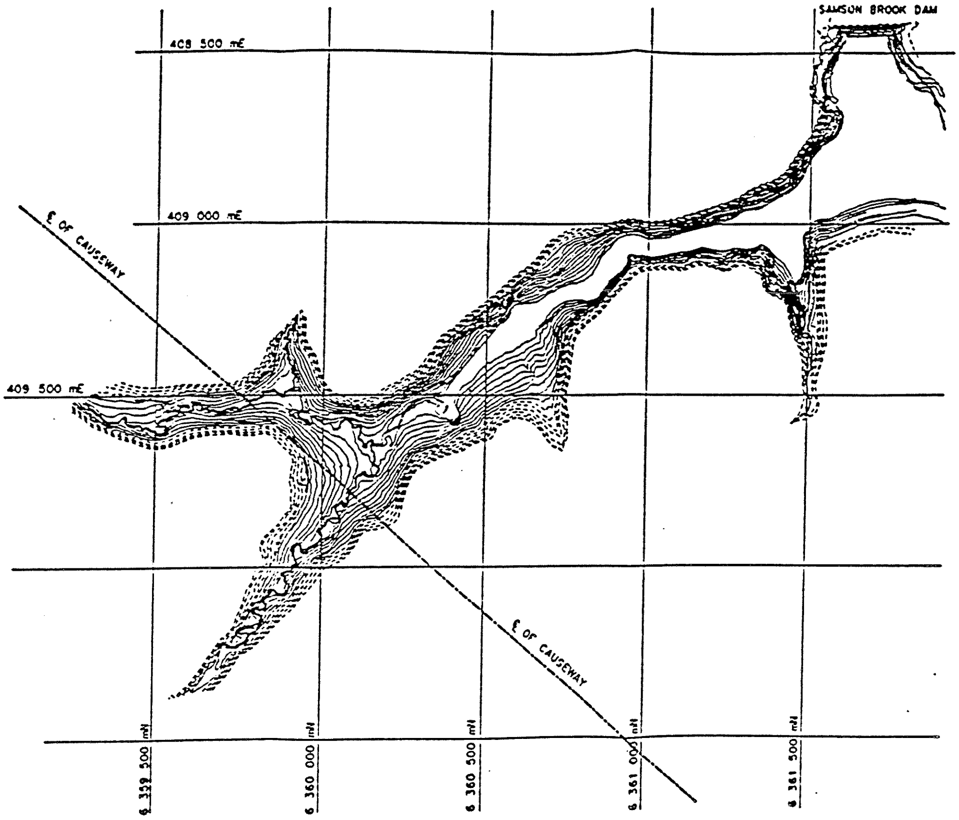


Figure 1. Site plan.

Phase 3 5 boreholes drilled to depths from 7.5 m to 10 m with thin walled tube samples taken at 1.5 m intervals, and PVC groundwater monitors installed in each hole.

Laboratory testing was carried out in three stages. Stage 1 testing was mainly to ascertain the suitability of borrow materials. The borrow area was located downstream (to the west) of the causeway and on either side of Samson Brook, between the edge of the forest and the low water mark. Details of the testing programme are summarised in Table 1.

An investigation of the mineralogy of the soft material was carried out on 4 samples by X-ray diffraction (XRD) testing. A thin section petrographic examination was also undertaken.

Table 1. Details of testing programme.

Test	No. of Tests		
	Stage 1	Stage 2	Stage 3
Moisture Content	9	8	-
Organic Content	-	1	10
Particle Size Distr.	9	-	-
Atterberg Limits	9	3	10
Compaction	7	-	-
Emerson Class Number	6	-	-
Percentage Fines	-	3	-
Triaxial	-	3	-
Oedometer	-	2	2
Shear Vane	-	8	-
Direct Shear	-	2	-

4. SUBSURFACE CONDITIONS

4.1 General

Following is a description of the subsurface conditions encountered within the causeway alignment.

4.2 Northern Tributary

The results of subsurface investigation within the northern tributary causeway alignment are summarised in Table 2.

Table 2. Subsurface conditions Northern Tributary.

Material	Depth (m)	Description
Silty Gravel	0-3	Loose becoming medium dense, and strongly cemented (laterite) in places up to 1 m thick.
Silt/Clay	3-19	"Soft" in 120 m section across the northern tributary becoming stiff and cemented in places to the north and south of this zone and in the south tributary.

A cross section through the soft material is shown on Figure 2.

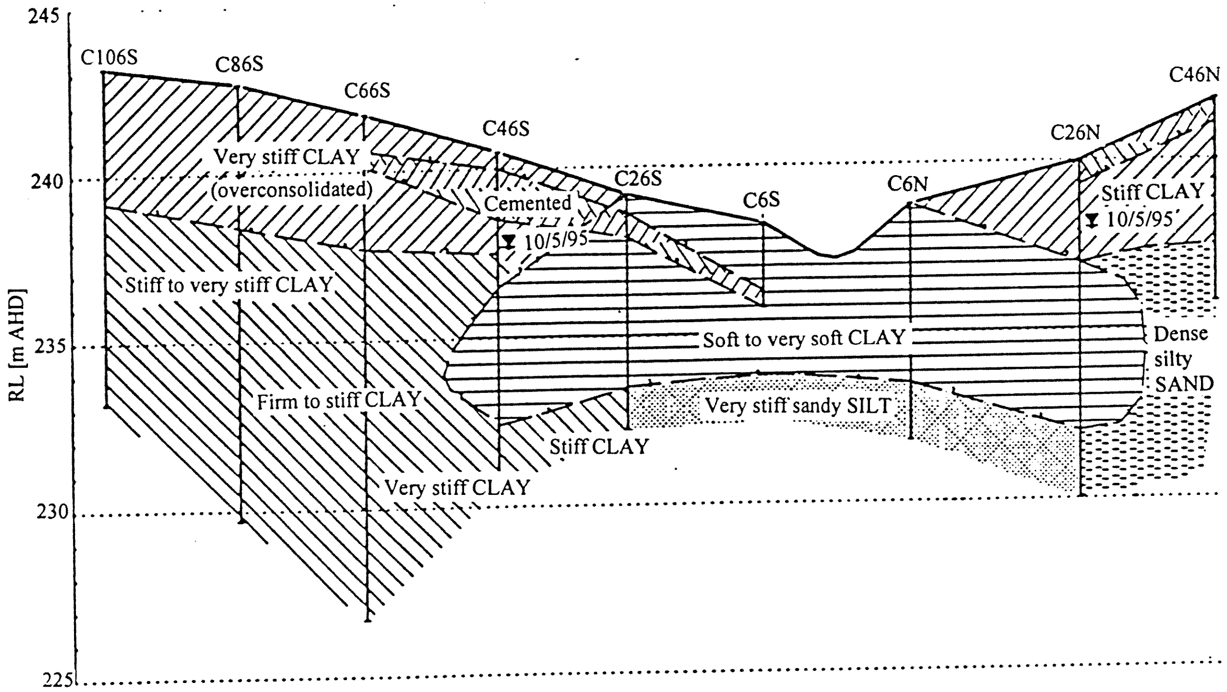


Figure 2. Cross section through soft material.

The term "soft" is used to describe material with a cone resistance (q_c) of 1 MPa or less.

The groundwater levels measured in the boreholes in the vicinity of the northern tributary where soft material was encountered were typically 1-3 m below ground level and in the range RL 237 m AHD to RL 238 m AHD.

4.3 Southern Tributary

Table 3 summarises the results of investigation within the southern tributary causeway alignment.

Table 3. Subsurface conditions Southern Tributary.

Material	Depth (m)	Description
Silty Gravel	0-2	Loose becoming medium dense, and strongly cemented (laterite).
Silt/Clay	2-11	Stiff to very stiff within the entire area, with cemented lateritic bands intermittent, permeable root channels to depths of at least 5 m.

5. ENGINEERING PROPERTIES OF SOFT MATERIAL

5.1 General

Due to the concern surrounding the implications for the project in respect of the soft materials, the laboratory testing programme focussed predominantly on determining the engineering properties of these materials.

The origin of the material was considered vital to its characterisation. An alluvial material of soft consistency and liquidity index greater than 1 may be considered to be inadequate to support the causeway either in the short term or long term without ground improvement. With the cost of ground improvement expected to be very high, extensive testing was warranted to allow for quantification of appropriate design procedure.

5.2 Classification Testing

The results of Atterberg limit testing are presented on Figure 3.

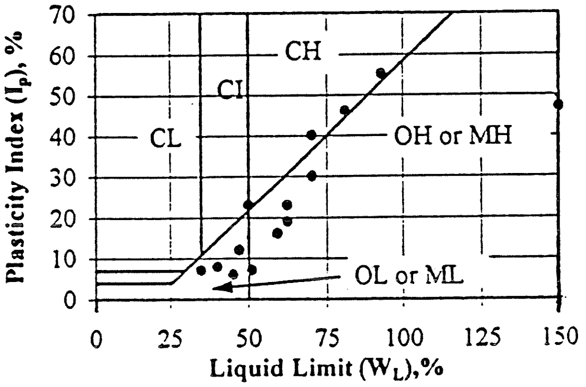


Figure 3. Results of Atterberg limit testing and inferred mineralogy.

The soft materials classified as medium to high plasticity silts, plotting near the A-line.

5.3 Dry Density, Moisture Content and Organic Content

A summary of the results of dry density and moisture content testing is presented in Table 4.

Table 4. Dry density, moisture content and organic content.

Property	No. of Samples	Range	Mean	Standard Deviation
Dry Density (t/m^3)	6	0.89- 1.09	0.99	0.11
Moisture Content (%)	11	37-80	60	15
Organic Content (%)	6	8.4-11.6	9.6	1.3

The low dry densities may be attributed to a combination of leaching of minerals from the clay material by groundwater, and the somewhat unusual clay mineralogy. The high moisture contents can be attributed to the filling of voids left in the material after leaching has occurred, and the presence of clay mineral halloysite which contains chemically bound water which is additional to the free water.

5.4 Mineralogy

5.4.1 X-ray Diffraction

Four samples were selected for quantitative X-ray diffraction (XRD). In two of these samples, diffraction peaks disappeared after drying in a dessicator or after being dried at 65°C. This indicated that the 10A-halloysite in the two samples was extremely unstable. The tests were replicated and the results are shown on Table 5.

5.4.2 Atterberg Limits

The results of Atterberg limit testing are presented in Figure 3. The inferred mineralogy was as shown in Table 6.

Table 5. X-ray diffraction mineralogy.

Borehole	A66S	B6N	C46S	D26S
Depth	4.0-4.4	4.0-4.4	2.8-3.2	3.5-3.9
Gibbsite	0	16	0	35
Halloysite	0	16	2	0
Kaolinite	92	37	44	36
Quartz	8	15	25	27
Amorphous	0	16	29	2

Table 6. Mineralogy of soft clay.

Inferred Mineral	Number of Samples
Halloysite	5
Kaolinite	4
Illite	2
Montmorillinite	1

NB: Sample Population N = 12

5.4.3 Thin Section Petrographic Examination

The classification from examination of a thin section was extremely weathered clay saprolite representing an original high grade metamorphosed amphibolite assemblage with gneissic texture. The original composition was possibly dolerite which has been strongly hydrothermally altered.

6. GEOTECHNICAL PROPERTIES OF SOFT MATERIAL

6.1 Undrained Shear Strength

The undrained shear strength of the soft material was estimated or measured as follows:

(i) hand shear vane (SV) on 'undisturbed' samples (peak and residual strength);

(ii) laboratory direct shear (DS) on 'undisturbed' samples;

(iii) from CPT using the empirical relationship

$$S_u = \frac{Q_c}{20} \text{ (kPa)}$$

The results are summarised in Table 7.

6.2 Drained Shear Strength

The results of consolidated isotropic undrained (CIU) triaxial testing are summarised in Table 8.

6.3 Consolidation Testing

The coefficients of consolidation (C_v), volume compressibility (M_v) and permeability (k) as determined by oedometer testing are presented in Table 9.

Table 7. Undrained shear strength.

Method	No. of Samples	Range (kPa)	Mean (kPa)	Standard Deviation
SV $^{(P)}$ Peak	10	20-64	41	15
SV$^{(R)}$ Residual	10	5-17	11	3
DS	2	53-62	57	-
CPT	10	20-65	39	14

Table 8. Drained shear strength.

Borehole	Depth	Drained Cohesion (c', kPa)	Drained Angle of Friction (ϕ', degrees)
A66S	4.0 - 4.4	0	35.5
B6N	4.0 - 4.4	3	36
C46S	5.8 - 6.2	2	29.5

Table 9. Primary consolidation test results.

Borehole	Depth	C_v (m^2/yr)	M_v (m^2/kN)	k (m/s)
C26N	5.5- 5.9	430	1.7×10^{-4}	2.3×10^{-8}
C46S	5.8- 6.2	220	1.5×10^{-4}	1.0×10^{-8}

The coefficients of long term consolidation (creep) $C\alpha$ are presented in Table 10.

Table 10. Secondary consolidation test results.

Borehole	Depth	k (m/s)	$C\alpha$
A66S	6.0-6.4	3.4×10^{-9}	6.2×10^{-4}
C46S	4.3-4.7	2.9×10^{-9}	6.3×10^{-4}

7. EMBANKMENT DESIGN

7.1 General

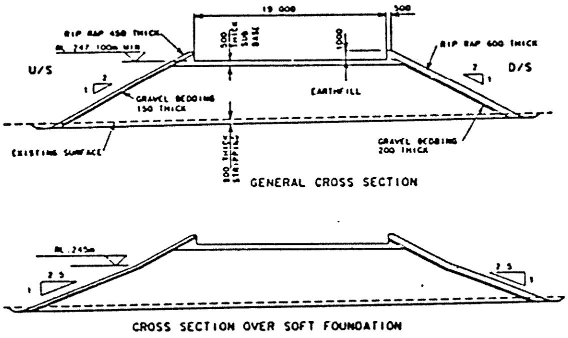
A homogeneous earthfill comprising fine grained material derived from the borrow area was adopted for the causeway (Water Authority of WA, 1995).

A 20 m crest width was set by Alcoa to accommodate:

- the mine haul road;
- the conveyor; and
- transport of a 1000 tonne crusher.

Rip rap was used on external faces of the embankment to protect against wave action.

The design cross sections are shown on Figure 4.



GENERAL CROSS SECTION

CROSS SECTION OVER SOFT FOUNDATION

Figure 4. Embankment design cross sections.

7.2 Foundation Materials

The soft materials encountered within the northern tributary (120 m × 80 m) were characterised by:

(i) low dry density (average 1.0 t/m^3);
(ii) high moisture content in some cases greater than liquid limit; and
(iii) organic content (average 10%).

The X-ray diffraction testing and Atterberg limit testing showed that the clay mineral halloysite was present. A reduction in the Atterberg limits occurred when the material was pre-dried at 105°C. It is inferred from this that the pre-heating caused the conversion of hydrated halloysite ($4H_2O$) to meta halloysite ($2H_2O$). The apparently high organic content can be explained by the loss of water of crystallisation ($2H_2O$) during ignition.

The presence of chemically bound water, together with the 'free' or unbound water, has resulted in the high moisture contents measured. These high moisture contents have resulted in low in-situ dry densities.

The undrained shear strength (S_u) ranged between 20 kPa (soft) and 60 kPa (stiff). The residual shear strength was about 10 kPa (average). The sensitivity of the material was moderate to insensitive.

The design drained shear strength parameters for the soft material were based on lower bound values determined from effective stress triaxial testing:

- drained cohesion (c') = 2 kPa
- drained angle of friction (ϕ') = 30°

These parameters are consistent with a material which is slightly over-consolidated, and at the lower end of the measured undrained shear strengths.

The mineralogy suggests that the original rock may have been dolerite, subsequently metamorphosed and completely weathered to the present high grade amphibolite. The variable strength within the soft material may have been due to banded migmatites, similar to those seen at Waroona Dam where amphibolite, amphibolite schist and banded gneiss occur with near vertical dip. The presence of halloysite is consistent with amphibolite rock under conditions of prolonged saturation.

The laboratory consolidation tests indicated that the soft material was moderately compressible and the drained Young's modulus (E') was estimated to be about 6 MPa. The high C_v values indicated that the soft material would consolidate very rapidly (i.e. during construction). Field evidence was that soft material was of high permeability due to root holes, solution channels, fissures and relic joints, and discontinuities associated with banded material. The hydraulic gradient measured between the causeway and Samson Dam was very low (about 0.002) suggesting a high permeability material in between. Subsurface conditions at the Harris Dam showed similarities to these, and permeability was of the order of 10^{-5} m/s. Excess porewater pressures at Harris Dam were observed to dissipate almost overnight, and settlement of 600 mm occurred with 95% completed during construction.

The following predictions were made based on the laboratory oedometer tests, and data and experience from other sites with similar materials present:

- Settlement due to primary consolidation: 350-500 mm, occurring over a period of 3 months.
- Settlement due to secondary consolidation: 20-30-mm, occurring over a period of 2 years.

In order to decrease consolidation time, either a surface sand blanket or sand finger drain was recommended. Wick drains and stone columns were considered, but were prohibitively expensive.

7.3 Stability Analysis

7.3.1 General

A stability analysis was undertaken by the Water Authority of WA (1995). The analysis was based on the use of the slip circle (2 dimensional) limit equilibrium method attributable to Bishop. The foundation conditions analysed were the stiff clay and soft clay. Embankment slopes of 1:2 for the stiff clay foundation and 1:2.5 for the soft clay foundation were adopted.

7.3.2 Material Properties

The material properties assumed are shown in Table 11.

Table 11. Material properties for stability analysis.

Total Stress	Undrained Shear Strength (Su, kPa)		
Earthfill	50		
Stiff clay foundation	50		
Soft clay foundation	40		
Effective Stress	Drained cohesion (c', kPa)	Drained friction angle (ϕ')	Density (t/m^3)
Earthfill	10	30	1.96
Stiff clay foundation	10	26.5	1.85
Soft clay foundation	2	30	1.62

Excess pore pressures were not expected to occur in the foundation because of the high C_v's measured in the laboratory testing, and from experience at Harris Dam where foundations were quite permeable, and similar materials to Lake Kabbamup were encountered. Excess pore pressures at Harris Dam dissipated very quickly. In addition, the groundwater levels were similar to that of the Samson Dam, suggesting good hydraulic connection and high permeability materials in between.

Due to the significance of the structure and the design life of 15 years, the selection of strength parameters was based conservatively on lower bound values.

The undrained shear strength (S_u) adopted for the soft clay foundation for the total stress condition was based on the field testing, as discussed in Section 6.1. Given the satisfactory factor of safety (FS) derived for this condition, it was not necessary to compute FS for the stiff clay foundation. S_u for the compacted clay earthfill was estimated from experience at Harris and North Dandalup Dams.

The effective stress parameters for the compacted earthfill and stiff clay foundation were based on lower bound values from triaxial testing for Harris and North Dandalup Dams.

The effective stress parameters for the soft clay were based on the lower bound values from the present investigation.

7.3.3 Analysis

Six loading conditions were considered which included the transport of the 1000 tonne crusher applied under undrained conditions across the causeway.

Details of the six (6) loading conditions were:

1. Steady seepage, reservoir full. The reservoir is at full supply level (RL 245 m) and has been there long enough for the phreatic surface in the embankment and foundations to also be at RL 245 m.

2. Steady seepage, reservoir empty. The reservoir is empty (RL 237 m) and the phreatic surface in the bank and foundations is also at RL 237 m.

3. Rapid drawdown. The reservoir has been drawn down over a summer from RL 245 to RL 237 m. Pore pressure develops in the bank because it is very impermeable and the material cannot drain at the same rate as the lake level falls. The B parameter estimated was 0.3 at the centre of the bank and varies to 1.0 at the face. The foundations were assumed to be free draining (based on experiences with many dams in the Darling Range) so no pore pressures above RL 237 m were assumed to develop.

4. Alcoa plans to transport a 1000 tonne crusher across the causeway twice during its lifetime. It should be noted that the 1000 tonne load consists of the crusher load of 830 tonne plus the trailer load of 171 tonne. The model of the load was a 40 t load on the crest of the bank and increased pore pressure within the bank and foundation. For condition 4 this was combined with the steady seepage pore pressures.

5 & 6 The crusher load detailed above was combined with conditions 2 and 3 respectively.

The results of the analyses for the stiff clay and soft clay foundation are summarised in Table 12 and Table 13 respectively.

Table 12. Stability analysis results for stiff clay foundation - effective stress.

Condition	Factor of Safety	Required FS
1. Steady seepage, reservoir full	2.16	1.5
2. Steady seepage, reservoir empty	1.68	1.5
3. Rapid drawdown	1.41	1.3
4. Condition 1 + crusher load	1.75	1.5
5. Condition 2 + crusher load	1.48	1.5
6. Condition 3 + crusher load	1.24	1.3

Table 13. Stability analysis results for soft clay foundation - effective stress.

Condition	Factor of Safety	Required FS
1. Steady seepage, reservoir full	2.11	1.5
2. Steady seepage, reservoir empty	1.68	1.5
3. Rapid drawdown	1.45	1.3
4. Condition 1 + crusher load	1.70	1.5
5. Condition 2 + crusher load	1.49	1.5
6. Condition 3 + crusher load	1.28	1.3

The result of the total stress analysis is presented in Table 14.

Table 14. Stability analysis results for soft clay foundation - total stress.

Condition	Factor of Safety	Required FS
End of construction	1.35	1.3

8. CONSTRUCTION PERFORMANCE

Settlement of the culverts was measured during and after construction. Survey pins were established at 10 m intervals along each of the culverts. The maximum settlements measured at the mid-point of the northern and southern tributary culverts are presented in Table 15.

Table 15. Construction settlements.

Location	Foundation Condition	Maximum Settlement (mm)	Time for 90% Consolidation (t_{90}, months)
Northern Tributary Culvert	Soft Clay	200	3*
Southern Tributary Culvert	Stiff Clay	100	3*

* Includes construction time.

No monitoring of foundation porewater pressures was carried out.

The secant moduli back-calculated from the construction settlements were 6 MPa (soft material) and 12 MPa (stiff material).

Approximately 50% of the primary consolidation settlement occurred during construction.

9. CONCLUSIONS

- The geotechnical investigation has demonstrated the presence of a weathered laterite profile with highly variable engineering and geotechnical properties.
- In the surrounding borrow area and southern tributary, the materials were typically of stiff consistency, and had an average moisture content of around 30%. In the northern tributary, a distinct 'soft' zone about 120 m in length was investigated in some detail.
- The engineering properties of the 'soft' materials were significantly different in comparison to the regional materials investigated. A thorough investigation and characterisation of the soft materials was therefore justified. The testing confirmed that the material was metamorphosed amphibolite and was part of the lateritised weathered profile.
- The effective stress parameters measured in the laboratory triaxial tests showed that the material had reasonable strength, and stability analyses confirmed that factors of safety were adequate to support the causeway.
- Mineralogical assessment confirmed the presence of halloysite, which accounted for the very high moisture content. A significant proportion of the moisture content may therefore be accounted for as chemically bound water within the halloysite. The material would not be expected to behave as either a 'liquid', or as a highly compressible material (normally or under-consolidated) as the engineering properties might suggest.
- With regard to embankment design, the testing has demonstrated that the foundation material had sufficient strength to support the embankment both in the short term and the long term, and that ground improvement was not required.
- The predictions of magnitude and rate of settlement were based on vertical upwards drainage being permitted. To achieve this, sand finger drains were constructed in the base of the embankment.
- Based on the laboratory test results and the absence of shear failures during construction, it is concluded that the soft foundation materials have reasonable strength properties, notwithstanding low dry densities and high apparent moisture contents.
- Laboratory testing indicated that the soft material was moderately compressible. Settlement estimates based on oedometer testing were conservative, being about twice the actual settlement. Actual settlement may have been reduced by the presence of the upper cemented ferruginous layer. This was not considered in predictions due to its variable strength and discontinuous nature. The predicted rate of settlement, based on a design C_v of 100 m^2/yr, compared well with the measured rate. The actual time taken for 90% of primary settlement to occur was 3 months. About 50% of the primary settlement occurred during construction.
- Based on the field consolidation settlement data, the average permeability (k_v) was estimated to be 5×10^{-9} m/s. This agreed well with laboratory permeability data measured in oedometer and triaxial tests.

10. ACKNOWLEDGEMENTS

The Author wishes to thank Alcoa of Australia for its permission to publish this paper and for providing settlement data. Also the Water Authority of WA, in particular Michael Somerford for his assistance and providing the design report.

11. REFERENCES

Geological Survey of Western Australia (1980). 1:250,000 Geological Series Map, Pinjarra Sheet SI/50-2.

Water Authority of Western Australia (1995). Lake Kabbamup Causeway Design Report.