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Project 35046/1

14 August 2002

REPORT ON
GEOTECHNICAL INVESTIGATION
PROPOSED PRAWN FARM
GUTHALUNGRA

1.0 INTRODUCTION

This report presents the results of a geotechnical investigation for a proposed prawn farm approximately 5km north of Guthalungra, located between Bowen and Home Hill. The work was carried out for Sinclair Knight Merz (SKM) on behalf of Pacific Reef Fisheries.

An acid sulphate investigation was also conducted in parallel and is reported under separate cover (DP Report: 35046/2).

The scope of work performed by Douglas Partners Pty Ltd (DP) comprised a series of backhoe test pits at locations generally distributed throughout the proposed facility, followed by laboratory testing and reporting.

A number of samples collected by SKM from the site have previously been tested and were reported by DP in February 2000 (DP Report Ref: 21236). This earlier DP report is appended.

The purpose of the current geotechnical investigation was to provide the following:

- descriptive logs of all test pits plus a summary of subsurface conditions and geology;
- results of all laboratory testing;
- comments on suitably of the soils for pond lining;
- guidelines for earthworks placement, compaction and moisture control;
- comments on dispersivity of clay soils and advice on protective measures to be taken to reduce the risk of piping failure;
- comments on erosion protection;
- comments on suggested frequency of earthworks quality control testing during construction;
- estimates of liner material quantity; and
- site classification of the office/processing area in accordance with AS 2870-1996 (Ref 1).

Layout drawings showing the areal extent of the proposed development were provided by SKM to assist in the investigation.

2.0 SITE DESCRIPTION AND GEOLOGY

The site is located approximately 5km north of Guthalungra and 58km south east of Home Hill, on a relatively flat coastal plain with some shallow drainage channels or swales. Extensive tidal mud flats were observed bordering on and beyond the northern extremity of the site. Vegetation cover comprised relatively short grasses and scattered trees. The site covers an area of approximately 340ha (refer Drawing 1).

At the time of the investigation, the property was being used for cattle grazing. Several relatively small dams were located near the central western portion of the site (Stage 2 ponds) and a larger dam with cattle yards situated in the south eastern corner (near the proposed office and processing facility).

As indicated on attached Drawing 1 and discussed more fully in Section 4.0 below, the surface terrain was observed to comprise predominantly two main types as follows:

- terrain with a relatively smooth surface (easily traversed);

- terrain with shallow depressions and dried out cattle hoof prints (rough surface).

In addition, two discrete small areas were observed which appeared to differ from the above terrain. They are described as follows:

- A narrow strip of land, approximately 40m to 50m wide, and probably directly underlain by sand, extended northwards from the vicinity of Pit 54 for approximately 300m to 400m. This strip of land contained a group of trees not encountered outside the strip.
- A salt flat tributary of slightly depressed land in the vicinity of Pit 39, drained northward to the salt flats located beyond the northern boundary of the site, and was probably underlain by relatively soft clay.

The Queensland Department of Mines' 1:250,000 Geological Series Ayr Sheet and accompanying Explanatory Notes indicate the site to be underlain by alluvial and deltaic sedimentary deposits, of varying particle size (sand through clay with gravel) and "semi-consolidated in places". Close to and beyond the northern boundary of the site the map indicates coastal mudflats (littoral flats and salt pans) and superficial coastal sand dunes. All sediments are indicated to be of Quaternary age.

These descriptions are consistent with what was encountered during field work.

3.0 FIELD WORK METHODS

The field work performed by DP, as relevant to the geotechnical investigation, was undertaken between 25 June and 28 June 2002 and comprised the following:

- walkover appraisal by a senior geo-environmental engineer and experienced geo-environmental engineer from Douglas Partners (DP); and
- subsurface investigation by 54 backhoe test pits (designated Pits 1 to 54); directed by an experienced geo-environmental engineer from DP who logged the profiles encountered, took soil samples for testing and took groundwater depth measurements; and

Additional sampling was undertaken in some of these test pits for acid sulphate soils assessment which is reported separately.
The test pits were located on a general grid of 200m to 500m with occasional additional pits allocated as considered appropriate.

The test pits were set out by the field geo-environmental engineer using a hand-held GPS, based on AMG coordinates supplied by SKM. It should be noted that the coordinates required conversion to the WGS 84 datum, as handheld GPS units are not equipped to process the supplied AMG coordinates. The approximate locations of these pits are indicated on attached Drawing 1.

4.0 FIELD WORK RESULTS

The subsurface conditions encountered in Pits 1 to 54 are presented in the attached report sheets. These should be read in conjunction with the general notes preceding them, which explain descriptive terms and classification methods.

4.1 Soils

It is considered probable that most of the soils encountered are of alluvial or deltaic flood plain origins, although it is considered possible that soils of lower strength consistency (i.e. relatively more soft) may represent buried coastal or lagoonal deposits. Such coastal deposits may be represented in Pit 39, and the slightly depressed salt flat tributary feature draining northwards from there.

With the exception of Pit 39, where a layer of soft silty clay was encountered, and Pit 54, which was situated within a relatively narrow sandy strip (refer Section 2.0 above), the site can be divided broadly into two generalised profiles, based on near surface conditions as follows:

Shallow Profile 1 – Dark Grey Silty Clays (Rough Surface)

Approximately one half of the pits (22 no) encountered generally very stiff dark grey silty clay from the surface to depths between 0.35m and 1.4m depth (but more frequently between 0.8m to 1.4m depth). The ground surface at and surrounding these locations was generally very uneven, and considered to be highly moisture sensitive. Traversing the terrain underlain by this layer was generally slow, due to the large number of holes either creating by pooling water (previously evaporated/absorbed) or by hoof prints of the cattle grazing across the property, imprinted when the ground was probably moist. The approximate extent of the area underlain by this material is delineated on attached Drawing 1.

Underlying this upper clay was a **Transition Layer** of generally hard sandy/silty clays. These clays were sometimes fissured and usually grey brown/brown grey. This clay layer extended to depths between 0.9m and 3.2m depth (but more frequently 1.7m to 3.2m depth), although it was not apparent in Pits 12 and 51.

Shallow Profile 2 – Brown Grey Sandy / Silty Clay (Relatively Smooth Surface)

The remaining test pits (30 no) generally encountered very stiff to hard grey brown / brown grey slightly sandy to sandy silty clay to depths between 0.3m and 3.1m (but more frequently between 0.3m to 1.2m depth). This clay was commonly fissured and contained trace amounts of fine to coarse gravel sized carbonate nodules.

The upper clay layer was generally underlain by a **Transition Layer** of mottled orange brown slightly sandy to sandy silty clay, grading to clayey sand in places. Weak cementation was regularly observed within the orange brown layer. Some fine to coarse subrounded gravel was observed where the layer graded to clayey sand.

At two locations (Pit 4 and 41) excavation was terminated early due to virtual backhoe refusal on variably cemented gravelly sandy clay/clayey gravelly sand. It is considered that this cementation most probably represents a localised induration process and does not represent underlying bedrock.

Beneath the silty or sandy clay soils described under **‘Shallow Profiles 1 and 2’** above, many of the test pits (30 out of the 54 excavated) encountered clayey sand. These **‘Deep Profile’** soils, where encountered, were noted below depths of 0.8m to 2.8m (below an average depth of 1.8m).

4.2 Groundwater

Groundwater was encountered during excavation between depths of 2.3m to 3.1m in Pits 20, 22, 23, 25, 34, 36 and 37. It should be noted, however, that groundwater depths are affected by climatic conditions and soil permeability and, at this site, probably by tidal influence, and will therefore vary with time.

5.0 LABORATORY TESTING

Laboratory testing comprised the analysis of plasticity (Atterberg limits and linear shrinkage), dispersivity (Emerson Class), laboratory permeability and shrink-swell tests. The permeability and shrink-swells were performed on samples remoulded to 98% Standard compaction.

The detailed results of all tests are attached and are discussed in Section 6.0 below.

6.0 COMMENTS

6.1 Proposed Development

It is understood that the proposed development is to occupy approximately 340 ha and comprise 256 aquaculture ponds to be constructed in three stages, each pond covering an area of approximately 100m by 100m (refer attached Drawing 1). In addition, it is understood that there will be two sedimentation ponds and a seawater storage dam on the seaward side of the ponds and an office/processing storage facility near the entrance to the development in the south east corner.

Adjacent water supply and disposal pipelines are indicated on supplied drawings, extending north north east for approximately 1.5km across the tidal flats from the north east corner of Stage 3 ponds, and then 2km east north east across sand dunes and intermittent salt flats to the sea.

Crest heights for the pond-separating bund walls and depths of excavation had not been set at the time of preparation of this report. It should be noted, however, that the aquaculture ponds are likely to be 1m to 2m deep and the seawater storage and sedimentation dams up to a maximum of 5m deep, with a balanced cut to fill where possible.

It is understood that the bund walls are to be battered at no steeper than 2H to 1V with water maintained at a freeboard of no less than 0.3m. In this connection, it should be noted that cracking depths of the more reactive clays at this site are likely to be up to 0.8m approximately in prolonged dry conditions, unless well irrigated grass cover or other protection systems are adopted to reduce cracking.

6.2 Pond Lining Materials Selection

6.2.1 Location of Suitable Liner Material

With reference to Section 4.0 above, it is considered that all “Shallow Profile” silty or sandy clays, as encountered throughout the majority of the investigation area, are likely to be suitable for use as pond lining materials. It should be noted, however, that the dark grey clays (rough surface areas shown on Drawing 1) are more sensitive to moisture variation and are therefore more likely to be prone to cracking when dry.

Based on a visual appraisal of the moisture conditions within the pits, both of these materials appear to have relatively low insitu permeabilities. Those materials underlying the upper clays were generally observed to be considerably drier, comprised predominantly clayey sand (‘Deep

Profile'), and were generally considered unsuitable either for pond lining or for water retention in situ.

Areas to be avoided for borrow material have been described in Section 2.0 above and are summarised as follows:

- the strip of tree growth in the vicinity of and up to some 300m to 400m north of Pit 54, which is probably directly underlain by sand throughout;
- the salt flat tributary, draining northwards from Pit 39, which is probably underlain by acid sulphate soils throughout.

Selection of suitable lining materials should be made with due consideration of plasticity, dispersivity characteristics and propensity for shrink-swell movement. These characteristics are described in Sections 6.2.3 and 6.2.4 below.

6.2.2 Estimated Quantity of Suitable Liner Material

Approximate calculations were performed to estimate the amount of suitable clay liner material at the site for bund construction purposes. These were based on an average depth to base of "Shallow Profile" clays of 1.8m (Section 4.0 above) and a total site area of 340ha less the sand strip north of Pit 54 and the soft clay strip north of Pit 39.

This results in an estimated in situ volume of approximately 5.9 million cubic metres. This reduces to approximately 4.3 million cubic metres after allowing a nominal 0.5m cover of clay left over the underlying clayey sands.

In the event that only 'Shallow Profile 2' soils are used (smooth surface on attached Drawing 1), then the above quantities reduce to approximately 4.4 million cubic metres and 3.2 million cubic metres respectively. Initial on site appraisal indicates that the 'Shallow Profile 1' clays are more reactivity compared to the 'Shallow Profile 2' clays, as evident by rutting and cracking on the surface. This is supported by the results of limited shrink-swell tests but not by plasticity tests. Additional laboratory testing would therefore be required for verification.

6.2.3 Plasticity

It is recommended that suitable lining materials either be confirmed by an experienced geotechnical engineer or geotechnician, or verified by additional plasticity testing during construction. The results of plasticity testing should be used to confirm that the plasticity index and liquid limit properties of the lining materials plot above the ‘A’ line on Table A1 of AS 1726 (Ref 2). In addition, plasticity index values should be $\geq$ 8% and liquid limit $\geq$ 23%, as indicated on attached Drawing 2, which is adapted from Fell et al (Ref 3).

Upon inspection of the results of plasticity testing from the present investigation (Table 6.1 below), it is apparent that all materials tested during this investigation satisfy the above plasticity requirements. As discussed above, however, the ‘Shallow Profile 1’ soils appear to be more reactive (site inspection only) than the ‘Shallow Profile 2’ soils, although this is not supported by the results of plasticity testing. The results of limited shrink-swell testing, however, on two samples only (refer Section 6.4 below) do indicate there may be a higher cracking potential for Shallow Profile 1 soils (Pit 33 at 0.5m depth) and a much lesser cracking potential for Shallow Profile 2 soils (Pit 38 at 1.0m depth). It is particularly important to control cracking, even if the ponds are kept near full, since deep cracks developing along the crest of internal or external walls may lead to leakage at a high level.

Table 6.1 – Summary of Plasticity Test Results for Samples from Present Investigation

Pit No	Depth (m)	Description	Shallow Profile (1)	Liquid Limit (%)	Plasticity Index (%)
4	1.5	Brown slightly sandy silty clay	1T	42	27
5	0.5	Grey brown sandy silty clay	2	42	28
6	0.5	Dark grey silty clay	1	53	35
8	1.5	Light orange brown sandy silty clay	2T	35	22
12	0.5	Dark grey silty clay	1	47	32
15	1.5	Light brown mottled orange brown sandy silty clay	1T	42	29
20	1.0	Light brown mottled orange brown sandy silty clay	2T	36	24
27	0.5	Dark brown grey silty clay	2	50	34
28	1.0	Light brown grey silty clay	2	53	36
33	0.5	Dark grey silty clay	1	55	38

36	1.5	Mottled light grey and orange brown silty sandy clay	2T	34	20
37	0.5	Mottled light brown grey and orange brown slightly sandy silty clay	2T	32	16
42	0.5	Yellow brown slightly sandy silty clay	2	51	36
45	0.5	Brown grey mottled orange brown and black silty clay	2	47	32
45	1.5	Light orange brown sandy silty clay	2T	33	20
46	1.0	Brown grey sandy silty clay	2	32	20
47	0.5	Dark grey silty clay	1	46	32
53	1.5	Light brown sandy silty clay	1T	45	28

<u>Note</u>: (1) *Refer Section 4.1 above for definition of main soil types and Drawing 1 for location. 'T' denotes transition layer between Shallow and Deep Profile soils.*

Table 6.2 – Summary of Plasticity Test Results for Samples from Earlier SKM Investigation

Pit No	Depth (m)	Description	Profile (1)	Liquid Limit (%)	Plasticity Index (%)
G2A	0.5	Dark grey silty clay	-	40	23
G2B	1.0	Dark brown sand and clayey silt	-	23	1
G3B	1.5	Mottled grey and yellow brown silty clay with some fine sand	-	32	13
G6B	1.0	Dark brown fine sandy clayey silt	-	29	5
G7C	2.0	Grey brown silty clay with a trace of fine sand	-	36	21
G8A	0.3	Dark grey fine sandy clayey silt	-	25	2
G9C	2.0	Light grey silty clay	-	51	36
G10B	1.0	Mottled grey and yellow brown gravelly sand with clayey silt	-	28	5
G11B	1.5	Brown fine sandy clayey silt	-	25	3

<u>Note</u>: (1) *Profile not sighted or logged by DP. Samples provided by SKM (refer report in Appendix D).*
denotes unsuitable for lining purposes (refer Drawing 2 in Appendix C)

Five of the nine samples tested from the earlier report (Feb 2000) do not conform to the above liner plasticity requirements. These are highlighted in Table 6.2 above. Although DP did not sample these soils or log the pits from which they were taken, it is suggested that these soils are either representative of the transition layer or deep profile soils. Sample G8A may be from the salt flat tributary indicated above (Section 6.2.1) as unsuitable for lining materials.

6.2.4 Dispersivity

(a) Conclusions

A summary of Emerson Class dispersion test results is presented in Table 6.2 below.

Table 6.3 – Summary of Dispersion Test Results, Present Investigation

Pit No	Depth (m)	Description	Shallow Profile (1)	Emerson Class No.
4	1.5	Brown slightly sandy silty clay	1T	5
5	0.5	Grey brown sandy silty clay	2	4
6	0.5	Dark grey silty clay	1	7
8	1.5	Light orange brown sandy silty clay	2T	4
12	0.5	Dark grey silty clay	1	7
15	1.5	Light brown mottled orange brown sandy silty clay	1T	4
20	1.0	Light brown mottled orange brown sandy silty clay	2T	4
27	0.5	Dark brown grey silty clay	2	5
28	1.0	Light brown grey silty clay	2	5
33	0.5	Dark grey silty clay	1	7
36	1.5	Mottled light grey and orange brown silty sandy clay	2T	6
37	0.5	Mottled light brown grey and orange brown slightly sandy silty clay	2T	4
42	0.5	Yellow brown slightly sandy silty clay	2	5
45	0.5	Brown grey mottled orange brown and black silty clay	2	7
45	1.5	Light orange brown sandy silty clay	2T	4
46	1.0	Brown grey sandy silty clay	2	4
47	0.5	Dark grey silty clay	1	7
53	1.5	Light brown sandy silty clay	1T	6

Note: (1) *Refer Section 4.1 above for definition of main soil types and Drawing 1 for location. 'T' denotes transition layer between Shallow and Deep Profile soils.*

Table 6.4 – Summary of Dispersion Test Results, Earlier SKM Investigation

Pit No	Depth (m)	Description	Shallow Profile [1]	Emerson Class No.
G2A	0.5	Dark grey silty clay	-	4
G2B	1.0	Dark brown sand and silty clay	-	5
G3B	1.5	Mottled grey and yellow brown silty clay with some fine sand	-	5
G6B	1.0	Dark brown fine sandy clayey silt	-	4
G7C	2.0	Grey brown silty clay with a trace of fine sand	-	4
G8A	0.3	Dark grey fine sandy clayey silt	-	3
G9C	2.0	Light grey silty clay	-	5
G10B	1.0	Mottled grey and yellow brown gravelly sand with clayey silt	-	3
G11B	1.5	Brown fine sandy clayey silt	-	4

Note: [1] *Profile not sighted or logged by DP. Samples provided by SKM (refer report in Appendix D).*
denotes dispersive

The above results (Table 6.3) indicate that the soils tested during this investigation are unlikely to be highly dispersive, such as would be characterised by an Emerson Class number of 1, 2 or 3. Two of the earlier samples tested (Table 6.4) did prove dispersive. It is considered possible that these soils (Pits G8A and G10B) represent salt flat materials and hence are not suitable for lining.

Experience with water retention bunds and dam walls elsewhere in North Queensland indicates that dispersive-type tunnel or pipe erosion features can develop even in materials with Emerson Class numbers of 4 or higher, where they are subjected to fluctuating water levels, seasonal moisture variation and associated cracking. It is therefore suggested that these soils be treated as slightly dispersive.

(b) Preventative Action

The most secure line of defence against dispersive failure of cohesive soils is to ensure good compaction at a moisture content close to or wet of optimum and regular maintenance to prevent crack development. Hence, compliance with the summary construction specification presented in Section 6.3 below will greatly assist in this regard.

(c) Good Engineering Practice

In addition to the above, it is suggested that special care be taken where the clay soils are placed close to a fluctuating water level or where pipes are to be carried through water retention bunds.

In the former case (fluctuating water table) it would be prudent to take precautions to prevent cracking of the liner, leading to dispersion on rewetting and ultimate ‘blow-out’ through the core. These precautions may either comprise use of rip-rap and vegetation cover, as for erosion protection (Section 6.6 below) or the addition of an estimated 2% to 3% by weight of calcium sulphate (gypsum) to further inhibit dispersive erosion. Use of gypsum should, however, be checked against aquaculture requirements.

In the latter case (pipes through embankment), it is suggested that consideration be given to backfilling pipe trenches entirely with clay soil, treated with 2% to 3% by weight of calcium sulphate and carefully compacted in layers, as for the remaining embankment, but using hand tools or hand guided compaction equipment. Dependent upon the size and length of the pipe (and the driving head or hydraulic gradient), it may also be advisable to include buttress walls around the pipe at regular intervals to inhibit seepage close to the soil/pipe interface.

6.2.5 Permeability

It is understood that seepage from the ponds is to be minimised in order to reduce the impact on adjacent properties from elevated groundwater and varying salinity. Accordingly, the thickness, moisture conditioning and compaction of lining materials is critical in order to reduce permeability and hence seepage.

The results of permeability tests performed in the laboratory, on remoulded samples, are summarised in Table 6.3 below. All testing was performed on clay samples, remoulded to a relatively high compactive effort (98% dry density ratio). There were insufficient tests conducted to indicate the effects of moisture content variation on permeability. Based on experience with similar soils, however, it is considered in-house and well documented elsewhere that lowest permeabilities are generally achieved close to and wet of optimum.

Table 6.3 – Summary of Permeability Test Results for Remoulded Soil Samples

Pit No	Depth (m)	Description	Remoulded Density Ratio (%)	Relative Moisture Content* (%)	Permeability (m/sec)
20	1.0	Sandy silty clay	98.0 STD	0.1 WET	1×10^{-9}
28	1.0	Silty clay	97.5 STD	0.3 WET	3×10^{-11}
33	0.5	Silty clay	98.0 STD	3.0 WET	3×10^{-11}
42	0.5	Silty clay	98.0 STD	1.5 WET	5×10^{-11}
46	1.0	Sandy silty clay	98.0 STD	0.1 WET	2×10^{-10}
47	0.5	Silty clay	98.0 STD	4.9 WET	5×10^{-11}

Legend: *STD – denotes Standard Compaction*
WET – denotes wet of Standard optimum moisture content

Based on the above, it appears likely that the silty or sandy clays encountered throughout most of the site are likely to be of low permeability or even practically impermeable. This is provided earthworks construction is performed in accordance with Section 6.3 below.

6.3 Earthworks Construction

6.3.1 Trafficability

Trafficability is likely to be a problem at this site after rain, particularly in the areas underlain by the dark grey silty clay (rough surface), necessitating use of tracked vehicles in such conditions.

6.3.2 Dewatering

Based on limited groundwater observation during the investigation, it is concluded that groundwater is unlikely to be a problem over much of the site during the major part of construction. This is provided excavation is less than approximately 2.0m deep on the lower areas within approximately 50m of the northern site boundary and 3m elsewhere. Provision for pumping of excavations, however, should be made, in the event of encountering localised perched groundwater contained by fine lagoonal or flood sediments within the sands. Similarly, tidal influence may lead to a requirement for dewatering of excavations.

6.3.3 Subgrade

In order to maximise stability of the batter slopes and facilitate good liner construction across the pond floors, it is recommended that good subgrade preparation be undertaken under close supervision and testing to the following specification:

- excavate to proposed formation levels and remove any remaining deleterious soft, wet or highly compressible material;
- test roll the complete surface of the subgrade (with a roller of minimum 8 tonne static weight) in order to detect the presence of any localised soft or loose zones which should either be compacted (sand soils only) or excavated out (if soft clay or very silty) and replaced with approved filling;
- compact the tyned natural foundation soil to a minimum dry density ratio of 98% Standard.

6.3.4 Floor Liner

Where a selected floor liner (cohesive soil) is required to cover excavation exposed sandy soils, it should be placed in layers of maximum 200mm loose thickness and compacted to a minimum dry density ratio of 98% Standard and at a moisture content of between 1% dry to 2% wet of Standard optimum moisture content.

Final floor liner thickness should be selected not only in accordance with seepage considerations but in relation to potential damage incurred by machinery during maintenance periods when the ponds are temporarily emptied.

6.3.5 Bund Walls

The bund walls should be constructed by raising and compaction in horizontal lifts, followed by trimming back of the face of each batter afterwards to the required thickness.

Bund wall materials should be placed in maximum 200mm loose thickness layers and compacted to a minimum dry density ratio of 98% Standard.

6.4 Footings / Site Classification (Office & Processing Facility Only)

6.4.1 Testing

Two (2) shrink-swell tests were performed on disturbed samples obtained in the vicinity of the proposed office & processing facility. The samples were remoulded to 98% Standard Proctor.

The detailed results are attached and are summarised in Table 6.5 below.

Table 6.5 – Summary of Shrink-Swell Test Results

Test Pit	Depth (m)	Description	Shrink-Swell Index (I_{ss}) (% per DpF)
33	0.5	dark grey silty clay	4.7
38	1.0	grey brown silty clay	1.1

6.4.2 Footings

Provided that site preparation is carried out as recommended in Section 6.3.3 above it is recommended that small pad or strip footings, founded at depths generally 0.5m or greater, in very stiff to hard clays or silty/sandy clays, may be designated for an allowable bearing pressure of up to 300kPa. This is provided that all external footings (around the outer perimeter of the building) are protected by concrete paving or the ground surface otherwise appropriately sealed and shaped to guard against ponding of surface water. Where there is potential for water to infiltrate and soften the clay subgrade under prolonged wet weather, the allowable bearing pressure should either be limited to 200kPa, or the external footings founded at a minimum depth of 1.2m. Where there is potential for building foundations to be subjected to prolonged flooding then the bearing pressure may require to be further reduced to 100kPa for small pad or strip footings.

The above values are based on a factor of safety of approximately 2.5 to 3 against bearing capacity failure.

With respect to untied slab panels or slab-on-ground footings, it is suggested that design bearing pressures will be governed by settlement rather than bearing capacity failure, but should generally be no greater than 50kPa. Similarly, heavily loaded column pad footings (bearing pressure in excess of 100kPa) should be checked for tolerable settlement.

6.4.3 Site Classification

Site classification of foundation soil reactivity provides an indication of the propensity of the ground surface to move with seasonal variation in moisture. An in-house computer program, REACTIVE, was used to calculate characteristic surface movement (y_s) for the site, based on procedures presented in AS 2870-1996 (Ref 1) and recommended by the Townsville Footings Sub-Committee (Ref 5), the typical soil profiles revealed in the test pits, and the results of laboratory testing. Whilst this is strictly only valid for residential type structures, it provides some guidance at this site for the anticipated behaviour of brittle components such as brickwork, blockwork and concrete under such seasonal variation.

It should be noted that AS 2870 provides recommended values of change in suction (Δ_u) and depth of design suction (H_s) for major and regional centres throughout Australia. Values are not, however, included for North Queensland. Based on previous experience in the area and on recently published data by Fox (Ref 6) relating climatic conditions to suction, a value of 1.2pF was adopted for Δ_u and 2.3m for H_s in the REACTIVE calculations. This is based on a temperate climatic zone.

For the site in its natural form at present ground level, the above analytical approach suggests that the area office/processing facility site should be designated Class H to E (highly to extremely reactive).

This classification is also based upon proper site maintenance being carried out in accordance with CSIRO Sheet 10-91 (Ref 7).

In the event that the site were to be raised by compacted imported non-reactive or low reactivity granular soil, then the reactivity may be reduced to Class M or less, dependent upon the thickness of compacted granular soil placed.

6.5 Erosion Protection

Due to the requirement to minimise seepage, the face of the bund walls or liner should be protected against wind and wave erosion at and close to the design water level. Such protection may comprise use of selected rip-rap or stone filled gabions/reno mattresses. In the event that waves and water velocities generated by wind are not too great, it may be possible to use well maintained and irrigated grass cover, although shrubs and small trees are to be avoided as their roots are likely to penetrate the liner or bund width, leading to excessive seepage or 'blow-out'.

6.6 Quality Control During Construction

It is recommended that earthworks and geotechnical quality control testing comprise the following components:

- subgrade inspection after excavation and during initial proof rolling;
- inspection of liner materials as they are excavated and stockpiled prior to placement on the pond floors and walls;
- verification of liner suitability by visual and tactile assessment and, where materials are considered marginal, by plasticity testing (refer Section 6.2.2 above);

- field density testing of all bulk filling, liner filling, and treated backfilling around buried pipes through embankments.

Based on general guidelines presented in AS 3798 (Ref 4) it is recommended that density testing be carried out at the following frequencies:

- constructed areas - 1 test per layer of 200mm thickness (including subgrade) per 2500m^2;
- embankment filling - 1 test per layer of 200mm thickness per 100m of embankment length;
- pipe backfill - 1 test per 2 layers per 50m^2.

Due to the critical performance aspects of material selection and compaction, it is recommended that a NATA registered earthworks testing laboratory, such as provided by Douglas Partners Pty Ltd, be retained to carry out geotechnical inspections and testing during the construction of the ponds. The laboratory should have appropriate persons skilled in the materials control requirements of this report and be fully briefed as to the design and construction philosophy.

DOUGLAS PARTNERS PTY LTD

Reviewed by:

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