



EVALUATION OF DAMAGE TO A ROAD AND SPORTS COMPLEX ON EXPANSIVE CLAYS

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ABSTRACT

The paper discusses a road rehabilitation project and a sports complex, where the underlying in situ materials were typical African black cotton soils known for their expansive characteristics. Field investigations were undertaken to discern the main causes of the pavement distress and the serious damage to roads and structures encountered at a large sports complex with the support of quantitative data from test pits and with laboratory test results. In addition, available design documents and construction records were reviewed to identify key aspects that may have contributed to the observed excessive cracking of the road pavement and damage to both roads and structures at the sports complex. The results of the investigations indicated that the in situ materials were both potentially highly active. It was concluded that either inadequate precautionary measures were taken against heave during the design phase, an indication of the lack of appreciation of the behavior of expansive clay, or that poor supervision and control during construction and commissioning negated many of the design innovations.

INTRODUCTION

Expansive clays are notorious for the damage caused to structures by their changes in volume with fluctuating moisture conditions. When a road or structure is constructed over expansive soils, these volume changes can cause extensive deformation and cracking of the road pavement or structure. The paper discusses the evaluation of the damage to a road pavement in Ethiopia and a sports complex in Botswana, where the underlying in situ materials were typical African black cotton soils known for their expansive characteristics.

CASE 1: THE ROAD REHABILITATION PROJECT

Background to the problem

The project consisted of the widening and improvement of an old distressed pavement. Within a year or so after completion of construction of the northern end of the road, longitudinal cracking in the single sealed shoulders and in the outer portions of the asphalt carriageway was observed on a section of the road. Investigations were carried out and remedial

measures were then suggested and implemented. However, a further section constructed in accordance with the new revised design which called for the replacement of the black cotton soil on either side of the main carriageway of the old road, leaving the middle part of the road from the Telford base and beneath undisturbed, developed similar problems.

The authors were then requested by the contractor to conduct a further field investigation with the overall objective of confirming the findings of previous investigations with the support of quantitative data. The findings of the previous study were that the roadbed soil was potentially active, the moisture movement and variations were high, inappropriate materials were specified and consequently used for fills and that inadequate or no measures were specified to counter the expected heave.

Investigation methodology

The investigation was conducted in two phases. The first phase was an evaluation of existing data for the project, and comprised two components: a site visit to conduct a visual assessment of the road and data gathering followed by detailed evaluation and preparation of findings in a report. A review

was conducted of all available design data that formed the basis of the pavement design, which included materials investigation reports, traffic data, pavement design reports, field and laboratory test and other as-built data necessary for evaluation of design compliance.

Figure 1 shows an example of the longitudinal cracking in the shoulder. Figure 2 shows a typical longitudinal crack in the asphalt observed along the roadway.



Fig. 1. Longitudinal cracking in the shoulder



Fig. 2. Illustrative longitudinal crack in asphalt carriageway

The second phase, which is the focus of this paper, entailed a detailed inspection and recording of the profile in six test

trenches. The field investigation was supported by the collection of moisture content samples to fully define the moisture regime within and around the pavement structure at the selected sections. Disturbed samples for material characterization (Atterberg limits, and particle size distributions, etc) were collected and while it had been envisaged to collect undisturbed samples of the in situ roadbed, this proved to be very difficult as the in-situ material simply broke along existing slickensides and fractures in its moist and weak (expanded) condition. Considering the number of investigations that had previously been done, reference was made to previous test results on undisturbed samples, which must have been the basis of the project design. In situ densities of the base and the sub base layers were also determined. In addition to field and laboratory data collection, a review of previous investigation reports was conducted.

Investigation results

Roadbed characteristics. The roadbed was predominantly black cotton soil (BCS). The X-Ray Diffraction (XRD) analysis test results on the samples showed that the most abundant clay mineral was montmorillonite, but both kaolinite and halloysite were also present. The principle exchangeable cation was calcium. Magnesium was present in significant quantities with potassium occurring in minor quantities. The parent material of the BCS was a Cenozoic basalt. Both residual and transported clays from the weathered basalt occurred on the project.

While samples were collected at different positions across the test pits, the focus was on the roadbed material within the area under the shoulder, the edge of the pavement and the carriageway, considered to be the critical positions for evaluating the conditions contributing to the prevailing problem under investigation. Samples outside these areas were also collected to understand and quantify the typical moisture variations from the edge of widening the embankment towards the centre of the pavement. In total, 116 moisture content determination tests and 36 plasticity index tests were performed. It should be noted that the investigation was carried out at the end of the wet season.

The in-situ moisture content values for all BCS samples varied between 24 and 53%. The liquid limit and plasticity index values varied between 43 and 103% and 9 and 54% respectively for all the roadbed samples. The average clay content (percentage finer than 0.002 mm) was greater than 47%, with a maximum of 73%. The CBR of the roadbed was generally less than 4 %.

Fill and replacement fill material. Volcanic ash and a tuffaceous material were used as fill for widening the road shoulders and to raise the road. Red silty clay was also used as

backfill in sections where the partial replacement technique was used as a corrective solution following the early development of cracking. The laboratory permeability of the borrow pit material for granular fill ranged between 1.4×10^{-7} and 1.6×10^{-5} cm/sec at varying degrees of compaction, with maximum in-situ dry densities ranging between 1.58 Mg/m^3 and 1.66 Mg/m^3 .

The liquid limit and plasticity index values for the replacement red silty clay were 50% and 13% for the first borrow pit material and 63% and 25% for the second borrow material. The clay content was 16% for the material from the first borrow pit and 68% for the material from the second borrow pit, with 86% and 93% finer than 0.075 mm respectively.

Pavement support material. Red cinder gravel was used as subbase with crushed unweathered basalt as base aggregate. The in-situ maximum dry density for the subbase ranged between 1.60 Mg/m^3 and 1.80 Mg/m^3 , with a degree of compaction of between 98 % and 101 %, and CBR values ranging between 35 and 60%. For the base, the in-situ maximum dry density values ranged between 2.01 Mg/m^3 and 2.38 Mg/m^3 with a degree of compaction of between 96 % and 99 % and CBR values ranging between 77 and 94%.

Test pit observations. Figures 3 and 4 show the crack propagation with depth as observed in two of the test pits. Figure 3 shows the crack at a shoulder location while Fig. 4 shows the crack propagation underneath the carriageway near the outer wheel track. The cracks pass through all of the pavement layers. While the cracks could not easily be discerned in the roadbed itself, the highly slickensided structure was indicative of the seasonal movement taking place within the clay.



Fig. 3. Crack propagation in shoulder



Fig. 4. Crack propagation in outer wheel track

Discussion

The discussion focuses on the material characteristics and general observations made in relation to the problem that was investigated. Mention is made of the material property requirements given in the project specifications, in particular with respect to the replacement material.

Roadbed. The values of the liquid limit and plasticity index were used to classify the roadbed with respect to degree of expansion and swelling potential, based on the different methods of classification presented in Chen (1988) and Nelson and Miller (1992). The roadbed was classified as having a potential expansiveness of high to very high and degree of potential swell of medium to high. The category of high and medium for the potential expansiveness and degree of potential swell respectively, applied to the section of the

roadbed material that showed less severe distress. The samples from the rest of the locations were categorized as having very high potential expansiveness and high potential swell. The general classification was therefore that of highly expansive material.

Fig. 5 shows the moisture content data point distribution within the in-situ BCS at one of the test pits. Similar moisture content data profiles were obtained for all the points of investigation.

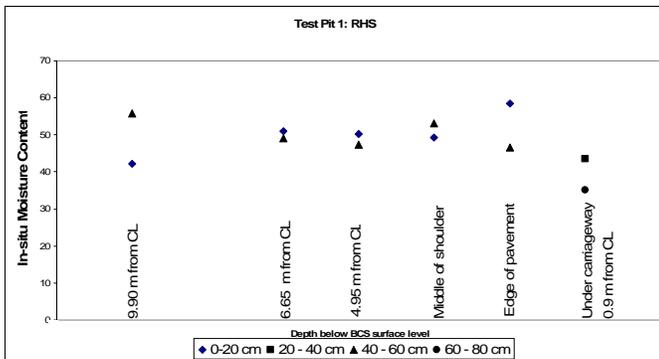


Fig. 5. Lateral moisture content distribution within the BCS roadbed

The results generally showed a variation of the moisture content. Fig. 5 shows that at a depth of about 20 cm below the BCS, the moisture content was higher at the edge of the pavement than in the shoulder.

These values were, however, higher than at the toe of the embankment. At a depth of more than 40 cm, within the BCS, the shoulder was wetter than the side of the embankment as well as the edge of the pavement. Lower moisture content values were obtained from samples collected under the carriageway. The roadbed was also generally wetter under the embankment surface than under the carriageway, suggesting a vertical moisture migration from the embankment surface towards the roadbed.

An assessment of the relationship between moisture content and plastic limit for the roadbed along the route was conducted. The role of the moisture content and plasticity properties of the soil in the behavior of a pavement on expansive soil is discussed in Haliburton (1971). The analysis showed that the sections with the most distress had roadbed moisture contents greater than 1.10 to 1.30 times the plastic limits as established in Haliburton’s study. This confirmed the contribution of moisture content and plasticity properties of the roadbed to the problem on this project.

Since the classification of the roadbed was potentially highly expansive, there was therefore the need to predict the in situ

soil movement likely to occur under the pavement. The method presented in Rao et al (1988) and the Van der Merwe chart (1964; 1975) were used in estimating the expected heave at different points along the investigated road sections. The values ranged between 24 mm and 70 mm, with the Van der Merwe method estimating higher values. These values generally were indicative of conditions requiring some measure against movement.

Fill. According to the rating for road making materials, with respect to permeability (VSRT, 2004), the materials were rated as moderate. It should be noted that homogeneous clay will have a rating of very slow and densely graded granular material will have a rating of slow, while clean gravel will have a rating of very rapid. Under the project circumstances, with an expansive roadbed, the most appropriate material for fill should have had a rating of slow to very slow to prevent moisture ingress to the roadbed.

The results of the laboratory permeability tests on the fill material showed that at a compaction of greater than 95% Mod AASHTO, it was possible for the permeability of the material from one of the borrow pits, to be reduced to acceptable levels. On the other hand, the results showed that for the material from the second borrow pit, also used in the construction, even at a compaction of 100% Mod AASHTO, under laboratory conditions, it was not possible to reduce the permeability of this material to achieve a permeability rating of slow to very slow.

Replacement fill. Table 1 shows a comparison between the specification requirements and the material characteristics of the samples obtained from the two borrow pits and the in-place fill.

Table 1: Comparison of fill materials properties against specifications.

Parameter	Specification Requirement	Material			
		BP1	BP2	Fill A	Fill B
Plasticity Index (%)	≤ 20 %	13	25	20	18
Percent < 0.075	≤ 50 %	86	93	96	88
CBR	≥10 @ 95 % MDD	7	9	3	3
CBR swell	≤ 1 %	1.33	0.24	13.0	3.0
WPI*	-	12	25	20	17

*WPI = Weighted Plasticity Index (PI x Percentage passing 0.425 mm sieve)

The requirement for a plasticity index of $< 20\%$ would ensure that the materials selected would have a low swelling potential. However, in the revised design instruction, the limit was increased to $\leq 40\%$. The requirement for $PI \leq 40$, and correcting it for the P_{425} with the data for the borrow pit and in-place fill materials gives a WPI of about 38. According to the Van der Merwe (1964; 1975) classification of intrinsic activity, the revised specification thus allowed for a material that had very high potential activity to be acceptable.

It was recommended that material with a weighted PI less than 12 can be treated as normal while those with weighted PI greater than 28 should be replaced and that material having PI values between 12 and 28 require further examination. The materials had weighed PIs between 12 and 28, requiring further investigation.

The properties of the red silty clay used as replacement fill indicate that the materials used were not adequately inert and that they would probably require countermeasures to be taken in order to minimize movement within themselves. The investigators were of the opinion that there existed a possibility that, in addition to heave from the swelling of the roadbed, heave could also have originated from the replacement fill.

Drainage control. The generally flat topography of the area under investigation required that particular attention be paid to drainage to ensure that the surface rainfall does not accumulate near the roadway. However there were sections along the route where there was water ponding alongside the road (Fig. 6). This photo was taken during the last month of the rainy season and no rain had fallen for a week.



Fig. 6. Ponding of water adjacent to road

Construction. Generally, expansive soils will shrink on drying and swell when allowed to absorb moisture. The exposure of

the roadbed during excavation followed by the placement of the fill, expectedly introduced various changes in the moisture condition. No records existed to show that placement of a barrier was recommended to protect the roadbed from experiencing moisture variation and neither was there mention in the specifications of the specific requirements in dealing with the disturbed black cotton soil surface following its excavation and preparation. Instructions to the contractor to ensure minimum exposure of the roadbed were only given after problems were first experienced.

During the course of the investigation, some aspects of the construction, for example layer thicknesses, were found to not entirely conform to the technical specifications. However, several instances were found where the layer thickness constructed was significantly higher than the specified design values. The variability of the layer thicknesses found in the course of the investigation, was not considered out of the ordinary in construction practice, and would not contribute to the development of the observed longitudinal cracks.

Conclusions

The preparation of the project specifications should to a great extent anticipate as many of the problems as possible that may occur on the project. The obviously high expansive potential characteristics of the roadbed certainly required special design and construction strategies to counter the anticipated damage due to volumetric movements of the roadbed associated with moisture changes. The review of the documents revealed that the necessary laboratory tests were performed on the roadbed, from which its expansive potential characteristics should have been identified.

Calculations could have been made using several methods to estimate the potential heave along the project route. From such an exercise, sections of the road on which countermeasures were required could have been identified. The design should then have taken into account the impact of the estimated movement for the pavement. There was no evidence that such an exercise was performed in the pavement analysis. This suggested to the investigators that special solutions for dealing with the expansive roadbed had not been adequately considered at the design phase.

Additionally, various countermeasures could have been specified to minimize the impact of the anticipated movement of the expansive soil on the pavement during construction and after construction while a moisture content equilibrium condition was being achieved. The designers could have stipulated that this equilibrium moisture content be maintained during and after construction as closely as possible.

The evaluation showed that there was a lack of appreciation of the impact of the roadbed characteristics on the performance

of the pavement in the original design and during the early phases of the project until problems were first noted.

CASE II: SPORTS COMPLEX

Background to the problem

This case study relates to the construction of a large sports complex and stadium in Botswana. The complex is built on transported soils derived from surrounding weathered Mesozoic basalts. During the original site investigation, the potential problems with the site were recognized, but the client was adamant that this site should be used for the sports complex. For this reason a number of additional specialist experts were consulted during the design phase. Despite the added costs involved, the structure for the stadium and the athletic track were constructed on piles and these currently show no indications of distress or excessive differential movement.

The complex, however, also includes administration buildings, change rooms, store rooms, tennis courts, boundary walls, etc. Despite the design engineers taking specific precautions to minimize potential damage to these structures, nearly all of them illustrate distress of various forms and degree. The typical precautions taken were, among others, to place structures on soil rafts, to provide vertical moisture barriers in the form of plastic sheeting at the edges of the rafts, to provide flexible joints for wet services, to provide concrete aprons around buildings, to construct reinforced concrete groundbeams and ringbeams above windows and to pay attention to the control of stormwater on the site. Generally single storey structures were not articulated to cater for differential movements within the structures.

It is evident that the precautionary measures had an impact although the extent of differential movement of the foundations has a significant impact on many of the buildings. The installation of the concrete ring beams reduced the tendency for corner cracking but led to cracking parallel to them (Fig. 7) and between the groundbeam and window-level ringbeams. Corner cracking and movement of the ceilings, however, affected internal walls (Fig. 7), window frames and many of the floors were cracked.

Although the wet services made use of flexible joints, the placement of concrete slabs and block paving against them “locked” them in and resulted in failures and subsequent leakage (Fig. 8).

Other facilities such as tennis courts were placed on soil rafts. Despite this, the edges of the courts were subject to movement and cracking (Fig. 9).



Fig 7. Cracking parallel to ring beam (above window) and on internal wall



Fig 8. Damage to wet services as a result of leaking paving



Fig 9. Damage to tennis court – note distortion of fencing supports

Unfortunately, there also appeared to be inadequate liaison between the engineers and the landscape gardeners, resulting in a lot of the good design being negated by conflicting irrigation design. The later placement of the irrigation system for the gardens resulted in the addition of significant volumes of water adjacent to and on the concrete and block aprons surrounding the structures, leading to differential movement and cracking. Similar problems were related to the location of the drainage systems for the air conditioning units added to the buildings.

Investigation methodology

An initial site inspection was carried out during February 2004 (near the end of the wet season). No testing or material sampling was done but discussions were held with various stakeholders and the damage to the structures was inspected.

The follow-up fieldwork was undertaken in June 2005, 2 to 3 months after the end of the wet season. Four test pits were excavated within the boundaries of the stadium complex to various depths between 1.7 and 3.4 m.

The test pits were positioned to obtain information on the natural in situ subsoils as well as the nature and thicknesses of the founding materials for various structures. The depth to and nature of the bedrock could not be ascertained from the pits (mechanically limited to about 3 m depths) but it had been determined during earlier investigations that expansive clays existed to a depth of between 6 and 8 meters overlying residual sandstone and sandstone bedrock at depths up to about 15 meters. The subsoils exposed in the test pits were examined and logged.

Representative disturbed and undisturbed soil samples were collected from all test pits for testing in a local laboratory. The laboratory testing included Atterberg limits, particle size distribution, moisture content and oedometer testing on selected materials.

Investigation Results

In Situ Moisture Content. Fifteen samples were taken at various depths from the test pits for the determination of the in situ moisture content using standard gravimetric methods and the results are summarized in Table 2.

Test Pit	Depth (m)	Moisture content (%)
TP 1	0 – 0.20	14.6
TP 1	0.20 – 0.30	16.5
TP 1	1.0	19.8
TP 1	2.0	18.6
TP 2	0 – 0.50	4.0
TP 2	0.50 – 1.20	21.6
TP 2	1.20 – 2.50	18.2
TP 3	0 – 0.25	10.8
TP 3	0.25 – 1.20	18.6
TP 3	1.20 – 3.00	18.5
TP 4	0.20 – 0.30	16.6
TP 4	0.60 – 0.80	21.2
TP 4	1.20 – 1.30	16.4
TP 4	1.30 – 1.40	32.9
TP 4	1.70	29.2

The in situ moisture contents varied between 10.8 (in the surface materials) and 32.9 per cent, with one result of 4 per cent recorded at TP2. This low moisture content was determined on material imported as “inert fill” for the soil raft and consisted of crushed rock quartzite and soil. This test pit was excavated along the line of a tennis court fence.

In general the moisture contents tend to increase with depth as would be expected from soils sampled during the early stages of the dry season. The wettest soil was recorded at TP4 which was excavated one meter away from the foundation of the Administration building. The pit was located on the south side of the structure in an area that received little direct sunlight during the day. Fig. 10 illustrates the relationship between the moisture content and depth.

Table 2. Natural moisture content

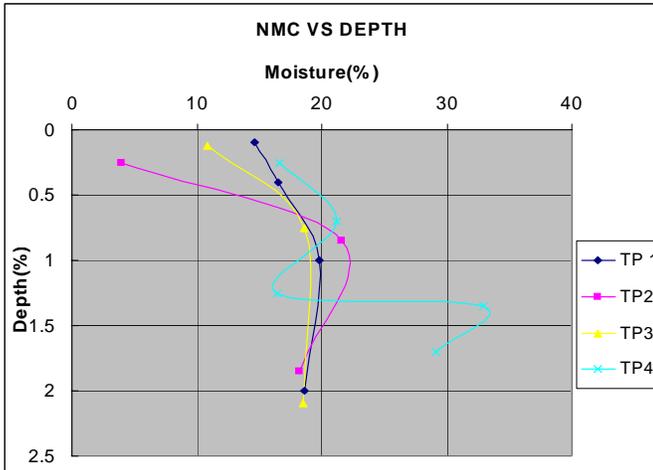


Fig. 10. In situ moisture content versus depth

The “equilibrium” moisture content (into the dry season) appears to be about 18 to 20 per cent. The wet material from TP4 had a moisture content of about 30 per cent, which is probably close to the equilibrium moisture content during the wet season but is somewhat lower than the data determined for the road in Ethiopia (Case I) where the average moisture content at a depth below one meter was closer to 50%. This is in line with the generally higher Liquid Limits and Plasticity Indices determined on the Ethiopian materials. The moisture content of the oedometer samples after full saturation varied between 30 and 44 % with a mean of 38 %, indicating that volumetric changes would typically occur on site as the moisture fluctuated between about 18 and 30 %, unless full saturation at depth occurred and was constant, which is unlikely.

Gradings and Indicator Tests. Ten full indicator tests comprising Atterberg Limits, bar linear shrinkage and particle size analyses to two microns were carried out on the disturbed samples recovered at varying depths, from the test pits.

The particle size distribution curves are given in Figs. 11 (TP1&TP2) and 12 (TP2, TP3 & TP4).

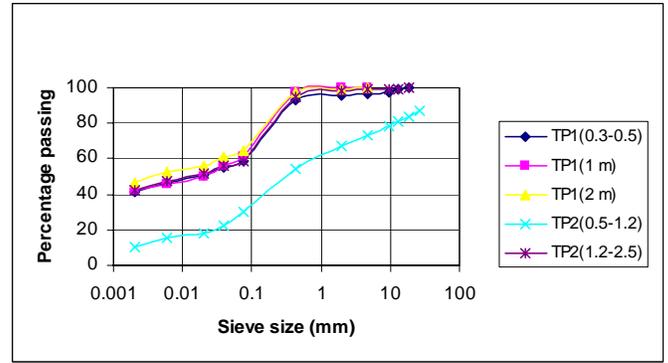


Fig. 11: Particle size distribution of test pit samples

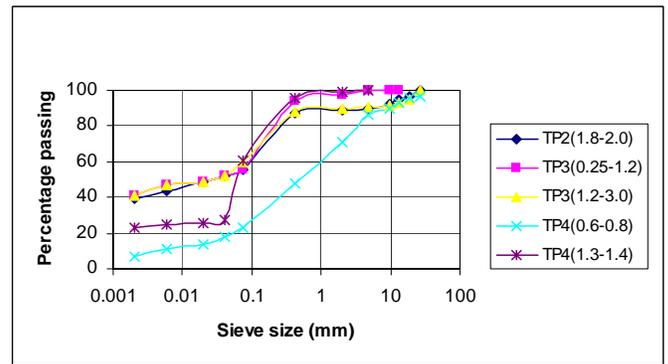


Fig. 12: Particle size distribution of test pit samples

The results show that except for the imported materials from TP2 and TP4; which were classified as silty or clayey gravel and sand; the rest of the materials tested were classified as clayey soils with greater than 35 per cent passing the 75 micron (No. 200) sieve. The materials are classified as A-7-6 using the PRA Soil Classification System.

The Liquid Limits of the samples vary between 40 and 65 %. The Plasticity Index of the residual clays from test pit TP1 and TP2 ranges from 26 to 29 %. Material extracted from TP3, which was essentially undisturbed natural ground situated away from any infrastructure development, had a plasticity index at 1.2 m of 30 %, while at 2.4 m the Plasticity Index was 38 %. Material from the surface down to 0.8m in TP2 and TP4 which was imported (soil raft) had a Plasticity Index of 13 %.

In addition to these tests, two small samples of material from TP3 and TP4 were tested by the CSIR soils laboratory. These gave Plasticity Indices of 47 and 26 % respectively, which compared better with the results of one of the independent experts.

The Atterberg limit results and moisture contents are plotted against depth on Figs. 13 to 16.

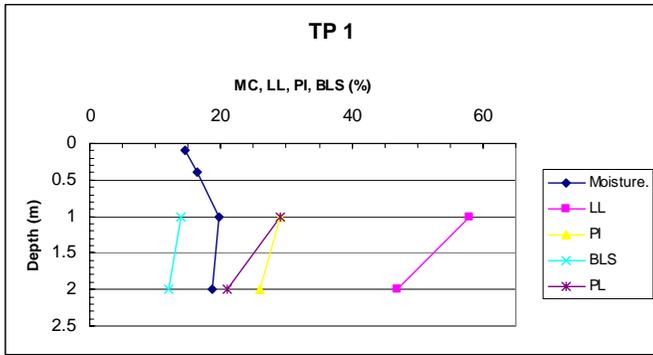


Fig. 13: Moisture content and Atterberg limits versus depth for TP1

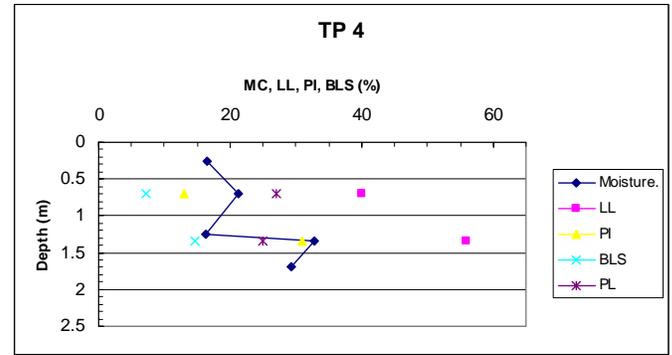


Fig. 16: Moisture content and Atterberg limits versus depth for TP4

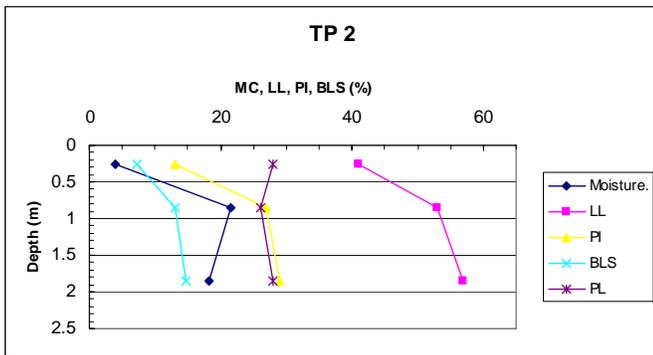


Fig. 14: Moisture content and Atterberg limits versus depth for TP2

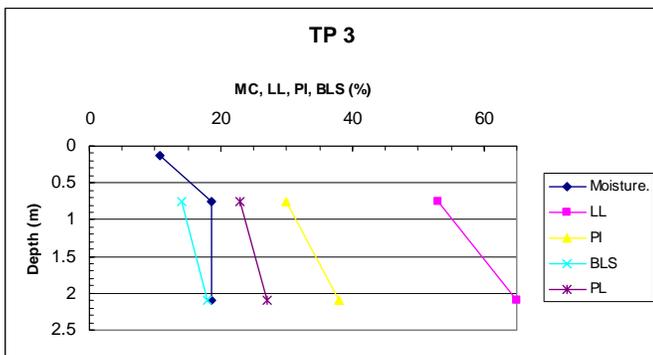


Fig. 15: Moisture content and Atterberg limits versus depth for TP3

Oedometer Tests: Potential Expansiveness/Heave: Block samples from three test pits, namely TP1, TP3 and TP4, were collected for oedometer tests. Three methods were used to predict the heave potential of the sampled materials, namely, the Brackley (1975), Weston (1978) and Van der Merwe (1964; 1975) methods. The results are summarized and given in Table 3.

Brackley 1 and 2 are the full model as published and a modified version used in the original geotechnical report respectively.

The oedometer heave predictions vary between 0.2 and 6.7 % with small differences resulting from the differing loading pressures. Two of the results are very low (0.2 and 1.5) and are attributed to the sampling technique. The slickensided nature of the material made collection of block samples by cutting in situ material almost impossible and eventually large blocks were excavated using the back-hoe and selected from the material removed. It is suggested that the low values have been obtained on material cut perpendicular to the in situ horizontal position, reducing the effect of the horizontal layering of the expansive clays. Excluding these values the predicted swells are between 4.6 and 6.7 per cent.

Swells have been predicted using the methods used in the original geotechnical report for comparative purposes. Although there are numerous models available, these three are those commonly used in South Africa, although they all have specific weaknesses. As discussed previously, the moisture fluctuations on site would probably range between 18 and about 30 per cent. This would influence the actual maximum swell/heave which is typically predicted without taking the actual moisture content range adequately into account.

Table 4. Swell Potential from Oedometer tests and predicted Heave potential

TP No	Swell (%) ¹	Load (kN)	Depth (m)	Brackley 1 (%) ²	Brackley 2 (%) ³	Van der Merwe (%)	Weston (%)
TP1	5.8	25	1.0	27	1.5	2.7	4.5
TP1	6.5	40	1.0	24	1.5	1.9	1.3
TP3	1.5	25	1.5	34	8.3	5.1	2.8
TP3	6.7	25	1.5	34	8.3	5.1	2.8
TP3	4.6	50	1.5	29	8.3	3.1	2.2
TP3	0.2	50	1.5	29	8.3	3.1	2.2
TP4	4.9	25	1.3-1.7	9	0	2.7	0.6

1 – Oedometer swell at load shown in column 3
 2 – Brackley Model : Swell = (32.4 - 0.85moisture) (5.3 - 2.77 e - log10p)
 3 - Simplified Brackley Model : Swell = (.525PI + 4.1 - 0.85moisture)

Comparison of Results: Comparison of the results obtained with those provided in the original Geotechnical Report for the project indicate somewhat higher plasticity indices (26 – 29% compared with 16.5 – 25%) although the CSIR results were significantly higher (26 – 47%). It is interesting that a number of samples obtained during one of the earlier investigations from between 1 and 6 meters depth had plasticity indices in the range 29 – 52%, considerably higher than any of the other measurements. This is indicative of the problems with the accurate determination of the Plasticity Index (Sampson, 1984).

The percentages passing 0.075 mm provided in the original Geotechnical Report were in the range 49 - 76 compared with the 55 - 66 per cent obtained during this investigation. No determinations of the clay content were carried out in the earlier work but this testing indicated that about 40 per cent of the material was generally finer than 0.002 mm.

Plots of the results of the indicator tests on the modified Williams Activity chart showed that all of the materials except the imported fill had a high potential expansiveness with one material being very high (Fig. 17).

The very high predicted swell values determined using Brackley 1 result from the low void ratios (0.6 - 0.7) associated with such highly over-consolidated materials, compared with the values in excess of 1.0 used to determine the model. Brackley 2 predicts swells of 1.5 - 8.3%, Van der

Merwe of 1.9 - 5.1% and Weston of 0.6 - 2.8%. Despite the problems associated with using such empirical methods, the results are generally within the 2 – 7% predicted in the original geotechnical report and the 4 – 5% predicted later by other experts.

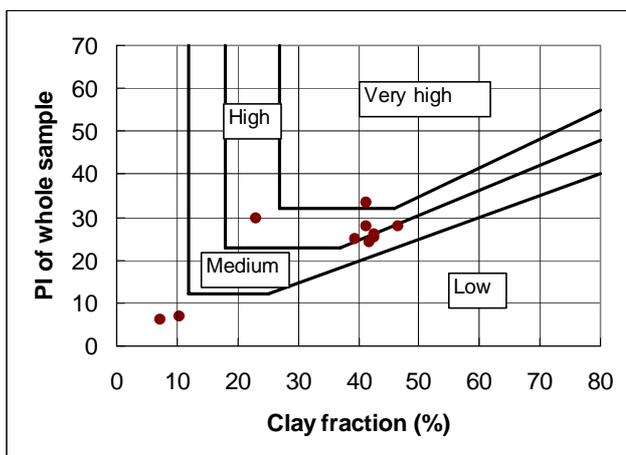
Fig. 17: Plots of predicted heave using modified van der Merwe chart

The imported inert fill used for the soil rafts was specified to have a maximum plasticity index of 10%. The two fill materials tested had plasticity indices of 13%, and were thus not compliant with the necessary requirements, possibly compounding the problem. In addition, the field investigation indicated that the rafts were adequately deep (1.5 meter) but only extended 1 meter beyond the footprint of the buildings: this was considered to be inadequate as heave of the edges of the relatively stiff raft certainly resulted in differential uplift of the edges of the buildings.

Aspects such as the placement of the block paving and/or concrete slabs adjacent to the wet services, the location of irrigation systems and drainage of water from air conditioning units need to be carefully considered. These measures are usually implemented at the end of the project under subcontract by people without a full understanding of the consequences of the actions.

Conclusions

From the limited additional soil investigation carried out, it is clear that the material, as advised by the original geotechnical consultants and experts, would not provide suitable founding conditions for structures and significant movement of structures could be expected, even after taking precautionary measures. A number of precautions and countermeasures were recommended and these were



generally followed with the resulting damage to structures being mostly within that predicted. Despite best-practice procedures in terms of the geotechnical investigation being generally followed, the remedial works necessary on various parts of the complex have resulted in delays in take-over from the contractor, withheld payments and disputes as to who should pay for remedial works. Six years after the Certificate of Practical Completion being issued, the complex has yet to be properly commissioned.

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