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Power Station Foundations in Deep Expansive Soil

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On behalf of ESCOM - Lethabo Ad Hoc Foundation Committee

SYNOPSIS The site for a large (3 600MW) thermal power station is underlain by deep expansive soils which posed difficult conditions for the design of stable foundations. After describing the soil and ground-water conditions and attendant foundation problems, the paper goes on to describe how the amount and rate of heave were predicted. Most of the foundations are piled, but the piles will be subject to uplift as the soil swells. Large scale tests to acquire information for the prediction of uplift forces are described, as well as measures for reducing the uplift forces.

SITUATION, GEOLOGY AND SOILS

The Lethabo power station, currently under construction, will be coal fired and have an installed capacity of 3 600MW (6 sets each of 600 MW). It is situated adjacent to the Vaal river (the boundary between the Transvaal and Orange Free State provinces) near the town of Vereeniging (27°S, 28°E). The climate is temperate and semi-arid, having an average annual precipitation of 650mm. The average annual evaporation from a free water surface is 1500mm. The elevation of the site above sea level is 1 450m. The temperature ranges from 0°C to + 35°C.

The site is underlain by horizontally bedded sedimentary rocks of the Vryheid formation of the Ecca sub-group forming part of the Karroo supergroup. (These rocks were laid down 200 to 300 million years ago).

The site is crossed by a paleochannel of the Vaal river, and soils residual from Vryheid formation rocks are overlain by river alluvium over part of the site. Over the remainder of the site residual soils extend almost to the surface. Figures 1 and 2 show typical soil profiles at the West of the site (where there is no alluvium) and at the East (where a considerable deposit of alluvium exists).

As indicated in Figures 1 and 2, the composition of the alluvium varies considerably from bands, layers and lenses of clean sand to stiff and very stiff fissured and shattered claysands. The clay content of the alluvium generally decreases with depth and several metres of sandy material overly the siltstone to the east of the site.

Residual soils are mainly weathered siltstones and occur as stiff to very stiff fissured and shattered clayey silts. The residual siltstone grades with depth to very soft rock and the colour changes from banded orange and light grey to yellowish brown and brownish grey where it grades into the underlying carbonaceous shale.

The weathered carbonaceous shale is a dark grey intensely laminated very soft rock with a high mica content. Both the siltstone and the micaceous shale disintegrate or slake rapidly on exposure to the atmosphere. The underlying sandstone may have a capping of light grey slightly weathered hard rock varying in thickness up to 0,5m. Below the capping (if present) the sandstone is softer and can be classified as soft to medium hard rock. The sandstone contains occasional bands of very soft rock and dense residual sand.

GROUND WATER CONDITIONS

The area surrounding the power station site is partly covered by eucalyptus plantations and partly by natural grass cover or cultivated maize lands. A number of swampy areas, seepage zones and springs occur in the vicinity and appear to result from the seasonal formation of perched water tables over clayey zones in the alluvium. It is known that the water table is generally lower under an area of established eucalyptus trees because of the efficient transpiration and deep root zones of these plants. (Blight and Lyell, 1984).

Two distinct piezometric surfaces were found to exist under the site, as shown on the east-west section in Figure 3. The piezometric level in the sandstone is above that in the residual siltstone over that portion of the site where alluvium is absent. The piezometric levels in both horizons drop suddenly and merge under the eastern portion of the site where the residual siltstone has been eroded away and largely replaced by alluvium. It appears there is a general flow of ground-water in a easterly direction. Where the sandstone is overlain by shale and siltstone, it acts as a confined aquifer under an artesian head of about 7m. Where the siltstone and shale have largely been eroded away, the sandstone appears to behave as an unconfined aquifer.

The reason for the depression of the water table over the eastern portion of the site has not yet been satisfactorily explained. It is likely

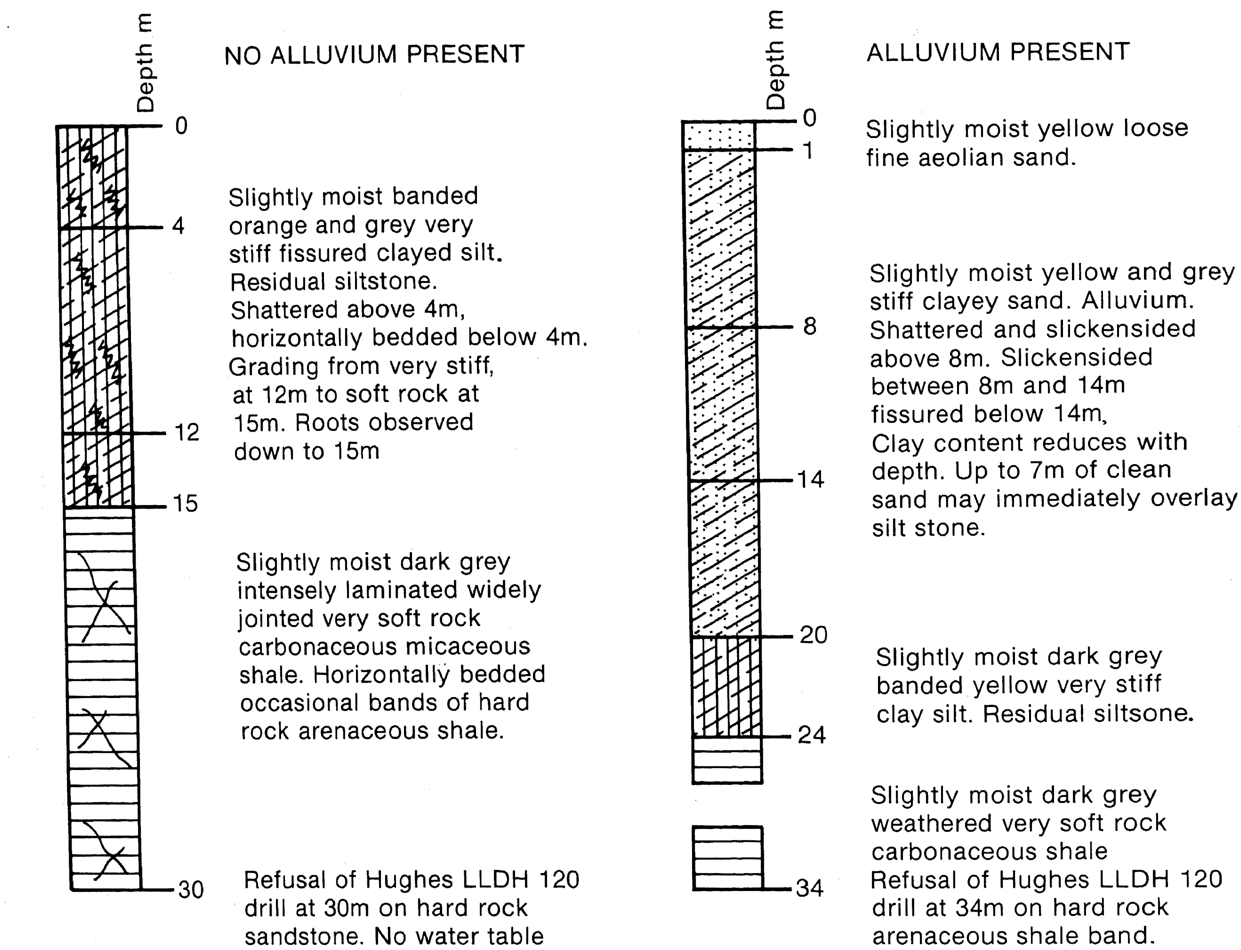


Fig.1. (left) and 2. (right) Typical soil profiles at site of Lethabo power station.

that the depression results from a combination of the following factors :

(i) Further to the east the alluvium may be in direct contact with the sandstone and may act as an underdrain draining away the limited flow of water emerging from the sandstone and thus depressing the piezometric surface in both the sandstone and the overlying siltstone.

(ii) Co-incidentally, the part of the site underlain by alluvium was densely covered by eucalyptus trees prior to site clearing. Tree roots have been found to depths of 15m, even in the residual siltstone. It is likely that transpiration by the trees has contributed to the depression of the water table.

As indicated by the description of the soil profiles (Figures 1 and 2), the soils above the water table are desiccated to a considerable lepth. When the site was cleared, the desic- :ating effects of vegetation were removed, and ıs construction proceeds and the power station :omes into operation, the moisture content of :he soil can be expected to increase with the result that heave will occur.

It is expected that the depth of the water tε over the western portion of the site will not vary by very much. However, there is at present (1983) no certainty as to what will hapı to the water table over the eastern portion c the site. One possibility, the worst scenar: from the point of view of foundation design, that the water table will rise and equalize ı that to the west. A more optimistic possibi: is that the state of desiccation in the clay alluvium will be relieved, that isolated per water tables will develop over clayey horizo within the alluvium, but that the underdrain: action of the sandy alluvium base will preve the permanent water table from rising signif: cantly.

GEOTECHNICAL PROBLEMS

The operation of power station plant and ins lations is particularly vulnerable to differ tial movement of its foundations. Hence vir tually all plant and building foundations at Lethabo are (or will be) piled. In general, bored cast-in-situ reinforced concrete piles are being used throughout the station. Many these piles carry extremely heavy loads (e.g boiler house and turbo-alternator foundation while others (e.g. induced draught fan found tions) carry relatively light loads.

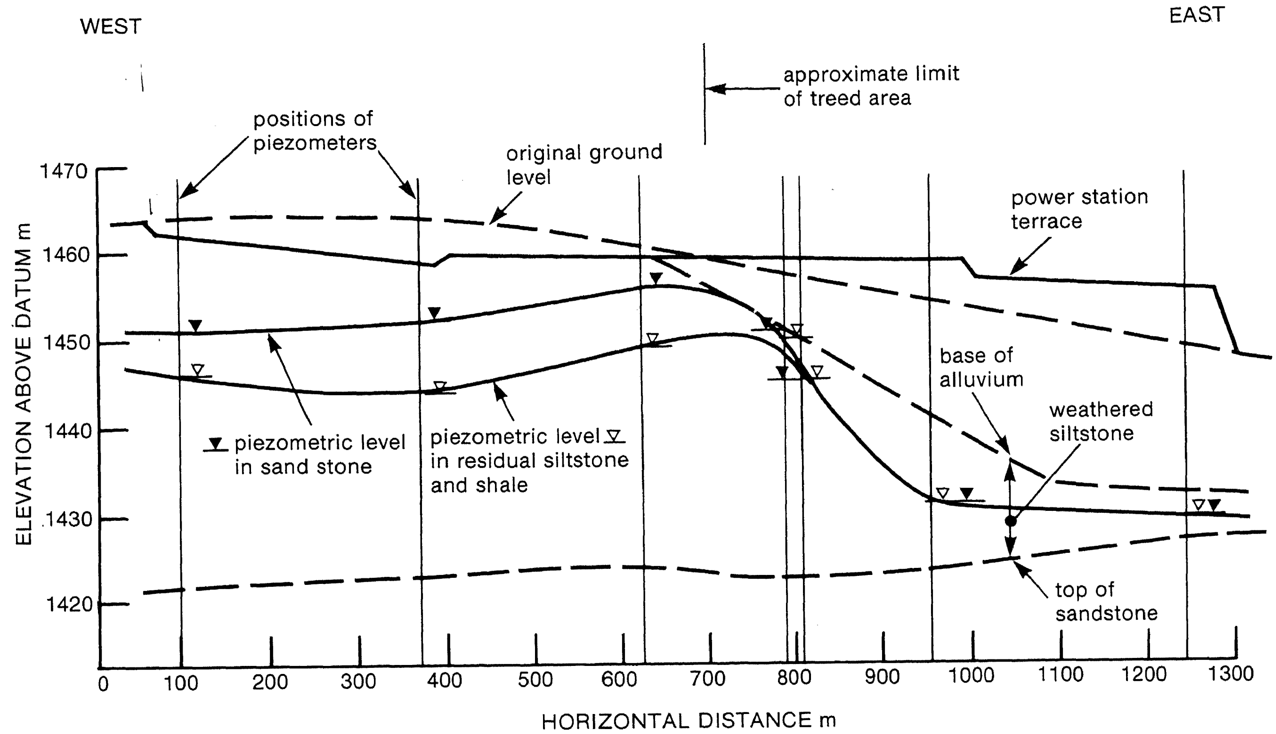


Fig.3. East-west section through site showing depression of water table under trees.

Design for heavy loads is not problematic as piles can be socketed into the carbonaceous shale at an allowable shaft friction of 100kPa, or alternatively can bear on the sandstone at pressures varying up to 7 500kPa. However, problems arise with the design of lightly loaded piles and piles that, because of the construction sequence, will be installed two to three years prior to receiving their full dead load. As the desiccation of the residual siltstone and the clayey alluvium is relieved, these piles will become subject to frictional uplift and therefore have to be designed as tension anchor piles.

Apart from the design of the piles, the expectation that the site will be subject to a general heave means that installations such as circulating water ducts and cooling towers have to be supported on piles to prevent distortion from differential foundation movement, and then have to be isolated from the effects of heave by creating suitably sized voids between the underside of the pile caps and the underlying soil.

PREDICTION OF THE AMOUNT AND RATE OF HEAVE

It was important to predict both the amount and rate of heave of the site. The amount of heave was required to assess, amongst other things :

- the dimensions of voids to be provided under pile caps;
- the extent to which shear strength would be mobilized on pile shafts; as well as to design the foundations for minor structures on the site.

The rate of heave was required to assess the rate of increase of uplift forces on piles, whether economies could be obtained by rescheduling the piling so that where possible, piles would only be installed after partial heave of the profile had occurred, etc.

Amount of Heave

Rather than use one of the empirical or semi-empirical methods of predicting heave (e.g. Jennings and Knight, (1957), Brackley (1975)), it was decided to predict on the basis of expected changes of effective stress in the profile and rebound parameters measured in terms of effective stresses. The prediction was thus carried out in a similar manner to that of a conventional settlement analysis.

Initial effective stresses in the profile were estimated from swelling pressures measured in the oedometer apparatus :

After setting up the specimen in the oedometer, the swelling pressure was estimated by inundating the soil and then adjusting the vertical applied stress to maintain a condition of zero vertical strain. The equilibrium stress under this condition was taken to represent the in-situ vertical effective stress. Figure 4 summarizes the results of these determinations.

The results were very scattered and many indicated that the vertical effective stress in the soil equalled the total overburden stress i.e. that the soil suction was zero. Nevertheless, sufficient specimens indicated the existence of an excess swelling pressure to enable a reasonable "worst case" estimate to be made of the initial effective stress in the profile.

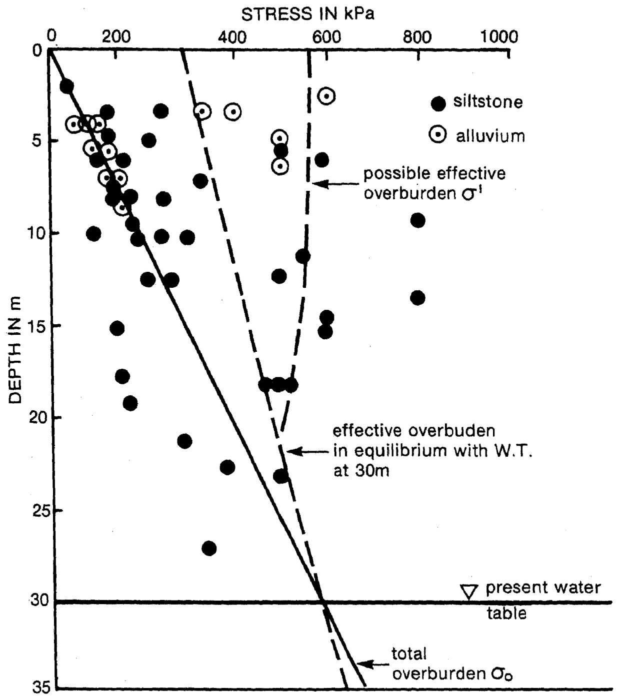


Fig.4. Measured effective stresses in soil specimens from Lethabo.

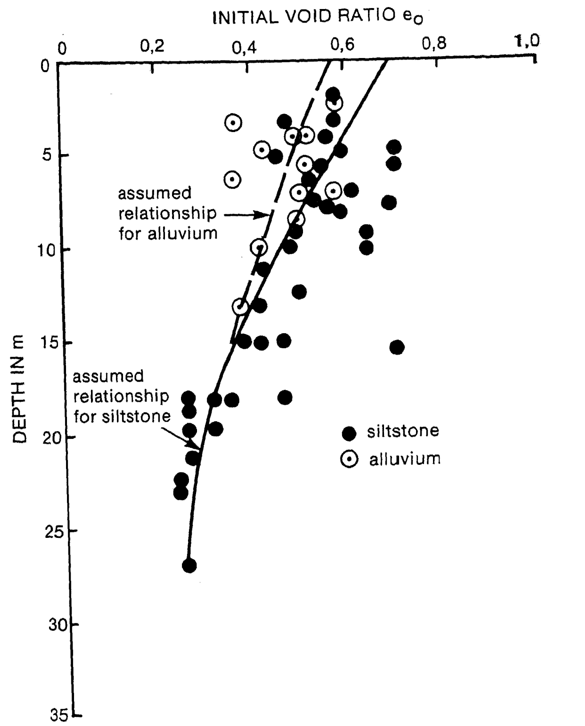


Fig.5. Relationship between initial void ratio and depth for Lethabo.

The other parameters necessary for the heave analysis were the initial void ratio e_o in the profile and the swell index C_s, i.e. the slope of the void ratio-log pressure rebound curve. Figures 5 and 6 summarize measurements of the initial void ratio and the swell index.

Because of the difference in water table depths and profiles, the eastern and western halves (see Figures 1,2 and 3) of the site had to be treated speparately. The idealized effective stress diagrams on which the heave calculations were based are shown in Figure 7. Over the western side of the site, heave is expected to occur as a result of alleviation of desiccation with the water table remaining approximately constant at 16m. Over the eastern half it has been assumed that not only will the desiccation be relieved, but also that the water table will rise from - 25m to reestablish an equilibrium position at - 11m. The amounts of heave calculated on this basis are

western side of site : heave = 120mm
eastern side of site : heave = 85mm

Rate of Heave

The rate of heave depends on the rate of accumulation of moisture in the profile. Over an extensive area like a power station site with a deep water table, the major source of moisture recharge is infiltration from the surface, with upflow from the water table making a minor contribution. The time for recharge to occur can be assessed by calculating the time necessary for available recharge to fill :

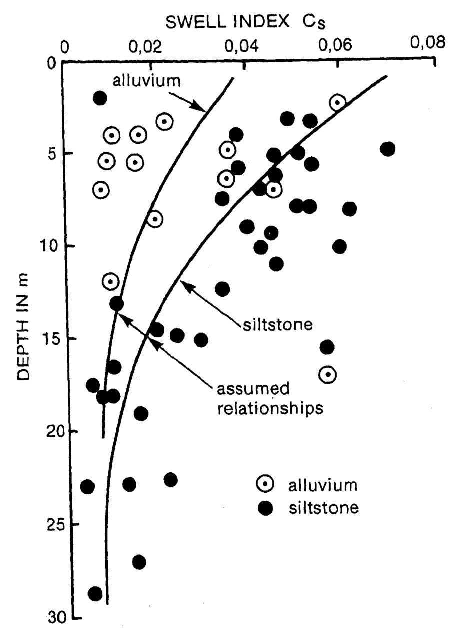


Fig.6. Relationship between swell index and depth for Lethabo.

(a) air-filled pore space in the profile above the water table; and

(b) additional void space created by swelli

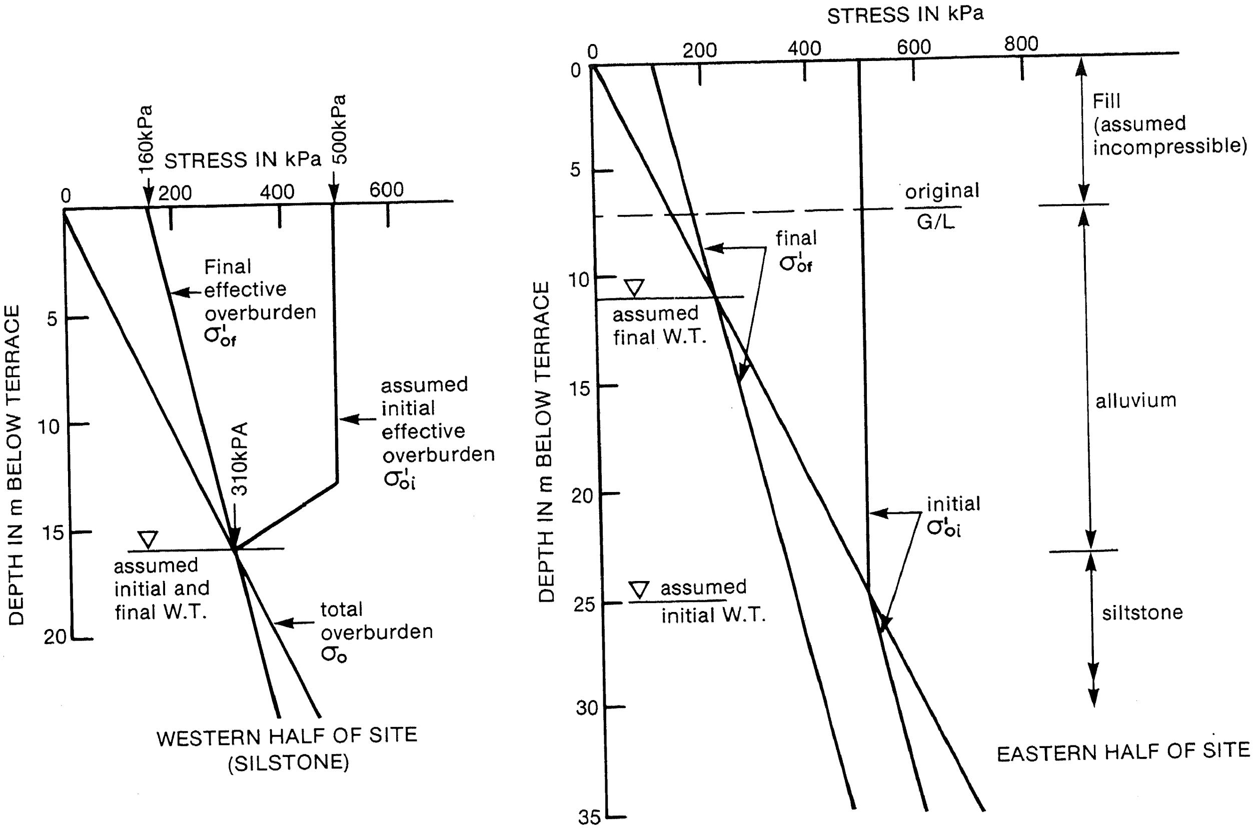


Fig. 7. Idealized effective stress diagrams for calculation of surface heave.

The additional air-filled void space per unit volume of soil is

$$V_{a}/_{V} = e\,\frac{(1-S)}{1+e}$$

while the pore space resulting from swell is given by

$$V_{s}/_{V} = \frac{\Delta e_s}{1+e}$$

Figure 8 shows the distribution of $V_a/_V$ and $V_s/_V$ in the profile at Lethabo. It is clear from the diagram that the void space created by swelling is a negligible component of the total void space available for recharge.

The time t taken for the available recharge to fill this void space is calculated by dividing the pore space by the recharge rate, i.e.

$$t = \frac{V_a/_V + V_s/_V}{\frac{1}{V}\cdot\frac{dV_r}{dt}}$$

in which $\frac{dV_r}{dt}$ is the rate of recharge.

In this procedure, it is assumed that the recharge enters the profile in the form of a sharply defined wetting front (e.g. Morgenstern and de Matos, (1975)) and hence that heave

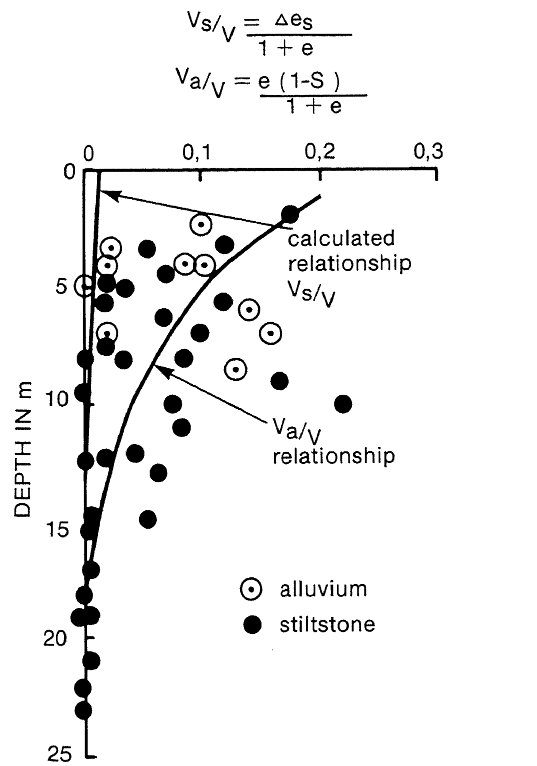
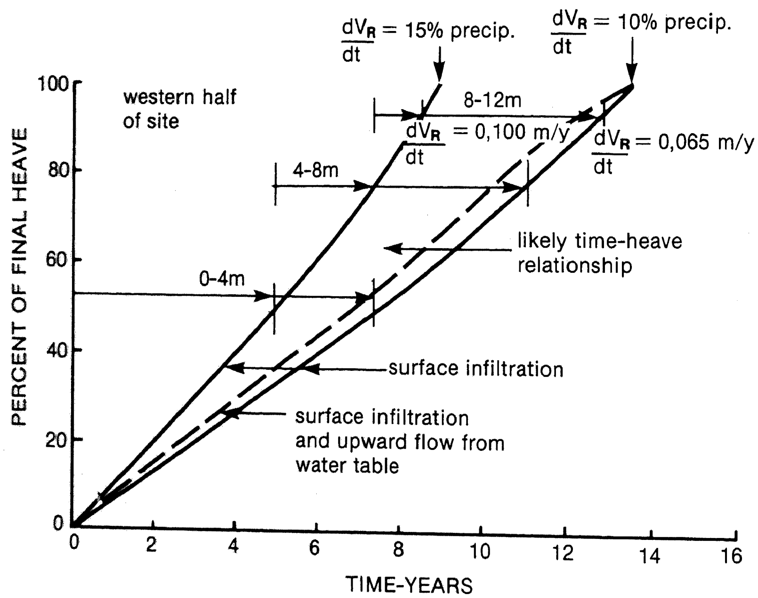


Fig.8. Relationship between unsaturated void space and depth at Lethabo.

largely proceeds from the surface downwards. The biggest uncertainty in the procedure lies in assessing the recharge rate dVr/dt. In the case of Lethabo, it was assessed at between 10% and 15% of the mean average precipitation of 650mm. However, quite apart from the correctness of the proportion, one or two exceptionally wet or dry years could have a profound influence on the prediction.

If the shear strength parameters of the expansive soil are uniform with depth, equation (6) becomes

$$P = \Pi D\ (c'z + K\sigma_v' \tan\phi') \qquad (7)$$



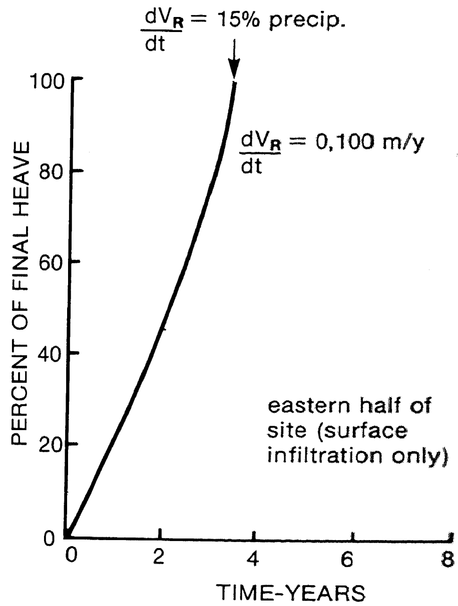


Fig. 9. (left) and 10. (right) Predicted time-heave curves for Lethabo.

Figures 9 and 10 show the time-heave curves predicted for the western and eastern sides of the Lethabo site. Figure 9 (western) illustrates the effect of uncertainty as to the actual infiltration rate as well as the small size of the contribution of upward flow from the water table.

UPLIFT FORCES ON PILES

It was decided to calculate the uplift forces on piles by means of the effective stress approach originally proposed by Collins (1953). If the shear strength of the soil in contact with the pile at a given depth is

$$\tau = c' + K\sigma_v' \tan\phi' \qquad (4)$$

in which c' and ϕ' have their usual meaning, σ_v' is the effective overburden stress and K is the product of the ratio of the horizontal to vertical effective stresses at that depth and a factor for adhesion between the pile shaft and the soil. The increment in uplift force ΔP over a length of dz of pile of diameter D is

$$\Delta P = (c' + K\sigma_v' \tan\phi')\ \Pi D dz \qquad (5)$$

and the total uplift force is

$$P = \Sigma\Delta P \text{ (over the length of pile affected by heave)} \qquad (6)$$

DEVELOPMENT OF UPLIFT FORCE WITH TIME

Equations (5) and (6) can be used in conjunction with the time heave curve and typical shear stress-deformation curves to predict the development of uplift force on a pile with time: The sequence of operations is as follows :

(i) At time 1, assess the movement of the soil relative to the pile by means of the time-heave curve.

(ii) From the shear stress-deformation curve assess the shear stress developed up the length of the pile shaft.

(iii) Apply equations (5) and (6) to assess the uplift force.

(iv) Repeat for time 2.

The results of such a set of calculations for a 1m diameter pile in residual siltstone at Lethabo are shown in Figure 11. This curve will be referred to again later in the paper.

PLUG PULLING TESTS

The two questions that arise in applying equations (5) and (6) are :

(a) What values of the parameters c' and ϕ' are appropriate, and

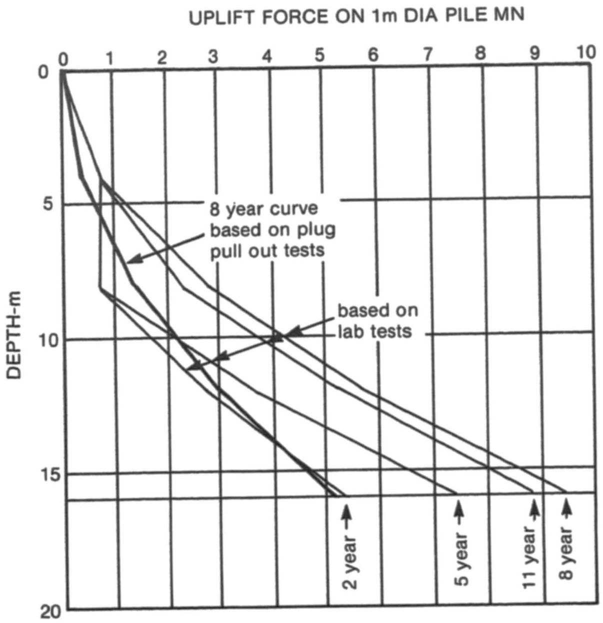


Fig.11. Predicted uplift forces on a 1m diameter pile resulting from soil heave.

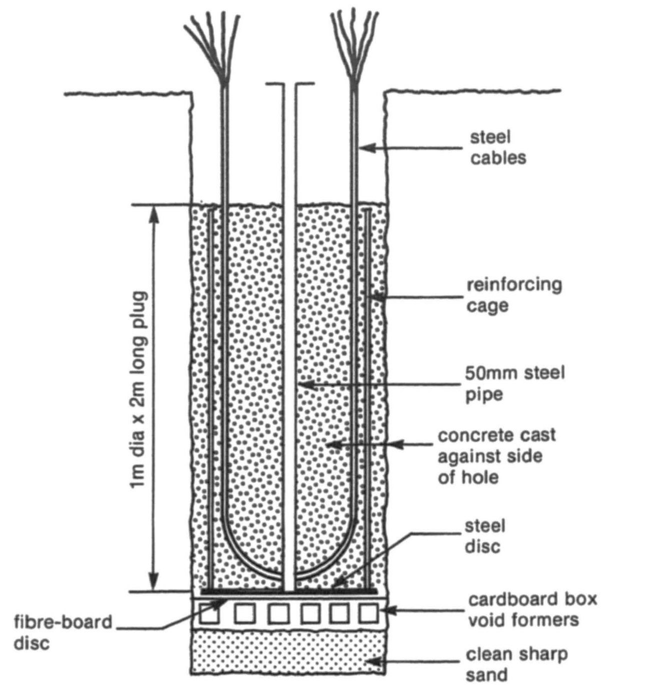


Fig.12. Construction of concrete test plugs.

(b) What value of K should be used ?

Experience with large-diameter bored cast-in-situ piles in saturated stiff fissued clays (e.g. Burland et al (1966) and O'Riordan (1982)) have shown that the appropriate strength to use in design corresponds to the lower limit of laboratory strength measurements, while 1,0 is an appropriate value of K.

Experience in South African of small diameter (up to 225mm) piles (Donaldson, 1967), has also indicated a value for K of 1,0.

However, as the circumstances and pile sizes at Lethabo are very different to those dealt with by other investigators, it was decided to carry out two series of large scale plug pulling tests, one in residual sandstone and one in alluvium, to establish values for shear strength and lateral pressure coefficients that would be appropriate to Lethabo.

Dimensions of 1 050mm diameter by 2m long were chosen for the plugs, which were constructed as shown in Figure 12. The object of the cardboard void formers was to ensure that no adhesion developed between the bottom of the hole and the plug, while the 50mm diameter steel pipe served the dual function of enabling the movement of the plug to be measured during pulling and of preventing any suction developing at the base of the hole.

The soil surrounding one of the plugs was kept at natural water content while that around the remaining plugs was soaked by filling the hole to a depth of 0,5m above the top of the plug as well as maintaining the central 50mm diameter hole full of water. The period of soaking was 3 to 4 weeks, depending on when the plugs were tested.

The pulling operation was achieved by jacking up a beam around which the cables from the plug were fastened. Figure 13 shows the arrangement of the jack and beams during the course of a pulling test, while Figure 14 illustrates

Fig.13. Arrangement of jack & beams for plug-pulling tests.

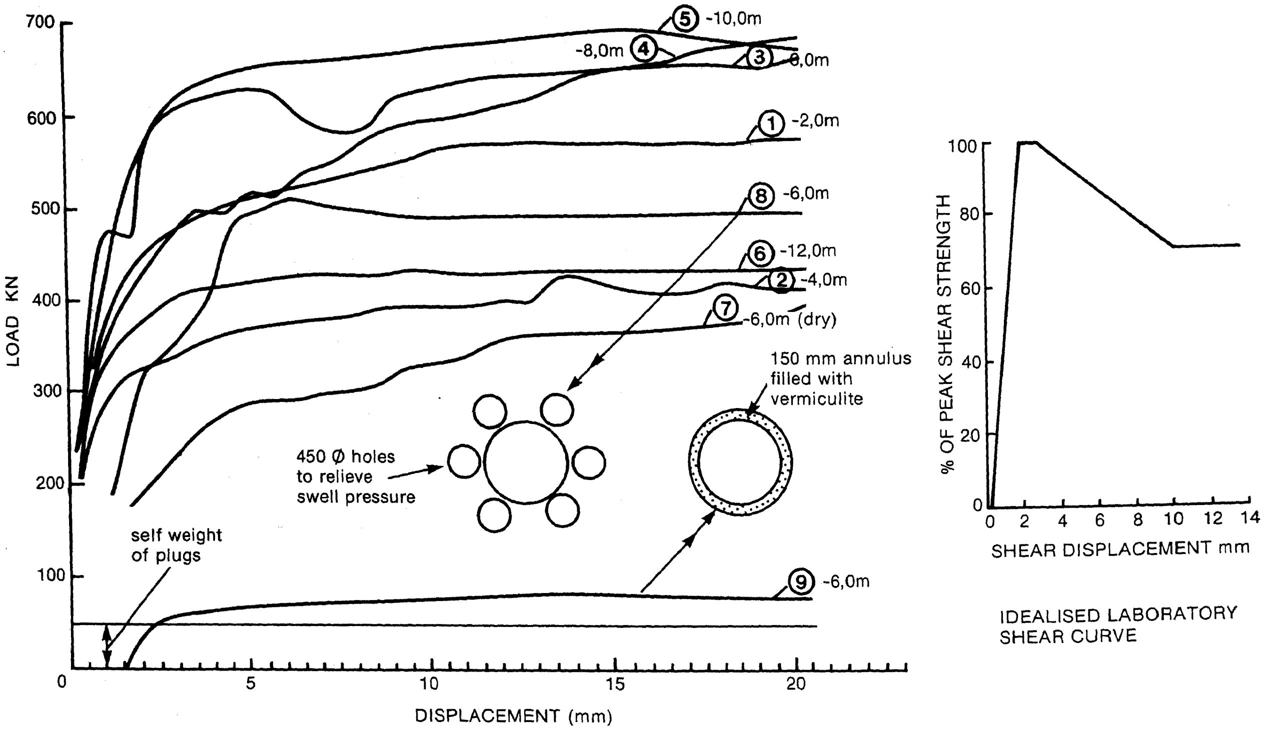


Fig. 14. Load-displacement curves for plug-pulling tests.

the load-deflexion curves recorded for the series of plugs. In general, the load increased rapidly to a yield value at a displacement of between 2mm and 5mm. Thereafter the pulling resistance remained approximately constant. This confirmed the indications from laboratory tests (inset on Figure 14) that full strength of the siltstone would be mobilized at displacements of between 2mm and 3mm. However, the plug pulling tests showed no peak strength but appeared to develop only a residual value. Most of the laboratory tests showed a peak strength of about 25% more than the residual value.

In Figure 14, curves 1 to 6 represent tests on plugs in soaked and fully swollen soil. The corresponding strengths have been interpreted as corresponding to total overburden stress with zero pore water pressure. Plug 7 in the series was left at natural water content and was installed at the same depth as plug 3. However, the pull-out strength was considerably less. This is almost certainly a reflection of the difference in the lateral stress against the plug walls in the two cases. In terms of total stresses, plug 3 developed a lateral stress equal to, or slightly more than the vertical stress, whereas the lateral stress on plug 7 was approximately half the vertical stress.

The plug-pulling experiment was also used to explore two possible methods of relieving the uplift forces on a pile shaft. After casting plug 8 (at the same depth as plugs 3 and 7) a series of 6 equally spaced 450mm diameter holes were drilled tangentially to the plug shaft on the theory that expansion of the siltstone would occur into the holes, thus reducing the lateral stress on the pile as well as the uplift force. Figure 14 shows the measure was almost completely ineffective.

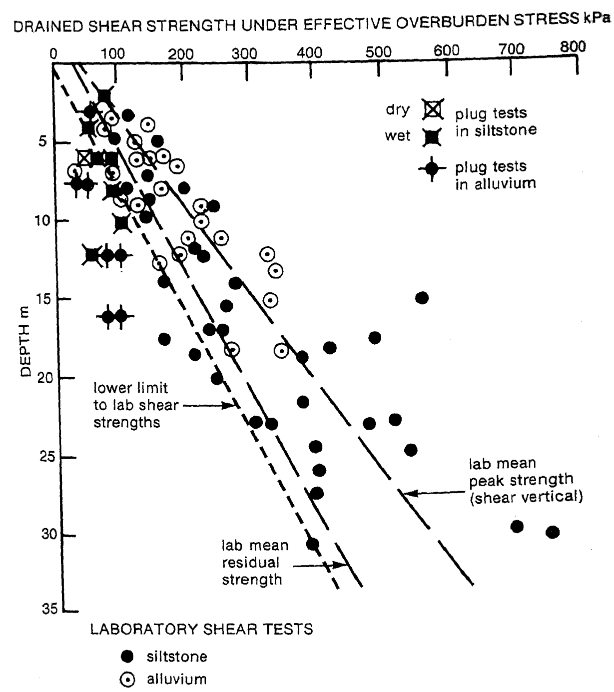


Fig. 15. Shear strengths measured by means of plug-pulling tests compared with laboratory shear strengths.

Plug 9, at the same depth as plugs 3,7 and 8 was cast inside a 1 005mm diameter light casing placed concentrically in a 1 200mm diameter hole. The annulus around the plug was then filled with exfoliated vermiculite to act as a bond breaker. Figure 14 shows that this measure was very succesful. The maximum pull-out force was only 90kN of which 50kN represented the weight of the plug and pulling cables.

Figure 15 summarizes all the drained shear strength tests performed on siltstone from Lethabo, as well as the results of the plug pull-out tests. This confirmed the trend (mentioned earlier) for large scale shear strengths to con-incide with the lower limit of shear strengths measured in the laboratory on small specimens. As the laboratory tests had been carried out under effective overburden stress, the indication was also given that K in equations (5) and (7) was close to 1,0.

(The time taken to reach maximum load in the pull out tests was about 5 hours which may have been too rapid for fully drained conditions to obtain on the shear surface. However, as the pore pressure parameter for the siltstone is close to zero this would not have had much effect on the results).

A similar series of plug-pullout tests was performed on plugs cast in the alluvium. These results are also shown on Figure 15. As the drained strength parameters measured in the laboratory were very similar for both siltstone and alluvium, these results indicate that K in the alluvium was about 0,5. This is presumably because of the lower lateral swelling pressures developed in the alluvium.

The time-uplift force curves shown in Figure 11 were calculated on the basis of mean laboratory strengths and the shear stress-deformation curve inset on Figure 14. The curve for maximum uplift force calculated on the basis of the plug pulling tests illustrates the considerable economies in reinforcing steel that could be effected by designing on the basis of strengths assessed from the plug pulling tests.

RELIEVING UPLIFT FORCES ON PILES

The test on plug 9 showed that uplift forces on piles could be very effectively reduced by sleeving them with vermiculite. Figure 16 shows a typical design of a vermiculite-sleeved pile which is being used extensively at Lethabo.

COOLING TOWER PONDS

Voids to isolate foundations and other structures e.g. circulating water ducts, have been formed at Lethabo by using waxed corrugated cardboard void-former boxes. The contractors for the cooling towers, however, have elected to cast the cooling tower pond on the ground and then to form an isolating void by undermining the pond floor. Figure 17 is a view showing the undermining in progress.

CONCLUSION

The foundation conditions at the Lethabo power station have posed some unusual problems, that, in turn, have led to an unusual programme of investigation. There are still many unknowns to be resolved and further tests are in progress to unravel these (see Blight et al, 1984). In addition, a programme of monitoring has been instituted to check on the long term performance of structures and the accuracy of the various predictions that have been made.

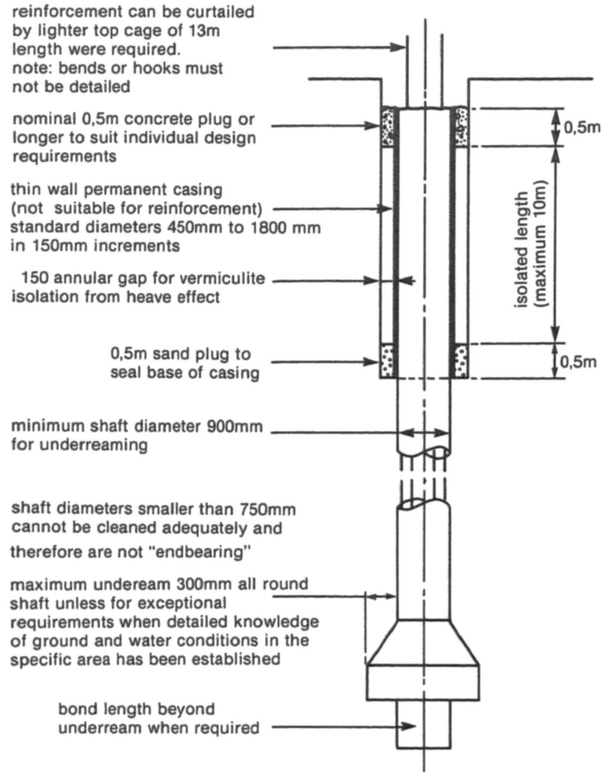


Fig.16. Vermiculite-sleeved pile designed to avoid uplift forces.

Fig.17. Creation of void under cooling tower ponds by undermining.

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