02 Jun 1993, 9:00 am - 12:00 pm

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Proceedings: Third International Conference on Case Histories in Geotechnical Engineering, St. Louis, Missouri, June 1-4, 1993, Paper No. 1.40

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SYNOPSIS

The paper describes tests on rigid square pads at the UK national soft clay research site at Bothkennar, Scotland. The work was performed as a low cost adjunct to the instrumented pile research described by Lehane and Jardine (1992). Its aims were to investigate (i) bearing capacity, (ii) load-displacement response to short and long term loading, (iii) the applicability of relevant theories and (iv) relationships between soil properties determined in-situ and those measured in high quality laboratory tests.

INTRODUCTION

The UK Science and Engineering Research Council (SERC)'s Bothkennar site is situated on the south bank of the Firth of Forth. Extensive site investigations have been performed there using the most modern techniques, as reported in fifteen co-ordinated papers in June 1992 in Geotechnique. The soils profile is summarised in Table 1.

Strata & depths	Soil type	Typical Index Parameters: % clay	w/c %	I_P %	I_L	γ kN/m³
0-1m **A**	Weathered clay-SILT crust	15	40	20	0.4	18
1-1.3m **B**	Shelly layer	----------N/A---------				
1.3-1.2m **C**	Soft clayey SILT with shell fragments	15	50	30	0.6	17
2.2-7m **D**	Soft black CLAY with mottling and occasional silt laminae.	20 to 40	60 to 75	30 to 50	0.6 to 1.0	15.5 to 16.5

Table 1. Summary of soils profile to 7m depth

Test	C_u, kPa
Triaxial UU, 100mm piston sample.	17.5
As above, Laval sample	25.0
K_0CU triaxial compression, Laval sample	22.5
As above, Sherbrooke sample	27.5
K_0CU triaxial extension, Laval sample	8.8
Undrained DSS test, Laval sample	16.3
Field Vane	25.0
CPT (for N_k = 12.5)	17.5
Self Boring Pressuremeter	19.5
DSS & triax. comp; mean C_u at large strain	15.5

Table 2. Average peak C_u values; 2 to 6m depth.

Profiles for the peak undrained shear strengths (C_u) obtained using a range of techniques are shown in Figure 1, whilst Table 2 emphasises the effects on C_u of the test method and sample type. Tubed piston samples are weaker than over-cored Laval or 'block' quali~~ty Sh~~erbrooke samples. The clay is lightly cemented by hematite and is noticeably brittle, giving remoulded C_u values of only 3 to 4 kPa.

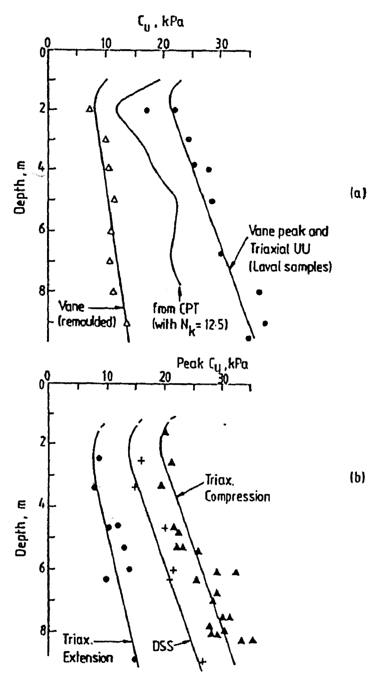
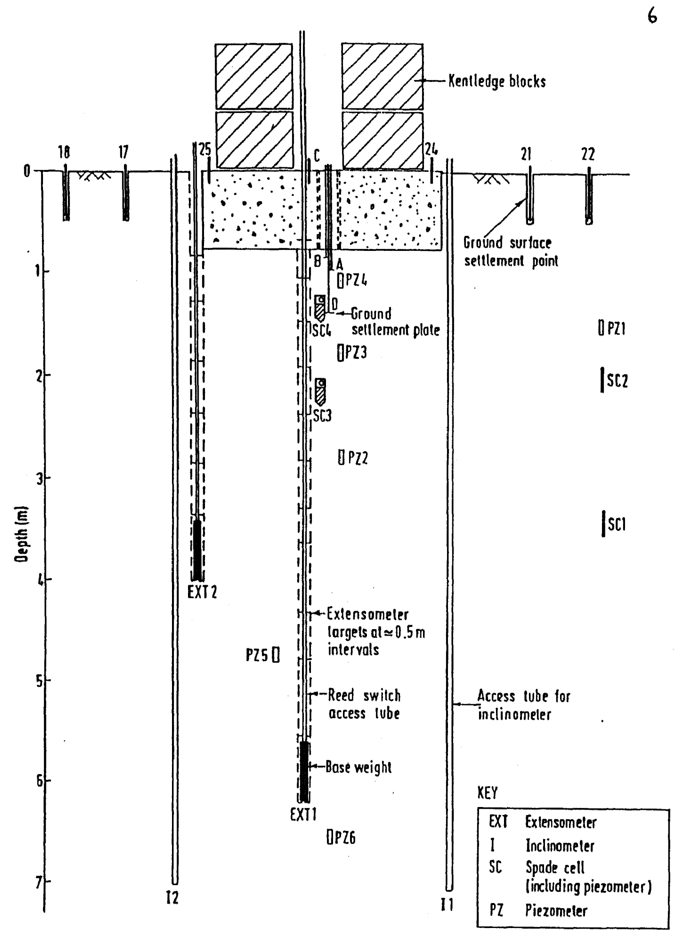


Figure 1. C_u profiles for Bothkennar; part b shows data from CK_0U laboratory tests.

Standard oedometer tests on samples from 1 to 4m depth gave relatively uniform vertical yield stresses (σ'_{vy}) between 40 and 60 kPa and confirmed that Strata B and C are less compressible than the clay layer (D). Hight et al (1992) provide further information on a wide spectrum of other soil parameters.

FIELD LOADING TESTS

The field tests involved applying vertical loads to five rigid, square, 0.8m thick reinforced concrete pads which were either 2.2m or 2.4m square in plan. Loading was applied by placing a series of relatively small kentledge blocks over periods of one to four days. Soil instruments were placed under two of the pads. The suite used for test B is illustrated in Figure 2; a simpler scheme was adopted for Test A. Pad A was taken to failure, whilst Pad B was loaded to 67% of the ultimate bearing capacity, with observations being continued for two years. Supplementary long-term tests were performed on the un-instrumented pads.



re 2. Instruments installed for Test B.

RESULTS

/erall load-displacement-time behaviour of .ve tests is summarised in Figure 3, which .tes curves relating bearing pressure (q_L) ettlement for a range of conditions. q_L d 138 kPa in the short term test to failure xcellent agreement was found between the ıg portions of the five tests.

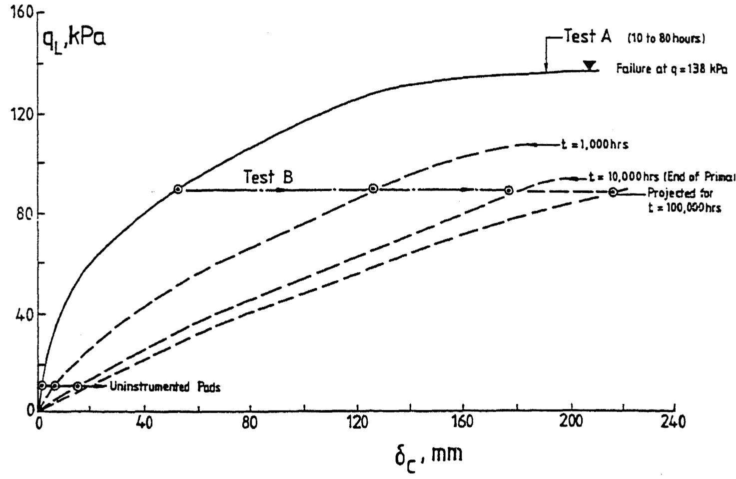


Figure 3. Overall summary of load-settlement data

Summaries of the data gathered from the various instruments are given in the following paragraphs, together with an interpretation developed from a simplified non-linear numerical analysis presented earlier by Jardine et al (1986). The predictions are identified by the code LPC2, which is the acronym of the non-linear elastic-plastic soil model employed for the study. Reference is also made to classical elastic and bearing capacity theory, and to linear consolidation analyses. Further details of the test data are given by Jardine et al (1993).

LOADING STAGES

Pore-pressure and radial stress changes during loading

The pore water pressures (u) and radial total stresses (σ_r) measured under the centre-line of Test A are plotted against applied load in Figure 4. For saturated clays, undrained pore pressure changes, Δu, can be separated into components associated with deviatoric and spherical stress changes: $\Delta u = B\ \Delta p + \alpha\ \Delta q$, where B can be taken as close to 1. Smith et al (1992) show that in triaxial compression Bothkennar clay gives $\alpha \approx 0$ and $\Delta u \approx \Delta p$, up to the peak deviator stress. The LPC2 theoretical solutions for the undrained centre-line stresses allow Δp and $\Delta\sigma_r$ to be calculated as functions of q_L for the full range of load factors L_f, where $L_f = q_L/q_L^{maximum}$. Predictions and measurements for Δu are summarised in a non-dimensional format in Figure 5. Points to note include:

1. The ratios $\Delta u/\Delta q_L$ (measured at given depths) generally increased with load, as anticipated by the non-linear LPC2 analysis.

2. Good agreement was seen between the LPC2 predictions and measurements at depths below foundation level, Z*, greater than ≈ 0.6D.

3. Upwards partial drainage was occurring at higher levels, with pore pressures falling well below the LPC2 predictions. An analysis of this dissipation indicated a consolidation coefficient, c_v, of ≈ 100 m^2/year for strata A and B.

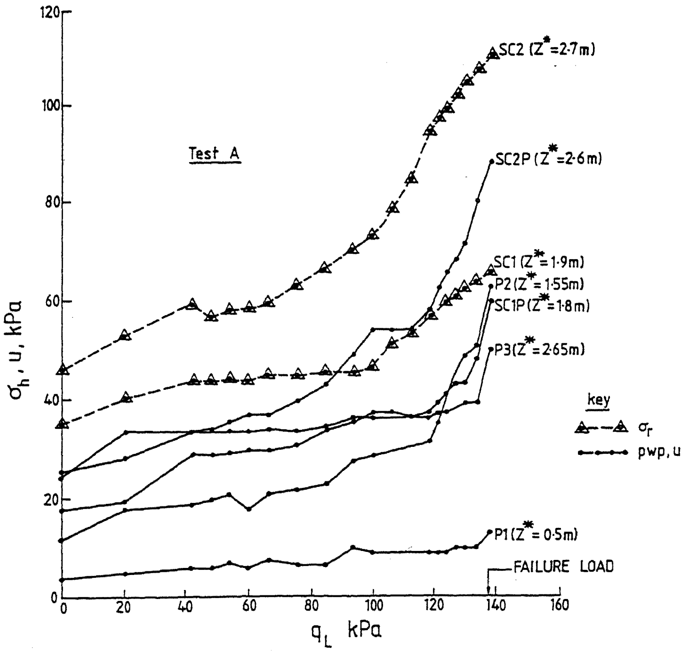


Figure 4. Pore pressures and radial stresses during loading; Test A

An analysis of the spade cell measurements is presented by Jardine et al (1993) which confirms that the influence factors relating q_L to measured stresses are non-linear. Further evidence was also given of partial drainage in the upper layers during loading.

Ground movements

Settlements

Examples are given in Figure 6 of the ground surface settlement profiles established by precision surveying in Tests A and B. The non-dimensional settlement bowls become progressively more steeply curved towards the pad edge as the load factor increased; almost no vertical heave was seen at failure. These trends were anticipated in the LPC2 numerical study, as shown by the theoretical curves reproduced in Figure 6. (Note that the equivalent pad radii r and diameters D are calculated on the basis of giving equal surface areas to the square pads).

Good agreement was also found between the centre-line settlements observed during loading in Tests A and B. Non-dimensional settlement-depth profiles are presented in Figures 7 a) and b) using data from both tests. Predictions from the LPC2 analysis are also shown.

The classical linear elastic theory greatly overestimates the volume of soil experiencing significant movements. Excepting the case of final failure, the settlement-depth gradients become progressively steeper as the load factor increased, giving more direct evidence of soil non-linearity. The LPC2 curves also over predict remote movements at depth, but to a much smaller extent; factors that may explain this over-prediction include the stiff surface crust, the finite depth of embedment, the non-uniform C_u profile and partial drainage during loading. None of these site specific factors was modelled in the LPC2 analysis.

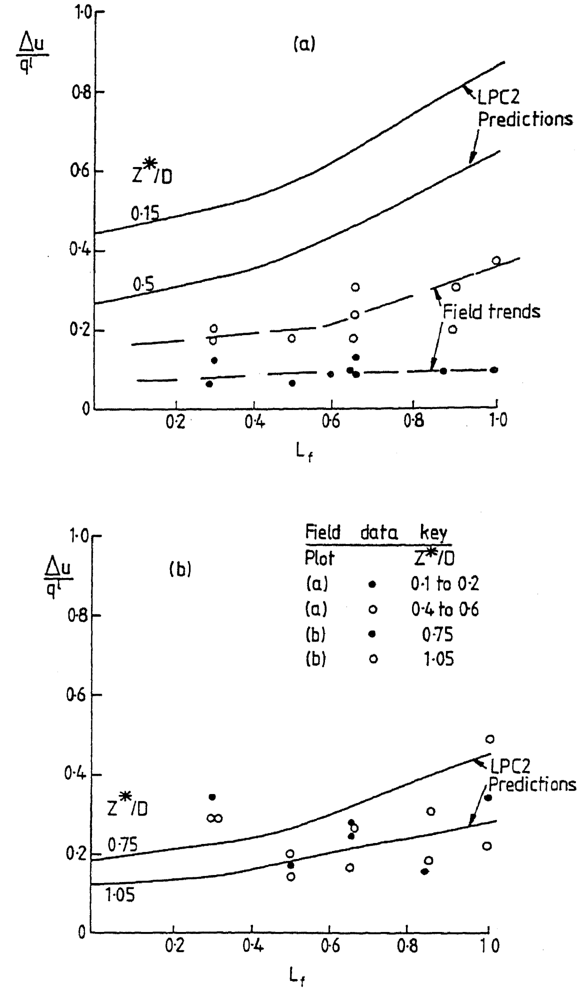


Figure 5. Predictions and measurements for pore pressures during loading.

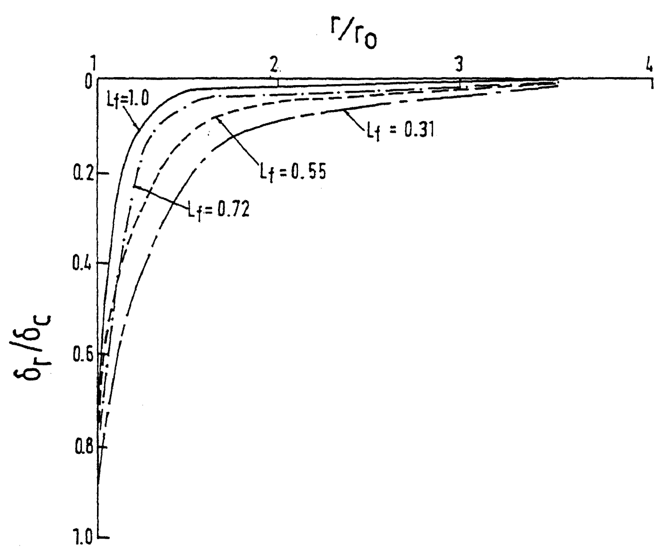


Figure 6. Ground surface settlements.

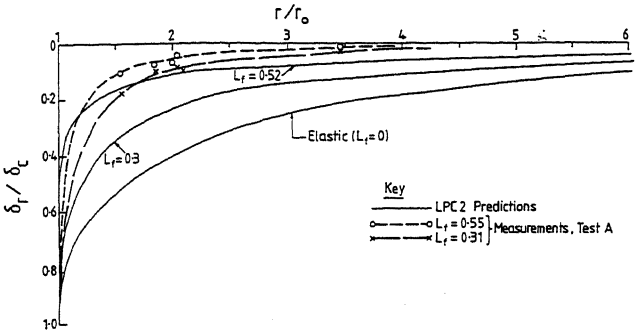


Figure 7. Predictions and measurements for centre-line settlements; Tests A and B.

In-situ shear stiffness

Axial stress-strain data can be deduced from the field measurements by differentiating the centre-line settlement-depth plots and adopting the LPC2 analytical results for deviator stress change; see Jardine et al (1993). If undrained conditions are assumed, then secant shear stiffness-strain curves may be derived from the loading data, as shown in Figure 8. Here G is calculated as $\Delta q/3\epsilon_{axial}$ and p' is the undisturbed in-situ mean effective stress. Also shown are data from a typical K_0CU triaxial test given by Smith et al (1992): the agreement is generally very good.

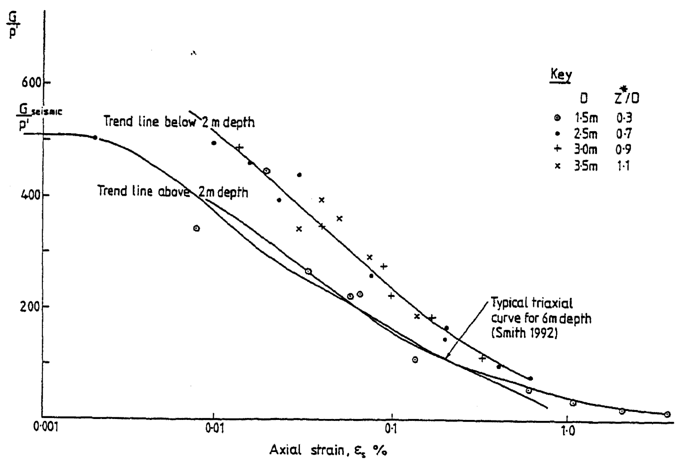


Figure 8. Shear stiffness-strain plots deduced from Test B, loading stage.

Lateral movements

Non-dimensional profiles representing the average of the radial displacements, Y, measured in Test B are shown in Figure 9, with the profile predicted for $L_f = 0.67$ by the LPC2 analysis. The shape of the measured profile hardly changed with load or time and the LPC2 analysis predicts rather more shallow movements than were measured. The factors listed above in connection with centre-line settlement profiles might also explain this discrepancy.

Figure 10 charts the relationship between centre-line settlement and Y_{max}. Stages 0-to-2 and 2-to-4 cover the loading and consolidation periods respectively. The line 0-X indicates the relationship expected if the volume displaced by the rigid pad's settlement matched that swept out by radial movements under the perimeter. The 'undrained' $\Delta Y_{max}/\Delta\delta_c$ gradient is 0.25, whilst that measured during loading was 0.19, showing that radial movements account for at least 75% of the volume displaced by the pad's short-term settlement. $\Delta Y_{max}/\Delta\delta_c$ fell to 0.055 during the consolidation stage, with drainage accounting for almost 80% of the volume displaced by the additional settlements.

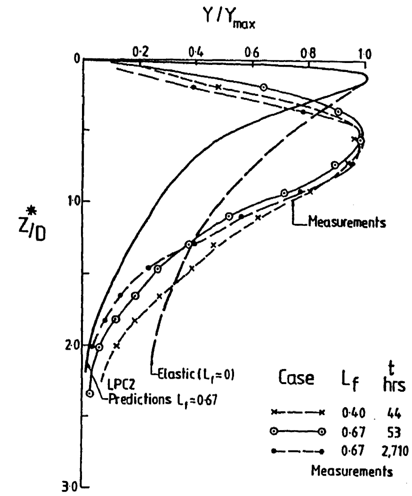


Figure 9. Non-dimensional radial displacement profiles; Test B.

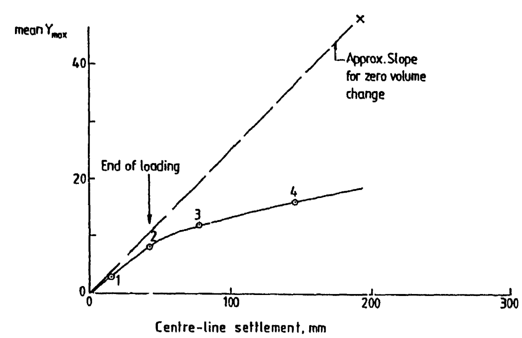


Figure 10. Trends of lateral movement with settlement.

Failure

The conventional bearing capacity equation for failures on clay is $q_{Lmax} = N_c C_u + \sigma$, where σ is the overburden pressure at founding depth. Following the LPC2 numerical analysis and classical theory, N_c was taken as 6.1 and the depth factor was assumed to be $1 + 0.4Z/D$. Accounting for the unit weights of the concrete and soil gives the operational C_u as 20.3 kPa. This strength falls below the peaks found in triaxial tests on high quality samples (see Table 2) and reflects the strain softening that develops during the progressive failure of the pad's foundations. The LPC2 numerical study showed that local failure is likely beneath even lightly loaded rigid footings. Jardine et al (1993) present an approximate analysis of the punching failure mechanism that was observed in the field.

BEHAVIOUR UNDER MAINTAINED LOAD

Variations of pore pressure, radial stress and settlement

The long term variations of radial total stresses σ_r, pore water pressures, pad settlement and radial displacement Y_{max} under the centre-line of Pad B are presented in Figures 11 and 12; Jardine et al (1993) provide further details of the other measurements. The following features are evident:

1. Excess pore water pressures took about 14 months (10,000 hrs) to dissipate, with the main clay layer controlling the overall rate of settlement.

2. Radial total stresses fell by larger amounts than pore pressures during dissipation.

3. The semi-logarithmic settlement-time graph has an almost constant gradient after 80 hours and shows no change at the end of pore-pressure dissipation.

4. Radial displacements were relatively small during the dissipation stage.

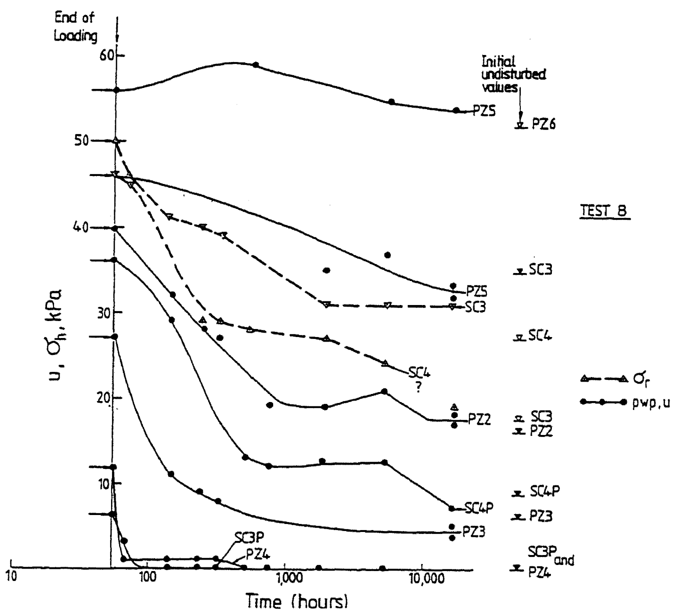


Figure 11. Long term variations in pore pressures and radial stresses; Test B.

A back-analysis has been made of the 'primary' settlement-time curve using the coupled Biot consolidation solutions of Davis and Poulos (1972). Good agreement was found between the field and theoretical curves when an 'oedometer' c_v of $8.5m^2$/year was assumed. This coefficient is consistent with laboratory data for clay samples that were on the verge of large scale yielding.

Non-dimensional profiles of long term centre-line settlements are given in Figure 13. Strains were concentrated initially at the upper and lower boundaries, but slowly became more uniform with depth as pore pressures dissipated. Given that the applied stresses reduced steeply with depth, the results show that the soil's drained stress-strain response is non-homogeneous.

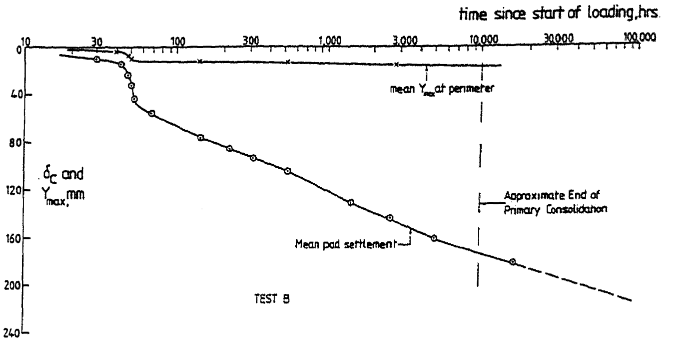


Figure 12. Long term radial movements and settlements; Test B.

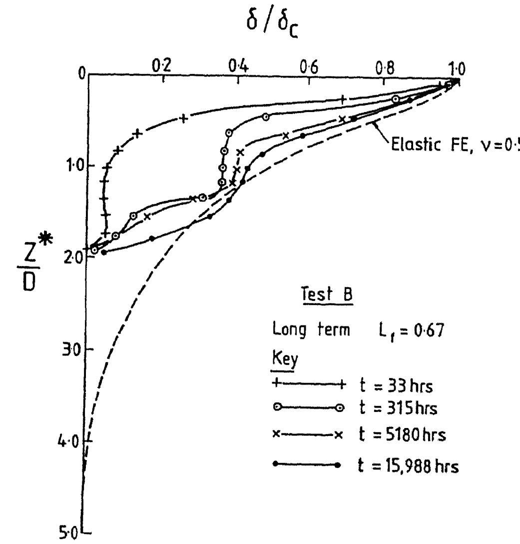


Figure 13. Long term centre-line settlements; Test B.

Stress-strain data

Calculations similar to those described for the loading stages have been performed for the two centre-line spade cells monitored in Test B. The stress and strain conditions are less well defined in the long term and alternative assumptions have been made when calculating σ_z in-situ. The resulting stress-strain plots, identified as Cases I and II, are given in Figure 14, where they are compared with drained stress path test data provided by Smith et al (1992). Generally reasonable agreement is seen between the laboratory and field data. In-situ yield points can also be identified from the field curves.

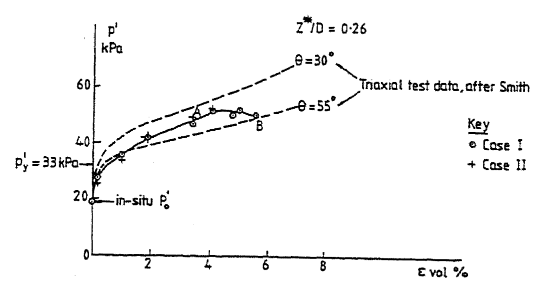


Figure 14. Long term stress-strain data; Test B.

In-situ yield envelope

Four independent large strain yield points were interpreted from Tests A and B; their effective stress states are plotted together on Figure 16, which also shows the envelope reported by Smith, et al (1992) from stress path probing tests on 'block' samples taken at 6m depth. A scaled down version of the same surface is also shown which passes through the mean oedometer yield point of the layers between 1 and 4m depth. The field envelope lies roughly between the laboratory surface for 6m depth and its scaled down derivative. But the envelope's shape appears to have been shifted upwards, possibly reflecting differences in the role that chemical bonding plays at different levels in the Bothkennar profile.

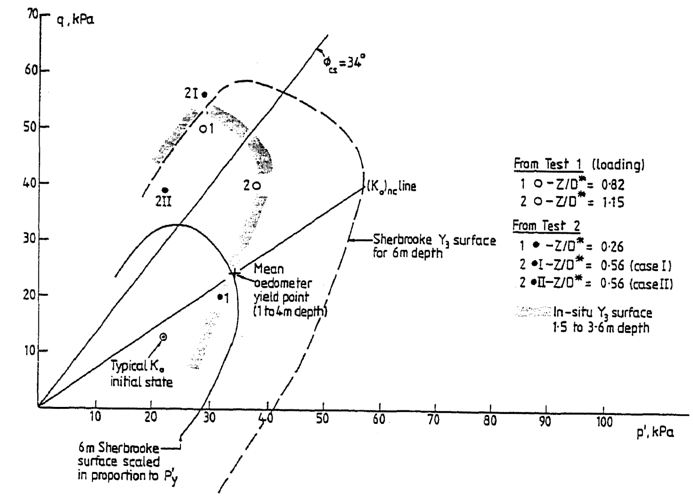


Figure 15. In-situ and laboratory yield envelopes.

CONCLUSIONS

1. The foundation behaviour was non-linear, in-elastic and time dependent, even under light loads.

2. The foundation response over a loading period of two to three days, was not truly undrained.

3. Settlements were generally concentrated progressively closer to the loading area as the load factor increased.

4. Failure was progressive. The measured bearing capacity was 35% lower than calculated from triaxial tests on the highest quality samples.

5. Consistent patterns were seen in the ratio of radial movements to pad settlements. Lower ratios ratio applied to long term consolidation than to loading.

6. Linear elastic theory seriously underestimates the centre-line stress changes caused by loading. It also greatly over-predicts the radial and vertical extents of significant ground movement.

7. Simplified non-linear analyses predict many of the phenomena seen in the field loading tests.

8. Linear coupled and uncoupled consolidation analyses were useful for predicting approximate rates of settlement and pore pressure decay.

9. Shear stiffness-strain data back-figured from the load tests matched stress-path triaxial data. Reasonable agreement was also obtained between the p' - ϵ_{vol} curves and yield envelopes determined in the field and in the laboratory.

ACKNOWLEDGEMENTS

The Authors thank the Science and Engineering Research Council (SERC), who funded the work; the Building Research Station who assisted with the site operations, the University of Glasgow group, who gave valuable practical help, and to colleagues at Imperial College.

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