Instrumented footing load tests on soft sensitive Bothkennar clay  
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Introduction

Shallow foundations are seldom used in cases where the foundation soil consists of soft sensitive clay. This is due to both the risk of bearing capacity failure and the unacceptable deformations experienced, even under relatively light loads. However, it is necessary to found many other types of structure such as embankments or bridge abutments on this type of material. Due to the complex stress-strain behaviour of natural soft clays, much care is required in estimating both the bearing capacity and the load-displacement behaviour of any structure founded on soft clays. Though much research has been carried out into the behaviour of embankments on soft clays (ie Jardine & Hight, 1987; Leroueil, Magnan & Tavenas, 1990) assuming plane strain conditions, much less work has been directed towards the investigation of full scale foundations on soft clay under essentially axisymmetric conditions, similar to those encountered in the triaxial test.

The intention of this paper is to describe the behaviour of an instrumented footing loaded to failure while founded on soft, sensitive estuarine clay. This will be done in terms of both its load displacement history and its bearing capacity. The field behaviour will then be compared with both theoretical predictions as well as predictions based on high quality laboratory testing. Finally a series of conclusions will be made regarding the observed and predicted behaviour of the footing.

Site location and ground conditions

The SERC soft clay test bed site is situated at Bothkennar on the south bank of the Forth Estuary, approximately 1.5km from Kincardine Bridge (Figure 1). The general geology consists of a thin desiccated crust overlying between 14m and 22m of lightly overconsolidated, weakly cemented estuarine clays and silts (Carse clay sequence). These in turn overlie a layer of gravels. An extensive programme of site investigation was carried out at the site between 1987 and 1990 incorporating both specialist in situ testing and laboratory testing on high quality samples (for details see Geotechnique Symposium in print, 1992).

In the specific area of the test footings the geology and typical index parameters can be summarised as shown in Table 1.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Strata description</th>
<th>% clay</th>
<th>mc</th>
<th>I_p</th>
<th>I_L</th>
<th>γ (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 1.0</td>
<td>firm to stiff very clayey silt (desiccated crust)</td>
<td>15</td>
<td>40</td>
<td>20</td>
<td>0.4</td>
<td>18</td>
</tr>
<tr>
<td>1.0 to 1.3</td>
<td>shelly layer in matrix of soft very silty clay</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>1.3 to 2.2</td>
<td>soft very clayey silt, occ. shell fragments</td>
<td>15</td>
<td>50</td>
<td>30</td>
<td>0.6</td>
<td>18</td>
</tr>
<tr>
<td>2.2 to 7.0</td>
<td>soft black mottled very silty clay, occ. silt laminae</td>
<td>20-60</td>
<td>60-75</td>
<td>30-50</td>
<td>0.6-1.0</td>
<td>15.5-16.5</td>
</tr>
</tbody>
</table>

Table 1.

Footing load test procedure

During the period May/June 1990, two reinforced concrete footings were constructed with approximate dimensions 2.2m × 2.2m × 0.78m. Footing A was instrumented and subsequently loaded to failure (Gildea, 1990). Footing B was loaded to approximately 90% of the failure load of Footing A and monitored for two years (Jardine, Lehane,
LEFT: Figure 2a. In situ strength profiles.
Figure 2b. Laboratory derived strength profiles.
BOTTOM: Figure 3a. Plan view of footing and instrumentation.
BELOW and INSET RIGHT: Figure 3b. Surface settlement point positions.
Only the results of the footing loaded to failure (footing A) will be discussed here.

**Instrumentation**

The instrumentation was installed approximately two and a half weeks after the footing had been poured and consisted of the following (see Figure 3a):

- one magnet extensometer with nine target magnets at various depths;
- two earth pressure cells (spade cells) with internal pneumatic piezometer units;
- three pneumatic piezometers;
- four settlement points cast into the footing at the centre of each edge.

All instruments were calibrated on site prior to and following installation. In addition, 25 surface settlement points were installed in the ground at various positions in order to investigate the extent of the area stressed surrounding the test footing (Figure 3b). All settlement points both on and surrounding the footing were monitored using precise levelling techniques. A schematic cross section of the footing instrumentation is shown on Figure 4.

**Loading Procedure**

Loading was applied using a combination of both concrete and steel kentledge blocks, placed in symmetrical sequence. All kentledge was weighed using a load cell suspended from a crane prior to placement. Loading of the test footing to failure was carried out over a period of four days. The criteria used to control the rate of loading was for 40 kPa to be added per day unless the creep rate exceeded 8 mm/h. At this point no further load would be added until the creep rate had stabilised. As the test approached the expected failure load the load increments were significantly reduced. Once the test footing had failed, it was left overnight before unloading was carried out in two separate stages, over a period of three hours.

**Test results**

**Ground Movements**

The settlement-time behaviour for the four points monitored on the footing is shown in Figure 5, while the load-maximum displacement plot is shown on Figure 6. The observed load at failure ($q_{(max)}$)
was 138kPa. It can be seen from Figure 5 that the settlement of the footing was relatively even until the load factor, $L_t (= q_l / q_{l(max)})$ was greater than about 0.7. As the load was further increased and the footing approached failure, the footing began to tilt preferentially to one side. At failure (ie $L_t = 1$) the maximum differential movement across the footing was 50mm corresponding to a maximum tilt angle of 1.3°. This relatively even settlement profile is indicative of a punching type failure mechanism. On unloading the footing, 11% elastic rebound was observed indicating the high degree of plastic deformation at failure.

Figure 7 shows the normalised surface settlement ($\delta_r / \delta_c$) plotted against the distance from the edge of the test footing normalised to the equivalent circular radius of the square footing ($a_0 = 1.24m$). This has enabled the field data to be compared with that predicted by finite element analysis of a circular rigid footing on an undrained, non-linear, low plasticity clay at various load factors (Jardine et al., 1986). It can be seen that the influence of the pre-yield stiffness variation is to concentrate the ground movements very strongly around the loaded area. The close comparison between the field data and the movements predicted by the non-linear finite element model is clear evidence of the high degree of non-linearity of the soil. It should be noted that almost zero ground heave was observed in the area surrounding the footing, a phenomena also predicted by the finite element analysis.

The variation of settlement with depth was monitored directly beneath the centre of the footing. The data was interpreted by differentiating the settlement depth plot (Figure 8) to provide a plot of variation in vertical strain with depth below base of footing, $Z^*$ (Figure 9). This indicates large vertical strains developing (>12%) in a zone between 1.8m and 2.8m below the base of the footing, as load factors increased. This suggests the development of a punching type failure mechanism at this depth, where a thin relatively compressible layer of soil has been progressively 'squeezed' out and displaced laterally.

- Variations in pore water pressure and radial stresses

Figure 10 shows the variations in observed pore water pressures and radial stresses with applied surface load, while Figure 11 indicates the variation of both total and excess pore water pressures with depth below footing level and increase in load factor. It should be noted that the observed field ratio of both $\Delta u / \Delta q_o$ and $\Delta \sigma_r / \Delta q_l$ increased with increasing load factor, as would be
Figure 9. Variation of vertical strain with depth and load factor.

Figure 11. Variation of pwp and excess pwp with depth and time.
expected during undrained loading. However, the behaviour in both cases was significantly non-linear. The ratio \( \Delta u / \Delta q_L \) remains relatively low (0.1 to 0.3) before rapidly increasing towards 1.0 once the load factor increases above approximately 0.6. The same general trend applies to the change in radial stresses.

**Stress-strain behaviour during loading**

Plots of deviator stress \((\sigma_d - \sigma_0)\) against axial strain \((E_a)\) are shown in Figure 12. The influence factors, \( I_0 \), used to calculate the field changes in deviator stress are taken directly from the FE analyses presented by Jardine et al (1986). These influence factors were adopted because they were found to be much less sensitive to constitutive laws and load factor than those calculated for changes in vertical stress. These plots indicate peak undrained shear strength values of between 20-25kPa which is directly comparable with results obtained from laboratory tests on high quality samples of Bothkennar clay.

**Stiffness variations**

Undrained secant Young's moduli \((E_u^{sec})\) normalised to the in situ mean effective stress \((\bar{\sigma}_v')\) have been calculated directly from the respective stress-strain plots and are shown in Figure 13, plotted against log axial strain. At low strain levels the clay exhibits high stiffnesses which rapidly drop off with increases in axial strain. The

\[ \text{Figure 10. Plot of radial stress and pwp changes against load.} \]
normalised stiffness vs axial strain curve derived from undrained triaxial compression tests on high quality samples is also shown (see Smith, Jardine & Hight, 1992) and generally indicates slightly lower stiffness values than the field curves at equivalent strain levels. The likely reason for this is a combination of lower strain rates in the triaxial test, partial drainage in the field producing a stiffer response and the accuracy of the extensometer system at low strain levels (<0.1%). However the general trends are directly comparable and indicate that the Bothkennar clay behaves in a highly non-linear manner.

Figure 13. Field and laboratory derived stiffness profiles.

- Stress paths during loading
Both total and effective stress paths for the footing loaded to failure are shown in Figure 14, plotted in terms of deviator stress, \( q = \sigma_A - \sigma_R \) and mean effective stress, \( p' = (\sigma_A + 2\sigma_R)/3 \). It can be seen that as loading progresses there is only a relatively small increase in \( p' \) until yielding occurs at relatively large strains. At this point the effective stress paths move sharply to the left due to the rapid decrease in mean effective stress, \( p' \). The critical state envelope derived from laboratory tests is also shown (where \( \phi'_{cs} = 34^\circ \)). The yield points indicated by these two effective stress
paths can be seen to straddle this envelope. This observation of the field stress paths may be explained by: i) local variations in undrained shear strength; and ii) inaccuracies in the field measurements of radial stress which can be magnified during the calculation of deviator stress.

Discussion

- Bearing capacity
A bearing capacity prediction was made based on the theory outlined by Davis & Booker (1973) using a profile of undrained shear strength varying with depth derived from Figure 2. This method predicted an ultimate bearing capacity, \( q_u \) (max) of 161kPa, an overprediction of the observed field capacity of 17%. Back analysis of the field results using conventional bearing capacity theory for undrained failure on clay was also carried out (ie \( q_u \) (max) = \( N_c \, C_u + P_o \) where \( N_c \) is the undrained bearing capacity factor and \( P_o \) is the overburden pressure at base of footing level). This resulted in a back calculated average undrained shear strength of 20.3kPa. This value is lower than the peak values measured in undrained triaxial compression tests on high quality Laval and Sherbrooke samples. One explanation is that some degree of progressive failure has occurred, which is at variance with predictions made using the classical Prandtl wedge and its associated plastic failure mechanism. However, this is not surprising since conventional bearing capacity theory relies on the assumptions that the founding medium behaves as an isotropic rigid elastic—perfectly plastic material. This is not the case, especially for soft, sensitive clays which behave in a highly non-linear manner, with a considerable degree of irrecoverable plastic strain developing at even relatively light loads.

- Failure mechanism
Settlement of the footing occurred in an even manner until load factors exceeded approximately 0.7 and failure appears to have occurred via a punching type mechanism, rather than the classical wedge mechanism (see Figure 15 for comparison of theoretical and postulated failure mechanisms). The pattern of axial strain development (Figure 9) indicates that large strains developed fairly rapidly and at relatively low load factors initially. This occurred in a zone 2.8m to 2.8m below the base of the footing. It is suggested that local yielding occurred in this zone due to the progressive 'squeezing out' of a thin, locally weak zone of clay confined by marginally stronger material above and below. The contrast in strength between zones need not be very large, since once local yielding begins to develop, strain softening occurs and reduced post peak strengths in this thin, locally weak zone begin to govern the overall failure mechanism of the footing.

It is postulated that this type of failure is extremely difficult to predict, even with the aid of complex non-linear constitutive models.

Therefore,
inherent, often apparently random, variations in the properties of natural soft clays are extremely difficult to model effectively and care must be exercised in the interpretation of any theoretical predictions of behaviour at failure.

Conclusions

- The field load displacement behaviour indicates that Bothkennar clay displays highly non-linear, inelastic and hysteretic behaviour even at low strain levels.
- As the load factor increases, non-linear behaviour causes vertical deformations to be concentrated closer to the edge of the footing than predicted by elastic theory.
- Failure mechanisms implicit in classical bearing capacity theory do not accurately model the observed progressive, punching type failure mechanism.
- Back analysis of the field behaviour using classical bearing capacity theory leads to calculated undrained shear strengths up to 30% less than peak undrained shear strengths derived from triaxial compression tests on high quality samples.
- The field variation of shear stiffness with axial strain shows similar trends to those derived from laboratory tests on high quality samples. However, the field tests show slightly higher stiffnesses than laboratory tests at equivalent strain levels.
- Small variations in natural soft clay properties can initiate local yielding under load. This is problematical if predictions of behaviour have been made based on standard theory. Therefore the possibility of local and progressive failure should always be considered when dealing with soft, sensitive clays.

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References


