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EARTH PRESSURES AGAINST AN EXPERIMENTAL RETAINING WALL BACKFILLED WITH SILTY CLAY

by

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ABSTRACT

The measurement is described of the lateral soil pressures produced by the compaction of a silty clay backfill against a 2 metre high retaining wall. In addition to monitoring the soil pressures and the total vertical and horizontal thrusts acting on the wall, measurements of pore water pressure were made to establish the effective stress state of the soil fill.

Following completion of the filling operations, the conditions were maintained for several months to observe changes in pressure acting on the wall as a result of excess pore pressure dissipation. Thereafter, active and passive failures of the soil were induced by rotating the wall about its toe.

The investigation indicated that significantly higher lateral pressures were developed on the 2 metre high wall than could be created by the self weight of the soil fill. These high pressures were attributed to the influence of the compaction plant. The study also demonstrated that only a very small rotation of the experimental wall ($\sim 1/20$ degree) was required to achieve the active state. The passive condition, which was investigated using 1 metre height of fill, required in excess of 7.5 degrees to mobilise the full passive resistance of the soil.

1. INTRODUCTION

If on-site fill materials were more extensively used in the construction of earth retaining structures, the cost of these structures would be substantially reduced. At present the performance of cohesive-frictional fill materials both during construction and during the longer term is uncertain. Until more reliable information is available an increased use of these materials is unlikely. To help provide this information investigations are being carried out on the lateral pressures developed in a range of materials during construction as well as at limiting states of active and passive pressure. Preliminary results using a sand have already been presented[1] and in this report results with a silty clay as backfill are discussed. Future work will include a study of the lateral stresses on retaining structures backfilled with a heavy clay soil.

Although research[1,2,3] has demonstrated that the compaction of granular fill behind rigid retaining walls can induce high lateral earth pressures on the structure, the behaviour of cohesive soils is somewhat more complex and in need of detailed investigation. Whereas with a free draining granular soil only the effective stresses need generally be considered, in the case of cohesive soil the excess pore water pressure may also have to be taken into account. For the purpose of this experiment using silty clay backfill, instruments were installed to evaluate both the magnitudes of the pore water pressure and the total stress produced by the construction plant. Following completion of the filling operations, the conditions were maintained for several months to observe changes in pressure acting on the wall as a result of pore water pressure dissipation.

2. EXPERIMENTAL RETAINING WALL

A detailed description of the experimental retaining wall has been given in an earlier publication[1]. The facility included a moveable metal wall (height 2 metres, length 6 metres) and also a one-metre thick reinforced concrete wall of similar dimensions to allow an assessment to be made of the influence on the results of wall rigidity.

3. INSTRUMENTATION

The total thrust on the centre panel of the metal retaining wall was recorded by load cells and the distribution of stress was measured using soil pressure cells set flush with the panel surface. Three vertical profiles of six pressure cells were used and each profile incorporated a different type of cell. As the operating performance of the different versions of pressure cell depended upon the soil condition[4], an in-situ calibration was achieved by comparing the total thrust on the wall with that determined from the stress distribution obtained from the vertical profile involving those cells. Cell factor, which is defined as the ratio of measured to applied stress, is normally influenced by the level of applied stress. However, with the exception of the passive case, only a limited range of pressures (5—30 kPa) occurred in this study and it was considered justified to assume a constant factor. The cell factors in silty clay were calculated on this basis to be 0.77, 1.04, 1.42 for the strain gauge, hydraulic, and pneumatic cells respectively. These same latter two factors were applied to correct the readings of the hydraulic and pneumatic cells installed in the reinforced concrete wall.

The pore water pressures in the silty clay were monitored by vertical profiles of piezometers placed in the fill at a distance of 1 metre from the instrumented retaining wall panels. The piezometers used were of the ceramic filter candle type connected by twin lengths of polythene-coated nylon tubing to mercury-filled manometers. Piezometer tips with a high air entry pressure were chosen as the silty clay was only partly saturated, and checks were regularly made to see if de-airing was required.

Measurements of wall movement were also carried out using a dial gauge arrangement referred to a datum line independently established using a laser system. A more detailed description of this system is reported elsewhere[1].

4. PROPERTIES OF THE SOIL

The soil used was a silty clay from Iver, Buckinghamshire. The particle-size distribution and the results of plasticity and specific gravity tests are shown in Figure 1. The soil was intended to be used at the wetter end of the moisture content range allowed in the then current specification for earthwork construction in the UK[5] namely 1.2 times the plastic limit corresponding to a moisture content of 20 per cent. Although care was taken to control the moisture content by adding water and rotovating each layer of soil, exceptionally hot weather at the time of the experiment led to some drying out and measurements on the compacted fill showed that its average moisture content and bulk density were 18.5 per cent and 2.0 Mg/m^3 respectively.

Triaxial tests were carried out on specimens remoulded to this moisture content and density. The shear strength parameters in terms of effective stress were found from drained tests to be 37 degrees for the angle of shearing resistance (ϕ') with no cohesion intercept. Undrained triaxial tests using the silty clay gave a c_u value of 25 kN/m^2 and a ϕ_u value of 13 degrees. The A and B pore pressure parameters were also determined from triaxial tests and found to be zero and 0.72 respectively.

Using the ϕ' value of 37 degrees, the coefficient of earth pressure at rest (K_o) was calculated from the formula $(1 - \sin \phi')$ as 0.40. The values of the active (K_a) and the passive (K_p) earth pressure coefficients were determined as 0.25 and 4.02 respectively.

A consolidation test carried out on the fill material showed that the coefficient of consolidation was 0.6 m^2/year over the pressure range of 20–30 kPa.

5. EXPERIMENTAL PROCEDURE

The silty clay was placed and then compacted with six passes of a 3.25 Mg smooth wheeled roller (Plate 1) to 125 mm thick layers. During this operation, the closest the roller edge came to the wall was 100 mm. After compaction of each layer the plant was removed and measurements of earth pressure and wall deflection were made. When construction was completed the fill was carefully sheeted to minimise any loss of moisture and readings were taken over a period of four months during which time dissipation of pore water pressures was expected to occur.

The previous experiments using a sand[1] had demonstrated that active wall movements as small as 0.5 mm had a considerable effect on the stresses acting on a 2 metre high wall. For this reason the active study was therefore carried out after first surcharging the fill with concrete blocks to produce a uniform surcharge stress of 27 kPa which corresponded to an additional height of fill of 1.4 metres. The amount of wall movement and the changes in pressure on the wall were thus larger and could be more accurately recorded. The active condition studied corresponded to a rotation of the wall about its base and away from the soil. The rotation took place in increments of one minute of arc until at an angle of 0.5 degrees the stresses on the retaining wall were significantly below the predicted active values. Measurements were continued after the rotational movement was completed to monitor the changes which occurred as the silty clay gradually relaxed to recover intimate contact with the wall and produce the stresses corresponding to the active condition.

For the passive study it was necessary to remove the surcharge and the top metre of silty clay as calculations had indicated that the full two metres height of fill would cause the jacking system to overload. However, even with such a low height of fill it was only possible to rotate the wall into the soil by 7.5 degrees about its base before the stresses had increased to their permitted maximum. It was therefore not possible to exceed the limiting shear strength of the soil and fail the soil in the passive mode.

6. RESULTS

6.1 Lateral stresses after compaction

The distributions of stress immediately after the completion of construction are shown in Figure 2 for the three pressure cell types installed in the metal wall. In these figures the horizontal stresses acting on the wall have been corrected using the appropriate cell factors referred to previously. Also shown in Figure 2 is the wall deflection measured from the stage at which soil was first compacted at each particular level until the completion of backfilling. This wall deflection was expected to relieve some of the stresses on the metal wall and the results in Figure 3 verify that slightly higher stresses were recorded on the more rigid concrete wall. In both cases significantly higher stresses were measured than would be calculated if earth pressure at rest (K_o) conditions were assumed.

6.2 Decrease of stress with time

After the construction operation was completed, a gradual decrease in the lateral thrust acting on the wall occurred as shown in Figure 4. After a period of four months the lateral thrust was approaching an asymptotic state at which time the stresses on both the metal and concrete retaining walls had fallen to a magnitude very close to the calculated earth pressure at rest value (Figure 5).

In Figure 6 the changes in both the total stress and pore water pressure distribution over this four month interval are plotted for the metal wall. Similar results were obtained with the reinforced concrete retaining wall. Consideration of the relative areas under the appropriate graphs showed that, to within 9 per cent, the decrease in thrust on the wall face could be correlated with the dissipation of excess pore pressure.

6.3 Active condition

In Figure 7 the variation in the horizontal and downward vertical thrusts on a metre length of retaining wall are given for both the 0.5 degree rotation of the wall away from the soil and the subsequent recovery of these forces with time towards the active values. A steady horizontal force on the wall of 20.4 kN was recorded after nearly a week, although 80 per cent of this force was developed within 12 hours.

6.4 Passive condition

The results for the passive study, which was investigated employing only one metre height of fill in the test facility, are shown in Figure 8. A horizontal thrust of about 125 kN per metre length of wall was recorded after a rotation of 7.5 degrees at which point the study had to be terminated because of the danger of overloading the load cells, although the results appear to indicate that the limiting passive value had nearly been reached.

7. DISCUSSION

Adequate compaction of the backfill behind retaining walls such as bridge abutments is essential if subsequent settlement is to be avoided. However the proximity of plant to the wall is likely to result in pressures in excess of those calculated assuming earth pressure 'at rest' conditions. The occurrence of high lateral stresses towards the top of an experimental wall has been reported in an earlier study using granular backfill[1], and the results summarised in Figure 9 confirm that such high stresses are also produced with cohesive fill. In Figure 9 close agreement is observed between the measured stresses and the stresses calculated from elastic theory taking account of the influence of both construction plant and pore water pressure. For the purpose of these calculations the twin drums on the rear axle of the roller were considered as two finite length line loads acting at the appropriate distances from the retaining wall. Further details of the method are given in the Appendix.

In contrast with the sand backfill in which the high stresses at the top of the structure were a permanent feature, in the case of the silty clay the high stresses began to decrease when the construction work was halted. After a period of about four months the distribution of total stress on the wall had nearly reached the K_0 condition (Figure 5) and this reduction in stress correlated well with the dissipation of the excess pore water pressure. In order to confirm that it was a consolidation process which was occurring, the experimental observations are compared in Figure 4 with a simple prediction based on a 1-dimensional consolidation analysis for a 2 metre thick layer of fill. In reality there was a further metre thickness of silty clay below this, but as this had been placed at an earlier stage it was considered to be fully drained. For the purpose of the analysis a drainage path of 1 metre was assumed and reasonable agreement was obtained after making the appropriate allowance for the construction period[6].

The magnitude of the movement required to produce the active soil condition is shown in Figure 7, where it was found that a small outward rotation of only three minutes was required. This result corresponded to a deflection at the top of the wall of about 1/1000 of its height and agrees well with the value first reported by Terzaghi[7].

In the calculation of active design conditions for a soil (such as silty clay) which has both internal friction and cohesion, the effect of cohesion plays an important part in the classical equation for the horizontal pressure at any depth on the wall:–

$$\text{Horizontal pressure} = K_a(\gamma z + q) - 2c\sqrt{K_a}$$

Evaluation of this formula using the undrained parameters for the silty clay shows that the soil is in tension to a depth of 1.8 metres. If the effects of wall friction and wall adhesion were included, this depth would be even larger.

In the active study reported herein, the total force on the wall approached zero during the rotation (Figure 7) as would be expected from the above formula. However this situation was only temporary as positive horizontal pressures were recorded on the experimental wall within hours of its movement being halted. After a few days had transpired a steady state level was reached which was considered to correspond to the active condition anticipated in the field. A wedge type analysis was then performed to determine the mobilised angle of internal friction using the measured values of 20.4 kN and 12.5 kN for the horizontal and vertical forces on the wall. On this basis and assuming no cohesion on the failure plane, a mobilised angle of internal friction of 34° was calculated for the silty clay. This compares well with the residual value of 33° measured by Pomfret[8] during drained shearbox tests on normally consolidated specimens prepared from a slurry of the silty clay. The specimens, which were sheared slowly, showed no peak strength but indicated a steady increase in shear resistance up to the residual value. These results suggest that during the active failure study, strain softening was induced with a consequent reduction in the angle of internal friction from the peak to residual value. On this basis therefore, it would appear most appropriate to determine the active earth pressure distribution on the basis of effective stress considerations employing the residual value of ϕ'.

Although it was not possible to complete the study of the passive resistance of the silty clay, the data were valuable in demonstrating the large magnitude of the forces created. After a rotational movement of only 7.5 degrees a horizontal force of nearly 125 kN and a vertical force of 51 kN were developed over a metre length of wall. Much larger forces were expected to be generated at failure. The long term behaviour can be assessed by assuming a curved plane of failure and carrying out an effective stress analysis based on a progressive type of passive failure[9,10]. In this case a coefficient of passive resistance of ~ 6.7 and hence a horizontal force of ~ 66 kN would be calculated on the basis of the apparent wall friction angle of 22 degrees. However as this study was carried out relatively quickly over a period of one day, a higher force than 66 kN would be anticipated because of the generation of high excess pore water pressure. A study in which the shape of the failure plane was recorded would be required in order to make a more detailed analysis of the passive state.

8. CONCLUSIONS

The performance of a silty clay backfill behind an experimental retaining wall has been evaluated. The following conclusions were reached:–

1. The compaction of a silty clay behind a rigid retaining wall produced lateral stresses at the top of the wall considerably in excess of those calculated for the 'at rest' condition. A more realistic assessment of the lateral stress distribution can be obtained from elastic theory taking account of the influence of both construction plant and pore water pressure.

2. On completion of construction the high lateral stresses decreased as consolidation occurred with the dissipation of excess pore water pressures. In this particular case a period of about four months was sufficient to relieve the high stresses at the top of the wall.

3. The rotation required to reach the active pressure distribution corresponded to about 1/20 degree. Interpretation of the results for the cohesive-frictional fill employing the observed boundary forces indicated that strain softening had occurred. Thus, the active earth pressure distribution corresponded best with that determined on the basis of effective stress considerations employing the residual value of ϕ'.

4. A preliminary investigation using a one metre height of fill showed that a rotation in excess of 7.5 degrees was required to mobilise the full passive resistance of the soil.

9. ACKNOWLEDGEMENTS

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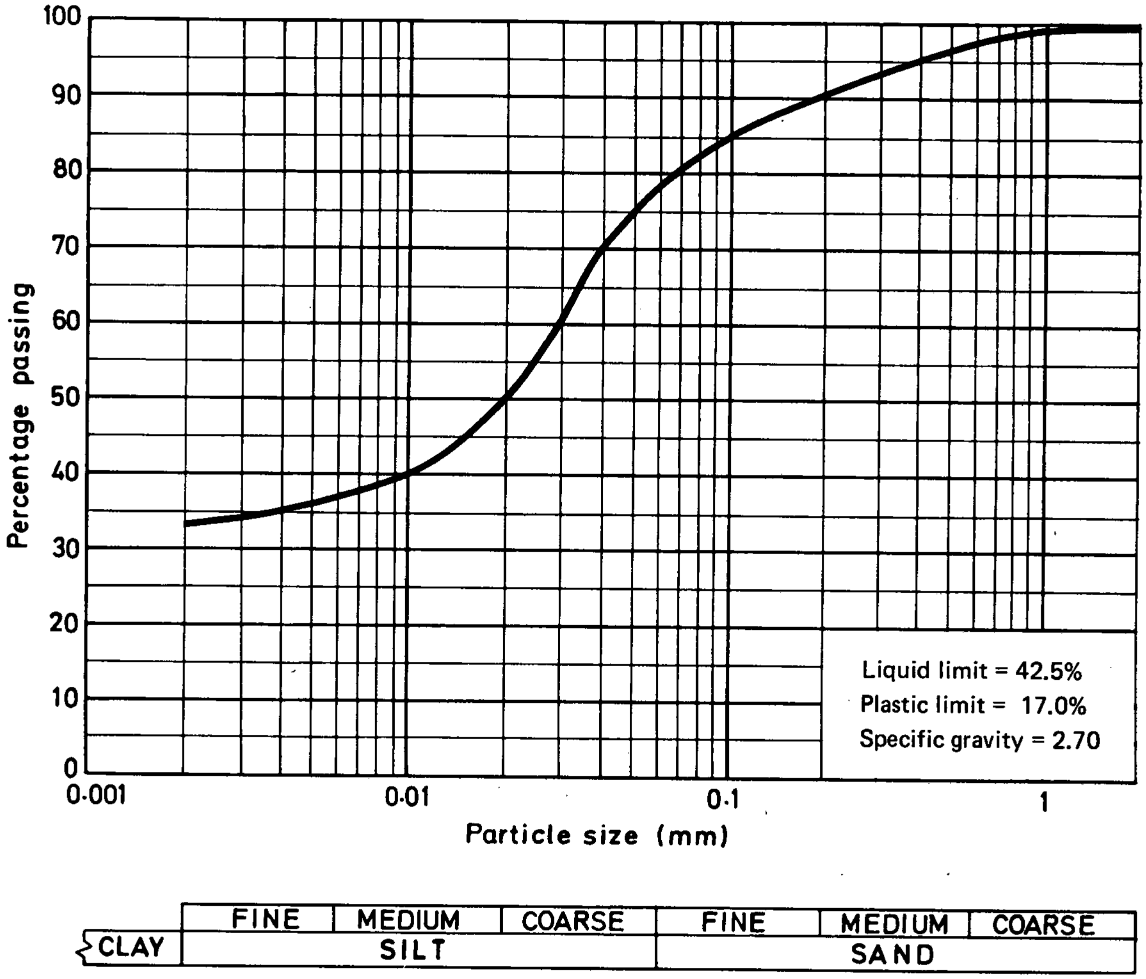

Fig.1 PARTICLE SIZE DISTRIBUTION FOR THE SILTY CLAY

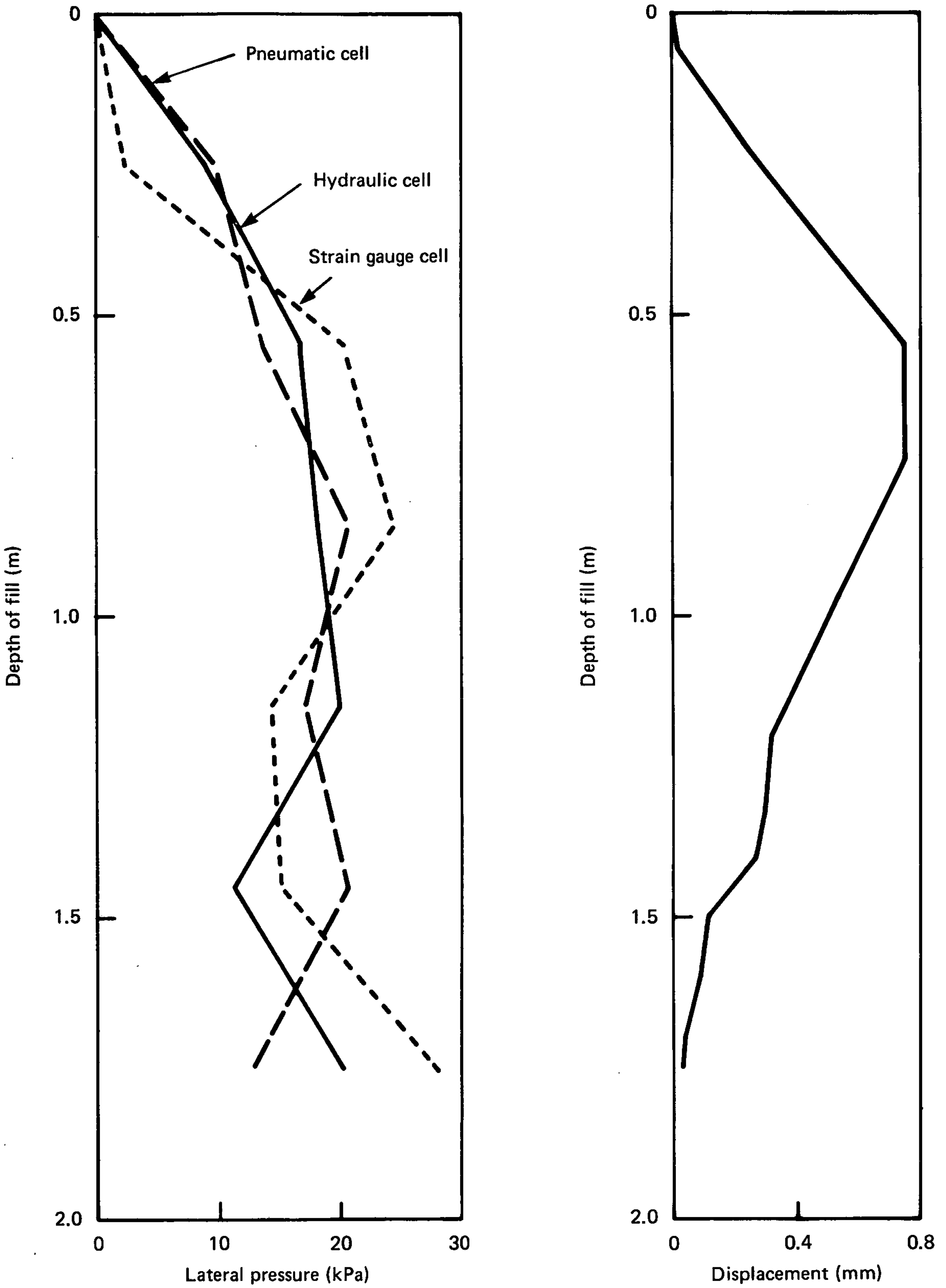

Fig.2 COMPACTION OF SILTY CLAY

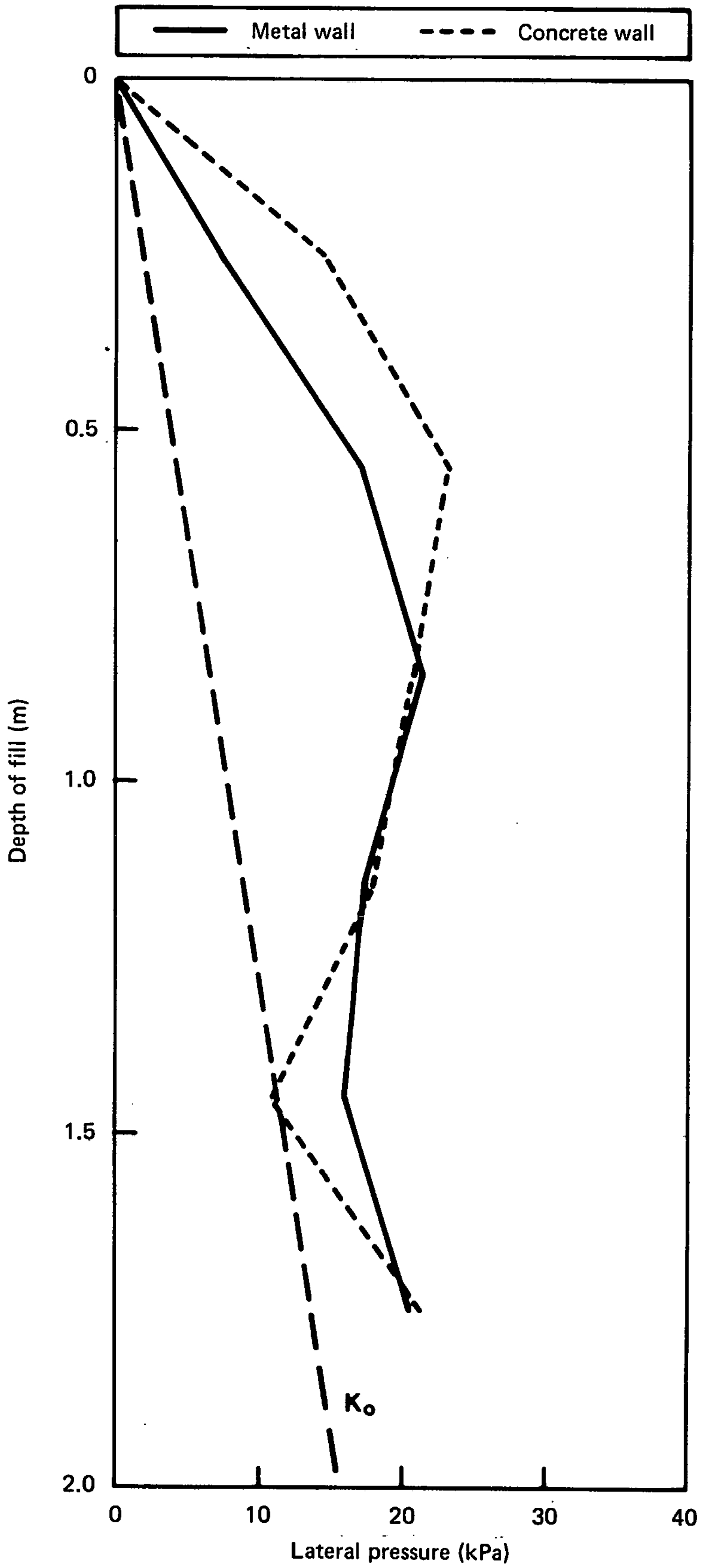

Fig.3 COMPACTION OF SILTY CLAY

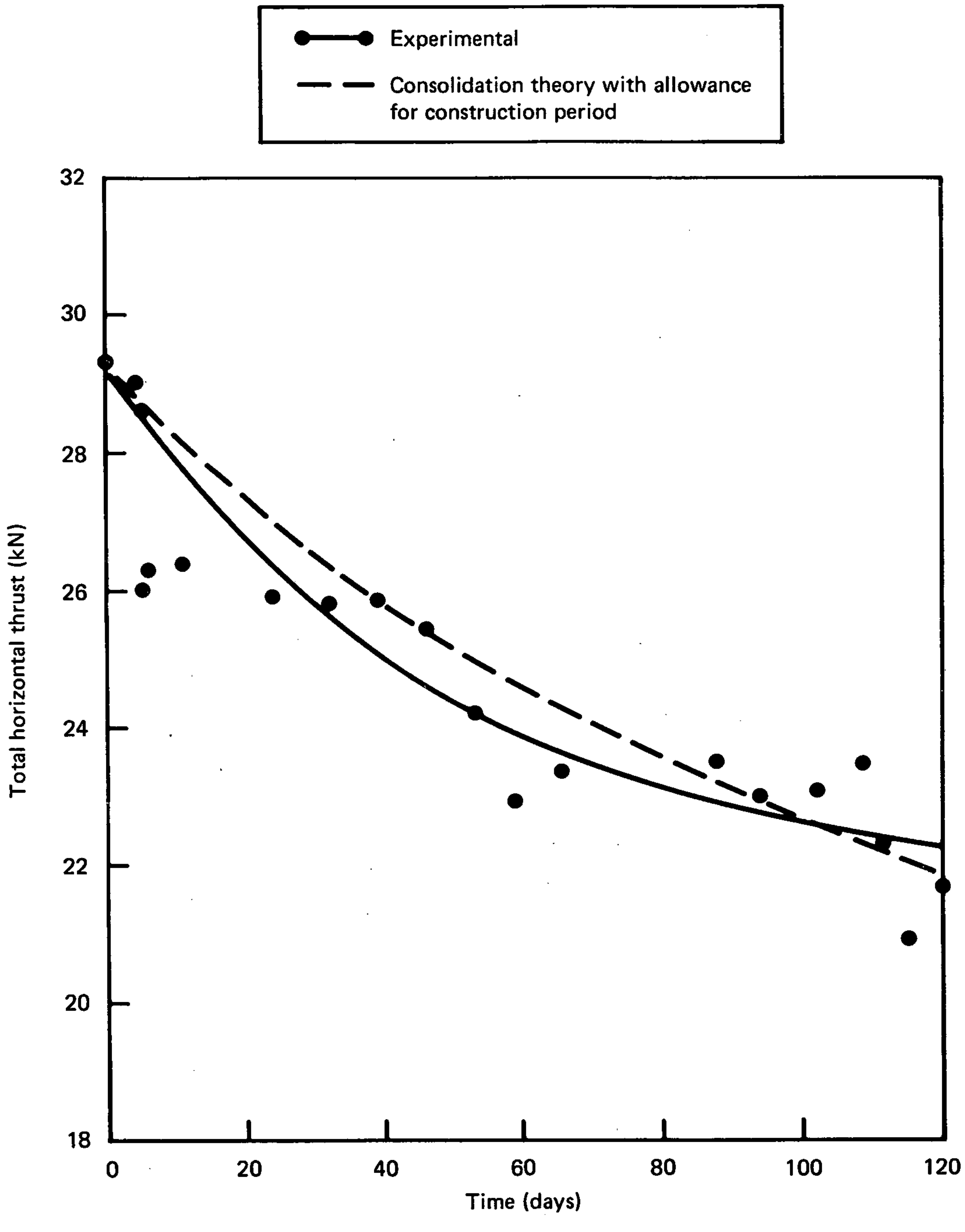

Fig.4 DECREASE IN THRUST WITH TIME

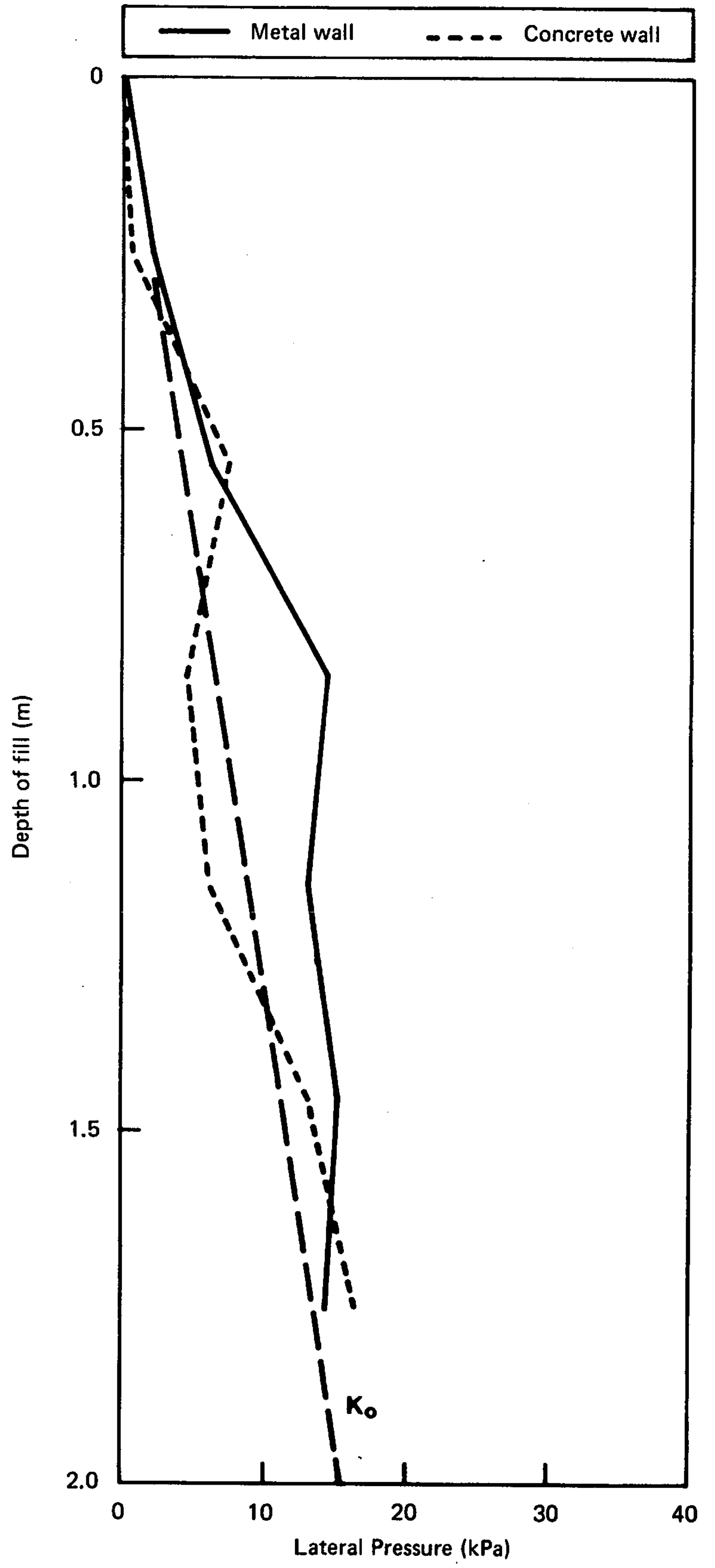

Fig.5 STRESS DISTRIBUTION FOUR MONTHS AFTER COMPLETION OF COMPACTION

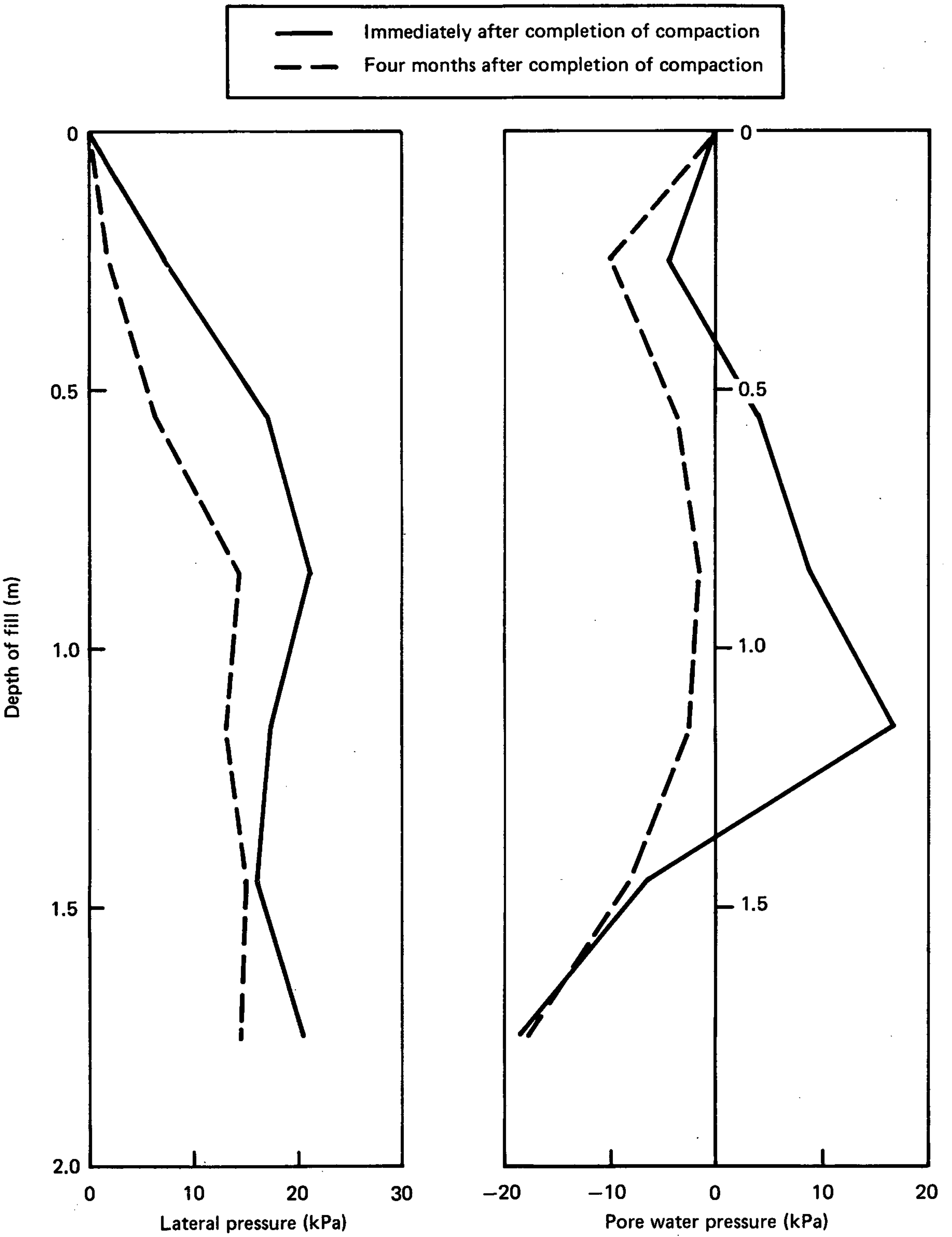

Fig.6 COMPARISON OF CHANGES IN LATERAL PRESSURE AND PORE WATER PRESSURE FOR METAL WALL

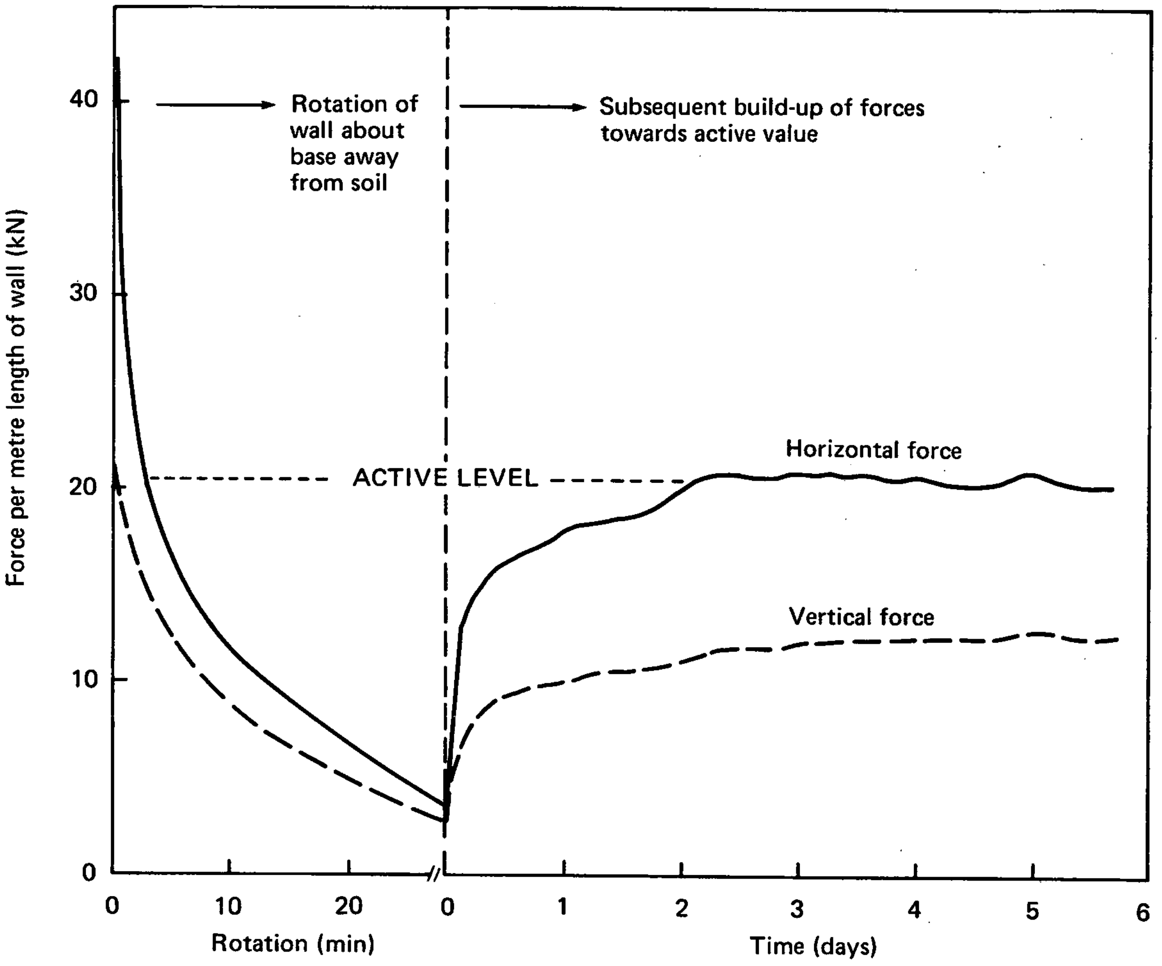

Fig.7 THE ACTIVE CONDITION

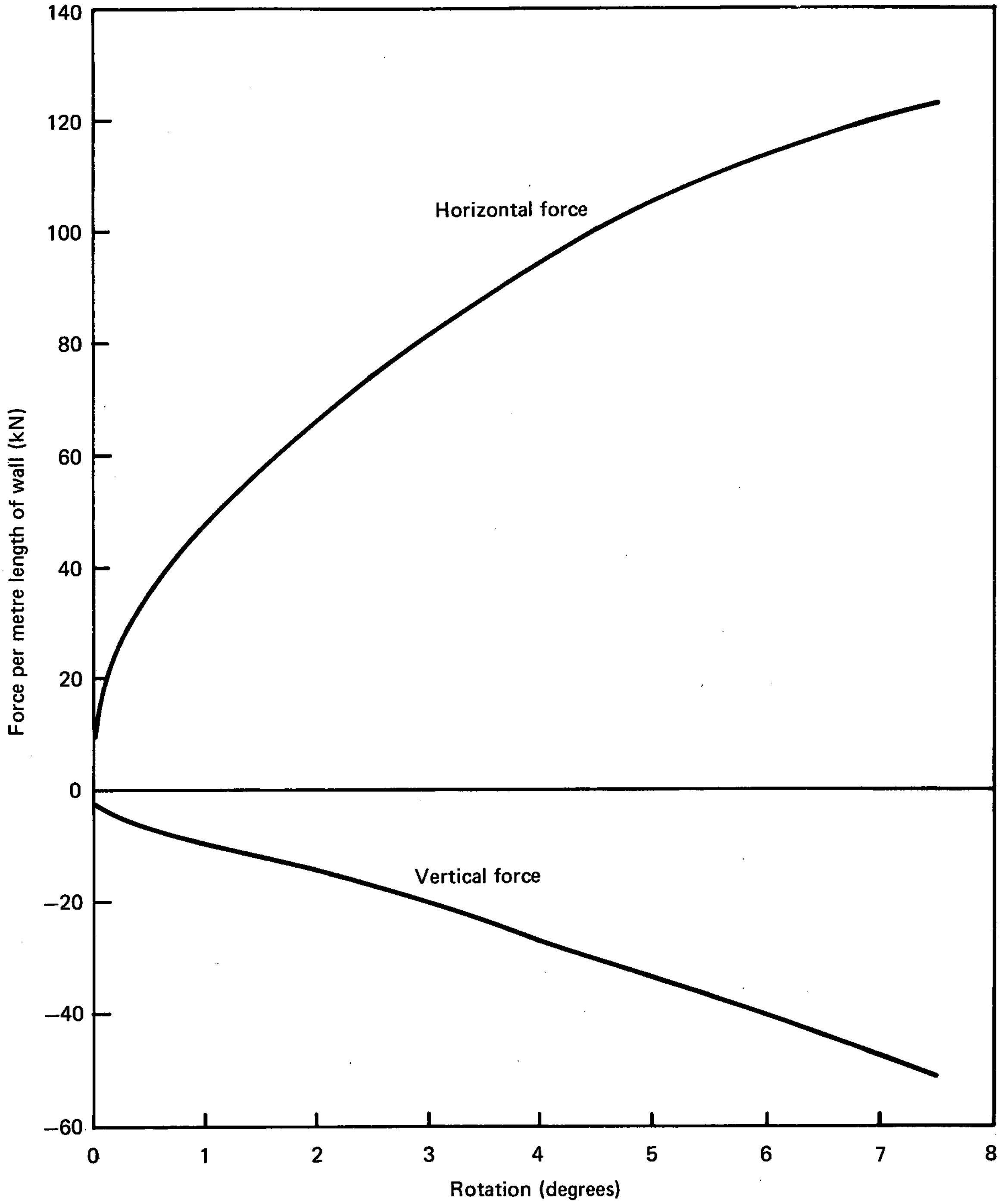

Fig.8 THE PASSIVE CONDITION

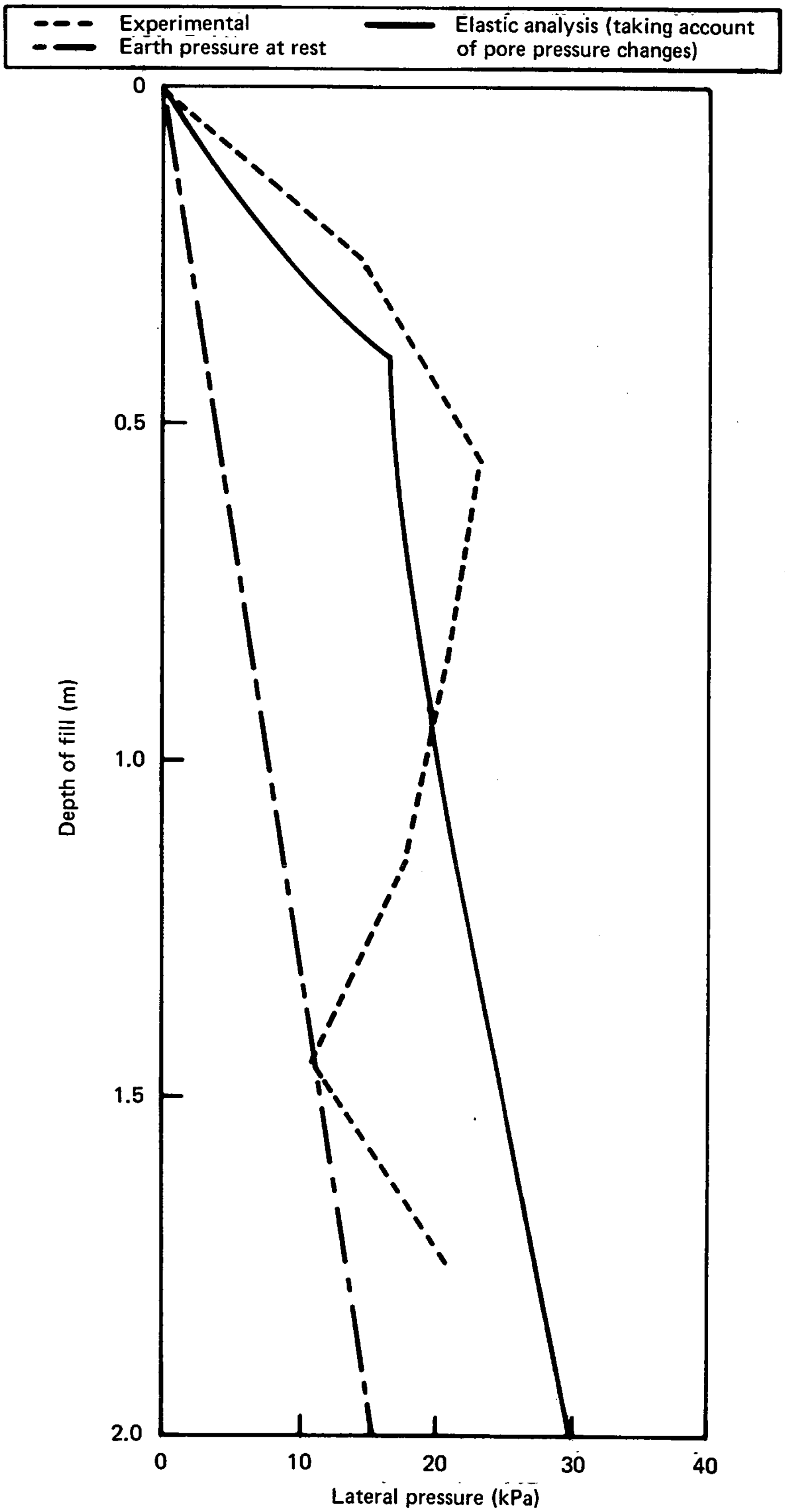

Fig.9 COMPACTION OF SILTY CLAY

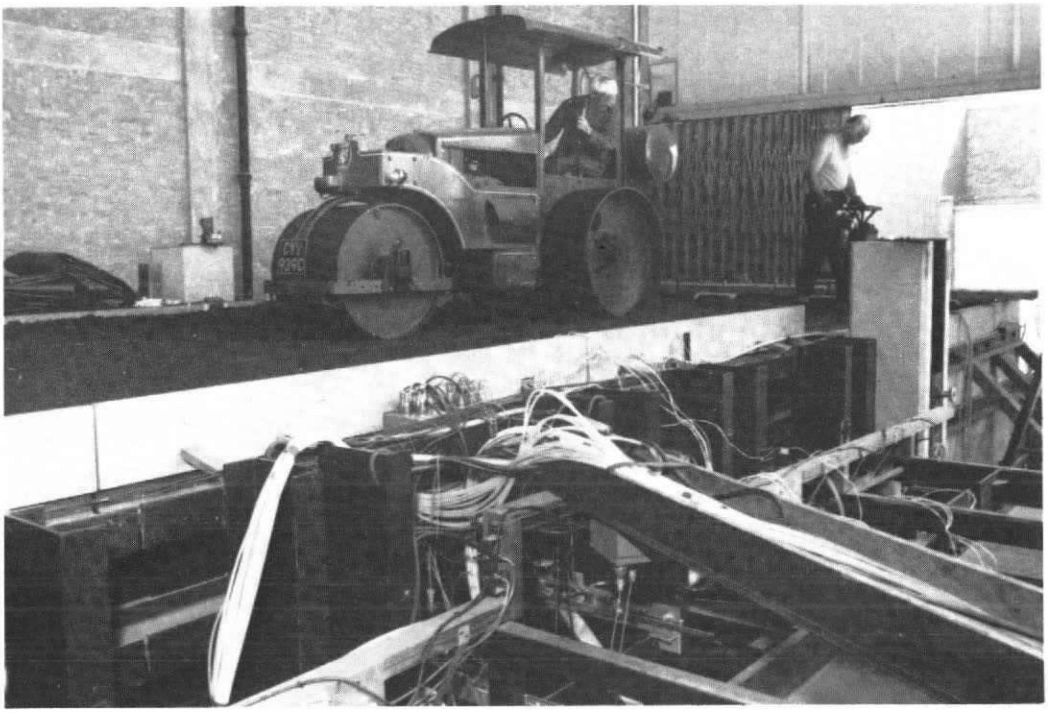

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Plate 1 ROLLER COMPACTING THE FINAL LAYER OF SILTY CLAY

11. APPENDIX

CALCULATION OF LATERAL STRESSES ON RIGID STRUCTURES DUE TO COMPACTION

A method is described for calculating the influence on the stresses of the proximity of plant to the wall. It is possible by using this approach to obtain a more realistic assessment of the horizontal pressure distribution acting on a rigid wall. Alternatively, this technique may be employed for walls designed on the basis of active yielding, as a guide for assessing the permissible distance of construction plant from the wall face.

For the purpose of the following calculations the wall has been assumed frictionless and completely rigid. Consequently earth pressure at rest conditions apply and an elastic analysis will provide a reasonable assessment of the influence of compaction plant on the horizontal and vertical pressures. The elastic equations employed are derived from the integration of the formulae for a point load near the surface of a semi-infinite solid, ie:–

$$\sigma_h = \frac{p}{\pi z}\left[\left(\frac{x}{R}\right)^3 - \frac{(1-2.\nu_u)\,x}{(R+z)}\right]_{x1}^{x2} \qquad \ldots\ldots\ldots\ldots\ldots \quad (1)$$

$$\sigma_v = \frac{p}{2\pi.z}\left[\frac{x}{R}\left(\left(\frac{z}{R}\right)^2 + 2\right)\right]_{x1}^{x2} \qquad \ldots\ldots\ldots\ldots\ldots \quad (2)$$

where σ_h = horizontal stress on wall induced by line load perpendicular to wall face

σ_v = vertical stress at the wall face induced by line load

p = line load per unit length

x1 = distance to nearest point of line load

x2 = distance to furthest point of line load

z = depth below surface

R = $\sqrt{x^2 + z^2}$

ν_u = Undrained value of Poisson's ratio

An additional factor of two has been incorporated in the formula for horizontal stresses to make allowance for the 'reflected' stress situation which arises at the wall surface. When using equations (1) and (2) account has to be taken of the type of plant being employed. For example, if vibrating plant is being used an increased value of p will be required to allow for the centrifugal force being generated. For the case discussed in this report, the smooth-wheeled roller incorporated twin drums on the rear axle and it was necessary to summate the stresses calculated for each drum.

The undrained value of Poisson's ratio used in equation (1) is determined from the following formula for saturated soil which was derived by Bishop and Hight[11]:–

$$\nu_u = \frac{[\,3\nu + (1-2\nu).B\,]}{3-(1-2\nu).B}$$

where ν = Poisson's ratio calculated from the angle of internal friction (ϕ')

B = Pore pressure coefficient

It was considered justified to use this formula for the silty clay fill (degree of saturation = 83 per cent) as the soil is likely to become locally saturated beneath the concentrated load. The undrained value of Poisson's ratio for the silty clay was accordingly determined as 0.43.

The distribution of the applied stress between the effective stress and pore water pressure is then calculated at different depths of fill using the equation:–

$$\Delta u = B . \left\{ \frac{1}{3} (\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3) + (A - \frac{1}{3}) . \sqrt{\frac{1}{2}\left((\Delta\sigma_1 - \Delta\sigma_2)^2 + (\Delta\sigma_2 - \Delta\sigma_3)^2 + (\Delta\sigma_3 - \Delta\sigma_1)^2\right)} \right\} \quad \ldots\ldots (3)$$

where A, B = Pore pressure coefficients
Δu = Pore pressure increment
$\Delta\sigma_1, \Delta\sigma_2, \Delta\sigma_3$ = Principal stress increments

This equation, which is a modification of the relationship derived by Skempton[12], permits the A and B coefficients determined from triaxial loadings to be applied to a 3-dimensional situation. In this particular instance there is no significant stress change in a direction parallel to the wall length and at points in the line of action of the loading, ie $\Delta\sigma_2 \approx 0$. The elastic analysis shows that the major principal stress will rotate as the vertical stress is greater than the horizontal stress at depth, whilst the converse is true at the top of the wall. However, as $\Delta\sigma_1$ and $\Delta\sigma_3$ are interchangeable in equation (3), the value predicted for the pore pressure increment will be unaffected regardless of whether the horizontal or vertical stress is assumed to be the major principal stress. Some error may nevertheless result when the major principal stress is at an intermediate orientation between the two directions. Figure 10(a) shows the results for the roller compacting the silty clay fill with the roller edge at a distance of 0.1m from the retaining wall face. In this figure the appropriate corrections have been made for the stresses due to the self-weight of soil.

The high pressures recorded whilst the compaction plant is close to the wall are however only transitory, and stress levels will reduce when the plant has passed by. The total vertical stress will return to its original value calculated according to the self-weight of fill, whilst horizontal effective stresses up to a maximum value corresponding to the earth pressure at rest for unloading (K_o^1) will remain locked in[13,14]. Equation (3) can then be employed to determine the corresponding fall in pore water pressure. The results for silty clay are shown in Figure 10(b) and have been calculated assuming the A and B pore pressure coefficients remain constant throughout both the loading and unloading conditions.

By summating the effective stress and pore water pressure values, the total stress distribution is then calculated. However this is not the final stress distribution which will be recorded on the wall, because no account has been taken of the construction procedure in which backfilling and compacting occurs in layers. Allowance for this method of construction can be made by considering the outer envelope of the stresses in a similar manner to that reported by Broms[13]. The stress distribution predicted for the silty clay fill is compared with the experimental results in Figure 9. A reasonable fit was obtained between the theory and practice, suggesting that this approach may have an application in current design practice.

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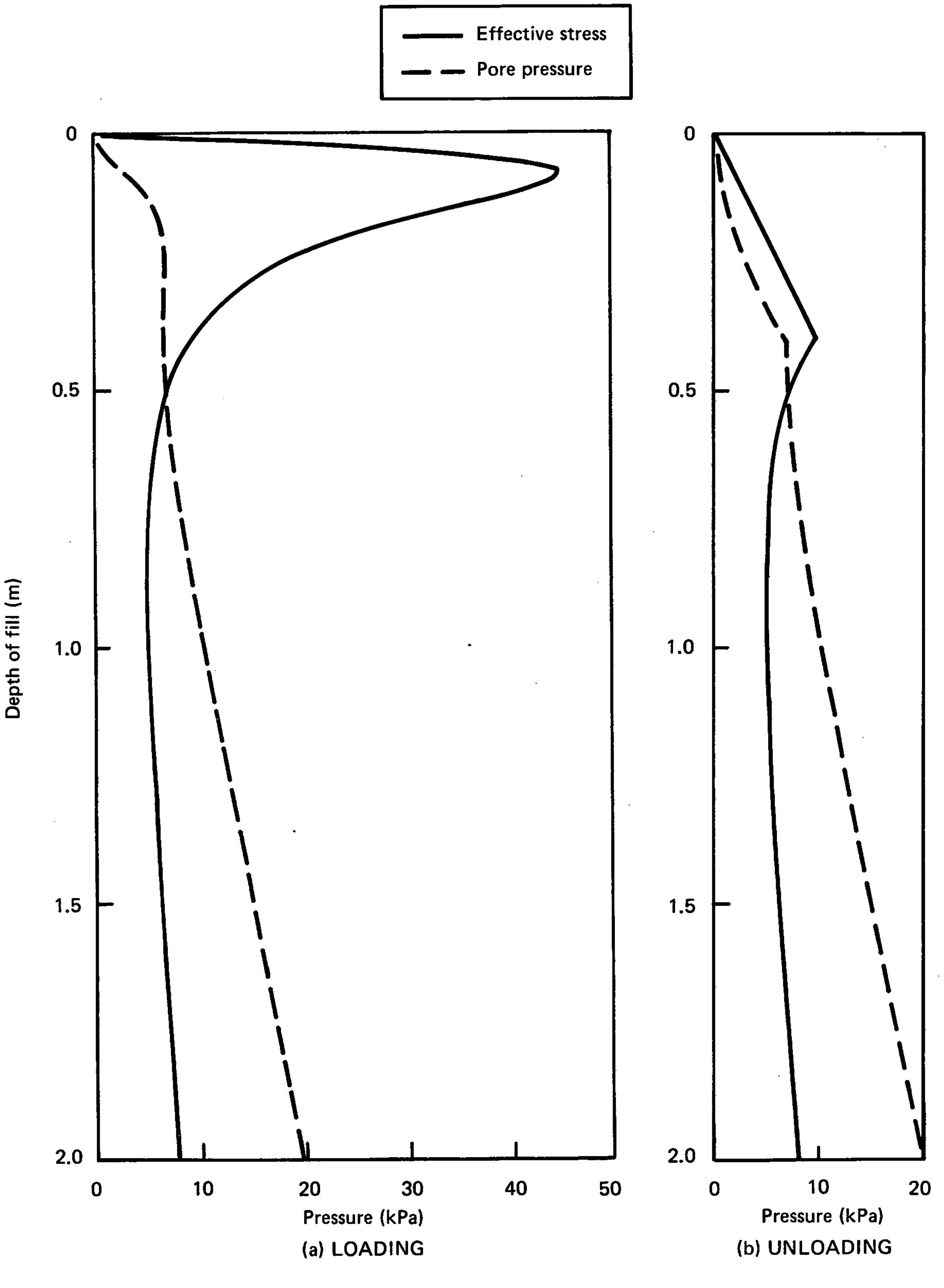

Fig.10 RESULTS FOR SILTY CLAY

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