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Analytical and Laboratory Investigations of Reinforced Clay

Recherches analytiques et en laboratoire sur l'argile renforcée

The current design theories for reinforced granular soils employ a discrete analysis in which soil and reinforcement are considered separately. This design philosophy could in principle be extended to reinforced clays where the short term stability might be assessed in terms of Cu the undrained shear strength of the clay. A novel deviation from this approach involves the concept of a composite theory in which the effects of the reinforcement are assumed to impart an equivalent undrained shear strength Cu'. Ideally structures could then be designed using existing total stress theories with Cu' substituted for Cu. To assess this possibility simple theories are developed to model plane-strain compression of a reinforced clay cube, reinforced clay foundations and finally a reinforced clay wall. Following this a series of model tests are described and test data presented to allow comparison with the theories. Such a comparison shows sufficiently reasonable agreement overall to warrant further research with the aim of defining and calibrating this potentially simple design technique.

Les théories actuelles d'étude des sols granuleux renforcés utilisent une analyse discrète suivant laquelle on examine séparément le sol et le renforcement.Cette philosophie de l'étude pourrait en principe être étendue aux argiles renforcées où la stabilité à court terme peut être évaluée en fonction du facteur Cu (la résistance à la déformation de l'argile non asséchée).Une déviation nouvelle à partir de cette approche implique le concept d'une théorie composite selon laquelle on suppose que les effets du renforcement transmettent une résistance à la déformation non asséchée équivalente Cu'.Dans l'idéal on pourrait céer des structures utilisant les théories existantes de tension totale en substituant Cu' à Cu. Pour juger de cette possibilité on développe des théories simples pour reproduire la compression sous tension plane d'un cube en argile renforcée,de fondations en argile renforcée et enfin d'un mur en argile renforcée.Après cela on décrit une série de tests modèles et on présente des données expérimentales pour permettre de les comparer aux théories.

INTRODUCTION

In designing reinforced soil structures employing non-cohesive fill the strength components of the soil and reinforcement are considered separately using discrete theory. Since drained conditions are deemed to prevail both during construction and the service life of the structure design is based on effective strength parameters. The same approach could in principle be adopted for the design of structures employing cohesive fill but this would only be relevant to the assessment of long term stability. Obviously in the short term, that is during and immediately after construction, design would need to be based on total stress analysis, employing amongst other parameters, the undrained shear strength of the fill Cu. Basic design philosophy would remain unchanged with the reinforcement being considered separately from the fill. A novel deviation from this discrete approach involves the concept of a composite theory in which the effects of reinforcement are assumed to impart an equivalent undrained shear strength Cu'. To quantify this possibility a simple theory was derived based on the plane-strain compression of clay between adhesive plattens. Subsequently plane-strain laboratory tests were carried out to provide test data for comparison with the theory with a view to extending this approach to other applications.

COMPOSITE THEORY

The basic theory developed relates to the plane-strain compression of clay between a pair of rigid adhesive plattens as depicted in Figure 1 which shows plattens of width B bounding an element of clay of thickness S.

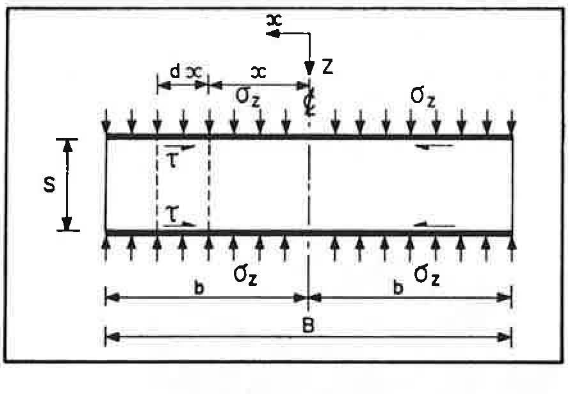

Fig1 Compression between Adhesive Plattens

Compression of these plattens under a vertical normal stress σ_z will ultimately lead to yielding of the clay of undrained shear strength Cu, which will attempt to displace laterally. This movement will mobilise restraining shear stresses between the clay and plattens. The magnitude of these shear stresses may be related to Cu by an adhesion factor α. Consideration of the equilibrium of an element of clay of width dx, Figure 1, leads to equation (1)

$$2\alpha Cu dx + S d\sigma_z = 0 \quad (1)$$

At the free edges of the plattens the vertical stress σ_z may be assumed to locally equal the compressive strength of the clay 2Cu. Thus integrating equation (1) and applying the boundary conditions $x=B/2, \sigma_z=2Cu$ leads to equation (2)

$$\sigma_z = 2Cu\left[1 + \frac{\alpha(B/2-x)}{S}\right] \quad (2)$$

The expression in equation (2) represents local values of σ_z. Further integration leads to the mean value given in equation (3) which is deemed to represent the compressive strength, p, of the composite.

$$p = 2Cu\ (1 + \alpha B/4S) \quad (3)$$

Extension of the normal convention for unconfined loading which defines compressive strength p=2Cu leads to the proposition that equation (3) may be used to represent Cu' the equivalent undrained shear strength of the reinforced clay composite, equation (4)

$$Cu' = Cu\ (1 + \alpha B/4S) \quad (4)$$

2 PLANE-STRAIN TESTS AND RESULTS

To provide test data to compare with the theoretical analysis a number of plane-strain tests were carried out on reinforced and unreinforced remoulded London Clay using a specially constructed 150mm plane-strain apparatus. The basic apparatus, shown in Figure 2, comprises a concrete cube mould with two opposite sides welded to a large rigid base plate and stiffened by heavy welded webs. During preparation of the sample the two remaining sides of the mould are bolted firmly in position. The London Clay employed was thoroughly remoulded by drying, pulverizing in a jaw crusher and subsequently a mechanical grinder, following which water was added to achieve an undrained shear strength of approximately 45kN/m². The clay was prepared well in advance of testing and double sealed in plastic bags to allow for equalisation of moisture content. Reinforcement, which was cut to the exact internal dimensions of the mould, was in the form of a plastic geogrid with a 6mm diamond shape mesh and a total structure depth of 4mm. With the four sides of the mould in position alternate layers of reinforcement and London Clay were placed and compacted to a bulk density consistent with full saturation. Each sample was constructed with a layer of reinforcement top and bottom with the intervening reinforcement layers being at a constant spacing within any one sample. With the sample formed two sides of the mould were unbolted and carefully removed following which a rigid 150mm square loading platten was positioned on top of the sample. The whole assembly was then located in a compression machine and loaded, through the platten, at a vertical rate of strain of 2% per minute. A record was made of the applied vertical pressure, p, and vertical axial strain. To assess the true undrained shear strength of the clay in each reinforced sample an unreinforced 150mm cube sample of clay was prepared from the same clay and at the same time as the reinforced sample. Sets of 76mm x 38mm diameter samples were subsequently taken from the unreinforced sample and tested over a range of cell pressures compatible with the range of vertical pressure p to take account of any lack of saturation. Although the inner surfaces of the plane-strain cell were highly polished and lubricated to obviate any strength enhancement induced by side friction this was no guarantee of compatibility between unreinforced strengths measured using this apparatus and 38mm diameter samples tested in the conventional triaxial apparatus. To quantify any difference the first two tests using the plane-strain apparatus were carried out using no reinforcement. When the resulting drained shear strengths were compared with those obtained using the triaxial test they were found to be only 8% higher and thus for the purpose of the investigation confirmed the compatibility of the two test methods. Following this four tests were carried out with reinforcement at nominal spacings of 19mm, 25mm, 38mm and 50mm. The resulting measured values of p at failure were normalised by dividing by 2Cu the corresponding unreinforced compressive strength measured using the triaxial apparatus. Reinforcement spacing, S, was also rendered dimensionless by dividing by platten width B. These test results are shown in Figure 3 together with the theoretical line obtained from equation (3). The adhesion factor of 0.89 employed in the evaluation of equation (3) was obtained by direct measurement using a 60mm square shear box. Since the comparison of theoretical and test data showed promise it was decided to extend the basic composite theory to the specific cases of reinforced clay foundations and walls with a view to conducting laboratory tests to model these applications and provide test data for comparison.

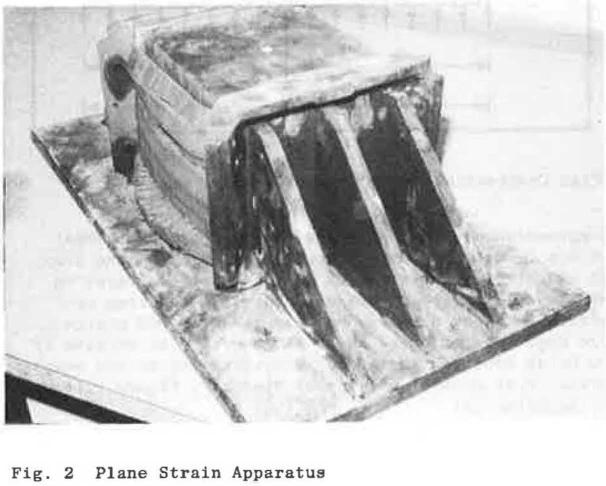

Fig. 2 Plane Strain Apparatus

3 FOUNDATION THEORY

The generation of a composite theory for reinforced clay foundations deviates from that of the previous simple theory in two fundamental points. Firstly since there is diminution of applied vertical stress with depth the

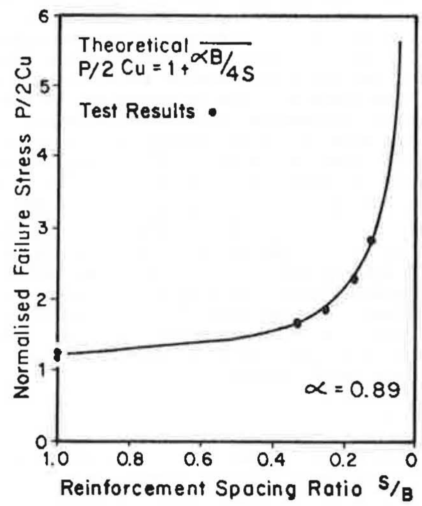

Fig. 3 Plane Strain Test Results

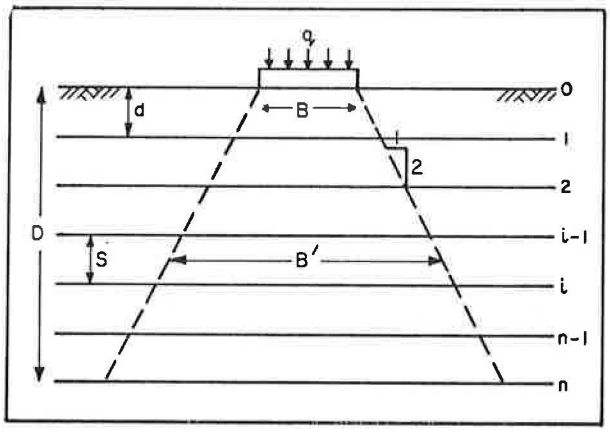

Fig. 4 Diagrammatic Representation of Model Footing

stress intensity at each level of reinforcement will be different. Secondly, since the width of the reinforcement in the clay is large compared with the width of the footing the clay cannot be considered to be unconfined at any horizontal boundary. The first of these problems can be overcome by making an assumption regarding the vertical stress distribution. In the theory presented a very simple 1:2 spread has been taken, Figure 4, which shows that the effect of assuming this, or indeed any other distribution, is to effectively increase the platten width with depth. Consequently it is necessary to define a mean effective platten width over the depth of the reinforced zone D. It follows from Figure 4 that the effective platten width B' between the (i-1)th and ith reinforcing layer is:

$$B' = B + (i - \tfrac{1}{2})S$$

For n layers of reinforcement the mean effective platten width is then given by equation (5)

$$\bar{B} = B + \frac{1}{n}\sum_{1}^{n}(i-\tfrac{1}{2})S = B + \tfrac{1}{2}nS \qquad (5)$$

Obviously the magnitude of n is known, since for a reinforced zone of depth D with a regular reinforcement spacing S it follows that $n = D/S$.

Having defined a notional platten width $\bar{B}$ consideration may be given to the internal equilibrium of an element such as that shown in Figure 1. This leads to the same generic equation, namely equation (1), which can again be integrated, however, the boundary condition $x=\bar{B}/2$, $\sigma_z=2Cu$ will not apply since in the case of a foundation the clay is confined at the boundary $x=\bar{B}/2$. If the general boundary condition $x=\bar{B}/2$, $\sigma_z=(2+\beta)Cu$, is substituted then equation (1) can be integrated as before to render the mean value of σ_z. This results in equation (6) which is of similar form to equation (3)

$$q = 2Cu\,(1 + \alpha\bar{B}/4S + \beta/2) \qquad (6)$$

Now if the foundation soil were unreinforced equation (6) would reduce to $q=(2+\beta)Cu$ whence taking a direct correspondence between this and the classical expression $q=(2+\pi)Cu$ leads to $\beta=\pi$ and equation (7)

$$q = 2Cu\,(1 + \alpha\bar{B}/4S + \pi/2) \qquad (7)$$

Applying the classical bearing capacity coefficient $N_c=(2+\pi)$ to the equivalent undrained shear strength leads to equation (8)

$$q = N_c Cu' = 2Cu(1 + \alpha\bar{B}/4S + \pi/2) \qquad (8)$$

Evoking equation (5) to allow substitution for $\bar{B}$ and remembering that $n=D/S$ allows equation (8) to be rearranged to form an expression for equivalent undrained shear strength

$$Cu' = Cu\left[1 + \frac{\alpha(B+D/2)}{2S(2+\pi)}\right] \qquad (9)$$

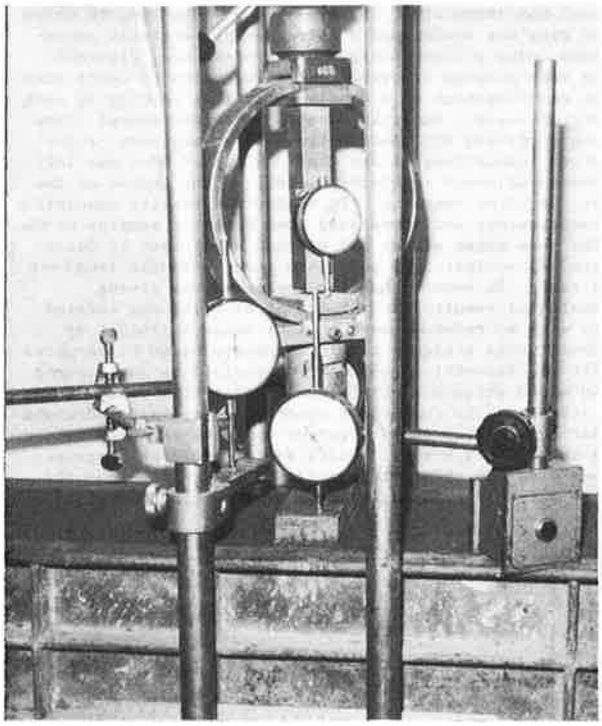

Fig. 5 General View of Model Footing Apparatus

4 FOUNDATION TESTS AND RESULTS

Model footing tests were conducted using the same clay and geogrid reinforcement employed in the earlier plane strain cell tests. The apparatus consisted of a rigid

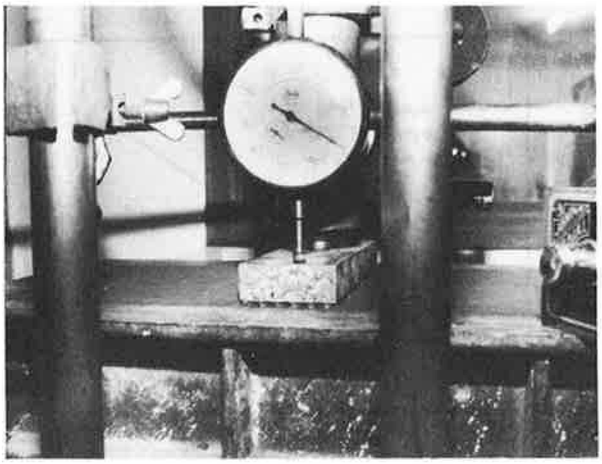

Fig. 6 Detail of Model Footing

steel box 150mm wide, 150mm deep and 710mm long in which the clay was loaded under undrained plane-strain conditions using a rigid strip footing 50mm wide, Figure 5. The test program involved conducting pairs of tests with the reinforcement at a constant vertical spacing in each pair of tests. Employing sheets of reinforcement 325mm long permitted the construction of two adjacent reinforced foundations at one time. A gap of 60mm was left between adjacent reinforced zones, at the centre of the box, to allow sampling. To render the results completely dimensionless and normalised 38mm diameter samples of the clay were taken at the end of each model test to determine any variation in undrained shear strength from test to test. To ensure that the apparatus was giving meaningful results the first pair of tests was carried out with no reinforcement so that measured values of unreinforced ultimate bearing capacity could be compared with the theoretical value N_cCu obtained using measured undrained shear strengths and the classical value of $N_c=(2+\pi)$. This comparison rendered an average undrained bearing capacity coefficient of 5.0 with measured values in the range 4.9 to 5.1 which was in acceptable agreement with the theoretical value of 5.14. Subsequently four pairs of reinforced tests were carried out with reinforcement at constant spacings of 19mm, 25mm, and 50mm. As well as using reinforcement at the prescribed vertical spacings within the clay a layer of reinforcement was attached to the underside of the footing to render a rough base, Figure 6.

For comparison with the theory summarised in equation (9) the measured failure stress q was normalised by dividing by N_cCu using measured values of Cu for each test. The corresponding theoretical plot of q/N_cCu versus S/B is given in Figure 7 together with the normalised test results. As can be seen the agreement between theory and test data is reasonable. The results were further analysed by plotting the normalised bearing capacity ratio, R, this being the ratio of reinforced to unreinforced bearing capacity, against number of reinforcing layers, n, Figure 8, and depth ratio d/B, where d is the depth to the top reinforcement, Figure 9. Consideration of Figure 8 shows bearing capacity ratio generally increasing with number of reinforcing layers as might be expected, however, at low settlement ratios, namely 5%, for $n<5$ the reinforcement appears to weaken the foundation as indicated by bearing capacity ratios less than unity. This tendency is repeated in Figure 9 which shows $R<1$ for $d/B>0.65$ and ρ/B =5%. A more significant

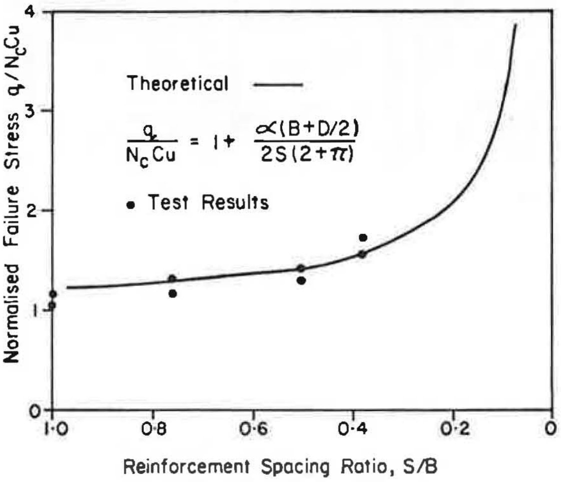

Fig. 7 Model Footing Test Results(q/N_cCu vs. S/B)

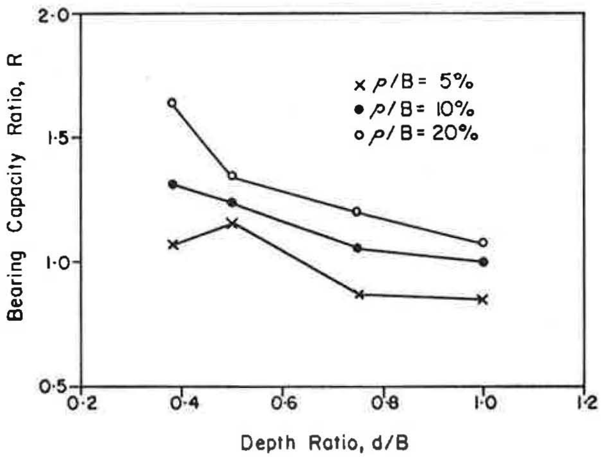

Fig. 9 Model Footing Test Results (R vs. d/B)

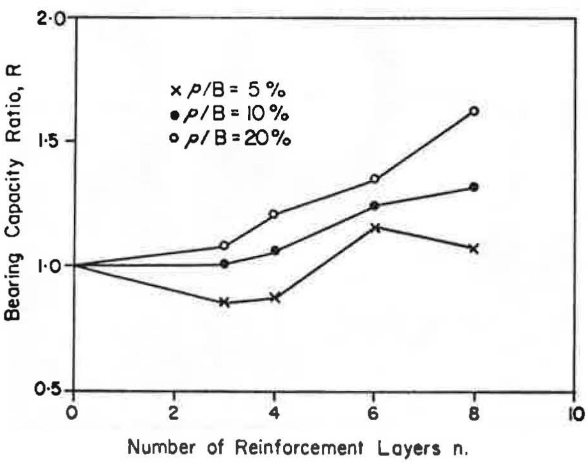

Fig. 8 Model Footing Test Results (R vs. n)

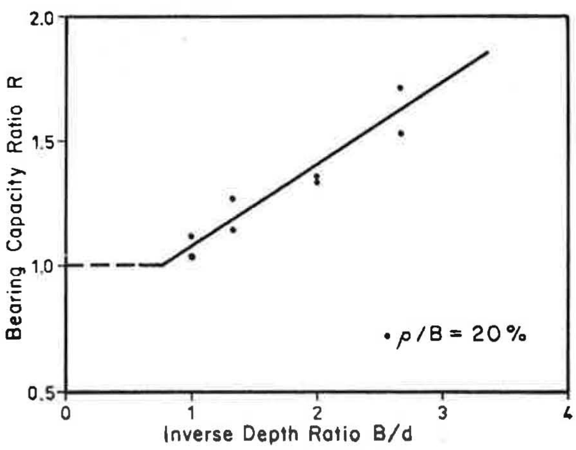

Fig. 10 Model Footing Test Results (R vs. B/d)

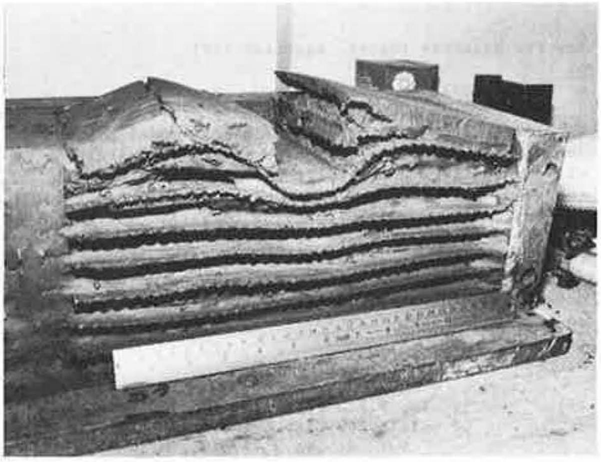

Fig. 11 Detail of Failure Mode for n = 8

relationship emerges from Figure 10 which shows a plot of bearing capacity ratio against B/d for $\rho/B=20\%$. Accepting that at this settlement ratio R would not be less than unity Figure 10 indicates that for $B/d<0.75$ the reinforcement has no effect, however, as B/d increases, that is the top reinforcement rises nearer to the foundation, there is an approximately linear increase in bearing capacity ratio. Since B/d appears to be of major significance the theory presented earlier would need to be extended to allow for $d \neq S$. Additionally there is a need to optimise the number of reinforcing layers. This is evident from Figure 11 which shows the failure mode for eight layers of reinforcement. As can be seen the lowest three or four layers of reinforcement do not appear to have contributed to enhancing bearing capacity. In this case the effective depth, D, of the reinforced zone would be less than assumed in the theoretical evaluation. If D was decreased this would have the effect of slightly lowering the theoretical line shown in Figure 7.

5 WALL THEORIES AND TEST RESULTS

Since the wall theories developed here are, to an extent, a function of the test method subsequently described it is appropriate to present the two simultaneously. In order to put composite theory in clearer focus it is presented in parallel with conventional, or discrete, theory with both of these theories being compared with test results obtained. To assess the theories a series of simulated wall tests was carried out using Kaolin clay again reinforced with plastic geogrids. Due to the impracticality of bringing a laboratory model to failure by self weight only the simulated walls were failed under the application of a vertical surcharge Figure 12. To minimise the vagaries of subsequent total stress analyses the surcharge was applied using a rigid platten that has the effect of inducing failure along a preselected plane. The justification for this and detailed description of the apparatus is given elsewhere, Ingold (1). In essence the objective of the tests conducted was to observe the surcharge intensity, p kN/m^2, required to cause failure in walls with and without reinforcement. The walls, which were formed in a long, rigid, open ended box to maintain plane-strain conditions, were 150mm high and reinforced at 19mm, 25mm, 38mm or 75mm vertical centres.

One approach to predicting the magnitude of surcharge intensity to cause failure is to apply conventional theory which considers the restoring forces in a reinforced clay wall to be the sum of two discrete forces namely those due to the clay fill and those developed by the reinforcement. To demonstrate this Figure 12 shows a wall of height H with n reinforcing layers at a vertical spacing S. For the particular series of tests reported the reinforcement strength and geometry were selected to obviate tensile failure or pull-out from Zone II, Figure 12. Remembering that the wall has no facing units the remaining mode of failure involves pull-out of the reinforcement from Zone I. Assuming the generation of soil-reinforcement adhesion αCu the pull-out resistance of the ith layer is represented by equation (10)

$$T_i = 2(L-L_i)\,\alpha Cu = 2H\cot\theta\,\alpha Cu[(2i-1)/2n] \qquad (10)$$

The total horizontal restoring force T developed by n layers of reinforcement is obtained by summing equation (10) over n terms

$$T = nH\cot\theta\,\alpha Cu \qquad (11)$$

For the particular platten width employed, b=100mm, $\theta=45°+\phi'/2$ for which equation (11) reduces to equation (12)

$$T = nH\sqrt{K_a}\,\alpha Cu \qquad (12)$$

A theoretical value of p, the surcharge intensity at failure, can be obtained by a simple Coulomb total stress analysis incorporting T, equation (13)

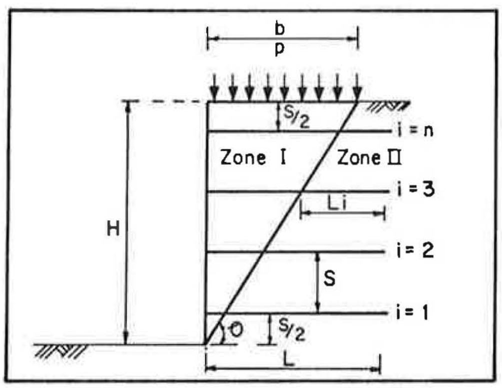

Fig. 12 Model Wall-Arrangement of Reinforcement

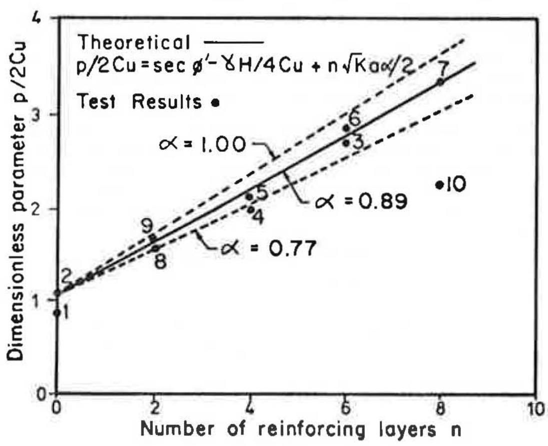

Fig. 13 Model Wall Test Results (p/2Cu vs. n)

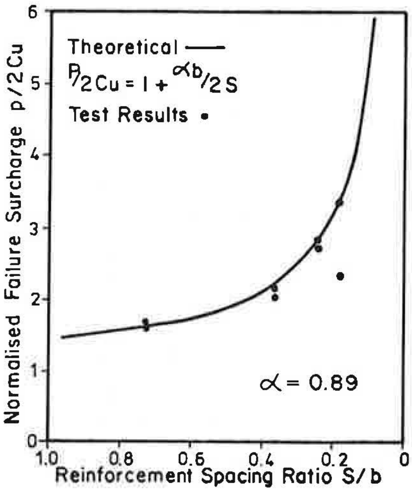

Fig. 14 Model Wall Test Results(p/2Cu vs. S/b)

$$p = 2Cu\sec\phi' - \tfrac{1}{2}\gamma H + n\sqrt{K_a}\,\alpha\, Cu \qquad (13)$$

This equation is rendered dimensionless by dividing throughout by 2Cu and is compared with test data in Figure 13. The range of adhesion factors cited are the maximum, mean and minimum values obtained from shear box tests.

A possible alternative to the above conventional or discrete analysis is composite theory which defines an equivalent undrained shear strength Cu'. Now for an unrenforced wall simple total stress theory predicts a failure surcharge intensity defined by equation (14)

$$p = 2Cu - \tfrac{1}{2}\gamma H \qquad (14)$$

From consideration of internal stability similar to that shown in Figure 1 it is possible to derive equation (15) which gives an expression for equivalent undrained shear strength

$$Cu' = Cu\,(1 + \alpha b/2S) \qquad (15)$$

Substitution of Cu' for Cu into equation (14) leads to equation (16)

$$p = 2Cu\,(1 + \alpha b/2S) - \tfrac{1}{2}\gamma H \qquad (16)$$

For the particular wall tests conducted the $\frac{1}{2}\gamma H$ term was approximately 1 kN/m^2 and was therefore neglected. The resulting theoretical expression is compared with the test results in Figure 14 and shows generally good agreement save for one test where the reinforcement became contaminated with grease thus causing premature failure.

A final reflection on the vagary of total stress analysis concerns the apparent compatibility between the discrete and composite theory if failure is considered to occur along a plane at 45° to the horizontal. In this case, which is equivalent to assuming a surcharge width of b=H as opposed to $b=\sqrt{K_a}H$, the discrete theory predicts a surcharge intensity at failure defined by equation (17)

$$p = 2Cu - \tfrac{1}{2}\gamma H + n\alpha Cu \qquad (17)$$

For a platten width b=H the composite theory may be used to define an equivalent undrained shear strength Cu' for the reinforced clay, equation (18)

$$Cu' = Cu(1 + \alpha H/2S) \qquad (18)$$

In the case of an unreinforced wall the surcharge p is defined by equation (14) as before. Remembering that H/S=n substitution of Cu' from equation (18) into equation (14) leads to an expression identical to that for the discrete theory, equation (17).

6 CONCLUSIONS

Simple theories have been developed to model plane-strain compression of a reinforced clay cube, reinforced clay foundations and finally a reinforced clay wall. Following this a series of model tests were described and test data presented to allow comparison with the theories. Although there are obvious shortcomings the comparisons in general show sufficiently reasonable agreement to warrant further research with the aim of developing and calibrating this potentially simple design technique.

REFERENCES

(1) Ingold, T.S. "A Laboratory Simulation of Reinforced Clay Walls" Geotechnique Vol. 31. No.3. 399-412. (1981).