

PULLOUT CAPACITY OF STEEL GRIDS IN LATERITIC SOIL BACKFILL

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SYNOPSIS

A study of the interaction behavior between lateritic soil backfill and steel grid reinforcements was made through a number of laboratory and field pullout tests. The diameter and spacing to diameter (S/D) ratios of the transverse bars; moisture content of the compacted soil; and applied normal stress levels affected the type of bearing capacity failure mechanism developed in front of the transverse bars, and, consequently, the magnitude of the pullout resistance. Our test data indicated that the frictional resistance over the longitudinal wires on average contributed only 10 to 15% of the total pullout resistance and the interference between the passive resistant zones of the transverse members seemed to be less significant for spacing to diameter (S/D) ratios greater than 60 to 100. Field pullout tests were also conducted on dummy reinforcements embedded at different elevations in a test embankment resting on soft clay foundations. The laboratory pullout tests, in general, were found to give conservative values of the pullout resistances compared to the field pullout tests. The proposed empirical prediction equations were found to have good agreement with the experimental data.

INTRODUCTION

The behavior of three different locally-available, cohesive-frictional soils namely: clayey sand, lateritic soil, and weathered Bangkok clay were studied during the pullout tests at the Asian Institute of Technology (AIT), to explore their potential for use as backfill materials in the construction of high and steep reinforced embankments. This paper discusses the interaction behavior between steel grids and lateritic soil backfill. Lateritic soils are abundant in many tropical countries including the Southeast Asian Region. These materials have been primarily used as construction materials (backfills) for embankments, highway fills, and dams. The lateritic soils are regarded as strong and excellent construction materials. Additional strengthening by way of earth reinforcement (mechanical stabilization) would be required for safe construction of steep and high embankments of soft ground conditions. Hence, welded steel grids are proposed for use as reinforcements in backfill soils to create a composite material strong in both tension and compression.

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INTERACTION BETWEEN REINFORCEMENT AND BACKFILL SOIL IN MECHANICALLY STABILIZED EARTH (MSE)

Soil is strong in compression, but weak in tension. To impart sufficient tensile strength to the soil, a composite construction material can be developed by the inclusion of a wide variety of materials which are strong in tension in various forms such as strips, sheets, grids etc. This composite construction material is commonly termed as mechanically stabilized earth (MSE). MSE is, therefore, a generic term applicable to all forms of soil reinforcing systems. MSE also possess improved compressive and shear strengths. MSE in its modern form was first applied to "Reinforced Earth" retaining walls (Vidal, 1969) which utilize granular backfills and horizontal steel strip reinforcements. Subsequently, the use of cheaper, low-quality, cohesive-frictional soils also attracted much attention with the advent of the grid reinforcements, especially the steel grid (welded steel bar) reinforcements. The problem of corrosion with these steel grid reinforcements can be tackled effectively by providing "sacrificial steel" or by providing galvanizing and ensuring appropriate backfill soils with good and uniform compaction during construction. The steel grid reinforcements, which are normally regarded as inextensible, can generate the required pullout resistance without undergoing large strains and also without the excessive mobilization of the soil shear strength. It was observed that the grid type of reinforcements can generate about 5 to 6 times greater pullout resistance than the corresponding value for the strip reinforcements (Chang et al. 1977; Peterson & Anderson, 1980; Nielsen & Anderson, 1984; Shivashankar, 1991). The pullout resistance of a steel grid reinforcement is derived largely from the passive resistance due to the soil bearing in front of the transverse members (F_b). The frictional resistance (F_f) over the longitudinal members constitute the rest. Therefore, the total pullout resistance (F_t) can be expressed as:

$$F_t = F_b + F_f \quad (1)$$

The transverse members of a grid reinforcement can be regarded as a series of deeply embedded strip footings which have been rotated by 90 degrees to the horizontal and pulled through the soil. Two types of bearing capacity failure model in front of the transverse members have been proposed. The first is the general bearing failure model (Peterson & Anderson, 1980) in which the slip planes are fully developed. The prediction equation for the pullout resistance for this case based on the Terzaghi-Buisman bearing capacity equation is as follows:

$$F_b/NWD = C^*N_c + \sigma_v^*N_q \quad (2)$$

$$\text{where: } N_q = e^{(\pi \tan \phi)} \tan^2 (45 + \phi/2) \quad (3)$$

$$\text{and } N_c = (N_q - 1)/\tan \phi \quad (4)$$

The symbols are defined in Appendix 1. This prediction seems to form an apparent upper bound envelope for the pullout capacities of the grid reinforcements (Palmeira & Milligan, 1989; Jewell, 1990; Shivashankar, 1991). The second model proposed by Jewell et al. (1984) is based on the punching shear failure mode of deeply embedded

foundations. The prediction equation for the bearing capacity factor, N_q , for this case based on the equation of Vesic (1963) is as follows:

$$N_q = e^{(\pi/2 + \phi) \tan \phi} \tan (45 + \phi/2) \quad (5)$$

This prediction seems to form an apparent lower bound envelope for the pullout capacities of the grid reinforcements (Palmeira & Milligan, 1989; Jewell, 1990; Shivashankar, 1991).

Ospina (1988) conducted pullout tests with radiographic measurements. A predominantly general bearing failure was observed with fine sands in their loose states, while a punching shear failure was observed in the dense states. It was also observed that by increasing the confining pressures, the failure mechanism approached a general bearing failure mechanism. Palmeira & Milligan (1989) concluded from their pullout tests using dry Leighton Buzzard sands that the failure model changes from punching shear to a generalized bearing failure when D/D_{50} exceeds 7.5, where D is the diameter of the transverse bar and D_{50} is the particle size of the backfill soil corresponding to 50% finer. It was observed that interference between the passive resistant zones of the bearing members become negligible for S/D values beyond about 50, where S is the spacing between the transverse members and D is as defined previously.

LATERITIC SOIL BACKFILL

The lateritic soil used in the present study was reddish brown in colour and classified as clayey to sandy gravel. It contained about 18% passing the U.S. sieve No. 200. The standard Proctor compaction test gave an optimum moisture content of 11.5% and corresponding maximum dry density of 19.3 kN/m³. The specific gravity of the soil particles was 2.61.

Direct shear tests were conducted at low normal pressures ranging from 2 to 18 kPa (Amin, 1989). A special direct shear apparatus with low friction based on the design of Onitsuka et al. (1987) was fabricated for this purpose. Macatol (1990) extended the study of Amin (1989) by conducting direct shear tests on the same lateritic backfill soil, but at higher normal stresses ranging between 10 to 130 kPa. The combined results of the above two studies suggested bilinear failure envelopes as shown in Figs. 1a to 1c for the dry side, optimum, and wet side compactions, respectively. The bilinear failure envelope was also reflected in the pullout test results. This can be attributed to the particle crushing phenomenon inherent in these lateritic backfill soils under high confining stresses.

LABORATORY PULLOUT TEST SET-UP AND TESTING PROCEDURE

Pullout tests were conducted in the laboratory at AIT in a pullout box with inside dimensions measuring 1.25 m × 0.75 m × 0.51 m (length × width × depth). The pullout box was made up of steel plates and rolled steel beams, built for this study, using both welded and bolted connections. A detailed and schematic view of

the laboratory pullout test set-up is shown in Fig. 2, the components of which are listed in Table 1.

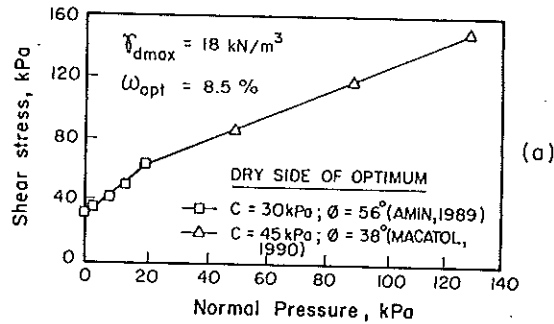


Fig. 1a Direct Shear Test Results of Lateritic Soil (Dry Side)

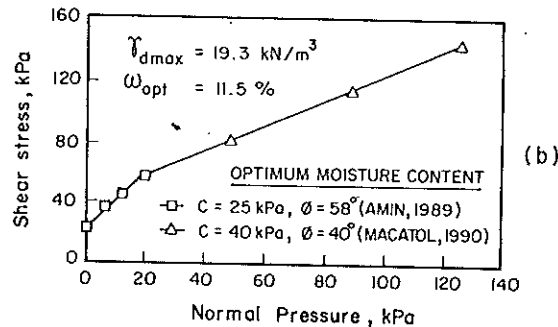


Fig. 1b Direct Shear Test Results of Lateritic Soil (OMC)

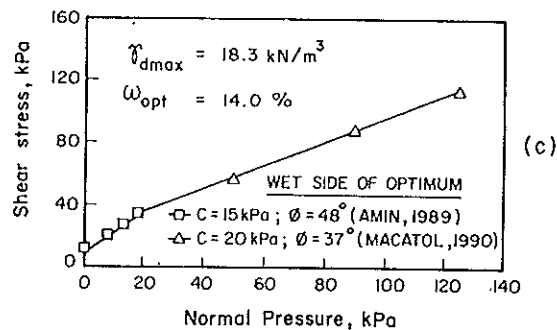


Fig. 1c Direct Shear Test Results of Lateritic Soil (Wet Side)

PULLOUT CAPACITY

Horizontal metal plates or sleeves were used on the inside part of the opening of the pullout box, extending 0.15 m behind the face, to decrease the horizontal stresses on the front face near the slot inside the pullout box during the pullout and to minimize the arching effects over the grid specimens, as shown in Fig. 2. These sleeves were positioned across the full width of the pullout box, above and below the reinforcements for a length of 0.15 m near the front. The top metal sleeve rested on spacers provided at the front corners of the pullout box to avoid direct contact with the reinforcements.

In each set-up, the soil was compacted in two equal lifts of 0.15 m thickness each using a hand (Wacker) compactor. After compacting the first lift, the level of the compacted soil surface reached nearly up to the center of the pullout slot. The grid specimen was then placed in position at the center of the slot and checked for its horizontality and its alignment. Then, the second lift of soil was placed over it and compacted again. The uniformity in the degree of compaction and moisture contents in both lifts was monitored using a Troxler nuclear gage densitometer and sand cone method.

Pullout tests in the laboratory were conducted on the dry side and on the wet side of optimum compacted to 95% of the standard Proctor density. A variation of $\pm 2\%$ about the mean value was observed on the degree of compaction, while a corresponding variation of $\pm 1\%$ was recorded in the moisture contents (Fig. 3). Pullout tests were also done at optimum moisture contents.

A multi-stage pullout testing program was followed. In each set-up, three pullout tests in three corresponding stages were conducted by increasing the vertical

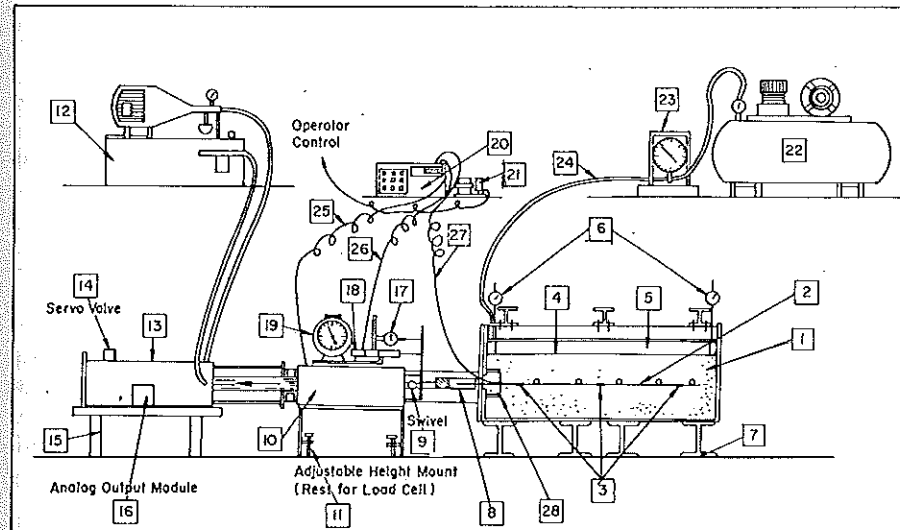


Fig. 2 Schematic of the Laboratory Pullout Test Apparatus

Table 1 Components of the Pullout Test Set-up (refer to Fig. 2).

1. Compacted soil in the pullout box
2. Grid specimen
3. Strain gauges
4. Flexible plate placed over the compacted soil
5. Air bags for applying normal pressures
6. Dial gauges at top to measure dilatancy
7. Airloc pads
8. Clamping mechanism
9. Swivel joint
10. Load cell
11. Mount for load cell, of adjustable height
12. Piston pump and motor with filters and valves
13. Hydraulic cylinder
14. Servo valve
15. Mount for the cylinder
16. Analog output module
17. Front dial gauges
18. LVDT for measuring front displacements
19. Stop clock
20. 21X datalogger with multiplexer
21. EMRS power supply module and EMD amplifier module
22. Air compressor
23. Regulator
24. Stiff tubing connecting air compressor to air bag through regulator
25. Lead wire connecting load cell to datalogger
26. Lead wire connecting LVDT to datalogger
27. Lead wires connecting all the strain gauges to the datalogger through the multiplexer
28. Horizontal sleeve

normal stress at each stage. In the first stage, the mat was pulled out under a given normal stress. After this, the pulling force was released and the normal stress was increased for the next stage. After allowing the normal stress to stabilize for about 30 minutes in the subsequent stages (second or third), the mat was pulled again. At each stage the mat was pulled out by 25.4 mm. Usually, the maximum pullout force is obtained after a pull of 8 to 10 mm (see Fig. 5).

In the case of the lateritic soils, a new batch of material was used each time at the different test set-ups of the pullout tests and also in the shear strength tests. After compacting the second lift in the pullout box, a thin layer of clean and dry sand,

PULLOUT CAPACITY

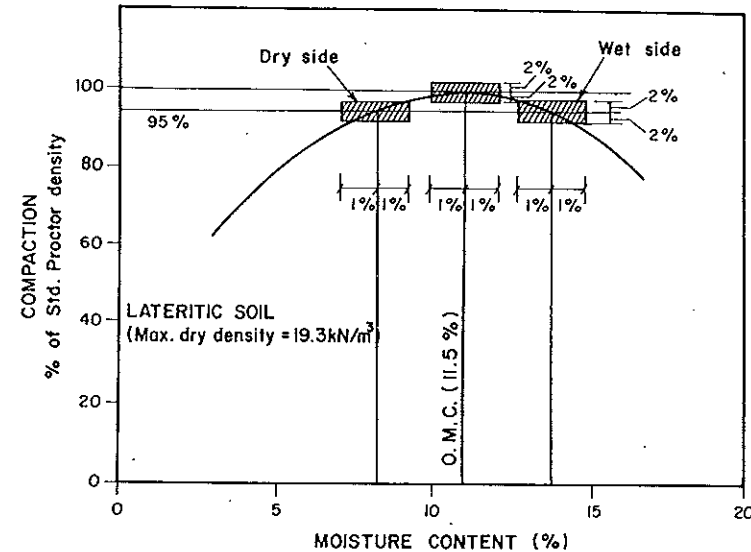


Fig. 3 Range of Moisture Contents and Compaction Densities Used in the Pullout Tests

about 25 to 50 mm thick was placed over it to even up the surface. The lower top flexible cover plate, 6.35 mm thick, was then placed over the sand layer. Inflated paper air bags placed on the lower flexible plate were used to apply the normal stresses. These air bags would take reaction from I-beams above it bolted on to the top of the pullout box through an upper top cover plate.

A uniform pullout rate of 1 mm/min was used and maintained with the help of electronic controls, dial gage and a stop clock. Two strain gages were fixed at each instrumentation point on the grid specimen, diametrically opposite of each other to cancel out the bending stresses. There would be three instrumentation points on a longitudinal wire of a grid specimen, at varying distances from the face of the pullout box, and one instrumentation point on a transverse wire. A seating load of about 0.1 to 0.2 tons (1 to 2 kN) was applied at the beginning to remove any slack in the system. Pullout tests in the laboratory were conducted using three types of reinforcements as follows:

- (a) Only the four longitudinal bars spaced at 0.15 m laterally (Fig. 4a), consisting of different bar diameters and surface characteristics as galvanized and smooth, non-galvanized (black) and smooth. These tests are termed as the "friction pullout tests".
- (b) Only the four longitudinal bars with transverse ribs on them (Fig. 4b). In this case, the portion of the transverse bars between the longitudinal bars of a grid reinforcement were clipped. Therefore, only the transverse ribs on the longitudinal bars were left in place. The length of these ribs were kept at twice the diameter of

the longitudinal bars. These tests are termed as the *ribbed friction pullout tests*. These tests were conducted for comparison with the results of the field pullout tests.

- (c) Welded-wire grid reinforcements of varying bar sizes and mesh geometries (Fig. 4c). These tests are termed as the "*grid pullout tests*". Typical results of pullout tests using lateritic soils as backfill materials are shown in Fig. 5 using the configuration in Fig. 4c.

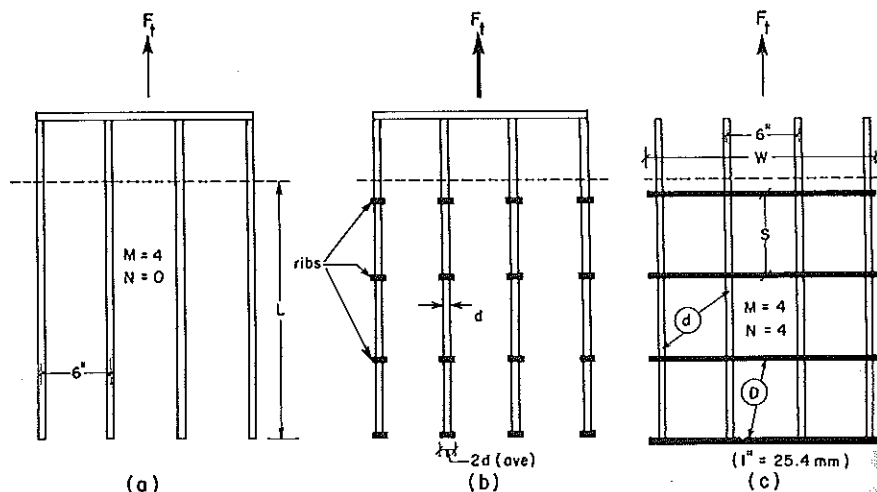


Fig. 4 Types of Reinforcements Used in the Pullout Tests

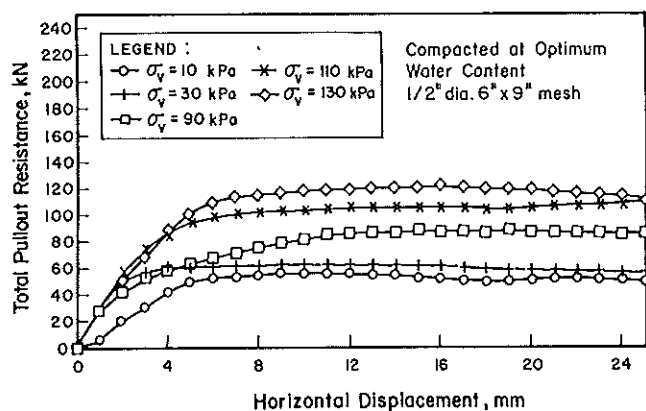


Fig. 5 Typical Pullout Force-Displacement Relationship

FULL-SCALE MSE TEST EMBANKMENT

A full-scale mechanically stabilized wall/embankment (MSE) system with welded steel geogrid reinforcements was constructed inside the campus of the Asian Institute of Technology (AIT), located about 42 kms north of Bangkok (Bergado et al. 1991a,b). It has a vertical welded steel grid facing in front and a sloping back. It consisted of three sections corresponding to the three different backfill materials used, namely: clayey sand in section I, lateritic residual soil in section II and weathered Bangkok clay in section III. The backfill materials were compacted to 95% standard Proctor densities on the dry side of optimum (see Fig. 3). Dummy reinforcement specimens, protruding from the vertical face of the wall/embankment system were installed during construction, at different levels in all the three sections, for field pullout tests. The foundation subsoil consists of a layer of soft clay, about 6 m thick, sandwiched between a surficial 2 m thick layer of weathered Bangkok clay and an underlying 6 m thick layer of stiff clay.

FIELD PULLOUT TESTS

Constant strain (constant displacement rate) field pullout tests were conducted about 8 months after the construction of the test embankment. By this time, both the foundation subsoil and the wall/embankment system had undergone substantial vertical and lateral movements. Five dummy reinforcement specimens embedded in lateritic soil section were pulled out. A typical dummy reinforcement is shown in Fig. 6. The pullout test procedure followed was the same as adopted in the laboratory pullout tests. The pullout force was applied by means of a specially designed reaction frame butting against the wall face. A wooden platform was built to support the pullout equipment.

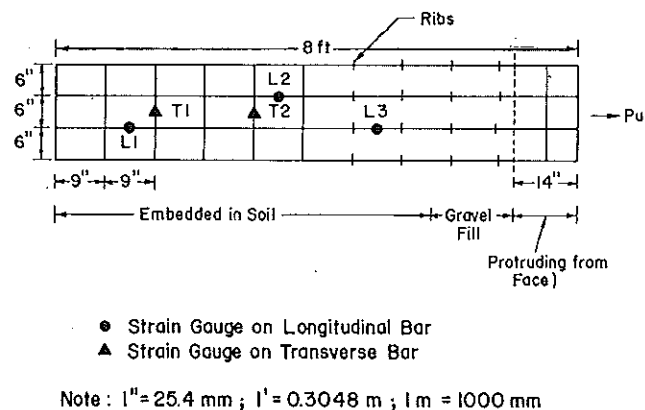


Fig. 6 Typical Dummy Mat Used in the Field Pullout Tests

FRICTIONAL PULLOUT RESISTANCES

It was observed that the frictional resistances over the longitudinal wires contribute on average of only 10 to 15% of the total pullout resistances of the grid reinforcements (Shivashankar, 1991). Figure 7 shows the results of the laboratory friction pullout tests at all the three compaction moisture conditions, namely: the dry side of optimum, at OMC, and on the wet side of optimum. It was observed that the frictional resistances increased with the increase in the normal stress levels at all three moisture conditions. However, the frictional resistances were found to decrease with the increase in the bar sizes at all three moisture conditions. This is because, the larger diameter bars by virtue of their greater rigidity may tend to rebound upon the withdrawal of the compacting plant after compaction which may result in the reduction of the contact area with the surrounding soil, thereby, lowering the frictional pullout resistances. On the other hand, the smaller diameter and less rigid bars may remain undulated even after the compacting plant has been removed. This may result in somewhat intact contact areas and, thus, greater frictional pullout resistances. Furthermore, the frictional resistances were found to decrease with the increase in the moisture content as expected.

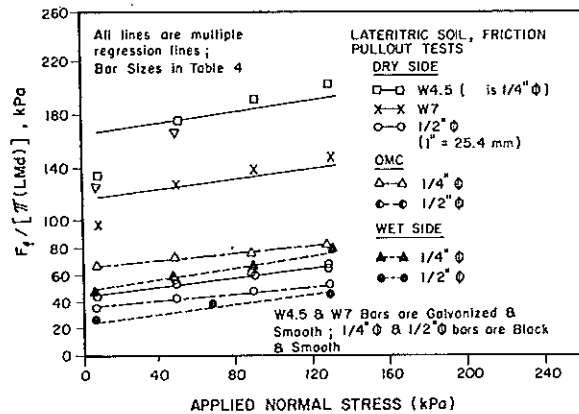


Fig. 7 Friction Pullout Test Results at Different Compaction Moisture Conditions

APPARENT FRICTION COEFFICIENT

The apparent friction coefficient, μ^* , as defined in Equation 6 was found to decrease with the increase in the normal stress level. In Fig. 8, the curves flatten out beyond a normal stress level of about 100 kPa. The apparent friction coefficient has been defined in this study as:

$$\mu^* = \frac{f}{\sigma_{ave}} \tag{6}$$

$$\text{and } f = \frac{F_f}{M \pi d l} \tag{7}$$

where F_f is the maximum pullout force from the friction pullout tests. The friction force usually reached the peak after the pullout displacement of 2 mm.

The average overburden pressure, σ_{ave} , was taken as 1.15 times the applied vertical normal stress (σ_v) to account for the compaction induced stresses and also the circular cross section of the longitudinal wires assuming lateral at-rest pressure coefficient, K_o , of 1.3. Compaction is said to represent a form of overconsolidation (Duncan & Seed, 1986). An OCR value of 8 was found to be quite appropriate for the compacted lateritic soil, which corresponds to a lightly overconsolidated soil (Dunn et al. 1980; Bergado et al. 1991 a,b).

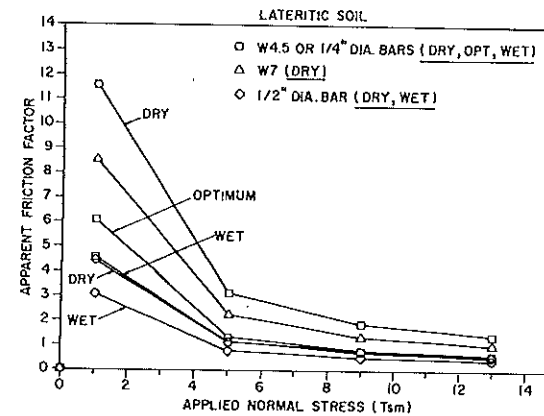


Fig. 8 Variation of the Apparent Friction Coefficient with the Applied Normal Stress Level at Different Moisture Conditions

BEARING PULLOUT RESISTANCES

The bearing resistances in front of the transverse members were obtained by subtracting the corresponding frictional resistances (from the friction pullout tests) from the total pullout resistances (from the grid pullout tests). These bearing resistances, represented by F_b/NWD , were found to increase with the increase in the spacing to diameter (S/D) ratios of the transverse members of the grid reinforcements. These results were found to be in between the upper and lower bound envelopes proposed by Peterson & Anderson (1980) and Jewell et al. (1984), respectively. With the smaller diameter transverse bars at larger spacings, i.e. larger (S/D) values, the pullout resistances at the end of 25.4 mm pull tended to approach the upper bound prediction of the general bearing failure mechanism. On the other hand, in the case of the grid mats with closely spaced larger diameter transverse wires i.e. smaller (S/D) values, the pullout resistances at the end of 25.4 mm seemed to approach the lower bound prediction of the punching shear failure mechanism.

DEGREE OF INTERFERENCE

Figure 9 shows a decrease in the degree of interference (DI) with the increase in the (S/D) ratio. Although there is some scatter in the data, the trend is clearly seen. This interference between the passive resistance of the adjacent transverse bars in the grid reinforcement are less significant for S/D ratios greater than about 60 to 100 depending on the compaction moisture contents. In calculating the values of DI from the grid pullout test data, the maximum bearing resistance that can be generated in front of the transverse bars with no interference ($(F_b/NWD)_{lt}$) was obtained from

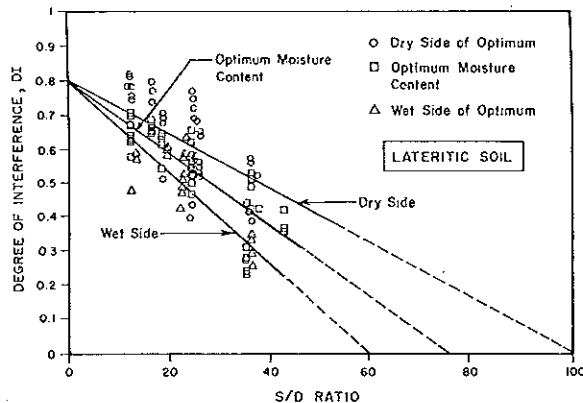


Fig. 9 Degree of Interference (DI) versus (S/D) Values from the Laboratory Pullout Tests

Equations (2) to (4). The degree of interference (DI) from the grid pullout test data as shown in Figs. 9 was calculated as:

$$DI = 1 - \frac{(F_b/NWD)_{ave}}{(F_b/NWD)_{lt}} \tag{8}$$

where $(F_b/NWD)_{ave}$ is the average bearing resistance mobilized in front of all the transverse bars of the grid reinforcement obtained from the grid pullout test, and $(F_b/NWD)_{lt}$ is the upper bound value for the bearing resistances. The first transverse bar will have no interference effects and the bearing resistance mobilized in front of the other transverse bars $(F_b/NWD)_{grid}$, are proportional to the S/D ratios of the grid reinforcement. Therefore, the degree of interference (DI) can also be expressed in terms of only the grid parameters, for a grid with N number of transverse bars, similar to Jewell (1990) as:

$$DI = 0.8 - \frac{(S/D)}{(S/D)_{lt}} \tag{9}$$

where $(S/D)_{lt}$ varies for different water content.

PREDICTION EQUATIONS

The frictional resistance, F_f , has been expressed as:

$$F_f = \pi L M d \{C_a^P + (\sigma_{ave}) \tan(\delta^P)\} \tag{10}$$

Dividing both sides of Equation (10) by NWD yields the following expression:

$$F_f/NWD = \pi S_f \{C_a^P + (\sigma_{ave}) \tan(\delta^P)\} \tag{11}$$

where S_f is the grid shape factor. The terms C_a^P and $\tan \delta^P$ were obtained by a simple linear regression of the friction pullout test data with F_f/NWD as the dependent variable and the average overburden pressure, $\sigma_{ave} = 1.15 \sigma_v$, as the independent variable. The values of C_a^P and $(\tan \delta^P)$ obtained for lateritic soil were, respectively, as follows: 92 kPa and 0.25 on the dry side, 51 kPa and 0.15 at optimum moisture content, and 38 kPa and 0.23 on the wet side.

The bearing resistance in front of the transverse bars has been expressed in the general form as:

$$F_b/NWD = (C N_c + \sigma_v N_q) (1 - DI) \tag{12}$$

The term F_b/NWD represents the bearing resistance per unit area of the transverse members normal to the direction of pullout. The bearing capacity factors, N_q and N_c , were obtained from Equations 3 and 4, respectively. The shear strength parameters were derived from the direct shear tests in Fig. 1a,b,c. The degree of interference (DI) in the above prediction equation (Equation 12) can be estimated from

Equation 9, by considering a suitable value of the $(S/D)_t$. The prediction lines which are also shown in Figs. 10 to 12 were found to have good agreement with the experimental data. The empirical relationship for total pullout resistance in the case of lateritic soil and the particular range of stresses can be expressed in the general form as:

$$F_p/NWD = \{[(C N_c + \sigma_v N_q) (1 - DI) + \pi S_f \{C_a^P + (\sigma_{ave}) \tan(\delta^P)\}]\} \quad (13)$$

DISCUSSION ON THE GRID PULLOUT TEST RESULTS

On the dry side of optimum and also at the optimum moisture content, at low normal stresses, the data points for low S/D values agreed well with the punching shear failure model. Even at high normal stresses, especially when the S/D values are low, the data points are somewhat closer to the lower bound prediction. This can be attributed to the particle crushing phenomenon. The increase in the moisture contents and (S/D) ratios, in general, seemed to steer the pullout failure mechanism in front of the transverse bars towards the general bearing failure model (Figs. 10 to 12). The typical grain size distribution curves were obtained before and after the pullout tests with the lateritic backfill soil. The samples for the grain size distribution after completing the pullout tests were taken from within the grid apertures in front of the transverse members. It was observed that the percent passing the U.S. sieve No. 200 increased. Thus, the D/D_{50} values ranged between 1 before the start of the pullout test to about 6 at the end of pullout test in the vicinity of the transverse bars.

The pullout capacities at the end of 25.4 mm pull, showed a small increase, with the increase in the compaction moisture contents from the dry side of optimum (8.5%) up to the optimum moisture content (11.5%). However, it dropped considerably with further increase in the moisture contents in the wet side of optimum (14%).

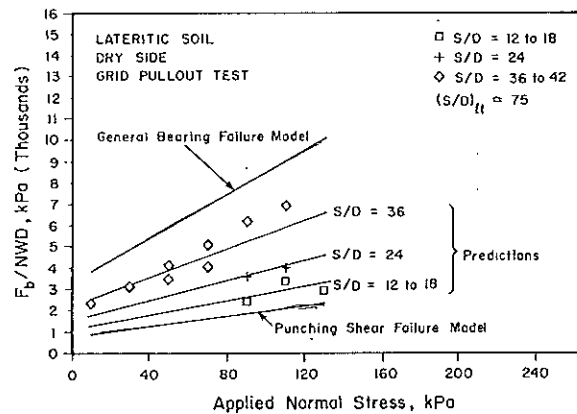


Fig. 10 Grid Pullout Test Results on the Dry Side

PULLOUT CAPACITY

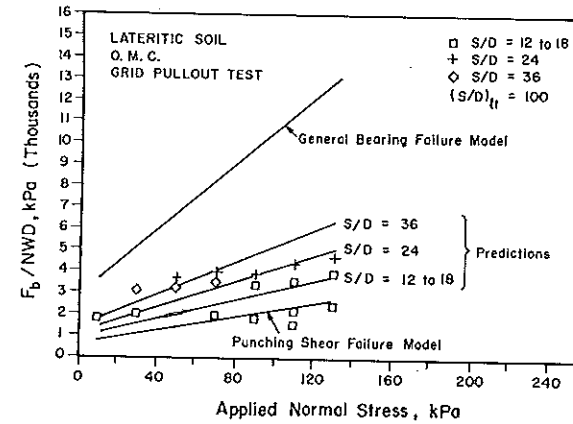


Fig. 11 Grid Pullout Test Results at the Optimum Moisture Content

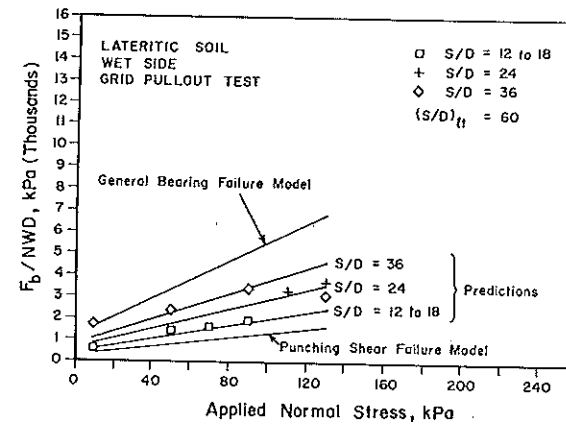


Fig. 12 Grid Pullout Test Results on the Wet Side

In the field condition, the arching effects, especially, in the lower portions of the middle lateritic section of the AIT wall, due to the complex subsoil movements and the presence of the inextensible reinforcements, severely affected the field pullout resistances (Shivashankar, 1991; Bergado et al. 1991 a,b; Bergado et al. 1992). Due to the arching effects, the field pullout resistances of the middle lateritic section were found to decrease with the increase in the overburden pressures, unlike in the two outer sections, and contrary to the theoretical expectations and what was observed from the laboratory pullout tests (Fig. 13). Also, due to the arching effects, the weathered clay and clayey sand backfill soils gave higher pullout resistances from the field pullout tests than from the laboratory pullout tests (Bergado et al. 1992).

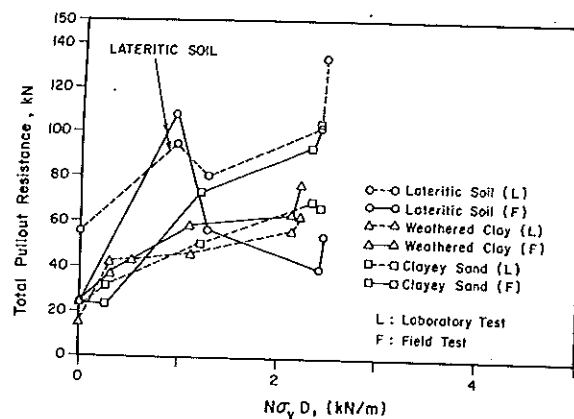


Fig. 13 Comparison between Laboratory and Field Pullout Test Results

CONCLUSIONS

Based on the results obtained from this study, the following conclusions can be drawn:

- 1) In the case of the lateritic soil and the particular range of stresses, the proposed empirical equations were found to have good agreement with the experimental data. The equation for total pullout resistance is given as follows:

$$F/NWD = (C N_c + \sigma_v N_q) (1 - DI) + \pi S_f \{C_a^p + (\sigma_{ave}) \tan (\delta^p)\}$$
- 2) From the test data, the bearing resistance in front of the transverse members of the grid reinforcement constitute about 85 to 90% of the total pullout resistance, while the frictional resistances over the longitudinal wires constitute the rest.
- 3) With the lateritic backfill soil, the type of pullout failure in front of the transverse members of the grid reinforcement tends towards the general bearing failure model when using smaller diameter transverse bars, increasing spacing to diameter (S/D) ratios of the transverse bars; and increasing compaction moisture contents. Otherwise, the pullout failure tends towards the punching shear failure model.
- 4) Interferences between the passive resistant zones of the adjacent transverse bars in the grid reinforcement are less significant for S/D ratios greater than about 60 to 100 depending on the compaction moisture contents.
- 5) Pullout capacities at the end of 25.4 mm pull during grid reinforcement pullout tests with lateritic backfill soil increased with the increase in the moisture contents up to the optimum moisture contents and then decreased considerably towards the wet side of optimum.

PULLOUT CAPACITY

ACKNOWLEDGEMENTS

This work formed a part of a three-year research project conducted at the Asian Institute of Technology (AIT), Bangkok, Thailand, in collaboration with Utah State University, U.S.A. The research project was funded by the U.S. Agency for International Development (USAID) to which unbounded gratitude is due. The steel grids reinforcements were donated by the Hilfiker Welded Wire Co. Ltd., U.S.A.

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APPENDIX I. NOTATION

- The following symbols are used in this paper:
- C' = effective cohesion intercept of backfill soil (direct shear test)
- C_a^p = adhesion value between the reinforcement and the soil (from pullout test)
- D = diameter of the transverse wires of the grid reinforcement
- D_{50} = particle size corresponding to 50% finer
- d = diameter of the longitudinal wires of the grid reinforcement
- F_b = bearing force in front of the transverse wires
- F_f = frictional force over the longitudinal wires
- F_t = total pullout resistance (force) of the grid reinforcement
- f = frictional resistance over the longitudinal wires
- L = length of embedment of the grid reinforcement in backfill soil
- M = number of longitudinal wires in the grid reinforcement
- N = number of transverse wires in the grid reinforcement
- N_c = bearing capacity factor
- N_q = bearing capacity factor
- S = spacing between the grid bearing members
- S_f = grid shape factor
- W = width of each transverse wire
- δ^p = angle of friction between the reinforcement and the soil from the pullout test
- μ^* = apparent friction coefficient
- ϕ = effective angle of internal friction of the backfill soil (direct shear test)
- σ_v = vertical normal stress
- σ_{ave} = average overburden pressure on the members of the grid reinforcement