Interaction of lateritic soil and steel grid reinforcement


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Received October 29, 1991

Accepted November 13, 1992

Laboratory and field pullout tests were carried out to study the interaction of welded steel grid reinforcements embedded in lateritic residual soil backfill. The laboratory pullout tests were conducted on various reinforcement sizes, mesh geometries, normal pressures, and compaction conditions of the backfill material. Field pullout tests were conducted at representative overburden, field-moisture, and density conditions. From the test results, it was found that the longitudinal members yielded frictional resistance from 8 to 15% of the total grid pullout resistance. Thus, the major contribution to the pullout resistance of grid reinforcements consists of the passive resistance mobilized in front of the transverse members. The maximum pullout resistance is shown by a bilinear curve which displayed similarity with the failure envelope from direct shear tests of the backfill material. This bilinear envelope reinforced the previous observation regarding the effect of particle breakage phenomenon inherent to lateritic residual soils subjected to high normal pressures. Comparisons between laboratory and field pullout resistances and between the predicted passive resistance and the laboratory test data are also presented.

Key words: reinforcement, laboratory test, backfill, compaction, friction resistance.

Des essais d’arrachement en laboratoire et sur le terrain ont été réalisés pour étudier l’interaction d’armatures en grillage d’acier soudé enfouies dans un remblai de sol résiduel latéritique. Les essais d’arrachement en laboratoire ont été conduits sur différentes dimensions d’armature, géométries de mailles, pressions normales, et conditions de compaction du matériau de remblai. Des essais d’arrachement sur le terrain ont été conduits à des valeurs représentatives de pression des terres susjacentes, de teneur eau sur le terrain, et de densité. En partant des résultats, l’on a trouvé que les membranes longitudinales ont donné des résistances de 8 à 15% de la résistance totale à l’arrachement du grillage. Ainsi, la contribution majeure de la résistance à l’arrachement des armatures en grillage est fournie par la résistance en butée mobilisée en avant des membranes transversales. La résistance maximale d’arrachement a donné une courbe bilinéaire qui a montré une similarité avec l’enveloppe de rupture d’essais de cisaillement direct du matériau de remblai. Cette enveloppe bilinéaire ajoute du poids à l’observation antérieure concernant l’effet du phénomène de fracture des particules inhérente aux sols résiduels latéritiques soumis à de fortes pressions normales. L’on présente également des comparaisons entre les résistances d’arrachement en laboratoire et sur le terrain, de même qu’entre la résistance en butée prédite et les données des essais en laboratoire.

Mots clés : armature, essai de laboratoire, remblai, compactage, résistance en frottement.

[Traduit par la rédaction]
Modern earth reinforcement was first envisaged by Vidal (1969) and has been widely used in geotechnical engineering infrastructures such as retaining walls and embankments. Various reinforcements in the form of steel strips and steel grids to polymer grids and geotextiles are presently being used. Among the types of reinforcing materials, grid reinforcement has six times higher pullout capacity than strip reinforcement (Chang et al. 1977). Granular backfill materials are commonly used.

In the Chao Phraya Plain of Thailand, where Bangkok Metropolis is located, ideal granular materials are scarce and expensive because sources are distant from construction sites. Also, the use of polymer materials as reinforcement is considered expensive due to high import tax. Consequently, it is imperative to utilize cheaper, locally available but poor-quality backfill materials in conjunction with steel geogrid reinforcement that can be fabricated locally.

In this paper, cohesive-frictional, lateritic residual soils are utilized as backfill materials. Lateritic residual soils are commonly used as construction materials for earth structures such as road embankments and dikes. Some of the prominent properties of this soil are the hardening on exposure to air, susceptibility to change in properties due to wetting-drying process, particle breakage, and stress dependency of the failure envelope. Interactions between the lateritic residual soil backfill and the steel geogrid reinforcements are investigated by laboratory and field pullout tests. Laboratory pullout tests were conducted in a pullout box designed and fabricated for that purpose. The field pullout tests were conducted on dummy steel grid specimens embedded at different levels along the face of a full-scale geogrid reinforced wall-embankment system. The reinforced wall-embankment system was constructed to investigate the behavior of mechanically stabilized earth (MSE) constructed with cohesive-frictional backfills on soft ground (Bergado et al. 1991).

**Fig. 1.** Grain-size distribution before and after the pullout test.

**Properties of lateritic residual soil**

Residual soils are formed *in situ* from the products of chemical decomposition, physical disintegration, and biological weathering of rock. Mostly found in humid tropics, these soils occur when environmental conditions such as abundant rainfall, high temperature, and low pH are conducive to rapid and intense weathering, and the rate of decomposition exceeds the rate of erosion (Sowers 1963). Vallarga and Rananand (1969) defined lateritic soil as a hardened material formed by primary weathering of non-transported soils or by secondary enrichment and cementation of transported or nontransported soils containing sesquioxides of aluminum and iron and large amounts of quartz and kaolinite.

In this study, the lateritic soils were generally gap-graded to well-graded, yellowish brown, and clayey to gravelly. The particle size is erratic (Vallarga et al. 1969). Figure 1 shows the particle-size distribution before and after the pullout test. It was found that after compaction, the gravel-size particles broke down, transforming the soil to a well-graded material. Similar results were also obtained by several investigators (Brand and Hongsmoii 1969; Onitsuka et al. 1987; Bergado et al. 1988). The index properties of lateritic soil used in this study are listed in Table 1. Because of their extreme variations, the Atterberg limits seem to be of limited assistance in classifying lateritic soil. The moisture-density relationships for lateritic soils are influenced by many factors, namely grading, predominant clay mineral, crushing strength of coarse material, degree of weathering, and placement conditions of the material. In this study, the standard Proctor compaction test yielded a maximum dry density of 18.8 kN/m², with corresponding optimum water content of 11.5%, and when compaction water content is within a range from 8.5 to 14%, over 95% of compaction can be obtained.

Since the lateritic soils are found in a variety of forms, their engineering properties are also varied. The strength of
### Table 1. Basic properties of lateritic soil

<table>
<thead>
<tr>
<th>Specific gravity</th>
<th>Unit weight (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Coarse fraction (10-0.07 mm)</td>
</tr>
<tr>
<td>Test No.</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>2.66</td>
</tr>
<tr>
<td>2</td>
<td>2.66</td>
</tr>
<tr>
<td>3</td>
<td>2.65</td>
</tr>
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</table>

### Table 2. Strength parameters of compacted lateritic soil

<table>
<thead>
<tr>
<th>Type of test</th>
<th>Compaction water content (%)</th>
<th>Degree of compaction (%)</th>
<th>Normal stress range (kPa)</th>
<th>Cohesion (kPa)</th>
<th>Friction angle (deg.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>UU</td>
<td>10.00</td>
<td>85</td>
<td>50-200</td>
<td>42.0</td>
<td>29.2</td>
</tr>
<tr>
<td></td>
<td>10.30</td>
<td>90</td>
<td>50-200</td>
<td>66.0</td>
<td>31.0</td>
</tr>
<tr>
<td></td>
<td>10.60</td>
<td>95</td>
<td>50-200</td>
<td>80.0</td>
<td>32.5</td>
</tr>
<tr>
<td>CIU</td>
<td>10.90</td>
<td>85</td>
<td>50-200</td>
<td>11.0</td>
<td>15.3</td>
</tr>
<tr>
<td></td>
<td>10.80</td>
<td>90</td>
<td>50-200</td>
<td>24.0</td>
<td>19.0</td>
</tr>
<tr>
<td></td>
<td>10.80</td>
<td>95</td>
<td>50-200</td>
<td>30.0</td>
<td>21.0</td>
</tr>
<tr>
<td>Direct shear</td>
<td>9.88</td>
<td>95</td>
<td>2-18</td>
<td>35.0</td>
<td>56.8</td>
</tr>
<tr>
<td></td>
<td>12.20</td>
<td>95</td>
<td>2-18</td>
<td>27.0</td>
<td>65.8</td>
</tr>
<tr>
<td></td>
<td>14.60</td>
<td>95</td>
<td>2-18</td>
<td>14.0</td>
<td>47.5</td>
</tr>
<tr>
<td></td>
<td>8.50</td>
<td>95</td>
<td>10-130</td>
<td>51.0</td>
<td>38.5</td>
</tr>
<tr>
<td></td>
<td>11.30</td>
<td>95</td>
<td>10-130</td>
<td>32.0</td>
<td>41.4</td>
</tr>
<tr>
<td></td>
<td>14.00</td>
<td>95</td>
<td>10-130</td>
<td>20.0</td>
<td>37.2</td>
</tr>
</tbody>
</table>

*aUU, unconsolidated-undrained; CIU, isotropically unconsolidated - undrained.*

Compressed lateritic soil used in this study was investigated by unconsolidated-undrained (UU) and isotropically consolidated - undrained (CIU) triaxial compression tests (Bergado et al. 1987) and a large number of direct shear tests (Amin 1989; Macaulay 1990). In the triaxial tests, the soil was compacted to 85, 90, and 95% of the maximum standard Proctor dry density at 1-2% of dry side of optimum water content. The specimen was 102 mm in diameter and 203 mm in height. The confining pressure ranges from 50 to 200 kPa. In the direct shear tests, the conventional direct shear box was modified with a square shear box of 150 mm in side length and 127 mm in height. The soil was compacted to 95% of the maximum standard Proctor dry density with water content varying from 8.5 (dry side) to 14% (wet side). The same corresponding conditions were used for pullout tests. Two series of normal pressures were used, namely low normal pressures, with 2, 8, 13, and 18 kPa, and relatively high normal pressures, with 10, 50, 90, and 130 kPa. The test results are listed in Table 2 in terms of cohesion and friction angle values. These results clearly show that the strength parameters are significantly influenced by the compaction water content, degree of compaction, and applied normal pressures. As expected, the strength of the soil samples compacted at the dry side of optimum was much higher than that of compacted at the wet side of optimum. The triaxial test results show that the higher the degree of compaction, the higher the strength of the soil. The applied normal pressures mainly influenced the friction angle of the soil. The direct shear test results show that at low applied normal pressure, the friction angle is very high, around 50°. At this condition, the laternite soil behaves more like a gravelly soil, since its particles are not broken under low normal pressure. At relatively high normal pressure, the friction angle reduced by about 10° due to particle breakage (Onitsuka et al. 1987). Comparing the UU with the CIU test results, it can be seen that the soaking of the compacted lateritic soil to 100% saturation (water content increased from 10.3 to 18.0%) greatly reduced the strength. Soaking also accelerates the particle breakage phenomenon.

The theoretical background of soil - grid reinforcement interaction

The pullout resistance of grid reinforcement comprises two components. One component is the frictional resistance $P_f$ developed between the soil and the frictional surface of the longitudinal members of the grid reinforcement. The frictional resistance depends on the angle of skin friction and the normal effective stress between the soil and the reinforcement surface as follows:

\[ P_f = A_s \sigma_n \tan \delta \]

where $A_s$ is the surface area of the longitudinal bars, $\sigma_n$ is the average normal stress, and $\delta$ is the friction angle between the reinforcement and the soil. The other component of the pullout resistance is the passive resistance of the soil bearing...
### Table 3. Bearing capacity factors for pullout passive bearing resistance

<table>
<thead>
<tr>
<th>Theory</th>
<th>Bearing capacity factor</th>
<th>Author</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>General shear</td>
<td>$N_a = \exp(\pi \tan \phi) \tan^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right)$</td>
<td>Peerson and Anderson 1980</td>
<td>Provides apparent upper bound for experimental data</td>
</tr>
<tr>
<td></td>
<td>$N_c = (N_q - 1) \cot \phi$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Punching shear</td>
<td>$N_a = \exp\left[\left(\frac{\pi}{2} + \phi\right) \tan \phi\right] \tan\left(\frac{\pi}{4} + \frac{\phi}{2}\right)$</td>
<td>Jewell et al. 1984</td>
<td>Provides apparent lower bound for experimental data</td>
</tr>
<tr>
<td></td>
<td>$N_c = (N_q - 1) \cot \phi$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deep footing in clay</td>
<td>$\sigma_b = N_c C_u$, where $N_c = 7.5$, and $C_u$ is undrained shear strength</td>
<td>Bergado et al. 1987</td>
<td>For predicting the pullout resistance of grid in cohesive backfill</td>
</tr>
<tr>
<td>Anchor capacity (finite element analysis)</td>
<td>Graphical chart</td>
<td>Rowe and Davis 1982a, 1982b</td>
<td>Strength parameters, dilatancy properties, and initial stress state of soil are needed to use the chart</td>
</tr>
</tbody>
</table>

### Table 4. The conditions for laboratory pullout tests

<table>
<thead>
<tr>
<th>Spacing of grid specimens</th>
<th>152 × 152 mm and 152 × 225 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bar diameter</td>
<td>6.3, 9.5, and 12.7 mm</td>
</tr>
<tr>
<td>Compaction water content of backfill soil</td>
<td>Dry side of optimum, optimum, and wet side of optimum</td>
</tr>
<tr>
<td>Degree of compaction</td>
<td>95% of standard Proctor compaction</td>
</tr>
<tr>
<td>Embedment length</td>
<td>1.0 m</td>
</tr>
<tr>
<td>Normal pressure series</td>
<td>2, 8, 13, and 20 kPa; 10, 30, and 50 kPa; 50, 70, and 90 kPa; and 90, 110, and 130 kPa.</td>
</tr>
<tr>
<td>Capacity of pullout machine</td>
<td>225 kN</td>
</tr>
<tr>
<td>Pullout rate</td>
<td>1 mm/min</td>
</tr>
</tbody>
</table>

Fig. 2. Schematic diagram of grid specimens with instrumentation points. ●, wire extensometer; x, strain gage.

The transverse member of the grid reinforcement. The passive resistance mechanisms similar to the base pressure on a deep foundation in the soil (Jewell et al. 1984). To evaluate the passive or bearing resistance, the bearing capacity theory has been used, and the bearing resistance can be expressed as follows:

$\sigma_b = c N_c + \sigma_p N_q$

where $\sigma_b$ is the bearing resistance of a unit bearing area, $c$ is the cohesion of backfill soil, $\sigma_p$ is the applied normal pressure at the soil – grid reinforcement interface; and $N_c$ and $N_q$ are the bearing capacity factors. Different assumptions have been used for determining the bearing capacity factors, and they are summarized in Table 3 together with comments on their accuracy.

For grid reinforcement, the bearing members have interference to each other. A dimensionless parameter of spacing ratio $S/D$ is introduced to express the influence of grid geometry on pullout resistance (Jewell et al. 1984), where $S$ is the centre-to-centre spacing between two neighboring transverse members, and $D$ is the thickness of the transverse member. Generally, the higher the spacing ratio, the higher the pullout passive bearing resistance from one bearing member or less interference between bearing members. Palmira and Milligan (1989) found that when $S/D$ is larger than 50, the interference becomes negligible.

The total maximum passive resistance $P_\phi$ of a grid reinforcement can be obtained as bearing resistance $\sigma_b$ of a unit bearing area multiplied by total bearing area. The total pullout resistance $P_t$ is then expressed as the sum of the frictional and passive resistances as follows:

$P_t = P_f + P_\phi$

### Laboratory pullout test

**Testing program and procedures**

A total of 189 laboratory pullout tests were carried out on welded steel grid reinforcements embedded in compacted lateritic residual soil subjected to normal pressures ranging from 2 to 130 kPa. The test conditions are summarized in Table 4. The typical schematic diagram of the steel grid specimen with instrumentation points is shown in Fig. 2. For each setup, three to four pullout tests were done corresponding to different normal loading stages.
FIG. 3. Typical laboratory pullout test setup.

![Diagram of laboratory pullout test setup.]

FIG. 4. Total pullout resistance vs. horizontal displacement from laboratory pullout test. $C_0$, overburden pressure. Compacted at optimum water content; 12.7 mm bar diameter; 152 x 225 mm mesh.

The pullout box was made of steel plates and rolled steel beams with inside dimensions of 1.3 x 0.8 x 0.5 m. Details of the pullout box have been reported elsewhere (e.g., Bergado et al., 1992). A typical pullout test setup is shown in Fig. 3. A data-acquisition system consisting of a 21 x micrologger was employed to record the mat displacements, pullout forces, and axial strains in the bar by means of strain gages. The backfill soil was compacted using a hand-operated tamper. The density and moisture content of the compacted soil were measured by a nuclear gage densitometer. The normal pressure was applied by inflating an air bag fitted inside the pullout box over a flexible metal plate, 6.3 mm thick, which was placed on top of the compacted backfill soil, with a 30-mm layer of fine sand placed in between.

Pullout resistance of grid specimens

Figure 4 shows a typical stress-strain relationship from laboratory pullout tests. Generally, the pullout resistance increased with increasing applied normal pressures. As shown, the pullout force increased quickly before the pullout displacement reached 8 mm. Thereafter, the pullout force increment was very small until the maximum pullout force was reached. Test results indicated that the increments of pullout force corresponding to the increase of normal pressures at low normal pressure range generally seemed to be higher than at high normal pressure range. This coincides with the direct shear test results of the compacted lateritic soil wherein the friction angle at low normal pressure range was higher than the corresponding value at higher normal pressure range. Furthermore, test results indicated that the backfill soil compacted at the dry side and the wet side of optimum provided the highest and lowest total pullout resistances, respectively. At the dry side, the suction in the soil mass is thought to be much higher than that of the wet side.

The strain in the longitudinal bar and the mat displacement along the grid pullout direction were measured by strain gages. The maximum strain in the longitudinal bar was within 0.2%. Figure 5 is a typical front displacement versus the displacements of different points on the grid which had different displacement from the face of the pullout box. The mat displacements along the grid were measured by piano wire - dial gage system. Figure 5 indicates that the grid moved practically as a rigid body.

Pullout resistance of longitudinal members

Pullout tests were also conducted using only longitudinal
bars to investigate the frictional resistance between the steel bar and the compacted lateritic soil. Figure 6 shows a typical horizontal displacement – frictional resistance curve of a 12.7-mm-diameter bar with the backfill soil compacted at optimum water content. The specimen was tested with four longitudinal bars embedded 1.0 m in the soil. It can be seen that the mobilized frictional resistance developed very quickly within 2–4 mm pullout displacement. After reaching the peak strength, the friction resistance was approximately constant. The pullout resistance of the longitudinal bar only consisted of about 8–15% of the total pullout resistance. The frictional resistance was analyzed in terms of skin friction angle $\phi$ and adhesion $c_a$. For the soil compacted at dry side of optimum, optimum, and wet side of optimum water content, the skin friction angle was 17, 10, and 8$^\circ$ and the adhesion 85, 75, and 52 kPa, respectively.

**Influence of grid dimensions and bar size**

Two configurations were used to investigate the influence of grid dimension on pullout resistance. Figure 7 shows the maximum pullout resistance of two grid specimens with the same bar diameter and under the same backfill condition but different bearing member spacing ratio, i.e., 152 × 152 mm grid with seven transverse bars and $S/D$ of 16, and 152 × 225 mm grid with five transverse bars and $S/D$ of 23.7. The total pullout resistances were nearly the same, with the former (152 × 152 mm grid) being only 10% higher than the latter (152 × 225 mm grid). The maximum pullout resistances with the same grid spacing and backfill soil condition but with different bar diameters or different $S/D$ ratios are shown in Fig. 8. It can be seen that the resistance increased with the increase in bar diameter, but the increase was not directly proportional to the increase in bar diameter. These results confirmed that the grid geometry, i.e., $S/D$ ratio, has a strong effect on the grid pullout passive bearing resistance and indicated that increasing the bar diameter and the number of transverse bars have limited benefit to increasing the total pullout resistance.

**Mechanically stabilized earth (MSE) embankments and field pullout tests**

A full-scale, mechanically stabilized wall–embankment system with welded steel geogrid reinforcements was constructed within the campus of the Asian Institute of Technology, located 42 km north of Bangkok (Bergado et al. 1991, 1992). It consists of three sections with different backfill materials, namely weathered clay, clayey sand, and lateritic soil, with a base length of 26 m. The lateritic soil was placed in the middle section about 5 m in length. The five dummy pullout specimens, instrumented with strain gages, were embedded at different levels along the face of the wall for field pullout tests. Field pullout tests were conducted to investigate the pullout resistance of reinforcements at representative overburden, field-moisture, and density conditions. The steel grid specimens have mesh sizes of 152 × 225 mm with four longitudinal bars and four or five transverse bars. The bar diameter varied from 5.4 (W3.5) to 9.9 mm (W12). The embedment length is 2 m, but with limited capacity of the pullout equipment, only some of the transverse bars near the face were cut off. The pullout force was applied in a manner similar to that of the laboratory pullout test, with a specially designed reaction frame butting against the wall face (Lo 1990). In addition, a wooden platform was built to support and position the pullout equipment corresponding to the locations of the grid pullout specimens. A typical setup of a field pullout test is shown in Fig. 9. The maximum pullout displacement was 125 mm. The backfill soils of the wall–embankment were compacted to 95% of standard Proctor compaction at moisture content near the optimum.

Figure 10 shows the field pullout test results in terms of load–displacement curves. The pullout resistance decreased...
with the increase of the overburden height. Since the lateritic residual soil was located at the middle section of the test wall-embankment system, an arching effect occurred in the reinforced soil mass due to the compression of the soft clay foundation which changed the vertical stress distribution within the reinforced embankment. Thus, at the lower part of the middle section the vertical pressure was less than the unit weight multiplied by the corresponding distance from the top of the wall (Bergado et al. 1991).

Field pullout tests were also done with ribbed longitudinal bars under the overburden height of 3.8 m. The result was analyzed in terms of apparent friction coefficient $f$, which is defined as the frictional pullout resistance $P_t$ divided by the surface area of the longitudinal bars $A$ and the average normal pressure $σ_n$ acting on the soil–steel bar interface, which was taken as 0.75 times the overburden pressure $σ_o$, taking lateral pressure coefficient as 0.3 ($σ_n = (1 + k) σ_o / 2$). The apparent friction coefficient is given as

$$f = \frac{P_t}{Aσ_n}$$

The value of $f$ was found to be 2.65 for lateritic soil.

As mentioned previously, the embedment length of the grids in the field was larger than that of laboratory tests, but for most grids the number of transverse bars was the same. To compare the laboratory and field pullout test results, the resistance of ribbed longitudinal bars was calculated using the apparent friction coefficient of 2.65 and assuming that the vertical stress acting on the soil–grid interface can be calculated by the unit weight of the soil multiplied by the overburden depth. The calculated resistance of ribbed longitudinal bars was subtracted from the total field pullout resistance. Finally, the field and laboratory pullout resistance with the same number of transverse bars was compared. Figure 11 shows the comparison of field and laboratory maximum pullout resistance of grids with bar diameter of 6.3 mm and spacing of 152 × 225 mm. Because of the arching effect in the wall-embankment system, it was difficult to make a precise comparison. However, Fig. 11 shows that the laboratory, large-scale pullout test can give a good estimation of pullout resistance of grid reinforcement in the reinforced earth structure.

Comparison of the calculated values with the laboratory test data

The frictional resistance was subtracted from the total pullout resistance to get the maximum passive or bearing resistance which was used to compare with calculated data. The loading condition of the soil in front of the transverse bar is close to triaxial compression. The strength parameters selected for passive resistance calculation are derived from an unconsolidated undrained (UU) triaxial test at high normal pressure range. However, since no triaxial test data were available for low normal pressure range, the strength parameters from direct shear tests were used. For specimens compacted at optimum water content, the strength param-
Fig. 12. Comparison of theoretical passive resistance and observed data.

$s$ are as follows: (i) at high normal stress range, $c = 80$ kPa and $\phi = 32.5^\circ$; (ii) at low normal stress range, $c = 32$ kPa and $\phi = 41.7^\circ$. Several methods are available to calculate the passive resistance as discussed earlier. Using Rowe's chart (Rowe and Davis 1982a, 1982b), it was assumed that the at-rest earth pressure coefficient $k$ = 0.5 and the soil dilatancy angle was equal to the friction angle (associate flow rule). Figure 12 shows the comparison of calculated and test data for backfill soil compacted at optimum water content. In this figure, $P_0$ is the passive resistance of the soil, $n$ is the number of transverse bars, $w$ is the width of the grid, and $d$ is the diameter of the transverse bar. It can be shown that the general shear failure mode and punching shear failure mode provided apparent upper and lower bounds of the test data. The calculated values using Rowe's chart generally agreed well with the test data but were about 20% lower than the average values of the test data, especially at low normal pressure range. The pullout resistance with the backfill soil compacted at dry side and wet side of optimum water contents indicated trends similar to that at optimum water content. A simple empirical equation is proposed to predict the passive resistance and is expressed as follows:

$$P_0 = (c + \sigma_N \tan \phi)N_c$$

where $N_c$ is an empirical bearing capacity factor for the compacted lateritic soil, and was found to be 22. This empirical equation is only applicable using strength parameters at high normal pressure range. Figure 12 also indicates that because of the particle-size variation, the pullout passive bearing resistance showed considerable scatter, forming a band.

Conclusions

(1) The pullout test results of welded steel bar grids embedded in compacted lateritic soil showed that the dominant component of pullout resistance is passive bearing resistance. The pullout resistance is influenced by the backfill soil compaction water content and the grid geometry, i.e., bearing resistance ratio $S/D$. The test results also indicate that the pullout resistance increased with the increase of normal pressure, and locally available lateritic soil compacted at optimum or dry side of optimum water content can be used as backfill material for MSE constructions.

(2) The maximum strain in longitudinal bars of the welded wire used in laboratory tests was only in the order of 0.1–0.2%, proving that the mat moved almost as a rigid body and that the resistance along the mat was mobilized uniformly.

(3) The field pullout tests on the dummy steel grid embedded in the lateritic soil section of MSE wall embankment system showed that the maximum pullout resistance of the steel grid decreased with the increase of overburden height due to an arching effect. However, comparison between the laboratory and field pullout test results indicated that the laboratory large-scale pullout test can give a good estimation of the actual pullout resistance of the grid reinforcement in the field.

(4) Comparing the predicted maximum pullout resistance with the laboratory pullout test data, it can be shown that the general shear and punching failure modes provided the apparent upper and lower bounds for laboratory pullout test results. Rowe's chart (Rowe and Davis 1982) yields the better prediction but generally underpredicted the average test values. An empirical equation for calculating the passive resistance has been proposed, and it was found that the corresponding empirical bearing capacity factor for compacted lateritic soil is 22.

Acknowledgments

This research was done as part of a project sponsored by the U.S. Agency for International Development (USAID) conducted at the Asian Institute of Technology (AIT), Bangkok, Thailand, with joint cooperation from Utah State University, Logan. The financial assistance given by USAID, Bangkok, Thailand, and the facilities provided by AIT are gratefully acknowledged.


