Pull Resistance and Interaction of Earthwork Reinforcement and Soil

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Results are presented of tests on different kinds of earthwork reinforcement to study pull resistance and soil-reinforcement interaction in reinforced earth. Three kinds of pull tests were conducted. Results indicate that the soil will not be significantly strained until a proportional limit is reached on a load-deformation curve. For the same surface area, bar-mesh reinforcement has nearly six times the pull resistance as that of flat steel-strip reinforcement. Bar-mesh reinforcement embedded in a dense cohesive soil exhibited greater pull resistance than that embedded in a less dense cohesionless soil. An increase in size of mesh opening will substantially reduce the pull resistance of the bar mesh. The minimum length of steel-strip reinforcement required for a low-height, reinforced earth wall was found to be at least 3.1 m (10 ft).

The use of steel strips as earthwork reinforcement has been reported by Vidal (8, 9), Lee and others (6, 7), and Chang and others (1, 2, 3, 4). More than a dozen reinforced earth walls have been constructed in the United States in recent years under a patented design (5). The California Department of Transportation built the first of these walls with steel-facing and steel-strip reinforcements in 1972 on Cal-39 in the San Gabriel Mountains of Los Angeles County. A second reinforced earth wall with concrete-facing and steel-strip reinforcements was constructed on I-5 near Dunsmuir, California, in 1974.

Two other experimental projects designated as mechanically stabilized embankment with concrete-beam facing and bar-mesh reinforcements were also constructed on I-5 in September 1975 and May 1976 respectively (Figures 1 and 2). These experimental projects were designed and constructed by the California Department of Transportation under an agreement with the Reinforced Earth Company of Washington, D.C. Future construction that uses this kind of system will be under the license of that company.

For the purpose of studying the pull resistance and the interaction of different kinds of reinforcement and soil for these construction projects, field and laboratory pull tests were conducted on both large and small scales. These tests and the results are discussed.

FIELD PULL TESTS ON STEEL STRIPS

The pull resistance of soil reinforcements in the field was studied by installing additional dummy steel strips in the reinforced earth wall located on Cal-39 in the San Gabriel Mountains, California, at different elevations during construction. Three steel strips, 1.5, 3.1, and 4.6 m (5, 10, and 15 ft) long, were embedded at each of three levels under overburden heights of 2.3, 3.8, and 5.6 m (7.5, 12.4, and 18.2 ft) respectively. Three 7 and 14-m (23 and 46-ft) steel strips were also embedded at depths of 5.5 and 11.60 m (18 and 38 ft) respectively. Both the 7 and 14-m (23 and 46-ft) strips were instrumented with strain gauges on both top and bottom at 1.5-m (5-ft) intervals. All of the steel strips are 3 mm (0.118 in) thick and 60 mm (2.362 in) wide. The fill material is primarily decomposed granite. The physical properties of soil samples obtained from a nearby borrow site are given in Table 1.

Figure 3 shows six typical load-deformation curves obtained from field pull tests for 1.5, 3.1, 4.6, 7, and 14-m (5, 10, 15, 23, and 46-ft) steel strips. Three pull loads selected for analysis are indicated on these curves:

1. Yield load, which represents the proportional limit of the load-deformation relation,
2. Peak load, which represents the maximum pull load observed, and
3. Residual load, which represents the pull load when deformation increased appreciably without changing pull load.

Tensile strength tests on a small-sized sample of the reinforcing strip resulted in yield and ultimate capacities of 45.8 and 61.8 kN (10.3 and 13.9 kips) respectively. At the site, one of the 7-m (23-ft) strips broke at a pull load slightly over 62.3 kN (14 kips). Three of the 14-m (46-ft) strips broke at pull loads of 59.9, 63.2, and 64.3 kN (13.46, 14.20, and 14.45 kips). The rest of the steel strips failed by slipping.

The straight line portion of the load-deformation curve represents the elastic properties of the steel. When the steel strip has sufficient length and overburden, i.e., when the frictional grip is great enough, the proportional limit will reflect the yield capacity of the steel. If the frictional grip is not sufficient at the loaded end of the strip, the soil will start to strain at a proportional limit lower than that for reaching the yield strength of the steel. Therefore, it appears that the yield load represents either the yield capacity of the steel or the initial point of soil-steel interaction as a composite material. The values of the yield load depend on the length of the reinforcement and the overburden load. The peak load represents the maximum mobilized pull resistance of the composite material of reinforcement and soil. Once the peak load is reached, the strips begin to slip and the pull load drops to the residual or ultimate level. Figure 4a shows the relations between the peak pull load and the overburden load and the height of overburden and the length of the strips. At the same height of overburden, the peak pull load is proportional to the overburden load, shown by the solid straight lines. The slopes of these lines decrease as the overburden height increases. For the same strip length, the relation between the peak pull load and overburden load is approximately linear, indicated by the dashed lines. The solid lines suggest that the longer strips have higher peak pullout resistance for the same height of overburden. For a given strip length, the pull resistance increases as overburden heights increase. The rate of increase in peak pull load caused by an increase in overburden is much smaller than that caused by an increase in strip length. Figure 4b shows similar characteristics for the yield load.

Since the peak load represents the maximum mobilized friction force, the factor of safety can be evaluated by using the peak load as failure load and the factor of safety against slippage can be evaluated by using the following equation proposed by Chang (1, 2, 3, 4).

Factor of safety against slippage = peak failure pull load $\div K_a \gamma H d \Delta H$ (1)

where

K_a = coefficient of active earth pressure,
γ = unit mass of soil,
H = height of fill above the reinforcement,
d = horizontal spacing of reinforcement, and
ΔH = vertical spacing of reinforcement.

The relations between overburden height (H), strip length (L), and the factor of safety (FS) are shown in Figure 4c, which can be used as a guide for selecting the minimum length of strip reinforcement required for a given height of fill.

The relation between the peak pull load and the skin friction force is shown in Figure 4d and calculated by using the following equation.

$$2F = 2\gamma HbL \tan \delta \quad (2)$$

where

F = friction force on one face of the reinforcement,
b = width of reinforcing strip,

Figure 1. Concrete-beam facing of mechanically stabilized embankment on I-5 at Dunsmuir, California.

Figure 2. Bar-mesh reinforcement in mechanically stabilized embankment on I-5 at Dunsmuir, California.

Table 1. Physical properties of soil samples.

Location and Soil Classification	Sample	Effective Friction Angle (degrees)	Cohesion (kPa)	Max. Dry Density (kg/m^3)	Sand Equivalent Value
Highway 39					
Sand gravel	72-RE8	40	29	2227	28
I-5					
Gravel sand	73-1803	35	55	1233	27
Highway 101					
Silty clay and gravel	74-1231	34	97	2034	15

Notes: 1 kPa = 0.145 lbf/in^2 and 1 kg/m^3 = 0.624 lb/ft^3.

Figure 3. Load-deformation curves from field pull tests on Cal-39.

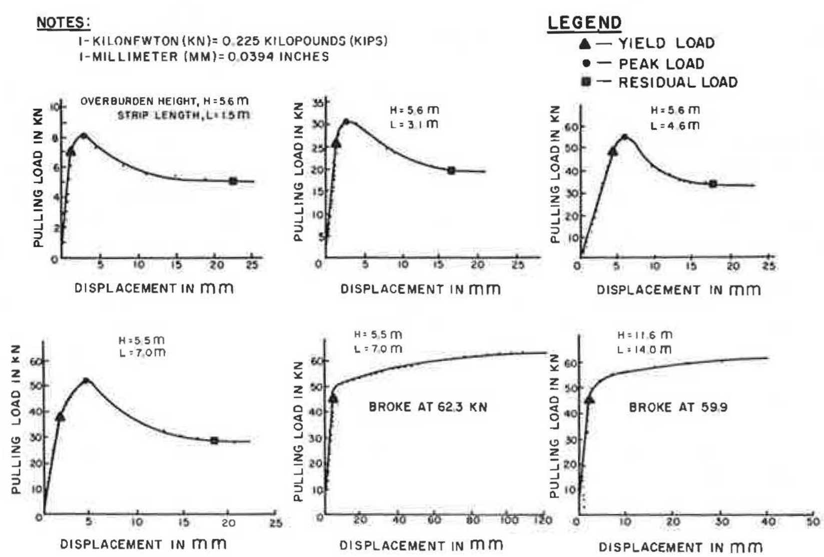


Figure 4. Summary results of field pull test.

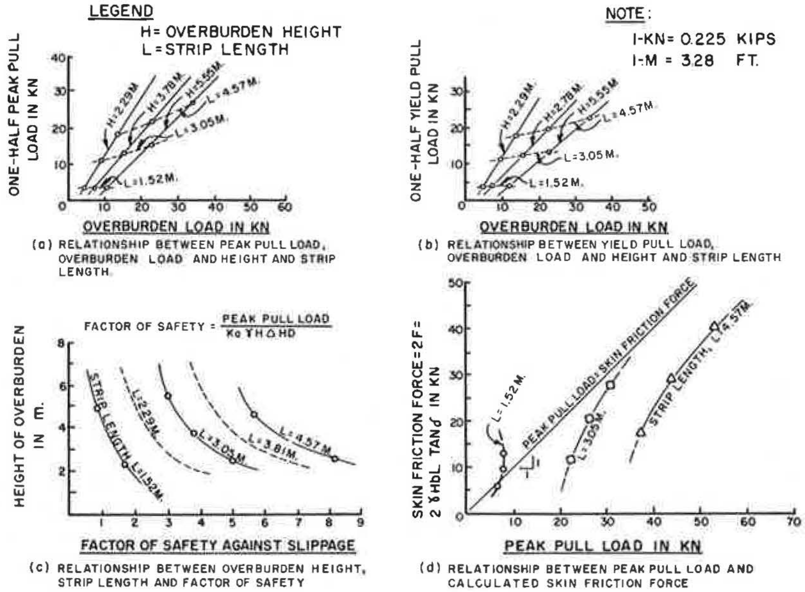


Figure 5. Axial force in 7 and 14-m dummy steel strips.

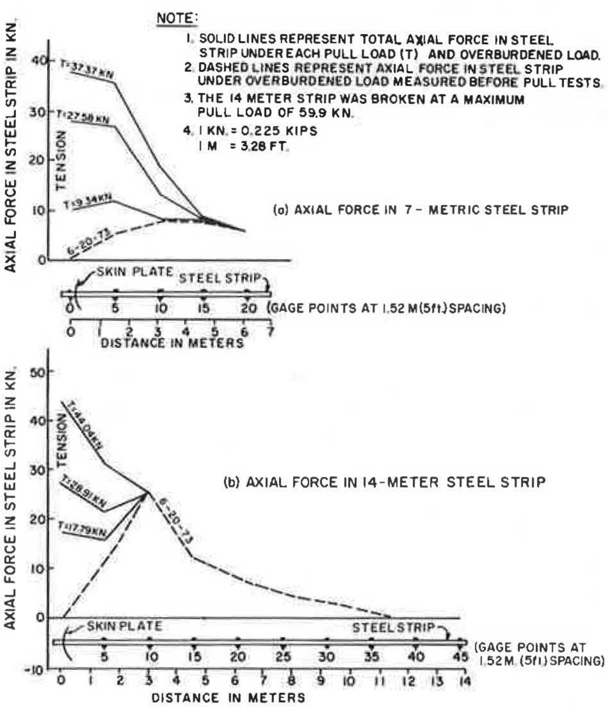
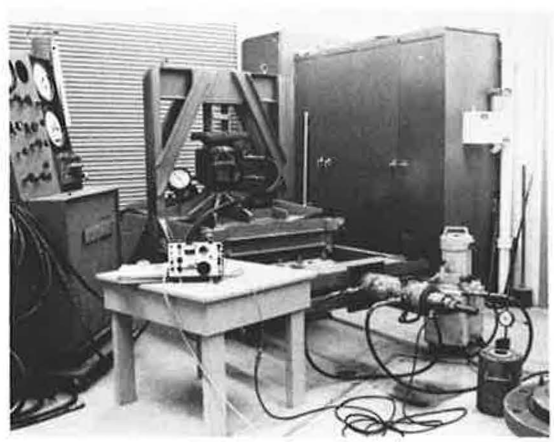


Figure 6. Laboratory facility for pull test.



L = length of reinforcing strip, and

δ = skin friction angle between soil and steel reinforcement.

It can also be seen that the peak pull loads exceed the calculated skin-friction force when the strip length exceeds 3.1 m (10 ft). Thus, it may be concluded that the minimum strip length should be 3.1 m (10 ft).

Figure 5 shows the distribution of tensile and compressive forces measured by strain gauges spaced at 1.5-m (5-ft) intervals along the length of the 7 and 14-m (23 and 46-ft) strips respectively. The dashed lines indicate the tensile or compressive forces in the steel strip measured at each strain gauge location under the embankment load. Each solid line represents the distribution of the total tensile forces along the length of strip that includes the forces induced by each applied pull load and the existing forces induced by the measured embankment load. Figure 5a shows that each external-applied pull load induced additional tensile forces for almost the entire length of the 7-m (23-ft) strip. However, the external-applied pull load only stressed the 14-m (46-ft) strip to gauge point that is a 3.1-m (10-ft) distance into the fill. There were no additional forces measured beyond gauge point 10 other than the existing forces induced by the embankment load before testing. The 14-m strip broke at the external-extended portion under the maximum pull load of 59.9 kN (13.46 kips). Because the 14-m strip was longer and under a heavier embankment load, the strip appeared to develop a fixed point at gauge point 10; therefore, no additional tensile forces developed beyond 3.1 m (10 ft) of the strip length within the fill.

LARGE-SCALE LABORATORY PULL TESTS

Large-scale laboratory pull tests were conducted so that the interaction between the soil and reinforcements could be understood. The test facility consists of a rigid steel box, 45.70, 91.40, and 137.20 cm (18, 36, and 54 in) high, wide, and long respectively, with pro-

Figure 7. Test runs with each kind of reinforcement.

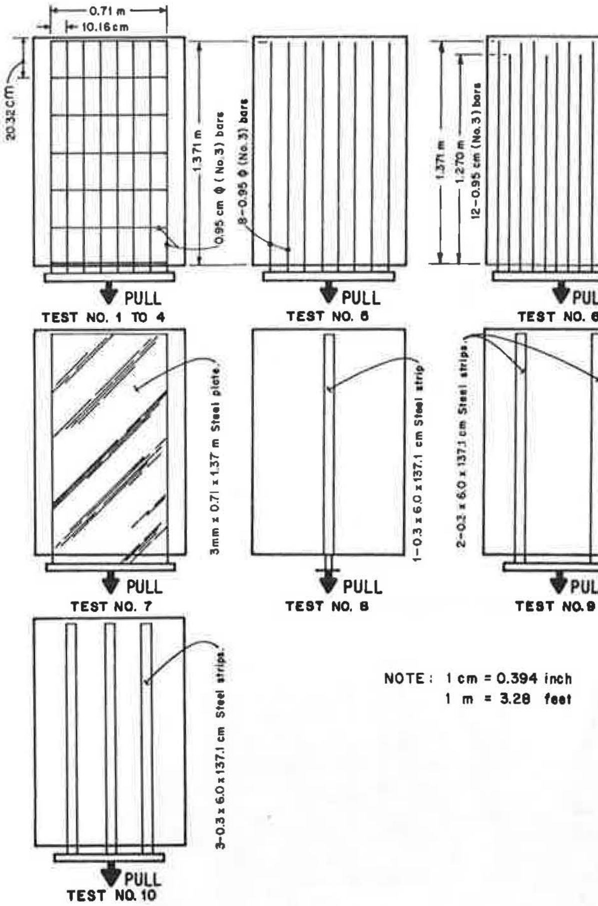
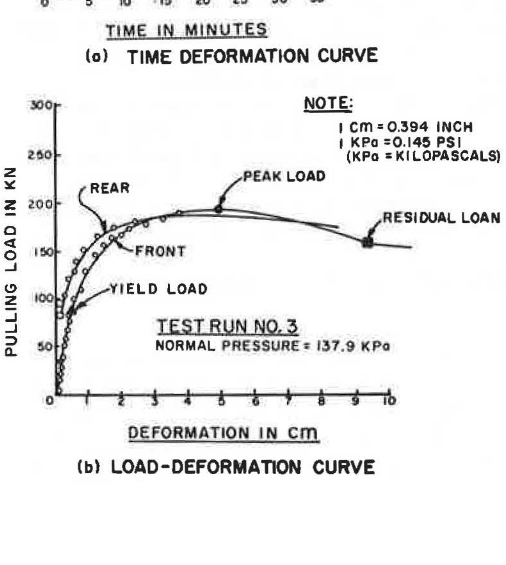


Figure 8. Typical time and load-deformation curves for bar mesh with 10.16 by 20.32-cm openings embedded in gravel-sand soil.



visions for applying vertical pressure to simulate the overburden load up to 15.2 m (50 ft) of earth fill (Figure 6). The test specimen is compacted in the box with reinforcement placed horizontally in the middle height of the specimen. A vertical normal load is applied to consolidate the specimen. Reinforcement can then be pulled out at a controlled rate of 0.05 mm/1min (0.002 in/1 min). During pull testing, the front face of the box is removed so that a free unrestrained face of the soil specimen is provided.

Figure 7 shows the 10 tests that were conducted with four different kinds of reinforcements. A description of the reinforcements follows:

1. Bar mesh made by crossing and welding smooth longitudinal bars with 0.95-cm ($^3/_8$-in) diameters to provide 10.20 by 20.30-cm (4 by 8-in) openings for tests 1 through 4;
2. Longitudinal smooth bars with 0.95-cm ($^3/_8$-in) diameters and 1.37-m (54-in) lengths were spaced differently for tests 5 and 6;
3. Solid steel plate that was 3 mm ($^1/_8$ in) thick, 71.1 cm (28 in) wide, and 1.37 m (54 in) long was used for test 7; and
4. Steel strips that were 3 mm ($^1/_8$ in) thick, 6 cm (2.3 in) wide, and 1.37 m (54 in) long were used for tests 8 through 10.

Figure 9. Typical load-deformation curves for bar mesh embedded in silty clay with gravel soil.

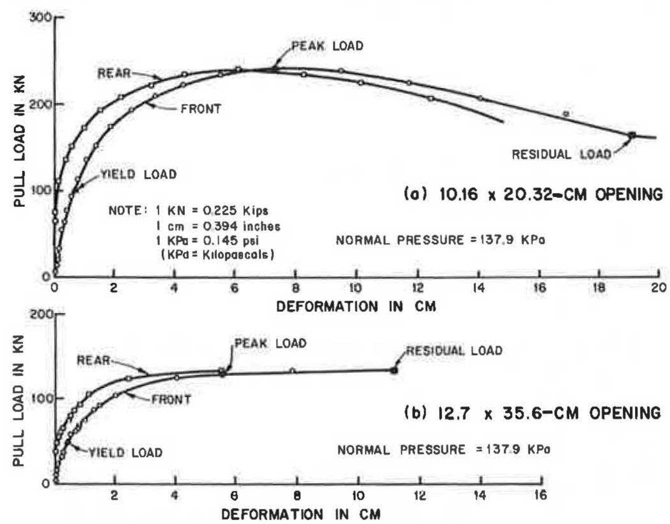
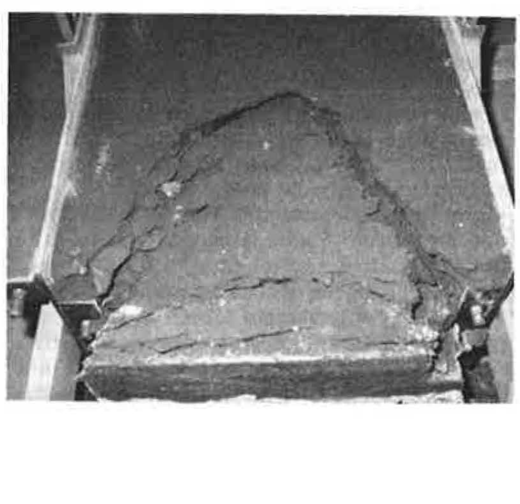


Figure 10. Cone-shaped failure in specimen with bar-mesh reinforcement.



Tests 1 to 4 were performed under normal pressures of 34.47, 68.95, 137.90, and 172.34 kPa (5, 10, 20, and 25 lbf/in^2) respectively. Tests 5 through 10 were conducted under a normal pressure of 68.95 kPa (10 lbf/in^2).

The soil samples used in the tests were obtained from the job site on I-5. This material is primarily poorly graded gravelly sand. The physical properties of the soil samples are given in Table 1.

Figure 11. Failure in specimen with eight longitudinal-bar reinforcement for test 5.

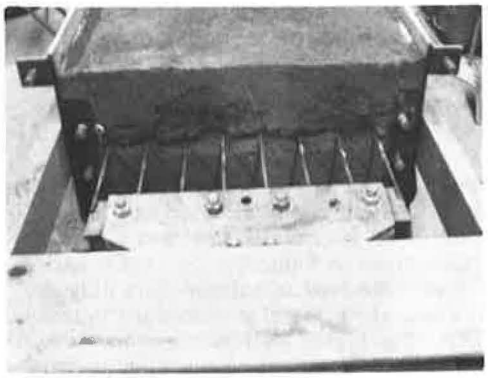


Figure 12. Failure in specimen with three steel-strip reinforcement for test 10.

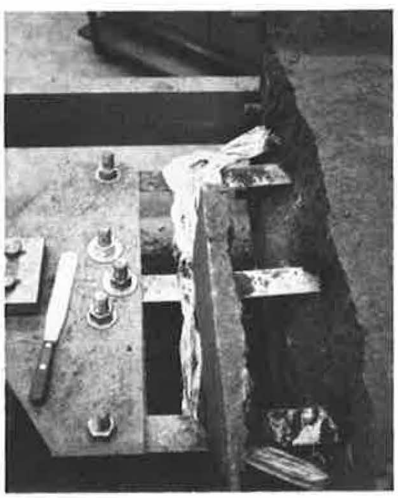


Figure 13. Cone-shaped failure in specimen with solid steel-plate reinforcement for test 7.

Time- and Load-Deformation Curves

During the pull tests, the deformations of the reinforcements were measured by two extensometers: one located at the front and at the rear of each specimen. The normal and pull loads were measured by load cells. A typical time-deformation curve for test 3 on bar mesh under a normal pressure of 137.90 kPa (20 lbf/in^2) is shown in Figure 8a. A typical load-deformation curve for the same test is also shown in Figure 8b. The load-deformation characteristics were found to be quite different for each kind of reinforcement.

The following observations were noted.

1. The strain response at the rear of the specimen is always behind that at the front or loaded end of the specimen (Figure 8); and
2. When the pull load is sufficient to cause an abrupt change in deformation at the rear of the specimen (time-deformation curve), the proportional limit or yielding point (load-deformation curve) is attained, i.e., the yield point occurred at the same strain level measured at the front end.

It can be hypothesized that the applied pull load only strains the reinforcement without causing any significant strain in the soil until the yielding load is reached when the whole length of the reinforcement exhibits an abrupt change in deformation.

Figure 9 shows the typical load-deformation curves obtained from pull tests that used bar-mesh reinforcement with mesh openings of 10.20 by 20.30 cm (4 by 8 in) and 12.7 by 35.6 cm (5 by 14 in) respectively that were embedded in silty clay soil under a normal pressure of 137.90 kPa (20 lbf/in^2). This material was obtained

Figure 14. Relation between normal and pull loads for bar mesh with 10.16 by 20.32-cm opening embedded in gravel-sand soil.

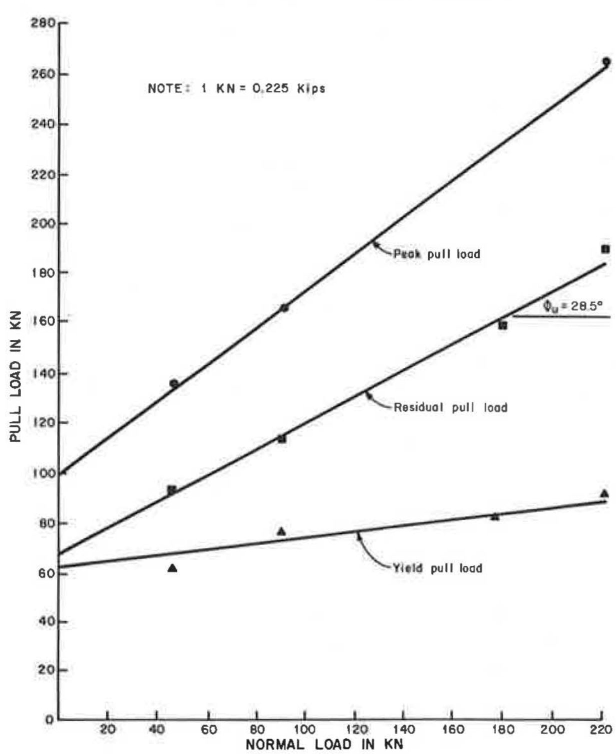
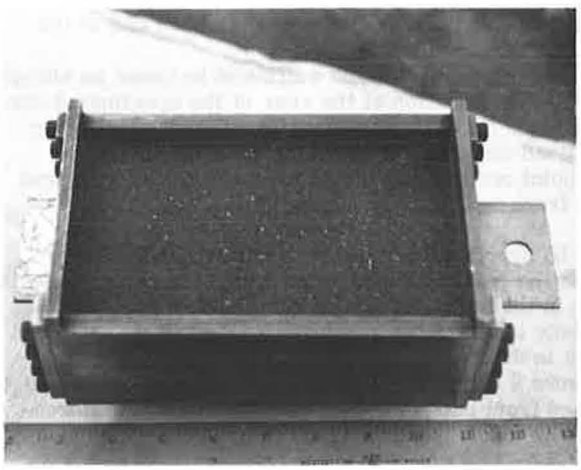


Table 2. Pull resistance for different reinforcements and soils.

Test	Reinforcement	Total Surface Area (Percent of Bar Mesh With 10 by 20-cm Openings)	Peak Resistance (kN) Under Normal Pressure (70 kPa)	
			Gravel Sand	Silty Clay and Gravel
2	Bar mesh (10 by 20-cm openings)	100	167	176
5	8 longitudinal bars	69	15	–
6	12 longitudinal bars	100	30	23.1
7	1 solid steel plate	409	79	–
8	1 galvanized steel strip	34	12	–
9	2 galvanized steel strips	68	24	22.2
10	3 galvanized steel strips	102	30	25.8
–	Bar mesh (13 by 36-cm openings)	97	–	129

Notes: 1 kN = 0.225 kips, 1 kPa = 0.145 lbf/in^2, and 1 cm = 0.394 in.

Figure 15. Shear-box, test for skin friction.



from a project site on US-101 at Cuesta Grade near San Luis Obispo, California.

For the same bar-mesh reinforcement, the peak pull loads were higher in a dense silty clay soil (Figure 9a) than those in a less dense gravelly sand soil (Figure 9). However, in the same soil, the peak pull load decreased substantially when mesh openings increased to 12.7 by 35.6 cm (5 by 14 in), as shown in Figure 9b.

Failure Mode

The failure mode resulting from laboratory tests indicated that all the specimens with bar-mesh reinforcement failed because a cone-shaped soil wedge developed (Figure 10) while the specimens with longitudinal-bar and steel-strip reinforcements failed because of slippage, with only a local slump of the soil at the front face of the specimens (Figures 11 and 12). The specimen with a solid steel-plate reinforcement also failed because small cone-shaped soil wedges developed (Figure 13) that are similar to the specimen with bar-mesh reinforcement (Figure 10). This mode of failure is believed to represent full mobilization of soil resistance, i.e., the development of a passive pressure wedge. The cone-shaped failure mode indicates that the soil and bar-mesh reinforcement failed as a unit.

Comparison of Test Results

The pull test results for the smooth bar-mesh reinforcement with 10.20 by 20.30-cm (4 by 8-in) openings embedded in gravelly sand soil under three different normal loads are shown in Figure 14. The yield, peak, and residual loads are all proportional to the normal loads.

The comparison of the pull resistance for different kinds of reinforcement embedded in gravelly sand soil are given in Table 2. The smooth bar-mesh reinforcement has a much higher pull resistance than the other kinds. For the same surface area, the bar-mesh reinforcement has a peak pull resistance about six times that of the longitudinal-bar and steel-strip reinforcement.

A comparison of the pull resistance for bar-mesh reinforcement embedded in gravelly sand and silty clay respectively is also given in Table 2. The same bar-mesh reinforcement embedded in a dense silty clay soil has greater pull resistance than that embedded in the gravelly sand soil. The higher pullout resistance could reflect the higher density or effect of cohesion present in the silty clay sample. For the same soil material, the bar-mesh reinforcement with larger mesh openings produced lower pull resistance.

SMALL-SCALE LABORATORY PULL TESTS

Small-scale laboratory pull tests were also conducted in a specially designed shear box (Figure 15) to determine the skin friction between the galvanized reinforcing steel strip and two types of soil, namely, decomposed granite from Cal-39 and gravelly sand material from I-5. These test results (Table 1) suggest that the skin friction angle is slightly smaller than the internal friction angle of the soil for granular material.

FIELD AND LABORATORY TEST CONCLUSIONS

Field Test

1. The soil will not be strained significantly until the proportional limit (or yield point) is reached. At this point, the load-deformation curve becomes nonlinear for the composite steel strip and soil material.
2. The maximum tensile stress in the reinforcement is developed near the front face of the wall for any external-applied pull force.
3. The required minimum length of steel strip is about 3.1 m (10 ft) for a reinforced earth fill lower than 3.1 m (10 ft) in height.

Laboratory Test

1. The proportional limit on the front-end, load-deformation curve can be determined from the displacement on the front-end, time-deformation curve at the point where an abrupt increase in deformation occurs on the time-deformation curve for the rear of the specimen. Thus, the soil will not be significantly strained until reaching a proportional limit that defines the initial point of soil-steel interaction as the composite material.
2. For the same surface area, plain bar-mesh reinforcement has nearly six times the pullout resistance

as steel-strip or plain longitudinal-bar reinforcements in gravelly sand soil.

3. Bar-mesh reinforcement embedded in dense silty class soil exhibited greater pull resistance than bar-mesh reinforcement embedded in less dense gravelly sand soil.

4. An increase in mesh opening will substantially reduce the pullout resistance of the bar-mesh reinforcement.

5. The skin friction angle between a galvanized steel strip and soil for granular material is only slightly smaller (6 to 13 percent) than the internal friction angle of the soil. For practical design purposes, the skin friction angle between the galvanized steel strip and soil material can be assumed to be 10 percent smaller than the internal friction angle of the soil.

6. Cohesive soil of low plasticity can be used in reinforced earth providing that bar-mesh reinforcement is used.

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Some Uncertainties of Slope Stability Analyses

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Some practical limitations of total stress and effective stress analyses are discussed. For clays having a liquidity index of 0.36 or greater, ϕ-equal-zero analyses based on laboratory undrained shear strengths give factors of safety close to the actual factor of safety. However, ϕ-equal-zero analyses based on field vane sheer strengths may yield factors of safety that may be too high. The difference between field vane and calculated shear strengths increases as the plasticity index increases. For clays having a liquidity index less than 0.36, ϕ-equal-zero analyses that use laboratory undrained shear strengths give factors of safety that are too high; however, the strength parameters can be corrected by the empirical relation presented here. An empirical relation for correcting field vane shear strength is also presented. A method is proposed for predicting the probable success of ϕ-equal-zero analysis. Data suggest that overconsolidated clays and clay shales or clays having a liquidity index less than 0.36 pose a slope design dilemma for engineers. An effective stress analysis based on peak triaxial shear strength parameters generally yields factors of safety that are too high; residual shear strength parameters frequently yield factors of safety that are too low. The theoretical strength of an overconsolidated clay that has undergone a softening process is approximated by using the effective stress parameters that might be obtained from triaxial tests performed on remolded, normally consolidated clay. It is suggested the soil be remolded to a moisture content equal to the plastic limit plus the product of 0.36 and the plasticity index.

Two limiting conditions (2) must be considered when designing a cutting in a clay or an embankment on a clay foundation to ensure against a first-time failure (no preexisting shear plane). The first condition is the short-term or end-of-construction case in which the water content of the clay does not change. In this case, excess pore pressures are controlled by the magnitude of the stresses acting in the clay or tending toward instability; therefore, significant dissipation of pore pressure does not occur. However, it is difficult to predict the excess pore pressures. Consequently, the short-term design is made by using the ϕ-equal-zero analysis and the undrained shear strength obtained from unconsolidated-undrained (UU) triaxial tests, unconfined compression (U) tests, field vane shear (FV) tests, or a combination of these tests.

The second condition is the long-term, steady seepage case. In this case, pore pressure do not depend on the magnitude of total stresses but are controlled by the flow pattern of underground water or the groundwater level. Excess dissipation of pore pressure occurs and the clay exists in a drained state. Long-term design is performed in terms of effective stress and the drained shear strength parameters (ϕ' and c') that are conventionally obtained from consolidated isotropically, drained (CID) triaxial tests; consolidated isotropically, undrained (CIU) triaxial tests with pore pressure measurements; consolidated-drained, direct shear (CDS) slow tests; or a combination of these tests.

For a cutting in a clay, the long-term stability is considered critical because pore pressures are initially