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Cohesive soil retaining walls reinforced with geotextile

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ABSTRACT: Centrifuge model tests were performed to study the behavior of soil retaining walls reinforced with a nonwoven geotextile simulant and backfilled with cohesive soil. Models were constructed on either firm or rigid foundations reinforced with different lengths of reinforcement,and loaded to failure under increasing self weight in the geotechnical centrifuge. Two modes of failure, designated as overturning and rotational sliding, were identified in this study. The reinforced models on firm foundations showed higher prototype equivalent heights at failure compare to the identical models with rigid foundations. A limit equilibrium stability analysis incorporating tangential or horizontal reinforcement was carried out to predict the behavior of the model walls.

1 INTRODUCTION

Soil being used as the backfill comprises about 35 to 45 percent of the total cost of a reinforced system, depending on the type of project and the availability of good quality backfill material. Accordingly, the economic attraction of a reinforced soil retaining wall is reduced when backfill must be imported to the site. In that case using on-site marginal material could be a cost effective option in terms of the economy of the project. Design which can accommodate, instead, marginal or even cohesive soil as backfill has become a very attractive possibility, with results from experimental backing up the likely feasibility of such walls. Along these lines an experimental study was carried out to examine the behavior of geotextile reinforced walls with low-quality backfill material. Geotechnical centrifuge modeling technique, following the work of Mitchell et al. (1988) and Bolton et al. (1978), was applied in this research. The geotechnical centrifuge at the University of Maryland used in this study is a 2.7m diameter Genisco model 1230-1 G-accelerator with an acceleration of 200g.

2 MATERIAL PROPERTIES

The geotextile used in this study is a non-woven fabric manufactured by Pellon Co. as interfacing material. The maximum tensile strength of the geosynthetic simulant, using ASTM wide-width test (D4595), was measured to be 0.053 kN/m at 18% strain.

The soil used in all models as the backfill, the retained fill and, when appropriate, for the foundation, was Hydrite Kaolin type "R" processed by Dry Branch Kaolin Company in New Jersey, USA. The liquid limit of the kaolin is 49% and the plastic limit 33%. The maximum dry unit weight in a standard Proctor test is 14.2 kN/m^3 at an optimum moisture content of 29%.

3 EXPERIMENTAL DETAILS

All model walls were constructed in a rigid aluminum container with inside dimensions of 400 mm by 300 mm in area, by 300 mm in depth. Models were constructed either on the rigid base of the model container, on which sand had been glued to provide a rough interface between the model and the foundation, or on a firm compacted clay foundation. For these models the foundation soil was mixed at optimum moisture content and then compressed, increasing stress slowly using a single loading plate to reach a maximum vertical stress of 337 kN/m^2 over a period of 5 minutes. When that stress was reached, the load was immediately removed. This produced a clay foundation with a dry unit weight of 13.5 kN/m^3 and a degree of saturation of 78%. The result was a foundation layer that was firmer than the same kaolin prepared for the retained fill and the backfill of the model walls.

3.1 Model construction

The inside vertical side boundaries of the container were sprayed with silicon, and overlain with a thin plastic film to reduce boundary friction effects. After foundation preparation, an aluminum block was laid on the foundation at the toe of the wall to be constructed, to provide lateral support during model construction. The first layer of reinforcement was then placed on the exposed portion of the foundation, a layer of soil placed, in turn, on it, and the geotextile folded back into the soil to provide a flexible facing for the wall. The length of the geotextile overlap, stretching back into the wall was 32 mm in all models. A compressive stress was then applied increasing slowly to a maximum of 175 kN/m^2 over a period of 5 minutes. The result was a lift of backfill and retained fill with dry unit weight of 12.3 kN/m^3, and degree of saturation of 94% with an average water content of 35%. This process was repeated for successive layers, each of which had finished thicknesses of 19 mm, until the model wall reached the desired height. A profile of a model is shown in Figure 1. The lateral support blocks were then removed before the centrifuge test. A full height lateral support during construction is not desirable in the field, since it is beneficial to develop gradual tensioning of the reinforcement as a wall is constructed. This gradual tensioning is achieved in the centrifuge models during the steadily increasing self-weight loading. The top of the model was sprayed with dark paint to highlight the development of tension cracks on the surface of the white clay. On the vertical side of the model, which would be visible during the test through a Plexiglas window, a grid of dots was painted to highlight cross-sectional deformation.

3.2 Testing procedure

Each test involved loading a model by increasing gradually the self-weight of the model until model failure occurred. The rate of these increases was 2g/minute until cracking was observed, at which point the rate was decreased, allowing any slope movement to cease before further increases were made. After a test, the model was disassembled to examine the deformations of the reinforcements at different elevations. The coordinates of the failure surfaces were recorded using a profilometer, measuring the vertical profile at 10 mm horizontal intervals through various model cross-sections. Direct shear tests were performed on specimens retrieved at various depths in the unfailed rear portion of the model after failure, subjecting each to normal stresses equal to the maximum experienced by the specimen during a test due to overburden load. The straight line Mohr-Coulomb strength parameters fit to the data from all models were $\phi=20.6^0$, and c=19.4 kN/m^2, although in the stability analysis, c and ϕ were developed individually for each model for stability analysis.

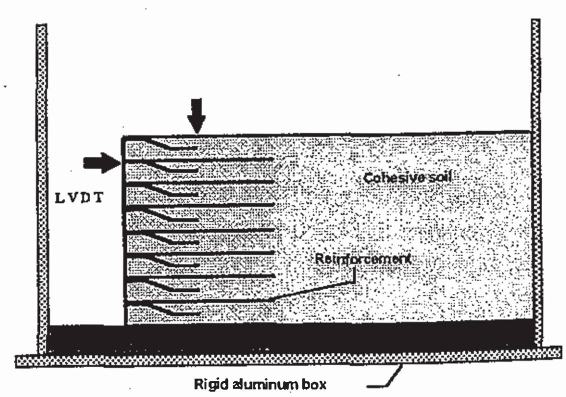

Figure 1: Schematic diagram of a typical model

4 DISCUSSION OF RESULTS

Table 1 presents the model geometry and the results of the model walls, 152 mm high, built on either firm or rigid foundations. Reinforcement configurations varied from no reinforcement, to a maximum reinforcement length of 114 mm or 0.75 times model wall height, with eight layers of reinforcement in every model.

4.1 Failure characteristics

The failure mechanisms and the prototype equivalent heights at failure are interconnected functions of reinforcement length. Figure 2 shows a plot of the prototype equivalent heights at failure ($H_f = H_m \times N_f$). Unreinforced vertical models M-34 on firm foundations failed by overturning at prototype equivalent height of 5.3m. In this mode of failure a block of soil at the front of the model toppled forward, rotating around the toe of the wall (Figure 3). When the

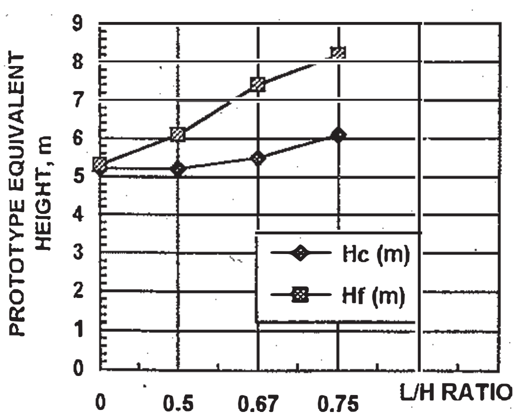

Figure 2: Prototype equivalent height

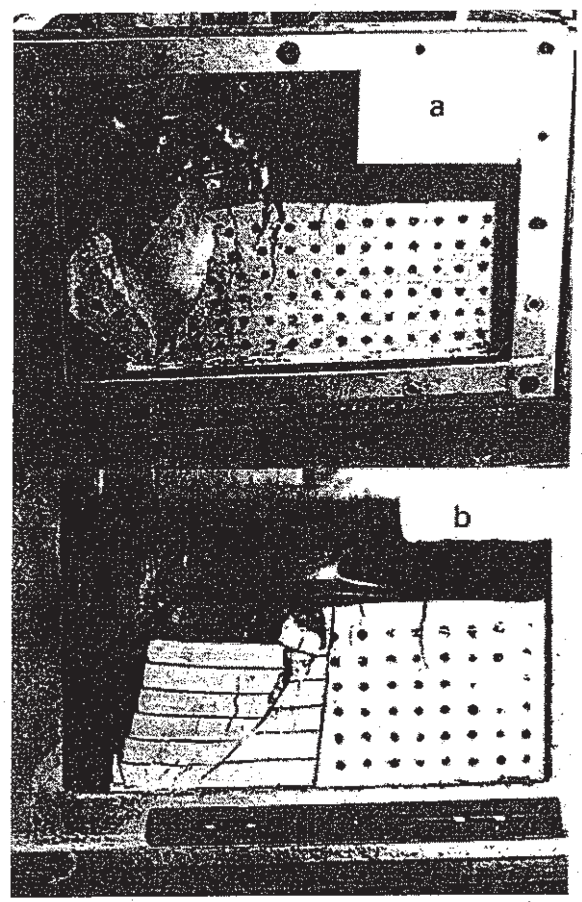

Figure 3: Modes of failure observed after centrifuge tests: (a) Overturning, (b) Rotational sliding

Table 1: Geometrical configurations of tested models

MODEL NO.	FOUNDATION TYPE	LENGTH (mm)	L/H RATIO	H_c (m)	H_f (m)
M-34	FIRM	0	0	5.2	5.3
M-48	FIRM	76	0.50	5.2	6.1
M-49	FIRM	100	0.67	5.5	7.4
M-28	FIRM	114	0.75	6.1	8.2
M-14	RIGID	0	0	5.3	5.3
M-56	RIGID	114	0.75	5.6	7.3

L/H=length of reinforcement as a multiple of model height
H_c=prototype equivalent height when tension crack occurred
H_f=prototype equivalent height at failure

wall was reinforced with geotextile of length equal to 0.50H, as in model M-48, the prototype equivalent height at failure increased 15% to 6.1m, and failure occurred not by toppling but by rotational sliding, in which a block of soil mass slipped along a concave upwards surface. In this case, the reinforced zone acted as an unsatisfactory retaining wall, leaning outward due to differential compression in the lower layers of the reinforced soil, and leaving partially unsupported the retained fill behind it. The reinforced zone, however, postponed development of failure because movement of that now partially unsupported retained fill, was only possible when a failure surface developed which passed through the narrow reinforced zone. Rotational failure is usually preferable to toppling failure in terms of the damage resulting (Figure 4).

In vertical model with longer reinforcement (M-49:L/H=0.67; M-28:L/H=0.75) failure occurred at progressively greater prototype equivalent heights of 7.4m and 8.2m, corresponding to 40% and 55% improvements, respectively, over the unreinforced vertical models. The lower layer of the reinforced soil also compressed differentially. For model M-49 failure began immediately behind the reinforcement, however, when the L/H ratio increased to 0.75, as in model M-28, the slip surface passed entirely within the reinforced zone, leading to internal failure.

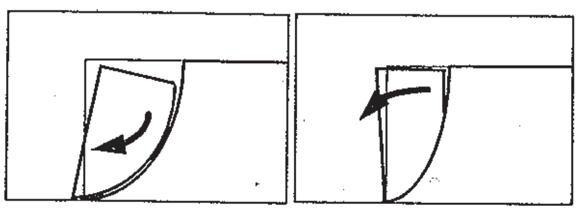

Figure 4: Failure mechanisms

4.2 Effect of foundation

A retaining wall based on a deformable foundation will undergo some settlement, some deformation and some internal redistribution of stresses which can be a benefit for reinforced soil structures, provided the deformations are not excessive (see Porbaha and Goodings (1994) for the adverse influence of soft foundations on model wall stability). Data from this work are presented in Table 2 to show a comparison of the behaviors of models grouped for identical geometry but with firm and rigid foundations.

When vertical models of identical geometry reinforced with L/H=0.75 are compared, as in model M-28 on a firm foundation and M-56 on a rigid foundation, the prototype equivalent height was 8% less at development of the tension crack and 11% less at failure for the model on rigid foundation. However, in unreinforced vertical walls, the foundation had no identifiable, effect on the model behavior. This is attributed to the pore water pressure at the interface, nonlinearity of soil-geotextile interaction, and stress redistribution within the reinforced zone due to change in the stiffness of the foundation.

Table 2: Effect of foundation rigidity on prototype equivalent height

MODEL NO.	FOUNDATION TYPE	L/H RATIO	H_c (m)	H_f (m)	H_c/H_f
M-34	FIRM	0	5.2	5.3	0.97
M-14	RIGID	0	5.3	5.3	1.00
M-28	FIRM	0.75	6.1	8.2	0.74
M-56	RIGID	0.75	5.6	7.3	0.77

L/H=length of reinforcement as a multiple of model height
H_c=prototype equivalent height when tension crack occurred
H_f=prototype equivalent height at failure

4.3 Stability analysis

The analysis was based on the two-dimensional limit equilibrium technique using simplified Bishop method, modified for reinforcement in a computer program prepared by Geocomp Corporation (1992). This method was based on the U.S. Federal Highway Administration (FHWA) guidelines for design of reinforced soil walls and slopes, as outlined in Christopher et al. (1989).

Soil properties input to the program included cohesion, c, angle of internal friction, ϕ, and soil unit weight, γ. One set of total stress soil strength parameters, c and ϕ, were developed for the backfill of each model using direct shear strength data from undisturbed soil specimens retrieved from that model. Total stress input parameters for firm foundations were c = 30 kN/m^2, ϕ=28^o and γ = 18.2 kN/m^3, derived from specimens tested in direct shear. High dummy numbers were used for the rigid foundations.

Table 3 summarizes the results of the limit equilibrium stability analyses. Safety factors were calculated for the stress levels at which the first tension crack developed (case I), and at which failure was observed to occur (cases II and III). The direction in which the geotextile is assumed to act must be selected by the program user. Case II assumed the geotextile provided a horizontal force, whereas case III assumed the geotextile force acted tangentially to the slip surface. Examining safety factors at failure the general conclusion is that the analysis proved to be a very acceptable predictor of model behavior. Moreover, the actual slip surfaces are closely consistent with those predicted by the analyses, as shown in Figure 5.

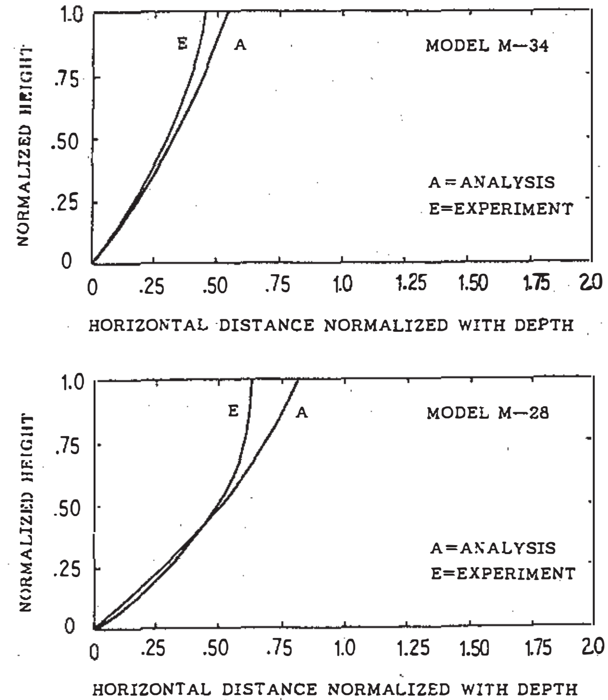

Figure 5: Actual vs. predicted failure surfaces

The success of this analysis sheds light on the suitability of the input parameters. As noted earlier, total stress soil strength parameters were input. These parameters contained within themselves the effect of negative pore pressures likely to be present in the backfill during a test. They were, then, short term in nature and in this case less conservative than long term.

Case III was intended to explore the effect on calculated factor of safety when the reinforcement was assumed to act horizontally. When the geotextile is assumed to act horizontally, there is a component of its strength acting tangentially to the failure surface and

Table 3: Results of stability analysis

MODEL NO.	FOUNDATION TYPE	L/H RATIO	COHESION (kN/m^2)	FRICTION ANGLE (deg.)	FS CASE I	FS CASE II	FS CASE III
M-34	FIRM	0	16.3	21.3	0.98	0.96	-
M-48	FIRM	0.50	18.6	20.1	1.09	0.98	0.97
M-49	FIRM	0.67	17.8	21.5	1.07	0.87	0.84
M-28	FIRM	0.75	20.0	20.8	1.07	0.87	0.83
M-14	RIGID	0	17.8	20.7	1.02	1.02	-
M-56	RIGID	0.75	22.8	19.4	1.28	1.00	0.99

FS= Factor of safety

another component of force acting normal to the failure surface which provides increased resistance to failure by increasing frictional soil resistance. The sum of these two components of force is less than when the full strength of the geotextile is assumed to act tangentially to the failure surface. The result in this analysis is that when geotextile strength is assumed to act horizontally, the assessment of wall stability is more conservative by about 10 % or less, when safety factors of the two cases are compared. Deformations of models prior to failure suggest that it is likely these geotextiles did stretch along the failure surface before failure, thereby acting tangentially. Further details of the behaviors of vertical and slopping walls are discussed by Porbaha and Goodings (1995).

5 CONCLUSIONS

The behavior of geotextile reinforced cohesive vertical walls was investigated using reduced scale centrifuge modeling technique. Model walls on firm or rigid foundations showed two modes of failure: overturning and rotational sliding. When backfill is cohesive soil, to derive maximum benefit from the presence of the geotextiles, the length of reinforcement for vertical walls should be at least 75% of the height to contain the failure surface within the reinforced zone. Regarding the foundation rigidity, the reinforced models on firm foundations showed higher prototype equivalent heights at failure compare to the identical models with rigid foundations.

The suitability of Bishop's simplified method of limit equilibrium analysis modified for soil reinforcement, was tested and the factors of safety at failure was close to one for overwhelmingly cases for these models at failure.

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