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港湾技術研究所報告
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Stability of Reinforced Retaining Systems under Artificial Gravity

Ali PORBAHA*

要　旨

ジオテキスタイルなどを用いた補強土工法は，擁壁，斜面安定や盛土工事に広く用いられている．補強盛土の挙動に関しては，多くの研究がなされている．しかし，盛土下の基礎地盤の剛性が盛土挙動に及ぼす影響については，解析的に検討した研究例はいくつかあるものの，実験的な検証を行った研究は数少なく，基礎地盤の剛性の影響について未だ十分に解明されていないのが現状である．

そこで，本研究ではジオテキスタイルによる補強盛土の破壊挙動に及ぼす基礎地盤の剛性の影響について，遠心模型実験により明らかにすることを目的とした．

実験では，盛土下の基礎地盤の剛性を2種類に変化させるとともに，盛土勾配を90度から45度まで変化させた模型地盤を作製した．作製した模型地盤に付加する遠心加速度を一様に増加させ，盛土に発生するクラック，破壊現象に及ぼす基礎地盤の剛性の影響を調べた．さらに，簡易Bishop法による円弧すべり計算を行い，ジオテキスタイルの強度の盛土安定に及ぼす影響についても検討した．

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Stability of Reinforced Retaining Systems under Artificial Gravity

ALI PORBAHA*

Synopsis

This study investigates the failure characteristics of reinforced and unreinforced soil retaining structures backfilled with low quality fill material. Reduced scale model retaining structures reinforced with geotextile simulants were constructed on the firm and rigid foundations and then were exposed to an artificial gravity resulted from geotechnical centrifuge. The retaining structures are vertical walls, sloping walls, steep slopes and embankments with slope angles of 45^0. By placing the models in an artificially increased gravitational field, however, to increase the self-weight of the soil, the stresses can be made equal at geometrically corresponding points in the models and the full-scale prototypes. The results indicate an overall better performance for models on compacted clay foundations compared with the cases of unyielding rigid foundations in terms of prototype equivalent heights and relative improvements. On the other hand, the rigid foundations pushed the failure surfaces back from the faces of the walls and slopes. Stability analyses incorporating the tangential and horizontal effect of reinforcement were found to be in good agreement with the experimental results.

KEY WORDS: Geotextile, Foundation, Centrifuge modeling, Retaining structure, Cohesive soil.

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1. Introduction

"Reinforced soil" is a generic name that is applied to combinations of soil and distributed linear or planar inclusions (e.g., steel strips, steel or polymeric grids, geotextile sheets, steel nails, etc.) that are capable of withstanding tensile loadings, and in some cases, bending and shear stresses as well. This composite system has found its greatest applications for earth retaining structures (**Figure 1**).

Reinforced retaining structures are more cost-effective than conventional concrete retaining systems. Therefore, it is desirable to understand the mechanistic behavior of reinforced retaining systems under various conditions.

The rigidity of foundation plays a significant role in the performance of soil retaining systems in terms of both stability and serviceability (Chou and Wu[1]). This effect is more pronounced for geosynthetically reinforced soil retaining structures which are inherently flexible, and have the capability to withstand large deformations prior to failure. Bell et al.[2] reported the Glenwood Canyon geosynthetic test wall, in which large settlements, more than 60 cm was observed with only hairline cracks detected.

Several analytical and numerical studies have been conducted to investigate the effect of foundation on the behavior of retaining structures (Clough and Woodward[3]), and reinforced soil retaining structures (see, for example, Chou and Wu1[1], Michalowsky[4]). However, few experimental investigations have been reported (Goodings and Santamarina[5]) and therefore, the effect of foundation rigidity on the performance of soil retaining walls and slopes is not fully understood.

The purpose of this experimental investigation is to study the effect of foundation rigidity on the failure characteristics of geotextile reinforced soil retaining structures backfilled with low quality cohesive soil. Following the work of Bolton et al.[6], Mitchell et al.[7], Taniguchi et al.[8] centrifuge modeling technique was applied in this investigation.

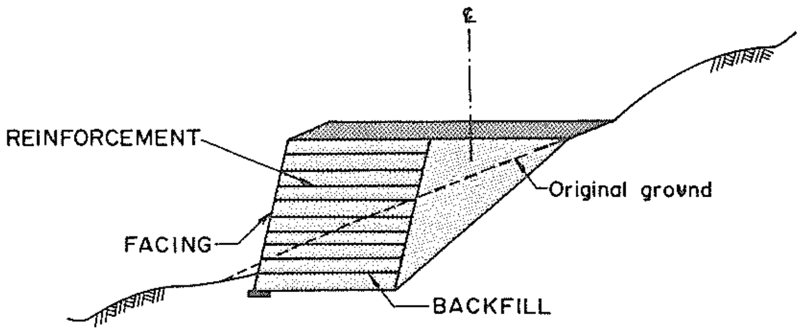

Figure 1: Components of a soil retaining structure

2. Basic Concept of Centrifuge Modeling Technique

The idea of making a small-scale model to study a physical phenomenon is common in many fields of engineering, including geotechnical engineering which started practically couple of decades ago. The behavior of soil is dependent on the stress due to the self-weight which increases substantially with depth. The reduced scale model, geometrically similar to the models of soils at 1g, demonstrates much different behavior than that occurs in the full-scale situation because the stress gradient in 1g models does not replicate the one in a full-scale prototype. By placing the model in an artificially increased gravitational field, however, to increase the self-weight of the soil, the stresses can be made equal at geometrically corresponding points in the model and the full-scale prototype. Centrifuge modeling, which uses this technique, has proven to provide a realistic approach for examining complex geotechnical problems (see for example: Schofield[9]; Fuglsang and Ovesen[10]). Small stress correct models present special opportunities to study the response of soil to a wide range of possible loading conditions which cannot be tested in full-scale engineering projects. Application of the centrifuge for modeling geosynthetically reinforced soil retaining structures is further discussed by Porbaha[11]. **Figure 2** shows the basic concept of centrifuge modeling technique.

3. Laboratory Investigation

3.1 Geotextile and Soil Properties

The soil used in all models, as the backfill and the retained fill, 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%.

The shear strength of the kaolin was obtained from direct shear tests on specimens taken from the model after failure occurred in the centrifuge. For kaolin clay cohesion ranged between 17.3 kN/m^2 and 23.8 kN/m^2 with friction angle ranging between 18.2^0 and 21.7^0.

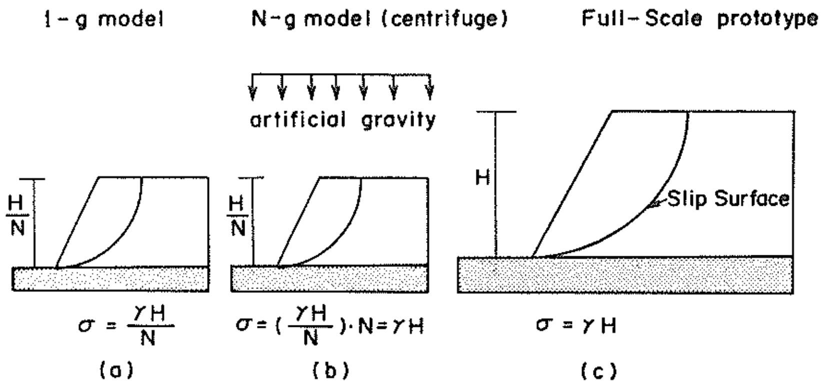

Figure 2: Basic concept of centrifuge modeling technique

The geotextile simulant used in this study is a non-woven polyester fabric manufactured by Pellon Co. as interfacing material. Nonwoven geotextile is a planar and random textile structure produced by bonding and or interlocking of fibers by mechanical, chemical or thermal means (Koerner[12)]). The tensile strength of this fabric is many times less than the full-scale geotextile strength. This is because while all stresses in reduced-scale centrifuge models are similar to the stresses at geometrically corresponding points in full-scale prototypes N times larger in scale, the forces at 1g in a model are still N times less than those in the prototype. For similarity, then, a model geotextile simulant should be N times weaker than a prototype geotextile. The maximum tensile strength of the geosynthetic simulant, using the ASTM standard wide-width test (ASTM D4595), was measured to be 0.053 kN/m at 18% strain.

3.2 Model Construction

All model walls and slopes 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 (rigid foundation), on which sand had been glued to provide a rough interface between the model and the foundation, or on a firm compacted clay foundation (firm 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.

The inside vertical sides of the container were sprayed with silicon, and overlain with a thin plastic film to reduce boundary friction effects. Sand had been glued to the bottom of the box to provide a rough interface between the model and the foundation. This foundation simulated an unyielding stable foundation, typically expected in design to provide the best support for a structure.

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. This process was repeated for successive layers, each of which had a finished thickness of 19 mm, until the model wall reached the desired height. 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. A profile of a model is shown in **Figure 3.** The top of the model was sprayed with dark paint to highlight the development of tension cracks on the surface of the white clay. The cross-section of the model was visible during the test through a Plexiglas window.

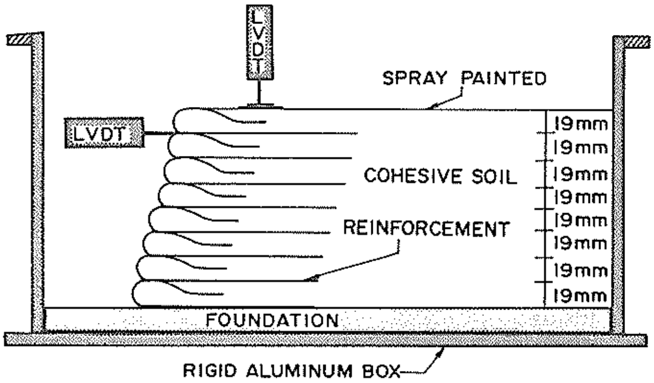

Figure 3: Geometrical configuration of a typical model

3.3 Centrifuge Test

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 wall movement to cease before further increases were made. After a test, the model was disassembled to examine the pattern of 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 pressure.

4. Test Results

Table 1 lists the gravitational accelerations and prototype equivalent heights both at tension cracking and at failure of the pairs of unreinforced and reinforced (length per height ratio, L/H=0.75) walls and slopes of 90^0 to 45^0 with firm and rigid foundations. All models were 152 mm in height and were constructed in 8 lifts of 19 mm in vertical spacings. The behavior of individual models in terms of crack development and failure mechanisms are discussed by Porbaha and Goodings[13,14].

5. Discussion of Results

5.1 Prototype Equivalent Height

Figures 4 and **5** show the plots of prototype equivalent heights, respectively, at first tension crack and at failure for slopes of different inclinations on firm and rigid foundations. A reinforced retaining system 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). Leshchinsky[15] also suggested that modest wall deformations may lead to force redistribution in reinforcement, improving overall stability and reinforcement effectiveness. Those data shown in Table 1 support this observation in both reinforced and unreinforced cases for the development of tension crack and at failure.

In general, failure in reinforced models with L/H=0.75 occurred on average at prototype equivalent heights 3 m higher in models with firm foundations, with the exception of vertical walls where the difference was 0.9 m. A similar although lesser effect was observed in terms of the development of cracking and the difference was progressively greater as slope angle decreased.

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 first tension crack and 11% less at failure for the model on rigid foundation. When sloping models are reinforced, the effect of the foundation is more pronounced. When reinforcement length is L/H=0.75, model M-32 on a firm foundation showed cracking at a prototype equivalent height of 7.7 m and failure at 11.4 m, 50% beyond the stress level at which cracking occurred. Compare this behavior to model M-57 on a rigid foundation, in which cracking occurred at 6.9 m and failure developed at stresses 28% beyond the point of first cracking at 8.8 m. The prototype equivalent stress level at failure, then, was 30% greater in the model on a firm foundation than in the model on the rigid foundation.

In unreinforced vertical walls, the foundation rigidity had no identifiable effect on the model behavior. In unreinforced sloping walls, such as model M-29 on a firm foundation and model M-27 on a rigid foundation, the development of cracks occurred at prototype equivalent heights of 6.5 m and 5.9 m, respectively, in those cases. A larger difference existed in their prototype equivalent heights at failure by how soon the failure occurred after the crack development. For model M-29 on a firm foundation the failure occurred at 7.2 m, 11% beyond the height at cracking, and for model M-27 on a rigid foundation, the failure occurred at 6.1 m, only 3% beyond the height at cracking.

Table 1: Effect of foundation on prototype equivalent height

MODEL NO.	SLOPE ANGLE (deg.)	FOUNDATION TYPE	L/H RATIO	N1 (g)	Nf (g)	Hp1 (m)	Hpf (m)
M-56	90.0	RIGID	0.75	37	48	5.6	7.3
M-57	80.5 (1H:6V)	RIGID	0.75	45	58	6.8	8.8
NA	71.6 (1H:3V)	RIGID	0.75	-	-	-	-
M-26	63.4 (1H:2V)	RIGID	0.75	71	82	10.8	12.5
M-30	45.0	RIGID	0.75	114	>124	17.3	>18.8
M-14	90.0	RIGID	0	35	35	5.3	5.3
M-27	80.5 (1H:6V)	RIGID	0	39	40	5.9	6.1
M-25	71.6 (1H:3V)	RIGID	0	46	48	7.0	7.3
M-24	63.4 (1H:2V)	RIGID	0	51	55	7.8	8.4
M-22	45.0	RIGID	0	65	72	9.9	10.9
M-28	90.0	FIRM	0.75	40	54	6.1	8.2
M-32	80.5 (1H:6V)	FIRM	0.75	51	75	7.8	11.4
M-47	71.6 (1H:3V)	FIRM	0.75	67	86	10.2	13.1
M-20	63.4 (1H:2V)	FIRM	0.75	92	102	14.0	15.5
M-34	90.0	FIRM	0	34	35	5.2	5.3
M-29	80.5 (1H:6V)	FIRM	0	43	47	6.5	7.1
M-19	71.6 (1H:3V)	FIRM	0	52	58	7.9	8.8
M-21	63.4 (1H:2V)	FIRM	0	65	67	9.9	10.2
M-23	45.0	FIRM	0	>110	NF	-	-

N1 = Centrifugal acceleration @ first tension crack (g)
Nf = Centrifugal acceleration @ failure (g)
Hp1 = Prototype equivalent height @ first tension crack (m)
Hpf = Prototype equivalent height @ failure (m)
L/H = Length of model reinforcement as a multiple of model height.
NA = Not available

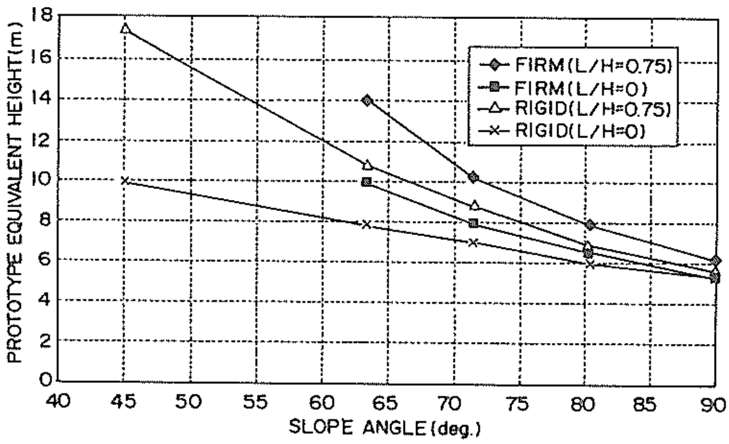

Figure 4: Prototype equivalent height at tension crack

Unreinforced model slopes of 71.6°, 63.4°, and 45° (M-25, M-24, M-22) were constructed on rigid foundations. Two reinforced slopes at 63.4° with L/H=0.75 (M-26) and at 45° with L/H=0.75 (M-30) were also built on a rigid foundation. In every case, models on rigid foundations were inferior in their performance. Models on rigid foundations tended both to fail and to crack at accelerations at least 10% less than otherwise identical models on firm foundations. The flatter the slope was, the greater the negative influence was on stability, irrespective of being reinforced or unreinforced. The locations of cracks and failure surfaces were also somewhat different, developing further behind the crests of the slopes with rigid foundations than those with firm foundations. Note that model M-30, with a rigid foundation, L/H=0.75, and β=45°, cracked at a prototype equivalent height of 17.3 m, whereas model M-23 on a firm foundation with β=45° but without reinforcement showed no signs of cracking when experiencing a maximum prototype equivalent height of 16.7m. Failures and cracks in unreinforced models also occurred at greater prototype equivalent heights when the models were founded on firm foundations. This is attributed to the development of pore water pressures and stress distributions between the superstructure and the foundation.

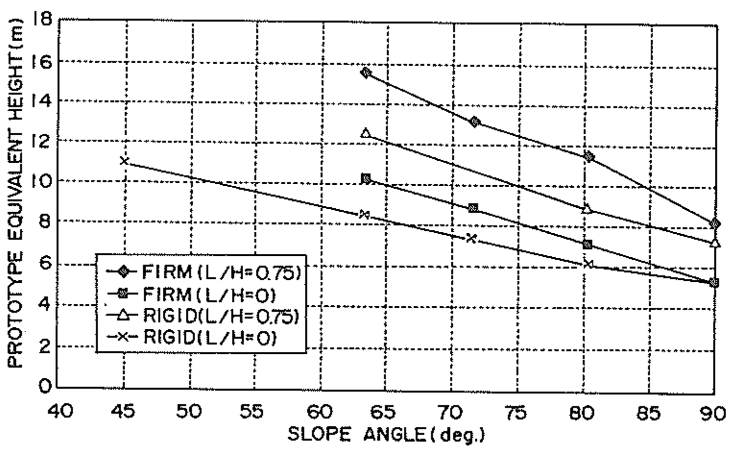

Figure 5: Prototype equivalent height at failure

5.2 Effect of Foundation Rigidity on Walls and Slopes

The better performance of firm foundation models is attributed to the opportunity that a more deformable foundation, compared to rigid condition, offers for redistribution of stresses within the flexible wall or slopes, allowing "safe" strain to take place which permits development of the tensile resistance of the geotextile -- the same phenomena occurring for prestressing of steel in reinforced concrete -- which can then be transferred to the soil, strengthening the structure as a whole. This pattern of behavior, then, was consistent with the general patterns observed by Goodings and Santamarina[5].

The increase in stress vector at the toe of an embankment due to change in foundation modulus at the toe of an embankment was investigated by Clough and Woodward[3] which supports the findings in this study. Through numerical simulation the stresses at the toe were calculated for the embankment shown in **Figure 6**. Values of foundation modulus ranged from infinity (the rigid foundation case) to 100 ksf (=4788 kPa), which is the same as the embankment material. The analyses were carried out by the incremental construction finite element procedure, considering ten construction lifts. Stresses calculated at the base of the embankment in the four cases are presented in **Figure 7**. Both the normal stresses (σ_x) and shear stresses (τ_{xy}) can be seen to vary with the foundation flexibility.

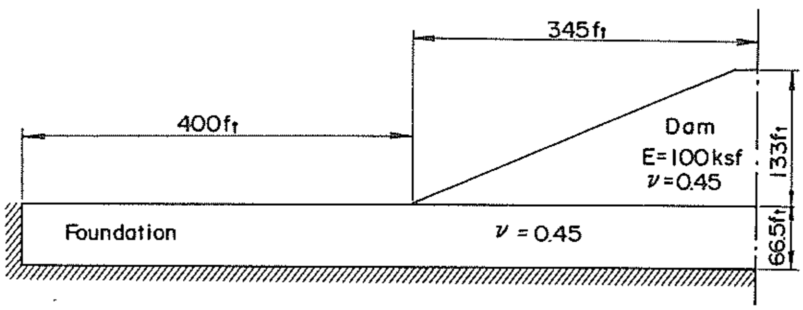

Figure 6: Foundation-embankment system (after Clough and Woodward[3])
(1ft = 0.3048, 1ksf = 47.88 kPa)

The stress concentration at the interface of rigid foundation was also observed by Leshchinsky and Marcozzi[16] who studied the effect of foundation rigidity on bearing capacity of a shallow foundation. They reported that flexible foundation generates smaller pressure concentrations that potentially result with a collapse governed by peak strength along most of the failure surface; i.e. approximately uniform shear strength mobilization occurs (see **Figure 8**). Consequently, the potential for a significant nonsimultaneous mobilization of peak strength may result in a lower load-bearing capacity for the rigid foundation as compared with the flexible one.

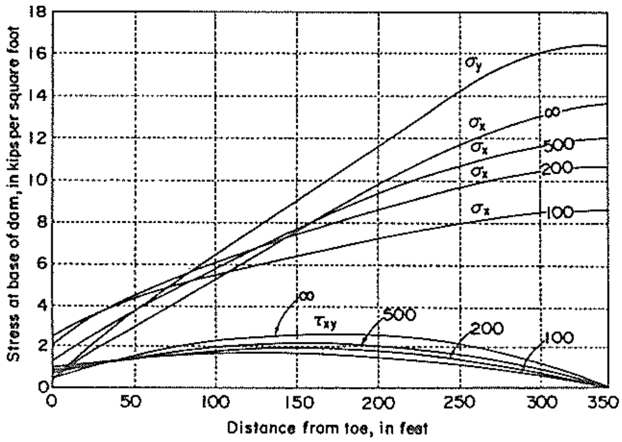

Figure 7: Effect of foundation flexibility on stress at the base :σ_x= normal stress, τ_{xy}= shear stress, E= modulus of elasticity (after Clough and Woodward,[3)]) (1ft = 0.3048, 1ksf = 47.88 kPa)

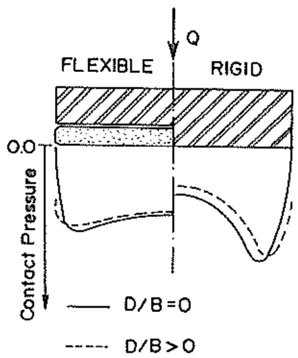

Figure 8: Contact pressures for centrally loaded foundations (after Leshchinsky and Marcozzi[16)])

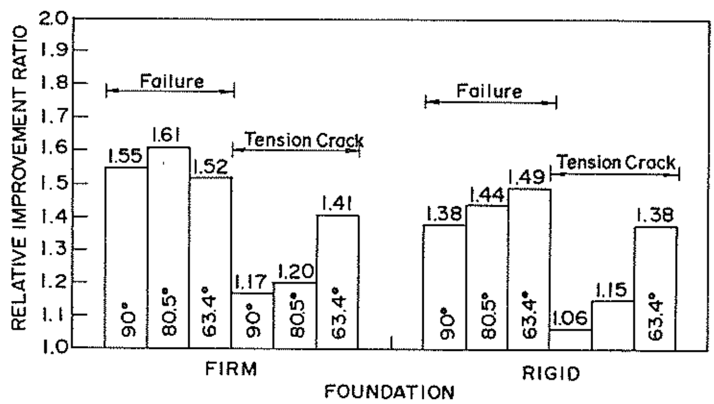

Figure 9: Variation of relative improvement ratio with foundation rigidity

5.3 Relative Improvement

The ratio of prototype equivalent height of the reinforced model relative to the corresponding prototype equivalent height of the unreinforced model with identical geometry and foundation condition is a measure of reinforcement efficiency and denotes here as relative improvement ratio (Ψ). The higher the value of Ψ implies a better measure of reinforcement performance and thereby a more desirable reinforced system, when the reinforced volumes are equal. **Figures 9** and **10** show better improvement ratios for models on firm foundations compared with those on rigid foundations when tension cracks occur and at failure, respectively. This statement is true for all slopes and walls, and the effect is more pronounced for 80^0 models on firm foundations with equal reinforced volumes. This means that a more deformable foundation is more efficient configuration for sloping walls than for the slopes. This is mainly attributed to the nonlinearity of soil-geotextile interaction and the variation of induced earth pressures due to changes in slope angles.

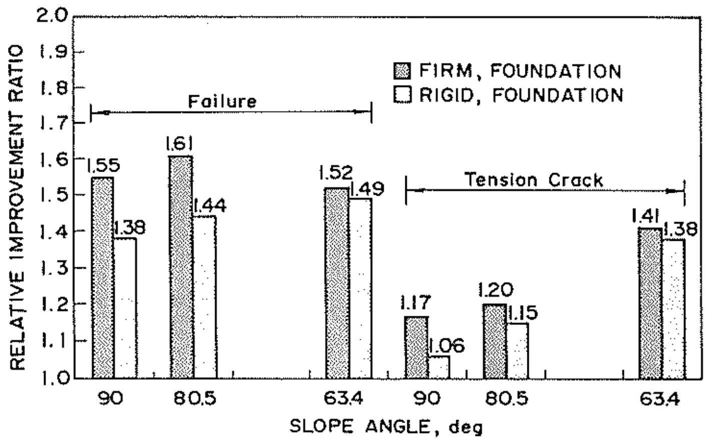

Figure 10: Variation of relative improvement ratio with slope angle

5.4 Slip Surfaces

The positions and traces of failure surfaces are functions of the length of reinforcement and rigidity of the foundations for different slope inclinations. Post-test inspection of models indicated that a rigid foundation tends to result in the slip surface developing further behind the front of the wall for both reinforced and unreinforced models. However, at all times the failure surfaces of reinforced walls and slopes are behind the positions of failure surfaces of unreinforced models.

Figure 11 shows the positions and traces of slip surfaces for the walls and slopes reinforced to a maximum length equivalent to 0.75 of the height, observed after failure in the centrifuge loaded under self-weight. The failure surfaces of reinforced models are comprised of two portions; a vertical tension crack and a curvilinear slip surface which extends to the toe of the wall or slope. The development of tension cracks and curvilinear slip surfaces are attributed to the intrinsic nature of cohesion in soil, which is not commonly observed in retaining structures with granular backfill. The reason why the slip surfaces are more smooth for the case of firm foundation is most likely attributed to the stress concentration at the interface of the reinforced retaining system and the rigid base.

Porbaha[18)] reported the behavior of lime-treated cohesive soil retaining walls, and observed identical phenomenon for development of failure surfaces when the length of reinforcement increased from L/H=0.50 to 0.75 of the height. Similarly, in those cases the failure surfaces moved toward the face of the retaining structure, although the mode of failure, the shape of slip surfaces and the collapse mechanisms were quite different from untreated cohesive fills.

6. Analysis of Results

Virtually all design methods of geosynthetically reinforced structures are based on limit equilibrium analyses. The popularity of limit equilibrium methods among design engineers to perform stability analysis is the main incentive to apply Bishop method in this investigation. The analytical method used in this work was based on the two-dimensional simplified Bishop method, modified for reinforced slopes in a computer program prepared by Geocomp Corporation. This method was based directly on the U.S. Federal Highway Administration (FHWA) guidelines for design of reinforced soil walls and slopes, as outlined in Christopher et al.[19].

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 2 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. The concept of tangential and horizontal effect of reinforcement on slip surface is shown in **Figure 12**. Examining safety factors at failure the general conclusion is that the analysis proved to be a very acceptable predictor of model behavior considering the fact that limit equilibrium techniques do not usually take the effect of foundation rigidity into consideration when the slip surface is not deep-seated, as the case of this study.

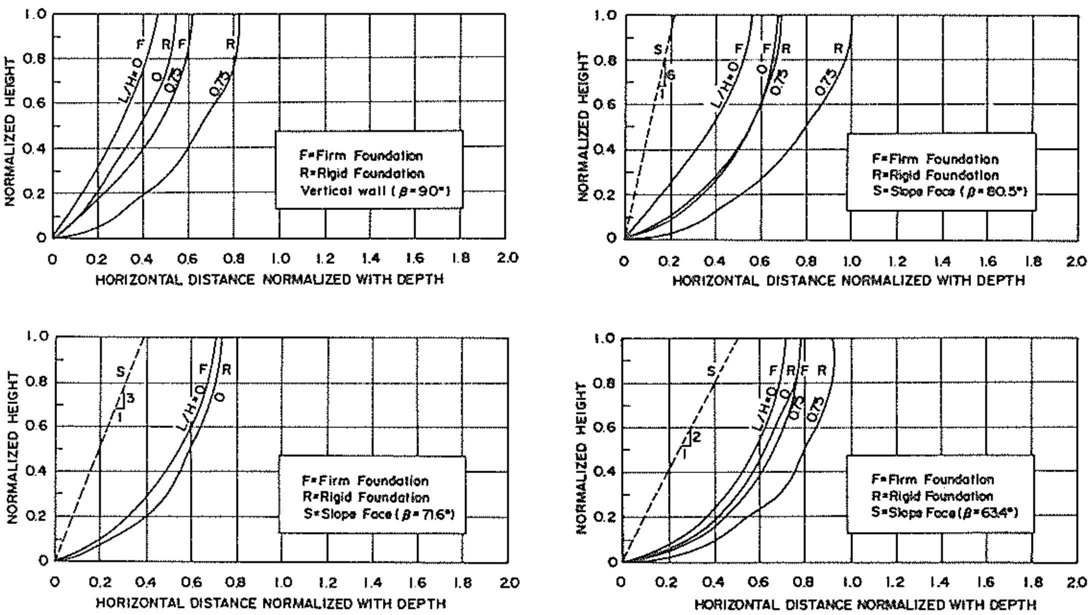

Figure 11: Traces of slip surfaces for different slope angles

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 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.

Table 2: Results of stability analysis

MODEL NO.	SLOPE ANGLE (deg.)	FOUNDATION TYPE	L/H RATIO	COHESION (kN/m^2)	FRICTION ANGLE (deg.)	FS CASE I	FS CASE II	FS CASE III
M-56	90.0	RIGID	0.75	22.8	19.4	1.28	1.00	0.99
M-57	80.5 (1H:6V)	RIGID	0.75	22.9	18.2	1.17	0.99	0.98
M-26	63.4 (1H:2V)	RIGID	0.75	22.3	19.6	1.08	1.00	0.99
M-14	90.0	RIGID	0	17.8	20.7	1.02	1.02	1.02
M-27	80.5 (1H:6V)	RIGID	0	17.3	21.4	1.09	1.07	1.07
M-25	71.6 (1H:3V)	RIGID	0	17.2	20.8	1.08	1.05	1.05
M-24	63.4 (1H:2V)	RIGID	0	16.7	20.6	1.09	1.04	1.04
M-28	90.0	FIRM	0.75	20.0	20.8	1.07	0.87	0.83
M-32	80.5 (1H:6V)	FIRM	0.75	23.8	20.6	1.16	0.90	0.88
M-47	71.6 (1H:3V)	FIRM	0.75	23.3	19.6	1.05	0.91	0.90
M-20	63.4 (1H:2V)	FIRM	0.75	22.4	20.1	0.96	0.91	0.90
M-34	90.0	FIRM	0	16.3	21.3	0.98	0.96	0.96
M-29	80.5 (1H:6V)	FIRM	0	17.8	21.7	1.05	0.97	0.97
M-19	71.6 (1H:3V)	FIRM	0	19.3	18.7	1.02	0.96	0.96
M-21	63.4 (1H:2V)	FIRM	0	19.5	18.4	0.98	0.96	0.96

FS= Factor of safety

7. Conclusions

An experimental investigation was carried out to study the behavior of reinforced retaining structures on firm and rigid foundations. Reduced-scale models of geotextile reinforced cohesive soil retaining structures were constructed and exposed to an artificial gravitational acceleration using geotechnical centrifuge. The following conclusions were drawn from this study:

1. Model walls and slopes founded on firm compacted clay foundations showed higher prototype equivalent heights than those on rigid foundations.The improvement ratio was higher for those models on firm foundations at development of tension crack and at failure.

2. Rigid foundations pushed back the failure surface of both reinforced and unreinforced models compared with firm foundations.

3. Stability analysis using simplified Bishop method incorporating the horizontal and tangential effect of reinforcement was found to be a good predictor of the model behavior, particularly when the tangential geotextile force was taken into account.

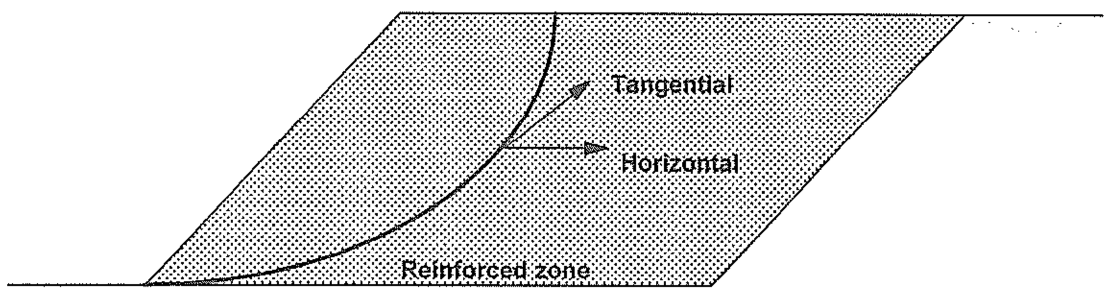

Figure 12: Tangential and horizontal resisting force of reinforcement that contribute to stability

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List of Symbols

c	:cohesion (N/m^2);
H	:model height (m);
H_{p1}	: prototype equivalent height at first tension crack (m);
H_{pf}	: prototype equivalent height at failure (m);
L	:length of the model reinforcement (mm);
L/H	:length of model reinforcement as a multiple of model height;
N_1	:centrifugal acceleration at first tension crack (g);
N_f	:centrifugal acceleration at failure (g);
n	:number of reinforcement layers,
β	:slope angle (degree);
ϕ	:internal friction angle (degree);
γ	:unit weight (kN/m^3).

Key Words: Geotextile, Foundation, Centrifuge modeling, Retaining structure, Cohesive soil.

Stability of Reinforced Retaining Systems under Artificial Gravity

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This study investigates the failure characteristics of reinforced and unreinforced soil retaining structures backfilled with low quality fill material. Reduced scale model retaining structures reinforced with geotextile simulants were constructed on the firm and rigid foundations and then were exposed to an artificial gravity resulted from geotechnical centrifuge. The retaining structures are vertical walls, sloping walls, steep slopes and embankments with slope angles of 45^0. By placing the models in an artificially increased gravitational field, however, to increase the self-weight of the soil, the stresses can be made equal at geometrically corresponding points in the models and the full-scale prototypes. The results indicate an overall better performance for models on compacted clay foundations compared with the cases of unyielding rigid foundations in terms of prototype equivalent heights and relative improvements. On the other hand, the rigid foundations pushed the failure surfaces back from the faces of the walls and slopes. Stability analyses incorporating the tangential and horizontal effect of reinforcement were found to be in good agreement with the experimental results.

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