Preliminary Investigation of Bearing Capacity of Layered Soils by Centrifugal Modeling

A. C. HERDY AND F. C. TOWNSEND

The bearing capacity of circular footings founded on a sand layer overlying a clay layer was investigated by using centrifugal model tests. Model footings corresponding to prototypes ranging from 0.6 to 1.5 m were loaded to failure. Comparisons between observed model behavior and Meyerhoff's bearing-capacity theory for layered soils were made. These comparisons showed excellent agreement (less than 13 percent), which when considering the sensitivity of bearing-capacity coefficients to ϕ, is well within engineering acceptance.

Traditional bearing-capacity analysis of footings is limited to homogeneous soils extending to a depth B beneath a footing. To overcome this limitation, several theories and analyses have been presented. Button (1) presented a theory for nonhomogeneous (layered) soils; however, his analysis was applicable to clays only ($\phi = 0°$). Reddy and Srinivasan (2) extended Button's work to include anisotropic strength properties. Desai and Reese (3) applied the use of finite-element techniques to analyze the bearing capacity of footings founded on various clay layers with nonhomogeneous and nonlinear stress-strain properties. Brown and Meyerhof (4) performed laboratory model tests to investigate Button's theory. These previous investigations only considered clay layers, and only recently have layers involving friction and cohesion been analyzed. Meyerhof and Hanna (5) described analyses of footings subjected to inclined load on layered soils. Later in 1979, their work on two-layered soils was extended to include three layers (6). Purushothamaraj, Ramiah, and Rao (7) and Vesić (8) have also investigated the subject.

As can be seen, the bearing capacity of footings founded on layered soils has received considerable attention. However, verification of these theories is restricted by the inability to perform full-scale tests. In this context, centrifugal model testing offers a practical means for providing laboratory models that when subjected to a gravitational field will be in similitude with prototype structures.

PURPOSE AND SCOPE

The objective of this investigation was to verify Meyerhof's (9) theory for the bearing capacity of footings on a sand layer overlying a clay. This verification was accomplished by performing centrifugal model tests on a circular footing founded on a sand layer overlying clay.

DESCRIPTION OF THEORY

Based on model tests and some field observations, Meyerhof (9) proposed a semiempirical theory for two cases: layered soil composed of a dense sand layer overlying a soft clay and a loose sand overlying a stiff clay.

Case 1: Dense Sand on Soft Clay

Meyerhof observed that if a footing rests on a relatively thin dense sand layer above a soft clay deposit, failure may occur by breaking through the sand stratum into the clay. An approximate solution was developed by considering the failure to be an inverted uplift problem. This theory was found to hold true when the ultimate bearing capacity of a homogeneous thick bed on sand (q_t) is much greater than that of the underlying clay deposit (q_b).

According to Meyerhof (9), the ultimate bearing capacity for a rough strip footing of width B and depth D_f founded on a dense sand overlying a soft clay can be calculated by the following equation:

$$q_o = cN_c + 2P_p \sin\delta/B + \gamma D_f \quad (1)$$

where

$$P_p = 0.5\gamma H^2 (1 + 2D_f/H)K_p/\cos\delta \quad (2)$$

and K_p is the coefficient of passive earth pressure and H is the depth of the sand layer underlying the footing. Meyerhof suggested that the angle δ may be estimated as 0.67ϕ.

Meyerhof also suggested that a coefficient K_s, which represents the punching shearing resistance on the vertical plane through the footing edge, can be used. This coefficient is related to the corresponding earth pressure coefficient K_p by $K_s \tan\phi = K_p \tan\delta$

This relationship was found by Caquot and Kerisel (10) and is shown in Figure 1 (9) for various friction angles ϕ. It is of interest to note that K_s increases rapidly with ϕ from about one to two times the Rankine value of $\tan^2 (45 + \phi/2)$.

By substituting all of the aforementioned values into Equation 1, it becomes the following:

$$q_o = cN_c + \gamma_s H^2 (1 + 2D_f/H) [(K_s \tan\phi)/B] + \gamma D_f \quad (3)$$

with a maximum value that corresponds to a single layer of sand of the following:

$$q_{o\,max} = q_t = (\gamma/2) BN_\gamma + \gamma D_f N_q \quad (4)$$

For circular footings of diameter B, depth D_f, and distance H above the clay surface, the bearing capacity can be approximated by the following:

$$q_o = 1.2cN_c + 2\gamma_s H^2 (1 + 2D_f/H) [(SK_s \tan\phi)/B] + \gamma D_f \quad (5)$$

with a maximum of the following:

$$q_{o\,max} = q_t = 0.3\gamma_s BN\gamma + \gamma D_f N_q \quad (6)$$

The above formula is an extension of the one for strip footings by considering the passive resistance P_p inclined at δ on a vertical cylindrical surface through the footing edge.

The S-factor that appears in the second term on the right-hand side of Equation 5 is a shape factor governing the passive earth pressure on a cylindrical wall. It may be conservatively taken as unity (1.0), especially for small ratios of H/B. Figure 1 also shows values of SK_s as a function of ϕ.

For rectangular footings of width B and length L, the first and second terms on the right-hand side of Equation 3 defined for strip footings should be multiplied by $(1 + 0.2B/L)$ and $(1 + B/L)$, respectively. This is an approximate interpolation between the values for bearing capacity obtained for circular and strip footings.

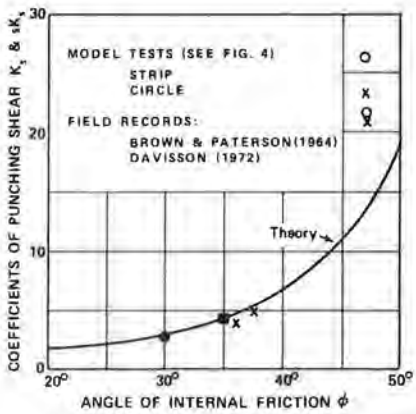


Figure 1. Coefficient of punching shearing resistance.

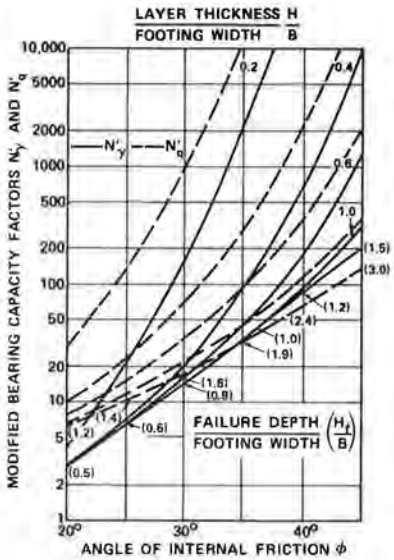


Figure 2. Theoretical modified bearing-capacity factors for strip footings.

Case 2: Loose Sand on Stiff Clay

In this case the bearing-capacity failure may be limited to the sand layer. By considering that the ultimate bearing capacity of a thick bed of sand (q_t) is less than that of a thick bed of clay (q_b), the bearing capacity of the stratum may be estimated by assuming that the sand layer rests on a rigid base. The sand layer in this case may fail laterally by a squeezing action.

The bearing capacity of a rough strip footing founded on loose sand overlying a stiff clay layer may be expressed as follows:

$$q_o = \gamma B N'_\gamma/2 + \gamma D_f N'_q \quad (7)$$

with a maximum value that corresponds to a single layer of clay of the following:

$$q_o = q_b c N_c + \gamma D_f \quad (8)$$

The theoretical modified bearing-capacity factors N_γ' and N_q' that are dependent on ϕ, the ratio of H/B, and the degree of roughness of the rigid base are shown in Figure 2 (9). One should note that these factors increase with ϕ from the lower limits of N_γ amd N_q and as the ratio of H/B decreases from the depth ratio of H_f/B of the failure surface in a thick bed of sand.

For the case of circular footings, a solution was

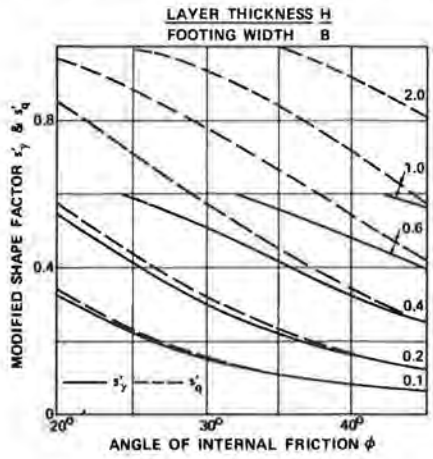


Figure 3. Theoretical modified shape factors for circular footings.

found by considering that in radial planes the stresses and shear zones are identical to those in transverse planes of a corresponding strip footing; i.e., the contact pressure distribution beneath the circle may be assumed to be similar to that for a strip. Therefore, the bearing capacity may be calculated from the following:

$$q_o = S'_\gamma \gamma B N'_\gamma/2 + S'_q \gamma D_f N'_q \quad (9)$$

with a maximum of the following:

$$q_o = q_b = 1.2cN_c + \gamma D_f \quad (10)$$

The modified shape factors S_γ' and S_q' are given in Figure 3 (9). These factors decrease from the conservative upper limits of $S_\gamma' = 0.6$ and $S_q' = 1$ for a thick bed of sand. Also, as ϕ increases and the ratio H/B decreases, both factors tend to similar values for small ratios of H/B.

For rectangular footings, the bearing capacity may be estimated by taking Equation 7 for strip footings and multiplying the first term on the right-hand side by $[1 - (1 - S_\gamma')B/L]$ and the second term by $[1 - (1 - S_q')B/L]$.

If the bearing capacity of the sand (q_t) approaches that of the clay (q_b), the failure surface beneath a footing extending into the clay and the shear zones becomes discontinuous at the interface, and the bearing capacity may be estimated by using an empirical parabolic intersection relationship of the following type:

$$q_o = q_t + (q_b - q_t)(1 - H/H_f)^2 \quad (11)$$

with a maximum for H/B = 0 when $q_o = q_t$.

EQUIPMENT

The University of Florida geotechnical centrifuge was manufactured by the Rucker Company and is a machine with a 1-m radius capable of accelerating a 25-kg payload to 85 g (Figure 4). It currently does not have a swinging bucket; the model is bolted to the end of the arm instead. For these tests, the soil was placed in a cylinder 25 cm in diameter. Loading of the model footings was accomplished by using a double-acting air piston to which regulated compressed air was supplied via two hydraulic slip rings. The load applied by the air piston was measured by using a miniature 226-kg (500-lb) load cell, while deformations were measured by using a

Figure 4. Principal layout of the University of Florida centrifuge.

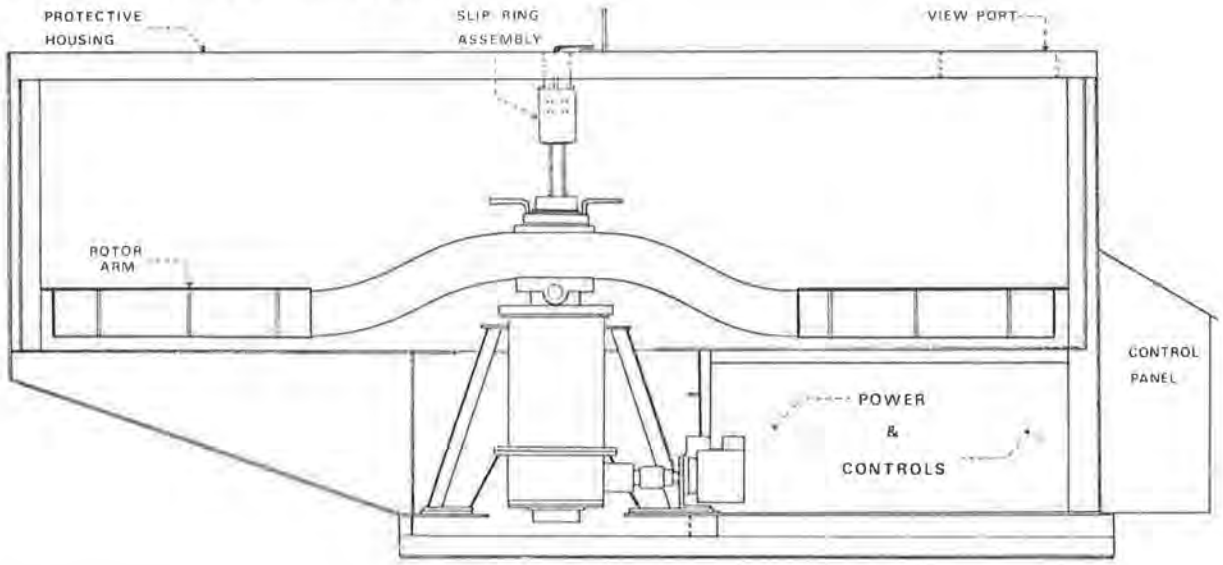


Figure 5. Schematic diagram of the centrifuge model.

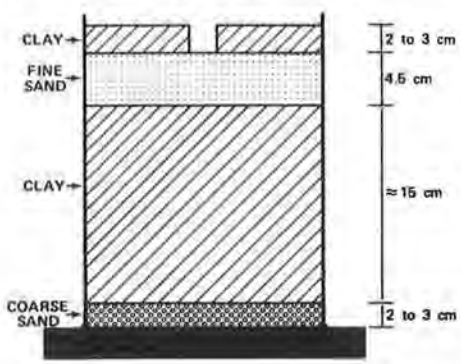
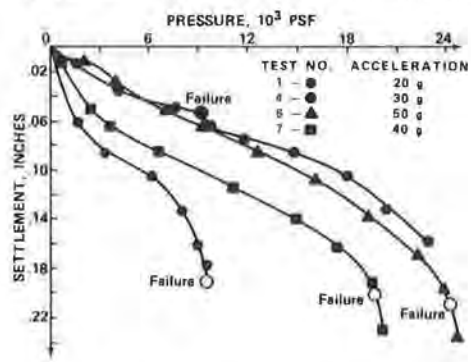


Figure 6. Relationships of stress versus deformation for model footings.



2.54-cm (1.0-in) linear variable differential transformer (LVDT). The model footing was an aluminum disk 19 cm in diameter to which sandpaper was glued to provide a rough contact surface.

SOILS AND MODEL PREPARATION

The clay base used in the models was kaolinite from Edgar, Florida; it had a liquid limit (LL) of 54 and a plasticity index (PI) of 24. The optimum water content was 26 percent. The sand used was an SP with D_{50} = 0.35 mm, e_{max} = 0.836, and e_{min} = 0.616. The ϕ-values determined by direct shear tests were 23° and 40° for the lowest and densest states, respectively. However, a correlation presented by Schmertmann (11) between ϕ and D_r was used to estimate ϕ for the model tests.

Model preparation consisted of placing the clay base into the model cylinder and statically consolidating the clay to a pressure equivalent to γHN (N = 50 g) by using a Tinius Olsen loading machine. To accelerate consolidation, a thin layer of coarse sand was placed in the bottom of the model container, and filter paper was placed over this sand. After consolidation (usually two days), the fine-sand upper layer was rained through a No. 30 sieve from a 1.5-m height. The sand layer was 4.5 cm thick, giving an H/B ratio of 1.5 as recommended by Tcheng (12).

Since the University of Florida centrifuge requires that the model container be mounted perpendicular to the radial acceleration plane, a thin layer of clay of 2-3 cm was placed on the clay surface to act as a membrane to prevent the sand from spilling out of the container. The influence of this layer was calculated as a surcharge. Figure 5 shows a sketch of the model after completion.

The soil weights and volumes were measured for each phase of model construction, and the corresponding densities of each layer were determined from them. Subsequent to each centrifuge model, test samples were taken from the clay layer and the unconfined compressive strength, water content, and density were determined. These values are presented in Table 1.

TEST PROCEDURES

Model tests were performed by assembling the loading piston and LVDT to the model footing, observing the zero readings, and accelerating the model to the test acceleration (either 20, 30, 40, or 50 g). The model was allowed to equilibrate for approximately 5 min at this acceleration, after which loading was initiated by supplying air pressure in 1-psi increments to the air piston, and the load and deformation values were recorded. Loading was continued until a sharp increase in deformation corresponding

Table 1. Summary of soil properties and model dimensions.

Test No.	Soil Property: Sand γ_s (pcf)	Sand D_r (%)	Sand ϕ[a] (degrees)	Clay γ_c (pcf)	Clay c[b] (psf)	Footing Dimension: Acceleration Level (g)	Prototype Footing Diameter B (ft)	Surcharge: Thickness (cm)	Surcharge γ_q (pcf)
1	98.3	73.6	37.8	105.2	178	20	2.0	1.9	112.5
4	91.5	16.8[c]	32.5	114.2	1440	30	3.0	2.8	123.8
7	104.0	114.5[c]	40.0	106.2	249	40	4.0	2.2	101.9
6	99.6	83.1	38.5	106.3	439	50	5.0	2.3	103.1

[a]From correlations by Schmertmann (11).
[b]Based on four U/C tests.
[c]D_r values appear in error.

Table 2. Comparison of calculated and observed bearing capacities.

Test No.	N_γ	N_q	N_c	D_f[a] (ft)	Sand[b] q_f (psf)	Clay[c] q_b (psf)	SK_s[d]	H (ft)	Calculated[e] q_o (psf)	Observed[f] q_o (psf)	Difference (%)
1	75.7	47.6	5.14	1.25	11 230	1238	6.8	2.95	9 575	9 135	4.6
4	32.7	24.6	5.14	2.76	11 070	9224	3.5	4.4	10 232[g]	9 410	8.0
7	109.4	64.2	5.14	2.87	32 183	7820	7.1	5.9	23 104	20 330	12.0
6	85.1	52.4	5.14	3.8	32 942	3095	7.1	7.4	28 074	24 450	12.9

[a]Surcharge thickness x g-level.
[b]Equation 6.
[c]Equation 10.
[d]Figure 4.
[e]$q_{lo} = 1.2cN_c + 2\gamma_s H^2 (1 + 2D_f/H) [SK_s \tan\phi/B] + \gamma_q D_f$ (Equation 5).
[f]From Figure 6.
[g]Equation 5 + Equation 9 with $H_f/B \approx 2$.

to failure was observed. For the tests at highest accelerations, it was necessary to provide a balancing pressure to the air piston to prevent the increased weight of the footing and ram, prematurely loading the model. For these cases, the balancing pressure was adjusted to maintain the LVDT readings at their original null positions. A total of seven tests was performed, of which four were satisfactory

TEST RESULTS

The soil and model characteristics are listed in Table 1. The parameters used for calculating the bearing capacity and the calculated bearing capacities are compared with the observed values from the models in Table 2. Figure 6 presents the stress-deformation relationships for the various models as they were loaded in the centrifuge.

In all but test 4 (30 g), a punching failure occurred. Observations in the clay layer for all tests revealed a heaved section with a diameter of approximately 5B.

A comparison of predicted versus observed results presented in Table 2 shows excellent agreement. The difference was generally less than 20 percent; the best agreement was 4.6 percent for test 1. Considering the sensitivity of the SK_s and the bearing-capacity coefficients to ϕ, the agreement is well within engineering acceptance. For example, a value of $\phi = 37.8°$ instead of 38.5° (test 6) will result in a calculated bearing capacity of 22 794 psf or a difference of -7 percent for this test.

Two test conditions are modeled in this program. Tests 1, 6, and 7 are for a dense sand layer over a clay layer with varying values of cohesion. For these tests, a punching failure was observed. Test 4 represents a loose sand layer of a stiff clay, and no punching failure occurred for this test. The increased levels of acceleration (G) corresponded to an increase in model dimensions; thus prototype footings ranging from 0.6 to 1.5 m were modeled.

CONCLUSIONS

The generally good agreement between the test data and Meyerhof bearing-capacity theory indicates that the testing procedure developed is satisfactory for conducting centrifugal model tests to evaluate bearing capacity of footings.

The results show that, at least for the H/B ratio of 1.5, Meyerhof's bearing-capacity theory for layered soils is very good in spite of uncertainties involved in determination of the parameters used in his equation.

The results show clearly that for the tests at 20, 40, and 50 g, punching failure had occurred. But for 30 g, it seems that a lateral squeezing of the sand had occurred.

Because this study was of a preliminary nature, additional model tests involving different shapes and H/B ratios should be more thoroughly investigated.

ACKNOWLEDGMENT

The assistance of David Bloomquist in performing the centrifugal model tests and preparing this paper is acknowledged and greatly appreciated.

REFERENCES

1. S.J. Button. Bearing Capacity of Footings on a Two-Layer Cohesive Subsoil. Proc., Third International Conference on Soil Mechanics and Foundation Engineering, Zurich, Vol. 1, 1953, pp. 332-335.
2. A.S. Reddy and R.J. Srinivasan. Bearing Capacity of Footings on Layered Soils. Journal of Soil Mechanics and Foundation Division of ASCE, March 1967.
3. C.S. Desai and L. Reese. Ultimate Capacity of

Circular Footings on Layered Soils. Journal of Indian National Society of Soil Mechanics and Foundation Engineering, 1970.
4. J. Brown and G.G. Meyerhof. Experimental Study of Bearing Capacity in Layered Soils. Proc., Seventh International Conference on Soil Mechanics and Foundation Engineering, Mexico City, Vol. 2, 1969, pp. 45-51.
5. G.G. Meyerhof and A.M. Hanna. Ultimate Bearing Capacity of Foundations on Layered Soils Under Inclined Load. Canadian Geotechnical Journal, Vol. 15, 1978.
6. G.G. Meyerhof and A.M. Hanna. Ultimate Bearing Capacity of Foundations on a Three-Layer Soil, with Special Reference to Layered Sand. Cana-
7. P. Purushothamaraj, B.K. Ramiah, and K.N. Venkatakrishna Rao. Bearing Capacity of Strip Footings in Two-Layered Cohesive-Friction Soils. Canadian Geotechnical Journal, Vol. 10, 1973.
8. A.S. Vesić. Bearing Capacity of Shallow Foundations. In Foundation Engineering Handbook, H.F. Winterkorn and H.Y. Fang, eds., Van Nostrand Reinhold, New York, 1975.
9. G.G. Meyerhof. Ultimate Bearing Capacity of Footings on Sand Layer Overlying Clay. Canadian Geotechnical Journal, Vol. 11, May 1974.
10. A. Caquot and L. Kerisel. Traité de Méchanique des Sols. Gauthier-Villars, Paris, 1949.
11. J. Schmertmann. The Measurement of In Situ Shear Strength. State of the Art Presentation to ASCE Specialty Conference on In Situ Measurement of Soil Strength, Raleigh, NC, 1975.
12. Y. Tcheng. Shallow Foundation on a Stratified Soil. Proc., Fourth International Conference on Soil Mechanics and Foundation Engineering, London, Vol. 1, 1957, pp. 449-452.

Publication of this paper sponsored by Committee on Mechanics of Earth Masses and Layered Systems.

Field-Performance Comparison of Two Earthwork Reinforcement Systems

JOSEPH B. HANNON, RAYMOND A. FORSYTH, AND JERRY C. CHANG

A field-performance comparison of two different earthwork reinforcement systems constructed on Interstate 5 near Dunsmuir, California, is presented. Two mechanically stabilized embankment (MSE) walls and one reinforced-earth (RE) wall, all of comparable height, were constructed with the same type of backfill material. The MSE state system uses welded steel bar mats for reinforcement as compared with the flat steel strips of the proprietary RE system. All three walls were extensively instrumented. The steel stresses in the MSE bar mats were uniformly low; the maximum was 4 ksi as compared with the variable stress measured along the RE strips (9.3 ksi maximum). Erection times were comparable for both systems. The stress patterns developed in the MSE reinforcement tend to confirm the premise that MSE can be used with low-quality backfill, which would offer a significant economic advantage.

Construction experience in 1972 by the California Department of Transportation (Caltrans) with the first reinforced-earth (RE) wall in California suggested that alternative systems of soil reinforcement could possibly provide increased pullout resistance.

In 1973, a large direct-shear device was developed at the Transportation Laboratory of Caltrans to measure pullout resistance of various reinforcement systems, including the flat steel strips then used by the Reinforced Earth Company. The results, presented in some detail in 1977 (1), clearly indicated that the mat arrangement increased pullout resistance to an extent far in excess of that possible solely by friction between soil and earth. The failure mechanism observed involved the development of a passive pressure wedge of soil rather than the slippage or tensile breaks observed with the proprietary flat steel strips. This introduced the possibility of the use of a lower-quality backfill material, which, at given locations, could offer significant economic advantages.

During the design of the realignment and widening of I-5 near Dunsmuir, California, earthwork retention structures were found to be economically feasible at two sites. As a result of an agreement with the Reinforced Earth Company, the decision was made to construct a system designated as a mechanically stabilized embankment (MSE) that used bar-mat reinforcement with concrete facing for two walls at one location. At the second location, a proprietary RE wall with concrete facing would be constructed. The Dunsmuir project presented the initial opportunity to fully evaluate a prototype mat reinforcement system, including construction characteristics, response to load, cost, and corrosion resistance. In addition, it would be possible to compare the two systems on installations of approximately the same configuration and environment although somewhat differing foundation conditions. All three installations were instrumented to monitor stresses and deformations of these two soil-reinforcement systems. Instruments also monitored steel loss due to corrosion.

This report summarizes construction, instrumentation, field behavior, and cost data for the RE and MSE systems constructed at Dunsmuir. Detailed information on these aspects plus foundation conditions and design are covered in the final research report on the project (2).

GENERAL DESCRIPTION

The MSE system is a Caltrans development licensed under the Reinforced Earth Company patent in accordance with an agreement dated May 1976.

The two MSE walls on this project are located along the northbound lanes of I-5 below the Siskiyou Elementary School. The site, designated location A, consists of an upper and a lower wall. The upper wall, constructed between May and August 1976, is