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

Field-Performance Comparison of Two Earthwork Reinforcement Systems

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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.

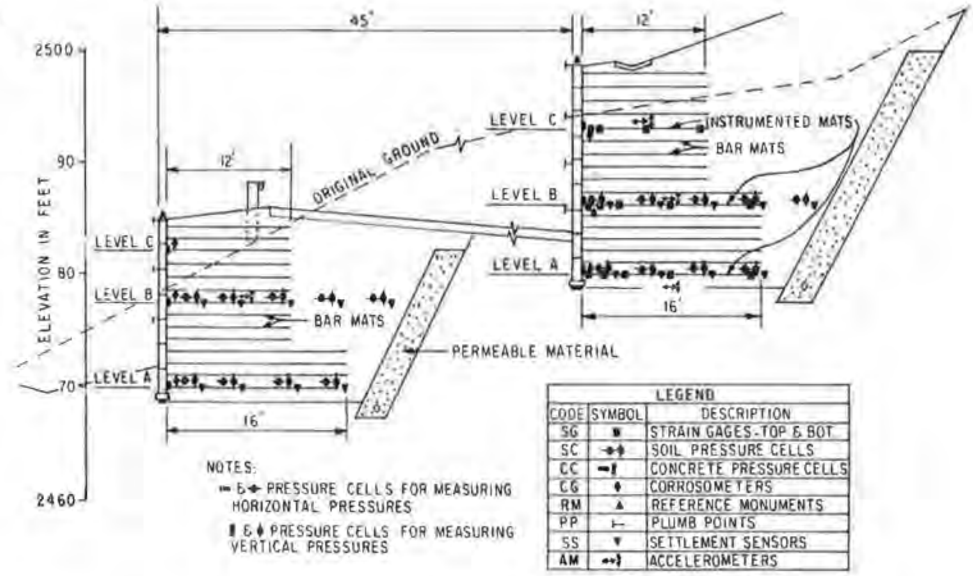
This report summarizes construction, instrumentation, field behavior, and cost data for the RE and MSE systems constructed at Dunsmuir. Detailed information on these aspects plus foundation conditions and design are covered in the final research report on the project (2).

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

Figure 1. Typical section and instrumentation plan for MSE walls at location A.



located above a frontage road and serves as an earth-retaining structure for the slope below the Siskiyou Elementary School.

The upper wall has a maximum height of approximately 20 ft. The lower wall, which was begun in September 1974 and completed in October 1975, is located below the frontage road along the northbound lane of I-5. The maximum height of the lower wall is approximately 18 ft. Figure 1 shows typical sections of the two MSE walls together with the instrumentation plan.

The RE wall is a system of earthwork reinforcement in which internal stability is achieved through friction between flat steel strips and soil backfill. This system was developed in the 1960s by a French engineer, Henry Vidal (3), and is marketed in the United States by the Reinforced Earth Company of Washington, D.C.

The RE wall, located along the southbound lanes of I-5, was designated location B in the contract plans. The distance between locations A and B is approximately 2000 ft. The maximum height of the RE wall is approximately 20 ft. Figure 2 shows a typical section of the RE wall together with the instrumentation plan.

The material for the backfill for both locations A and B consisted of Class 4 (Type 1) aggregate subbase. Specification requirements and test results are shown in Table 1.

MSE WALL

External and Internal Stability

External stability for the MSE walls consisted of analyzing the resistance to both sliding and overturning for static and for earthquake conditions.

Once the requirements for internal stability have been satisfied, the MSE wall is presumed to act as a solid gravity mass; its weight resists the overturning moment produced by the earth pressure behind the gravity mass. Resistance to sliding is provided by adequate embedment depth to mobilize the shear strength of both the foundation and backfill materials. For preliminary analyses, the soil parameters used for the backfill were ϕ=35°, c=360 psf, and γ=130 pcf. Calculations of overturning moment and sliding resistance were made at depths below top of wall of 10 and 20 ft to evaluate the effects of reduced reinforcement lengths as shown in Figure 1.

For the design of both MSE walls, advantage was taken of the gravity section design with reduced bar-mat lengths in the upper portion of the walls. The factors of safety were 2.0 or higher in all cases.

The stability of the foundation was analyzed by using ϕ=20°, c=1100 psf, and γ=120 pcf. The factors of safety exceeded 1.5 for sliding and 2.0 for overturning.

Horizontal earthquake acceleration of 0.2 g was also considered in a pseudostatic stability analysis that produced factors of safety exceeding 1.8. Internal stability was accomplished by longitudinal reinforcement by using sufficient cross-sectional area to preclude tensile failure with sacrificial steel for corrosion loss during the design life of the facility. The transverse members were selected to withstand pullout under an assumed pressure distribution.

Pull resistance was not a controlling factor in design of the bar-mat reinforcement since laboratory tests indicated that the bar mat has more than five times the pullout resistance than a corresponding surface area of flat steel reinforcing strips (1).

Embedment depth of the bar mat was determined from the requirement for external stability.

Corrosion

Before a decision on bar size could be made for the Dunsmuir walls, the metal losses throughout the design life of the structures required estimation so that sacrificial steel could be provided.

In underground metal facilities, such as gasand liquid-carrying pipelines where local pitting and eventual complete perforations can result in failure of the facility, the problem is of great concern. However, in buried load-bearing structures, such as piles or reinforcement, overall weight loss or average penetration was considered more critical than maximum penetration of pitting.

Tests on preliminary native soil samples of proposed backfill materials during the feasibility study produced a pH of 5.4 and resistivity of 17 000 ohm•cm and a pH of 5.8 and a resistivity of 30 000 ohm•cm. These results were determined from California Test 643. With this information as criteria, a rate of metal loss from pitting of 0.0018 in/year (1.8 mils/year) was assumed for preliminary design of the Dunsmuir bar mat based on corrosion of steel culverts. For 50 years' service life, a total depth of metal loss was estimated to be 0.09 in in terms of surface pit development. However, corrosion parameters obtained from tests on progress samples of the actual backfill material during construction (Table 1) were a pH of 5.4 and resistivity of 8700. This equated to a much higher metal loss of 7.2 mils/year in terms of pit development or 0.8 mil/year of uniform surface loss. This rate, although in excess of the original estimated corrosion rate, was compensated for as indicated later. The chart in Figure 3 by Stratfull (4) presents the basic relationship used in the above test method and equates weight loss to surface pitting.

The final bar-mat design for resistance to tensile stress and the effects of corrosion called for 3/8-in-diameter bars at 6-in longitudinal spacing. The mats were 4 ft wide with nine 3/8-in-diameter bars to resist a maximum tensile load of 3.1 kips/mat and the corrosion rate of 0.0018 in/year for a 50-year life as estimated initially. The transverse bars were spaced at 1-ft 10-in centers for the longer mats and 2-ft centers for the shorter mats (Figure 4). The length of the mats was governed by previously described stability considerations. The final mat length was 10 ft for the top five panels and 15 ft for all panels below the top five or in excess of a 10-ft vertical wall height. The vertical spacing between mats was 12 in.

Figure 2. Typical section and instrumentation plan for RE wall at location B.

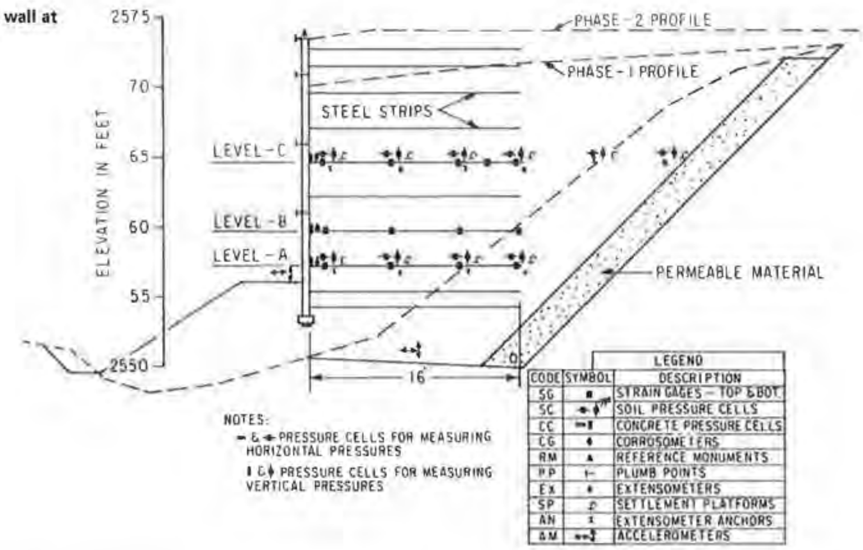


Table 1. Physical soil properties and shear strength of backfill materials for locations A and B.

	Proposed Material						Material Used[a] (74-1628, Sept. 1974)		
	MM 08433, June 1971			73-1803, Dec. 1973					
Property	UU	CU Eff	CU Total	UU	CU Eff	CU Total	UU	CD	Specification
Angle of internal friction ϕ (degrees)	24	33.5	15.5	35	33.0	10	34	36	
Cohesion C (psf)	2160	720	720	1150	400	1600	1000	200	
Plasticity index PI (%)		3			NP		5		
pH									
Resistivity (ohm-cm)		-			-		8700		
Sand equivalent		15			27		36		18 minimum
Maximum dry density (pcf)		116.8			77		94 (121 wet)		
Sieve size (% passing by weight)									
4 in		-			-		100		100
3 in		90			97		100		-
2½ in		86			94		97		-
2 in		83			92		97		-
1½ in		80			90		94		-
1 in		76			86		87		-
3/4 in		73			83		83		-
1/2		70			79		79		-
3/8 in		68			77		77		-
No. 4		65			67		74		30-94
No. 8		59			60		69		-
No. 16		55			52		64		-
No. 30		51			44		60		-
No. 50		47			33		54		-
No. 100		43			23		46		-
No. 200		38			15		39		0-29
5 μ		14			9		16		-
1 μ		6			4		-		-

Note: UU = unconsolidated undrained; CU Eff = consolidated undrained effective; CD = consolidated drained.

[a]Sampled from location B.

Concrete Facing

The connector and concrete facing design are covered in detail in the aforementioned final research report (2). The specific shape of the precast concrete panel that was finally used was the product of discussions with the contractor based on a cost-reduction proposal. The configuration ultimately selected was 2x12.5 ft, which allowed for four mats per panel (Figure 5).

Construction Operation

An unreinforced concrete leveling footing was placed in an excavated trench along the entire wall length at the prescribed foundation elevation to provide a level foundation for alignment control of face panels during erection. Handling and placement of the wall panels were accomplished with a small crane and two laborers. The wall panels were plumbed with wooden wedges, hammer, and level and were set to correct spacing. Placement time for one panel was approximately 5 min. Neoprene sheets 1/8 in thick and 3.5x12 in were placed on each shear key groove of the panels as a bearing pad between subsequent panels. The fill height was then raised to the elevation of the bottom mat connector holes in the panels.

Mats were then placed on the fill (Figure 6), and their threaded rods were inserted through the lower holes in the panels. Two washers and a nut were fastened to the end of each threaded rod, and a waterproofing sealant was pumped through the holes in panels between the washer and the outside wall face. The mats were then pulled back from the wall until the nut on the end forced the washers to rest tightly against the countersunk holes on the outside wall face. Backfill material was placed over the mats and compacted. When the backfill thickness was approximately 12 in over the mats, the nuts on the end of the threaded rods were tightened. The countersunk holes on the outside wall face were filled flush by using a cement grout mixture.

The maximum length of a row of panels was 469 ft and the minimum length was 37 ft. The average time required to place one panel height with its accompanying mats, backfill, and permeable drainage blanket behind the backfill was approximately 45 min.

A total of 11 working days was required to construct the lower wall, which was 455 ft in length and 18 ft in maximum height. The upper wall (406 ft in length and 20 ft in maximum height) was erected

Figure 3. Chart for estimating corrosion rate of steel buried in soil.

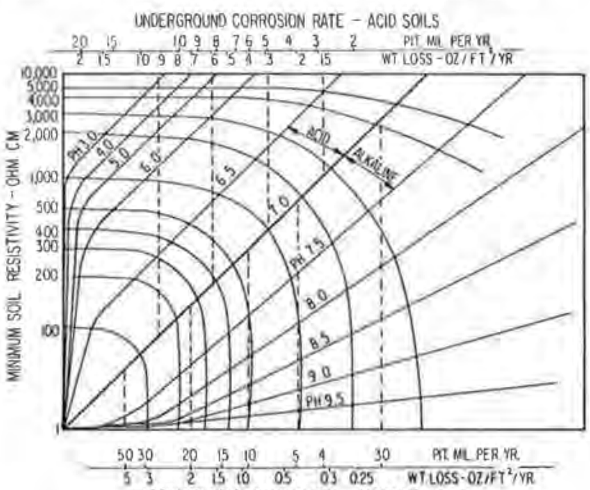


Figure 4. MSE bar-mat configuration and attachment to concrete facing.

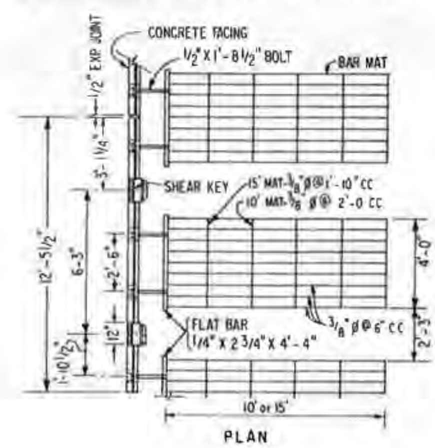


Figure 5. Isometric view, cross section, and typical stacking arrangement for MSE concrete facing panels.

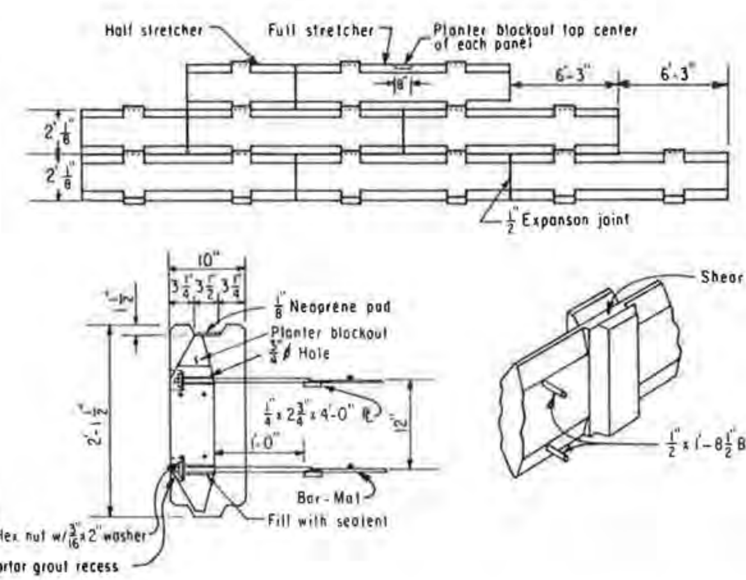


Figure 6. Steel bar mats for MSE in position on fill.

Figure 7. Completed MSE walls.

in 12 days. The completed installation is shown in Figure 7.

RE WALL

External and Internal Stability

External stability was analyzed by using triaxial strength properties for the foundation soils of ϕ=24°, c=200 psf, and γ=136 pcf. Factors of safety for the foundation soil exceeded 4.2. The gravity mass of the wall was designed with factors of safety that exceeded 1.5 for sliding and 2.0 for overturning.

Design of reinforcement for internal stability was accomplished by the Reinforced Earth Company. Smooth galvanized steel reinforcing strips 3 mm thick by 80 mm wide were used at a uniform 16-ft embedment depth. Lateral and vertical strip spacing was 3 and 2.5 ft, respectively.

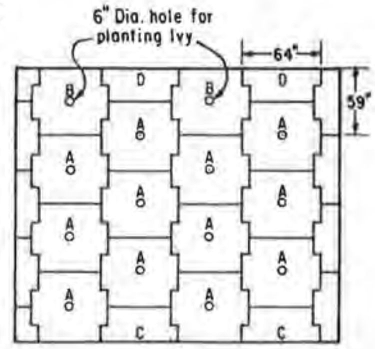
Concrete Facing Panels

The precast concrete facing panels for the reinforced earth wall were as shown in Figure 8. They were approximately 5x5 ft in greatest dimensions and 7.5 in thick. Four types of panel elements were used.

Construction Operation

A nonreinforced concrete leveling footing was first placed along the entire wall location. Once the concrete footing had cured a minimum of one day, panel placement began; full and half-panels were alternated (Figure 8). Panel placement was accom-

Figure 8. Typical layout for RE facing panels.



plished by using a truck-mounted boom, hoist, and sling. As the panels were lowered into position, correction was made for horizontal distance between panels and their vertical alignment by using a spacer bar, crowbar, wedges, and level. Placement time for each panel was approximately 5 min. Once the backfill elevation reached the top of the half-panels, another set of full panels was introduced and held in position with clamps. Each clamp remained in place until placement of the next elevation of panels. As the fill elevation reached each level of tie strips on the wall panels, reinforcing strips were attached (Figure 9). Backfill placement was then continued to the next level of tie strips.

A 4-ft length of 1x4-in cork board was placed on the top edge of each panel without glue or adhesive. A 9-ft strip of 2x2-in Polyether foam, Grade 1035, was pushed into the vertical joints on the fill side by using a blunt-ended instrument (see Figure 9). The Polyether foam prevents fine particles in the backfill from washing through the joints but allows water to drain freely.

Backfill and permeable materials were transported to the wall site in bottom dumps and compacted with a 12-ton vibratory steel drum roller. Continuous grading and leveling by using a motor grader complimented the compaction operation. A water truck provided additional moisture. Laborers shoveled material larger than 2 in away from the wall face, and compaction along the edge of the wall was achieved by a small hand-operated vibratory steel roller. The completed wall is shown in Figure 10.

INSTRUMENTATION

Instrumentation for both reinforcement systems included (a) weldable strain gages with half-bridge circuits to determine the stresses developed in the bar mats and steel strips, (b) Magna corrosometer probes to measure corrosion rate of steel in a soil environment, (c) concrete pressure cells to measure horizontal soil pressures developed behind the concrete facing panels, (d) hydraulic and pneumatic soil pressure cells to measure soil stresses, (e) settlement platforms and mercury-pneumatic-type settlement sensors to measure vertical settlement within the backfill, (f) Statham accelerometers to measure dynamic response to steady-state vibration in a later testing program, (g) reference monuments to measure horizontal and vertical movement at the top and at the base of the walls, and (h) plumb points to measure horizontal and vertical movement at the face of the walls.

All instruments were read periodically during construction and after completion of construction through June 1978. Strain gages on the bar mats of the MSE walls were monitored through December 1980.

The final research report (2) presents the data

Figure 9. RE face panels held in position by wooden clamps with reinforcing strips attached.

Figure 10. Completed RE wall.

collected at both sites in detail. Only the more significant portions will be summarized here. Also, for purposes of comparing the responses of the two systems, only the data developed on the MSE upper wall will be presented, since they represent a loading condition similar to that of the RE wall.

Soil Pressure Against Concrete Wall Faces

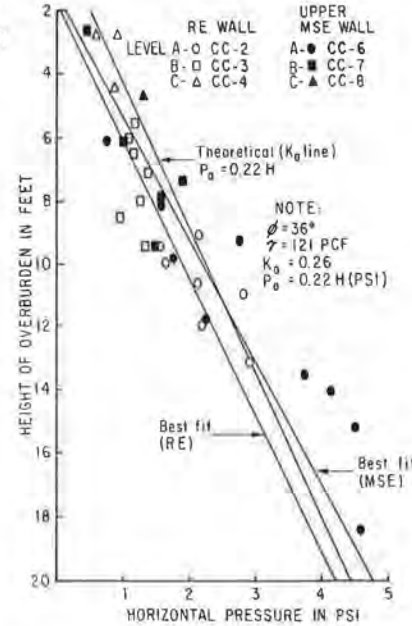
The pressures at the wall face at both sites were measured by Carlson soil stress meters (concrete cells) at three levels. A plot of horizontal pressure versus height of overburden during construction can be seen in Figure 11. Here a best-fit line through the measured data points results in a maximum horizontal pressure at 20-ft overburden of about 4.2 psi for the RE wall, which is comparable with the pressure measured with the same overburden for the upper MSE wall.

The pressure values appear relatively consistent and conform to an active pressure state.

Reinforcement Stresses

To provide the basis of comparison, reinforcing steel stress levels from the RE wall and the MSE upper wall for the three instrumentation levels are superimposed on Figure 12. As shown, stresses in the MSE bar mats (maximum average of four gage loca-

Figure 11. Horizontal soil pressure on wall face during construction of MSE and RE walls.



tions) are uniformly low; the maximum is 4 ksi as compared with the variable stress measured on the RE strips (9.3 ksi maximum) at individual locations. Compressive stress was also measured on the RE strips near the face at level A, possibly a result of the toe buttress. The maximum connection-bolt stress for the bar mat was 6.65 ksi at level A following construction. However, this stress relaxed and stabilized at 4.5 ksi, which was similar to the maximum bar-mat stress at level A. The other MSE connection-bolt stresses at levels B and C also stabilized with a value similar to the bar-mat maximum stresses or went into compression. The overall long-term trend is a general relaxation of stress in the MSE system. The RE system showed minimal relaxation with time.

The stresses in the bar mats were also much more uniform throughout the depth of horizontal embedment than in the RE steel strips, possibly due to greater stiffness of the system. This characteristic will permit reduced steel quantities on future designs.

Pull Resistance of Reinforcement

The primary difference between the two systems constructed on this project was in the nature of reinforcement and, subsequently, pullout resistance and failure mode. Laboratory pullout resistance of the two types of reinforcement when Dunsmuir backfill material was used was measured in a series of tests with a large direct-shear device (1).

Test data pertinent to this report are presented in Figure 13, which shows load-deformation curves for a 3/8-in-diameter bar mat compared with those for three flat smooth steel strips of the same width and thickness used in the RE wall. Both reinforcement systems had the same area of steel exposed to soil for development of pullout resistance. Since tensile failure of the reinforcements did not occur, cross-sectional area was not critical.

The peak pullout resistance of the bar mat was 5.6 times that of the smooth steel strips with equivalent surface area (longitudinal plus transverse bars). These data and the results of tests on silty clay backfill presented in the aforementioned report clearly indicate that a lower-quality cohesive soil backfill would have been suitable for MSE walls.

The Reinforced Earth Company now uses a ribbed strip that provides some increase in pullout resistance. The ribbed strip was not tested as part of this study.

Corrosion Rate

As mentioned previously, corrosometer probes (CG) were installed behind the facing of the MSE and RE walls (Figures 1 and 2) to monitor and predict the uniform rates of corrosion and compare them with the rate estimated for design. The discussion below will cover both the upper and lower MSE walls as well as the RE wall.

In the lower MSE wall, Figure 14 shows a corrosion rate that varied with depth. CG 1 in level A corroded at a projected uniform rate of 3.4 mils/year, CG 2 in level B corroded at 1.5 mils/year, and CG 3 in level C corroded at 0.3 mil/year. The average rate for these three sensing probes is equal to 1.7 mils/year.

The upper MSE wall contained two corrosometer probes in each of its three levels (Figure 14). In level C, the two probes indicate a projected corrosion rate of 0.4 mil/year. This corresponds well with the 0.3 mil/year computed for level C of the lower wall. However, the pairs of probes in levels A and B of the upper wall gave projected corrosion rates that did not correlate well. Level B indicated a corrosion rate of 3.1 mils/year for CG 5 and 1.1 mils/year for CG 8. Level A resulted in 2.9 mils/year for CG 4 and 0.7 mil/year for CG 7. It should be noted that a different type of probe was used for corrosometers CG 4, 5, and 6 than for the other six sensing probes. Most of the sensors used a 4-mil corrodible element, whereas CG 4, 5, and 6 contained a 1-mil element. The results from the 1-mil elements are somewhat questionable due to their high sensitivity and short life. They may not, therefore, realistically reflect the long-term corrosion effects.

When all data are included, the average uniform corrosion rates in mils per year for each pair of probes were 1.8, 2.1, and 0.45 for levels A, B, and C, respectively. The average uniform corrosion rate for the upper wall is 1.5 mils/year.

If one considers the average uniform rates of 1.7 and 1.5 mils/year from the corrosometer probes in the lower and upper walls, respectively, projected uniform metal losses of 0.86 oz/ (ft^2•year) and 0.78 oz/(ft^2•year) are predicted. These values can be compared with the predicted corrosion loss in terms of pitting of 7 mils/year for the actual backfill sample. From corrosion criteria (Figure 3), this equates to a loss of 0.5 oz/ (ft^2•year) (pH of 5.4 and resistivity of 8700) for the composite

Figure 12. Comparison of steel stresses in earth reinforcement (RE and MSE walls).

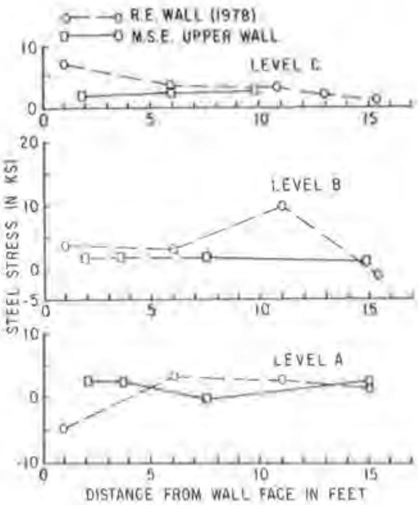


Figure 13. Laboratory pullout tests on steel strips and bar mat.

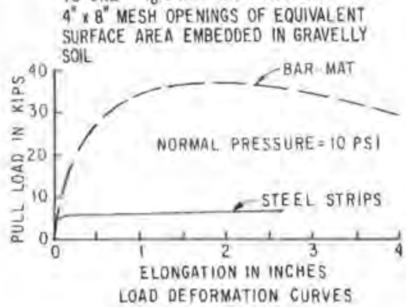


Figure 14. Estimated and actual corrosion rates, location A (MSE).

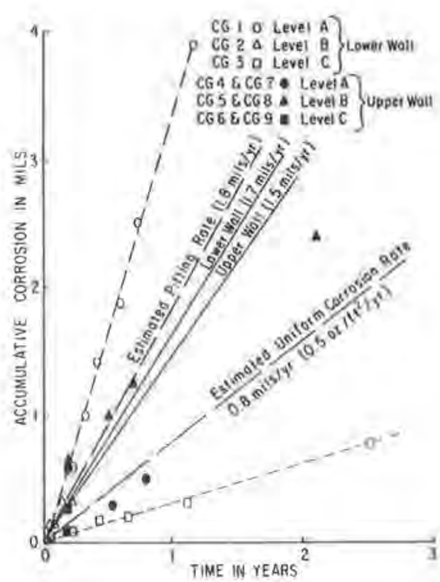
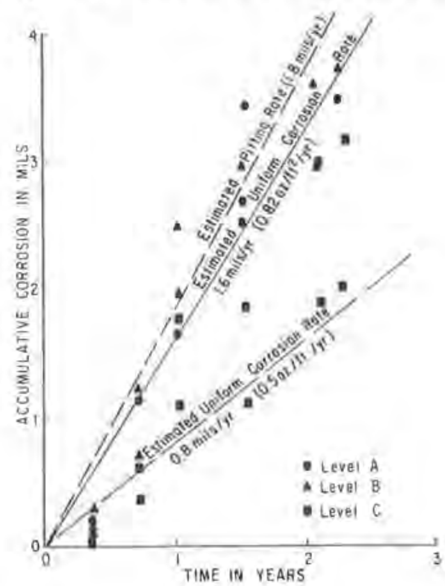


Figure 15. Estimated and actual corrosion rates, location B (RE).



progress sample of backfill material. This is approximately 60 percent of the actual rates measured by the corrosometer probes.

Even though the above rates of metal loss are greater than the original estimated value, the 0.375-in-diameter bar mat is more than adequate for the actual field stress conditions. This bar mat will also provide sufficient sacrificial steel for the average measured uniform rate of 1.7 mils/year.

The corrosometer data also appear to indicate that corrosion rates increase with depth. This may be due to the phenomenon of corrosion stress; i.e., under greater pressures, the corrosion rate is increased. There may also be more moisture retained at the lower levels, which would tend to accelerate corrosion.

In April 1980, approximately 46 months after the completion of level C of the upper MSE wall, a segment of the bar mat at this level was retrieved for further corrosion analyses.

The bar-mat segment was covered with a light layer of rust. As stated previously, the average corrosion rate at this level of the embankment was 0.4 mil/year, which is about one-fourth of the entire MSE average. The analysis was limited to level C since no samples could be obtained from lower levels.

Various specimens were cut from the retrieved bar-mat segment for tests to measure weld shear resistance and tensile strength of the longitudinal reinforcement.

The results indicated that strength of the steel was virtually unchanged. Also, the welds' minimum shearing strength was 1800 lb, which is well above the approximate 70-lb design requirement (assuming a uniform distribution of stress throughout the bar mat). A microscopic examination of the light rust on the welds showed that it had not penetrated or weakened the welds or the mat.

The rate of corrosion at location B (RE) was measured in the fill by means of two corrosometers per level in levels A, B, and C. Since the same backfill material as that for the MSE walls would be used, a 1.8-mil/year rate of corrosion in terms of pitting was predicted initially from laboratory tests for pH and resistivity as defined previously.

Measured and estimated corrosion rates are plotted in Figure 15 for levels A, B, and C. Although the corrosion rates vary somewhat from those measured in the MSE walls, the average for all the gages is very nearly the same.

The average uniform corrosion rates at each level range from 2.0 to 1.18 mils/year for levels A, B, and C, respectively. The average of these rates is 1.6 mils/year [0.82-oz/(ft^2·year) uniform rate], which is somewhat higher than the pitting rate of 7 mils/year [0.5 oz/(ft^2·year)] estimated for the actual backfill samples.

CONSTRUCTION COST

A valid comparison of construction cost for MSE and RE walls on this project based on contract bid items is not possible due to the vagaries of the bidding process, construction timing (location A was constructed after location B), and the fact that location A was the first MSE project whereas the construction industry was generally familiar with RE projects. As an example, steel for the RE wall was bid at \$0.53/lb as compared with \$1.00/lb for the MSE walls.

If we take these factors into consideration, an appropriate means of developing a cost comparison of the two systems is believed to be quantities of principal items and erection time. Excavation quantity was not considered, since this was primarily dictated by site geometry rather than by system type. Also not included were items associated with the subsurface drainage since the same system was used for both RE and MSE walls.

Actual costs for wall facing for the two systems were closely comparable (\$4.60/ft^2 for the RE wall versus \$4.50/ft^2 for the MSE walls). To normalize the data for comparison, quantities are presented below by amount per 100 ft^2 of wall face; phase 1 of the RE construction involved 10 563 ft^2 and the lower MSE wall construction involved 5825 ft^2:

Item	System RE	System MSE
Select backfill/100 ft^2	80.5	89.2
Steel for reinforcement (lb/100 ft^2)	490.5	946.2
Erection time (days/100 ft^2)	0.27	0.19

These data indicate that reinforcement steel quantities for MSE walls per unit area of wall face were approximately double that of the RE wall due to the presence of the transverse members. In terms of erection time, the MSE wall was found to offer a slight advantage.

In assessing the relative costs of the two systems used on the project, two key factors should be borne in mind with respect to the MSE installation. First, a subbase-quality backfill was used on both systems for the purpose of comparing response to load. The results of laboratory pullout tests referred to earlier indicated that a low-plasticity ($PI \leq 10$) nonexpansive native material would have been satisfactory for the MSE installation. Also, the extremely low and relatively uniform steel stresses measured in the MSE installation indicate that No. 2 rebar (W5 wire) rather than No. 3 rebar (W11 wire) would have provided adequate tensile strength and a corrosion life equivalent to that of the RE strips. The use of No. 2 bars (W5 wire) would have resulted in a net reduction in steel quantity of 55 percent for the MSE project.

CONCLUSIONS

1. Horizontal pressure on the upper wall at location A (MSE system) and the RE wall at location B approximated the active Rankine state, which verified design theory.
2. The stresses in the MSE bar mats suggest a redistribution of stress in the reinforced soil mass and a uniformly low steel stress level at a maximum of 4 ksi as compared with the variable stress measured on the RE strips (9.3 ksi maximum). This is possibly due to the greater stiffness of the MSE system.
3. The extremely low and relatively uniform stresses measured in the bar-mat steel for the MSE system suggest that a 0.25-in-diameter rather than a 0.375-in-diameter bar would have been adequate for tensile strength and a corrosion life equivalent to that of the RE system.
4. Laboratory pullout tests suggest that MSE bar mats provide a pullout resistance that is greater than five times that of the smooth RE strips of the same surface area. These results also suggest that lower-quality backfill materials would be suitable for the MSE system.
5. Corrosion rates determined by corrosometer probes buried in the backfill soil were somewhat questionable and provided a poor correlation to predicted soil corrosion rate based on Caltrans steel-culvert criteria.
6. Both the MSE and the RE systems performed satisfactorily.
7. The MSE system will require more steel due to the presence of transverse members but can apparent-

ly be erected more quickly than the RE system can.

8. Where quality backfill would be a major cost item, the mesh (bar-mat) reinforcement could offer a significant economic advantage.

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

Design and Construction of Fabric-Reinforced Retaining Walls by New York State

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The experience of New York State in the design and construction of two fabric-reinforced retaining walls is described. Crushed-stone fill is reinforced by horizontal layers of fabric placed at intervals dependent on the height of the wall, the strength of the fabric, and the internal friction angle of the fill. Design and construction procedures are detailed; the emphasis is on practical construction techniques. The design and construction are based on methods described by the U.S. Forest Service. The construction technique, although not commonplace, can be quickly mastered without special equipment or labor requirements. Instrumentation installed during construction to monitor vertical and horizontal movements indicates satisfactory performance 18 months after completion. The cost of this type of construction at this site compared favorably with alternative designs. Suggestions for cost reductions are offered for future installations, which may include embankment repair, similar to this project, or temporary works, such as construction detours.

New York State Department of Transportation (NYSDOT) has designed two retaining walls in which geotextile fabric is used as a reinforcing material. The walls, completed in August 1980, repair shallow failures in a side-hill embankment of NY-22 in Columbia County. The failures were observed in early 1976, when the easterly shoulder settled several inches in two areas 125 ft apart. The areas, designated A and B, extend for 110 ft and 150 ft, respectively.

A monitoring program was established in the fall of 1976 to measure any vertical or horizontal movements of the pavement, shoulder, and embankment slope. A subsurface investigation program was initiated in 1977; it consisted of a number of cased drill holes, one of which was converted into a long-term observation hole. The subsurface profile (Figure 1), determined from visual identification of soil samples and analysis of boring logs, shows 5-10 ft of loose clayey silt, sandy with gravel, overlying similar compact material; ledge rock is encountered at depths varying from 15 to 25 ft.

Movements up to 0.2 ft horizontally and 1.2 ft vertically, detected from September 1976 to April 1978, and distortion of observation hole casing at depths of 4-5 ft (see Figure 1) indicated a shallow failure in the loose material rather than a sliding failure along the rock surface. Subsurface water, due to side-hill seepage at area B and a broken box culvert at area A, was the primary cause for failure of the 1 vertical on 1.5 horizontal embankment slopes. This assumption was supported by the existence of 125 ft of unaffected 1 on 1.5 slope between the two failure areas and the natural flattening of slopes in the wet areas to 1 on 2.5.

The extent of the failures would have required remedial treatment beyond the capability of maintenance forces. Consequently, NYSDOT considered a number of design solutions. The criteria for an acceptable treatment included positive stabilization of the failure areas with low future maintenance, additional shoulder width, and safe traffic control during construction. Traffic restrictions by construction equipment and/or cost considerations eliminated a pile-and-lagging wall and extensive slope treatments. The earth-fabric wall concept was selected as the lowest-cost solution that met the criteria.

DESIGN

The design is based on methods described in a U.S. Forest Service publication (1, Chapter 5). The wall is designed as lifts of alternating fabric reinforcement and stone fill, as shown in Figures 2 and 3. The fabric within the theoretical failure zone cannot mobilize tensile strength to resist internal failure and is therefore discounted when the reinforcing length required is calculated. The fabric length embedded behind the theoretical failure plane is the fabric that reinforces the fill. The lift thickness is formed by the fabric, which overlaps the face of the wall and retains the fill material.

Site conditions controlled the length and the height of each wall necessary to stabilize the failure areas. The cross-section dimensions were determined to satisfy the internal and external stability of the wall. The minimum dimensions for internal stability were calculated with the strength parameters of the fabric and fill material by using appropriate factors of safety. These dimensions were increased to adequately resist lateral pressures acting on the wall, computed according to Rankine theory and the Boussinesq equation.

The reinforcing selected was Bidim C-34, a nonwoven, needle-punched, continuous-filament polyester fabric with high strength and permeability. The design tensile strength of 75 lb/in width of material represents approximately one-third the grab-test value (ASTM D-1117-69) reported by the manufacturer, a ratio that agrees with the Oregon State University ring-test results for other Bidim weights as report-