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Performance of an Earthwork Reinforcement System Constructed with Low-Quality Backfill

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ABSTRACT

A preliminary evaluation is presented of a retaining-wall system constructed on Interstate 80 at Baxter, California, using low-quality backfill materials. Four mechanically stabilized embankments were constructed. Two of these walls were instrumented to monitor performance over a 3-year period. Dummy bar-mats of different configurations were installed during construction and were subject to field pullout testing. These field pullout results are compared with laboratory tests conducted with the same backfill material at representative overburden and field moisture and density conditions. Field pullout test results are also compared with laboratory tests conducted before the project design. The results of these tests suggest that the laboratory pullout test values provide a conservative design. The transverse bars governed the pullout capacity of the bar-mat and the overburden stress did not significantly affect the pullout capacity in cohesive backfill. The contractor's method of construction significantly influenced the overall response of the soil-reinforced wall system. The satisfactory performance of this wall system after one severe winter season suggests that mat- (or mesh-) type soil reinforcement systems can be constructed successfully by using low-quality on-site materials as backfill.

The evolution of the mechanically stabilized embankment (MSE) has been described in some detail in the literature (1,2). Results of large-scale laboratory pullout tests by the California Department of Transportation (Caltrans) and others (3) have convincingly demonstrated the greatly increased pullout resistance of mesh-type reinforcement in that less steel is exposed to soil as compared with the case of flat reinforcing strips. Recognition of this may have been a factor in the introduction of ribbed reinforcing strips by the Reinforced Earth Company in 1978 (4). These early tests revealed not only a different failure mechanism as compared with the flat reinforcing strips but also extremely high pullout resistance in poor-quality backfill. Pullout resistance in a silty clay (Table 1) was found to compare favorably with that obtained in gravelly sand when strain criteria were used. These data confirmed the original premise that the use of mesh reinforcement could result in significant savings when quality backfill was not readily available.

TABLE 1 Comparison of Pull Resistance for Bar Mesh Embedded in Gravelly Sand and Silty Clay Soils *(1)*

Soil Type	Mesh Opening[a] (in.)	Confining Pressure (psi)	Yielding Load (kips)	Peak Load (kips)	Residual Load (kips)
Gravelly sand	4 x 8	10	17.3	37.5	25.7
Gravelly sand	4 x 8	20	20	44	35.8
Gravelly sand	4 x 8	25	20	60	43
Silty clay	4 x 8	10	19	39.5	33.5
Silty clay	4 x 8	20	21	55.5	37.5
Silty clay	4 x 8	40	24	66	59
Silty clay	5 x 14	10	10	29	27
Silty clay	5 x 14	20	11	30	30
Silty clay	5 x 14	40	12	41	41

[a]No. 3 reinforcing bars.

For the first MSEs, constructed on I-5 near Dunsmuir, California, in 1974, high-quality backfill was used so that a direct comparison in performance could be made with a Reinforced Earth (RE) wall of approximately equal height on the same project. The results, based on extensive instrumentation of both systems, were presented in some detail in 1982 (5). Although there was a great deal of interest in constructing a prototype MSE with marginal-quality backfill, either subsequent MSE projects were in areas where high-quality backfill was readily available or the nature of the installation was unsuitable for a long-term evaluation.

At the request of Caltrans District 3, a feasibility study for the construction of four MSEs near Baxter, California, was initiated in April 1979.

These walls, varying in length from 183 to 490 ft, were to provide 0.5 mile of additional lane on eastbound I-80 for the purpose of installing chains on vehicles before they entered the heavy snow of the Trans Sierra Highway (Figure 1). The presence of Canyon Creek adjacent to the highway at this location precluded construction of embankments for the full length of the widening. A total of 1,375 lineal ft of wall at a maximum height of 16 ft was necessary for the additional paved width.

Foundation exploration consisted of three rotary sample borings, twelve 2.25-in. cone penetration tests, and eleven 1-in. soil borings. Foundation material consisted of a compacted granular embankment underlain by a stream-deposited alluvium and between stations 383± and 388 colluvial silt and clays. Within these limits groundwater was encountered at a depth of 4 ft. The results of laboratory classification tests on samples considered representative for preliminary design purposes are shown in Table 2. The results of the feasibility study indicated that MSEs could be constructed at the site by using native material for backfill.

The project was delayed for approximately 2 years because of limited funding and higher-priority work. In April 1981 approval was given to begin detailed design. This was accomplished in a cooperative effort by the Transportation Laboratory and design personnel from District 3 (Marysville) and District 9 (Bishop). In addition to the MSE systems, alternative designs for reinforced-concrete crib walls and RE walls were included in the bidding package.

INTERNAL AND EXTERNAL WALL STABILITY

External stability analysis for the MSEs at Baxter consisted of checking both the resistance to sliding and overturning moments for the static condition. Once internal stability requirements are satisfied, the MSE is assumed to act as a solid gravity mass with its weight resisting the overturning moment. The overturning moment is due to the earth pressure behind the gravity mass. Resistance to sliding is provided by adequate horizontal embedment depth to mobilize the shear strength of the backfill material. Minimum factors of safety of 1.5 for sliding and 2.0 for overturning were required.

The site is located approximately 35 miles east of the historically active Stampede Valley fault. The maximum credible bedrock acceleration is estimated at 0.1 g as a result of activity on this fault. Because of the conservative design approach applied for the saturated backfill condition as defined in the following, the seismic load effect on stability was discounted in the final design. No potential exists for liquefaction.

Initial laboratory testing of on-site embankment material performed as part of the feasibility study for this project in 1979 provided the information shown in Table 2. The existing embankment material was determined to be an SM (sandy silt) by the Unified Soil Classification System. Because up to 48 percent of the material passed the No. 200 sieve, it was considered inappropriate for conventional RE backfill construction. The backfill specifications on Caltrans contracts for the patented RE alternative allows up to 25 percent passing the No. 200 sieve. The on-site materials of this project are not free draining and are subject to considerable strength loss when saturated. This potential strength loss is suggested by comparison of the undrained strength (CUtotal) with the effective or drained strength (CUeff) (Tables 3 and 4). A subsurface drainage system was required for positive drainage and long-term wall performance.

For the initial feasibility study, an angle of internal friction (ϕ) of 20 degrees and a cohesion of 500 psf were assumed at partial saturation. The minimum factors of safety for sliding (1.5) and overturning (2.0) could be satisfied with a bar-mat embedment length providing a 12-ft base for the maximum wall height of 16 ft. However, additional triaxial testing of on-site materials under saturated conditions suggested that a more conservative wall design should be used in the event that the proposed drainage blanket should malfunction, allowing saturation of the low-quality backfill. An angle of internal friction of 10 degrees and a cohesion of 800 psf were assumed for the final design but to be more conservative, the cohesion value was neglected when a Rankine triangular active pressure distribution was applied behind the wall face.

This conservative design required that the wall base width be increased by 2 ft to provide the minimum requirements for external stability. In finalizing this design, large-scale preliminary laboratory pullout tests were performed at 90 percent relative compaction and with variable moisture contents by using on-site materials from the existing embankment (proposed for backfill).

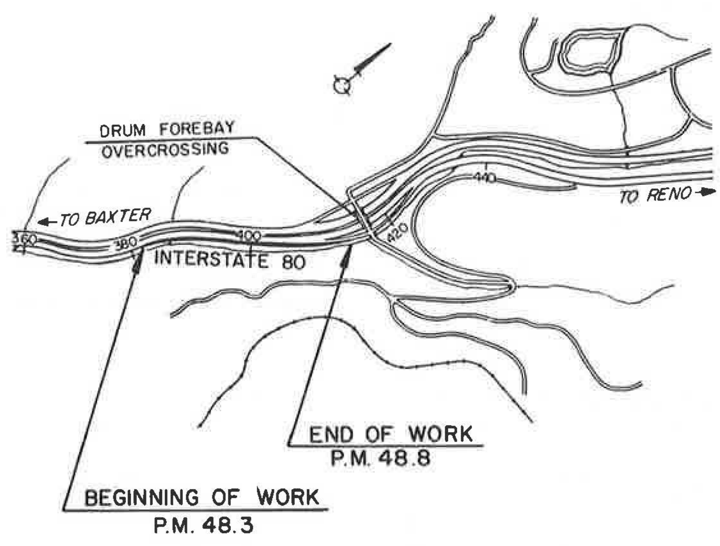

FIGURE 1 Vicinity map.

TABLE 2 Physical Soil Properties of Backfill Materials for Location 1

	Initial Test on Proposed Backfill (Embankment Material)		Progress Sample from Location 1 (Wall 1)		
Property	Sample 79-1190	Sample 79-1191	Sample 82-1208	Sample 82-1232	Specifications
Plasticity index PI (%)	8	11	7	9	10 max.
pH	5.9	5.5	–	–	–
Resistivity (ohm-cm)	14,500	7,300	–	–	–
Sand equivalent	23	13	19	13	–
Maximum wet density (pcf)	115	115	114	114	–
Sieve size (% passing by weight)					
6 in.	–	–	–	–	100
3 in.	100	–	100	99	–
2½ in.	92	100	99	97	–
2 in.	84	98	99	94	–
1½ in.	78	97	98	93	–
1 in.	75	95	95	90	–
3/4 in.	74	94	93	86	–
1/2 in.	72	93	91	82	–
3/8 in.	70	92	90	80	–
No. 4	67	90	87	77	–
No. 8	62	85	75	71	–
No. 16	56	78	65	64	–
No. 30	50	72	57	57	–
No. 50	44	64	51	49	–
No. 100	37	55	44	42	–
No. 200	32	48	38	36	–
5 μ	14	21	24	18	–
1 μ	5	7	9	10	–
Unified Soil Classification	SM	SM	SM	SM	–

TABLE 3 Shear Strength of Backfill Materials: Location 1

	Initial Test on Proposed Backfill (Embankment Material)						Progress Sample from Location 1 (Wall 1)					
	Sample 79-1190			Sample 79-1191			Sample 82-1208			Sample 82-1232		
Parameter	UU	CUeff	CUtotal	UU	CUeff	CUtotal	UU[a]	CUeff	CUtotal	UU[a]	CUeff	CUtotal
Angle of internal friction ϕ (degrees)	31	32	20	31	32	6	28	32	20	33	16	5
Cohesion c (psf)	1,600	500	700	1,200	300	900	2,700	300	600	700	700	1,000

Note: UU = unconsolidated undrained; CUeff = consolidated undrained effective. All strength tests performed on remolded specimens: 82-1208 sampled from backfill at station 383+60 9 ft below finished grade, 82-1232 sampled from backfill at station 383+60 5 ft below finished grade.
[a]Staged test.

TABLE 4 Shear Strength of Backfill Materials: Location 2

	Progress Sample from Location 2 (Wall 3)					
	Sample 82-1256			Sample 82-1261		
Parameter	UU[a]	CUeff[a]	CUtotal[a]	UU[a]	CUeff	CUtotal
Angle of internal friction ϕ (degrees)	35	31	19	13	33	18
Cohesion c (psi)	1,100	500	1,200	1,200	360	1,400

Note: UU = unconsolidated undrained; CUeff = consolidated undrained effective. All strength tests performed on remolded specimens; 82-1256 sampled from backfill at station 399+30 10 ft below finished grade, 82-1261 sampled from backfill at station 399+30 5 ft below finished grade.
[a]Staged test.

The proposed embedment length for the reinforcement was verified by the foregoing preliminary laboratory pullout tests by varying the saturation levels of the test backfill under overburden pressures equivalent to 5, 10, 15, and 20 ft of embankment. Based on these tests, a maximum pullout value of 4 kips was assumed at 1-in. lateral movement for the laboratory test mat 2 ft wide by 4 ft long (8-ft^2 vertical projected area) using three transverse and five longitudinal W7 bars on 6 x 24-in. grid and 10 ft of overburden. Because the bar-mats proposed for the construction (Figure 2) would be placed on 2-ft vertical spacings (Figure 3) and cover an effective area 5 ft wide by 12 ft long (60 ft^2), the available pullout resistance was conservatively estimated at (60/8) x 4 kips or 30 kips per mat for 16 ft of overburden. The mats for the actual design would provide a factor of safety for internal stability exceeding 2.0 for pullout.

It is the opinion of the authors that when sufficient steel reinforcement is provided within the reinforced soil block and external stability is satisfied, potential failure planes will be forced beyond the reinforced soil block. Information collected and now under study on MSE systems constructed by Caltrans with quality backfill has verified this. For the design of the MSE walls on this project where low-quality backfill was proposed, it was assumed that this condition also existed.

CORROSION

Table 2 presents the preliminary test results for soil pH and resistivity. Minimum values for 5.5 for pH and 7,300 ohm-cm were selected for determining corrosion loss of the buried bar-mats. Additional soil samples were also obtained along the existing shoulder before construction to reflect concentrations of deicing salts and the worst possible corrosion conditions. The resulting test values were less critical than the previous values for pH and resistivity. A uniform rate of surface corrosion of 0.5 oz/(ft^2 • yr) was therefore estimated for

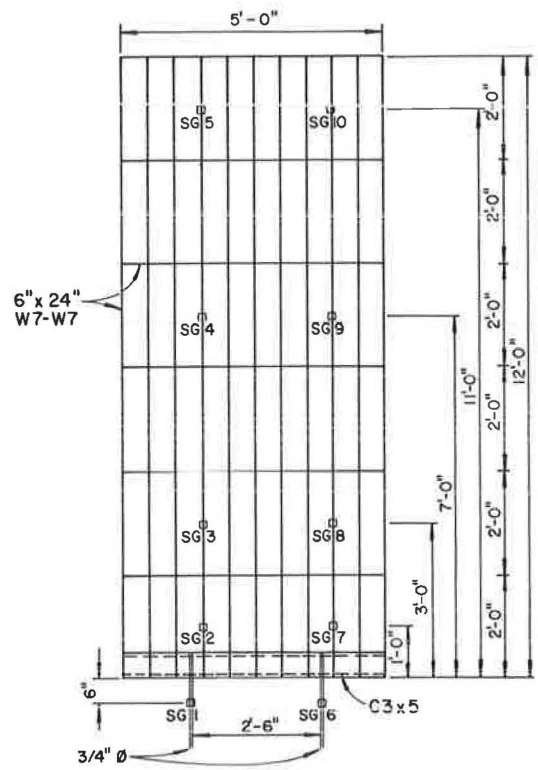

FIGURE 2 Instrumented bar-mat with strain gauge locations.

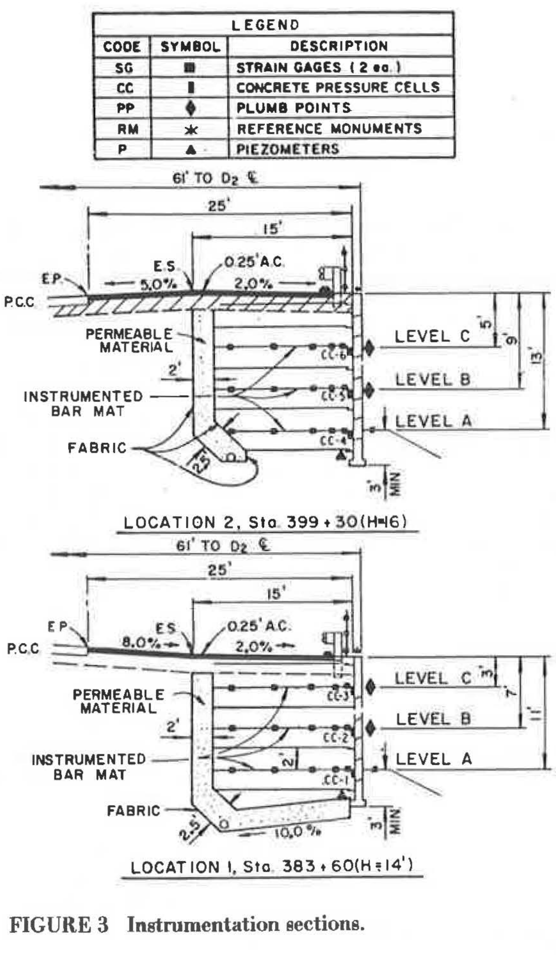

FIGURE 3 Instrumentation sections.

design by using the previous values (6). Accordingly, the W7 bar size was selected as adequate for the 50-year design life of the facility based on working stresses. Criteria now used by Caltrans are somewhat more conservative.

Progress tests for pH and resistivity made during actual construction indicated soil resistivity as low as 1,400 ohm-cm with a pH of 5.1, which is a more critical corrosion condition than originally estimated. A series of steel rods (W7 bars) were placed in the wall backfill at various levels during construction to monitor corrosion rate. These rods will be pulled at future intervals for inspection and determination of corrosion loss.

CONSTRUCTION

Plans and specifications for the project were completed in May 1981. Because of funding constraints, construction was deferred until the 1982 construction season. The project was advertised in March, with a bid opening of April 20, 1982. The successful bidder selected the MSE alternative with a bid of \$816,930 for the 17,626 ft² of wall and associated road work. The second low bidder submitted the reinforced-concrete crib-wall alternative (17,497 ft²) with a total bid of \$882,621. An RE wall was bid a close third at \$883,226 (17,278²). The contract was awarded to the Teichert Construction Company on May 28, 1982.

The difference in bid price was due primarily to the smaller amount of excavation (6,000 yd³ versus 7,350 yd³) and use of the on-site lower-quality backfill for the MSEs. The RE walls required 2 ft of additional horizontal embedment with imported backfill material. The RE backfill requirements included a maximum rock size of 6 in. and up to 25 percent passing the No. 200 sieve with a sand equivalent value of 25 minimum; that is, with the exception of the maximum rock size, the backfill was of subbase quality. The only quality requirement for MSE backfill, with the exception of 6-in. maximum size, was a maximum limitation on the plasticity index of 10.

The MSE construction was to be instrumented and subject to evaluation and monitoring under a federally financed research project as a type-B study.

On July 9, 1982, wall erection was under way; the initial delivery of prefabricated concrete facing elements and W7 welded wire reinforcing mats had been made. The contractor's operation began with excavation and stockpiling of the existing embankment material from wall 1 (location 1, see Figure 4). This material was used as backfill; additional materials came from the excavations for walls 2 and 3 (location 2). Test results on progress samples of backfill from wall 1 (location 1) are shown in Tables 2 and 3. These results are comparable with the initial test results.

Construction continued until mid-September, when progress was interrupted by intermittent rains. In August the project was shut down for 1 week because of unsatisfactory plasticity index tests on the native backfill material intended for wall 1, which were found to exceed the specification limit of 10 by as much as 8. This degree of plasticity was unanticipated based on the results of the original exploration in 1979 (Table 2). However, there was minimal concern with respect to internal stability because of the conservative design criteria used, the presence of a positive subsurface drainage system, and the results of the laboratory pullout tests under saturated conditions. The decision was ultimately made to allow the contractor to proceed with the operation after the assessment of a \$2,000 rebate.

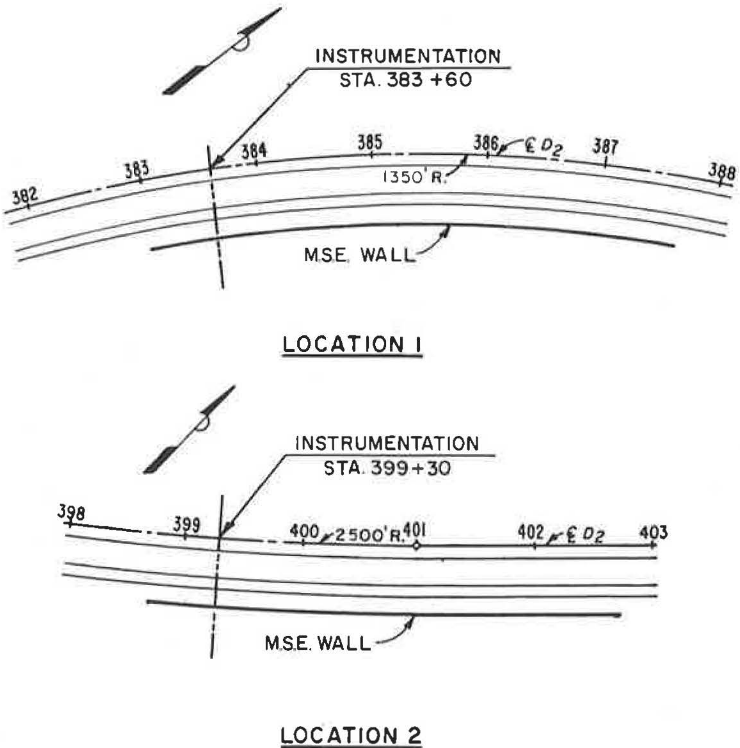

FIGURE 4 Plan of instrumentation locations.

Wall 3 (location 2) was the last wall to be completed, and materials from excavation had already been used for the other three walls. Local borrow was required as backfill to complete this facility. Initial tests on the proposed borrow material are presented in Table 5. Progress tests made during erection of wall 3 (location 2) are also presented. The materials placed as backfill were coarser than those initially tested. The backfill is also reported as nonplastic. The corrosion parameters for pH and resistivity also favor a longer-term performance for wall 3 (location 2) compared with the same parameters in Table 2, which suggest a shorter-term performance (location 1).

The walls were completed and the roadway was paved during November 1982. During construction, several rainstorms occurred that delayed the work. The fine-grained backfill material became partially saturated and additional time was required before work could resume. The backfill also required some reworking for proper compaction.

The method of construction for the permeable

TABLE 5 Physical Soil Properties of Backfill Materials for Location 2

Property	Progress Sample from Location 2 (Wall 3): Sample 82-1256	Progress Sample from Location 2 (Wall 3): Sample 82-1261	Proposed Imported Local Borrow (Location 2, Sample 82-1203)	Specifications
Plasticity index PI (%)	NP	NP	5	10 max.
pH	6.1	6.1	6.4	–
Resistivity (ohm-cm)	23,400	29,200	22,800	–
Sand equivalent	9	12	–	–
Maximum wet density (pcf)	118	117	–	–
Sieve size (% passing by weight)				
6 in.	–	–	–	100
3 in.	–	–	–	–
2½ in.	100	–	–	–
2 in.	99	100	–	–
1½ in.	99	99	–	–
1 in.	97	98	–	–
3/4 in.	96	98	–	–
1/2 in.	95	97	–	–
3/8 in.	94	97	–	–
No. 4	92	96	100	–
No. 8	89	94	98	–
No. 16	87	92	96	–
No. 30	85	89	95	–
No. 50	81	83	93	–
No. 100	70	66	91	–
No. 200	57	50	86	–
5 μ	12	12	25	–
1 μ	5	6	9	–
Unified Soil Classification	ML	ML	ML	–

blanket had a significant influence on the instrumentation results, which will be described in subsequent sections of this report.

INSTRUMENTATION

Two of the four walls to be constructed were selected for instrumentation. One critical section on each of these walls was instrumented in detail (Figures 3 and 4). Station 383+60 for wall 1 (instrument location 1) combined a high groundwater table and possible seepage problems. The maximum height of the wall was 14 ft. Station 399+30 for wall 3 (instrument location 2) represented the highest wall section at 16 ft.

Strain gauges, pressure cells, reference monuments, plumb points, and open standpipe piezometers were installed as shown in Figure 3. Steel inspection rods (W7 bars) were also installed to monitor corrosion rate. Two Ailtech weldable SG129 strain gauges were installed on the steel bar-mats at each strain gauge location as shown in Figure 2. Three levels of instrumentation (A, B, and C) were installed at both wall instrumentation locations to determine anchor bolt and bar-mat stresses (see Figure 3).

One Carlson stress meter was installed behind the concrete wall face at each bar-mat level to monitor lateral soil pressure. Vertical and lateral wall movements were monitored by reference points on the top and on the face of each wall and on the toe buttress. All instrumentation was monitored during and after construction at scheduled intervals.

WALL PERFORMANCE

Reinforcement Stresses

Stresses determined from strain gauge measurements are presented in Figures 5 and 6 for the completed walls at instrument locations 1 and 2, respectively, after pavement placement on November 24, 1982, and 6 months after construction on May 19, 1983. Anchor bar stresses are shown for strain gauge measurements 0.5 ft back from the face. All other points on Figures 5 and 6 represent bar-mat stresses in the W7 bars. The highest bar-mat stresses were recorded in level B for both walls. These stresses are considerably less than the design working stress of 24 ksi. The bar-mat stress patterns are relatively uniform at each level and conform to stress results found with other MSE walls, which show no significant peak stresses (5). The higher stresses at level B probably reflect some consolidation of the fine-grained backfill soil within the reinforced-soil block.

Soil Pressure Against Concrete Wall Face

Lateral soil pressure on the wall face as determined from pressure cell measurements is shown in Figures 7 and 8 for locations 1 and 2, respectively. The higher lateral pressures at level B, location 1, are consistent with the higher bar-mat stresses (Figure 5). The exception is the first pressure reading after 1.5 ft of overburden, which shows overregistration, possibly due to excessive compaction near the wall face. Subsequent readings show pressure relaxation with additional fill placement. These lateral pressures are somewhat confirmed by Figures 9 and 10, which present, for comparison, the theoretical wall pressures determined from actual anchor bar stresses (0.5 ft from the face) during backfilling. These wall pressures were determined by distributing the average tensile force on each 3/4-in.-diameter anchor bar over the contributing area of the concrete face panel. Since four anchor bar connectors attach per panel, the contributing area is 2 ft high by 12.5 ft long divided by 4, which equals 6.25 ft^2. The magnitudes of pressure are not comparable in all cases because of possible overstressing of the pressure cells during compaction operations near the face, that is, levels A and B for location 1. However, these data provide information on the coefficient of earth pressure during the early stages of construction.

A significant reduction in lateral pressure was noted for the lower portion of both walls due in part to the contractor's method of operation. The permeable drainage blanket shown behind the reinforced soil block in Figure 3 would normally be placed concurrent with the wall backfill. The contractor opted to place the lower portion of the vertical blanket concurrent with the wall backfill, cover the permeable material and filter fabric with plywood sheeting, and continue the construction of

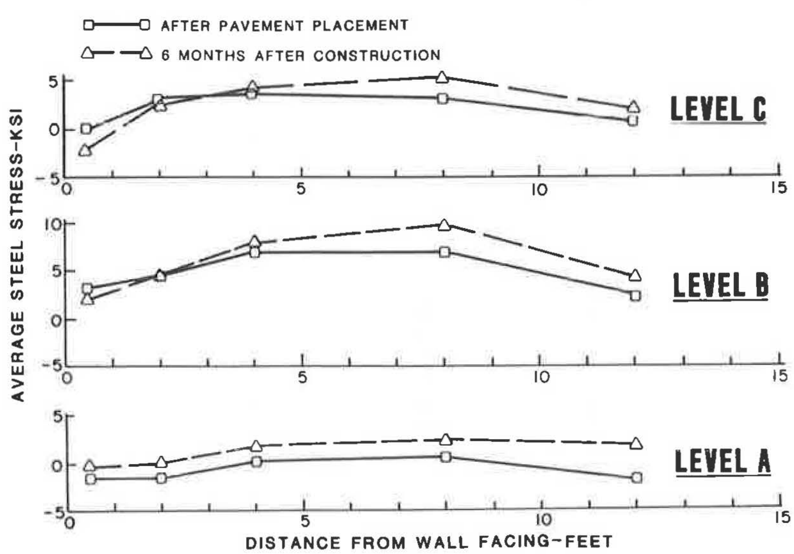

FIGURE 5 Steel stresses in earth reinforcement location 1.

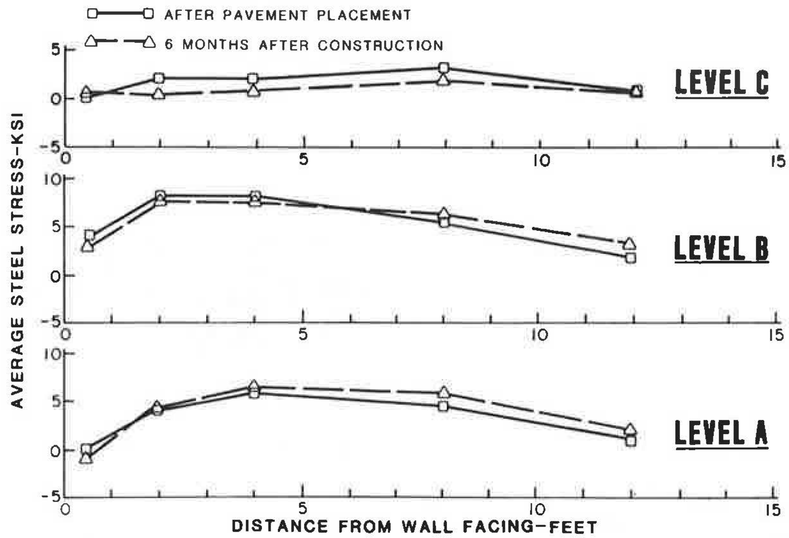

FIGURE 6 Steel stresses in earth reinforcement location 2.

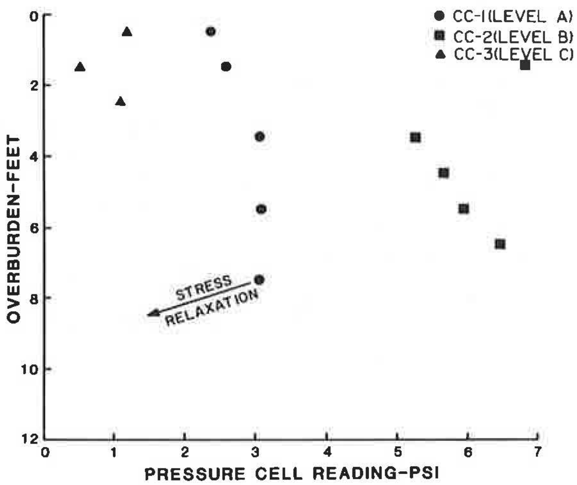

FIGURE 7 Soil pressure on wall face from pressure cell measurements at location 1.

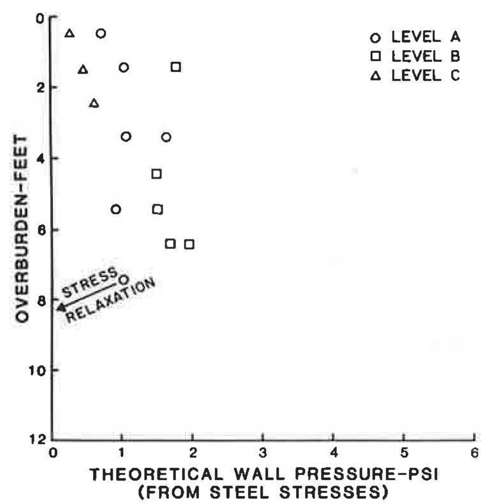

FIGURE 9 Soil pressure on wall face from strain gauge measurements at location 1.

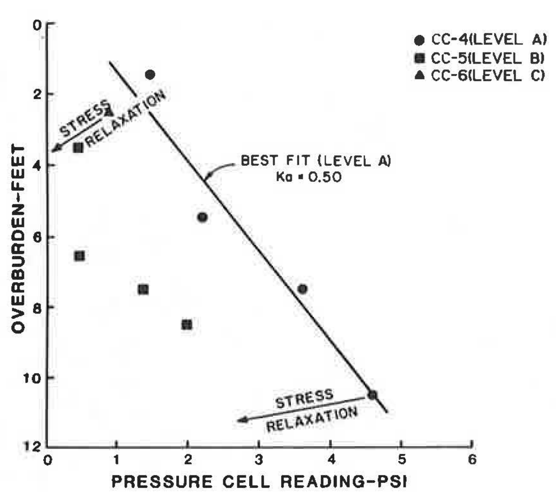

FIGURE 8 Soil pressure on wall face from pressure cell measurements at location 2.

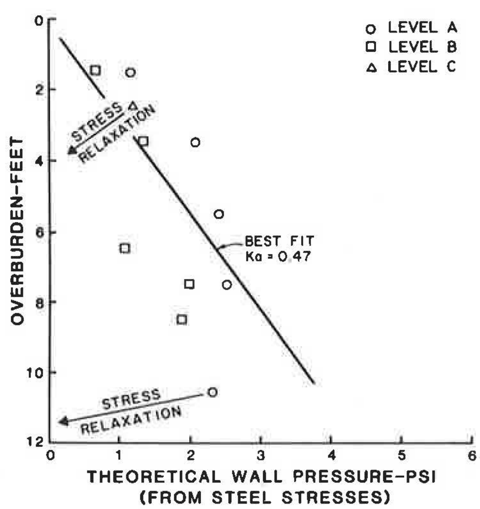

FIGURE 10 Soil pressure on wall face from strain gauge measurements at location 2.

the wall. When the top of wall was reached, a vertical trench was excavated down to the plywood. The plywood was removed, filter fabric was placed on the trench walls, and permeable material was placed (Figure 11).

The steel bar-mat stresses in Figures 5 and 6 show an increasing trend in tensile stress near the rear of the reinforced-soil block (8 and 12 ft back from wall face) after 6 months. This could be the result of soil creep of the reinforced-soil block toward the permeable material in the trench, which received a lower compactive effort than the reinforced-soil fill.

FIGURE 11 Placement of filter fabric and permeable material in vertical trench.

The two wall locations are evidently readjusting to these stress conditions as indicated by both soil pressure and steel stress relaxation. However, a reasonable approximation of earth pressure distribution with depth can be determined for location 2 by using both pressure cell readings for level A (Figure 8) and theoretical pressures (Figure 10). Lines of best fit for these data suggest coefficients of assumed active pressure (Ka) equivalent to 0.50 and 0.47 for Figures 8 and 10, respectively. These values are somewhat less than the conservative value of 0.70 used for design.

The foregoing analysis is considered preliminary because some stress adjustment will continue to occur within the reinforced-soil system as a result of the vertical trench excavation. This will be determined from additional instrumentation monitoring.

Lateral and Vertical Wall Movement

The results of monitoring lateral and vertical reference points indicate no significant wall deformations since completion of construction. After one winter season, which provided near-record rainfall, the walls are performing satisfactorily.

FIELD PULLOUT TESTS

Dummy bar-mats were installed at various depths in wall 3 (near instrument location 2) during its construction. These mats were placed in the backfill and extended beyond the facing as shown in Figures 12 and 13 at five levels between stations 398+97 and

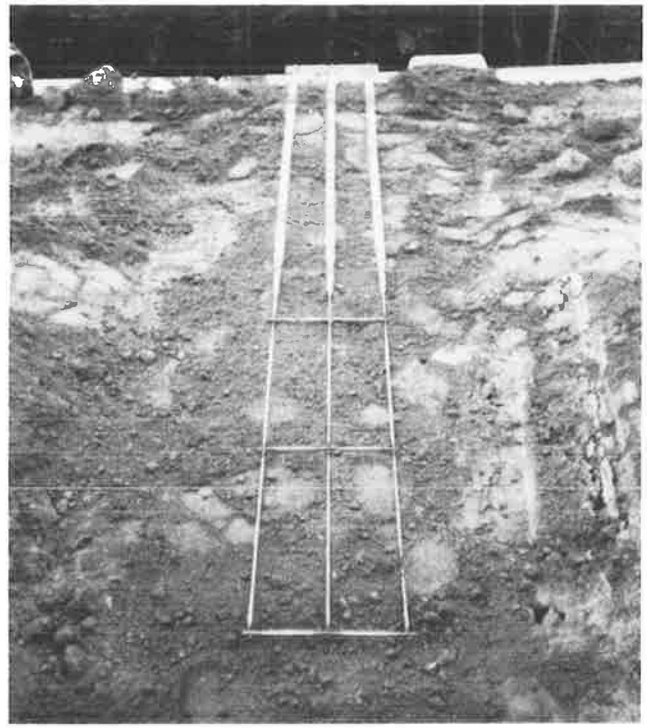

FIGURE 12 Typical dummy bar-mat (three transverse bars) placed in backfill during construction.

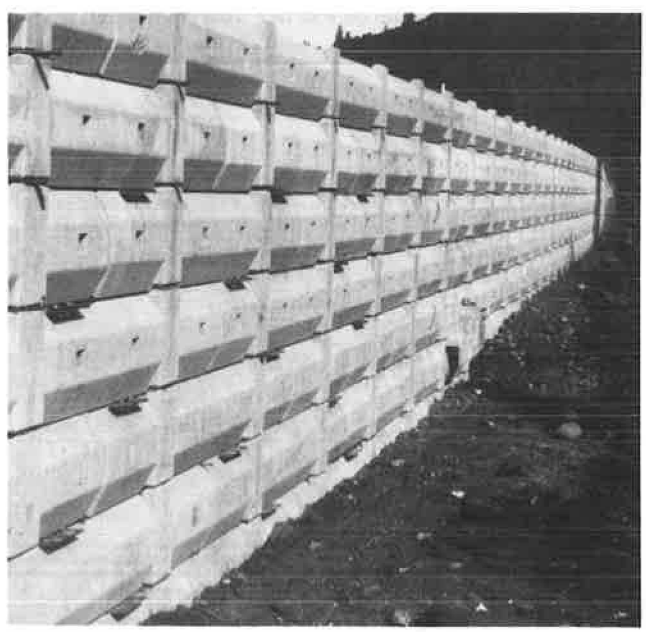

FIGURE 13 Front face of wall at instrument location 2 (wall 3) with dummy bar-mats extending from face.

399+22. The dummy bar-mat configurations consisted of three longitudinal bars and one, two, or three transverse bars to form a 6 x 24-in. grid (Figure 14). The outer 6 ft 2 in. of the longitudinal bars was equipped with greased sleeves to prevent soil bond. The mats extended a maximum of 10 ft 8 in. back from the front face of the wall with overburden heights of 4, 6, 8, 10, and 12 ft for each of the three bar-mat configurations. The object was to develop pullout information on the relative effect of individual transverse bars at various overburden pressures and then relate it to laboratory pullout test results.

Field pullout testing was performed during June 1983 by attaching a hydraulic jack and load cell and applying load to the face through a timber frame as shown in Figure 15. Loading continued until 8 in. of extension or failure occurred.

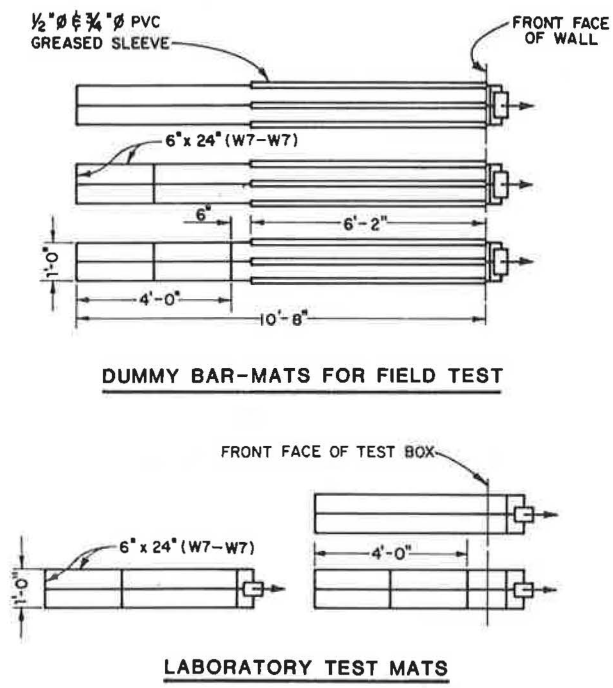

FIGURE 14 Bar-mat configurations for laboratory and field tests.

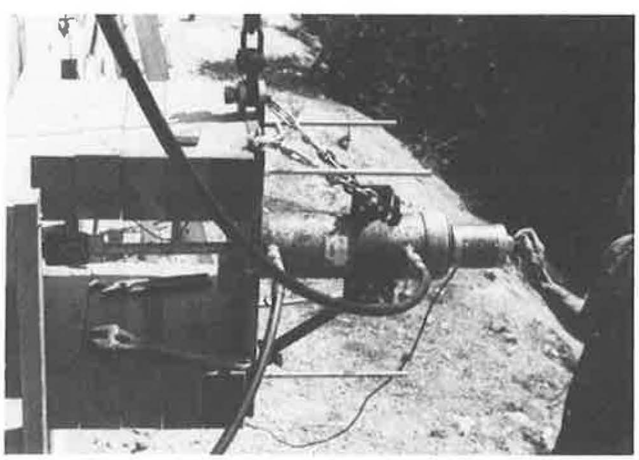

FIGURE 15 Apparatus for conducting field pullout tests.

A comparison of test results is reported here for the 6- and 10-ft overburden heights in Figures 16 and 17, respectively, with one, two, and three transverse bars. In these figures the first number on the curve represents the number of transverse bars. The second number represents the overburden height in feet. These results confirm previously reported laboratory tests that suggest that almost all of the total pullout resistance of grid-type reinforcement is mobilized by the transverse bars (3).

A comparison of field pullout resistance for the five different overburden heights is shown in Figure 18 for the dummy bar-mats with three transverse bars. The peak pullout loads are quite variable and are not consistent with theory, that is, that there is increasing pullout with increased overburden. This inconsistency is due partially to strength variability of the low-quality backfill resulting from the moisture regime within the reinforced mass.

Undisturbed soil sample tubes were obtained just before pullout testing from borings made 10 ft back from the wall face near the position of the dummy bar-mats. Field density, moisture content, plasticity index, and the results of laboratory triaxial tests are shown in Table 6.

A series of laboratory strain controlled pullout tests were performed in November 1983 with the same bar-mat configurations as those of the field dummy bar-mats (see Figure 14). This work provides a direct relationship between laboratory and field pullout tests.

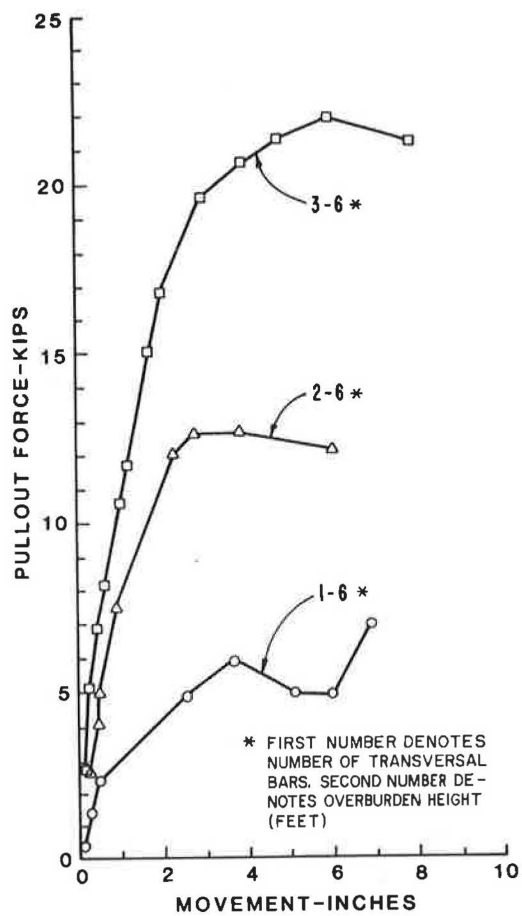

FIGURE 16 Field pullout resistance of dummy bar-mats at 6 ft overburden.

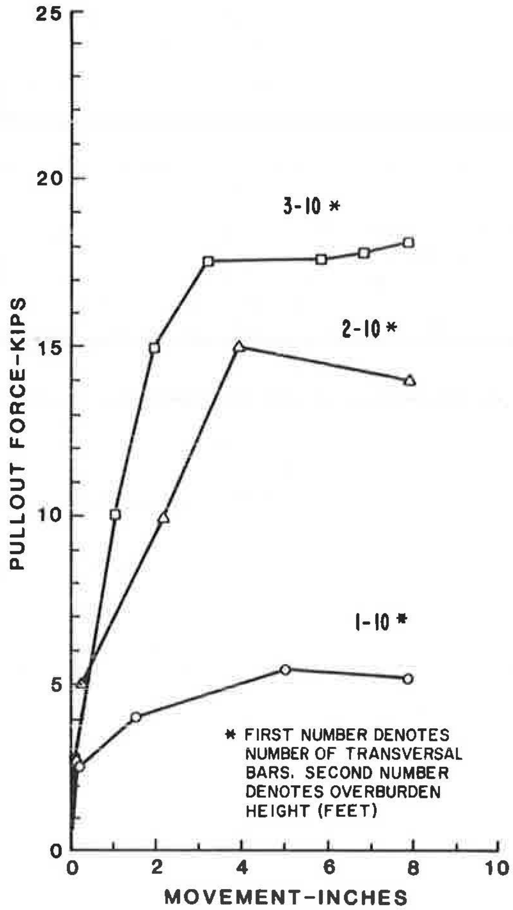

FIGURE 17 Field pullout resistance of dummy bar-mats at 10 ft overburden.

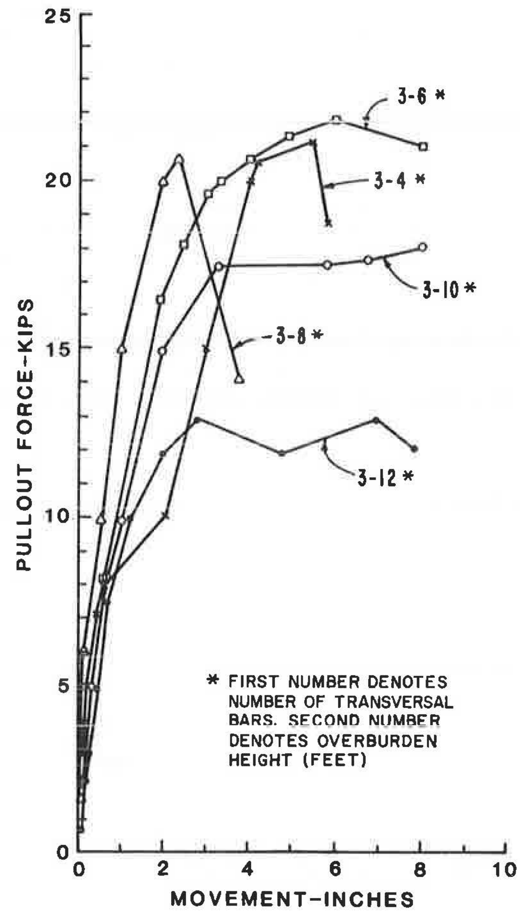

FIGURE 18 Field pullout resistance of dummy bar-mats with variable overburden.

TABLE 6 In Situ Properties of Undisturbed Field Soil Samples from Location 2 (Wall 3)

	Station 398+97			Station 399+10			Station 399+22		
Property	B1-1C	B1-4C	B1-5D	B2-1C	B2-3B	B2-5B	B3-1C	B3-3D	B3-4C
Depth[a] (ft)	4	12	14	4	8	13	3	11	15
Type of test	CU	CU	UU	CU	CU	UU	CU	CU	UU
Angle of internal friction ϕ (degrees)	34	34	21	34	28	19	34	26	18
Cohesion c (psf)	1,300	1,000	700	2,000	1,800	950	1,000	1,200	900
Plasticity index PI (%)	2	3	3	–	5	2	–	1	1
Field wet density (pcf)	124	122	118	125	120	116	114.4	122	119
Field moisture content (%)	21.6	10.2	18.6	19.3	29.2	25.5	24.2	22.2	26.1

Note: UU = unconsolidated undrained; CU = consolidated undrained; B1-1C, etc. = sample numbers.
[a]Depth below finished grade 10 ft back from wall face.

Backfill for the laboratory tests was obtained from the borrow site for wall 3 (location 2) during its construction. Laboratory pullout specimens were fabricated with the same field moisture content and were compacted to the field densities shown in Table 4. The procedure for all tests was similar to that described by Chang et al. (1).

A comparison of laboratory and field pullout tests with one, two, and three transverse bars at 8 ft of overburden is presented in Figure 19. The field tests in all cases provided pullout results in excess of those produced in the laboratory under the same backfill conditions.

The field test with three transverse bars produced a pullout resistance value at 1 in. movement (strain) that was more than twice the laboratory value. This result is also shown in Figure 19 for the configurations with one and two transverse bars. The peak field pullout resistance also shows a similar trend.

The maximum bar-mat stress (σ_{max}) determined from strain gauge measurements in wall 3 (location 2) was about 8 ksi at level B (Figure 6). For three longitudinal bars equivalent to the field dummy bar-mats (Figure 14), the maximum observed tensile load (T) per bar-mat was

$$T = \sigma_m \text{ (3) (area of W7 bar)},$$
$$T = 8 \text{ ksi (3) } (0.07 \text{ in}^2) = 1.68 \text{ kips}.$$

Assuming a maximum field pullout resistance of 18

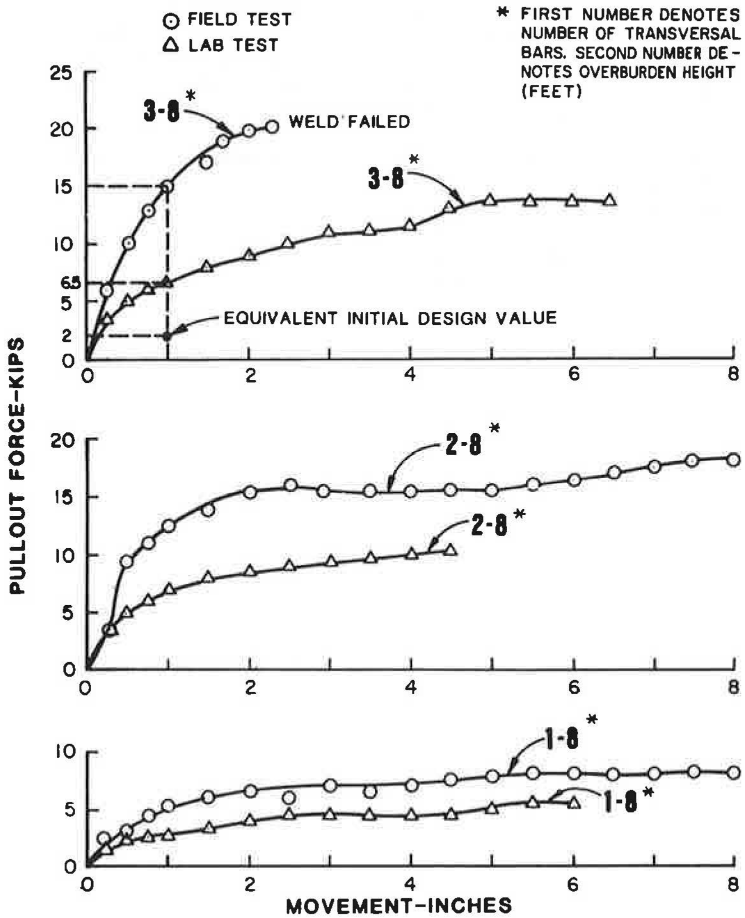

FIGURE 19 Comparison of laboratory and field pullout tests with one, two, and three transverse bars.

kips for three transverse bars (Figure 17), the maximum observed tensile load per bar-mat was

(1.68 kips/18 kips) (100) or 9.3 percent of the maximum developed pullout resistance.

The conservativeness of this particular wall design is also illustrated by referring to the previous section of this paper. A design value of 4 kips was assumed for the original preliminary laboratory pullout tests at 1 in. lateral movement. The bar-mats for these tests were 2 ft wide by 4 ft long as opposed to the laboratory test bar-mats (Figure 14) used for the field test comparison (1 ft wide by 4 ft long) with W7 bars on a 6 x 24-in. grid. An equivalent initial design value of 2 kips is shown for comparison in Figure 19 with the dummy bar-mat configuration for three transverse bars. These results indicate considerable conservativeness. However, the backfill material used for the initial tests is more representative of the materials actually placed for wall 1 (location 1) rather than for wall 3 (location 2).

These pullout tests conclusively illustrate the conservativeness of using laboratory pullout test values for design. The lower laboratory values are due in part to the free face test condition as opposed to the restraint provided by the concrete face panels in the field test.

CONCLUSIONS

1. This project illustrates that mesh-type earth reinforcement systems when properly designed and constructed can function satisfactorily with low-quality backfill.
2. The design criteria used for low-quality backfill on this project were conservative.
3. Laboratory pullout tests provide a conservative approximation of actual pullout resistance of mesh or bar-mat reinforcement.
4. Pullout resistance of mesh-type earth reinforcement in poor-quality soil does not necessarily increase with depth as it does with good-quality backfill.
5. The transverse bars are the major contributor to pullout resistance.
6. Mesh-type earthwork reinforcement systems offer considerable savings in wall construction costs by using on-site materials considered unsuitable for backfill construction.

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