Technical Report Documentation Page

1. REPORT No.

FHWA/CA/TL-86/06

2. GOVERNMENT ACCESSION No.

3. RECIPIENT'S CATALOG No.

4. TITLE AND SUBTITLE

Mechanically Stabilized Embankment Constructed with Low Quality Backfill

5. REPORT DATE

June 1986

6. PERFORMING ORGANIZATION

7. AUTHOR(S)

Terrie Bressette and George Chang

8. PERFORMING ORGANIZATION REPORT No.

633368

9. PERFORMING ORGANIZATION NAME AND ADDRESS

Office of Transportation Laboratory
California Department of Transportation
Sacramento, California 95819

10. WORK UNIT No.

11. CONTRACT OR GRANT No.

R82TL14

12. SPONSORING AGENCY NAME AND ADDRESS

California Department of Transportation
Sacramento, California 95807

13. TYPE OF REPORT & PERIOD COVERED

Final

14. SPONSORING AGENCY CODE

15. SUPPLEMENTARY NOTES

This study was conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration.

16. ABSTRACT

This report presents the results of a three year evaluation of Mechanically Stabilized Embankment (MSE) constructed using low quality backfill (greater than 32% passing the #200 sieve).

Four MSE walls were constructed in mid-1982 along Interstate 80 near Baxter, California utilizing the clayey silt and clayey sand embankment available within the project limits. Two walls were instrumented to monitor stresses in the reinforcement, both horizontal and vertical movement, and lateral pressure on the wall face.

The bid price for the MSE alternative was 12% lower than Reinforced Earth walls and 15% lower than concrete crib walls. The lower cost is attributed to reduced excavation and on-site availability of backfill.

Dummy bar-mats were buried in the backfill at various depths and pulled out a year later to determine pullout resistance values for various configurations. Laboratory pullout tests were conducted in a direct shear device using bar-mats with the same configurations. Pullout test results are compared. No definitive relationship exists between field, laboratory, or theoretical results; number of transverse bars and resistance; or resistance and overburden.

The walls have performed well. Stresses measured in the reinforcement were distributed in a manner similar to expected distribution.

17. KEYWORDS

Mechanically Stabilized Embankment, bar-mat reinforcement, stress, soil pressure distribution, instrumentation, pullout resistance

18. No. OF PAGES:

134

19. DRI WEBSITE LINK

http://www.dot.ca.gov/hq/research/researchreports/1981-1988/86-06.pdf

20. FILE NAME

86-06.pdf

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF ENGINEERING SERVICES
OFFICE OF TRANSPORTATION LABORATORY

MECHANICALLY STABILIZED EMBANKMENTS CONSTRUCTED WITH LOW QUALITY BACKFILL

Study Supervised by........................Robert W. Reynolds, R.E.G.
Chief, Geotechnical Branch

Co-Principal Investigators................Joseph B. Hannon, P.E.
George H. Chang, P.E.

Report Prepared by........................Terrie Bressette, P.E.
George H. Chang, P.E.

RAYMOND A. FORSYTH, P.E.
Chief, Office of Transportation Laboratory

86-06

10-28

TECHNICAL REPORT STANDARD TITLE PAGE

1. REPORT NO.	2. GOVERNMENT ACCESSION NO.	3. RECIPIENT'S CATALOG NO.
FHWA/CA/TL-86/06		

4. TITLE AND SUBTITLE
Mechanically Stabilized Embankment Constructed with Low Quality Backfill

5. REPORT DATE
June 1986

6. PERFORMING ORGANIZATION CODE

7. AUTHOR(S)
Terrie Bressette and George Chang

8. PERFORMING ORGANIZATION REPORT NO.
633368

9. PERFORMING ORGANIZATION NAME AND ADDRESS
Office of Transportation Laboratory
California Department of Transportation
Sacramento, California 95819

10. WORK UNIT NO.

11. CONTRACT OR GRANT NO.
R82TL14

12. SPONSORING AGENCY NAME AND ADDRESS
California Department of Transportation
Sacramento, California 95807

13. TYPE OF REPORT & PERIOD COVERED
Final

14. SPONSORING AGENCY CODE

15. SUPPLEMENTARY NOTES
This study was conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration.

16. ABSTRACT

This report presents the results of a three year evaluation of Mechanically Stabilized Embankment (MSE) constructed using low quality backfill (greater than 32% passing the #200 sieve).

Four MSE walls were constructed in mid-1982 along Interstate 80 near Baxter, California utilizing the clayey silt and clayey sand embankment available within the project limits. Two walls were instrumented to monitor stresses in the reinforcement, both horizontal and vertical movement, and lateral pressure on the wall face.

The bid price for the MSE alternative was 12% lower than Reinforced Earth walls and 15% lower than concrete crib walls. The lower cost is attributed to reduced excavation and on-site availability of backfill.

Dummy bar-mats were buried in the backfill at various depths and pulled out a year later to determine pullout resistance values for various configurations. Laboratory pullout tests were conducted in a direct shear device using bar-mats with the same configurations. Pullout test results are compared. No definitive relationship exists between field, laboratory, or theorectical results; number of transverse bars and resistance; or resistance and overburden.

The walls have performed well. Stresses measured in the reinforcement were distributed in a manner similar to expected distribution.

17. KEY WORDS
Mechanically Stabilized Embankment, bar-mat reinforcement, stress, soil pressure distribution, instrumentation, pullout resistance.

18. DISTRIBUTION STATEMENT
No restrictions. This document is available to public through the National Technical Information Service - Springfield, VA 22161

19. SECURITY CLASSIF. (OF THIS REPORT)	20. SECURITY CLASSIF. (OF THIS PAGE)	21. NO. OF PAGES	22. PRICE
Unclassified	Unclassified		

DS-TL-1242 (Rev.6/76)

CONVERSION FACTORS

English to Metric System (SI) of Measurement

Quality	English unit	Multiply by	To get metric equivalent
Length	inches (in)or(")	25.40 .02540	millimetres (mm) metres (m)
	feet (ft)or(')	.3048	metres (m)
	miles (mi)	1.609	kilometres (km)
Area	square inches (in^2) square feet (ft^2) acres	6.432×10^{-4} .09290 .4047	square metres (m^2) square metres (m^2) hectares (ha)
Volume	gallons (gal) cubic feet (ft^3) cubic years (yd^3)	3.785 .02832 .7646	litre (1) cubic metres (m^3) cubic metres (m^3)
Volume/Time (Flow)	cubic feet per second (ft^3/s	28.317	litres per second 1/s)
	gallons per minute (gal/min)	.06309	litres per second (1/s)
Mass	pounds (lb)	.4536	kilograms (kg)
Velocity	miles per hour (mph) feet per second (fps)	.4470 .3048	metres per second (m/s) metres per second (m/s)
Acceleration	feet per second squared (ft/s^2)	.3048	metres per second squared (m/s^2)
	acceleration due to force of gravity (G) (ft/s^2)	9.807	metres per second squared (m/s^2)
Density	(lb/ft^3)	16.02	kilograms per cubic metre (kg/m^3)
Force	pounds (lbs) (1000 lbs) kips	4.448 4448	newtons (N) newtons (N)
Thermal Energy	British termal unit (BTU)	1055	joules (J)
Mechanical Energy	foot-pounds (ft-lb) foot-kips (ft-k)	1.356 1356	joules (J) joules (J)
Bending Moment or Torque	inch-pounds (in-lbs) foot-pounds (ft-lbs)	.1130 1.356	newton-metres (Nm) newton-metres (Nm)
Pressure	pounds per square inch (psi)	6895	pascals (Pa)
	pounds per square foot (psf)	47.88	pascals (Pa)
Stress Intensity	kips per square inch square root inch ($ksi\sqrt{in}$)	1.0988	mega pascals$\sqrt{metre}$ ($MPa\sqrt{m}$)
	pounds per square inch square root inch ($psi\sqrt{in}$)	1.0988	kilo pascals$\sqrt{metre}$ ($KPa\sqrt{m}$)
Plane Angle	degrees (°)	0.0175	radians (rad)
Temperature	degrees fahrenheit (F)	$\frac{+F - 32}{1.8} = +C$	degrees celsius (°C)

NOTICE

The contents of this report reflect the views of the Office of Transportation Laboratory which is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the State of California or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

Neither the State of California nor the United States Government endorse products or manufacturers. Trade or manufacturers' names appear herein only because they are considered essential to the object of this document.

ACKNOWLEDGEMENTS

This report was prepared by the Special Research and Study Unit, Geotechnical Branch, Transportation Laboratory, Department of Transportation, under the direction of Raymond A. Forsyth and his successor, Robert W. Reynolds.

The authors wish to acknowledge the contributions of the following individuals: Preeda Sitachitta and Morris Tatum for designing and modifying the system; Gary Purcell for construction inspection and technical assistance; Dave Castanon for instrumentation monitoring, data collection, and data reduction; Jack Abcarius and Darrell Beddard for technical support; the Translab graphic services staff; and Tina Paschke, Marlene Glaspey, and Kennie Callahan for word processing.

A special thanks to resident construction engineer, Brian R. Schonlank, Office of Structures Construction, and the Survey Crew in District 3 for their contributions to the success of this study.

TABLE OF CONTENTS

TABLE OF CONTENTS (Cont'd)

LIST OF PHOTOGRAPHS

LIST OF FIGURES

LIST OF FIGURES (Cont'd)

LIST OF FIGURES (Cont'd)

LIST OF TABLES

LIST OF TABLES (Cont'd)

INTRODUCTION

A. Background

Mechanically Stabilized Embankment (MSE) is an earth reinforcing system which uses both longitudinal and transverse reinforcing elements to obtain the pullout resistance required for internal stability. This design is a variation of the Reinforced Earth (RE) design originated by Henri Vidal (1) which uses only longitudinal reinforcing strips. Examples of the two configurations can be found in Photographs 1 and 2. The California Department of Transportation (Caltrans), constructed its first RE wall in 1972 (2) (3). In 1973, a large direct shear device was developed at the Transportation Laboratory (Translab) to measure the pullout resistance of reinforcing elements. Testing in this device supported a developing hypothesis that a bar-mat reinforcement configuration could develop much greater pullout resistance than the flat longitudinal strips of the RE system, in terms of steel area exposed to soil. The MSE system was developed in 1974 based on this observation.

In 1974 the hypothesis was further tested with the construction of two MSE walls and one RE wall along Interstate 5 near Dunsmuir, California. All three walls were constructed using high quality backfill. One MSE wall and one RE wall of similar size and configuration were instrumented and carefully observed. The results of this research were presented in detail in 1982 (4). The evaluation of the field data from this project also supported the premise that MSE bar-mats developed more pullout resistance than RE strips.

Taking these observations one step further, it was suggested that MSE walls could be constructed using lower quality backfill than that required by RE walls and still develop the pullout resistance necessary for internal stability. Considering the increasing cost and decreasing availability of quality backfill material, this proposal was very appealing. As early as the mid-1970's, Caltrans was aware that RE walls could be less expensive than traditional retaining walls at locations where the retaining system is very high and a cantilever retaining wall would require extensive foundation preparation. The use of lower quality backfill material in MSE construction could represent even greater savings, particularly if onsite excavation material could be used.

In 1979 an appropriate site at which to test both hypotheses was encountered when Caltrans District 3 requested a feasibility study for the construction of MSE walls at four locations on Interstate 80 near Baxter, California (Figure 1). A proposed roadway widening of approximately one half mile in length could not be constructed without retaining walls to support the fill at the four locations where Canyon Creek impinged on the toe of the proposed embankment (Figure 2). Several forms of retaining systems were considered in the preliminary design phase and the contract plans contained alternative designs for MSE walls, RE walls, and Concrete Crib walls. Figure 3 shows the typical section as contained in the contract plans. The MSE design was chosen by the successful bidder at a total project cost considerably lower than the proposed costs of bidders who used either of the other two systems.

Construction of the project began in July 1982 and was completed in November of the same year (Photo 3).

All four MSE walls were constructed of marginal backfill material which was either excavated at the wall sites or obtained at an onsite borrow area. All walls were constructed without problems and have performed satisfactorily. The history, design, construction, and early research results were reported in 1984 (5).

A research proposal submitted in April 1982 provided funding for evaluation of the design, performance, and cost savings of the Baxter MSE walls. The research program proceeded in three phases:

1. Two of the four walls constructed were instrumented and monitored for three years after construction.

2. Dummy bar-mats were placed in the embankment during construction and pulled almost a year later.

3. Laboratory pullout tests were conducted to compare laboratory and field performance.

This report is the summary and evaluation of observations and data from all three phases of the research program.

B. Research Objectives

The specific objectives of the research project as presented in the research proposal dated April 1, 1982, were as follows:

1. To determine the actual field stresses developed in the bar-mats and soil pressures against the face members for comparison with theoretical design.

2. To measure vertical and horizontal movements of the embankment and facing members.

3. To evaluate the potential savings of MSE construction using marginal backfill materials compared to other types of retaining wall construction with select backfill materials.

4. To help establish an MSE design requirement for backfill material which provides an adequate safety factor with low construction cost.

In fulfilling the details of the proposed work plan additional research objectives developed which were as follows:

1. To verify the initial design criteria by conducting large scale pullout tests both in the laboratory and in the field.

2. To compare field bar-mat pullout test data to laboratory bar-mat pullout test data to help establish a testing procedure for future designs involving marginal backfill materials.

DISCUSSION

A. Mechanically Stabilized Embankment

After the successful construction and performance of a Reinforced Earth (RE) wall in 1973, Caltrans developed a direct shear device to test reinforcement strengths. Large scale laboratory pullout tests using this equipment were conducted on several configurations of reinforcement members (5). From these tests and tests by others (6), Caltrans developed the Mechanically Stabilized Embankment (MSE) system which uses a bar-mat configuration of reinforcement to provide pullout resistance. The MSE design was successfully field tested near Dunsmuir, California in 1974 (4). Information gained from this project and from pullout tests conducted at the Translab facility led to the hypothesis that MSE walls could be constructed using marginal to low quality backfill material and still obtain the pullout resistance required to maintain internal stability.

It was not until five years later that an appropriate reinforced earth wall project was located. In April 1979, Caltrans District 3 requested a feasibility study for the construction of four reinforced earth walls. The proposed project met the criteria of potential cost savings through use of low quality onsite backfill and a location at which extensive postconstruction evaluation was feasible.

1. The Project Site

The project site is about two miles east of Baxter, California on eastbound Interstate 80 (Figure 1). At this location it was proposed that the roadway be widened by one lane to provide an area in which to install chains on vehicles before entering the heavy snow area of the trans sierra highway. Canyon Creek meanders adjacent to the highway at this location and prevented extension of the existing fills throughout the full length of the roadway widening. It was proposed that retaining structures support the fill at the four places where slopes would encroach on the streambed (Figure 2).

2. Preliminary Soils Information

Preliminary design recommendations were based on control test data from construction of the existing fill and strength tests from remolded samples. The construction testing was done in 1956 and consisted of sieve analysis, sand equivalent, and R-value testing. Based on the results of these tests, the soils were described as silty or clayey and were given an assumed internal friction angle of 25 degrees. Later, in 1979, tests conducted using bag samples from the existing embankment led to a reclassification of the soil as clayey sand with an internal angle of friction between 20 and 32 degrees and cohesion ranging from 300 to 700 psf (Table 1).

In 1979 a foundation investigation was conducted by Translab Engineering Geology involving three rotary sample borings, twelve 2.25 inch cone penetrometer tests, and eleven 1 inch soil borings. In part the foundation report stated:

> "The foundation soil encountered at the site consists of two units. The upper unit is compacted granular highway embankment which is composed of slightly compact to dense mixtures of silt, sand, and gravel with occasional boulders up to two feet in diameter... The lower unit underlying the embankment consists of coarse, granular, stream deposited alluvium, and in the vicinity of Station 385+00, colluvial silts and clays. The granular alluvium located immediately below the embankment appears to be saturated.
>
> ...The embankment between Station 383+ and 388+ appears to be saturated. Water is flowing out of the horizontal drain installation located in the embankment."

Prior to final design, 1982, a large quantity of onsite embankment material was brought to the Translab soils laboratory to be used for large-scale pullout tests. Samples from this material were tested for remolded soil strengths, density, resistivity, pH, sand equivalent, plastic index, and gradation (Table 2). Soil strength parameters of this material are similar to those determined earlier, 1979.

The large percentage of material passing the No. 200 sieve meant the backfill could not be considered free-draining. A subsurface drainage system would be required to prevent additional stresses resulting from a hydraulic head within the embankment behind the MSE wall.

Soil samples taken from the existing embankment in both 1979 and 1982 were tested for soil pH and resistivity values to determine theoretical corrosion losses for the buried steel elements. Soil samples were also obtained from the shoulder to determine concentrations of deicing salts. These preliminary corrosion test values were within an acceptable range to permit use of onsite materials in the MSE wall construction (Tables 1 to 3). A pH below 4.5 and resistivity values below 1000 ohm-cm are indicative of a corrosive soil and cannot be allowed in reinforced embankment construction .

3. Design

The MSE walls at Baxter were designed in a cooperative effort involving engineers from the Translab District 3 (Marysville), and District 9 (Bishop).

a. Foundation and Slope Stability

Foundation stability was analyzed using the slip circle method invoking limit equilibrium conditions. The analysis was applied to a representative cross section with approximately 35 feet of fill and an 11-1/2 foot high retaining system founded on the embankment. The analysis assumed an angle of internal friction of 10 degrees and a cohesion intercept of 800 psf. The minimum safety factor determined using the computer program, SOILX, was 1.4.

The foundation investigation report from Engineering Geology recommended an allowable bearing capacity of 1.5 tons per square foot for the foundation design of a gravity type retaining wall.

The site is located approximately 35 miles southwest of the active Stampede Valley Fault (M = 6 1/2). Maximum credible bedrock acceleration is estimated at 0.1g (gravity).

Virtually no potential for seismic damage existed and any seismic design consideration was, therefore, eliminated in the final design.

b. Reinforcement Design

The MSE walls at Baxter were the first reinforced earth structures that had been designed using a cohesive backfill. Out of concern for the potential problems that could arise should the backfill become saturated and for want of a precise design procedure involving cohesive soils, it was decided that the reinforcement would be designed using an internal angle of friction of 20 degrees and a cohesion intercept of 500 psf based on preliminary design soil strength parameters (Table 1). The external stability requirements, safety factor of 2.0 against overturning and 1.5 against sliding, were met with mats of the following embedment lengths:

Height of Wall (ft)	Embedment Length (ft)
4 to 10	8
12	10
14 to 16	12

Bars for longitudinal reinforcing members were sized at 0.299 inch in diameter (W7) to carry calculated loads and meet sacrificial steel corrosion loss requirements. (A discussion of corrosion design procedures is included in a later section of this report.) Transverse reinforcing members were W7 wire for ease of construction.

Laboratory pullout tests were conducted using samples from the proposed backfill and various mesh configurations. It was determined that bar-mats constructed of W7 wires in a mesh configuration of 6 inches between longitudinal members and 24 inches between transverse members, having embedment lengths listed earlier, would be sufficient to prevent pullout at the design loads (Table 10). The two threaded bars used to connect the mat to the facing elements were sized at 0.75 inch in diameter (Figure 4).

A proposal for a design procedure for MSE walls constructed in cohesive backfill is outlined in a later section of this report.

c. Concrete Facing

Concrete facing members were designed as a slight modification of the facing used on the MSE walls at Dunsmuir, California. The full face panels are 2 feet high and 12.5 feet wide and are connected to a single layer of bar-mats at midheight. The panel was designed using conventional reinforced concrete criteria for a simply supported beam. The design load was assumed to be a uniform soil pressure acting on the face of the wall. An isometric view and a cross-section of the concrete facing panels with a typical stacking arrangement are shown in Figure 5.

4. Construction

The objectives of this research do not include evaluation of the techniques involved in MSE wall construction. Therefore, only the particular details that may have influenced the performance of the structures or the cost analysis will be discussed. A detailed discussion of MSE construction can be found in the final performance report for the MSE and RE walls at Dunsmuir, California (4).

Construction of the first MSE wall at Baxter began on July 9, 1982 when the prefabricated concrete facing elements and W7 welded wire reinforcing mats were delivered to the project site. At the same time the contractor began excavating and stockpiling existing embankment material for inclusion in the MSE wall. Walls 1, 2 and 4 were constructed using existing embankment as backfill. However, an unanticipated shortage of existing embankment material required that Wall 3 be constructed in part using materials excavated from a local borrow site.

Two walls, Wall 1 and Wall 3, were selected for instrumentation. One critical section of each wall was instrumented in detail. Figure 6 shows locations of instruments. Station 383+60 at Wall 1 was chosen because it was constructed using existing embankment materials and because the foundation investigation had found a high ground water table at this location which could create a seepage problem. Station 399+30 at Wall 3 was chosen because it was constructed in part with backfill from the local borrow site and contains the highest wall section, 16 feet.

All four MSE walls were designed with a permeable drainage blanket between the reinforced earth section and the backslope. A common construction practice is to place this blanket concurrent with construction of the reinforced earth wall. However, the contractor opted to place the vertical drainage curtain by trenching and backfilling after the MSE wall was constructed (Photo 3).

Several rainstorms occurred which delayed construction of the MSE walls. At these times, the fine-grained backfill material became partially saturated and construction was stopped for 2 reasons; (1) the fine grained materials were difficult to compact, and (2) the bar-mats would slide about during compaction of the overlying layer if they were placed at the interface between a wet and dry layer.

In general, construction proceeded smoothly. The walls were completed and the roadway was paved in November 1982.

5. Instrumentation

Various monitoring devices were installed during construction at Station 383+60 of Wall 1 and Station 399+30 of Wall 3 (Figure 6). These instruments included (1) strain gages to determine the stresses that developed in the bar-mats, (2) pressure cells to measure horizontal soil pressures that developed behind the concrete facing panels, (3) reference monuments to measure horizontal and vertical movement at the top of the walls, (4) plumb points on the wall face to measure horizontal and vertical movement , and (5) open standpipe piezometer stations to measure water levels and water pressure within the reinforced embankment. Details of the instrumentation can be seen in Figures 6 and 7. Two Ailtech, Weldable SG 129 strain gages were installed on the bar mats at each strain gage location (Figure 9 - Detail).

All instruments were read periodically during construction and for approximately three years after completion of construction.

6. Postconstruction Observations

Following is a summary of the data collected to evaluate the performance of the two instrumented MSE walls.

a. Soil Properties

In situ soil samples were taken from three locations behind Wall 3 after completion of construction. The soils at these locations were predominantly clayey silts and sands, Unified Soil Classification CL. They had field wet densities of 114 to 125 pounds per cubic foot and moisture contents of 10 to 29 per cent. The soil property information is summarized in Table 4.

Samples taken from 4 to 12 feet below the top of the wall were tested for consolidated undrained strengths at the field moisture content. These results represent the soil strengths that can be expected during the field pullout tests and are probably representative of the soil strengths throughout the postconstruction period. Under these conditions the soils exhibited angles of internal friction, ranging from 26 to 34 degrees with cohesion values ranging from 1000 to 2000 pounds per square foot.

b. Soil Pressure Against the Wall Face

Direct pressure cells were placed at the soil/wall interface to measure lateral earth pressures on the concrete wall panels. Each cell was placed in a recess in the panel face (Photo 5) and the panel was placed during the course of construction. Readings were taken both during and after construction. Because there is some concern about possible erroneous pressure readings created by adjustments made to the wall panels during the construction phase, only pressure readings taken after project completion have been included in this report.

It should be noted that the pressures recorded throughout the observation period can only be used as indicators of pressure distribution. The pressure cells have limitations that directly affect the accuracy of the data collected and are, therefore, not considered as reliable as measured stresses in the bar-mats. Pressure cell accuracy is considered limited due to a number of factors. These factors are as follows:

1. Although every attempt was made to place the pressure cells flush with the concrete panel surface, any surface discontinuity would affect the pressure readings.

2. If the pressure cell face is not absolutely parallel with the panel surface, the cell will measure an eccentric loading.

3. Pressure cells of the type used at Baxter are constructed to read only compressive forces and are unable to read tensile forces.

4. Pressure cells are very susceptible to damage from water and aging. It is often considered that pressure cells have a limited life of approximately 2-1/2 years.

The pressure readings recorded at four intervals over the three years after completion of construction are shown in tabular form in Figure 8. Figure 8 also shows a plot of pressure readings vs depth. The last readings for three of the six pressure cells are considered erroneous because they recorded negative (tension) pressures.

The concrete face panels of the MSE walls are designed to carry a theoretical load characterized by an active state pressure distribution of the backfill. However, the wall face is a flexible surface constructed of separate panels that are not firmly connected one to the other. This means that the wall face, as a total unit, does not act as the restraining force against movement or failure like a cantilever retaining wall. It is also known that the reinforced earth mass behaves like a coherent gravity mass and is by itself stable. The face, therefore, serves three primary purposes; to provide lateral resistance during compaction of the backfill, to prevent erosion of the backfill material along its face, and to present a more aesthetically pleasing treatment than the reinforced soil alone. With this in mind, it would be inaccurate to attempt to draw any conclusions about the pressures measured at the face of the wall as they relate to the loads carried by the reinforcing members. The inconclusive and even contradictory nature of the pressure cell readings is supported by a comparison with stresses measured in the connector bolts only 6 inches away (Firgures 10 to 13). Some explanations for the pressures that were read at the wall face are:

1. The pressures were induced by the compactive effort directly behind the wall face and represent only localized soil pressures.

2. The wall panels are adjusted at various times throughout the construction period to maintain an even face. Some of the pressures measured may be created by the tightening of the face panels against the backfill during these adjustments, rather than the movement of the soil against the wall face.

3. Pressure changes after the end of construction could be the result of the rotation of the wall face panels into or away from the backfill.

4. Pressure changes could be the result of the lateral expansion of the backfill both in reaction to compression of the fill and change in pore water pressures.

c. Stresses in Bar-Mats

Strain gages were attached to the longitudinal reinforcing members of the bar-mats at three levels in each wall; 3, 7, and 10 feet above the base; and at five locations; 0.5, 2, 4, 8, and 12 feet behind the concrete face. To balance bending stresses, these gages were attached in pairs, one on the top and one on the bottom. Two pairs of gages, one on each side of the bar-mat, were attached at equal distances behind the wall face (Figures 7 and 9.) Initial readings of these gages were taken when 2 to 3 feet of soil had been placed over the bar-mat. This reading was chosen to eliminate strains induced both by adjustments made to the face of the wall and by construction equipment distorting the bars during compaction of the overlying layers. Strain gage readings were taken at intervals throughout construction and over three years after the project was completed.

Figures 10 and 11 are plots of stress vs depth at all five locations for four intervals throughout the observation period. The first reading for these curves, November 1982, was taken immediately following completion of the wall and paving of the roadway. Figures 12 and 13 are plots of stress vs location along the embedment length for the three levels at three dates throughout the observation period.

As expected from the theoretical stress distribution, stress increases with depth to approximately midheight of the wall (Figures 10 and 11). A decrease in stress at the lowest level would be expected due to the counteracting stresses developed by the berm at the toe of the walls. However, the lowest layer of strain gages is located at the top of the berm and therefore, the stresses recorded at this level are much smaller than the theoretical stress distribution would suggest.

When the stresses are plotted against their locations along the embedment length, as in Figures 12 and 13, the increase in bar-mat stresses near midlength of the bar-mats delineates the developing failure plane as would be expected in an active state analysis.

In both sets of curves, stress vs depth and stress vs location along the embedment length, a generalized pattern of stress increase followed by decrease over time can be observed. Both the increase and later decrease are to be expected. In the period immediately following construction, the recorded stresses in the reinforced mass are extremely high due to compaction induced stresses and increase or remain high as the mass adjusts itself to gravity forces. However, after the mass has adjusted there follows a period of time in which the induced stresses relax and the mass begins to counteract the pressures brought about by movement in the unstable mass that the reinforced soil is buttressing. These forces acting behind the reinforced soil block are compressive and are absorbed by the more rigid reinforcement.

d. Horizontal Wall Movement

Horizontal movement of the MSE wall faces were monitored for three years following the end of construction. Figures 14 and 15 are plots of movements for Wall 1 and Wall 3 respectively. A review of these figures shows that the maximum displacement was 0.06 foot movement into the backfill at several locations along Wall 3 and that, in general, neither wall experienced any great amount of movement. It would appear that for the most part, the displacement was into the backfill at Wall 3 and variable at Wall 1. There are two possible explanations for the apparent inward movement.

1. The material immediately behind the wall face was not compacted properly allowing the wall face to settle back against the reinforced soil mass. The compressive forces and reduction of tensile forces immediately behind the wall face seem to support this explanation.

2. The contractor elected to place the permeable blanket after completion of construction of the MSE wall. The permeable material was placed in a 2 foot wide trench to approximately total depth. Because the trench was so narrow the material was compacted using a tamper. Thus the reinforced mat could be rotating back into the less compacted material.

Horizontal movement was measured only at the wall face. This movement could reflect the movement of the total reinforced earth structure or it may be only movement of the wall face. There were no provisions made to monitor movement in the reinforced soil mass. Visual inspection of the roadway surface shows no distress in the pavement and there are no signs of movement at the toe of the walls.

e. Settlement

Reference monuments were placed at the top of the wall faces to use for monitoring settlement (Figure 7). Elevation readings were taken at intervals throughout the study period.

Figure 16 is a plot of the maximum settlement recorded at each monument. The maximum settlement at Wall 1 was .06 feet (0.72 inch). The maximum settlement, at wall 3 was 0.03 foot (0.36 inch). The large amount of settlement at the northwesterly end of Wall 1, 0.04 to 0.06 foot, was probably a result of localized erosion from a surface drainage problem that has been corrected.

Most of the settlement took place within the first 6 months after completion of construction.

Again, this settlement may be only settlement at the wall face. No means of monitoring settlement within the MSE mass was provided.

f. Water Table Determination

The selection of instrumentation Station 383+60 at Wall 1 was based on a combination of high ground water and possible seepage problems. In order to reduce hydrostatic pressure on the MSE wall, a 2-foot thickness of permeable material was applied both behind and under the wall section (Figure 7). Instrumentation Station 399+30 at Wall 3 was at less risk from ground water pressures and no permeable materials were added under this wall section (Figure 7). To monitor the water level, piezometers were installed at both locations. During the period of field observations, there were no water levels indicated by the piezometers.

B. Field Pullout Tests

Dummy bar-mats were installed at Wall 3 during construction. These mats were placed at five levels having overburden heights of 4, 6, 8, 10, and 12 feet (Figure 17). These mats extended 11 feet into the fill (Photo 5). Three configurations of bar-mats, 1, 2, and 3 transverse bars, were buried at each level. All bar-mats were pulled in mid-1983, approximately one year after placement.

1. Equipment and Procedure

The dummy bar-mats were constructed of W7 wire (0.299 inch diameter) welded together. All bar-mats were fabricated with three longitudinal bars at 6-inch spacings. The two and three transverse bar configurations formed 6-inch by 24-inch grids. Approximately 6.25 feet of the longitudinal bars near the wall face were enclosed in greased pvc (polyvinyl chloride) sleeves to prevent soil bonding during pull out. See Figure 18 for details of the bar-mat construction.

The pulling apparatus used in the field pullout tests was designed and fabricated to meet the requirements of this study. A 60-ton hydraulic jack and load cell were mounted on a timber and steel frame (Figure 22) to allow the application of loads on the wall face. A truck-mounted boom was used to raise and lower the jack into place against the wall face (Photo 7). A U-type universal connector joined the pulling apparatus to the dummy bar-mat (Figure 19). The force on the load cell, the rate of loading, and the displacement were all recorded by an automatic plotter situated at the top of the wall.

During the same period of time at which the pullout tests were being conducted at Wall 3, borings were made at the three locations of the dummy bar-mats. Continuous sampling was conducted from 3 to 16 feet beneath the surface. A 2-inch California sampler was used to obtain undisturbed specimens for triaxial testing and soil property analysis. The results of those tests are presented in Table 4.

Each dummy bar-mat was tested by attaching the pullout apparatus to the longitudinal bars that extended beyond the wall face. A load was applied to the bar-mat that counteracted a force distributed on the wall face by the timber frame. The procedure called for a constant displacement of 0.20 inch per minute to be applied by the hydraulic jack. However, the limitations of the system used, and the erratic resistance forces encountered made control of the displacement rate difficult. Each bar-mat was pulled until either approximately 8 inches of extension was achieved, resistance forces decreased significantly, or a reinforcement member broke.

2. Test Results

Table 5 presents a summary of the field pullout test results for all the dummy bar-mats. This table includes pertinent soil information based on tests of the in situ soil samples retrieved from the fill near the ends of the bar-mats (Table 4). The soil throughout the depth at all three locations is classified as combinations of silts, sands, and clays with some pebbles and small gravel throughout. Contract specifications allowed acceptance of all materials smaller than 6 inches. Large sample gradations from construction control sampling encountered rock-sized material. Also, Photographs 1 and 5 show large rocks in the fill. Therefore, it can be assumed that rock-sized material was placed near the dummy bar-mats but was not retrieved by the 2-inch sampler. The presence of rocks in the fill is of great importance because they could greatly increase pullout resistance if they become trapped ahead of a transverse bar during pullout testing.

The maximum pullout resistances for all the dummy bar-mats is plotted against overburden heights in Figure 20. More detailed curves showing pullout resistance vs movement for bar-mats with 1, 2 and 3 transverse bars at 6 and 10 feet of overburden are presented in Figures 21 and 22. Figure 23 is a summarization of the pullout force compared to displacement for the bar-mats with three transverse bars at all five overburden heights.

From Figures 20 through 22 it appears that pullout resistance is a function of the number of transverse bars, particularly at displacements greater than 0.5 inch. At smaller overburdens, less than 7 feet, it appears that the pullout resistance is approximately proportional to the number of transverse bars. However, at larger overburdens, greater than 8 feet, a decrease in pullout resistance was encountered and the relationship between resistance and number of transverse bars is less clear.

The decrease in pullout resistance with increase in overburden is contrary to expected behavior and has not been encountered in pullout tests conducted on reinforcing elements in non-cohesive backfill (5,8,9,10). There are three possible explanations.

1. The backfill at greater depths contained smaller quantities of rock-sized material, or less rock-sized material was present within the area surrounding the lower bar-mats.

2. Greater displacement rates and erratic displacement rates at the lower level bar-mats combined to lower the resistance forces.

3. The soil was nearly saturated at time of test, thus, at large overburden pressures excess pore water pressures could develop as the mats are pulled forward due to soil compression in the rupture zone. These pore water pressures could induce a localized state of liquefaction which can significantly reduce the shearing resistance of the soil. Conversely, dilational soil characteristics would induce negative pore pressure development leading to increased soil resistance at low overburden pressures.

C. Laboratory Pullout Tests

Laboratory pullout tests were conducted using the direct shear device designed by Translab personnel. A truckload of fill material taken from behind Wall 3 during construction was used to conduct the tests. Three configurations of bar-mats; 1, 2, and 3 transverse bars were pulled at simulated overburden loads equivalent to 4, 8, 10, 12, and 16 feet.

1. Equipment & Procedure

The bar-mats used in the laboratory pullout tests were fabricated of W7 wire (0.299 inch diameter) welded together to form the same 3 configurations as the dummy bar-mats that were pulled in the field tests (Figure 18).

The direct shear device is an 18-inch x 36-inch x 54-inch steel box which is open at the top and one end. A face is placed on the open end during the compaction process. A 9-inch layer of soil, compacted to 90 percent relative compaction (California Test 216) with 21 to 23 percent moisture content to simulate field conditions, was placed in the lower half of the box. A bar-mat was placed on this layer and an additional 9 inches of soil was placed and compacted to the same specifications. A nuclear gage was used to check densities and moisture contents. In several tests, soil samples were taken to check the nuclear gage values.

A hydraulic jacking system was placed on top of the soil and a load applied to simulate the desired overburdens. The soil in the shear box was allowed to consolidate under this load for at least 24 hours.

The face plate was removed from the box and a hydraulic jack and load cell attached to the bar-mat. Each bar-mat was subjected to a displacement loading of approximately 0.25 inch per minute until the resistance force levelled off. A record of overburden loading, displacement rate, and resistance force was kept by a mechanical plotting device.

At the end of the test for a lower overburden, 4 to 12 feet, the overburden was increased and the soil/bar-mat mass was allowed to consolidate for at least 3 to 4 hours. The bar-mat pullout resistance was again tested as described earlier. The overburden loads were "bumped" on all tests.

2. Test Results

Table 6 is a summary of the results from the laboratory pullout tests. The densities and moisture contents for all the tests were relatively similar. Figure 24 is a plot of maximum pullout resistance compared to overburden height.

A review of Table 6 and Figure 24 suggests that no direct relationship exists between number of transverse bars and pullout resistance. Nor does there seem to be any discernible relationship between overburden height and pullout resistance.

The failure of these tests to produce expected results may be explained as follows.

1. The face plate was removed from the shear box prior to displacement loading. Therefore, less lateral resistance counteracted the forward movement of the bar-mats relative to tests with the face plate in

place. The consequence of this is most noticeable in the tests with a transverse bar far forward in the box. In these tests there was little or no resistance on the forward bar. In these tests there was little or no resistance on the forward bar.

2. The results of the tests conducted on the bar-mats after the overburden had been increased could be lower than expected because of insufficient consolidation time and soil rupture zones that were not obliterated during the consolidation period after the additional loading. In test #5 and #7 the pullout resistances for the initial overburdens are 15 to 25 percent higher than the pullout resistance of tests #3 and #2 for the same overburdens which were the results of "bumped" loading.

D. <u>Comparison of Theoretical, Laboratory, and Field Pullout Resistance</u>

Graphical comparisons of field pullout tests, laboratory pullout tests and theoretically calculated values are presented in Figure 25. It can be seen from these curves that there is very little correlation between laboratory, field or theoretical results.

In 1979, Bishop concluded that "primarily, the pullout resistance is a function of the overburden pressure, soil density, number of transverse wires..., and length of transverse wires..."(9). In 1980, Peterson proposed that the pullout resistance for the transverse bars can be most closely approximated by the Terzaghi-Buisman bearing capacity equation (10).

$$Q_{ult} = BcN_c + 1/2 \quad D_f B^2 N \quad + \quad D_f N_q B \qquad ......(1)$$

where:

B = footing width

c = cohesion

= unit weight of soil

D_f = height of overburden

Nc, N , and Nq are bearing Capacity factors

This equation has been modified to more closely model the total pullout resistance of the MSE bar-mats (5). The modification takes the following form.

$$Q_{ult} = L_r \times A_s + T_r \times A_p \qquad ......(2)$$

where:

Q_{ult} = Ultimate resisting capacity of bar-mat, kips

L_r = Frictional resistance of longitudinal bar per unit area, ksf

$L_r = 0.5$ ($_v$Tan +c), ksf

$_v$ = Effective overburden pressure, ksf

0 = Soil friction angle, degrees

c = Soil cohesion, ksf

0.5 = Coefficient of friction between soil and bar

A_s = Surface area of longitudinal bar, sq ft

$A_s = N_L \times \quad \times D \times L_L$

N_L = Number of longitudinal bars

D = Bar diameter, ft

L_L = Length of longitudinal bar in soil, ft

T_r = Passive resistance per unit projected area of transverse bar, ksf

$T_r = CN_c + 2/3 \quad {}_v N_q$, ksf

N_c = Bearing capacity factor

N_q = Bearing capacity factor

A_p = Projected area of transverse bar, sq ft

$A_p = N_T \times D \times T_L$, sq ft

N_T = Number of transverse bars

D = Bar diameter, ft

T_L = Length of transverse bar, ft

The theoretically calculated resistances in Figure 25 were determined using equation (2) and the soil strength parameters determined from in situ soil samples (Table 4). The soil strengths were determined from consolidated undrained triaxial tests of soil samples at field moisture contents. The graphs in Figure 25 show that the theoretically calculated values are as little as fifteen percent of either the field or laboratory test values.

It would be difficult to draw any conclusions regarding the relationship of the proposed model for pullout resistance to the actual pullout resistance. Also, the field and laboratory results are very different and it would be inappropriate to draw conclusions based on these tests.

E. Evaluation of the Design Procedure

Mechanically Stabilized Embankment (MSE) is designed to meet two design criteria; external and internal stability. Each of these criteria requires a complete analysis of the expected loading and a determination of reinforcement size to insure a stable structure. Accepted practice has been to design a reinforced structure following a procedure outlined by Henri Vidal in 1966 (1) which assumes that the backfill material is non-cohesive. One objective of this study is to evaluate the appropriateness of this procedure for walls constructed using cohesive backfill in light of the performance of the MSE walls at Baxter.

When designing an MSE wall to be constructed using cohesive backfill and acting as a retaining system for an embankment of cohesive soil, it becomes necessary to reconsider the parts of the design procedure which involve soil strength values. An "active state" analysis of the pressures within an embankment of cohesive soil could lead to the assumption that a retaining system is unnecessary. Theoretically, the embankment is able to stand unsupported for some height due to the affects of apparent cohesion (12). In the case of the embankments at Baxter, this unsupported height exceeds the recommended design height. This, however, would be an erroneous assumption because variations in strengths of cohesive soils due to soil dilation, changing water content and the ever-present concern regarding creep require that some form of mechanical support be used.

Appendix A contains calculations which have been used to check the design of both Wall 1 and Wall 3. The check is based on a soil with an internal angle of friction of 26 degrees and a cohesion intercept of 1200 psf. This is the lowest soil strength value determined during postconstruction soil testing at Wall 3 (Table 4), and is higher than most of the soil strengths determined from remolded soil samples which were used in the design of the walls (Tables 1 to 3).

1. External Stability

A basic premise of all reinforced earth structure designs is that the reinforced soil mass behaves as a total unit, or gravity wall, when determining external stability. First, the driving forces that act on the soil mass are determined, then the size of the structure, i.e. length of reinforcing elements, is determined. The reinforced mass is sized to develop a resistance to sliding and overturning which meets predetermined factors of safety, 1.5 against sliding and 2.0 against overturning.

To determine the resistance to overturning only an analysis of driving forces is necesary. The calculations to determine the resistance to sliding involve considering soil properties for quantifying both driving and resisting forces. For design purposes, the driving forces acting at the back of the reinforced mass are calculated using an "active state" stress analysis of the backfill material which does not include any consideration of the cohesive nature of the soil. The forces which resist sliding along the base of the wall are a combination of both cohesion and friction.

In the procedure used to design the walls at Baxter, lower friction and cohesion values meant that the external stability of the walls is controlled by sliding. An embedment-to-height ratio of 0.75 is required to prevent sliding along the base as compared to a 0.62 ratio to prevent overturning. However, in the design check (Appendix A) higher friction and cohesion values make prevention against overturning the controlling criteria. An embedment-to-height ratio of 0.62 is required to prevent overturning as compared to a 0.32 to prevent sliding along the base.

2. Internal Stability

The internal stability of a reinforced earth structure is dependent on two criteria. The mats must extend far enough beyond the assumed failure plane to develop resistance to pullout and the longitudinal bars must have a cross-sectional area which is large enough to carry loads created by the earth pressures that act on the face of the wall. For internal stability, as well as external stability, it is conservative to develop the earth pressures using an "active state" analysis of the backfill without cohesion.

Calculations for the design of the Baxter walls assumed that external stability controlled; and, therefore, only the size of the bars required to carry the tensile load was evaluated in the internal stability analysis. It was determined that a W7 bar had 9 cross-sectional area which was adequate to carry the load and allow for corrosion losses over the life of the structure. The backfill was assumed to be a cohesionless material with an angle of internal friction of to 20 degrees.

Calculations in Appendix A and B are a check on the adequacy of the design procedure for the internal stability of the Baxter walls. In Appendix A the length of embedment is determined to develop the required pullout resistance. In Appendix B the adequacy of the W7 bars is checked to insure they will carry the loads throughout the life of the structure.

The check of the pullout resistance is based on the assumption that the reinforcement must extend beyond the failure plane and provide mat to soil contact which can develop shearing resistance to counteract the stresses which act at the face of the wall. The pullout resistance of the mats can be calculated using the equations presented in the previous section of this report or can be determined from laboratory testing. The theoretical failure plane as defined by the Rankine failure surface is determined. Then the length of the reinforcement is specified to provide for the pullout resistance required.

In Appendix A, the pullout resistance of the reinforcement was calculated using the laboratory pullout test results from 1982 (Table 11). In the time since the Baxter design, it has become apparent that the laboratory results are generally lower than field test results (Figure 25 and Table 5). Therefore, the highest values shown in Table 10 and 11 are probably conservative. However, as determined in Appendix A, the reinforcement needs to extend only 2 feet beyond the assumed failure plane.

In Appendix A, the failure plane is located using the Rankine method. As a check on the location of the failure plane, the increase in stresses along the embedment length as shown in Figures 12 and 13 could be considered the approximate location of the developing failure plane. This plane is in fact a lesser distance from the wall face than the distance determined using the Rankine method. Therefore, the required theoretical embedment length of 12 feet for a 16-foot high wall appears to be sufficient.

Twelve feet of embedment is the specified length for the 16-foot Baxter Walls. Thus, the original design which specified a minimum 0.75 embedment to height ratio is adequate.

The check for the required cross-sectional area of the bars can be found in Appendix B. Using the corrosion loss criteria which was the standard in 1982, the W7 bars have sufficient cross-sectional area to carry the theoretical loads.
A review of Figures 10 and 11 also shows that the longitudinal reinforcement at some locations in both walls is or has been carrying loads which are as much as 10 percent greater than theoretical design loads. It should be noted, however, that the W7 bars are Grade 65 steel and are capable of carrying 65 ksi at construction and up to 36 ksi after 50 years - well in excess of the maximum stress of 9.3 ksi recorded at any location. Therefore, the W7 bars are more than adequate.

Ultimately, the only way the accuracy of the design and built-in factors of safety could be checked would be by failing the wall - an impractical solution in this instance.

F. Corrosion

A primary concern in the design of steel reinforced earth structures is the susceptibility of metal reinforcing elements to corrosion losses. Before final design can be completed a determination must be made regarding the corrosive nature of the backfill, the rate at which metal losses will occur throughout the life of the project, and the manner in which the losses will occur.

Corrosion of buried metals is measured in two ways; depth of pitting and surface area loss. At the time the MSE walls at Baxter were designed, corrosion losses were determined based on total surface losses calculated from soil resistivity vs pH curves developed at Translab in 1962 (12) and revised in 1979. In 1984, the Translab issued interim design criteria (13) in which corrosion loss is defined based on resistivity and pH for two soil groups, normal and select backfill. The 1984 interim corrosion criteria developed exclusively for steel reinforced earth structures are more conservative than those used in 1979. Appendix B contains both 1979 the corrosion loss curves and the 1984 interim design criteria.

All buried steel reinforcement must conform to the requirement that the service load on each bar at the end of 50 years may not exceed 55 percent of its yield strength. In the case of the W7 wires used for reinforcement in the MSE walls at Baxter, the maximum allowable stress per bar at the end of the 50 year service-life is 36 ksi. At the uniform 0.00075 inch per year corrosion loss rate determined from the corrosion loss curves, 56 per cent of the original cross sectional area remains after 50 years. Therefore, under peak traffic loading and maximum wall height the maximum theoretical stress per bar would be 19.9 and 22.4

ksi at Wall 1 and 3 respectively (Appendix B).

By way of comparison, the current (1984) corrosion criteria provides for 0.0013 inch per year corrosion loss rate leaving only 32 per cent of the original cross-sectional area after 50 years. This increases the theoretical stress per bar to 35.2 and 39.7 ksi at Wall 1 and 3, respectively, which means that W7 wires used at wall 3 do not meet current design criteria at the 50 year service life (Appendix B).

It should be noted, however, that throughout the three year study period, the maximum stress measured on the bar-mats was 5.6 and 9.3 ksi (Level B at Walls 1 and 3 respectively). Assuming that these stresses typify the maximum throughout the life of the structure and that the bar-mats corrode at the expected rate, the bare steel W7 wires will be stressed to only approximately 16 ksi (Wall 1) and 24 ksi (Wall 3) at the end of the 50-year service life. This is only 45 and 67 percent of the end of service life allowable stress.

G. Cost Analysis

Tables 7 through 9 are compilations of the pertinent contract bid items for the three retaining system alternatives; Mechanically Stabilized Embankment (MSE), Reinforced Earth (RE), and concrete crib walls. In each case quantities are based on the engineers estimates of quantities. Unit prices are the prices bid by the three different contractors who proposed using the alternative in their construction proposal.

Each table presents an item and cost break down. The proposed cost of construction for each wall has been broken down to per square foot of wall facing for relative comparison. With drainage systems included the per square foot of wall face costs were $28.30, $32.30, and $32.90 for MSE, RE and concrete cribwall respectively. Without drainage systems being included the per square foot of wall face costs were $25.23, $28.20, and $26.60 for MSE, RE, and concrete crib wall respectively. In the comparison of the total system costs the MSE bid represented a $60,244 cost savings for this project.

Much of the cost saving can be attributed to the decreased cost of backfill material because the MSE walls were constructed using onsite borrow material and required less excavation than the RE alternative.

IV. CONCLUSIONS

1. The stresses that developed in the bar-mats were distributed in a manner similar to expected distributions. The stresses increase with depth and decrease in the area of the berm. The stresses also show an increase near the plane which approximates the location of the Rankine active failure zone.

2. The peak measured bar-mat stresses were much more than the theoretical stresses approximated by an "active" Rankine state analysis of a cohesive soil, and generally less than those approximated by an "active" Rankine analysis using only the friction angle of the soil.

3. Peak bar-mat tensile stresses (assumed a direct indicator of lateral earth pressures) occurred early in the construction period and decreased with time signifying early history compaction stress domination. After three years, bar-mat stresses became compressive at some instrumented points.

4. Measurements of lateral pressures at the wall face by soil pressure cells are indicators only of localized conditions and are not reflections of the pressures that develop in the reinforced soil mass.

5. Horizontal and vertical movements at the wall face of both Wall 1 and Wall 3 were insignificant.

6. It is impossible to draw any conclusions about movements of the embankment because no precise means of observing movement was provided. However, visual inspection of the roadway surface suggests that any movement that may have occurred is insignificant as no pavement distress has been noted.

7. The construction of MSE walls using low quality, onsite backfill can be more economical than constructing other types of retaining systems which require standard, or high quality, backfill material. The cost of constructing MSE walls for this project represented an 11% savings compared to the next lowest bid, which would have used the patented RE system.

8. The design criteria used for these walls are adequate based on their performance.

9. Large scale pullout tests in both the laboratory and field suggest that existing load capacity models are extremely conservative.

10. No correlation could be made between field, laboratory and theorectical pullout resistance values.

11. The pullout resistance of the dummy bar-mats in the field tests increased with overburden height to approximately 8 feet and decreased with additional overburden. These results are contrary to other investigations and should be researched to determine if they were an anomaly or a phenomenon of the bar-mat configuration in cohesive soils.

V. RECOMMENDATIONS

Mechanically Stabilized Embankment Systems should be utilized to reduce the cost of retaining wall construction. Where MSE walls can be constructed using available onsite materials they should be included in the project bid package.

Research should be implemented to determine the adequacy of the existing corrosion loss criteria. Long term research should be implemented which will assess the corrosion rate for the steel reinforcing elements of MSE systems in various backfill materials.

Further research should be implemented to examine the relationship between laboratory pullout test data and field pullout test data in various backfill materials.

Further research is necessary before any changes are made in the existing design procedure for MSE walls.

VI. IMPLEMENTATION

Mechanically Stabilized Embankment (MSE) walls have been constructed in California along I-5 at Dunsmuir (3), along I-80 at Baxter (4), along Rte 192 at Montecito (1), along Rte 140 in Mariposa County (1), and along Rte 99 at Delhi (1). Of these only the walls at Baxter have used low quality backfill. The finding of this research project will guide future research on, and design of, MSE walls and other forms of reinforced earth structures.

VII. REFERENCES

1. Vidal, Henri, "Reinforced Earth (A New Material for Public Works)", Annales de L'Institut Technique du Batiment et des Travaux Publics, Vol. 19 Nos. 223-224, Jul - Aug 1966 (Also published as Translation No. 67-12, Dec 1967, U.S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.)

2. Chang, Jerry C., Joseph B. Hannon and Raymond A. Forsyth, "Pull Resistance and Interaction of Earthwork Reinforcement and Soil", TRB, Transportation Research Record 640, pp. 1-7 1977.

3. Forsyth, Raymond A., "Alternative Earthwork Reinforcements", Symposium on Earth Reinforcement, American Society of Civil Engineers, pp. 358-370, April 1978.

4. Hannon, Joseph B., Raymond A. Forsyth and Jerry C. Chang, "Field-Performance Comparison of Two Earthwork Reinforcement Systems", TRB, Transportation Research Record 872, pp. 24-32, 1983.

5. Jackura, Kenneth A., "Results of Minor Research on Bar-Mat Pullout Test", Final Report CA/TL - 84/08, 643364, Transportation Laboratory, Caltrans DOT, May 1984.

6. Al-Yassin, Zaynab, "Laboratory Pullout Test Results on VSL Retained Earth", VSL Corporation, 1980.

7. Chang, Jerry C., Joseph B. Hannon and Raymond A. Forsyth, "Field Performance of Earthwork Reinforcement", Final Report CA/TL-81/06, 462155, Transportation Laboratory, Caltrans, Appendix, November 1981.

8. Nielsen, Mark R. and Loren R. Anderson, "Pullout Resistance of Wire Mats Embedded in Soil", for the Hilfiker Company, April 1984.

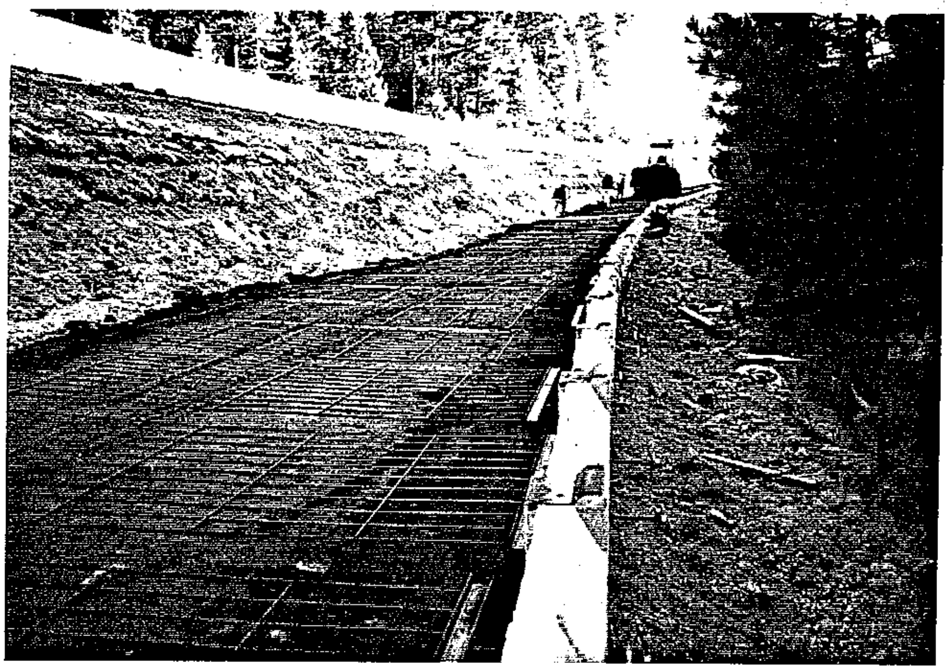
9. Peterson Larry W. , "Pullout Resistance of Welded Wire Mesh Embedded in Soil", Masters Thesis, Utah State University, 1980.

10. Bishop, Jerold A., "Evaluation of a Welded Wire Retaining Wall", Masters Thesis, Utah State University, 1979.

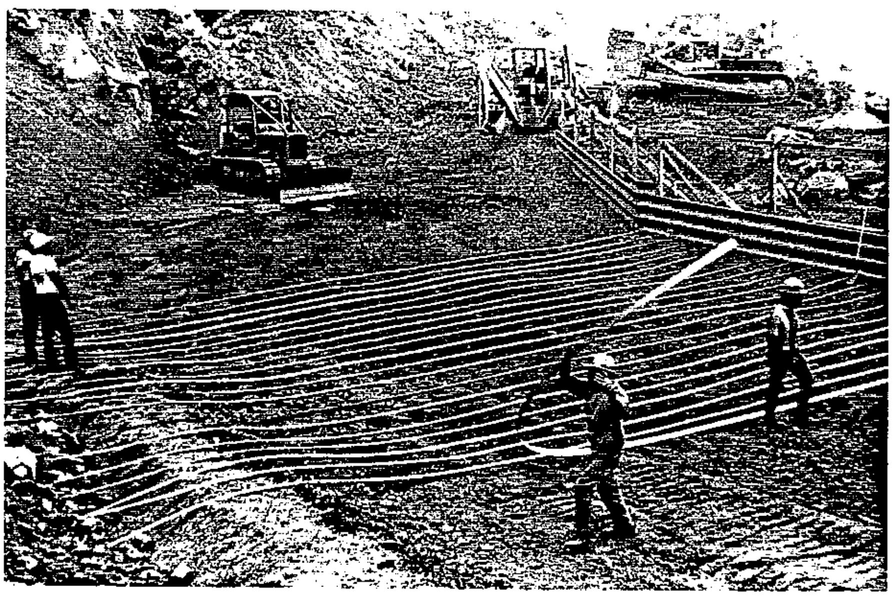
11. Lambe, William T., and Robert V. Whitman, Soil Mechanics, Si Version, John Wiley & Sons, New York, pp. 328-351, 1979

12. Beaton, J. L.,and R. Stratfull, "Field Test for Estimating the Service Life of Corrugated Metal Pipe Culverts", Proceedings, 41st Annual Meeting Highway Research Board, pp. 225-272, 1962.

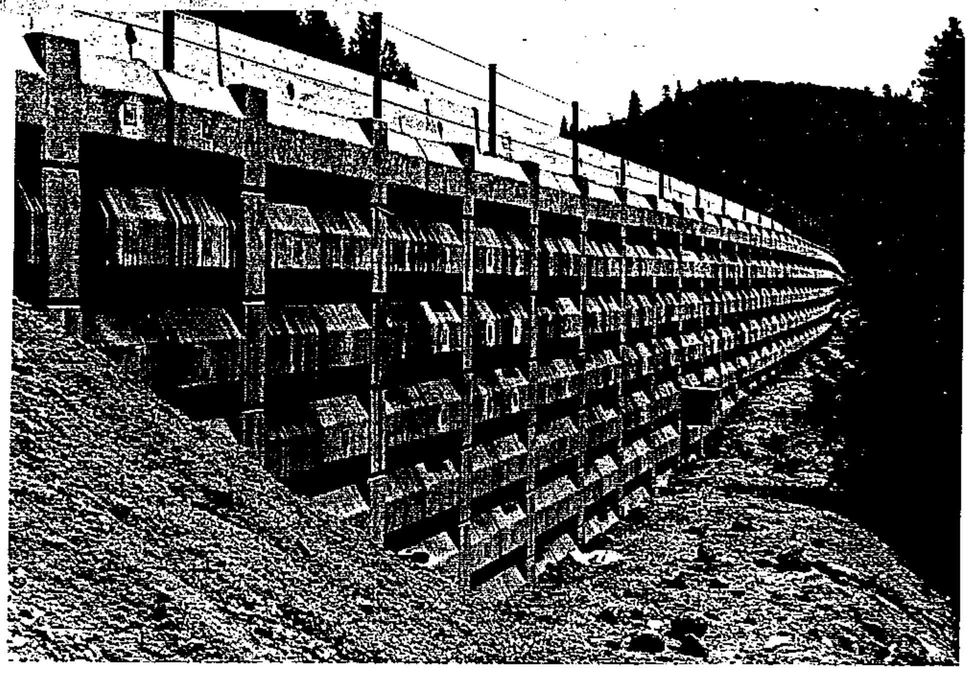
VIII. A <u>PHOTOGRAPHS</u>



1 Mechanically Stabilized Embankment
Reinforcing Bar-mats

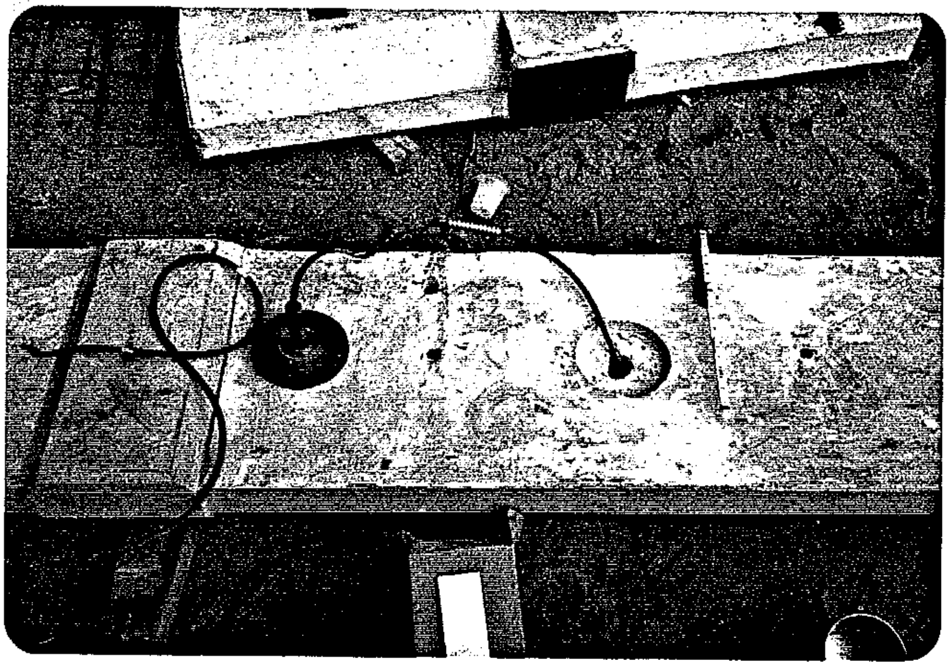


2 Reinforced Earth Reinforcing Steel Strips

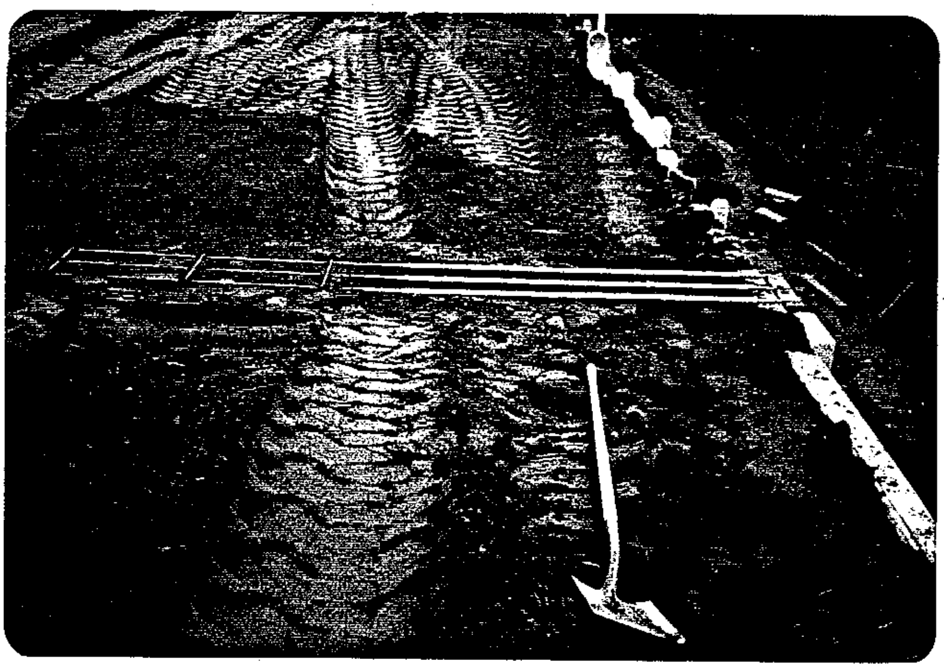


3 A Completed MSE Wall at Baxter, California

4 Installation of the Permeable Blanket
After MSE Wall Completion



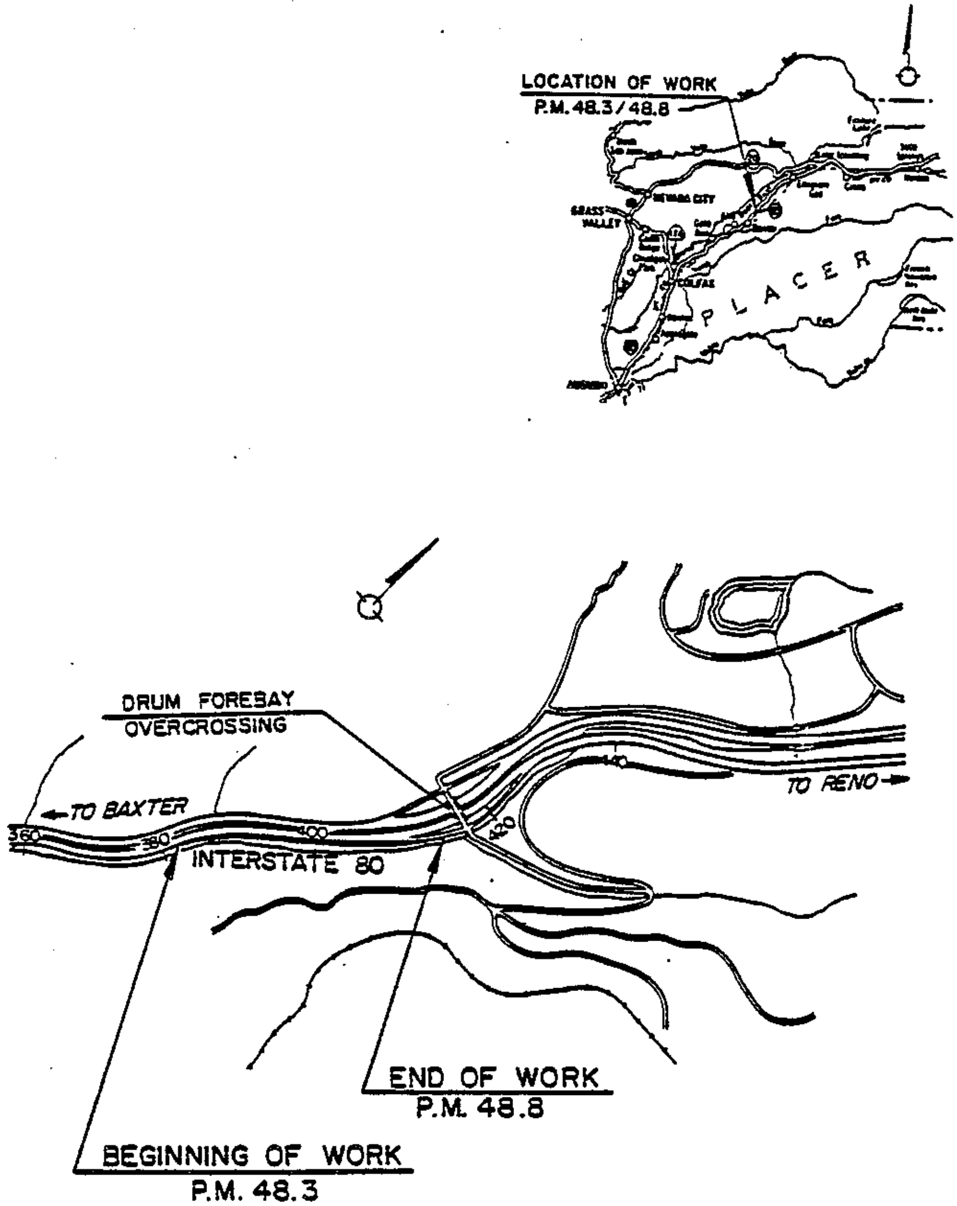
5 Pressure Cells in Face Panels



6 Dummy Bar-mats in Backfill

7 Boom Arragnement for Pullout Testing Equipment

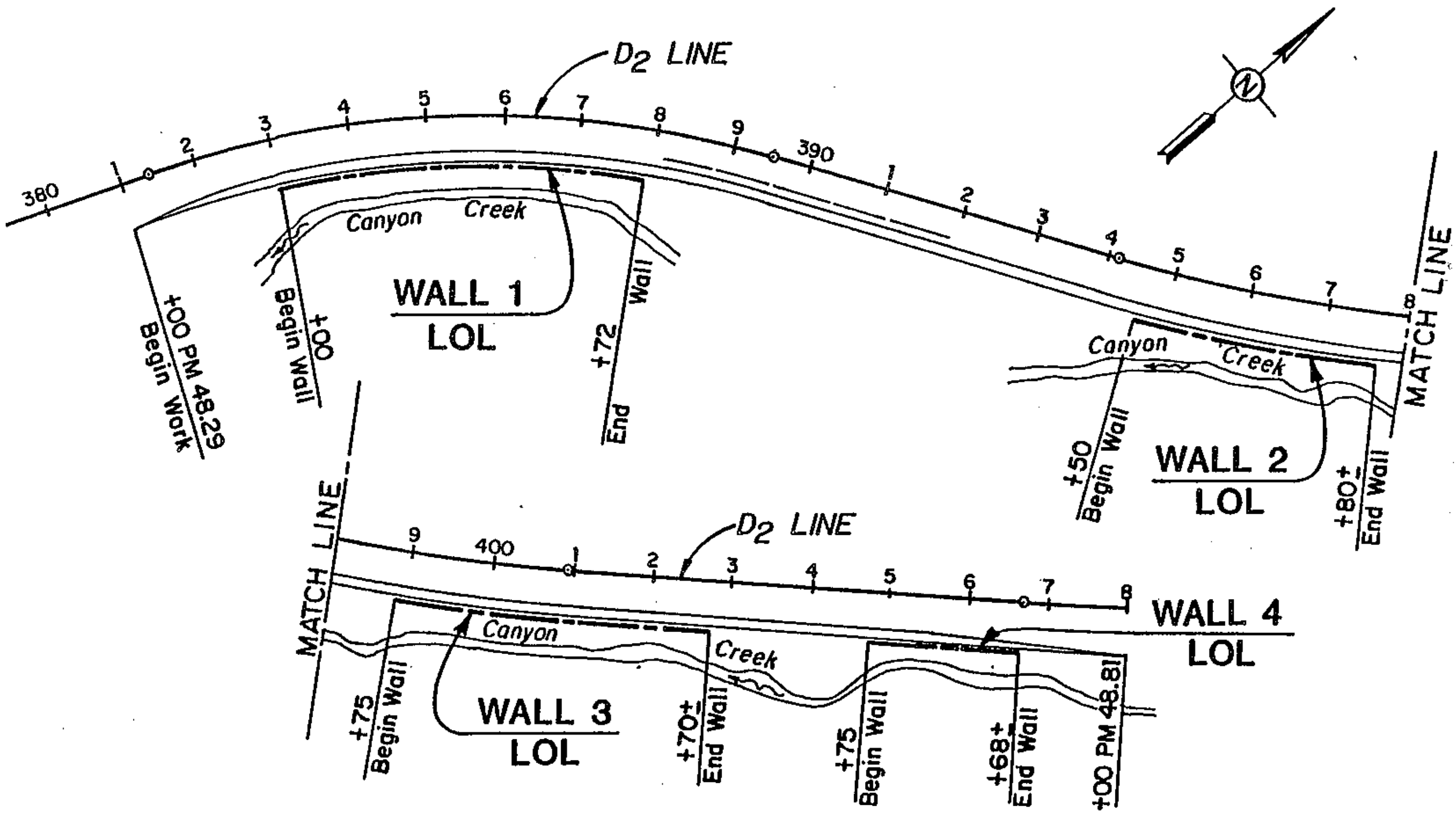
VIII. B. FIGURES



IN PLACER COUNTY ABOUT 2.4 MILES EAST OF BAXTER
FROM 0.7 MILE WEST TO 0.2 MILE WEST
OF DRUM FOREBAY OVERCROSSING

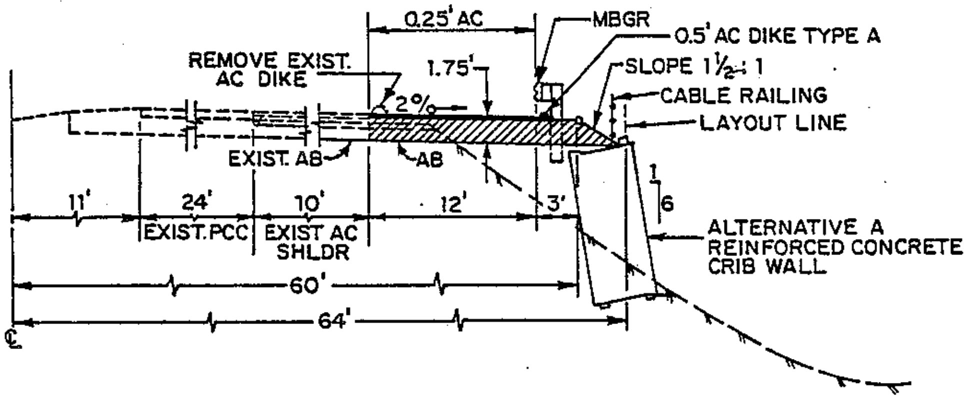
PROJECT LOCATION

FIGURE 1

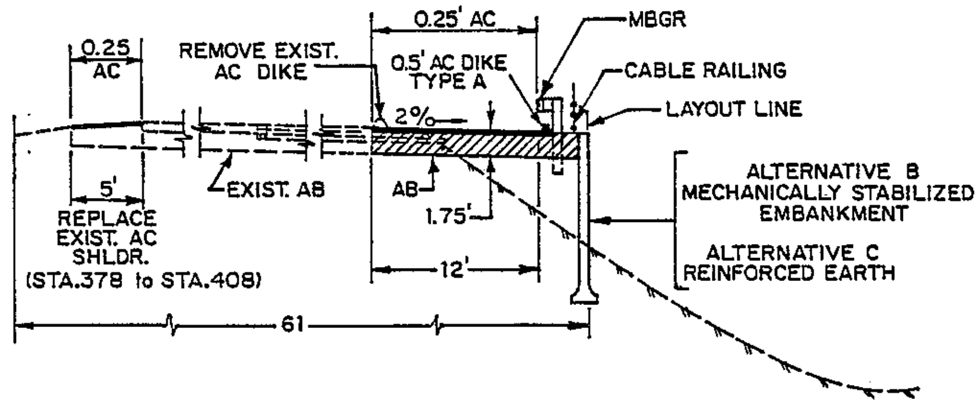


WALL LOCATIONS

FIGURE 2



Concrete Crib Wall

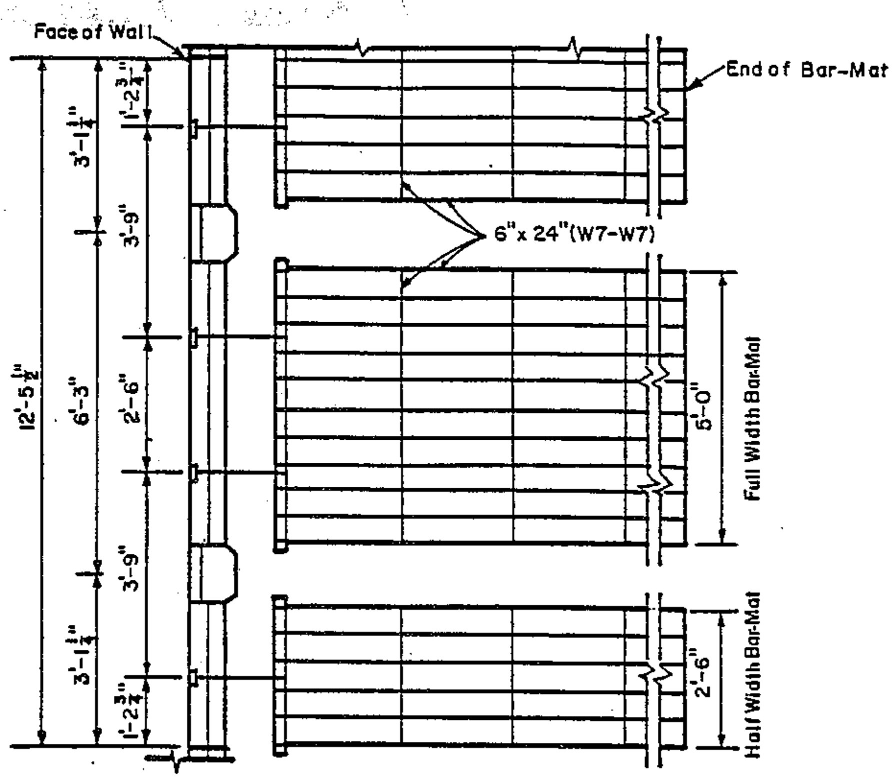


Mechanically Stabilized Embankment and Reinforced Earth

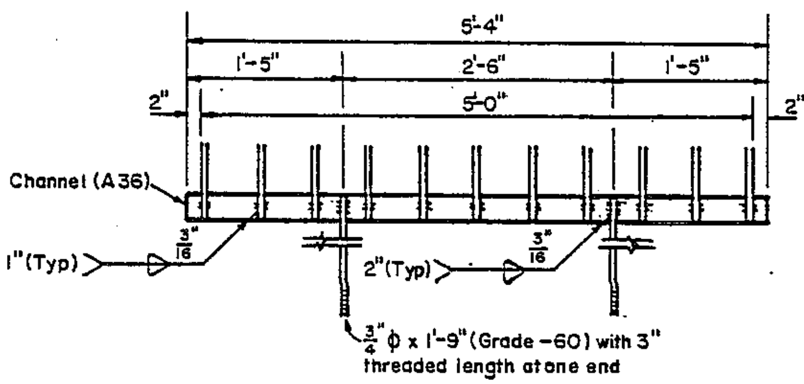
TYPICAL CROSS SECTIONS

RETAINING WALL ALTERNATIVES

FIGURE 3



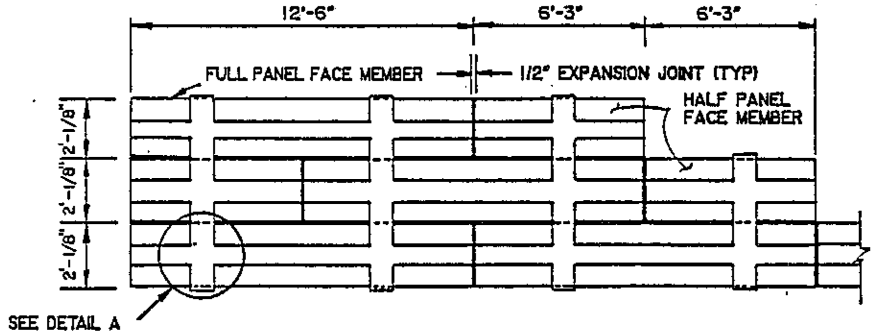
BAR-MATS



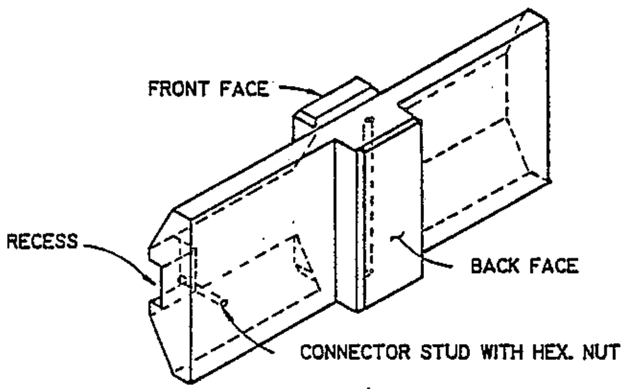
BAR-MAT CONNECTOR DETAILS

BAR-MAT & BOLT CONNECTION DETAILS

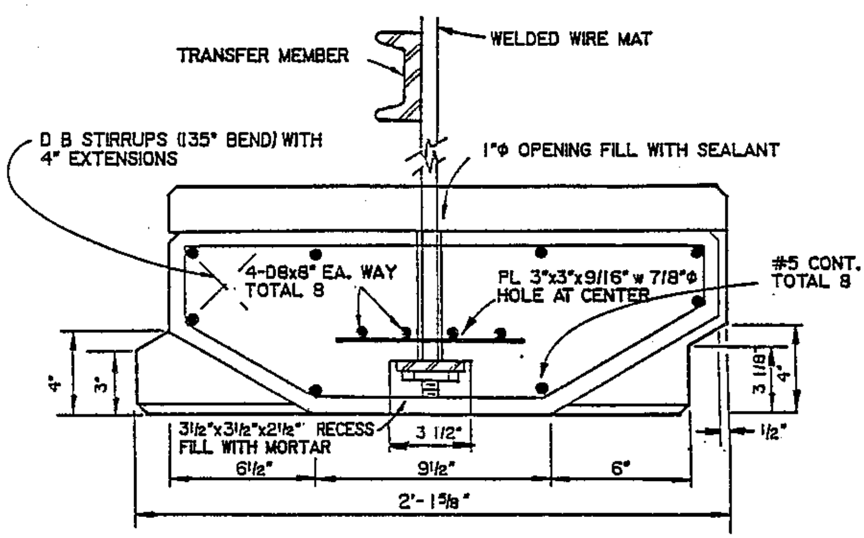
FIGURE 4



TYPICAL STACKING ARRANGEMENT



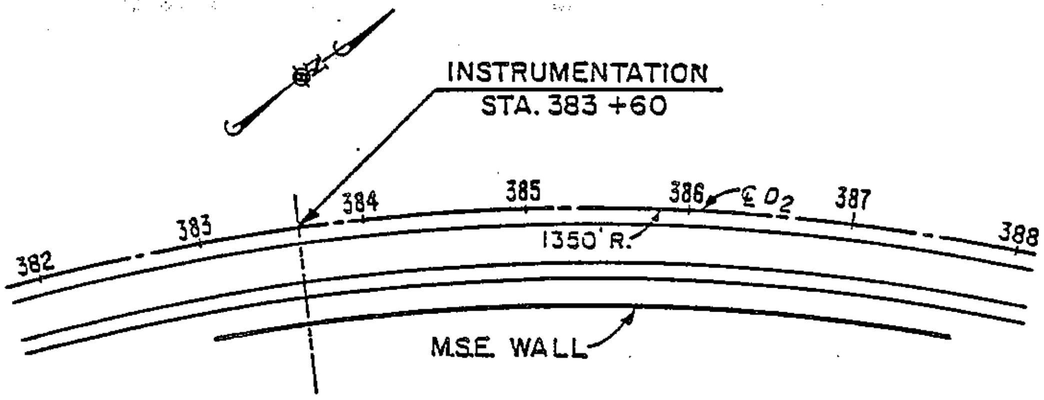
DETAIL A



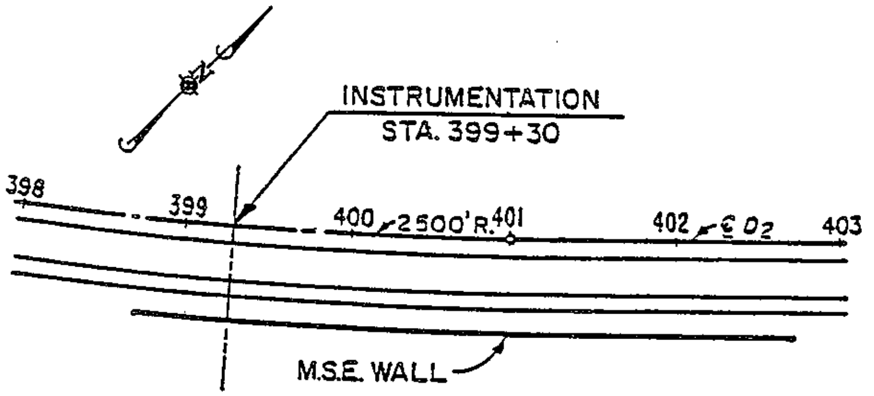
TYPICAL CROSS SECTION

CONCRETE FACE PANELS

FIGURE 5



WALL 1

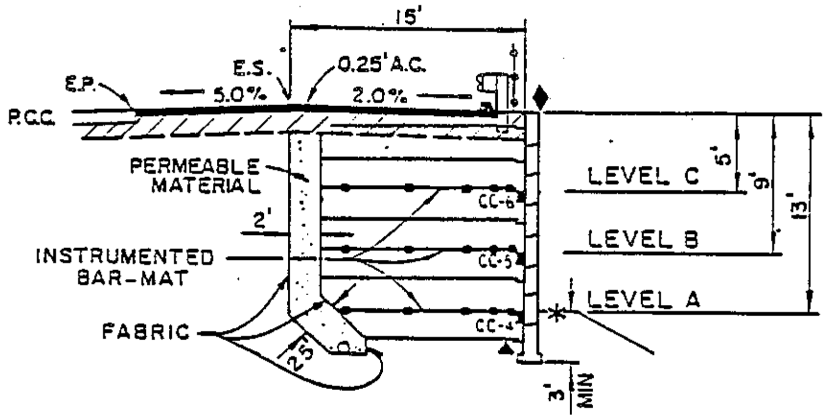


WALL 3

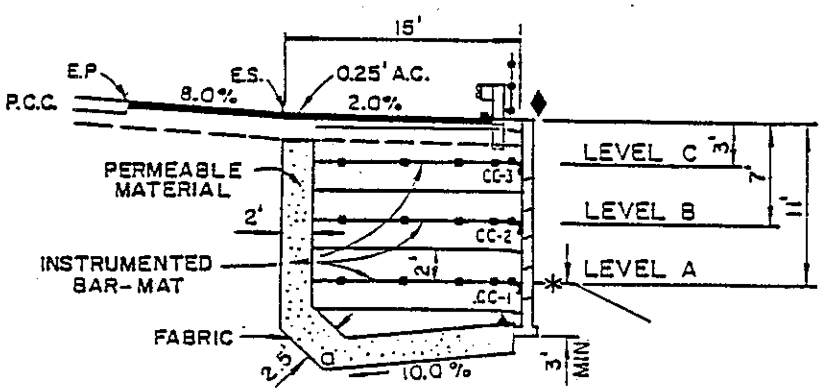
INSTRUMENTATION LOCATIONS

FIGURE 6

LEGEND		
CODE	SYMBOL	DESCRIPTION
SG	■	STRAIN GAGES (2 ea.)
CC	▮	CONCRETE PRESSURE CELLS
PP	♦	PLUMB POINTS
RM	✳	REFERENCE MONUMENTS
P	▲	PIEZOMETERS



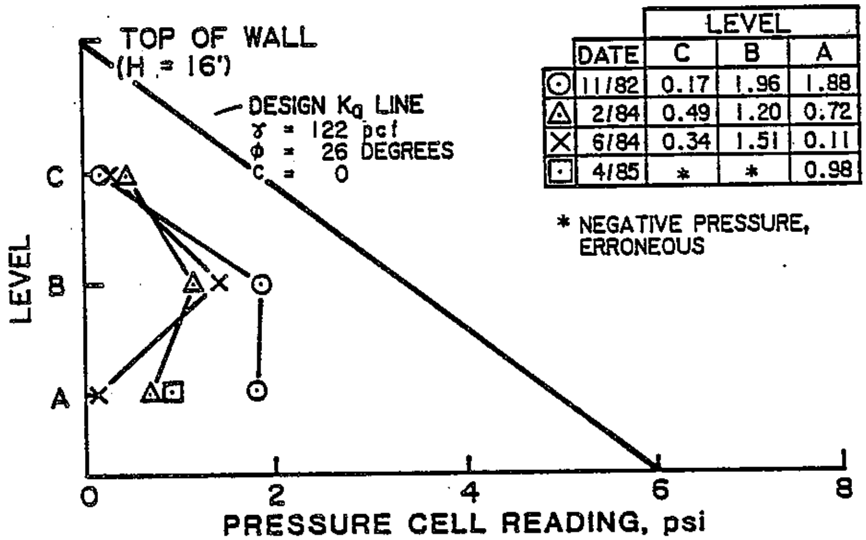
WALL 3, STA. 399+30(H=16')



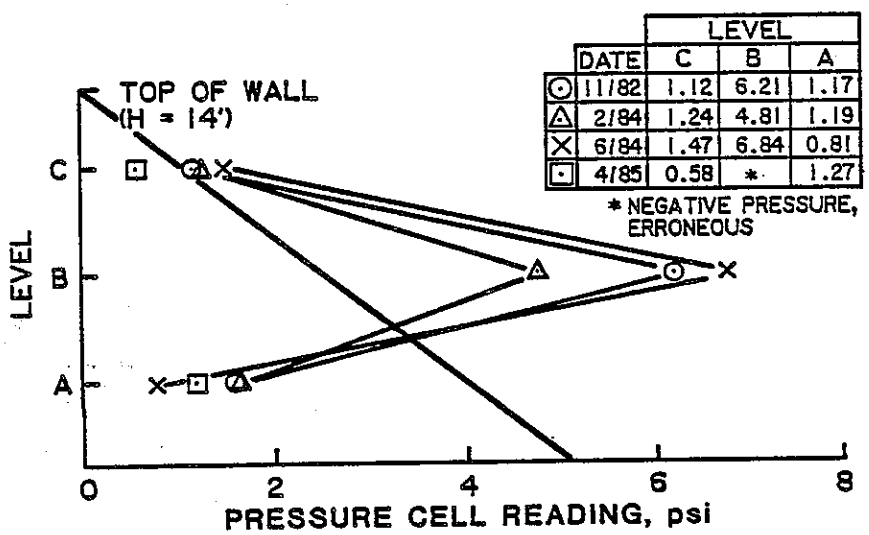
WALL 1, STA. 383+60 (H=14')

INSTRUMENTATION SECTIONS

FIGURE 7



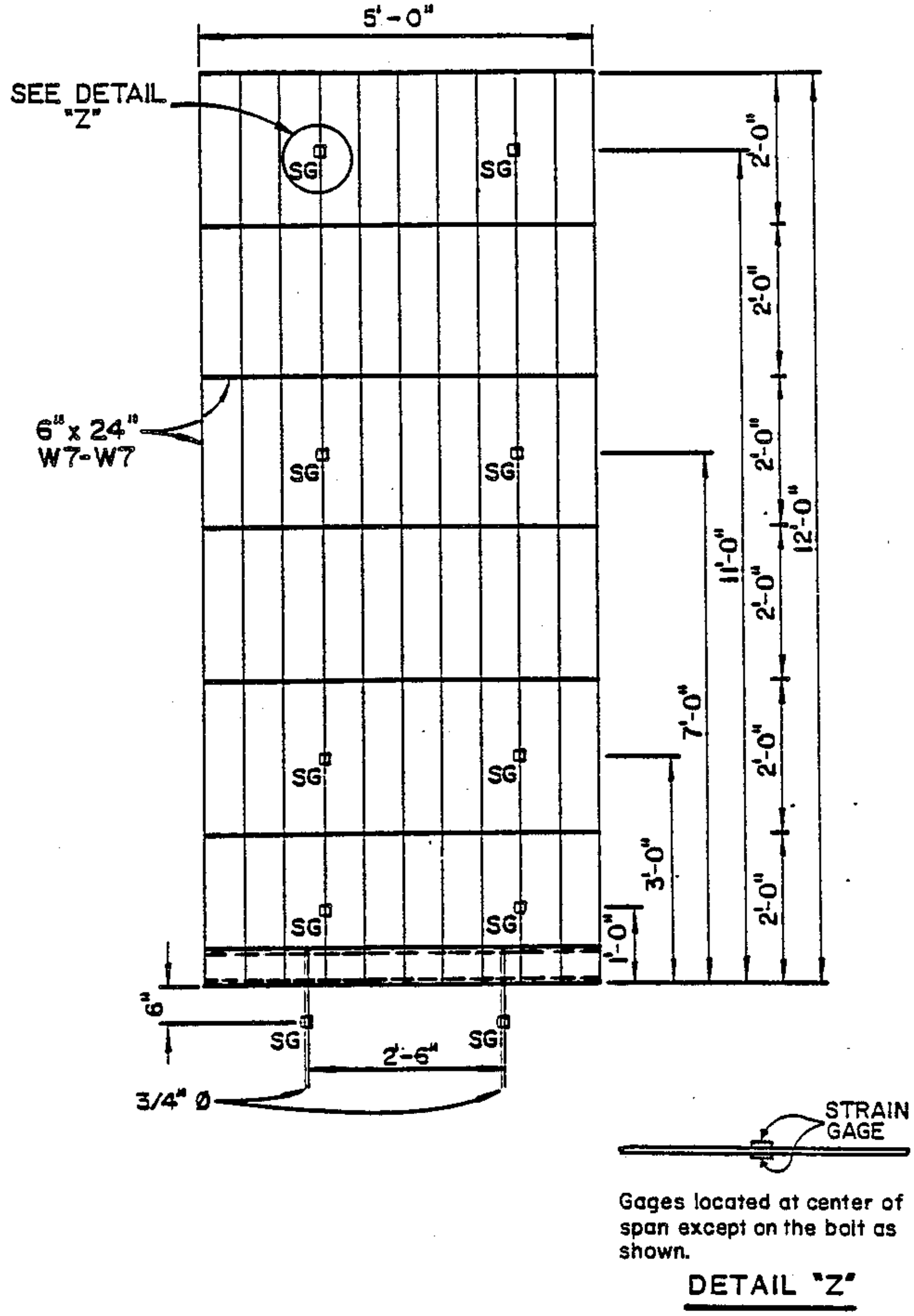
WALL 3



WALL 1

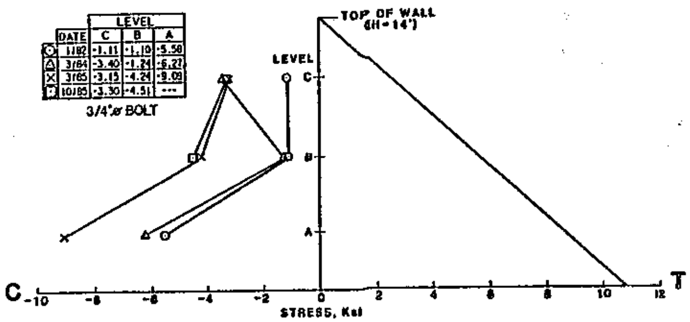
PRESSURE CELL READINGS ON WALLS

FIGURE 8



LOCATION OF STRAIN GAGES ON INSTRUMENTED BAR-MATS

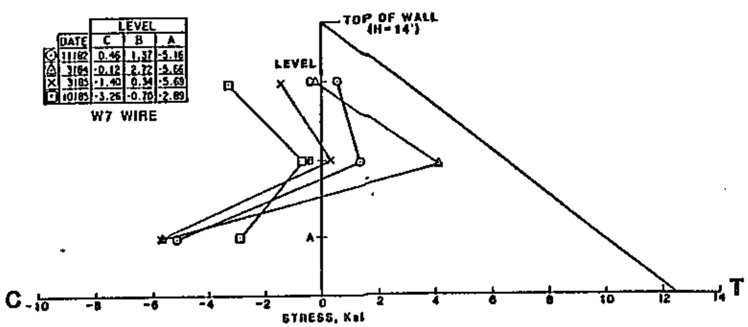
FIGURE 9



	DATE	LEVEL C	LEVEL B	LEVEL A
⊙	11/82	-1.11	-1.10	-5.58
△	3/84	-3.40	-1.24	-6.27
X	3/85	-3.15	-4.24	-9.09
⊡	10/85	-3.30	-4.51	---

3/4"ø BOLT

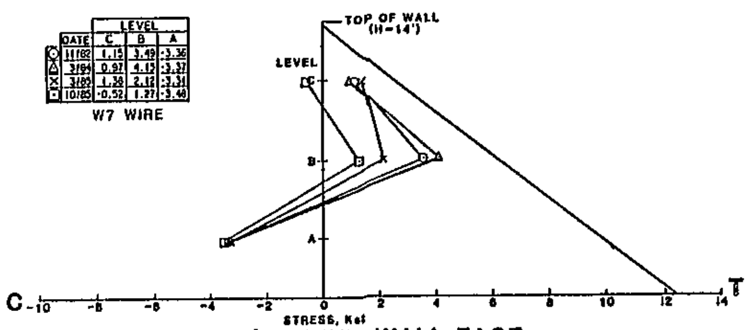
ROW 1 - 6" BEHIND WALL FACE



	DATE	LEVEL C	LEVEL B	LEVEL A
⊙	11/82	0.46	1.37	-5.16
△	3/84	-0.12	2.72	-5.66
X	3/85	-1.40	0.34	-5.69
⊡	10/85	-3.26	-0.70	-2.89

W7 WIRE

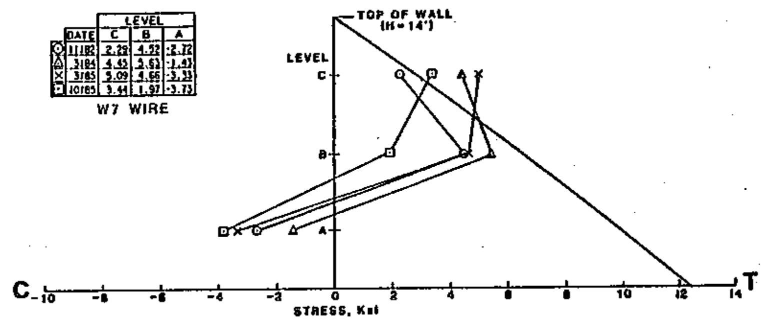
ROW 2 - 2' BEHIND WALL FACE



	DATE	LEVEL C	LEVEL B	LEVEL A
⊙	11/82	1.15	3.49	-3.36
△	3/84	0.97	4.15	-3.37
X	3/85	1.38	2.12	-3.31
⊡	10/85	-0.52	1.27	-3.48

W7 WIRE

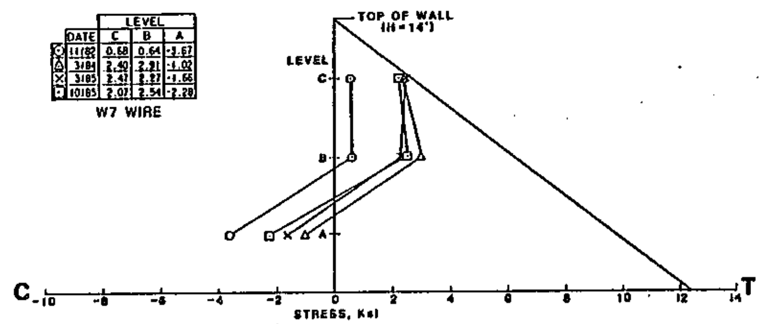
ROW 3 - 4' BEHIND WALL FACE



	DATE	LEVEL C	LEVEL B	LEVEL A
⊙	11/82	2.29	4.52	-2.72
△	3/84	4.45	5.63	-1.43
X	3/85	5.09	4.66	-3.33
⊡	10/85	3.44	1.97	-3.73

W7 WIRE

ROW 4 - 8' BEHIND WALL FACE



	DATE	LEVEL C	LEVEL B	LEVEL A
⊙	11/82	0.68	0.64	-3.67
△	3/84	2.40	2.21	-1.02
X	3/85	2.47	2.27	-1.66
⊡	10/85	2.07	2.54	-2.28

W7 WIRE

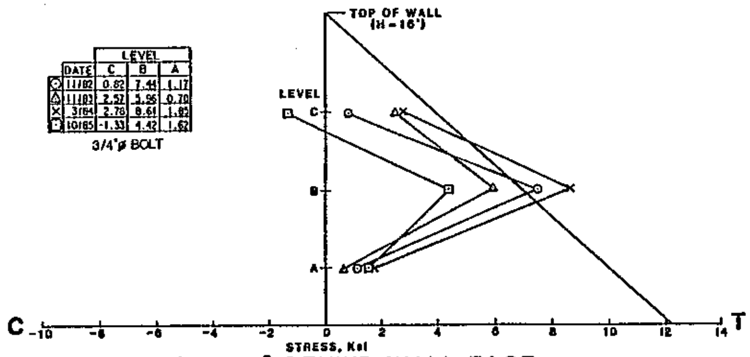
ROW 5 - 12' BEHIND WALL FACE

T = Tension (+)
C = Compression (-)

Initial Reading at
2 ft. Overburden

WALL 1
STRESS IN BAR-MATS

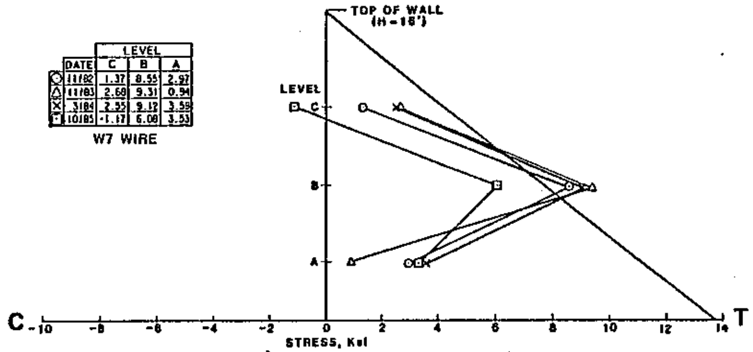
FIGURE 10



	DATE	LEVEL C	LEVEL B	LEVEL A
○	11/82	0.82	7.44	1.17
△	11/83	2.52	5.96	0.70
×	3/84	2.78	8.61	1.85
⊡	10/85	-1.33	4.42	1.62

3/4"ø BOLT

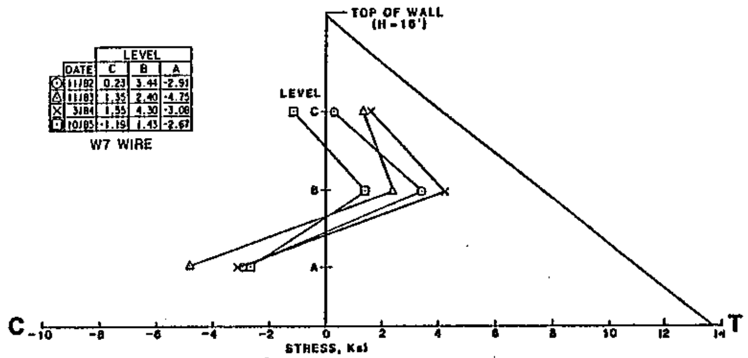
ROW 1 – 6" BEHIND WALL FACE



	DATE	LEVEL C	LEVEL B	LEVEL A
○	11/82	1.37	8.55	2.97
△	11/83	2.68	9.31	0.94
×	3/84	2.55	9.12	3.59
⊡	10/85	-1.17	6.08	3.53

W7 WIRE

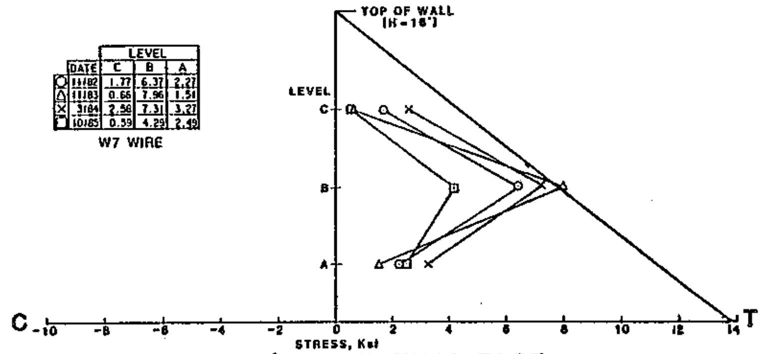
ROW 2 – 2' BEHIND WALL FACE



	DATE	LEVEL C	LEVEL B	LEVEL A
○	11/82	0.23	3.44	-2.91
△	11/83	1.35	2.40	-4.75
×	3/84	1.55	4.30	-3.08
⊡	10/85	-1.19	1.43	-2.67

W7 WIRE

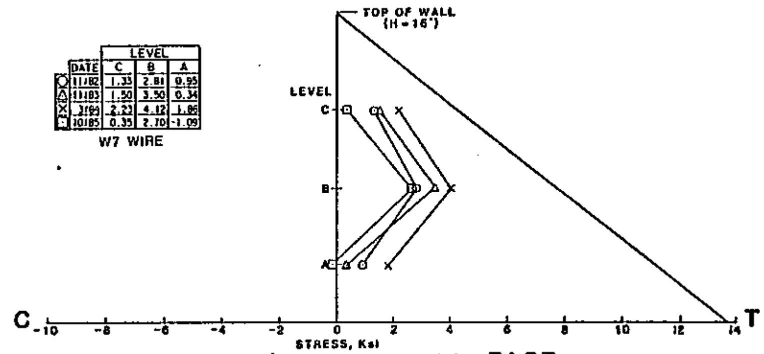
ROW 3 – 4' BEHIND WALL FACE



	DATE	LEVEL C	LEVEL B	LEVEL A
○	11/82	1.77	6.37	2.27
△	11/83	0.66	7.96	1.51
×	3/84	2.58	7.31	3.27
⊡	10/85	0.59	4.29	2.49

W7 WIRE

ROW 4 – 8' BEHIND WALL FACE



	DATE	LEVEL C	LEVEL B	LEVEL A
○	11/82	1.33	2.81	0.95
△	11/83	1.50	3.50	0.34
×	3/84	2.23	4.12	1.86
⊡	10/85	0.35	2.70	-1.09

W7 WIRE

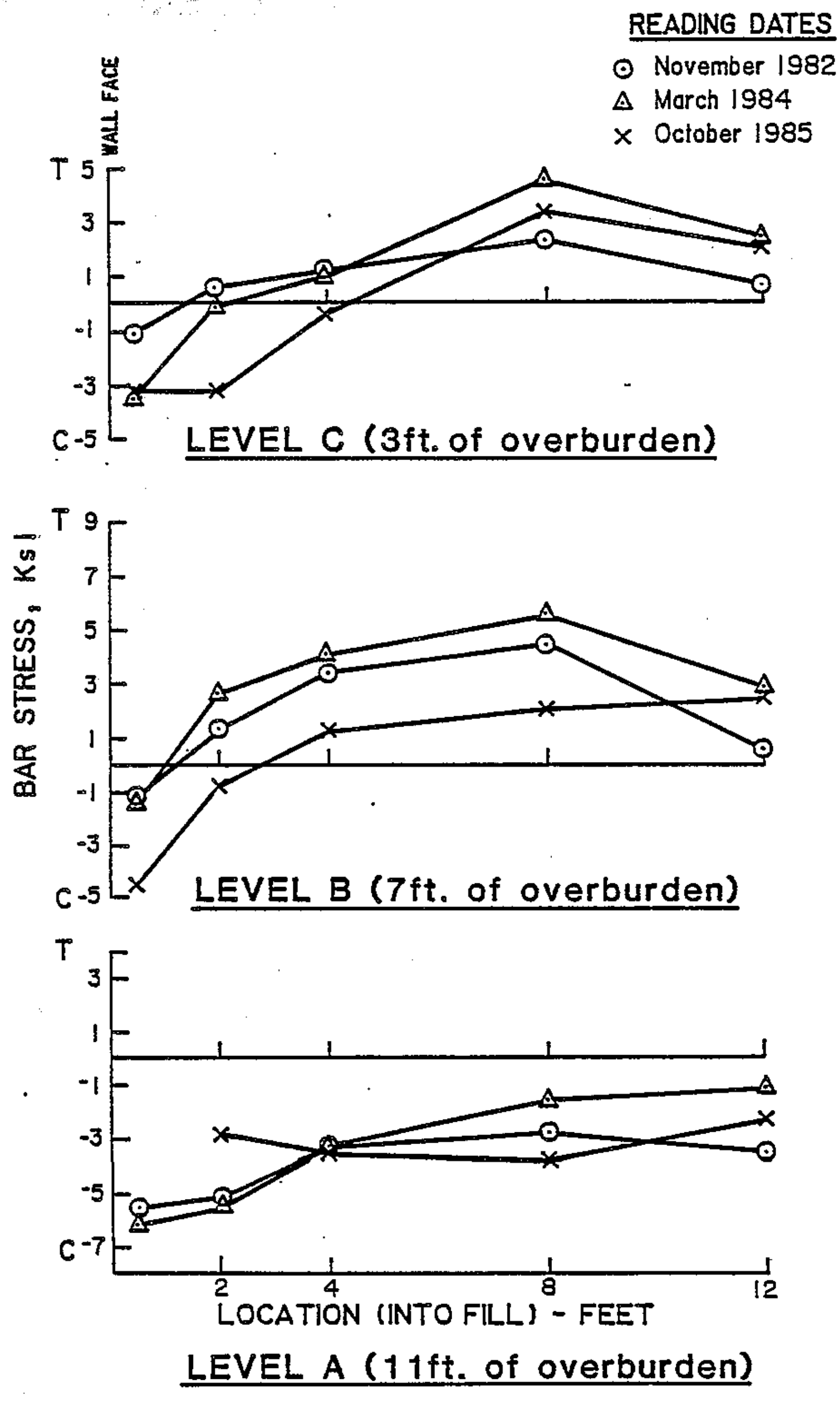
ROW 5 – 12' BEHIND WALL FACE

T = Tension (+)
C = Compression (-)

Initial Reading at
2 ft. Overburden

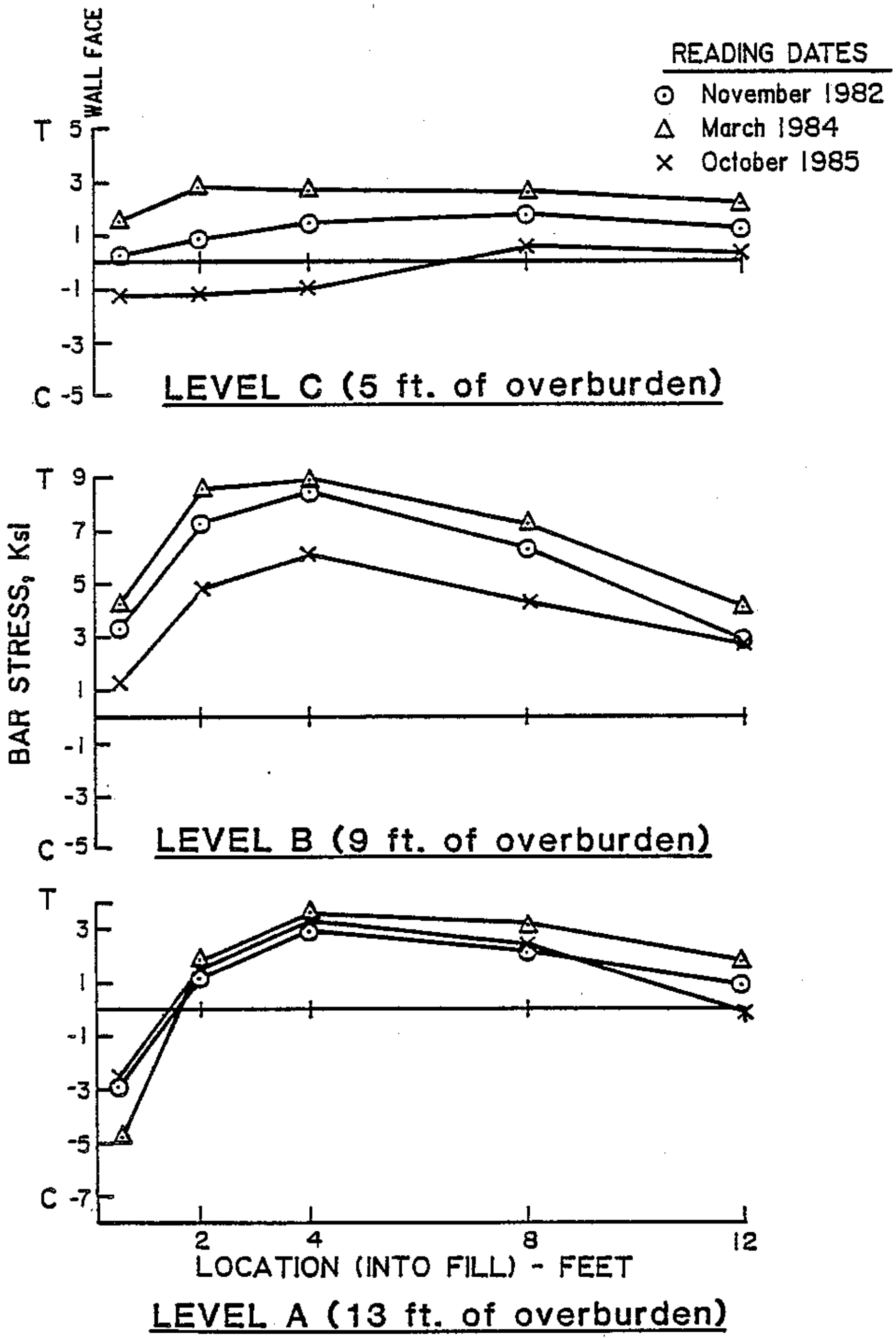
WALL 3
STRESS IN BAR-MATS

FIGURE 11



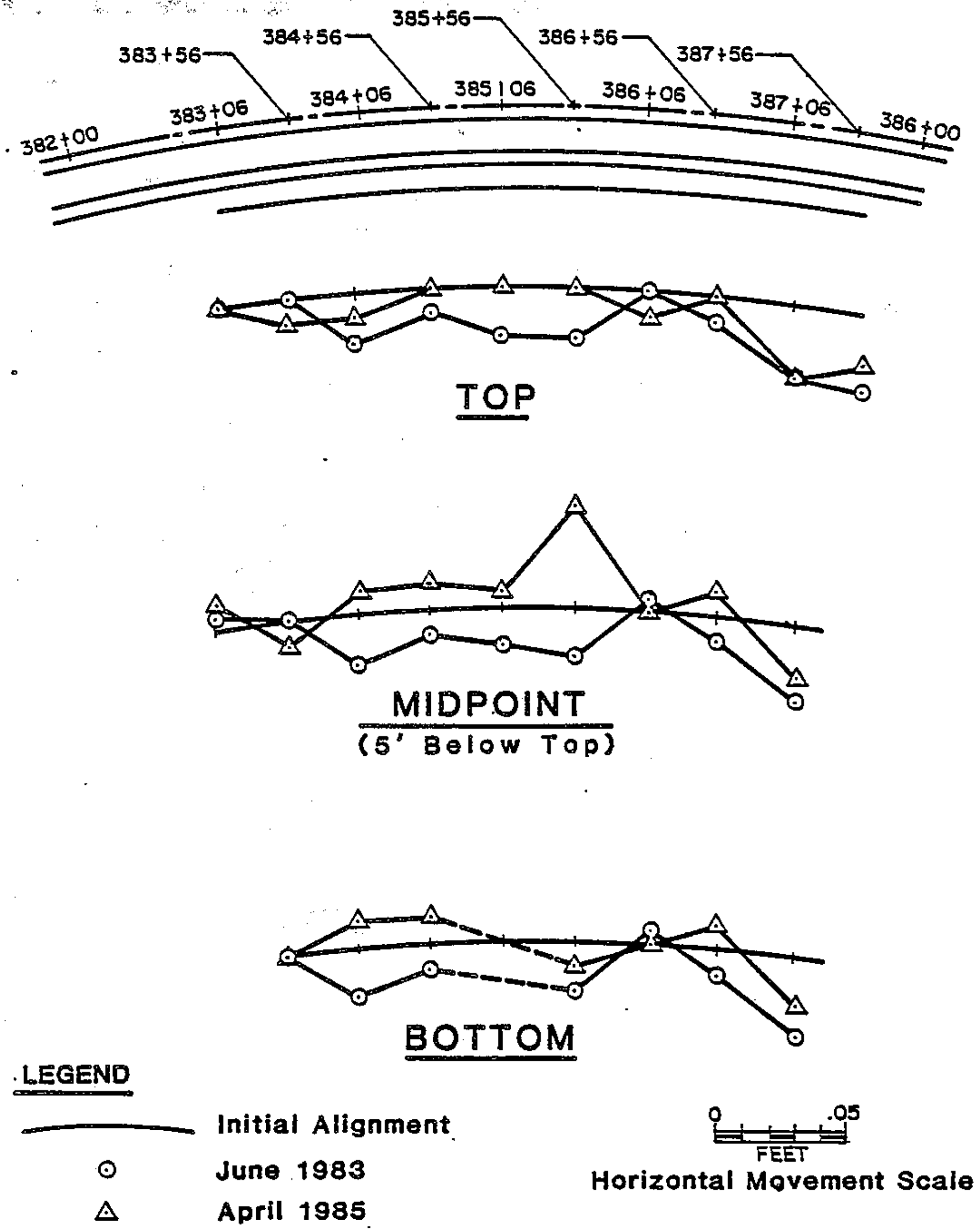
WALL 1, STRESS IN BAR-MATS

FIGURE 12



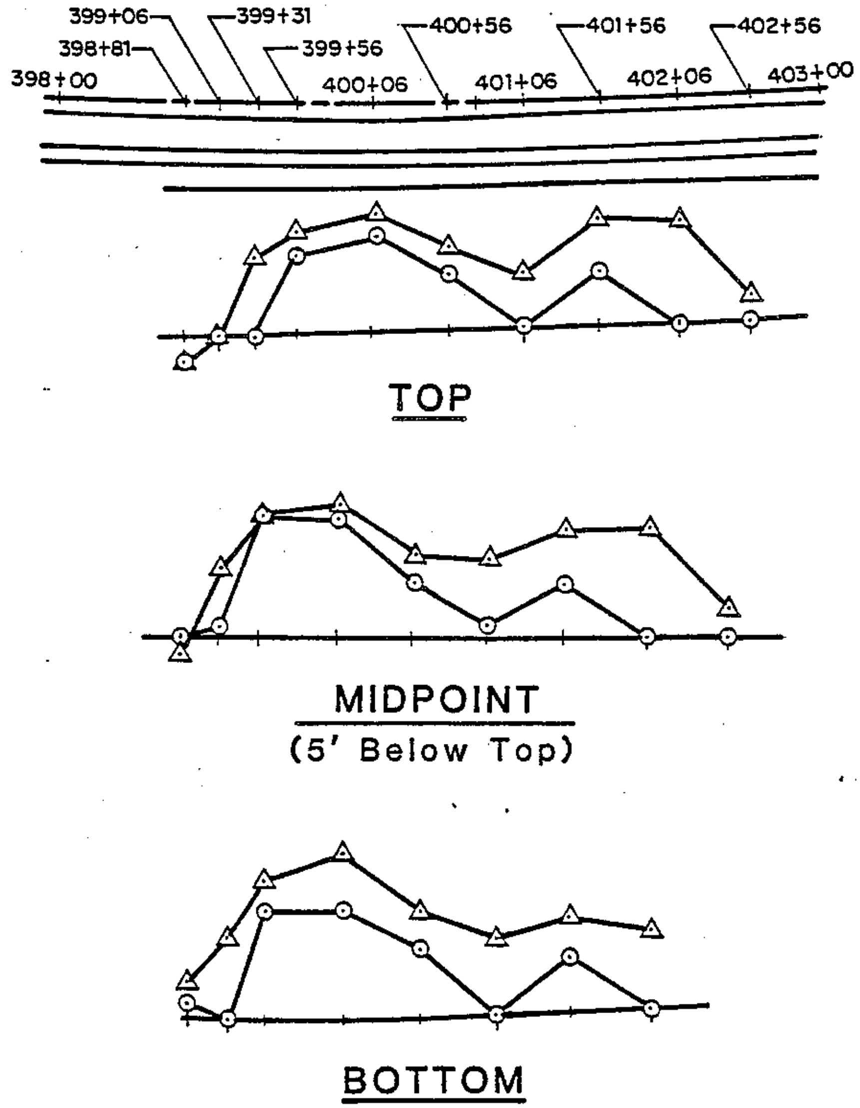
WALL 3, STRESS IN BAR-MATS

FIGURE 13



WALL 1
HORIZONTAL WALL MOVEMENT

FIGURE 14



LEGEND

Initial Alignment

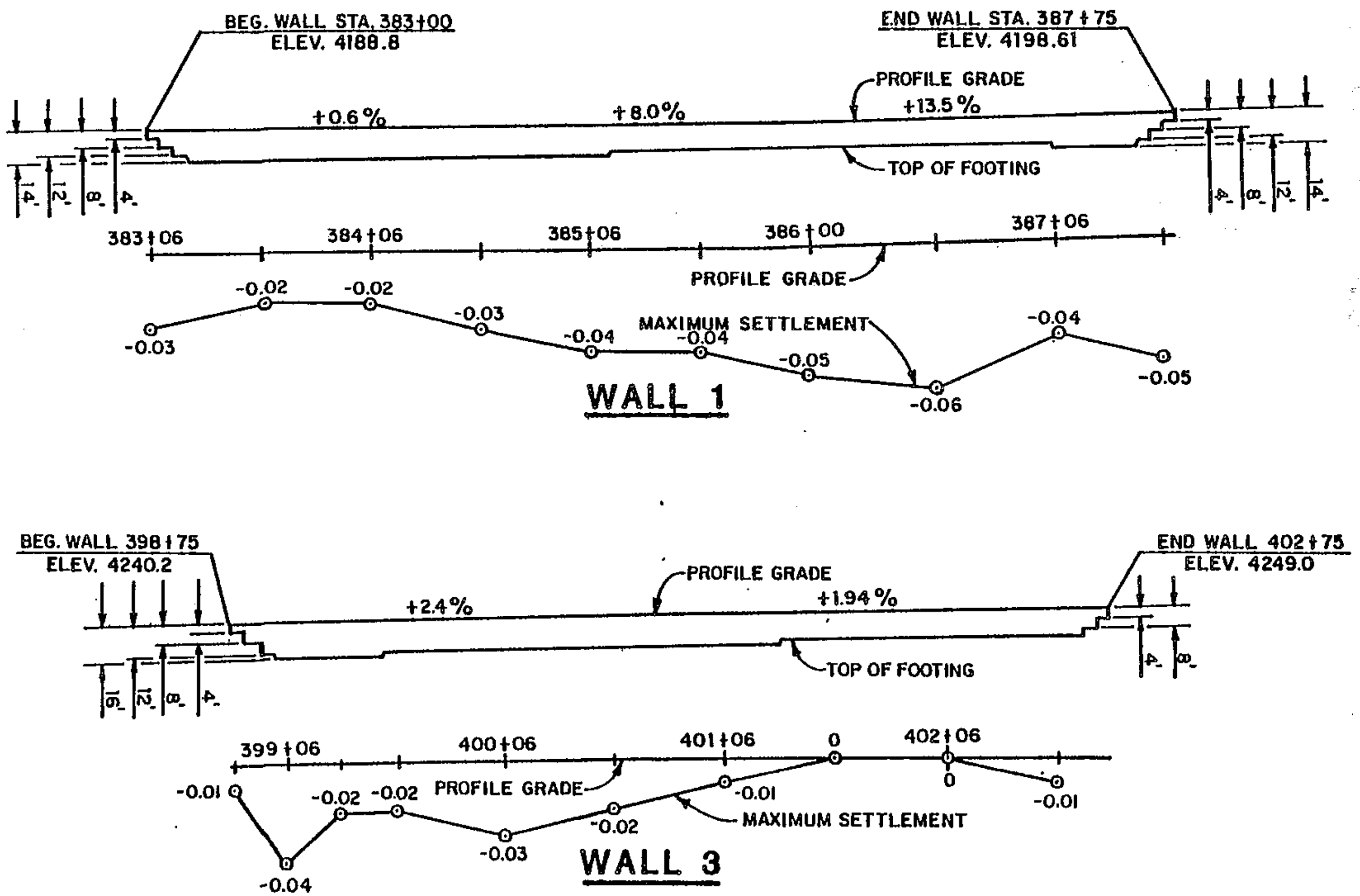
⊙ June 1983

△ April 1985

Horizontal Movement Scale

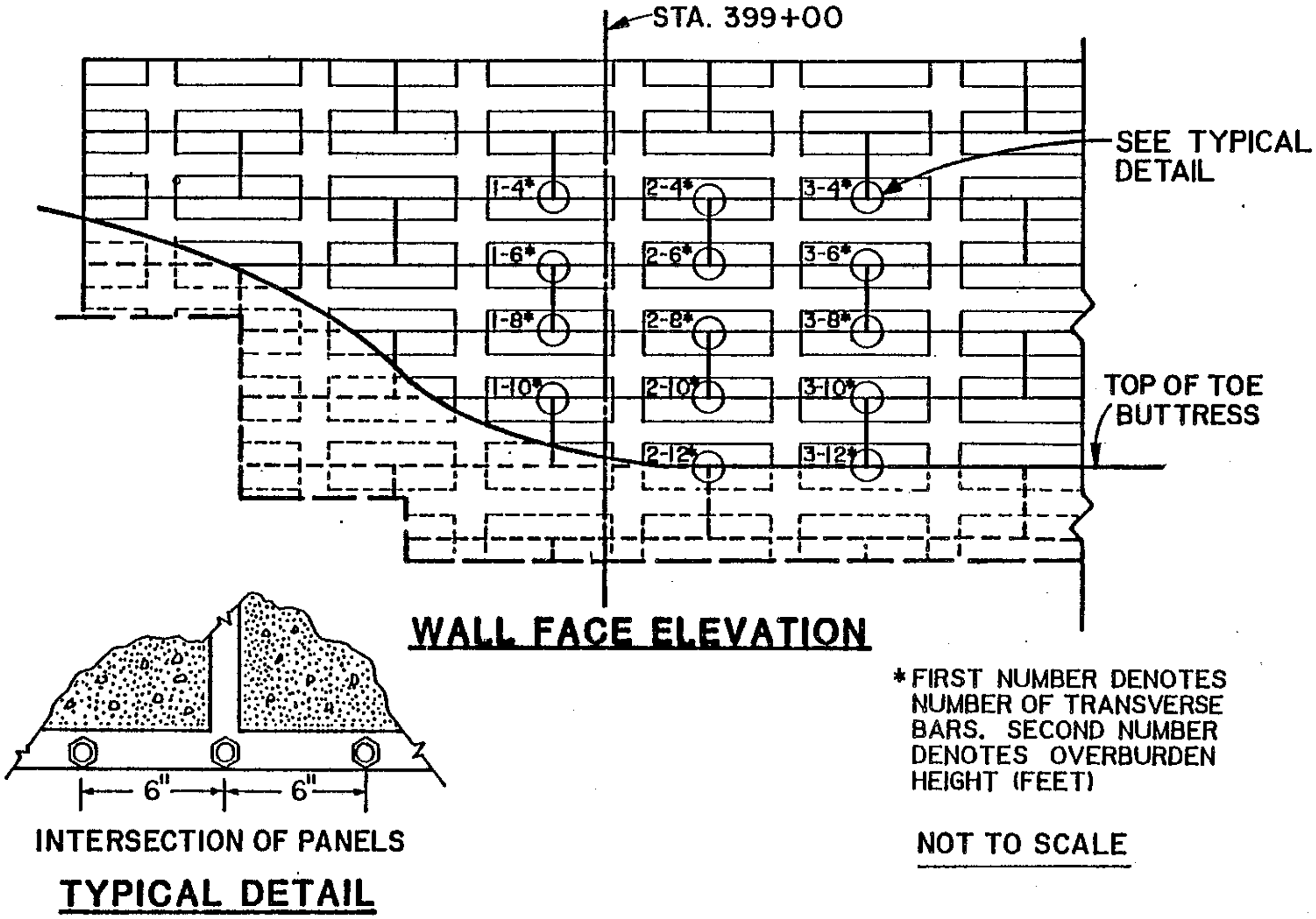
WALL 3
HORIZONTAL WALL MOVEMENT

FIGURE 15



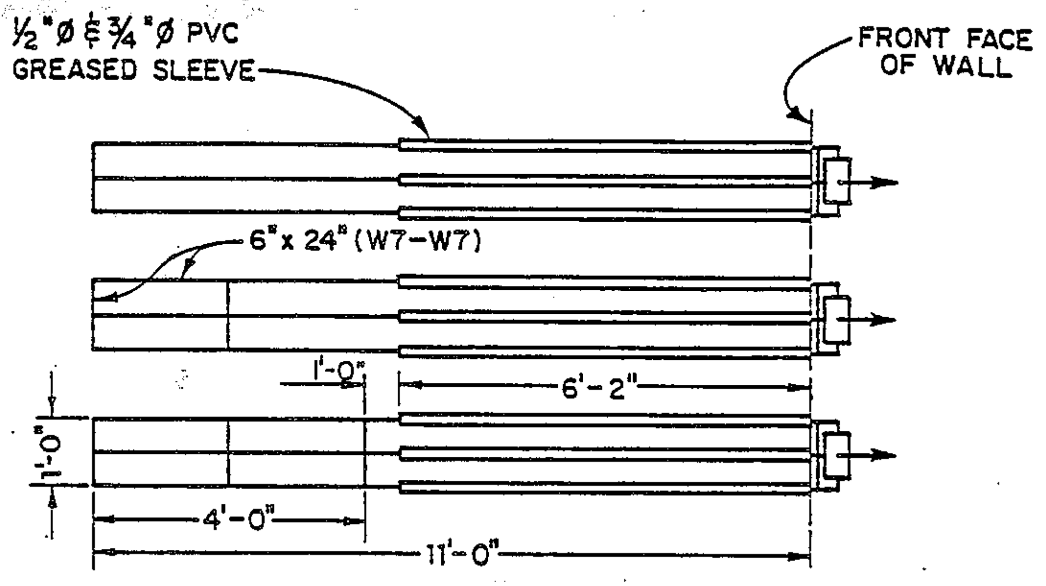
SETTLEMENT AT WALL FACE

FIGURE 16

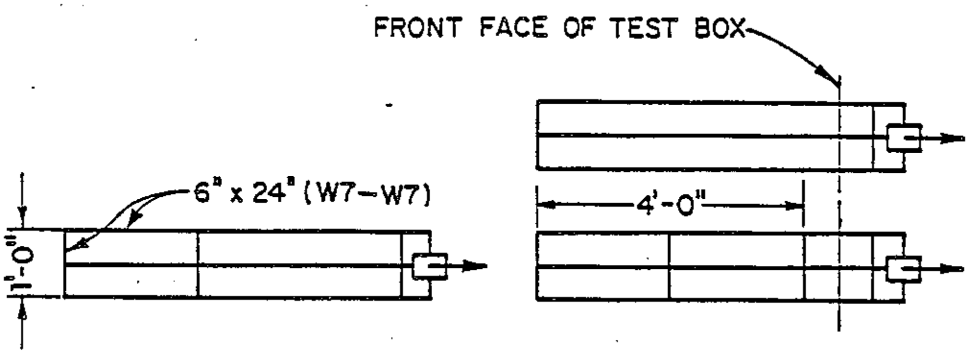


DUMMY BAR-MAT LOCATIONS

FIGURE 17



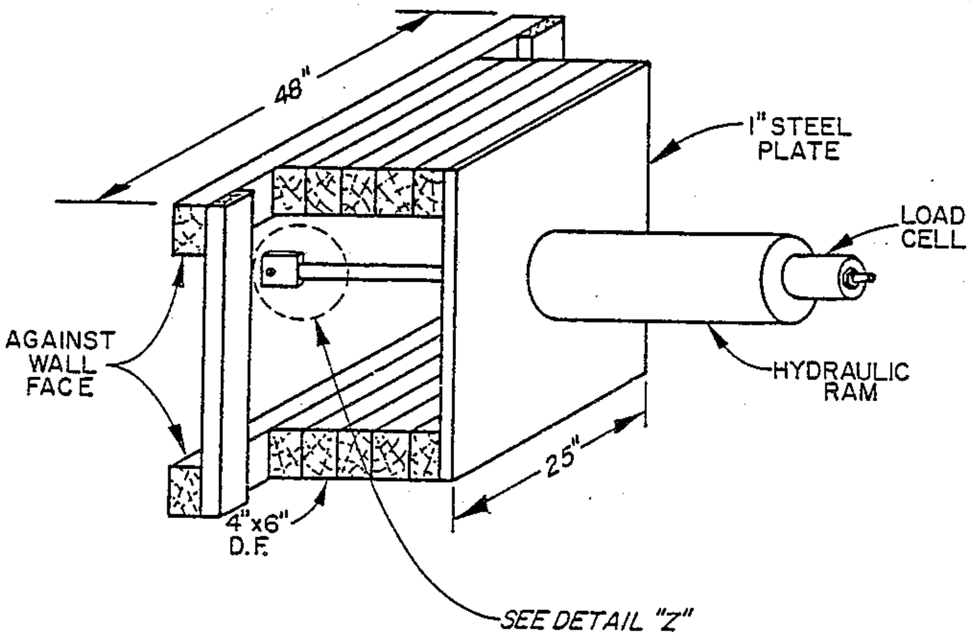
DUMMY BAR-MATS FOR FIELD TEST

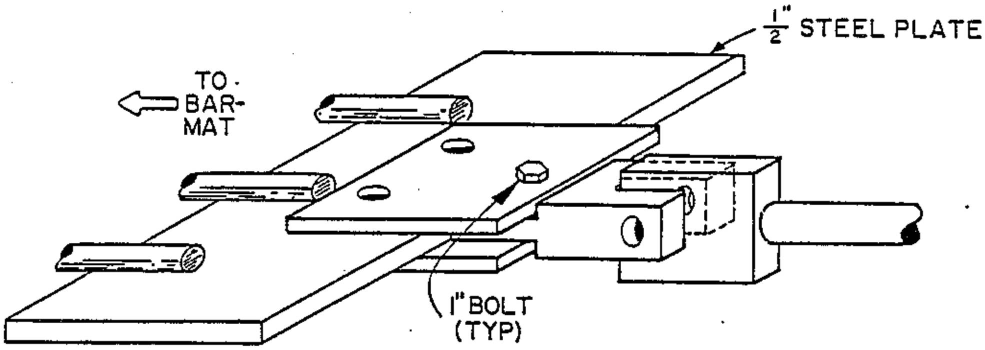


LABORATORY TEST BAR-MATS

BAR-MAT CONFIGURATIONS FOR
LABORATORY AND FIELD TESTS

FIGURE 18



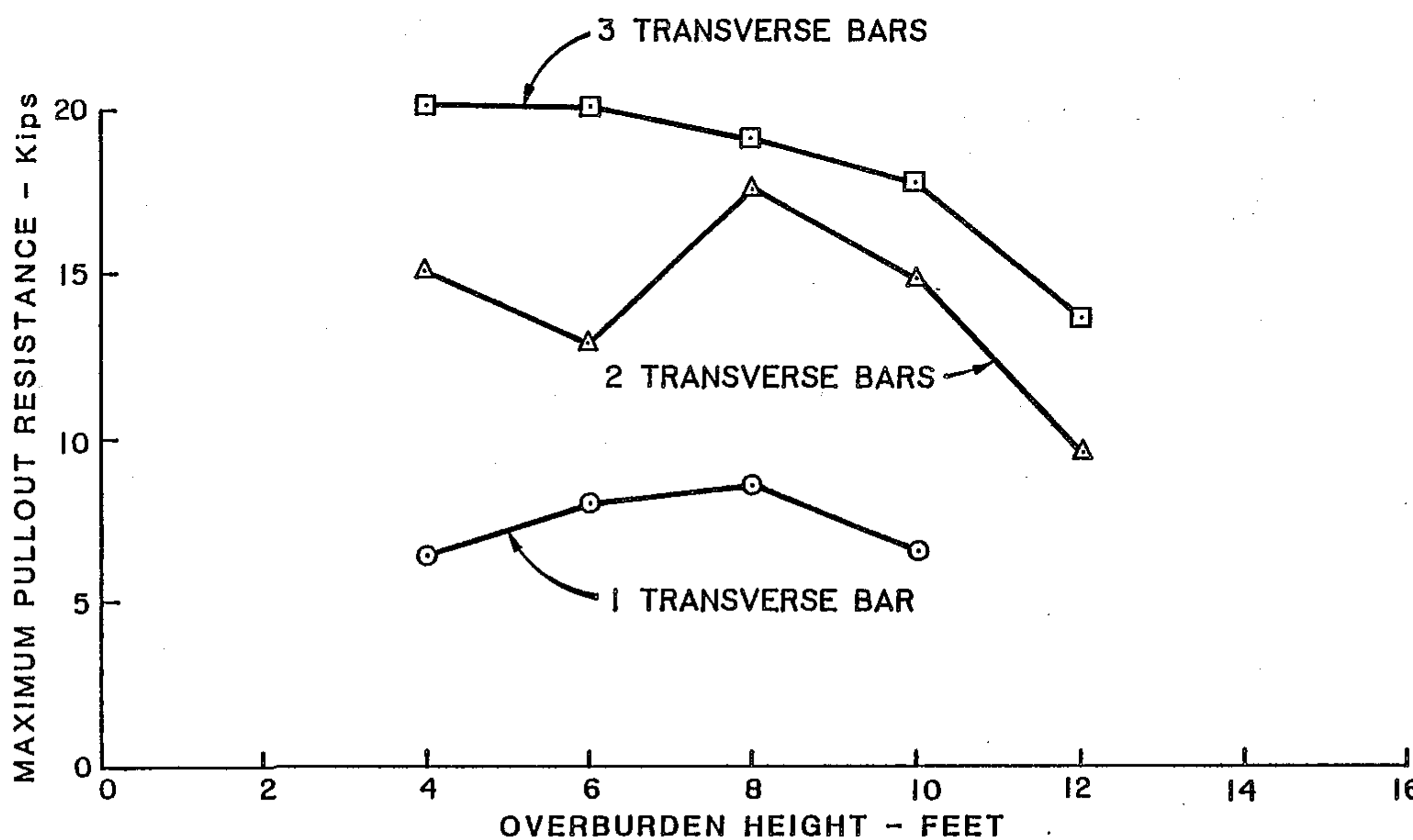


UNIVERSAL CONNECTOR

DETAIL "Z"

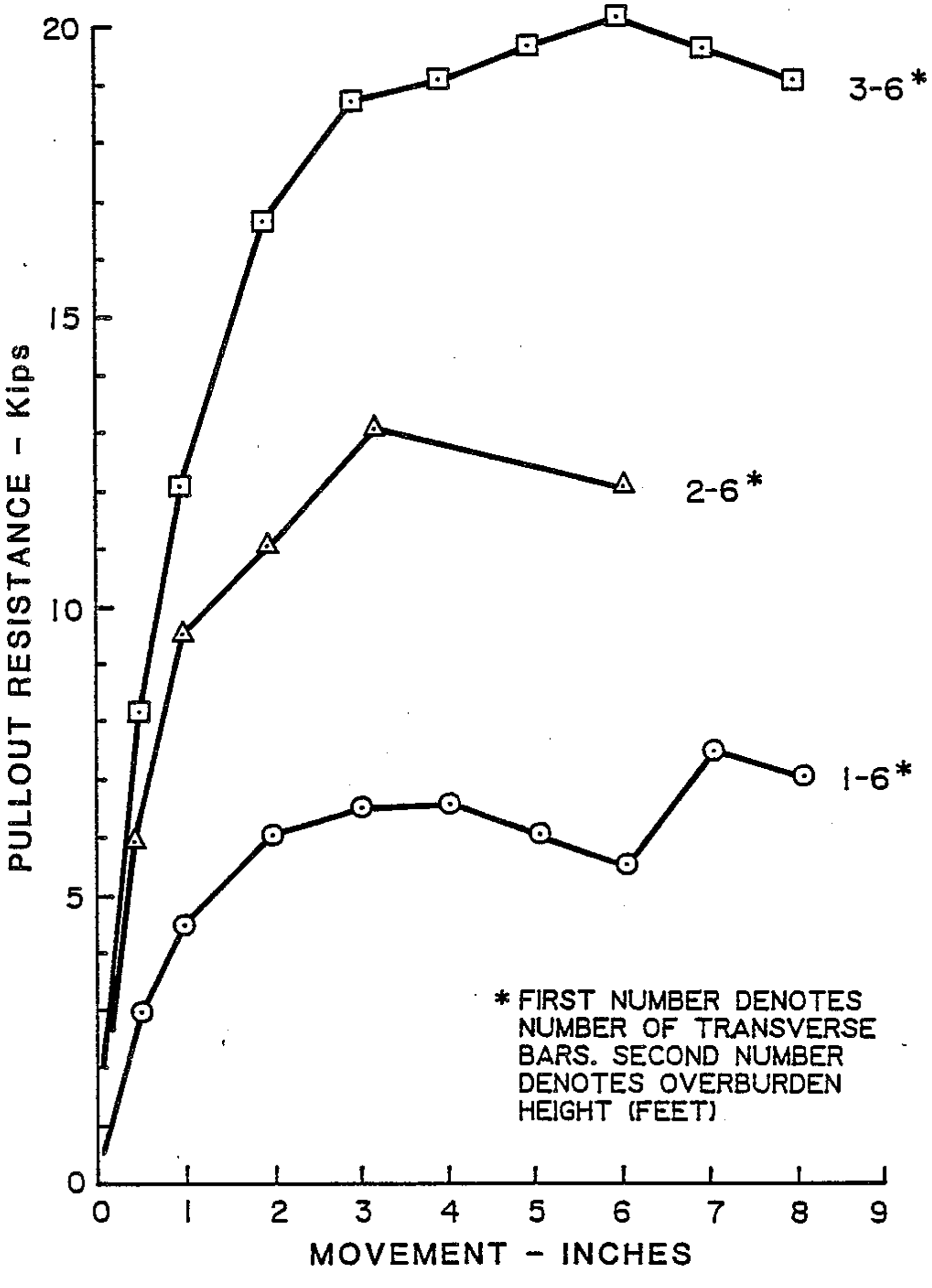
FIELD PULLOUT APPARATUS

FIGURE 19



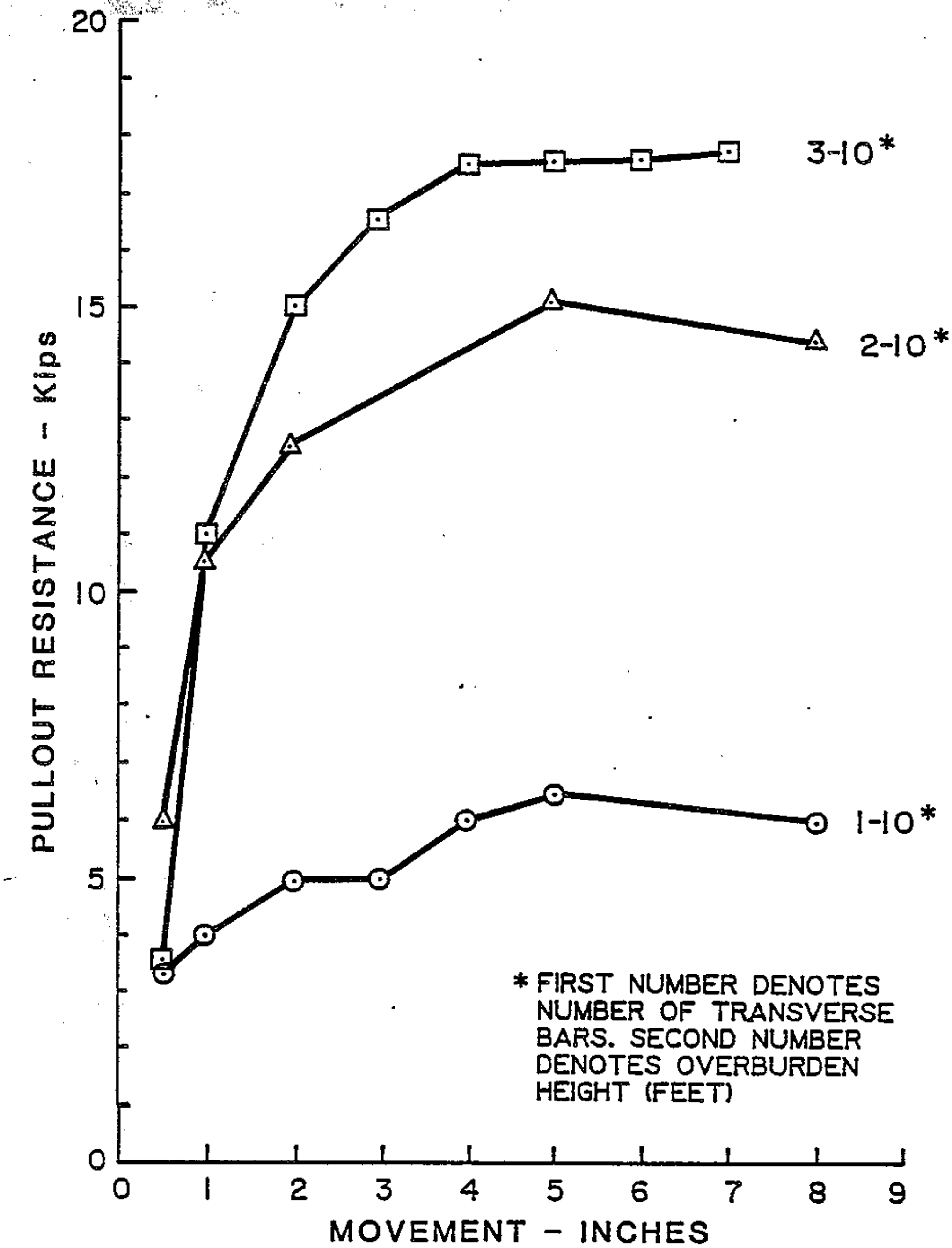
MAXIMUM PULLOUT RESISTANCE FIELD TESTS

FIGURE 20



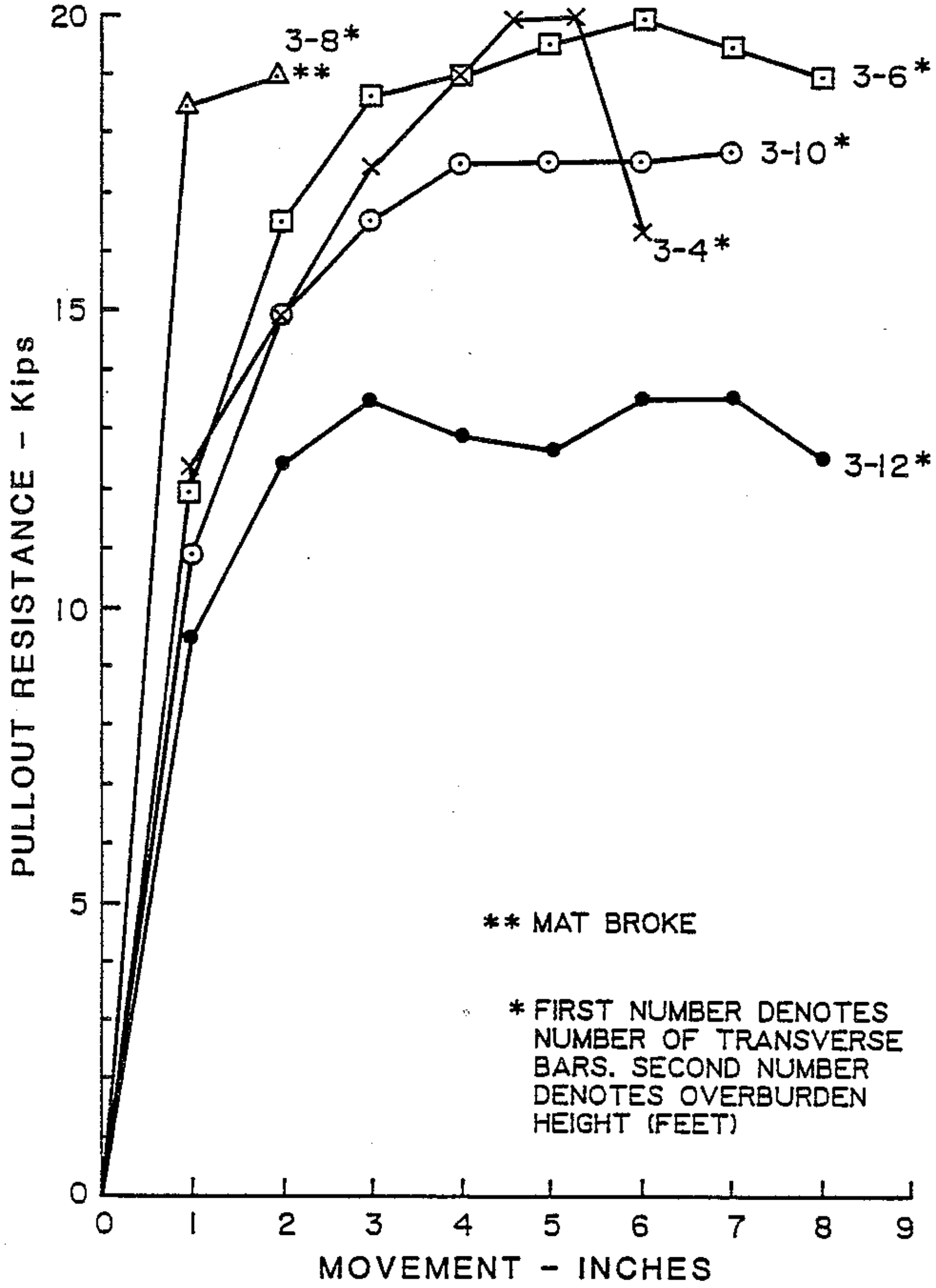
FIELD PULLOUT RESISTANCE OF DUMMY BAR-MATS AT 6 FEET OVERBURDEN

FIGURE 21



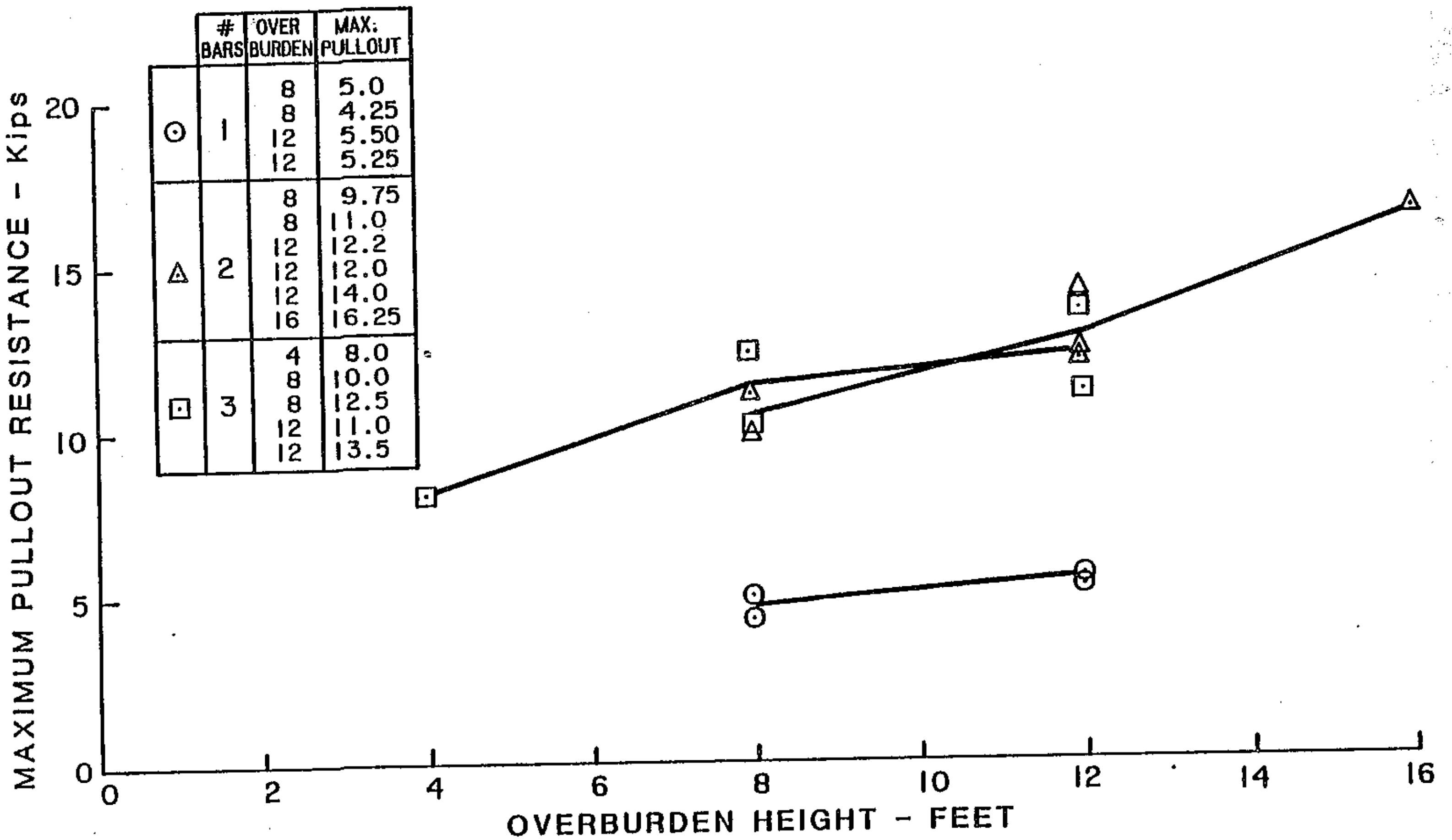
FIELD PULLOUT RESISTANCE OF DUMMY BAR-MATS AT 10 FEET OVERBURDEN

FIGURE 22



FIELD PULLOUT RESISTANCE OF DUMMY BAR-MATS UNDER VARIABLE OVERBURDENS

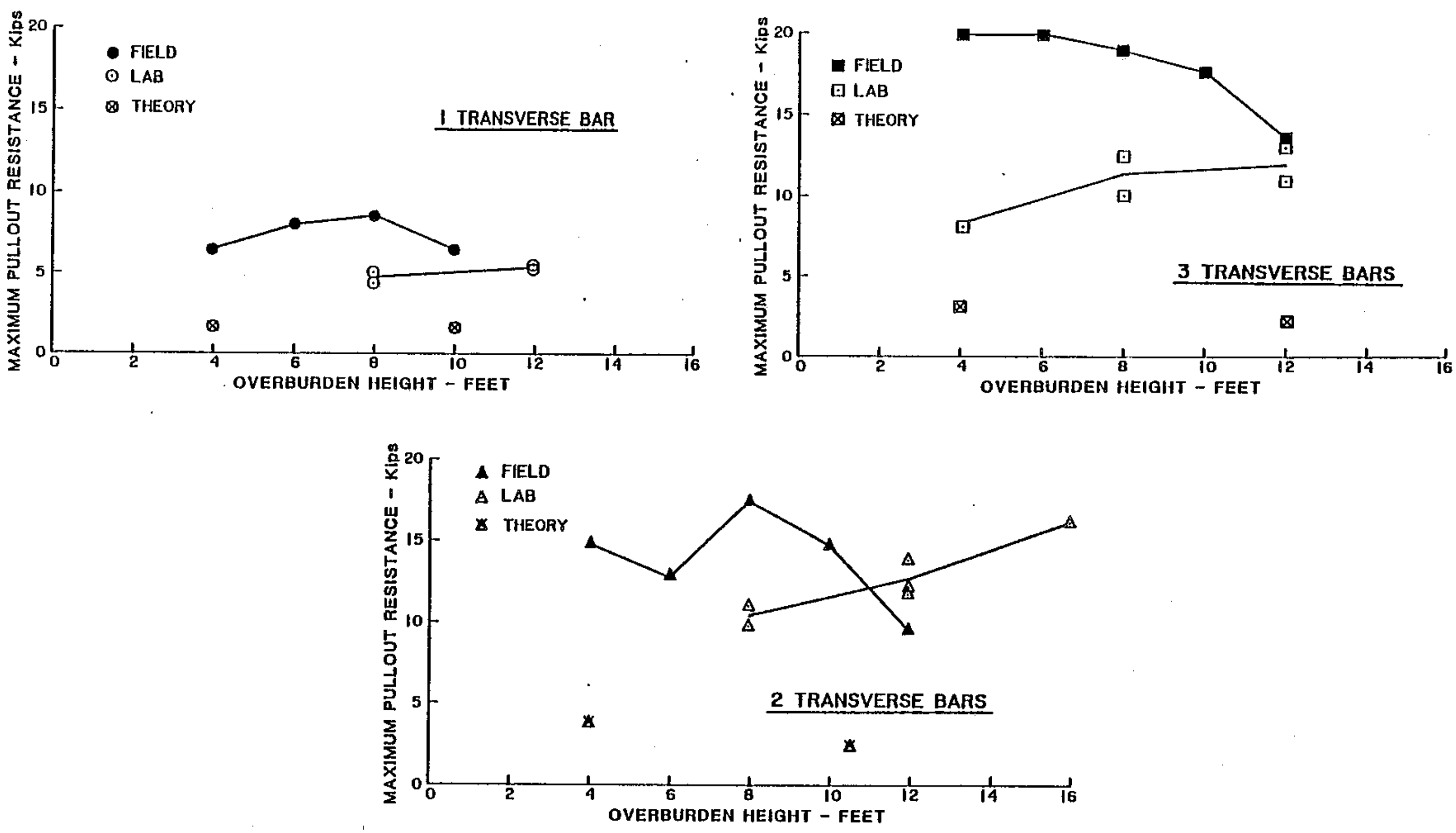
FIGURE 23



	# BARS	OVER BURDEN	MAX. PULLOUT
⊙	1	8	5.0
		8	4.25
		12	5.50
		12	5.25
△	2	8	9.75
		8	11.0
		12	12.2
		12	12.0
		12	14.0
		16	16.25
⊡	3	4	8.0
		8	10.0
		8	12.5
		12	11.0
		12	13.5

MAXIMUM PULLOUT RESISTANCE VS OVERBURDEN
LABORATORY TESTS

FIGURE 24



MAXIMUM PULLOUT RESISTANCE VS OVERBURDEN (FIELD, LAB, AND THEORY)

FIGURE 25

IX. TABLES

Table 1

SHEAR STRENGTHS AND PROPERTIES OF BACKFILL MATERIAL

Preliminary Design

	Sample Number			
	79-1190		79-1191	
Soil Strengths	Effective	Total	Effective	Total
Friction Angle, Ø (deg)	32	20	32	6
Cohesion, c (psf) (Remolded specimens)	500	700	300	900
Plasticity Index, PI	8		11	
Sand Equivalent, SE	23		13	
Wet Density (pcf)	115		115	
pH	5.9		5.5	
Resistivity (ohm-cm)	14,520		7,300	
	Soil Gradations (% Passing by Weight)			
Sieve Size				
6-in	—		—	
3-in	100		—	
2 1/2-in	92		100	
2-in	84		98	
1 1/2-in	78		97	
1-in	75		95	
3/4-in	74		94	
1/2-in	72		93	
3/8-in	70		92	
#4	67		90	
#8	62		85	
#16	56		78	
#30	50		72	
#50	44		64	
#100	37		55	
#200	32		49	
5 µm	14		23	
1 µm	5		9	
Unified Soil Classification	SC		SC	

Table 2

SHEAR STRENGTHS AND PROPERTIES OF BACKFILL MATERIAL

Progress Samples Wall 1

	Sample Number			
	82-1208		82-1232	
Soil Strengths	Effective	Total	Effective	Total
Friction Angle, ø (deg)	32	20	16	5
Cohesion, c (psf) (Remolded specimens)	300	600	700	1000
Plasticity Index, PI	7		9	
Sand Equivalent, SE	19		13	
Wet Density (pcf)	114		114	
pH	—		—	
Resistivity (ohm-cm)	—		—	
	Soil Gradations (% Passing by Weight)			
Sieve Size				
6-in	—		—	
3-in	100		99	
2 1/2-in	99		97	
2-in	99		94	
1 1/2-in	98		93	
1-in	95		90	
3/4-in	93		86	
1/2-in	91		82	
3/8-in	90		80	
#4	87		77	
#8	75		71	
#16	65		64	
#30	57		57	
#50	51		49	
#100	44		42	
#200	38		36	
5 μm	24		18	
1 μm	9		10	
Unified Soil Classification	SC		SC	

Table 3

SHEAR STRENGTHS AND PROPERTIES OF BACKFILL MATERIAL

Progress Samples Wall 3

	Sample Number			
	82-1256		82-1261	
Soil Strengths	Effective	Total	Effective	Total
Friction Angle, Ø (deg)	31	19	33	18
Cohesion, c (psf) (Remolded specimens)	500	1200	360	1400
Plasticity Index, PI	NP		NP	
Sand Equivalent, SE	9		12	
Wet Density (pcf)	118		117	
pH	6.1		6.1	
Resistivity (ohm-cm)	23,400		29,200	
	Soil Gradations (% Passing by Weight)			
Sieve Size				
6-in	—		—	
3-in	—		—	
2 1/2-in	100		—	
2-in	99		100	
1 1/2-in	99		99	
1-in	97		98	
3/4-in	96		98	
1/2-in	95		97	
3/8-in	94		97	
#4	92		96	
#8	89		94	
#16	87		92	
#30	85		89	
#50	81		83	
#100	70		66	
#200	57		50	
5 μm	12		12	
1 μm	5		6	
Unified Soil Classification	ML		ML	

Table 4

SHEAR STRENGTHS AND PROPERTIES FROM UNDISTURBED SOIL SAMPLES

WALL 3

	Station 398+97		Station 399+10		Station 399+22	
Sample No.	B1-1C	B1-4C	B2-1C	B2-3B	B3-1C	B3-3D
Depth* (ft)	4	12	4	8	3	11
Type of Test	CU	CU	CU	CU	CU	CU
Angle of Internal Friction, (degrees)	34	34	34	28	34	26
Cohesion, c (psf)	1300	1000	2000	1800	1000	1200
Plasticity Index, PI	2	3	—	5	—	1
Wet Density (pcf)	124	122	125	120	114	122
Moisture Content (%)	21.6	10.2	19.3	29.2	24.2	22.2

* Depth below finished grade 10 feet back from wall face.

Notes: CU = Consolidated Undrained partially saturated

Table 5

FIELD TESTS
MAXIMUM PULLOUT RESISTANCE

No. Trans Bars	Overburden (ft)	Density Wet (pcf)	Density Dry (pcf)	Moisture Content (%)	Percent Saturation (%)	Displacement Rate (in/min)	Maximum Resistance (kips)	Displacement @ Max. Resist. (in)
1	4	121	98.7	22.6	85	0.14	4.6	1.6
	6	124	106.6	16.3	74	0.45	5.9	6.8
	8	124	99.1	25.1	95	0.46	6.7	6.0
	10	124	99.6	24.5	94	0.72	4.8	5.0
2	4	124	101.7	21.9	89	0.47	12.6*	6.4
	6	121	95.7	26.4	93	0.29	10.1	2.7
	8	112	90.2	24.2	75	0.20	15.1	8.0
	10	—	—	—	—	0.40	12.6	3.6
	12	—	—	—	—	0.44	7.6	4.0
3	4	117	98.8	18.4	69	0.50	18.1	5.6
	6	124	102.4	21.1	87	0.35	18.1	6.0
	8	121	98.9	22.3	84	0.22	17.2*	2.4
	10	126	103.1	22.2	93	0.62	15.5	7.6
	12	121	96.0	26.1	92	0.60	10.9	7.0

* Mat broke during test

Table 6

LABORATORY TEST
MAXIMUM PULLOUT RESISTANCE

No. Trans Bars	Overburden (ft)	Density		Moisture Content (%)	Percent Saturation (%)	Test #	Maximum Resistance (kips)
		Wet (pcf)	Dry (pcf)				
1	8	122.7	100.8	21.7	86	3	5.0
	8	123.8	104.4	18.6	81	6	4.25
	12	122.7	100.8	21.7	86	3	5.5
	12	123.8	104.4	18.6	81	6	5.25
2	8	125.5	105.3	19.2	85	8	9.75
	8	127.2	105.3	20.8	92	4	11.0
	12	125.5	105.3	19.2	85	8	12.2
	12	127.2	105.3	20.8	92	4	12.0
	12	125.1	103.9	20.4	88	5	14.0
	16	125.1	103.9	20.4	88	5	16.25
3	4	123.0	101.0	21.8	87	2	8.0
	8	123.0	101.0	21.8	87	2	10.0
	8	122.1	—	—	—	7	12.25
	10	122.1	100.2	21.8	85	1	7.0*
	12	123.0	101.0	21.8	87	2	11.0
	12	122.1	100.2	21.8	85	1	6.0*
	12	122.1	—	—	—	7	13.50

* Welds broke during test

Note: All tests were bump tests for greater overburdens.

Table 7

CONTRACTOR BID SUMMARY

MECHANICALLY STABILIZED EMBANKMENT

Item	Estimated Quantity	Unit	Unit Price (Dollars)	Amount (Dollars)
Structure Excavation	8,000	CY	7	56,000
Structure Backfill	6,000	CY	7	42,000
Imported Borrow	5,000	CY	4	20,000
Minor Concrete (footing)	42	CY	300	12,600
Precast Concrete Facing	17,626	SF	13	229,138
Welded Wire Mat	100,000	LB	0.85	85,000
Sub Total				444,738
8" Underdrain Pipe	2,000	LF	6	12,000
Filter Fabric	5,400	SY	1	5,400
Class 3 Perm. Material	1,450	CY	25	36,250
Sub Total				53,650
Grand Total				$498,388

Unit Cost per Square Foot of wall facing:

A. With Drainage System - $28.28
B. Without Drainage System - $25.23

Table 8

CONTRACTOR BID SUMMARY

REINFORCED EARTH WALLS

Item	Estimated Quantity	Unit	Unit Price (Dollars)	Amount (Dollars)
Structure Excavation	8,250	CY	10	82,500
Structure Backfill	7,350	CY	8	58,800
Minor Concrete (footing)	26	CY	400	10,400
Precast Concrete Facing	17,278	SF	19	328,282
Reinforcing Strips	69,000	LB	0.10	6,900
Sub Total				486,882
8" Underdrain Pipe	2,000	LF	4	8,000
Filter Fabric	6,500	SY	1.50	9,750
Class 3 Perm. Material	1,800	CY	30	54,000
Sub Total				71,750
Grand Total				$558,632

Unit Cost per Square Foot of wall facing:

A. With Drainage System - $32.30
B. Without Drainage System - $28.20

Table 9

CONTRACTOR BID SUMMARY

REINFORCED CONCRETE CRIB WALLS

Item	Estimated Quantity	Unit	Unit Price (Dollars)	Amount (Dollars)
Structure Excavation	5,650	CY	9.00	50,850.00
Structure Backfill	5,000	CY	21.50	107,500.00
Imported Borrow	1,700	CY	3.00	5,100.00
Reinforced Concrete Crib Wall (Type II)	14,773	SF	17.25	254,834.25
Reinforced Concrete Crib Wall (Type III)	2,724	SF	17.25	46,989.00
Sub Total				465,273.25
Underdrain Pipe	1,600	LF	6.50	10,400.00
Filter Fabric	6,350	SY	3.50	22,225.00
Class 3 Perm. Material	2,000	CY	39.00	78,000.00
Sub Total				110,625.00
Grand Total				$575,938.25

Unit Cost per Square Foot of wall facing:

A. With Drainage System - $32.90
B. Without Drainage System - $26.60

Table 10

Preliminary Laboratory Pullout Tests, 1979

Bar-Mats with 3 Transverse Bars

Overburden (ft)	Density (pcf) Wet	Density (pcf) Dry	Moisture Content (%)	Max Pullout Resistance (kips)
5	115	96	19.7	13.5
10	113	91	24.3	20.8
15	110	89	24.3	10.6
20	117	96	22.2	13.1

Average Load Calculations

Area of Test Bar = 2 ft x 4 ft = 8 sq ft

Pullout resistance per sq ft, $f_r = \dfrac{10.6 \text{ kips}}{8 \text{ sq ft}}$

$= 1.3$ ksf

Test Bar-Mat configuration

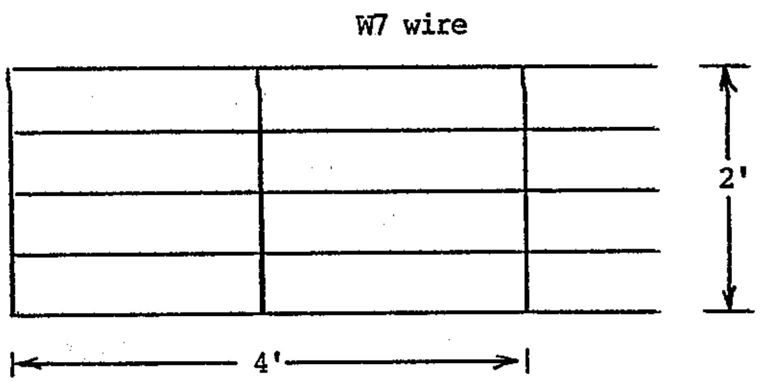


Table 11

Laboratory Pullout Tests, 1982

Bar-Mats with 3 Transverse Bars

Overburden (ft)	Density (pcf)		Moisture Content (%)	Max Pullout Resistance (kips)
	Wet	Dry		
4	117	95.1	23	21.4
6	124	106.9	16	21.8
8	121	95.3	27	20.5
10	126	102.4	23	18.5
12	121	96.8	25	13.0

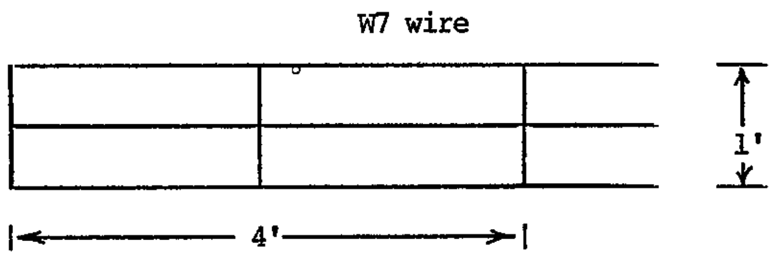
Average Load Calculations

Area of Test Bar = 1 ft x 4 ft = 4 sq ft

Pullout resistance per sq ft, $f_r = \dfrac{13.0 \text{ kips}}{4 \text{ sq ft}}$

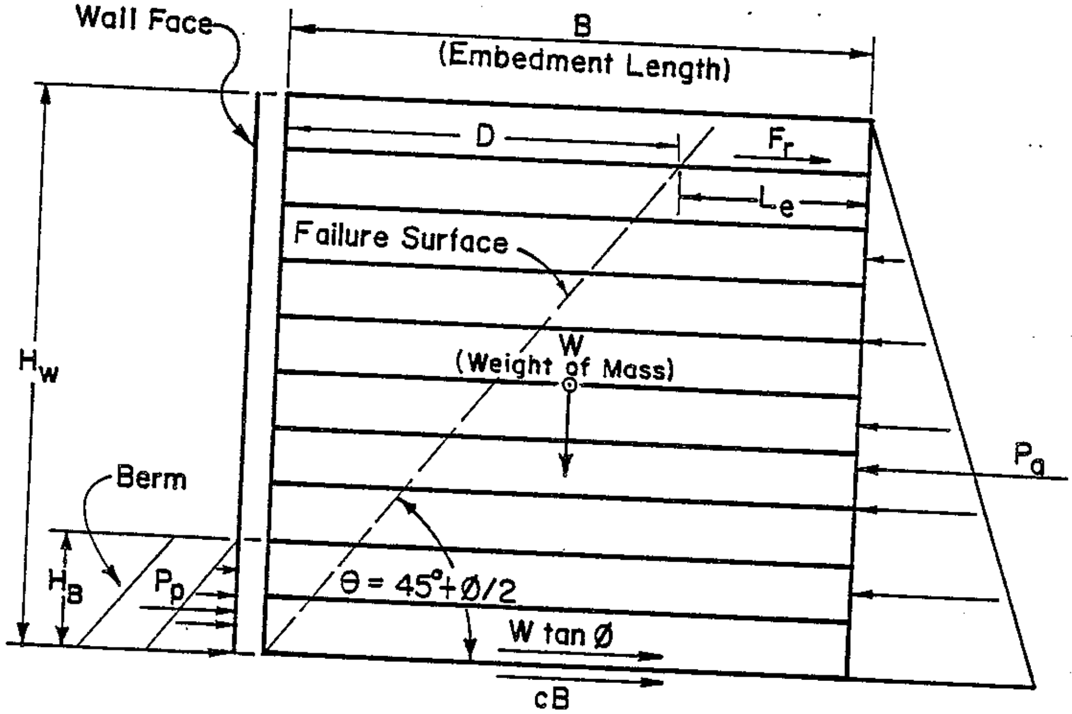
$= 3.25$ ksf

Test Bar-Mat configuration

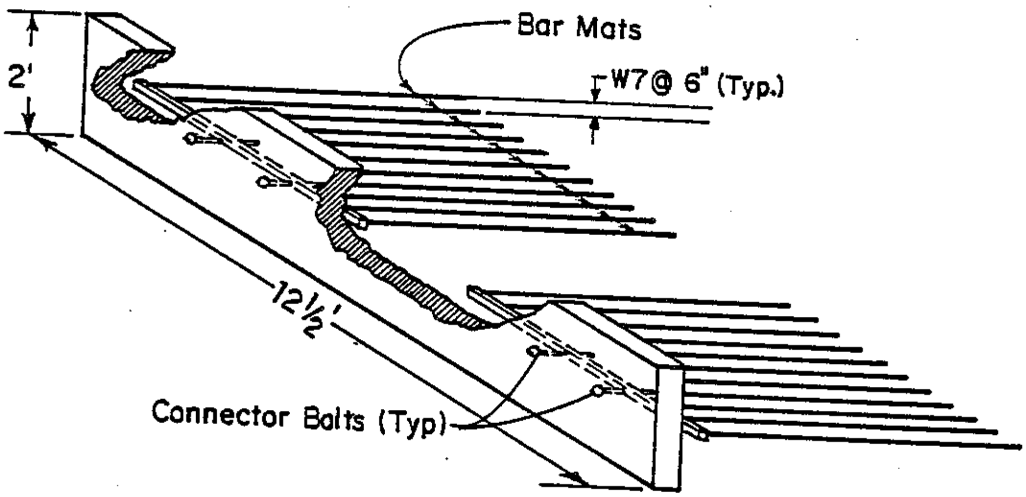


APPENDIX A

DESIGN CALCULATIONS



MSE FORCE DIAGRAM



CONTRIBUTORY AREA

FIGURE A-1

Check Design

Assume: Active State Pressure Distribution
$\phi=26°$ and $c=0$

1. Calculate Maximum Horizontal Stress (including traffic loading)

$$\sigma_h' = (\gamma_t h + q_s h_s)k_a$$

where: γ_t = weight of soil, 122 pcf
h = height of wall
$q_s h_s$ = surcharge loading for traffic
$q_s h_s$ = 2ft x 122 pcf = 244 psf
k_a = coefficient of active earth pressure using minimum ϕ = 26° (Table 4)
k_a = 0.3905 (Assuming c=0)

Maximum Horizontal Stress, σ_h'

	Full Height	Level B
Wall 1, H = 14'	0.76 ksf	0.43 ksf
Wall 3, H = 16'	0.86 ksf	0.52 ksf

2. Calculate Driving Force At Back of Wall (per foot of wall length)

$$P_a = 1/2\ (\gamma_t h^2 k_a) + (q_s h_s) h k_a$$

where: γ_t, h, $q_s h_s$, and k_a are defined above

Driving Force @ Back of Wall, P_a

	Full Height	Top of Berm
Wall 1, H = 14'	6.00 kips	3.53 kips
Wall 3, H = 16'	7.62 kips	5.34 kips

Check Design
(Continued)

1. EXTERNAL STABILITY

Trial #1:

Sliding Failure Along Base Without Cohesion

where: $\phi = 26°$ and $c = 0$

Factor of Safety, F.S. $\geq$ 1.5

Determine Embedment, B

$$F.S. \geq 1.5 = \frac{\text{Resisting Forces}}{\text{Driving Forces}}$$

$$= \frac{W\tan\phi + P_p}{P_a}$$

$$= \frac{BH\gamma_t\tan\phi + P_p}{P_a}$$

$$\therefore \quad B = \frac{1.5\,P_a - P_p}{H\gamma_t\tan\phi}$$

	Embedment Length Required, B	
	Full Height	Top of Berm, $P_p = 0$
Wall 1, H = 14 ft	10.1 ft	7.4 ft
Wall 3, H = 16 ft	11.4 ft	10.8 ft

1. EXTERNAL STABILITY (Continued)

Trial #2:

Sliding Failure Along the Base Using Cohesion

Where: $\phi=26°$ and $c=1.2$ ksf

Factor of Safety, F.S. ≥ 1.5

Determine Embedment, B

$$F.S. \geq 1.5 = \frac{\text{Resisting Forces}}{\text{Driving Forces}}$$

$$= \frac{W\tan\phi + cB + P_p}{P_a}$$

$$= \frac{BH\gamma_t \tan\phi + cB + P_p}{P_a}$$

$$\therefore \quad B = \frac{1.5\,P_a - P_p}{H\gamma_t \tan\phi + c}$$

Embedment Length Required, B

	Full Height	Top of Berm, $P_p=0$
Wall 1, H = 14 ft	4.1 ft	2.6 ft
Wall 2, H = 16 ft	5.0 ft	4.4 ft

Check Design
(Continued)

1. EXTERNAL STABILITY (Continued)

Overturning of Gravity Block

Where $\phi = 26°$ and $c = 1.2$ ksf

Determine Embedment, B

$$F.S. \geq 2.0 = \frac{\text{Resisting Moments About Toe}}{\text{Driving Moments About Toe}}$$

$$= \frac{W(B/2)}{P_a(H/3)}$$

$$= \frac{BH\gamma_t(B/2)}{P_a(H/3)}$$

$$= \frac{3B^2\gamma_t}{2P_a}$$

$$\therefore \quad B = \sqrt{\frac{4P_a}{3\gamma_t}}$$

Embedment Length Required, B

	Full Height	Top of Berm
Wall 1, H = 14 ft	5.4 ft	6.2 ft
Wall 3, H = 16 ft	9.5 ft	7.6 ft

Check Design
(Continued)

2. INTERNAL STABILITY

Pullout Resistance of Reinforcement

Where $\phi=26°$ and $c=0$

Factor of Safety, F.S. $\geq$ 2.0

Determine Embedment, B

Total Pullout Resistance, $F_r = f_r \times L_e$

Where: f_r = pullout resistance/foot^2 from lab (Table 10)
= 3.25 K/ft^2
L_e = embedment beyond failure plane

$$F.S. \geq 2.0 = \frac{F_r}{\sigma_h' \times \text{Contributory area/bar}}$$

$$= \frac{f_r \times L_e}{\sigma_h' \times 1.14 \text{ sq ft}}$$

$$\therefore \quad L_e = \frac{(2.0)(1.14)\sigma_h'}{3.25}$$

Embedment, L_e

Wall 1, H = 14 ft	L_e = 0.74 ft	say 1 ft
Wall 3, H = 16 ft	L_e = 0.83 ft	say 1 ft

$$D\text{, distance, wall to failure zone} = \frac{\text{Wall height, Hw}}{\tan \theta}$$

$\therefore$ Embedment $B = D + L_e$

Embedment Length Required, B

	Full Height
Wall 1, H = 14, D = 8.7 ft	9.7 ft
Wall 3, H = 16, D = 10.0 ft	11.0 ft

Check Design
(Continued)

3. Calculate Resisting Force from Berm at Toe (per foot of wall length)

$$P_p = \frac{1}{2} \gamma_t H_B^2 k_p$$

where: γ_t = total wt of soil, 122 pcf
H_B = height of berm
= 2 feet (allowing 1 foot for erosion)
k_p = coefficient of passive earth pressure
= $\tan^2\left(45° + \frac{\phi}{2}\right)$ or 2.56

P_p = 624 pounds or 0.624 kips

4. Calculate Contributory Area Per Bar

11 bars per mat
2 mats per face panel
area of face panel = 12.5 ft x 2 ft

$$\text{Contributory Area} = \frac{(12.5 \times 2) \text{ sq ft}}{22 \text{ bars}}$$

$$= 1.14 \text{ sq ft/bar}$$

5. Calculate Theoretical Stress In Bars

$$\text{Stress Per Bar} = \frac{\sigma_h' \times \text{Contributory Area}}{\text{area of bar}}$$

where: σ_h' = maximum theoretical stress

Bar Area = 0.070 sq in

Theoretical Bar Stress

	Level B		Full Height	
	σ_h' (ksf)	Bar Stress (ksi)	σ_h' (ksf)	Bar Stress (ksi)
Wall 1, H = 14 ft.	0.428	6.97	0.761	12.39
Wall 3, H = 16 ft.	0.524	8.53	0.858	13.97

APPENDIX B

CORROSION CALCULATIONS

Compare 1979 Design Criteria to 1984 Interim Design Criteria

A. Compute Cross-Sectional Area Remaining After 50 Years

Basic Equation: $A_{50} = \frac{[D-2K(Y-C)]^2}{D^2} \times 100\%$

where: A_{50} = % of original cross-sectional area remaining
D = original diameter, 0.299 inch
Y = time of exposure to soils, 50 years
K = corrosion rate factor, inch/year
see page B-4 for 1979 value
see page B-6 for 1984 value
C = useful life of coating
for bare steel, $C = 0$ (no coating)

1979 Design Corrosion Calculations

$$A_{50} = \frac{[0.299 - 2(.00075)(50)]^2}{(0.299)^2} \times 100\%$$

$$= 56\%$$

1984 Interim Design Criteria Calculations

$$A_{50} = \frac{[0.299-2(.0013)(50)]^2}{(0.299)^2} \times 100\%$$

$$= 32\%$$

B. Check Load Carrying Capacity of W7 Wires @ 50 Years, f_{s50}

1. Maximum Allowable Bar Stress After 50 Years

W7 Wires, Grade 65: $f_y = 65$ ksi

$f_{s50} = 0.55\ f_y$ (Bridge Design Specification)
$= 36$ ksi

2. Calculate Area of Steel @ 50 Years

A_{50}, Remaining Bar Area $= \frac{\pi D^2}{4} \times A(\%)$

1979 Design Criteria: $A_{50} = \frac{\pi \times (0.299)^2}{4} \times 56\%$
$= 0.039$ sq in

1984 Interim Design Criteria: $A_{50} = \frac{\pi \times (0.299)^2}{4} \times 32\%$

$= 0.022$ sq in

Check Corrosion
(Continued)

3. Calculate Maximum Stress Due To Soil Pressure

$$\sigma_{s50}\text{, Max. Stress/Bar} = \frac{\sigma_h' \times \text{contributory area per bar}}{A_{50}}$$

$$\sigma_h' = (\gamma_t h + q_s h_s)\, k_a$$

where: γ_t = weight of soil, 122 pcf
- h = height of wall
- A_{50} = area of bar remaining @ 50 years
- $q_s h_s$ = surcharge loading for traffic
  - $q_s h_s$ = 2ft x 122 pcf = 244 psf
- k_a = coefficient of active earth pressure using minimum $\emptyset$ = 26° (Table 4)
  - k_a = 0.3905 (Assuming c=0)

$$\text{Contributory area} = \frac{25\text{ sq ft}}{\text{panel}} \times \frac{1\text{ panel}}{2\text{ mats}} \times \frac{1\text{ mat}}{11\text{ bars}}$$

$$= 1.14\text{ sq ft/bar}$$

Maximum Stress Per Bar*, $\sigma_{s(50)}$

	σ_h (ksf)	1979 Criteria A_{50} (sq in)	1979 Criteria σ_{s50} (ksi)	1984 Criteria A_{50} (sq in)	1984 Criteria σ_{s50} (ksi)
Wall 1	0.762	0.039	22.3	0.022	39.5
Wall 3	0.858	0.039	25.1	0.022	44.5

*The two criteria are shown for comparison purposes only. The Baxter MSE walls were designed using the 1979 criteria and meet the requirements set forth under it.

Algorithms for computing the cross-sectional area (%) remaining on steel reinforcing elements after corrosion loss, given soil type and years of service:

A) Round Rod Types

$$A = \frac{[D - 2K\,(Y-C)]^2}{D^2} \times 100\%$$

B) Flat Strap Types

$$A = \frac{[W - 2K\,(Y-C)]}{(W)}\,\frac{[T - 2K\,(Y-C)]}{(T)} \times 100\%$$

where:

A = % of Original Cross-Sectional Area Remaining*

D = Original Diameter, inches

W = Original Strap Width, inches

T = Original Strap Thickness, inches

Y = Time of Exposure in Soils, years

K = General Corrosion Rate Factor

C = Useful Life of Coating, years. (For Bare Steel, C=0)

*Round calculated values of A to the nearest 5%.

Soil Type	K	C		
		Paint	Galvanized 2 oz.	Galvanized 3 oz.
Normal				
Neutral & Alkaline	.0011	5	10	15
Acidic	.0013	5	10	15
Corrosive	.0028	5	6	9
Select Granular **				
Neutral & Alkaline	.0005	5	20	30
Acidic	.0005	5	20	30
Corrosive	.0010	5	12	20

**Specifically selected, well-draining backfill

SELECT GRANULAR BACKFILL

When select granular backfill is chosen for added corrosion protection, materials shall conform to the following criteria:

Sieve Size	Grading Limits, Percent Passing
6 inches	100
3 inches	100 - 75
No. 4	25 - 0
No. 200	5 - 0

Plasticity Index < 6

NORMAL SOILS

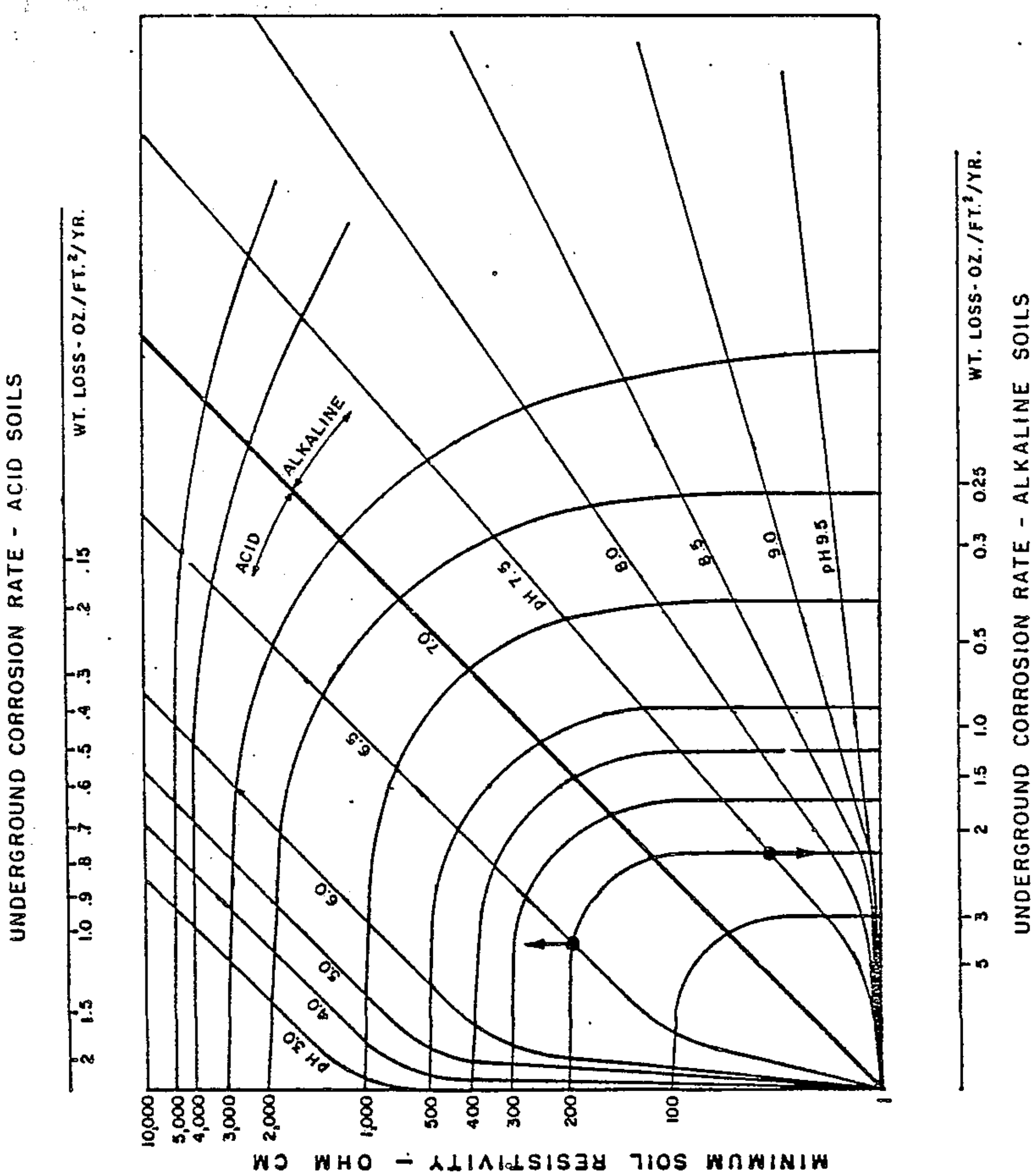
SOIL TYPE	COATING	CROSS-SECTIONAL AREA REMAINING, (PERCENT) AFTER 50 YEARS					
		HILFIKER (WIRE MESH)		M.S.E. (WELDED RODS)			REINFORCED EARTH (STRAPS)
		9 ga. (.148"φ)	7 ga. (.177"φ)	W5 (.252"φ)	W7 (.299"φ)	W11 (.375"φ)	60x5 mm (2.362"x.197")
NEUTRAL & ALKALINE R ≥1000 pH ≥7	BARE	5	15	30	40	50	40
	PAINT	10	20	35	45	55	50
	GALVANIZING (2oz/FT²)	15	25	40	50	60	55
	GALVANIZING (3oz/FT²)	25	30	50	55	65	60
ACIDIC R ≥1000 pH <7	BARE	5	5	20	30	40	30
	PAINT	5	10	30	40	50	40
	GALVANIZING (2oz/FT²)	10	20	35	40	50	45
	GALVANIZING (3oz/FT²)	15	25	40	50	55	50
CORROSIVE R <1000	BARE	0	0	0	0	5	0
	PAINT	0	0	0	0	10	0
	GALVANIZING (2oz/FT²)	0	0	0	5	10	0
	GALVANIZING (3oz/FT²)	0	0	0	5	15	0
VERY CORROSIVE R <1000 Cl⁻ >500 $SO_4^{=}$ >2000	NOT RECOMMENDED						

FOR SELECT GRANULAR BACKFILL*

SOIL TYPE	COATING	CROSS-SECTIONAL AREA REMAINING, (PERCENT) AFTER 50 YEARS					
		HILFIKER (WIRE MESH)		M.S.E. (WELDED RODS)			REINFORCED EARTH (STRAPS)
		9 ga. (.148"φ)	7 ga. (.177"φ)	W5 (.252"φ)	W7 (.299"φ)	W11 (.375"φ)	60x5 mm (2.362"x.197")
NEUTRAL & ALKALINE R ≥1000 pH ≥7	BARE	45	50	65	70	75	70
	PAINT	50	55	65	70	75	75
	GALVANIZING (2oz/FT²)	65	70	80	80	85	85
	GALVANIZING (3oz/FT²)	75	80	85	90	90	90
ACIDIC R ≥1000 pH <7	BARE	45	50	65	70	75	70
	PAINT	50	55	65	70	75	75
	GALVANIZING (2oz/FT²)	65	70	80	80	85	85
	GALVANIZING (3oz/FT²)	75	80	85	90	90	90
CORROSIVE R <1000	BARE	10	20	35	45	55	45
	PAINT	15	25	40	50	60	50
	GALVANIZING (2oz/FT²)	25	35	50	55	65	60
	GALVANIZING (3oz/FT²)	35	45	60	65	70	70
VERY CORROSIVE R <1000 Cl⁻ >500 $SO_4^{=}$ >2000	NOT RECOMMENDED						

* Specifically selected, well-draining backfill

CORROSION OF EARTH RETAINING SYSTEMS
INTERIM DESIGN CRITERIA – 1984 (Revised 7/25/84)


UNDERGROUND CORROSION RATE - ACID SOILS
WT. LOSS-OZ./FT.²/YR.
.15
.2
.3
.4
.5
.6
.7
.8
.9
1.0
1.5
2
UNDERGROUND CORROSION RATE - ALKALINE SOILS
WT. LOSS-OZ./FT.²/YR.
0.25
0.3
0.5
1.0
1.5
2
3
5
MINIMUM SOIL RESISTIVITY - OHM CM
10,000
5,000
4,000
3,000
2,000
1,000
500
400
300
200
100
1
ACID
ALKALINE
PH 3.0
4.0
5.0
6.0
6.5
7.0
PH 7.5
8.0
8.5
9.0
pH 9.5