Demonstration Projects Program
Technology Transfer
FHWA-DP-37-11
May 1980

DEMONSTRATION PROJECT NO. 37

DISCARDED TIRES IN HIGHWAY CONSTRUCTION

RNIA

FHWA-DP-79-29

Prepared for
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U.S. DEPARTMENT OF TRANSPORTATION
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REPORT NO. FHWA/CA/TL-79/22

FILL STABILIZATION USING NON-BIODEGRADABLE WASTE PRODUCTS-PHASE II

FINAL REPORT

NOV. 1979

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TECHNICAL REPORT STANDARD TITLE PAGE

1. REPORT NO.	2. GOVERNMENT ACCESSION NO.	3. RECIPIENT'S CATALOG NO.
FHWA/CA/TL-79/22		

4. TITLE AND SUBTITLE	5. REPORT DATE
FILL STABILIZATION USING NON-BIODEGRADABLE WASTE PRODUCTS-PHASE II	November 1979
	6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S) Wilfred S. Yee	8. PERFORMING ORGANIZATION REPORT NO. 19302-652124
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. D-3-44
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

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

16. ABSTRACT

A study has been conducted to evaluate the performance and behavior of a tire sidewall reinforced earth embankment. The fill material is an intermixture of clayey sandy silt. Instruments were installed and monitored to measure lateral soil pressure, stress in the connecting elements of the tire mat system, elongation of the sidewalls, movement of the soil within the fill mass and sidewalls, settlements, and lateral movements of the embankment with respect to depth. Laboratory pull-out tests conducted on a buried sidewall and clip are reported.

A year after the completion of the embankment, a local slope failure occurred. Reasons for the failure are discussed, and the repair of the slope is described.

Evidence is presented to show that, despite the slope failure, the proposed design procedures in this report are suitable and practical for this tire sidewall reinforced embankment. The apparent coefficient of friction between the soil and sidewall surfaces was found to be about 0.36 from a pull-out test in our laboratory. The embankment was observed to behave as an integral mass. The soil could not be compacted at the slope and a wire mesh was effective in retaining the soil during compaction. The calculated stresses based on the suggested design procedures did not agree with measured data because of slack and discontinuity in the tire/clip system.

17. KEY WORDS	18. DISTRIBUTION STATEMENT
Reinforced earth, tire sidewall, rebar clips, mats, stress, movement, stability, corrosion, pull-out test, friction, slope, cohesion, failure.	No restrictions.

19. SECURITY CLASSIF. (OF THIS REPORT)	20. SECURITY CLASSIF. (OF THIS PAGE)	21. NO. OF PAGES	22. PRICE
Unclassified	Unclassified	110	

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

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STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF CONSTRUCTION
OFFICE OF TRANSPORTATION LABORATORY

November 1979

FHWA No. D-3-44
TL No. 652124

Mr. C. E. Forbes
Chief Engineer

Dear Sir:

I have approved and now submit for your information this final research project report titled:

FILL STABILIZATION USING NON-BIODEGRADABLE WASTE PRODUCTS-PHASE II

Study made by ..... Soil Mechanics and Pavement Branch

Under the Supervision of ..... Raymond A. Forsyth, P.E.

Principal Investigators ..... R. H. Prysock, P.E. and Joseph B. Hannon, P.E.

Co-Investigator ..... Wilfred S. Yee, P.E.

Report Prepared by ..... Wilfred S. Yee, P.E.

Very truly yours,

Neal Andersen

NEAL ANDERSEN, P.E.
Chief, Office of Transportation Laboratory

Attachment

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ACKNOWLEDGEMENTS

This earth reinforcement Phase II field study was sponsored and financed by the Federal Highway Administration. Highway research project funds provided were supplemented with FHWA demonstration project funds to enable processing and placement of the tire mat reinforcement and associated instrumentation within the highway embankment. Phase I, the laboratory oriented portion of this study, was completed in 1973.

We wish to thank FHWA representatives Douglas Bernard, John Walkinshaw and Gene Stager for their counsel and assistance. Special thanks are extended to Messrs. Dino Cassinelli, Thomas Walsh and Dave Hayes of District 04 San Francisco, who acted to secure the site for the project. Appreciation is extended to William Schott, Project Resident Engineer, for his assistance during construction, instrumentation, and monitoring phases of the work; and to the Granite Construction Company for its cooperation. We further thank the experts in various disciplines, particularly the members of the Soil Mechanics and Pavement, and the Electrical, Corrosion, and Engineering Services Branches of the California Transportation Laboratory, for constructive suggestions tendered in the development of the tire clip and instrumentation systems for this project.

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CONVERSION FACTORS

English to Metric System (SI) of Measurement

Quanity	English unit	Multiply by	To get metric equivalent
Length	inches (in)or(")	25.40	millimetres (mm)
		.02540	metres (m)
	feet (ft)or(')	.3048	metres (m)
	miles (mi)	1.609	kilometres (km)
Area	square inches (in^2)	6.432×10^{-4}	square metres (m^2)
	square feet (ft^2)	.09290	square metres (m^2)
	acres	.4047	hectares (ha)
Volume	gallons (gal)	3.785	litres (l)
	cubic feet (ft^3)	.02832	cubic metres (m^3)
	cubic yards (yd^3)	.7646	cubic metres (m^3)
Volume/Time (Flow)	cubic feet per second (ft^3/s)	28.317	litres per second (l/s)
	gallons per minute (gal/min)	.06309	litres per second (l/s)
Mass	pounds (lb)	.4536	kilograms (kg)
Velocity	miles per hour(mph)	.4470	metres per second (m/s)
	feet per second(fps)	.3048	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)	9.807	metres per second squared (m/s^2)
Weight Density	pounds per cubic (lb/ft^3)	16.02	kilograms per cubic metre (kg/m^2)
Force	pounds (lbs)	4.448	newtons(N)
	kips (1000 lbs)	4448	newtons (N)
Thermal Energy	British thermal unit (BTU)	1055	joules (J)
Mechanical Energy	foot-pounds(ft-lb)	1.356	joules (J)
	foot-kips (ft-k)	1356	joules (J)
Bending Moment or Torque	inch-pounds(ft-lbs)	.1130	newton-metres (Nm)
	foot-pounds(ft-lbs)	1.356	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)	$\frac{tF - 32}{1.8} = tC$	degrees celsius (°C)

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INTRODUCTION

A great deal of research on reinforced earth systems has been conducted in recent years to establish design procedures that are consistent with engineering theory and observed behavior. The late Dr. Kenneth Lee had reference to this objective when he stated: "Rational design procedures have been developed and demonstrated on many successful structures - however, from a basic research point of view, many fundamental details still remain to be worked out.... Although conceptually simple, the behavior of reinforced earth is actually very complex..." (1). The subject of this report is an earth reinforcement system utilizing a waste product as the reinforcing element, i.e., discarded automobile tire sidewalls.

The major components of the tire-reinforced embankment are soil-fill materials, tire sidewalls and their connecting elements, and a wrap-around wire mesh slope face. The tire sidewall mats are placed at predetermined vertical intervals within the fill mass. The connecting element is a reinforcing steel bar bent into the form of a clip. The wire mesh is secured to the tire mats.

The use of tire mats to resist tension and deformation in embankments and thereby strengthen the soil structure, can provide several significant benefits. For example, a highway embankment could economically be built without a conventional wall facing, utilizing slopes that are steeper than normal practice. A steeper slope design requires a lesser volume of fill material and allows embankment construction on a narrower base, thereby reducing right-of-way requirements.

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In addition to low-cost roadway construction and a method of disposing of unwanted used tires, the tire-reinforced embankment offers the potential advantage of increased resistance to damage from earthquake-induced ground shaking.

Phase I of this research study was a laboratory investigation of the stabilizing effects of including various solid non-biodegradable waste materials in four typical highway embankment soils. Waste materials included portions of discarded rubber automobile tires, broken glass containers and tin and aluminum cans. Compacted soil specimens with various percentages of waste materials in different layering systems were subject to large-scale triaxial compression testing to determine changes in shear strength parameters. These tests were performed under high stress conditions representative of high embankment construction (over 100 feet).

It was concluded that certain embankment soils could be beneficiated by increased shear strength provided by inclusion of relatively flat high tensile strength waste materials. Flattened tin and aluminum containers were found to provide the greatest stabilizing benefit and strength improvement. Chopped rubber tire inclusions were beneficial to moderately plastic, silty clay embankment soil with low angle of internal friction. However, this waste product was also found to lower the strength parameters of better quality embankment soils. Broken glass performed as an aggregate, but crushed under heavy loading, thus producing no significant gain or loss in shear strength with the typical embankment soils tested.

This Phase II research study was initiated to demonstrate that placement of tire mats at vertical intervals within

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the embankment will increase the strength of a given soil mass and allow a steeper slope face. In addition, the objective was to develop a design philosophy from the gathered field data and observed behavior for the construction of structurally safe and low maintenace reinforced earth embankments using tire mats.

SUMMARY AND CONCLUSIONS

During the late fall of 1976, an experimental embankment approximately 300 ft in length was constructed on Route 236 in the Santa Cruz Mountains utilizing tire sidewall mats as earth reinforcements. It was built to a maximum height of 50 ft using 1/2:1 side slopes, as an FHWA Demonstration Project. Instrumentation and monitoring were carried out under Phase II of an HPR research project entitled "Fill Stabilization Using Non-Biodegradable Waste Products" (D-3-44).

The use of tire sidewalls as earthwork reinforcements evolved from the results of Phase I of the above study, experience from past research by the department on earthwork reinforcement and a 1974 mandate by the State Legislature to study means of disposal of discarded automobile tires.

In the winter of 1978, following a period of almost unprecedented rainfall, a slipout involving approximately 5% of the reinforced embankment occurred, due primarily to an inadequate subsurface drainage system.

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Repair of the embankment in the late fall of 1978 consisted of the construction of a tire reinforced buttress, improved slope protection, and the installation of additional subsurface drainage. The facility has been performing satisfactorily since completion of the repair in January 1979.

The following are specific conclusions resulting from the study:

1. The suggested design procedures set forth in Sections I, D and E, are suitable and practical.

2. Instrumentation data and observed field behavior indicated that no tire mat slippage or stretching occurred back of the slip plane or passive zone in spite of the eventual saturated condition of the embankment.

3. The proposed design procedure for computing the resisting force of a clip under varying overburden soil pressures is consistent with observed field results.

4. The apparent friction coefficient between the soil and surface area of sidewalls for this embankment is about 0.36 as determined from the laboratory pullout test.

5. The 2 ft vertical spacing is about the maximum allowable spacing between sidewall mats to develop the pseudo-cohesive strength from friction between soil and the surface area of the sidewalls and resisting force of the clips. In addition, a 2 ft or less spacing is needed to properly construct a wire mesh facing to retain the soil at the slope during and after construction.

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6. The tire-reinforced embankment cannot be constructed with a 1/2 to 1 or steeper slope without an erosion-protective facing.

7. The assumption of a vertical face provides an active pressure coefficient, Ka, which is conservative for the sloped embankment condition.

8. The embankment behaved as an integral mass.

9. The dimensions of the tire-reinforced embankment must be sufficient to insure both internal and external stability. The embedment or adherence length of the tire reinforcing mat is not critical for internal stability when external stability is satisfied. For the test embankment, maximum height was dictated by the strength of the rebar clips.

10. No appreciable settlements occurred under this embankment following construction, except at the zone of the slope where failure eventually occurred.

11. Slack in the tire mats influenced the clip strain gage and sidewall extensometer readings. The calculated stresses did not agree with measured stresses.

12. The principal lesson learned from this study was the importance of positive subsurface drainage and erosion control for a tire mat reinforced embankment.

RECOMMENDATIONS

Six parameters must be considered in the design and construction of a tire-reinforced earth embankment. They are:

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1. Coefficient of active earth pressure

2. Apparent friction coefficient between the soil and surface area of sidewall

3. Sizes of tire sidewalls and clips

4. Vertical spacing of the reinforcing mats

5. Embedment length of reinforcing mats

6. Potential soil erosion of the embankment slope face.

The use of Rankine's active pressure coefficient, Ka, with an assumed vertical face is recommended for design calculations. This assumption offers an added factor of safety if soil erosion occurs on the slope face.

Experience from this project indicates that a single tire rim diameter should be specified for the sidewall to provide a tighter sidewall/clip mat system. This will minimize slack in the mats.

The design procedure set forth in Equation 3 is recommended for calculating the resisting traction force of the sidewalls. The apparent coefficient of friction recommended is tan(2/3Ø) with a maximum of 0.40.

The design procedure proposed in Equations 4 and 5 is recommended to calculate the resisting traction force of the rebar clips. The minimum recommended rebar clip diameter is 3/8-inch. Clip design should consider both strength and corrosion loss for a specific service life.

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A maximum 2 ft vertical spacing for the reinforcing mats is recommended. The mats should extend to a sufficient embedment length to insure external stability.

A wire mesh or chain link mesh similar to that shown in Fig. 39 with aggregate or rock backfill is recommended to prevent soil erosion of the 1/2 to 1 slope. Steeper slopes may be considered if efficient slope protection systems can be utilized.

IMPLEMENTATION

The findings from this research project have been implemented in the design of one proposed tire-reinforced embankment on Route 39 in Los Angeles County. Other possible sites are under consideration in our Transportation District 09, north of Bishop; and in the San Francisco area (District 04).

SECTION I

A. BACKGROUND

In recent years, engineers have developed new techniques and adapted a variety of materials to earth reinforcement. The first proven and generally accepted method was reinforced earth, a patented construction procedure developed by the French Engineer Henri Vidal in the late 1950's (2). Vidal's system consists of a wall facing and compacted soil backfill reinforced with steel strips at predetermined intervals within the fill mass. The development of friction between the soil and surface area of the strips provides

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pseudo-cohesive strength within a relatively cohesionless backfill. This holds the soil together as an integral mass. For a soil to be satisfactory for reinforced earth construction, Vidal suggests that it be predominantly granular with a minimum angle of internal friction of 25 degrees. The wall facing, usually consisting of interlocking steel or concrete panels, serves to tie the system together and prevent local sloughing and erosion.

From the strip method introduced in the United States in late 1960's, other earth reinforcement concepts evolved; including the use of rebar welded mesh, wire mesh, fabrics, and nets. The purpose of all such systems is to increase the strength of the soil mass so as to permit vertical side slopes. The first reinforced earth project in the United States was constructed in 1972 by the California Department of Transportation on Route 39 in Los Angeles County (3).

The concept of tire sidewall reinforcement was derived from a California Transportation Laboratory research study in 1973. That study indicated that the inclusion of certain high tensile strength non-biodegradable materials increased the strength of the soil mass. The initial Phase I laboratory study was described in an interim report published in 1973 (4).

The Phase II study was also stimulated by a perplexing problem in California, and throughout the nation: the disposal of discarded automobile tires. It has been reported that approximately 200 million tires are discarded each year in the United States. This problem was of such magnitude that it prompted House Resolution #37 in the

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1973 California Legislative Session, which charged the California Department of Transportation to study the problem of abandoned tires and develop possible solutions for their disposal or recycling. It was speculated that tires could be disposed of in highway embankments and could also serve to increase the strength and stability of the soil element.

The major problems found in the burial of whole tires in landfill is that the tires have a tendency to work up to the surface and that soil is difficult to compact around the tires. However, these problems were resolved with the use of newly-developed equipment which separates the tire treads from the sidewalls. It was reasoned that sidewalls placed in strips with ties or laid and secured in mats would eliminate the compaction problem.

The sidewall concept of soil reinforcement was encouraged further by the results of a theoretical study to determine the effects of tire reinforcement on earthquake resistance of embankments. The analysis was accomplished with the QUAD-4 finite element program developed at the University of California, Berkeley. In addition to embankment geometry and mat placement, the elastic properties of the soil and tire reinforcements were introduced into the problem with a dynamic loading simulating a moderate intensity earthquake.

The results revealed a reduction in the dynamic shear stress and strain of about 33 percent by the inclusion of tire mats. The above analysis is treated in some detail in "Use of Waste Materials in Embankment Construction" (5).

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Thus the initiation of Phase II of this research project coincided with a California State Legislative mandate to investigate the problem of used tire disposal. The satisfactory performance of the reinforced earth embankment on Route 39 in Los Angeles County and the results of the QUAD-4 dynamic analysis provided further impetus for a field performance study to evaluate tire sidewalls as soil reinforcement.

In early 1976, a suitable location for an experimental embankment utilizing tire sidewall reinforcement was found in the vicinity of the redwood basin, near Santa Cruz. The site was a sidehill fill slide correction project located in a remote area, to which hauling imported fill material would be expensive and where limited right-of-way existed. The decision to use tire sidewalls as soil reinforcement was made after considering four different slide correction construction methods. Total cost estimates were based on 16,000 sq ft of face area and are presented in Table I. The cost reflected a requirement that complete excavation of the old embankment would be necessary to install a new subdrainage system.

TABLE 1 COST COMPARISONS OF SLIDE CORRECTION METHODS.

Tire Reinforcement (1/2:1 Slope)	Crib or Bin Wall	Reinforced Earth	2:1 Slope Fill
$11.60/sq ft	$22/sq ft	$15/sq ft	$25/sq ft
$186,000	$352,000	$240,000	$400,000
actual	estimated	estimated	estimated

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B. SITE DESCRIPTION

The project is located on Route 236, between Boulder Creek and Big Basin in Santa Cruz County, as shown in Figure 1. Route 236 is a two-lane highway which traverses a dense redwood-covered mountainous area. Adjacent to state parks and other recreation areas, this route experiences moderate to heavy passenger car traffic use, particularly in the summer months. Several years previously, a landslide caused by subsurface water movement occurred at PM 7.26. The loss of most of one lane and subsequent repairs resulted in an unusually tight curve. In 1976, Transportation District 04 (San Francisco) scheduled reconstruction of 350 ft of the embankment section to improve stability and alignment.

The profile is a vertical curve from Elevation 398 at Station 10+00 to Elevation 382 at Station 13+50. The new and old alignments are shown in Figure 2. A view of the site prior to reconstruction is shown in Photo 1.

At the critical section, Station 12+00, the height, top width, and base width (Elevation 343) of the fill are about 49, 40, and 27 ft, respectively. The average fill height is about 42 ft between Stations 11+50 and 12+50. At the base the width varies from 22 to 39 ft. At the top, the width varies from 25 to 44 ft. A typical cross-section is shown in Figure 3.

C. SOIL PROPERTIES AND STRENGTH PARAMETERS

In 1976, a soil investigation of the site was conducted with a hand-pushed 2-inch California sampler. The borings were made along the slope and at the toe of the existing

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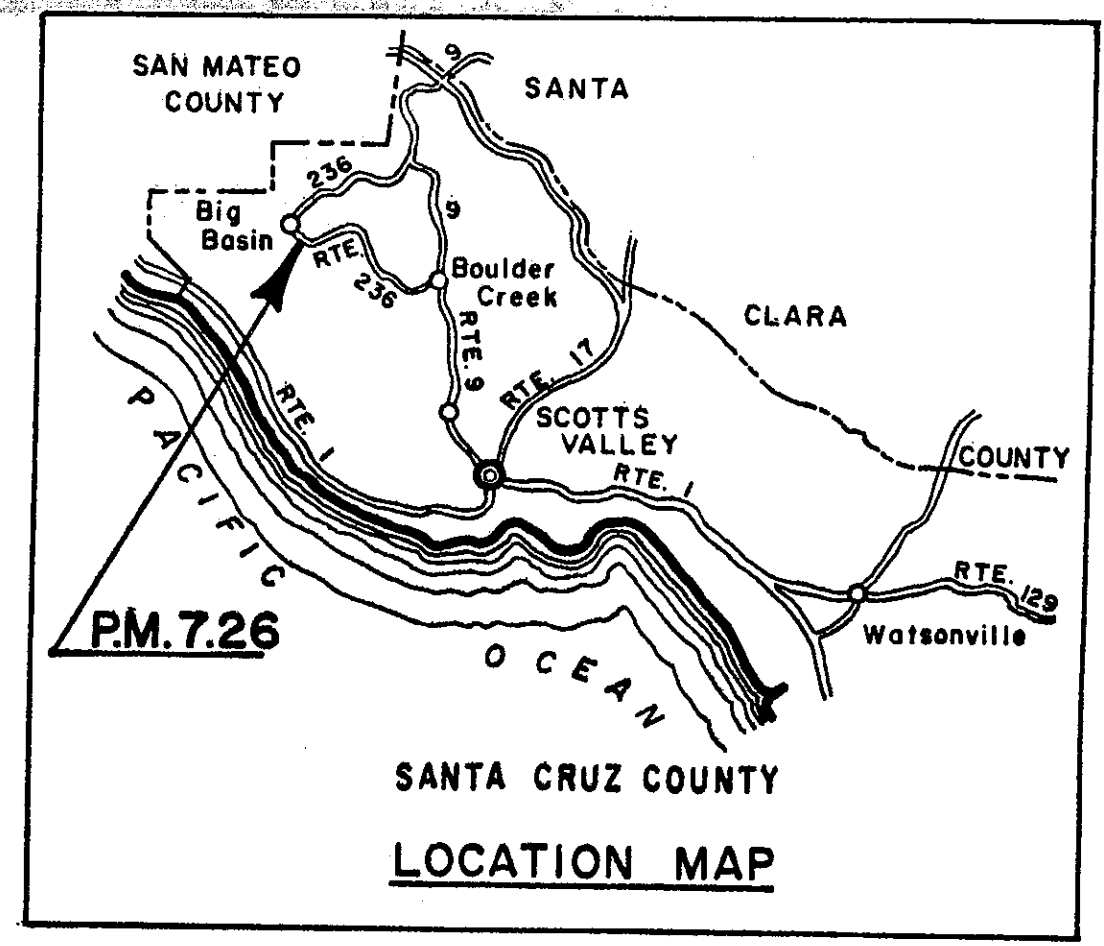

Fig. 1 SITE LOCATION

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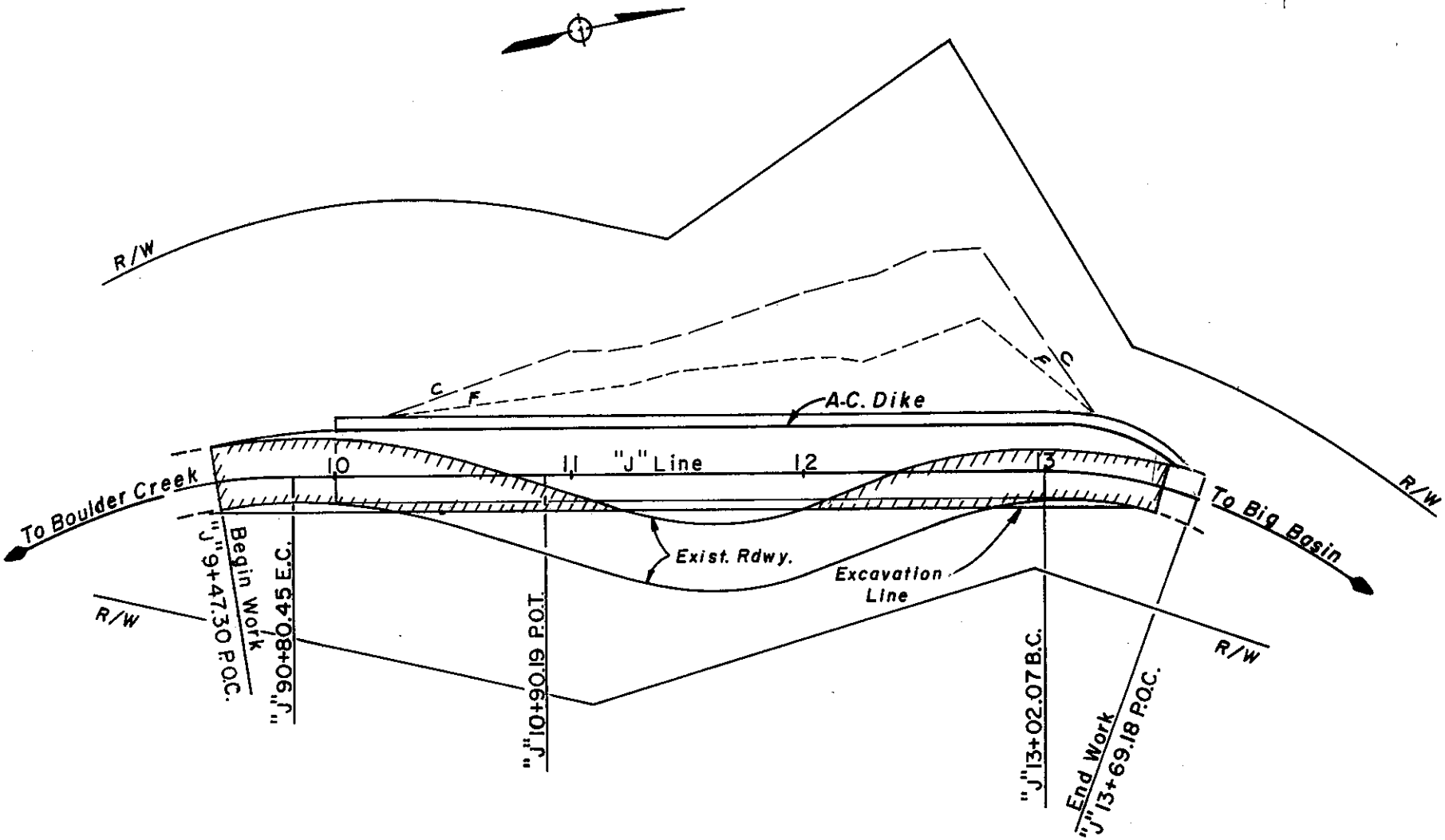

FIG. 2 SITE PLAN

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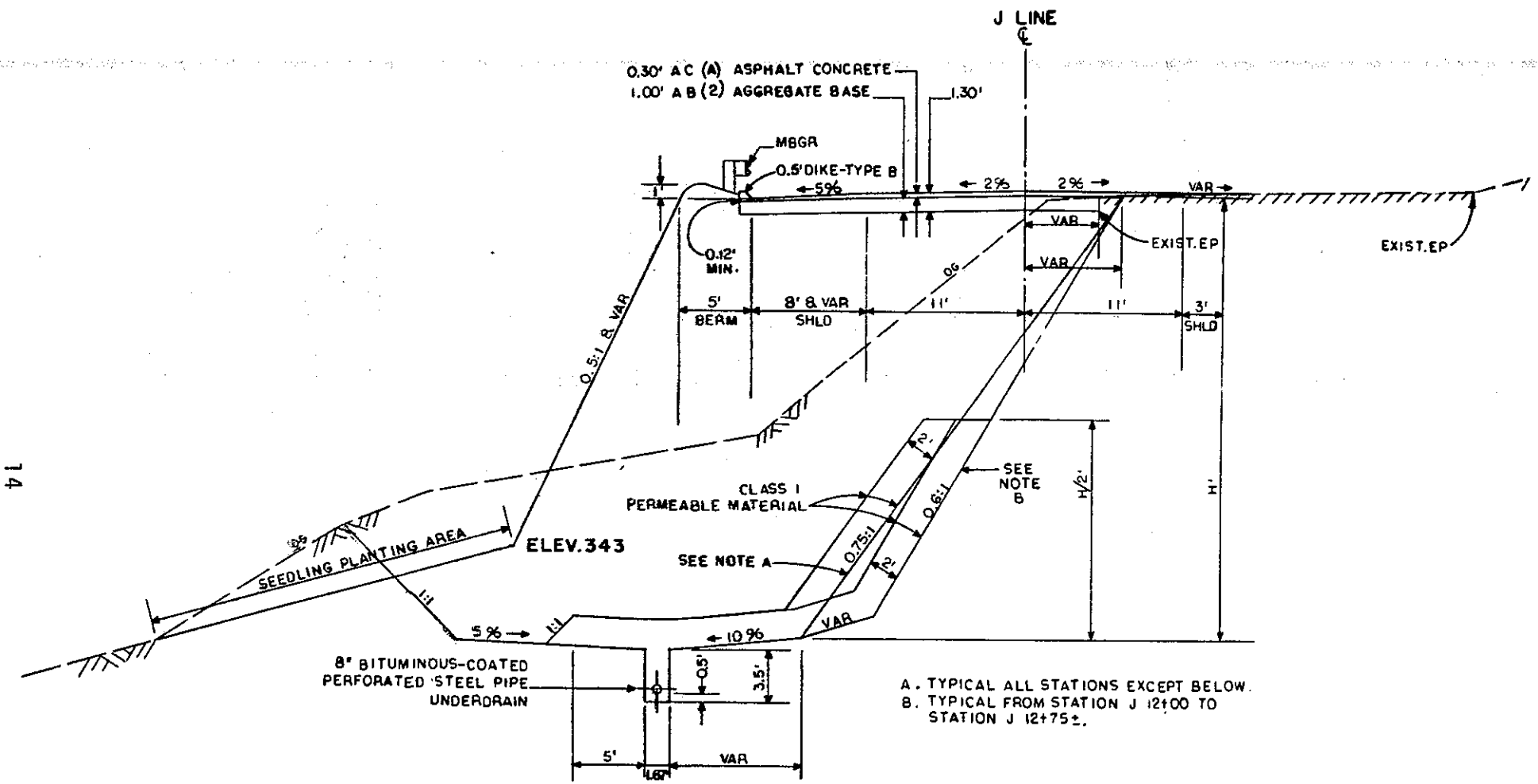

FIG. 3 TYPICAL X-SECTION NEW EMBANKMENT

embankment. The existing embankment slide material was to be used in construction of the new tire reinforced fill.

The existing embankment and foundation soils consisted of moist, firm to dense, brown, gravelly clayey silt with interbedded layers of dense sand. Laboratory analysis indicated the soil unit weight to be about 115 pcf with moisture contents averaging about 15 percent. Figure 4 shows the grain size distribution of the hand boring samples of the existing material. Groundwater was not encountered.

The upper 4 feet of the experimental embankment was to consist of imported material, the gradation of which is also shown by Figure 4. Consolidated undrained (CU) triaxial tests (Calif. Test Method 230) were performed on six hand-pushed undisturbed embankment samples. For each test, a confining pressure was first applied to a specimen and maintained overnight, before applying a deviator stress to failure at a controlled rate of strain of 0.06 inches per minute. Specimens were subjected to confining pressures of 1/2, 1, and 2 tons per sq ft, respectively. The six specimens were not saturated in the laboratory prior to the tests. The resulting strength parameters (c, and Ø), the cohesion intercept, and angle of internal friction are presented by Table 2. The Mohr strength envelopes are shown by Figure 5.

In addition, CU triaxial tests were performed on remolded embankment and imported fill samples compacted to 90 percent relative density. The remolded specimens consisted of both saturated and unsaturated samples, which

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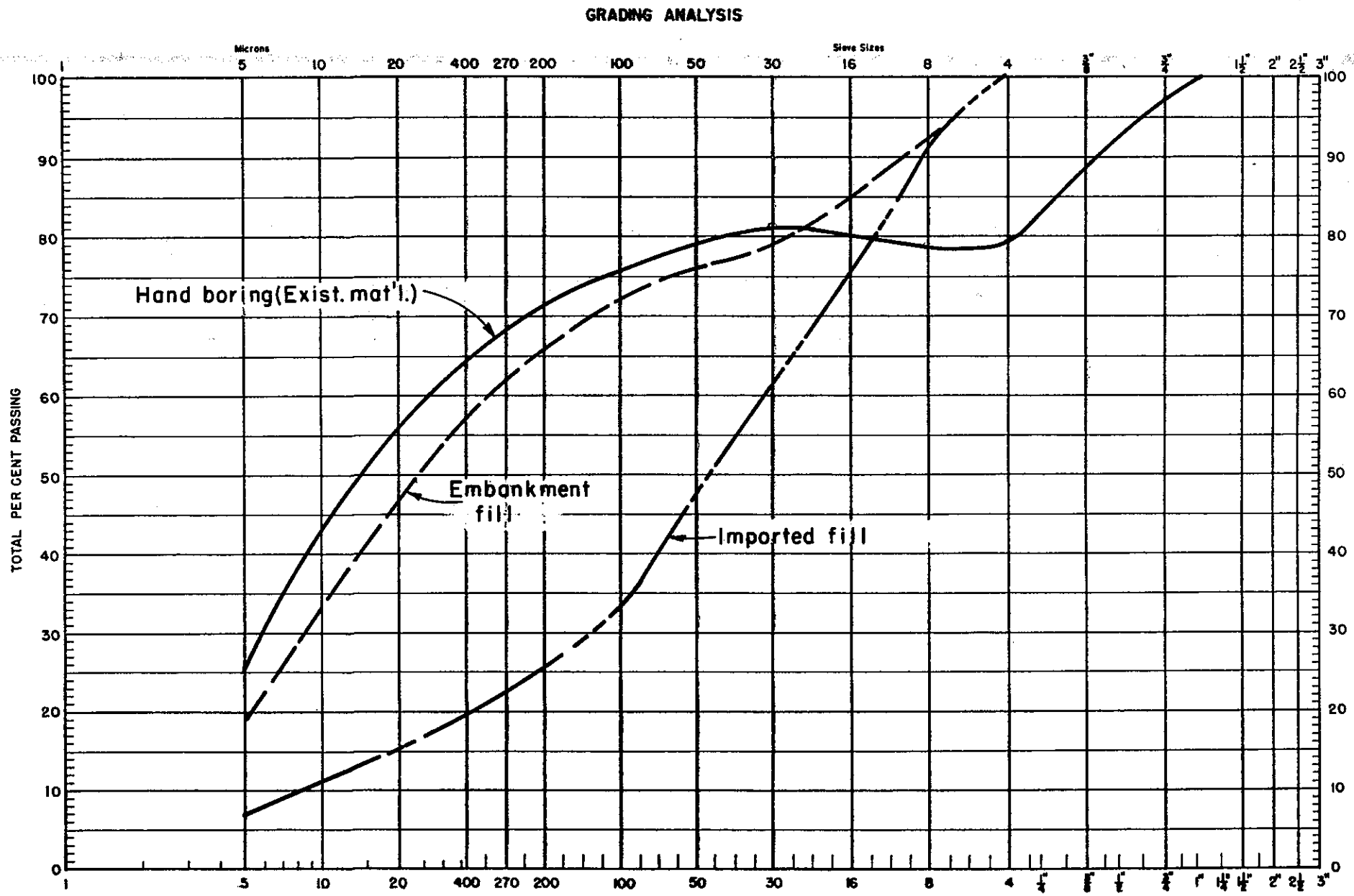

FIG. 4 GRAIN SIZE DISTRIBUTION

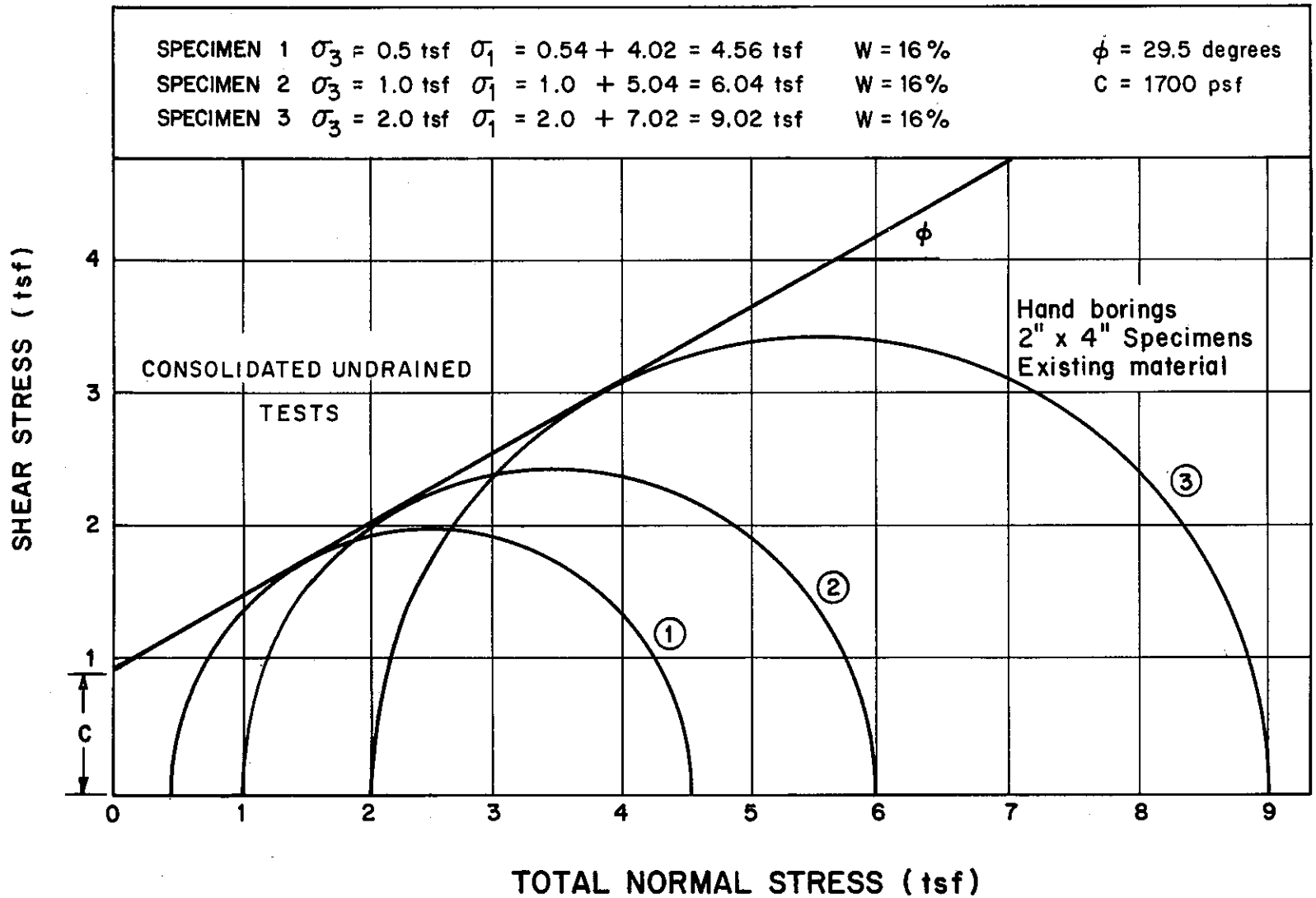

FIG. 5 MOHR STRENGTH ENVELOPE FOR EXISTING MATERIAL

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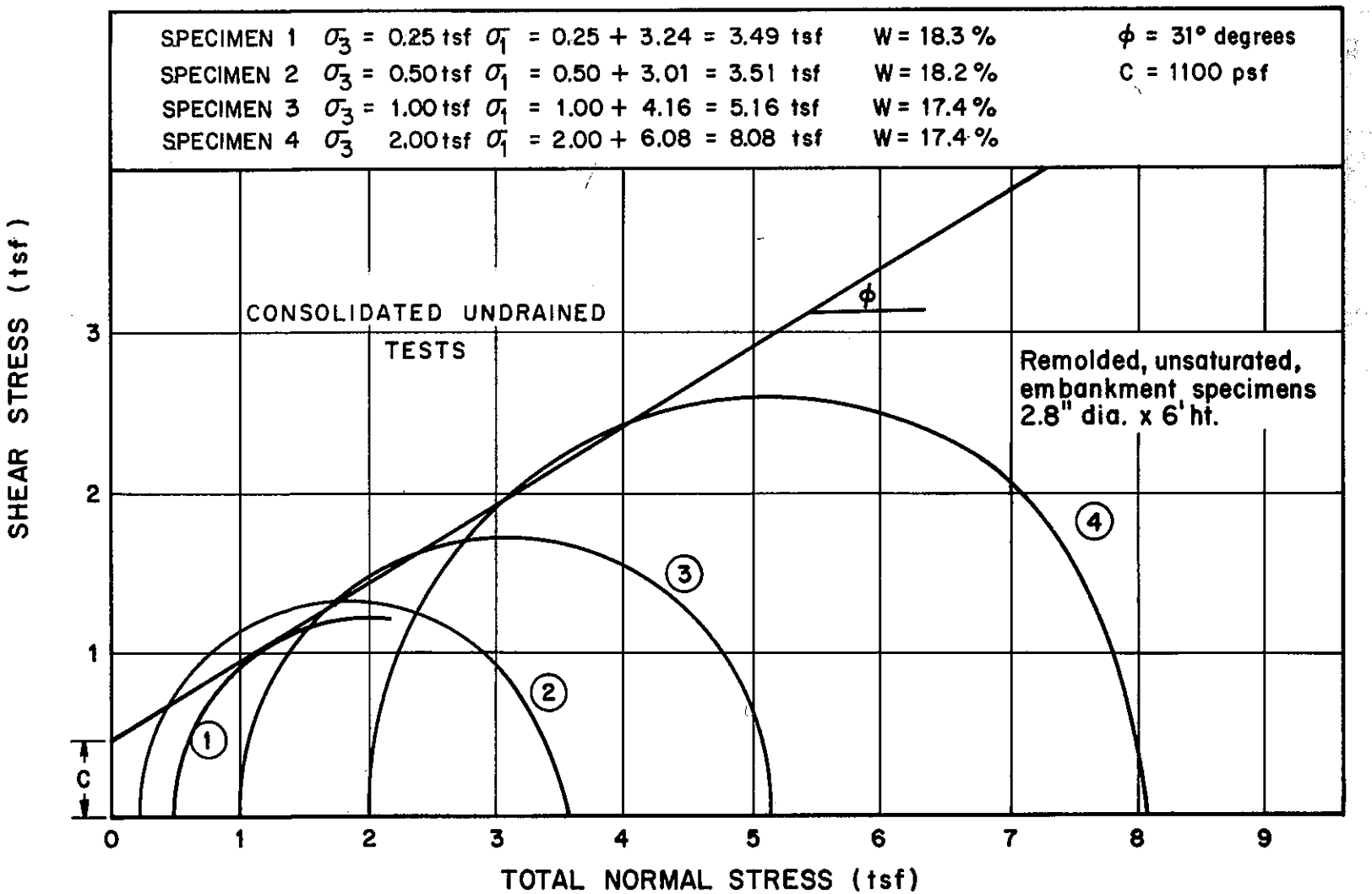

FIG. 6 MOHR STRENGTH ENVELOPE FOR EMBANKMENT FILL

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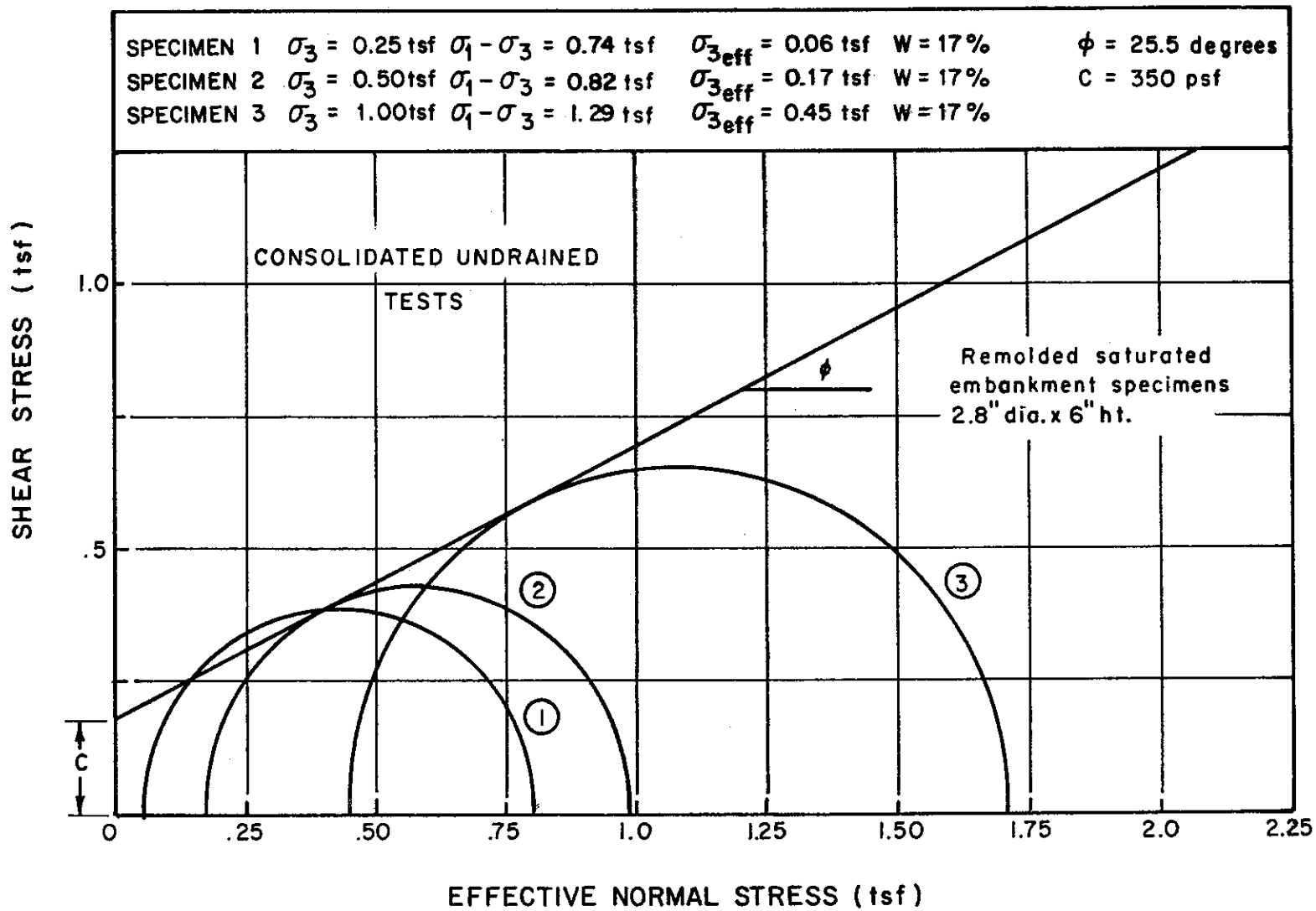

FIG. 7 MOHR STRENGTH ENVELOPE FOR EMBANKMENT FILL

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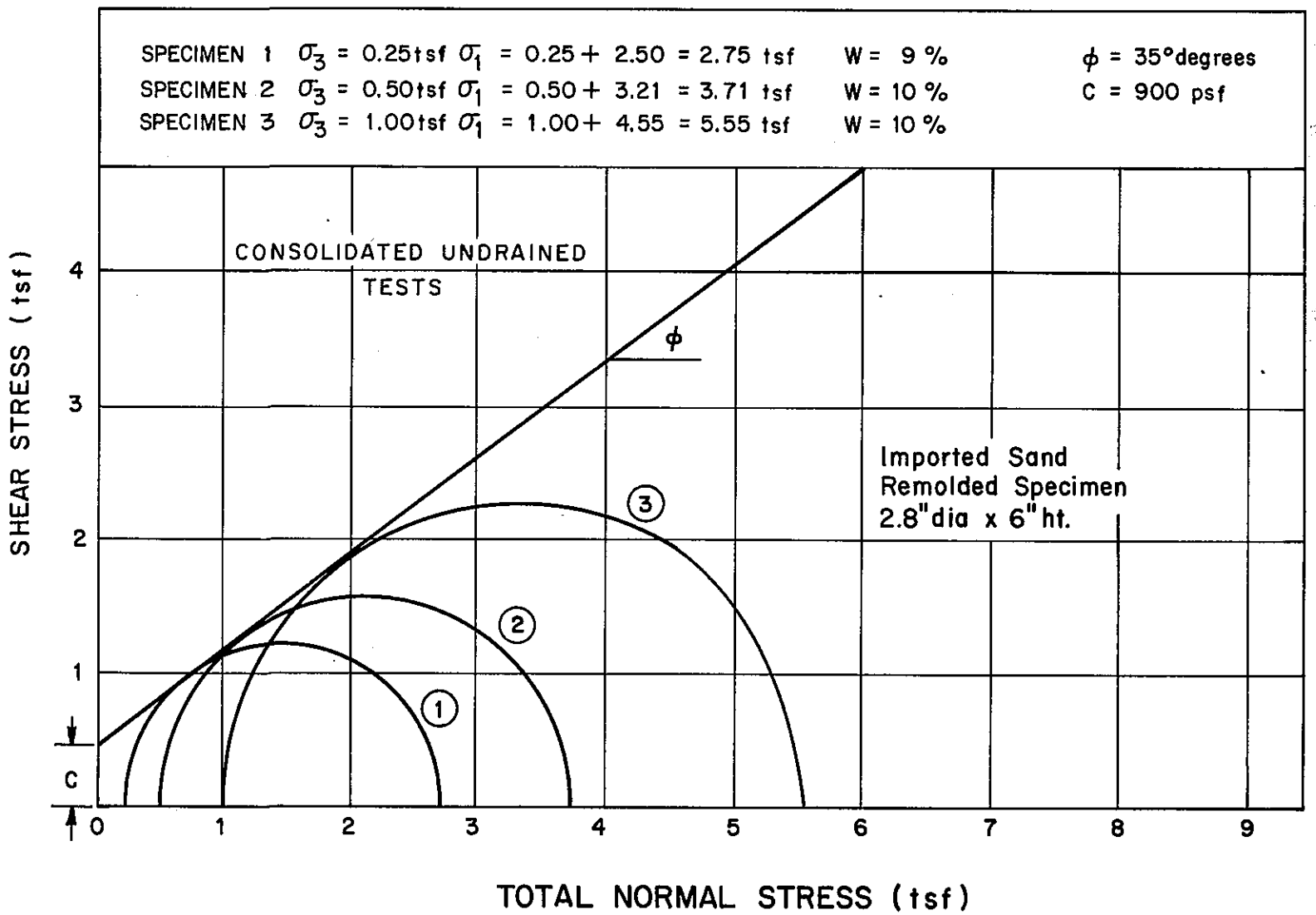

FIG. 8 MOHR STRENGTH ENVELOPE FOR IMPORTED FILL

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were subjected to 1, 2, and 4 tons per sq ft confining pressure, respectively. The unit weight of these specimens ranged from 120 pcf to 130 pcf. The imported fill specimens were not saturated in the laboratory and were subjected to 1/4, 1/2, and 4 tons per sq ft confining pressure. The resulting strength parameters are presented by Table 2. The Mohr strength envelopes are shown by Figures 6 through 8.

TABLE 2 SUMMARY OF STRENGTH PARAMETERS (CU TESTS)

	Ø degrees	c psf	Range of Confining Pressures (tsf)
Hand borings (Existing Material)	29.5	1700	1/2 to 2
Remolded fill	31	1100	1/4 to 4
Remolded fill (Saturated)	25.5	350	1/4 to 1
Imported fill	35	900	1/4 to 2

D. PRACTICAL DESIGN CONSIDERATIONS

The design concept of reinforced earth as presented by Vidal (2) assumes a composite material consisting of tensile reinforcement and sandy backfill. The reinforcement when properly oriented will significantly increase the shear strength of the composite material compared to that of unreinforced natural soils. This is accomplished by pseudo-cohesion developed from friction between the soil and the reinforcements. In simplified terms, the behavior can be explained by a soil element as shown in Figure 9.

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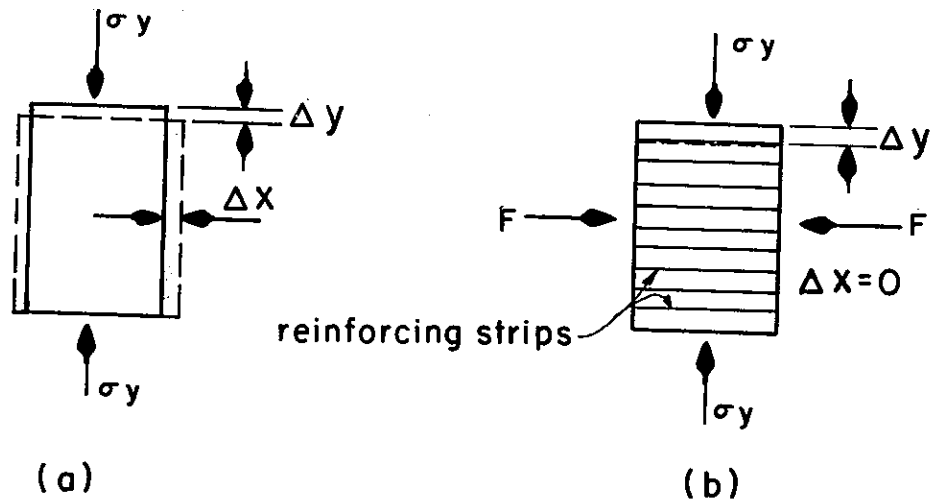

FIG. 9 SOIL ELEMENT

In Figure 9a, a lateral strain (ΔX) occurs when a vertical load is placed on the element due to dilation of the granular material. In Figure 9b. lateral strain cannot occur due to the reinforcing strips placed within the soil mass unless pull-out or tension break of the strip occurs. For vertical walls, it is necessary to provide a wall facing to prevent soil sloughing and erosion at the face. However, Hausman and Lee concluded from their study that a definite reinforcing effect exists without a wall facing (6).

1. Lateral Earth Pressure

In conventional reinforced earth walls, the state of stress of the lateral earth pressure depends on the validity of two assumptions. The first is that, initially, the wall will yield a sufficient amount to develop the maximum shearing

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strength of the backfill material. The second is that the strength of the backfill, once developed, will remain reasonably constant with the continuing wall movement. The state of shear failure is called the Rankine active pressure state (Ka).

Vidal reasoned that soil in contact with the long reinforcing strips in the upper part of the wall is in an at-rest state (Ko). As the soil confined by the wall facing and strips cannot expand laterally, the resulting horizontal shearing stresses are relatively small. For the lower part of the wall, horizontal shearing stresses are much greater due to higher overburden pressures. The soil here is in an active state (Ka).

Chang (3) and Schlosser (7) have shown that the state of stress in reinforced earth structures with wall facing cannot be predicted accurately on the basis of a single earth pressure coefficient. This has been demonstrated by data gathered from instrumented reinforced earth structures. Obrcian studied the influence of gradation, relative density, vertical stress level, and loading history on the coefficient of earth pressure (8). These factors were found to have a pronounced influence on lateral stresses. The value of Ko in particular, decreases with increase in vertical stress level (8).

A tire-reinforced embankment is somewhat different in its response to load. Although the flexible wire mesh facing effectively retains the soil during compaction procedures, it affords little resistance to lateral deformation of the soil. Some slack can also be anticipated in the rebar clip/side wall linkage of the tire mats. Until the slack has been taken up, some lateral expansion of the soil can

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be expected. This lateral soil expansion between layers of reinforcement would be sufficient to develop large horizontal shearing stresses resulting in an active state of stress condition within the embankment. Therefore, a single earth pressure coefficient may be appropriate for the embankment design.

For a steep-sloped embankment, the value of Ka may be calculated from the Mueller-Breslau equation (9). The equation is given below. Figure 10 defines the angles.

$$K_a = \frac{\cos^2(\phi-\beta)}{\cos^2\beta\,\cos(\delta+\beta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\omega)}{\cos(\delta+\beta)\cos(\omega-\beta)}}\right]^2} \quad \text{Eq. (1)}$$

where: Ka = coefficient of active earth pressure
ϕ = angle of internal friction
δ = angle of wall friction
β, ω = angles as shown in Fig. 10

As described in Section I(B), the fill material for the subject project was a fine-grained soil that can exhibit potential creep. Other potential problems were taken into account, such as environmental conditions (heavy rainfall), slope saturation, erosion, and instability. Since this project involved a system somewhat divorced from conventional reinforced earth, and for which there was no past experience, a conservative design approach was utilized. An active pressure state with a vertical face was assumed.

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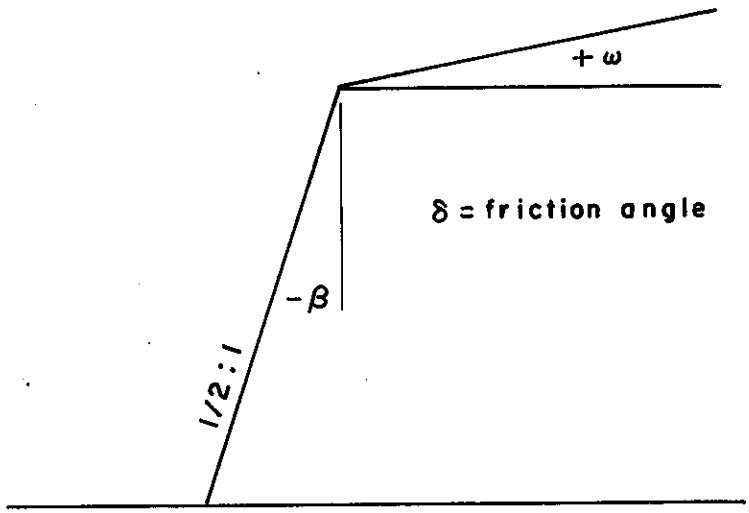

FIG. 10 MUELLER-BRESLAU ANGLES

2. Traction Forces in Sidewalls

As discussed previously, traction forces in the reinforcement produce horizontal shearing stresses in the earth as a result of adherence between soil and reinforcing strips. For conventional reinforced earth walls, the traction forces and tensile stresses in each layer of reinforcement are calculated independent of the stresses exerted in the other layers. A local equilibrium condition between wall facing and the layer of reinforcement must be achieved at the level under consideration on the assumption that the earth between the reinforcement is in a state of incipient failure. The vertical stress in the horizontal plane is assumed to have a uniform

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distribution. For a vertical wall, the direction of the principal stresses on a soil element are vertical and horizontal.

Chang (3) and Schlosser and Long(10) have shown that the maximum stress developed in the reinforcing strips does not always occur at the face of the wall. Al-Hussaini and Perry confirmed this distribution by measurements of the tensile forces in instrumented reinforcing ties (11). They also concluded that the maximum tensile stress that occurred in the reinforcing ties did not coincide with the theoretical Rankine failure plane. Hausman and Lee concluded from their study that the length of the ties affects the inclination of the failure plane (6). Schlosser and Long indicated that the points of maximum tensile stress separate the structure into two zones, as shown in Figure 11. These consist of: 1) an active zone, in which horizontal shearing stresses exerted by the earth on the reinforcements are directed outward towards the face; and 2) a passive zone, in which the horizontal shearing stresses are directed inwards from the face.

The length of reinforcement in the latter zone is defined as the adherence length. The boundary separating the two zones is dependent on geometry of the mass, foundation conditions, the magnitude and location of applied external loading, and the friction between the earth and reinforcements (10).

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The general formula for the traction force, T, assuming a constant vertical spacing, may be expressed as:

$$T = K\gamma H d \Delta h \qquad \text{Eq. (2)}$$

where K = coefficient of earth pressure
γ = unit weight of soil
H = depth of reinforcement below backfill level
d = horizontal spacing of reinforcement
Δh = vertical spacing of reinforcement.

The resisting traction force, F, may be expressed as:

$$F = (\gamma H)(2A)\tan \delta \qquad \text{Eq. (3)}$$

where A = surface area of reinforcement
$\tan \delta$ = apparent coefficient of friction between reinforcement and soil.

For a tire-reinforced embankment with the absence of a wall facing, the distribution of traction forces along the reinforcements is somewhat more complex and has not been determined. However, the basic design assumptions utilized for calculation of reinforcement traction forces for reinforced earth systems are believed to be applicable to tire sidewall-reinforced embankments.

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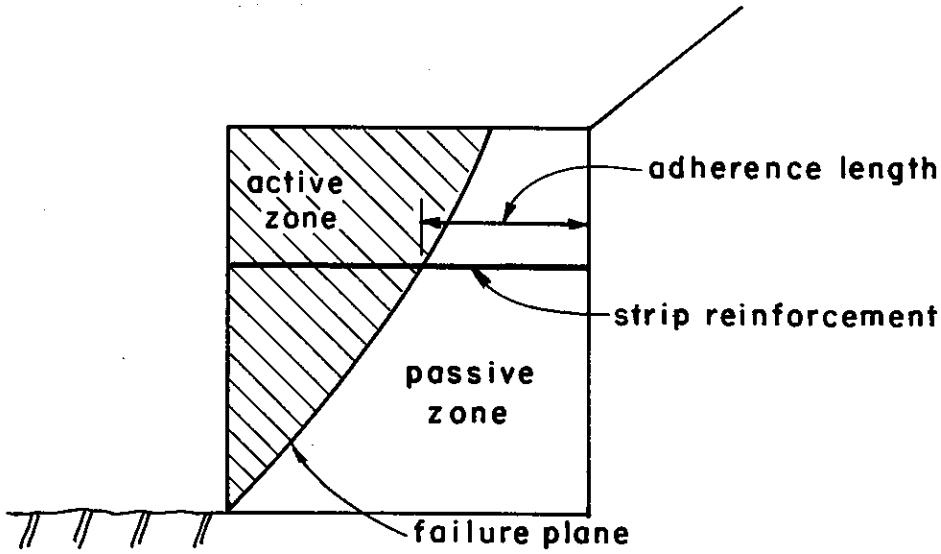

FIG. 11 ACTIVE AND PASSIVE ZONES WITHIN REINFORCED EARTH STRUCTURES (AFTER SCHLOSSER & LONG, 1974)

3. Friction Between Earth and Sidewall

The development of adequate friction between the soil and tire reinforcement is dependent on the characteristics of the surface of the sidewall, the backfill and the backfill density, and overburden pressures. Bacot, et al, concluded in their study that the apparent friction coefficient obtained from shear box tests is the minimal value for soil reinforcement (12).

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A pull-out test was conducted for this project using a special shear box for determining the apparent friction coefficient between the proposed embankment soil and the surface of the sidewalls. A 14-inch diameter automobile tire sidewall was placed in the shear box between two 9-inch layers of soil compacted to a relative density of 90 percent. The soil was similar to the material at the site. Photo 2 shows the sidewall resting on the first layer of soil. An overburden pressure equivalent to that imposed by 25 ft of embankment was exerted on the upper layer of soil with a hydraulic jack. The small exposed portion of the sidewall was pulled with a hydraulic jack and displacement versus pull-out force was measured and recorded at various tensile loads and as plotted in Figure 12. The pull force at yield strength was found to be about 4800 lbs. The sidewall failed at about 8500 lbs. At yield strength, the calculated apparent friction coefficient from Equation 3 was found to be 0.36 which is equivalent to a friction angle of 20 degrees.

This pull-out test demonstrated that the embedded sidewall could not be pulled out of the soil; therefore, any system failures would be as a result of tension break of the sidewall or the connection system.

4. Strength of Tire Clips

Originally, the use of heavy duty commercial staples was considered as a method for connecting the sidewalls into mats. However, laboratory trials of stapled connections using both 14- and 16-gauge staples, in combinations of 2 and 4 at each connection, failed to develop sufficient

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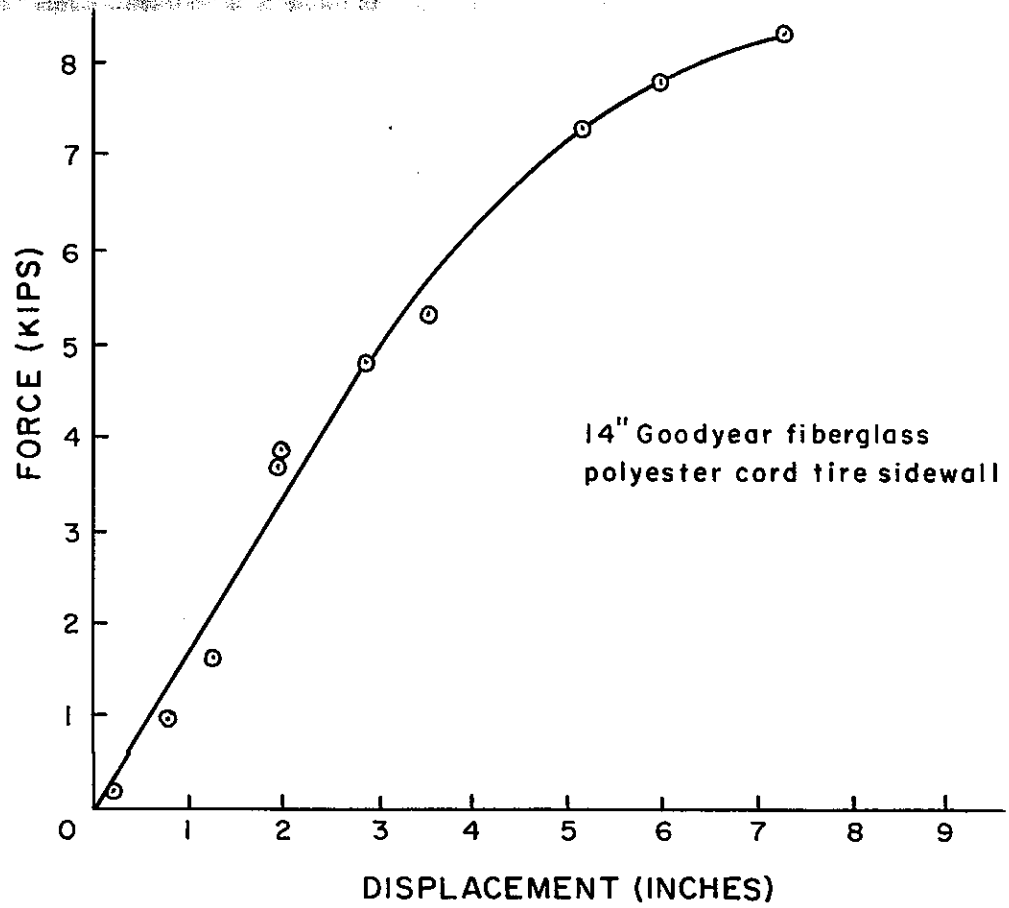

FIG. 12 SIDEWALL PULL-OUT TEST

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resisting force to mobilize the inherent tensile strength of the sidewalls. Other objectionable features of the stapled connection, apart from its lack of strength, were the necessity of overlapping the sidewalls and limited corrosion life. Mats so constructed would have required many more tires than the pattern eventually selected.

When the staples proved to be unsatisfactory, a No. 3 steel rebar clip was suggested. Details of the rebar clip bend are shown in Figure 13. Consideration was also given to using a No. 2 rebar. However, No. 3 rebar was selected to provide a longer corrosion life. (Discussion on the corrosion of the rebar clips follows this section.)

Pull-out tests were conducted to determine the resisting force of the tire clip before rebar yielding. The tests were conducted with the clip embedded in a layer of well-compacted, fine to medium sand in the shear box and subjected to pressure equivalent to 48 ft of overburden pressure. The sand layer thickness was 9 inches below and above the clip. The clip was placed in a vertical plane with the hook ends down. One end of the clip was fastened to the rear of the shear box. A pulling force was applied at the other end with a heavy duty chain. The pulling force was found to be about 6500 lbs at the yield point. The failure mode was by yielding at the bend of the clip.

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A second pull test was performed in air with a Universal Testing Machine. As anticipated, the failure mode was also by yielding at the bend of the clip under a load on the unconfined clip of about 700 lbs.

Results of both pull tests led to a rational method for calculating the resisting force of the clips at any given depth in an embankment. The procedure is based on the general bearing capacity equation with a straight-line Mohr envelope as shown below in Equation 4(13).

$$q_c = \frac{\gamma b}{2} \tan^5\alpha + 2c(\tan\alpha + \tan^3\alpha) + q\tan^4\alpha \qquad \text{Eq. (4)}$$

Where q_c = bearing capacity (psf)

$\alpha = 45 + \phi/2$

ϕ = angle of internal friction from laboratory triaxial tests (degrees)

c = cohesion intercept from laboratory triaxial tests (psf)

$q = \gamma H$

H = depth of clip below backfill surface (ft)

b = width of rebar (in.)

γ = unit weight of soil (pcf)

Examination of the bearing equation for a rebar indicates the first term has a very small value and can be neglected. The procedure to calculate the resisting force of the clip with the remaining two terms in Equation 4 is illustrated in Figure 13.

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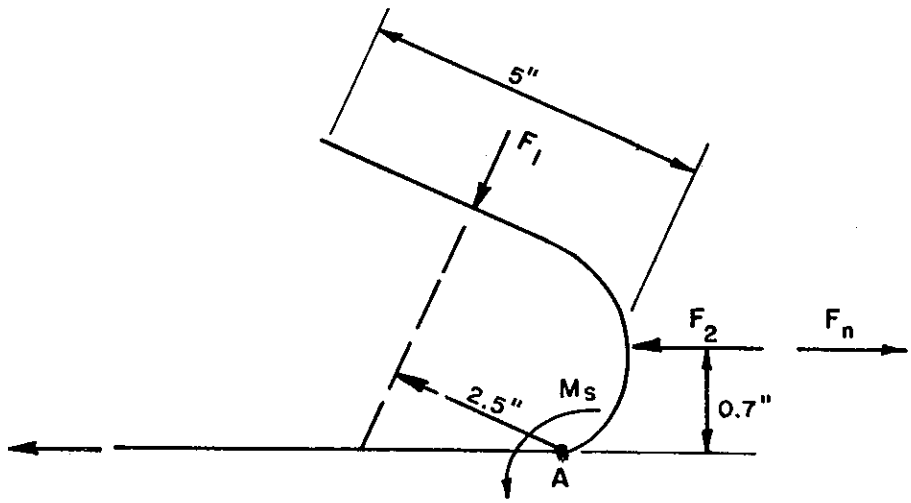

FIG. 13 FORCES ON TIRE CLIP

Given a height of fill above a clip, the forces F_1 and F_2 can be computed from Equation 4, where:

$$F_1 = q_c \times \frac{3/8 \text{ inch} \times 5 \text{ inches}}{144 \text{ in}^2/\text{ft}^2} \quad \text{(lbs)}$$

$$F_2 = q_c \times \frac{3/8 \text{ inch} \times 0.7 \text{ inch}}{144 \text{ in}^2/\text{ft}^2} \quad \text{(lbs)}$$

The resisting force, F_n, of the clip may be obtained from a summation of moments about point "A" divided by the lever arm of 0.7 inch. The internal resisting moment M_s of the rebar was assumed to be 700 lb-in. Thus,

$$M_A = F_1 \times 2.50 \text{ inches} + F_2 \times 0.7 \text{ inch} + 700 \text{ lb-inch}$$

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and,

$$F_n = \frac{M_A}{0.7 \text{ inch}} \qquad \text{Eq. (5)}$$

Based on Equation 5 and assumed strength parameters ϕ and c, the resisting force of the clip at various depths below the top of the backfill may be computed. A chart showing resisting force of clip versus depth will be shown later in this report (Figure 16).

5. Corrosion of Tire Clips

As mentioned earlier, the effect of corrosion on the clips was considered in determining the size of the rebar required. The California Transportation Laboratory estimates relative soil corrosivity by combining the relative influences of the hydrogen-ion concentration (pH) and the minimum resistivity of the soil(14). A chart developed for estimating underground corrosion rate of steel pipe (shown in Figure 14) presents the relationship between corrosion rate, minimum soil resistivity (ohm/cm), and pH values for acid and alkaline soils.

Samples of soil at the construction site scheduled for embankment placement were tested for potential corrosivity. The material was found to be acidic with a pH value of 5.5, and an average resistivity of 2400 ohms per centimeter. These data when applied to the chart, Figure 14, indicated that the metal loss would be about 10 mils per year. This can be interpreted also as a weight loss of about 0.8 oz. per sq ft, per year. Based on this information, the No. 3 rebar was estimated to have an effective life of over 40 years.

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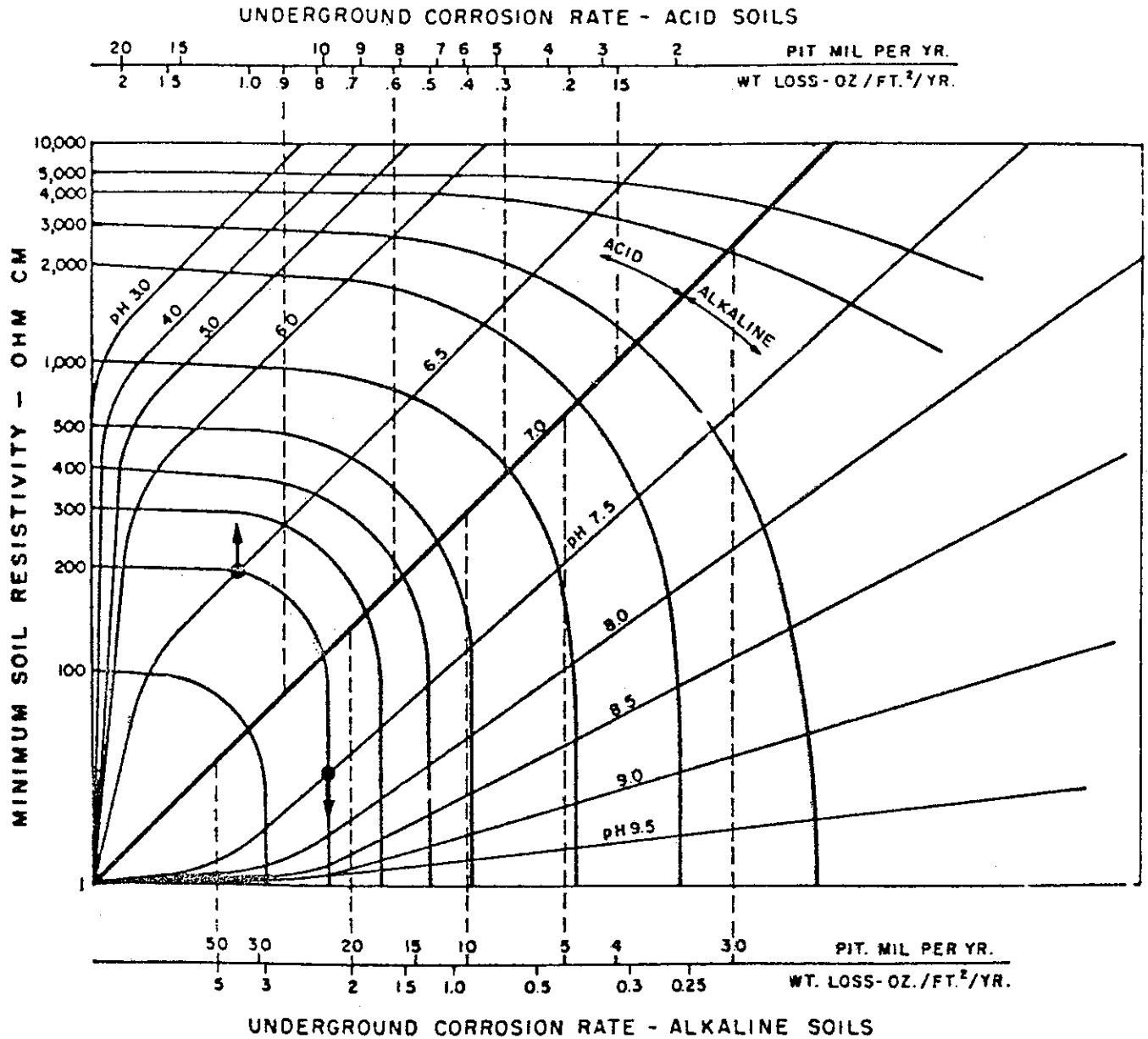

Figure 14—Chart for estimating corrosion rate of steel pipe. Example 1: with pH of 6.5 and resistivity of 200 ohm cm, the underground pipe loss is about 1.0 oz sq ft yr and the pitting rate about 0.013 ipy. Example 2: with pH of 7.5 and resistivity of 200 ohm cm, the underground pipe loss is about 2.2 oz; sq ft yr and the pitting rate is about 0.022 ipy.

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E. DESIGN OF TIRE-REINFORCED EMBANKMENT

In the design of the tire embankment, both external and internal stability must be investigated. Internal stability considers two failure modes, i.e., sidewall pull-out and clip tension break. As described in Section D, test results indicated that the governing failure mode is by clip tension break (yielding at the bend). Therefore, the tire-reinforced embankment design requires that the resisting clip and corresponding sidewall traction forces be able to resist the lateral earth pressures at all points within the embankment.

Figure 15 shows the tire embankment with a level backfill and frictionless wire mesh facing. The assumed failure wedge shown in Figure 15 is subjected to two sets of forces: earth pressure, and sidewall-mat traction forces. The distribution of earth pressures and traction forces with respect to depth is assumed to be linear. The mat is assumed to be pulled directly outward in the active zone or failure wedge. The wedge is assumed to be at equilibrium under its own weight, W; reaction, R, along the surface BC; and the sum of all the resisting forces, F_{n+i}, to the resultant earth pressure. How the wedge is influenced by the sidewall reinforcement has not been determined.

The active earth pressure must be known to determine the resisting clip and corresponding sidewall traction forces at any point or level in the embankment. As suggested in Section D, the coefficient of earth pressure may be calculated using Equation 1, and assuming a vertical face. Calculations for the active earth pressure at any point in the embankment are as follows:

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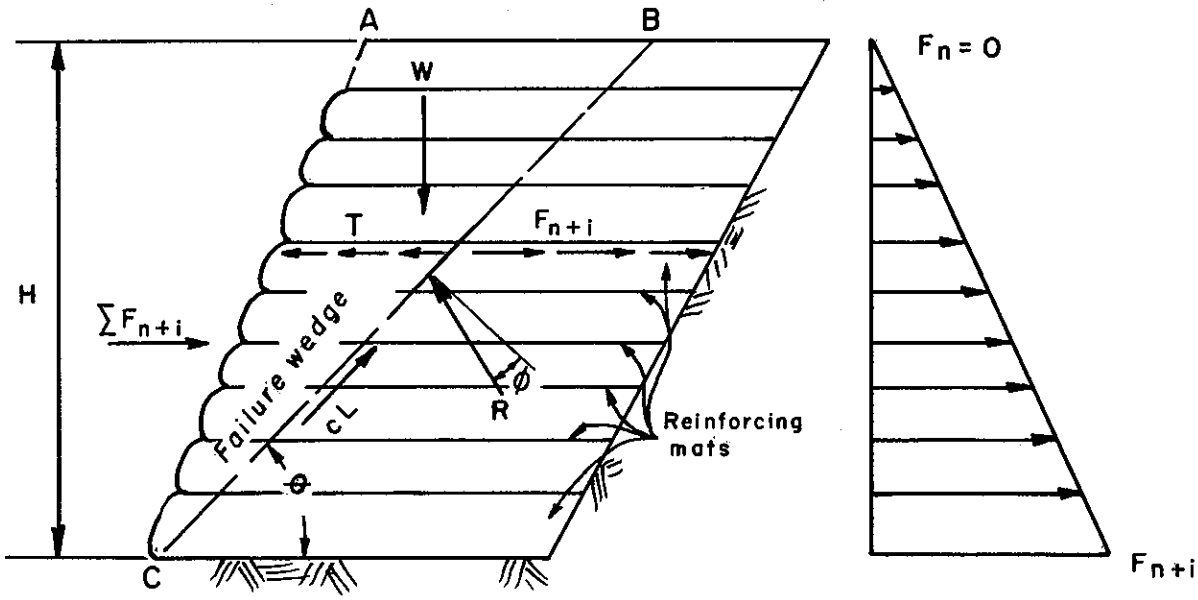

FIG.15 TRACTION FORCE AND LOADS

$$pa_{n+i} = Ka\gamma h - 2c\sqrt{Ka} \qquad \text{Eq. (6)}$$

where pa_{n+i} = earth pressure at depth (n+i) per ft depth

Ka = coefficient of active pressure

γ = unit weight of soil

c = cohesion intercept value of soil

h = depth to reinforcing mat

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Once the active pressure at a point has been computed, it can be compared to the corresponding resisting traction force of the clip which must be equal or greater than the active pressure for equilibrium. In other words,

$$F_{n+i} \geqq pa_{n+i} \qquad \text{Eq. (7)}$$

where F_{n+i} = clip resisting force at depth (n+i) per ft width

$F.S. = F_{n+i}/pa_{n+i}$

The critical angle θ in Figure 15 can be determined from a conventional sliding wedge analysis.

As an additional check of the total clip forces at the failure plane, the difference between the driving force and the sum of the cohesive and frictional forces would give the total clip forces. The equation may be written as:

$$\Sigma F_{n+i} \geqq W \sin\theta - (cL + W\cos\theta\tan\phi) \qquad \text{Eq. (8)}$$

Also, by taking incremental soil layers from top to bottom, each corresponding clip force can be determined at the failure plane. The process can be simplified by designing the clip connection for the maximum embankment depth.

Figure 16 shows the calculated traction forces of a sidewall and rebar clip, and the variation in active earth pressure with respect to depth for the given soil strength parameters. This chart is for a 2 ft vertical reinforcement spacing per ft width, developed for the design of the subject project and based on results of the pull-out tests and Equations 3 through 6. Note that the maximum height of the embankment

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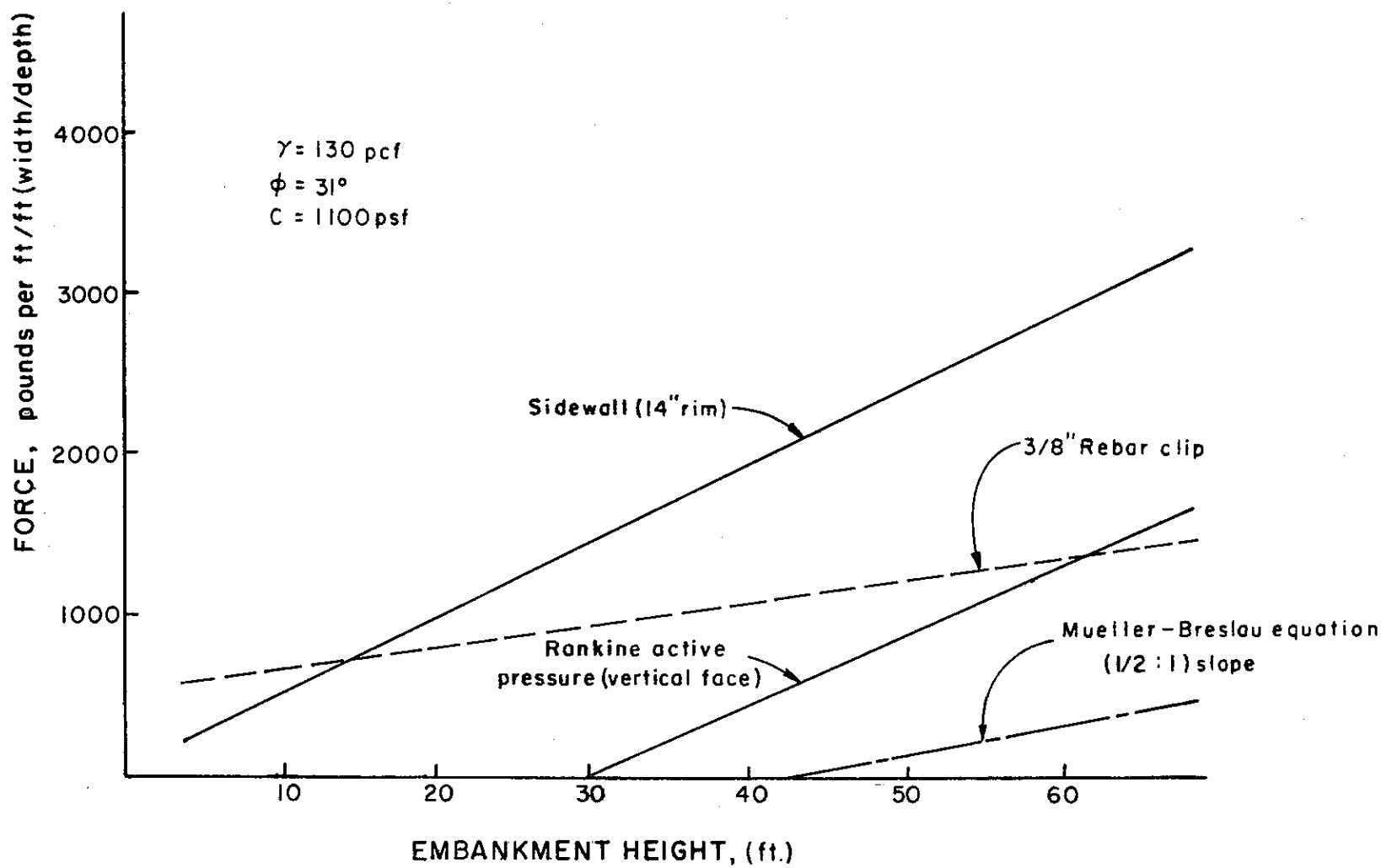

FIG. 16 RESISTING FORCES OF SIDEWALL AND REBAR CLIP, AND ACTIVE EARTH PRESSURE VS EMBANKMENT HEIGHT

is dependent on the resisting traction force of the rebar clip. Also, at shallow depths, the calculated resisting traction force for the sidewalls is smaller than that for the clips. However, since the results of pull-out test indicate that the buried sidewall cannot fail by pull-out, the resisting traction force of the clips is suggested as the basis of the design.

External stability of the embankment involves consideration of the behavior of the reinforced embankment as a gravity system. Bearing capacity; overturning; sliding at the base; and general slope stability, are all considered.

External stability dictates the minimum embedment length for the tire-reinforcing mat system. At Station 12+00, a minimum factor of safety of 1.80 was found for a sliding wedge failure at the toe with soil strength parameters ϕ = 31 degrees, c = 1100 psf and γ = 130 pcf. Under saturated conditions, the minimum factor of safety for a sliding wedge failure at the toe was found to be 1.03, with soil strength parameters ϕ = 26 degrees, c = 350 psf and γ = 130 pcf. This latter condition was not expected to occur because drainage provisions were included in the design, and embankment geometry provided lateral buttressing.

F. CONSTRUCTION OF TIRE-REINFORCED EMBANKMENT

Construction of the tire embankment began on September 22, 1976. During the ensuing two weeks, the designated portion of the existing embankment was removed. The material was stockpiled adjacent to the site. Excavation to a depth equal to Elevation 335 was necessary to permit installation of an underdrain system (see Figure 17).

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After the subdrainage system had been installed, the new fill was brought up some 5 ft to Elevation 340, levelled, and compacted to 90 percent relative density as determined by California Test Method 231; after which, the initial layer of sidewalls was placed (see Photo 3, and Figures 17 and 18). The sidewalls were installed in the horizontal plane at intervals of 2 ft, vertically. Each level of tire sidewalls was placed in rows, forming a continuous mat. Each sidewall was connected, using steel rebar clips, to the adjacent sidewall sections in the row. Tires were fastened in two directions to form a mat without overlap (see Photo 4). Each row extended the full width of the new embankment. Placement of the tire sidewalls started at the face of the slope. Each sidewall was placed with the inside facing up. No slack was allowed so that the clips were in contact with the beads of the tire sidewalls. Although an effort was made to eliminate all slack in the tire-mat system during placement of overlaying embankment material, some did occur. Tire mats were placed to within 4 ft of the profile elevation. Backfill material was placed and compacted according to Section 19 of the California Standard Specifications.

A total of 24 layers of mats were installed with approximately 1000 sidewalls per mat. Construction was completed December 17, 1976.

1. Materials

Passenger vehicle or pickup truck tires in wheel sizes of 14 or 15 inches only were utilized on the project. Sidewall sections were separated from the tire by cutting circumferentially at each shoulder (the point where the tread and sidewall meet). Tire sections in damaged condition were

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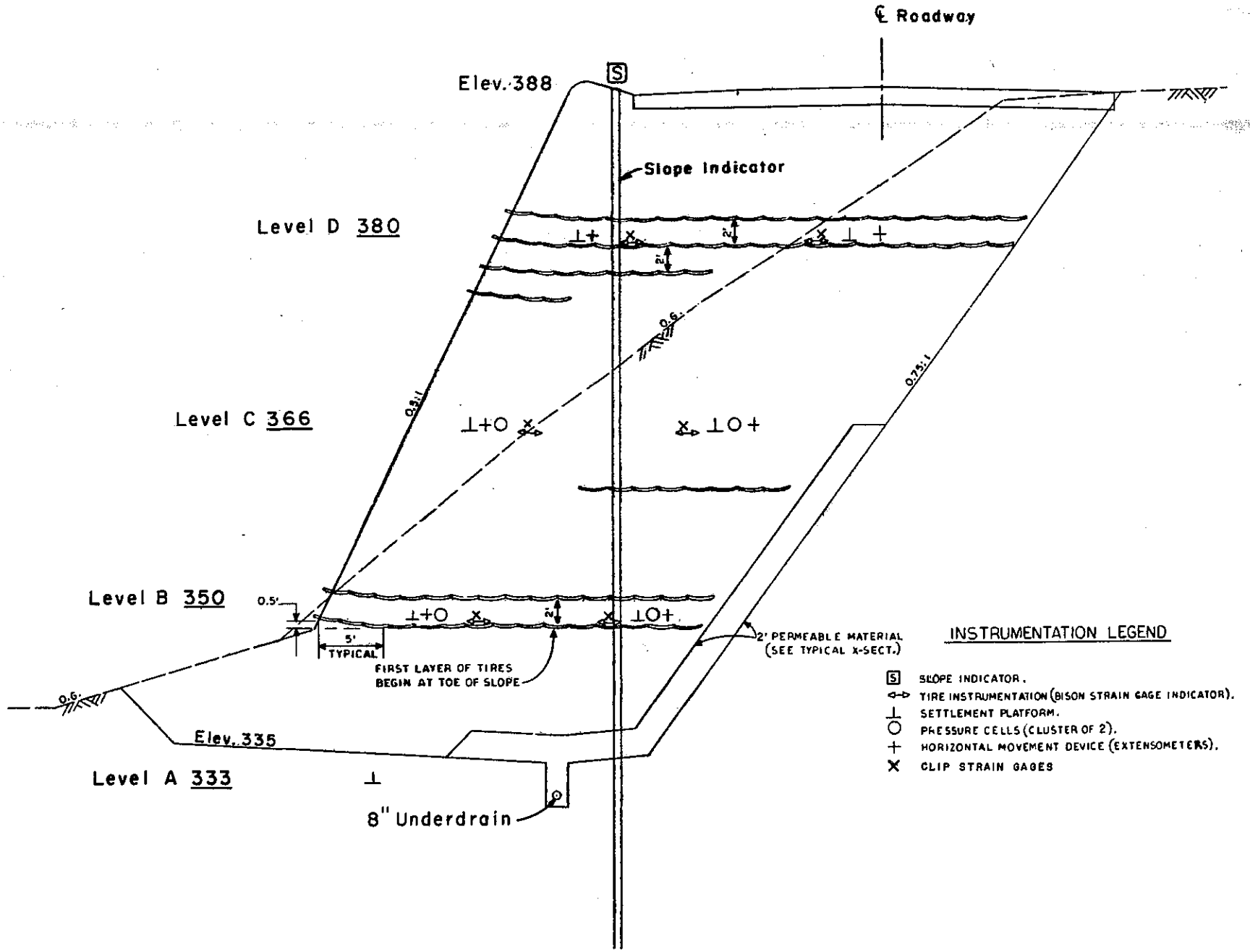

FIG. 17 X-SECTION (J 11+88), INSTRUMENTATION

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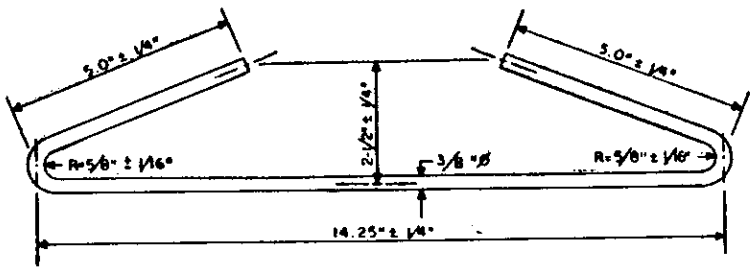
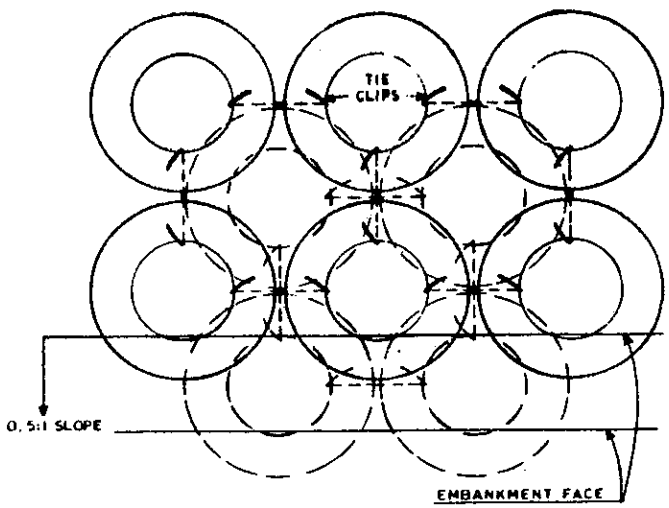


STEEL TIRE CLIP



PLAN VIEW OF TIRE PLACEMENT AND TIE ARRANGEMENT

FIG. 18 STEEL TIRE CLIP AND TIRE ARRANGEMENT

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accepted provided the bead area was intact for a minimum distance of 2-1/2 inches measured from the inside edge of the bead in a radial direction.

Although, the contract had originally disallowed 13-inch diameter sidewalls, a change order was written to permit their use. Only one layer (Layer 5) of the 13-inch diameter sidewalls was installed. Steel-belted tires were not used. Tire clips were fabricated from 3/8-inch, Grade 40, reinforcing steel bars (Figure 18).

Imported sandy material from a nearby quarry at Felton was utilized to construct the upper 4 ft of the embankment. Some 820 cy of this material was used, plus about 13,000 cy of excavated material for the lower portion of the fill. Class II aggregate base material, also from Felton, was used for the roadway structural section.

2. Cut Slope Erosion Protection

Some difficulty was experienced during construction in compacting the embankment zone adjacent to the slope face. The compaction effort forced the soil between layers of sidewall mats to displace laterally. The problem of preventing sloughing during compaction was solved by using 1-inch wire-mesh to contain the earth. The wire mesh was unrolled along the outer edge and secured to the sidewall mat. See Photo 5. Earth was then placed on the wire mesh and the free edge was pulled over the loose material and again secured to the sidewall mat. The material at the slope face was then densified with hand compaction equipment. The face of the slope was seeded to ground cover to prevent erosion. The completed embankment is shown in Photo 6.

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3. Construction Labor, Equipment, and Charges

The labor force for installing the tire mats consisted of two laborers and a loader-operator. The tires were brought down into the fill level with a loader and placed in the desired pattern by the two laborers, as shown in Photo 3. Sidewalls were then clipped together and manually tensioned. Following are the summaries of the labor, equipment, and material costs:

a. Construction Charges

Contract bid items for the project were as follows:

1. Tire sidewalls and clips installed - $2.00 each
2. Roadway excavation and fill - $6.00/cy
3. Imported fill material (in place) - $6.13/cy
4. Class II aggregate material (in place) - $10.00/cy
5. Additional charge for supplementary slope stabilization including labor and material - $8,605.19

b. Construction Equipment

The following construction equipment was used on this project:

2 - DC8 Caterpillar Dozers
1 - 815 Caterpillar Compactor
1 - 922 Caterpillar Loader
1 - 12 Caterpillar Grader
1 - 40 ton Lorraine Truck Crane with dragline
1 - Watertruck
1 - Hand compactor

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c. Construction Labor

The average labor force included the following:

1 - Superintendent or foreman
1 - Labor foreman
3 - Laborers
2 - Operating engineers for equipment
1 - Operator and oiler for truck crane
and others, such as truck drivers

G. INSTRUMENTATION

A single test section was selected (Station 11+88) for field instrumentation to monitor performance. The test section consisted of four levels of instrumentation, Levels A, B, C and D (see Figure 17).

On Level A, the lowest, one settlement platform was installed near the face of the excavated slope. Two fluid level type settlement platforms were installed on each of Levels B, C and D. Two soil pressure cells were placed at Levels B and C, only, as shown by Figure 17. Two extensometers were installed to measure the elongation of the tire sidewalls on Levels B, C and D. Two pairs of strain gages were installed to measure the tension forces in the clips in the longitudinal and lateral directions at Levels B, C and D. Two pairs of Bison strain gages were placed in the soil within the sidewalls to measure soil movements on Levels B, C and D. Two extensometers were installed to measure soil movements within the fill on Levels C and D. A slope indicator was installed to measure lateral embankment movements. The locations of all instrumentation within the embankment are shown in Table 3.

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TABLE 3 INSTRUMENTATION LOCATIONS

	Station "J" Line	Distance ft	Elevation ft
Level A (Oct. 7, 1976)			
Settlement Platform SI-1	11+75	Lt 22.5	332.87
Level B (Oct. 22, 1976)			
Settlement Platform SI-2	11+91	Lt 28.0	350.20
SI-3	11+91	" 18.5	350.07
Sidewall Extensometer #1	11+86	" 17.9	350
#2	11+86	" 19.1	350
Soil Extensometer #3	11+88	" 18.5	350
#4	11+86	" 28.0	350
Pressure Cells #1	11+90	" 17.2	350
#2	11+90	" 15.7	350
#3	11+90	" 33.3	350
#4	11+90	" 32.3	350
Bison Gages #1 and 2	11+88	" 17.2	350
#3 and 4	11+88	" 30.8	350
Clip Strain Gages #1	11+88	" 32.0	350
#2	11+89.2	" 30.8	350
#3	11+88	" 16.0	350
#4	11+89.2	" 17.2	350
Level C (Nov. 4, 1976)			
Settlement Platform SI-4	11+93.2	Lt 21.4	366.37
SI-5	11+93.5	" 3.5	366.39
Sidewall Extensometer #5	11+91.5	" 10.0	366
#6	11+91.5	" 11.2	366
Soil Extensometer #7	11+92.5	" 9.4	366
#8	11+93.0	" 20.1	366
Pressure Cell #5	11+92.5	" 4.9	366
#6	11+92.5	" 6.2	366
#7	11+93	" 23.0	366
#8	11+93	" 24.5	366
Bison Gages #5 and 6	11+89.2	" 28.2	366
#7 and 8	11+88.4	" 8.3	366
Clip Strain Gage #5	11+89.2	" 27.0	366
#6	11+90.4	" 28.2	366
#7	11+88.9	" 7.1	366
#8	11+90.2	" 8.3	366
Level D (Nov. 24, 1976)			
Settlement Platform SI-6	11+88	Lt 8.0	380.77
SI-7	11+92	Rt 7.0	380.36
Sidewall Extensometer #9	11+92.2	Rt 4.4	380
#10	11+92.2	Rt 5.5	380
Soil Extensometer #11	11+92.6	Rt 6.0	380
#12	11+88	Lt 8.2	380
Bison Gage #9 and 10	11+90.2	Lt 11.6	380
#11 and 12	11+90	Rt 4.6	380
Clip Strain Gage #9	11+90.2	Lt 10.4	380
#10	11+90	Rt 5.8	380
#11	11+90	Rt 5.8	380
#12	11+91.1	Rt 4.6	380
Slope Indicator	12+00	Lt 22.5	387.0 (Top)

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Instruments were installed during October and November, 1976. Installation of each level required two days. The instruments were placed in a two ft deep trench at each level. Backfill was placed above the instruments and compacted with a hand-held compactor prior to allowing heavy equipment to go over the trench. Photos 7, 8, 9, 10 and 11 show the settlement platform, pressure cells, extensometers, strain gages for tire clips, Bison strain gages, and slope indicator, respectively.

1. Field Data

All instruments were checked and calibrated in the laboratory and in the field prior to their installation. These instruments were read at frequent intervals during construction; and monthly, following the completion of construction, over a six month period. The following is a summary of the results of field instrumentation.

a. Settlement Platforms

Settlement data from the various levels is plotted in Figure 19.

1) Level A

Settlement platform SI-1 showed an initial settlement of 0.08 ft under a fill height of 8 ft. An additional 0.02 ft of settlement occurred during the balance of the construction period. No readings were made subsequent to January 4, 1977, at which time a mudslide covered the readout box.

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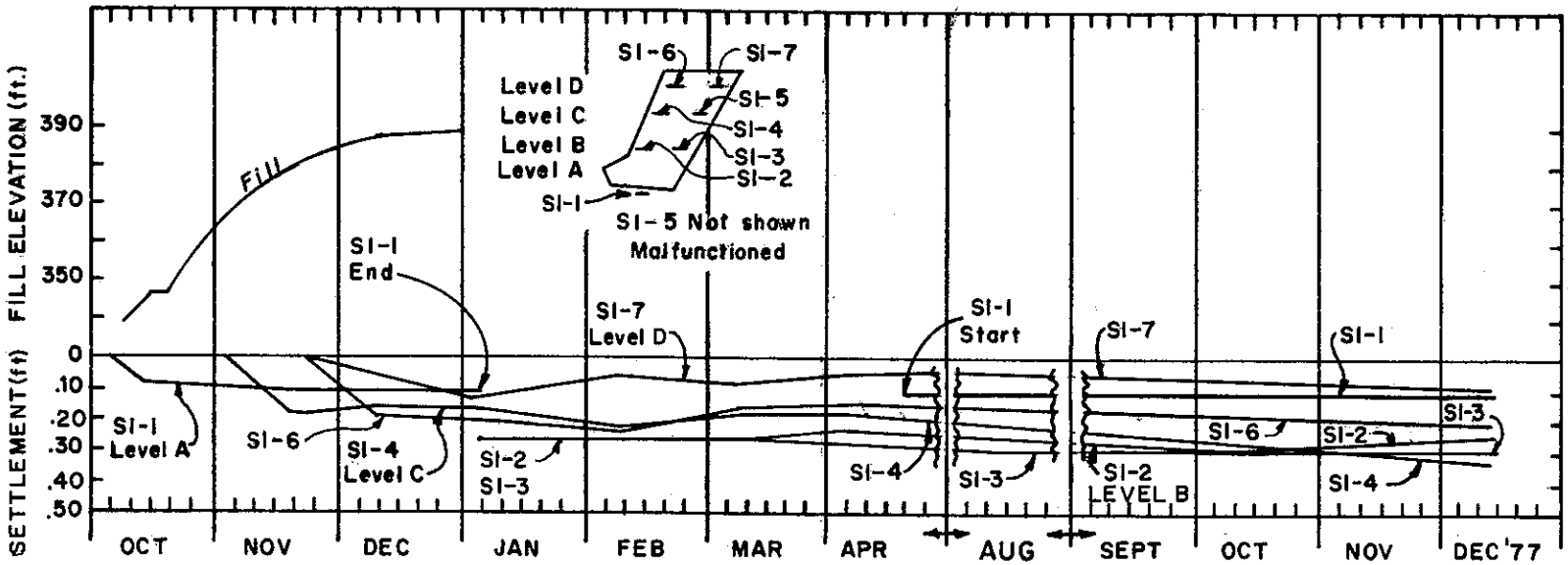

FIG. 19 SETTLEMENTS, LEVELS A, B, C AND D

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2) Level B

The settlement platforms SI-2 and SI-3 malfunctioned following installation and no readings were made prior to January 1977. After repairs were made, the reading showed 0.25 ft of settlement.

3) Level C

Measurements began on November 4, 1976. Platform SI-4 showed about 0.17 ft settlement during construction of the fill. After the fill was completed, an additional 0.05 ft occurred. Settlement platform SI-5 malfunctioned, and no measurements were recorded.

4) Level D

Measurements began on November 24, 1976. Initial measured settlement showed 0.20 ft for platform SI-6. Platform SI-7 showed about 0.10 ft settlement. After the fill was completed, no additional settlement occurred.

b. Slope Indicator

Horizontal movements of the embankment during and after construction were measured by a slope indicator. In the North-South direction, a total longitudinal movement of 1.0 inch occurred during and after construction of the fill, as shown in Figure 20. In the East-West orientation (towards the slope), lateral movement of 2.8 inches was measured during construction of the fill. From completion of the fill to November 1977, this lateral movement increased by an increment of 1.6 inches. The post-construction movement was apparently

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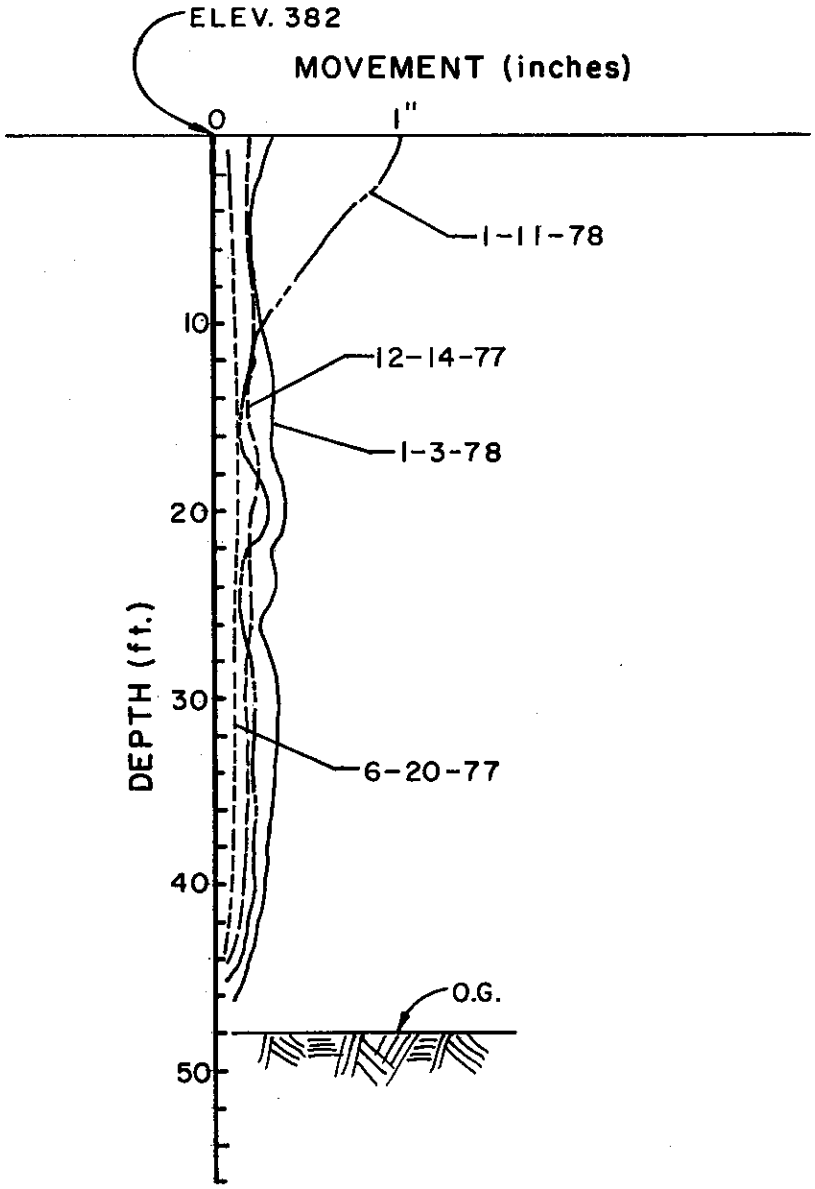

FIG. 20 SLOPE INDICATOR READINGS
(Longitudinal Direction Including Construction Of Fill)

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limited to the upper 18 ft of the embankment, as shown in Figure 21. As a result, a fissure approximately 1/2-inch wide developed along the embankment/cut slope interface, which required repairs.

After November 1977, large lateral movements occurred totalling over 20 inches, eventually resulting in localized slope failure (see Figure 22).

c. Bison Strain Gage

Bison gages were used to measure small soil movements within the instrumented tire sidewalls on Levels B, C and D. The gages consisted of two circular disks implanted in the soil which measured the movement between them by electro-magnetic coupling.

1) Level B

Gages 1 and 2, located near the interface of the old fill measured a maximum elongation of 0.027 inch during construction of the fill, but changed to compressive movement of 0.068 inch after construction and to an elongation again of 0.08 inch prior to slope failure. Gages 3 and 4, nearest to the new slope, showed consistent compressive movement up to 0.112 inch maximum after construction; changing to zero before slope failure. See Figure 23 for the plot of gages 1 through 4.

2) Level C

Gages 5 and 6, located near the interface measured elongation of 0.03 inch maximum during construction; but changed to

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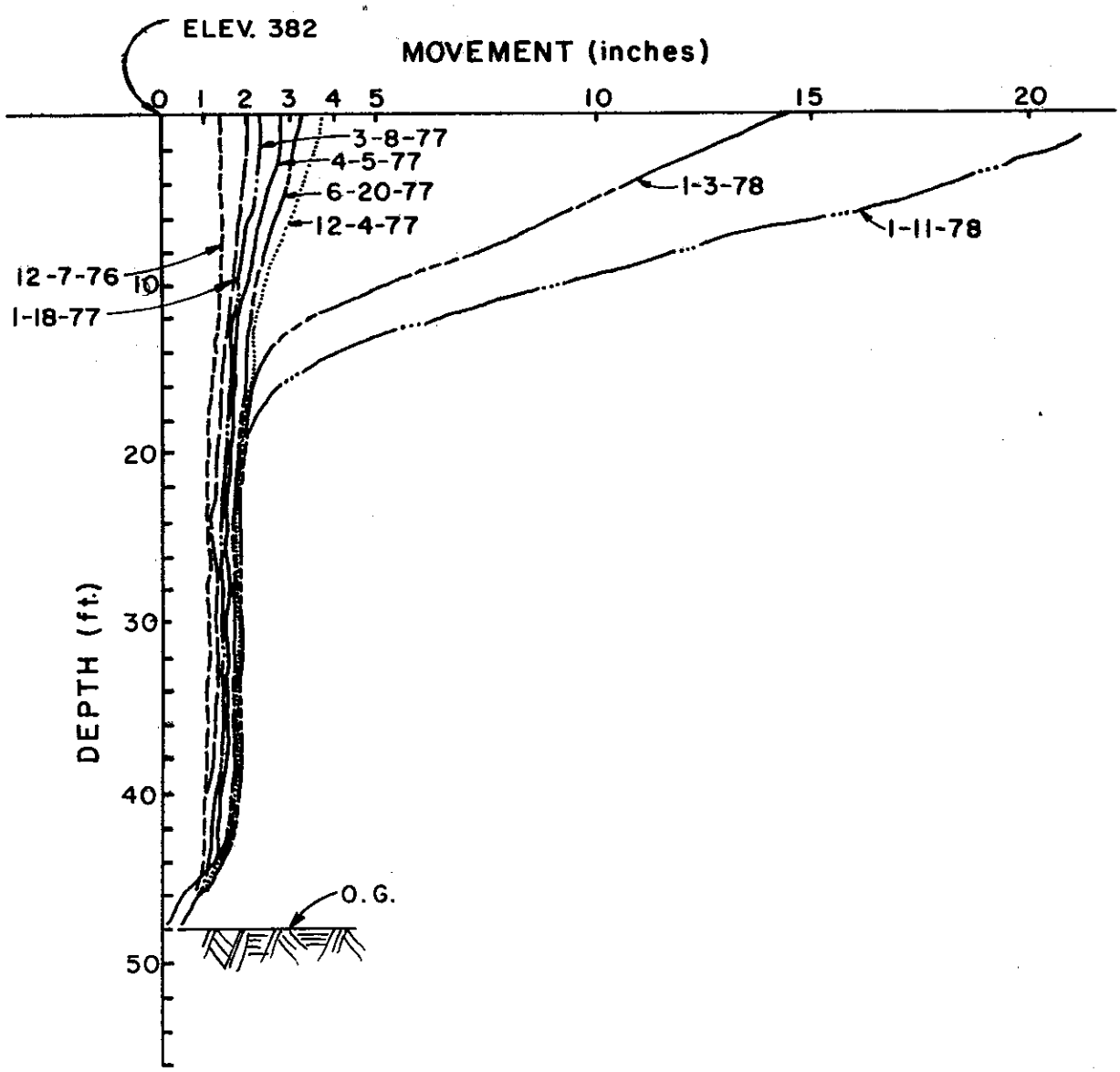

FIG. 21 SLOPE INDICATOR READINGS
(Lateral Direction Including Construction Of Fill)

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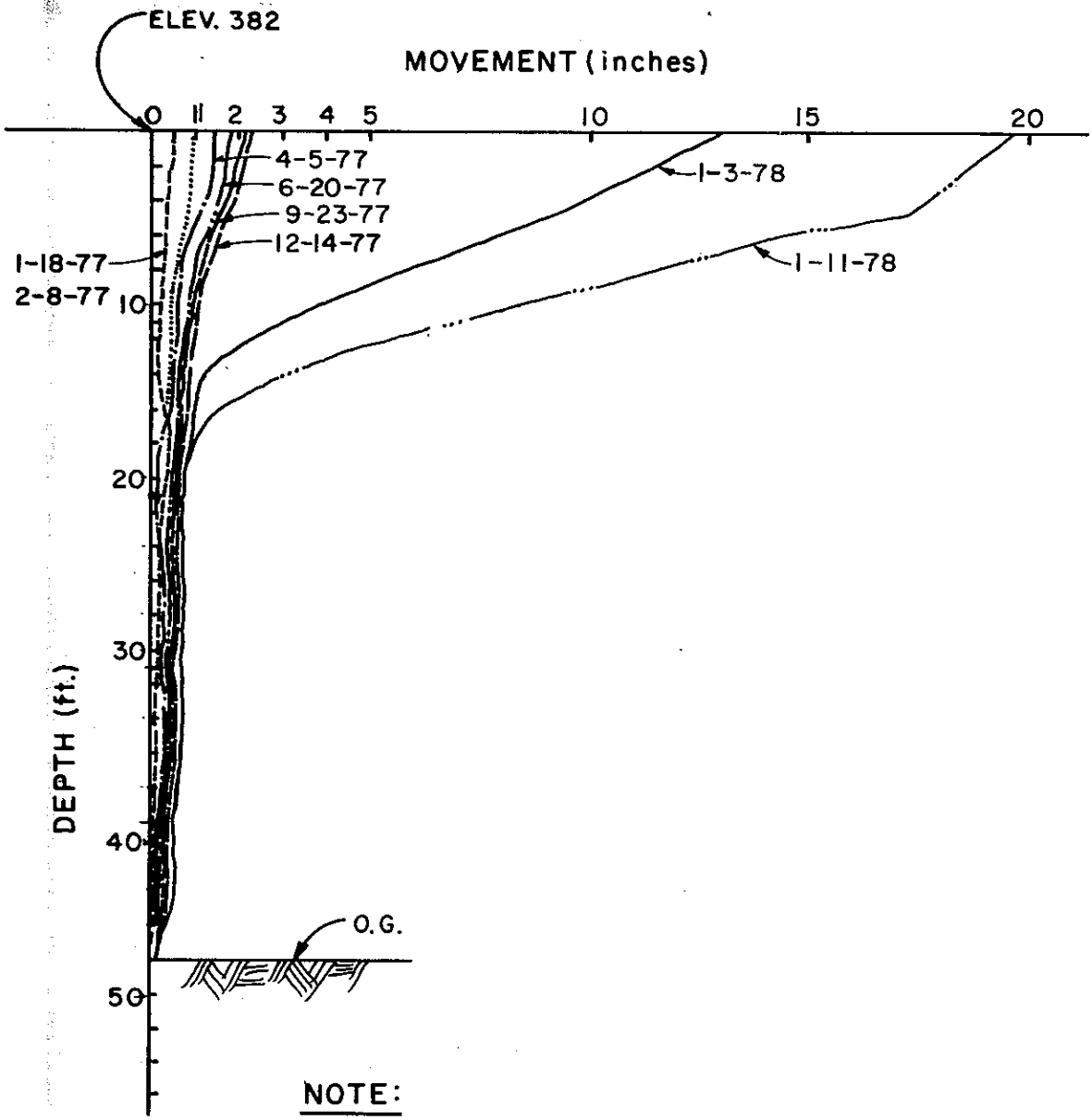

NOTE:

Longitudinal direction not shown because of very small displacements

FIG. 22 SLOPE INDICATOR READINGS
(Lateral Direction Corrected After Fill Completed 12-7-76)

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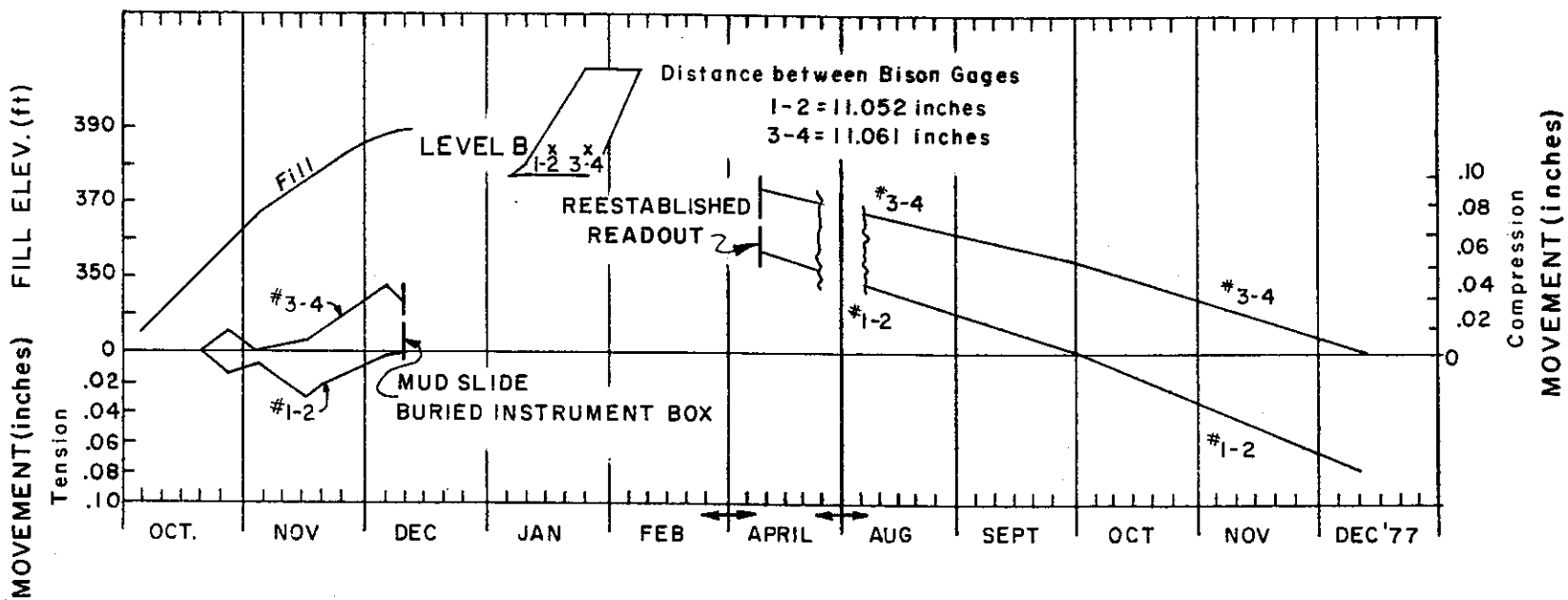

FIG. 23 SOIL MOVEMENT WITHIN SIDEWALL, LEVEL B
BISON GAGES

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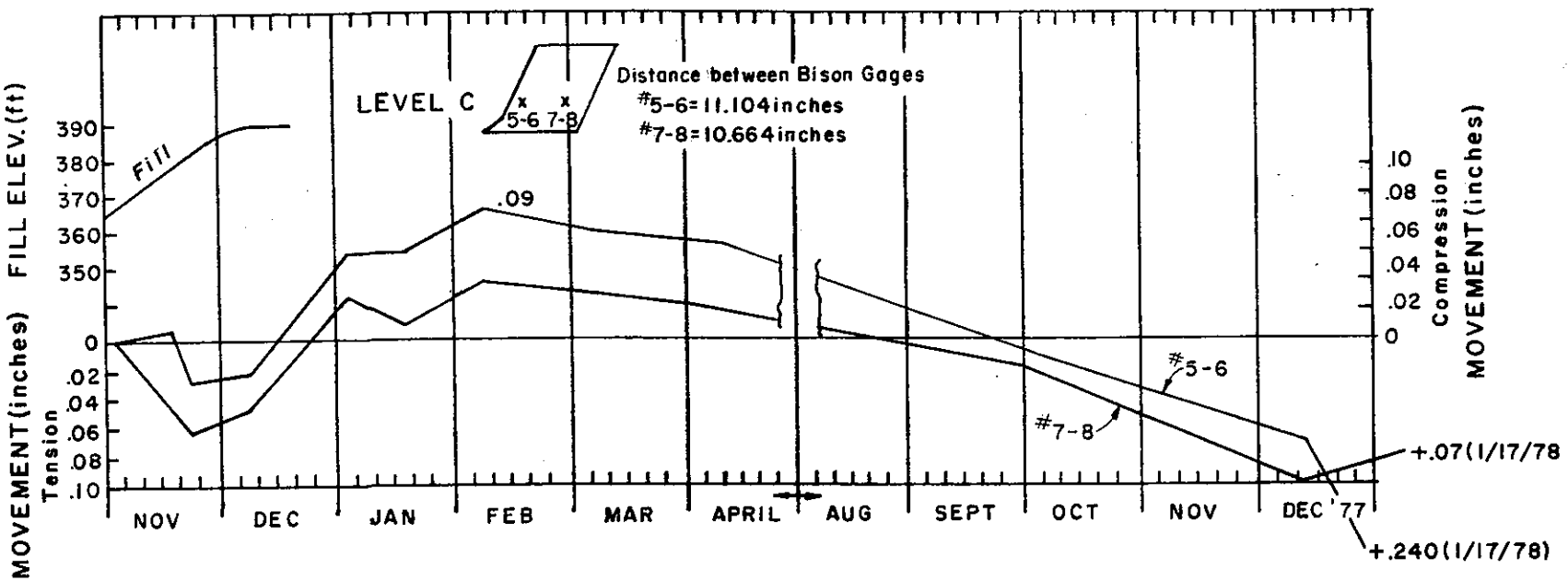

FIG. 24 SOIL MOVEMENT WITHIN SIDEWALL, LEVEL C
BISON GAGES

0.09 inch maximum compressive movement after completion of construction, and to an elongation again of 0.24 inch prior to slope failure. Gages 7 and 8 recorded elongation of 0.065 inch maximum during construction; but changed to compressive movement after construction of 0.041 inch maximum; and to an elongation again of 0.07 inch prior to slope failure (see Figure 24).

3) Level D

Gages 9 and 10, located near the interface, measured consistent compressive movements of 0.095 inch maximum, even after the fill had been completed; but changed to elongation of 0.36 inch prior to slope failure. Gages 11 and 12 recorded compressive movement of 0.105 inch maximum changing to an elongation of 0.149 inch maximum prior to slope failure (see Figure 25).

d. Extensometers

Extending rods were used to measure movement of the sidewalls and soil at Levels C and D. The rods were extended out to the slope face for measurements.

1) Level C

Elongations of only 0.05 inch were measured for the sidewalls prior to slope failure (Figure 26). About 0.44 inch of elongation occurred in the soil prior to slope failure (Figure 27).

2) Level D

Small elongations of 0.05 inch were measured for the sidewalls prior to slope failure (Figure 26). About 0.45 inch of elongation occurred in the soil prior to slope failure (Figure 27).

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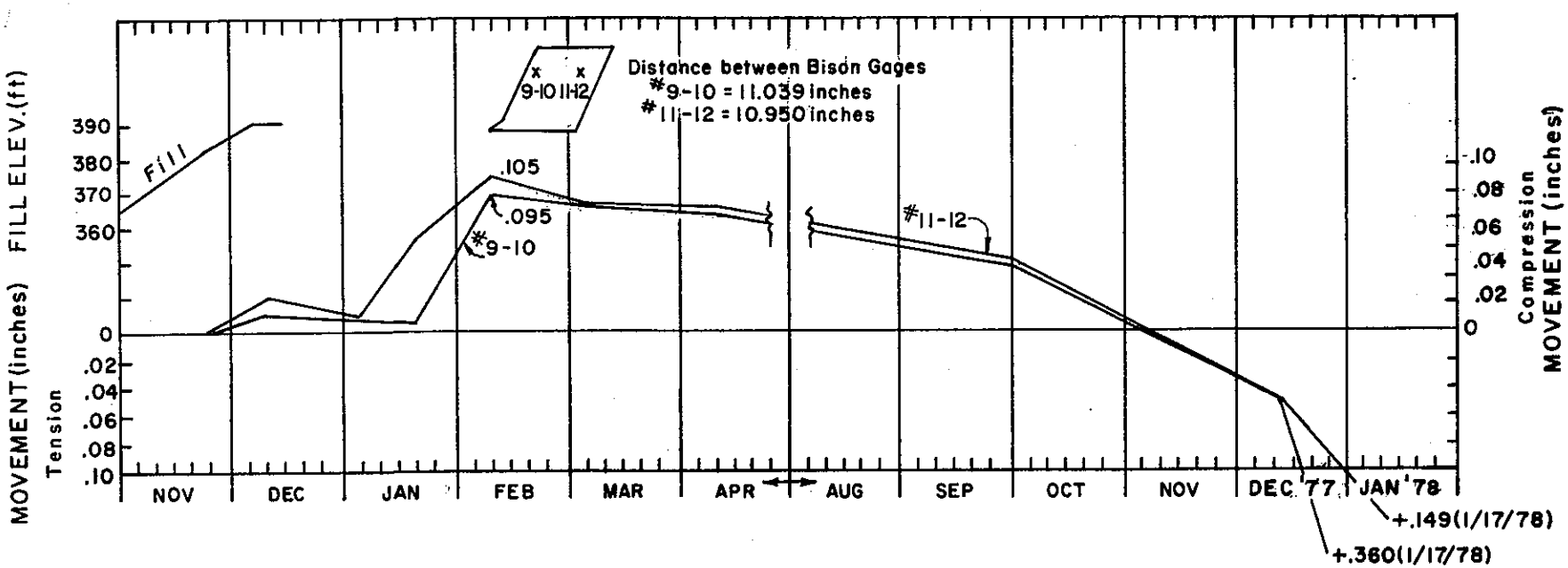

FIG. 25 SOIL MOVEMENT WITHIN SIDEWALL, LEVEL D
(BISON GAGES)

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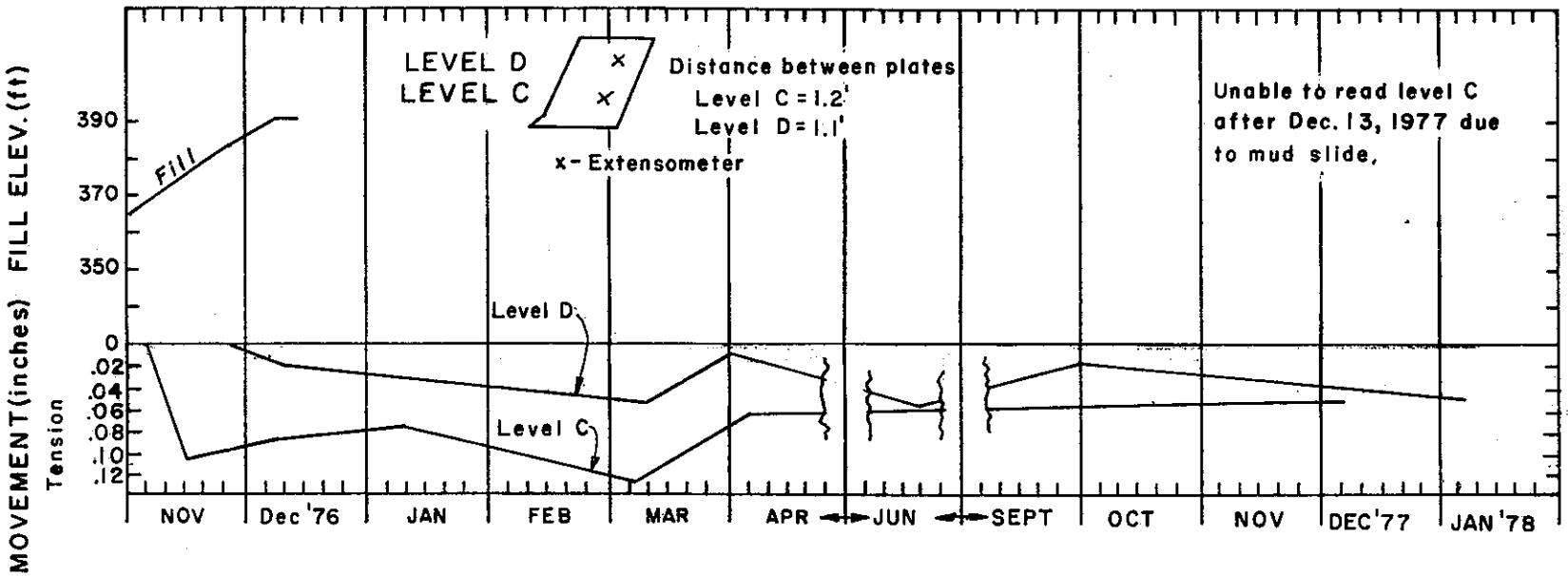

FIG. 26 SIDEWALL MOVEMENT, LEVELS C AND D (EXTENSOMETERS)

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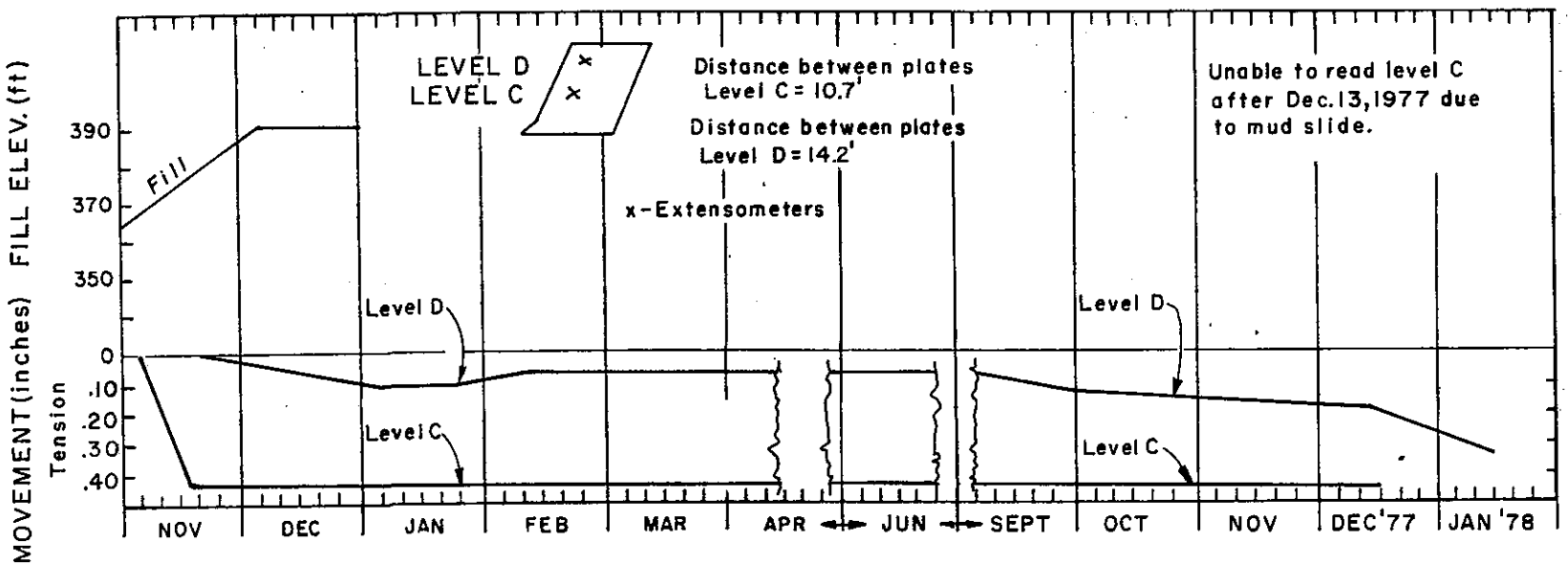

FIG. 27 SOIL MOVEMENT, LEVELS C AND D
(EXTENSOMETERS)

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e. Forces in Tire Clips

Forces in the clips were measured with SR-4 strain gages and found to be very small.

1) Level B

Figure 28 shows the history of the clip forces in Level B. Gages 1 and 3 in the lateral direction experienced a maximum tensile load of 29 lbs, then decreasing to a compressive load of 108 lbs. Gages 2 and 4 in the longitudinal direction also experienced a tensile load of 13 lbs and changed to a compressive load of 113 lbs.

2) Level C

Figure 29 shows the history of the clip forces in Level C. Gages 5 and 7 indicated the maximum tensile force of 32 lbs, then changing to a compressive force of 285 lbs maximum, prior to slope failure. Gages 6 and 8 showed tensile force of 152 lbs maximum and changing to compressive force of 115 lbs maximum prior to slope failure.

3) Level D

Figure 30 shows the history of the clip forces in Level D. Gages 9 and 11, which are in the lateral direction, showed tensile forces of 19 lbs maximum; then changing to compressive force of 481 lbs maximum prior to slope failure. Gages 10 and 12, which are in the longitudinal direction experienced compressive forces with a maximum of 136 lbs prior to slope failure.

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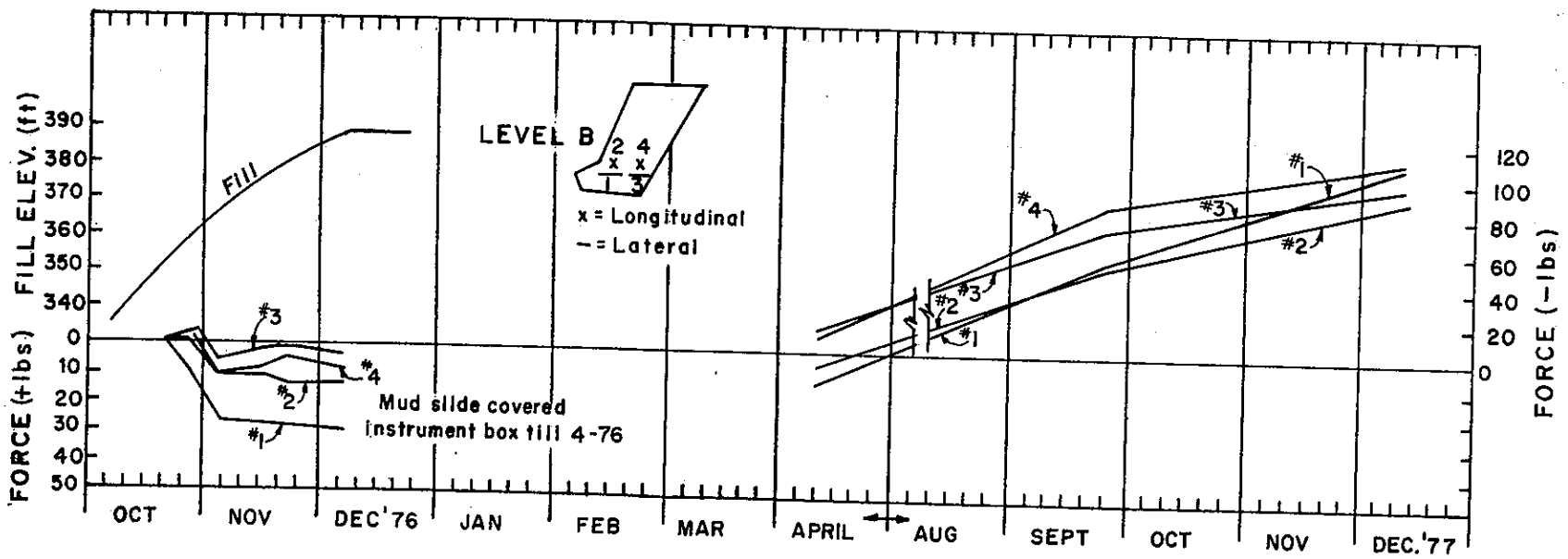

FIG. 28 TIRE CLIP FORCES, LEVEL B

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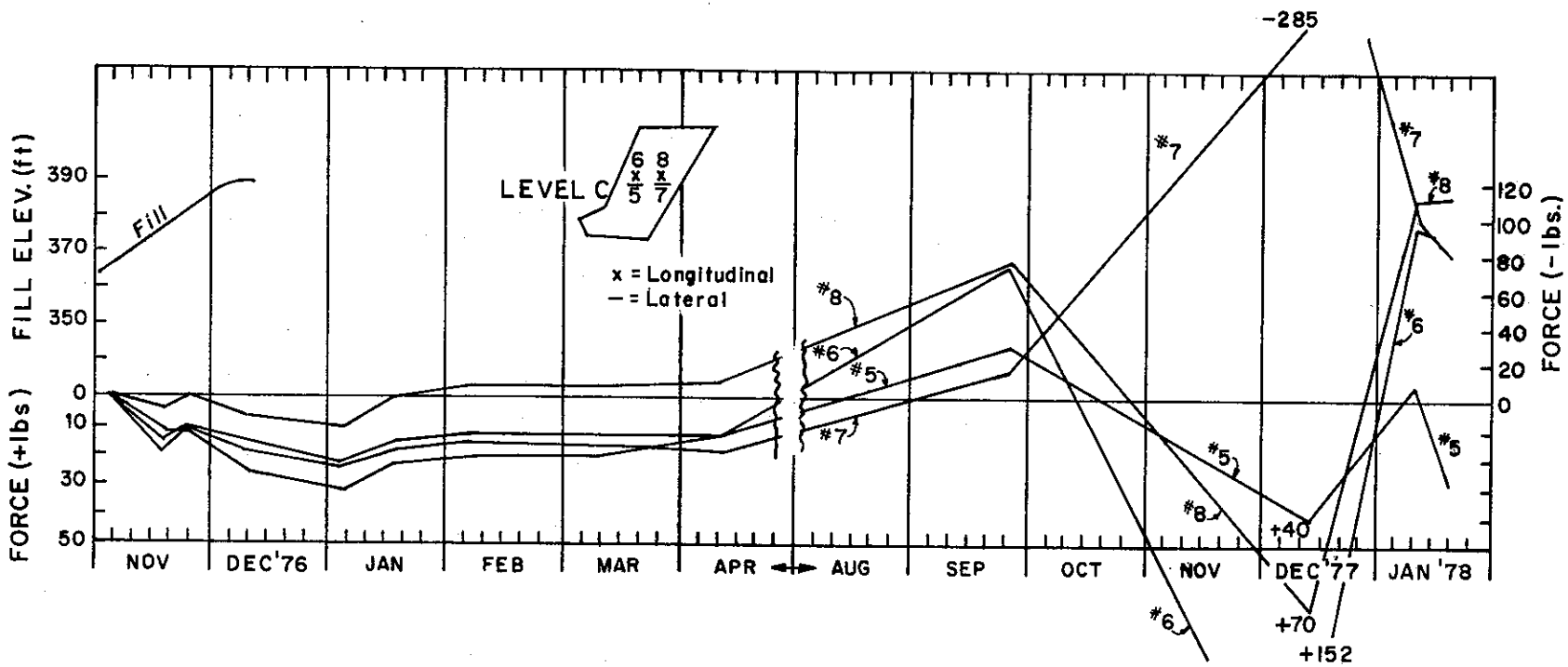

FIG. 29 TIRE CLIP FORCES, LEVEL C

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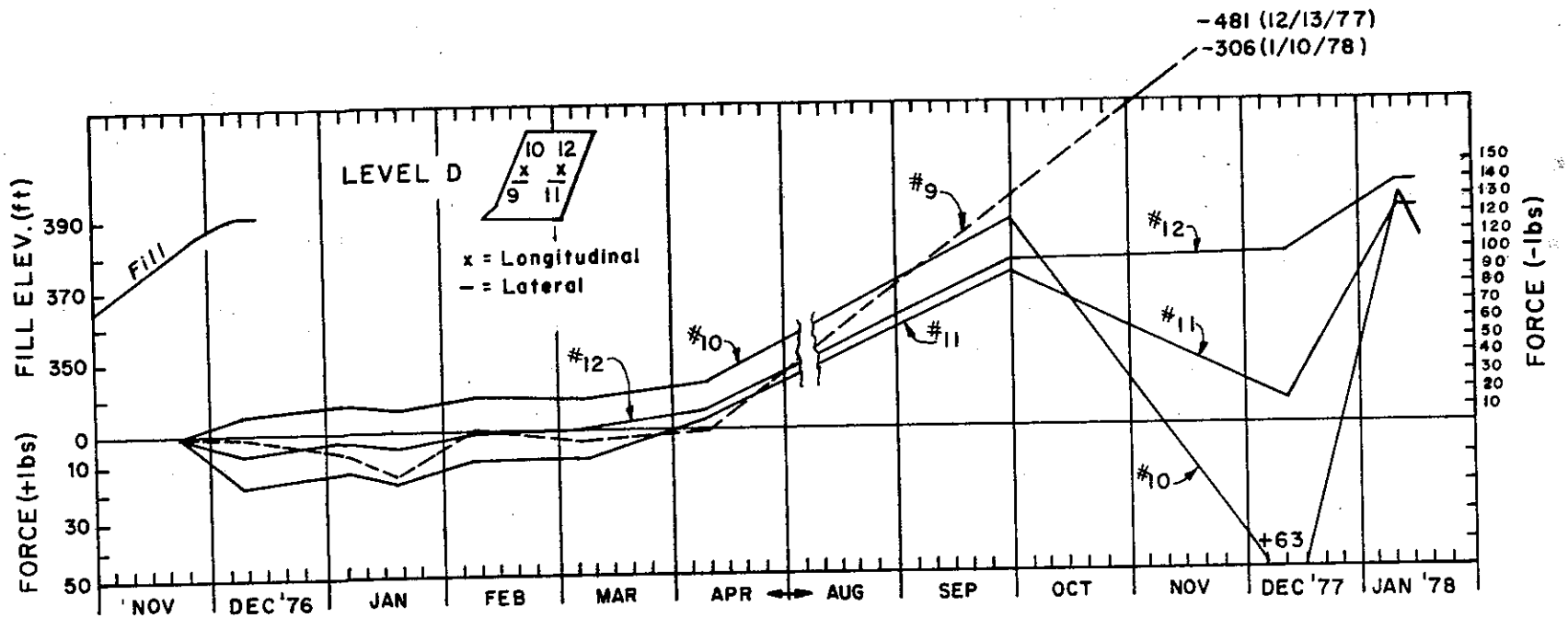

FIG. 30 TIRE CLIP FORCES, LEVEL D

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f. Pressure Cells

Horizontal and vertical soil stresses were measured hydraulically with modified general transducer oil-filled pressure cells in Levels B and C.

Figure 31 shows the soil stresses at Level B. At the end of construction, the vertical stresses varied from about 12 psi between the outside (Cell 1443) and inside (Cell 1449). There was reasonable agreement between cell 1449 and the overburden stress within the fill. Horizontal soil stresses were about 7 psi for both (Cell 1338) and (Cell 2240). The ratio K, as shown in Figure 31, was about 0.23 after the completion of fill for Cells 2240 and 1449. This would indicate that our suggested value of Ka (0.39) assuming a vertical face is conservative. Cell 1443 is in the slope wedge and does not offer a realistic K value. After September 1977, the cell readings were negative.

Figure 32 shows the soil stresses at Level C. After completion of the fill, the horizontal stress (Cell 974) was about 4 psi. The vertical stresses varied from 15 to 22 psi between the outside (Cell 1316) and inside (Cell 971). There was reasonable agreement between cell readings for 1316, and 971 and the overburden stress within the total fill. Cell 1387 malfunctioned. The ratio K, between 974 and 971 was found to be about 0.15 after the completion of the fill. After September 1977, the cell readings became negative.

2. Discussion of Field Data

The settlement platforms indicated no appreciable settlement of the embankment after the fill was completed, except at

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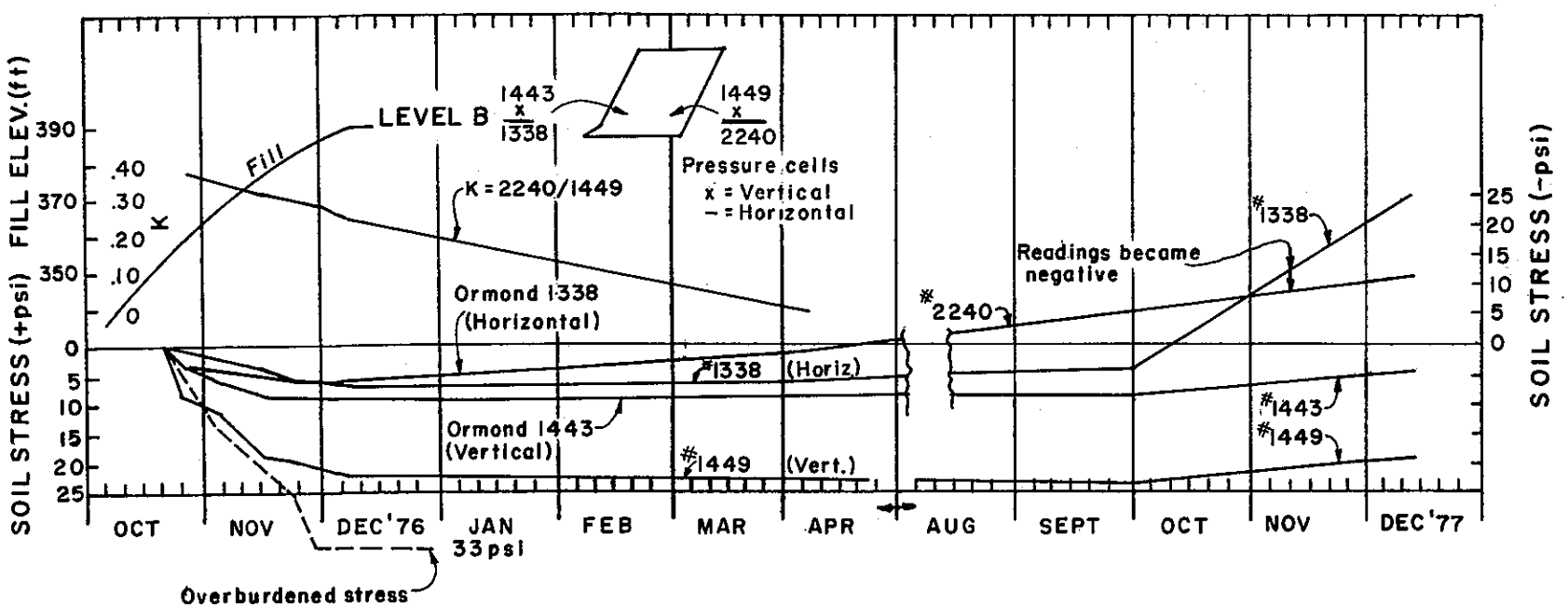

FIG. 31 HORIZONTAL AND VERTICAL SOIL STRESSES, LEVEL B (PRESSURE CELLS)

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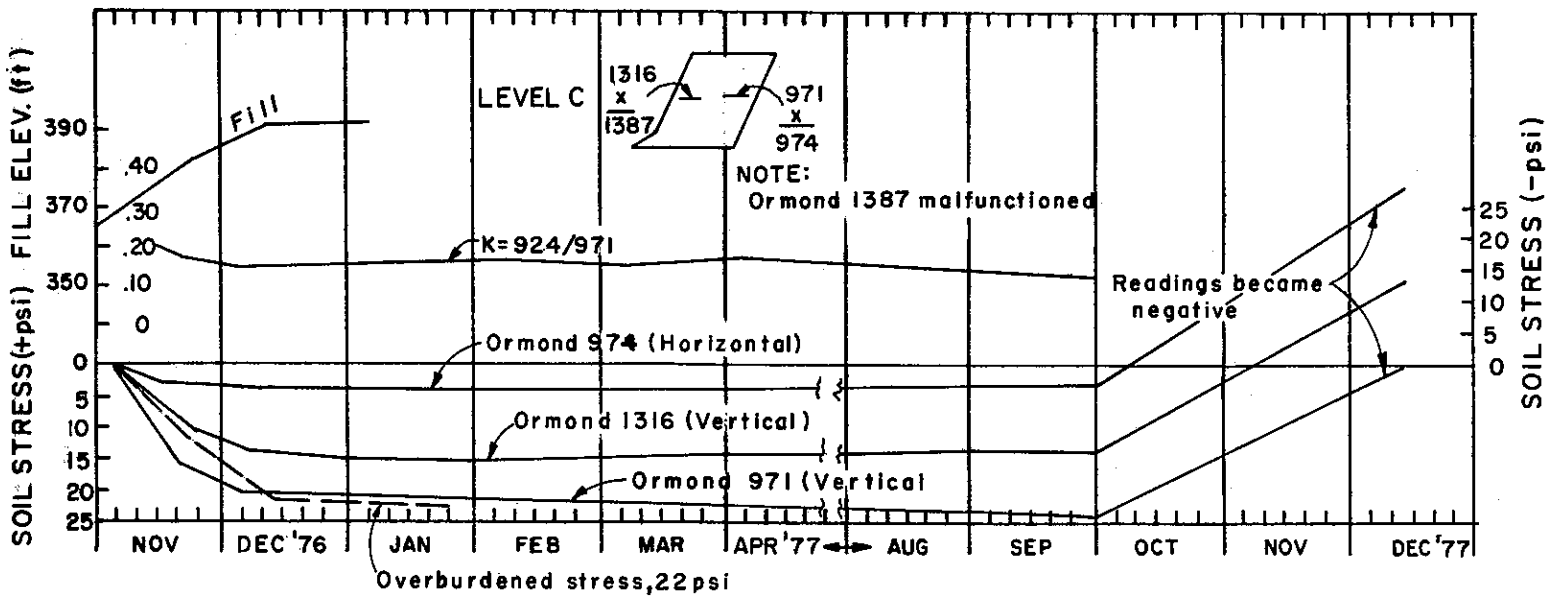

FIG. 32 HORIZONTAL AND VERTICAL SOIL STRESSES, LEVEL C (PRESSURE CELLS)

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SI-4. An additional settlement of 0.21 ft occurred near the slope wedge due to the pending slope failure.

The slope indicator showed about 1.6 inches of lateral movement after the fill was completed, and an additional 20 inches prior to failure of the slope.

In general, field data obtained from the Bison and clip gages, sidewall and soil extensometers were fairly consistent during and after construction. The Bison gages within the sidewalls showed soil compression. The confined soil would be compressed by the compaction effort. The clip gage data was consistent with the soil and sidewall extensometer data which showed elongation.

Prior to slope failure, field data collected were inconsistent in that Bison gage readings showed elongation while the clip gages showed compression, or relaxation. Relaxation of the mat might have occurred as the slope approached failure. There was little change in the sidewall and soil extensometer's readings as the slope approached failure. No conclusion was reached as to why the pressure cell readings became negative as the slope approached failure. It was concluded that the slack in the mat affected the clip gage and sidewall extensometer readings.

In summary, with the exception of that from the settlement platforms and slope indicator, most of the data resulting from the field instrumentation reflected too low a level of activity to permit meaningful analysis. The same is true of the data collected after reconstruction of the slope and buttress. However, the new slope indicator showed about one inch of lateral movement following reconstruction for the six month period through July 1979. Additional readings

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in September 1979 indicated no further movement. Horizontal control points on the pavement surface indicated that no movement had occurred in the upper level of the embankment during this time period. It is speculated that the one-inch movement detected by the slope indicator is a result of readjustment of the reconstructed fill and new buttress system. Instrumentation monitoring will continue over the long term to verify this assumption. No other instruments have reflected significant changes in data measurements since reconstruction.

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SECTION II

LOCAL EMBANKMENT SLIDE

A shallow local slide involving approximately 5 percent of the tire embankment occurred on February 7, 1978 (see Photo 12). The slide consisted of a triangular wedge about 80 ft in length, centered near Station 12+00. This movement was due primarily to seepage pressure resulting from an unanticipated rise in groundwater level as a result of unusually heavy rainfall between December 1977 and January 1978. This seepage was too high to be intercepted by the permeable blanket installed on the backslope or interface during construction (see Figure 33).

Water was observed dripping from the channels for the extensometers at Levels D and C, which are about 10 and 23 ft below the slope hinge-point (see Photo 13). The heaviest seepage appeared just below Level C where erosion was the most pronounced. The erosion undermined the slope and cracks appeared in the roadway shoulder from reduced lateral support. Settlement of the shoulder indicated the slope mass was becoming unstable (see Photo 14). The tire mats were not designed to resist the seepage force or the saturated soil condition in the embankment.

The lateral displacement at the hinge-point increased from 2 to 20 inches between December 14, 1977 to January 11, 1978. Shortly thereafter, the slope indicator (see Figure 22) was damaged to such an extent that additional measurements could not be taken. This failure was witnessed by a forest ranger who described it as a flow slide. The slide mass was completely saturated with water.

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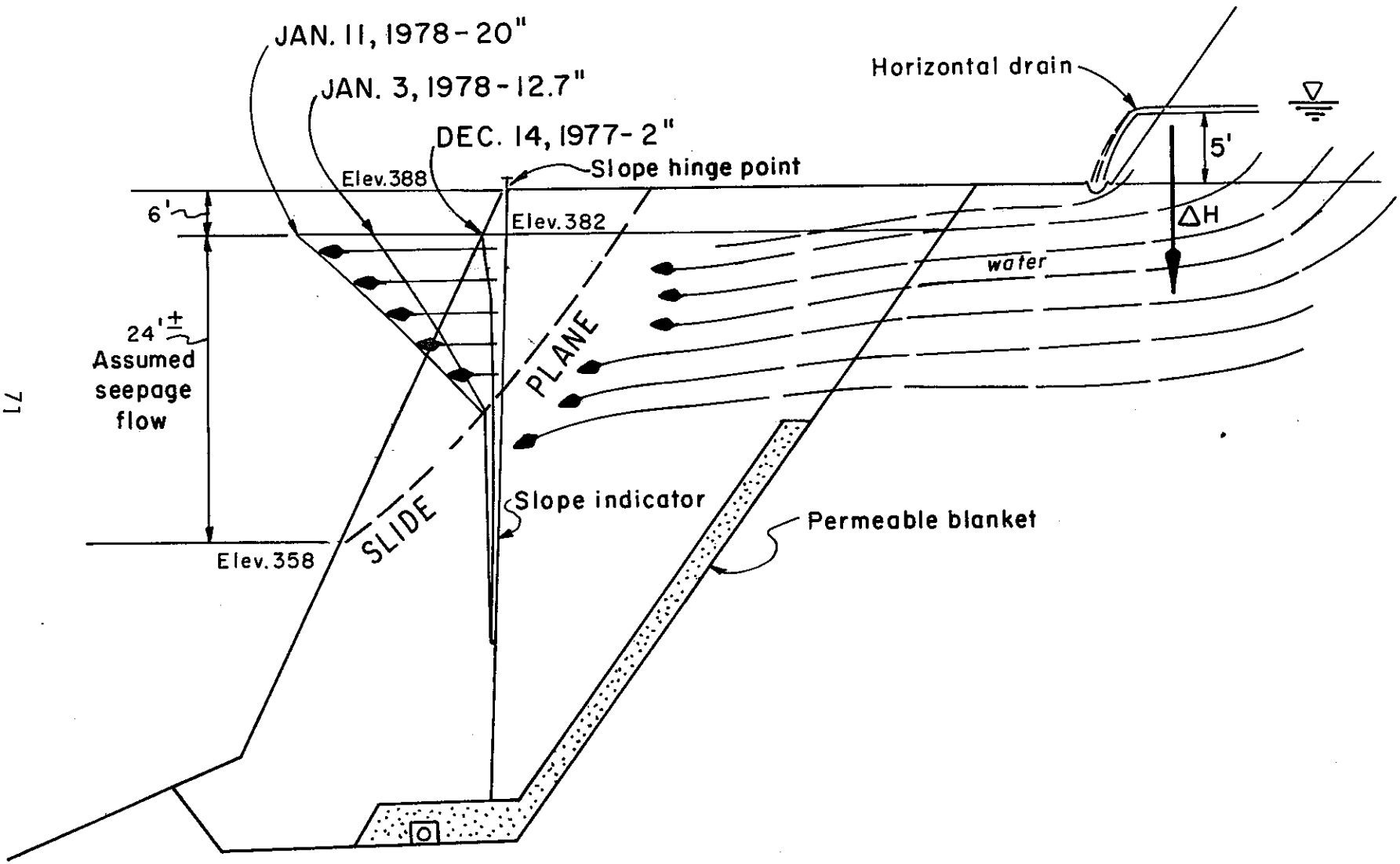

FIG. 33 SEEPAGE FLOW
(Sta. 12+00)

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Slide Analysis

A wedge analysis of the slide was performed using strength parameters determined from triaxial (CU) soil tests of remolded backfill samples as discussed previously in the Soil Properties Section. The analysis assumed the following:

Resisting force (RF) = $cL + \Sigma clips + (N-U)\tan\phi$
Driving force (DF) = $W\sin\theta$
Factor of Safety (F.S.) RF/DF = 1 (at equilibrium)

$$F.S. = \frac{cL+\Sigma clips+(N-U)\tan\phi}{W\sin\theta}$$

where:
N = normal force = $W\cos\theta$
U = seepage force = $\gamma_W \Delta HA$
γ_W = unit weight of water
ΔH = hydraulic head
A = cross-sectional area of seepage of 1-ft width
Other items are previously defined.

Seepage forces were assumed to have acted at depths between 6 ft and 30 ft below the slope hinge-point, as shown in Figure 33. These forces were assumed to be acting perpendicular to the slip plane. The computed resultant seepage force was 34,500 lbs (see Figure 34).

The clip and tire resisting forces were added from depth 4.7 and 30.7 ft from the hinge-point of the slope at 2-ft vertical intervals. This force totaled 12,200 lbs for 13 mats using equations 4 and 5, or Figure 35.

The total weight of the wedge shown in Figure 34 was estimated to be about 24,700 lbs excluding the eroded portion of the slope. The length of the slip face considered able to develop

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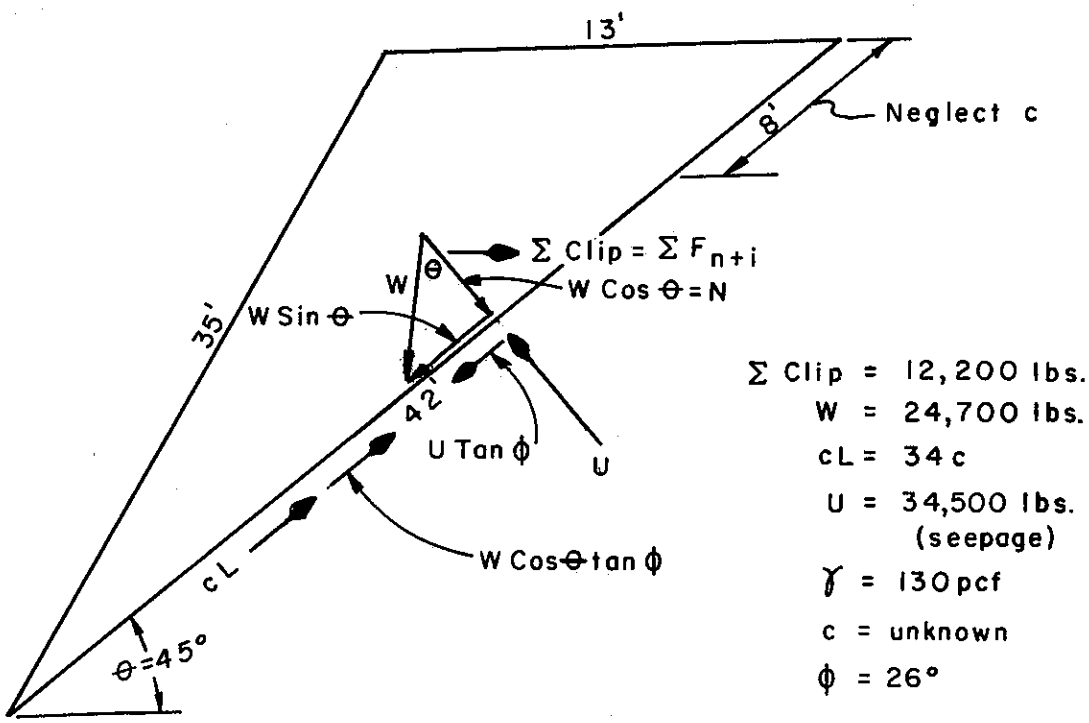

FIG. 34 FORCES ON FAILURE WEDGE

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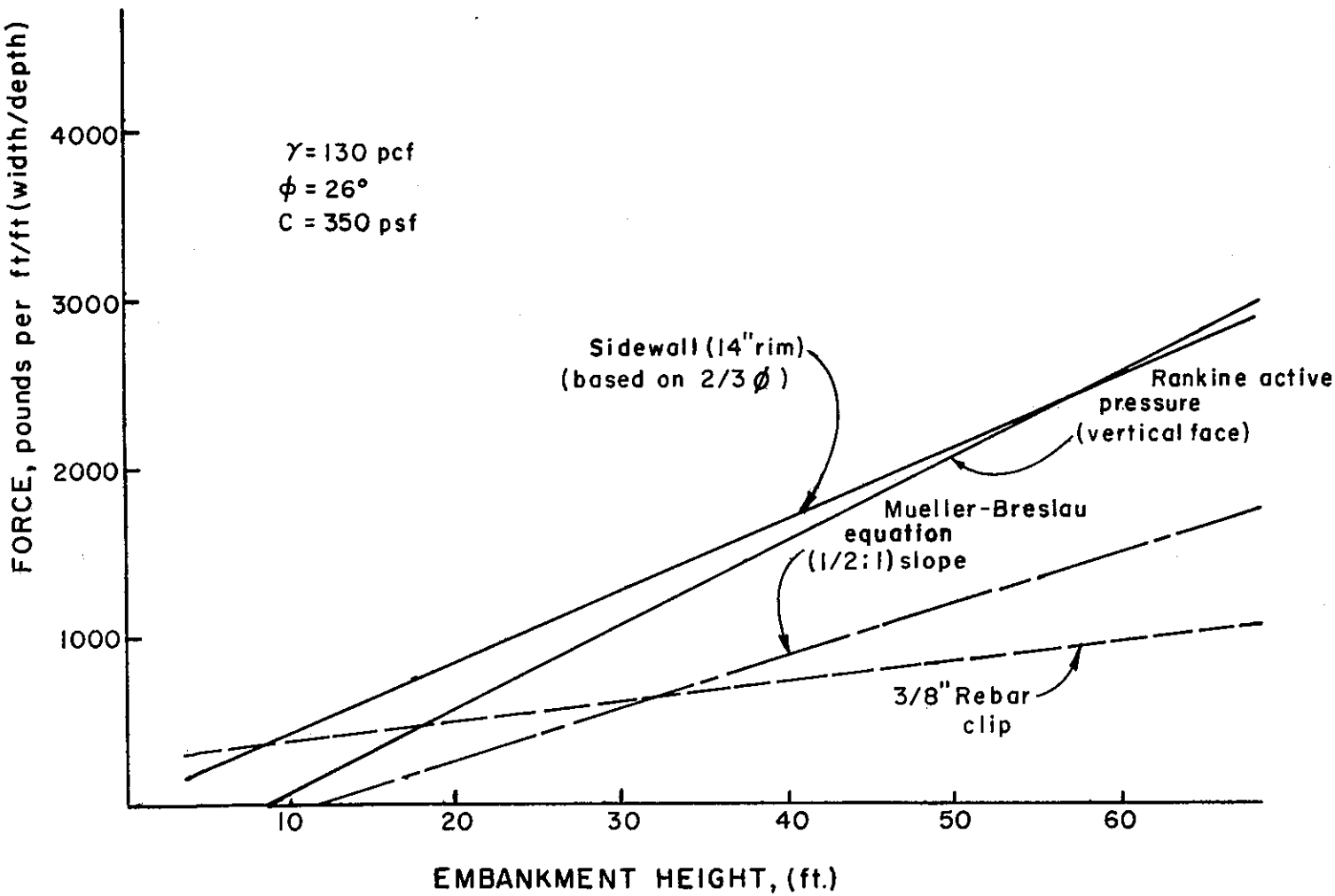

FIG. 35 RESISTING FORCE OF SIDEWALL AND REBAR CLIP AND ACTIVE EARTH PRESSURE VS EMBANKMENT HEIGHT

a resisting cohesive force was estimated at 34 ft. Tension cracks had developed on the paved shoulder and the depth was estimated to be about 8 ft. The angle of the slip face from the horizontal plane was estimated in the field to be about 45 degrees.

The results of the analysis are shown in Table 4.

TABLE 4 COMPARISON OF SOIL STRENGTH VALUES

Condition	c(req'd) psf	c(available) psf	ϕ degrees	Sample Condition
Non reinforced/ no seepage force	200	1100	31	moist
Non reinforced/ seepage force	760	350	26	saturated
Reinforced/no seepage force	0	1100	31	moist
Reinforced/seepage force	390	350	26	saturated

As can be seen from Table 4, the last condition (Reinforced/ seepage force) required a c value of about 390 psf for a stable slope which was in excess of the available c value (350 psf) from laboratory triaxial tests. As a result, the slope became unstable.

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SECTION III

EMBANKMENT REPAIR

Several alternatives were considered for repair of the embankment. One scheme involved rebuilding the embankment to the original slope and using large rocks to control erosion. This plan was discarded after further study indicated it would be more costly than other alternatives, since heavy construction equipment would be unable to work efficiently within the restricted area of the project. In addition, suitable rocks were not available near the jobsite. Another scheme consisted of constructing a stepped 1/2 to 1 slope utilizing treated wood facing, with provision for establishment of native plants in the step areas. This, too, was dropped for economic reasons.

The alternative selected embodied a strut placed along the bottom half of the existing embankment. The strut would be placed with a 1/2 to 1 slope. The upper half of the existing embankment, where local slope failure had occurred, would be cut back to a 3/4 to 1 slope (see Figure 36). This plan provided excavated soil at the site for the strut and allowed adequate maneuvering area for the use of heavy construction equipment. The strut fill also provided the highest safety factor of the various remedial measures considered.

An integral part of the repair was correction of an inadequate subsurface drainage condition.

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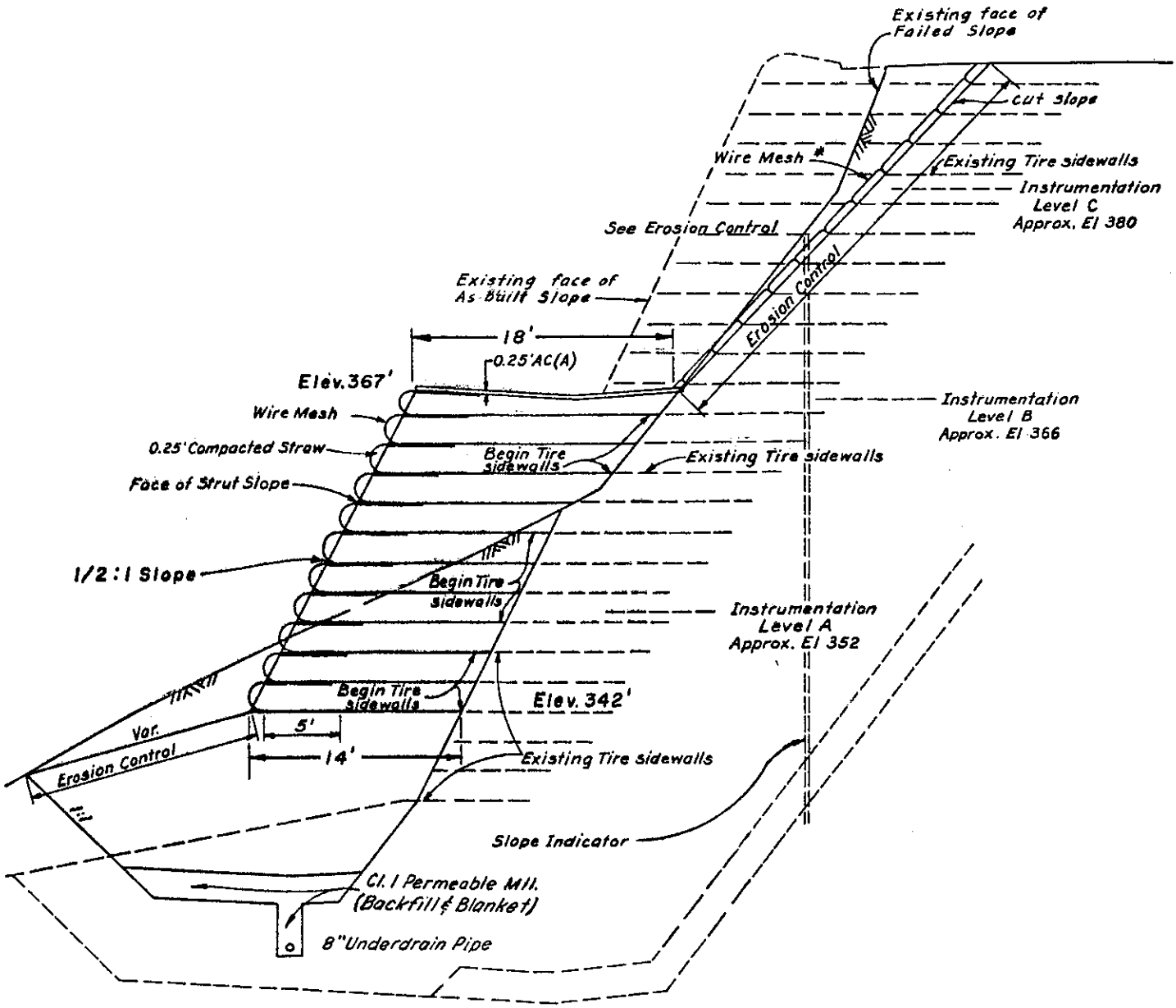

FIG. 36 TYPICAL STRUT SECTION
(Sta. J 11+99)

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As previously mentioned, the original plan carried the two ft permeable layer up the backslope of the reinforced embankment to within 15 ft of roadway level (see Figure 3). The decision to terminate drainage at this depth was based upon a lack of evidence of ground water migration at a higher elevation. However, the almost unprecedented rainfall of the winter of 1977-1978 raised ground water above roadway level at least locally as indicated by the seepage from an old horizontal drain approximately 5 ft above roadway level in the cut opposite the reinforced embankment (see Figure 33). The subject drain had been inactive for many years and was assumed to be plugged, if indeed it had ever intercepted ground water. Thus a substantial portion of the reinforced embankment was undrained under unusually wet conditions. That inadequate subsurface drainage was a prime factor in the local slide is demonstrated by the proximity of the undrained portion of the embankment to the zone of maximum movement as indicated by the slope indicator (Figure 33). To correct the problem, an additional permeable filled cut-off trench, 2 ft wide and about 16 ft deep, was constructed at the toe of the cut slope to intercept groundwater in the upper embankment zone (see Photo 15). Relief was accomplished by daylighting the trench downslope and interception by horizontal drains from the new bench.

1. Construction

Repair of the embankment was undertaken October 10, 1978 and completed January 18, 1979. The embankment was stabilized by installing sidewall sections cut from discarded automobile tires in the embankment strut as shown in Figures 36 and 37 similar to the original contract (see Photos 16 and 17). The

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limits of the tire reinforcement for the embankment were from Station 11+00 to Station 12+75.

The tire sidewall reinforcing mats were installed at 2-ft intervals. Tires were fastened in two directions to form a mat without overlap. Each row was extended through the full width of the strut and connected to the existing sidewall mats at the face of the original embankment. Placement of the tires started at the existing sidewalls and conformed to the original sidewall pattern (see Photo 18). Slack was eliminated or minimized by tensioning the mats so that clips were in contact with beads of the tire sidewalls. The first level of tire sidewalls was installed at Elevation 342 (see Figures 36 and 37). Relative compaction of the earth between layers of mats was 90 percent.

2. Materials

The tire material requirements were generally the same as the original contract. The rebar clips were increased to 1/2 inch diameter instead of the 3/8" that was used in the original contract. Clip details are shown by Figure 38.

3. Strut Slope Erosion Protection

Strut slope erosion control consisted of stabilizing the embankment with wire mesh and straw as shown in Figure 39. The face of the strut slope was wrapped with 5-ft wide strips of galvanized one-inch by 2-inch welded wire mesh. Placement of tire sidewalls, the outer edge of compacted embankment material, and compacted straw, were performed prior to wrapping with wire mesh. Straw and soil were placed in excess quantities so that when compacted by

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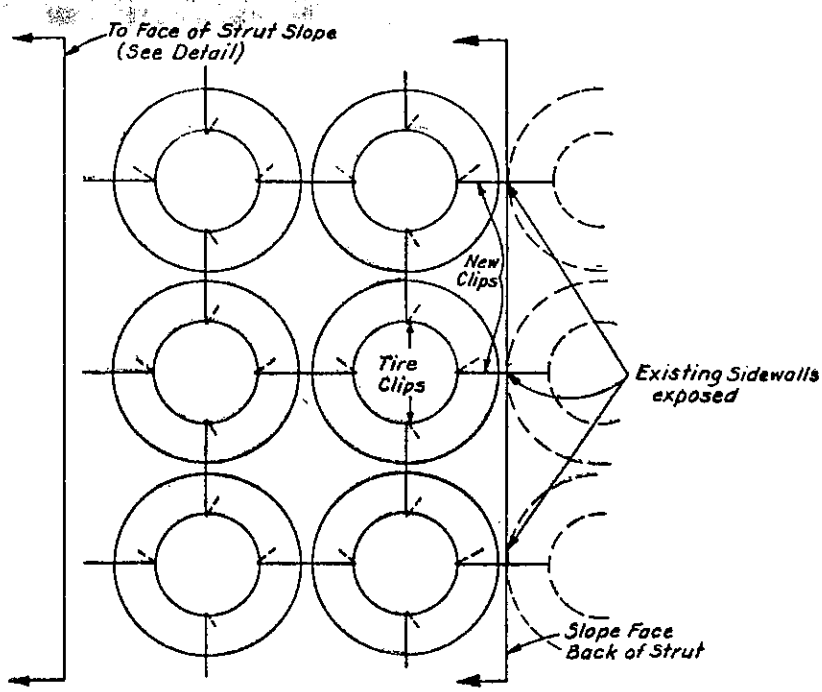

FIG. 37 TIRE ARRANGEMENT AND CLIP

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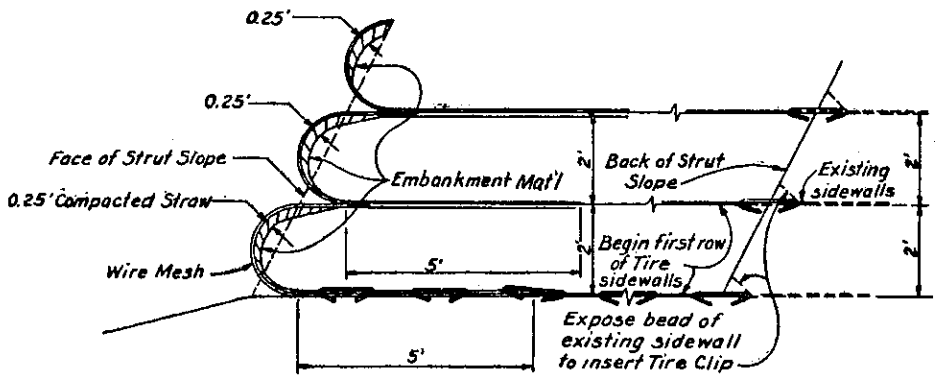

FIG. 39 STRUT SLOPE EROSION CONTROL

NOTES

(a.) All measurements except bend radius are to center of Bar. Bend radius is inside diameter.

(b.) Material is to be of "Cold-rolled" $\frac{1}{2}$" diameter steel.

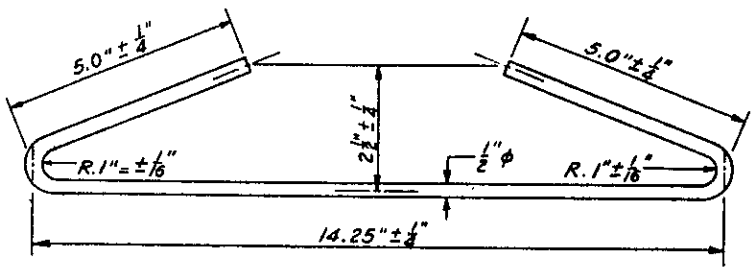

FIG. 38 TIRE CLIP

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wrapping of the wire mesh, they could achieve the minimum dimensions shown on Figure 39. Adjacent edges and ends of the wire mesh strips were overlapped a minimum of 6 inches and tied securely at 18-inch intervals with 14-gage wire or hog rings (see Photos 19 and 20).

4. Cut Slope Erosion Protection

Cut-slope erosion control consisted of a biodegradable mat, straw, and a welded-wire mesh (see Photo 21). Prior to placing straw, the biodegradable mat was placed horizontally across the slope and anchored in place with 4-inch steel staples. The biodegradable mat was in 3-ft wide rolls with a minimum length of 100 ft. The mat consisted of seed, fertilizer, and peat moss bonded together between a polyester insulating mesh and Number 11 tissue paper. Straw was spread evenly to an uncompacted thickness of not less than 6 inches over the areas to be stabilized. Following the straw, galvanized 1-inch by 2-inch by 60-inch, 14-gage welded wire mesh was laid carefully and evenly across the slope. The wire was drawn taut and securely anchored to existing tire sidewalls as shown in Figure 40. Adjacent edges of the wire mesh strips were securely tied at 18-inch intervals with 14-gage wire or hog rings (see Photo 21).

5. Instrumentation Repair

All instrumentation readout devices were damaged by the slide. A new slope indicator was installed in the strut embankment. Existing instruments at Levels A, B, and C were reconnected to readout devices.

The repair contract was awarded for a bid price of $140,000.

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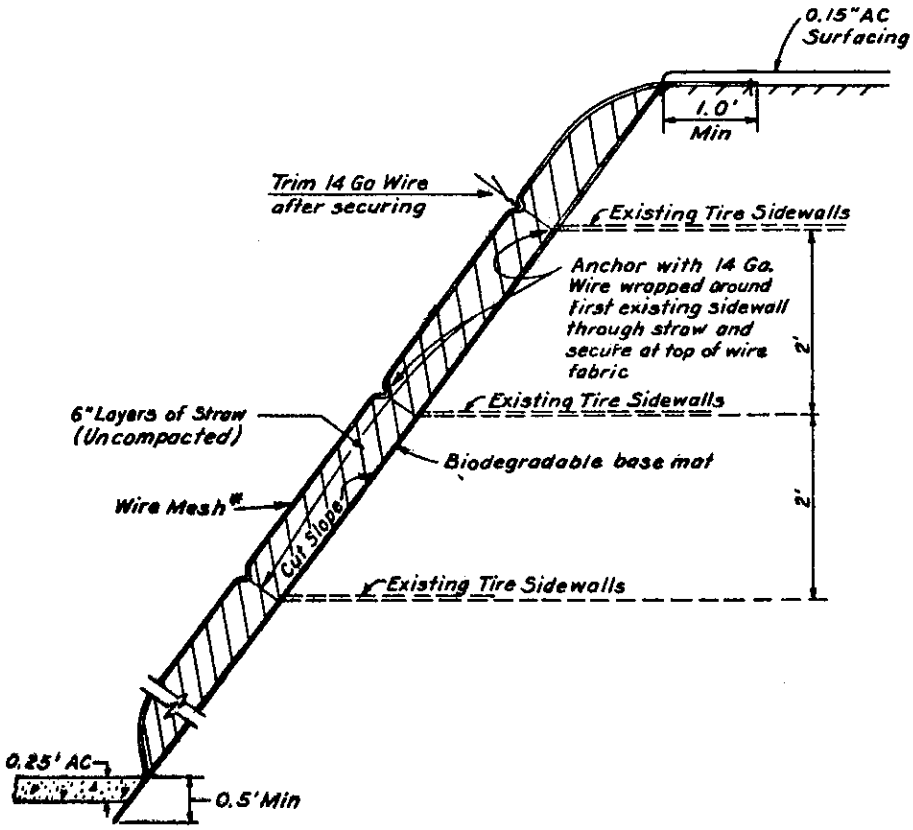

FIG. 40 CUT SLOPE EROSION CONTROL

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SECTION IV

PERFORMANCE

As stated earlier in this report, the embankment was completed in December 1976, and performed satisfactorily throughout 1977, except for some soil erosion near the lower half of the slope. In February 1978, an 80-ft portion of the slope experienced a sliding wedge failure. See Photo 12. The slide was caused by a combination of soil erosion, which undermined the upper portion of the slope, and water seepage into the fill from the adjacent hillside due to unusually heavy rainfall. The performance of the slope, mat reinforcement, tire clips, and the embankment, based on observed and instrumented behavior is described below.

Initially, the embankment was to be constructed with an unconfined slope. However, during construction, it was found that the embankment could not be compacted properly in the outer slope zone. This forced a change in the initial construction procedure of the slope. A 1-inch wire mesh was placed flat and anchored on the slope face to contain the earth for the lower half (see Photo 6). Despite the 1-inch wire mesh, soil erosion occurred, and was probably a factor in the local slide. The erosion, about 2 sidewalls deep, undermined the upper portion of the slope and caused cracks to open up in the roadway shoulder area. See Photo 14. Settlement and lateral movement of the slope occurred in the upper 20 ft. The slope indicator readings recorded the horizontal displacements. See Figure 22. The 1-inch wire mesh wrap-around between layers of earth for the upper portion of the slope performed satisfactorily until the local slide occurred.

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In spite of the saturated condition, there was no evidence of mat slippage beyond the slip plane or passive zone. This confirmed laboratory findings which indicated that embedded sidewalls subjected to an overburden pressure cannot fail by pullout. The apparent minimum friction coefficient is 0.36. Extensometers installed to the sidewalls showed no increase in elongation. Strain gages attached to the clips showed no increase in clip tension in the lateral direction. However, there were increases in clip tension in the longitudinal direction. The distribution of the traction force in the mat was not determined. Examination of the slide debris disclosed no sidewall failure due to tension break. The design procedure for the traction force calculation with respect to depth is considered satisfactory.

As indicated by pull tests, the tire clip proved to be the critical element in the sidewall mat system. Examination of failed clips showed that they were pulled straight from a bend position at depths of 5 to over 20 ft (see Photo 22).

Observations showed that the embankment behaved as an integral mass. Futhermore, Bison strain gages within the sidewalls, and extensometers recorded very small internal soil displacements. Only at near slope failure did the Bison gages show positive soil displacements. The roadway pavement experienced cracking only at two locations: at the shoulder, and at the interface or joint between the new and old existing embankment. See Photos 14 and 23. The cracks at the shoulder were caused by the unstable slope moving laterally and downward. The depth of the cracks at the interface were estimated to be about 3 ft. The cracks at the interface, 1/2 to 1 inch wide, gave evidence that the embankment was behaving as a unit and moving outward.

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Pressure cells at Level B suggest that the Ka design parameter (0.39) assuming a vertical face is conservative. However, this assumption may be justified for design due to possible soil erosion at the slope and loss of lateral support.

The performance of the reconstructed embankment with a strut has been satisfactory. Instrumentation and visual monitoring will continue for verification of long term performance.

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Photo 1 Site, Prior to Reconstruction

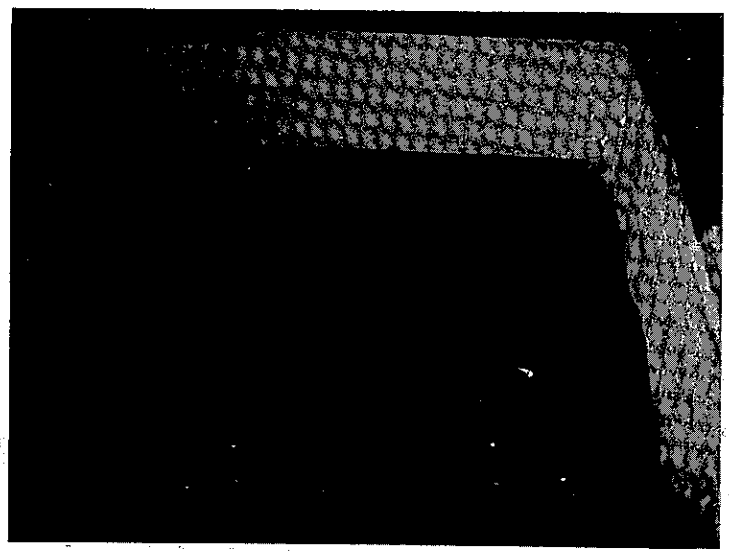

Photo 2 Sidewall Pull-out Test

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Photo 3 Initial Layer of Sidewall Mat

Photo 4 Sidewall Pattern

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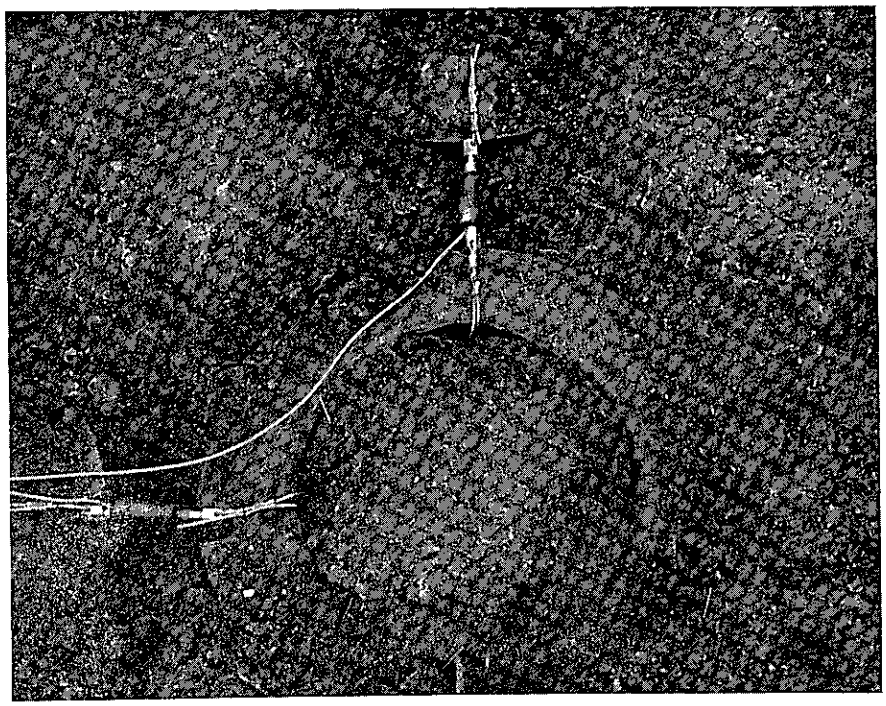

Photo 9 Clip Strain Gages

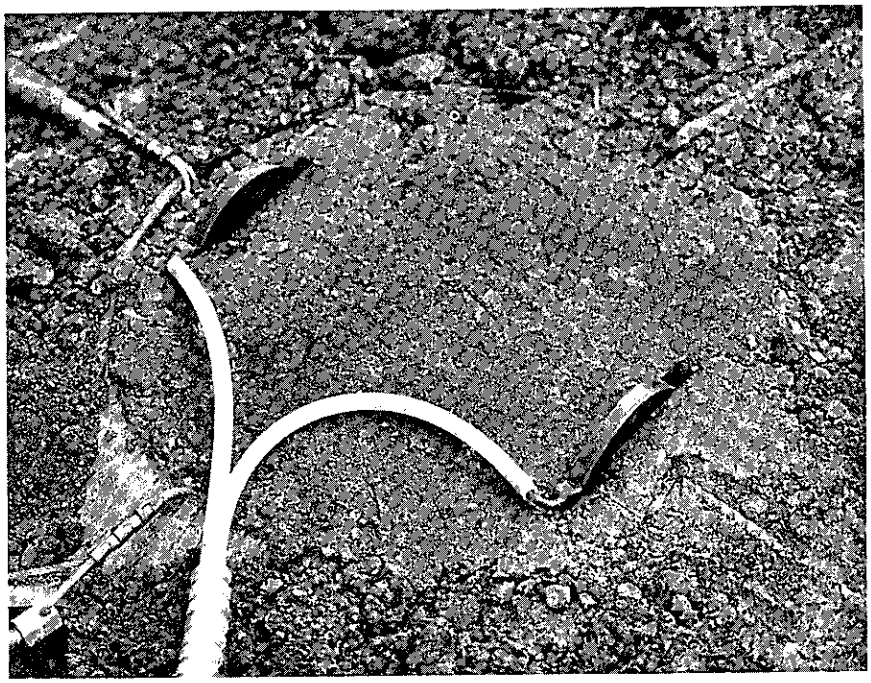

Photo 10 Bison Gages

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Photo 11 Slope Indicator

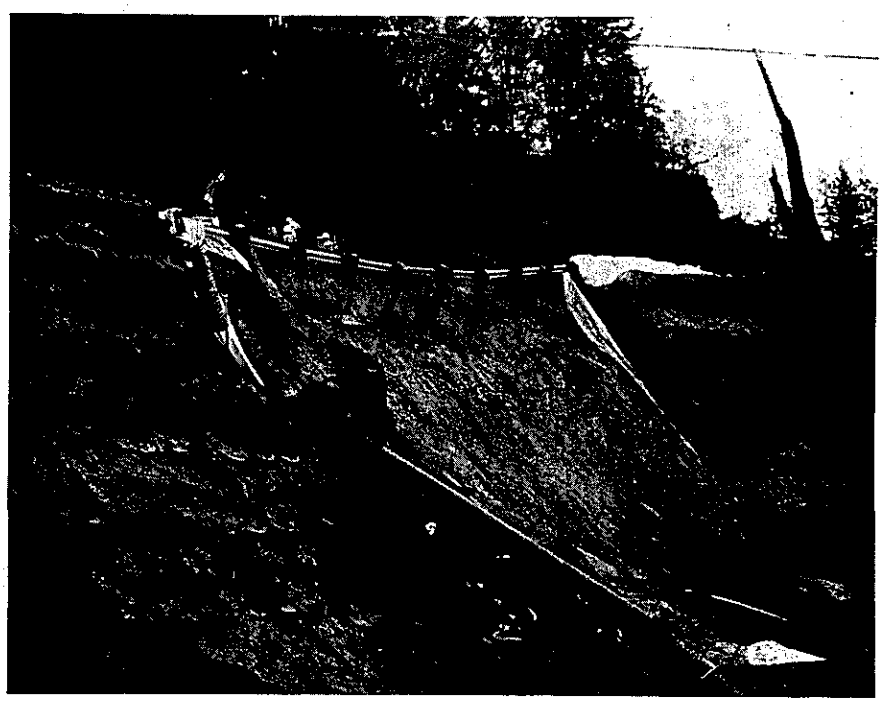

Photo 12 Local Embankment Slide

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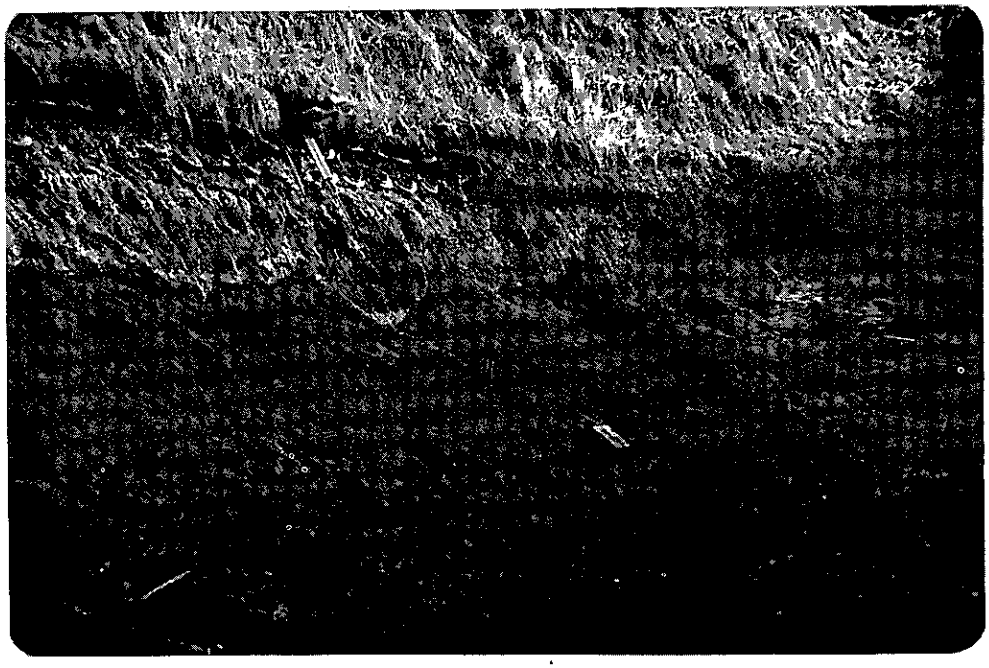

Photo 13 Seepage at Level C

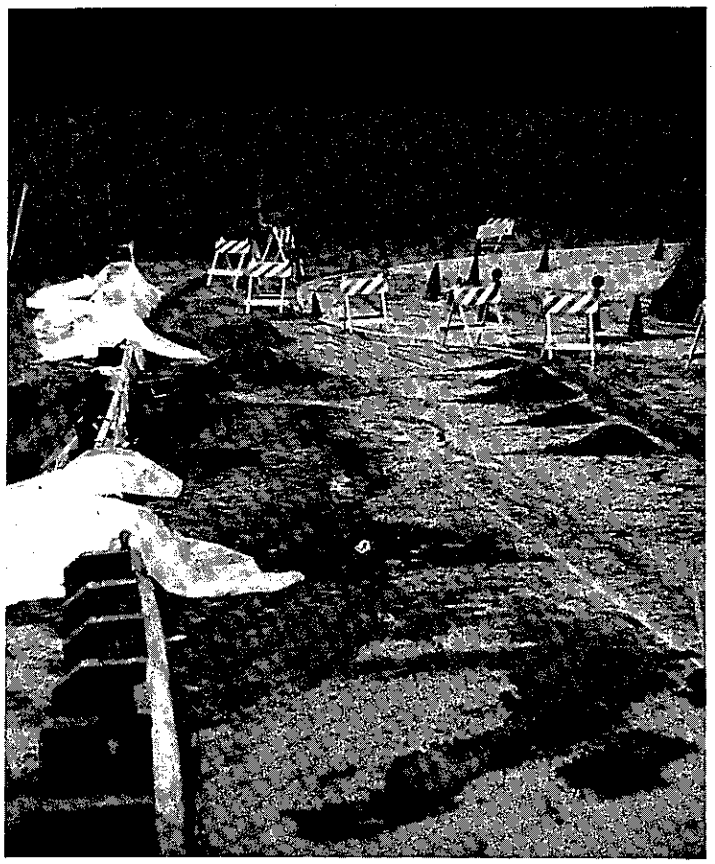

Photo 14 Settlement at Shoulder

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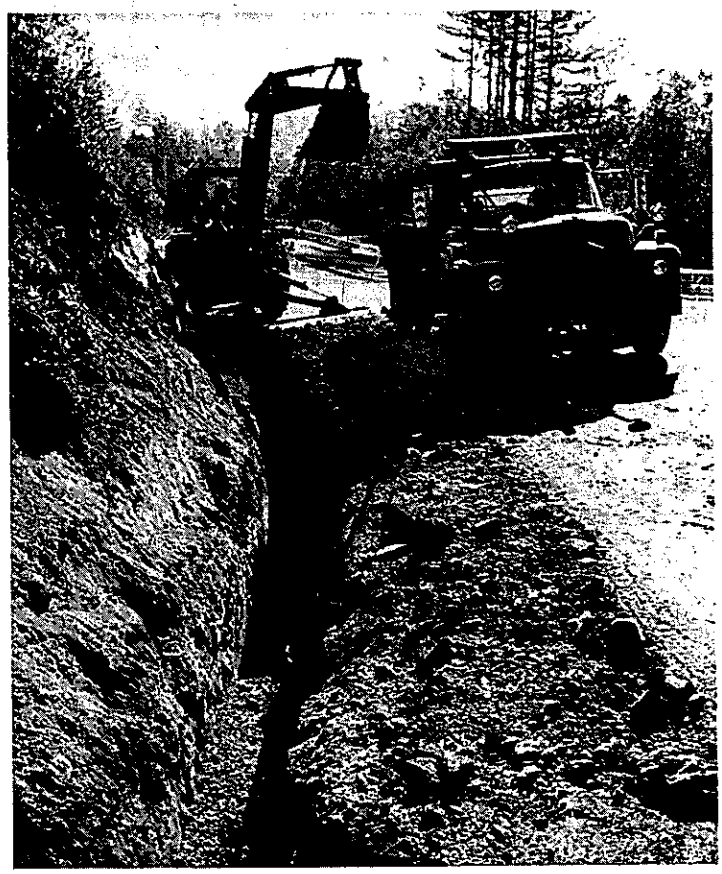

Photo 15 Cutoff Trench

Photo 16 Strut Construction

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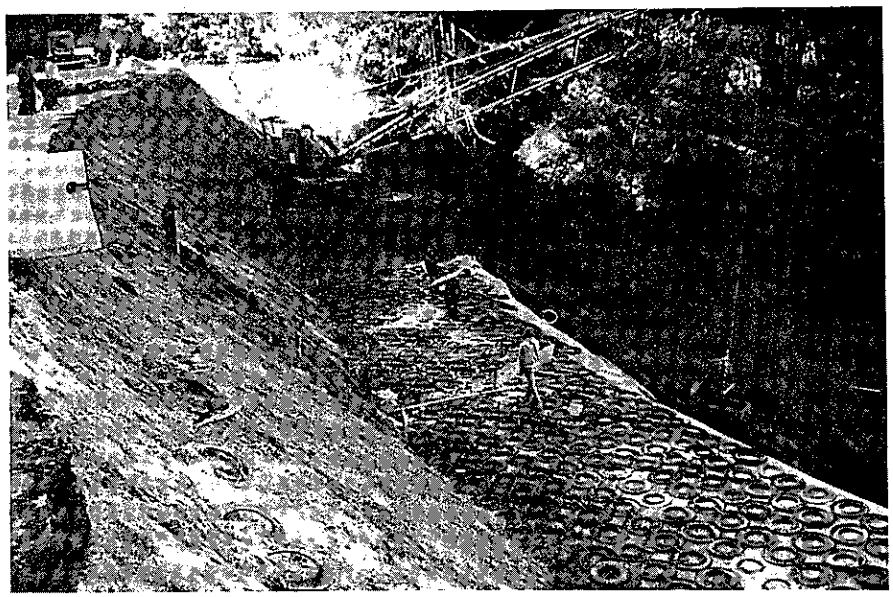

Photo 17 Strut Sidewall Mat

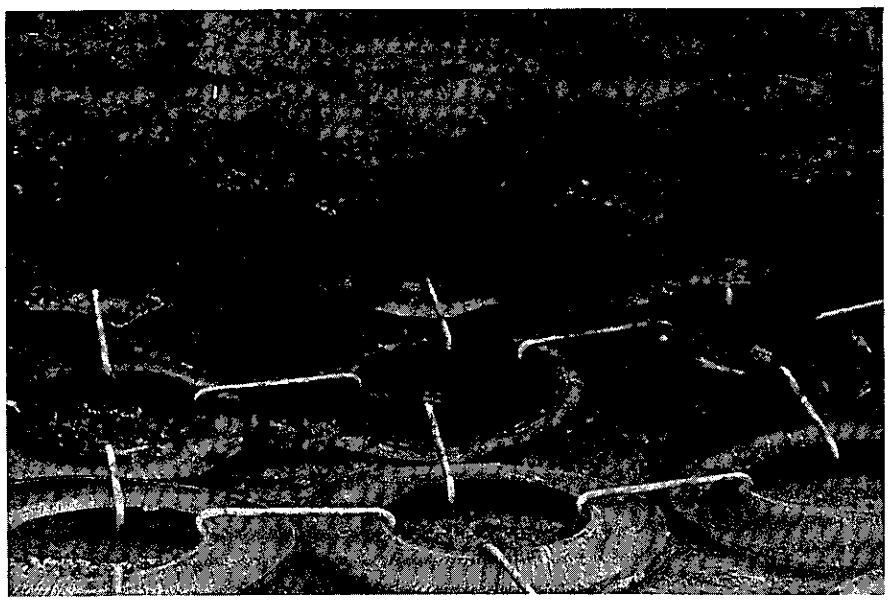

Photo 18 Sidewall Connected to Existing Mat

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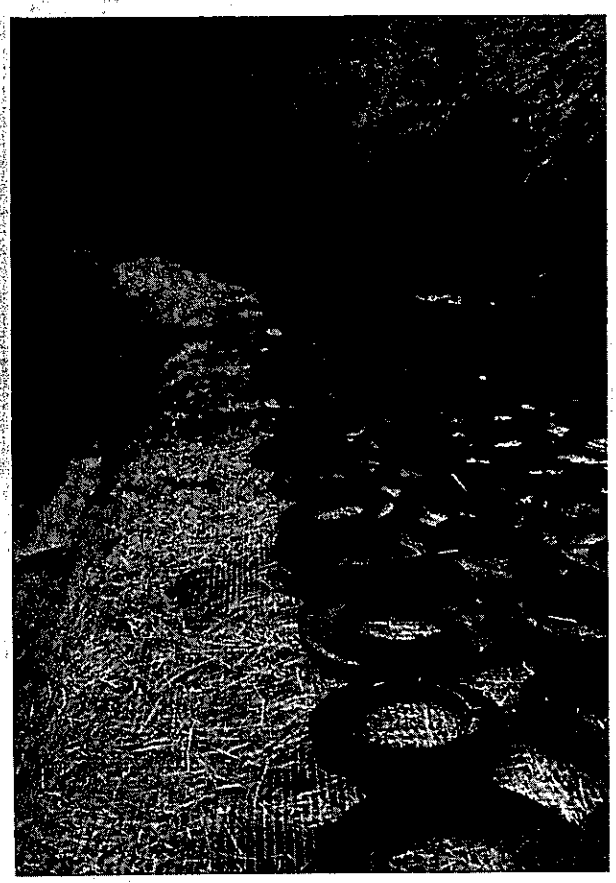

Photo 19 Strut Slope Erosion Control (Wire Mesh)

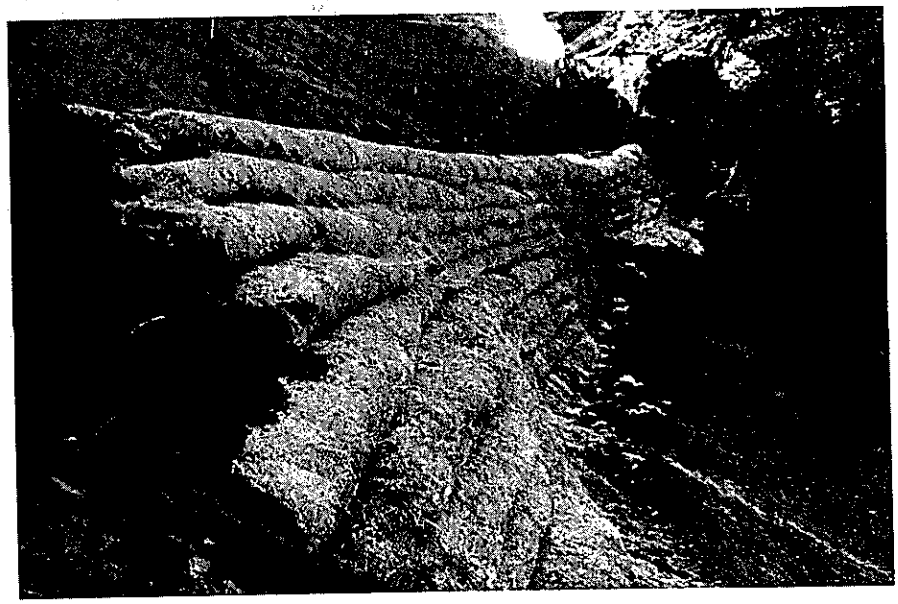

Photo 20 Strut Slope Erosion Control (Wire Mesh)

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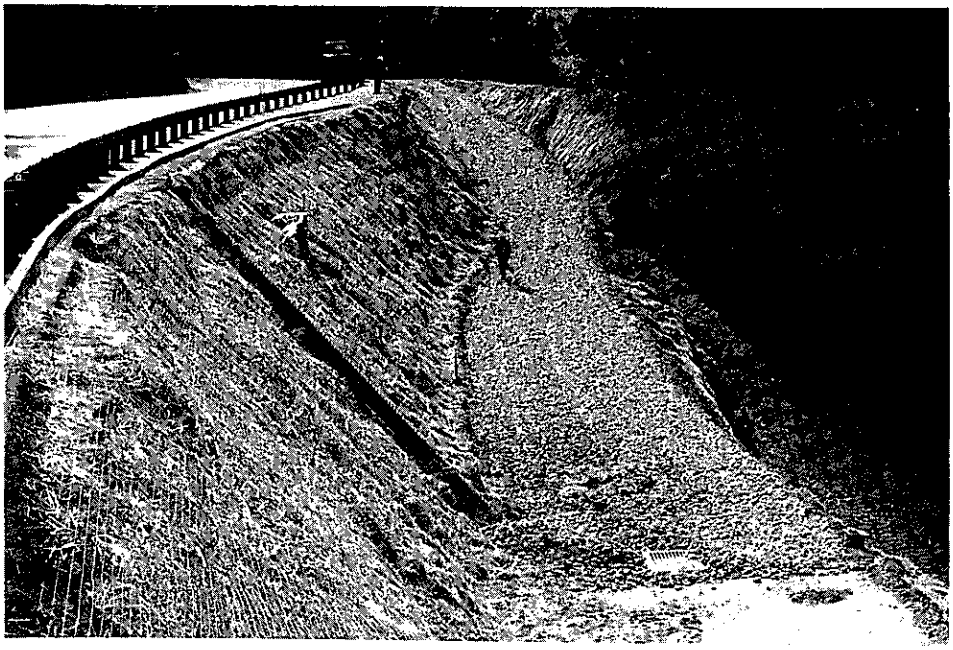

Photo 21 Cut Slope Erosion Control (Wire Mesh)

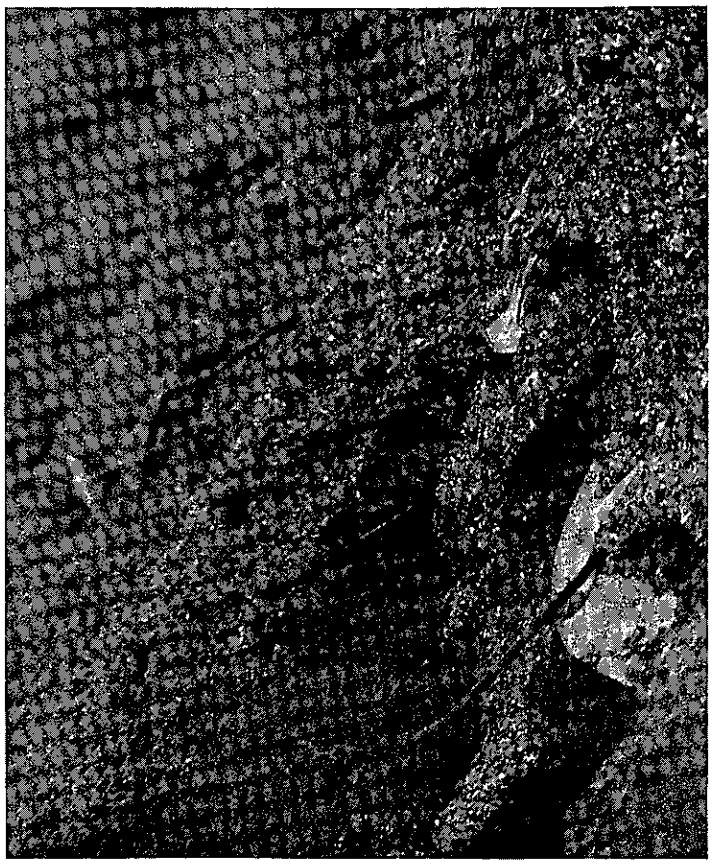

Photo 22 Clip Failure

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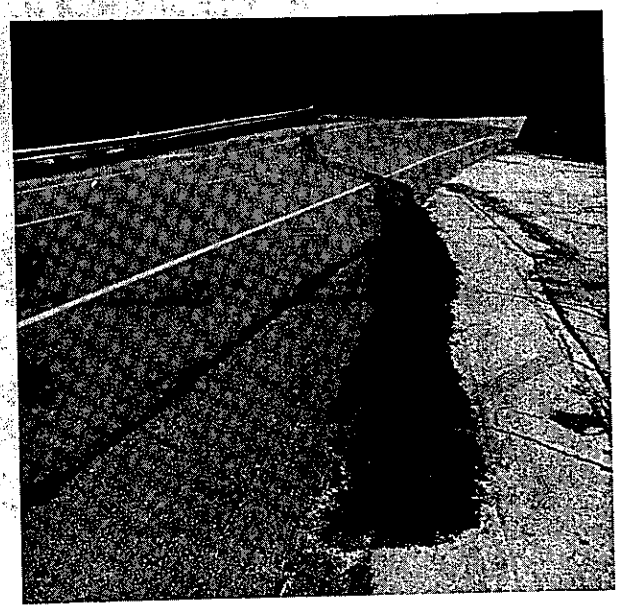

Photo 23 Cracking at Interface (Sealed)

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