Technical Report Documentation Page

1. REPORT No.

2. GOVERNMENT ACCESSION No.

3. RECIPIENT'S CATALOG No.

4. TITLE AND SUBTITLE

Landslide Investigation Near Forest Glen

5. REPORT DATE

January 1976

6. PERFORMING ORGANIZATION

7. AUTHOR(S)

R.H. Prysock

8. PERFORMING ORGANIZATION REPORT No.

9. PERFORMING ORGANIZATION NAME AND ADDRESS

State of California
Department of Transportation
Division of Structures & Engineering Services
Office of Transportation Laboratory

10. WORK UNIT No.

11. CONTRACT OR GRANT No.

12. SPONSORING AGENCY NAME AND ADDRESS

13. TYPE OF REPORT & PERIOD COVERED

14. SPONSORING AGENCY CODE

15. SUPPLEMENTARY NOTES

Prepared in cooperation with the U.S. Department of Transportation, Federal Highway Administration

16. ABSTRACT

Introduction

This report describes a recent natural slope earthslide on Road 01-Tri-36 near Forest Glen in the Trinity National Forest and summarizes the subsequent field and laboratory investigation by Caltrans' District 01 and Transportation Laboratory.

For several miles east of Forest Glen, Old Route 36 is generally parallel to the north bank of Rattlesnake Creek and suffers from acute horizontal alignment. In an effort to improve alignment and grade along this portion of the road, an FHWA construction project was in progress when the slide occurred. The slide is located about one-half mile east of Forest Glen, as shown on Figure 1, where the old road alignment includes a rather sharp "elbow" that juts southward. This elbow was eliminated by new construction which placed the new road about 250 feet north of and 50 feet higher than the old road. Construction of the realigned portion, however, required excavating a through cut about 350 feet long as measured along the north (uphill) cutface, and about 40 to 50 feet deep at the north edge of roadway.

Shortly after completion of the cut in July, 1975, FHWA Project personnel observed the development of a series of cracks on the north cutface at Station 1249+. Further investigation revealed visible distress was a manifestation of a large incipient slide which apparently was triggered by removal of lateral support as a result of construction of the cut. In order to arrest movement, the FHWA

17. KEYWORDS

Lab. Auth. 652798
01-Tri-36

18. No. OF PAGES:

102

19. DRI WEBSITE LINK

http://www.dot.ca.gov/hq/research/researchreports/1976-1977/76-69.pdf

20. FILE NAME

76-69.pdf

TRANSPORTATION LABORATORY

76-69

LANDSLIDE INVESTIGATION NEAR FOREST GLEN

01-Tri-36

Prepared in Cooperation with the U.S. Department of Transportation,
Federal Highway Administration

Caltrans
CALIFORNIA DEPARTMENT OF TRANSPORTATION

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF STRUCTURES & ENGINEERING SERVICES
OFFICE OF TRASNPORTATION LABORATORY

January 28, 1976

Lab. Auth. 652798

Mr. Jerry L. Budwig
Director, Office of Federal Highway Projects
Federal Highway Administration, Region 8
Denver Federal Center
Denver, Colorado 80225

Dear Sir:

Submitted for your consideration is:

REPORT
OF
LANDSLIDE INVESTIGATION
NEAR
FOREST GLEN, CALIFORNIA
01-TRI-36

Study made by . . . . Geotechnical Branch

Under the General Direction of . . . . Raymond A. Forsyth

Work Supervised & Report by . . . . R. H. Prysock

Very truly yours,

GEORGE A. HILL
Chief, Office of Transportation Laboratory

By Raymond A. Forsyth
R. A. Forsyth
Chief, Geotechnical Branch

Attachments

INTRODUCTION

This report describes a recent natural slope earthslide on Road 01-Tri-36 near Forest Glen in the Trinity National Forest and summarizes the subsequent field and laboratory investigation by Caltrans' District 01 and Transportation Laboratory.

For several miles east of Forest Glen, Old Route 36 is generally parallel to the north bank of Rattlesnake Creek and suffers from acute horizontal alignment. In an effort to improve alignment and grade along this portion of the road, an FHWA construction project was in progress when the slide occurred. The slide is located about one-half mile east of Forest Glen, as shown on Figure 1, where the old road alignment includes a rather sharp "elbow" that juts southward. This elbow was eliminated by new construction which placed the new road about 250 feet north of and 50 feet higher than the old road. Construction of the realigned portion, however, required excavating a through cut about 350 feet long as measured along the north (uphill) cutface, and about 40 to 50 feet deep at the north edge of roadway.

Shortly after completion of the cut in July, 1975, FHWA Project personnel observed the development of a series of cracks on the north cutface at Station 1249±. Further investigation revealed visible distress was a manifestation of a large incipient slide which apparently was triggered by removal of lateral support as a result of construction of the cut. In order to arrest movement, the FHWA authorized the installation of a series of horizontal drains from a natural drainage swale in the center of the slide mass near the roadway. These drains, while successfully intercepting some subsurface water, had no apparent effect on the visual rate of slide movement.

Because the project was scheduled to be accepted for maintenance by District 01 of the Department of Transportation in November, a field review of the slide was made by representatives of the Forest Service, FHWA, District 01, and the Transportation Laboratory on August 26, 1975. As a result of this review, it was concluded that in spite of efforts to drain the slide, it was continuing to move at the rate of approximately 1/2" per day at the north cutface.

Because this movement was occurring during the dry part of the year, it was apparent that movement would probably accelerate with the advent of wet weather. The possible consequence of a massive slide movement would be the closure of Route 36 and encroachment of slide debris into Rattlesnake Creek. This real possibility plus the massive nature of the slide, prompted District 01 to recommend that an investigation be conducted to determine the slide mechanism and to evaluate possible means of correction. Consequently, a Forest Highway Project Agreement was entered into by the FHWA and the California Department of Transportation (Transportation Laboratory) on September 17, 1975, to conduct the proposed investigation. Included in the proposed program was a geology evaluation, development of a topographic map, subsurface exploration, instrumentation, appropriate laboratory tests, and analytical studies.

DESCRIPTION OF SLIDE

Movement of the slide was first detected in the north cutslope, near the toe, as evidenced by the eastern flank crack which slowly began to lengthen up and beyond the top of cutslope. Also, the roadbed through the cut became uneven due to differential upthrusting action typical of slide toe zones and began to require frequent removal of debris from along the northedge of roadway.

The slide is about 1,000 feet long transverse to the new road alignment and some 400 feet wide at the toe, narrowing to about 250 feet in the upper portion, and covers an area of about seven acres as shown on Figure 2. Crack patterns and differential movement within the slide boundary indicate the presence of a secondary slide, the upper scarp of which is located about 550 feet uphill from the new road alignment. The slide is traversed generally along its axis by a ravine which serves as a drainage channel during periods of rainfall.

Topography of the slide area is very irregular as may be noted on Figure 2. Because of the fairly sharp bends and switchbacks in the ravine, there are several localized natural slopes with cracks that indicate tendencies for secondary movements in various directions. This phenomenon makes it extremely difficult to appraise on the site the overall slide movement pattern.

The slide is apparently a reactivation of an older slide that underwent slow movement over a fairly long period of time. During field reviews several large diameter trees were observed to exhibit the characteristic downslope leaning of the lower trunks. Also, the topography in the upper region of the present slide has the appearance of having slumped somewhat many years ago.

During a field review on December 2, 1975, water was seeping out of the cut at roadway level and flowing into the CMP crossdrain near Station 1249. Water was also flowing from the several horizontal drains previously mentioned and shown on Figure 2. At the head of the slide, surface water was seen flowing into the slide area where it quickly disappeared by percolating down into the subsoils.

SITE GEOLOGY

The site of the slide is located in the Klamath Mountain geologic province of California. Geologically, the area is comprised of undifferentiated Triassic and Paleozoic rocks consisting of volcanic, granitoid, and ultra-basic rock types. The granitic rocks are similar to those found to the south in the South Fork Mountain granitic zone. Textures and mineralogic content are extremely varied over short distances and rock types change rapidly. The rocks are highly fractured and deep weathering is common. Blue-gray to black clay is commonplace and can be found underlying much of the area. The slide is bordered to the northeast by fractured granitoid and plutonic rocks and to the southwest by hard fractured vesicular basalts.

Since being exposed in the cutfaces, the softer materials have eroded severely. This is especially true of the southerly cutface where large fractured rock masses were undercut and will lead to rockfall during the winter.

The slide mass itself is composed of a variable mixture of soil and different type rocks. The large pieces have an average diameter of about two inches, with most of the material having weathered to the consistency of a soil. Any fracturing or structural control for the slide is difficult to identify, although slickensides suggest that faulting may be present.

SITE INVESTIGATION

During October 7-15, 1975, a drill crew from District 01 made four borings (Nos. 246-249) through the slide mass to depths of 61,63,72 and 73 feet. Tube samples were taken at various depths for laboratory testing and then the boreholes were used for slope indicator installation. Examination of samples revealed the subsoils to consist primarily of brown and gray sandy clayey silt with a high percentage of rock fragments comprised of weathered gneiss, schist, quartz, and serpentine. Hard rock, as indicated by auger refusal, was found in three of the four borings at bottom depths of 63 to 73 feet. No well-defined stratification was evident and no reliable generalized soil profile could be developed, especially in view of the probable folds and distortions in the substrata.

Free groundwater was encountered during drilling at depths of 23, 43,51 and 57 feet, respectively, as noted on the boring logs, Figures 4-7.

The site investigation also included mapping the slide area to provide a geometrical basis for subsequent analytical work. The resulting map is included as Figure 2.

The pattern of slide movement as shown by slope indicator data, noted on Figure 2, substantiates visual observations of ground surface cracks. SI-1, for example, shows a resultant movement in a westerly direction almost 90 degrees from the direction of overall movement. This localized movement toward a free surface (the natural slope down to the drainage course) probably results from a pinching or squeezing action as the overall slide mass rides over or around firmer material. Indicators SI-2,-3, and -4 show resultant movements in the general direction of overall slide movement. SI-3 shows an interesting pattern in that from ground surface to a depth of about 52 feet, the resultant movement is in an easterly direction toward a free surface, whereas overall movement (below 52 feet) is in a southerly direction toward the new road alignment and slide pushup area. The seemingly erratic pattern of slide movement is believed to result from the subsurface twisting, squeezing, and overriding caused by a complex stratigraphy that probably is reflected by the topography.

The depth of movement is shown on Figures 8-11 and appears to be 55 to 65 feet below ground surface at slope indicators 3 and 4. Indicator 4, for example, indicates the presence of a slip surface at a depth of about 60 feet, at which depth the casing sheared between the October 21 and November 3 readings. Indicators 2 and 3 show maximum movements to depths of 20 and 50 feet, respectively. The casing deflection diagram for SI-1, decreasing more or less linearly from ground surface to zero at bottom of casing, indicates that either the slip surface is below the casing bottom or there is no definite slip surface or zone, and movement consists of shearing strains over the full depth of about 60 feet. The movement of SI-1, however, is believed to be, at least partially, of a localized nature due to the resultant direction.

Converting depths to elevations, the apparent slip surface at indicators 3 and 4 is at elevation 2493 and 2504, respectively, for the upper movement. The zone of lower movement at SI-3 appears to be at elevation 2481. If the slopes of these probable upper and lower slip surfaces are projected in easterly and southerly directions, interception with the cutslope near its eastern end would occur slightly above toe of slope and slightly below the toe to the south, which corresponds very well with field visual observations.

The total amount of differential movement along the scarps is relatively small considering the time period of some six months since first detection. Also, the maximum movement registered by the slope indicators is only six inches, over a three-month period. The toe area of the slide which includes the lower portion of the roadway cutslope and the roadbed itself is the only location where appreciable differential movement has been observed. While movement in this area has not been measured, it has been of sufficient magnitude to visibly note an increase from day to day and to require cleanup by the use of equipment.

The rates of movement at different depths have been rather erratic as shown by Figures 12-15. These plots show movement at depths of 10,30 and 50 feet for the four slope indicators. Generally, rates of one inch per week or slower were prevalent prior to the corrective treatment, whereas a slowing trend appears to be developing for the period after mid-December. No definite conclusions can be drawn at the present time.

As of December 30, 1975, groundwater surface was at depths of 14 feet in SI-1, and about 35 feet in the remaining three holes. These depths represent an increase in water level of only three and six feet in Holes SI-2 and -4, respectively, but larger increases of eight and 33 feet for Holes SI-1 and -3. Similar rises have been noted for the newer Holes SI-5 and -6, as shown in Table 1.

Readings made during mid-December showed a drop in water levels in Holes SI-2,-3 and -4. This drop is believed to have resulted from the diversion of surface water from the upper portion of the slide, as part of the temporary corrective treatment during the week of December 8. However, as may be noted from Table 1, the most recent readings indicate water is again rising. More time and additional readings will be necessary before a reliable assessment of stable groundwater levels can be made.

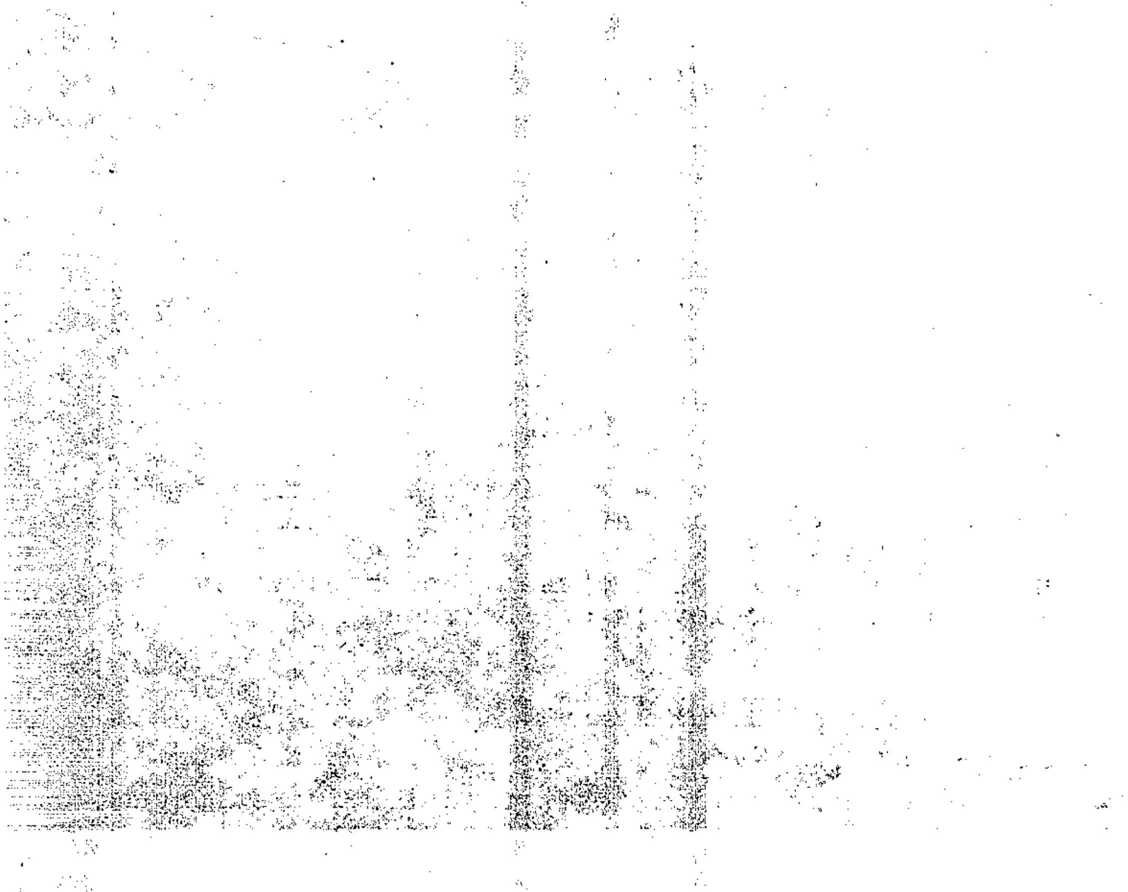
LABORATORY TESTS

Laboratory testing of retrieved tube samples was restricted to direct shear and triaxial compression tests to determine strength parameters for stability analyses. Samples tested were from depths of 59 to 62 feet in Boring 248 and were selected based on slope indicator data which suggested a slip surface or zone at that depth. Difficulty was experienced during the testing process because of the generally high percentage of rock fragments in the samples. Overall, the materals sampled were not completely suited for laboratory testing and the reliability of results must be considered questionable.

Two series of direct shear tests were run by repeatedly shearing the specimens along the same failure plane to determine residual strength values. One of the series did exhibit strength loss with repeated shearing and gave residual values of 700 psf and 12^{o} for unit cohesion and friction angle, respectively. The remaining series did not show appreciable strength loss and resulted in strength values of C=0 and $\emptyset$=26^{o}. These results are plotted on Figure 16.

A two-specimen series of consolidated-undrained triaxial compression tests with pore pressure measurement was conducted to determine effective strength parameters. The specimens were consolidated under confining pressures of 4 and 8 tsf to minimize possible over-consolidation effects, and then loaded to 42 percent axial strain. The purpose of extending the strain to 40 percent was to check for significant strength loss at the greater strains.

The results, shown plotted as a Mohr diagram in Figure 17 demonstrate that no appreciable loss of strength occurred. The common failure criterion of maximum principle effective stress ratio, which occurred at less than 20 percent strain, yielded a $\emptyset$ - angle of 33^{o} whereas 42% strain values produced a $\emptyset$ - angle of 29^{o}.



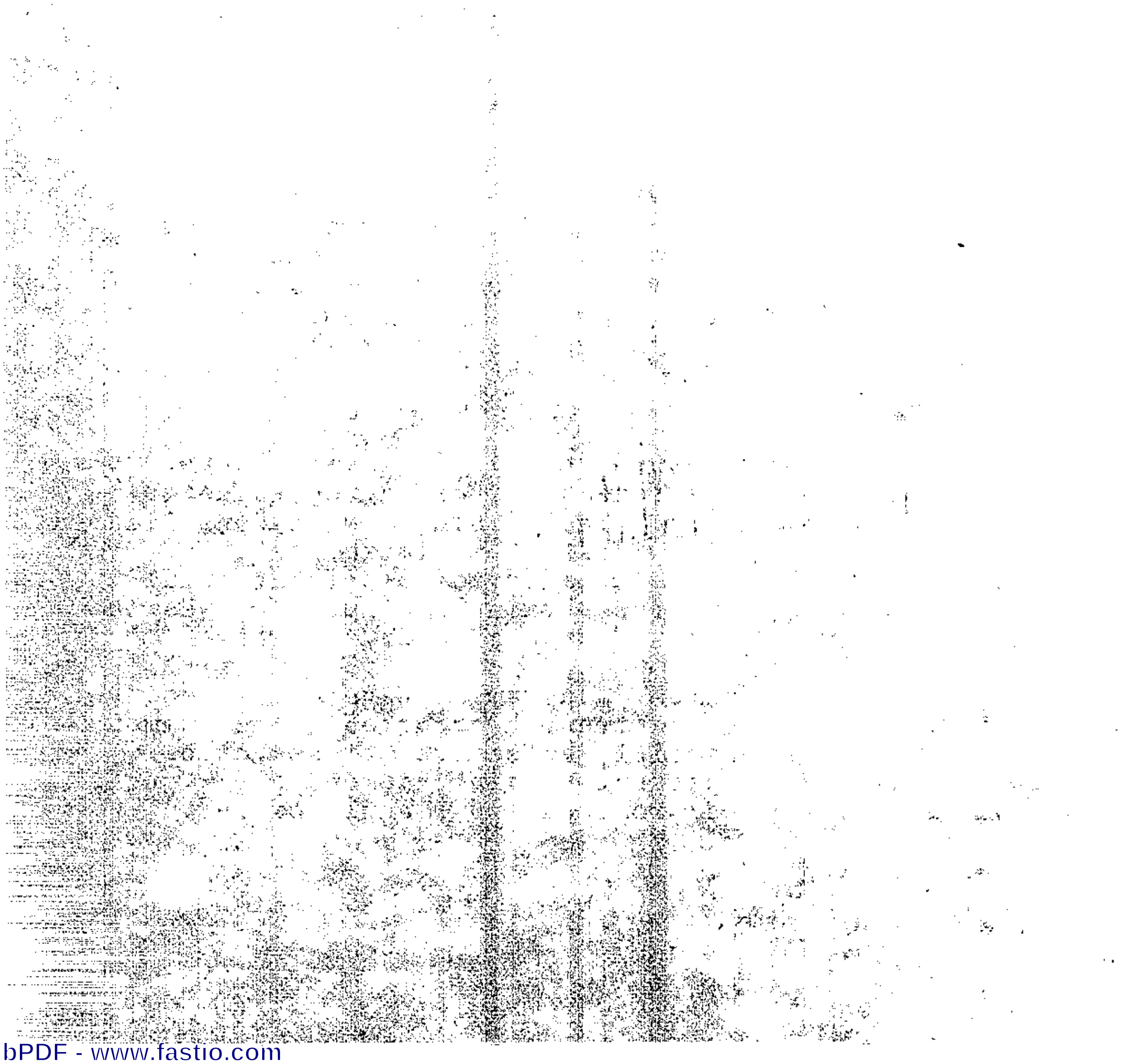


TABLE 1 - GROUNDWATER LEVELS IN SLOPE INDICATOR HOLES

DATE	WATER LEVEL DEPTH IN FEET					
	SI-1	SI-2	SI-3	SI-4	SI-5	SI-6
10-9-75	22.8	--	--	--		
10-16-75	22.0	40.0	67.5	40.0		
10-21-75	22.0	40.0	66.0	40.0		
11-3-75	21.0	39.0	33.1	39.0		
11-17-75	19.0	34.0	29.0	34.0		
12-9-75	19.1	29.1	31.1	37.1		
12-11-75	18.0	29.0	32.0	37.0		
12-17-75	--	38.0	37.0	38.0	50.0*	36.0**
12-30-75	14.0	37.0	34.0	34.0	20.0	30.6

* INITIAL READING TAKEN AT 12-19-75

** INITIAL READING TAKEN AT 12-22-75

After the above-described tests, it was concluded that further strength testing was not warranted since the remaining samples were very similar in composition and could be expected to yield similar strength values.

STABILITY ANALYSES

Failure Analysis

A stability analysis, using the Soil X Computer Program, was begun by first analyzing the slide in the failed condition. This analysis was performed (1) to determine what strength values would produce a safety factor of near unity so they could be used later in assessing possible correction schemes, and (2) to determine the most probable slide surface location over a representative section through the entire slide mass.

Using the contour map of Figure 2, several sections through the slide were plotted. Section E'-E, shown on Figure 2 and plotted as Figure 18 was selected as representative of the critical configuration. Initial failure arcs were selected based on (1) scarp locations, (2) roadway pushup, and (3) slope indicator data plus depth to apparent competent rock as determined from the four borings. Strength values of zero for unit cohesion and 15^{o} for friction angle were selected based largely on experience but with some consideration being given to one of the direct shear test series.

For the above conditions, a series of searches by the computer resulted in safety factors of 1.09 for the major scarp failure arc and 1.01 for the minor scarp failure arc. These values were judged to be sufficiently close to unity for the intended purpose of providing a basis for comparing corrective schemes. The two critical arcs are shown on Figure 18.

Correction Analyses

Three alternate treatments were input to the computer program using the arcs obtained from the failure analysis. Alternate I involved placement of a 25-foot high strut at the location indicated in Figure 18. Alternate II entailed unloading the soil mass in the minor scarp area by flattening the slope to 6:1, shown on Figure 18 as a shaded area. The third alternate combined Alternates I and II (strut plus unloading). The resulting safety factors plus those representing the failed condition are shown in Table 2.

TABLE 2 - COMPARISON OF CORRECTION ALTERNATE STABILITIES

Arc Analyzed	Description	Factor of Safety
Major Scarp	Incipient failure condition	1.09
Minor Scarp	γ=125, C=0, ϕ=15°	1.01
Major Scarp	Alternate I (25'x100' Strut)	1.19
Minor Scarp	γ=125, C=0, ϕ=15°	1.16
Major Scarp	Alternate II (Unloading)	1.13
Minor Scarp	γ=125, C=0, ϕ=15°	1.16
Major Scarp	Alternate III (Strut & Unloading)	1.36
Minor Scarp	γ=125, C=0, ϕ=15°	1.34

These data show that the safety factors with Atlernate I are 1.19 for the major scarp and 1.16 for the minor scarp. Slightly lower values (1.13 and 1.16, respectively) were obtained with Alternate II. The safety factors for Alternate III, 1.36 for the major scarp and 1.34 for the minor scarp, indicate significant improvement in stability over the other solutions.

Examination of field data showed that relatively shallower failure surfaces might be considered as another possible failure condition. To evaluate this possiblity, additional analyses were performed applying a block and wedge technique to an assumed translatory slide condition. This solution incorporates the E'-E configuration with slope indicator data. The results of this analysis were similar to those obtained by the circular arc method, with somewhat higher factors of safety for both minor and major scarps.

To check the validity of the block and wedge results, a failure arc fitted to approximate the translatory slide geometry was submitted to the computer. Results from the two methods are:

	Type of Analysis	Factor of Safety
Major Scarp	Circular Arc	1.25
	Block/Wedge	1.26
Minor Scarp	Circular Arc	1.18
	Block/Wedge	1.18

It would appear, therefore, that the deeper failure surfaces are indeed the more critical.

Based on stability analysis results, the timing of initial movement, and the amount of movement, there is little doubt that the direct cause of the slide was the construction of the cut which removed critical resisting forces from the lower portion of the slope. The relatively small amount of movement along the flanks and head, especially during the first five months subsequent to detection, indicates the mass was in a fairly stable state (with the possible exception of a slow creep-type movement) prior to removal of the roadway cut material. While ground water was present within the slide mass, it was not considered to be of sufficient quantity or pressure to have created a metastable condition prior to the cutting. However, to fully and permanently arrest movement, it may be necessary to provide means for removal of water from a sufficient depth to result in increased shearing resistance along the surface of movement.

TEMPORARY CORRECTION

Of the alternates analyzed, only those two involving strutting (or reloading the cut area) would be acceptable based on the results obtained from stability analyses. However, the strutting alternates were not completely acceptable because of the effect on the realigned roadway grade. This objection to strutting as a permanent correction, plus other factors, led to a recommendation for a temporary correction primarily to protect the roadway through the winter and early spring wet weather.

The temporary correction consisted of unloading an upper portion of the slide, building a 10-foot strut over the realigned roadway, and providing more positive surface drainage in the upper reaches of the slide. These recommendations were detailed in a memorandum dated December 4, 1975, a copy of which is included as an appendix to this report.

Work on the temporary correction began on December 8 and was essentially complete by the end of the same week. As constructed, the strut is about 350 feet long and of variable thickness, tapering near the middle to match existing roadway at each end. Maximum thickness is about 16 feet.

During the temporary correction operations, two additional slope indicators, SI-5 and -6, were installed in the upper portion of the slide. These holes, the locations of which are shown on Figure 2, were bottomed at depths of 52 feet. The temporary correction will allow time for consideration of other schemes for permanent correction, and will permit an evaluation of the effectiveness of the temporary scheme in slowing the slide movement.

PERMANENT CORRECTION

Five alternate schemes for permanently stabilizing the slide have been considered based on information obtained to date. These schemes are as follows:

1. Partial reloading of slide toe area.
2. Chemical stabilization.
3. Subsurface drainage.
4. Keying slide to firm material.
5. Combination of (1) and (3).

While these treatments may be physically effective, all have advantages, disadvantages, and uncertainties which must be evaluated qualitatively in relation to each other. Also, any scheme utilized will benefit from the partial unloading of the upper slide mass, executed as part of the temporary treatment during the week of December 8.

Scheme 1 - Partial Reloading

This scheme is essentially the same as that already utilized for temporary correction and consists of a buttress about 350 feet long between (approximately) Stations 1249+50 and 1253. A distinct advantage of the scheme is its cost since the temporary buttressing is already in place. The only disadvantage is that such a buttress raises the roadway profile over a relatively short distance and hence increases grades to undesirable values in the order of 10%. Another factor that must yet be evaluated is the effectiveness of the 10-foot thickness of the temporary buttress in arresting the slide movement. Future field data over the next few months may indicate that for permanent correction a 15-foot thickness, for example, may be required.

This alternate, because of the positive contribution of the in-place partial reloading, must be considered as a viable scheme for permanent correction. However, a reliable evaluation of its probable effectiveness must await additional field information to be obtained over the next several weeks.

Scheme 2 - Chemical Stabilization

This scheme involves the pumping of special chemicals into the subsurface soils of the sliding mass. The objective is to chemically alter the subsurface materials to increase their strength values and enhance their stress-deformation characteristics. The chemicals and methodology are proprietary and the potential success with the particular soils in question is unknown at the present time. However, Ion Tech, the private firm which has specialized in developing and implementing this technique, has been supplied with soil samples for evaluation. Preliminary indications are that chemical treatment does increase the soil strength to some extent. The strength of one sample, for example, was increased by 50 percent.

This method of stabilizing slides has been used in California on Interstate 5 north of Redding and on Route 280 in the San Francisco Bay Area. The Route 5 project was successful whereas the Route 280 application was unsuccessful. However, neither case was clear-cut with regard to the role of the chemical treatment. Horizontal drains were installed on Route 5 as part of the stabilizing treatment and their contribution relative to the chemicals is unknown. The weather conditions (excessive rain within six months of chemical application) immediately after treating the Route 280 slide were known to be detrimental but to an unknown degree. Therefore, the experience of the Department of Transportation in chemically stabilizing slides is inconclusive.

Assuming the technique to prove feasible, the scheme as presently envisioned would include the removal of the temporary buttress.

Scheme 3 - Subsurface Drainage

The purpose of this scheme would be to remove subsurface water from sufficient depths within the slide mass to permit development of additional sliding resistance. Although the presence of subsurface water was not considered to be the direct cause of movement, the removal of water would in all likelihood result in increased resistance to movement, especially in the future.

Specifically, the scheme calls for installing horizontal drains from two headings, A and B, as shown on Figure 19. These heading locations, along the old road alignment at approximate elevations 2445 and 2525, respectively, will permit fan coverage of the lower one-third of the slide mass. Existing horizontal drains, although functioning, are not deep enough to remove water from the zone or surface of slide movement. Nor do they cover an area sufficiently large to materially improve the sliding resistance of the earth mass.

The proposed scheme includes eleven drains from Heading A and five drains from Heading B, fanned as shown in Figure 19. The length of drains from A will vary from 280 feet to 450 feet whereas all drains from B will be 500 feet in length. The depth of slide movement as shown by slope indicators, suggests that the drains should terminate at depths of 50 to 60 feet which results in drain slopes of about 15 percent.

A more positive means of removing subsurface water consists of first collecting the water in a drainage gallery and then draining it away through horizontal drains. Such drainage galleries are typically constructed by drilling several large diameter (24-36") holes in line spaced at twice the hole diameter. The bottoms are belled to interconnect all borings, and the holes backfilled with pea gravel. The holes must be bottomed at or below the elevation from which water is to be removed, in the present situation about 55 to 60 feet maximum below ground surface. Potential layouts of two galleries of wells are shown on Figure 19.

The terrain and subsurface soil conditions at the subject site would probably prove difficult and expensive for such an installation, especially in view of the depth required.

As presented, a subsurface drainage scheme would include removing the temporary buttress and restoring the planned roadway grade.

Scheme 4 - Keying Slide

The concept of this scheme is to key the sliding mass to firm material below the zone of movement, to provide positive resistance to long-term movement. The only practical means to achieve the keying action is the Fondedile Pile System, a technique relatively new in this country. Essentially, the technique consists of boring (usually by auger) several small diameter holes to the depth desired and backfilling with concrete. A large rebar is placed in the hole prior to filling with concrete under pressure which tends to expand the boring walls. In principle, the sliding mass is thus "stitched" to firm underlying material.

A disadvantage of this scheme is the large slide area which would require numerous piles to be installed. Another possible disadvantage is the difficulty that may be realized in boring the holes into the more competent material immediately below the slide plane. Hence, the establishing of the necessary key may prove too difficult from an economic viewpoint.

Scheme 5 - Combination of Schemes 1 and 3

A corrective treatment combining buttressing (reloading in this case) and subsurface drainage is often used for difficult stability problems. Combining treatments usually results in less stringent requirements for each of the separate corrective measures. For example, one-fourth to one-third of the horizontal drains of Scheme 3 probably could be eliminated. The minimum amount of buttressing or reloading, however, cannot be determined until additional field data are evaluated.

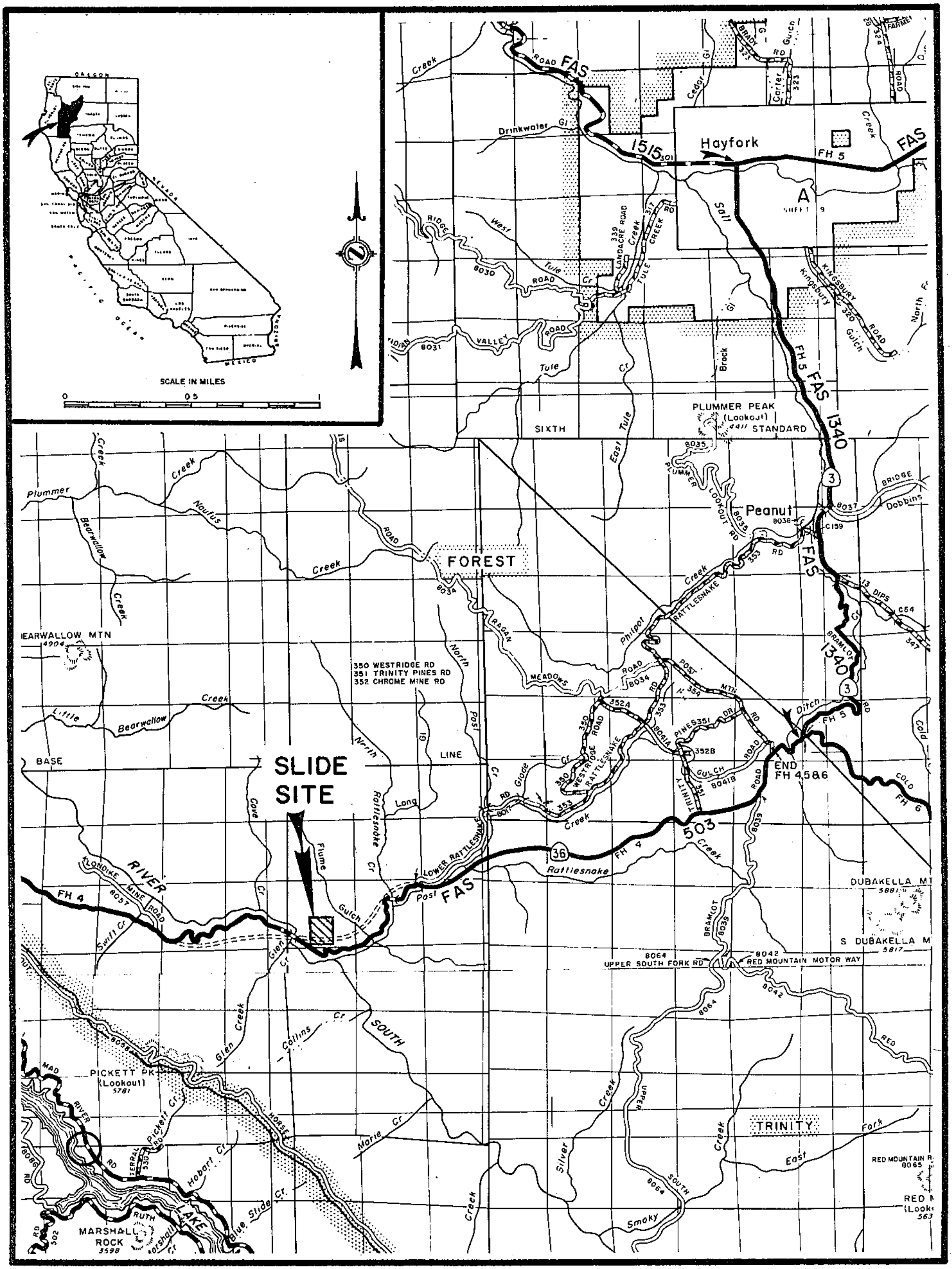
The advantages are as already discussed separately under (1) and (3). Another favorable factor, especially in view of the positive effects that would be realized, in the relatively low cost. Again, the only disadvantage is the effect on the roadway profile grade.

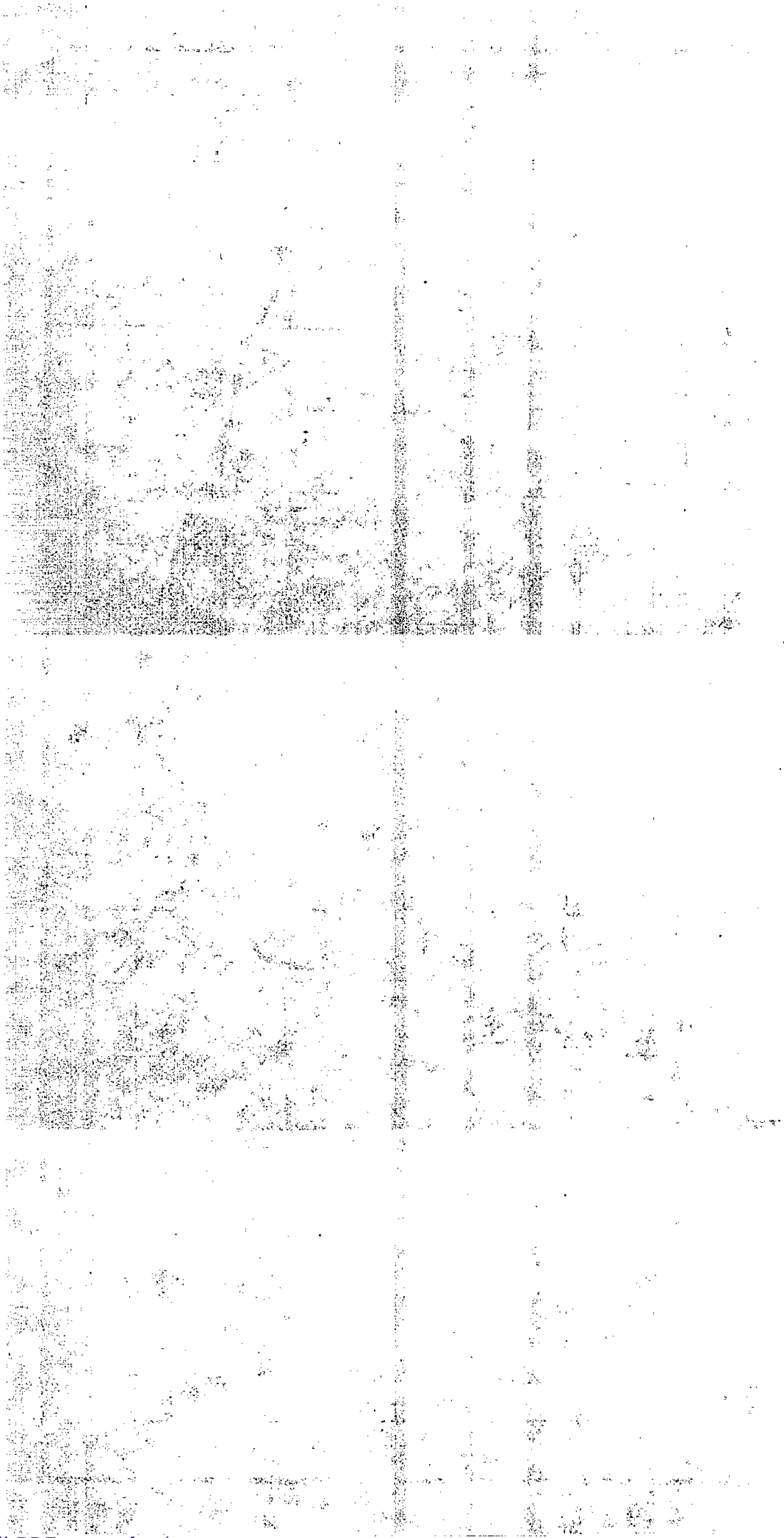
RECOMMENDATIONS

Because of the importance given to maintaining the planned profile grade, it is recommended that the selection of a permanent correction scheme be deferred to a later date, possibly in March of this year. This time delay would allow further evaluation of the temporary correction effectiveness in arresting movement which is of vital importance if a permanent scheme involving no reloading is to be selected. Also, groundwater fluctuations would be monitored.

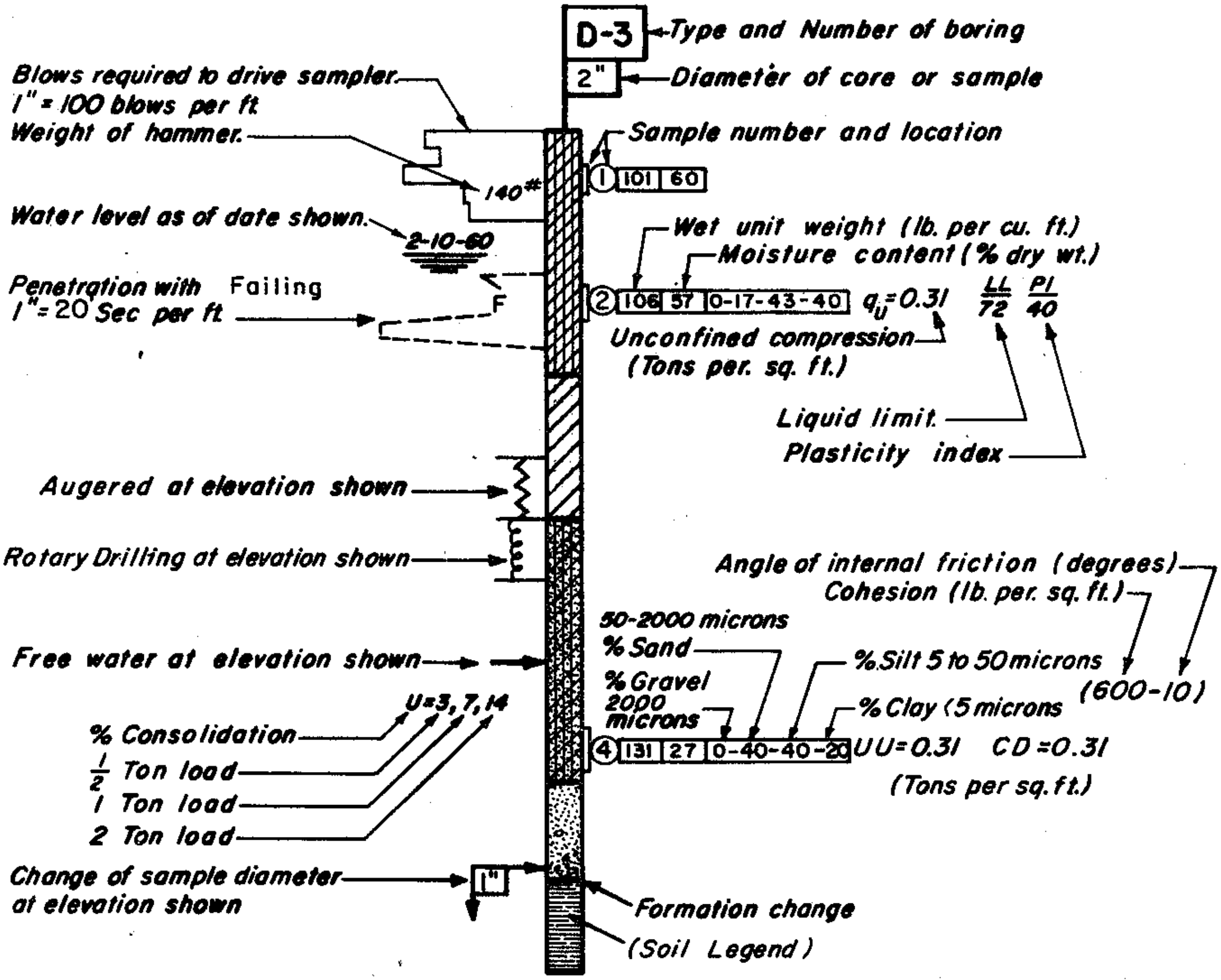
As a precautionary measure, it is recommended that design consideration be given to minimizing the problems that would be generated by raising the roadway profile, in the event partial reloading (10 to 15 feet) is deemed necessary.

Figure 1

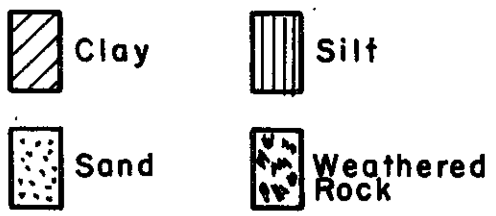

SLIDE
SITE
Hayfork
FH 5
FAS
1515
A
SHEET 9
Drinkwater Gl
Cedar Gl
Carter Gulch
BRADY RD
Creek
Salt
Kingsbury Gulch
KINGSBURY ROAD
320
North Fork
West
Tule Cr
TULE CREEK
LANDACRE ROAD
339
317
RIDGE
ROAD
8030
8031
VALLEY ROAD
Tule
East Tule
Cr
Brock Gl
FH 5
FAS
1340
PLUMMER PEAK
(Lookout)
STANDARD
SIXTH
8035
PLUMMER LOOKOUT RD
Peanut
8038
8037
BRIDGE
Dobbins
C159
13 DIPS
C54
347
BRAMLOT
3
Plummer
Creek
Naufus
Bearwallow
Creek
FOREST
ROAD
8034
RAGAN
MEADOWS
Philpot
Creek
RATTLESNAKE
353
RD
POST
MTN
EARWALLOW MTN
4904
350 WESTRIDGE RD
351 TRINITY PINES RD
352 CHROME MINE RD
North
Post
Gl
Little
Bearwallow
Creek
BASE
LINE
North
Rattlesnake
Cr
Long
Cove
Cr
Glade
Cr
352A
350
WESTRIDGE ROAD
RATTLESNAKE
8041A
PINES
351
DR
352B
GULCH
8041B
TRINITY
ROAD
Ditch
FH 5
END
FH 4,5&6
COLD
FH 6
8011
503
FH 4
36
Creek
Rattlesnake
8039
LOWER RATTLESNAKE
Flume
Gulch
Post
FAS
KLONDIKE
MINE
ROAD
8057
RIVER
FH 4
Swift Cr
Glen
Cr
DUBAKELLA MT
5881
S DUBAKELLA MT
5817
BRAMLOT
8039
8064
UPPER SOUTH FORK RD
8042
RED MOUNTAIN MOTOR WAY
8042
8064
Glen Creek
Collins Cr
SOUTH
8098
RED
PICKETT PK
(Lookout)
5781
MAD
RIVER
Pickett Cr
TERRAL
330
RD
Hobart Cr
HORSE
Marie Cr
Silver
Creek
UPPER
Creek
TRINITY
Fork
East
RED MOUNTAIN R.
8065
RED
SOUTH
8064
Smoky
Creek
RUTH
LAKE
Blue Slide Cr.
MARSHALL
ROCK
3598
Marshall Cr
502
8080
SCALE IN MILES
0
0.5
1
OREGON
NEVADA
PACIFIC OCEAN
MEXICO
ARIZONA



BORING LEGEND
CROSS-SECTION & PROFILE SHEETS



SOIL LEGEND



STRENGTH TESTS

q_u - Unconfined Compression

UU - Unconsolidated Undrained

CU - Consolidated Undrained

CD - Consolidated Drained

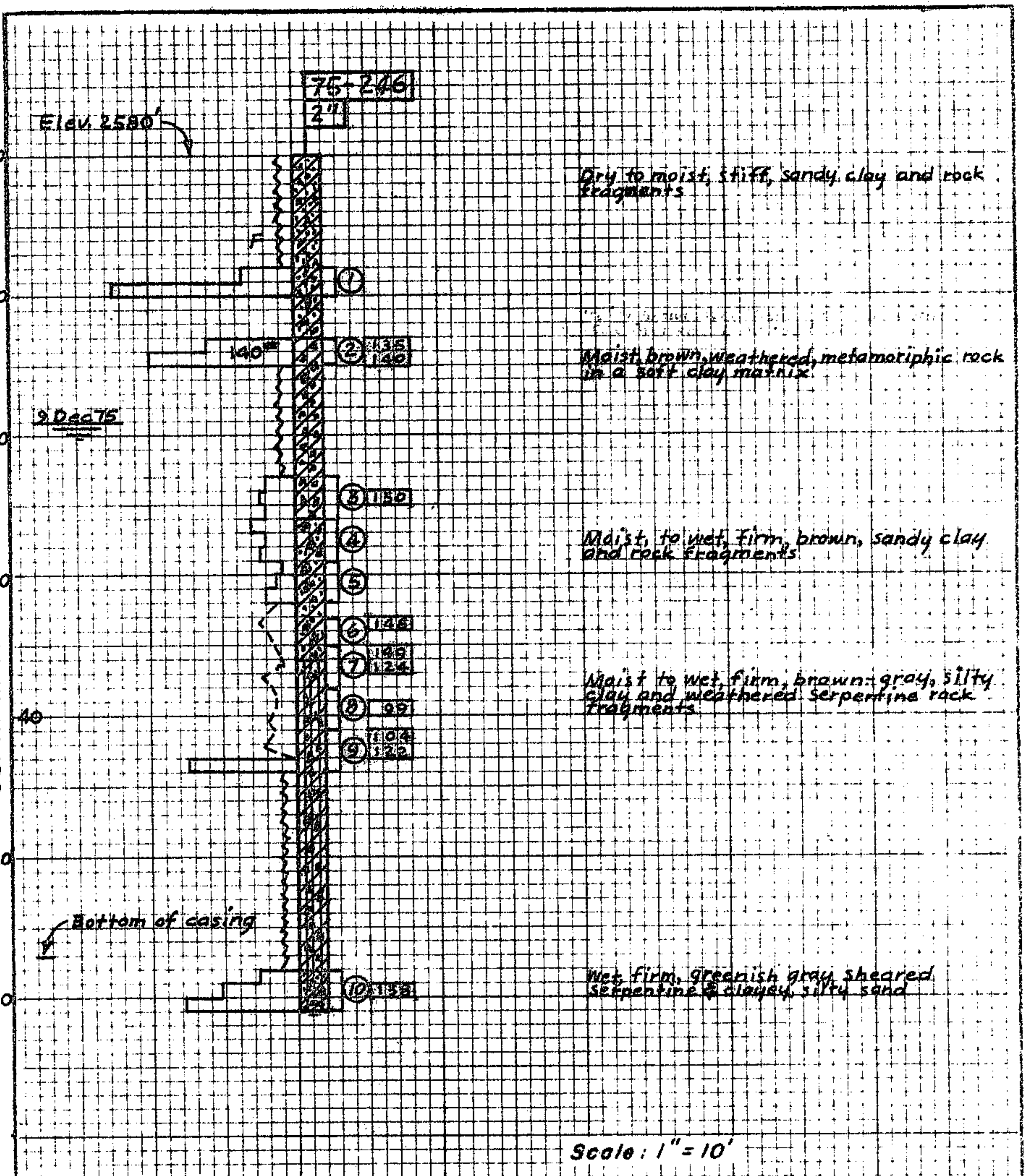
STATE OF CALIFORNIA — HIGHWAY TRANSPORTATION AGENCY
DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS
TRANSPORTATION LABORATORY

LANDSLIDE INVESTIGATION

NEAR FOREST GLEN

DATE	SUBMITTED BY:	DWG. NO.
28Jan76	[signature] SR. MAT'LS. & RES. ENG'R.	2798

DR. BY	CK. BY	APPROVED BY:	SHEET NO
MIT	AL	[signature] SUP'R. MAT'LS. & RES. ENG'R. - FDN	3 OF 19 SHTS.

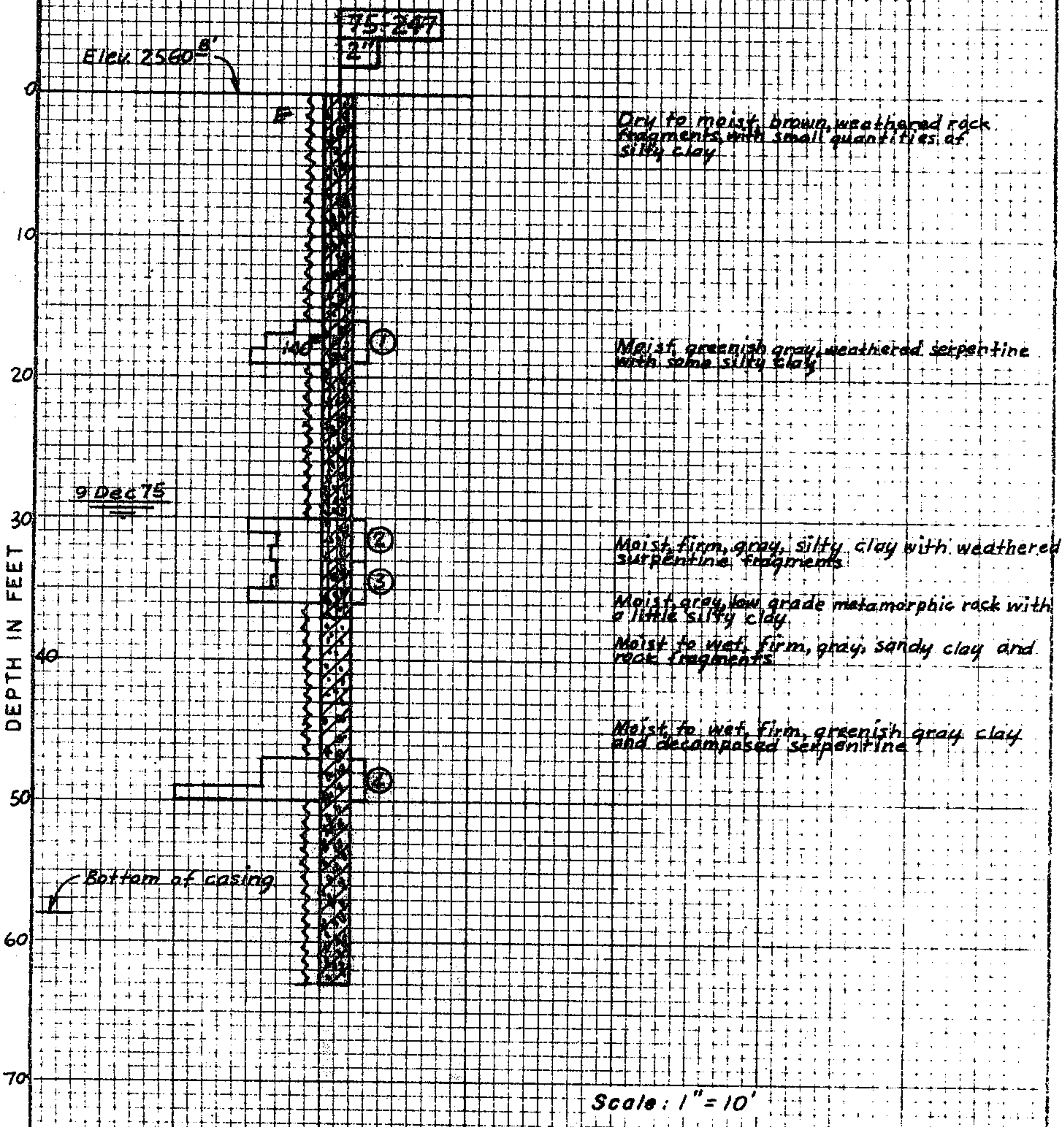


TRANSPORTATION LABORATORY
LANDSLIDE INVESTIGATION

NEAR FOREST GLEN

Profile of Boring 75-246

DATE	DIST.	COUNTY	ROUTE	POST MILE	DWG. NO.	TOTAL SHEETS
1-28-76	01	Tri	36		2798	4 of 19

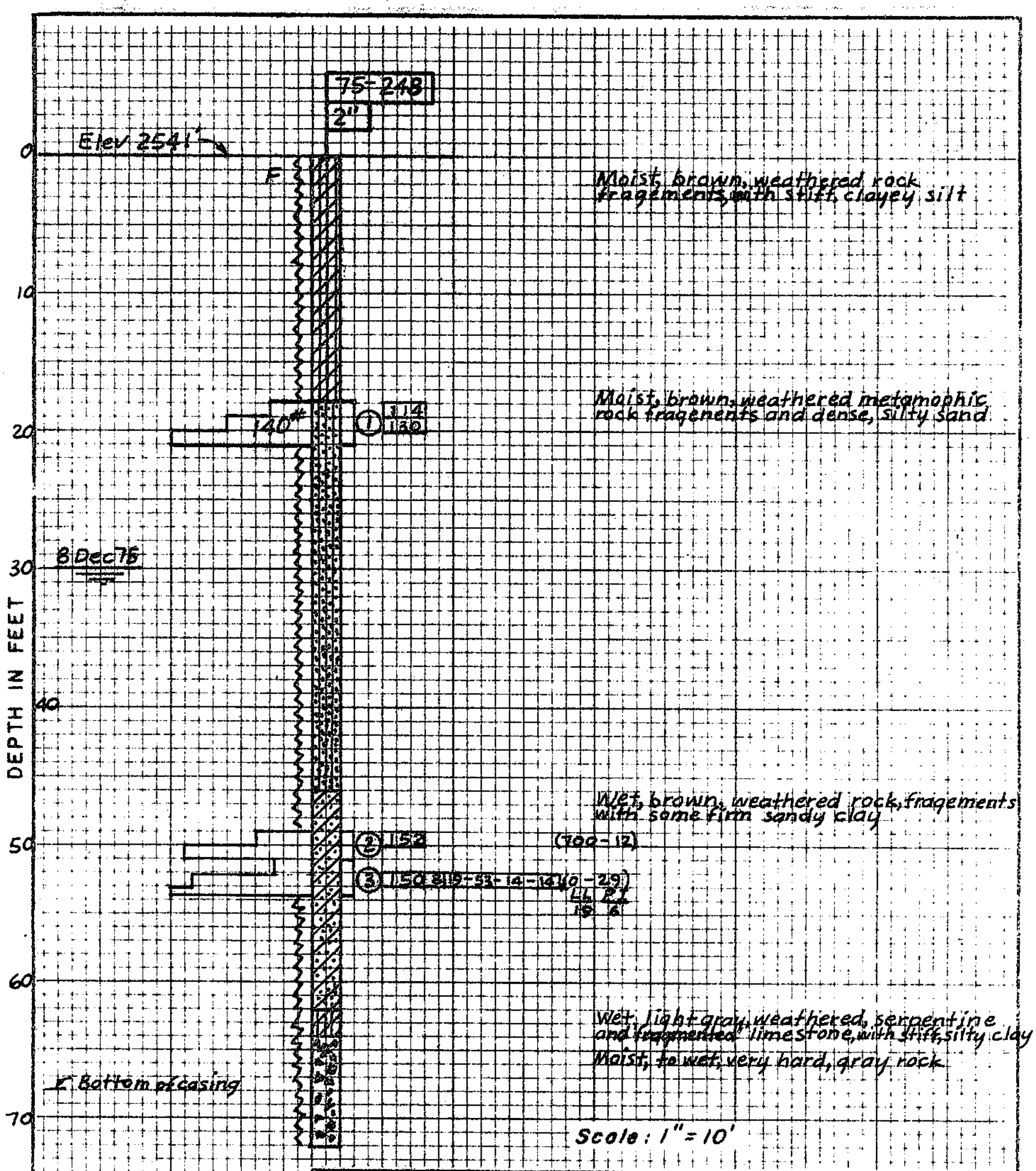


TRANSPORTATION LABORATORY
LANDSLIDE INVESTIGATION

NEAR FOREST GLEN

Profile of Boring 75-247

DATE	DIST.	COUNTY	ROUTE	POST MILE	DWG. NO.	TOTAL SHEETS
1-28-76	01	Tri	36		2798	5 of 19

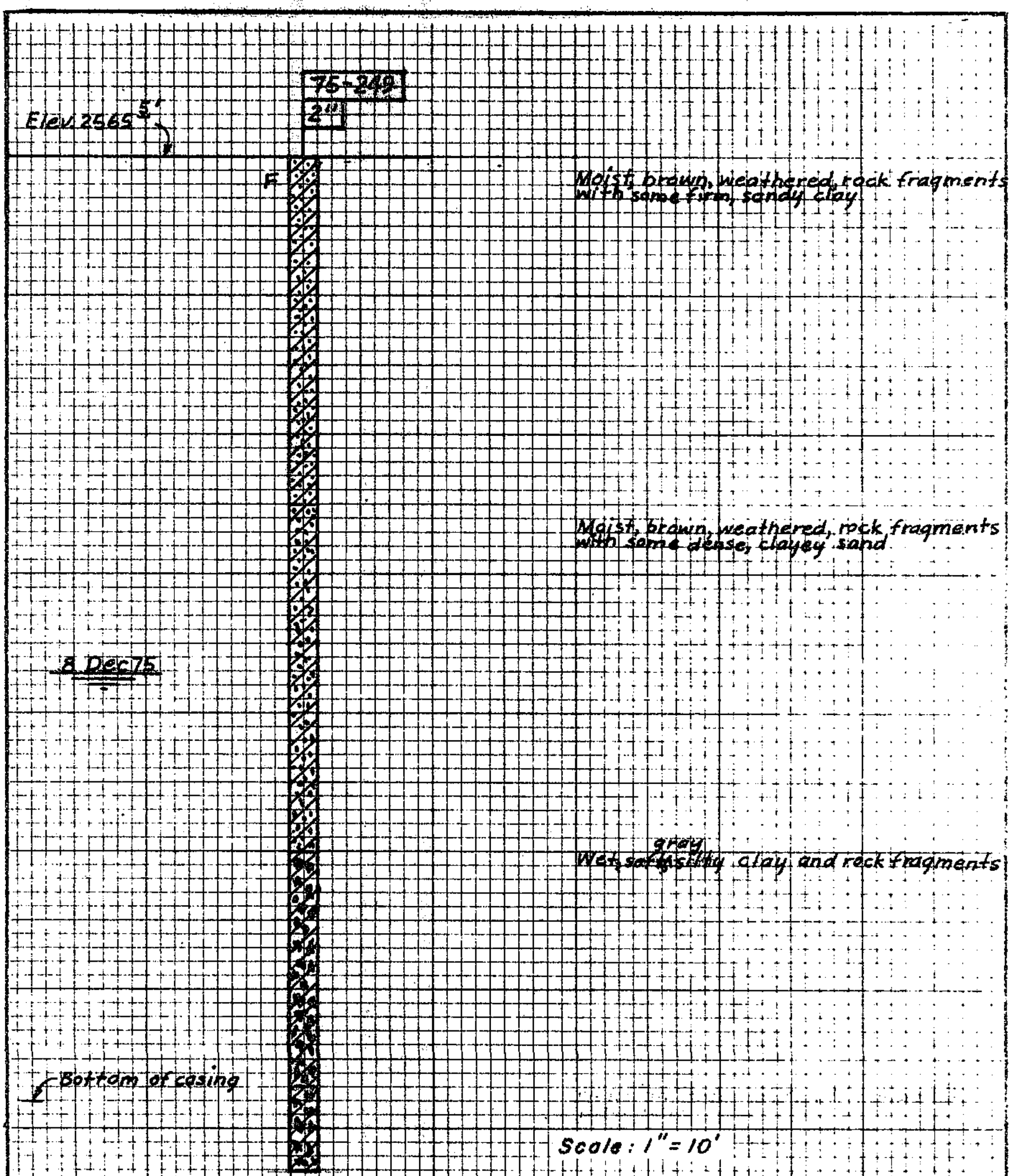


TRANSPORTATION LABORATORY
LANDSLIDE INVESTIGATION

NEAR FOREST GLEN

Profile of Boring 75-248

DATE	DIST.	COUNTY	ROUTE	POST MILE	DWG. NO.	TOTAL SHEETS
1-28-76	01	Tri	36		2798	6 of 19



TRANSPORTATION LABORATORY
LANDSLIDE INVESTIGATION

NEAR FOREST GLEN

Profile of Boring 75-249

DATE	DIST.	COUNTY	ROUTE	POST MILE	DWG. NO.	TOTAL SHEETS
1-28-76	01	Tri	36		2798	7 of 19

Figure 8

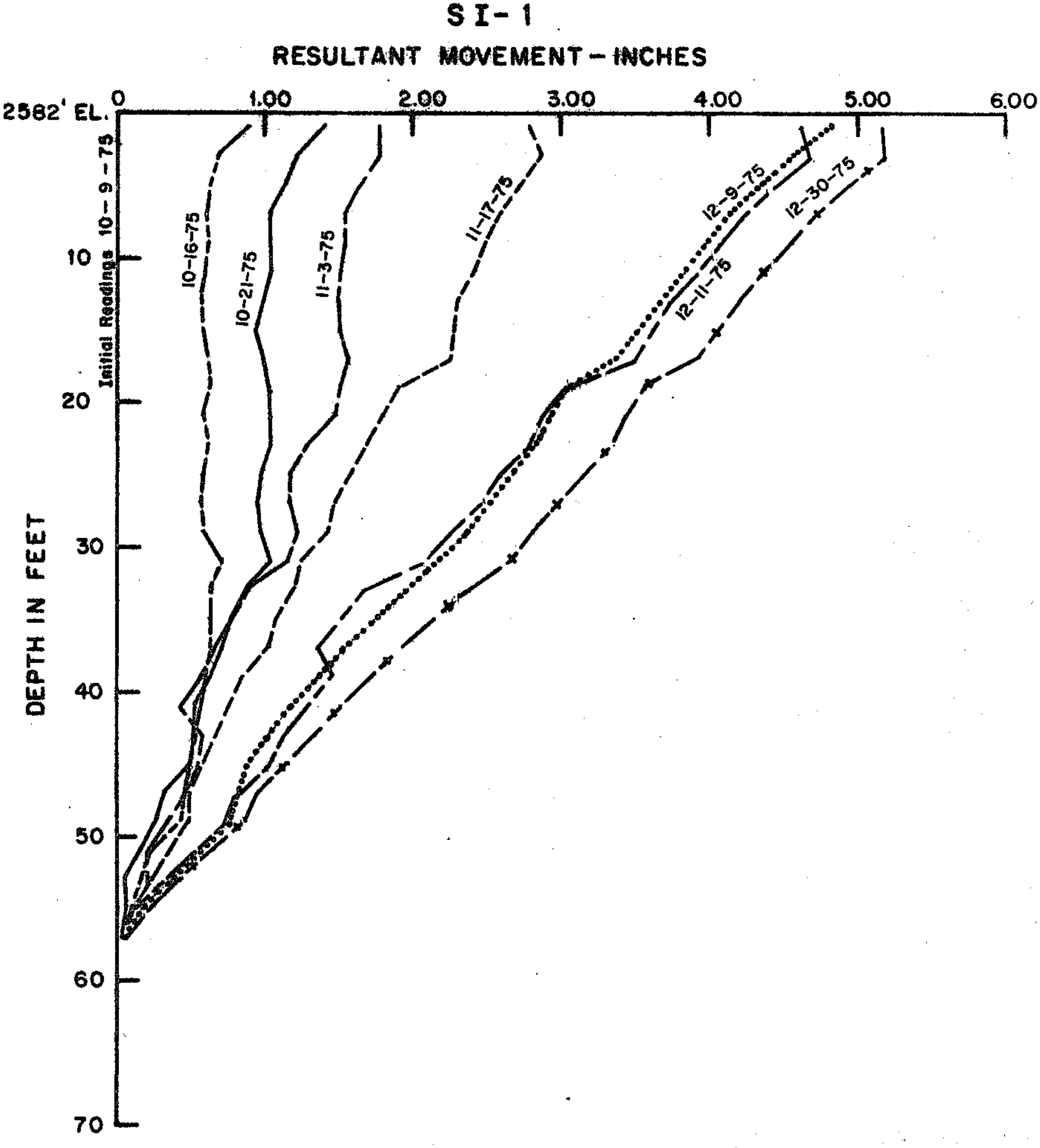

S I - 1
RESULTANT MOVEMENT - INCHES
0
1.00
2.00
3.00
4.00
5.00
6.00
2582' EL.
Initial Readings 10 - 9 - 75
10-16-75
10-21-75
11-3-75
11-17-75
12-9-75
12-11-75
12-30-75
DEPTH IN FEET
10
20
30
40
50
60
70

Figure 9

S I - 2

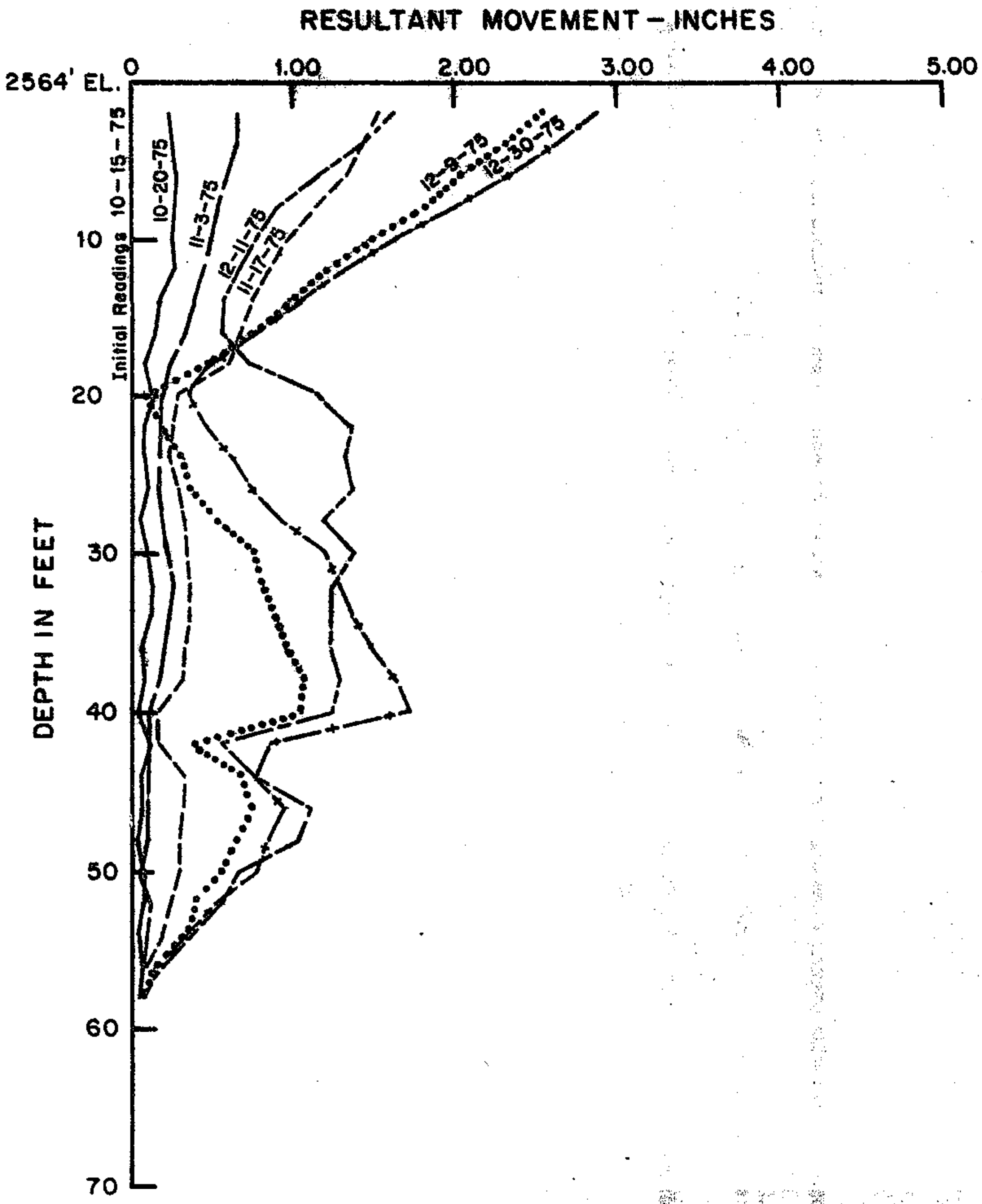

RESULTANT MOVEMENT - INCHES
0
1.00
2.00
3.00
4.00
5.00
2564' EL.
DEPTH IN FEET
10
20
30
40
50
60
70
Initial Readings 10-15-75
10-20-75
11-3-75
12-11-75
11-17-75
12-9-75
12-30-75

Figure 10

S I-3

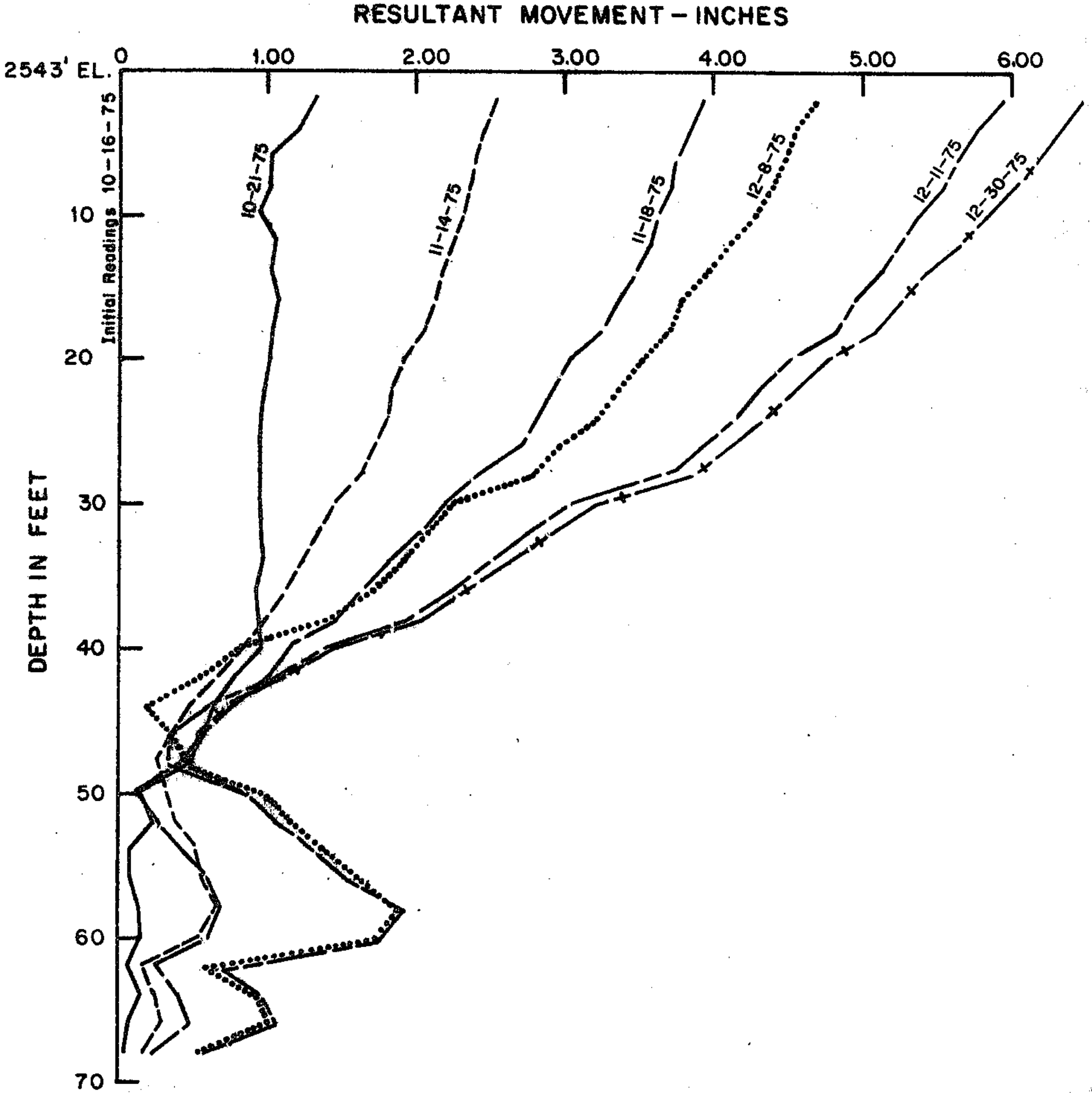
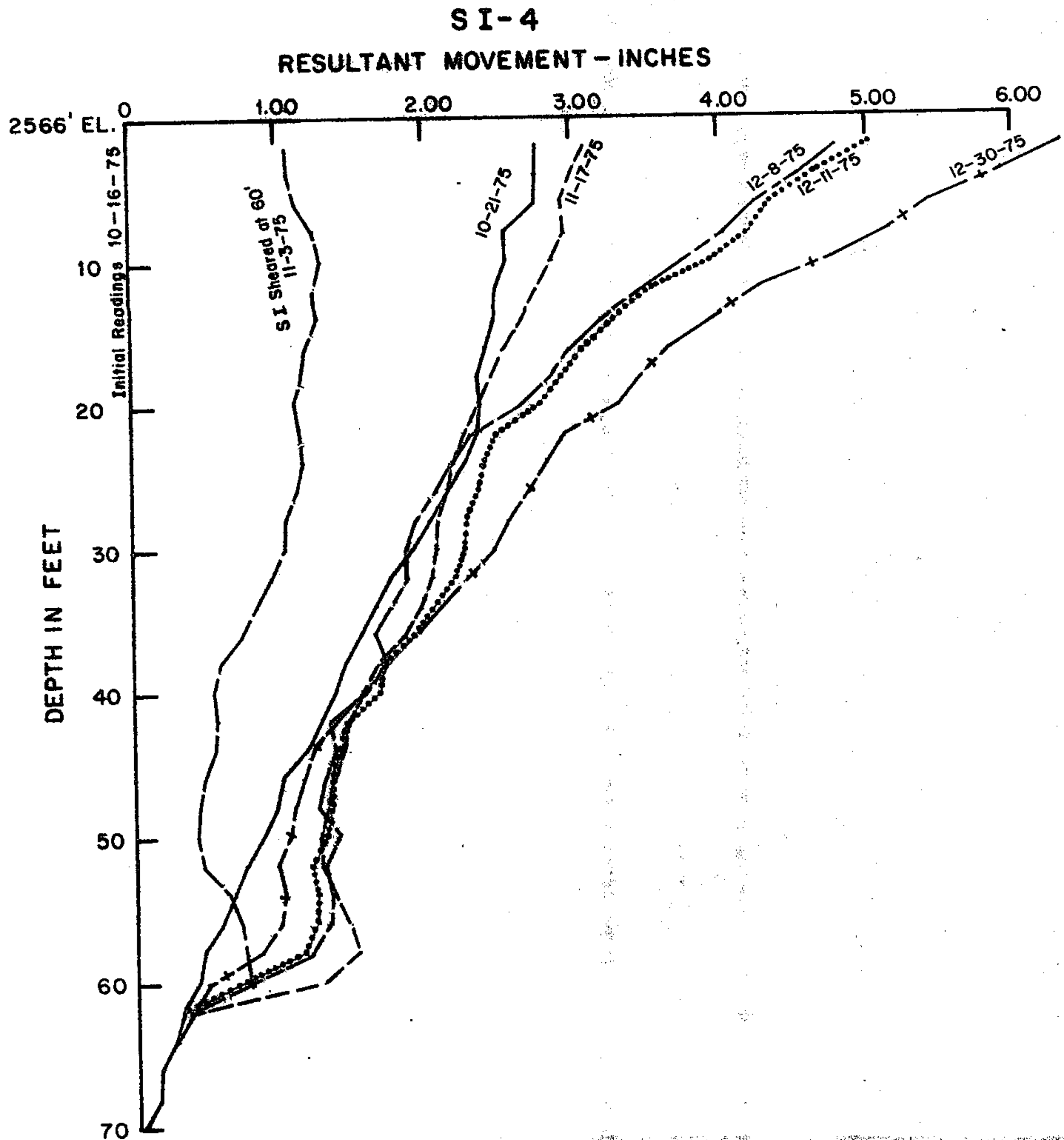


Figure II


S I - 4
RESULTANT MOVEMENT - INCHES
0
1.00
2.00
3.00
4.00
5.00
6.00
2566' EL.
Initial Readings 10-16-75
S I Sheared at 60'
11-3-75
10-21-75
11-17-75
12-8-75
12-11-75
12-30-75
10
20
30
40
50
60
70
DEPTH IN FEET

SI - 1

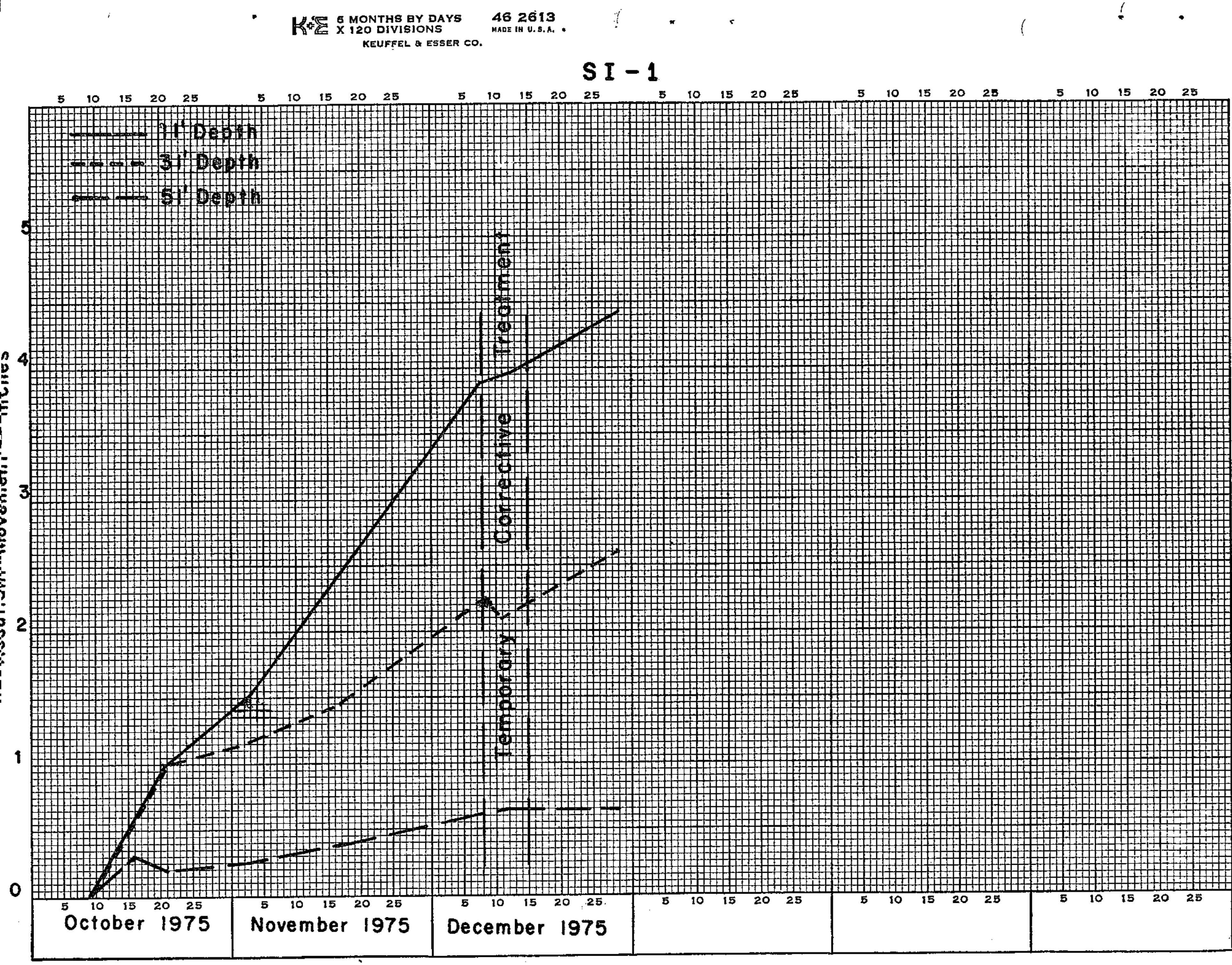


Figure 12

K+E 6 MONTHS BY DAYS X 120 DIVISIONS 46 2613 MADE IN U.S.A. KEUFFEL & ESSER CO.

SI - 2

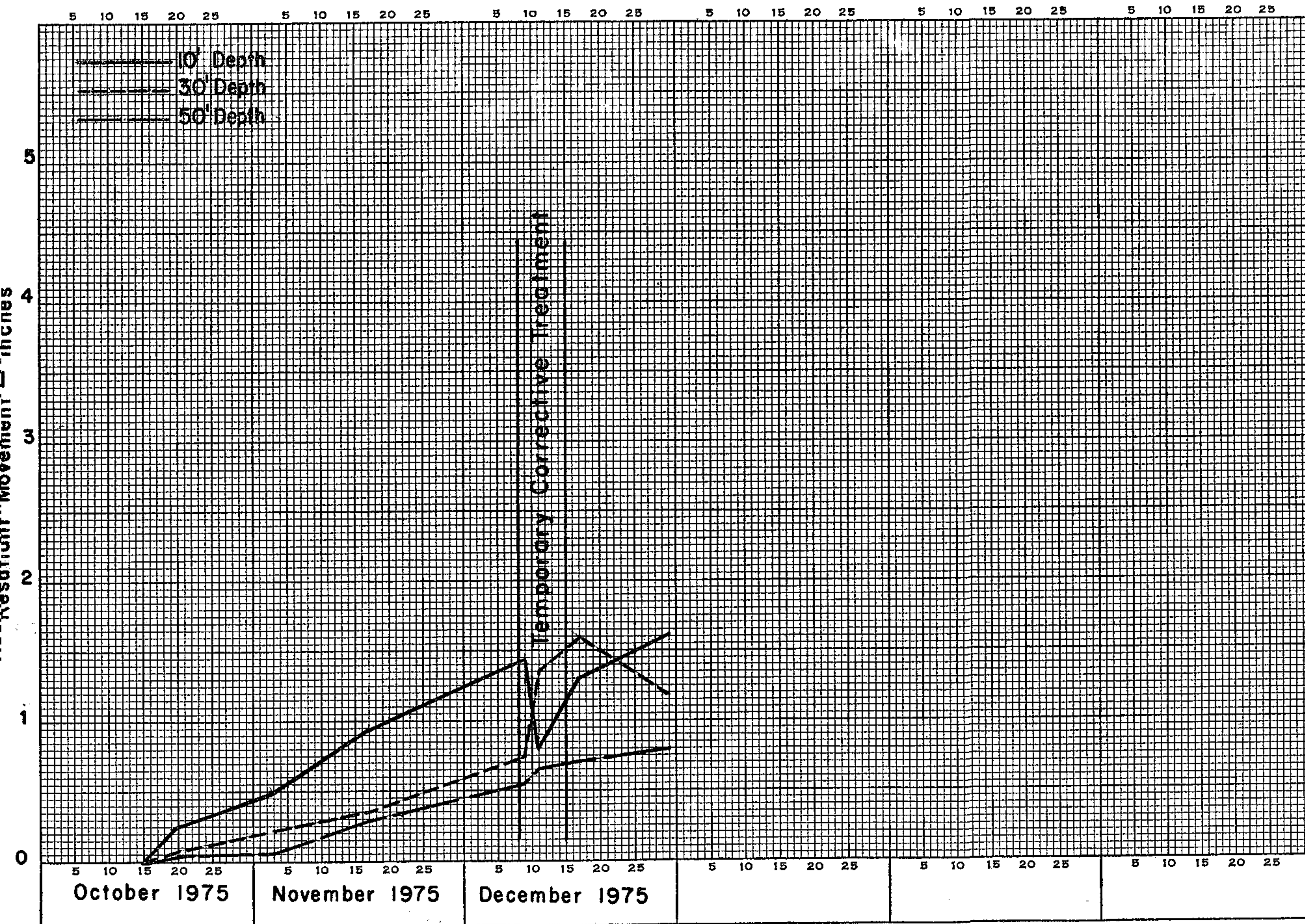


Figure 13

K+E 6 MONTHS BY DAYS X 120 DIVISIONS 46 2613 MADE IN U.S.A.
KEUFFEL & ESSER CO.

SI - 3

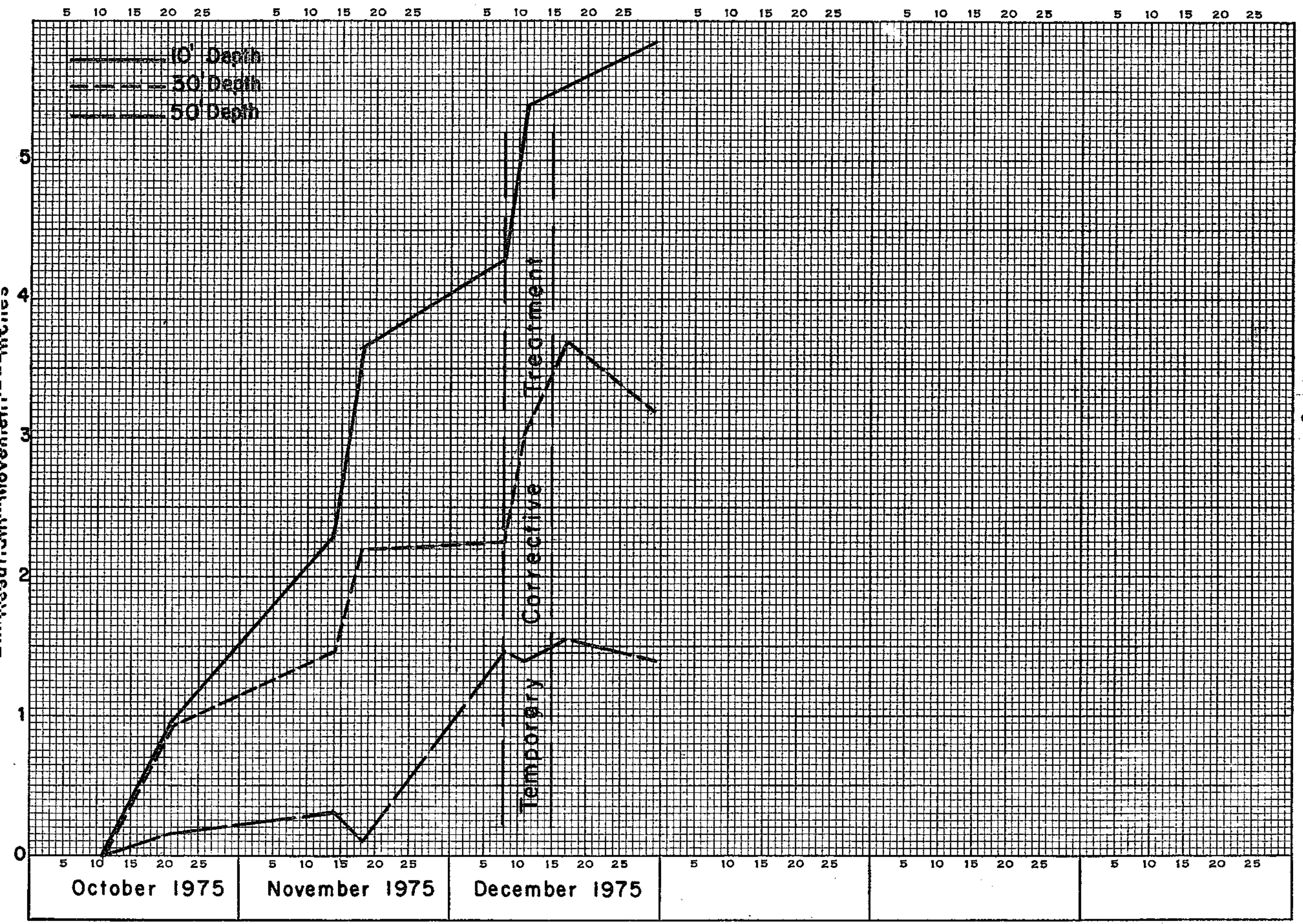


Figure 14

SI - 4

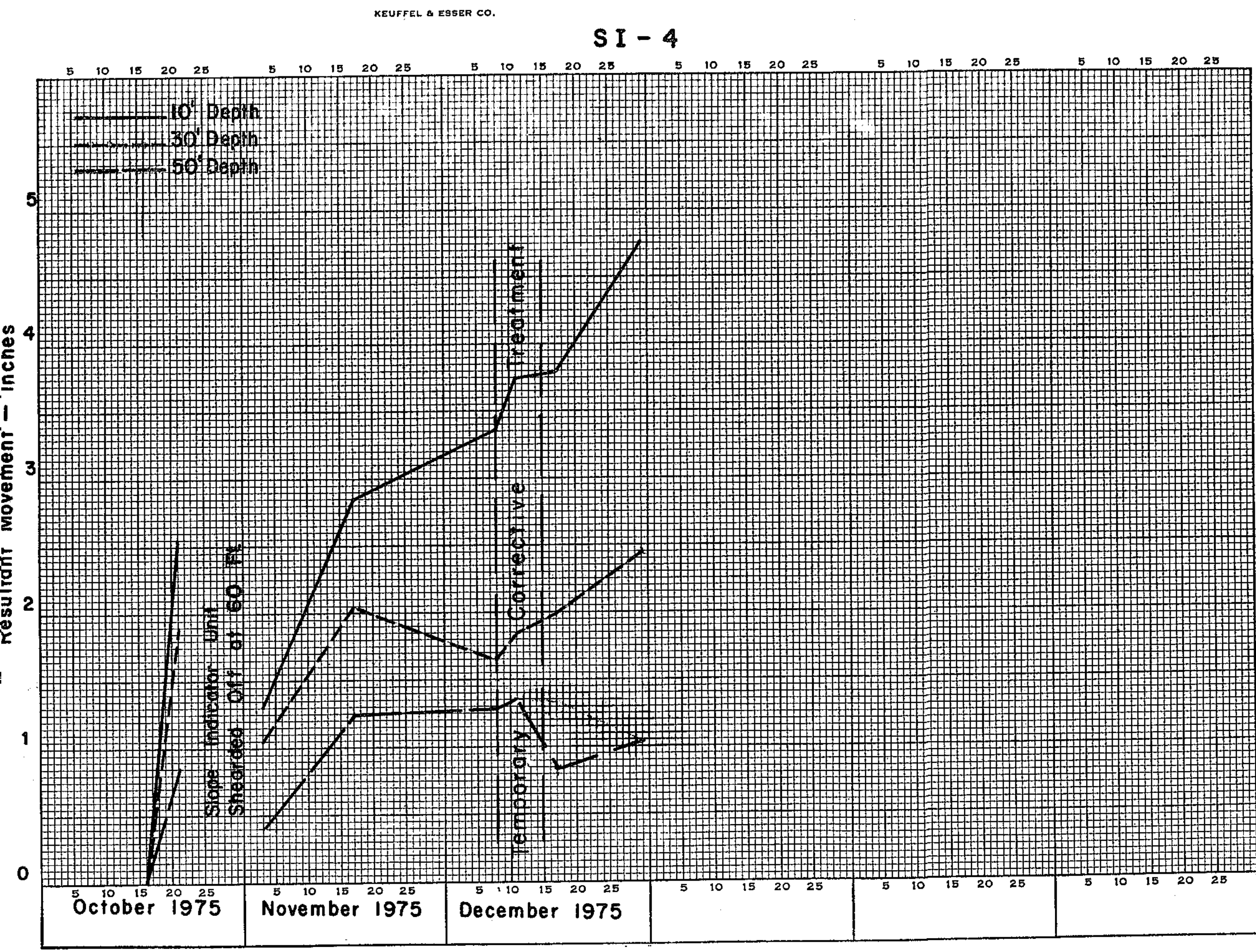


Figure 15

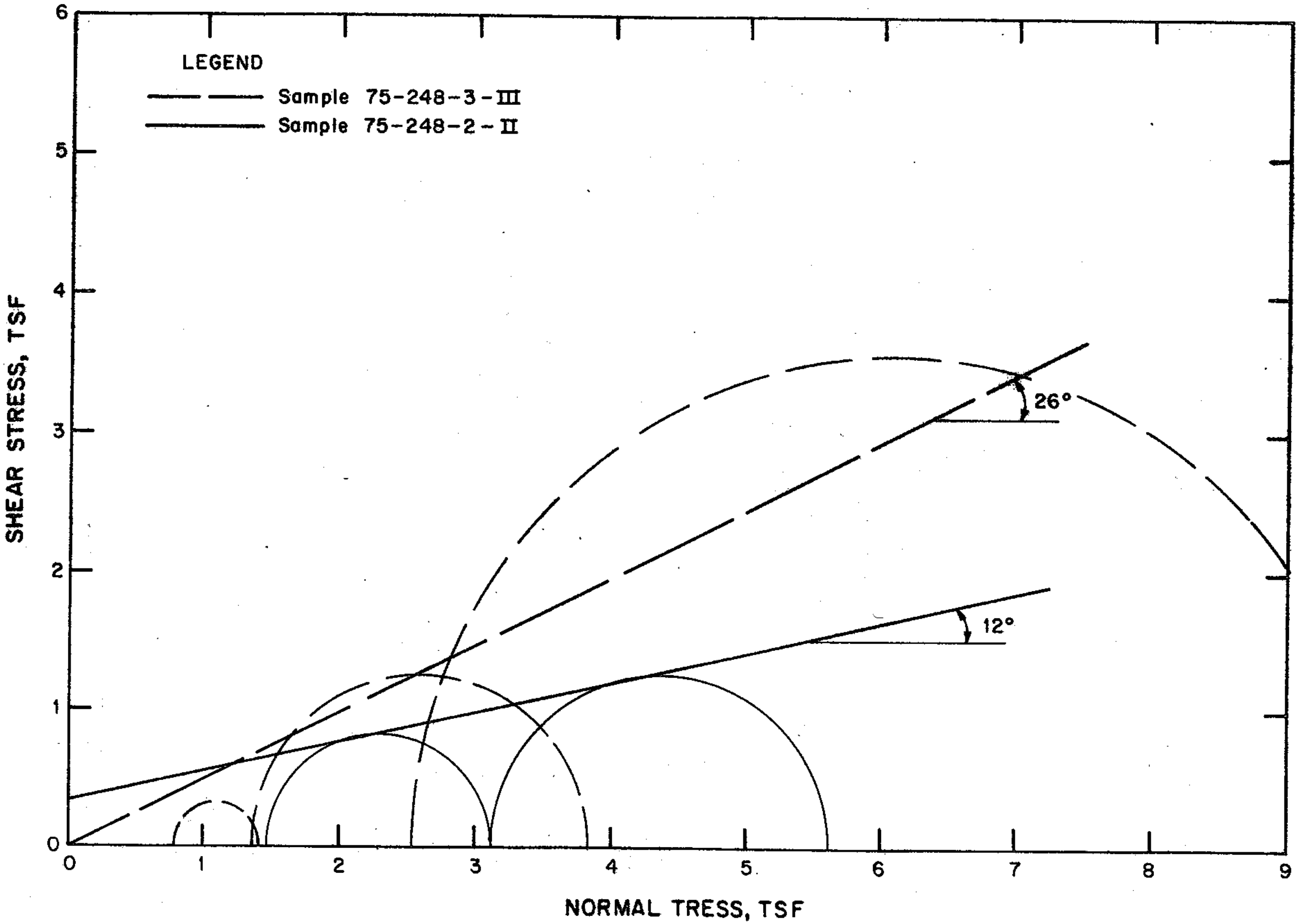

LEGEND
Sample 75-248-3-III
Sample 75-248-2-II
SHEAR STRESS, TSF
NORMAL TRESS, TSF
26°
12°
0
1
2
3
4
5
6
7
8
9

Figure 16

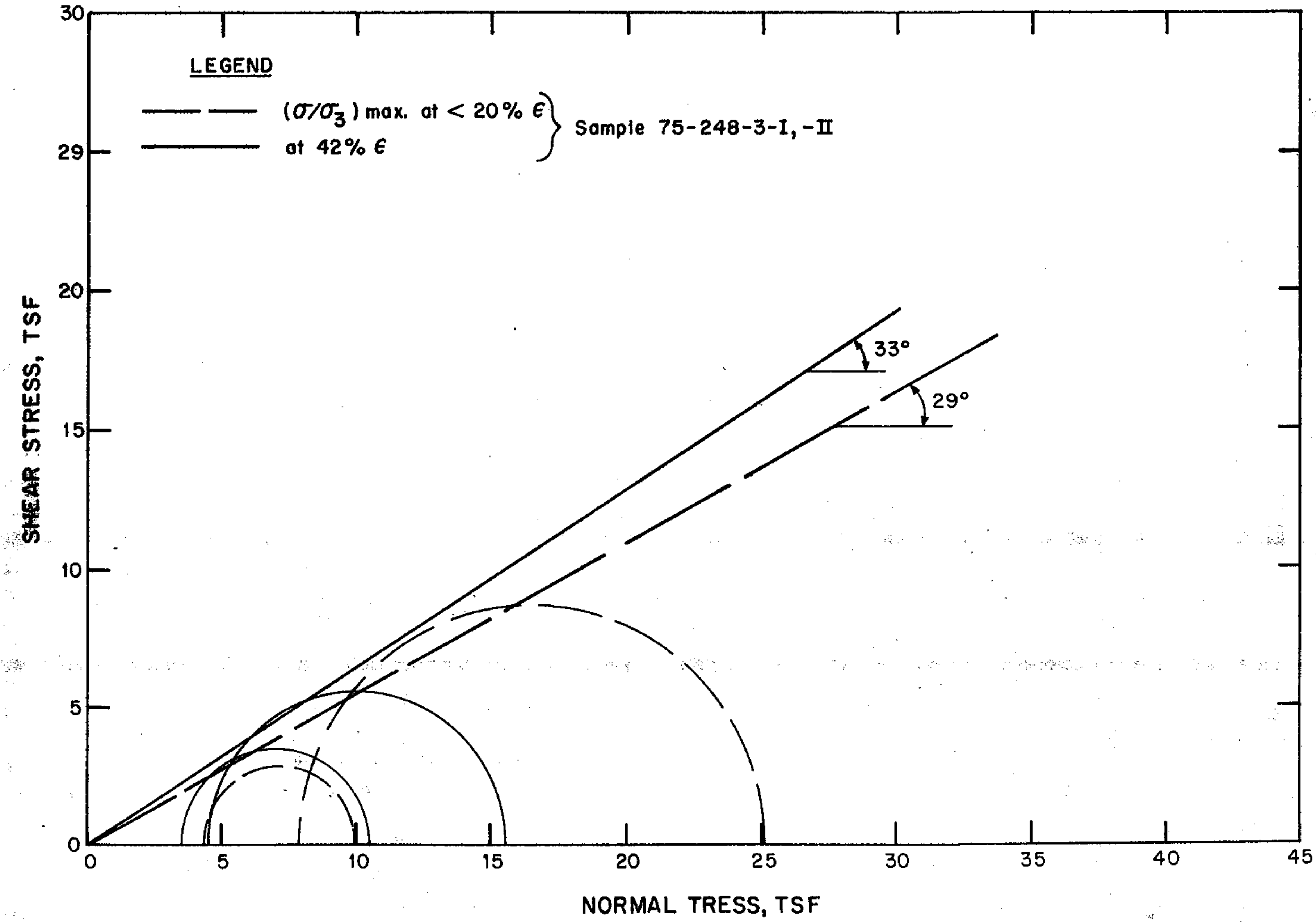

LEGEND
(σ/σ3) max. at < 20% ε
at 42% ε
Sample 75-248-3-I, -II
33°
29°
SHEAR STRESS, TSF
NORMAL TRESS, TSF
0
5
10
15
20
29
30
25
35
40
45

Figure 17

APPENDIX

Memorandum

To : Mr. W. Z. Hegy, District 01
District Director of Transportation

Attention Mr. R. K. Sweet
Chief, Operations Branch

Date: December 4, 1975

File : FHWA 4-2(5)
Van Duzen to Peanut
01-Tri-36, P.M. 12.49±
Lab. Auth. 662777

From : DEPARTMENT OF TRANSPORTATION
Transportation Laboratory

Subject: Slide condition on FHWA 4-2(5), Van Duzen to Peanut
01-Tri-36, P.M. 12.49

The purpose of this memorandum is to report the results of the December 2, 1975, field review of the slide condition at the above site by W. F. Wilson, D. W. Knittel and J. R. Lyon from the District, Dave Crane of Headquarters Maintenance, R. A. Forsyth, M. L. McCauley and R. H. Prysock of this Department.

As you know, the above slide has been the subject of an extensive investigation by District 01 Materials and this Department for the past two months. In the course of this investigation, the slide was mapped and instrumented with undisturbed samples being recovered for laboratory tests. These tests and subsequent analyses of the results are nearing completion. A detailed report on the investigation with recommendation for permanent correction of the slide will be submitted during the next two weeks.

The field review which is the subject of this memorandum was prompted by a recent development in which a substantial pushup (1½'±) was observed at roadway elevation. Visual observations at the site revealed that in addition to the activity at roadway, cracks within the slide mass have widened and scarps deepened appreciably within the past two weeks, indicating an acceleration of slide movement. In view of this, and the fact that the weather can be expected to become progressively wetter, it would appear that temporary corrective measures to stabilize the slide through the winter season are urgently required. Immediate action is strongly recommended for the following reasons:

1. It is very likely that a massive slide will occur during the winter as more subsurface water accumulates within the area of earth movement. Such a slide would close Route 36 for the winter and into late spring since any attempt to remove slide material during periods of excessively wet subsoils would only aggravate the situation.

2. If such a slide should occur, the cost for permanent correction during the next construction season will be substantially greater than that which would be required if temporary stability through the winter season is achieved.

3. There is a possibility that a massive slide would move the badly weathered material on the westerly part of the south cutface into Rattlesnake Creek.

It is, therefore, suggested that the following temporary remedial measures be taken as quickly as possible:

1. As shown by the attached cross section, construct a temporary buttress a minimum of 10' in height from existing roadway grade, keying into both north and south cutfaces.

2. If possible, from a construction and environmental standpoint, buttress material should be taken from the vicinity of the minor scarp approximately 400' from the head of the slide, as indicated on the attached cross section. This would increase overall stability.

3. Surface drainage should be modified to minimize ingress of water into the slide mass. This can probably best be accomplished by reshaping the existing cat road on the periphery of the slide in combination with the installation of perforated metal pipe. The importance of minimizing water entry into the slide mass subsoils cannot be over emphasized and should be attended to immediately.

It should be emphasized that the buttress at roadway level and modification of the existing surface drainage are the most critical elements of the proposed temporary correction. The suggested buttress thickness of 10' is beleved to be the absolute minimum required to achieve temporary stability. Additional material may be required if existing instrumentation reveals continued movement, particularly at roadway level.

It is believed that permanent correction can be achieved by some combination of unloading and positive subsurface drainage of the slide mass which would permit removal of the temporary buttress at roadway level.

GEORGE A. HILL
Chief, Transportation Laboratory

By Raymond A. Forsyth
Raymond A. Forsyth
Chief, Geotechnical Branch

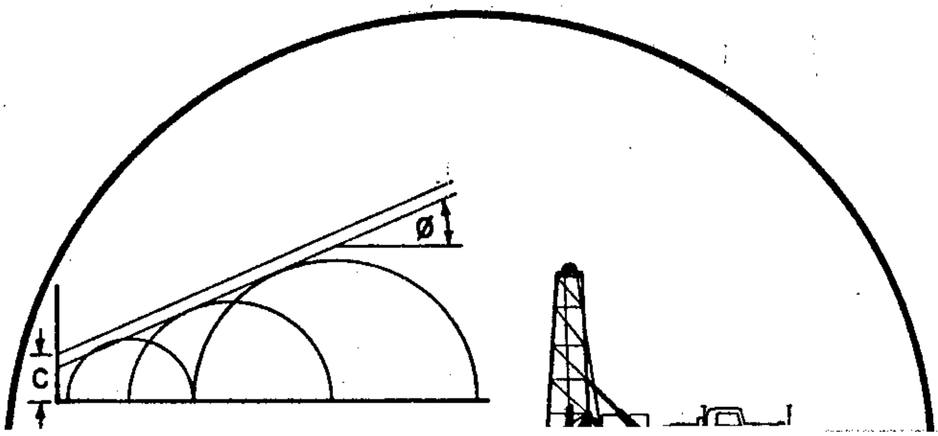
Attach.

REPORT DISTRIBUTION LIST:

cc C.E.FORBES, Chief Engineer, Caltrans
J.KOZAK, Chief, Office of Structures & Engineering Services
D. Crane, DOT, Maintenance
P.W.Welty, DOT, District 01 Maintenance
J.C.Bradfield, DOT, District 01 Materials
M.L.McCauley, DOT, Translab
R.H.Prysock, DOT, Translab
O.L.Homme, FHWA, Sacramento
J.M.Ellis, FHWA, Denver, Colorado
D.Maret, FHWA, Resident Engineer, Hayfork, CA
Alex ~~Terry~~ Tary, U.S. Forest Service, ~~Eureka~~ Redding, CA

5601

TRANSPORTATION LABORATORY



76-70

SLIDE INVESTIGATION AND GEOLOGICAL REVIEW 03-Gle-FOREST HIGHWAY 7

Prepared in Cooperation with the U.S. Department of Transportation,
Federal Highway Administration

Caltrans

CALIFORNIA DEPARTMENT OF TRANSPORTATION

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

September 1, 1976

ERFO 781(1)
03-G13-Forest Highway 7
(Mendocino Pass Road)
Lab. Auth. 662797

Mr. Jerry L. Budwig
Director, Office of Federal Highway Projects
Federal Highway Administration, Region 8
Denver Federal Center
Denver, Colorado 80225

A-446 Landslide
A-542 landslide [illegible]
A-830 [illegible]
B-61 [illegible]

Through Mr. Omar L. Homme, Division Administrator

Dear Mr. Budwig:

Submitted for your consideration is:

REPORT OF SLIDE INVESTIGATION
AND GEOLOGICAL REVIEW NEAR STATION 273±
03-GLE-FOREST HIGHWAY 7
WEST OF WILLOWS, CALIFORNIA

Study made by . . . . Geotechnical Branch

Under the General Direction of . . . . . . Raymond A. Forsyth

Work Supervised by . . . . Joseph B. Hannon

Report by . . . . Joseph B. Hannon and
Duane D. Smith

Very truly yours,

George A. Hill

GEORGE A. HILL
Chief, Office of Transportation Laboratory

Attachment

cc: O. L. Homme, FHWA, Sacto
G. Wishman, FHWA, Sacto
J. Walkinshaw, FHWA, SF
A. C. Estep, DOT, Local Asst.
G. A. Hill, DOT, Translab
H. L. Payne, DOT, District 03
L. V. Blackburn, DOT, Dist. 03
Rec'd. DRAFT COPY:
J.M.Ellis, Jr., FHWA, Colorado
Fondedile Company, Cambridge, Mass.

JBH:lb

INTRODUCTION

This report is the first of a series of three that will present the results of geological review and slide investigations performed at various sites on Forest Highway 7 (Mendocino Pass Road) located in Glenn County, west of Willows, California (Refer to Sheet 2). Their common purpose is to determine the feasibility of utilizing Fondedile Root Piles to correct the slope instability problems that exist on this facility and to consider other alternatives for correction. These locations were selected on April 8, 1976, during a field review conducted by personnel representing both FHWA and the Caltrans Laboratory. It was tentatively agreed that Caltrans would investigate these sites for feasibility of a Fondedile correction under an amended version of a previous agreement.

Forest Highway 7 has been the subject of various field reviews by County, State, and Federal agencies over the last four years due to storm damage in numerous unstable areas. Maintenance has become an ever increasing problem.

The various areas of instability of this project provide the opportunity to evaluate a proven European technique for increasing slope stability. FHWA is interested in the potential of the Fondefile system, also referred to as "Reticulated Pali Radice" or "Root Piles", as a possible alternative slope stabilization technique on Federal facilities. A previous investigation by Caltrans reported February 9, 1976, was conducted for a proposed experimental installation at Station 418. However, it was determined that instability at this particular location could be corrected more economically by a solution other than the Fondedile technique.

Based upon a field review of the Forest Highway 7 project on July 8, 1976, Dr. J. R. Sallberg of FHWA headquarters concurred that the three slides selected for the study had potential for correction with Fondedile piles. He also concurred on the desirability of an instrumentation and performance study of Fondefile root piles if that technique was employed at one of these sites.

The information in this report pertains to the investigation of the sidehill embankment between Stations 271± and 276±. This location appears to be the most challenging for the Fondedile correction. Additional site investigations will be covered in subsequent reports.

SITE GEOLOGY

The site of the slide under investigation between Stations 271± and 276± is located near the eastern edge of the Coast Ranges Geomorphic Province of California. This province consists of many separate ranges, coalescing mountain masses, and several major structural valleys.

Two entirely different basement complexes, one being the Jurassic-Cretaceous assemblage called the Franciscan Formation and the other consisting of Early(?) Cretaceous granitic intrusives and older metamorphic rocks, are present in this province.

The area of study has been mapped by the California Division of Mines and Geology as part of the Franciscan Formation. Geologic units mapped on the site by our department, however, are not the typical graywacke, shale, chert and conglomerate of the Franciscan but rather are characterized by metasedimentary rocks consisting predominantly of phyllite with secondary mica-quartz schist and slate. These rocks may be metamorphased Franciscan rocks, but are more probably part of an older basement complex mapped in adjacent areas as "Pre-Cretaceous Metasedimentary rocks".

Intermediate in metamorphic grade between slate and schist, the weathered surficial outcrops of phyllite display a rich golden silkly sheen on the surface of cleavage or schistosity and have a greasy feel when rubbed with the hand. Unweathered phyllitic core samples taken below the water table were dark blue-gray, with all gradations in color from the capillary fringe to the outcrops on the surface.

Interbedded with the phyllite are scattered thin layers of mica-quartz schist which are light to dark gray in color and are much harder than the phyllite. Local pods and veins of quartz are common, along with numerous zones of highly sheared and pulverized phyllite which take on the characteristics of clayey gouge.

The slide mass itself is composed of a mixture of reddish-brown clayey soil and rock fragments of the parent phyllitic and schistose bedrock.

Excellent exposures of the bedrock are visable on either side of the slide in the cut slopes. Here the thinly bedded meta-sediments display severe folding, faulting and fragmentation. This distortion is probably related to ancient movement along the major northwest-southeast trending Stony Creek Fault lying approximately four miles to the east.

DESCRIPTION OF SLIDE

Forest Highway 7 crosses the middle of an old landslide in the vicinity of Sta. 271 + 75 to 275±. Examination of aerial photos taken on October 10, 1964 during clearing and grubbing operations for the new alignment shows the main scarp to be located approximately 200 ft. right of centerline. The toe of the old slide may extend to Rattlesnake Creek at the base of the steep hillside (see attached Sheet 3).

Renewed movement occurred during the wet season of 1973. Cracks progressed up the cut slope and into the dense brush for a total distance of approximately 160 ft. right of centerline. The main,, arcuate shaped scarp is 5 to 6 ft. high with some cracking above it.

Movement on the downhill side also occurred with subsequent displacement of the roadway and sidehill embankment. Cracking and push outs of loose material can be observed for a distance of approximately 200 ft. down slope (see attached plan, Sheet 3).

Guardrail for some 100 ft. left of centerline has dropped a few feet and is partially suspended in the air. The underdrain which crosses the roadway near Sta. 273+60 was still functional. The out flow of this underdrain has deeply incised the steep hillside by the erosive action of runoff.

It appears that failure was precipitated by saturation of old, loose slide debris forming the 20 to 30 ft. high cut slope and the embankment foundation. Removal of lateral support by cutting undoubtedly contributed to the renewed instability.

A somewhat flat, amphitheater shaped area below the main scarp and above the top of cut serves as a catchment area for runoff, contributing to the saturation of the slide debris.

SITE INVESTIGATION

Field mapping of this slide began on May 6, 1976, by an engineering geologist of this department. Based on this geologic reconnaissance two borings were made, the location of which are shown on Sheet 3.

Drilling began at Boring D-1 on June 29, 1976, using a Failing 1500 drill rig and a crew of three drillers. An engineering geologist logged the boring. Continuous samples were taken using the 2-inch Modified California Sampler from the surface to refusal at 13 ft. From this level to the bottom of the boring at 55 ft., a 5 ft. Longyear core barrel with carboloy bits was used continuously. Groundwater was recorded at 36.5 ft.

A description of the materials encountered during the drilling operation is shown on the boring profiles (Sheets 1 and 4). Two soft, highly sheared, clayey gouge-like zones were noted at 13 and 45 ft. in boring D-1 and are thought to represent planes along which movement has occurred.

Boring D-2 was drilled, sampled and cored to a depth of 40 ft. An attempt was made to sample those soft zones which were encountered in Boring D-1 (Refer to Sheet 4). No representative samples could be obtained from the soft shear zones. Groundwater was encountered at the 31.2 ft. depth in this boring.

A slope indicator (S.I.) was also installed to monitor movement of the slide. The initial reading was taken on July 30, 1976, and the first follow-up measurement was made August 5, 1976. No movement was indicated during this time period. It appears the slide has stabilized for the interim but movement is highly probable with added lubrication of the slide mass. Subsequent readings will be made throughout the wet season to monitor any future movement. However, corrective action should probably be done prior to this winter season in order to prevent complete failure at this location.

FAILURE ANALYSIS

A slope stability analysis was performed using a cross-section at Station 273+50 provided by FHWA (Refer to Sheet 4). Since representative samples from the probable shear zone were not available for testing, the Soil X computer program technique was used to fit a failure arc to the assumed plane of movement. This zone was established from the geologic review and the boring logs. The process consists of assuming soil shear strength parameters which will yield a factor of safety of unity for the probable failure arc. For this condition, the groundwater table was also assumed to be elevated approximately 10 ft. above the indicated July 23, 1976 level. The soil strength parameters satisfying these conditions at failure were Ø = 13° and c = 500 PSF. Slope geometry, boring logs and location of the probable failure surface are shown on Sheet 4.

CORRECTION ANALYSIS

The following three alternate treatments are suggested as possible measures that could be applied to correct the instability that exists between Stations 271± and 276±:

1. Remove slide debris from cut slope above roadway. This could be accomplished by widening some 20 ft. at grade and using a variable cut slope from the edges of the unstable area to a maximum of 2:1 in the center of the slide as shown by attached Sheet 4. Positive drainage should be provided at grade to conduct the surficial water away from the slide area. This treatment would provide unloading of the slide and prevent surface water from ponding and entering the slide mass near the upper scarp. As part of this correction, horizontal drains could be drilled inward from locations adjacent to the lower periphery of the slide. More positive drainage could be achieved by constructing a 100 ft. length of cutoff trench 40 to 45 ft. right of centerline to a depth of 30 to 35 ft. The trench would be backfilled with permeable material with the upper 3 ft. depth sealed with native material to prevent infiltration of surface water. Water could be removed by means of an outlet pipe or intercepted by horizontal drains. This "deep trench" type of drainage installation was recently used successfully for the correction of a continuing slide problem on Interstate Route 80 near Vallejo. A tractor mounted backhoe with a digging capability to 34 ft. depth, opened the excavation with permeable material being placed immediately behind. Since no pipe was placed and so little of the excavation was opened at any given time, it was not necessary to use shoring, which, of course, would have made the price prohibitive. As would be the case for all three alternates, the down slope portion of the slide mass should also be dressed

and sealed to improve drainage and prevent saturation and erosion at the toe. The effect of the unloading as indicated by the stability analysis referred to earlier increased the factor of safety from unity (F.S. = 1.0) to F.S. = 1.18. When the water table is lowered to the July 23, 1976 level by dewatering and is combined with unloading, the factor of safety increases to 1.24. No additional instability is anticipated on the cut slope, if all loose material from previous movement is removed by unloading. This alternate would probably be the most economical of the three.

2. The second alternate would require a detour and would consist of removing the existing slide mass and constructing a stabilization trench keying into firm material and providing subsurface drainage by means of a permeable blanket and outlet trench. The approximate location and general detail of the stabilization trench is shown on attachement Sheet 4. This solution has a degree of risk since it would remove lateral support for the upper portion of the slide mass. When constructed, however, it would be quite effective since it would provide positive drainage and a shear key into relatively competent material.

3. The third solution involves the construction of a Fondedile reticulated pile structure founded below the slip plane. This particular slide appears to be the most active of the three studied. However, the location of the slip plane may require pile depths in excess of 50 ft. Drilling at this site will be more difficult than the other locations and will probably require a rock bit rather than a soil auger.

The feasibility and design of this correction will be determined by Warren-Fondedile, Incorporated of Cambridge, Massachusetts. A copy of this report will be forwarded to their office for information. A proposal for construction instrumentation and performance of this system will be prepared by the Caltrans Laboratory in the event that Alternate 3 is selected as the remedial measure.

INSTRUMENTATION PROPOSAL FOR FONDEDILE PILE
INSTALLATION ON FOREST HIGHWAY 7

INTRODUCTION

The Federal Highway Administration has recently encouraged various state highway agencies to study the Fondedile pile system for stabilization of slopes. Dr. Sallberg of FHWA Headquarters visited the FH-7 Project in July 1976. Based upon this review, he indicated that at least one of the three sites being investigated may be suitable for correction using the Fondedile technique. If this is confirmed by the results of the ongoing investigation, it is suggested that a minimum of five (5) piles within the proposed Fondedile Pile system be instrumented during the construction operation. This instrumentation can be used for control and long-term performance evaluation.

The proposed instrumentation work to be financed as part of construction control would consist of purchase of instrumentation, laboratory installation and calibration of instrumentation, field installation and initial monitoring. No data analysis or reporting would be conducted under this contract agreement. A follow-up HPR study would be utilized for subsequent monitoring of instrumentation, load testing and test of pile system response, analysis of data, establishment of design criteria and reporting. The HPR proposal is attached for information. It will be submitted for approval through normal research channels, if the construction instrumentation plan is acceptable.

SCOPE

The proposed instrumentation plan for construction control consists of instrumenting five (5) preselected piles and providing additional slope indicator monitoring locations and both horizontal and vertical survey points. Four (4) strain gages will be installed on the circumference at the 90 degree points on the reinforcing steel bar of each test pile at three elevation levels. Carlson M-4 Strain Meters will also be installed at the same levels and locations by means of a special jig attached to the reinforcing bar. These special jigs will hold the strain meters in position at a distance sufficient to provide approximately 1-inch of concrete cover between the outer nominal pile diameter and the strain meter. Load cells will also be welded to the bottom of each reinforcing bar of the five (5) test piles. The instrumented bars will be installed prior to concrete placement.

Load cells will be installed at the top of each instrumented reinforcing bar by means of a special attachment and grouted into the cap beam. All instrumentation wires will be routed to an instrumentation readout facility. A schematic of the proposed instrumentation plan is attached.

COST ESTIMATE

I. Purchase of Instruments

1. Strain Gages

    5 piles x 3 levels x 4 gages = 60 gages
    60 gages @ $15 ea. = $ 900

2. Carlson Strain Meters (M-4, 5/8-inch diameter x 4 inches long)

    5 piles x 3 levels x 4 meters = 60 meters

    60 meters @ $40 ea. = 2,400

3. Load Cells (Laboratory Fabricated)

    5 piles x 2 cells = 10 cells
    10 cells @ $500 ea. = 5,000

4. Slope Indicator Casing

2 installations x $300		600
Total		$ 8,900

II. Laboratory Installation and Calibration of Instruments

1. Strain Gages

60 gages x 4 M.H./gage x $30/Hr.	=	$ 7,200

2. Carlson Strain Meters

60 meters x 1 M.H./meter x $30/Hr.	=	1,800

3. Load Cells

10 load cells x 2 M.H./load cell x $30/Hr.	=	600
Total		$ 9,600

III. Field Installation of Instruments

1. Strain Gages

60 gages x 2 M.H. x $30/Hr.	=	$ 3,600

2. Carlson Strain Meters

60 meter x 4 M.H. x $30/Hr.	=	7,200

3. Load Cells

10 cells x 2 M.H. x $30/Hr.	=	600

4. Slope Indicators

2 indicators x 25 M.H. x $30/Hr.	=	1,500

5. Survey Monuments

20 M.H. x $30/Hr.	=	600
Total		$13,500

IV. Initial Instrumentation Monitoring

40 M.H. x $30/Hr.	=	$ 1,200

V. Travel Expenses

Field Installation and Monitoring

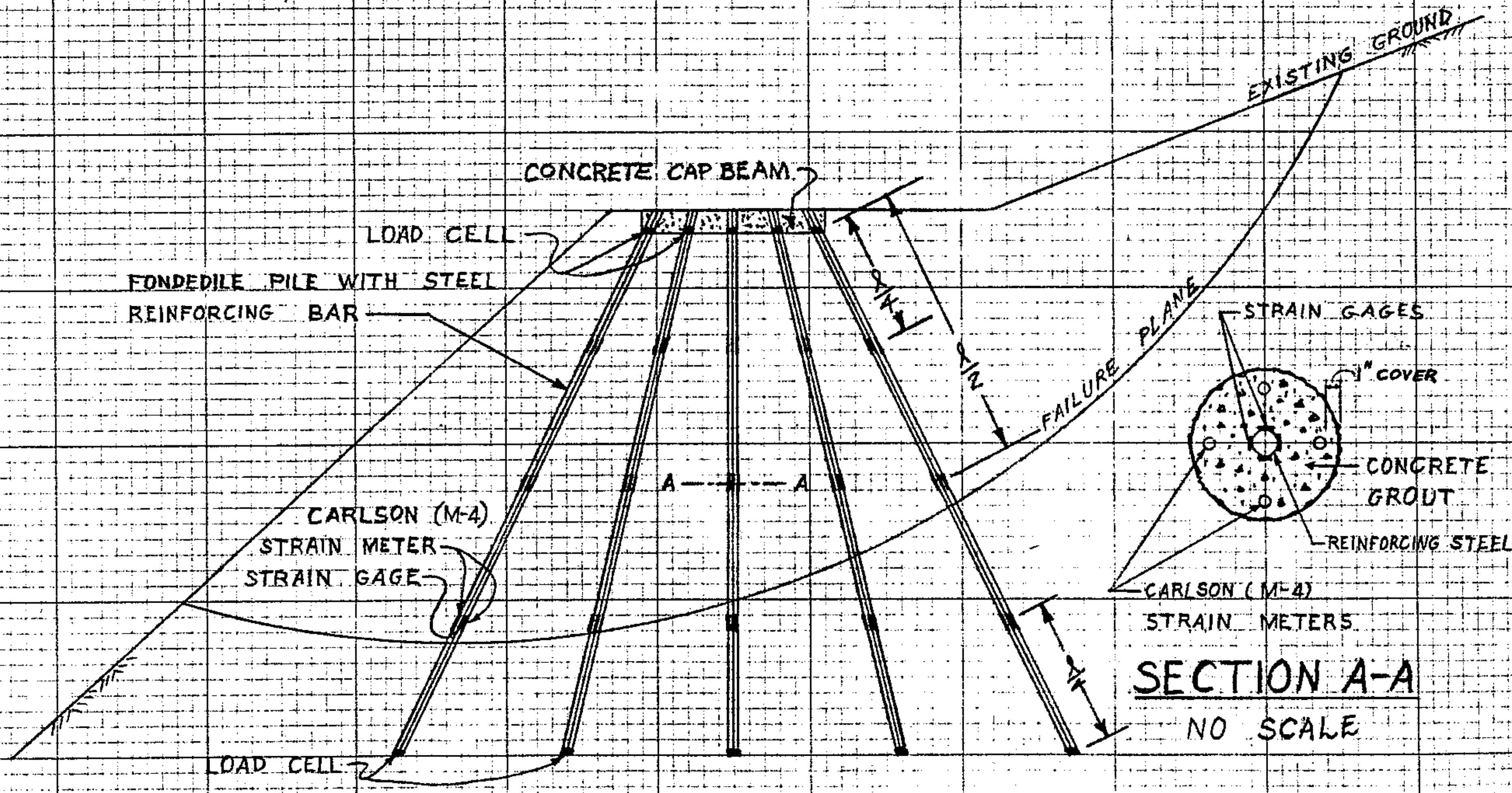
Per Diem 60 M.D. x $35/day	=	2,100

VI.	Equipment		
	Miscellaneous Vehicles		300
	Drilling Equipment		300
	Truck for Instrumentation		300
		Total	$ 900
VII.	Materials (Plugs, cable, wire, misc.)		$ 800
VIII.	Engineering Supervision		
	100 M.H. x $30/Hr.		$ 3,000
		Grand Total	$40,000

SUMMARY OF ESTIMATED COST

		MAN HOURS	COST
I.	Purchase of Instruments	--	$ 8,900
II.	Laboratory Installation and Calibration of Instruments	320	9,600
III.	Field Installation of Instruments	450	13,500
IV.	Initial Instrumentation Monitoring	40	1,200
V.	Travel Expenses	--	2,100
VI.	Equipment	--	900
VII.	Materials	--	800
VIII.	Engineering Supervision	100	3,000
	TOTAL	910	$40,000

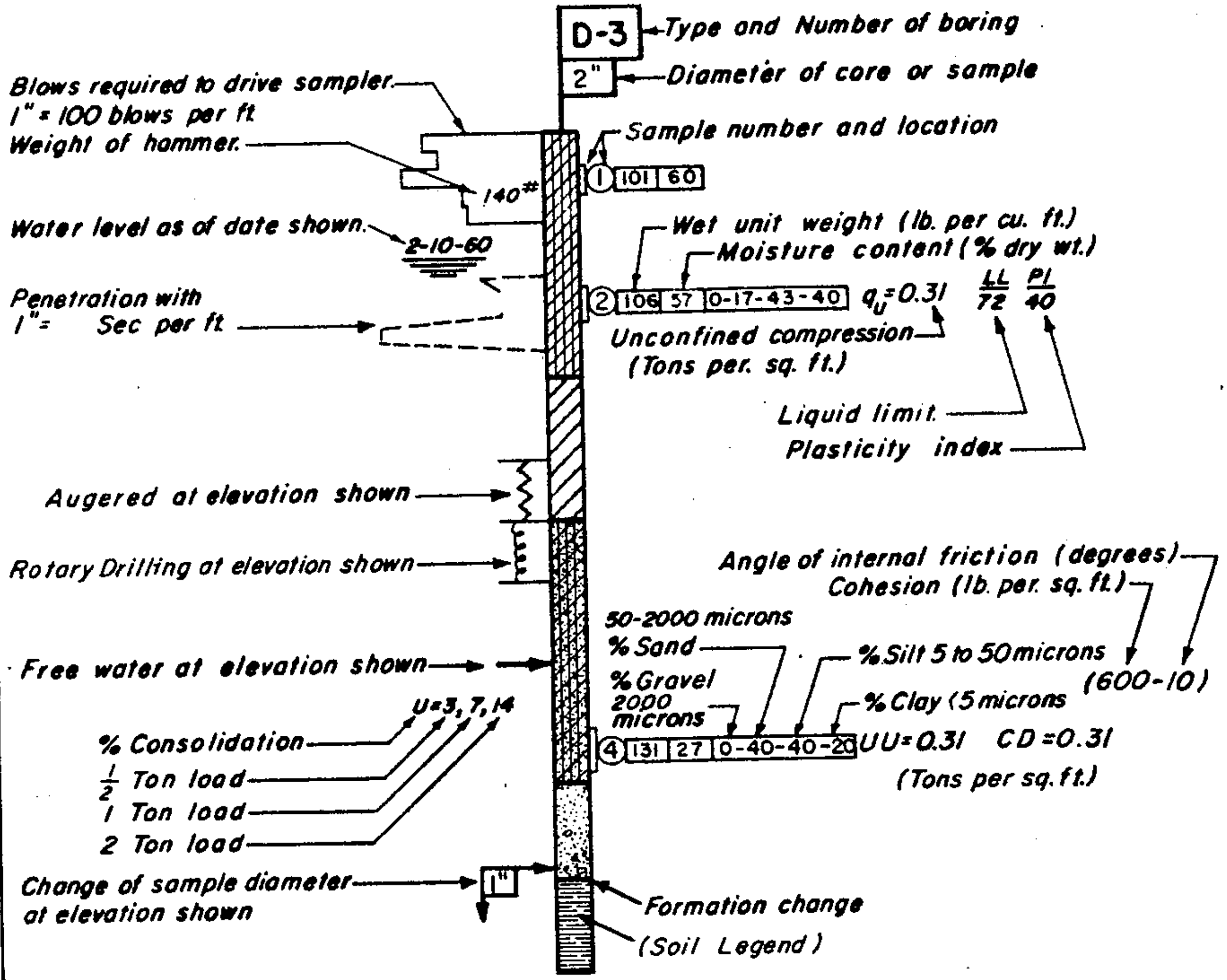
SCHEMATIC OF PROPOSED INSTRUMENTATION FOR FIVE PILES



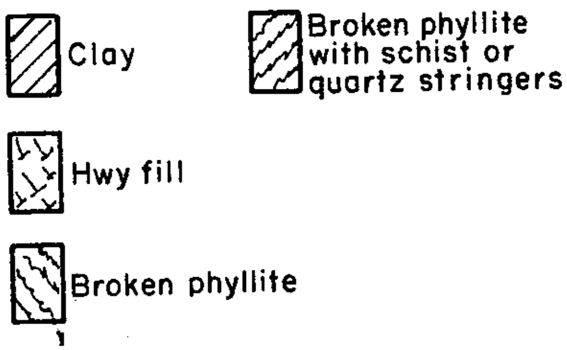
SECTION A-A
NO SCALE

TYPICAL SECTION
FONDEDILE PILES
NO SCALE

BORING LEGEND
CROSS-SECTION & PROFILE SHEETS



SOIL LEGEND



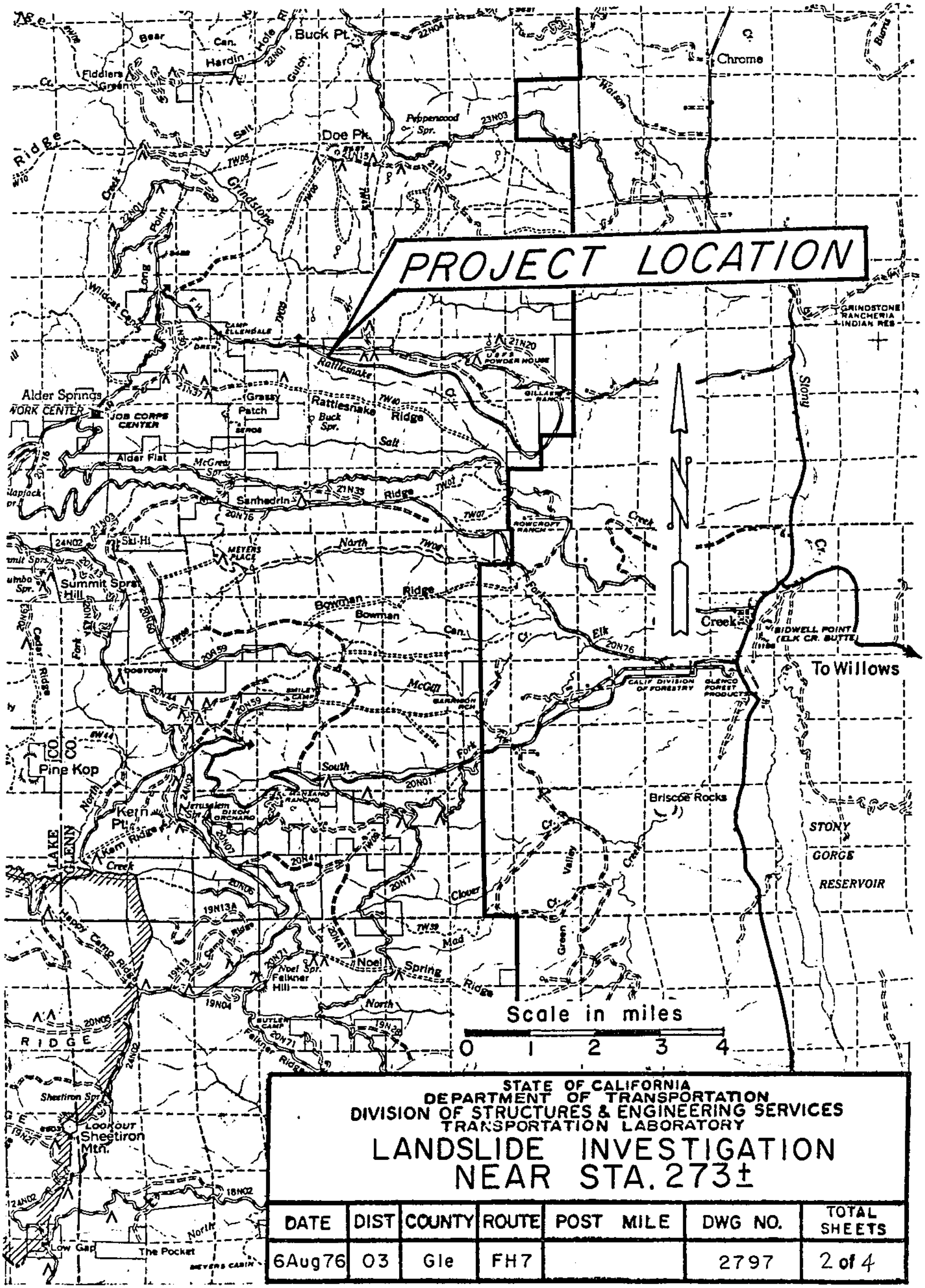
STRENGTH TESTS

q_u - Unconfined Compression
UU - Unconsolidated Undrained
CU - Consolidated Undrained
CD - Consolidated Drained

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF STRUCTURES & ENGINEERING SERVICES
TRANSPORTATION LABORATORY

LANDSLIDE INVESTIGATION
FOREST HIGHWAY 7
BETWEEN WILLOWS & COVELO

DATE 6 Aug 76		SUBMITTED BY: Joseph Hannon SR. MATLS. & RES. ENG'R.	DWG. NO. 2797
DR BY	CK. BY	APPROVED BY: CHIEF, GEOT. BR.	SHEET NO 1 OF 4 SHTS.



STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF STRUCTURES & ENGINEERING SERVICES
TRANSPORTATION LABORATORY

LANDSLIDE INVESTIGATION NEAR STA. 273±

DATE	DIST	COUNTY	ROUTE	POST MILE	DWG NO.	TOTAL SHEETS
6Aug76	03	Gle	FH7		2797	2 of 4

5634

NOT FOR PUBLICATION

DIVISION OF STRUCTURES AND ENGINEERING SERVICES

TRANSPORTATION LABORATORY

RESEARCH REPORT

PULL RESISTANCE AND INTERACTION OF EARTHWORK REINFO

77-38

DND

Presented at the 56th Annual Meeting of the Transportation Research Board

January 1977

Prepared in Cooperation with the U.S. Department of Transportation, Federal Highway Administration

Caltrans

CALIFORNIA DEPARTMENT OF TRANSPORTATION

5634

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

PULL RESISTANCE AND INTERACTION OF EARTHWORK REINFORCEMENT AND SOIL

By

Jerry C. Chang
Associate Materials and Research Engineer

Joseph B. Hannon
Senior Materials and Research Engineer

Raymond A. Forsyth
Supervising Materials and Research Engineer

Prepared for Presentation at the Annual Meeting
of the Transportation Research Board
Washington, D.C.

January 1977

REFERENCE: Chang, Jerry C., Hannon, Joseph B., and Forsyth, Raymond A., "Pull Resistance and Interaction of Earthwork Reinforcement and Soil".

ABSTRACT: Presented are results of tests on different types of earthwork reinforcement to study their pulling resistance and the soil-reinforcement interaction in reinforced earth. Three types of pulling tests were conducted. Results indicate that the soil will not be significantly strained until a proportional limit is reached on a load-deformation curve. For the same surface area, reinforcing bar mesh has nearly six times the pulling resistance of the flat reinforcing strip. Reinforcing bar mesh embedded in a more dense cohesive soil exhibited greater pulling resistance than that embedded in less dense cohesionless soil. An increase in size of mesh opening will substantially reduce the pulling resistance of the bar mesh. The minimum length of reinforcing strip required for a low height reinforced earth wall was found to be at least 3.1 meters (10 feet).

KEY WORDS: Bar-mesh, deformation, earthwork, friction angle, geotechnical engineering, highways, interaction, material testing, pulling resistance, reinforced earth, reinforcement, safety factor, shear strength, skin friction, strains.

PULL RESISTANCE AND INTERACTION OF EARTHWORK

REINFORCEMENT AND SOIL

Jerry C. Chang, Joseph B. Hannon, and Raymond A. Forsyth

California Department of Transportation

INTRODUCTION

The utilization of steel strips as earthwork reinforcement has been reported by Vidal (8, 9), Lee, et al., (6, 7), and Chang, et al., (1, 2, 3, & 4). More than a dozen reinforced earth walls have been constructed in the United States in recent years under the patented design of the Reinforced Earth Company (5). The California Department of Transportation built the first one of these walls with steel facing and steel strip reinforcements in 1972 on Highway 39 in the San Gabriel Mountains of Los Angeles County. A second reinforced earth wall with concrete facing and steel strip reinforcements was constructed on Interstate Highway 5 near Dunsmuir, California, in 1974.

Two other experimental projects designated as "Mechanically Stabilized Embankment" with concrete beam facing and bar-mesh reinforcements were also constructed on Interstate Highway 5 at Dunsmuir, California, in September 1975 and May 1976, respectively, (Photo 1-a and 1-b). These experimental projects were designed and constructed by the California Department of Transportation under an agreement with the Reinforced Earth Company of Washington, D.C. Future construction using this system will be under their license.

For the purpose of studying the pulling resistance and the interaction of different types of reinforcement and soil for these construction projects, field and laboratory large and small scale pull tests were conducted. These tests and the results are discussed in subsequent sections of this paper.

FIELD PULL TESTS ON STEEL STRIPS

General Description: In order to study the pull resistance of soil reinforcements in the field additional dummy steel strips for pulling tests were installed in the reinforced earth wall on California Highway 39 in the San Gabriel Mountains at different elevations during construction. Three strips, 1.5, 3.1, and 4.6 meters (5, 10, and 15 feet) in length, were embedded at each of three levels under overburden heights of 2.3, 3.8, and 5.6 meters (7.5, 12.4, and 18.2 feet), respectively. Three 7 meter (23 foot) strips were also embedded at a depth of 5.5 meters (18 feet) and three 14 meter (46 foot) strips were embedded at a depth of 11.60 meters (38 feet). One each of the 7 meter (23 foot) and 14 meter (46 foot) strips was instrumented with strain gages on both top and bottom at 1.50 meters (5 foot) intervals. All of the steel strips are 3 mm (0.118 inches) thick and 60 mm (2.362 inches) wide.

The fill material is primarily decomposed granite. The physical properties of soil samples obtained from a nearby borrow site are presented by Table 1.

Test Results and Discussion: Figure 1 shows 6 typical load-deformation curves obtained from field pull tests for 1.5, 3.1, 4.6, 7, and 14 meters (5, 10, 15, 23, and 46 foot) strips. Three pulling loads selected for analysis are indicated on these curves. They are: (1) Yield load representing the proportional limit of the load-deformation relationship; (2) Peak load representing the maximum pull load observed; (3) The residual load representing the pull load when deformation increased appreciably without changing the pulling load.

Tensile strength tests on a small size sample of the reinforcing strip resulted in yield and ultimate capacities of 45.8 kilonewtons (10,300 pounds) and 61.8 kilonewtons (13,900 pounds), respectively.

At the site, one of the 7 meter (23 foot) strips broke at a pull load slightly over 62.3 KN (14,000 pounds). Three of the 14 meter (46 foot) strips broke at pulling loads of 59.9, 63.2, 64.3 KN (13,460, 14,200, and 14,450 pounds). The rest failed by slipping.

The straight line portion of the load-deformation curve represents the elastic properties of the steel. When the steel strip has sufficient length and overburden, i.e., when the frictional grip is great enough, the proportional limit will reflect the yield capacity of the steel. If the frictional grip is not sufficient at the loaded end of the strip, the soil will start to strain at a lower proportion limit before the yield strength of the steel is reached. Therefore, it appears that the yield load represents either the yield capacity of the steel or the initial point of soil-steel interaction as a composite material. The values of the yield load depend upon the length of the reinforcement and the overburden load. The peak load represents the maximum mobilized pulling resistance of the composite material of reinforcement and soil. Upon reaching the peak load, the strips begin to slip and the pull load drops to the residual or ultimate level. Figure 2-a shows the relationships between the peak pull load, the overburden load, the height of overburden, and the length of the strips. At the same height of overburden, the peak pull load is proportional to the overburden load shown by the solid straight lines. The slope of these lines decrease with increasing overburden height. For the same strip length, the relationship between the peak pull load and overburden load is approximately linear as indicated by the dash lines. The solid lines suggest that the longer strips have higher peak pullout resistance for the same height of overburden. For a given strip length, the pull resistance increases with increasing in overburden heights. The rate of increase in peak pull load due to increasing overburden is much smaller than that due to increasing strip length. Figure 2-b shows similar characteristics for the yield load.

Since the peak load represents the maximum mobilized friction force, the factor of safety can be evaluated using the peak load as failure load and the factor of safety against slippage can be evaluated using the following equation proposed by Chang (1, 2, 3, & 4):

Factor of safety against slippage =

$$\frac{\text{Peak Failure Pulling Load}}{K_a \, \gamma \, H \, d \, \Delta H} \quad \text{.........................} \quad (1)$$

Where K_a = coefficient of active earth pressure

γ = unit weight of soil

H = height of fill above the reinforcement concerned

d = horizontal spacing of reinforcement

ΔH = vertical spacing of reinforcement.

The relationships between overburden height, H, strip length, L and the factor of safety (F.S.) are shown in Figure 2-c, which can be used as a guide for selecting the minimum length of reinforcement required for a given height of fill.

Figure 2-d shows the relationship between the peak pulling load and the skin friction force calculated by the following equation:

$$\text{Skin Friction Force} = 2F = 2 \gamma H b L \tan \delta \quad \text{.....................} (2)$$

Where F = friction force on one face of the reinforcement

b = width of reinforcing strip

L = length of reinforcing strip

δ = skin friction angle between soil and steel reinforcement.

H and γ were defined previously.

It will be noted from Figure 2-d that the peak pull loads exceed the calculated skin friction force when the strip length exceeds 3.1 meters (10 feet). It may thus be concluded that strip length should be a minimum of 3.1 meters (10 feet).

Figure 3-a and 3-b show the distribution of tensile and compressive forces measured by strain gages spaced at 1.5 meters (5 foot) interval along the length of strip for a 7 meter and 14 meter strip, respectively. The dashed lines indicate the tensile or compressive forces in the steel strip measured at each strain gage location under the embankment load. Each solid line represents the distribution of the total tensile forces along the length of strip which include the forces induced by each applied pull load and the existing forces induced by the embankment load measured. Figure 3-a shows that each externally applied pull load induced additional tensile forces for almost the entire length of the 7 meter strip. However, the external applied pull load only stressed the 14 meter strip to gage point to a distance of 3.1 meters (10 feet) into the fill. There were no additional forces measured beyond gage point 10 other than the existing forces induced by the embankment load before testing. The 14 meter strip broke at the external extended portion under the maximum pull load of 59.9 kilonewtons (13.46 kips). Because of its greater length, under a heavier embankment load, the 14 meter strip appeared to develop a fixed point at gage point 10, so that no additional tensile forces developed beyond 3.1 meters (10 feet) of the strip length within the fill.

LARGE SCALE LABORATORY PULLING TESTS

General Description: In order to understand the interaction between the soil and reinforcements large scale laboratory pull tests were conducted. The test facility consists of a rigid steel box, 45.70 cm (18 inches) high, 91.40 cm

(36 inches) wide, and 137.20 cm (54 inches) long, with provisions for applying vertical pressure to simulate the overburden load up to 15.2 meters (50 feet) of earth fill (Photo 2). The test specimen is compacted in the box with reinforcement placed horizontally in the middle height of the specimen. A vertical normal load is applied to consolidate the specimen. Reinforcement can then be pulled out at a controlled rate of 0.05 mm per minute (0.002 inches per minute). During pull testing, the front face of the box is removed providing a free unrestrained face of the soil specimen.

Ten tests were conducted with four different types of reinforcements shown in Figure 4, namely:

(1) Bar mesh consisting of eight smooth longitudinal reinforcing bars and seven welded cross bars 0.95 cm (3/8 inch) in diameter providing 10.20 cm x 20.30 cm (4 inch x 8 inch) mesh openings for Test Run No. 1 to 4.

(2) Longitudinal smooth bars 0.95 cm (3/8 inch) in diameter and 1.37 meters (54 inches) long with different spacing for Test Run No. 5 to 6.

(3) Solid steel plate 3 mm (1/8 inch) thick 71.1 cm (28 inches) wide and 1.37 meters (54 inches) long for Test Run No. 7.

(4) Steel strips 3 mm (1/8 inch) thick, 6 cm (2.3 inches) wide and 1.37 meters (54 inches) long for Test Run No. 8 to 10.

Test Runs No. 1 to 4 were performed under normal pressures of 34.47, 68.95, 137.90, 172.35 kilopascals (5, 10, 20, 25 psi), respectively, and Test Run No. 5 through 10 under 68.95 kpa (10 psi) normal pressure.

Soil samples employed in the tests were obtained from the job site on Interstate Highway 5, at Dunsmuir, California. This material is primarily poorly graded gravelly sand (SP). The physical properties of the soil samples are listed in Table 1.

Time-deformation and Load-deformation Curves: During pulling tests, the deformations of the reinforcement were measured by two extensometers: One located at the front and one at the rear of the specimen. The normal loads and pull loads were measured by load cells. A typical time-deformation curve for Test Run No. 3 on bar mesh under 137.90 kpa (20 psi) normal pressure is shown in Figure 5.

A typical load-deformation curve for the same test is shown in Figure 6. The load-deformation characteristics were found to be quite different for each type of reinforcement.

It is interesting to note that: (1) The strain response at the rear of the specimen always lags that at the front or loaded end (Figure 5); (2) At the time when the pulling load is sufficient to cause an abrupt change in deformation at the rear of the specimen on the time-deformation curve (Figure 5) the proportional limit or yielding point indicated by the load-deformation curve (Figure 6) is attained, i.e., the yield point occurred at the same strain level measured at the front end as shown by Figure 5 and 6. It can be hypothesized that the applied pulling load only strains the reinforcement without causing any significant strain in the soil until the yielding load is reached when the whole length of the reinforcement exhibits an abrupt change in deformation.

Figure 7-a and 7-b show the typical load-deformation curves obtained from pull tests on bar mesh with mesh opening of 10.20 x 20.30 cm (4x8 inches) and 12.7x 35.6 cm (5x14 inches), respectively and embedded in silty clay soil under normal pressure of 137.90 kpa (20 psi). This silty clay soil (Table 1) was obtained from a project site on Highway 101 at Cuesta Grade near San Luis Obispo, California.

It was noted that the peak pulling loads were higher in a rather dense silty clay soil (Figure 7-a) than those in a less dense gravelly sand soil (Figure 6) for the same bar mesh reinforcement. However, in the same soil, the peak pulling load decreased substantially when mesh openings increased to 12.7 cm x 35.6 cm (5x14 inches) (Figure 7-b).

Failure Mode: The failure mode resulting from laboratory tests indicated that all the specimens with bar mesh reinforcement failed by development of a cone shape soil wedge (Photo 3) while the specimens with longitudinal reinforcing bar and steel strips failed by slippage, with only a local slump of the soil at the front face of the specimens (Photos 4 and 5). The specimen with a solid steel plate reinforcement also failed in a small cone shape mode (Photo 6) similar to the specimen with bar mesh reinforcement (Photo 3). This mode of failure is believed to represent full mobilization of soil resistance, i.e., the development of a passive pressure wedge. The cone shaped failure mode indicates that the soil and bar mat reinforcement failed as a unit.

Comparison of Test Results: The pulling test results for the smooth bar mesh reinforcement with 10.20 cm x 20.30 cm (4x8 inch) openings embedded in gravelly sand soil under three different normal loads were plotted in Figure 8. The yield, peak, and residual load are all proportional to the normal loads.

The comparison of the pulling resistance for different types of reinforcement embedded in gravelly sand soil are shown in Table 2. It is seen that the smooth bar mesh reinforcement because of its anchoring affect has much higher pulling resistance than the other types of reinforcement. For the same surface area, the bar mesh reinforcement has a peak pulling resistance of about six times that of the longitudinal bars and steel strips.

A comparison of the pulling resistance for bar mesh embedded in gravelly sand and silty clay, respectively, is also shown in Table 2. The same bar mesh embedded in a rather dense silty clay soil has greater pulling resistance than that embedded in the gravel sand soil. The higher pullout resistance could reflect the higher density or effect of cohesion present in the silty clay sample. For the same soil material, the bar mesh with larger mesh openings produced lower pulling resistance.

SMALL SCALE LABORATORY PULL TESTS

Small scale laboratory pull tests were also conducted in a specially designed shear box (photo 7) to determine the skin friction between the galvanized reinforcing steel strip and two types of soil, namely, Highway 39 decomposed granite and gravelly sand material from Interstate 5 at Dunsmuir (Table 1).

These test results suggest that the skin friction angle is slightly smaller than the internal friction angle of the soil for granular material.

CONCLUSIONS

The field pulling test results substantiated the following conclusions:

1. The soil will not be strained significantly until the proportional limit (or yield point) is reached. At this point, the load-deformation curve becomes nonlinear for the composite steel strip and soil material.
2. The maximum tensile stress in the reinforcement is developed near the front face of the wall for any externally applied pull force.
3. The required minimum length of steel strip is about 3.1 meters (10 feet) for a reinforced earth fill lower than 3.1 meters (10 feet) in height.

From the laboratory testing results the following conclusions can be drawn:

1. The proportional limit on the front end load-deformation curve can also

be determined from the displacement on the front end time-deformation curve at the point when an abrupt increase in deformation occurs on the time-deformation curve for the rear of the specimen. Thus, the soil will not be significantly strained until reaching a proportional limit which defines the initial point of soil-steel interaction as the composite material.

2. For the same surface area, plain bar mesh has nearly six times the pullout resistance as steel strips or plain longitudinal reinforcing bars in gravelly sand soil.
3. Bar mesh embedded in a rather dense silty clay soil exhibited greater pulling resistance than bar mesh embedded in less dense gravelly sand soil.
4. An increase in mesh opening will substantially reduce the pullout resistance of the bar mesh.
5. The skin friction angle between a galvanized steel strip and soil for granular material is only slightly smaller (6 to 13 percent) than the internal friction angle of the soil. For practical design purposes the skin friction angle between the galvanized reinforcing strip and soil can be assumed to be 10 percent smaller than the internal friction angle of the soil.
6. Cohesive soil of low plasticity can be used in reinforced earth providing that bar mesh is to be utilized for reinforcement.

REFERENCES:

1. Chang, Jerry C., Forsyth, Raymond A., and Smith, Travis (1972), "Reinforced Earth Highway Embankment - Road 39", Highway Focus, Vol. 4, No. 1, Jan. 1972, Federal Highway Admin., U.S. Dept. of Transportation.
2. Chang, Jerry C., Beaton, J. L., Forsyth, Raymond A. (1974), "Design and Field Behavior of the Reinforced Earth Embankment - California Highway 39", presented at the Jan. 21-25, 1974, ASCE National Water Resources Engineering Meeting, held at Los Angeles, California, and submitted to the Journal of the Geotechnical Engineering Division, ASCE for publication.
3. Chang, Jerry C., and Forsyth, Raymond A. (1974), "Performance of A Reinforced Earth Fill", Transportation Research Record No. 510, Soil Mechanics, Transportation Research Board, National Research Council, Wash., D.C. 1974.
4. Chang, Jerry C., "Earthwork Reinforcement Techniques" (1974), Final Research Report No. CA-HY-TL-2115-9-74-1, Transportation Laboratory, Calif. Dept. of Transportation, Sept. 1974.
5. Gedney, David S., and McKittrick, David P. (1975), "Reinforced Earth: A New Alternative for Earth-retention Structures", Civil Engineering - ASCE, Oct. 1975, pp 58-61.
6. Lee, Kenneth L., Adams, Bobby Dean, and Vagneron, Jean-marie J. (1973), "Reinforced Earth Retaining Walls, Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 99, No. SM10, Oct. 1973, pp 745-764.
7. Lee, Kenneth L., and Richardson, Gregory N. (1974), "Seismic Design of Reinforced Earth Walls", presented at ASCE National Meeting on Water Resource Engineering at Los Angeles, Jan. 25, 1974, published in Journal of the Geotechnical Engineering Div., Vol. 101, No. GT2, Feb. 1975.

8. Vidal, Henri (1969), "The Principle of Reinforced Earth", Highway Research Record, No. 282, pp. 1-16, Wash., D.C., 1969.

9. Vidal, Henri (1969), "Reinforced Earth, Recent Application", English translation of the text entitled "La Terre Armee" as published in the Annales de l'Institute Technique du Batiment et des Travaux Publics, Series - Material (38) No. 259-260, July-Aug. 1969.

TABLE 1: PHYSICAL PROPERTIES OF SOIL SAMPLES

JOB LOCATION	HIGHWAY-39 San Gabriel Mountains	INTERSTATE-5 Dunsmuir	HIGHWAY-101 Cuesta Grade
SAMPLE NUMBER	72-RE8	73-1803	74-1231
CLASSIFICATION	Sandy Gravel (GW)	Gravelly Sands (SP)	Silty Clay With Gravel (CL)
EFFECTIVE FRICTION ANGLE	40°	35°	34°
COHESION (kpa)	29	55	97
MAXIMUM DRY DENSITY (Kg/m^3)	2227	1233	2034
SAND EQUIVALENT	28	27	15

NOTE: 1 kpa = 0.145 psi
1 kg/m^3 = 0.624 lb/ft^3

Jerry C. Chang, Joseph B. Hannon
Raymond A. Forsyth

TABLE 2: COMPARISON OF PULL RESISTANCE FOR DIFFERENT TYPES OF REINFORCEMENT AND TWO TYPES OF SOILS

NOTE:
1 Kilonewton = 0.225 Kips
1 Kilopascal = 0.145 Psi
1 cm = 0.394 In

TEST RUN NO.	TYPES OF REINFORCEMENT (See Fig. 4 for Detail)	TOTAL SURFACE AREA IN PERCENT OF BAR-MESH WITH 10x20 cm OPENINGS	PEAK RESISTANCE IN KILONEWTONS UNDER 70 kilopascal (10 psi) normal pressure	
			GRAVELLY SANDS (SP) (Dunsmuir)	SILTY CLAY WITH GRAVEL (CL) (Cuesta Grade)
2	Bar Mesh with 10x20 cm openings	100	167	176
5	8-longitudinal bar	69	15	-
6	12-longitudinal bar	100	30	-
7	1-solid steel plate	409	79	-
8	1-galvanized steel strip	34	12	-
9	2-galvanized steel strip	68	24	-
10	3-galvanized steel strip	102	30	-
	Bar-Mesh With 13x36 cm Openings	97	-	129

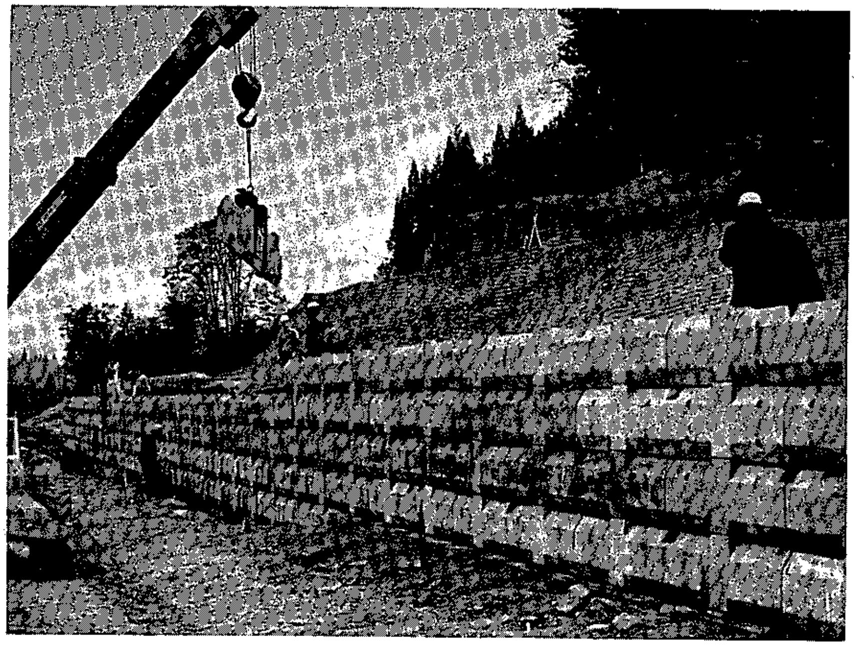


PHOTO 1-a: Concrete Beam Facing of the Mechanically Stabilized Embankment on Interstate Highway 5 at Dunsmuir, California.

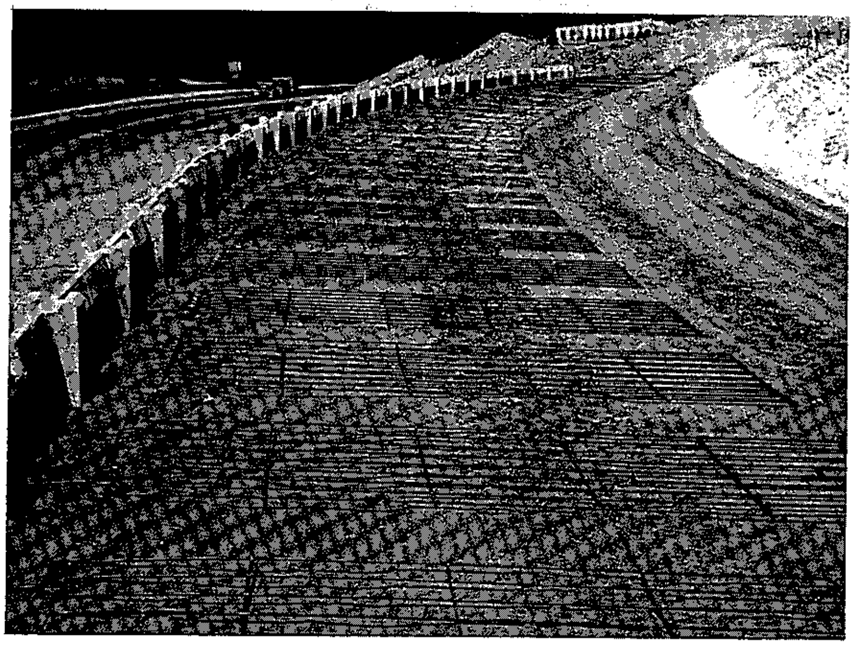


PHOTO 1-b: Reinforcing Bar-mesh in the Mechanically Stabilized Embankment on Interstate Highway 5 at Dunsmuir, California

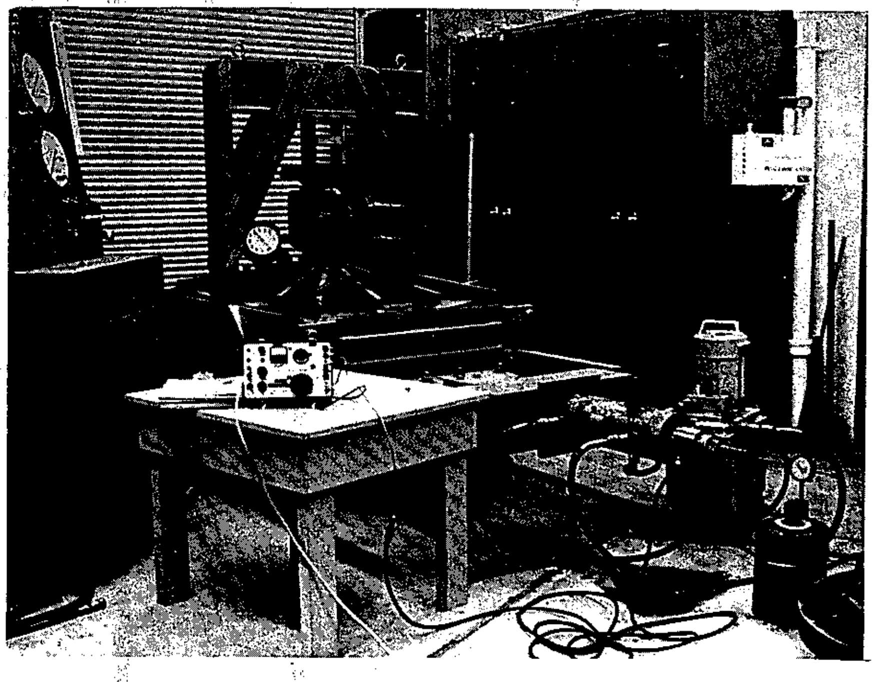


PHOTO 2: Laboratory pull test facility.

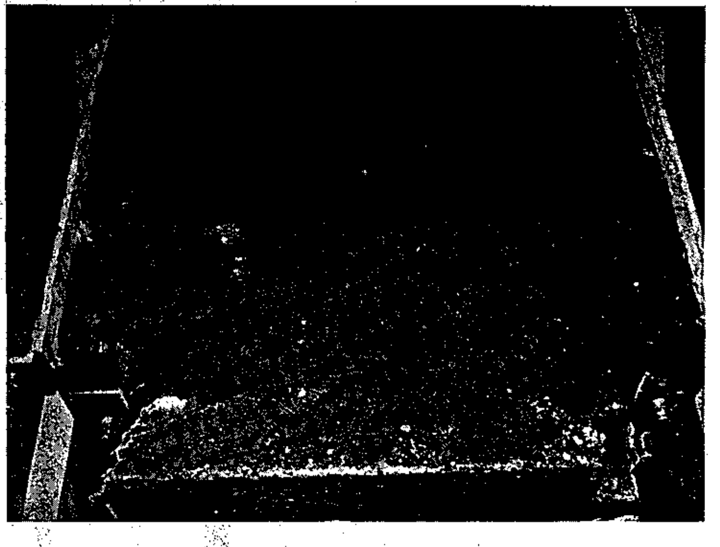


PHOTO 3: Cone-shaped failure mode exhibited in specimen with bar-mesh reinforcement.

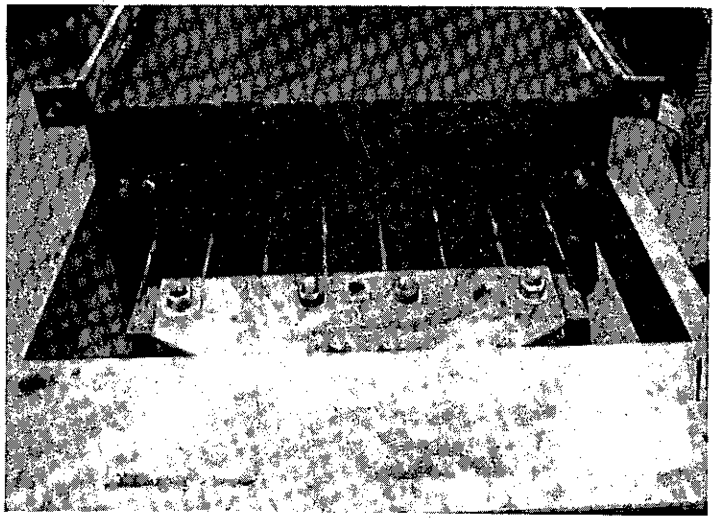


PHOTO 4: Failure Mode exhibited in specimen with 8-longitudinal bar (Test Run No. 5).

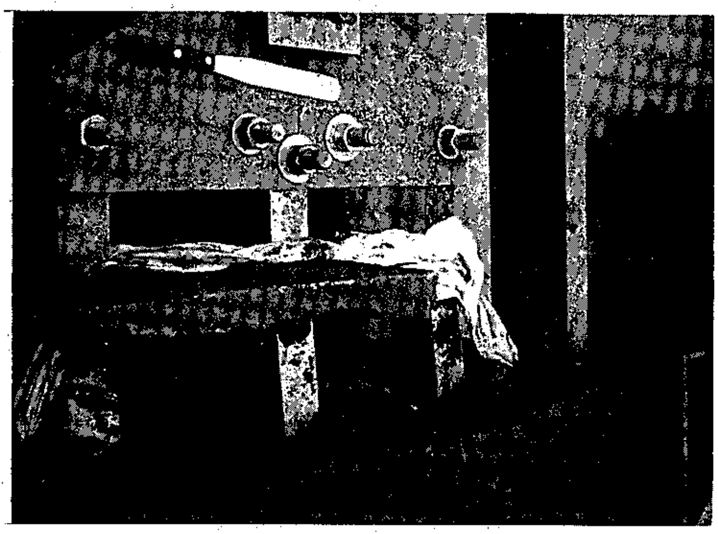


PHOTO 5: View of failure mode in specimen with three steel strips (Test Run No. 10).

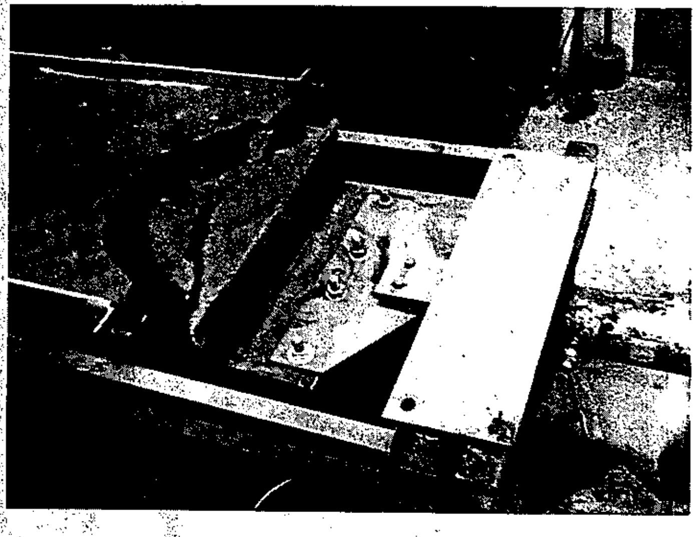


PHOTO 6: Cone-shaped failure mode exhibited in specimen with solid steel plate (Test Run No. 7).

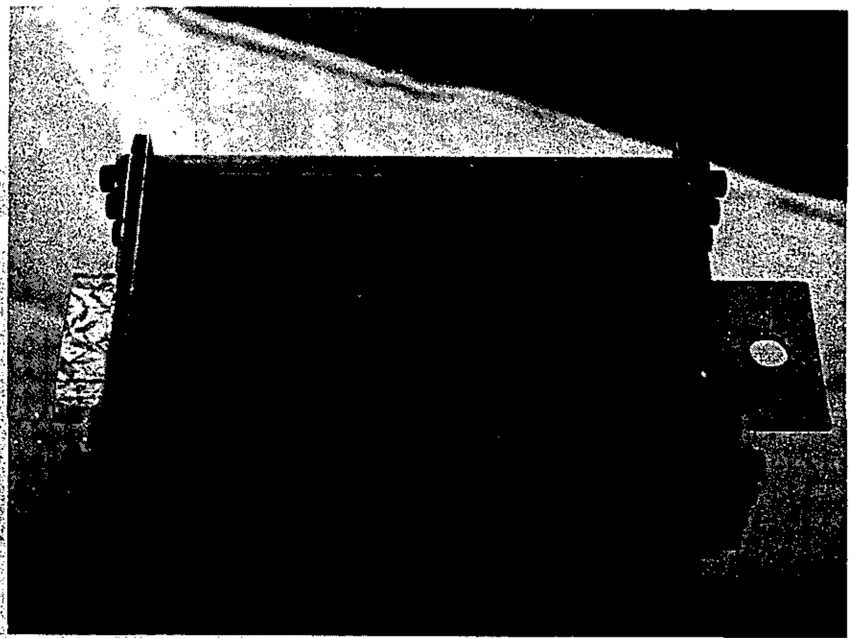


PHOTO 7: Shear box-skin friction tests.

NOTES:

1-KILONEWTON (KN) = 0.225 KILOPOUNDS (KIPS)
1-METER (M) = 0.394 INCHES

LEGEND

▲ — YIELD LOAD
● — PEAK LOAD
■ — RESIDUAL LOAD

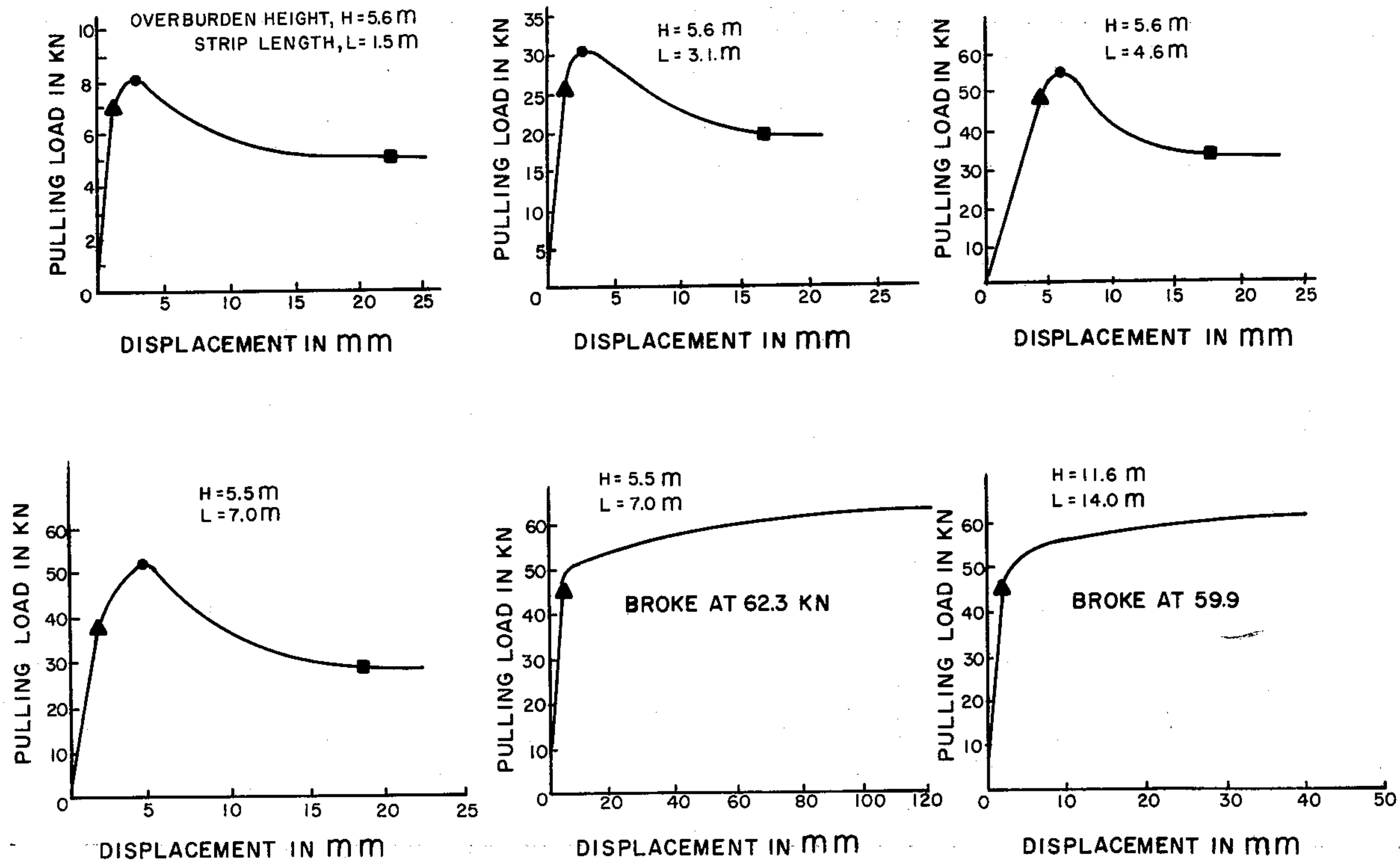
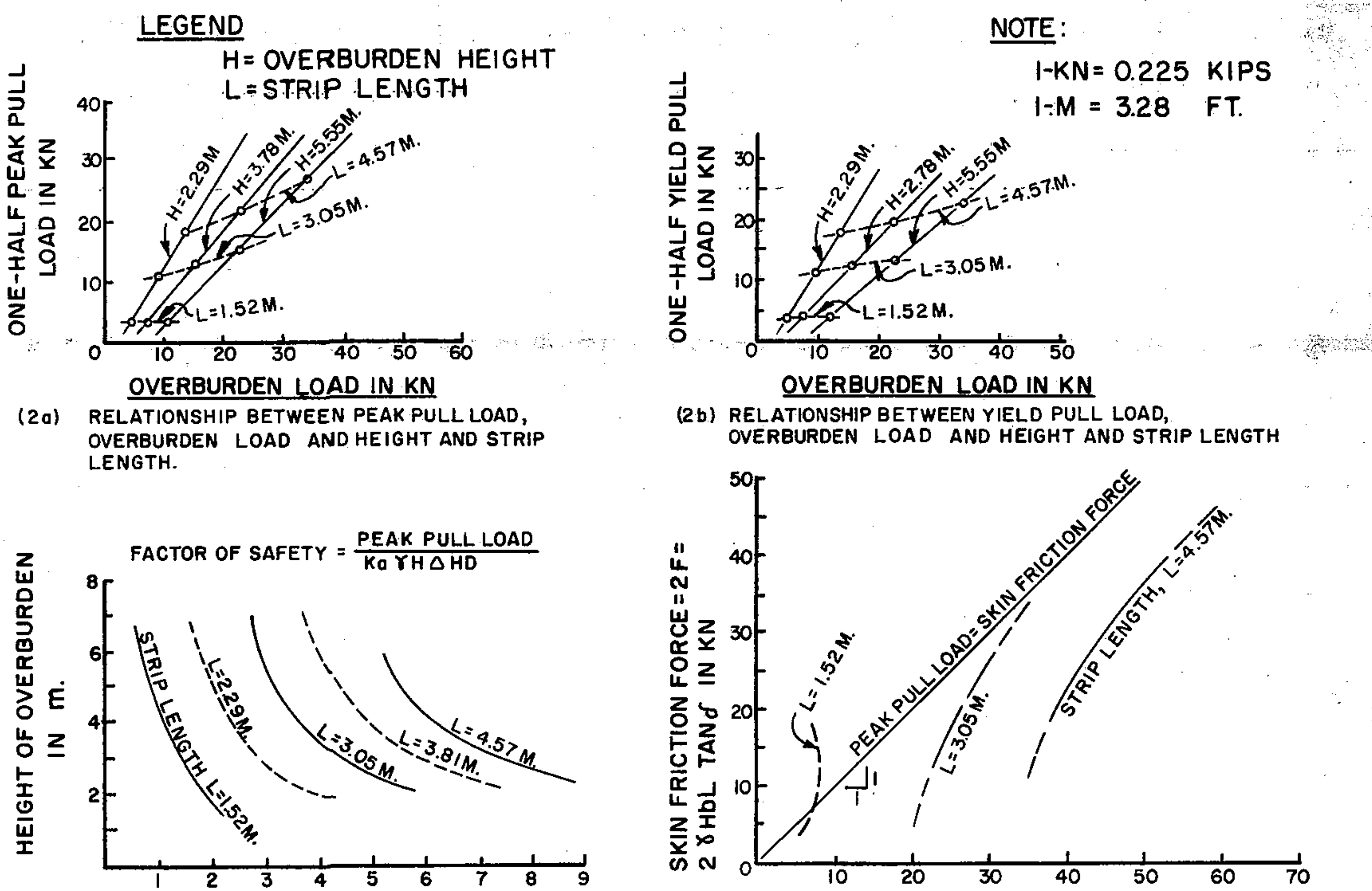


FIGURE 1 - LOAD DEFORMATION CURVES FROM FIELD PULL TESTS ON ROUTE 39



(2a) RELATIONSHIP BETWEEN PEAK PULL LOAD, OVERBURDEN LOAD AND HEIGHT AND STRIP LENGTH.

(2b) RELATIONSHIP BETWEEN YIELD PULL LOAD, OVERBURDEN LOAD AND HEIGHT AND STRIP LENGTH

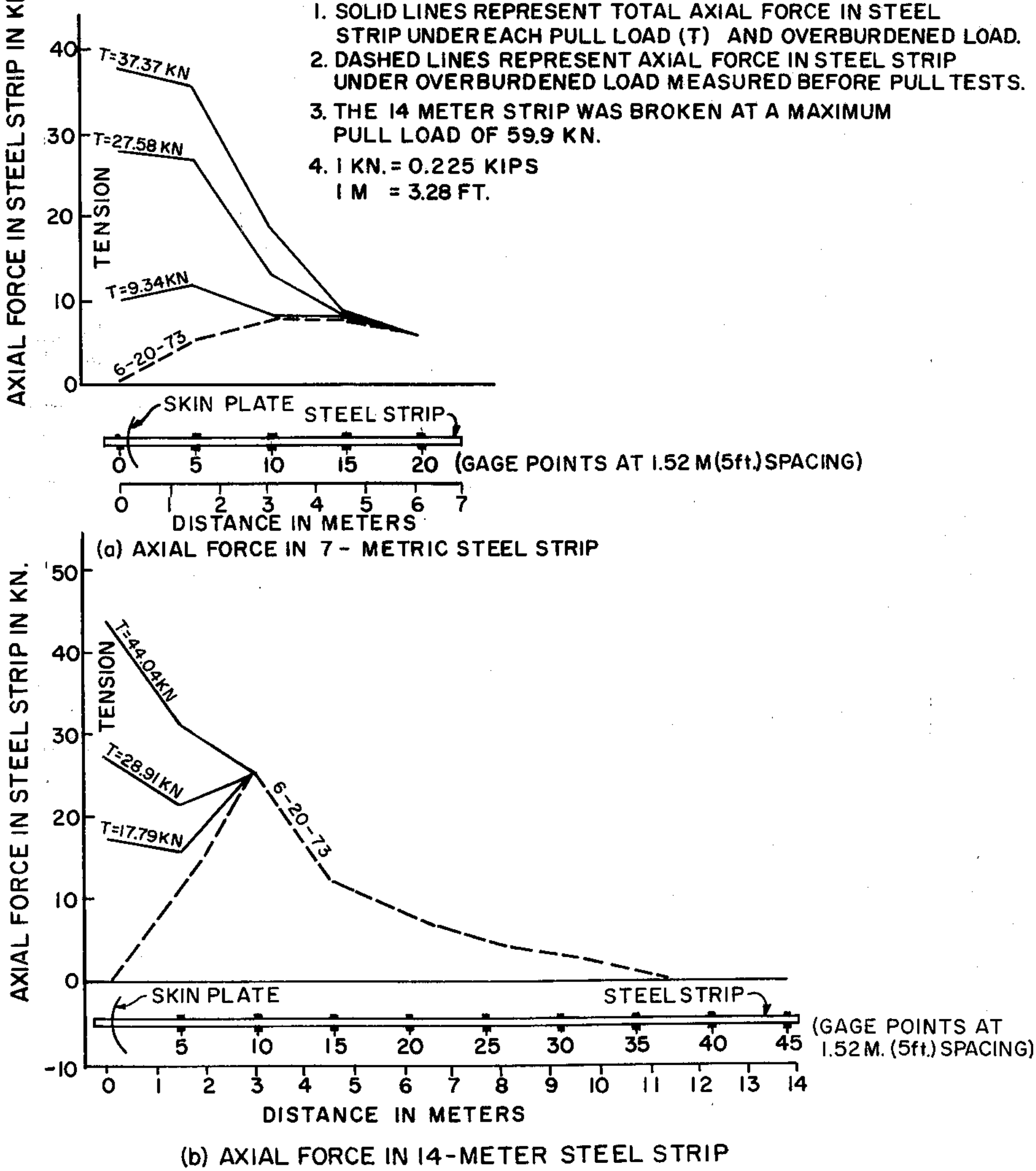
(2c) RELATIONSHIP BETWEEN OVERBURDEN HEIGHT, STRIP LENGTH AND FACTOR OF SAFETY

(2d) RELATIONSHIP BETWEEN PEAK PULL LOAD AND CALCULATED SKIN FRICTION FORCE

FIGURE 2- SUMMARY OF FIELD PULL TEST RESULTS

NOTE:

1. SOLID LINES REPRESENT TOTAL AXIAL FORCE IN STEEL STRIP UNDER EACH PULL LOAD (T) AND OVERBURDENED LOAD.
2. DASHED LINES REPRESENT AXIAL FORCE IN STEEL STRIP UNDER OVERBURDENED LOAD MEASURED BEFORE PULL TESTS.
3. THE 14 METER STRIP WAS BROKEN AT A MAXIMUM PULL LOAD OF 59.9 KN.
4. 1 KN. = 0.225 KIPS
   1 M = 3.28 FT.



(a) AXIAL FORCE IN 7 - METRIC STEEL STRIP

(b) AXIAL FORCE IN 14-METER STEEL STRIP

FIGURE 3-AXIAL FORCE IN DUMMY STRIPS

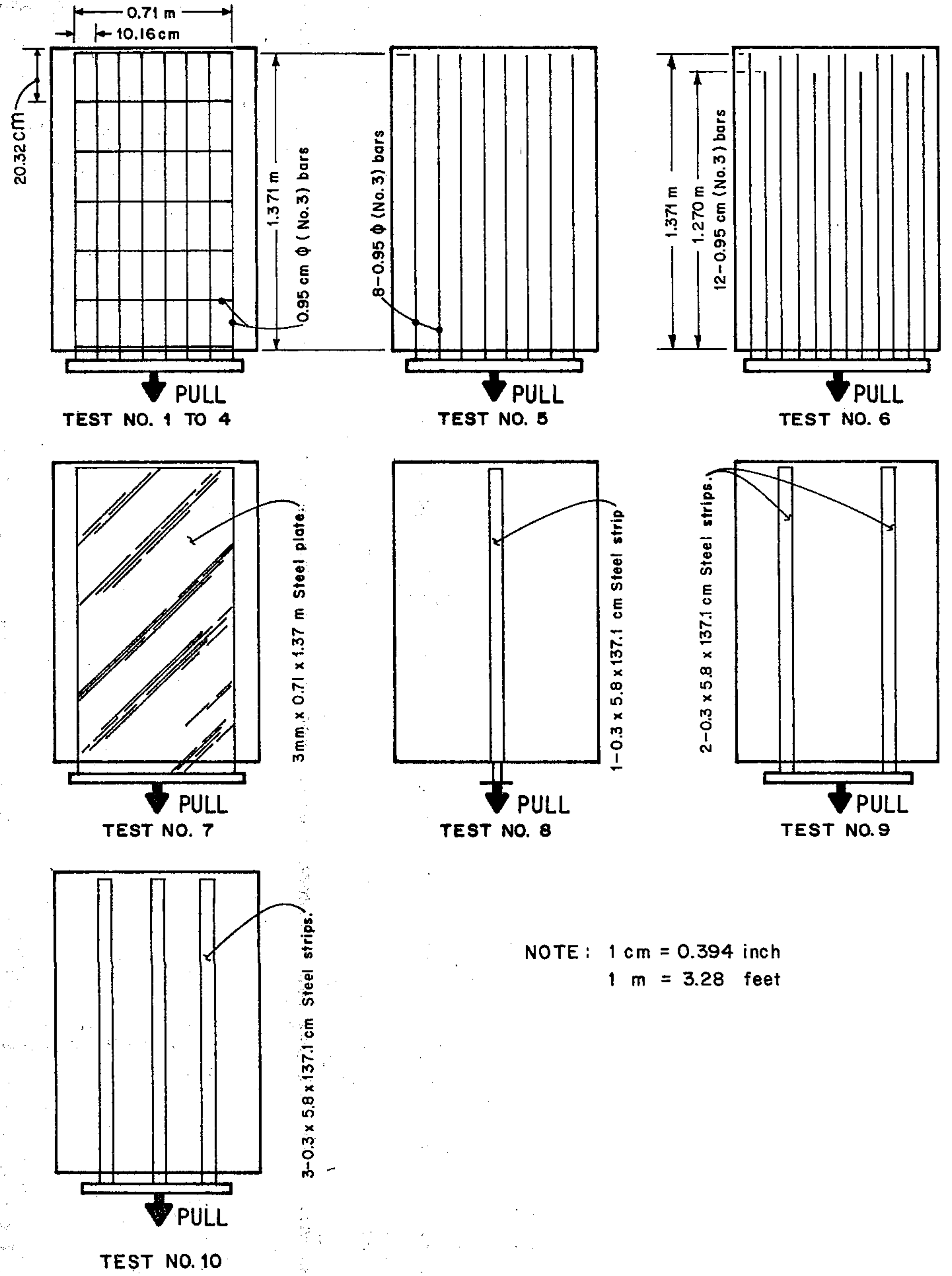


Figure 4.— TEST RUNS FOR EACH TYPE OF REINFORCEMENT

NOTE:

1 cm = 0.394 INCH
1 KPa = 0.145 PSI
(KPa = KILOPASCALS)

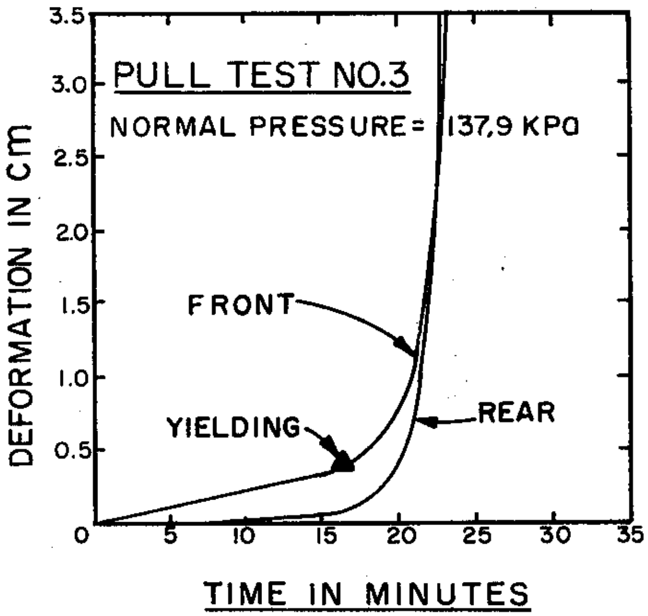


FIGURE 5 - TYPICAL TIME DEFORMATION CURVE FOR BAR MESH WITH 10.16 x 20.32 Cm OPENINGS EMBEDDED IN GRAVELLY SAND (SP) SOIL.

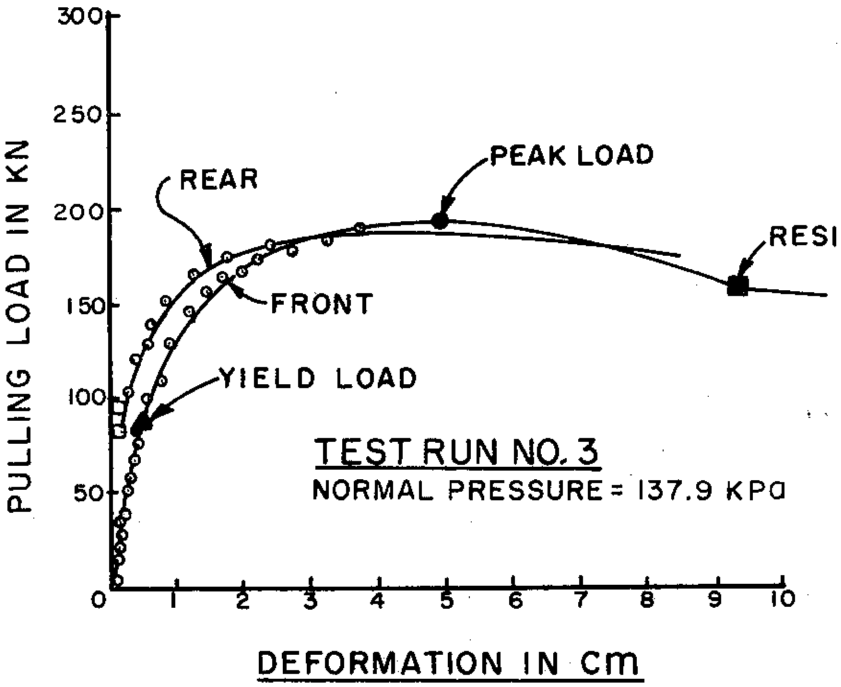


FIGURE 6— TYPICAL LOAD-DEFORMATION CURVE FOR BAR MESH WITH 10.16 x 20.32 Cm OPENING EMBEDDED IN GRAVELLY SAND (SP) SOIL.

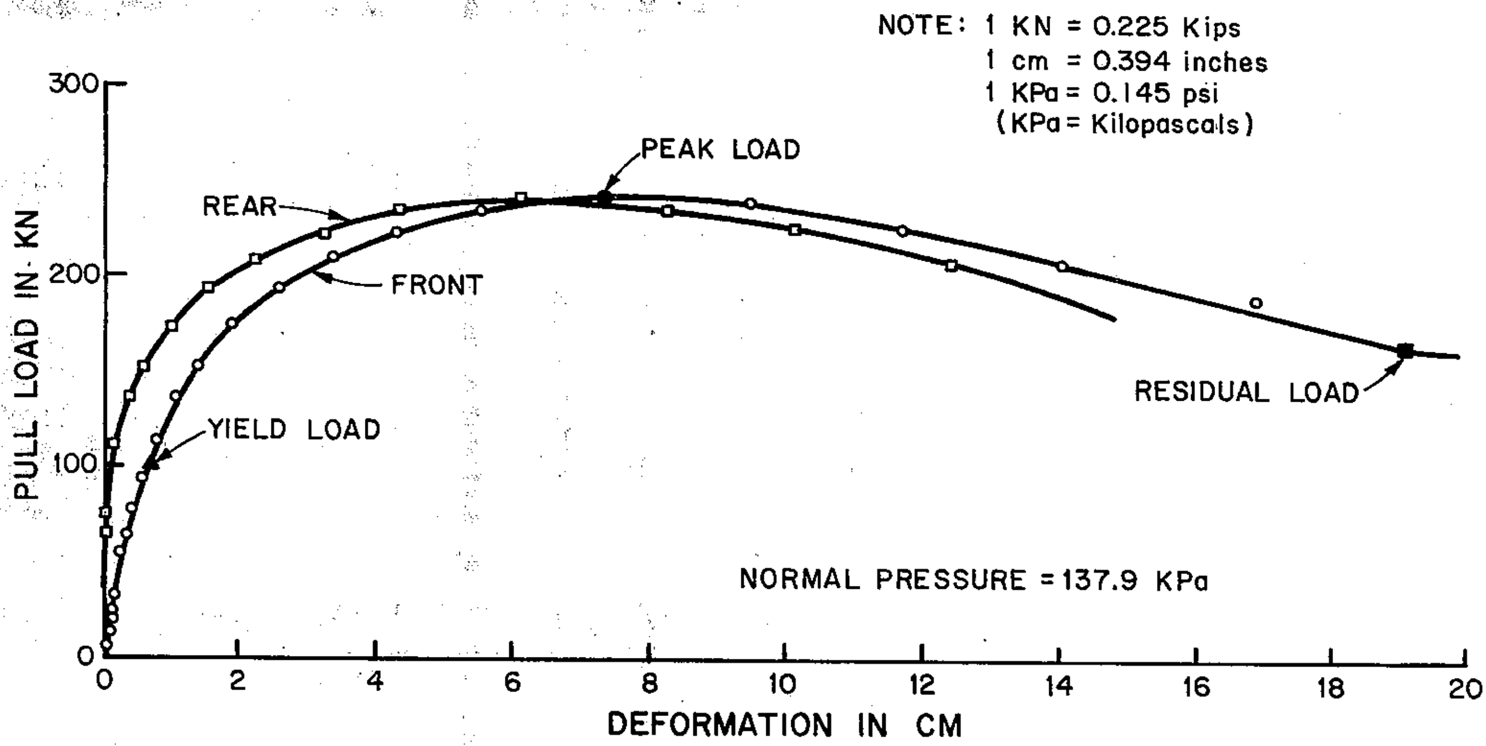


Figure 7a — TYPICAL LOAD — DEFORMATION CURVE FOR BAR MESH WITH 10.16 x 20.32 CM OPENING EMBEDDED IN SILTY CLAY WITH GRAVEL (CL) SOIL

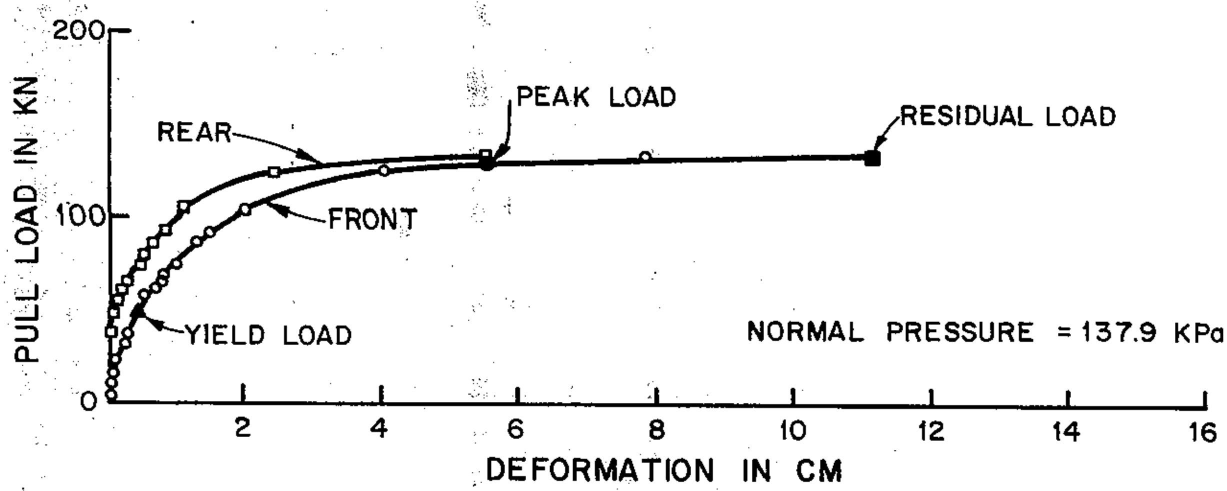


Figure 7b — TYPICAL LOAD — DEFORMATION CURVE FOR BAR MESH WITH 12.7 x 35.6 CM OPENING EMBEDDED IN SILTY CLAY WITH GRAVEL (CL) SOIL

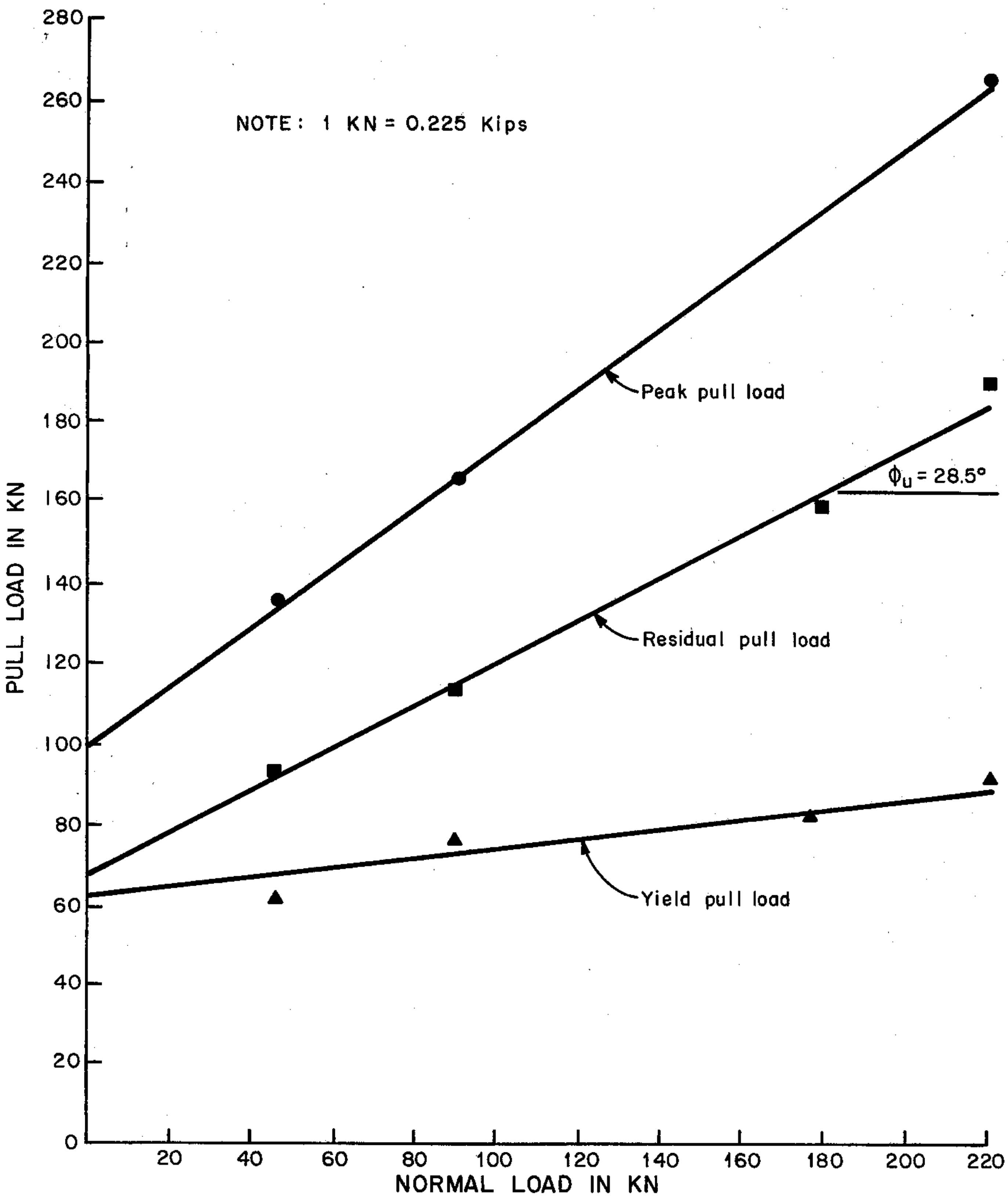


Figure 8—RELATIONSHIP BETWEEN NORMAL LOADS AND PULL LOADS FOR BAR MESH WITH 10.16 x 20.32 CM OPENING EMBEDDED IN GRAVELLY SAND (SP) SOIL