THERMAL EROSION OF CUT SLOPES IN ICE-RICH SOIL

FINAL REPORT

BY

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16. Abstract

Experience with cuts in ice-rich soil has shown that significant slope deformation and down-slope movement of soil can result as the frozen material along the slope face thaws. This report presents data from a test site constructed by Alyeska Pipeline Service Company in 1973 to evaluate the effect of various types of surface treatments on the performance of ice-rich cut slopes. The test site, located near Hess Creek, Alaska (approximately 95 miles north of Fairbanks), consisted of five test sections with the following surface treatments: 1) urea formaldehyde foam (4") over excelsior mulch, 2) two layers of excelsior mulch, 3) two layers of excelsior mulch over burlap or Dynel nylon fabric, 4) urea formaldehyde foam (2") or Dynel nylon fabric over sand, 5) untreated (bare soil).

The test sections were monitored during the 1973 and 1974 summer seasons. Longer term performance of the test sections was evaluated during two field investigations conducted during the 1983 summer season. Results from these two investigations and from 1973 and 1974 observations indicate that surface treatments with higher insulative properties (i.e., foam insulation) can reduce thermal erosion in ice-rich soil for a limited number of thaw seasons. Other surface treatments were relatively ineffective in reducing thermal erosion even during the first thaw season. Longer term stability in terms of soil displacement appears to be related to initial ice content of the frozen soil and shear strength of the thawing soil. All slopes at the Hess Creek test site appeared to have been stable in 1983. Vegetation was well established in most of the test sections.

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THERMAL EROSION STUDY
CUT SLOPES IN ICE-RICH SOIL

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1.0 SUMMARY

Experience with excavation of cut slopes in frozen ice-rich materials extends back in time to early development of the Fairbanks mining district in the 1920's and military construction of the Alaska Highway in the early 1940's. Unfortunately, technical documentation of ground conditions and slope performance is very limited or non-existent. Subsequent experience on major earthwork grading projects has resulted in significant emphasis being given to minimizing cuts in ice-rich frozen soil and, thus, evaluation and documentation of frozen soil cut slope performance in Alaska has been limited. This study addresses ice-rich cut slopes with regard to thermal degradation effect on slope deformation and/or stability and the performance of surface treatments selected with the intent of reducing the effect of natural seasonal thawing.

In 1969, major near vertical cuts were made in ice rich silts on the initial 56-mile Livengood to the Yukon River segment of the Dalton Highway. Performance of these near vertical cut slopes was monitored by Alyeska Pipeline Service Company (APSC) and the overall natural restabilization and revegetation of the thermally degraded slopes was considered to be satisfactory. Since similar performance could not be assured for other soil types and different organic cover and slope conditions, consideration was given to utilization of surface treatments on flattened slopes as a means for attaining similar restabilization and revegetation.

In 1973 a test site was constructed by APSC near Hess Creek on the Dalton Highway to evaluate the effect of various surface treatments in reducing thermal erosion on slopes cut at about 1.5:1 (horizontal to vertical) into ice-rich silt (APSC, 1974 and 1975). Five test sections were constructed using different surface treatments:

Section	Treatment
I	Urea formaldehyde foam (4") over one layer of excelsior mulch
II	Two layers of excelsior mulch
III	Two layers of excelsior mulch over burlap or "Dynel" nylon fabric sprayed with titanium dioxide.
IV	Urea formaldehyde foam (2") over "Dynel" nylon fabric over 6" sand layer
V	Untreated (bare soil)

Monitoring of test section performance was accomplished by APSC and their consultants during the summer seasons in 1973 and 1974. Slope elevation surveys, soil probes, temperature measurements and visual observations were made during these summer periods.

In 1983, ten years after test site construction, two field investigations were conducted for this project to evaluate longer term performance and general slope conditions at the Hess Creek Test Site. Based on these investigations, it appears that all slopes at the Hess Creek Test Site are stable at this time. However, significant slope deterioration - caused by large volume reduction and down-slope soil displacement - had occurred in most of the test section slopes since 1974. Revegetation was well established on slopes which had experienced significant soil displacements and consequent flattening of the slopes. Revegetation on steeper, less deformed slopes was not well established and, in some cases, non-existent.

From this study it appears that surface treatments which have higher insulation properties can reduce thermal erosion in ice-rich soil for a limited number of thaw seasons. Longer term stability (in terms of soil displacement) appears to be more related to the initial ice content of the soil and to the shear strength of the thawing soil. Excess pore water pressures due to thaw consolidation, while likely important for low compressible soils such as clay, apparently did not significantly affect stability for the low plastic silt encountered at the Hess Creek test site.

In comparing the performance of flattened treated cut slopes at the test site with that of the untreated restabilized near vertical cut slopes on the adjacent roadway, it appears that the long term end result is about the same. Advantages with treated flattened slopes appear to be associated with reducing the rate of thermal erosion and consequent lessening of siltation runoff conditions. This advantage, of course, is partially offset by higher first costs that may be involved with installation of a treated slope.

2.0 INTRODUCTION

Due to varying circumstances usually associated with route selection and geometric requirements, it sometimes becomes necessary to construct a road or highway through terrain underlain by perennially frozen soil (permafrost). Cuts required to achieve design grades in these permafrost areas can result in significant thaw of the frozen soil if adequate thermal protection is not applied to the cut-slope surface. A thawing cut slope may experience such conditions as; (1) deformations resulting from thaw settlement of the frozen ground, (2) erosion from surface runoff, (3) moderate downslope movement of thawed soil from sloughing or ablation, or (4) larger amounts of downslope movement of thawed soil due to landslides (flows, planar slides, or rotational slides).

The term "instability" is generally used to denote moderate or large downslope movement of explosed cut slopes (Condition Nos. 3 and 4 above). Primary concerns of thaw-induced slope instability are associated with potentially high maintenance costs, instability of a road or highway, possible damage to motor vehicles, loss or delays in vehicle passage, and unsightliness. Erosion (Condition No. 2), in addition to being unsightly, can result in additional maintenance costs related to cleaning out ditches and adverse environmental impacts resulting from increased turbidity in nearby rivers and streams. Settlement within a slope (Condition No. 1), in which little or no downslope movement of soil occurs, is generally not considered to be a concern for most highway projects.

Although their use has not been very extensive thus far, various construction and design techniques have been and can be utilized to limit or reduce mass movements within cuts made through ice-rich soils. These techniques have included:

1) Near vertical slopes with a widened ditch section to accommodated sloughing. Soil retention structures are normally necessary to prevent soil loss.

2) Cutting moderate slopes, 1.5:1 to 3:1, placing insulation, reflective, or other covering over the exposed slope surface to either reduce thaw penetration into underlying ice-rich soil or retard surface erosion while encouraging revegetation.

3) Cutting very flat slopes, such as 3:1 or flatter, normally without using protective covering. Thawing of lower angled slopes would result primarily in soil settlement with little or no lateral movement.

4) Buttressing exposed cut slopes with free-draining granular material to increase the effective normal stress along the potential failure plane. This may be done with or without insulation, depending on design requirements.

Specific subsurface soil conditions and design constraints tend to dictate which slope designs are most effective for a particular site. The use of surface coverings (Condition No. 2 above) is an attractive alternative in areas where maintenance may be limited, either by access or from a cost standpoint, or where low angle cut slopes (Condition No. 3 above) are not feasible. Ideally, covering the slope surface with an insulative material would limit thaw into the underlying permafrost, thereby reducing degradation of the cut-slope surface.

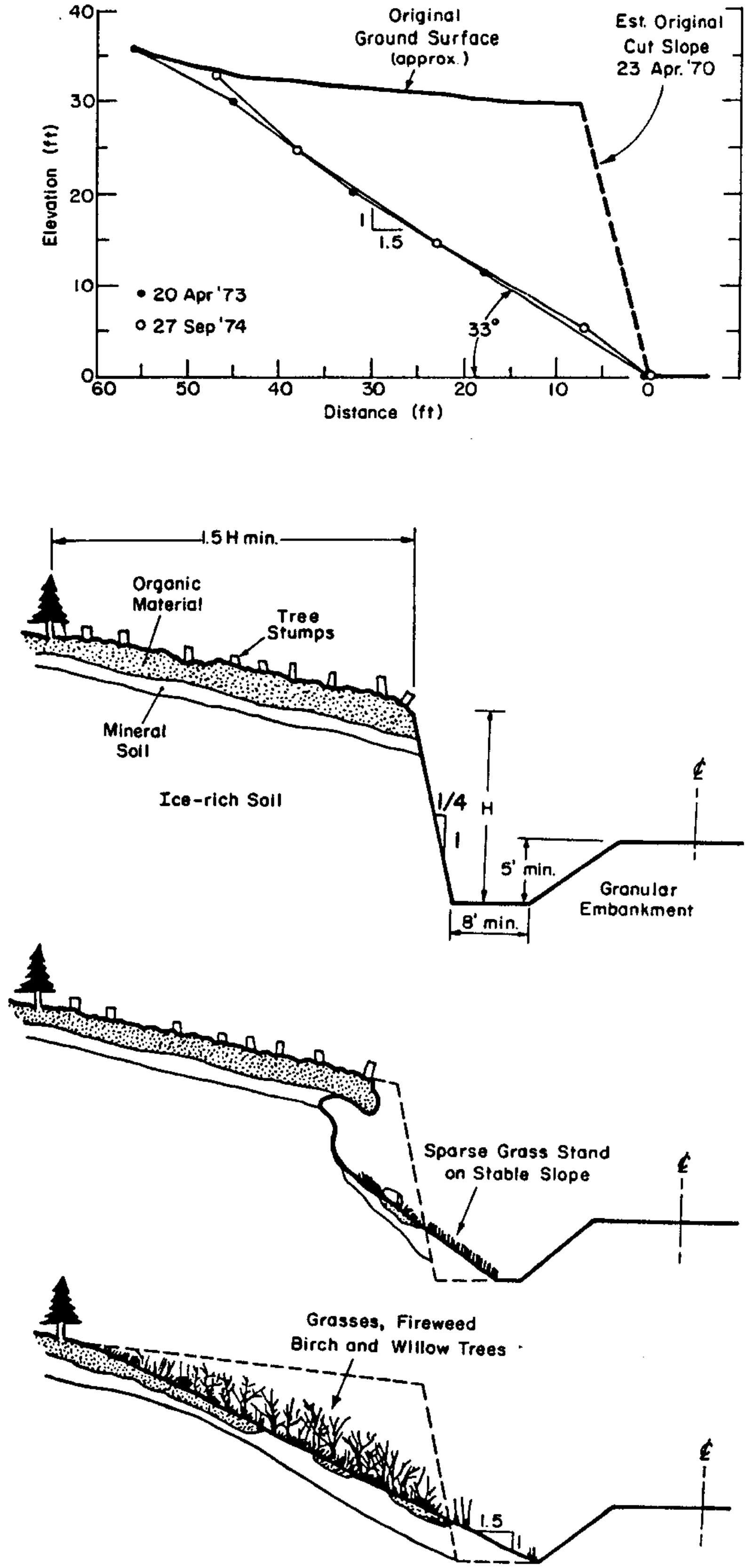
To date there has been only limited documentation with regard to the longer term performance of surface treatments on exposed ice-rich slopes. In 1969 the initial 56 mile segment of the North Slope haul road (Dalton Highway) from Livengood to the Yukon River was designed and constructed through ice-rich, fine-grained soils. Significant cuts in these ice rich soils were required. Based on experience with cuts made in similar ice-rich soils along the Alaska Highway, the design consultants recommended that near vertical cuts with extra wide ditch sections (to accommodate cut slope sloughing and erosion) be utilized. The anticipated cut slope restabilization and revegetation was well attained within several years. However, alternate design solutions that would possibly retard thaw degradation and thermal erosion during other construction operations in similar terrain were of interest to APSC.

To evaluate the effect of different surface treatments on cut-slope stability, a test site was constructed by APSC along the Dalton Highway near Hess Creek, Alaska. The test site, constructed in 1973, provides some insight as to the effect of various surface treatments on the performance of slopes cut into ice-rich fine-grained soil (APSC, 1974 and 1975).

Approximately 1000 feet north of this test section, Berg and Smith (1976) evaluated the performance of a near vertical cut constructed without any surface treatment in similar ice-rich material. As seen in Figure 1, significant deterioration of the slope surface was observed during the first thaw season . By the fifth or sixth thaw season, the slope had stabilized at a grade significantly less that the original cut. Vegetation was fairly well established at this time. It was anticipated by APSC that the performance of such cut slopes would be improved with the addition of surface treatments which would retard thermal erosion. Thus, it is of interest to examine the performance of the surface treatment at the nearby Hess Creek test site.

Monitoring of the test sections at the Hess Creek test site was accomplished by APSC during both 1973 and 1974, after which time monitoring was discontinued. Two field investigations were made in 1983 by R&M in order to evaluate the longer term performance of the surface treatments at the test site. The two field investigations were designed to obtain information regarding current slope geometry, soil properties, thaw depth profiles, vegetation growth on slopes, and document present condition of surface treatments and any unusual surface features along the slopes.

This report presents results from the two field investigations, an analysis of the results, and an evaluation of the performance of the ice-rich cut slopes and various surface treatments along with recommendations for treating cut slopes in similar ice-rich soils.



(a) *Slope profile measurements, left side of TAPS Road, mile 20.25 (from Berg and Smith 1976).*

(b) *Initial frozen cut profile.*

(c) *End of first thaw season. Slope is mostly unstable and very unsightly; ditch will require cleaning if massive ice is present.*

(d) *End of fifth or sixth thaw season. Slope stabilizes with reduced thaw and vegetation established. Free water from minimal thawing is used by plants whose root systems develop new organic material.*

Idealized development of stability in ice-rich cut (from Berg and Smith 1976).

FIGURE 1.

3.0 BACKGROUND

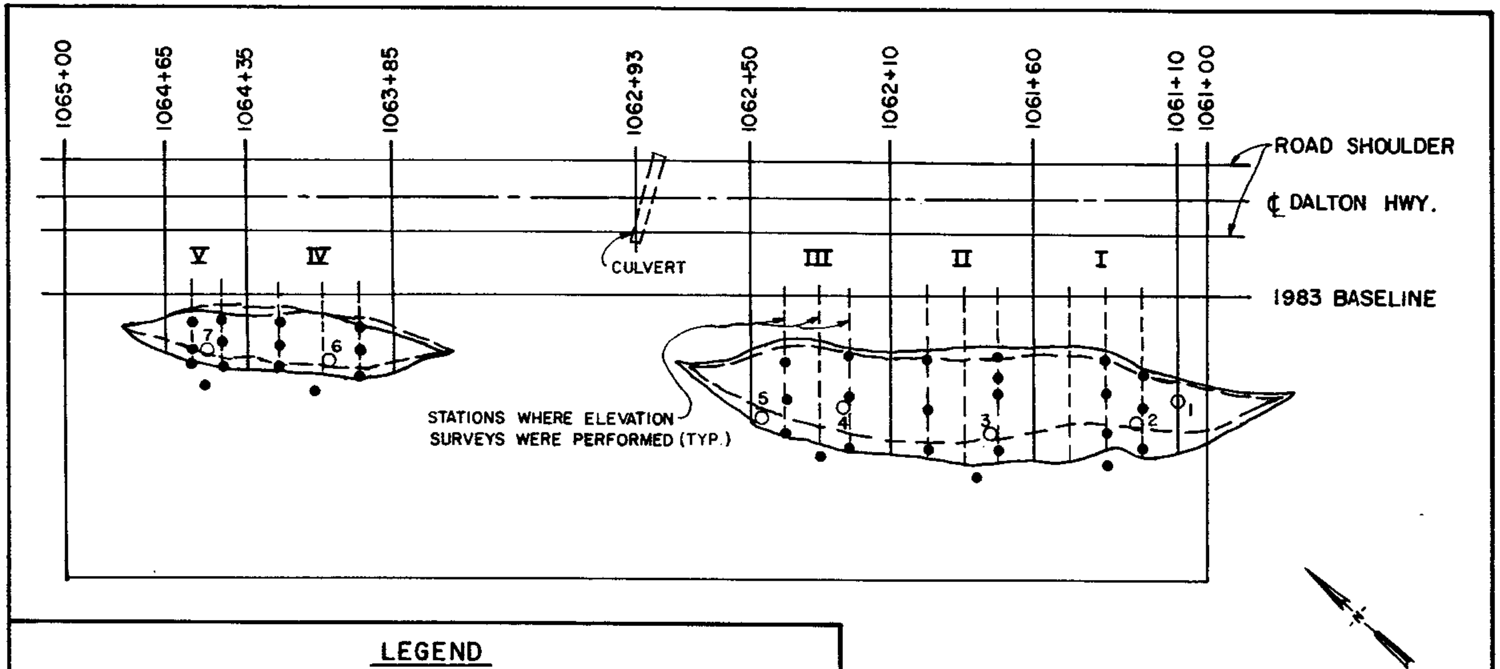
3.1 Hess Creek Test Site

In July 1973, Alyeska Pipeline Service Company began construction of a test program to study the effects of various surface treatments on the short term stability of slopes cut in ice-rich soil. The project site is located about three miles south of Hess creek and about 25 miles north of Livengood along the west side of the Dalton Highway between Station 1061+10 and Station 1064+65 based on the original 1969 construction survey. Five specific test sections were constructed on two separate slopes cut at approximately 1.5:1 (horizontal:vertical). A plan view of the site along with a summary of the surface treatment for each test section is presented in Figure 2.

3.2 Surface Treatments

Treatments on Sections I and IV were designed to retard slope thaw by reducing conductive heat transfer at the slope surface. The treated surface of Section I was intentionally left exposed to take advantage of the high reflectivity characteristics of the urea formaldehyde foam, while the surface of the Section IV insulation layer was treated with straw mulch and seed in an attempt to stimulate growth of vegetation. The sand filter beneath the Section IV "Dynel" nylon fabric material (manufactured by Dupont) was installed with the intent of intercepting melt water drainage and attempting to reduce potential excessive pore pressures that may occur in fine-grained soils at the thaw front during periods of high thaw rates.

The excelsior AMXCO "Curlex" blanket (manufactured by American Excelsior Company) placed on Section II was designed to reduce the heat transfer into cut slope soils by an evaporative or wicking action that reduces the seasonal heat reaching the cut face soils. In addition, the treatment was designed to enhance slope stabilization, at least surficially, by holding weaker, thawed soils on slopes.



LEGEND

———	CUT SLOPE LIMITS SURVEYED JUNE 1983
— — —	CUT SLOPE LIMITS SURVEYED AUGUST 1973
●	TEST PROBES DRIVEN SEPTEMBER 1983 (APPENDIX C)
○	TEST HOLES DRILLED SEPTEMBER 1973 (APPENDIX B)

SECTIONS	TREATMENT
I	4" UREA FORMALDEHYDE FOAM/1 LAYER EXCELSIOR
II	2 LAYERS EXCELSIOR BLANKET
III (EAST)	2 LAYERS EXCELSIOR / BURLAP & TITANIUM DIOXIDE
III (WEST)	2 LAYERS EXCELSIOR / DYNEL & TITANIUM DIOXIDE
IV	2" UREA FORMALDEHYDE FOAM / DYNEL / 6" SAND
V	CONTROL (UNTREATED)

R & M CONSULTANTS, INC.

HESS CREEK

THERMAL EROSION STUDY

LOCATION DIAGRAM

SCALE: 1" = 50'	DATE: 11-9-83	BY: DWM	FIGURE: 2

The treatment used on Section II, consisting of a layer of burlap fabric spray painted with a highly reflective titanium dioxide paint, was designed to lower the amount of heat reaching the cut surface soils by reflecting a portion of the solar radiation energy at the slope surface. The treatment on Section III failed repeatedly in the 1973 season, necessitating a considerable maintenance effort. Both Section II and III were subsequently covered in late 1973 with several layers of excelsior blanket that were carefully tacked to the slope in an attempt to minimize further slope surface instability.

3.3 Field Exploration Program - 1973 Season

A subsurface exploration program conducted in July 1973 provided subsurface soil information along the slopes prior to construction and included identification of soil profiles, moisture contents, and dry densities. Seven test holes were drilled in the area using a portable Haynes auger, tripod and split-spoon sampler. Logs of the test holes are presented in Appendix B. The approximate locations of the test holes are presented in Figure 2.

As seen from the 1973 test hole logs, the site was covered with a layer of organic material about 1.5 feet thick. This organic mat was underlain by ice-rich silt with visible ice varying from thin seams to massive ice lenses. In general, moisture content values and amounts of visible ice were lower in Test Hole No's. 5 and 6 than in the other five test holes. Average moisture contents within the top five feet of soil varied from 44% in Test Hole No. 6 to 138% in Test Hole No. 4.

3.4 Field Surveys - 1973 and 1974 Season

Following construction in 1973, field surveys were made along 14 cross sections - three within Test Section I through IV and two within Test Section V. Field surveys along these same cross sections were also conducted in 1974. Results from these field surveys, as summarized for August 1973, September 1974 and for June 1983 are presented in Figures A-1 through A-14.

4.0 FIELD PROGRAM (1983)

4.1 First Field Trip - June, 1983

Between June 27 and 29, 1983, R&M Consultants, Inc. conducted the first field investigation for the present study at the Hess Creek Test Site. The primary objective of this first field trip was to survey each of the original 14 cross sections in order to identify changes in slope elevation from the initial 1973 survey. In addition, surface treatment samples were obtained, photographs taken, and visual observations recorded.

Because of large slope movements, the 1973 reference baseline at the toe of slopes had been considerably distorted. A new baseline was therefore established approximately seven feet east of the 1973 baseline with stationing referenced to the center of the culvert (inlet side) at 1062+93 in accordance with the 1973 survey. A benchmark located about 21 feet northeast of the road at approximate Station 1065+50 was used as a datum point for the elevation survey. The reference elevation of this benchmark was 1650.0. Between 10 and 13 elevation points were measured along each of the selected 14 cross section stations, extending from the east toe of the Dalton Highway to the undisturbed area of the slope above the cut. Results from this elevation survey along with results from the 1973 and 1974 surveys, are presented in Figures A-1 through A-14.

Samples of urea formaldehyde foam insulation were obtained near the toe of slopes from Test Section I and IV. These samples were taken to the R&M laboratory for testing. Test results, including moisture content and dry density, are presented in Appendix D.

Photographs were taken of all test sections. Visual observations of distinctive features such as vegetation growth, deterioration of surface treatment and unusual slope geometry were recorded. Selected photographs are presented in Appendix E.

4.2 Second Field Trip - September, 1983

The second field trip for this study was conducted on September 28 and 29, 1983. The primary objectives of this second trip were to determine seasonal thaw depths and to obtain soil samples along the slopes. Other objectives included performing a second elevation survey, obtaining additional photographs, and recording visual observations made at the site.

Soil samples and seasonal thaw depths were obtained by driving a three-quarter inch (3/4") O.D. thin-walled probe into the ground using a 10 pound sledge hammer. The maximum depth of test probes ranged from 1.8 to 5.7 feet, the total length of the probe. Samples were obtained at selected intervals by extruding soil from the probe cavity. Test probes were made at three locations within the slope along two survey cross-section lines in each test section. In addition, one test probe was made in undisturbed natural ground at the top of each test section. The approximate locations of these test probes are shown in Figure 2.

Samples obtained from this investigation were visually identified and classified in the field by a soils engineer, sealed and returned to the R&M laboratory in Fairbanks for testing. Moisture content determinations were made on all samples. Logs of the test probes are presented in Appendix C. Results from laboratory tests are presented in Appendix D.

Typically, soils encountered during this site investigation consisted of unfrozen silt (active layer) overlying silt with varying amounts of ice. Some sand and gravel was encountered in several probes made at the toe of the slope. This granular material was probably placed during construction of the cut slope treatment sections or prior work on the Dalton Highway. Seasonal frost was encountered from about 4 to 12 inches below ground surface in all probes. Permafrost (perennially frozen soil) was encountered in most of the test probes at depths ranging from about 2.0 to 5.5 feet below ground surface. Permafrost was not encountered in six (6) probes. Groundwater was not encountered in any of the test probes, although the soil was wet in several of the samples taken below five feet where the probes were located near the toe of the slope. More detailed information regarding subsurface conditions at each test site is presented in the Appendix.

Based on interpretation of the test probe information, an estimated thaw depth profile was developed for each of the cross-sections where test probes were taken. These thaw depth profiles are presented in Figures A-1 through A-14. Also presented in these figures are logs of test probes (made in 1983) and test holes (drilled in 1973) showing moisture content values and zones of frozen and unfrozen soil. Zones of massive ice are also shown in the test hole logs.

A limited elevation survey was also made during this second field trip. The purpose of this survey was to determine whether any movements had occurred within the slopes following the June 1983 survey. Elevations were obtained at 10 to 14 positions along the cross-section line for each test section. The elevation points extended from the east edge of the road on up into the undisturbed area of the slope above the cut. Results from this second survey revealed no perceptible change in slope geometry or elevation from that measured in June, 1983. Because plots from the two surveys made in 1983 are nearly identical, the results from the second survey are not presented here.

Photographs were again obtained at all test sections. Selected photographs are presented in Appendix E.

5.0 TEST SECTION PERFORMANCE

A compilation of slope elevation data from the 1973, 1974 and 1983 field surveys, thaw depth profiles for September 1983, and test hole information are presented for each of the 14 cross sections in Appendix A. Also shown is the apparent change in unit volume of soil (cubic feet/lineal foot of slope) that occurred between 1973 and 1983. These approximated quantities were determined using a compensating polar planimeter. The following evaluation of each test section is based on; 1) the information presented in Figures A-1 through A-14, 2) reports by APSC (1974 and 1975), 3) photos presented in Appendix E, and 4) general field observations recorded in 1983.

5.1 Test Section I (Urea Formaldehyde over Excelsior Blanket)

This test section performed well during the early part of the first summer season (1973) with very little soil or water movement occurring on the slope. During the second summer season (1974), large thaw-induced depressions underlying the insulation layer had developed and subsequently large down-slope movements of thawed soil were observed. These slope failures destroyed the insulation cover and, by September 1974, only about thirty percent (30%) of the insulation remained intact on the slope face. Thaw depths, averaging 2.3 feet, were measured during August and September, 1974.

The 1983 elevation profile of the slope presented in Figures A-1 and A-3 shows a marked increase in degradation had occured along the slope face since 1974. The slope surface was very irregular with many scarps. Remnants of the insulation were visible over only about ten percent (10%) of the area. Much of the insulation that was visible was broken and covered with moss and lichen. The high moisture content value of 83% by weight (Appendix D) indicates the insulation had absorbed a considerable amount of moisture over the previous ten (10) years.

Vegetation on the slope face of Test Section I consisted primarily of grass and a dense growth of willow trees (approximately 6 - 10 feet in height).

Occasional small spruce trees were located near the bottom and middle of the slope while wild rose bushes were growing near the top of the slope. This vegetation appeared to be well established, especially on the flatter portion of the slope. Brush lines had to be cut to perform the June 1983 survey.

The September 1983 thaw depths presented in Figures A-1 and A-2, range from 1.7 feet in the undisturbed area at the top of the slope to about 4.5 feet at the toe. This represented an increase in observed thaw depth of about 1.5 feet from those observed in 1974.

5.2 **Test Section II (Excelsior Blanket)**

This slope experienced substantial soil movement during the 1973 summer season, especially near the toe, and continual maintenance efforts were required to preserve the initial treatment configuration. Much of the failed material flowed into the ditch throughout the 1973 summer season. Deterioration of the slope treatment also continued through the 1974 summer season and by September only fifty percent (50%) of the slope was covered with the excelsior blanket. A vertical scarp had formed at the top of the slope. The amount of sloughing and slope movements observed throughout this second thaw season was similar to that observed in Test Section I. Thaw depths, averaging 2.6 feet, were measured on September 12, 1974.

As seen from Figures A-4 through A-6, the 1983 slope configuration had changed significantly from that measured in 1974. The 1983 slope angle was very high at the top while the central portion of the slope in this test section had a more gentle, almost flat grade which transitioned to a steeper grade near the toe of the slope. Very little of the excelsior blanket could be identified due to degradation of the material and vegetation growth. The topography of the slope in Test Section II was irregular with many scarps and small fracture zones.

The current vegetation within Test Section II was almost indistinguishable from that in Test Section I. Vegetation consisted of grass and areas having a dense growth of willow trees. Survey brush lines also had to be cut through vegetation in this test section. Some small localized areas were observed to be either bare or only sparsely covered with grass.

Thaw depths measured in 1983 ranged from three feet in Test Hole II-3-2 to about 4.7 feet in Test Hole II-3-3. Deeper thaw depths generally occurred in the less vegetated areas near the top of the slope.

5.3 Test Section III (Excelsior Blanket over Burlap and Dynel)

The southeast portion of the test section was originally covered with burlap and sprayed with titanium dioxide. Dynel nylon fabric sprayed with titanium dioxide was originally placed on the northwest portion of the test section. Covering for this test section reportedly became soaked and distorted with mud flows during construction. Two layers of excelsior were subsequently placed over the material. The slope became badly eroded, mass movement of soil in this test section was observed to be greater than in any of the other four test sections during 1973. Material from the slope was transported up to 15 feet from the toe of the slope. During the 1974 thaw season, the slope in this test section settled noticeably but very little additional mass movement of soil was observed. The excelsior covered fabric remained substantially intact throughout the summer. The 1974 slope profile indicated some sloughing continued to occur below the burlap and dynel fabric. Thaw depths measured during 1974 ranged from about three feet at the toe to over five feet near the top of the slope.

The 1983 slope configuration appeared to be somewhat less distorted than that observed in 1974. The only visible abrupt feature along the slope surface was a step change in elevation at the toe, apparently the result of a previously failed soil mass. Several isolated depressions one to two feet deep were observed in the soil beneath the surface covering. Large areas of excelsior underlain by either burlap (southeast) or dynel fabric (northwest) were observed on the slope, primarily in the northwest part of this test section. The excelsior appeared to be deteriorated.

The amount of vegetation along the slope in this test section was considerably less than in Test Section I or II. The southeast portion of Test Section III was vegetated with thin grass and scattered willow trees six to ten feet in height. The vegetation on the northwest portion of the test section consisted primarily of patches of grass with a few alders four to eight feet in height near the top of the slope.

The thaw depths measured in this test section (Figures A-7 and A-9) were significantly deeper than those measured in Test Sections I or II. Thaw depths typically ranged from 3.5 feet to over 5.5 feet. Deeper thaw penetrations occurred near the toe of the slope. There was no significant difference in thaw depth profiles between the southeast (Figure A-7) and northwest (Figure A-9) areas of the test sections.

5.4 Test Section IV (Urea Formaldehyde Foam over Dynel Fabric over Sand Filter)

As seen in Figures A-10 through A-12, the amount of mass movement and/or soil settlement was significantly less that that observed in the other four test sections. Very little evidence of soil movements, soil sloughing or meltwater release was observed during 1973 or 1974. Thaw depths measured in 1974 ranged from two to three feet below the top of surface treatment. It appears that most of the change in elevation that occurred between 1973 and 1983 was the result of thaw settlement. Much of the foam insulation was intact in 1974; at least 70% of the slope face was still covered with the Urea foam insulation in 1983. Several large voids approximately six to twelve inches (6-12") deep were observed in 1983 below the dynel along the slope surface. The surficial sand layer placed during construction was generally intact; however, some movement of the sand may have occurred. The insulation that remained was generally broken into smaller pieces (approximately twelve inches across). Some of the insulation was covered with moss. Tension cracks, several inches wide and two to three feet (2-3') long, were observed near the toe and top of the slope in this test section.

As seen in photographs in Appendix E, the slope face in Test Section IV was virtually treeless. Approximately sixty to seventy percent (60-70%) of the slope surface was still covered with urea formaldehyde foam insulation or dynel fabric that has inhibited revegetation. The remaining slope surface was covered with vegetation consisting primarily of fireweed and grass.

The 1983 thaw depths shown in Figures A-10 and A-12 are significantly deeper than those in the other four test sections, ranging from three feet in the undisturbed area at the top of the slope to over 5.5 feet near the toe. The average thaw depth is about five feet (5'). It is expected that the low moisture content values and the lack of vegetation contributed to deeper thaw depths. These large seasonal thaw penetrations also suggest that the insulative properties of the urea formaldehyde foam have significantly deteriorated. This is substantiated from laboratory test results of this material (Appendix D). A moisture content of ninety-one percent (91%), which is considered to be very high for insulation foam, was obtained for a sample of insulation from this test section. Thaw depths were generally deeper at the toe and shallower at the top of the slope.

5.5 Test Section V (Control, Untreated)

During the 1973 thaw season, the slope in Test Section V experienced considerable caving and sloughing. The material, however, was not transported as far from the toe of the slope as was the failed soil in Test Sections I and II. By the end of the summer, a vertical scarp had developed near the top of the slope. No caving, sloughing, or hydraulic erosion of soil was observed during the second thaw season. Vegetation in the form of very sparse grass and moss appeared on the slope surface during 1974. Thaw depths of three to four feet (3-4') or greater were measured at the end of the 1974 summer season.

Some additional settlement had occurred since 1974 as shown in Figures A-13 and A-14. In 1983 there appeared to be two distinct grades within this slope; the grade of the upper portion was approximately twice that of the grade of the lower portion of the slope. Irregular topography, including large scarps two to three feet (2-3') high and sharp elevation changes, was evident throughout the test section particularly near the top of the slope. Occasional depressions were observed within the localized flat areas of the slope.

Vegetation along the southeastern portion of the test section in 1983 consisted primarily of grass with thinly spaced willow trees. The spacing of willow trees became more dense within the northwest portion of the site.

Thaw depths along this slope were slightly less than those measured in Test Section IV, ranging from 3.5 feet near the top of the slope to over 5.5 feet near the toe. The thaw depths along Section V-1 (Figure A-13) were generally deeper than those encountered along Section V-2 (Figure A-14).

6.0 DISCUSSION

6.1 Theoretical Background

The behavior of a thawing cut-slope is influenced by a number of factors - these include soil type, shear strength of the thawed soil, slope angle, initial ice content and distribution, local precipitation and drainage conditions, and rate and depth of thaw penetration. Depending on the particular set of conditions, an initially frozen cut slope may experience very little to somewhat extreme deformations during and after the thawing process. For purposes of discussion, the types of behavior exhibited by a thawing cut-slope are grouped into three general categories: (1) settlement, (2) surface erosion, (3) slope instability.

6.2 Settlement

As a frozen slope thaw, the ice that binds the soil particles together melts and (1) the volume of the soil mass decreases (relative to the initial frozen volume), and (2) the strength of the soil decreases. If the shear strength of the thawed soil is sufficient to resist downslope movements, cut slope instability will not occur. However, significant volume reductions and corresponding surface deformations can result from thaw-induced settlement in ice-rich soil. In addition, settlement within a thawing cut-slope can continue after slope instability (downslope movement of soil) has occurred. Typically, surface deformations are nonuniform (i.e., undulating surface) due to the variability of the initial ice content in the soil. Settlements along cut-slopes are generally not a primary concern for highway or road projects. However, thaw-induced settlements below a road or highway constructed through ice-rich soil may adversely effect the road structure and related drainage provisions.

6.3 Erosion

Erosion along a slope cut into either frozen or unfrozen soil occurs when surface runoff washes near-surface soil down slope. As the erosion process continues, rilling and subsequent drainage channels can develop along the slope where local drainage becomes concentrated. Severe surface erosion can lead to unsightly conditions, increased maintenance in order to remove soil deposited in the road ditch, and a potential increase in turbidity of nearby streams or rivers. Gullies created by surface erosion or crack openings resulting from ice lens melt can, in some cases, increase the stability of a slope by dissipating pore pressures below the slope surface (Wang, et.al. 1977). Erosion is typically more common in fine-grained soils although significant erosion gullies can develop in sand. Erosion can occur prior to, during, and after the onset of slope instability (described below).

6.4 Slope Instability

A serious concern related to cut-slopes in frozen soil is slope instability. Slope instability is a generalized term to describe all forms of downslope movement involving moderate to large volumes of soil. Various types of instability within a thawing slope include sloughing, flows, planar slide, and rotational slides. The term "thermal erosion" is sometimes used to describe the general process of thawing within the slope surface and subsequent transport of soil downslope. The primary differences between the various forms of slope instability are; (1) the volume of soil displaced, (2) physical form of the failure, and (3) rate of soil movement. Sloughing, and flows typically involve smaller quantities of soil over a longer period of time than planar and rotational slides, although the volume of soil involved with some flows can be significant (McRoberts and Morgenstern, 1974). A schematic diagram is presented in Figure 3 showing typical physical appearances of the various types of cut-slope instability.

The speed at which a failed mass of soil moves down a slope depends primarily on slope angle, shear strength of the failed soil, and water content of the failed soil. Some types of failure, such as sloughing and flows, occur during the thawing process. Planar and rotational slides generally do not occur until thaw penetration reaches a depth at which the shear strength of the soil cannot support the weight of the thawed soil mass.

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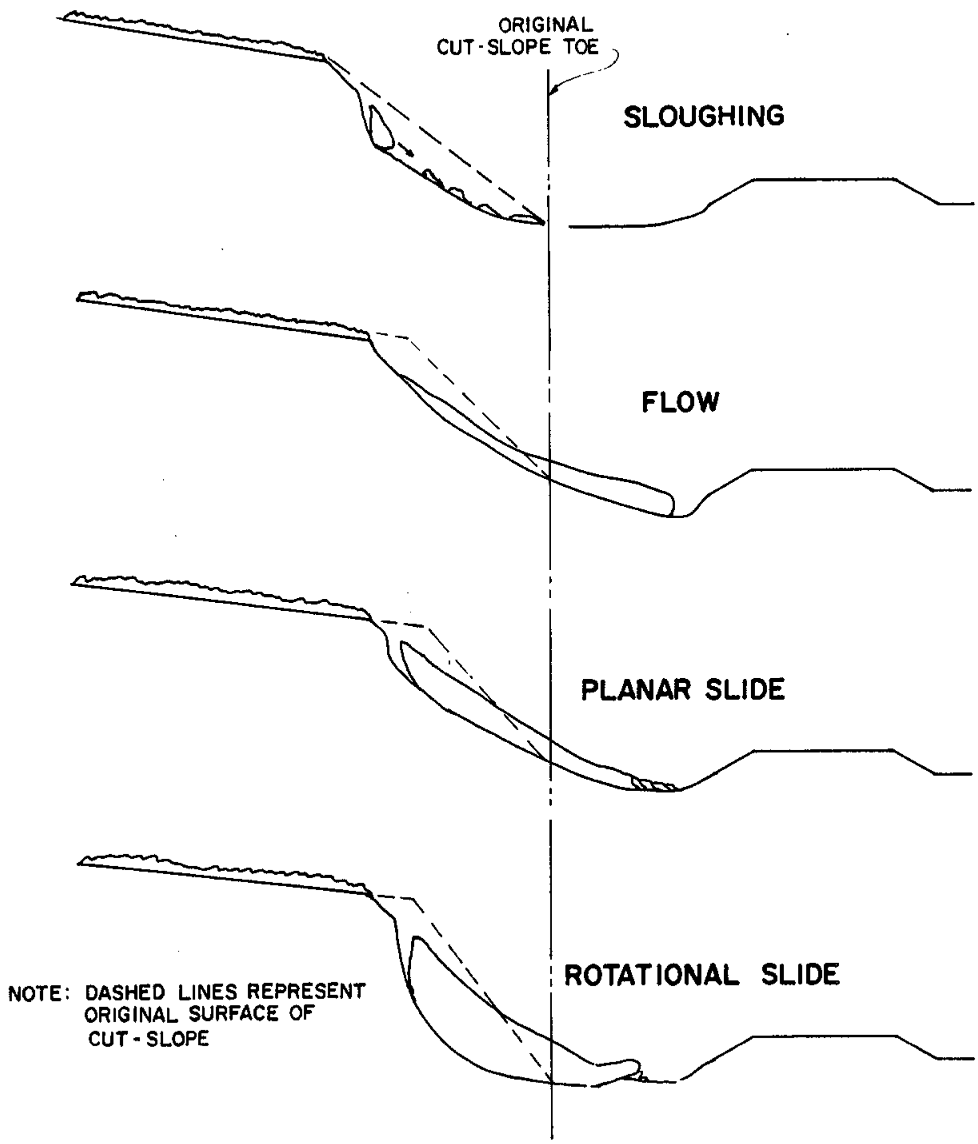
FIGURE 3
TYPES OF CUT-SLOPE INSTABILITY



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3-28-84	NONE	DWM	JWR	312163	3

Potential concerns related to instability of cut-slopes include: increased maintenance related to removal of failed soil from the road ditch and road surface, damage to the road, damage to motor vehicles, potential harm to persons involved in an accident caused by debris on the road surface, increased turbidity of nearby streams or rivers, unsightliness and cost of delays in traffic.

Instability of a thawing or thawed cut-slope results from the reduction or loss of shear strength of the thawed soil The degree of instability is usually expressed in terms of factor of safety (F.S.). Factor of safety can be defined as the ratio of forces that resist movement of a soil mass to forces that cause movement. When the F.S. is slightly above one, the slope is considered to be marginally stable; when the F.S. is less than one, the slope is considered to be unstable for defined conditions. This method of analyzing forces of a soil mass is termed "limit equilibrium" analysis. It provides a reasonable estimate of the level of stability of a slope when used in conjunction with representative soil properties, realistic field conditions and appropriate experience. Nevertheless, because of the difficulty in determining the appropriate soil properties and field conditions, factor of safety cannot be predicted with certitude and must be considered somewhat uncertain (Vita, 1983). The quantitative result of limit equilibrium analysis is expressed in terms of factor of safety. No quantitative information is obtained pertaining to: (1) displacement (amount of soil or slope movement), (2) type of failure (sloughing, flow, planar, etc.), or (3) rate of failure. Still, limit equilibrium analysis is useful in evaluating the expected performance of alternative cut-slope designs. The following discussion is directed to limit equilibrium analysis of thawing cut-slopes.

For the conditions where the depth of thaw is small compared to the length of slope, as is the case for a thawing cut-slope during the first several thaw seasons, it is appropriate to use the "infinite slope" method of limit equilibrium analysis. As thaw progresses deeper into a slope, other analytical techniques such as the method of slices (Bishop, 1955; Morgenstern and Price, 1965) may then be more appropriate to assess longer term stability.

6.4.1 Infinite Slope Analysis

Referring to infinite slope conditions, such as illustrated in Figure 4, the forces which "cause" soil movement result from the bulk weight of the thawed soil and any surcharge (if applicable). "Resisting" forces result from shearing resistance between soil particles. From classical soil mechanics the shear strength of the soil along a potential failure plane parallel to the slope surface may be expressed by:

$$\tau = \frac{c'}{F.S.} \left(b\right) \frac{1}{\cos\beta} + (N - UL) \frac{\tan \phi'}{F.S.} \qquad (6.1)$$

where:
- τ = shearing strength of soil along the potential failure plane (X)
- c' = effective cohesion of unfrozen soil
- ϕ' = effective angle of friction of unfrozen soil
- N = normal stress along failure plane
- U = total pore pressures at the potential failure plane (X)
- b = unit horizontal width of soil element
- L = length of soil element along ground surface
- β = slope angle

By introducing several relationships and rearranging terms, equation (6.1) becomes:

$$F.S. = \frac{c'}{(\gamma X + \gamma o D) \sin\beta \cos\beta} + \left[1 - \frac{\gamma w X Z + Us + Uo}{(\gamma X + \gamma o D)}\right] \frac{\tan \phi'}{\tan \beta} \qquad (6.2)$$

where:
- γ = bulk density of thawed soil
- γo = bulk density of free-draining surcharge
- w = density of water
- X = thaw depth
- D = thickness of surcharge
- Z = ratio of hydrostatic pore pressure (expressed in terms of height of water) to thaw depth = h/X
- Us = excess pore pressure during thaw due to self weight of soil
- Uo = excess pore pressure during thaw due to surcharge loading

It can be shown that when Us and Uo are zero (i.e., no excess pore pressures during thaw) equation 6.2 becomes identical to the conventional "infinite slope" stability equation for unfrozen slopes.

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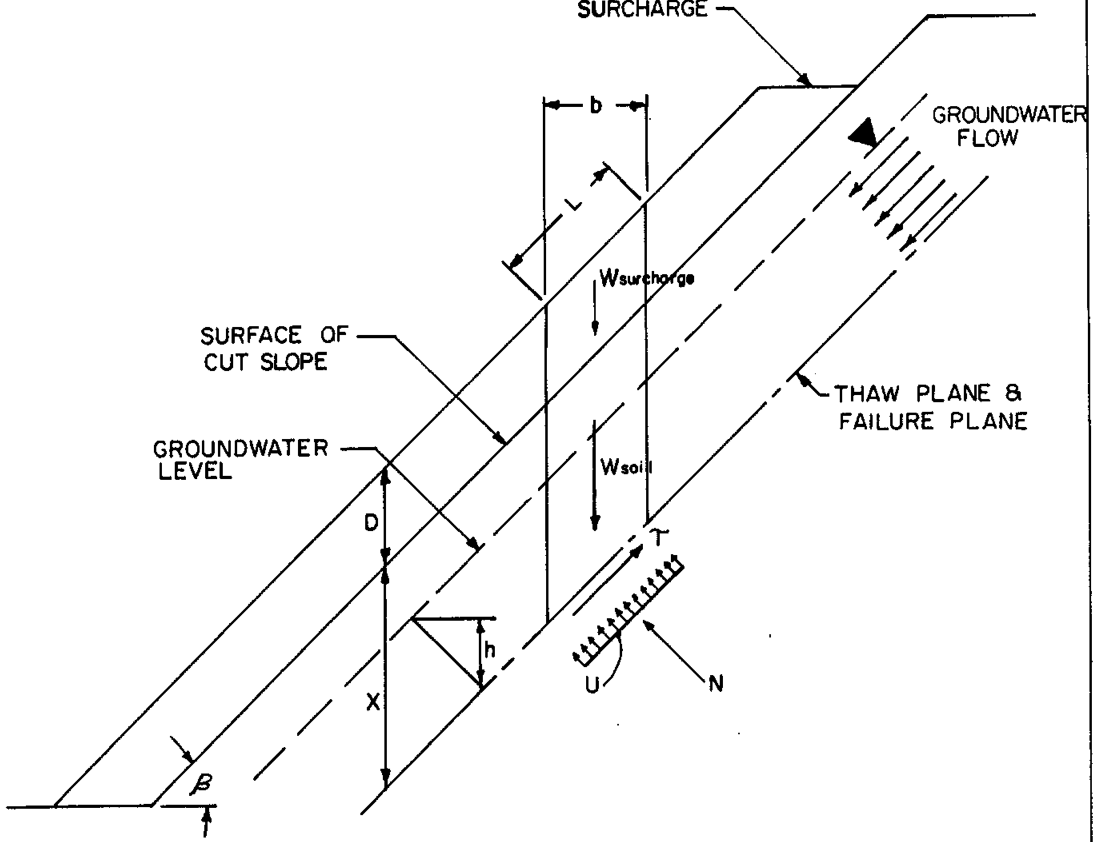

FIGURE 4

SCHEMATIC DIAGRAM
INFINITE SLOPE ANALYSIS

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Excess pore pressures, Us and Uo, are a function of the thaw rate and consolidation characteristics of a soil. Nixon and Morgenstern (1971) developed a thaw consolidation theory to evaluate excess pore pressures generated during thaw and proposed the following relationships:

$$Us = (\gamma - \gamma w)X \bullet R' \qquad (6.3)$$

$$Uo = \gamma oD \bullet R'' \qquad (6.4)$$

where: $$R' = 1/(1 + \frac{1}{2R^2}) \qquad (6.5)$$

$$R'' = erf(R)/(erf(R) + e^{-R^2}/R\pi^{\frac{1}{2}}) \qquad (6.6)$$

and: R = thaw consolidation ratio = $\alpha/2\sqrt{Cv}$ (6.7)
α = thaw rate parameter
Cv = coefficient of consolidation

By evaluating R' and R" at different values of R, it can be shown that:

$$R'' \sim 1.14 \bullet R' \qquad (6.8)$$

Substituting equations (6.3) though (6.8) into equation (6.2) results in the final limit equilibrium equation for analyzing stability of a thawing slope. A simple computer program was used to evaluate equation (6.2) for a number of different shear strength values, excess pore pressures, surcharge thicknesses, and thaw depths. A portion of the results from this parametric study are presented in Figures 5 through 8.

6.4.2 Influence of Thaw Consolidation Rate

From Figure 5, it is seen that, as expected, higher excess pore pressures result in lower factors of safety. The magnitude of the effect, however, is relatively insignificant for R values less than about 0.3. For a typical clayey silt (Cv = 500 ft/yr) and a moderate thaw rate (10 ft/yr) for "naturally" thawing slopes, the thaw consolidation ratio R, is only 0.22. The reduction in F.S. at this R value is less than 0.1 for all values of cohesion. The effect of excess pore pressures on stability, however, becomes more pronounced at higher values of R.

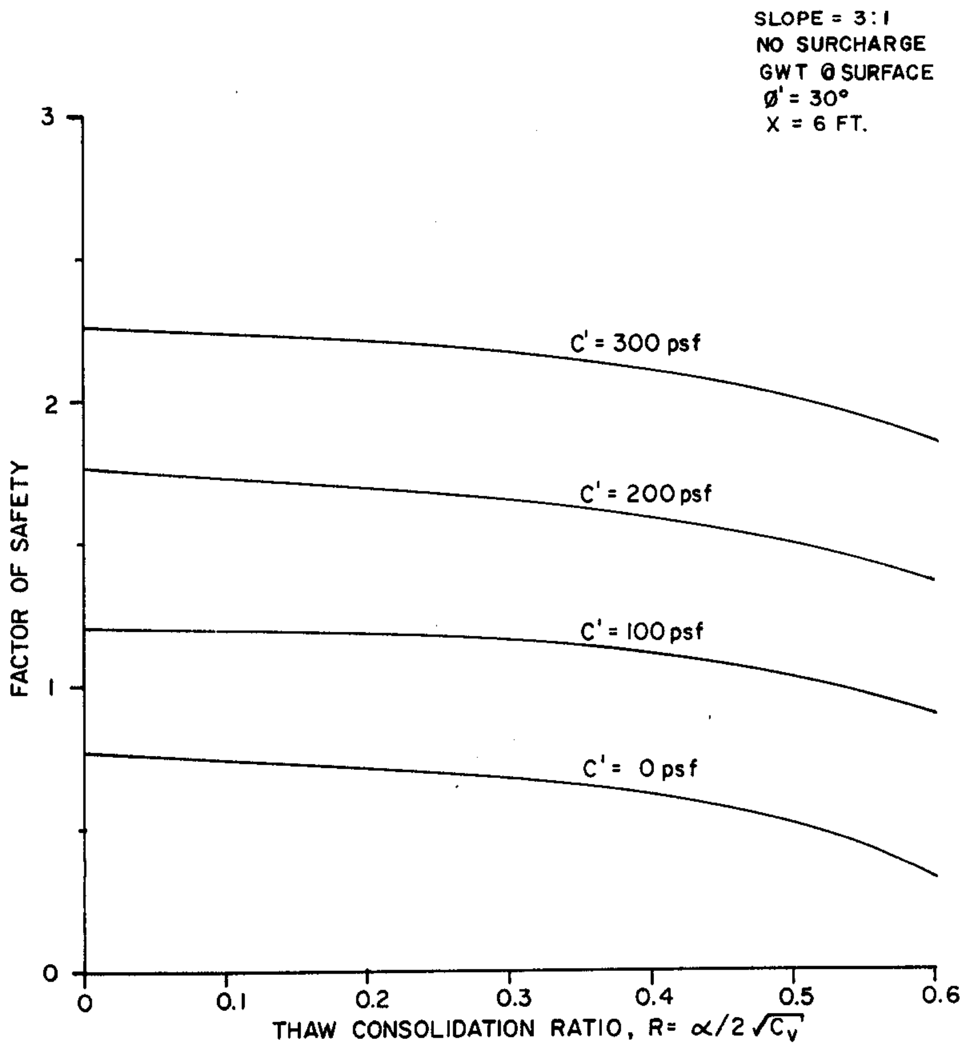

FIGURE 5

THAW CONSOLIDATION RATIO vs. FACTOR OF SAFETY

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6.4.3 Influence of Surcharge Thickness

Nixon and Morgenstern (1973), Pufahl and Morgenstern (1979), among others, have proposed that slope stability can be increased by adding a free-draining surcharge to the slope surface:

> "Although the weight of the surcharge does generate some excess pore water pressures, its presence has the effect of increasing effective normal stress disproportionately to any increase in shearing stress."
>
> (Pufahl and Morgenstern, 1979)

While this is true to some extent in the cohesionless soils investigated by Pufahl and Morgenstern, it is cautioned that, as seen in Figure 6, the opposite effect is attained in cohesive soils for certain conditions. For this particular case, the factor of safety of a slope - composed of soil with a friction angle of 30° and a cohesion of 300 psf - decreases approximately 14 percent (from 2.26 to 1.95) when a six foot surcharge is placed on the slope. The relationship between surcharge thickness and factor of safety is also influenced by groundwater level, slope angle, surcharge density and thaw depth.

6.4.4 Influence of Depth of Thaw

Another important parameter of stability analysis is the depth of thaw X. While X has no influence on F.S. for cohesionless soils, it has considerable effect for cohesive soils as is evident in Figure 7. The inclusion of cohesion in stability analysis at shallow depths results in very high factors of safety. The effect diminishes significantly as the depth of thaw progresses. Since thaw depths at the end of the first thaw season for more ice rich material generally do not exceed several feet, it is seen in Figures 5, 6, and 7 that the influence of effective cohesion on a thawing slope is very important. Moreover, it appears that the influence of cohesion on stability of thawing slopes is greater than that of excess pore pressures, particularly for R values less than about 0.3.

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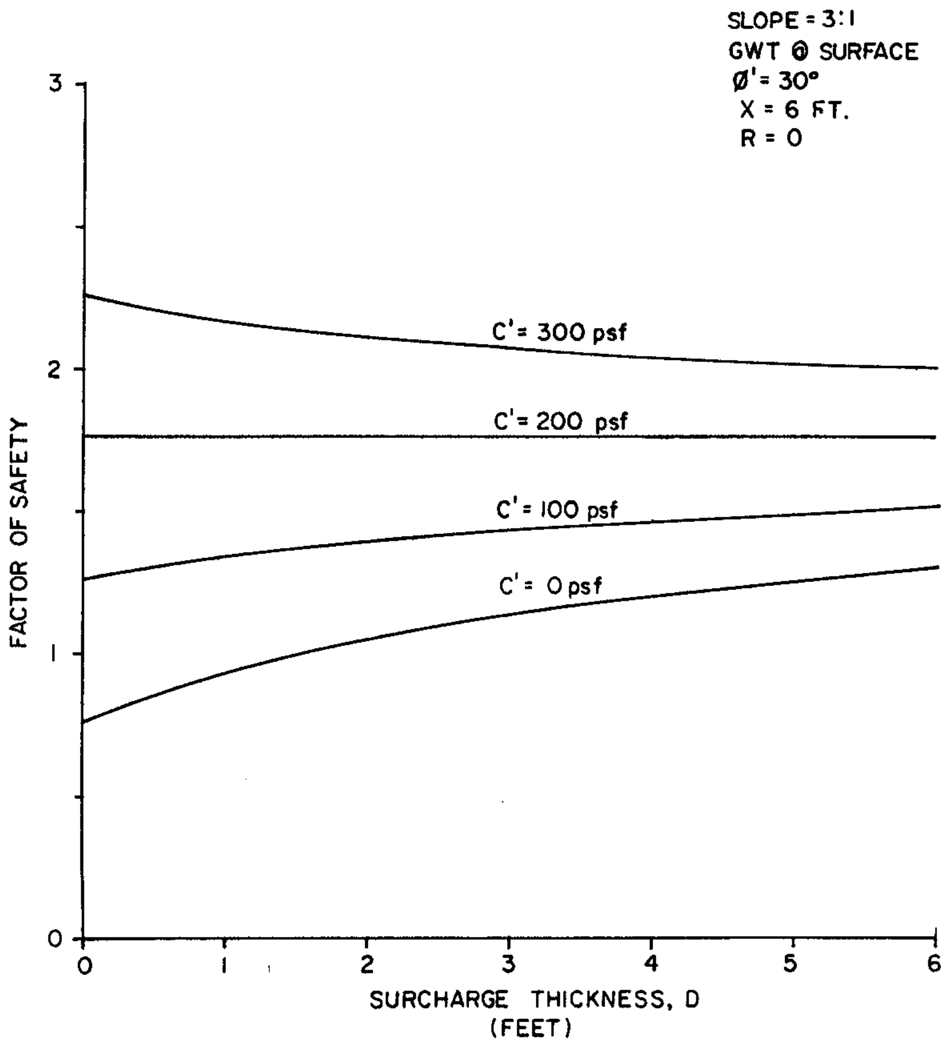

FIGURE 6

SURCHARGE THICKNESS vs. FACTOR OF SAFETY

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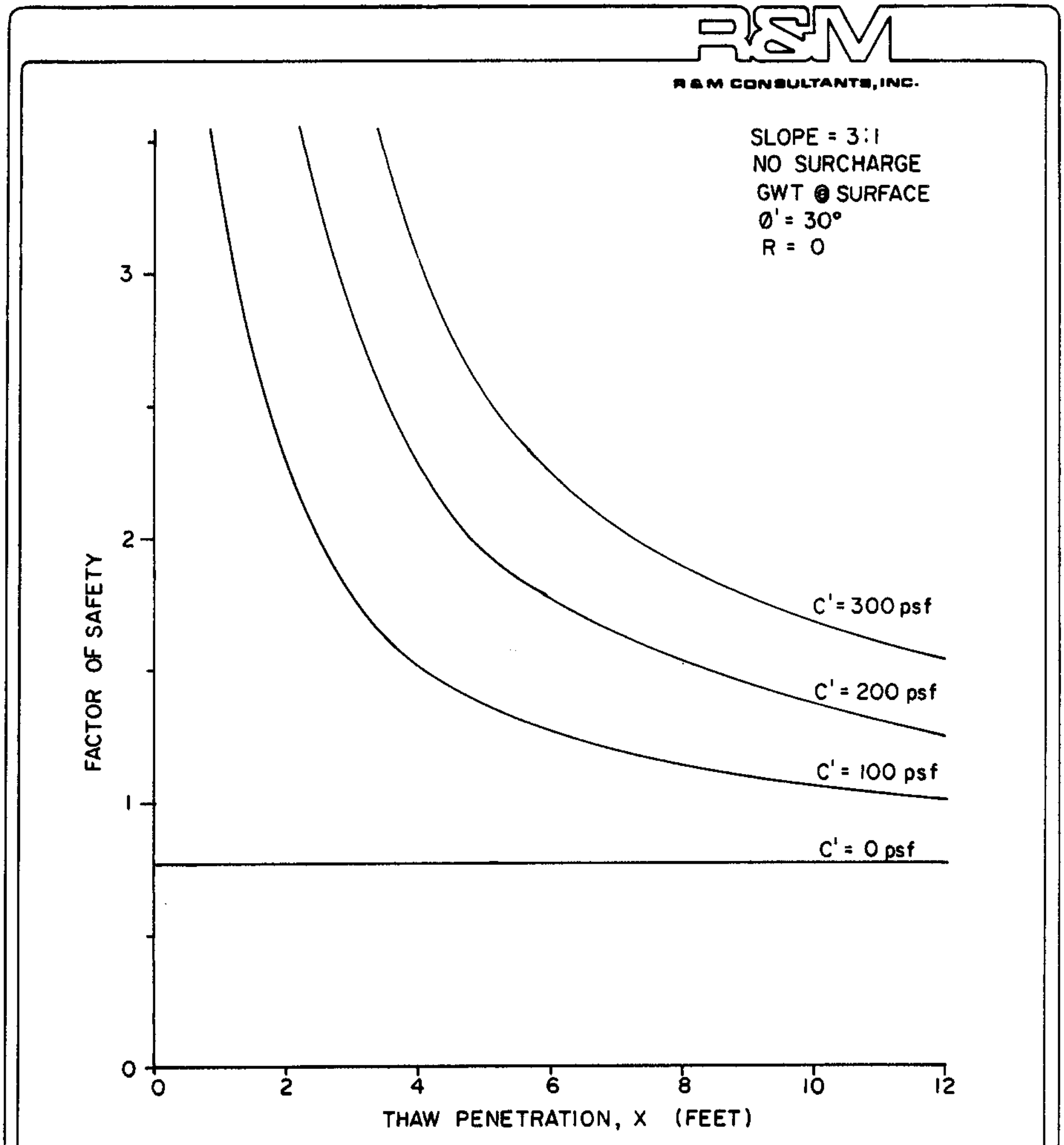

FIGURE 7

THAW PENETRATION vs. FACTOR OF SAFETY

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6.4.5 Influence of Groundwater Level

The above examples (Figures 5, 6, and 7) were all based on the assumption that the groundwater seepage within the surficial unfrozen layer is parallel to the slope and that the groundwater table is at the ground surface. It is well known that the level of phreatic surface significantly impacts factor of safety. What is not well known is how the phreatic surface varies with time in a thawing slope. Initial ice content, permeability of the thawed soil, slope angle, local precipitation and drainage conditions and ground thaw rate have considerable effect on groundwater levels in a thawing slope.

In order to investigate the relative importance of groundwater level on factor of safety, several computer runs were made using different values of Z as input (Z being the ratio of hydrostatic pressure, h, to thaw depth, X). Typical results are presented in Figure 8. As seen in Figure 8, the factor of safety is significantly reduced when the groundwater level is near ground surface (Z is approximately 0.9). Again, the importance of effective cohesion is seen in Figure 8. For this particular example, a slope composed of cohesionless soil would be considered to be unstable when the groundwater level rises to within the upper third portion of the thaw depth. If the effective cohesion of the soil is 100 psf, then, the factor of safety for the same slope conditions would be about 1.2, and would be considered to be marginally stable.

Most stability problems related to slopes cut into frozen soil are associated with fine-grained material. The permeability of thawed fine-grained soils is very low (typically less than 1×10^{-4} cm/sec for silt). If the groundwater level in a thawed slope is related to permeability, it follows that groundwater levels in fine-grained soil are likely to be high, especially during the first thaw season. Fissures and cracks along the slope surface would tend to lower the groundwater level. Although potentially conservative, it may be reasonable to assume that the groundwater level in a thawing slope composed of fine-grained soil is at the ground surface during the first thaw season. The groundwater level during subsequent

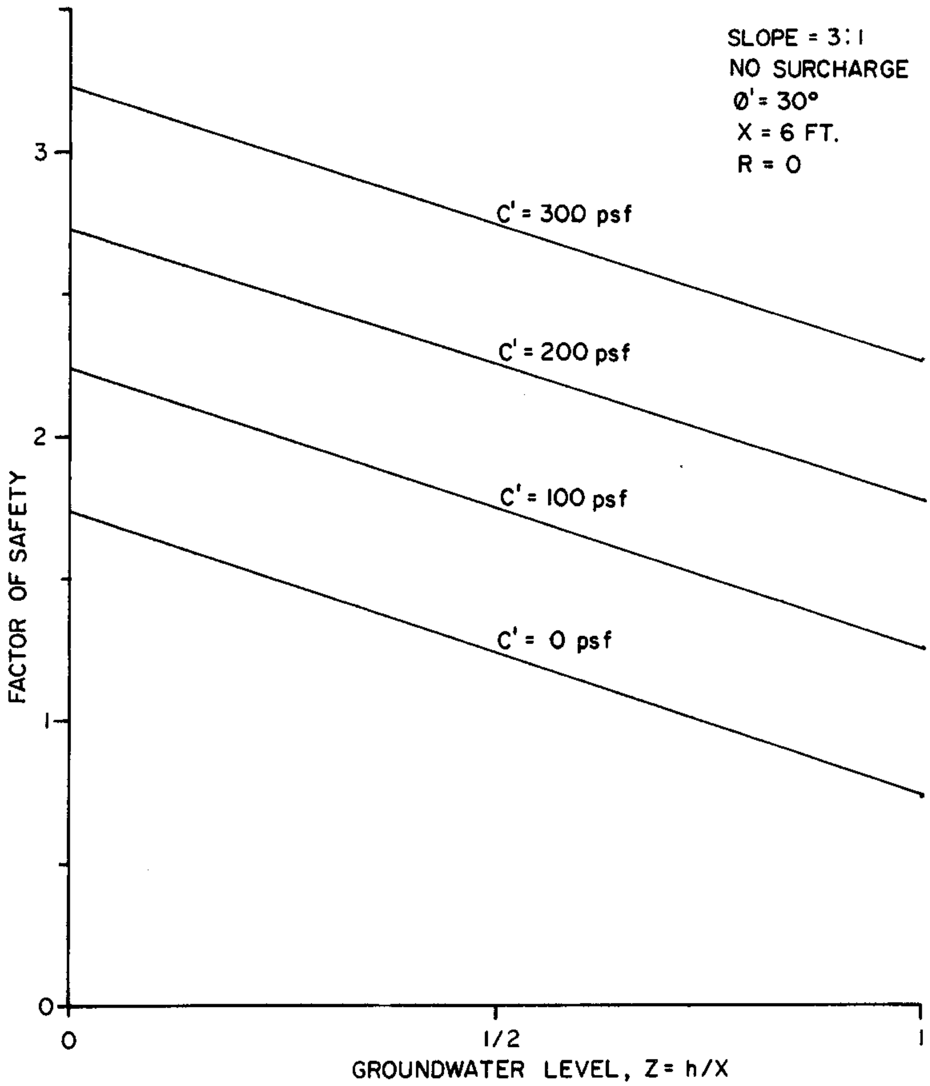

FIGURE 8

GROUNDWATER LEVEL vs. FACTOR OF SAFETY

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thaw seasons would depend on actual thaw melt rates, seasonal precipitation, local drainage paths and regional groundwater levels.

6.4.6 Summarization of Infinite Slope Evaluation

Summarizing some of the important points presented in this section:

1) Excess pore pressures due to thawing have only a limited effect on F.S. for values of R less than about 0.3. For naturally thawing silts typically encountered in Interior Alaska, values of R are typically not expected to exceed about 0.25. Excess pore pressures can be important, when considering R values greater than 0.3. The presence of low compressible soils such as clays, particularly in combination with high thaw rates, could result in R values greater than 0.3 (Figure 5).

2) Placement of a free-draining surcharge does increase short term stability of thawing slopes composed of low-cohesion soil As cohesion decreases and/or depth of thaw increases the effect of a surcharge can be to decrease stability (Figure 6).

3) For slopes composed of cohesive soils, the factor of safety decreases with increasing thaw depth (Figure 7).

4) In general, effective cohesion and groundwater level are the two factors which appear to dominate stability of thawing slopes when thaw depths are small. As thaw depths increase, the effect of cohesion diminishes. The influence of the groundwater level remains important at any thaw depth.

5) It is potentially conservative, but reasonable to assume that the groundwater level is at the ground surface during the first thaw season in a thawing slope composed of fine-grained soil. Groundwater levels during subsequent thaw seasons may vary and conservative values should be used in stability analysis.

6.5 Test Site Results

In order to discuss the performance of surface treatments at the Hess Creek Test Site, it is important to consider that the treatment applications were directed to the evaluation of shorter term performance and that the primary intent was to minimize thermal degradation impact while allowing longer term restabilization to occur through revegetation in the form of seeding and reinvasion of natural growth. With regard to short-term stability, several of the test sections performed rather well while others experienced considerable deterioration during the first thaw season. Regarding long-term stability, all slopes eventually stabilized; some, however, only after significant mass transport and slope deformation had occurred. In terms of revegetation, tree growth on several of the test slopes was very well established by 1984 while other slopes were virtually treeless. It is considered appropriate, therefore, to divide the discussion of the performance at the Hess Creek Test Site into three categories:

- Short-term stability
- Long-term stability
- Revegetation

6.5.1 Short-term Stability

Only index soil properties are available for the Hess Creek Test Site: moisture content, dry density, grain size and liquid limit. No shear strength tests or consolidation tests were performed on the silt soil from this site. Therefore, a detailed stability analysis has not been made for this study. However, in order to provide a preliminary estimate of stability in terms of factor of safety, various soil properties and thaw depths considered to be within the range expected for this site were used as input to equation 6.2. The results from this preliminary analysis are shown in Table I. The analysis was based on the following assumptions:

- slope = 1.5:1 (horizontal to vertical)
- groundwater level at ground surface
- seepage parallel to slope surface
- the potential failure plane is shallow (less than 4 feet), parallel to the slope surface and is along the thaw depth profile (X)
- soil is homogeneous

From Table I is is seen that factors of safety were less than one for nearly all cases. Only when an effective cohesion of 100 psf is used in the analysis at shallow thaw depths (X = 2 ft.) is the factor of safety greater than one. For thaw depths greater than about two feet, however, the F.S. decreases significantly. Also note the relatively small increase in factor of safety by neglecting excess pore pressures (e.g., R = 0).

TABLE I

SHORT-TERM STABILITY ANALYSIS FOR HESS CREEK

ϕ' (°)	C' (psf)	X (ft)	Factor of Safety R = 0.22	Factor of Safety R = 0
25	0	2	0.28	0.31
		4	0.28	0.31
	50	2	0.77	0.79
		4	0.52	0.55
	100	2	1.25	1.28
		4	0.77	0.79
30	0	2	0.35	0.38
		4	0.35	0.38
	50	2	0.83	0.87
		4	0.83	0.87
	100	2	1.32	1.35
		4	0.83	0.87
35	0	2	0.42	0.47
		4	0.42	0.47
	50	2	0.91	0.95
		4	0.66	0.71
	100	2	1.39	1.43
		4	0.91	0.95

Because of the lack of clay mineral in the soil, it is expected that effective cohesion of the silt encountered at the test site would be relatively small, probably less than 50 psf. According to Table I, all slopes would be considered to be unstable when thawed, assuming this value of cohesion regardless of the rate of thaw or thaw depth. A slope having a F.S.<1 does not necessarily mean that a large mass of soil will slide down the slope surface. As discussed earlier, the behavior of an unstable slope will vary significantly, depending on ice content, soil type, surface covering and other factors. However, this limited evaluation does suggest that all slopes at the Hess Creek Test Site were potentially unstable when thawed without consideration of thaw-induced excess pore pressures.

The relative degree of observed sloughing during the first thaw season (1973) was determined by APSC (1974) by visually examining the amount of sloughing for each test site and assigning a number to each section from one to ten. The results for each test section are shown in Figure 9. In Figure 10, thaw penetration is presented as a function of time for each test section. From these two figures, it can be seen that slopes with the lowest thaw penetration experienced the least amount of observable deterioration during the first thaw season.

Based on Table I results and from the discussion above, it is not likely that thaw-induced excess pore pressure had any significant impact on either stability or behavior of these slopes. It is concluded that the primary reason for slope instability during the first thaw season is due to the steepness of cuts and to high groundwater levels. In addition to downslope soil movement, a large fraction of slope deformation observed during the first two thaw seasons is associated directly with volume reductions (i.e. settlement) as ice masses within the slope thawed. While all test sections experienced some deformation, those slopes where thaw penetration was the deepest had developed the highest amount of thawed soil available to move down the slope or to settle. Therefore, by limiting thaw depth, short-term performance of a cut made through ice-rich soil (similar to that encountered at the Hess Creek Test Site) can be improved.

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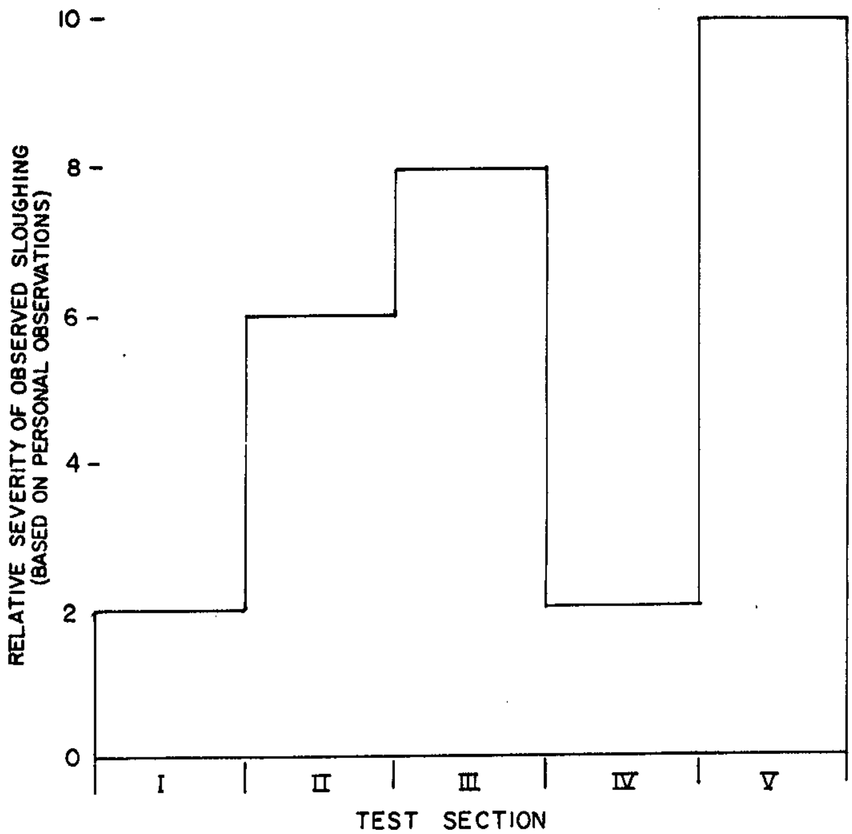

FIGURE 9

SLOPE SLOUGHING INDEX
(FROM APSC, 1974)

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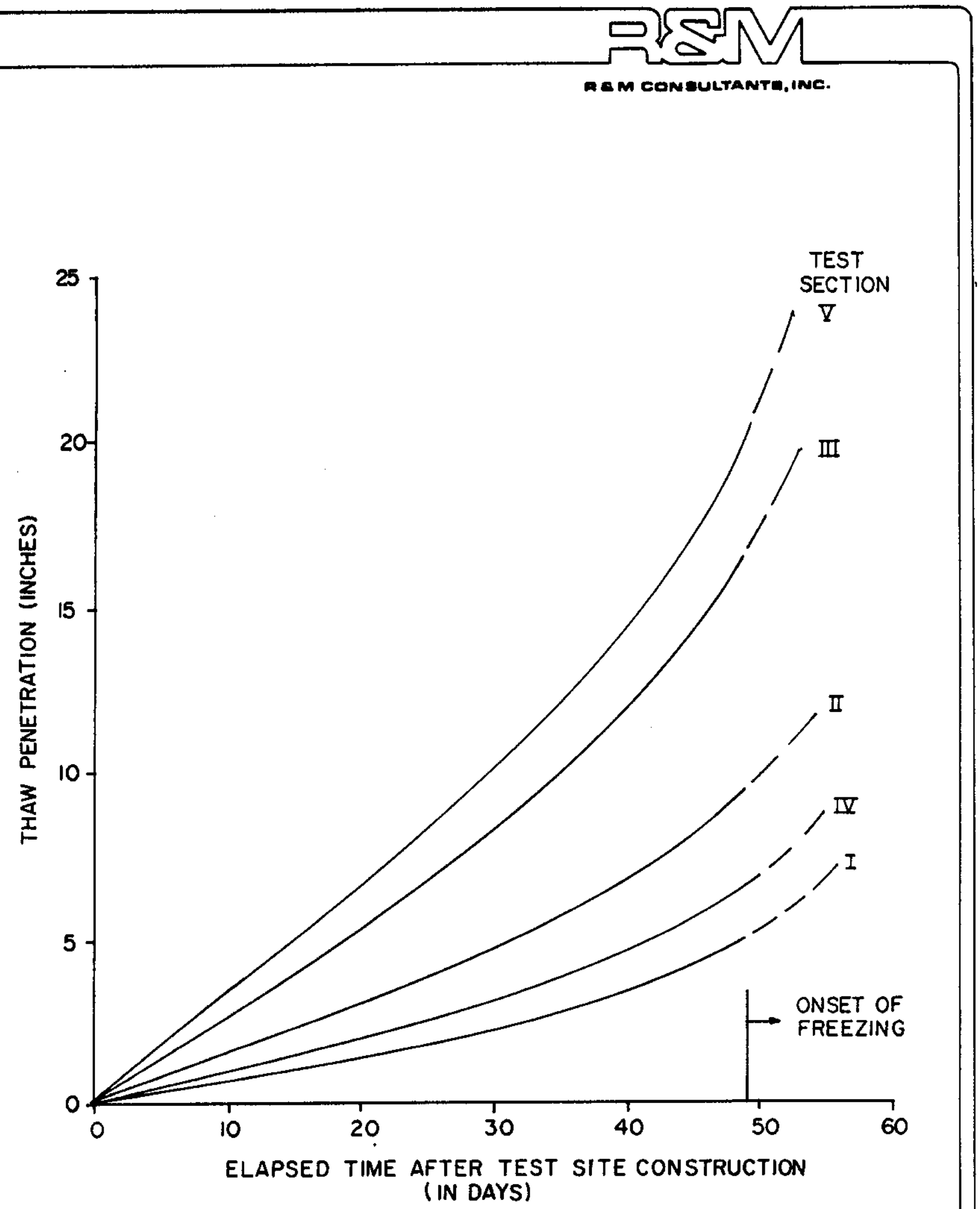

FIGURE 10

THAW PENETRATION AT MID SLOPE - 1973

(FROM APSC, 1974)

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There are other factors which influenced the behavior of the slopes at the Hess Creek Test Site suring the 1973 thaw season. Very little deterioration of the slope surface was observed in Test Section I even through this slope was underlain by soils with the highest initial ice content of all test sections. In addition to limiting thaw penetration, it is expected that the polyurethane insulation treatment masked much of the localized sloughing that may have occurred in this test section, so that to an observer it would appear the slope was relatively stable. Many voids were observed below this insulation cover during site investigations conducted in 1974.

A relatively small amount of soil transport or slope deformations was observed in Test Section IV during the first two thaw seasons. It is likely that the amount of soil transport or slope deformation would not have increased significantly even if no treatment was placed over the exposed slope surface. Soils encountered in this test section had significantly lower ice content than those encountered in the other test sections. Lower moisture contents probably resulted in higher shear strengths of the thawed soil. In addition, volume changes due to thawing of ice in the soils would be significantly less in this test section due to the apparent absence of visible ice features.

The reflective treatment used in Test Section III was supposed to reflect solar radiation, thus reducing heat transfer into the soil. It is apparent from the poor performance of Test Section III during the first thaw season that reflecting solar radiation alone is not sufficient to significantly retard thaw penetration.

Test Section V experienced the deepest thaw penetration and the most significant deterioration during the first thaw season, primarily because the lack of any surface treatment allowed a larger amount of solar radiation to reach the slope surface.

6.5.2 Long-term Stability

After the first thaw season the thermal-insulating effect of surface treatments apparently decreased as a result of material deterioration and all slopes experienced additional deformation from either settlement or transport of soil down the slope. Observations of slope performance for the 1975 through 1982 thaw seasons were not made. It is apparent from Figures A-1 through A-14; however, that the rate of deterioration was less during the first several thaw seasons for slopes with lower initial moisture contents (Sections IV and V-1) than for slopes with higher initial moisture contents (Section I, II, III, V-2). Eventually the rate of deterioration for all slopes decreased significantly. Based on elevation surveys conducted during the two 1983 field investigations, it is believed that the slopes in all five test sections are, under normal drainage conditions, relatively stable at this time. Some localized sloughing or shallow slope failures could occur in any of the test sections if groundwater levels within the slopes were to rise significantly as a result of changed drainage conditions or unseasonally high precipitation.

It is reasonable to assume that long-term slope deformation or soil displacement caused either by thaw settlement or by downslope soil movement are related to the amount of ice in the soil. The relationship to slope volume change was investigated for the Hess Creek Test Site. For each cross section where frozen soil samples were obtained in the 1973 investigation, moisture content values of the frozen soil were averaged. Where ice was encountered, a moisture content value of 500% was used for the analysis. Slope volume changes were normalized by dividing the unit volume of soil lost (values shown in Figures A-1 through A-14) by the initial length of the slope. The unit of measure for this ratio, termed V/L, is feet. The results from this analysis are shown in Figure 11.

It is apparent from Figure 11 that the amount of long-term slope volume change can at least partially be related to moisture content of the frozen soil. Although there is some scatter in data due to inherent variability of natural soils and statistical uncertainty, the relationship between the amount of slope volume change and the amount of ice (frozen moisture) in the soil is reasonable where frozen soil moisture content exceeds the unfrozen soil moisture content at saturation (estimated to be about 45% for Hess Creek silt). Initial frozen moisture contents less than saturation appeared to have much less effect on slope volume change. This phenomenon is attributed to the fact that a significant portion of volume change observed during the first two thaw seasons was a result of thaw settlement; while downslope soil transport contributed to the remaining portion of observed volume change. Thaw settlement, in turn, is primarily a function of initial ice (frozen moisture) content while downslope transport is more directly effected by shear strength of the thawed soil. Therefore, for this particular site where much of the change in volume is due to settlement, the relationship between long-term volume change and excess ice content (ice content above saturation) is expected to be reasonable. For other soil types where thawed shear strengths are very low (i.e., clay), slope volume reductions are likely to result primarily from downslope transport and a moisture content - volume change relationship would probably vary significantly from that shown in Figure 11.

The active layer as measured in September, 1983 along the test sections varied considerably. From examination of available data, it appears that the long-term active layer depth is associated with moisture content of near surface soil. Test sections in which moisture content was relatively low (III-2, IV, V-1) exhibited deeper seasonal thaw depths than did the other test sections where moisture content was higher. In addition, and perhaps as important, those test sections with shallow active layers also have well-established vegetation along the slope surface. Vegetation tends to limit the influx of solar radiation into the soil and dissipate heat through evapotranspiration, thereby retarding seasonal thaw penetration.

6.5.3 Revegetation

Although all five test sections were sprayed with the same grass seed mixture after construction, the amount, rate and type of revegetation varied for each section.

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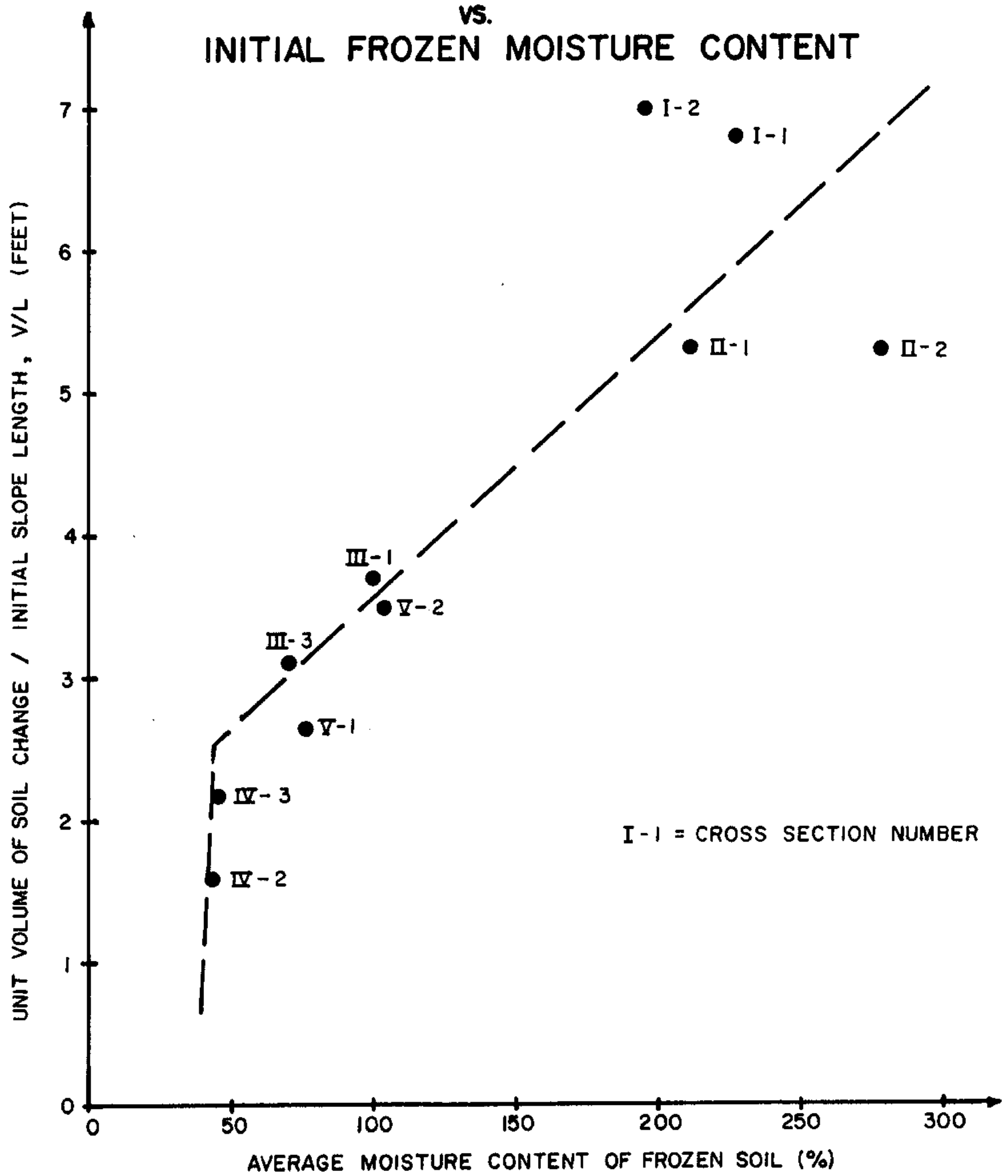
FIGURE 11

SLOPE VOLUME CHANGE
VS.
INITIAL FROZEN MOISTURE CONTENT



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No growth had occurred in Test Section I until late summer in 1974 when severe soil sloughing had occurred. At that time grass began germinating in the newly exposed soil and in portions of the wet foam insulation. Grass continued to develop in the soil but withered as the foam dried out. Eventually, as the slope flattened out with time, natural invasions from nearby willow and spruce began to develop. By 1983 follage in Test Section I was very developed with thick areas of willow trees ranging from 10 to 15 feet high.

At Test Section II, grass began growing in scattered areas throughout the 1973 summer season. As in Test Section I, willow trees began growing as the slopes flattened. In 1983, there appeared to be no difference in vegetation between these first two test sections.

The growth in Test Section III was noticeably thicker than that observed in Test Sections I and II in 1974. This is attributed to the flow of soil over the surface coverings in 1973. In the southwest portion of the test site where moisture contents were higher and soil displacements greater, willow growth had developed after 1974. While not as thick as in the first two test sections, the growth of medium-sized willow trees was significant on this slope by 1983. There was, however, a noticeable lack of tree growth in the north-west portion of the test site. Thick patches of grass had developed in areas where the surface covering was missing. This difference in vegetation from that encountered in the southeast portion of the test section is attributed to the steepness of the slope, relatively low moisture contents in this area, and the presence of the Dynel nylon fabric.

At Test Section IV, some grass was observed under the Dynel fabric where the foam insulation was missing in August 1974. Grass was not observed growing through the fabric or on the foam insulation. In 1983, the vegetation on this slope had not changed considerably since 1974 except for an increase in thickness and extent. Very little tree growth had occurred on this slope over the past ten years. This is probably due to the steepness of the slope (very little soil displacement had occurred in Test Section IV) and to the presence of the Dynel fabric and insulation over most of the slope. In addition, the moisture content values in this test section were significantly less than those encountered in other test sections.

At Test Section V, grass and other vegetation began growing in sporadic areas during the 1974 summer season. This growth, which was considered to be more substantial at that time than in other test sections, is attributed to the lack of any synthetic surface cover. As slopes flattened out, revegetation increased and eventually willow began growing along the flatter portions of the slope. The grass had developed into a thick mat by 1983.

Summarizing the above discussion, it appears that long-term revegetation was most complete in Test Sections I, II, III (southeast) and V where the greatest soil displacements were experienced. Revegetation was generally sporadic at best on those slopes in which soil movements were relatively small (Test Sections III (northwest) and IV). It is apparent that revegetation is retarded on slopes where surface coverings remain intact.

7.0 CONCLUSIONS

The following conclusions regarding thermal erosion of slopes cut into ice-rich soil are based on results from three field investigations conducted at the Hess Creek thermal erosion test site and subsequent theoretical analysis:

1) For low compressible soils such as silt, the amount of excess pore pressures generated during natural thawing may typically be small and does not appear to significantly influence stability of the slopes. Based on infinite slope analysis, most of the instability of the original cut slopes at the Hess Creek Test Site is attributed to the steepness of the cuts (1.5:1) and to high groundwater levels during thaw.

2) The primary effect of surface treatments with high insulative properties is to reduce the depth of thaw penetration on a short-term basis. Low thaw penetration depths resulted in less sloughing or mass transport of soil down the slope during the first thaw season. The effect of insulation on seasonal thaw penetration decreases with time as the insulating properties of the surface treatment deteriorate.

3) Surface treatments which allow thaw penetration into the underlying ice-rich soil are most effective in reducing thermal erosion and subsequent instability only on a short-term basis (one or two thaw seasons). Unless additional precautions are taken, deformation of the slope surface with time either by settlement or by mass transport will eventually cause the surface treatment to deteriorate, thus allowing thaw penetration and thermal erosion to progress.

4) Surface treatments that rely solely upon high reflectivity to reduce heat transfer into underlying frozen soil are ineffective in retarding thaw penetration.

5) Slopes within all five test sections continued to experience sloughing, mass transport of thawed soil down the slope, and settlement along the slope surface for at least several thaw seasons following

construction. All slopes appear to be relatively stable at this time, ten years following construction.

6) Based on survey elevations made in 1973, 1974 and 1983, it appears that settlement contributed to a large portion of long-term slope volume change at the Hess Creek Test Site. The remaining portion of volume change is attributed to downslope movement of thawed soil.

7) The amount of long-term slope volume change for cuts in ice-rich silt appears to be strongly related to initial moisture content of the frozen soil. This relationship may not be applicable for soils such as highly plastic clay which have very low shear strengths during thaw.

8) The most important factors that effect stability of naturally thawing slopes cut into ice-rich soil appear to be the shear strength of the soil, the groundwater level during thaw and the percent grade at which the slope is cut. Shear strength is derived from cohesion and friction between soil particles. Very small amounts of cohesion tend to have a significant impact on stability for shallow depths of thaw (i.e., during the first several thaw seasons). As thaw depths progress, the effect of cohesion diminishes. Groundwater levels during thaw are related to original moisture content of the frozen material. Fine-grained soils with moisture content values at or above saturation are expected to result in high groundwater levels during at least the first thaw season. It is considered potentially conservative, but reasonable, to assume that the groundwater level for fine-grained, ice-rich soil to be at the ground surface when slope stability for the first thaw season. Steep slope angles obviously result in lower stability; low slope angles result in higher stability.

9) Revegetation appears to be more successful in ice-rich slopes which have experienced considerable soil displacement and slope flattening. Slopes which have experienced little soil displacement, either because of low ice content or because of effective surface coverings, tend to lack developed vegetative growth. In addition to providing a better

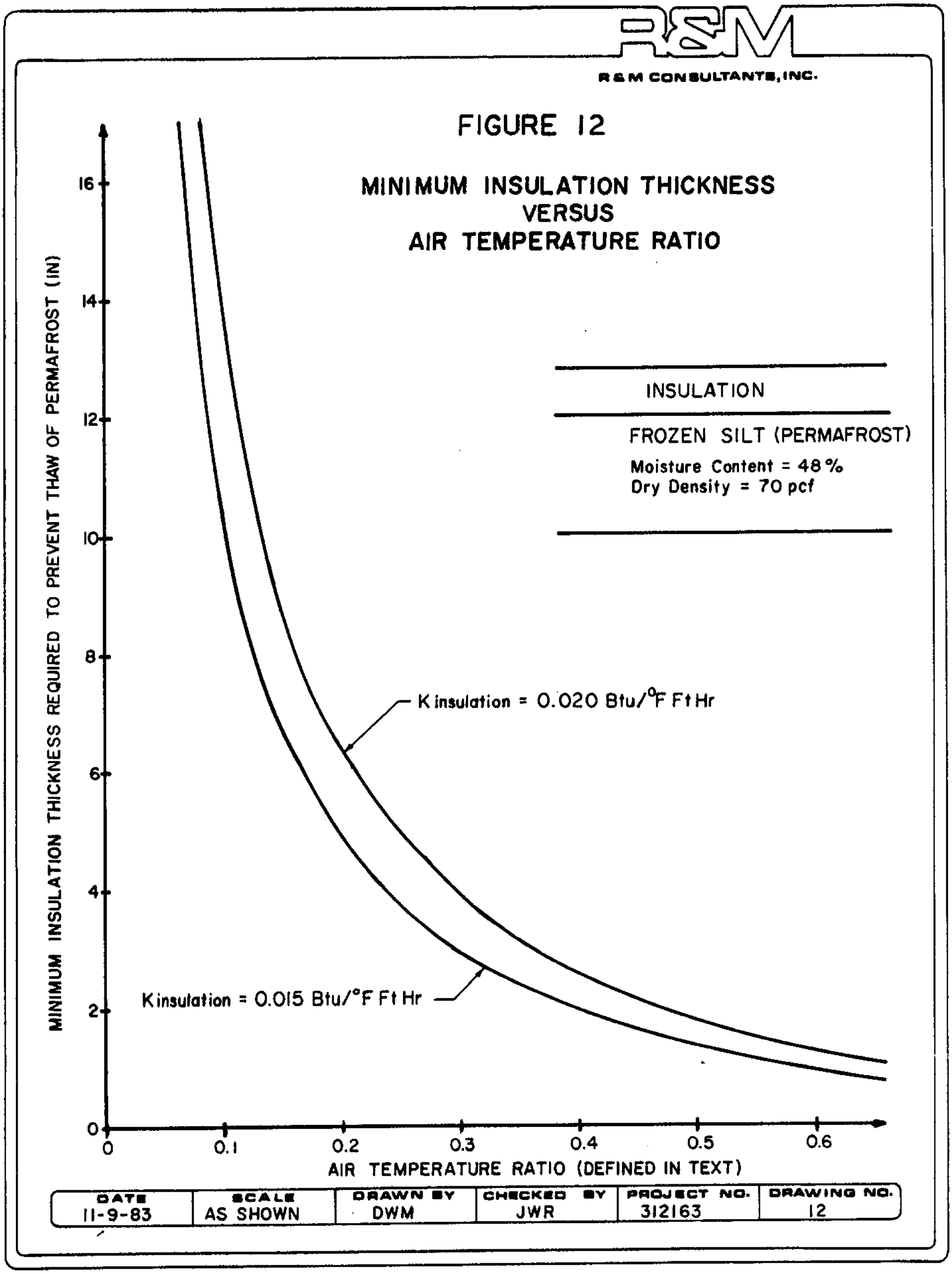
surface on which new seeds can collect, it is suggested that flatter slopes contain higher moisture contents resulting in better overall growing conditions. The presence of surface treatments appears to inhibit vegetative growth.

8.0 RECOMMENDATIONS

8.1 Short-term Stability

The use of surface treatments with high insulating properties on cut slopes can be effective in minimizing thermal degradation and erosion in ice-rich material for a limited period of time. However, the insulating value of most available materials may decrease with time because of water absorption, deterioration due to solar radiation, and deterioration due to natural processes (e.g., chemical action, vegetation). Moreover, even small amounts of thermal erosion below the insulation cover could result in localized overstressing of the treatment which, in turn, could cause cracking and further deterioration of most insulating materials. Therefore, the use of surface treatments is not generally expected to be effective for long-term stabilization of slopes cut into ice-rich soil unless additional precautions are taken. It is expected that, with proper design and application, certain surface treatments could effectively stabilize a cut slope in ice-rich soils for a specified number of thaw seasons. However, economic constraints may make costs for potential design solutions prohibitive.

Because of its high insulating properties and relative ease of application, sprayed foam insulation appears to be the most suitable material for surface treating in ice-rich cut slopes. Various types of foam insulation including polyurethane and sulphur have been used in other field applications (Berg, 1976; Smith and Pazsint, 1976). Urethane foam insulation appears suitable because of its availability, high insulating properties, relatively high strength, and durability. Insulating properties of insulation are typically measured in terms of thermal conductivity, k. Under ideal conditions, values of k for urethane insulation can vary from 0.015 to 0.020 Btu/°F Hr-ft (Berg, 1976). As discussed above, insulating values tend to decrease somewhat with time (k values increase with time). In addition, k values are a function of density of the insulation. Density, in turn, is affected by ground and air temperatures at the time of application. Colder air and ground temperatures result in higher density and thus higher thermal conductivity. Therefore, application during the


R&M
R&M CONSULTANTS, INC.
FIGURE 12
MINIMUM INSULATION THICKNESS
VERSUS
AIR TEMPERATURE RATIO
INSULATION
FROZEN SILT (PERMAFROST)
Moisture Content = 48 %
Dry Density = 70 pcf
K insulation = 0.020 Btu/°F Ft Hr
K insulation = 0.015 Btu/°F Ft Hr
MINIMUM INSULATION THICKNESS REQUIRED TO PREVENT THAW OF PERMAFROST (IN)
0
2
4
6
8
10
12
14
16
0
0.1
0.2
0.3
0.4
0.5
0.6
AIR TEMPERATURE RATIO (DEFINED IN TEXT)
DATE
11-9-83
SCALE
AS SHOWN
DRAWN BY
DWM
CHECKED BY
JWR
PROJECT NO.
312163
DRAWING NO.
12

winter results in lower quality insulation than application during the summer.

Thicker application of insulation will result in less thermal erosion. However, foam insulation can be expensive. Therefore, it is considered prudent to apply only the amount of insulation that will produce the desired performance. If minimizing thermal erosion is critical, then the thickness of insulation should be great enough to limit thaw penetration to a very small depth into the underlying frozen soil or preferably, to within the insulation. Lachenbruch (1959) developed an approach for evaluating thaw penetration into two or three layers of material where phase change does not occur. Using Lachenbruch's equations, a relationship was developed between the insulation thickness theoretically required to prevent thaw penetration into underlying permafrost and the air temperature divided by the amplitude of the air temperature sinusoidal curve (warmest monthly mean - coldest monthly mean). It was assumed in the development of these relationships that construction of the cut slope and placement of the insulation is done in early spring, just prior to the onset of above freezing average air temperatures. Graphs are presented in Figure 12 for two values of thermal conductivity of insulation.

It is emphasized that these curves should be used conservatively as a guideline for estimating insulation thickness when very little thermal erosion can be tolerated. It is very difficult to design for zero thaw penetration and limited thermal erosion over the entire slope, especially in warmer climatic regions south of the Brooks Range. Practical considerations that influence such thermal performance include; local variation in insulation thickness and density, seasonal fluctuations in temperature, and the potential for thermal erosion due to water flow below the insulation during periods of seasonal precipitation. In areas where limited thermal erosion can be tolerated, the design thickness of the insulation can be reduced. Beside increasing the potential for higher thermal erosion during the first thaw season, the use of less insulation will decrease the number of thaw seasons that the treatment will be effective in retarding further thermal erosion. Also, the positive influence of longer term revegetation effects should be considered and included in the design process.

To illustrate the range in insulation thickness required to limit thaw penetration into ice-rich, fine-grained soil, typical values have been calculated for various locations throughout Alaska (Table 2). The values shown in Table 2 (as well as in Figure 12) are based on the assumption that the foam insulation is applied during early spring, prior to the onset of above freezing temperatures. It is seen from both Figure 12 and Table 2 that, in general, a large thickness of insulation is required to minimize thaw of underlying permafrost in the "warmer" frozen ground regions and that in some areas (e.g., Anchorage) no amount of insulation would be effective.

8.2 Further Research

Quantitative methods for evaluating performance of cut slopes in ice-rich soil generally rely upon results from infinite slope analysis (factor of safety) taking into account thaw-induced excess pore pressures (McRoberts & Morgenstern, 1974; Pufhal & Morgenstern, 1979). This type of analysis is useful in evaluating the relative stability of alternate slope designs. However, limit equilibrium analyses does not provide information regarding the behavior of a potentially unstable slope. As evidenced by the Hess Creek Test Site results, slopes with a factor of safety less than one can behave quite differently. All slopes at the test site were believed to have factors of safety less than one during the first thaw season. The amount of slope displacement did vary during the thaw season - from very low (Test Section IV) to very high (Test Sections II and V). Therefore, it is recommended that, where possible, past performance of cuts in similar soil also be used in the slope evaluation and design process.

One method for incorporating the performance of previously constructed cut slopes would be to develop a means of correlating slope behavior to soil properties, initial ice content, slope angle, thaw rate, and perhaps other parameters. Further research should be directed toward the development of such possible correlations. It is envisioned that, in addition to conducting more field investigations of existing highway cut-slopes in previously frozen ice-rich soil, small-scale laboratory testing can be useful in identifying how each of the various parameters affect behavior of a thawing slope.

TABLE 2

MINIMUM INSULATION THICKNESSES TO MINIMIZE THERMAL EROSION FOR VARIOUS LOCATIONS IN ALASKA

Location	Latitude (°)	M.A.A.T.* (°F)	Thaw Index (°F-days)	Ao** (°F)	32°-M.A.A.T. / Ao	Minimum*** Insulation Thickness (in)
Barrow	71.0°	5	400	35	0.77	0.75
Prudhoe	70.2°	8	600	35	0.69	1.0
Galbraith Lake	68.5	12	1100	35	0.57	1.5
Wiseman	67.5°	15	2000	40	0.43	2.25
Kotzebue	66.9°	21	1700	30	0.37	3.0
Bettles	66.9°	21	2400	36	0.31	3.75
Ft. Yukon	66.7°	19.5	2650	41	0.30	4.0
Nome	64.5°	25	1900	26	0.27	4.5
Hess Creek	65.7°	23.5	2700	33	0.26	4.75
Fairbanks	64.9°	26.2	3400	37	0.16	8.0
Northway	62.9°	27.4	2991	33	0.14	9.0
Bethel	60.9°	30	2500	24	0.08	17.0
Anchorage	61.3°	35	3000	21	0	∞

* M.A.A.T. = Mean Annual Air Temperature

** Ao = Amplitude of Air Temperature Sinusoidal Curve

*** From Figure 2 (K insulation = 0.20)

8.3 Implementation (This subsection prepared by Alaska Department of Transportation, Research)

This study of 1.5:1 cut slopes in ice-rich silt permafrost soils has shown that none of the surface treatments investigated yielded appreciable long term improvements in stability. These treatments included sand blankets, urea formaldehyde foam insulation, reflective surfacing, burlap, synthetic permeable fabric, and synthetic mulch, alone and in combinations. Designers for the Department of Transportation and Public Facilities therefore should not rely on these techniques in attempts to produce cut slopes with long term stability in ice-rich permafrost materials. Utilization of insulative materials may be required to provide longer term mitigative thermal design solutions for certain site specific conditions. Such solutions should only be considered upon verifying anticipated long term thermal behavior of the proposed design.

The current technique for ice-rich silt slopes is to make a near-vertical cut combined with a wide ditch to catch slope material which sloughs as it thaws. This requires maintenance to clean up slopes and ditches until such time as the slope stabilizes. While this is inconvenient, use of this technique should be continued unless and until a superior one is found.

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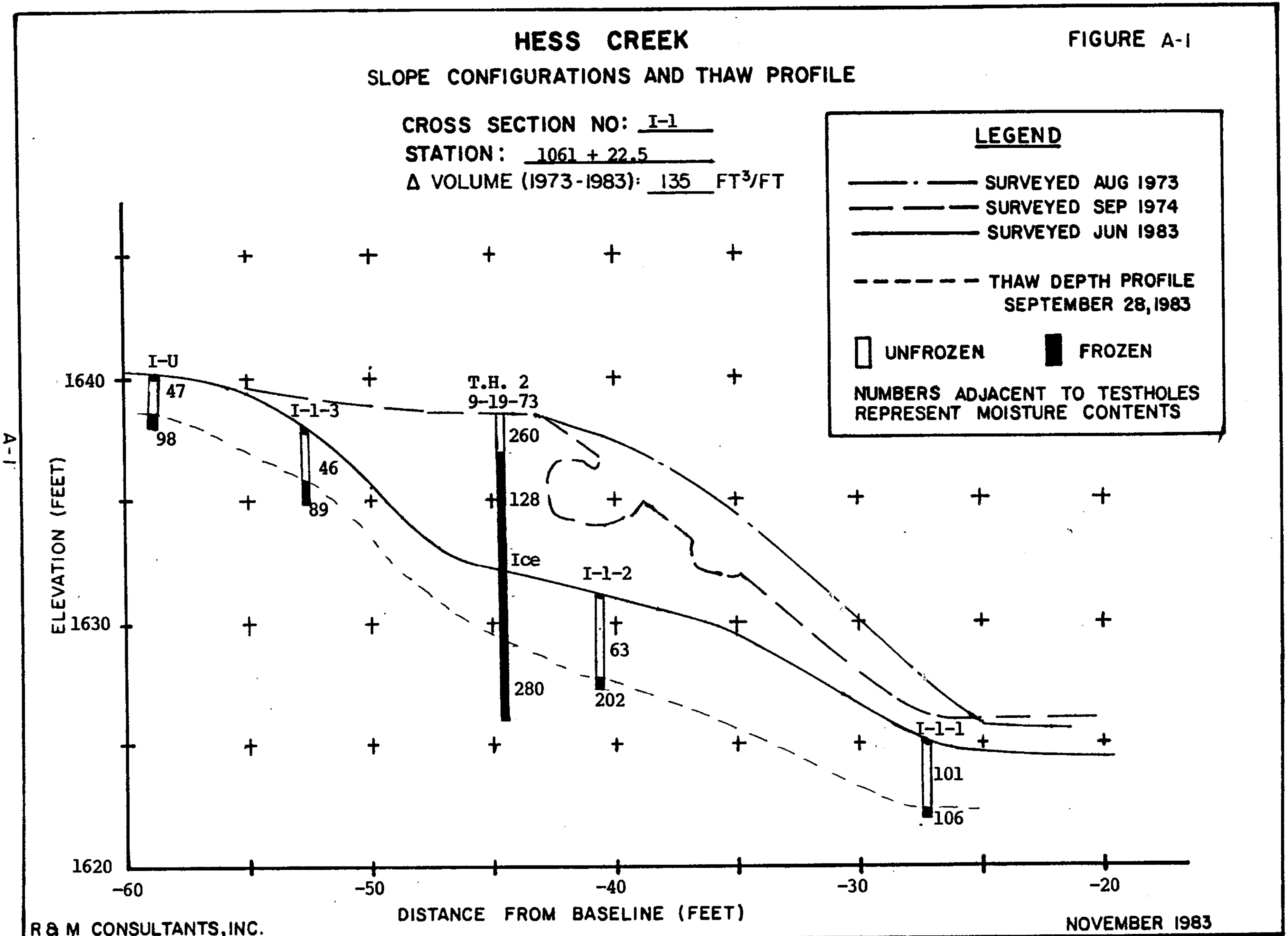
APPENDIX A

SLOPE CONFIGURATIONS AND THAW PROFILES


HESS CREEK
FIGURE A-1
SLOPE CONFIGURATIONS AND THAW PROFILE
CROSS SECTION NO: I-1
STATION: 1061 + 22.5
Δ VOLUME (1973-1983): 135 FT³/FT
LEGEND
SURVEYED AUG 1973
SURVEYED SEP 1974
SURVEYED JUN 1983
THAW DEPTH PROFILE SEPTEMBER 28, 1983
UNFROZEN
FROZEN
NUMBERS ADJACENT TO TESTHOLES REPRESENT MOISTURE CONTENTS
I-U
47
98
I-1-3
46
89
T.H. 2
9-19-73
260
128
Ice
280
I-1-2
63
202
I-1-1
101
106
1640
1630
1620
-60
-50
-40
-30
-20
ELEVATION (FEET)
DISTANCE FROM BASELINE (FEET)
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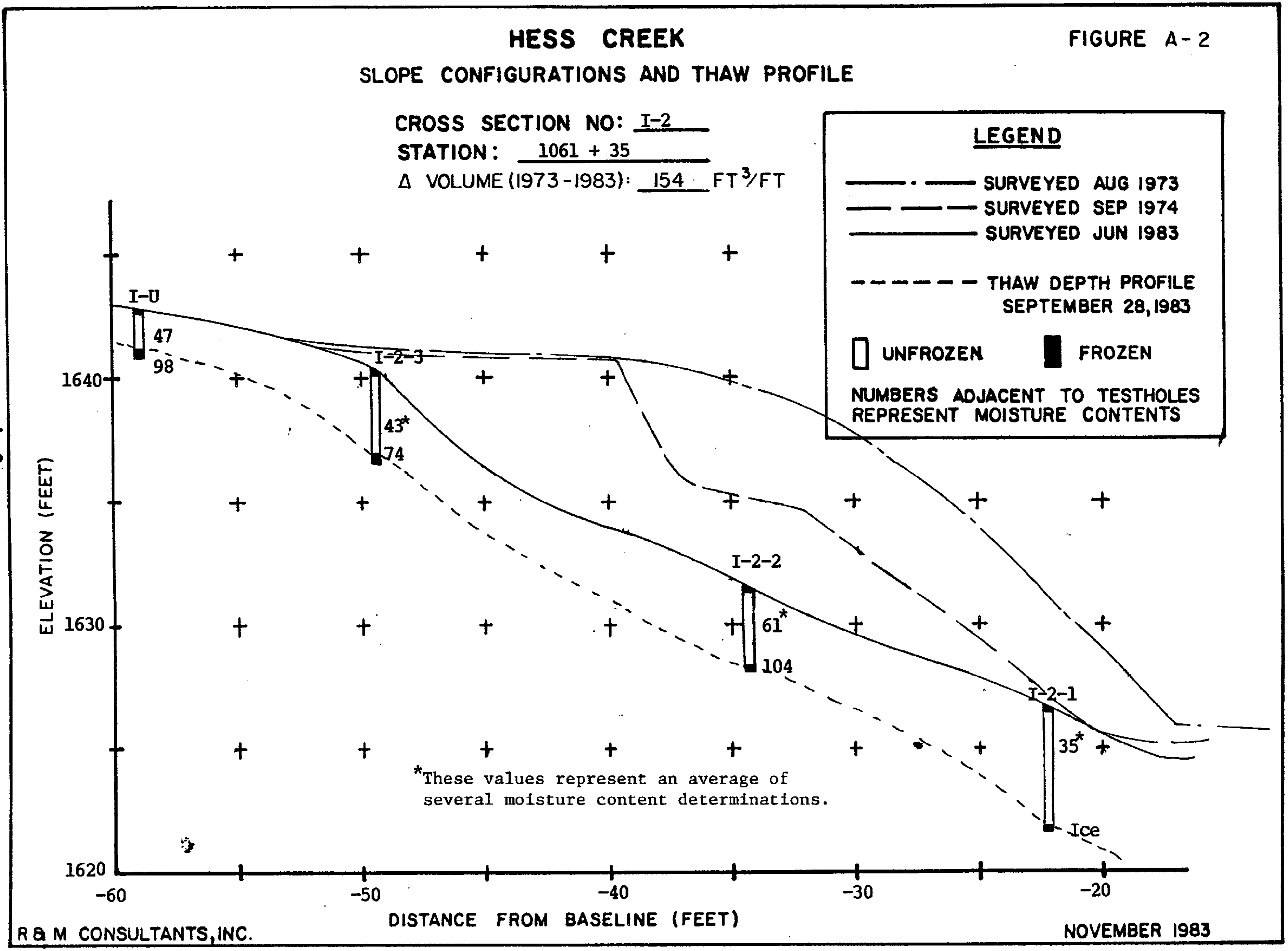
HESS CREEK
FIGURE A-2
SLOPE CONFIGURATIONS AND THAW PROFILE
CROSS SECTION NO: I-2
STATION: 1061 + 35
Δ VOLUME (1973-1983): 154 FT³/FT
LEGEND
SURVEYED AUG 1973
SURVEYED SEP 1974
SURVEYED JUN 1983
THAW DEPTH PROFILE SEPTEMBER 28, 1983
UNFROZEN
FROZEN
NUMBERS ADJACENT TO TESTHOLES REPRESENT MOISTURE CONTENTS
I-U
47
98
I-2-3
43*
74
I-2-2
61*
104
I-2-1
35*
Ice
1640
1630
1620
-60
-50
-40
-30
-20
ELEVATION (FEET)
DISTANCE FROM BASELINE (FEET)
*These values represent an average of several moisture content determinations.
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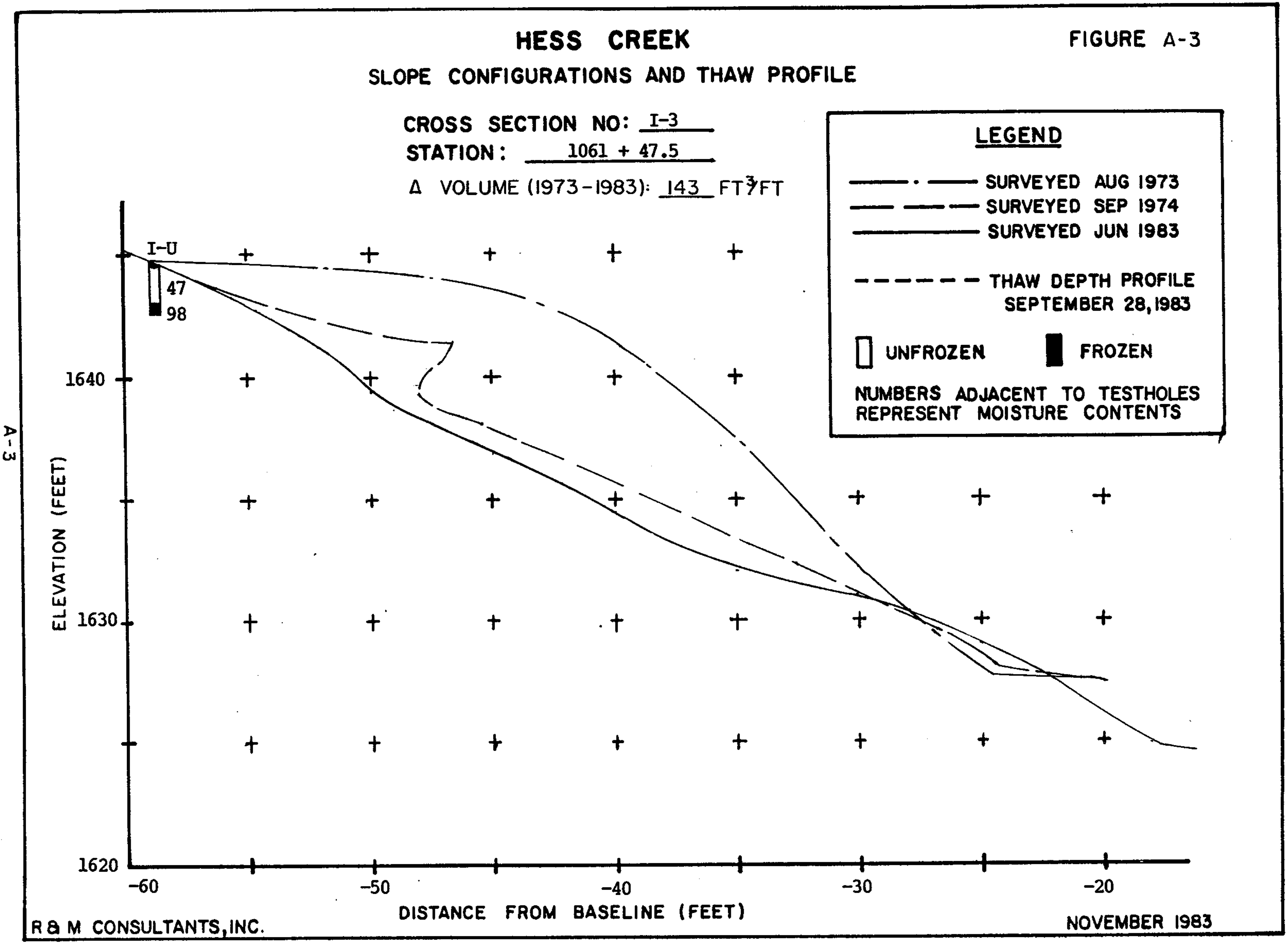
HESS CREEK
FIGURE A-3
SLOPE CONFIGURATIONS AND THAW PROFILE
CROSS SECTION NO: I-3
STATION: 1061 + 47.5
Δ VOLUME (1973-1983): 143 FT³/FT
LEGEND
SURVEYED AUG 1973
SURVEYED SEP 1974
SURVEYED JUN 1983
THAW DEPTH PROFILE SEPTEMBER 28, 1983
UNFROZEN
FROZEN
NUMBERS ADJACENT TO TESTHOLES REPRESENT MOISTURE CONTENTS
I-U
47
98
1640
1630
1620
ELEVATION (FEET)
-60
-50
-40
-30
-20
DISTANCE FROM BASELINE (FEET)
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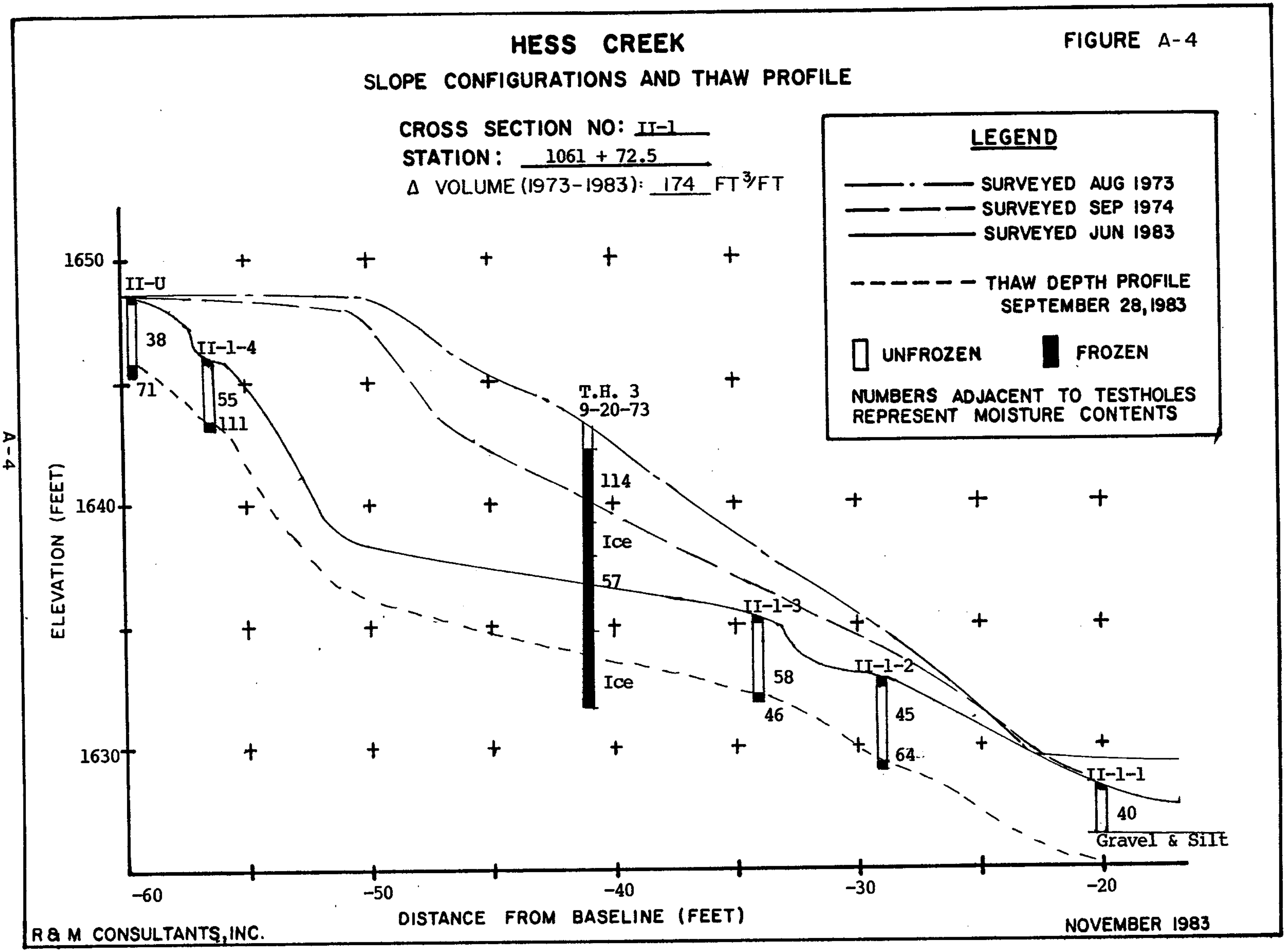
HESS CREEK
SLOPE CONFIGURATIONS AND THAW PROFILE
FIGURE A-4
CROSS SECTION NO: II-1
STATION: 1061 + 72.5
Δ VOLUME (1973-1983): 174 FT³/FT
LEGEND
SURVEYED AUG 1973
SURVEYED SEP 1974
SURVEYED JUN 1983
THAW DEPTH PROFILE SEPTEMBER 28, 1983
UNFROZEN
FROZEN
NUMBERS ADJACENT TO TESTHOLES REPRESENT MOISTURE CONTENTS
II-U
38
71
II-1-4
55
111
T.H. 3
9-20-73
114
Ice
57
Ice
II-1-3
58
46
II-1-2
45
64
II-1-1
40
Gravel & Silt
1650
1640
1630
ELEVATION (FEET)
-60
-50
-40
-30
-20
DISTANCE FROM BASELINE (FEET)
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HESS CREEK

FIGURE A-5

SLOPE CONFIGURATIONS AND THAW PROFILE

CROSS SECTION NO: II-2
STATION: 1061 + 85
Δ VOLUME (1973-1983): 152 FT^3/FT

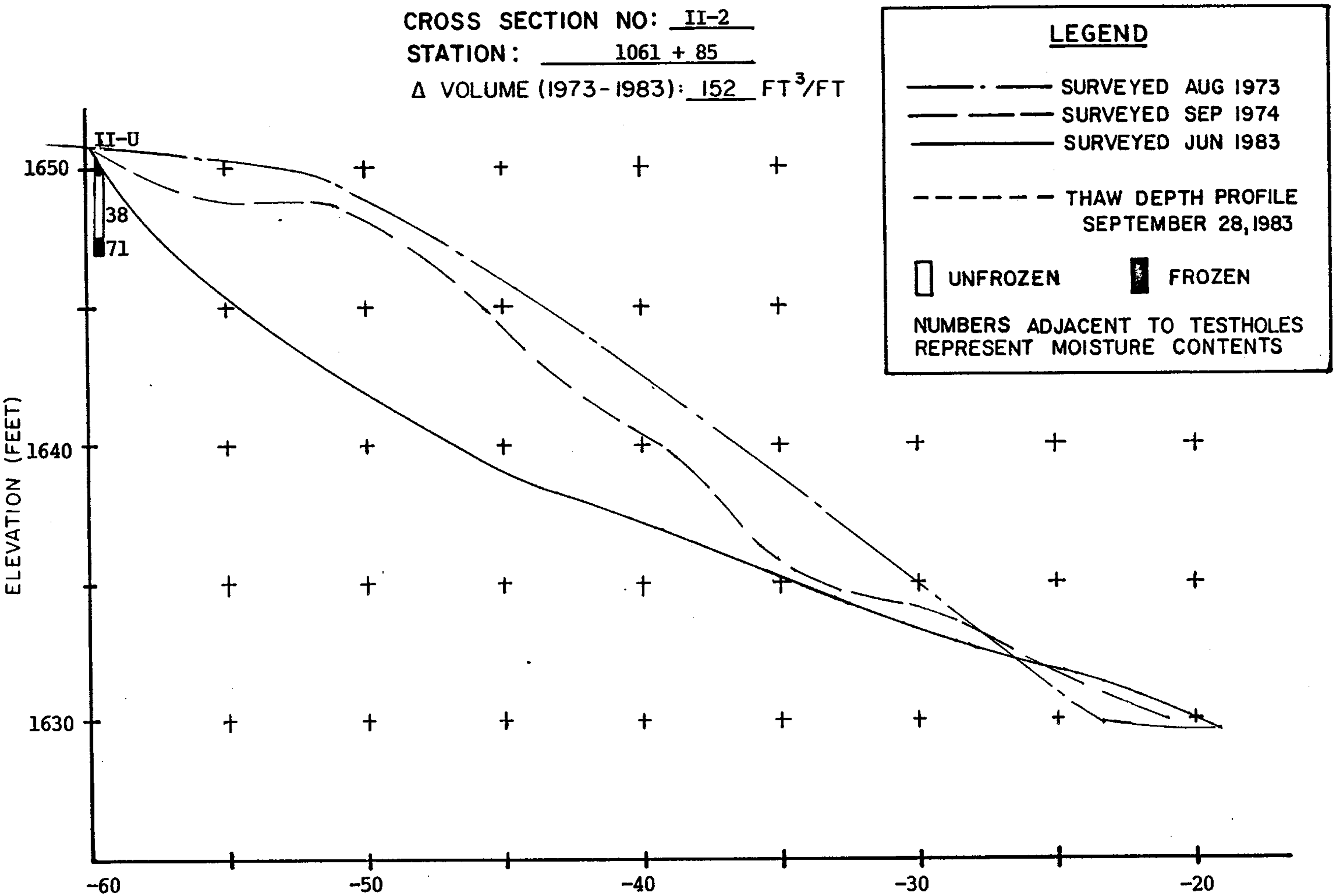

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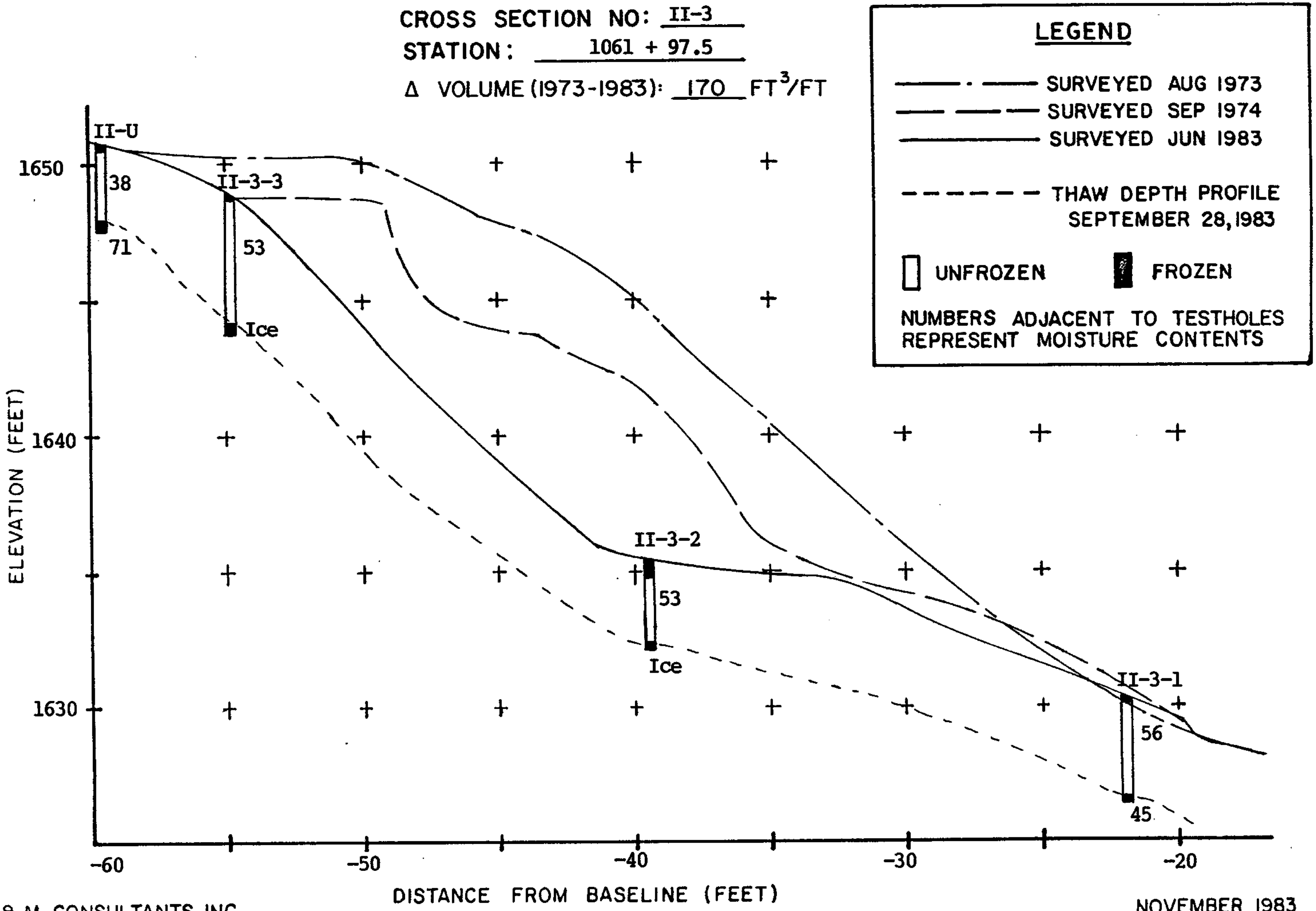
FIGURE A-6

HESS CREEK

SLOPE CONFIGURATIONS AND THAW PROFILE

CROSS SECTION NO: II-3
STATION: 1061 + 97.5
Δ VOLUME (1973-1983): 170 FT^3/FT



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HESS CREEK

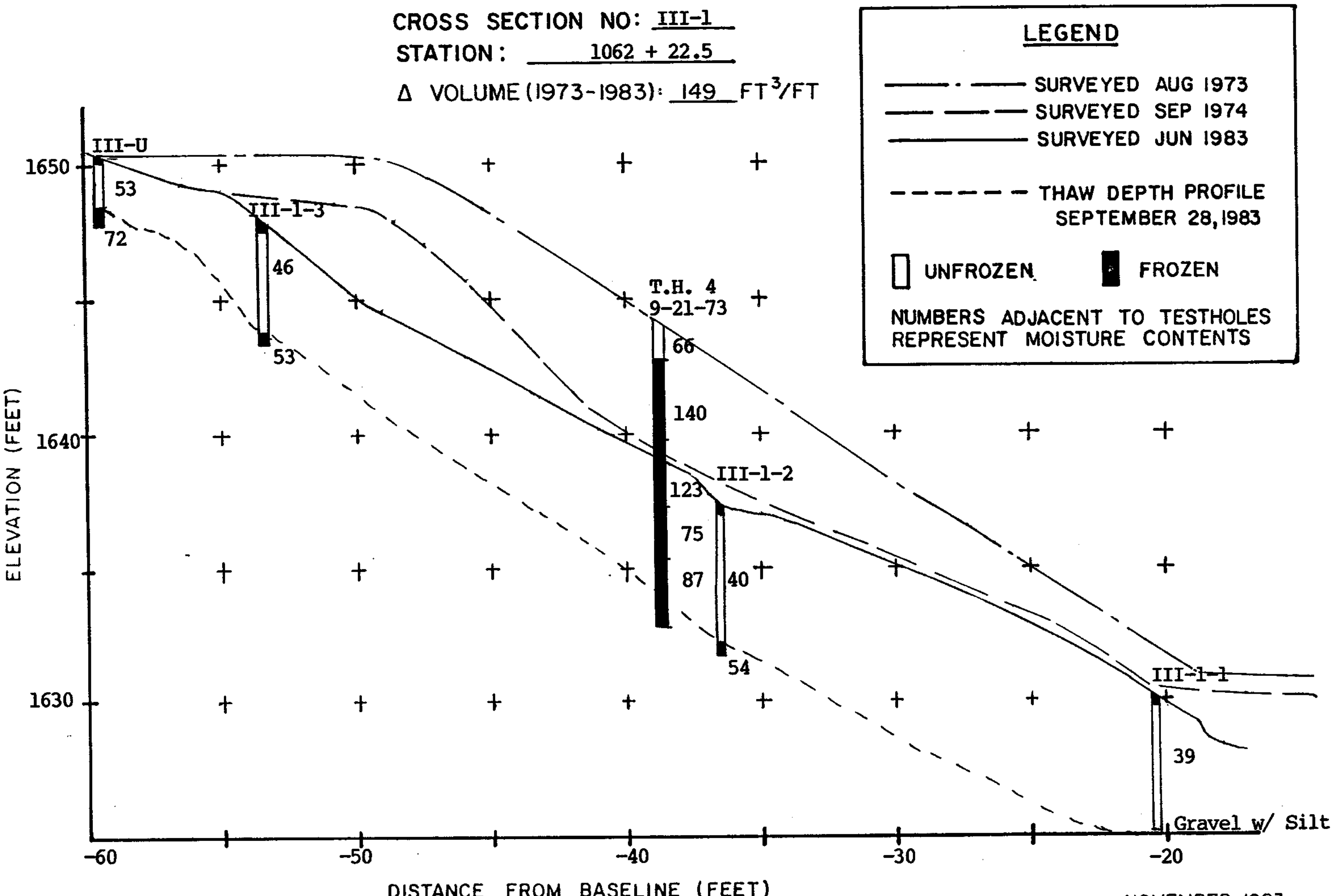
FIGURE A-7

SLOPE CONFIGURATIONS AND THAW PROFILE

CROSS SECTION NO: III-1

STATION: 1062 + 22.5

Δ VOLUME (1973-1983): 149 FT^3/FT



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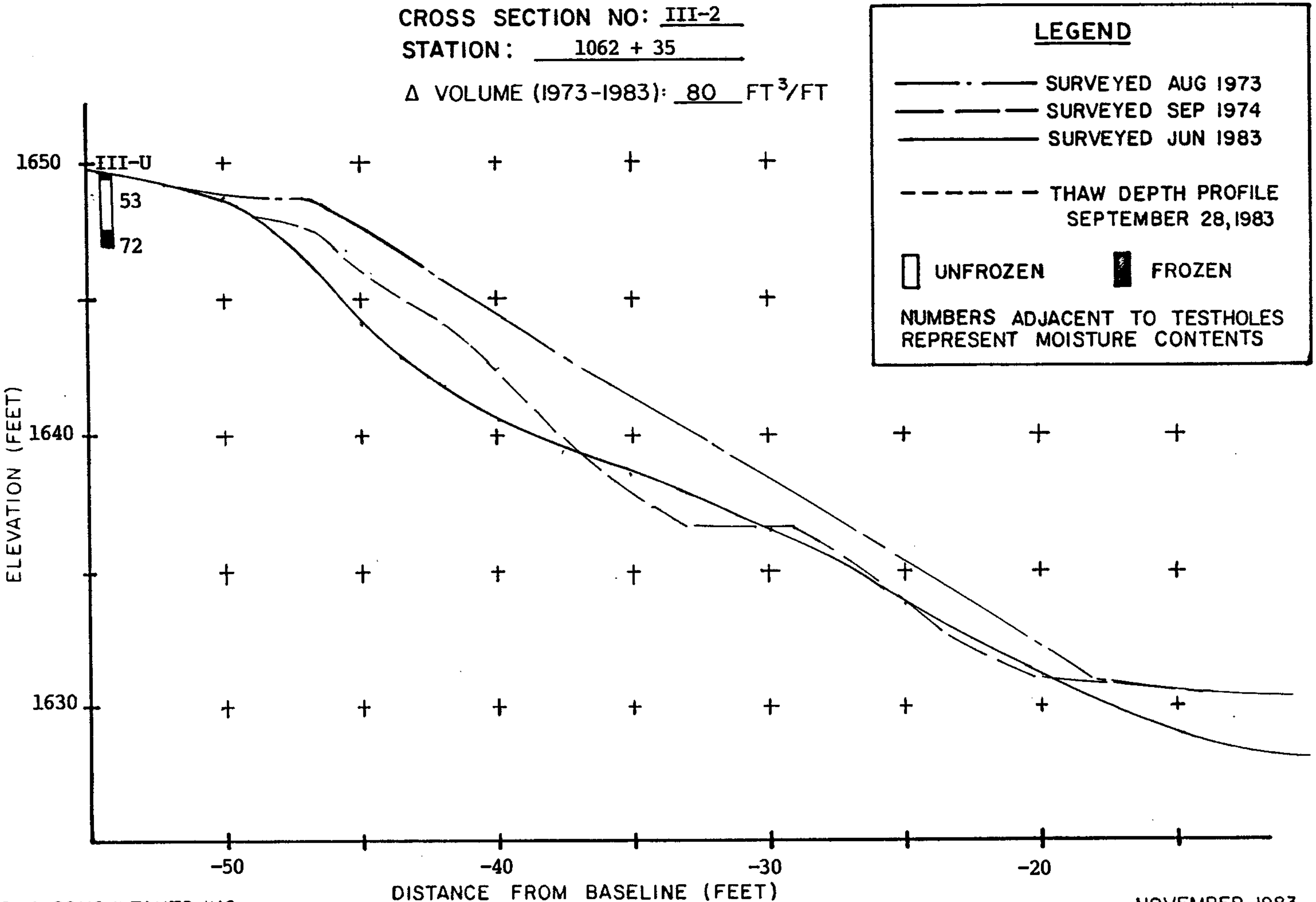
HESS CREEK

FIGURE A-8

SLOPE CONFIGURATIONS AND THAW PROFILE

CROSS SECTION NO: III-2
STATION: 1062 + 35
Δ VOLUME (1973-1983): 80 FT^3/FT



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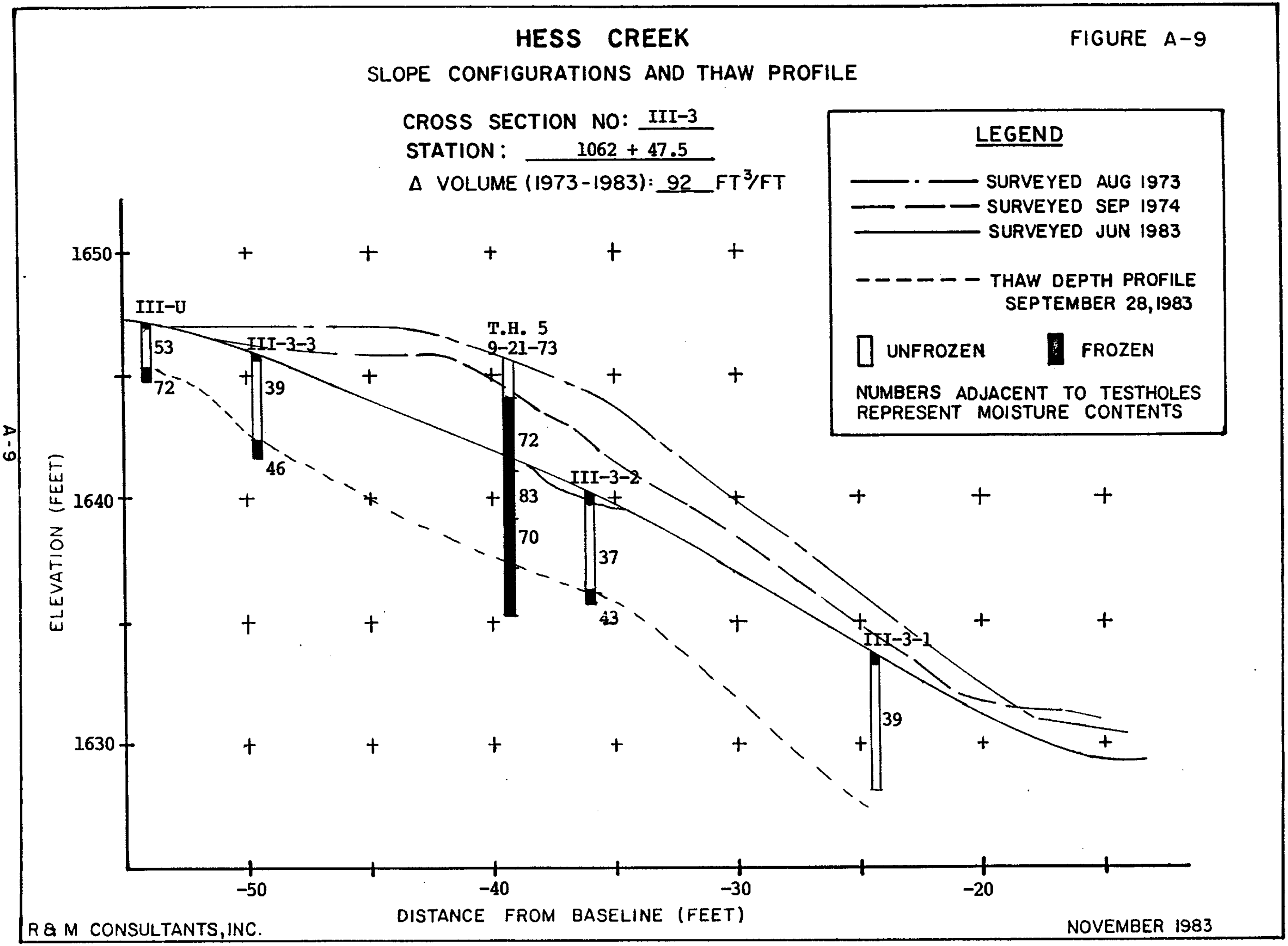
HESS CREEK
FIGURE A-9
SLOPE CONFIGURATIONS AND THAW PROFILE
CROSS SECTION NO: III-3
STATION: 1062 + 47.5
Δ VOLUME (1973-1983): 92 FT³/FT
LEGEND
SURVEYED AUG 1973
SURVEYED SEP 1974
SURVEYED JUN 1983
THAW DEPTH PROFILE SEPTEMBER 28, 1983
UNFROZEN
FROZEN
NUMBERS ADJACENT TO TESTHOLES REPRESENT MOISTURE CONTENTS
III-U
53
72
III-3-3
39
46
T.H. 5
9-21-73
72
83
70
III-3-2
37
43
III-3-1
39
1650
1640
1630
ELEVATION (FEET)
-50
-40
-30
-20
DISTANCE FROM BASELINE (FEET)
R & M CONSULTANTS, INC.
NOVEMBER 1983

HESS CREEK

FIGURE A-10

SLOPE CONFIGURATIONS AND THAW PROFILE

CROSS SECTION NO: IV-1
STATION: 1063 + 97.5
Δ VOLUME (1973-1983): 31 FT^3/FT

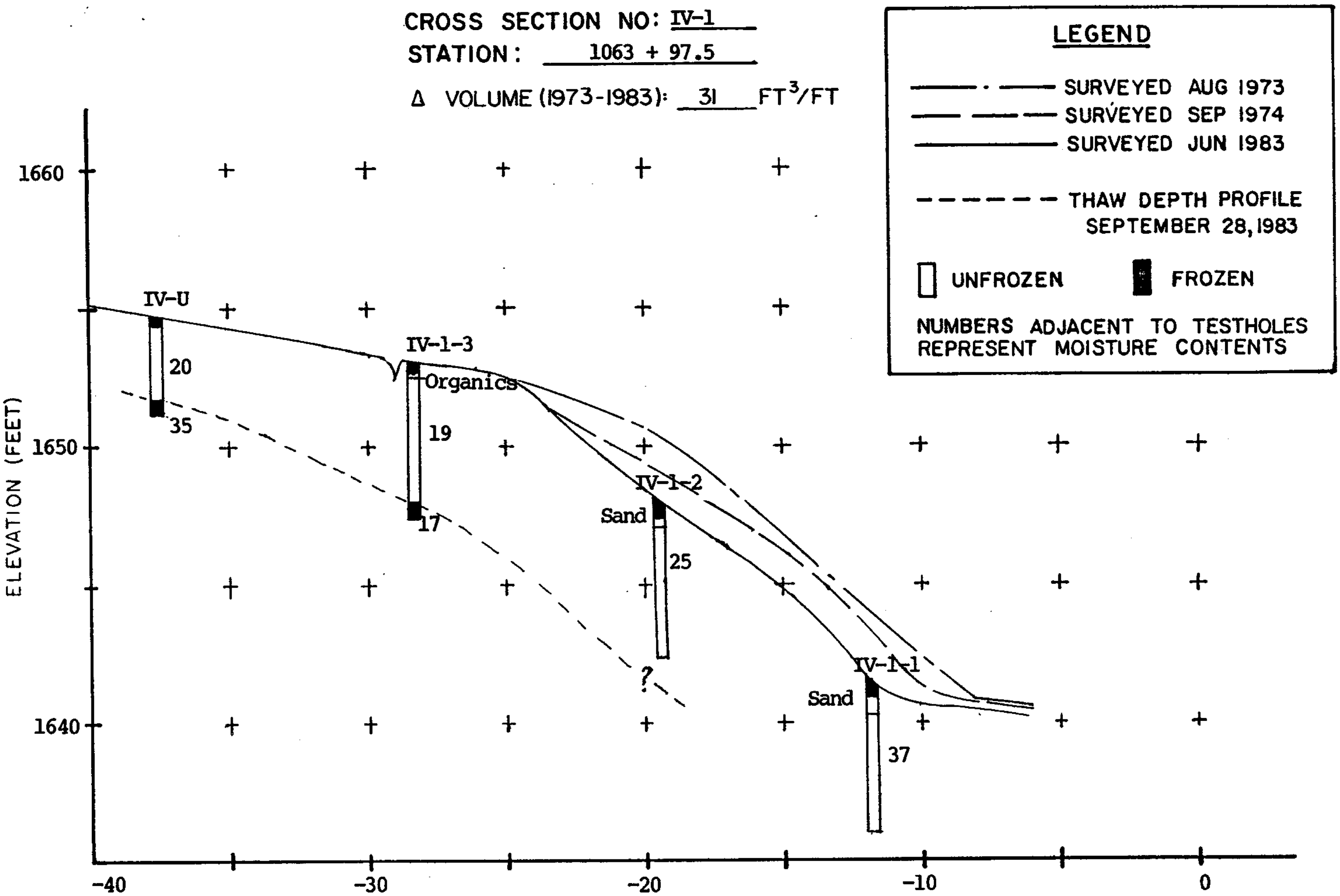

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FIGURE A-11

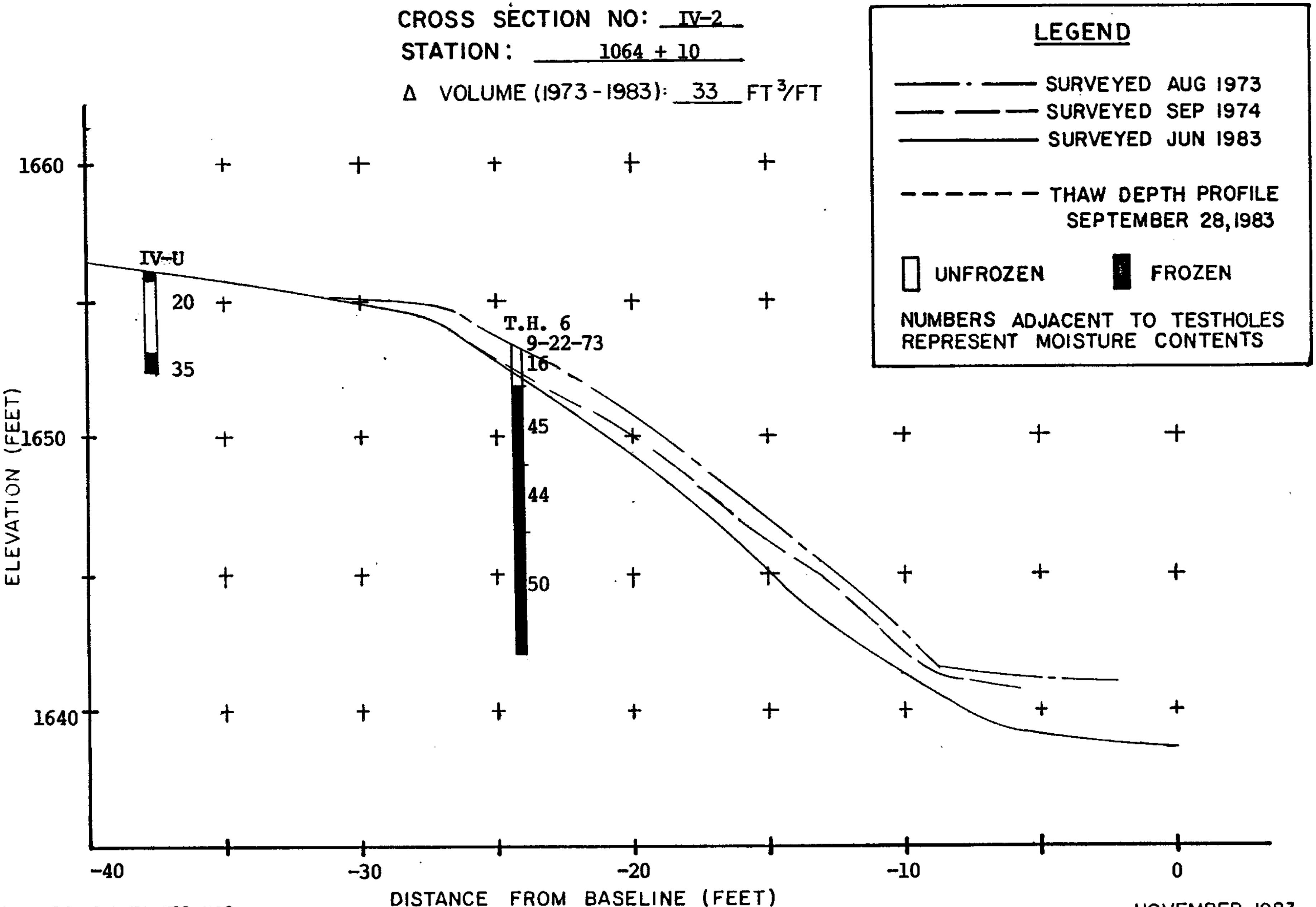
HESS CREEK

SLOPE CONFIGURATIONS AND THAW PROFILE

CROSS SECTION NO: IV-2

STATION: 1064 + 10

Δ VOLUME (1973-1983): 33 FT³/FT



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HESS CREEK

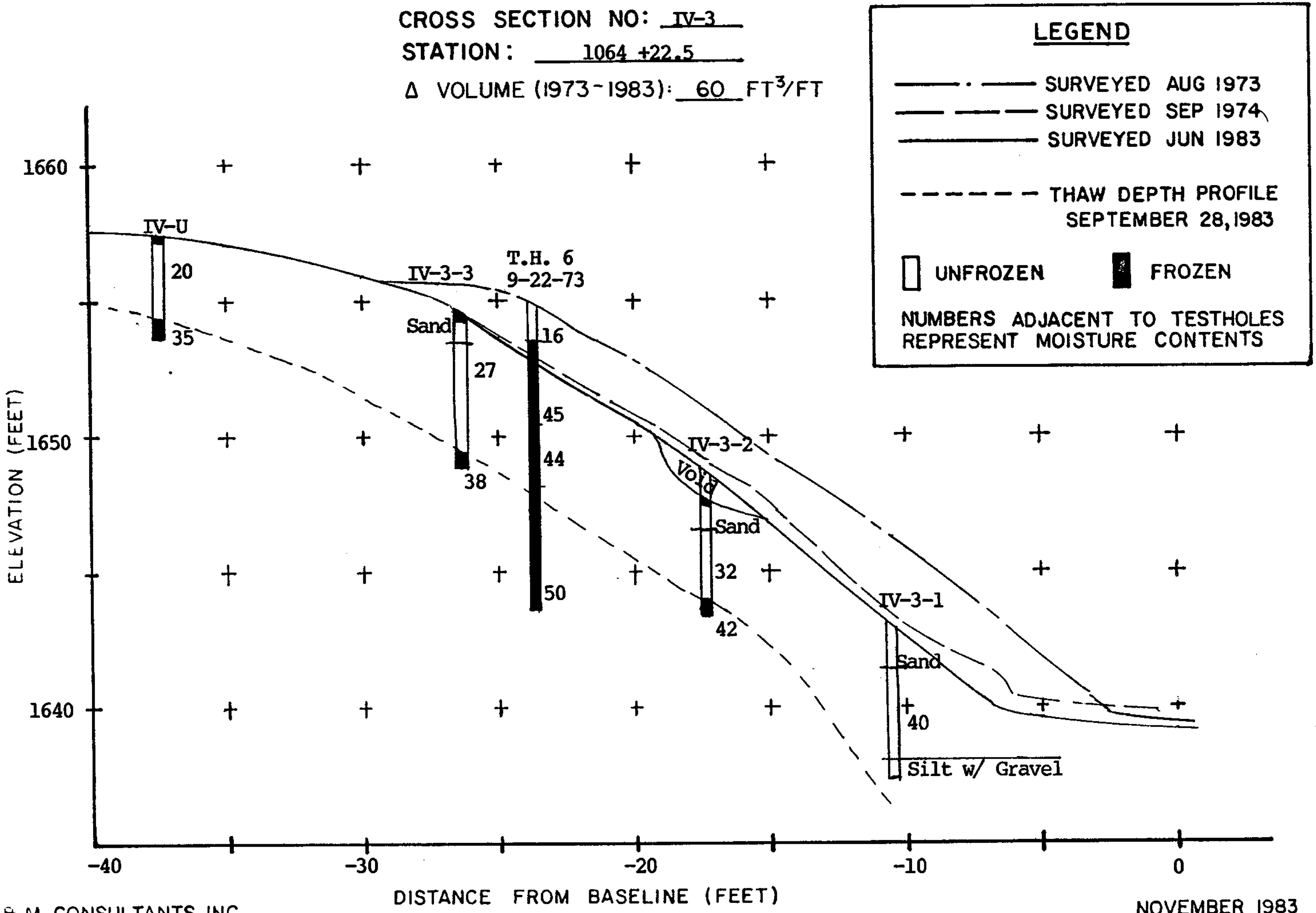
FIGURE A-12

SLOPE CONFIGURATIONS AND THAW PROFILE

CROSS SECTION NO: IV-3

STATION: 1064 +22.5

Δ VOLUME (1973-1983): 60 FT^3/FT



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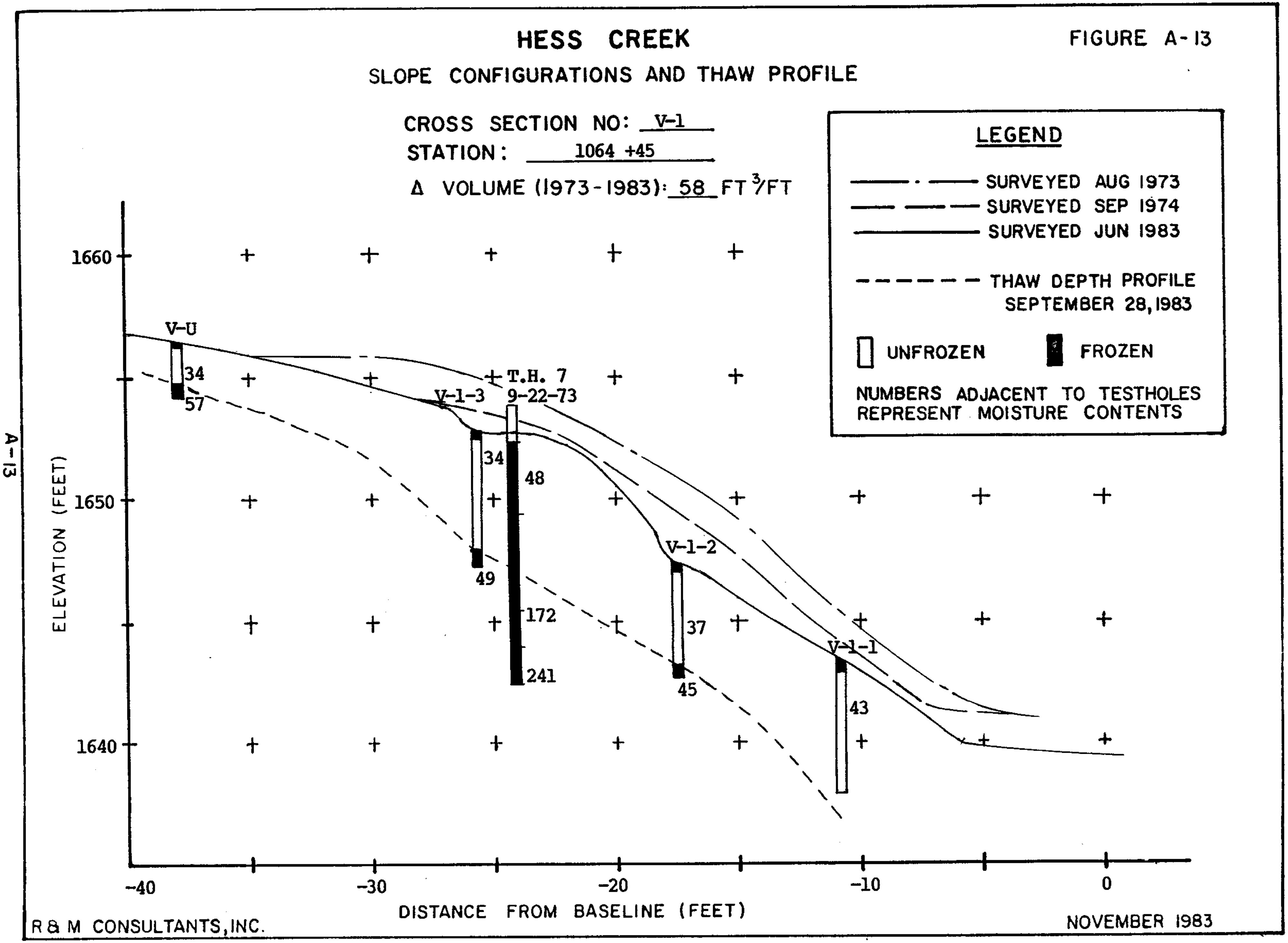


HESS CREEK
FIGURE A-13
SLOPE CONFIGURATIONS AND THAW PROFILE
CROSS SECTION NO: V-1
STATION: 1064 +45
Δ VOLUME (1973-1983): 58 FT³/FT
LEGEND
SURVEYED AUG 1973
SURVEYED SEP 1974
SURVEYED JUN 1983
THAW DEPTH PROFILE SEPTEMBER 28, 1983
UNFROZEN
FROZEN
NUMBERS ADJACENT TO TESTHOLES REPRESENT MOISTURE CONTENTS
ELEVATION (FEET)
1660
1650
1640
V-U
34
57
V-1-3
34
49
T.H. 7
9-22-73
48
172
241
V-1-2
37
45
V-1-1
43
-40
-30
-20
-10
0
DISTANCE FROM BASELINE (FEET)
R & M CONSULTANTS, INC.
NOVEMBER 1983

HESS CREEK

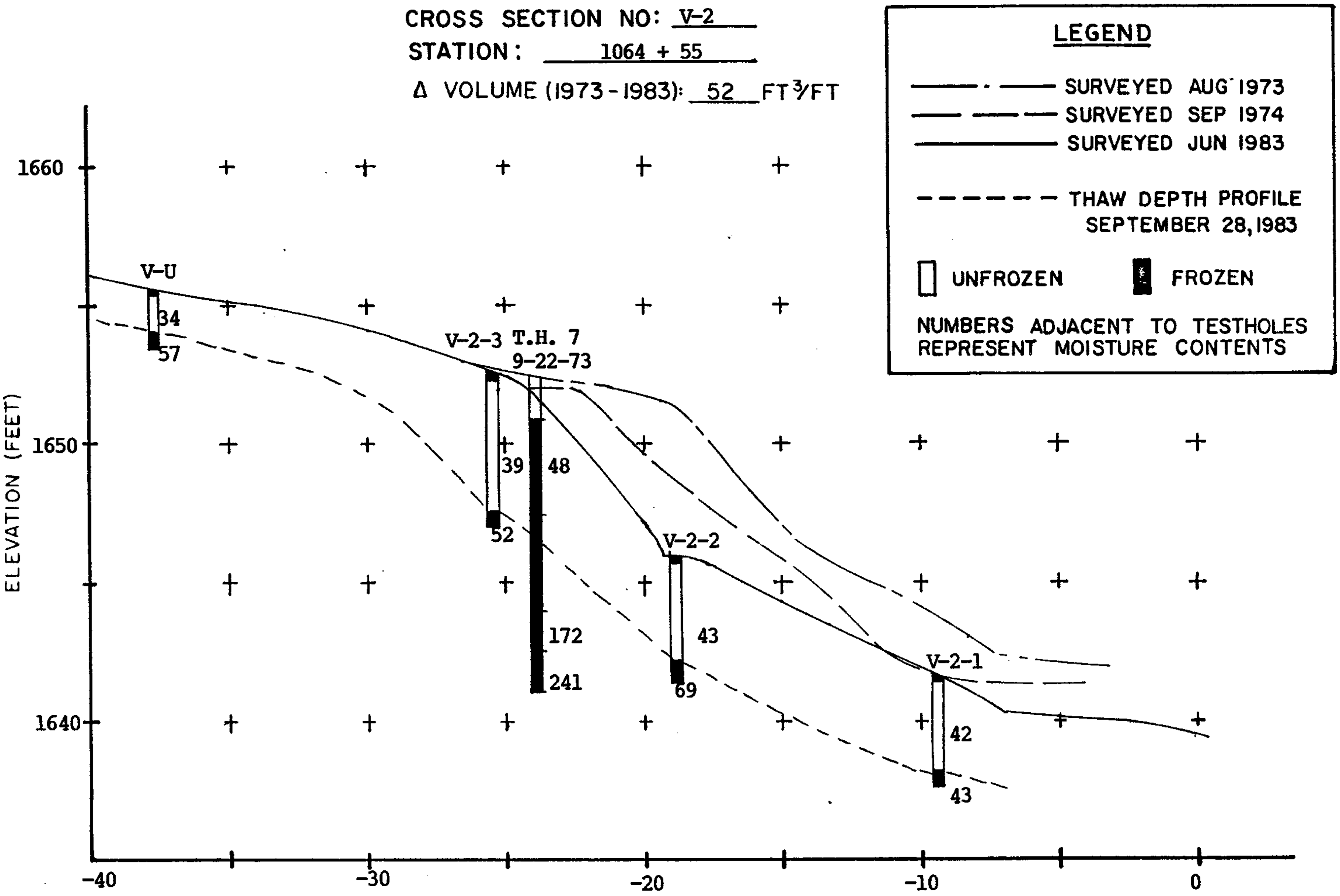
FIGURE A-14

SLOPE CONFIGURATIONS AND THAW PROFILE

CROSS SECTION NO: V-2

STATION: 1064 + 55

Δ VOLUME (1973-1983): 52 FT³/FT



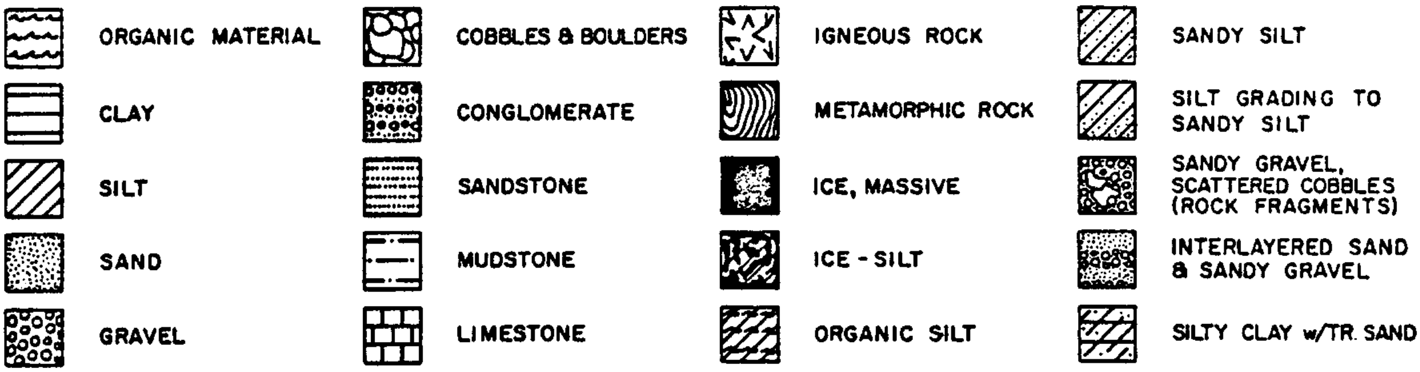
R & M CONSULTANTS, INC.

NOVEMBER 1983

APPENDIX B

LOGS OF TEST HOLES (1973)

STANDARD SYMBOLS



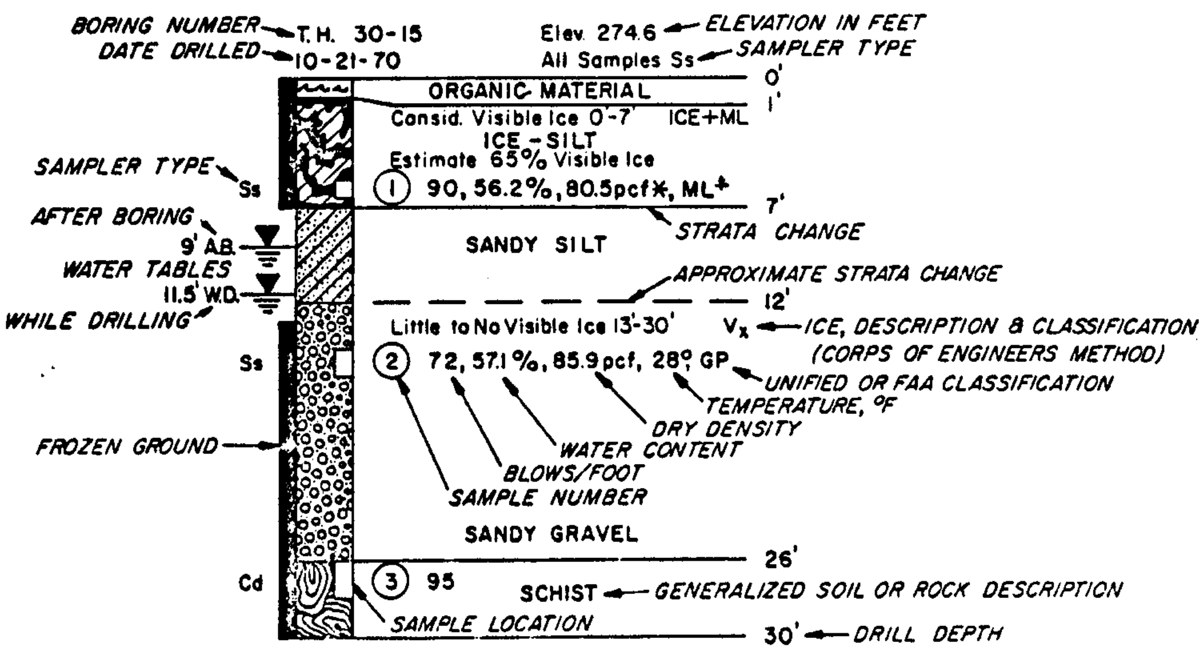
SAMPLER TYPE SYMBOLS

St . . . . . 1.4" SPLIT SPOON WITH 47 # HAMMER
Ss . . . . . 1.4" SPLIT SPOON WITH 140 # HAMMER
Sl . . . . . 2.5" SPLIT SPOON WITH 140 # HAMMER
Sh . . . . . 2.5" SPLIT SPOON WITH 340 # HAMMER
Sx . . . . . 2.0" SPLIT SPOON WITH 140 # HAMMER
Sz . . . . . 1.4" SPLIT SPOON WITH 340 # HAMMER
Sp . . . . . 2.5" SPLIT SPOON, PUSHED
Hs . . . . . 1.4" SPLIT SPOON DRIVEN WITH AIR HAMMER
Hl . . . . . 2.5" SPLIT SPOON DRIVEN WITH AIR HAMMER

Ts . . . . SHELBY TUBE
Tm . . . . MODIFIED SHELBY TUBE
Pb . . . . PITCHER BARREL
Cs . . . . CORE BARREL WITH SINGLE TUBE
Cd . . . . CORE BARREL WITH DOUBLE TUBE
Bs . . . . BULK SAMPLE
A. . . . . AUGER SAMPLE
G. . . . . GRAB SAMPLE

NOTE: SAMPLER TYPES ARE EITHER NOTED ABOVE THE BORING LOG OR ADJACENT TO IT AT THE RESPECTIVE SAMPLE DEPTH.

TYPICAL BORING LOG

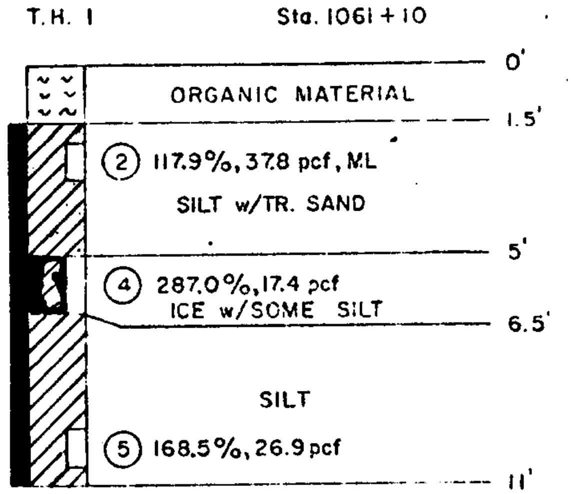


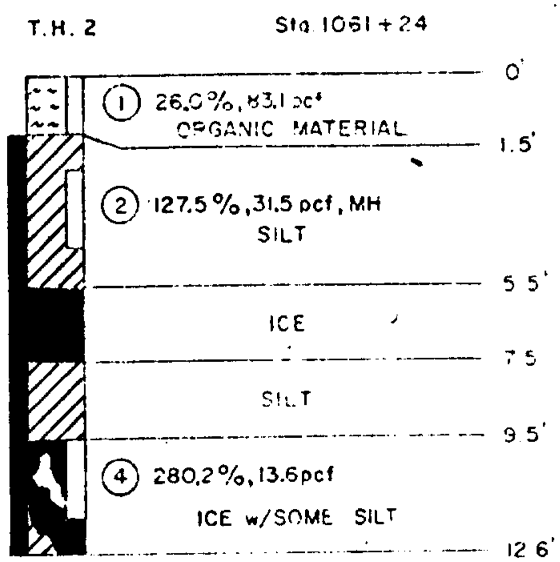
* *WEIGHTED AVERAGE*
+ *ADDITIONAL DATA AVAILABLE ON SUPPLEMENTAL LAB SHEETS*

R & M CONSULTANTS, INC.

EXPLANATION OF SELECTED SYMBOLS

2-16-72	SCALE NONE	DWN BY LDS	CHKD BY GLB	PROJ. NO GENERAL	DWG. NO B-1





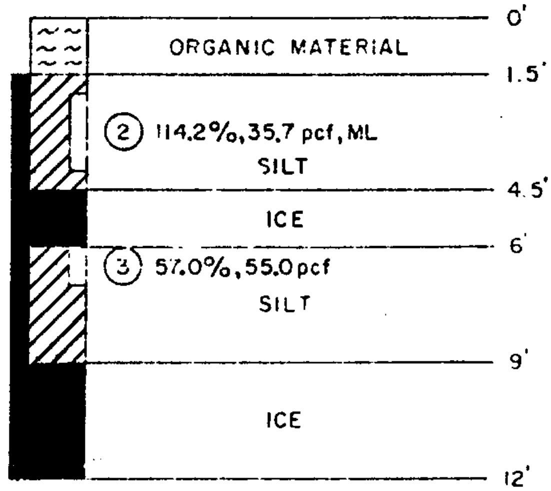
R & M CONSULTANTS, INC

HESS CREEK THERMAL EROSION STUDY

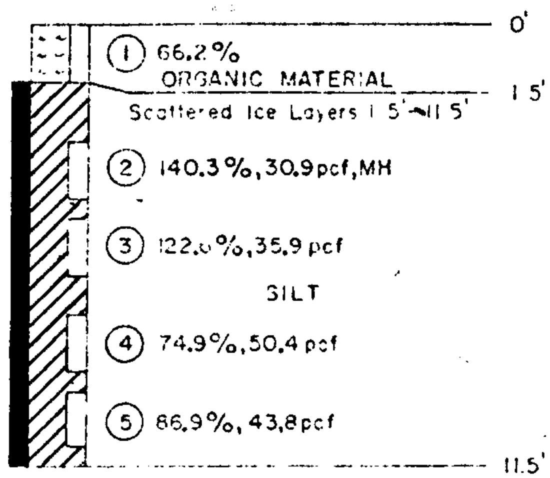
1973 BORING LOGS

[illegible]	SCALE 1"=5' VERT.	DWN BY LDS	CHKD BY JSS	PROJ. NO. 452930	DWG. NO. B-2

T.H. 3 Sta. 1061 + 74



T.H. 4 Sta. 1062+24

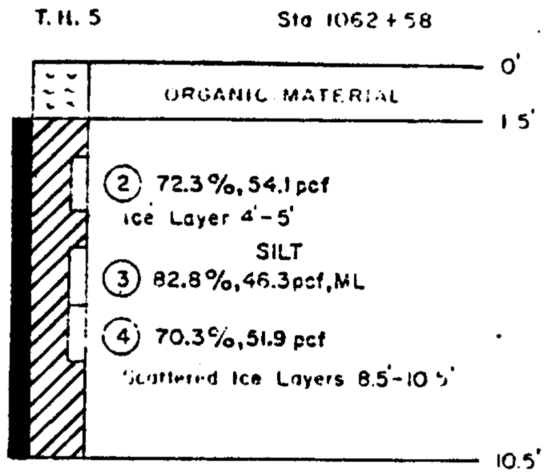


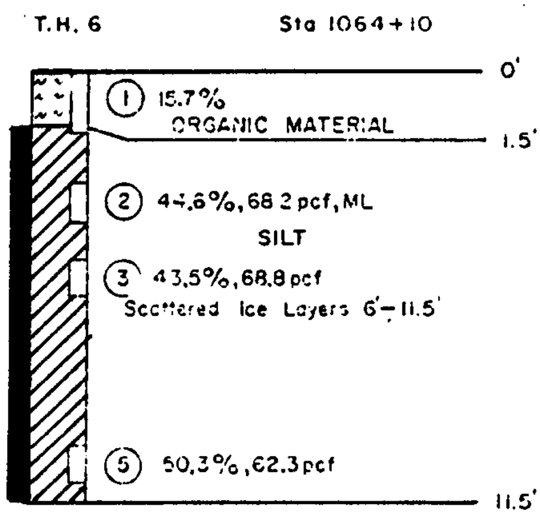
R & M CONSULTANTS, INC.

HESS CREEK THERMAL EROSION STUDY

1973 BORING LOGS

DATE 5-14-75	SCALE 1"=5' VERT.	DWN BY LDS	CHKD BY JSS	PROJ. NO. 45[illegible]	DWG NO. B-3



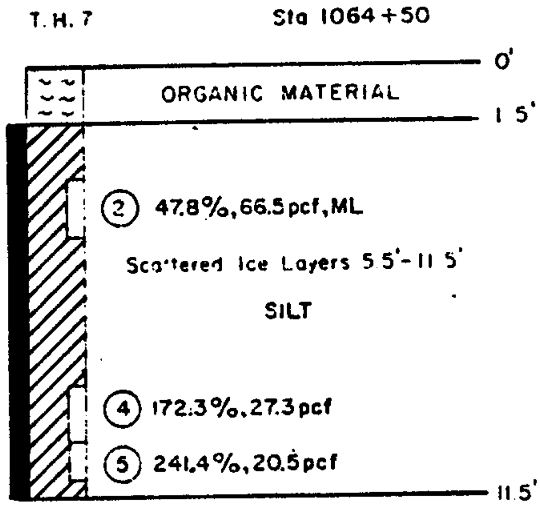


R & M CONSULTANTS, INC.

HESS CREEK THERMAL EROSION STUDY

1973 BORING LOGS

DATE 5-14-75	SCALE 1"=5' VERT.	DWN BY LDS	CHKD BY JSS	FILE NO 452930	DWG NO. B-4



HESS CREEK THERMAL EROSION STUDY

1973 BORING LOGS

R & M CONSULTANTS, INC.

[illegible] -75	SCALE 1" = 5' VERT	DWN BY LDS	CHKD BY JSS	PROJ NO 452930	DWG NO. B-5

APPENDIX C

LOGS OF TEST PROBES (1983)

STANDARD SYMBOLS

ORGANIC MATERIAL	COBBLES & BOULDERS	IGNEOUS ROCK	SANDY SILT
CLAY	CONGLOMERATE	METAMORPHIC ROCK	SILT GRADING TO SANDY SILT
SILT	SANDSTONE	ICE, MASSIVE	SANDY GRAVEL, SCATTERED COBBLES (ROCK FRAGMENTS)
SAND	MUDSTONE	ICE - SILT	INTERLAYERED SAND & SANDY GRAVEL
GRAVEL	LIMESTONE	ORGANIC SILT	SILTY CLAY w/TR. SAND

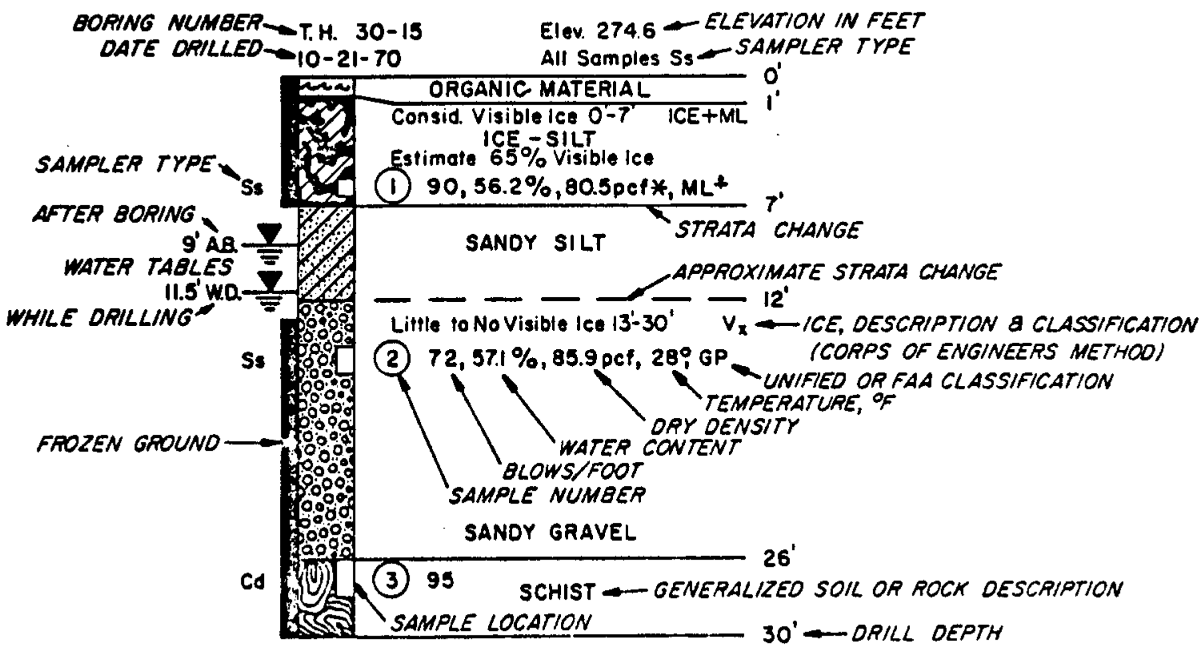
SAMPLER TYPE SYMBOLS

St . . . . . 1.4" SPLIT SPOON WITH 47 # HAMMER
Ss . . . . . 1.4" SPLIT SPOON WITH 140 # HAMMER
Sl . . . . . 2.5" SPLIT SPOON WITH 140 # HAMMER
Sb . . . . . 2.5" SPLIT SPOON WITH 340# HAMMER
Sx . . . . . 2.0" SPLIT SPOON WITH 140 # HAMMER
Sz . . . . . 1.4" SPLIT SPOON WITH 340# HAMMER
Sp . . . . . 2.5" SPLIT SPOON, PUSHED
Hs . . . . . 1.4" SPLIT SPOON DRIVEN WITH AIR HAMMER
Hl . . . . . 2.5" SPLIT SPOON DRIVEN WITH AIR HAMMER

Ts . . . . SHELBY TUBE
Tm . . . . MODIFIED SHELBY TUBE
Pb . . . . PITCHER BARREL
Cs . . . . CORE BARREL WITH SINGLE TUBE
Cd . . . . CORE BARREL WITH DOUBLE TUBE
Bs . . . . BULK SAMPLE
A. . . . . AUGER SAMPLE
G. . . . . GRAB SAMPLE

NOTE: SAMPLER TYPES ARE EITHER NOTED ABOVE THE BORING LOG OR ADJACENT TO IT AT THE RESPECTIVE SAMPLE DEPTH.

TYPICAL BORING LOG



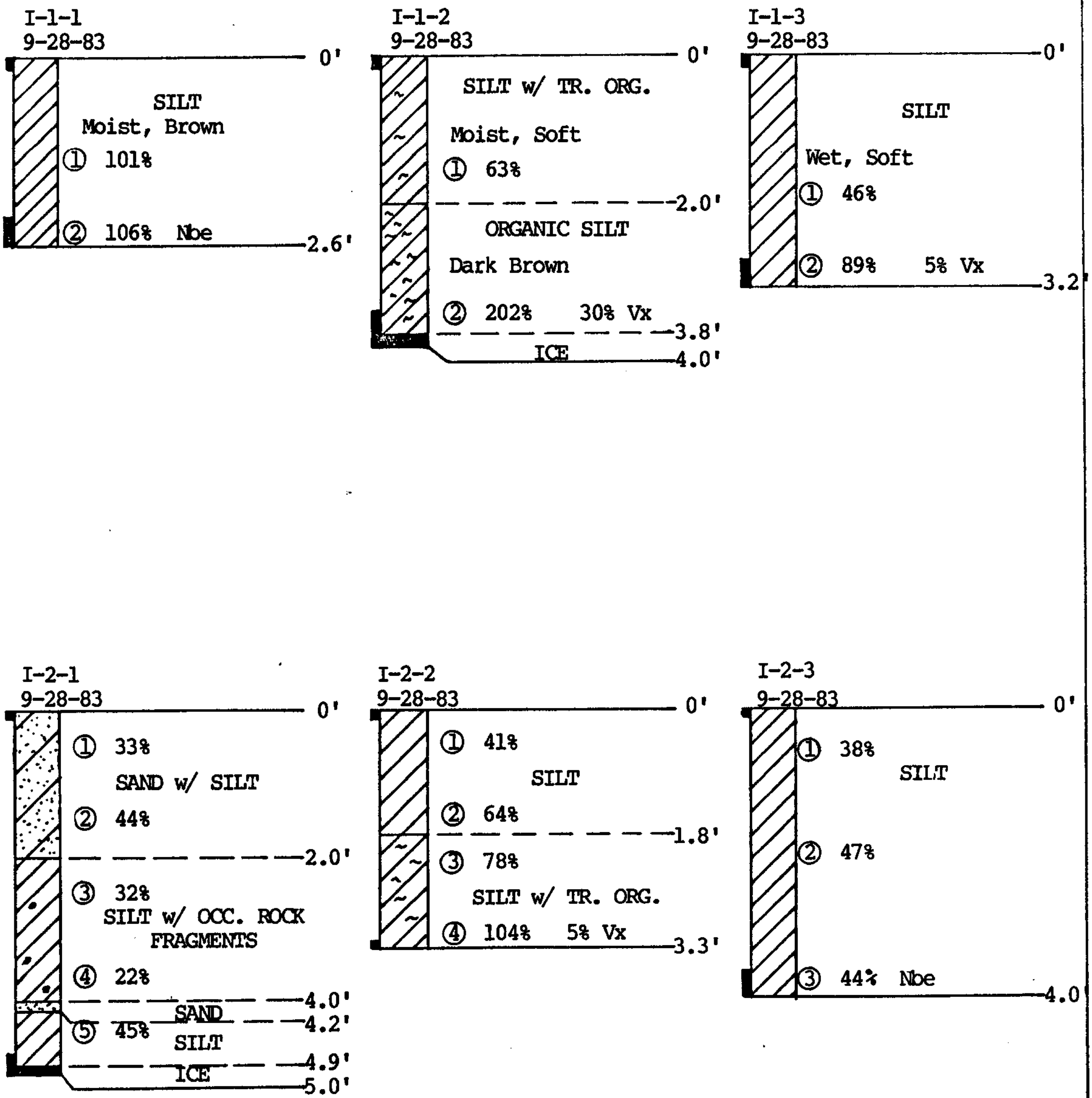
* WEIGHTED AVERAGE
+ ADDITIONAL DATA AVAILABLE ON SUPPLEMENTAL LAB SHEETS

R & M CONSULTANTS, INC.

EXPLANATION OF SELECTED SYMBOLS

DATE 2-16-72	SCALE NONE	DWN BY LDS	CHK'D B GLB	PROJ. NO. GENERAL	DWG NO. C-1

R & M CONSULTANTS, INC.

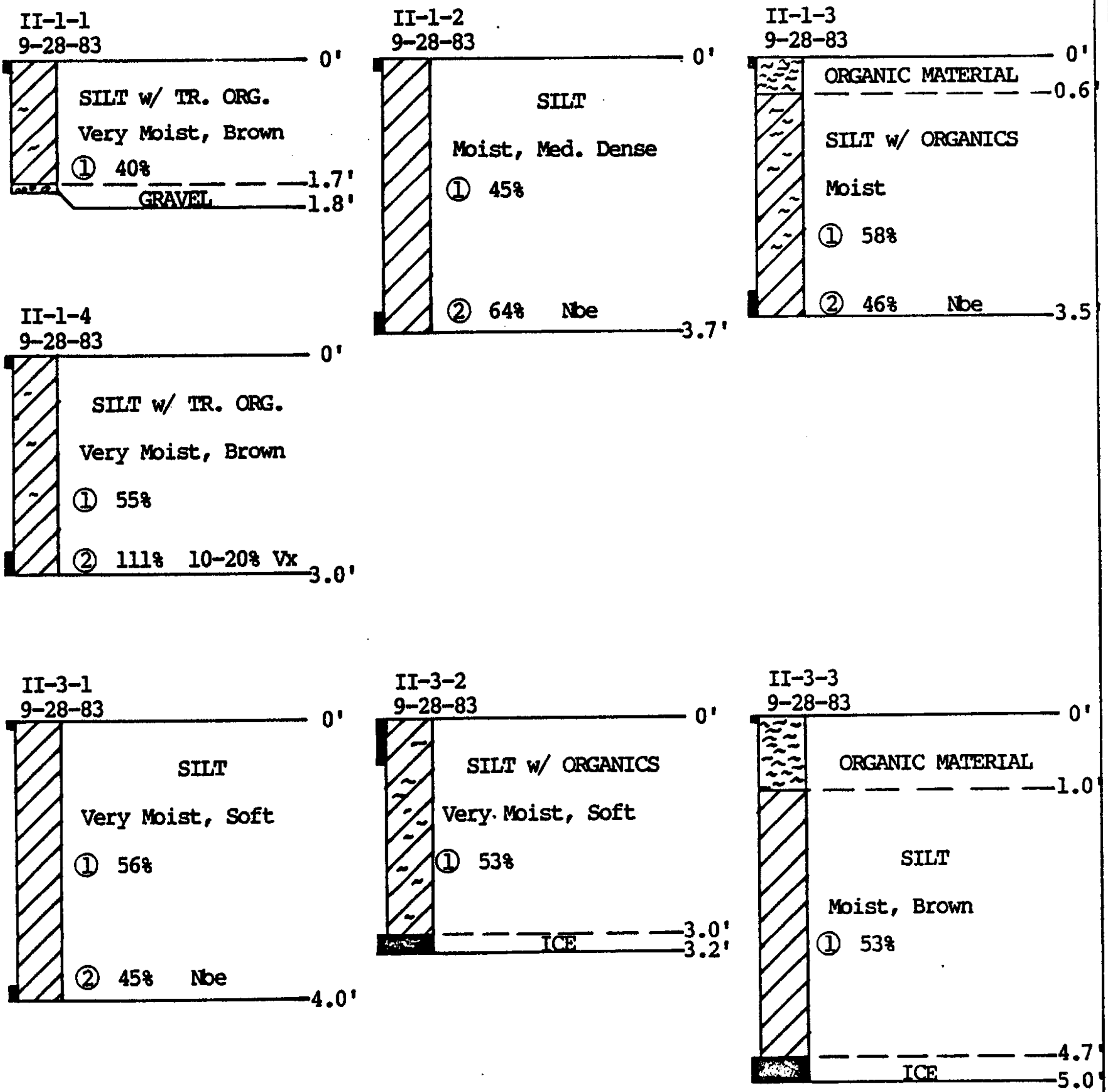


LOGS OF TEST PROBES

TEST SECTION I

DATE	SCALE	DRAWN BY	CHECKED BY	PROJECT NO.	DRAWING NO.
11-10-83	1" = 2'	DWM	JWR	312163	C-2

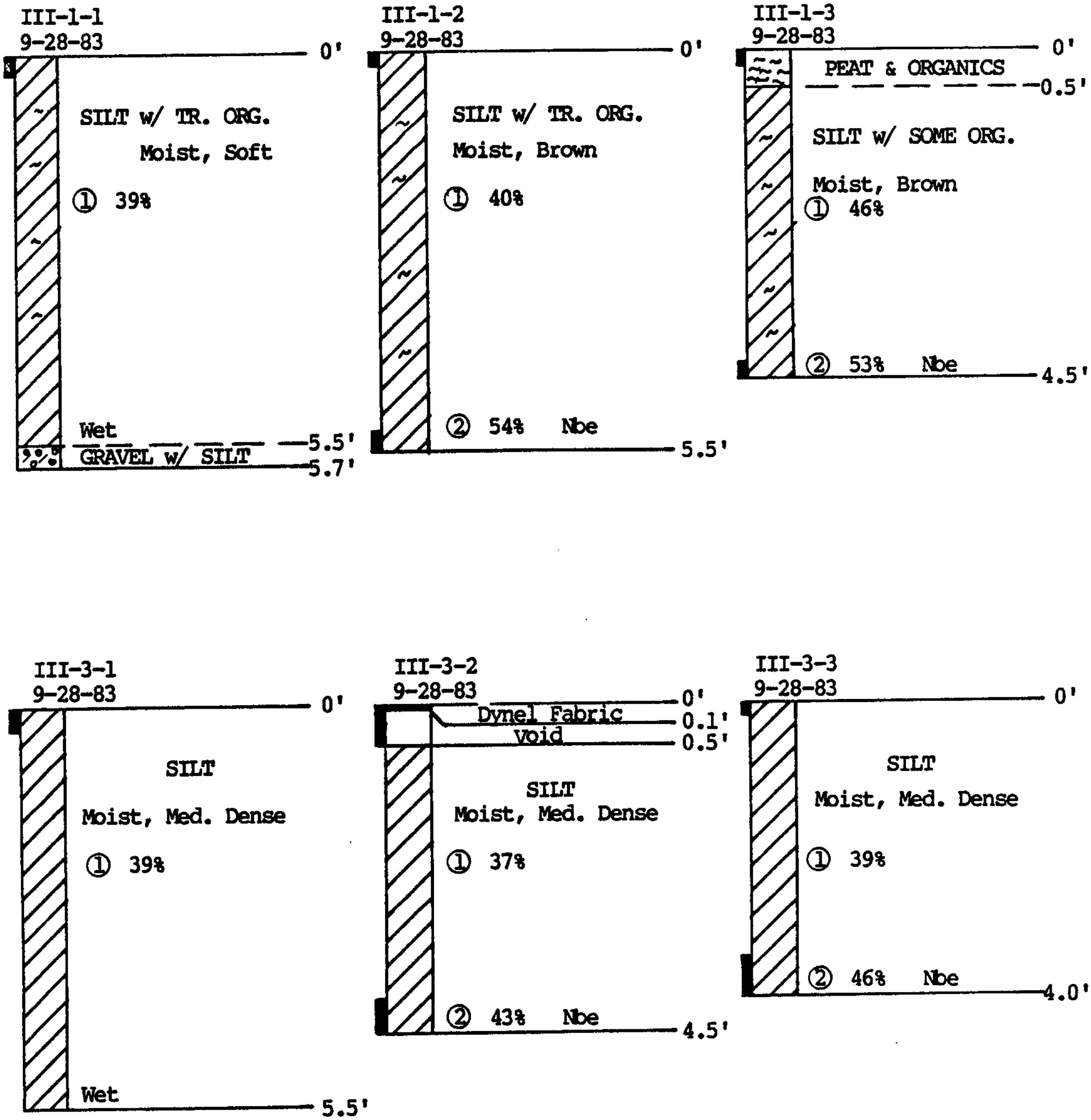
R & M CONSULTANTS, INC.



LOGS OF TEST PROBES
TEST SECTION II

DATE	SCALE	DRAWN BY	CHECKED BY	PROJECT NO.	DRAWING NO.
11-10-83	1" = 2'	DWM	JWR	312163	C-3

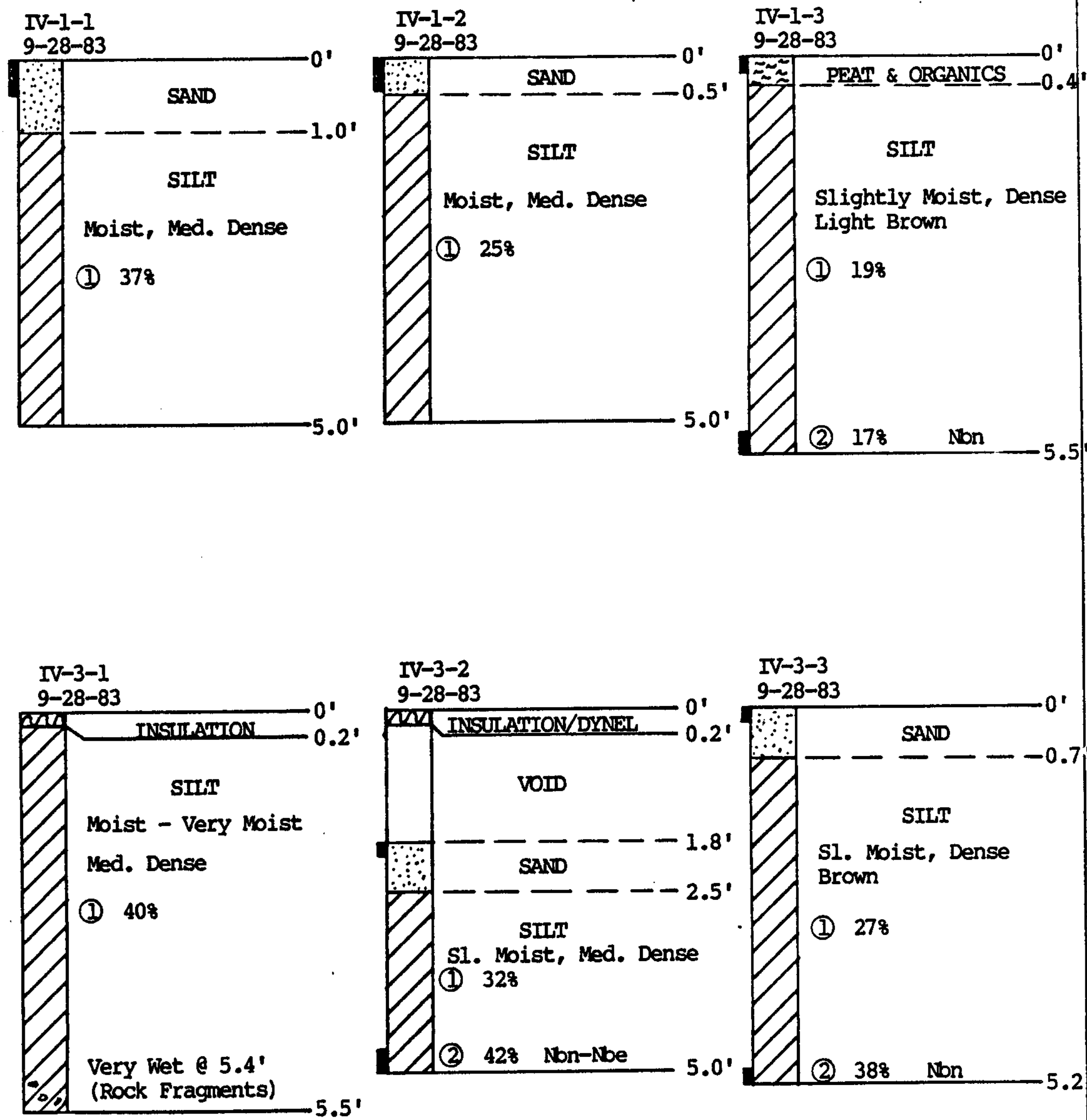
R&M CONSULTANTS, INC.



LOGS OF TEST PROBES
TEST SECTION III

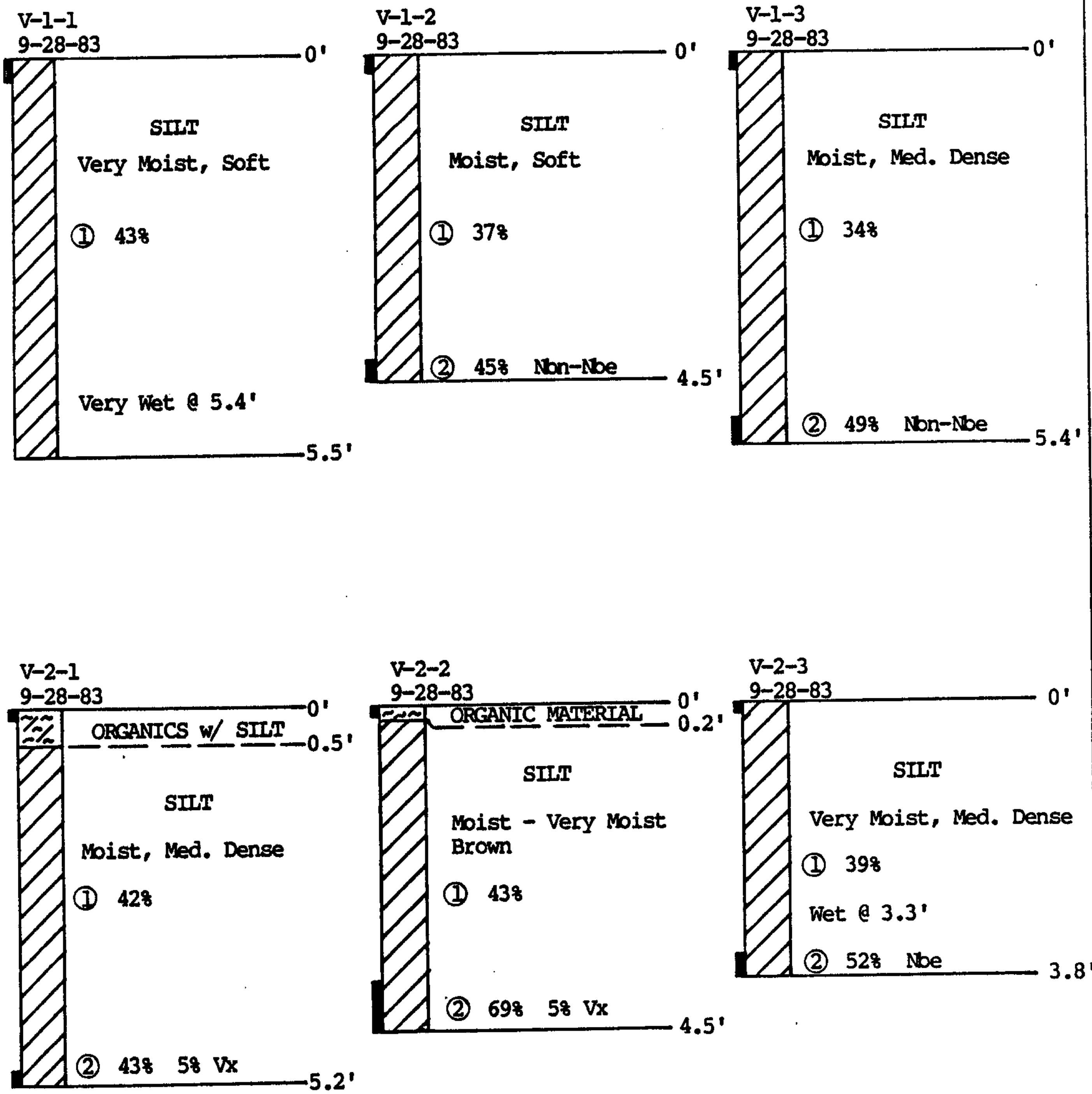
DATE	SCALE	DRAWN BY	CHECKED BY	PROJECT NO.	DRAWING NO.
11-10-83	1" = 2'	DWM	JWR	312163	C-4

R&M
R & M CONSULTANTS, INC.



LOGS OF TEST PROBES
TEST SECTION IV

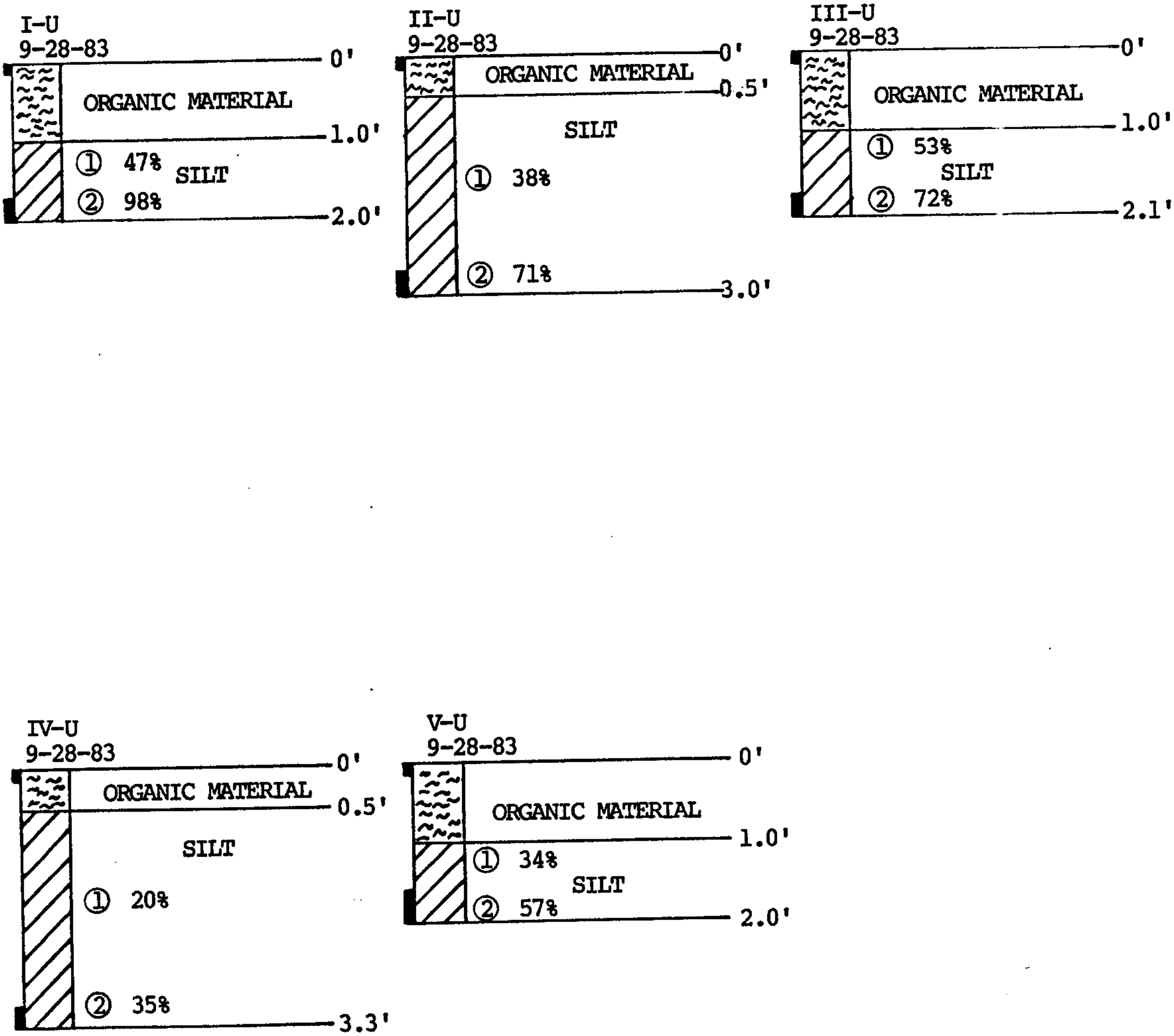
DATE	SCALE	DRAWN BY	CHECKED BY	PROJECT NO.	DRAWING NO.
11-10-83	1" = 2'	DWM	JWR	312163	C-5



LOGS OF TEST PROBES
TEST SECTION V

DATE	SCALE	DRAWN BY	CHECKED BY	PROJECT NO.	DRAWING NO.
11-10-83	1" = 2'	DWM	JWR	312163	C-6

R&M CONSULTANTS, INC.



LOGS OF TEST PROBES
TEST PROBES IN UNDISTURBED AREAS

DATE	SCALE	DRAWN BY	CHECKED BY	PROJECT NO.	DRAWING NO.
11-10-83	1" = 2'	DWM	JWR	312163	C-7

APPENDIX D

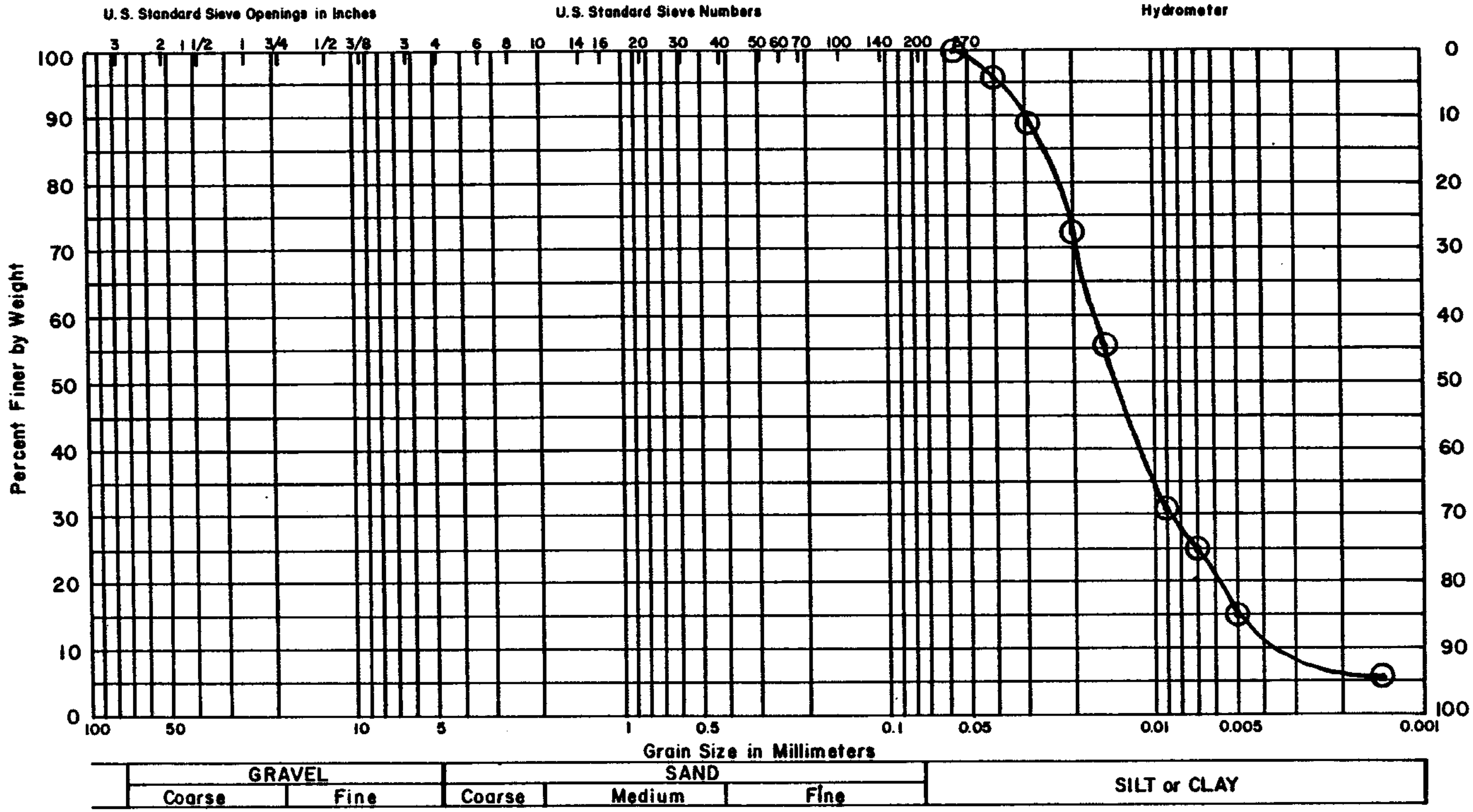
LABORATORY TEST RESULTS (1983)

APPENDIX D

Summary of Laboratory Test Results (1983)

Test Probe No.	Sample No.	Interval (Feet)	Thermal State (U/F)	Moisture Content (%)	Comments
I-1-1	1	0.2-2.2	U	101	
	2	2.2-2.6	F	106	
I-1-2	1	0.2-3.5	U	63	
	2	3.5-4.0	F	202	
I-1-3	1	0.4-2.8	U	46	
	2	2.8-3.2	F	89	
I-2-1	1	0-1.0	U	33	
	2	1.0-2.0	U	44	
	3	2.0-3.0	U	32	Ave = 35%
	4	3.0-4.0	U	22	
	5	4.2-4.7	U	45	
I-2-2	1	0-1.0	U	41	
	2	1.0-2.0	U	64	Ave = 61%
	3	2.0-3.0	U	78	
	4	3.2-3.3	F	104	
I-2-3	1	0-2.0	U	38	Ave = 43%
	2	2.0-3.6	U	47	
	3	3.6-4.0	F	44	Clay Content = 7%; LL = 35%
I-U	1	1.0-1.7	U	47	
	2	1.7-2.0	F	98	
II-1-1	1	0.1-1.7	U	40	
II-1-2	1	0.1-3.4	U	45	
	2	3.4-3.7	F	64	
II-1-3	1	0.1-3.2	U	58	
	2	3.2-3.5	F	46	Clay Content= 12%
II-1-4	1	0.1-2.7	U	55	
	2	2.7-3.0	F	111	
II-3-1	1	0-3.8	U	56	
	2	3.8-4.0	F	45	
II-3-2	1	0.5-3.0	U	53	
II-3-3	1	1.0-4.7	U	53	
II-U	1	0.5-2.7	U	38	
	2	2.7-3.0	F	71	
III-1-1	1	0.2-5.2	U	39	
III-1-2	1	0.2-5.2	U	40	Clay Content = 11%; L = 26%
	2	5.2-5.5	F	54	
III-1-3	1	0.5-4.3	U	46	
	2	4.3-4.5	F	53	
III-3-1	1	0.4-5.0	U	39	
III-3-2	1	1.0-4.0	U	37	
	2	4.0-4.5	F	43	
III-3-3	1	0.5-3.5	U	39	
	2	3.5-4.0	F	46	
III-U	1	1.0-1.8	U	53	
	2	1.8-2.0	F	72	

Test Probe No.	Sample No.	Interval (Feet)	Thermal State (U/F)	Moisture Content (%)	Comments
IV-1-1	1	1.5-5.0	U	37	
IV-1-2	1	1.5-5.0	U	25	
IV-1-3	1	1.0-5.2	U	19	
	2	5.2-5.5	F	17	
IV-3-1	1	0.8-5.0	U	40	Clay Content = 9%; LL = 23%
IV-3-2	1	2.6-4.7	U	32	
	2	4.7-5.0	F	42	
IV-3-3	1	0.7-5.0	U	27	
	2	5.0-5.2	F	38	
IV-U	1	0.5-3.0	U	20	
	2	3.0-3.3	F	35	
V-1-1	1	0.4-5.0	U	43	
V-1-2	1	0.4-4.2	U	37	
	2	4.2-4.5	F	45	
V-1-3	1	0.3-5.0	U	34	
	2	5.0-5.4	F	49	
V-2-1	1	0.5-5.0	U	42	
	2	5.0-5.2	F	43	
V-2-2	1	0.2-3.8	U	43	Clay Content = 11%; LL = 31%
	2	3.8-4.5	F	69	
V-2-3	1	0.5-5.0	U	39	
	2	5.0-5.2	F	52	
V-U	1	1.0-1.6	U	34	
	2	1.6-1.8	F	57	
I (Insulation)		0-0.4	U	83	Dry Density = 1.42 pcf
IV (Insulation)		0-0.4	U	91	Dry Density = 1.35 pcf



SAMPLE NO.	MOISTURE CONTENT	DRY DENSITY	LL	PI	CLASSIFICATION & DESCRIPTION
I-2-2-3			35		

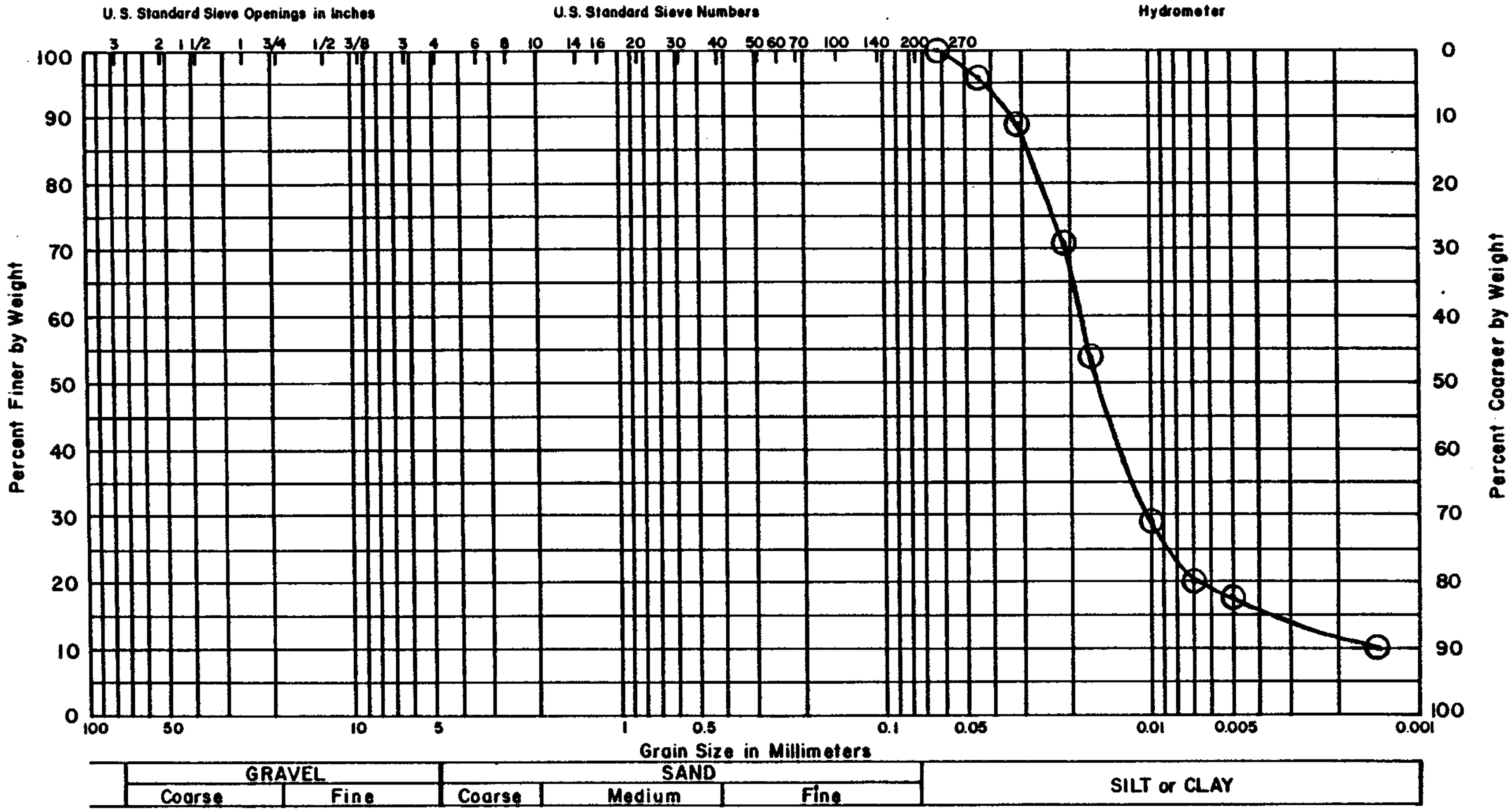
R&M

R&M CONSULTANTS, INC.

HESS CREEK - THERMAL EROSION TEST SITE

DRAWN BY R.J.L.
APPROVED BY D.W.M.
DATE 11-10-83
PROJECT NO. 312163

D-3

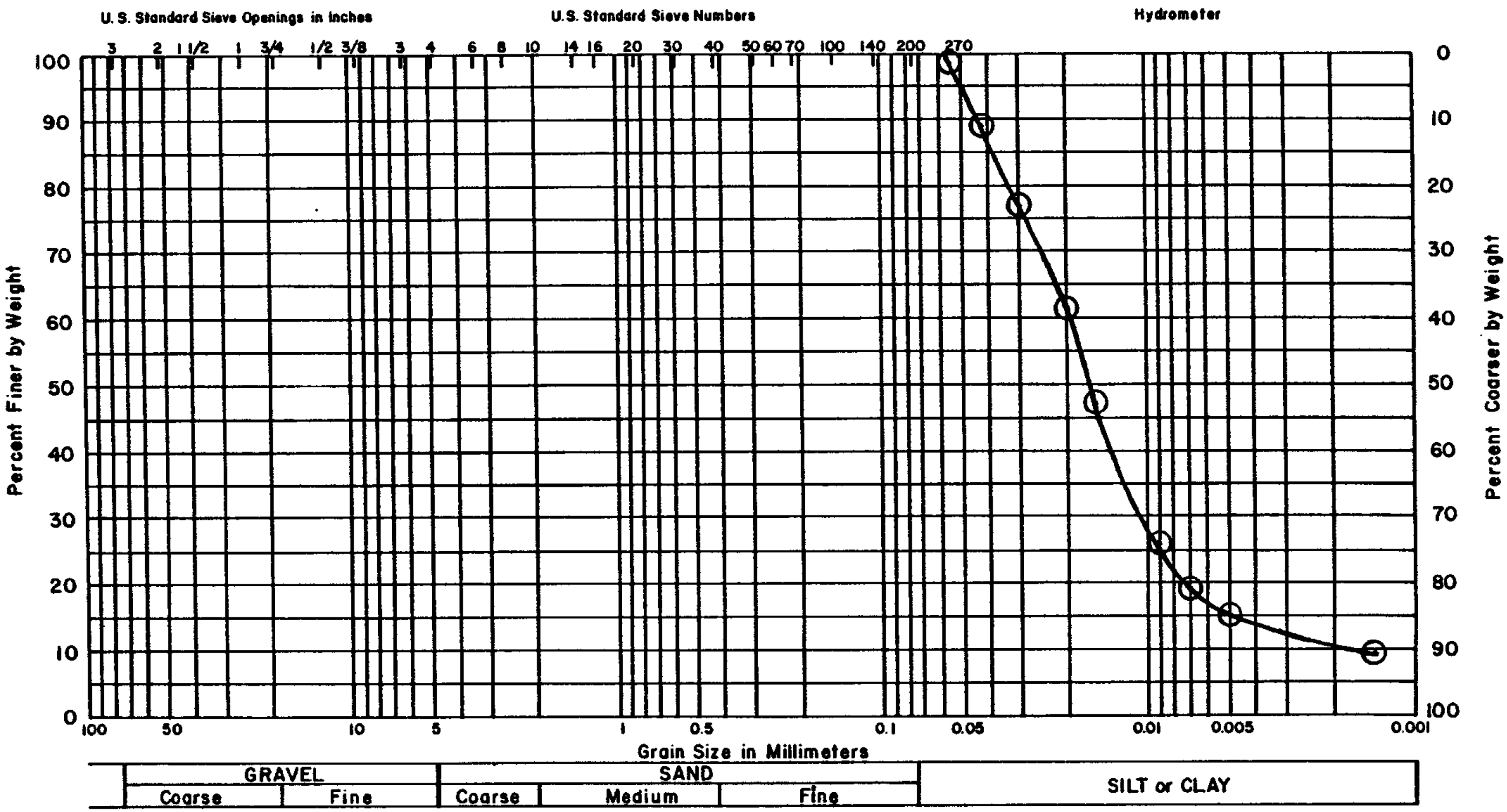


SAMPLE NO.	MOISTURE CONTENT	DRY DENSITY	LL	PI		CLASSIFICATION & DESCRIPTION
II-1-3-2						

R&M CONSULTANTS, INC.

HESS CREEK – THERMAL EROSION TEST SITE

DRAWN BY R.J.L.
APPROVED BY DWM
DATE 11-10-83
PROJECT NO. 312163

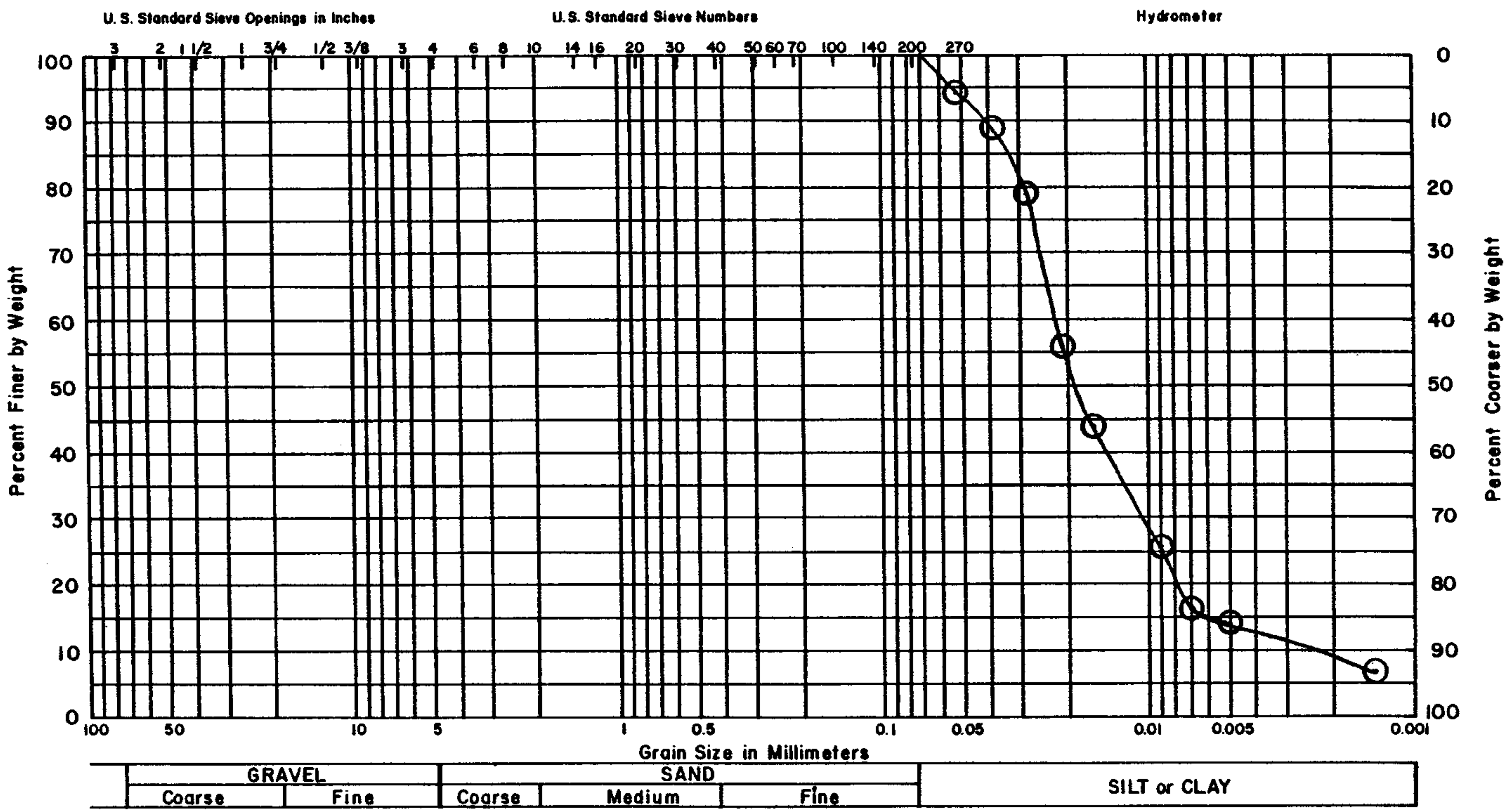


SAMPLE NO.	MOISTURE CONTENT	DRY DENSITY	LL	PI		CLASSIFICATION & DESCRIPTION
III-1-2-1			26			

R&M CONSULTANTS, INC.

HESS CREEK - THERMAL EROSION TEST SITE

DRAWN BY R.J.L.
APPROVED BY DWM
DATE 11-10-83
PROJECT NO. 312162



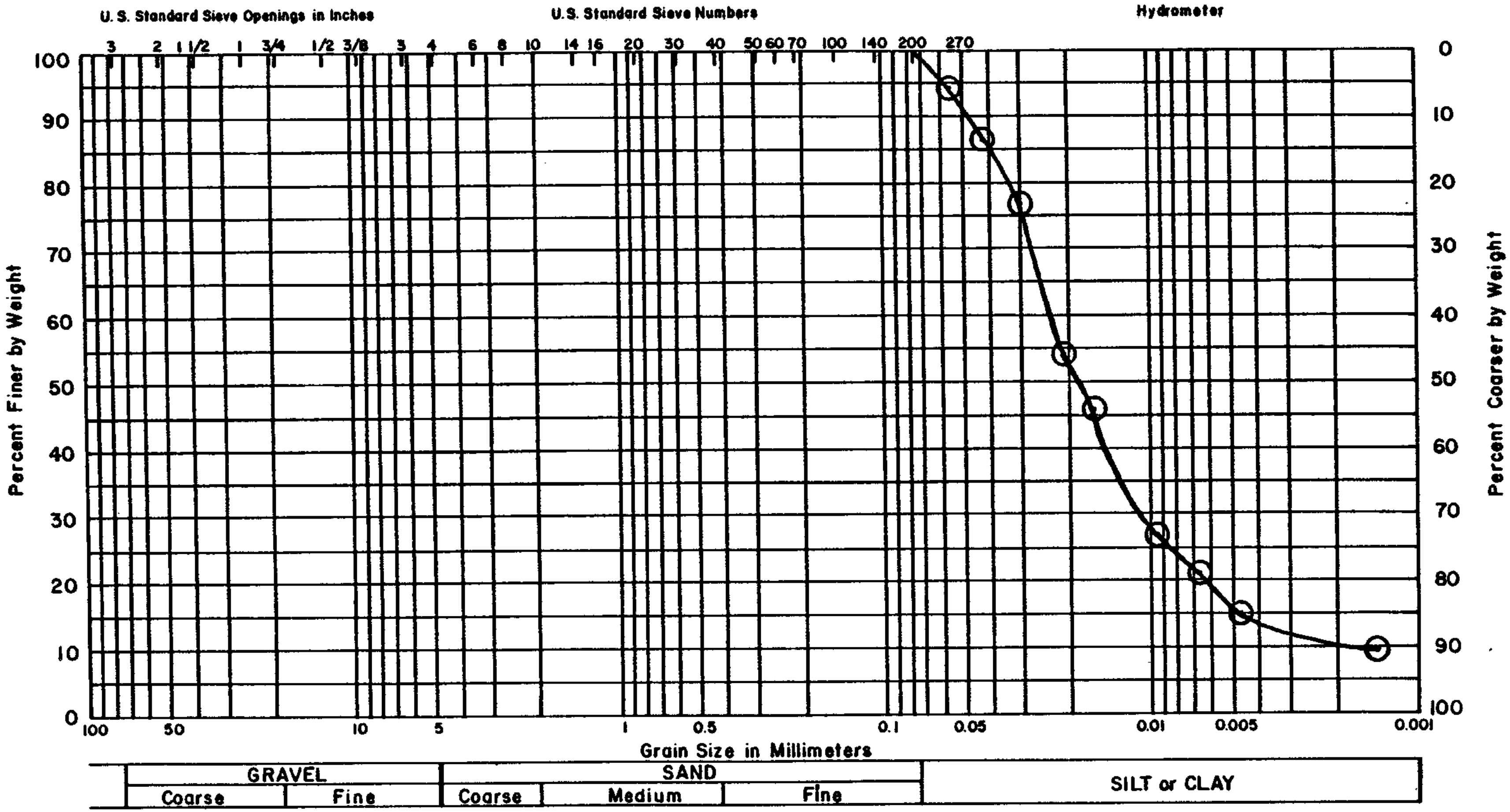
SAMPLE NO.	MOISTURE CONTENT	DRY DENSITY	LL	PI	CLASSIFICATION & DESCRIPTION
IV-3-1-1			23		

R&M CONSULTANTS, INC.

HESS CREEK - THERMAL EROSION TEST SITE

DRAWN BY R.J.L.
APPROVED BY DWM
DATE 11-10-83
PROJECT NO. 312163

D-7



SAMPLE NO.	MOISTURE CONTENT	DRY DENSITY	LL	PI	CLASSIFICATION & DESCRIPTION
V-2-2-1			31		

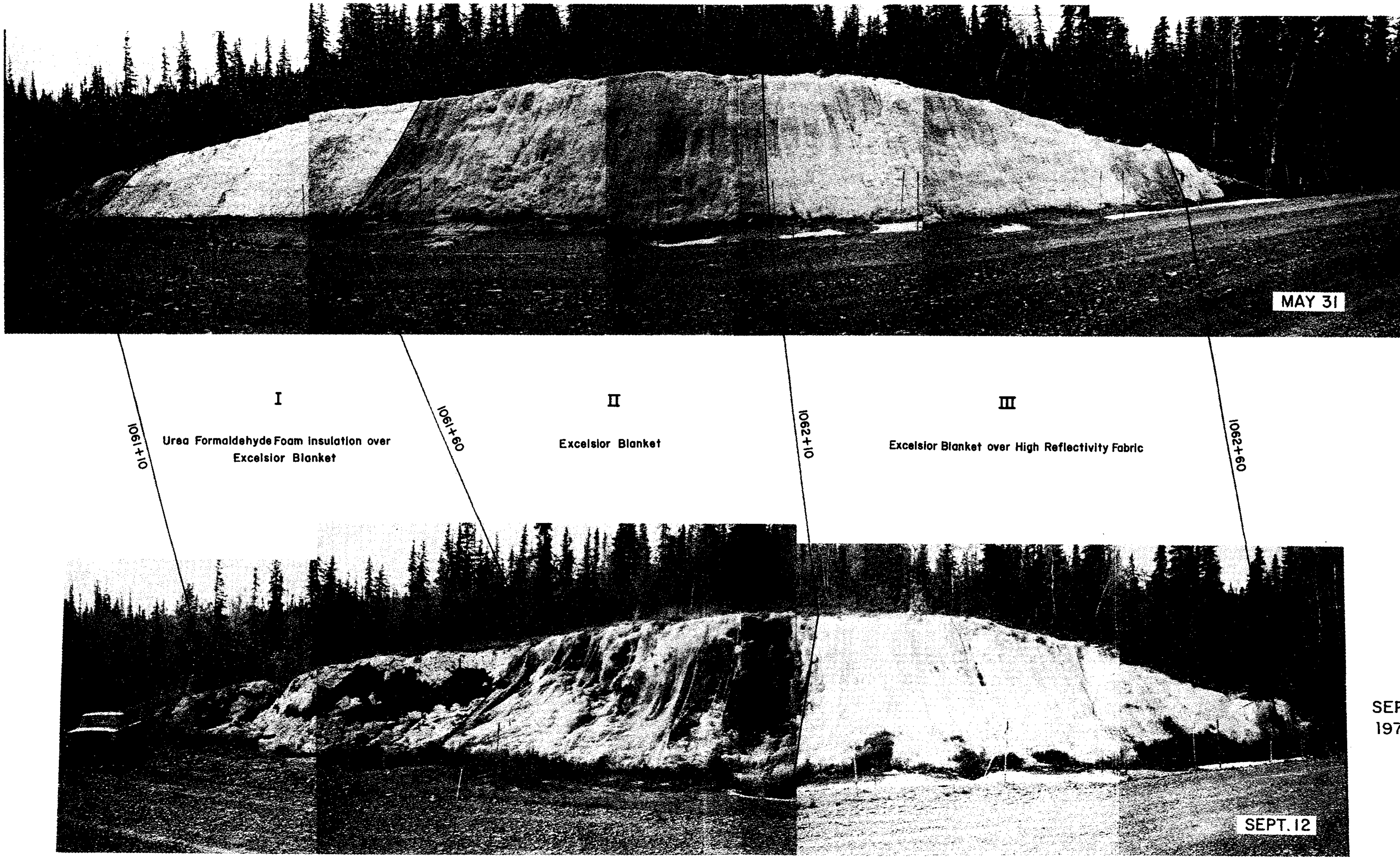
R&M
R & M CONSULTANTS, INC.

HESS CREEK - THERMAL EROSION TEST SITE

DRAWN BY R. J. L.
APPROVED BY DWM
DATE 11-10-83
PROJECT NO. 312163

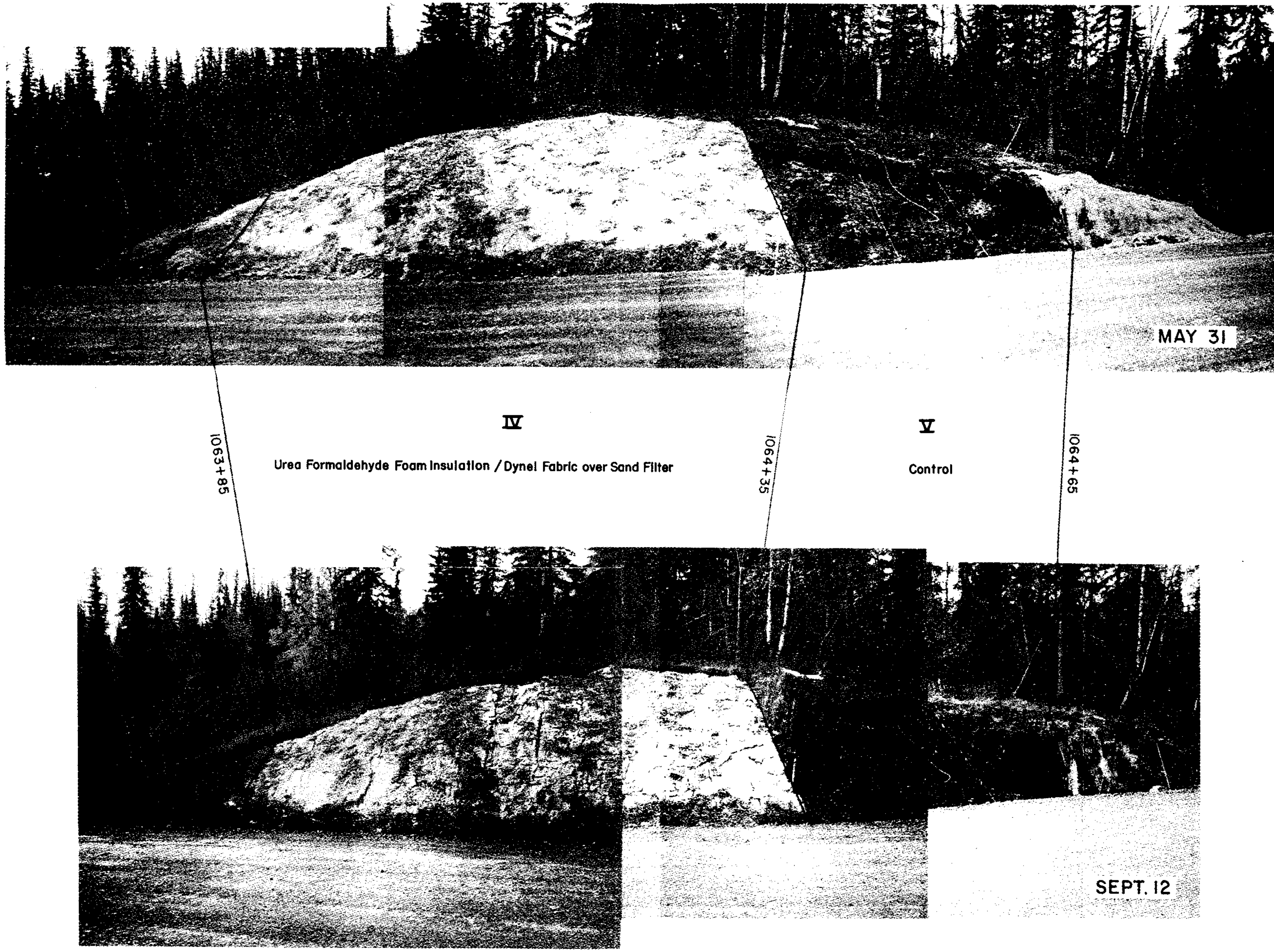
APPENDIX E

PHOTOGRAPHS



R & M CONSULTANTS, INC.

ALYESKA PIPELINE SERVICE COMPANY
HESS CREEK THERMAL EROSION STUDY
SLOPE CONDITIONS - 1974
SECTION I, II & III

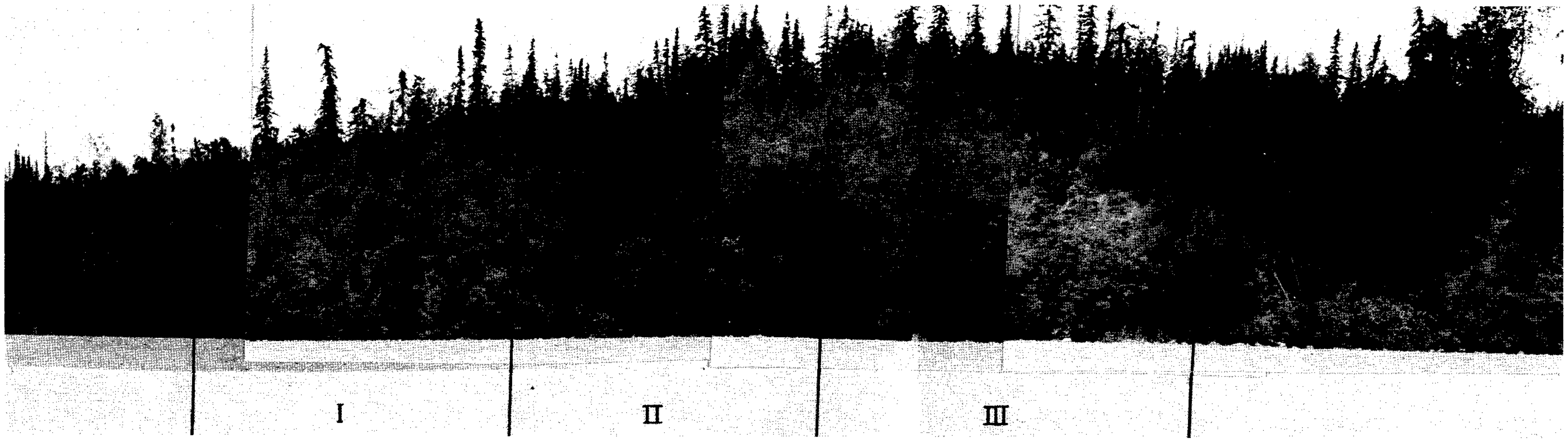


MAY
1974

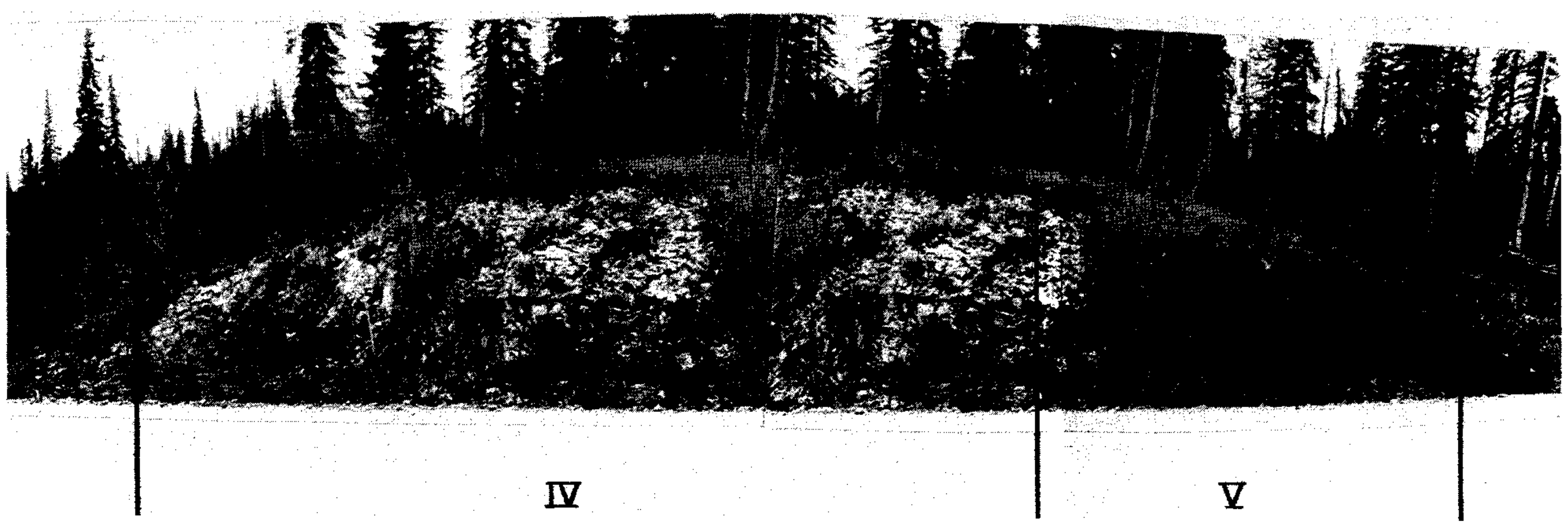
SEPT
1974

R & M CONSULTANTS, INC.

ALYESKA PIPELINE SERVICE COMPANY
HESS CREEK THERMAL EROSION STUDY
SLOPE CONDITIONS – 1974
SECTION IV & V



JUNE
1983



SEPT
1983

R & M CONSULTANTS, INC.

ALYESKA PIPELINE SERVICE COMPANY
HESS CREEK THERMAL EROSION STUDY
SLOPE CONDITIONS-1983
SECTIONS I, II, III, IV, V

R&M CONSULTANTS, INC.

TEST SECTION I

VEGETATION AT TOE OF SLOPE

(JUNE 1983)

TEST SECTION II

TEST PROBE USED TO OBTAIN SOIL SAMPLES TO 5 FT.

(SEPT. 1983)

HESS CREEK TEST SITE

DETAILED PHOTOS

1983

DATE	SCALE	DRAWN BY	CHECKED BY	PROJECT NO.	DRAWING NO.
APRIL 1984	NONE	DWM	JWR	312163	E-4

R&M CONSULTANTS, INC.

TEST SECTION IV

GRASS GROWING IN BETWEEN CRACKS OF FOAM INSULATION SURFACE TREATMENT

(JUNE 1983)

TEST SECTION IV

CLOSE-UP VIEW OF MOSS-COVERED FOAM INSULATION OVERLYING DYNEL FABRIC

(JUNE 1983)

HESS CREEK TEST SITE
DETAILED PHOTOS
1983

DATE	SCALE	DRAWN BY	CHECKED BY	PROJECT NO.	DRAWING NO.
APRIL 1984	NONE	DWM	JWR	312163	E-5

R&M

R & M CONSULTANTS, INC.

TEST SECTION V

VIEW OF GRASS-COVERED VERTICAL SCARP (~3 FT. IN HEIGHT) NEAR TOP OF SLOPE.

(SEPT. 1983)

TEST SECTION V

VIEW OF 6 INCH DEEP SURFICIAL CRACK NEAR BOTTOM OF SLOPE

(SEPT. 1983)

HESS CREEK TEST SITE

DETAILED PHOTOS

1983

DATE	SCALE	DRAWN BY	CHECKED BY	PROJECT NO.	DRAWING NO.
APRIL 1984	NONE	DWM	JWR	312163	E-6