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ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

January 18, 1984

Plateau Mining Corporation
P.O. Box PMC
Price, UT 84501

Attn: Robert G. Lauman

Gentlemen:

A soil and foundation investigation has been completed for the existing haul roads at the Plateau Mine. The investigation was performed to determine the cause of the road instability at various locations along the existing alignment. The work has been completed in accordance with a written proposal submitted to your organization for the work, and the results of the investigation along with pertinent recommendations for corrective action are outlined in the following sections of this report.

The information contained in the report is discussed under the following headings: (1) Geological and Existing Site Conditions Along the Mine Haul Road, (2) Subsurface Soil and Water Conditions in Areas Where the Roadway Has Failed, (3) The Results of Stability Analysis Performed at Critical Sections Along the Alignment, (4) Analysis of the Problem and Recommended Corrective Action, and (5) The Results of Field and Laboratory Tests.

1. GEOLOGICAL AND EXISTING SITE CONDITIONS ALONG THE MINE HAUL ROAD

The Plateau Mine is located in the north-central part of the Wasatch Plateau. While the Wasatch Plateau is composed of over 10,000 feet of strata ranging in age from Upper Cretaceous to Eocene, of particular interest to this investigation are the Emery Sandstone and Masuk Shale members of the Mancos Shale and the Star Point Sandstone and Blackhawk formations of the Mesa Verde group, all of which are Upper Cretaceous. The Lion Deck Access Road has been cut into these formations and rises approximately 900 feet from the stockpile to the bathhouse and shop area.

The Mancos Shale forms the lowland east of the Wasatch Plateau. The Emery Sandstone member of the Mancos Shale consists of massive to thin-bedded buff-gray sandstone and is exposed near the coal stockpile at the base of the Lion Deck Access Road. Overlying the Emery Sandstone member is the Masuk Shale member of the Mancos Shale. The Masuk Shale consists of blue-black to gray sandy marine shales and is approximately 400 feet thick.

Above the Mancos Shale lies the Mesa Verde group of which only the Star Point sandstone and part of the Blackhawk Formation are involved with the access road. The Star Point Sandstone overlies the Masuk Shale and usually forms the lowermost cliff in the escarpment of the Wasatch Plateau. It consists of buff to gray medium-grained beach and near-shore sandstone units which locally may grade to shale and is also separated in places by tongues of Mancos Shale. The result is an irregular sandstone-shale sequence which weathers to produce cliffs and ledges separated by narrow slopes. Total thickness of the Star Point Sandstone is about 500 feet.

Overlying the Star Point sandstone is the coal-bearing Blackhawk formation. It consists of massive- to thin-bedded fine- to medium-grained buff to gray quartz sandstone, brown or black to smokey-gray shales and coal and represents continental and fresh-water swamp deposits.

The existing haul road is located between the coal handling facilities and the mine approximately as shown in Figure No. 1. It will be noted that the haul road traverses the existing terrain in a switchback fashion and that most of the roadway consists on the consolidated portion of the geological formation.

In areas where drainage channels exist, however, it appears as if only a portion of the roadway is located on the natural material, and the remainder of the roadway is located on fill material excavated from the side of the mountain. Water collecting in the drainage channels above the roadway, in most cases, is free to seep through the roadbed into the unconsolidated materials. This condition appears to be particularly true when the snow which accumulates along the roadway melts during the spring runoff. In each of the areas where a portion of the roadway is located on unconsolidated materials, a partial failure of the roadway has occurred. The failures are characterized by a subsidence of the roadway along with a breakup of the asphalt pavement. The failure at each of these locations has been caused by either a slope stability failure or a settlement of the existing fill material.

2. SUBSURFACE SOIL AND WATER CONDITIONS IN AREAS WHERE THE ROADWAY HAS FAILED

In order to obtain an indication of the nature of the failure along the proposed alignment, nine test holes were drilled at locations where failure was incipient. The location of each of the test holes is presented in Figure No. 1. The test holes ranged in depth from 30 to 60 feet, and the logs for each of the test holes are presented in Figures 2 through 7.

The characteristics of the fill material varies somewhat from test hole to test hole, which would be expected because of the stratified nature of the parent material. Test Holes 2, 3, 4, and 7 consisted predominantly of granular material, while Test Hole No. 8 consisted predominantly of cohesive material. In Test Holes 1 and 9, the upper portion of the profile consisted of granular material, while the lower portion of the fill material consisted of cohesive material. In Test Holes 5 and 6, the upper portion of the fill material consisted of cohesive soils, while the lower portion of the fill material consisted of granular-type material. In Test Holes 1, 4, 5, 6, and 7, the subsurface material beneath the fill consisted of shale. In Test Holes 2 and 3, the natural material below the fill consisted of sandstone, while in Test Holes 8 and 9, the test holes did not penetrate the natural material.

During the subsurface investigation, sampling was performed at 3-foot intervals throughout the upper 15 feet of the soil profile and at 5-foot intervals thereafter. Both disturbed and undisturbed samples were obtained during the field investigations. Disturbed samples were obtained by driving a 2-inch split-spoon sampling tube through a distance of 18 inches using a 140-pound weight dropped from a distance of 30 inches. The number of blows to drive the sampling spoon through each 6 inches of penetration is presented on the boring logs. The sum of the last 2 blow counts, which represents the number of blows to drive the sampling spoon through 12 inches, is defined as the standard penetration value. The standard penetration value provides a reasonable indication of the in-place density of sandy material, however considerable care must be exercised in determining the density of gravelly-type soils from standard penetration values, particularly where the inside diameter of the sampling spoon is less than the particle size. The standard penetration value in cohesive materials is only an indication of the relative stiffness of these materials since the penetration resistance of is a function of the moisture content.

Undisturbed samples were obtained by pushing a 2½ inch thin-walled shelby tube into the subsurface material using the hydraulic pressure on the drill rig. The location at which undisturbed samples were obtained in cohesive materials is presented on the boring logs.

The results of the standard penetration values generally indicate that the granular material is in a relatively loose state, while the cohesive material is in a soft- to medium-stiff condition.

Each sample obtained in the field was classified in the laboratory according to the Unified Soil Classification System. The symbol designating the soil type according to this system is presented on the boring logs. A description of the Unified Soil Classification System is presented in Figure No. 8, and the meaning of the various symbols shown on the boring logs can be obtained from this figure.

It will be noted that the cohesive materials within the fill general classify as ML or CL-1 type soils, while the cohesive materials in the shale material in the lower portion of the soil profile classifies as a CL-2 type material. The classification of the granular material varies from an SM- to a GM-type soil.

During the subsurface investigation, field permeability tests were performed in each test hole to obtain an indication of the permeability characteristics of the subsurface materials. The field permeability tests were performed in accordance with designation E18 of the U.S. Bureau of Reclamation Earth Manual. Both open-end tests and packer-type tests were performed in the drill holes. The results of the field permeability tests expressed in terms of the permeability coefficient in feet per year are shown on the boring logs. It will be noted that the permeability characteristics of the fill material is relatively high, with values frequently in excess of 4,000 feet per year. Some of the cohesive material throughout the soil profile had relatively low permeability characteristics, however in most cases, the cohesive material is interbedded with permeable granular strata and it is expected that the cohesive materials will drain relatively quickly into the more pervious materials.

No groundwater was encountered in any of the test holes drilled at this site which indicates that the groundwater generally drains out of the fill material within a reasonably short time. It is possible, however, that during periods of high precipitation some groundwater may accumulate in the cohesive materials.

3. THE RESULTS OF STABILITY ANALYSIS PERFORMED AT CRITICAL SECTIONS ALONG THE ALIGNMENT

A. Introduction

As indicated earlier in this report, the area where the pavement appears to have failed generally exists in more or less drainage channels traversing the slopes of the hillside. In areas where drainage channels do not exist, it appears as if the roadway exists almost entirely on the natural material. However, in the drainage areas, a portion of the roadway appears to exist partially on the natural material and partially on fill material excavated from the adjacent hillside. Since the natural formation throughout the hillside consists of interbedded sandstone and shale, it can also be assumed that the fill material placed during the road construction may be all cohesive material, all granular material, or parts of each type of material.

As a prelude to performing the stability computations, a profile at each drilling location has been prepared. The profile uphill and downhill from the roadway was taken from the contour map provided us by your organization. The actual cross-section of the roadway was obtained by field surveys. The location at which failure was occurring on the road, along with the location of the drill hole relative to the roadway cross-section, was determined in the field. The cross-sections at each of the nine test holes are presented in Figures 9 through 17. An examination of each boring location indicates that the natural material in the roadbed is sandstone for locations corresponding to Test Holes 1, 2, and 3 and shale at Test Holes 4 through 7.

It will be observed that the natural slope generally varies from about 1.3 horizontal to 1 vertical to 2 horizontal to 1 vertical and that the slope of the fill material along the roadway area varies from about 1.2 horizontal to 1 vertical to 1.6 horizontal to 1 vertical. In all cases, the test boring was drilled within the failure zone; and in each case except for Test Holes 8 and 9, it was possible to define the interface between the fill and the natural material in the test borings. Using the location of the failure in the pavement, the location of the interface between the fill and the natural material in the test boring, and the original ground slope, an estimate has been made of the failure surface for each of the nine test holes.

The assumed failure surface is presented in each of the profiles.

The stability analysis has been performed using a computer model of Spencer's Method. Spencer's Method satisfies both force and moment equilibrium and is an accepted method for performing limiting equilibrium slope stability calculations. The results of the stability calculations for each of the test holes are discussed below.

B. Test Hole No. 1

The subsurface material in Test Hole No. 1 consists of granular material in the upper portion of the fill material and cohesive material in the lower portion of the fill area. The shear strength parameters used in the stability analysis were based upon laboratory tests for the cohesive material and the standard penetration tests for the granular material. An average friction angle of 32 degrees was assumed for the material along the failure surface, along with a total unit weight of 120 pounds per cubic foot. The results of the stability analysis indicate a factor of safety of 1.15 for the failure surface shown in Figure No. 9.

C. Test Hole No. 2

Test Hole No. 2 characterizes the subsurface material within the fill area in the vicinity of Test Hole No. 2, and it will be observed that all of the subsurface material within the fill area in this test hole is granular-type material. The assumed failure surface for the fill area for Test Hole No. 2 is presented in Figure No. 10. The shear strength parameters used in the stability analysis were based on the standard penetration values recorded in Test Hole No. 2 during the field investigation. A friction angle of 32 degrees was used for the granular material, along with a total unit weight of 125 pounds per cubic foot. The results of the stability analysis indicate a factor of safety of 1.15.

D. Test Hole No. 3

It should be noted that Test Hole No. 3 was drilled in front of the bath house and that the excavated area at this location is quite large. Test Hole No. 3 defines the characteristics of the subsurface material within the fill material at this location, and it will be observed that essentially all of the fill material is granular-type soils. The natural ground surface in Test Hole No. 3 was encountered at a depth of about 53 feet below the existing grade, and the assumed failure surface for this area is presented in Figure No. 11. The shear strength parameters used in the stability analysis were based upon the results of the standard penetration tests performed in the granular material throughout the soil profile at this site. Since the granular material within the profile at this location is relatively loose, a friction angle of 31 degrees and a total unit weight of 120 pounds per cubic foot was used in the analysis. The results of the stability analysis indicate a factor of safety of 1.19.

E. Test Hole No. 4

The characteristics of the subsurface material within the fill at this location consisted of interbedded layers of granular material and cohesive material. The assumed failure surface for the fill area at this location is presented in Figure No. 12. The results of the standard penetration test performed in the subsurface material at this site indicate that the subsurface soils are relatively loose, and as a consequence of this situation, a friction angle of 30 degrees and a total unit weight of 120 pounds per cubic foot were used in performing the analysis. The results of the stability computations indicate a factor of safety of 1.0. A factor of safety of 1.0 indicates that the subsurface material is in an incipient failure state. This fill areas has the lowest factor of safety of any of the slopes investigated during this study.

F. Test Hole No. 5

The fill material at this location is defined by the soil profile associated with Test Hole No. 5. The interface between the fill and the natural material appears to be located in the vicinity of 22 feet below the existing ground surface. Cohesive material predominates in the upper and lower portions of the fill with a zone of granular material in the center of the fill. The granular fill is in a relatively low density state, however the cohesive material appears to be in a relatively stiff condition. An overall friction angle of 31 degrees and a total unit weight of 120 pounds per cubic foot was used in the stability analysis for this slope. The failure surface associated with this slope is presented in Figure No. 13, and the results of the stability analysis indicate a factor of safety of about 1.2.

G. Test Hole No. 6

The soil profile defining the characteristics of the fill at this location consists predominantly of cohesive material in the upper 19 feet of the soil profile followed by granular material. The clay material appears to be in a medium-stiff condition, however the granular material is in a relatively loose state. The assumed failure plane for this location is presented in Figure No. 14, and the shear strength parameters used in the stability analysis assumes a friction angle of 29 degrees and a cohesion of 40 pounds per cubic foot. A total unit weight of 115 pounds per cubic foot has been used in the analysis. The results of the stability computations indicate a factor of safety of 1.0 for this location.

H. Test Hole No. 7

The subsurface material in Test Hole No. 7 is predominantly granular-type soils with some interbedded clay zones. The assumed failure surface for this location is presented in Figure No. 15, and it will be noted that the contact between the fill material and the underlying shale is about 24 feet below the roadway grade. The shear strength parameters used in this analysis are based primarily upon the results of the standard penetration tests performed in the test holes and a friction angle of 31 degrees and a total unit weight of 120 pounds per cubic foot have been used. The results of the stability analysis indicate a factor of safety of 1.25.

I. Test Hole No. 8

The characteristics of the fill material at Test Hole No. 8 consists essentially of cohesive material throughout the entire depth investigated. The assumed failure surface for a slope failure at this location is presented in Figure No. 16. The assumed failure surface at this location is strongly influenced by the natural slope downhill from the fill as well as the location of the failure surface in the pavement. The results of the field investigation indicate that the cohesive material within the soil profile at this location is in a medium-stiff condition. The shear strength parameters used in the stability analysis were based upon triaxial shear tests performed on representative samples of the cohesive material obtained in Test Hole No. 8. A friction angle of 32 degrees, along with a total unit weight of 115 pounds per cubic foot, was used in the stability computations. The results of the stability computations indicate a factor of safety of 1.6 for the existing slopes at this location.

J. Test Hole No. 9

It will be noted from Test Hole No. 9 that the material within the fill at this location consists predominantly of granular material in the upper 19 feet of the soil profile and cohesive material throughout the remainder of the depth investigated. The granular material appears to be in a relatively loose state, however the cohesive material appears to exist in a medium-stiff condition. The assumed failure surface for a slope failure at this location is defined by Figure No. 17. Again it will be noted that the failure surface is strongly influenced by the natural slope downhill from the roadway grade. The shear strength parameters used in this analysis are based primarily on the results of triaxial shear tests performed for samples of the clay in Test Hole No. 8. A friction angle of 32 degrees, along with the total unit weight of 115 pounds per cubic foot was used in the analysis. The results of the stability analysis indicate a factor of safety of 1.31 for this location.

It is our opinion that the shear strength parameters used in the stability analysis for all slopes are conservative and that the actual factor of safety for the slopes may be greater than indicated above.

4. ANALYSIS OF THE PROBLEM AND RECOMMENDED CORRECTIVE ACTION

A. Introduction

During our original visit to the site, it was postulated that in the drainage areas along the access road alignment that the roadbed existed on both natural material and fill material. It was also postulated that the fill material may be sufficiently impervious that water seeping into the fill material would saturate these materials and result in a buildup of pore pressures within the fill material resulting in a considerable loss in strength of the materials within the fill zone. Based upon the above assumptions, it appeared as if horizontal drains would provide a beneficial affect by permitting the fill material to drain. During the subsurface investigation, however, the results of the permeability tests performed in the bore holes indicated that most of the fill material had moderately high permeability characteristics. Based upon these tests, it is our opinion that the fill material is sufficiently permeable that drainage will occur by gravity without the necessity of horizontal drains. It is our opinion that the pavement failure which has occurred at a number of locations along the roadway alignment is due either to a slope stability failure of the fill material or to a subsidence of the fill material, combined with settlement associated with truck traffic.

B. Stability Considerations

The results of the stability analysis performed for each of the sites investigated indicated a factor of safety of greater than 1 for all locations, except at Borings 4 and 6. The factors of safety varied from about 1.19 to 1.6. Even at Borings 4 and 6, a factor of safety of greater than 1 would have been obtained if friction angles of 31 degrees had been used for the material at these locations. The natural angle of repose for most granular materials is in the vicinity of 1.4 horizontal to 1 vertical. The slopes associated with the fill zones are steeper, in some cases, and flatter than others than those slopes associated with the natural angle of repose. Since the factor of safety for each of the slopes is equal to or greater than 1 and since it is our opinion that the shear strength parameters used in the analysis are conservative, we believe that a slope failure is not the cause of the pavement failure at various locations along the existing alignment.

C. Settlement Considerations

The results of the field investigations indicate that the granular material is in a relatively loose condition at a number of locations. It is a well-known fact that water percolating through relatively loose granular soils will cause settlement, particularly under prolonged conditions. It is also well established that if cohesive materials become wet, subsidence will occur under overburden loads. The drainage conditions throughout the existing alignment are not sufficient to prevent water from percolating into the fill material along the roadway alignment.

It is our opinion, therefore, that the pavement failures which have occurred along the alignment are due primarily to the subsidence of the fill material. The subsidence has occurred most likely due to the prolonged action of the force of gravity on the loose material, combined with the densification action of the heavy truckloads which traverse the area. The subsidence associated with the above factors have been increased and hastened by the wetting, saturation, and percolation of waters in the fill material.

D. Recommended Corrective Action

(1) Excavate and Replace Fill in Failure Areas

In areas where the pavement has failed, we recommend that at least 2½ feet of the fill material be removed and replaced with granular backfill. The granular backfill should be a well-graded material with a maximum size less than 3 inches and with not more than 5 percent passing a 200 sieve. The granular material should be densified to an in-place unit weight equal to 90 percent of the maximum laboratory density as determined by ASTM D 1557-78. The upper 6 inches of the granular fill should be road base conforming to the following specifications:

Sieve Size	Percent Passing
1"	100
.5"	70 - 100
No. 4	41 - 68
No. 16	21 - 41
No. 50	10 - 27
No. 200	4 - 13

The road base should also be densified to an in-place unit weight equal to 90 percent of the maximum laboratory density indicated above.

The granular fill should be capped with a 3-inch layer of plant mixed asphalt densified in accordance with the standard specifications of the Utah State Department of Transportation.

(2) Improve the Existing Surface Drainage System to Substantially Reduce or Eliminate the Percolation of Waters Into the Subsurface Fill Material

Culverts to prevent ponding of water in those areas where the fill materials exist should receive serious consideration. Lining the drainage channel with an impervious liner along the inside of the roadbed in those areas where the drainage channels exist should also be considered.

It is our opinion that prevention of percolating waters into the fill area in cut and fill sections is absolutely necessary to prevent deep settlement of the existing fills.

(3) Possible Road Realignment

The Above Recommendations are designed to reduce the settlement of the fill associated with gravity and percolating waters as well as the settlement induced in the fill material by heavy truck loads. If the deep settlement in the fills cannot be eliminated by proper drainage, it may be necessary to realign the access road so that the roadbed is all located on natural, consolidated materials. Since it cannot be precisely determined if deep settlement of the fill material can be terminated, it may be advisable to cover the compacted granular fill placed in problem areas with a thin asphalt surface coarse until it is fully determined if deep subsidence of the fill material has terminated. Placing the thin asphalt surface coarse will prevent the percolation of surface waters into the subsurface soils and will provide a satisfactory surface for current traffic.

5. THE RESULTS OF FIELD AND LABORATORY TESTS

Field and laboratory tests performed during this investigation, to define the characteristics of the subsurface material, included standard penetration tests, miniature vane shear tests, in-place unit weight, natural moisture content, Atterberg limits, mechanical analysis, unconfined compressive strength, direct shear tests, and triaxial shear tests.

The standard penetration tests have been previously discussed, and the results of these tests are presented on the boring logs.

Miniature vane shear tests, which provide an indication of the undrained shearing strength of saturated, cohesive soils were performed on a number of the cohesive samples obtained throughout the profile. These tests were performed primarily on samples obtained from Test Holes 1, 7, and 8. The miniature vane shear tests are designated as the torvane value on the boring logs and are expressed in tons per square foot. A summary of all other tests performed during the investigation with the exception of the direct shear tests and the triaxial shear tests are presented in Table No. 1, Summary of Test Data.

It will be observed that the in-place unit weight of the cohesive material in Test Hole No. 1 varied from about 107 to 122 pounds per cubic foot and that the natural moisture content was a few percentage points above the plastic limit. The plastic index of all of the cohesive materials performed during the investigation were generally less than 11 or 12 percent, indicating that these materials have low-plasticity characteristics.

Mechanical analysis were performed on a number of the granular samples obtained from Test Holes 2, 3, 4, 6, and 8, and the results of these tests indicate that a substantial amount of material in the silt- and clay-size range existed in these materials. The fact that the granular materials were moderately permeable indicates that they were not in a high density state.

In order to obtain an indication of the strength characteristics of the cohesive material in the fill throughout the roadway alignment, three consolidated drain direct shear tests were performed on representative samples obtained from Test Hole No. 1 at a depth of 20 to 21.5 feet below the ground surface. The results of these tests are presented in the form of a Mohr envelope in Figure No. 18, and it will be observed that a friction angle of 33.7 degrees and a cohesion of 4 pounds per square inch was obtained.

In order to evaluate the strength characteristics of the subsurface material in Test Hole No. 8, 6 consolidated drained triaxial shear tests were performed on representative samples obtained at depths of 5 feet to 11.5 feet and 40 feet to 46 feet in Test Hole No. 8. The results of these tests are presented in the form of a Mohr envelope in Figures 19 and 20. It will be noted that friction angles in the vicinity of 33.6 degrees and a cohesion of 10 psi was obtained for both of these samples. In performing the stability analysis, friction angles slightly smaller than the values obtained during the tests indicated above were used. It should also be noted that while some cohesion was obtained as shown in Figures 19 and 20, no cohesion values were assumed in the stability computations.

The conclusions and recommendations presented in this report are based upon the results of the field and laboratory tests which, in our opinion, define the characteristics of the subsurface material at this site in a satisfactory manner. If there are any questions relative to the information contained herein, please advise us.

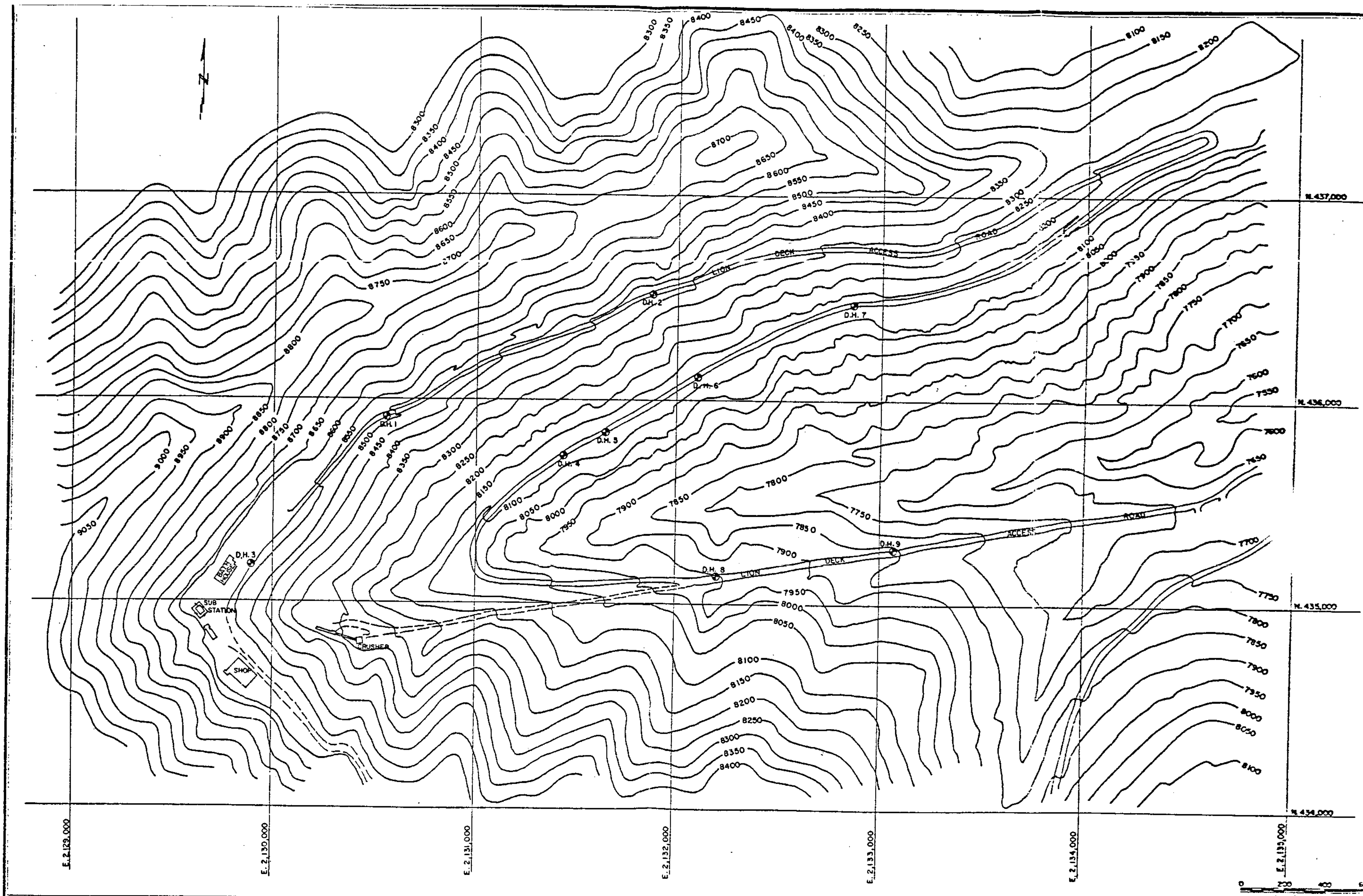
Yours truly,

ROLLINS, BROWN AND GUNNELL, INC.

Ralph L. Rollins

RLR/lah

Enclosures



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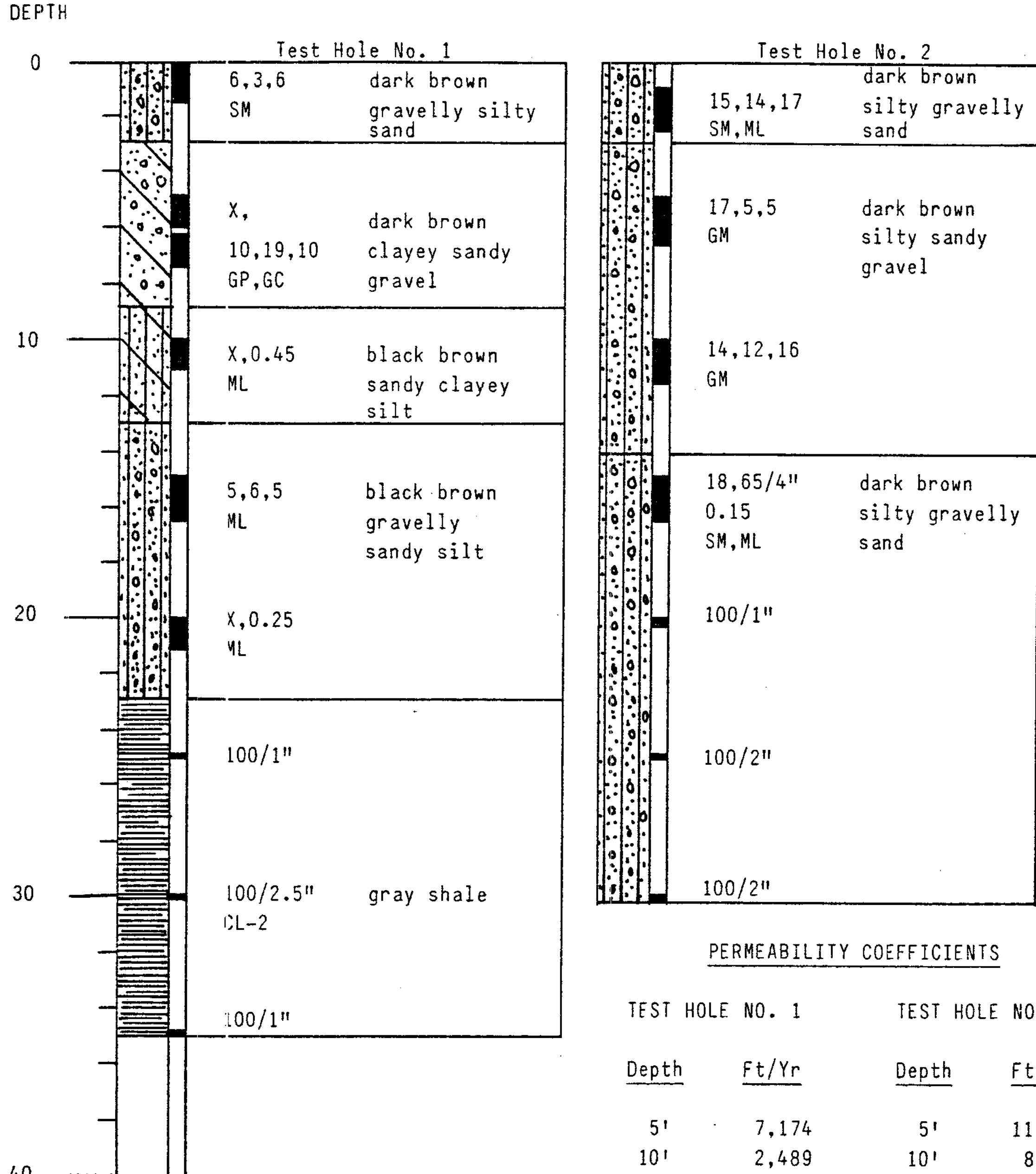
Plateau Mine Road

PLATEAU MINING COMPANY
PRICE, UTAH

Location of Test Holes

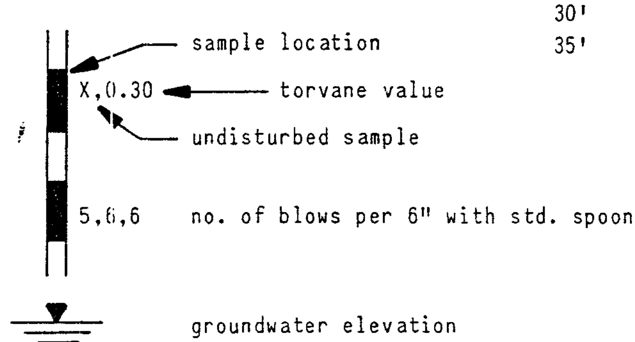
Figure No. 1



PERMEABILITY COEFFICIENTS

TEST HOLE NO. 1 Depth	Ft/Yr	TEST HOLE NO. 2 Depth	Ft/Yr
5'	7,174	5'	11,525
10'	2,489	10'	8,911
15'	>12,250	15'	1.0
20'	22.7	20'	>9,188
25'	0.2	25'	>8,167
30'	2,970		
35'	2,130		

LEGEND



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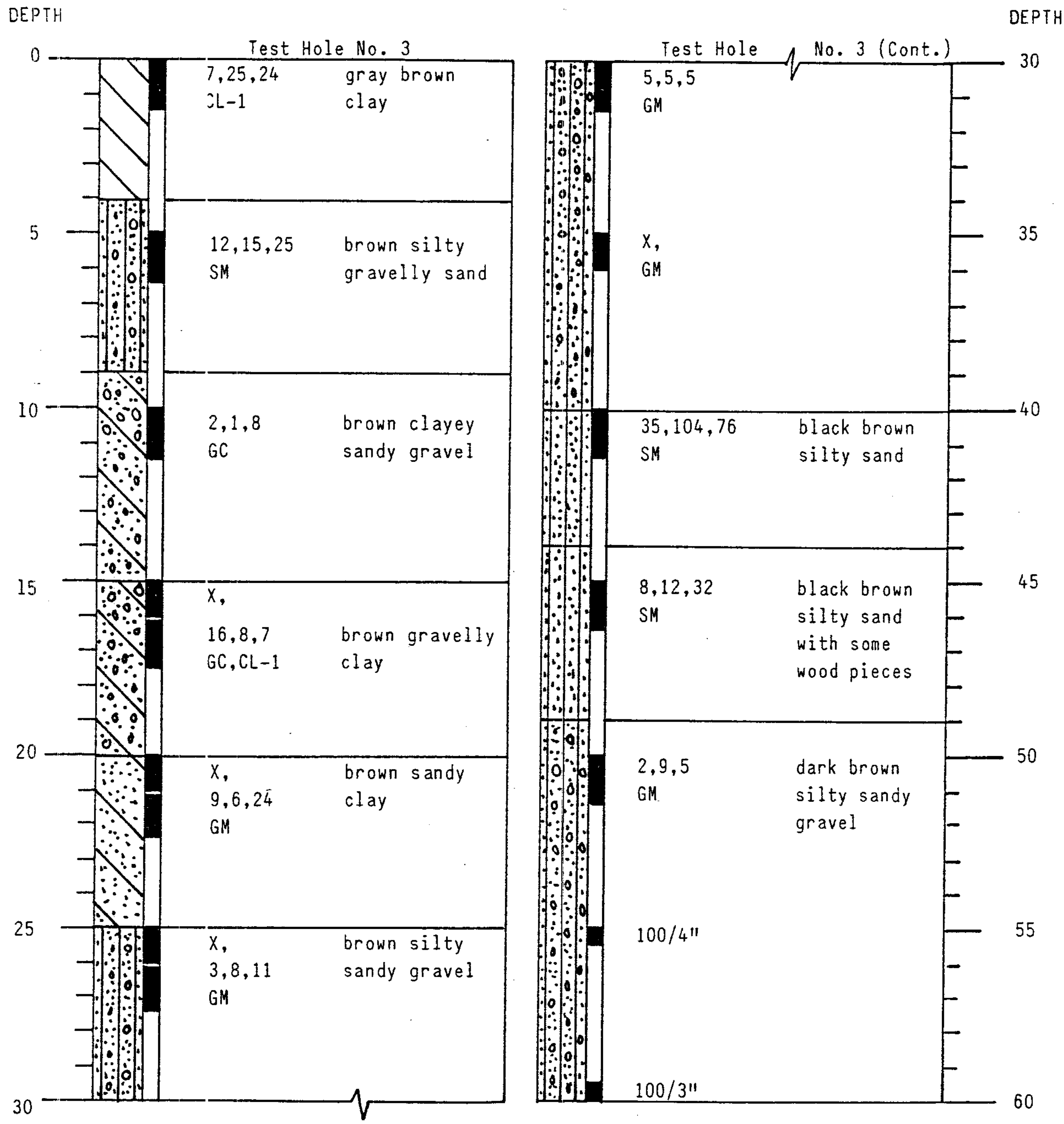
Log of Borings for:
Plateau Mine Road
Price, Utah

Figure No. 2



LEGEND

sample location

X,C.30 torvane value

undisturbed sample

5,6,6 no. of blows per 6" with std. spoon

groundwater elevation

PERMEABILITY COEFFICIENTS

Depth	Ft/Yr
0-5'	451
10'	>18,375
15'	>12,250
20'	>9,187
25'	>7,350
30'	>6,125
35'	>5,250
40'	0.2
45'	>4,324
50'	3,828
55'	0.4
60'	8.0

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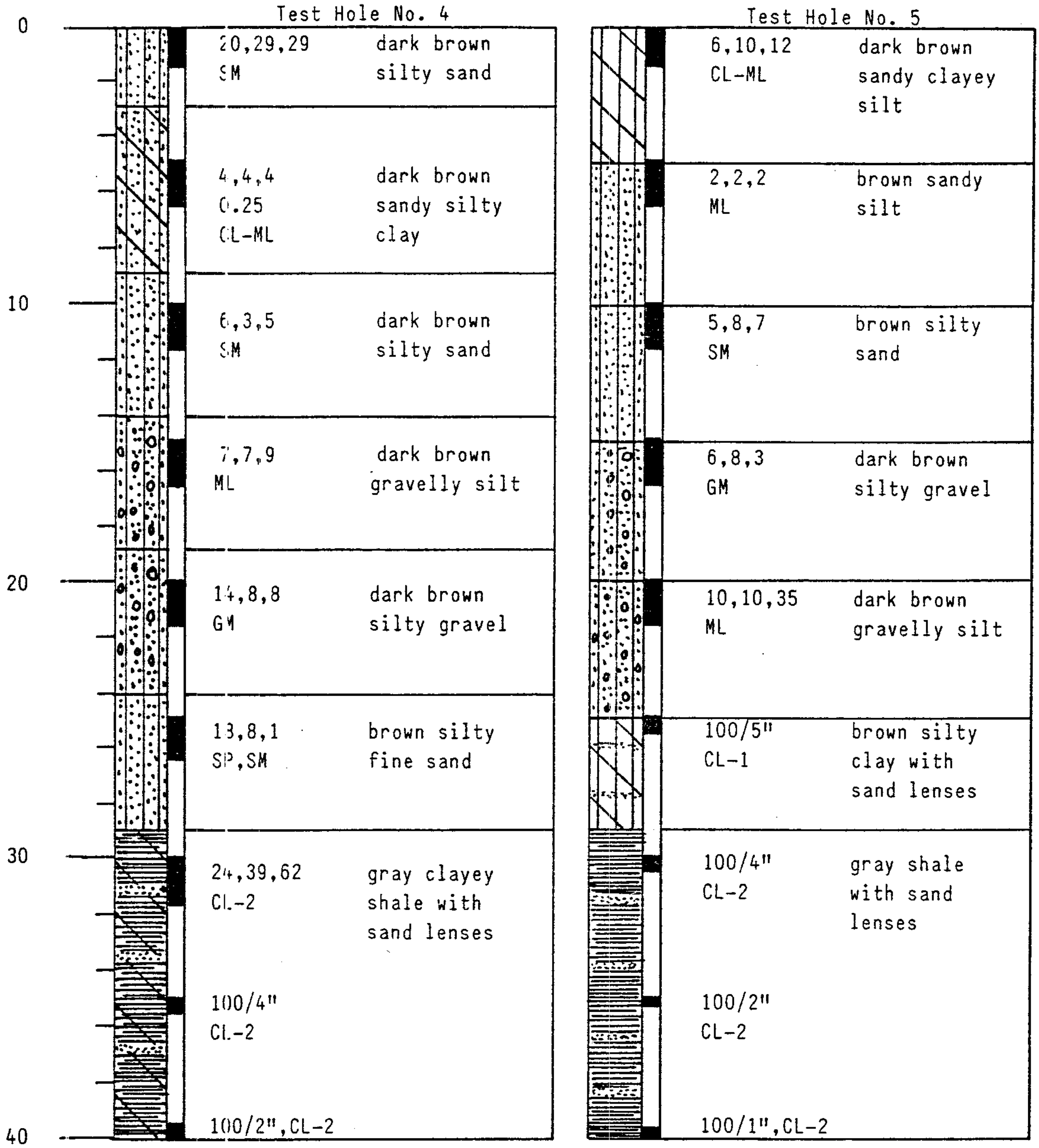
Log of Borings for:
Plateau Mine Road
Price, Utah

Figure No. 3

DEPTH



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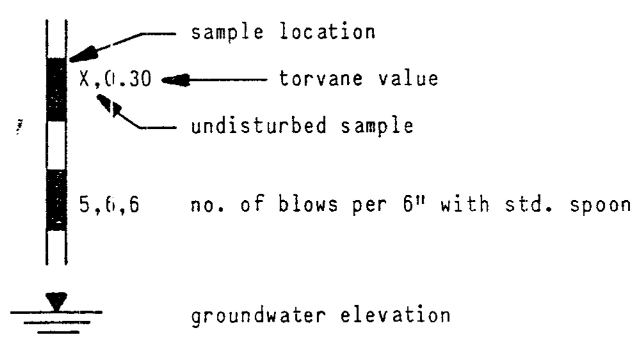

PERMEABILITY COEFFICIENTS

TEST HOLE NO. 4		TEST HOLE NO. 5	
Depth	Ft/Yr	Depth	Ft/Yr
0-5'	447	0-5'	18,967
5-10'	8,911	10'	5,444
10-15'	>14,700	15'	>12,250
20'	5.6	20'	9,188
25'	>7,350	25'	0.7
30'	3.5	30'	10.2
35'	0.3	35'	3.6
30-40'	0.6	40'	3.5

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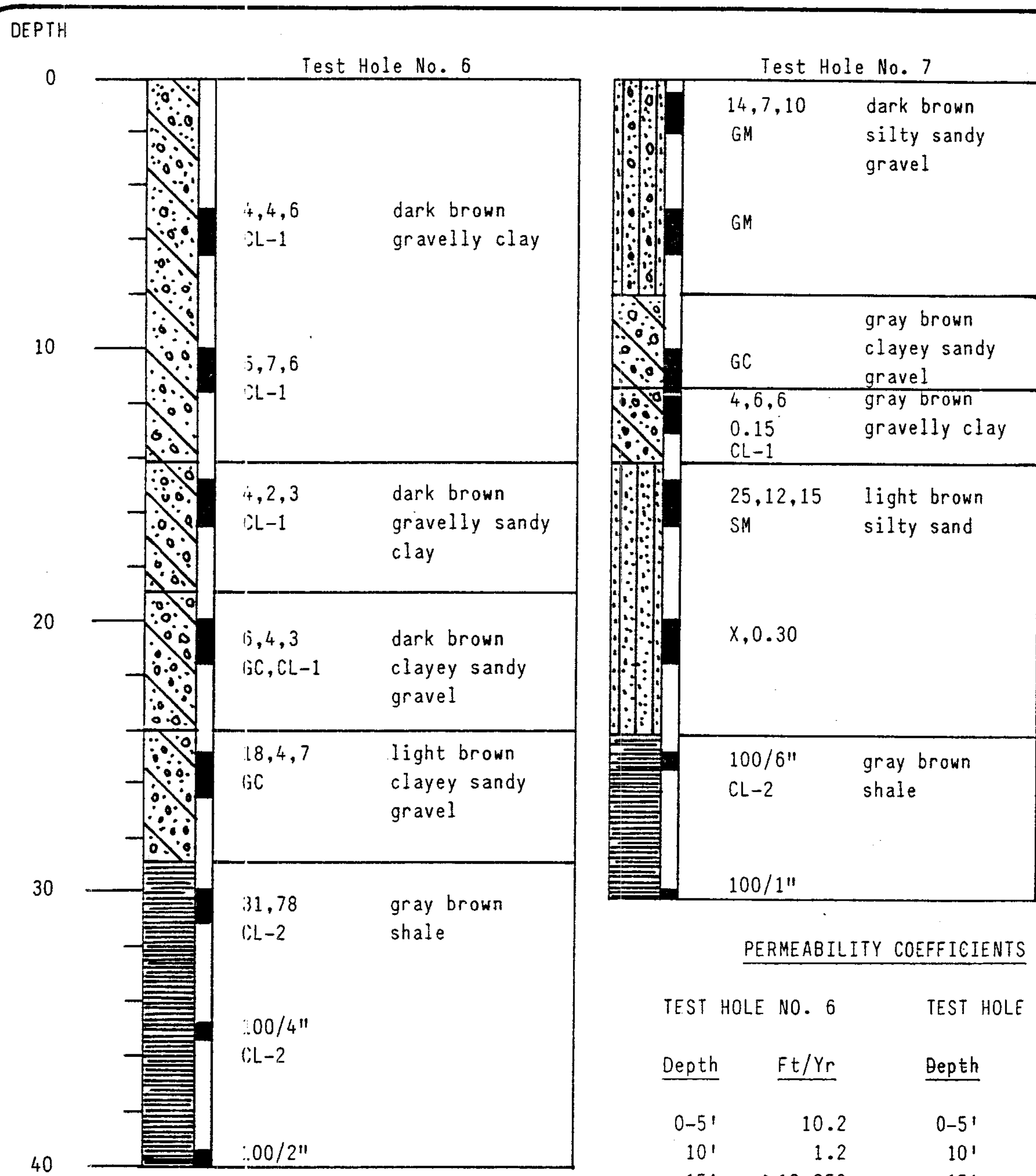
Log of Borings for:
Plateau Mine Road
Price, Utah

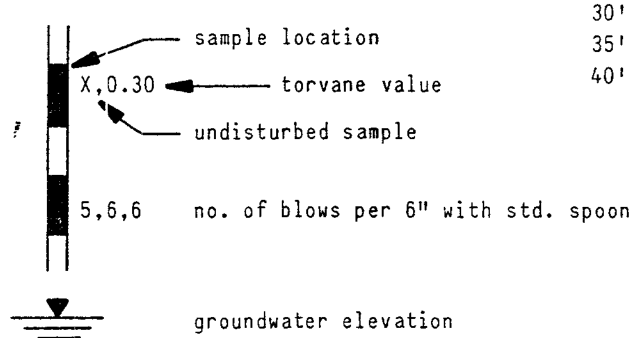
Figure No. 4



PERMEABILITY COEFFICIENTS

TEST HOLE NO. 6 Depth	Ft/Yr	TEST HOLE NO. 7 Depth	Ft/Yr
0-5'	10.2	0-5'	4,455
10'	1.2	10'	1,652
15'	>12,250	15'	1.6
20'	>9,187	20'	>9,187
25'	>7,350	25'	NML
30'	0.2	30'	0.3
35'	0.3		
40'	70		

LEGEND



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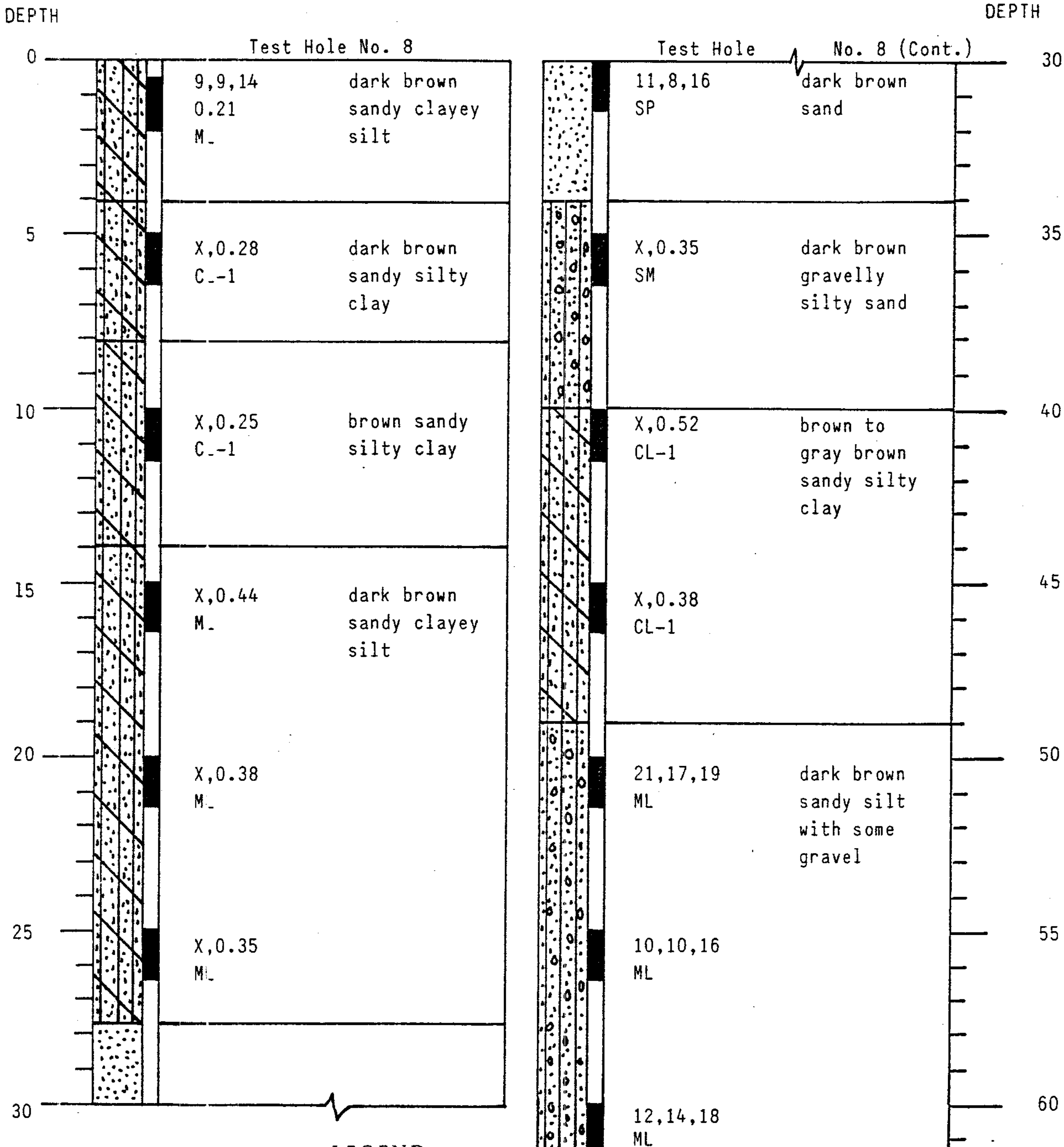
Log of Borings for:
Plateau Mine Road
Price, Utah

Figure No. 5



LEGEND

sample location

X,0.30 torvane value

undisturbed sample

5,6,6 no. of blows per 6" with std. spoon

groundwater elevation

PERMEABILITY COEFFICIENTS

Depth	Ft/Yr	Depth	Ft/Yr
0-5'	2,007	35'	>5,250
5-10'	10,891	40'	1.0
10-15'	>18,375	40-45'	303
15-20'	>12,250	45-50'	>3,868
25'	>7,350	55'	>3,341
30'	>6,125	55-60'	>3,196

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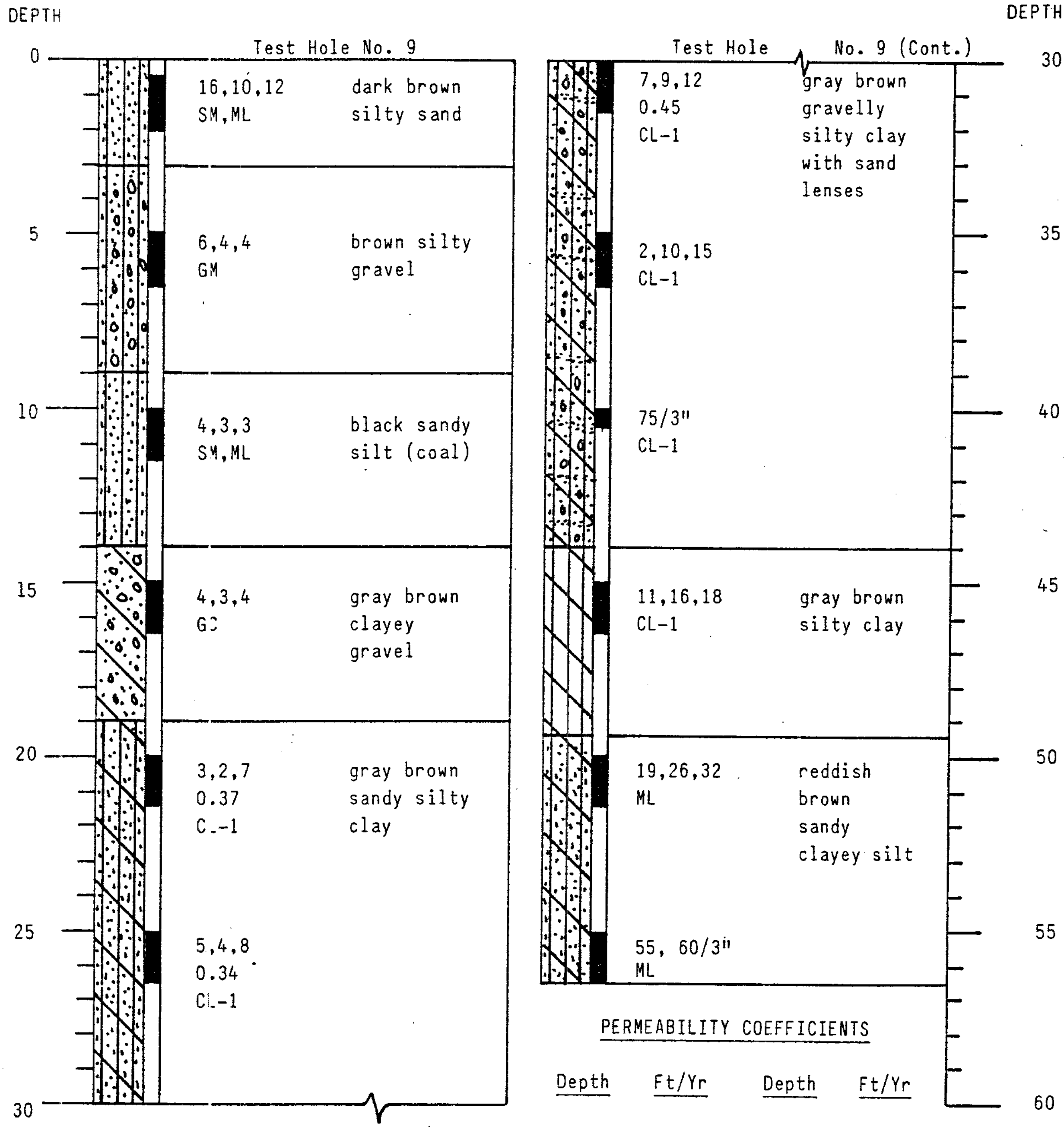
Log of Borings for:
Plateau Mine Road
Price, Utah

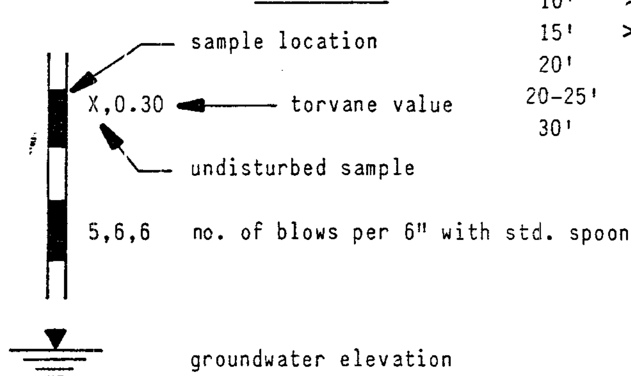
Figure No. 6



PERMEABILITY COEFFICIENTS

Depth	Ft/Yr	Depth	Ft/Yr
0-5'	58,800	35'	>5,654
10'	>18,375	40'	583
15'	>12,250	45'	288
20'	28.7	50'	>4,083
20-25'	>8,166	55'	>3,868
30'	30.7	60'	>3,675

LEGEND



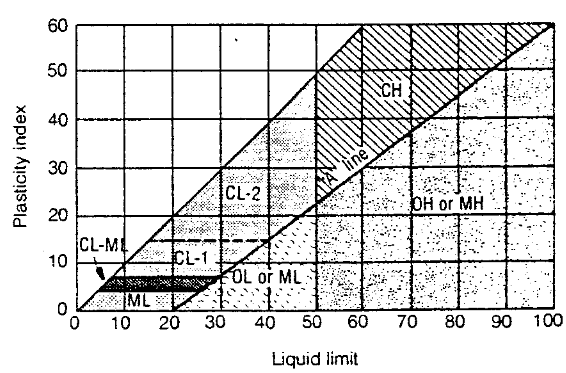
ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

Log of Borings for:
Plateau Mine Road
Price, Utah

Figure No. 7

Unified Soil Classification System

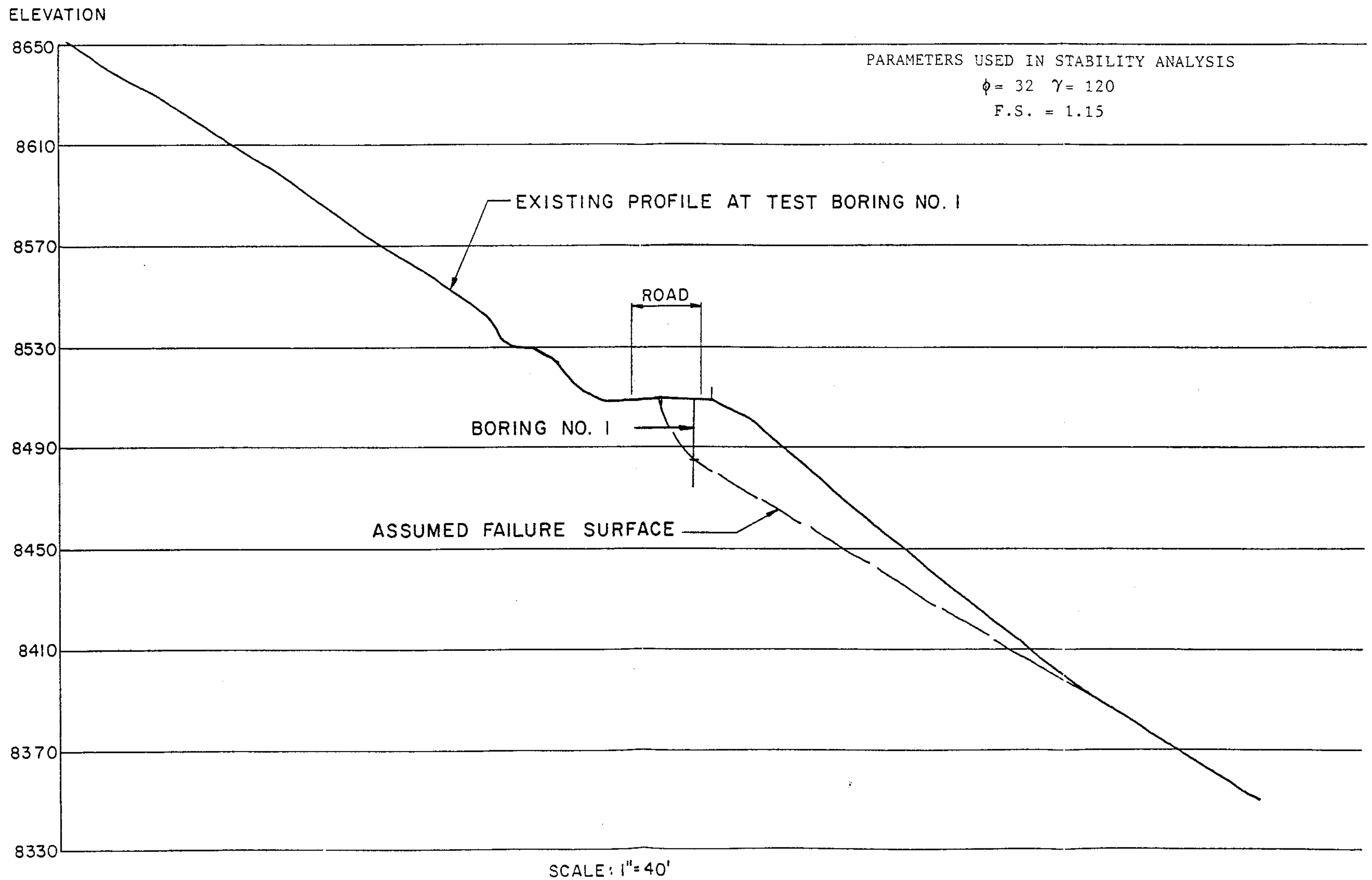
Major Divisions			Group Symbols	Typical Names	Laboratory Classification Criteria		
Course-grained Soils More than half of material is *larger* than No. 200 sieve	Gravels More than half of coarse fraction is larger than No. 4 sieve size	Clean Gravels (Little or no fines)	*GW*	Well graded gravels, gravel-sand mixtures, little or no fines.	Determine percentage of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5% .... *GW, GP, SW, SP* More than 12% .... *GM, GC, SM, SC* 5% to 12% .... *Borderline* cases requiring use of dual symbols**	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 4 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for *GW*	
		Gravels with fines (Appreciable amount of fines)	*GM** d / u	Silty gravels, poorly graded gravel-sand-clay mixtures		Atterberg limits below "A" line, or *PI* less than 4	Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring uses of dual symbols
			GC	Clayey gravels, poorly graded gravel-sand-clay mixtures		Atterberg limits above "A" line, or *PI* greater than 7	
	Sands More than half of coarse fraction is smaller than No. 4 sieve size	Clean Sands (Little or no fines)	*SW*	Well graded sands, gravelly sands, little or no fines		$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			SP	Poorly graded sands, gravelly sands, little or no fines.		Not meeting all gradation requirements for *SW*	
		Sands with fines (Appreciable amount of fines)	*SM** d / u	Silty sands, poorly graded sand-silt mixtures		Atterberg limits below "A" line, or *PI* less than 4	Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring uses of dual symbols
			SC	Clayey sands, poorly graded sand-clay mixtures		Atterberg limits above "A" line, or *PI* greater than 7	
Fine-grained Soils More than half of material is *smaller* than No. 200 sieve	Silt and Clays Liquid limit less than 50		*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity			
			CL 1 / 2	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL	Organic silts and organic silt-clays of low plasticity			
	Silts and Clays Liquid limit greater than 50		*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH	Inorganic clays of high plasticity, fat clays			
			OH	Organic clays of medium to high plasticity, organic silts			
Highly Organic Soils			*Pt*	Peat and other highly organic soils			



Plasticity Chart
For laboratory classification of fine-grained soils

*Division of GM and SM groups into subdivisions of d and u for roads and airfields only. Subdivision is based on Atterberg limits; suffix d used when liquid limit is 28 or less and the *PI* is 6 or less, the suffix u used when liquid limit is greater than 28.
***Borderline classification*: Soils possessing characteristics of two groups are designated by combinations of group symbols. For example GW-GC, well graded gravel-sand mixture with clay binder.

Figure No. 8



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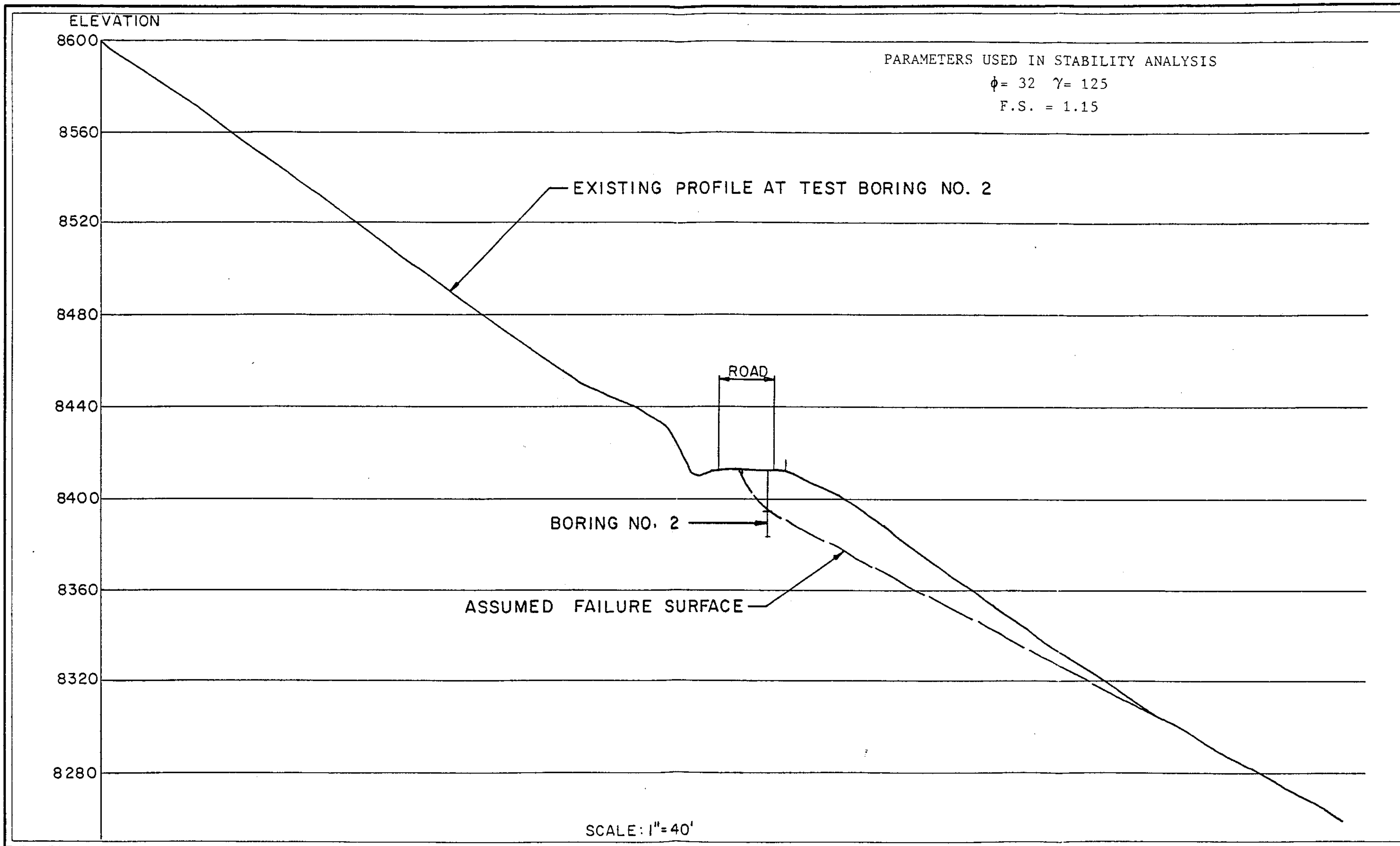
PROFILE AND ASSUMED FAILURE SURFACE FOR PLATEAU MINE ROAD

FIGURE NO. 9



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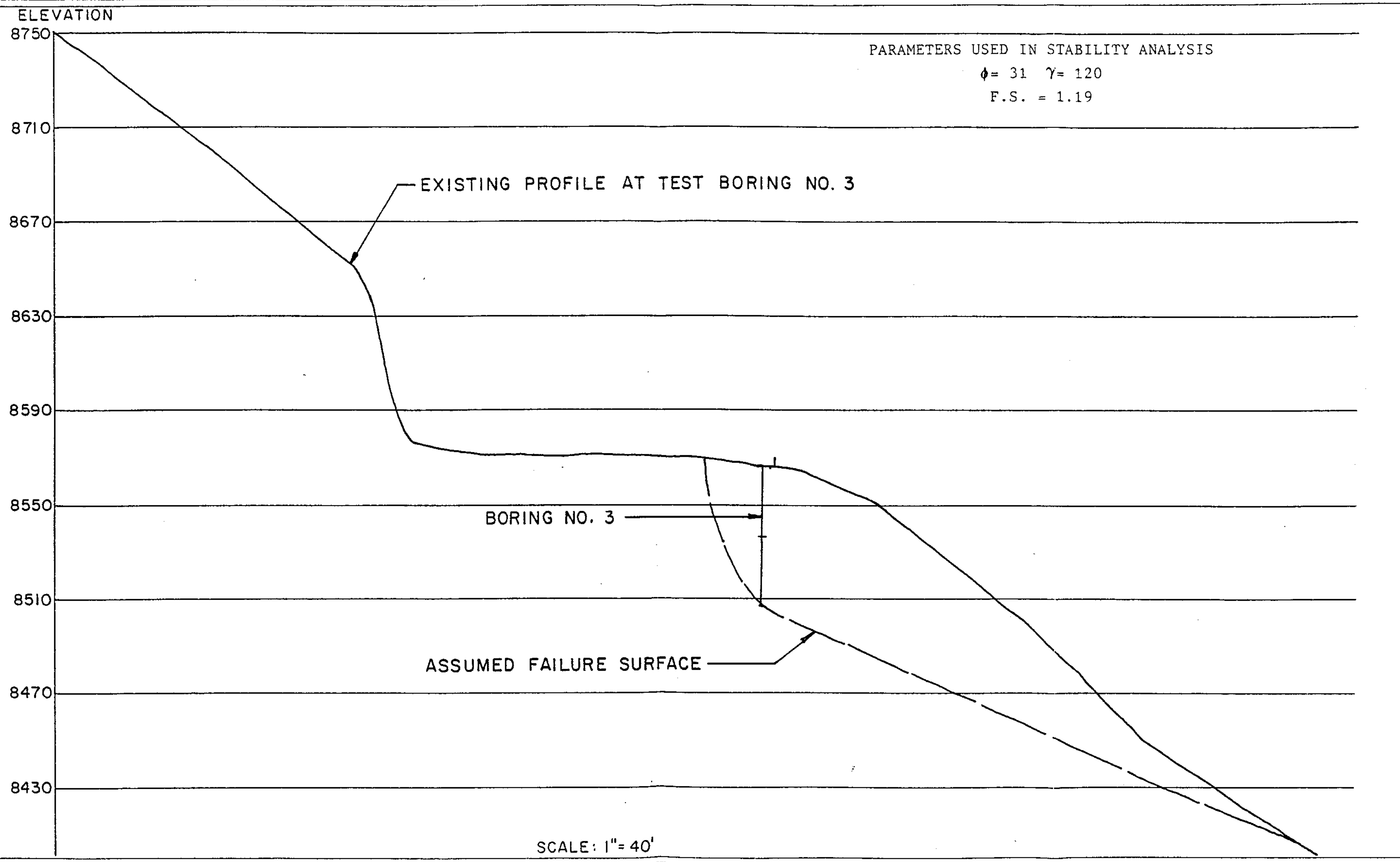
PROFILE AND ASSUMED FAILURE SURFACE FOR PLATEAU MINE ROAD

FIGURE NO. 10



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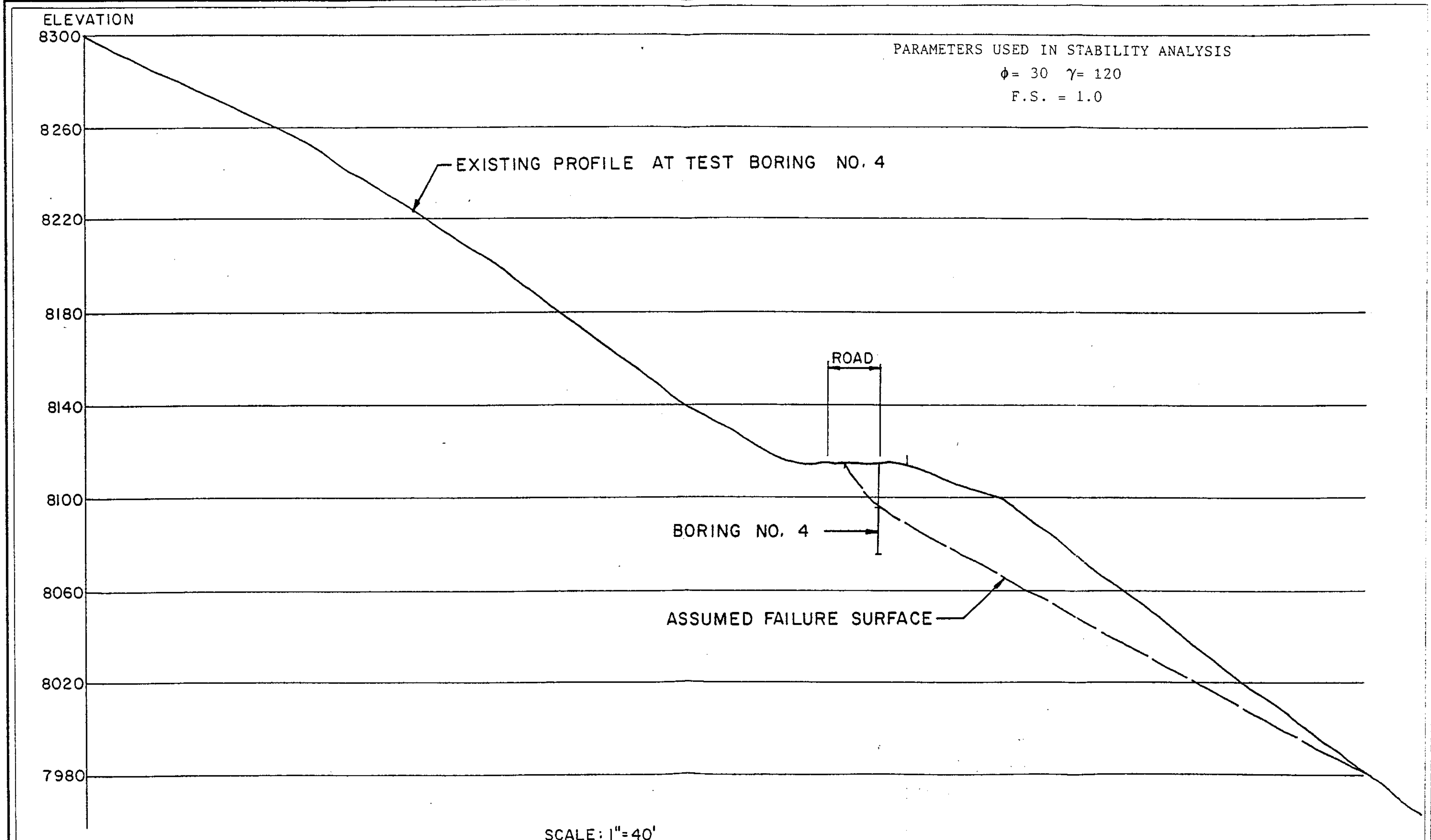
PROFILE AND ASSUMED FAILURE SURFACE FOR PLATEAU MINE ROAD

FIGURE NO. 11



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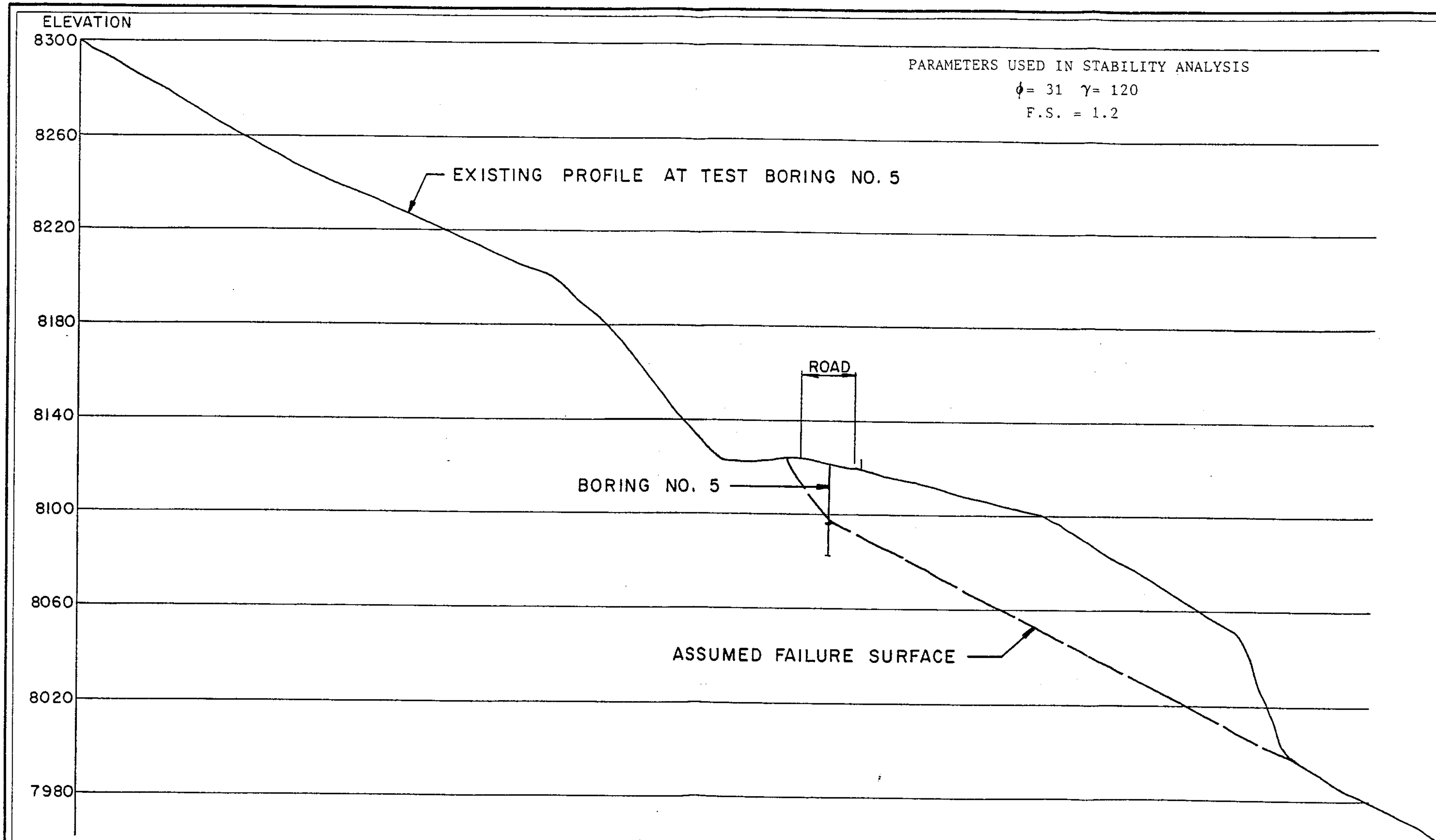
PROFILE AND ASSUMED FAILURE SURFACE FOR PLATEAU MINE ROAD

FIGURE NO. 12



SCALE: 1" = 40'

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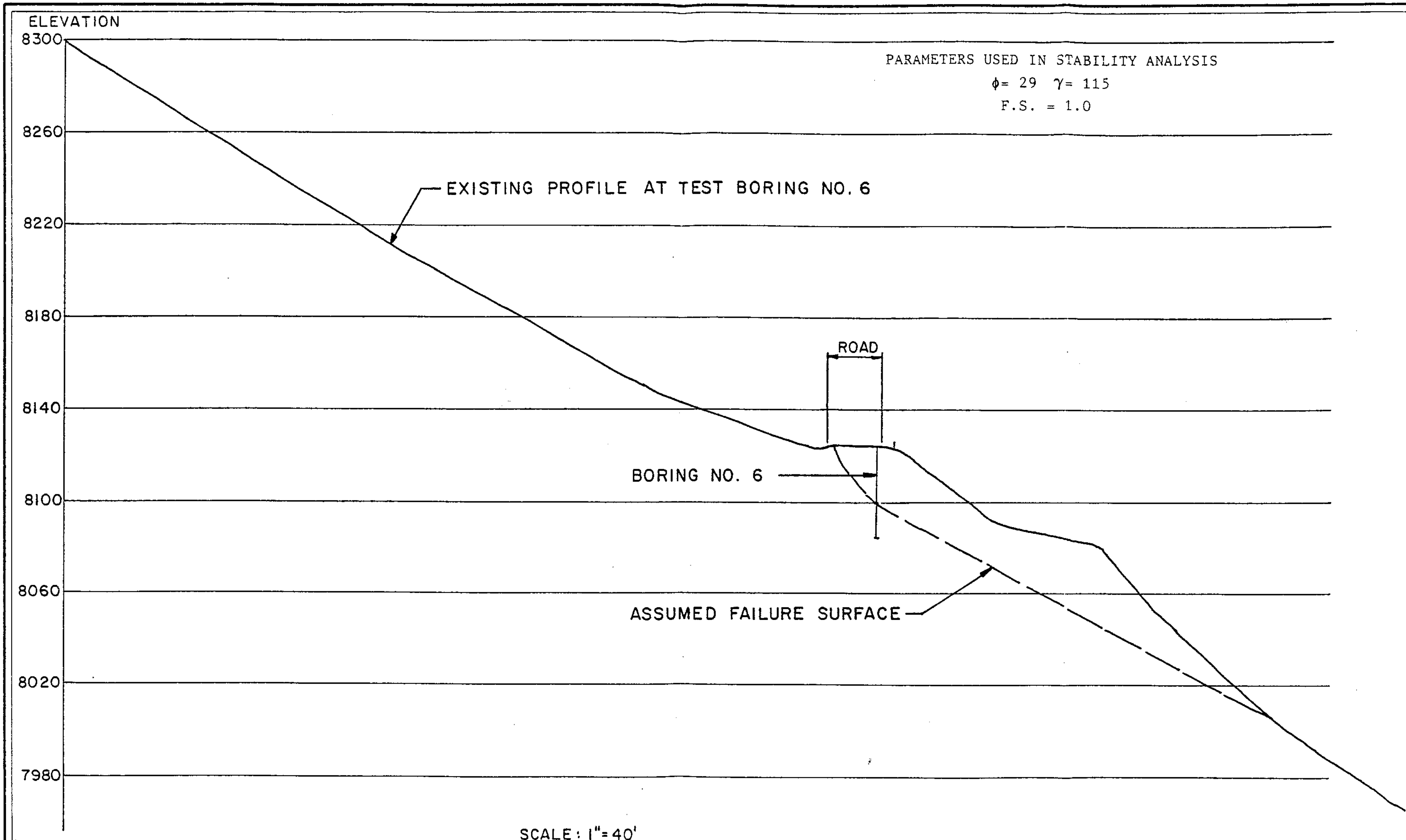
PROFILE AND ASSUMED FAILURE SURFACE FOR PLATEAU MINE ROAD

FIGURE NO. 13



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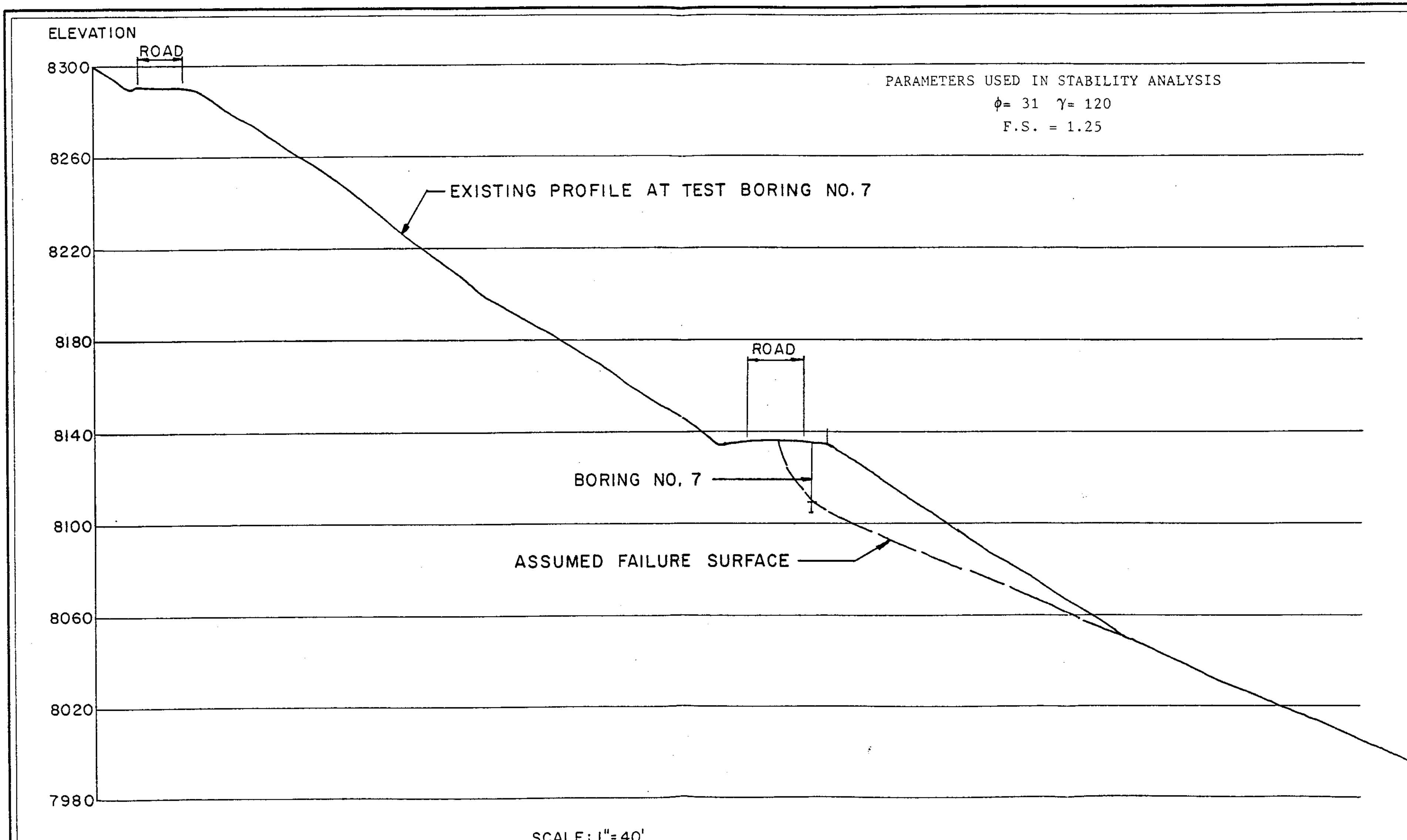
PROFILE AND ASSUMED FAILURE SURFACE FOR PLATEAU MINE ROAD

FIGURE NO. 14



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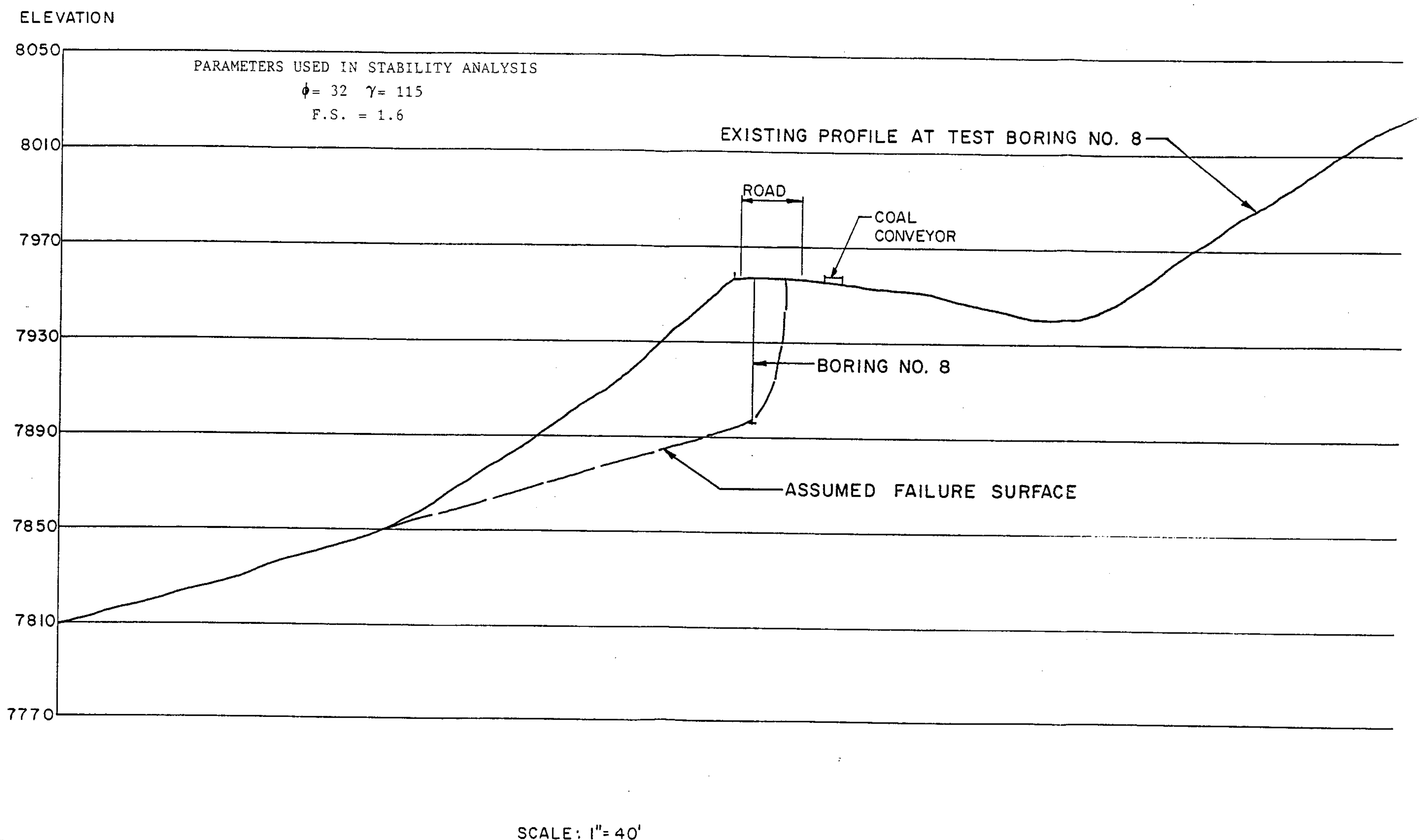
PROFILE AND ASSUMED FAILURE SURFACE FOR PLATEAU MINE ROAD

FIGURE NO. 15



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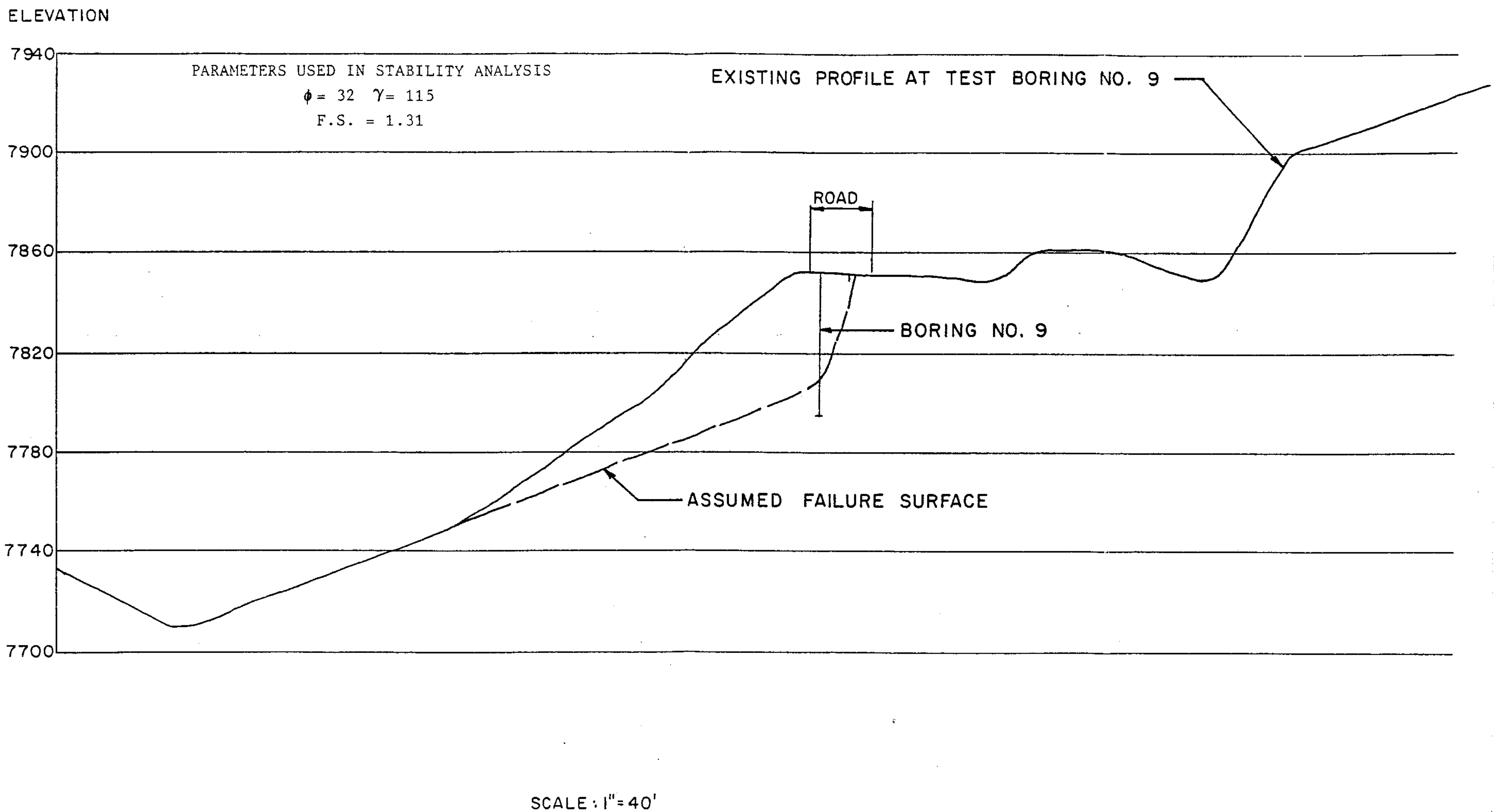
PROFILE AND ASSUMED FAILURE SURFACE FOR PLATEAU MINE ROAD

FIGURE NO. 16



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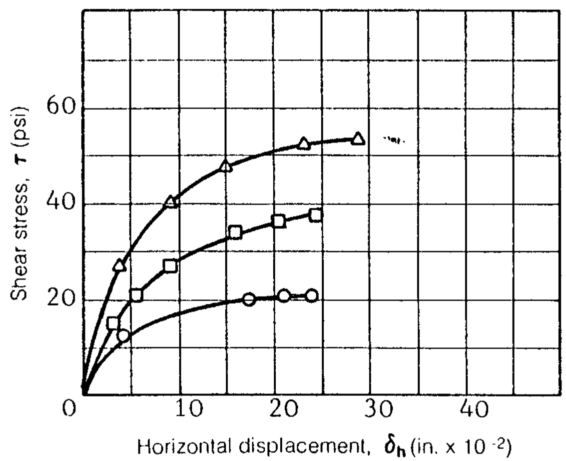
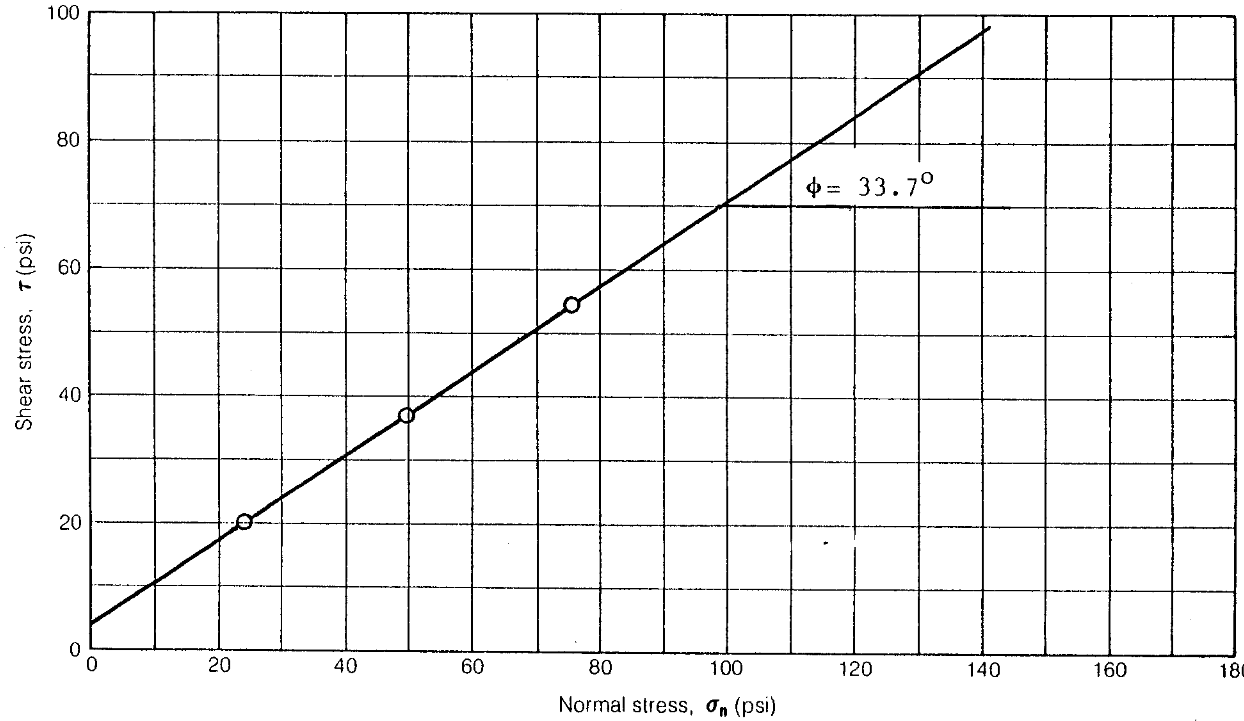
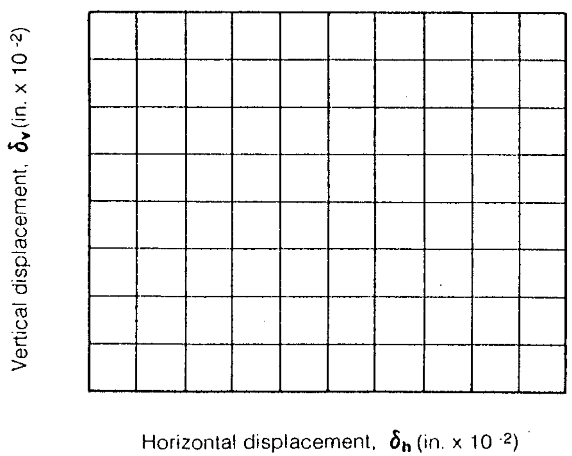
PROFILE AND ASSUMED FAILURE SURFACE FOR PLATEAU MINE ROAD

FIGURE NO. 17







Test no. or symbol	Sample size (inches)	Sample data: Dry density (pcf)	Sample data: Moisture content (%)	Degree of saturation (%)	Normal stress σ_n (psi)	Maximum shear stress τ (psi)	Strain rate (inches / minute)	Shear strength parameters: Friction angle ϕ (degrees)	Shear strength parameters: Cohesion (c / psi)
	2	97.6	17.2	100	24.5	20.4	.0047		
	2	97.6	17.2	100	49.9	37.5	.0047	33.7°	4.0
	2	97.6	17.2	100	77.1	53.8	.0047		

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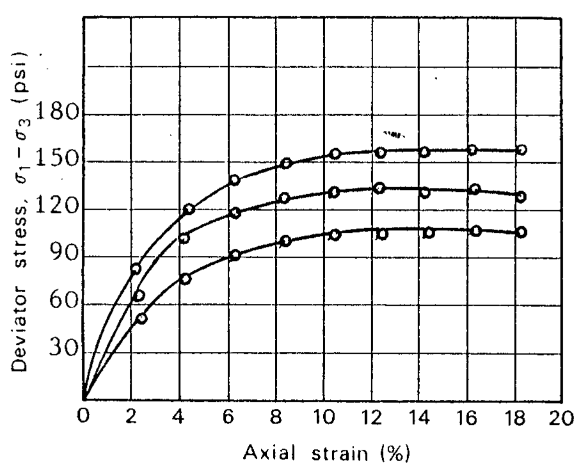
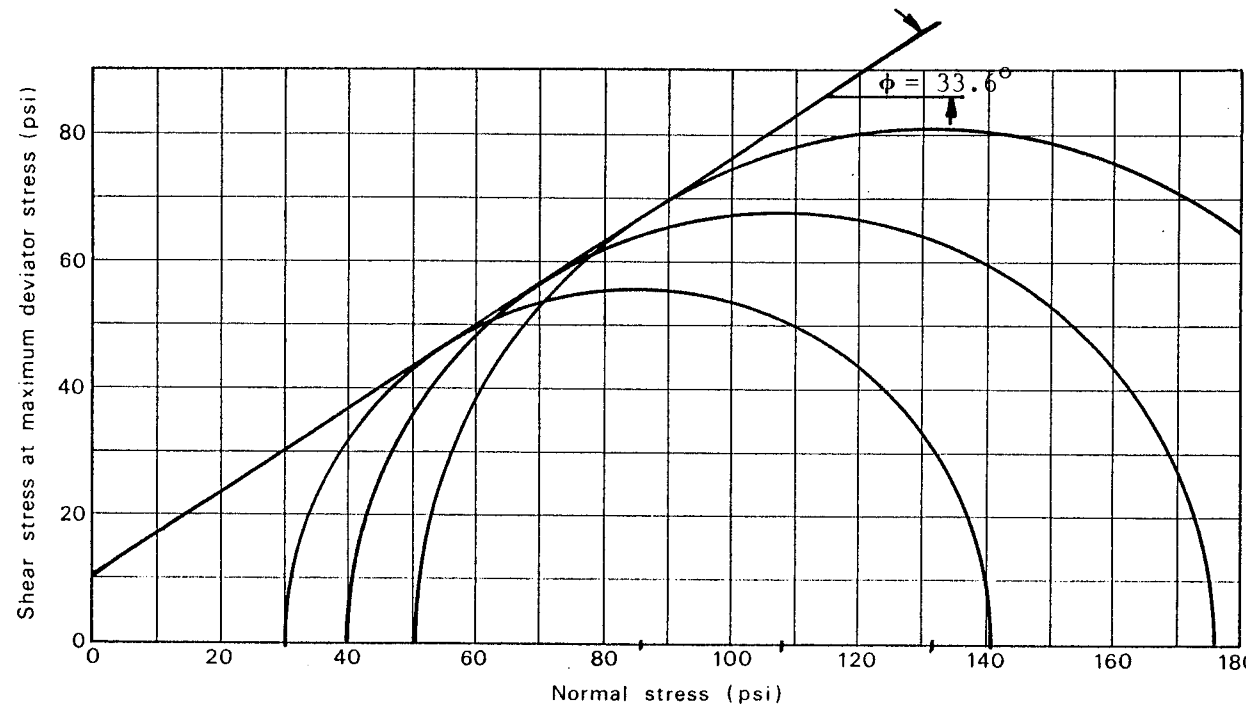
DIRECT SHEAR TEST

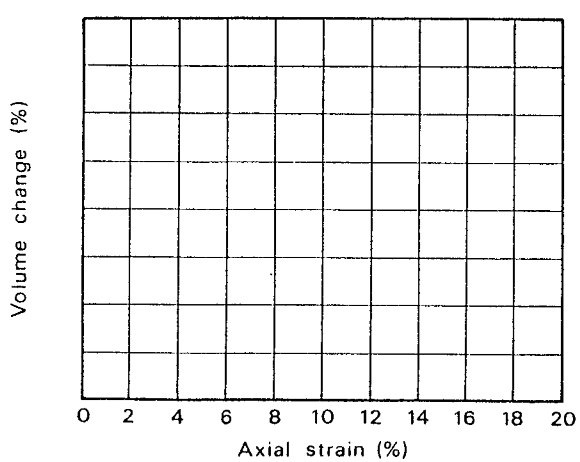
Project: Plateau Mine Road
Price, Utah

HOLE NO. 1
DEPTH: 20-21.5'

FIGURE NO. 18







Test no. or symbol	Boring no. or depth	Sample data: Dry density (pcf)	Sample data: Moisture content (%)	Degree of saturation (%)	Confining pressure (psi)	Maximum deviator stress (psi)	Strength values at failure: Friction angle ϕ (degrees)	Strength values at failure: Cohesion (c/psi)	Sample size, L/D (inches)	Strain rate (inches/minute)
8	10-11.5'	118.5	13.0	100	30	111.6	33.6	10	5/2.5	.0025
8	5-6.5'	122.1	11.8	100	40	134.0	33.6	10	5/2.5	.0025
8	10-11.5'	118.5	13.0	100	50	162.0	33.6	10	5/2.5	.0025

ROLLINS, BROWN AND GUNNELL, INC.
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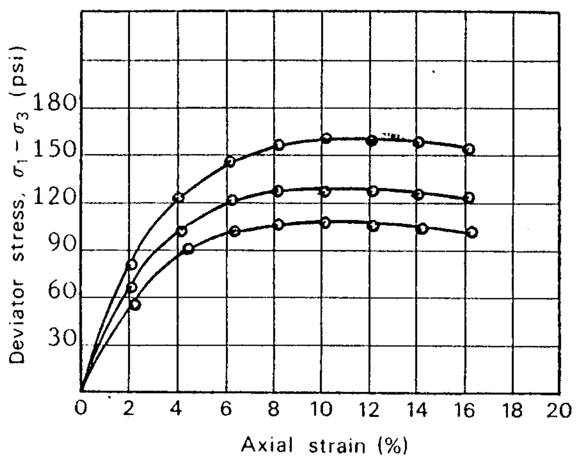
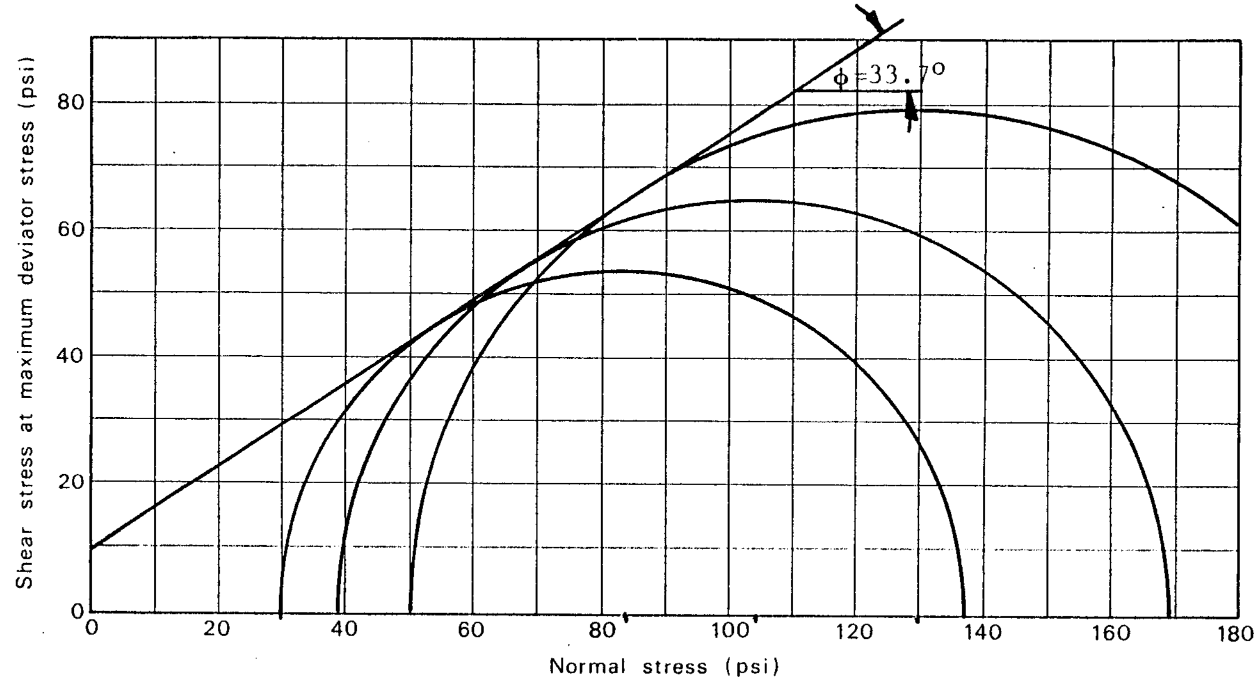
TRIAXIAL SHEAR TEST

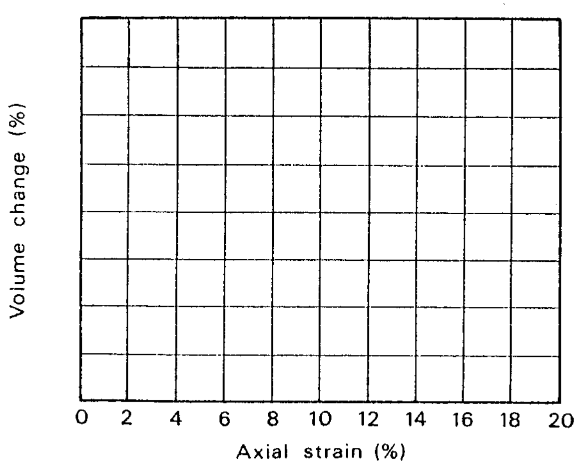
Project: Plateau Mine Road
Price, Utah

HOLE NO. 8
DEPTH: 5-11.5'

FIGURE NO. 19







Test no. or symbol	Boring no. or depth	Sample data		Degree of saturation (%)	Confining pressure (psi)	Maximum deviator stress (psi)	Strength values at failure		Sample size, L/D (inches)	Strain rate (inches/minute)
		Dry density (pcf)	Moisture content (%)				Friction angle ϕ (degrees)	Cohesion (c/psi)		
8	40-41'	117.3	13.3	100	30	107.9	33.7	10	2.8/1.32	.0016
8	40-41'	117.3	13.3	100	40	130.4	33.7	10	5/2.5	.0025
8	45-46'	118.7	13.1	100	50	163.0	33.7	10	5/2.5	.0025

ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

TRIAXIAL SHEAR TEST

Project: Plateau Mine Road
Price, Utah

HOLE NO. 8
DEPTH: 40-46'

FIGURE NO. 20

TABLE NO. 1 SUMMARY OF TEST DATA

PROJECT Plateau Mine Road FEATURE ______ LOCATION Price, Utah

HOLE NO.	DEPTH BELOW GROUND SURFACE	STANDARD PENETRATION BLOWS PER FOOT	IN-PLACE UNIT WEIGHT LB/FT^3	IN-PLACE MOISTURE PERCENT	IN-PLACE VOID RATIO	UNCONFINED COMPRESSIVE STRENGTH LB/FT^2	FRICTION ANGLE ϕ	CONSISTENCY LIMITS L.L. %	CONSISTENCY LIMITS P.L. %	CONSISTENCY LIMITS P.I. %	MECHANICAL ANALYSIS % GRAVEL	MECHANICAL ANALYSIS % SAND	MECHANICAL ANALYSIS % SILT & CLAY	UNIFIED SOIL CLASSIFICATION SYSTEM
2	1-2.5'	31									28.5	31.7	39.8	SM
	5-6.5'	22									66.3	15.6	18.1	GM
	10-11.5'	28									57.6	17.4	25.0	GM
	15-16.5'	65/4"									14.5	38.5	47.0	SM
3	0-1.5'	49									20.0	26.7	53.3	CL-1
	5-6.5'	40									33.7	35.6	30.7	SM
	10-11.5'	9									35.2	22.1	42.7	GC
	16-17.5'	15				3397					26.0	30.0	44.0	SC
	21-22.5'	30									24.3	33.6	42.1	SC
	25-26.5'	19									62.7	17.5	19.8	GM
4	0-1.5'	58									28.9	41.5	29.6	SM
	5-6.5'	8						21.4	15.9	5.5				CL-ML
	10-11.5'	8									30.9	43.0	26.1	SM
	15-16.5'	16									29.1	27.5	43.4	SM
	20-21.5'	16									37.3	28.3	33.9	GM

TABLE NO. 1 SUMMARY OF TEST DATA

PROJECT Plateau Mine Road FEATURE ____ LOCATION Price, Utah

HOLE NO.	DEPTH BELOW GROUND SURFACE	STANDARD PENETRATION BLOWS PER FOOT	IN-PLACE			UNCONFINED COMPRESSIVE STRENGTH LB/FT^2	FRICTION ANGLE ϕ	CONSISTENCY LIMITS			MECHANICAL ANALYSIS			UNIFIED SOIL CLASSIFICATION SYSTEM
			UNIT WEIGHT LB/FT^3	MOISTURE PERCENT	VOID RATIO			L.L. %	P.L. %	P.I. %	% GRAVEL	% SAND	% SILT & CLAY	
4	25-26.5'	9									20.5	67.9	11.6	SP,SM
6	5-6.5'	10						26.2	17.3	8.9				CL-1
	10-11.5'	13						27.8	16.3	11.5				CL-1
	15-16.5'	5						23.6	16.1	7.5				CL-1
	20-21.5'	7									34.9	23.6	41.5	GC
	25-26.5'	11									42.2	32.3	25.5	GC
8	0.5-2.0'	23						18.7	16.0	2.7				ML
	5-6.5'	Shelby	122.1	11.8				27.2	16.2	11				CL-1
	10-11.5	Shelby	118.5	13.0			33.6	19.9	15.1	4.8				CL-ML
	40-41.5	Shelby	117.3	13.3				23.7	16.7	7.0				CL-ML
	45-46.5'	Shelby	118.7	13.1			33.7	22.6	16.4	6.2				CL-ML
9	.5-2.0'	22									21.7	39.8	38.5	SM
	5-6.5'	8									44.7	30.4	24.9	GM
	10-11.5'	6									30.7	37.2	32.1	SM
	15-16.5'	7									45.8	22.1	32.1	GC

TABLE NO. 1 SUMMARY OF TEST DATA

PROJECT Plateau Mine Road FEATURE ______ LOCATION Price, Utah

HOLE NO.	DEPTH BELOW GROUND SURFACE	STANDARD PENETRATION BLOWS PER FOOT	IN-PLACE UNIT WEIGHT LB/FT^3	IN-PLACE MOISTURE PERCENT	IN-PLACE VOID RATIO	UNCONFINED COMPRESSIVE STRENGTH LB/FT^2	FRICTION ANGLE ϕ	CONSISTENCY LIMITS L.L. %	CONSISTENCY LIMITS P.L. %	CONSISTENCY LIMITS P.I. %	MECHANICAL ANALYSIS % GRAVEL	MECHANICAL ANALYSIS % SAND	MECHANICAL ANALYSIS % SILT & CLAY	UNIFIED SOIL CLASSIFICATION SYSTEM
9	20-21.5'	9						23.5	15.5	8.0				CL-1
	25-26.5'	12						26.8	17.0	9.8				CL-1
	35-36.5'	25						23.7	18.6	5.1				CL-ML
	45-46.5'	34						26.2	16.6	9.6				CL-1
	55-56'	60/3"						16.6	16.1	0.5				ML
1	10-11.5'					2258								
	20-21.5'		97.6	17.2										