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DIVISION OF
OIL, GAS & MINING

SOIL AND FOUNDATION INVESTIGATION

PLATEAU MINE EXPANSION PHASE II
WATTIS, UTAH

February 1984

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PROFESSIONAL ENGINEERS

February 24, 1984

Getty Oil Company
Coal Department
5250 South 300 West, Suite 200
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Attn: Bill Whitney

Gentlemen:

This report outlines the results of a supplementary soil and foundation investigation performed at the site of the Plateau Mine Expansion Phase II in Wattis, Utah. A soil investigation was previously performed in this area for the contemplated coal handling facilities. Since performing the original soil investigation, some changes have been made in the location of the proposed facilities; and this report outlines the results of a soil and foundation investigation to define the characteristics of the subsurface material in the area where the facilities will now be located. Information obtained in the original investigation is used in this report where applicable to arrive at foundation design recommendations. Substructures for which foundation recommendations are provided in this report include: (1) the conical storage pile and reclaim facility, (2) the refuse bin, (3) the secondary crusher building, (4) the county road bridge, (5) the sampling building, (6) the loadout silo, and (7) a conveyor transfer tower.

The information contained in the report is discussed under the following headings: (1) Existing Site and Geological Conditions, (2) Subsurface Soil and Water Conditions, (3) Foundation Considerations and Recommendations for the various facilities indicated above, (4) Slope Stability Considerations, (5) Recommended Flexible Pavement Design for the Relocated County Road, and (6) Site Preparation, Use of On-Site Materials and Compaction Recommendations, (7) The Results of Field and Laboratory Tests.

1. EXISTING SITE AND GEOLOGICAL CONDITIONS

The proposed site is located Wattis, Utah; and the new location of the facilities indicated above are presented in

Figure No. 1. The proposed crushing plant, the preparation building and the refuse bin are all located on the south side of the main drainage channel through the area.

The subsurface material immediately adjacent to the drainage channel is recent alluvial deposits consisting of both fine- and coarse-grained material. A coal pile currently exists in a portion of the area where the conical coal pile and reclaim facility will be located. A considerable amount of coal refuse has been placed throughout the development area, and the loadout feed conveyor will cut directly across an existing refuse pile.

Wattis is located on the eastern front of the Wasatch Plateau. Geological formations existing throughout the development area include the Mesa Verde Group and the Mancos Shale. The Star Point Sandstone and the Masuk Shale were the only stratigraphic units encountered throughout the proposed site. The Star Point Sandstone is a fine- to medium-grained resistant material, while the Masuk Shale is dark gray to black, characteristic of the low-energy shallow marine environment in which it was formed. The shale exists in thin, horizontal beds throughout the site; and at some locations, it is highly fractured.

Streamflow in the drainage channel along the north side of the development area is intermittent, and it is not anticipated that any flow in this channel will affect foundation performance for any facilities in the development area. Other than the information provided above, no environmental factors appear to exist at this location which would adversely affect foundation performance.

2. SUBSURFACE SOIL AND WATER CONDITIONS

The investigative program contemplated for the development area included 15 test borings to be drilled at locations as shown in Figure No. 1. Test Holes 9, 12, 13, and 14 are located on Federal lands; and at the time the drilling was completed, access had not yet been obtained for drilling on this property. Test Hole No. 9 was inaccessible; and when the subsurface investigation was performed, it was not drilled. The logs for the eleven other test holes drilled throughout the site, are presented in Figures 2 through 10. The elevation of the ground surface for each test boring is shown on the boring logs.

During the subsurface investigation, sampling was performed at 5-foot intervals throughout the depth investigated. 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-type material, however it only provides an indication of the stiffness of cohesive materials.

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 throughout the soil profile are presented on the boring logs.

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. 11, and the meaning of the various symbols shown on the boring logs can be obtained from this figure.

Test Holes 1 through 4 were drilled in the area where the conical coal pile and the reclaim tunnel will be located. It will be observed that Test Holes 1 and 2 were both located on the coal stockpile. The depth of the coal was approximately 47 feet at the location of Test Hole No. 1 and about 4 feet at the location of Test Hole No. 2. Refuse existed at the ground surface at the location of both Test Holes 3 and 4. It will be observed that the thickness of the refuse material in Test Holes 1 through 4 varies from about 30 feet in Test Hole No. 2 to about 59 feet in Test Hole No. 3. The subsurface material underlying the refuse in each of the Test Holes generally consisted of granular-type material classifying as either an SM- or a GM-type soil.

Test Holes 5 and 6 define the characteristics of the subsurface material in the vicinity of the refuse bin and the crusher building. Test Hole No. 5 consists predominantly of cohesive-type materials with some interbedded granular zones. Test Hole No. 6, which was drilled to a depth of about 27 feet, consists of a cohesive zone bounded by about 9 feet of granular material in the upper portion of the soil profile and a brown, silty gravel in the bottom part of the profile.

The boring logs for Test Holes 7 and 8, which were drilled in the vicinity of the highway bridge, are presented in Figure No. 7; and it will be observed that Test Hole No. 7 consisted predominantly of granular material throughout the entire depth investigated, while Test Hole No. 8 consisted of interbedded layers of cohesive material and granular material in the upper 20 feet of the soil profile underlain by a gray-brown, weathered shale. The gray-brown weathered shale was cored during the field investigations, and the percent recovery and the Rock Quality Designation (RQD) for the shale material are shown on the boring logs.

Test Hole No. 10, which was drilled in the vicinity of the sampling building consisted of a surface layer of cohesive material approximately 6 feet thick underlain by a gray to brown sand which extended to the depth at which the borings were terminated.

The log for Test Hole No. 11, which was located at the loadout silo, is presented in Figure No. 9. It will be observed that the subsurface material throughout the entire depth investigated at this site consisted of a gray shale. The upper 17 feet of the soil profile consisted of a gray, weathered shale which could be sampled using a split-spoon sampling spoon. Below this depth, however, coring was necessary to obtain a sample. The percent core recovery along with the Rock Quality Designation (RQD) is shown for the shale throughout the depth investigated.

Test Hole No. 15, which is located at a transfer station, is presented in Figure No. 10. It will be observed from Figure No. 10 that the upper 7 feet of the soil profile consists predominantly of a dark brown, silty sand. The remainder of the profile consisted of a gray shale. The shale material was cored, and the percent core recovery along with the Rock Quality Designation (RQD) are presented on the boring log. It will be observed that the upper portion of the shale zone is in a weathered state.

No groundwater was encountered in any of the test holes excavated at this site, and it is not anticipated that the zone of significant stress will be saturated for any of the structures by natural ground water.

3. FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

As indicated earlier in this report, foundation recommendations are provided for the conical coal pile and reclaim tunnel, the refuse bin, the secondary crusher building, the county road bridge, the sampling building, the loadout silo, and a conveyor transfer tower. As of the preparation of this report, the structural loads for all of the facilities discussed in this report are not available. Foundation loads for a portion of the facilities, however, have been determined from "Specifications for Soils Investigations" prepared by Dravo, where the details of the structural loads and the configuration of the proposed facility is not known, various assumptions have been made relative to the loading conditions for these facilities. Recommendations for each of the facilities outlined above are discussed below as follows:

(A) Conical Storage Pile and Reclaim Facility

This facility has been moved to the south and west of the location considered in the previous investigation. The results of the test borings indicate that 30 to 60 feet of refuse covers the area where the reclaim tunnel will be located. While the depth of the refuse material is somewhat less than the depth of the refuse in the area where the storage pile and the reclaim tunnel were originally contemplated, the borings still indicate that the refuse material is in a relatively loose condition and a considerable variation occurs in the depth of this material throughout the development area which could lead to differential movement.

We understand that the reclaim tunnel will have an overall width of about 12 feet, an overall height of about 14 feet, and a length of 326 feet. It is anticipated that the conical coal pile will be about 95 feet high and that the maximum load intensity on the tunnel will be 6,600 pounds per square foot. Since the base of the conical coal pile covers a considerable area, stresses for this facility will extend completely through the refuse material.

The results of a settlement analysis performed using the data obtained during the original soil investigation indicate that the settlement of the refuse material, due to the weight of the coal pile, may approach 3 to 4 inches. It is our opinion that a settlement of this magnitude may result in undesirable differential settlement within the reclaim tunnel. The hazards associated with the differential settlement for this facility could be considerably reduced if a portion of the refuse material is excavated and replaced with compacted granular fill. It is apparent from the size of the tunnel that an excavation at least 14 feet deep will be required to provide room for the tunnel. If an additional 16 feet of refuse material is excavated, most of the refuse material will be removed in areas where Test Holes 1, 2, and 4 are located; and the amount of settlement occurring in the remainder of the soil profile would be within tolerable limits.

In order for the compacted fill to serve adequately, the width of the compacted fill should be equal to at least twice the width of the footing. We, therefore, recommend that the minimum width of the compacted fill supporting the reclaim tunnel be approximately 35 feet. Recommendations for densification of the fill material beneath the reclaim facility is outlined in the following section of this report.

(B) Refuse Bin

The subsurface materials in the vicinity of the refuse bin are defined by Test Hole No. 5, and it will be noted that the subsurface material at this location consists predominantly of cohesive-type soils with some interbedded sand and gravel layers. The exact size of the refuse bin is not known as of the preparation of this report, however we understand that the bin will be supported on 4 to 6 legs and that the live load for the bin will be approximately 200 tons. For a 4-legged structure, this would amount to 50 tons per leg. For the purposes of this report, it has been assumed that the total dead and live load per leg would not exceed 200 kips.

It is apparent from the log for Test Hole No. 5 that if the foundations are located at a depth below ground just sufficient to provide frost protection, which is about 4 feet in this area, the zone of significant stress will exist within cohesive material. The results of field and laboratory tests indicate that the allowable soil pressure for the cohesive material should not exceed about 2500 pounds per square foot. For a 200 kip load, the size of the footing would be approximately 9 feet square. If the foundations for the proposed facility are proportioned in accordance with the above recommendations, the maximum settlement will not likely exceed 1 inch; and differential settlement throughout the structure will not likely exceed ½ inch, which should be tolerable for the proposed facility. It is concluded, therefore, that the proposed structure can be supported using spread footings and that other foundation types such as mats or piles will not be necessary unless the uplift requirements to prevent overturning necessitates such foundations. Some uplift resistance can be obtained from the spread foundations if they are imbedded to a sufficient depth within the subsurface material.

Recommended uplift capacities for various footing sizes and depth of imbedment are outlined in the following table:

Footing Size	Depth of Imbedment	Uplift Capacity (kips)
6 x 6	4'	37
6 x 6	8'	75
8 x 8	4'	52
8 x 8	8'	104
10 x 10	4'	68
10 x 10	8'	136

(C) Secondary Crusher Building

We understand that the secondary crusher building has been moved in a southerly and an easterly direction to a location in the vicinity of the thickener tank. The characteristics of the subsurface material at this location is defined by Test Hole No. 6. It will be noted that the subsurface material throughout this test boring, except for a clay layer in the lower portion of the profile, consists of granular material and low-plasticity silt.

We understand that the crusher building will be approximately 28 feet wide and 48 feet long. We also understand that the crusher will transmit its loads to the building columns and that the maximum down load will be 195 kips and that the maximum shear load will be 52 kips. It is apparent from the log for Test Hole No. 6 that approximately 4 feet of fine coal refuse covers the area where the test hole was drilled.

If the foundations for the proposed facility are located at a depth below ground surface just sufficient to provide frost protection, which is about 4 feet in this area, the bottom of the footings will be located on the gray, gravelly sand. The gravelly sand is in a medium-dense condition, and it is our opinion that the proposed facility can be supported using spread foundations on the granular material. In the event that the granular material is not encountered at the foundation level, we recommend that the existing material be excavated and replaced with compacted granular fill. At least 5 feet of compacted granular fill should exist beneath all foundations for the proposed structure. The bearing capacity chart as shown in Figure No. 12 has been prepared so that the allowable soil bearing pressure of the cohesive material beneath the granular layer will not be exceeded. It will be noted from the bearing capacity chart that the bearing capacity is a function of the width of the footing and that the allowable soil bearing pressure decreases as the footing width increases.

If the foundations for the proposed facility are proportioned in accordance with the above recommendations, the maximum settlement of any footing should not exceed 1 inch, and differential settlement throughout the structure should not exceed ½ inch. If this magnitude for the differential settlement is not satisfactory for the proposed facility, it is requested that we be advised in order that appropriate modifications can be made in the foundation recommendations.

(D) County Road Bridge

The characteristics of the subsurface material in the zone of significant stress for the bridge foundations are defined by Test Holes 7 and 8. It will be observed that the subsurface material in the entire soil profile for Test Hole No. 7 is granular material. The soil profile for Test Hole No. 8 consists of granular material interbedded with some cohesive material and underlain with a gray-brown weathered shale. Visual observation of the subsurface material in the railroad cut also indicated that the subsurface materials were predominantly granular-type soils.

It is our opinion that the foundations for the abutments for the proposed bridge can be supported using spread foundations on the natural material. It is recommended that prior to constructing the bridge over the railroad that the side slopes in the railroad excavation be cut back so that the slope is 1.5 horizontal to 1 vertical. In order to provide frost protection for the foundations for the proposed structure, we recommend that the bottom of the footing be placed at least 4 feet below the finished grade.

If these requirements are met, the allowable soil bearing capacity can be determined from Table No. 2. It will be noted that the bearing capacity is a function of the width of the footing as well as the distance of the footing from the edge of the slope. Under these conditions, the differential settlement between abutments should not exceed about 3/4 of an inch, which should be satisfactory for the proposed facility.

(E) Sampling Building

We understand that the sampling building will be approximately 22 feet wide and 30 feet long and that the wall height for this facility will be approximately 50 feet. Test Hole No. 10 defines the characteristics of the subsurface material at the proposed site for this facility. If the foundations are located at a depth below ground surface just sufficient to provide frost protection, which is about 10 feet in this area, the zone of significant stress will most likely exist within both the silt zone and the underlying sandy material. If the footing widths are narrow, most of the zone of significant stress will exist within the silts; however if the footings are wide, a considerable portion of the zone of significant stress will exist within the sandy material. The magnitude of the structural loads are not known as of the preparation of this report, however we understand that the proposed structure will be a metal building and that the proposed facility will be supported primarily on spot footings. For the purposes of this report, it is assumed that the maximum column load will not likely exceed 75 kips. The assumption has also been made that the major portion of the zone of significant stress will likely exist within the silty material. In view of this fact, we recommend that an allowable soil bearing pressure of 1500 pounds per square foot be used to proportion the foundations for the proposed facility. If the foundations for the proposed structure are proportioned in accordance with this value, the maximum settlement will not likely exceed 1 inch; and differential settlement throughout the structure will not likely exceed ½ inch, which should be tolerable for the proposed facility.

(F) Loadout Silo

It is our understanding that the loadout silo will have an inside diameter of 70 feet, a height of 210 feet, and a coal load of 10,500 tons. The entire profile in both test holes was a gray shale; however in Test Hole No. 11, sampling could be performed with a split-spoon sampling tube to a depth of about 17 feet below the existing ground surface, while in Test Hole No. 23, which was drilled in the same general area during the original investigation, sampling could only be performed with a split-spoon sampling tube to a distance of about 8 feet.

The rock quality designation below a depth of about 35 feet is similar in each hole, with the values generally greater than 90 percent. Above 35 in Test Hole No. 11, the rock quality designation between 17 and 20 feet and 27 and 37 feet varies from 13 to 28 percent, which indicates relatively pour rock. In Test Hole No. 23, however, the rock quality designation between 7 and 38 feet varied from about 63 percent to 100 percent, which indicates that the rock in this test hole is somewhat better than the rock in Test Hole No. 11. Part of the differences, however, may be accounted for in the difference in the elevations between the two holes. The elevation of the ground surface for Test Hole No. 11 was at 7067, while the elevation of the ground surface for Test Hole No. 23 was 7060.

The unconfined compressive strength of the shale material below elevation 7050 in Test Hole No. 11 varied from 1,522 psi to 3,396 psi. The unconfined compressive strength in Test Hole No. 23 also had similar strengths. The bearing capacity of this material is well in excess of the 10,000 pounds per square foot required for the silo. It is our understanding that the elevation of the bottom of the footings for the silo will be located at between 7054 and 7057. If this elevation is correct, it is apparent from Test Hole No. 11 that some of the highly weathered shale will exist beneath the foundations for the proposed facility. We recommend that all of the highly weathered shale be excavated and replaced with compacted granular fill. It should be noted that Test Hole No. 11 was drilled along the hillside at the periphery of the silo and it is entirely possible that when the knoll where the silo will be located is cut down to the appropriate elevation that hard, unweathered shale will exist at the foundation elevation and that no filling with granular material will be required.

Laboratory tests performed on the shale beneath the foundation elevation of the silo indicate that these materials have expansive characteristics and recommendations made in the original report relative to waterproofing the site should be followed.

(G) Conveyor Transfer Tower

It is apparent from Figure No. 1 that a transfer tower will be located in the area where Test Hole No. 15 was drilled. The magnitude of the power loads are not known as of the preparation of this report, however we understand that this is a relatively small structure, with light loads. It is apparent from Test Hole No. 15 that the zone of significant stress will most likely exist within granular materials. The thickness of the granular material will depend upon the depth of the cut at this particular location.

In order to proportion the foundations in this area, a bearing capacity chart as shown in Figure No. 13 has been prepared for this site. The bearing capacity chart assumes that the material within the zone of significant stress will be granular-type soils. The chart has similar characteristics to other bearing capacity charts presented earlier in this report.

If the foundations for the transfer tower are proportioned in accordance with Figure No. 13, the maximum settlement of any foundation will not exceed 1 inch and differential settlement throughout the structure will not exceed ½ inch, which in our opinion will be satisfactory for the proposed facility.

4. SLOPE STABILITY CONSIDERATIONS

When the supplementary soil investigation was initiated, it was contemplated that test borings would be drilled at locations 12, 13, and 14. These test borings were to be drilled to a sufficient depth that basic information would be available to determine the slope stability characteristics where cuts will be required through this area. The area where these test holes are located are on Bureau of Land Management property and the test holes have not been drilled at this date.

In the absence of basic information relative to the subsurface material at this site, the slopes at which cuts will be stable in this area have been deduced by other means. It can be observed throughout the area that the Star Point Sandstone generally forms cliffs and that the Masuk Shale weathers to a stable slope over an extended period of time. Observations throughout the general area indicate that the existing slopes can be classified as short-term and long-term slopes. In order to obtain some indication of the long-term slopes throughout the area, four cross-sections have been considered at locations as shown in Figure No. 14. The profiles for these cross-sections are presented in Figures 15 and 16. It will be observed that the long-term slopes generally vary from 1.3 horizontal to 1 vertical to 1.7 horizontal to 1 vertical.

It is our understanding that the railroad located on the south side of the development area in the vicinity of the loadout silo was constructed in 1923. Two cuts along this alignment are shown in Figure No. 14 and are designated as E-E and F-F. The cross-section for these slopes is presented in Figure No. 17, and it will be observed that the slopes for these cross-sections vary from 0.5 horizontal to 1 vertical to 0.9 horizontal to 1 vertical. Based on these observations, it is our opinion that the cuts along the conveyor alignment will be entirely stable for slopes of 1 horizontal to 1 vertical throughout the life of the proposed facility.

5. RECOMMENDED FLEXIBLE PAVEMENT DESIGN FOR THE RELOCATED COUNTY ROAD

The development of the proposed facilities in the Plateau Mine area require that the county road leading to the mine be relocated. The relocation is not shown in Figure No. 1, however the area where the roadway crosses the railroad tracks is shown. It is anticipated that some cutting and filling will be required to establish the finished grade for the proposed roadway. No test holes were drilled along the proposed alignment, however a review of all of the test holes drilled throughout the development area indicates that the overburden material generally consists of interbedded silt, sand, and clay layers. Silt and clay zones are frequently found at the ground surface. It is our opinion, therefore, that the thickness of the flexible pavement will depend primarily upon the characteristics of the surface silt, sand, or clay materials. Satisfactory inorganic fill is relatively scarce throughout the area, however an abundance of coarse coal refuse exists throughout the development area. In order to obtain some indication of the characteristics of the coarse coal refuse for use in fill areas, moisture density relationships and CBR tests were performed on typical samples of the refuse material obtained throughout the area. The results of these tests indicate that the maximum density of the refuse material as determined by ASTM D 1557-78 varied from about 92 to 94.5 pounds per cubic foot. CBR tests performed on these materials indicate values varying from 22.5 to 28.3 percent. It is our opinion, therefore, that this material can be used in fill areas along the proposed county road, if it is bounded on the outside of the fills by natural inorganic materials occurring throughout the area.

The flexible pavement design has been made based upon the fact that no data relative to the traffic distribution likely to use the county road is available as of the preparation of this report. In the absence of such data, it has been assumed that 100 vehicles will use the road per day and that 10 percent of these vehicles would have an 18,000 pound axle load. It has also been assumed that the subgrade soils would have a CBR value of 3.5 percent and that the coarse coal refuse and the untreated granular base would have CBR values of 20 and 50 percent respectively. The flexible pavement analysis has been performed using the procedure developed by the Utah State Department of Transportation. A summary of the flexible pavement design determined for the roadway is as follows:

Situation	Asphalt Thickness	Untreated Base Thickness	Coarse Coal Refuse
fill	3.5"	6"	13"
cut	3.5"	15"	0
fill	2.5"	8"	14"
cut	2.5"	18"	0

It should be recognized that the recommended flexible pavement design is only valid for the assumed traffic distribution. If it is determined that the actual distribution is significantly different than assumed in this report, it is requested that we be advised in order that the flexible pavement recommendations can be modified.

6. SITE PREPARATION, USE OF ON-SITE MATERIALS AND COMPACTION RECOMMENDATIONS

It is apparent that considerable excavation and grading will be required to accommodate the proposed facilities throughout the development area. It is anticipated that the foundations for several structures will be located using foundations on compacted fill. It is recommended that all compacted fill supporting structural foundations be a well-graded granular material with a maximum size less than 4 inches and with not more than 10 percent passing a 200 sieve.

It is anticipated that considerable excavation, filling, and grading will be required throughout the development area to locate the proposed facilities as shown in Figure No. 1. Excavation and backfilling will be required for the reclaim tunnel facilities, some excavation and fill may be required for the secondary crusher foundations, a considerable cut will be required through the refuse pile and the natural material along the conveyor alignment and a substantial amount of excavation will be required where the loadout silo will be located. All structural backfill placed in the reclaim tunnel area and at the secondary crusher site should be a well-graded granular material with a maximum size less than 3 inches and with not more than 10 percent passing a 200 sieve. The cut slopes in the refuse material and in the natural overburden should not be steeper than 1.5 horizontal to 1 vertical. Cuts in shale should stand satisfactorily for the short range condition at slopes of 1 horizontal to 1 vertical.

It is anticipated that the coarse coal refuse and the natural overburden material removed in cut areas will be used as fill throughout the project development. In the area where the loadout silo will be located, the existing knoll should be quite highly weathered and should be satisfactory for use in fill areas. It is anticipated that the fractured shale at this location will break down readily under the action of heavy construction equipment. In cut areas along the relocated county highway, we recommend that the natural materials be scarified and redensified to an in-place unit weight equal to 90 percent of the maximum laboratory density as determined by ASTM D 1557-78. Fill areas where the coarse coal refuse is used should be densified to 95 percent of the maximum laboratory density specified above. The granular base in the county road should be densified to an in-place unit weight equal to 90 percent of the maximum laboratory density specified above and should conform to the following gradation specifications. Mineral aggregates used in the asphalt surface course should conform to Section 402 of the Standard Specifications of the Utah State Department of Transportion. Mixing, placing, and densifying the asphalt surface course should also conform to state standards.

7. THE RESULTS OF FIELD AND LABORATORY TESTS

Field and laboratory tests performed during this investigation to define the characteristics of the subsurface material throughout development area included standard penetration tests, in-place unit weight, natural moisture content, mechanical analysis, Atterberg limits, unconfined compressive strength, consolidation tests, soil moisture density relationships, and CBR tests. A summary of all tests performed during the investigation with the exception of the consolidation tests, the moisture density relationships, and the CBR tests is presented in Table No. 1, Summary of Test Data.

It is significant to note that in general the granular material existing throughout the area is relatively well-graded with a considerable amount of material in the silt- and clay-size ranges. It will also be noted that the cohesive overburden throughout the development area generally classifies as an ML a CL-ML or as CL-1 type material, which indicates that these materials have low-plasticity characteristics.

The unconfined compressive strengths are generally in excess of 2,000 pounds per square foot. The unconfined compressive strength of the shale material obtained from Test Hole No. 11 varied from 1,522 pounds per square inch to over 3,000 pounds per square inch. The compressibility characteristics of the overburden clays and the shales were defined by performing 19 consolidation tests on representative samples obtained from Test Holes 5, 11, and 15. The consolidation tests performed on the overburden materials in Test Hole No. 5 indicate that these cohesive materials are not highly compressible and that they are slightly overconsolidated. The results of the consolidation tests performed on the shale material in the upper 17 feet of Test Hole No. 11 indicate that these materials are moderately compressible and that considerable settlement will occur in these materials for large load intensities. None of the samples in this region, however, indicated highly expansive characteristics. Below a depth of 20 feet, the shales are relatively incompressible, however they do indicate some expansive characteristics.

It should be recognized that the zone below a depth of 20 feet below the ground surface is within the zone of significant stress for the silo, and it is recommended that care be taken to prevent saturation of the subsurface shales in this area. Recommendations made in the original soil report relative to waterproofing the foundation materials in the silo area should be complied with. Consolidation tests performed on the shales in the lower portion of Test Hole No. 15 indicate that these materials do not have significant expansive characteristics and that they are relatively incompressible. The results of all the consolidation tests are shown in Figures 18 through 36.

An abundance of coarse coal refuse exists throughout the development area. Considerable cost savings would occur if this material could be used in the fill for the county road. In order to obtain some indication of the physical characteristics of this material, moisture density relationships were determined for two representative samples of this material. The moisture density relationships were determined in accordance with ASTM D 1557-78, and the results of these tests are presented in Figures 37 and 38. It will be noted that the maximum densities of between 92 and 94 pounds per cubic foot were obtained for these materials. Moisture density relationships were also performed on two samples of the proposed natural fill material. These tests indicate maximum densities of 118 and 124 pcf respectively as shown in Figures 39 and 40.

CBR tests were also performed on each of these samples, and the results of the CBR tests are presented in Figures 41 and 42. It will be noted that the CBR value varied from 22 to 28 percent. Based upon these tests, it is our opinion that the coarse coal refuse could serve satisfactory in fill areas along the county road.

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 throughout the development area. If, during construction, conditions are encountered which appear to be different than those presented in the report, it is requested that we be advised in order that appropriate action may be taken.

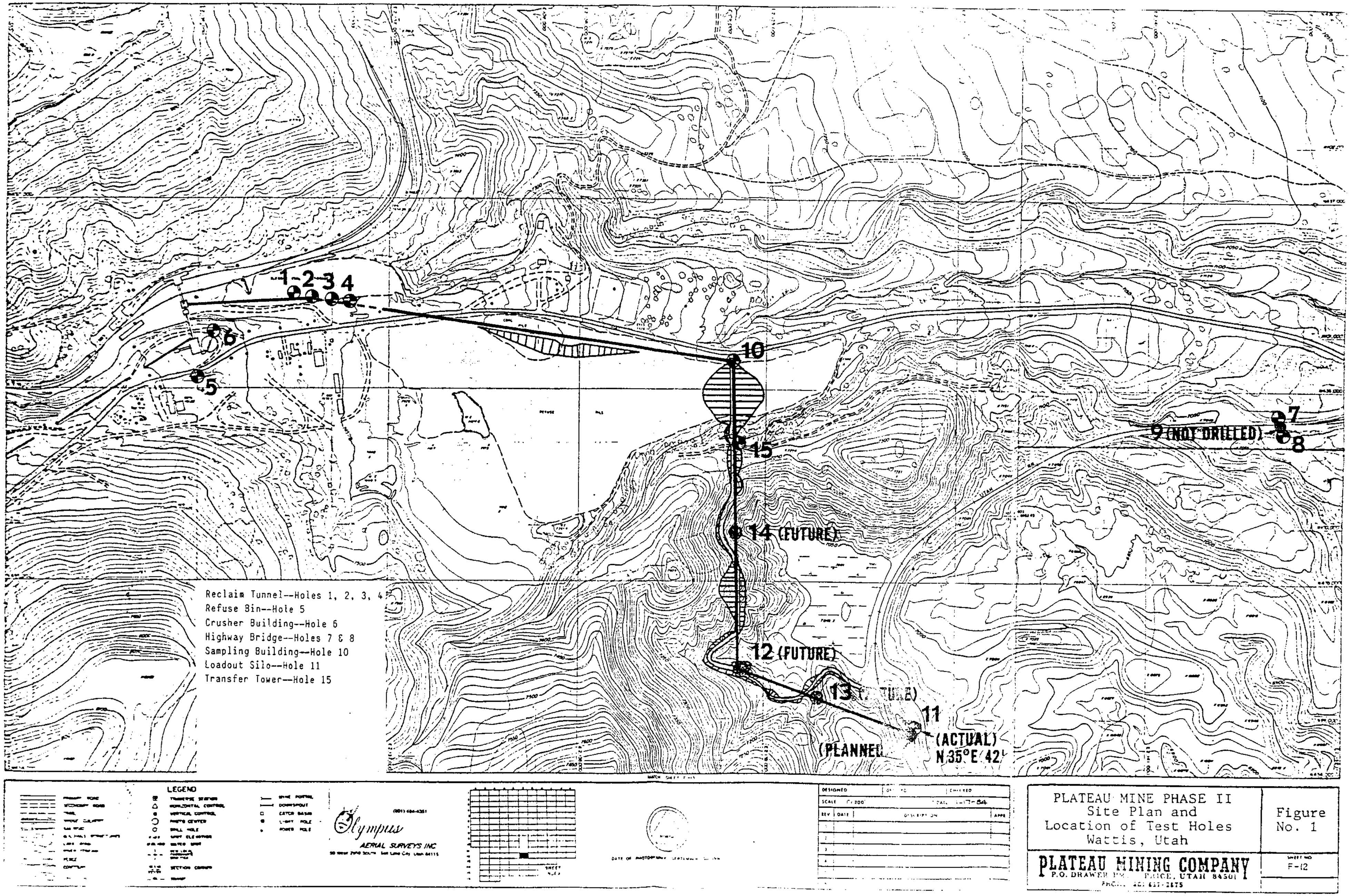
Yours truly,

ROLLINS, BROWN AND GUNNELL, INC.

Ralph L. Rollins

RLR/lah

Enclosures

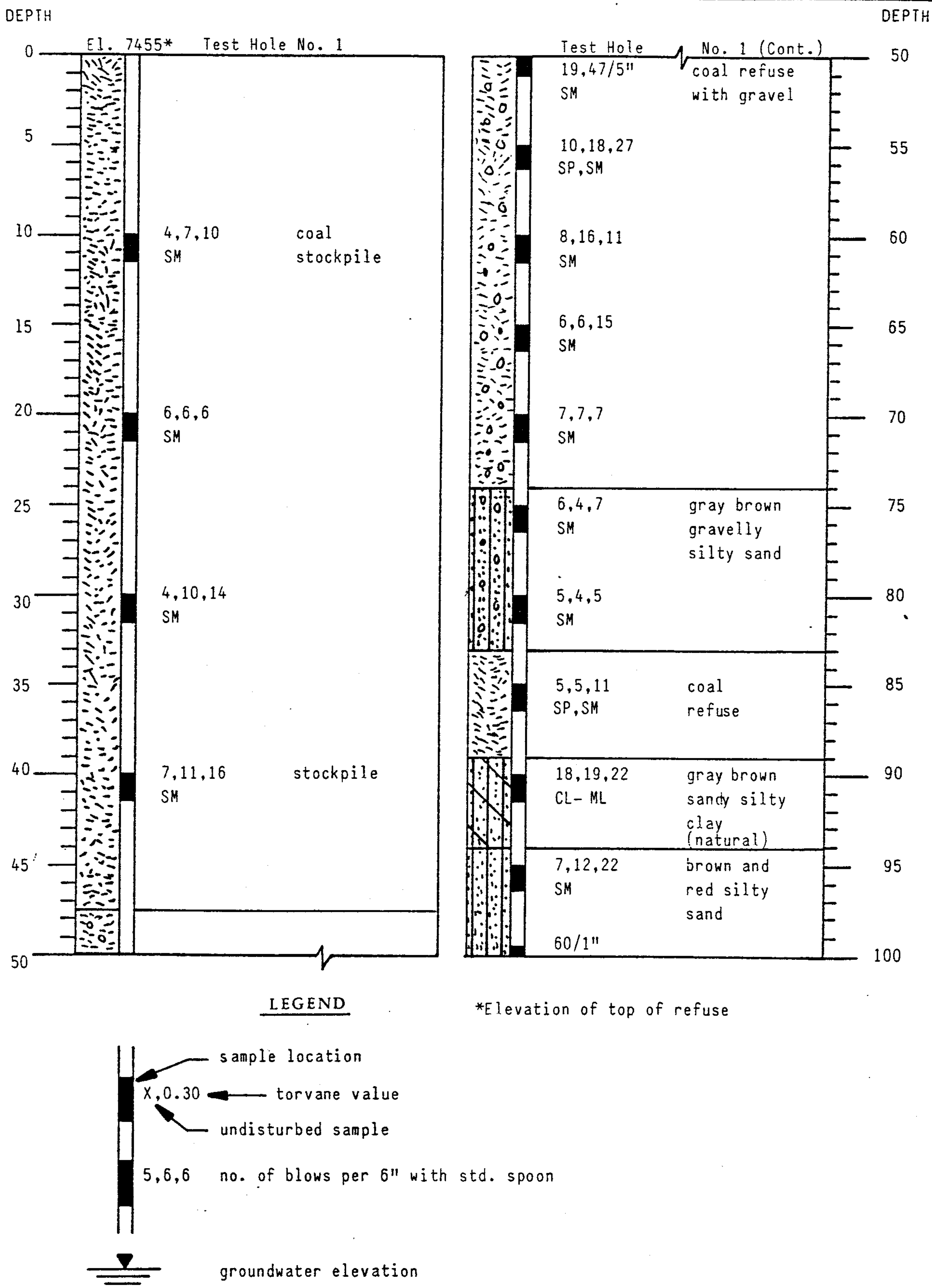


PLATEAU MINE PHASE II
Site Plan and
Location of Test Holes
Wattis, Utah

Figure No. 1

PLATEAU MINING COMPANY

SHEET NO F-12



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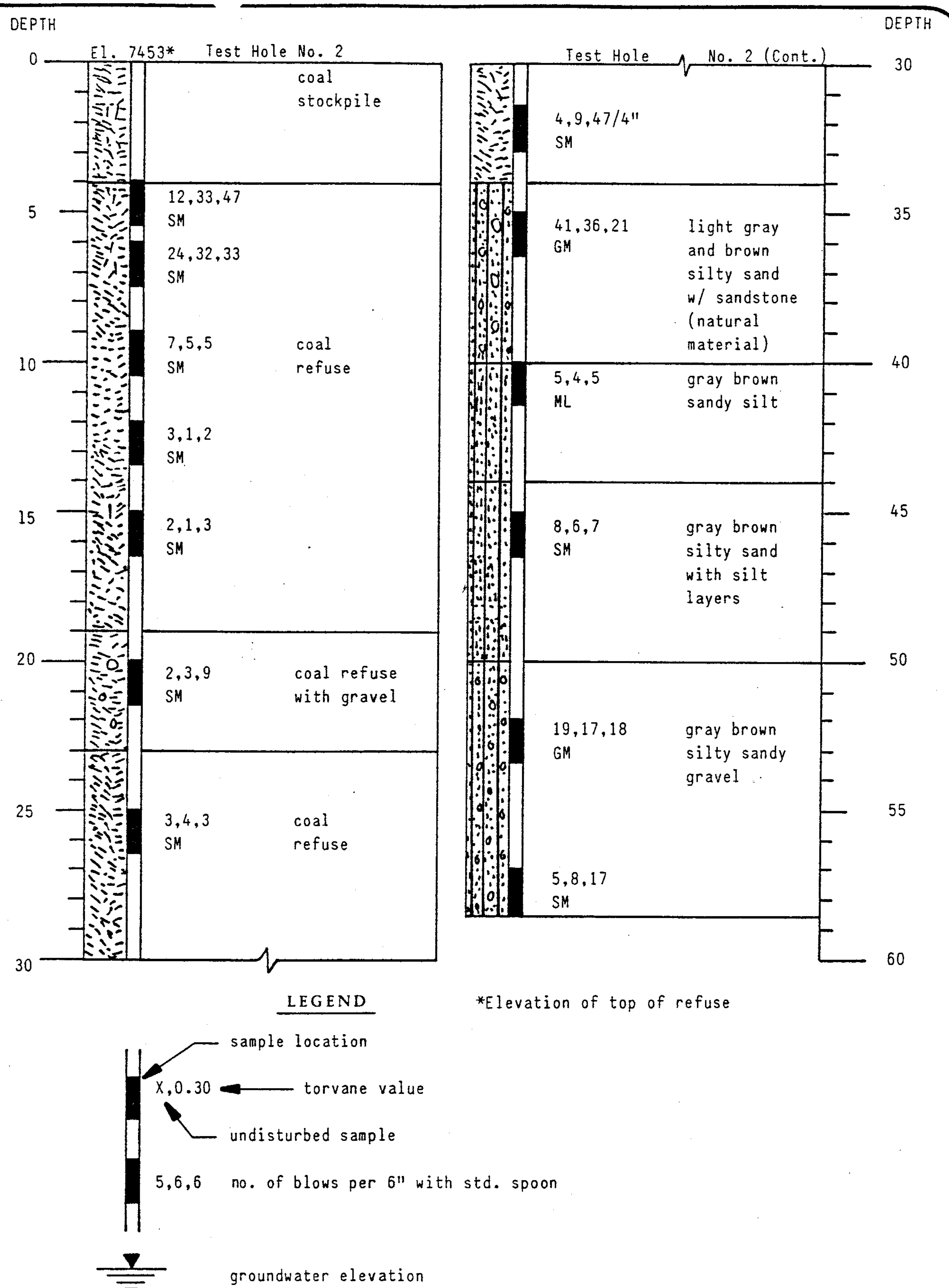
Log of Borings for:
Plateau Mine Phase II
Wattis, Utah

Figure No. 2



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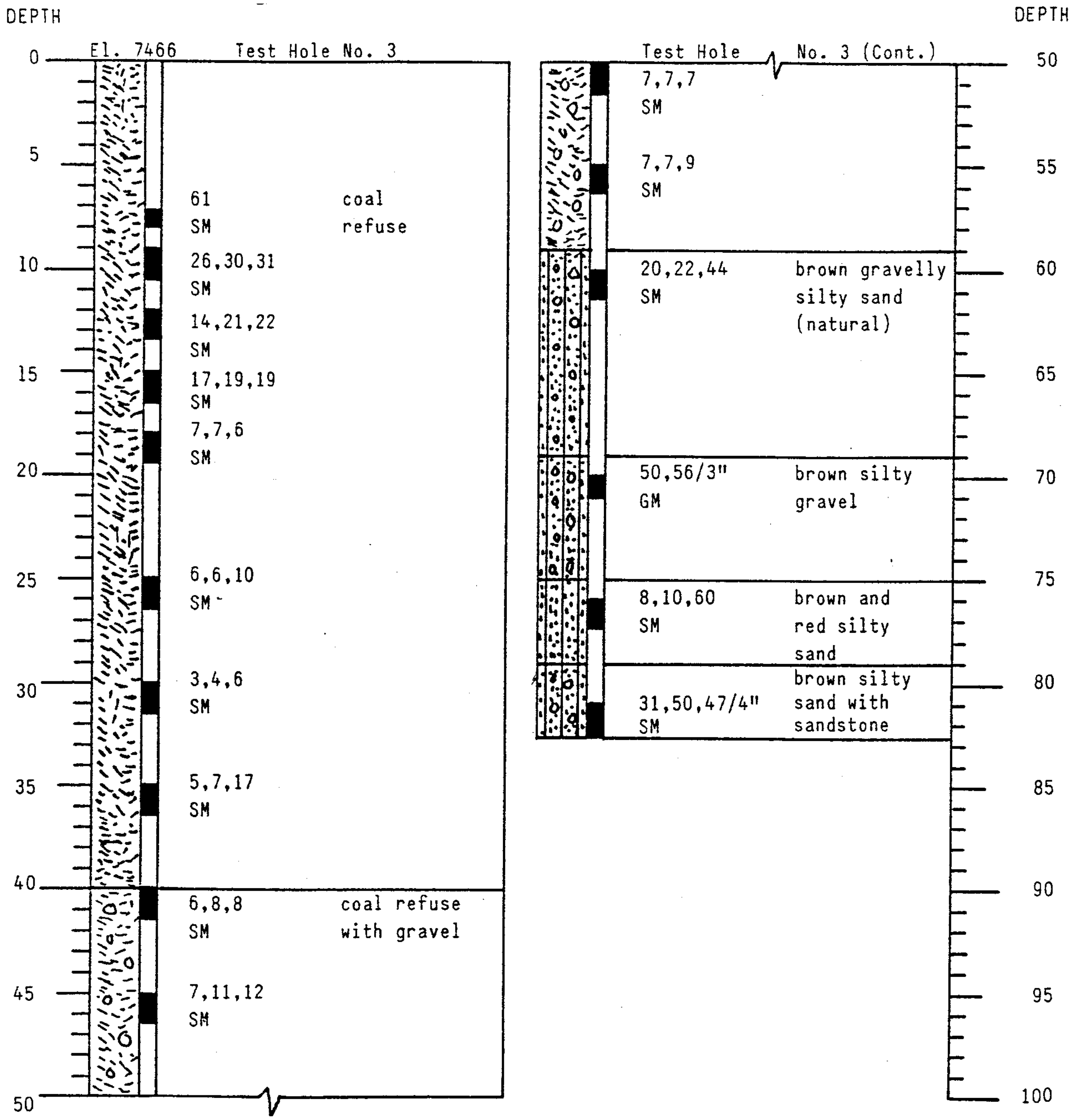
Log of Borings for:
Plateau Mine Phase II
Wattis, Utah

Figure No. 3



LEGEND

sample location

X,0.30 torvane value

undisturbed sample

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

groundwater elevation

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Log of Borings for:
Plateau Mine Phase II
Wattis, Utah

Figure No. 4

Test Hole No. 4

El. 7451

DEPTH	Blows	Description
0	18,14,14 SM	black fine coal refuse
5	11,12,9 SM	
10	2,2,2 SM	
15	3,2,2 SM	
20	6,6,5 SM	
25	7,6,6 SM	
30	4,6,9 SM	
35	6,6,6 SM	
40	5,32,30 GM	black sandy silty gravel
45	5,5,9 SM	black fine coal refuse
50		

Test Hole No. 4 (Cont.)

DEPTH	Blows	Description
50	19,60/2" ML	(natural) brown sandy silt
55	7,14,21 GM	brown silty gravel
60	42,30,35 SM	brown silty sand with gray sandstone fragments
65	16,32,60/3" ML	brown gravelly sandy clayey silt
70	100/3"	brown silty gravel
75	42,60/1"	
80		

LEGEND

sample location

X,0.30 torvane value

undisturbed sample

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

groundwater elevation

ROLLINS, BROWN AND GUNNELL, INC.
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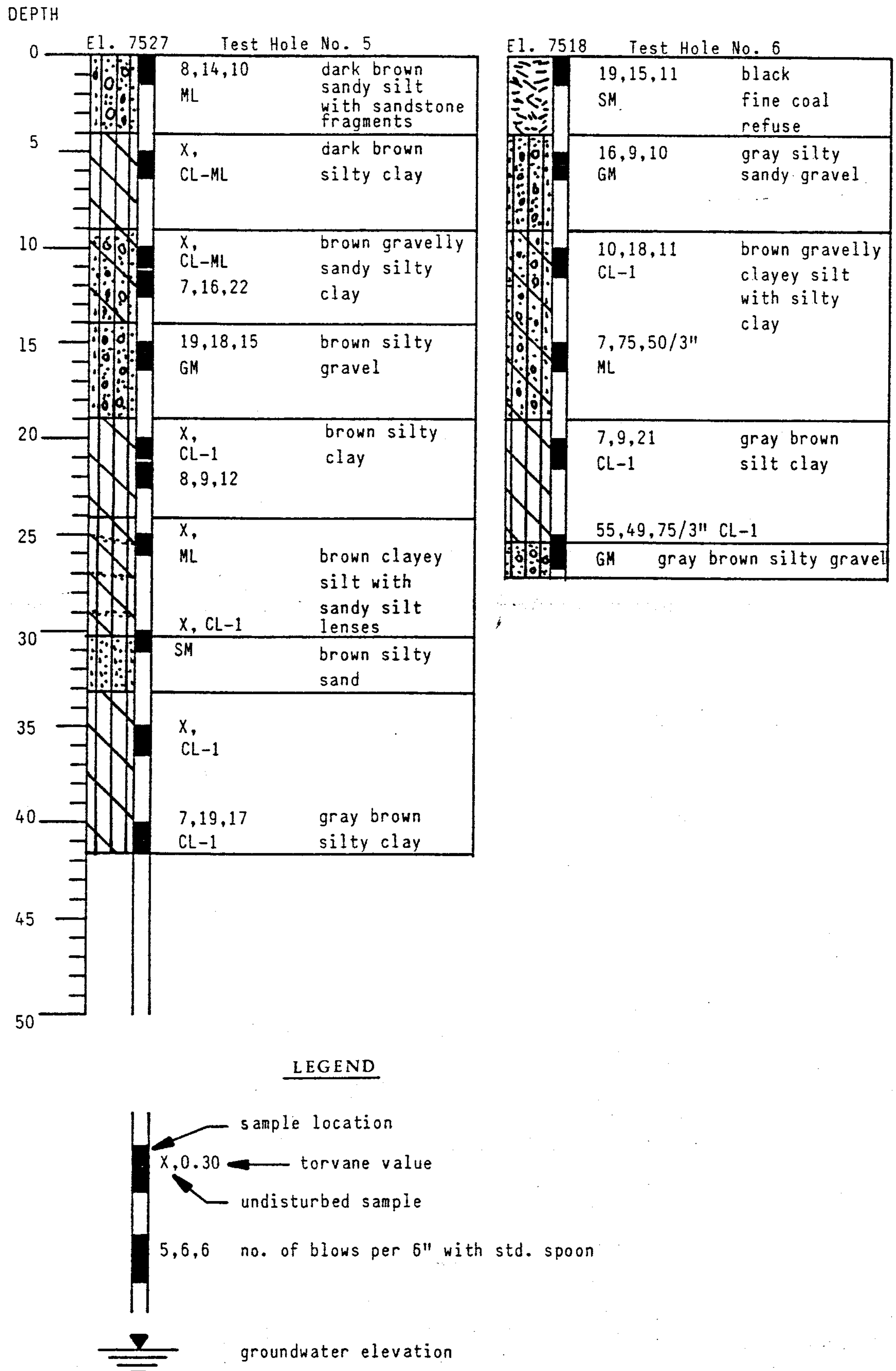
Log of Borings for:
Plateau Mine Phase II
Wattis, Utah

Figure No. 5



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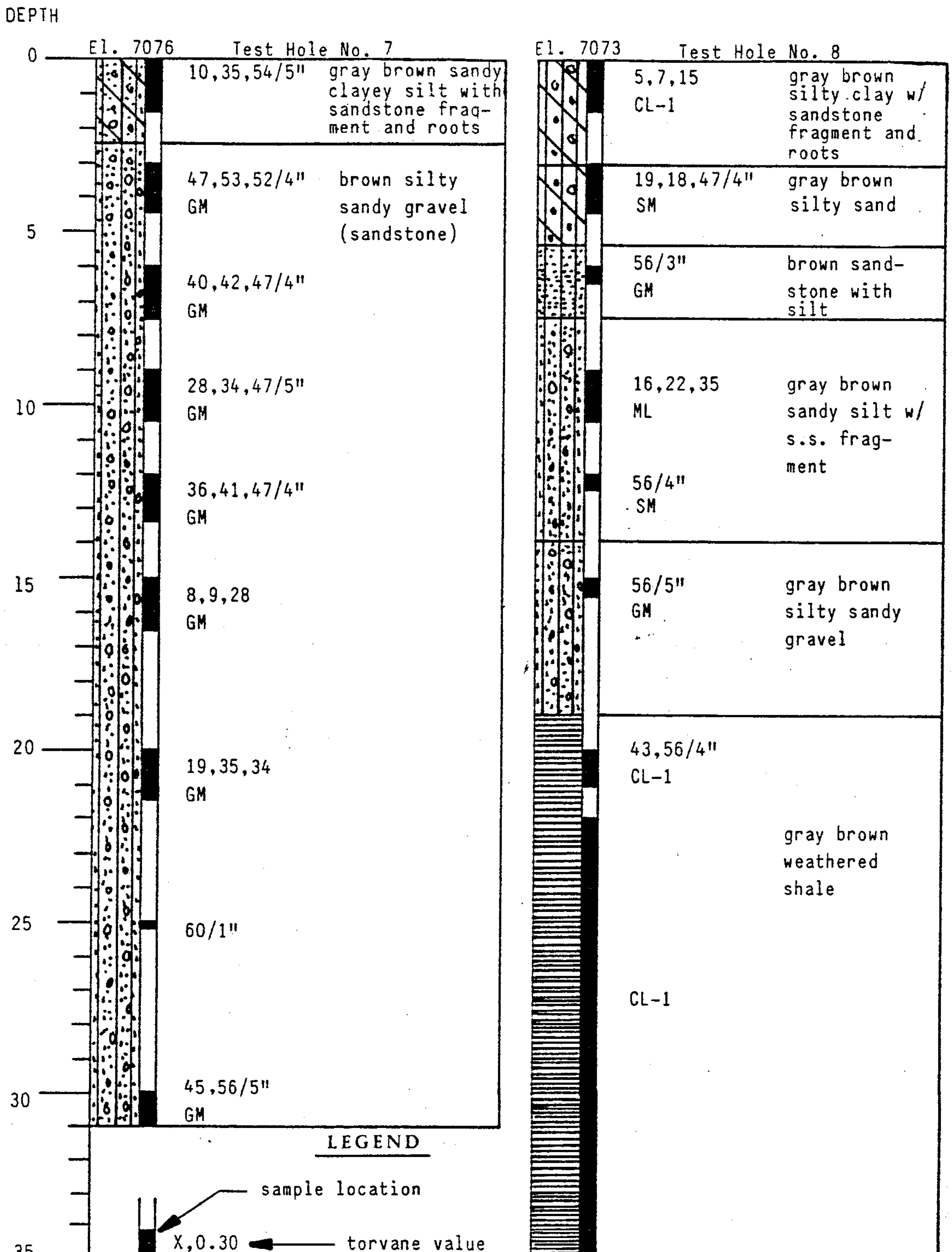
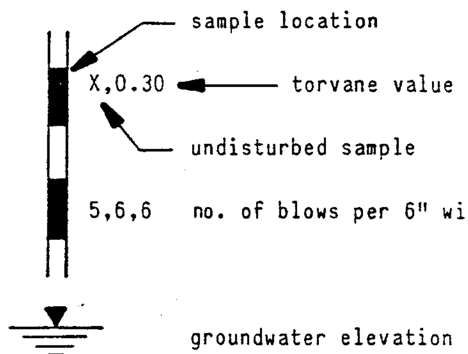
Log of Borings for:
Plateau Mine Phase II
Wattis, Utah

Figure No. 6



LEGEND



Depth	%Core Recovery	RQD
22-26	100	63
26-30	38	0
30-35	92	60

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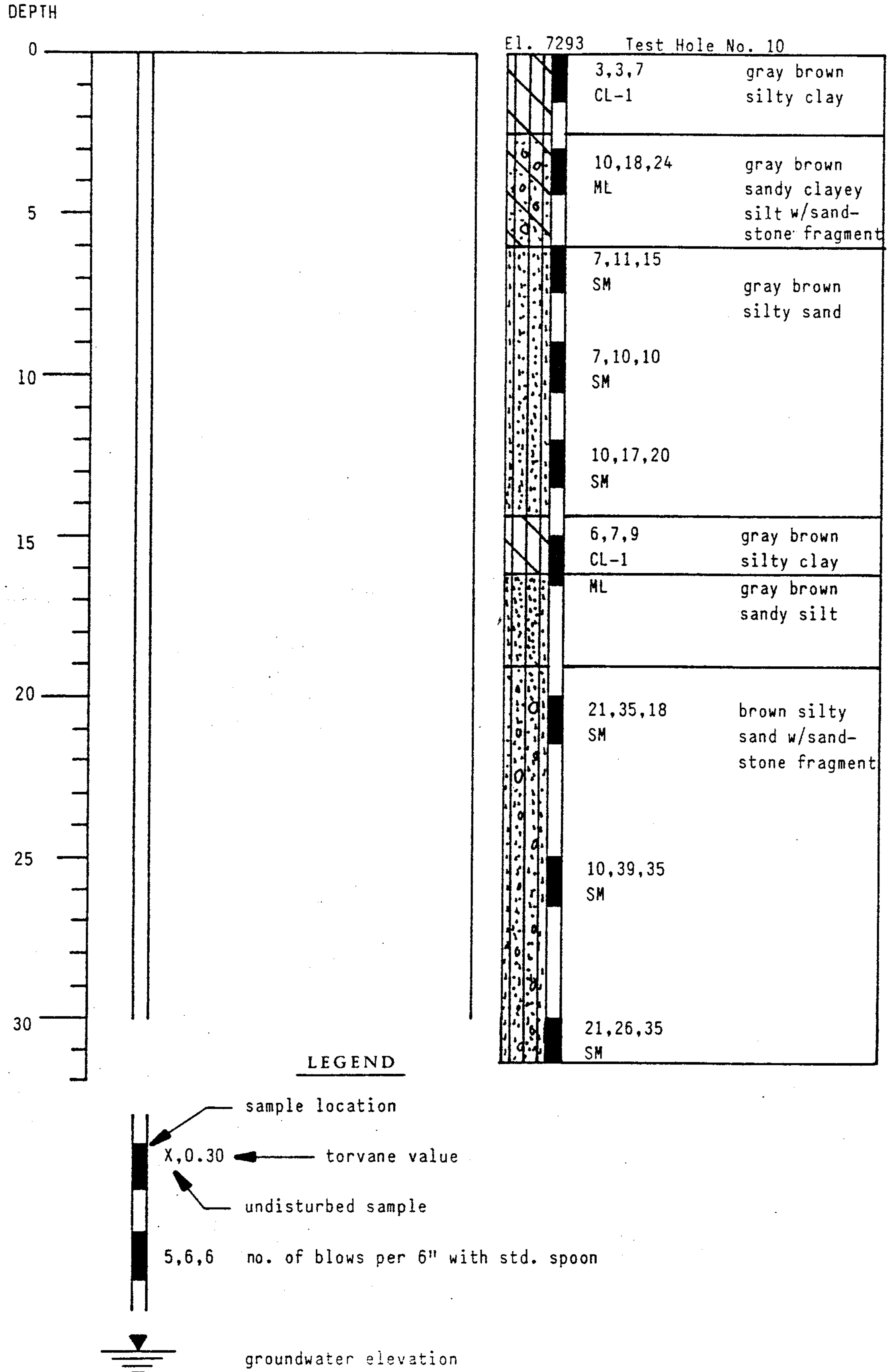
Log of Borings for:
Plateau Mine Phase II
Wattis, Utah

Figure No. 7



ROLLINS, BROWN AND GUNNELL, INC.
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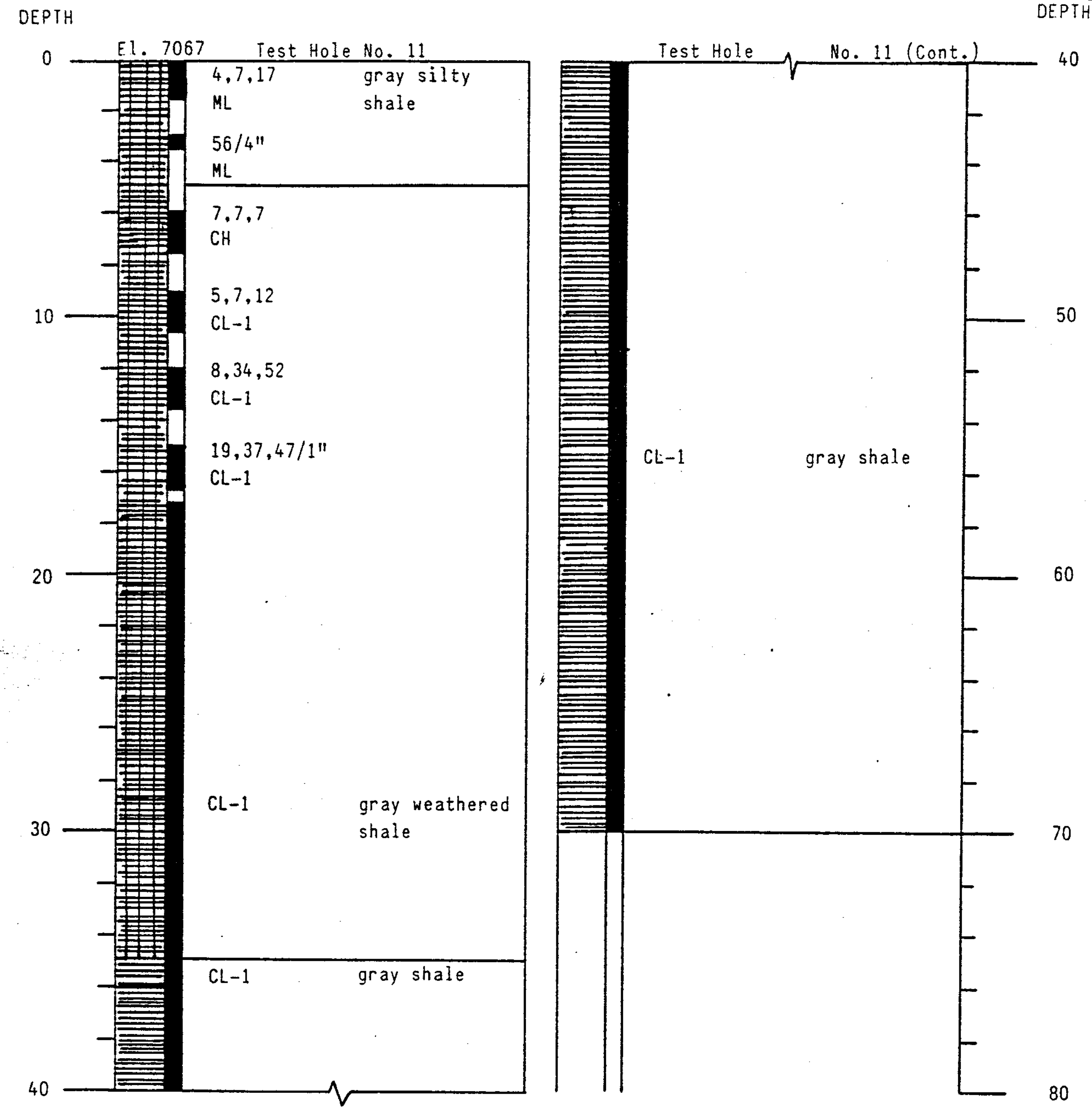
Log of Borings for:
Plateau Mine Phase II
Wattis, Utah

Figure No. 8



LEGEND

sample location

X,0.30 torvane value

undisturbed sample

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

groundwater elevation

Depth	%Core Recovery	RQD
17-20	100	17
20-22	100	96
22-27	100	100
27-32	100	28
32-37	70	13
37-42	100	100
42-47	100	100
47-52	100	100
52-57	100	100
57-62	100	100
62-67	100	97
67-70	100	100

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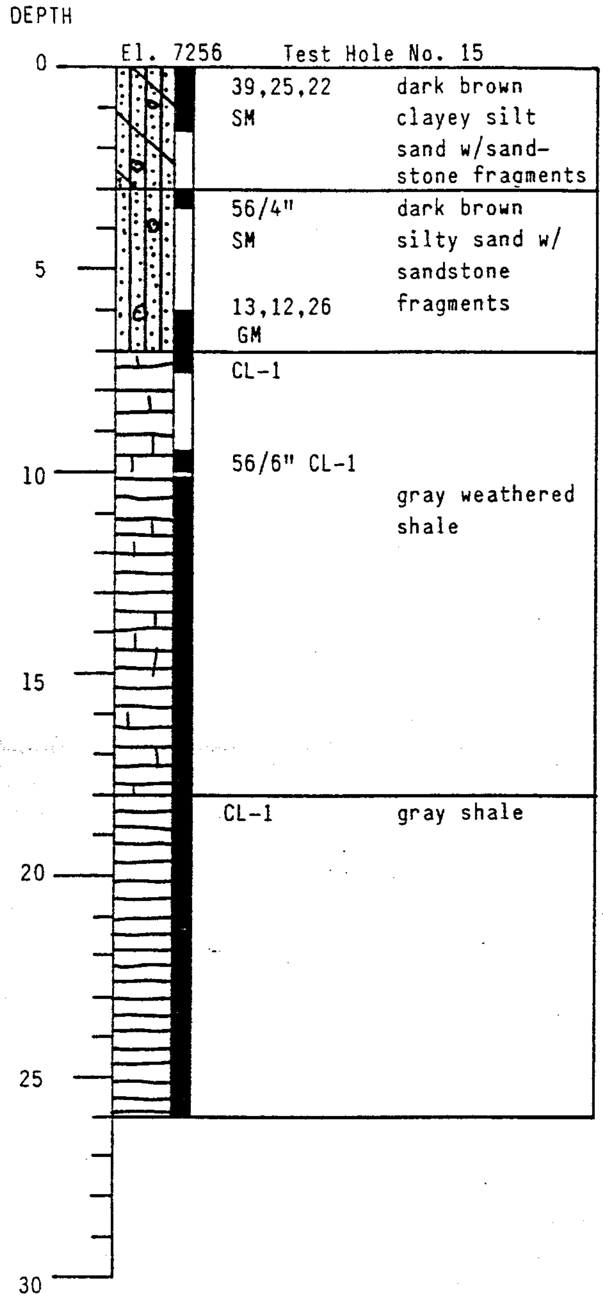
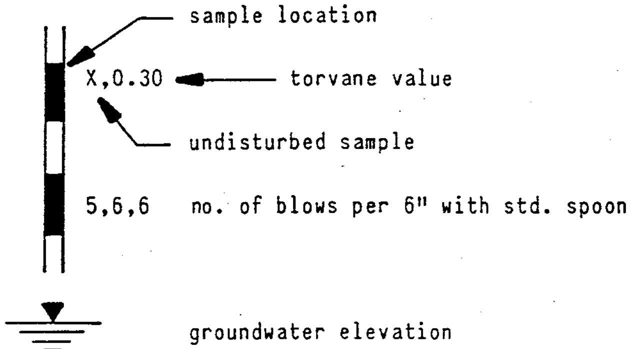
Log of Borings for:
Plateau Mine Phase II
Wattis, Utah

Figure No. 9



Depth	%Core Recovery	RQD
10-11	83	0
11-16	100	78
16-21	100	90
21-26	100	90

LEGEND



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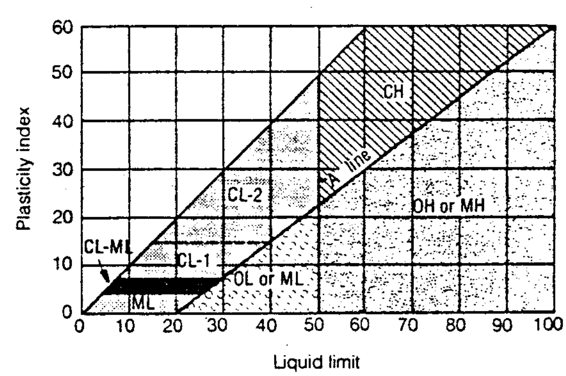
Log of Borings for:
Plateau Mine Phase II
Wattis, Utah

Figure No. 10

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	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
				u				
			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	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
				u				
			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	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
				2				
			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			

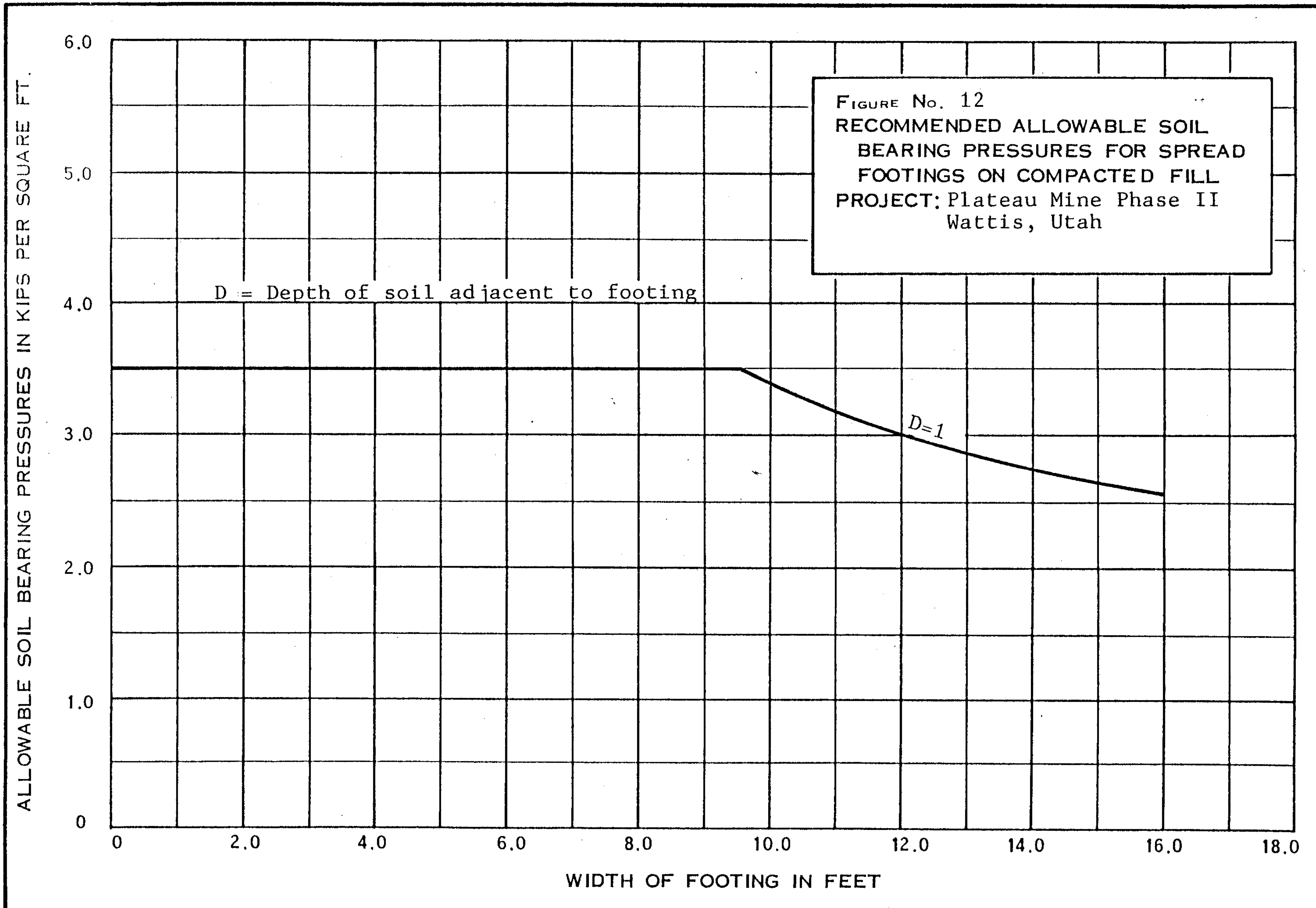


Plasticity Chart
For laboratory classification of fine-grained soils

*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. 11



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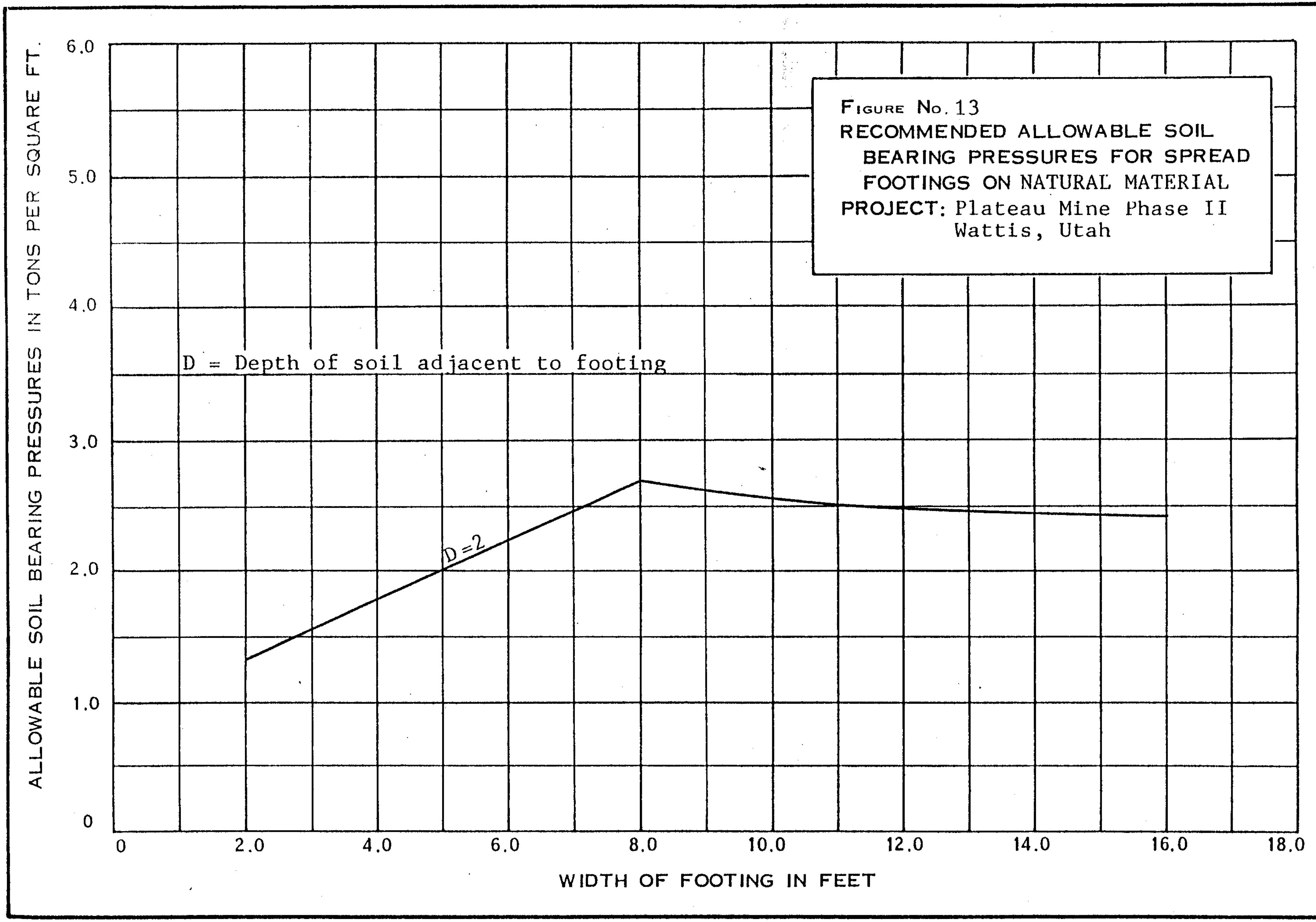

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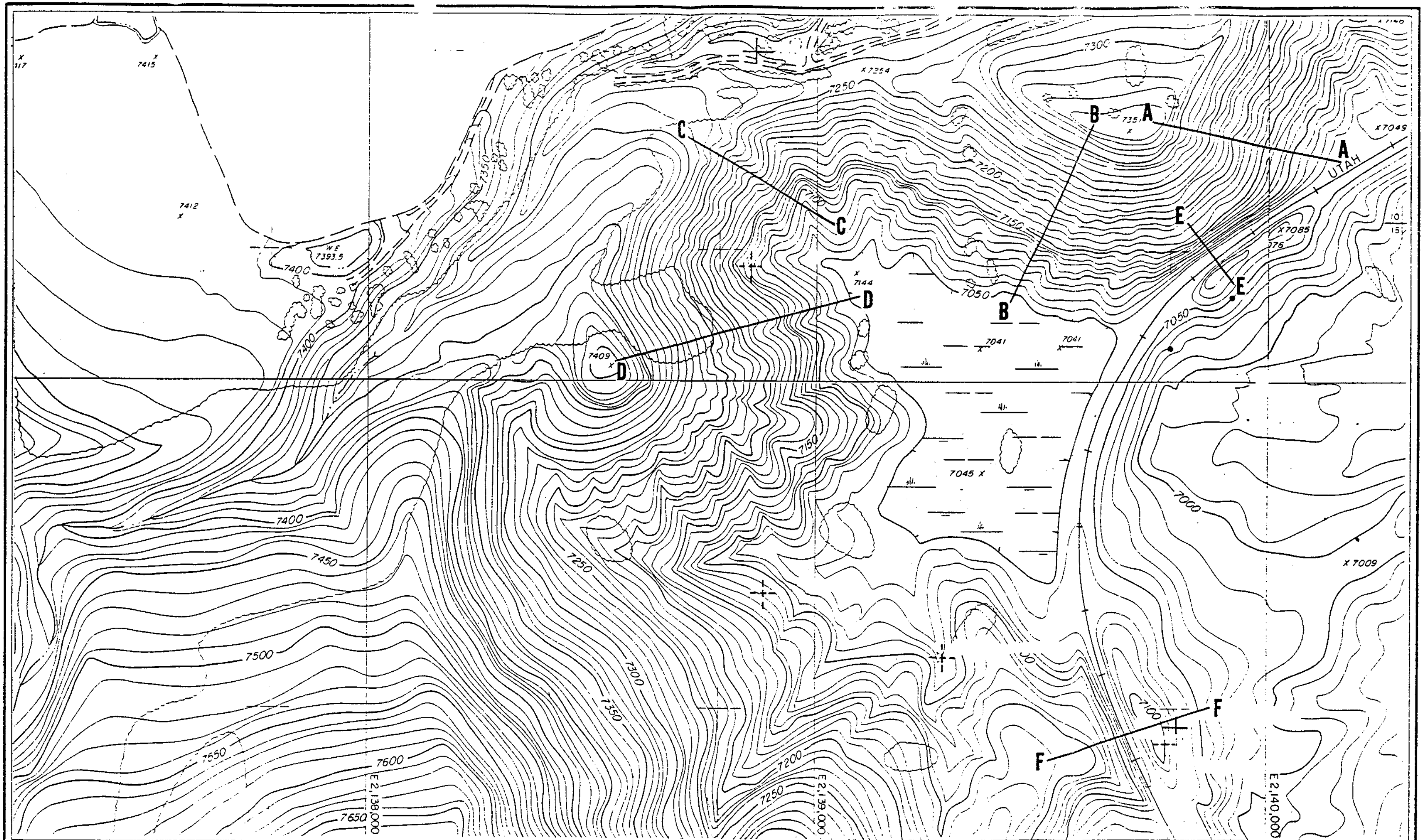

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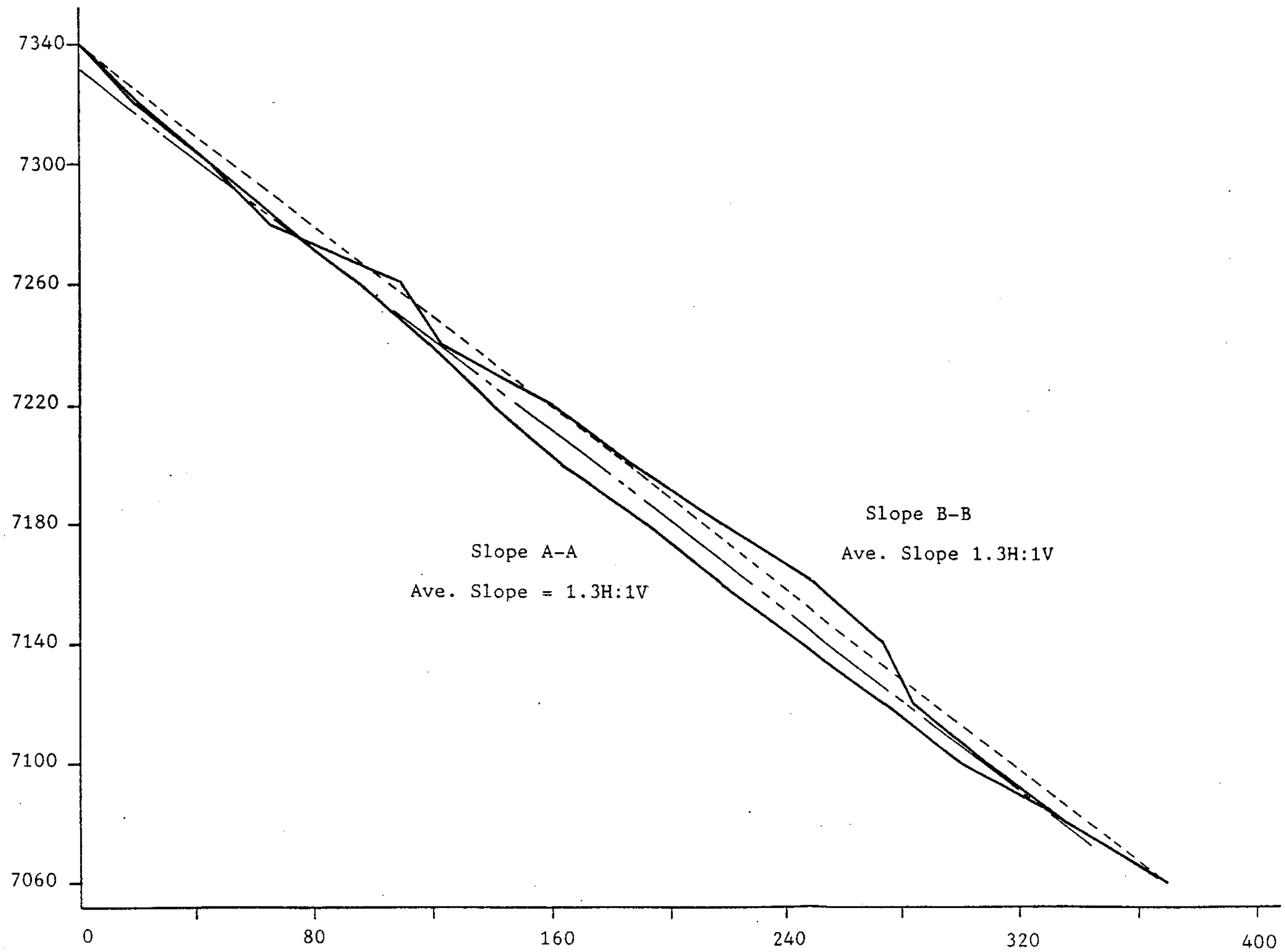
PLATEAU MINE PHASE II
Location of Slopes
Wattis, Utah

FIGURE
NO. 14



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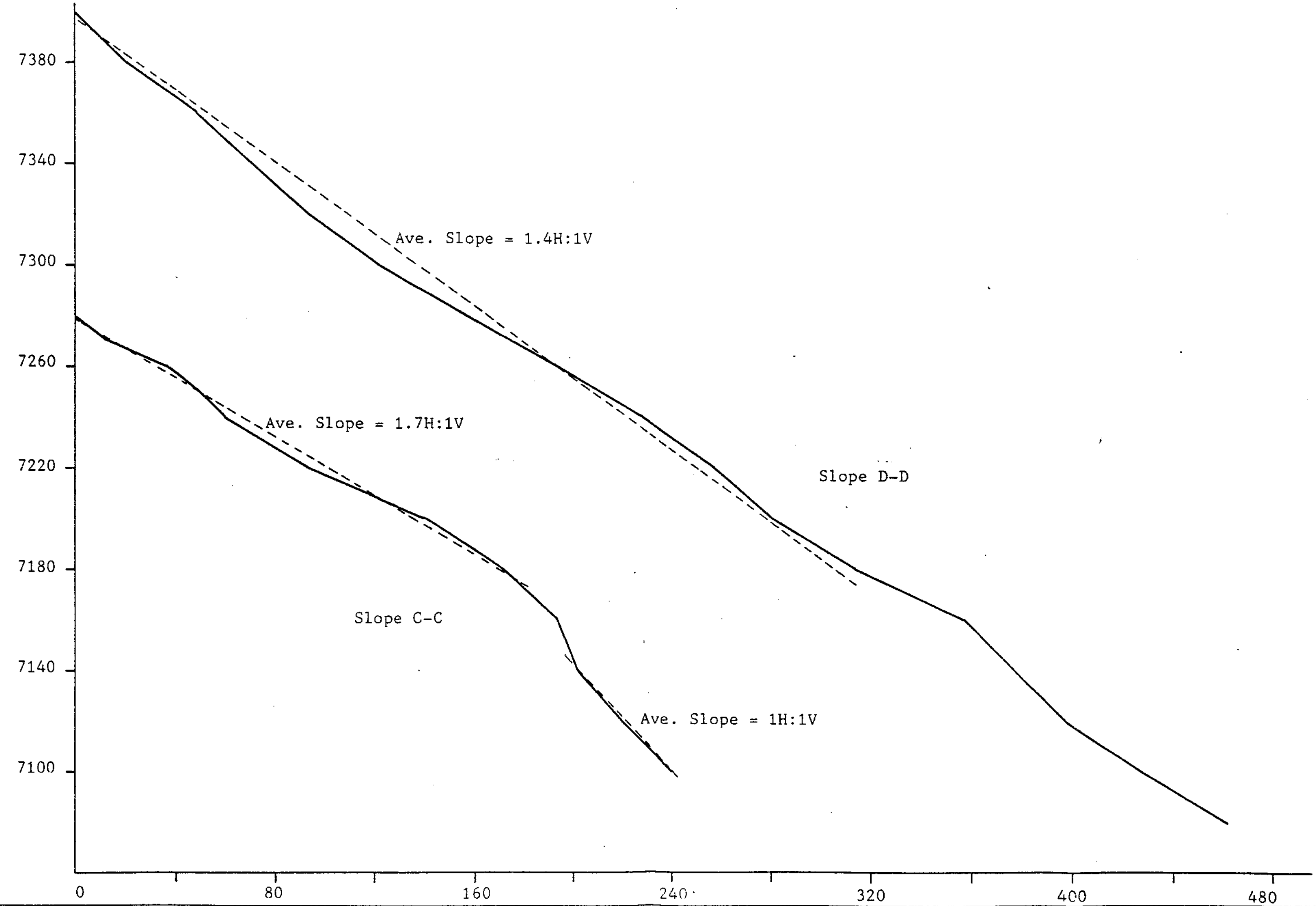
PLATEAU MINE PHASE II
Profiles of Slopes A-A and B-B
Wattis, Utah

FIGURE NO. 15



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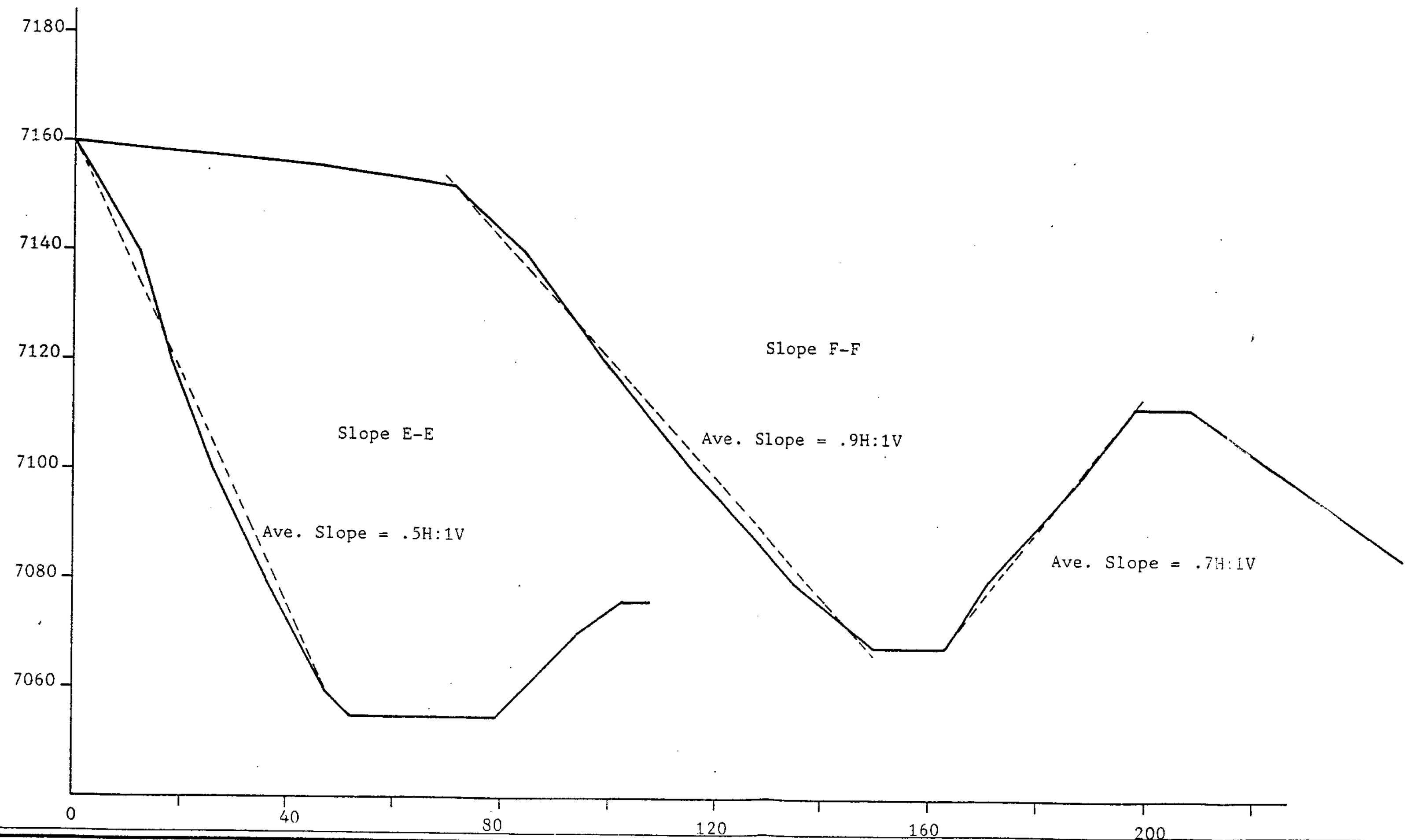
PLATEAU MINE PHASE II
Profiles of Slopes C-C and D-D
Wattis, Utah

FIGURE NO. 16



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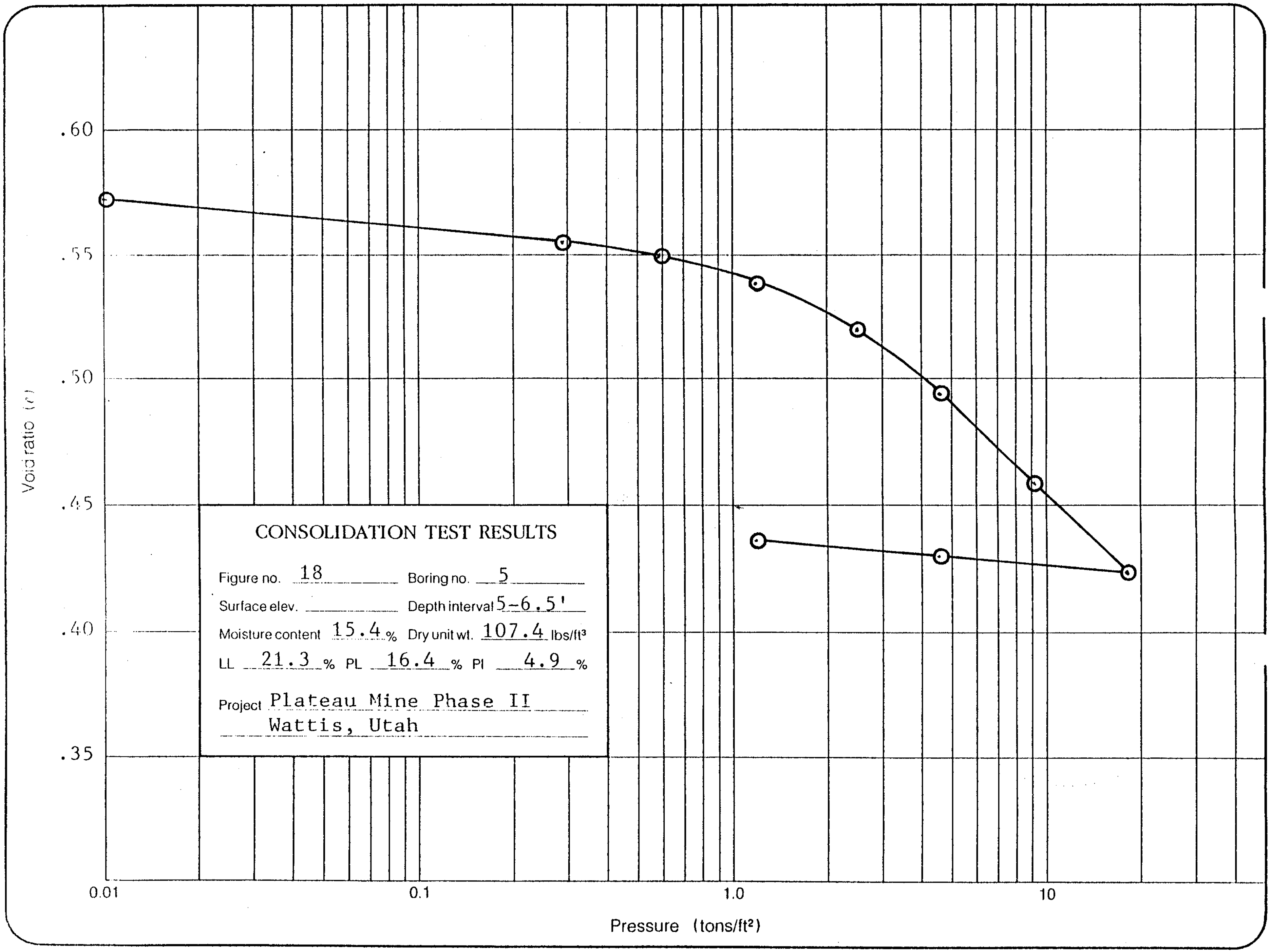
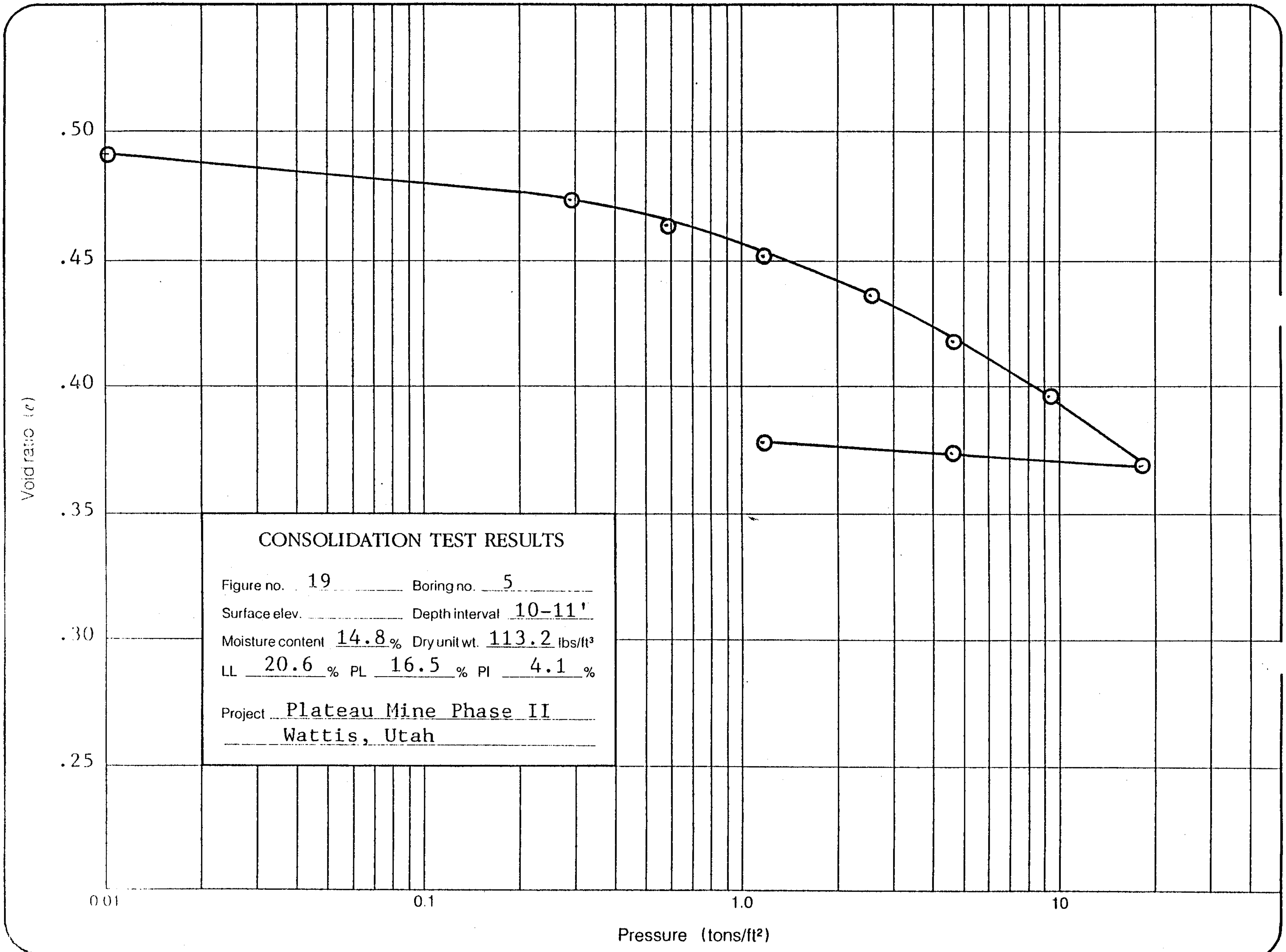
PLATEAU MINE PHASE II
Profiles of Slopes E-E and F-F
Wattis, Utah

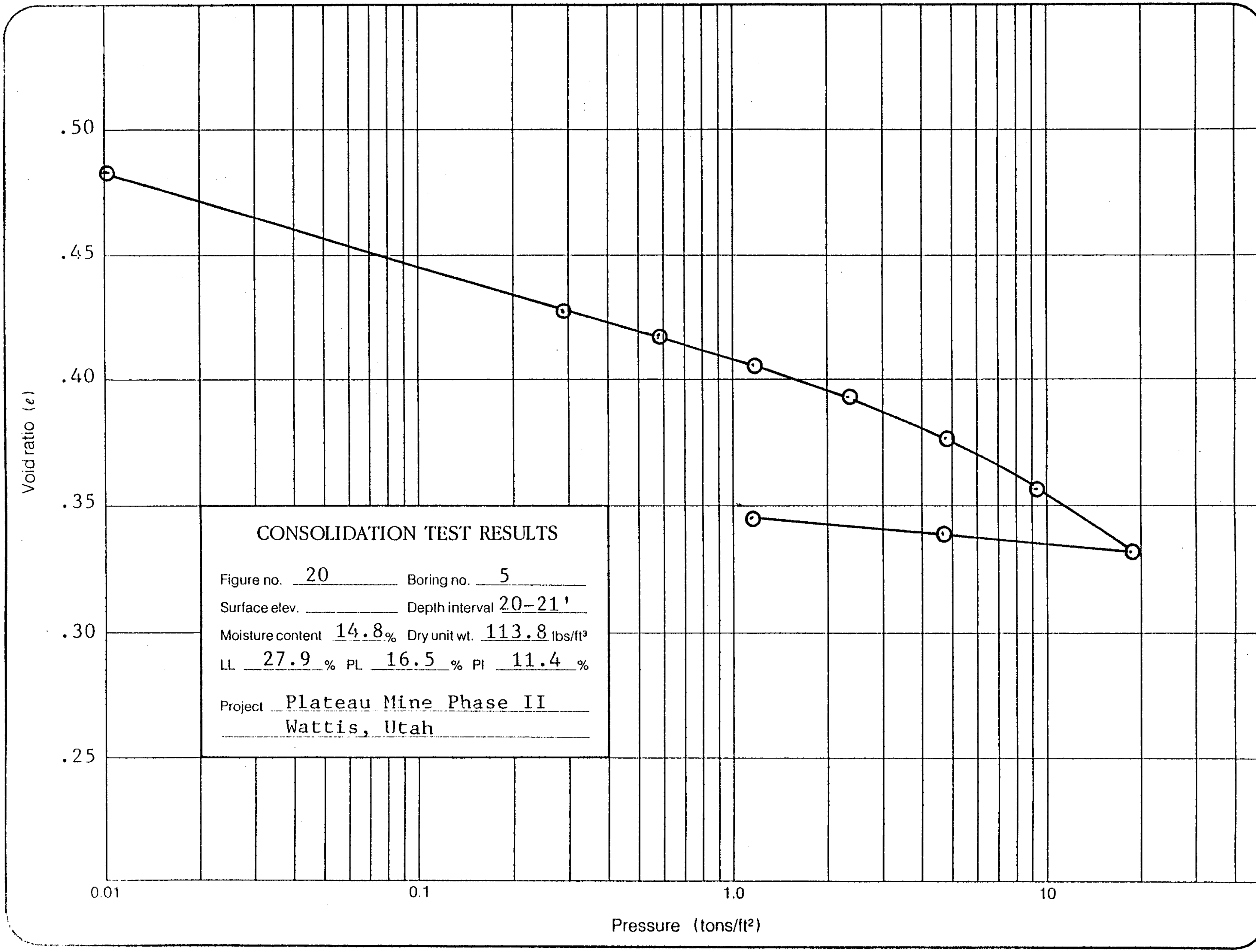
FIGURE NO. 17

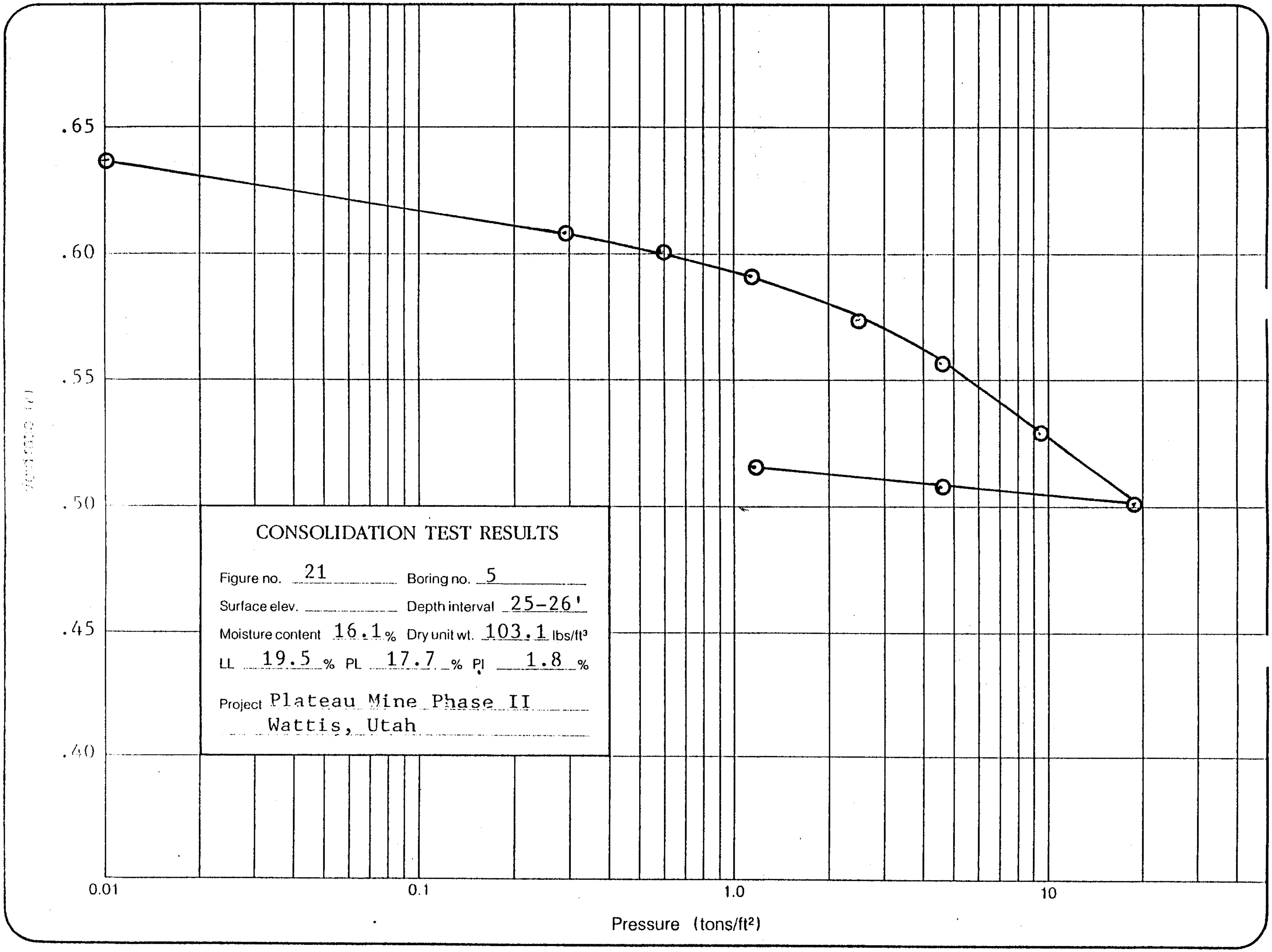


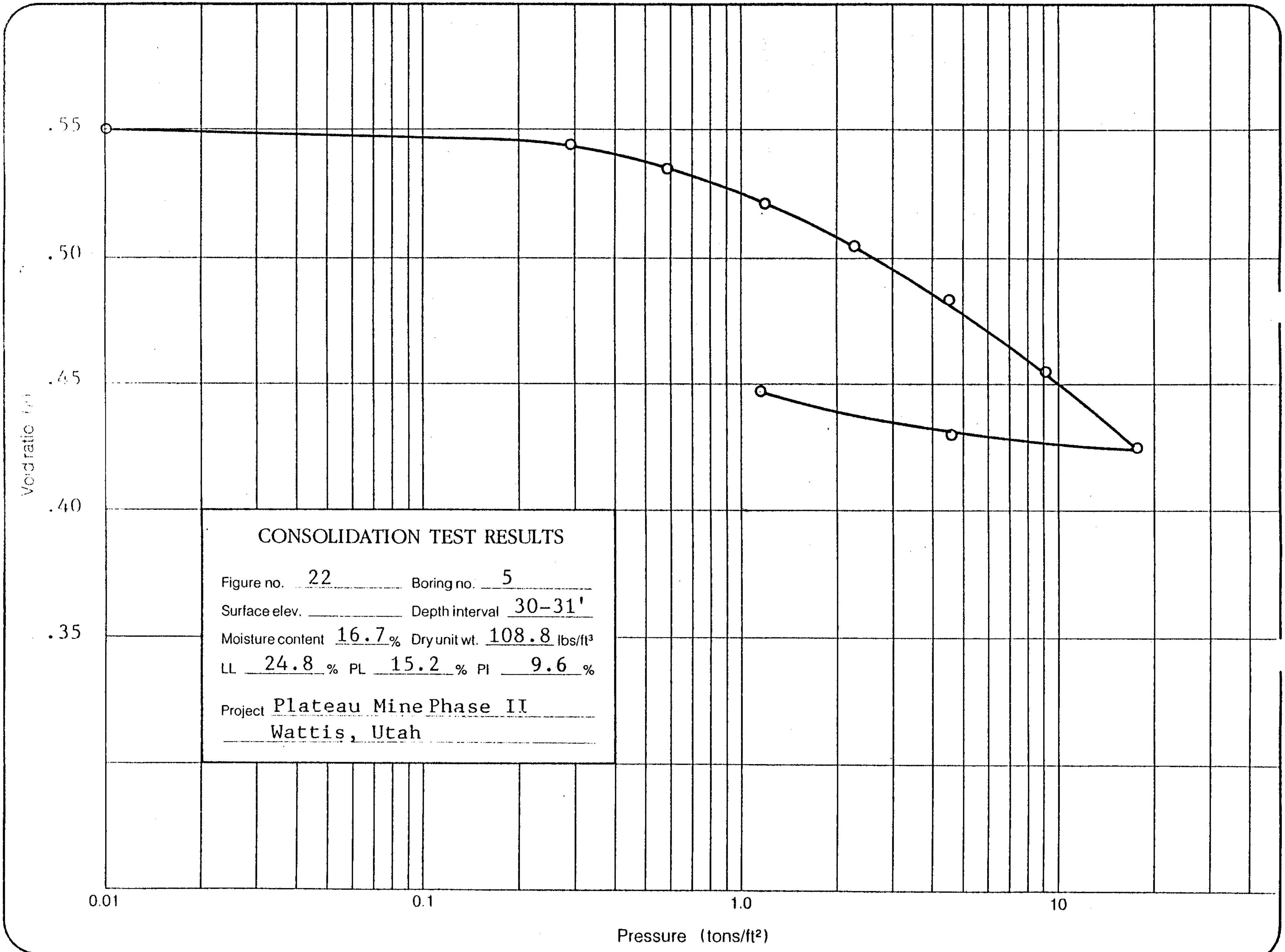
.60
.55
.50
.45
.40
.35
Void ratio (e)
0.01
0.1
1.0
10
Pressure (tons/ft²)
CONSOLIDATION TEST RESULTS
Figure no. 18
Boring no. 5
Surface elev.
Depth interval 5-6.5'
Moisture content 15.4 %
Dry unit wt. 107.4 lbs/ft³
LL 21.3 %
PL 16.4 %
PI 4.9 %
Project Plateau Mine Phase II
Wattis, Utah



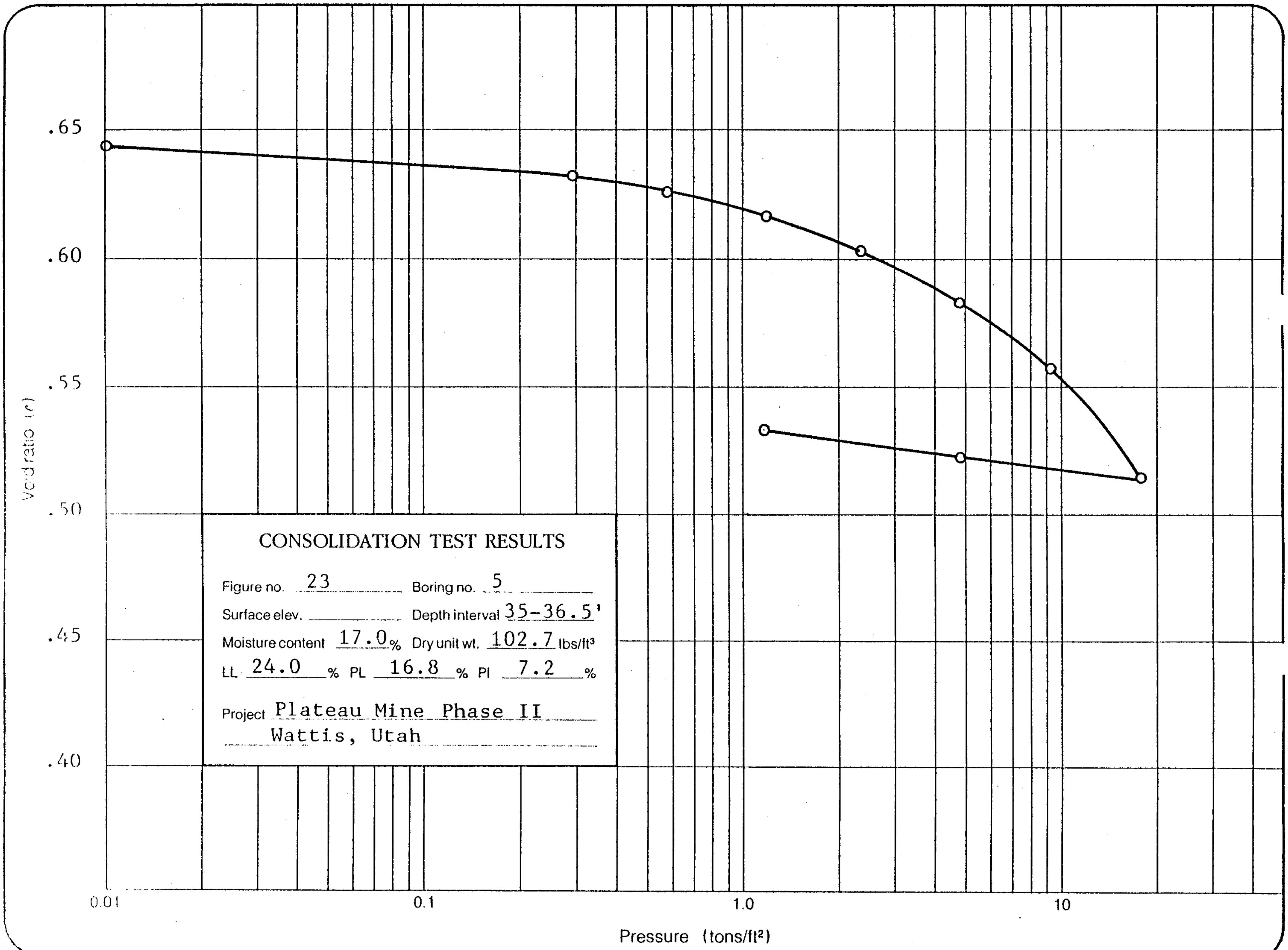
.50
.45
.40
.35
.30
.25
Void ratio (e)
0.01
0.1
1.0
10
Pressure (tons/ft²)
CONSOLIDATION TEST RESULTS
Figure no. 19 Boring no. 5
Surface elev. Depth interval 10-11'
Moisture content 14.8 % Dry unit wt. 113.2 lbs/ft³
LL 20.6 % PL 16.5 % PI 4.1 %
Project Plateau Mine Phase II
Wattis, Utah

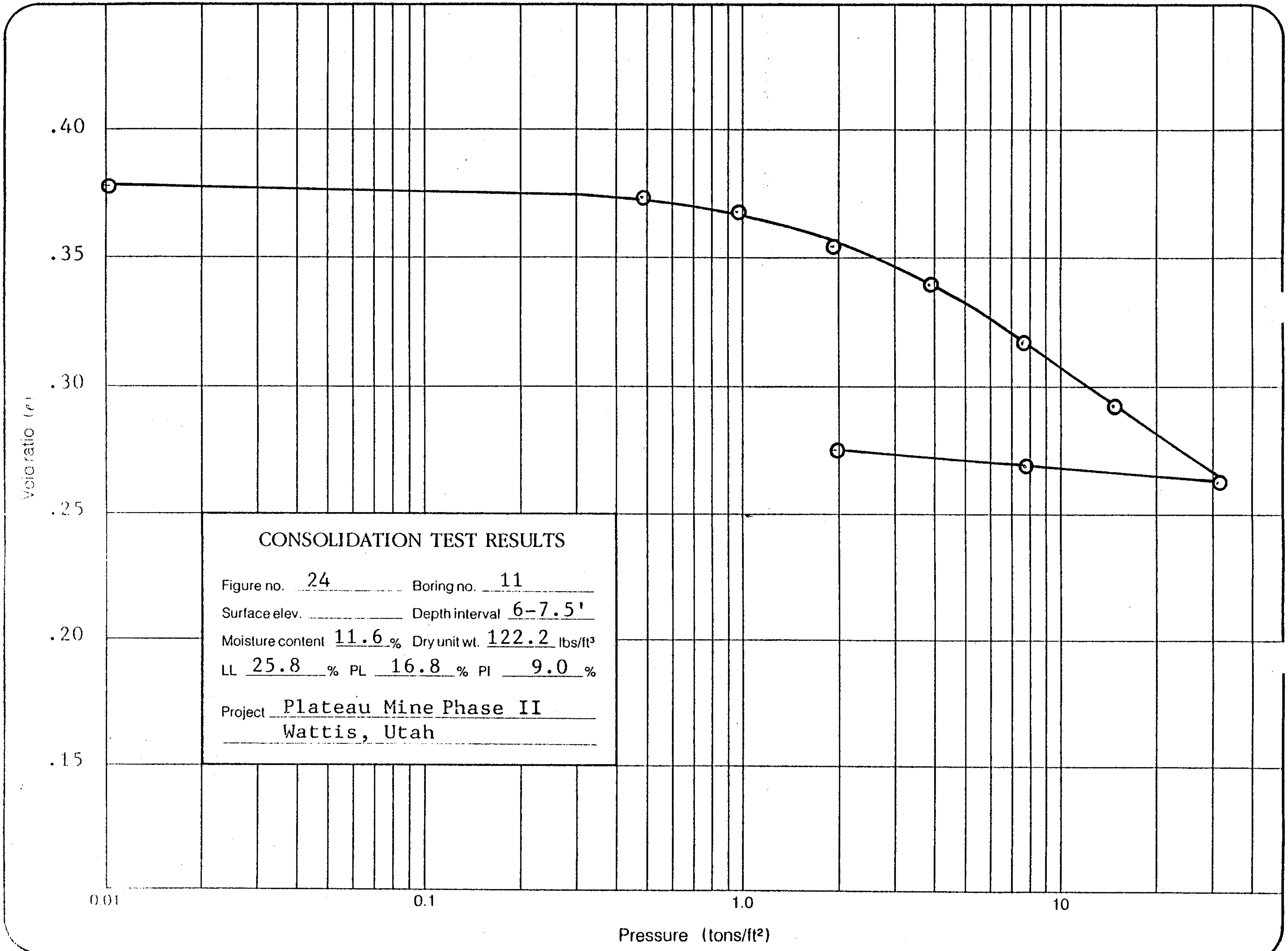

.50
.45
.40
.35
.30
.25
Void ratio (e)
0.01
0.1
1.0
10
Pressure (tons/ft²)
CONSOLIDATION TEST RESULTS
Figure no. 20
Boring no. 5
Surface elev.
Depth interval 20-21'
Moisture content 14.8%
Dry unit wt. 113.8 lbs/ft³
LL 27.9 %
PL 16.5 %
PI 11.4 %
Project Plateau Mine Phase II
Wattis, Utah

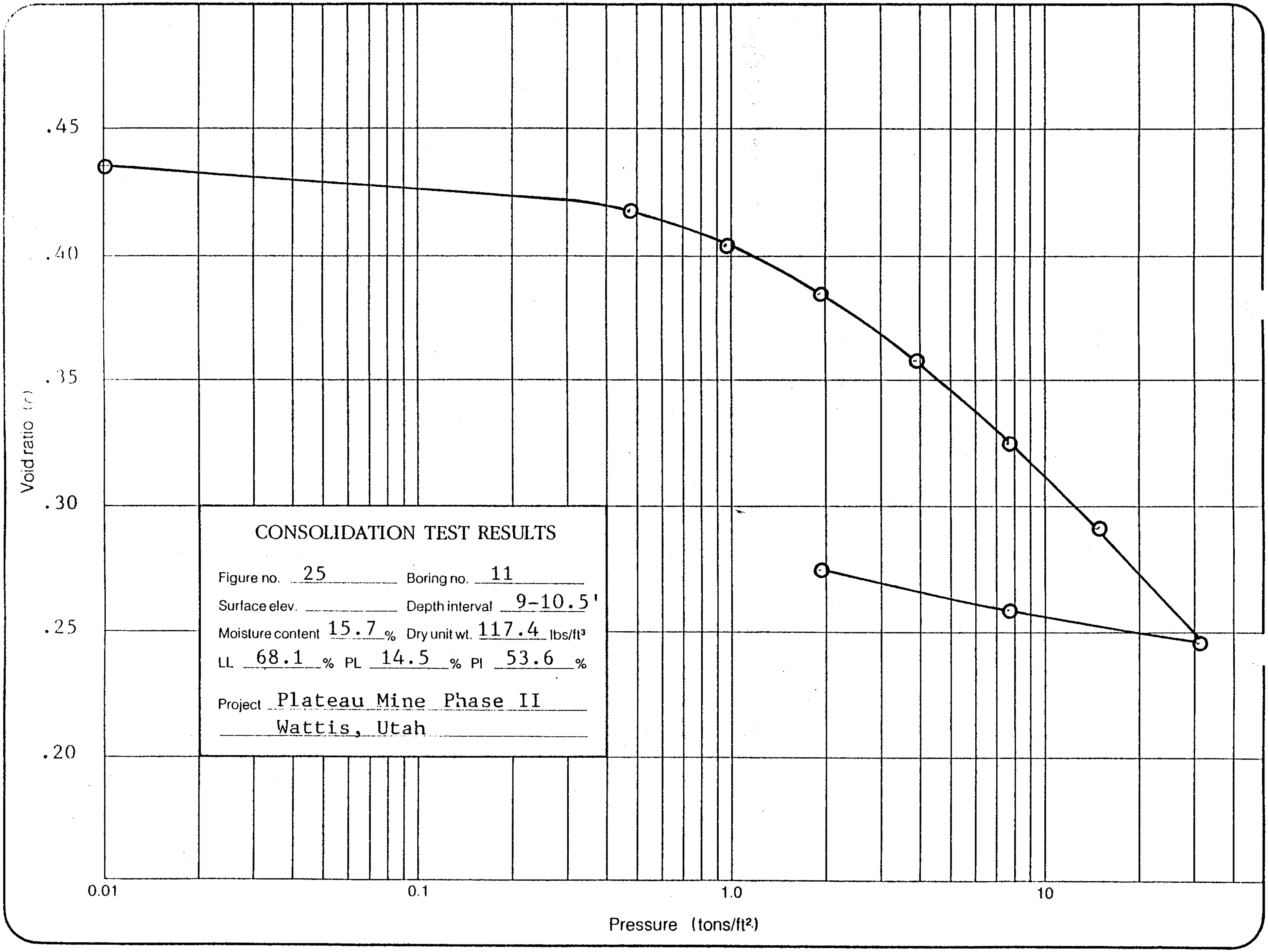

.65
.60
.55
.50
.45
.40
0.01
0.1
1.0
10
Pressure (tons/ft²)
CONSOLIDATION TEST RESULTS
Figure no. 21
Boring no. 5
Surface elev.
Depth interval 25-26'
Moisture content 16.1 %
Dry unit wt. 103.1 lbs/ft³
LL 19.5 %
PL 17.7 %
PI 1.8 %
Project Plateau Mine Phase II
Wattis, Utah



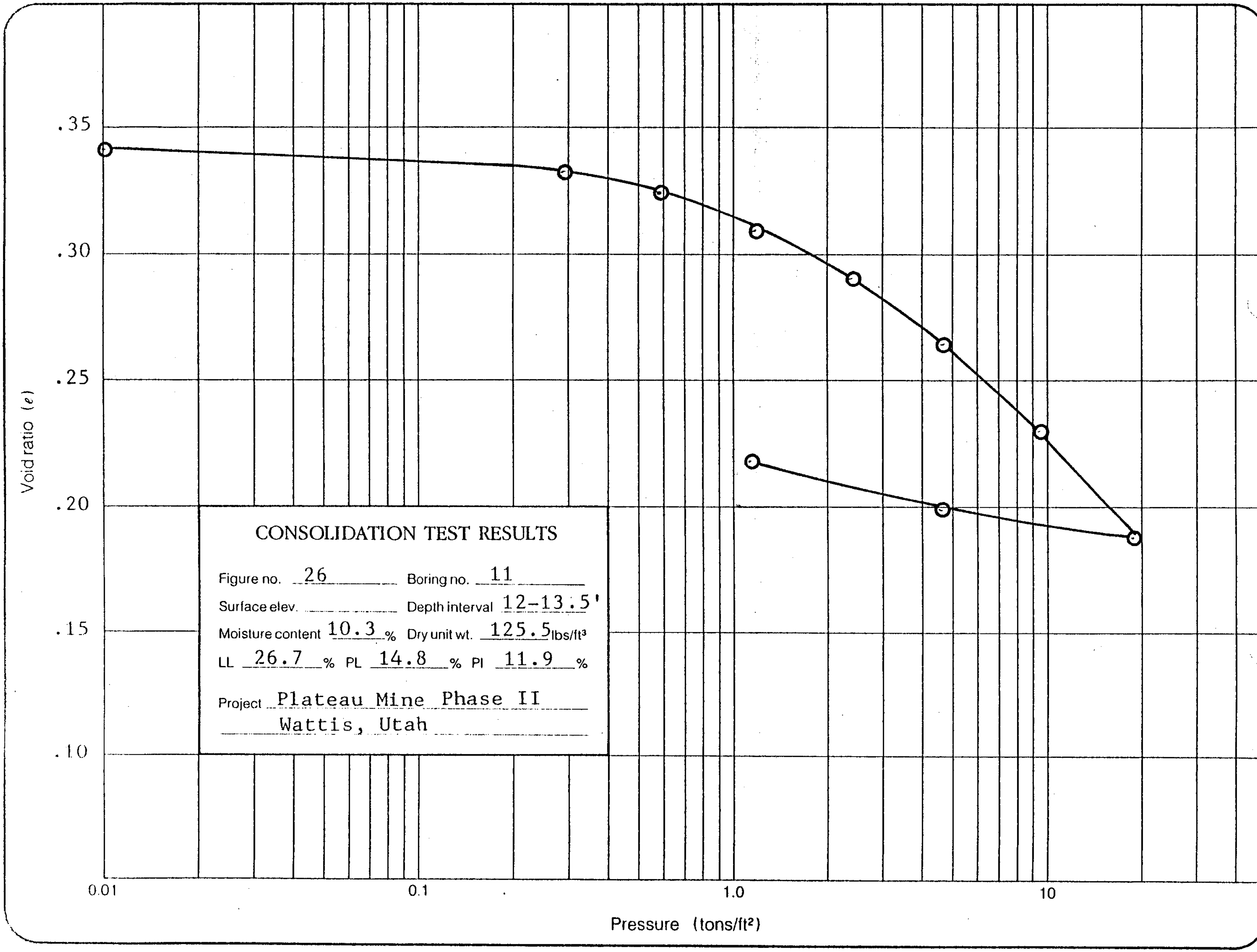
.55
.50
.45
.40
.35
Void ratio
0.01
0.1
1.0
10
Pressure (tons/ft²)
CONSOLIDATION TEST RESULTS
Figure no. 22 Boring no. 5
Surface elev. Depth interval 30-31'
Moisture content 16.7 % Dry unit wt. 108.8 lbs/ft³
LL 24.8 % PL 15.2 % PI 9.6 %
Project Plateau Mine Phase II
Wattis, Utah

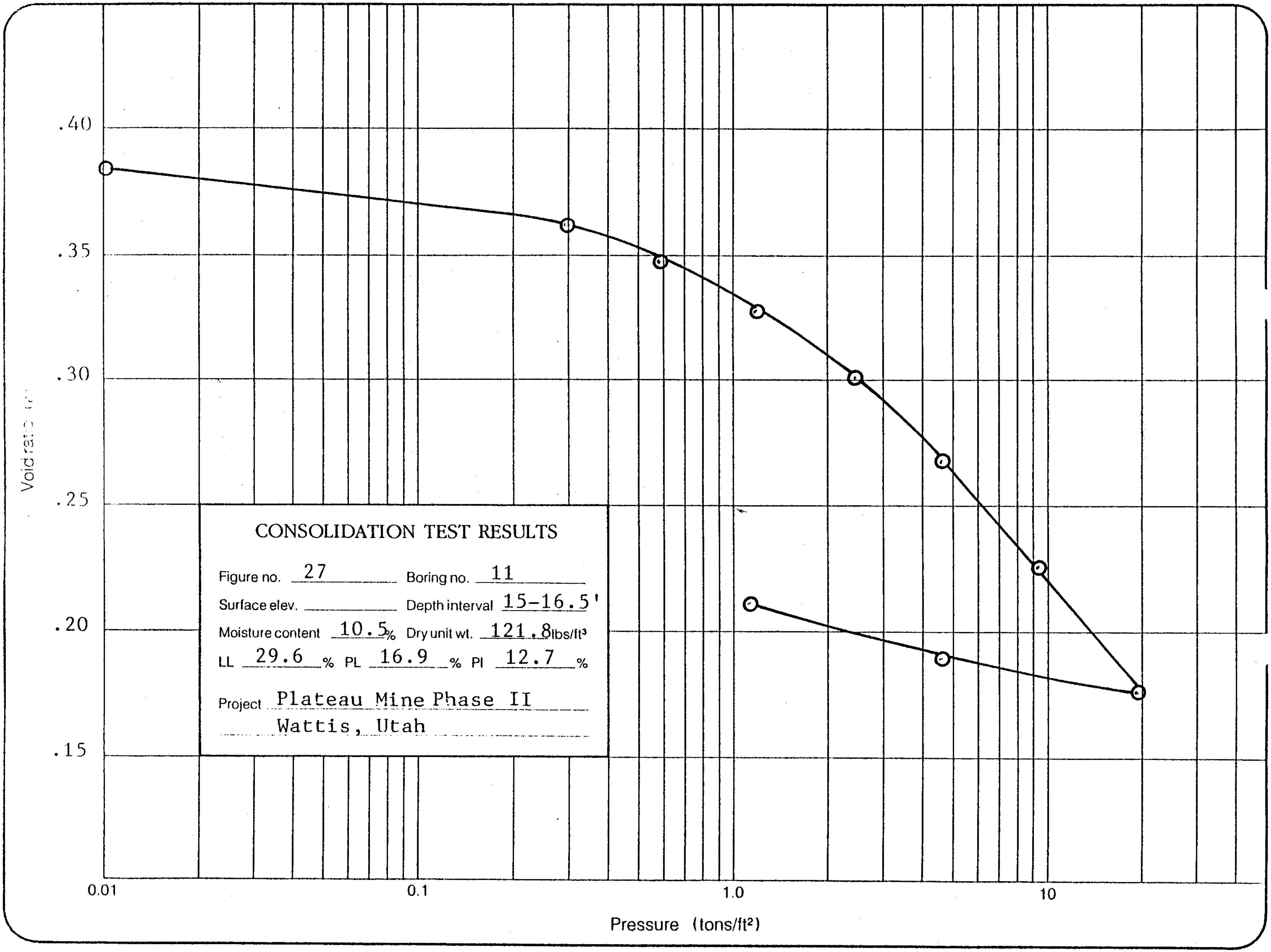

CONSOLIDATION TEST RESULTS
Figure no. 23 Boring no. 5
Surface elev. Depth interval 35-36.5'
Moisture content 17.0% Dry unit wt. 102.7 lbs/ft³
LL 24.0 % PL 16.8 % PI 7.2 %
Project Plateau Mine Phase II
Wattis, Utah
.65
.60
.55
.50
.45
.40
Void ratio (e)
0.01
0.1
1.0
10
Pressure (tons/ft²)


.40
.35
.30
.25
.20
.15
Void ratio (e)
0.01
0.1
1.0
10
Pressure (tons/ft²)
CONSOLIDATION TEST RESULTS
Figure no. 24
Boring no. 11
Surface elev.
Depth interval 6-7.5'
Moisture content 11.6 %
Dry unit wt. 122.2 lbs/ft³
LL 25.8 %
PL 16.8 %
PI 9.0 %
Project Plateau Mine Phase II
Wattis, Utah

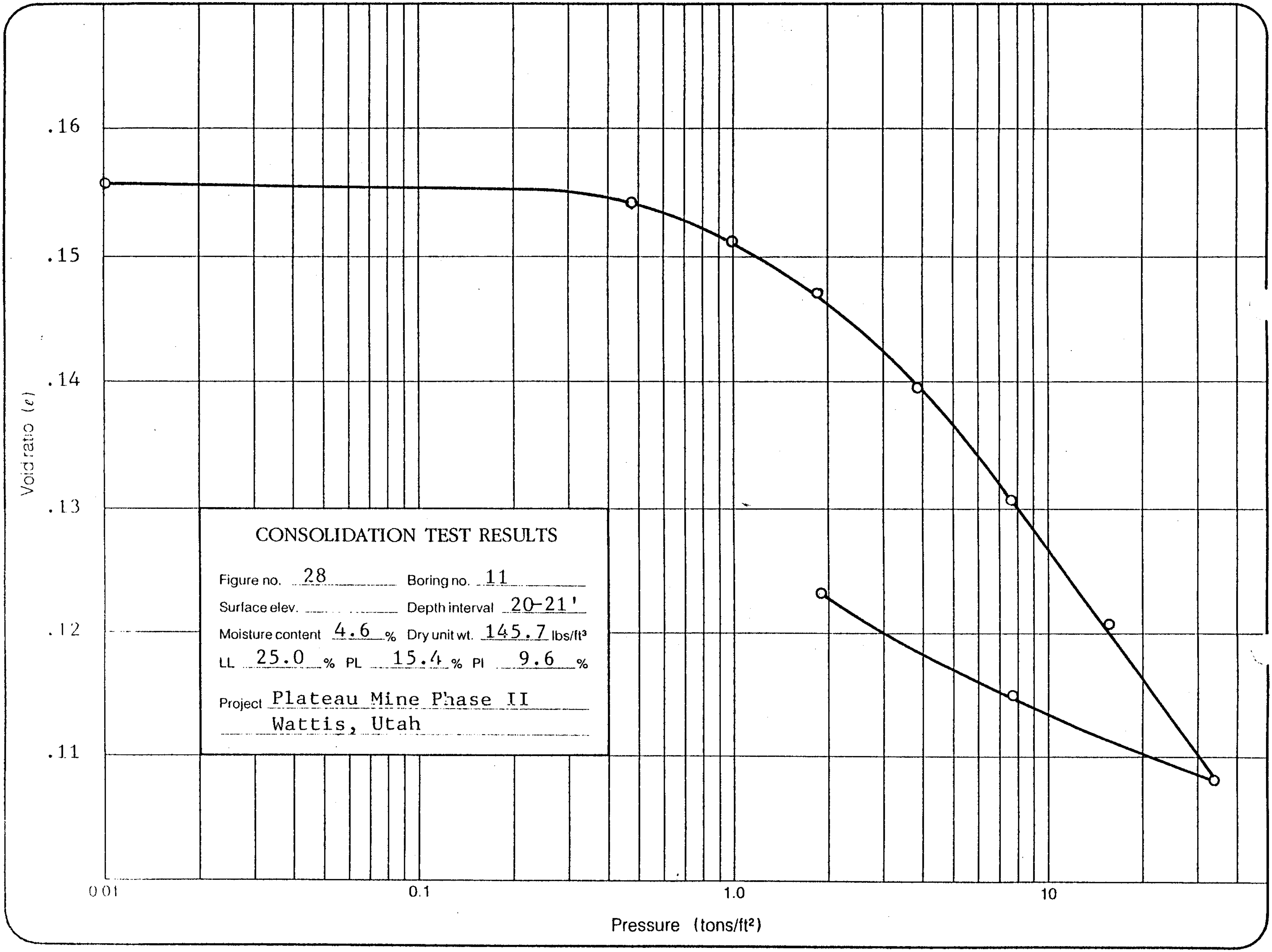


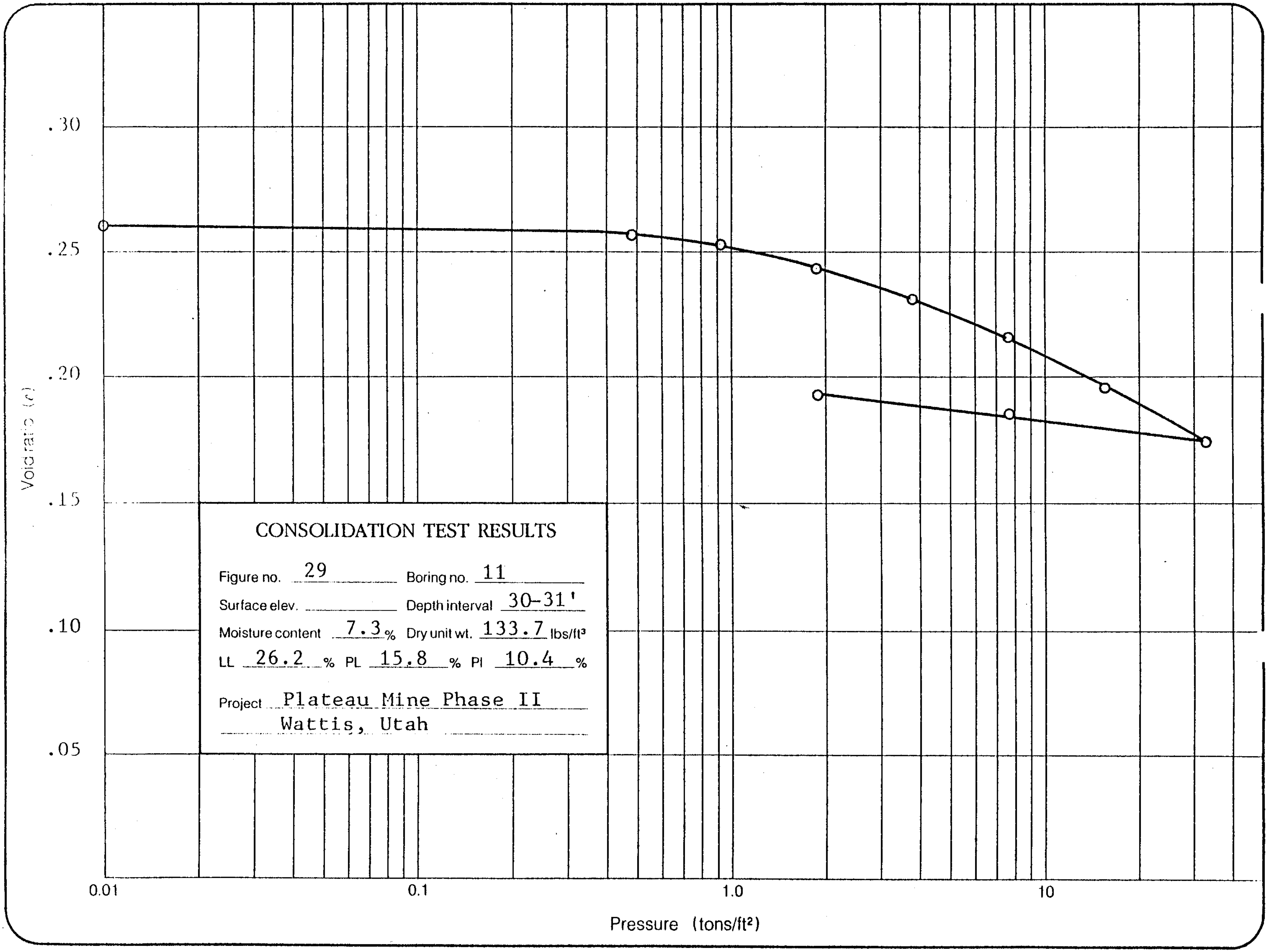
CONSOLIDATION TEST RESULTS
Figure no. 25
Boring no. 11
Surface elev.
Depth interval 9-10.5'
Moisture content 15.7 %
Dry unit wt. 117.4 lbs/ft³
LL 68.1 % PL 14.5 % PI 53.6 %
Project Plateau Mine Phase II
Wattis, Utah
.45
.40
.35
.30
.25
.20
Void ratio
0.01
0.1
1.0
10
Pressure (tons/ft².)

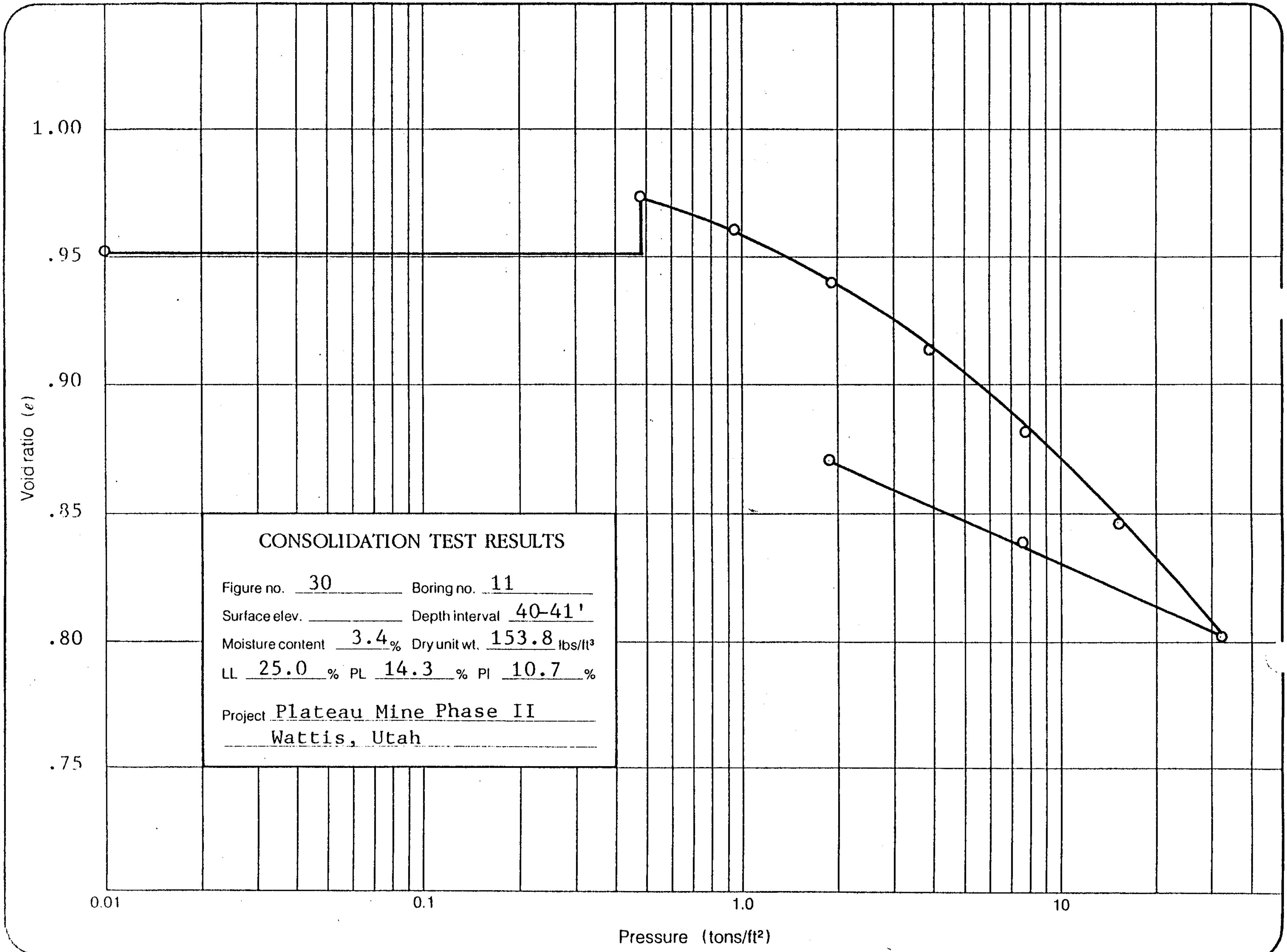

CONSOLIDATION TEST RESULTS
Figure no. 26 Boring no. 11
Surface elev.
Depth interval 12-13.5'
Moisture content 10.3 %
Dry unit wt. 125.5 lbs/ft³
LL 26.7 % PL 14.8 % PI 11.9 %
Project Plateau Mine Phase II
Wattis, Utah
Void ratio (e)
.35
.30
.25
.20
.15
.10
0.01
0.1
1.0
10
Pressure (tons/ft²)

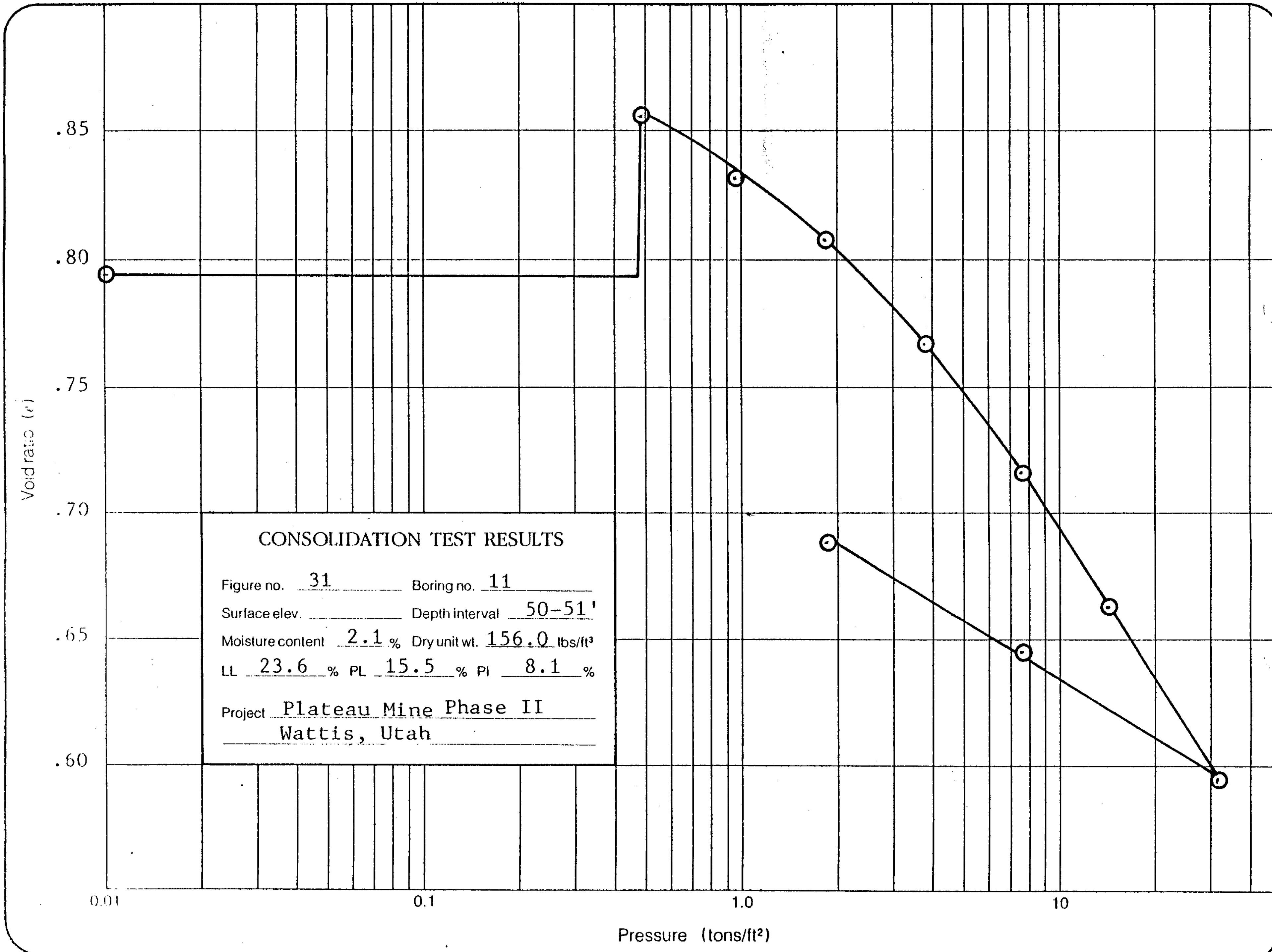


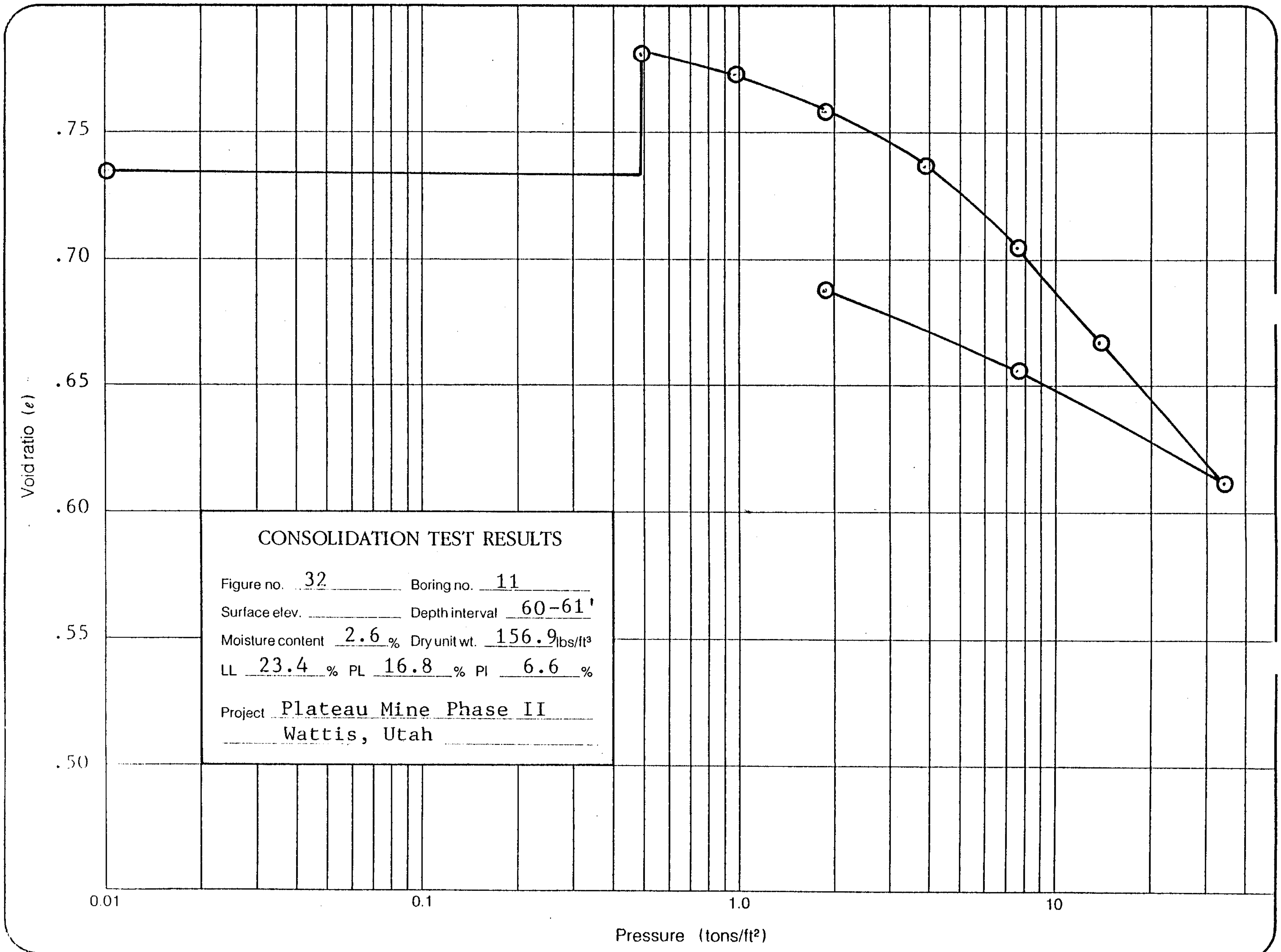
.40
.35
.30
.25
.20
.15
Void ratio (e)
0.01
0.1
1.0
10
Pressure (tons/ft²)
CONSOLIDATION TEST RESULTS
Figure no. 27
Boring no. 11
Surface elev.
Depth interval 15-16.5'
Moisture content 10.5%
Dry unit wt. 121.8 lbs/ft³
LL 29.6 % PL 16.9 % PI 12.7 %
Project Plateau Mine Phase II
Wattis, Utah


CONSOLIDATION TEST RESULTS
Figure no. 28
Boring no. 11
Surface elev.
Depth interval 20-21'
Moisture content 4.6 %
Dry unit wt. 145.7 lbs/ft³
LL 25.0 %
PL 15.4 %
PI 9.6 %
Project Plateau Mine Phase II
Wattis, Utah
Void ratio (e)
.16
.15
.14
.13
.12
.11
0.01
0.1
1.0
10
Pressure (tons/ft²)

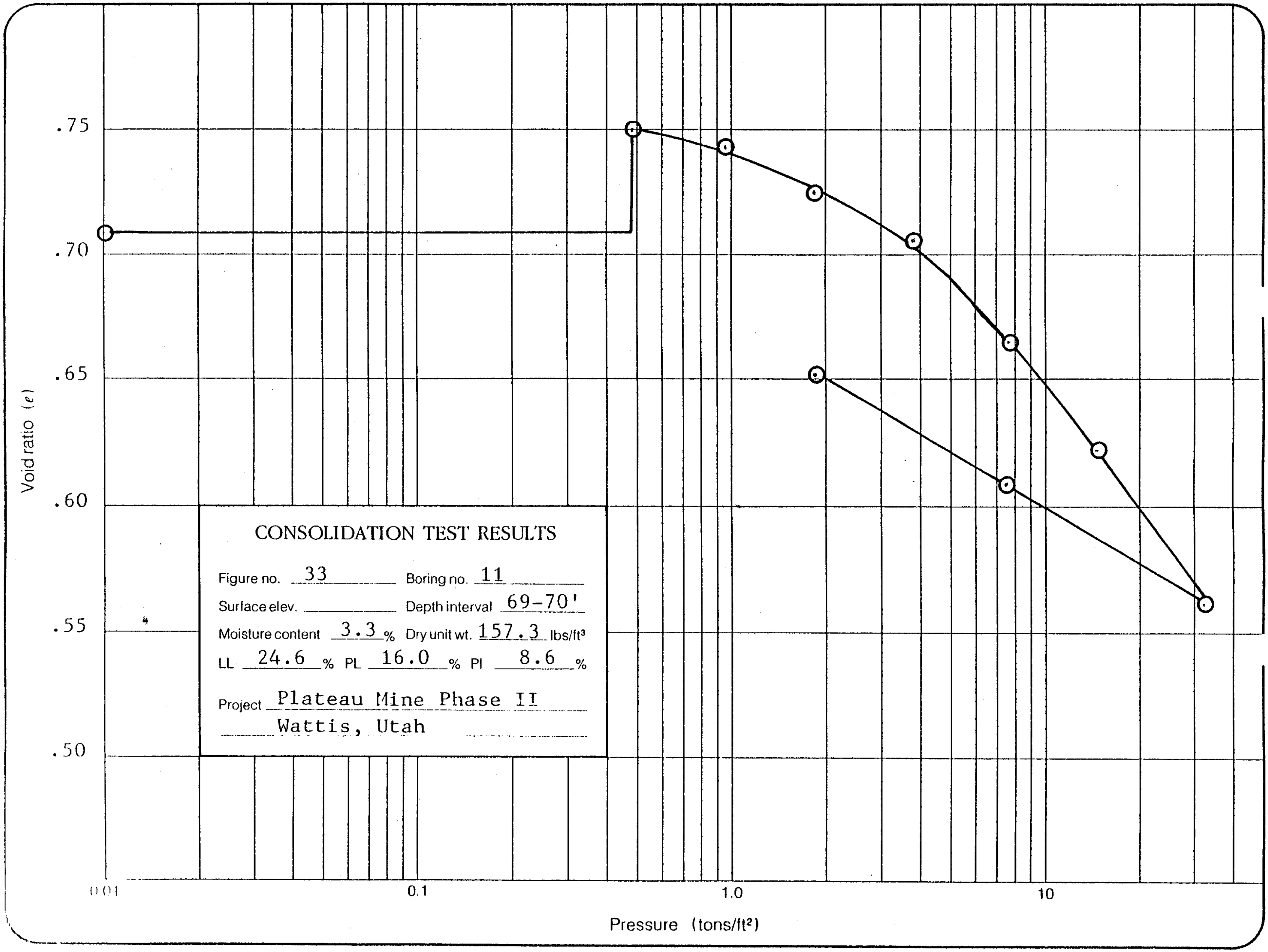

CONSOLIDATION TEST RESULTS
Figure no. 29 Boring no. 11
Surface elev.
Depth interval 30-31'
Moisture content 7.3 %
Dry unit wt. 133.7 lbs/ft³
LL 26.2 % PL 15.8 % PI 10.4 %
Project Plateau Mine Phase II
Wattis, Utah
.30
.25
.20
.15
.10
.05
Void ratio (e)
0.01
0.1
1.0
10
Pressure (tons/ft²)

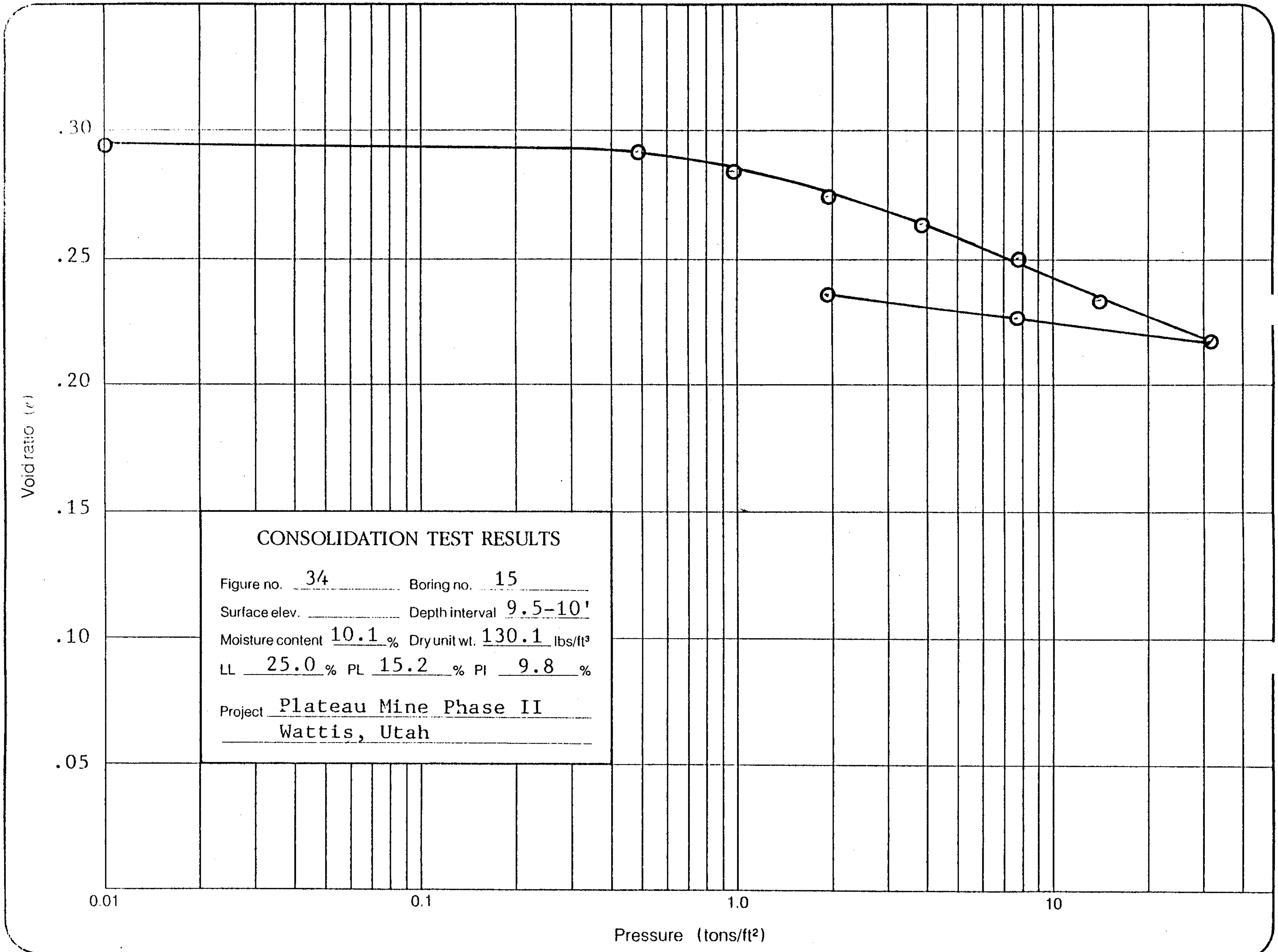

1.00
.95
.90
.85
.80
.75
Void ratio (e)
0.01
0.1
1.0
10
Pressure (tons/ft²)
CONSOLIDATION TEST RESULTS
Figure no. 30
Boring no. 11
Surface elev.
Depth interval 40-41'
Moisture content 3.4 %
Dry unit wt. 153.8 lbs/ft³
LL 25.0 %
PL 14.3 %
PI 10.7 %
Project Plateau Mine Phase II
Wattis, Utah

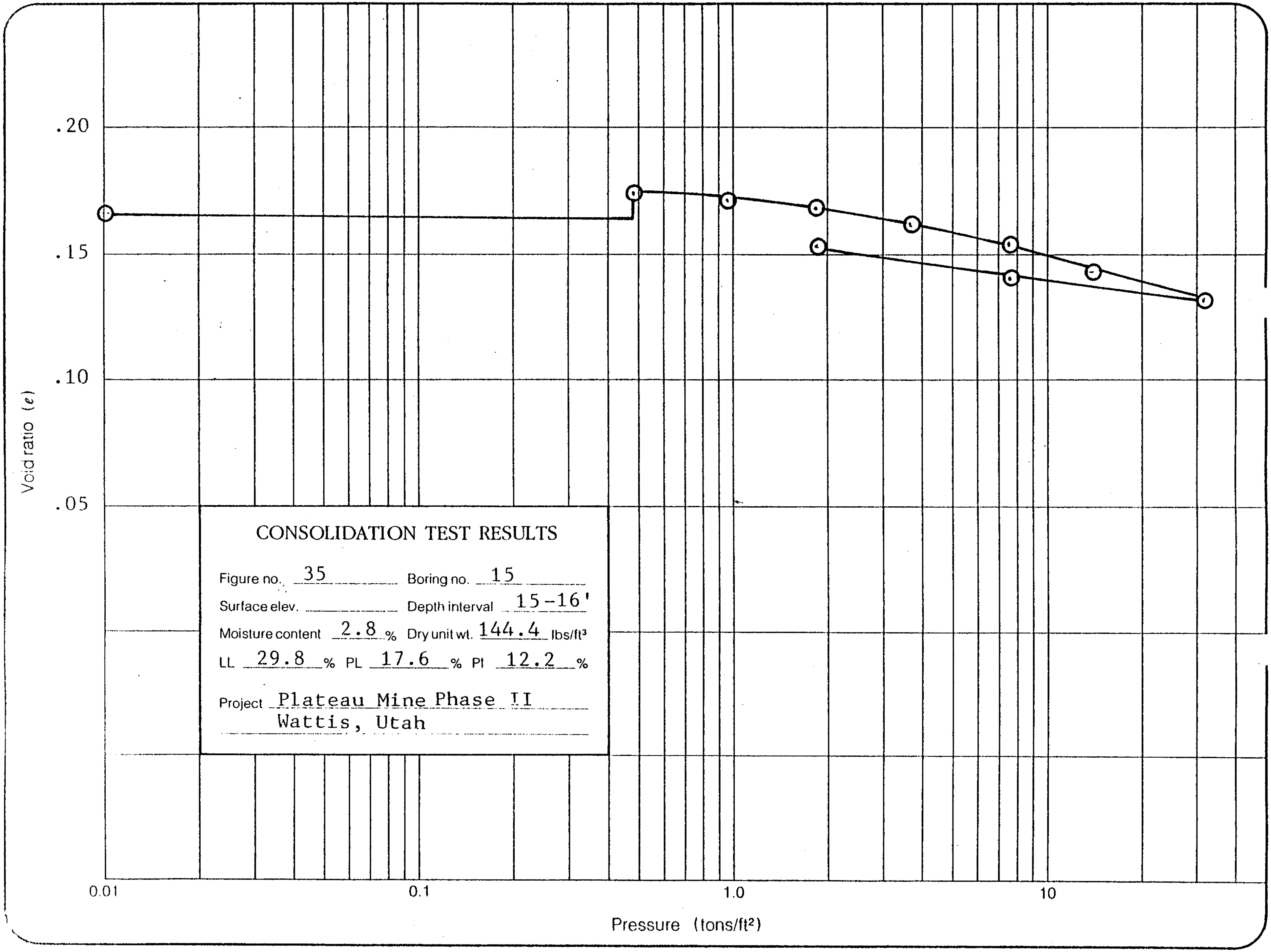

CONSOLIDATION TEST RESULTS
Figure no. 31
Boring no. 11
Surface elev.
Depth interval 50-51'
Moisture content 2.1 %
Dry unit wt. 156.0 lbs/ft³
LL 23.6 %
PL 15.5 %
PI 8.1 %
Project Plateau Mine Phase II
Wattis, Utah
Void ratio (e)
.85
.80
.75
.70
.65
.60
0.01
0.1
1.0
10
Pressure (tons/ft²)



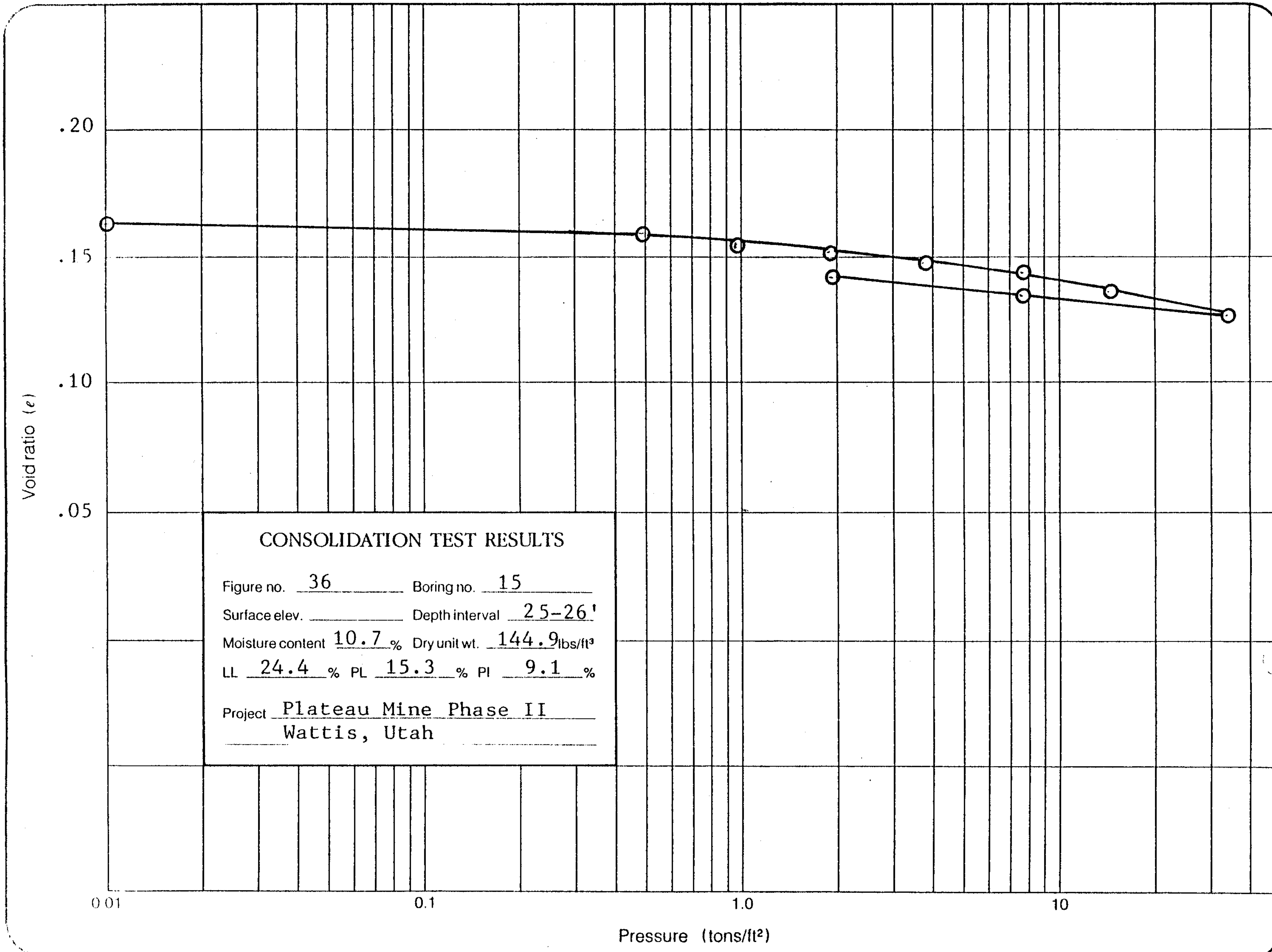
.75
.70
.65
.60
.55
.50
Void ratio (e)
0.01
0.1
1.0
10
Pressure (tons/ft²)
CONSOLIDATION TEST RESULTS
Figure no. 32
Boring no. 11
Surface elev.
Depth interval 60-61'
Moisture content 2.6 %
Dry unit wt. 156.9 lbs/ft³
LL 23.4 %
PL 16.8 %
PI 6.6 %
Project Plateau Mine Phase II
Wattis, Utah


.75
.70
.65
.60
.55
.50
Void ratio (e)
0.01
0.1
1.0
10
Pressure (tons/ft²)
CONSOLIDATION TEST RESULTS
Figure no. 33 Boring no. 11
Surface elev. Depth interval 69-70'
Moisture content 3.3 % Dry unit wt. 157.3 lbs/ft³
LL 24.6 % PL 16.0 % PI 8.6 %
Project Plateau Mine Phase II
Wattis, Utah


.30
.25
.20
.15
.10
.05
Void ratio (e)
0.01
0.1
1.0
10
Pressure (tons/ft²)
CONSOLIDATION TEST RESULTS
Figure no. 34
Boring no. 15
Surface elev.
Depth interval 9.5-10'
Moisture content 10.1 %
Dry unit wt. 130.1 lbs/ft³
LL 25.0 %
PL 15.2 %
PI 9.8 %
Project Plateau Mine Phase II
Wattis, Utah



.20
.15
.10
.05
Void ratio (e)
0.01
0.1
1.0
10
Pressure (tons/ft²)
CONSOLIDATION TEST RESULTS
Figure no. 35 Boring no. 15
Surface elev. Depth interval 15-16'
Moisture content 2.8 % Dry unit wt. 144.4 lbs/ft³
LL 29.8 % PL 17.6 % PI 12.2 %
Project Plateau Mine Phase II
Wattis, Utah

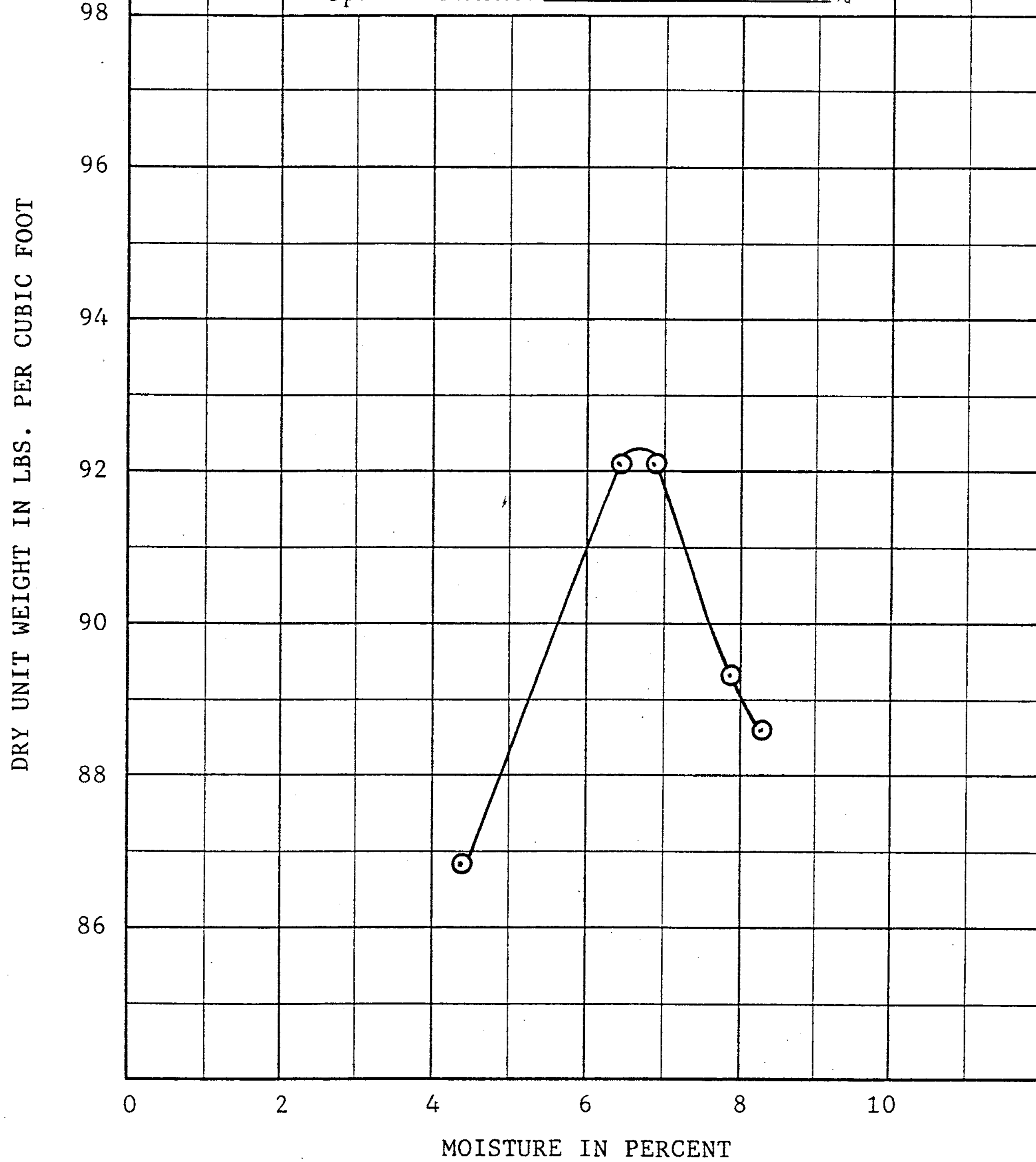

.20
.15
.10
.05
Void ratio (e)
0.01
0.1
1.0
10
Pressure (tons/ft²)
CONSOLIDATION TEST RESULTS
Figure no. 36 Boring no. 15
Surface elev. Depth interval 25-26'
Moisture content 10.7 % Dry unit wt. 144.9 lbs/ft³
LL 24.4 % PL 15.3 % PI 9.1 %
Project Plateau Mine Phase II
Wattis, Utah

SOIL MOISTURE DENSITY RELATIONSHIP

ASTM D1557-78

Maximum Density 92.2 lbs. per cubic foot

Optimum Moisture 6.8 %



ROLLINS, BROWN AND GUNNELL, INC.

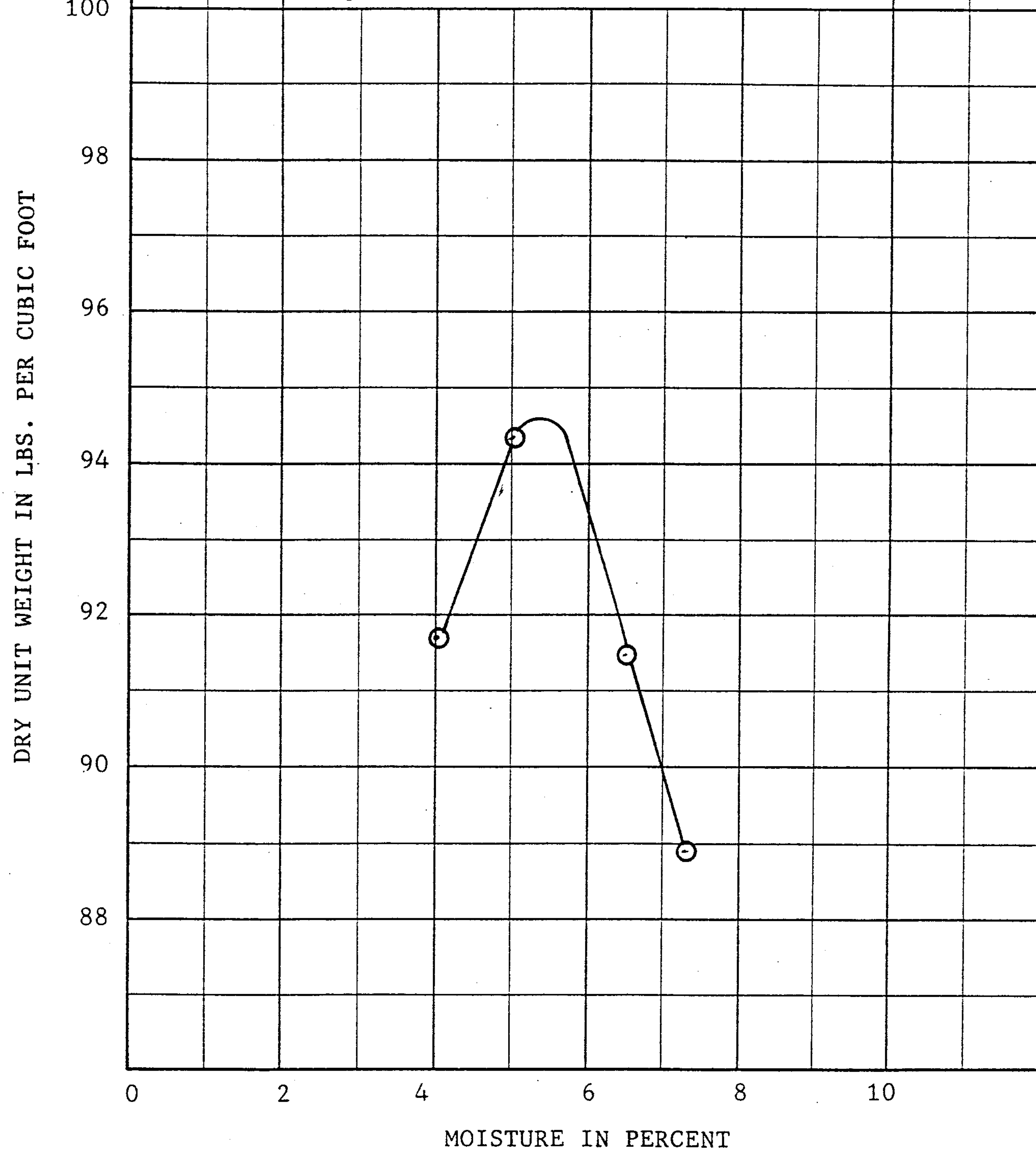
PROFESSIONAL ENGINEERS

Project: Plateau Mine Phase II
Wattis, Utah

Location: Refuse #1

Figure No. 37

SOIL MOISTURE DENSITY RELATIONSHIP
ASTM D1557-78
Maximum Density 94.6 lbs. per cubic foot
Optimum Moisture 5.3 %



ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

Project: Plateau Mine Phase II
Wattis, Utah

Location: Refuse #2

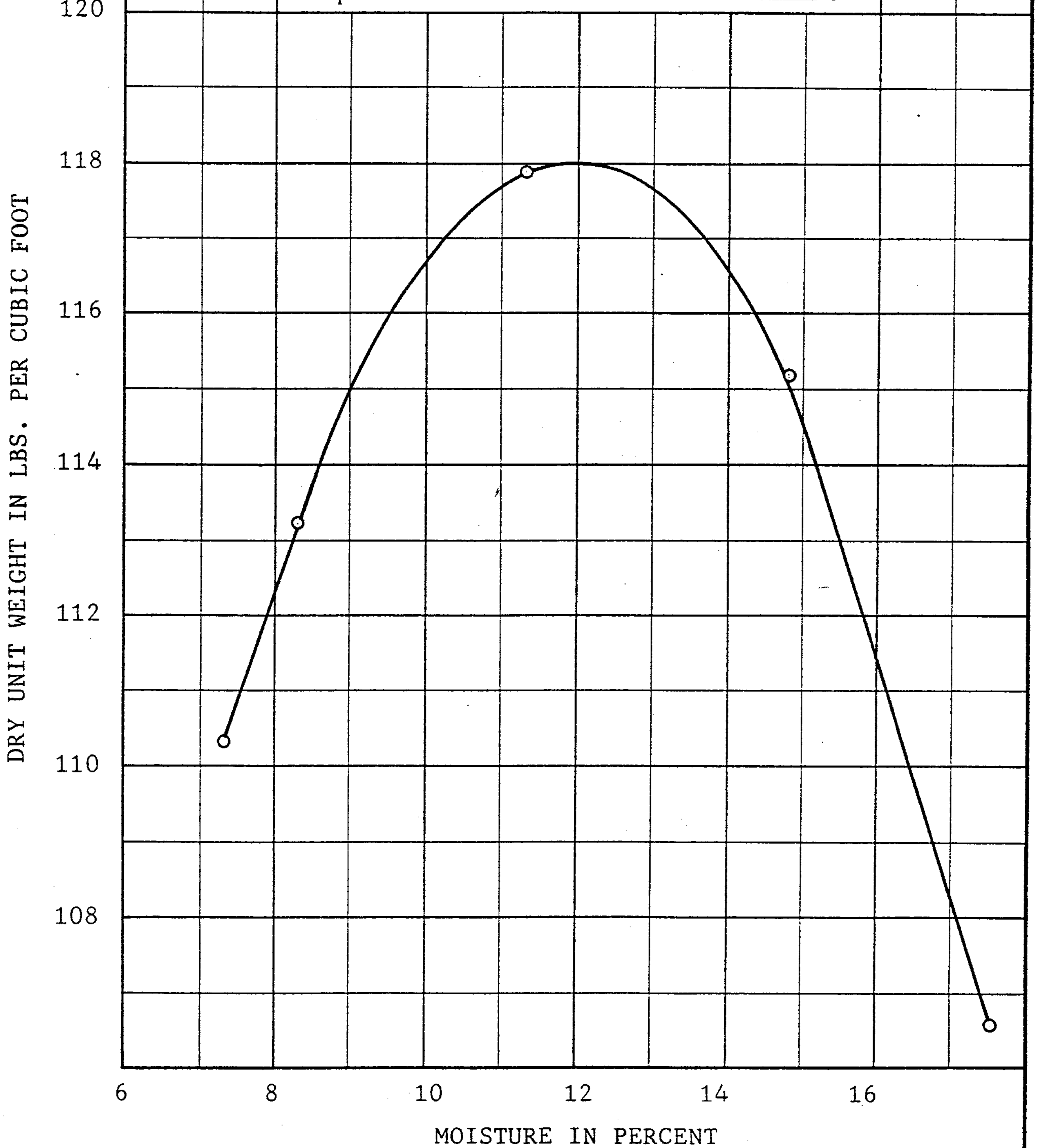
Figure No. 38

SOIL MOISTURE DENSITY RELATIONSHIP

ASTM D 698

Maximum Density 118.0 lbs. per cubic foot

Optimum Moisture 12.0 %



ROLLINS, BROWN AND GUNNELL, INC.

PROFESSIONAL ENGINEERS

Project: Plateau Mine Phase II
Wattis, Utah

Location: Borrow #1

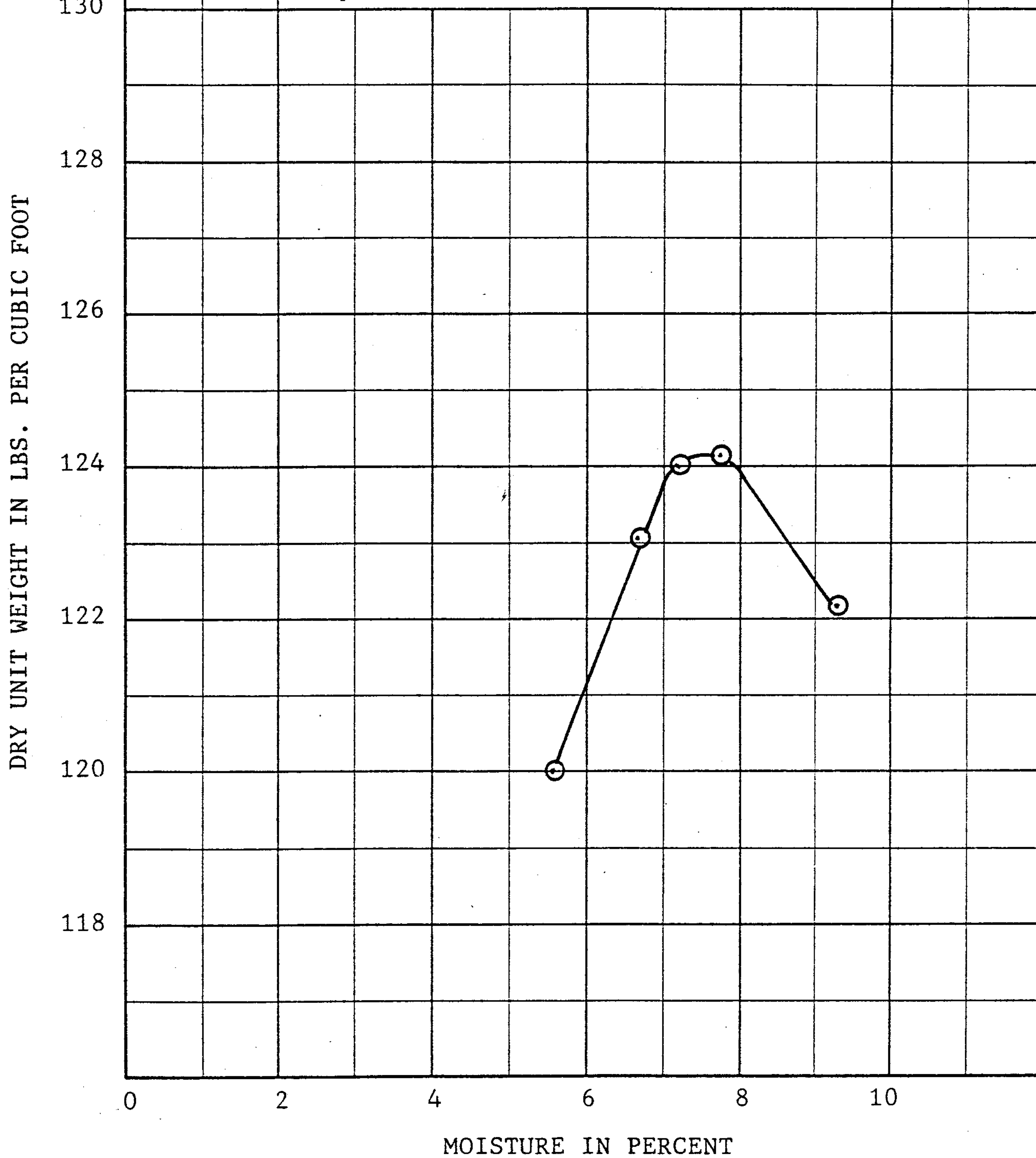
Figure No. 39

SOIL MOISTURE DENSITY RELATIONSHIP

ASTM D 698

Maximum Density 124.1 lbs. per cubic foot

Optimum Moisture 7.5 %



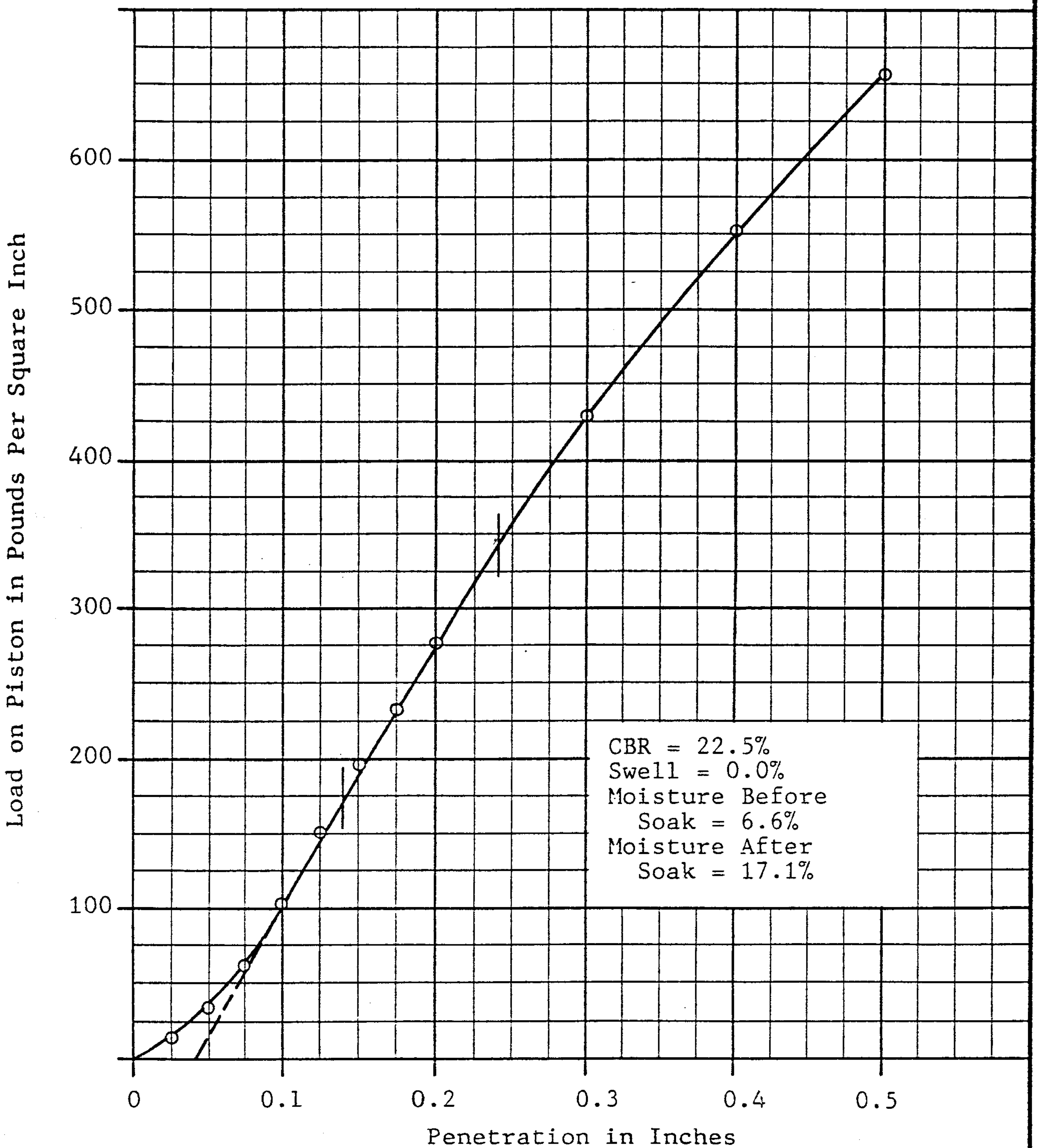
ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

Project: Plateau Mine Phase II
Wattis, Utah

Location: Borrow #2

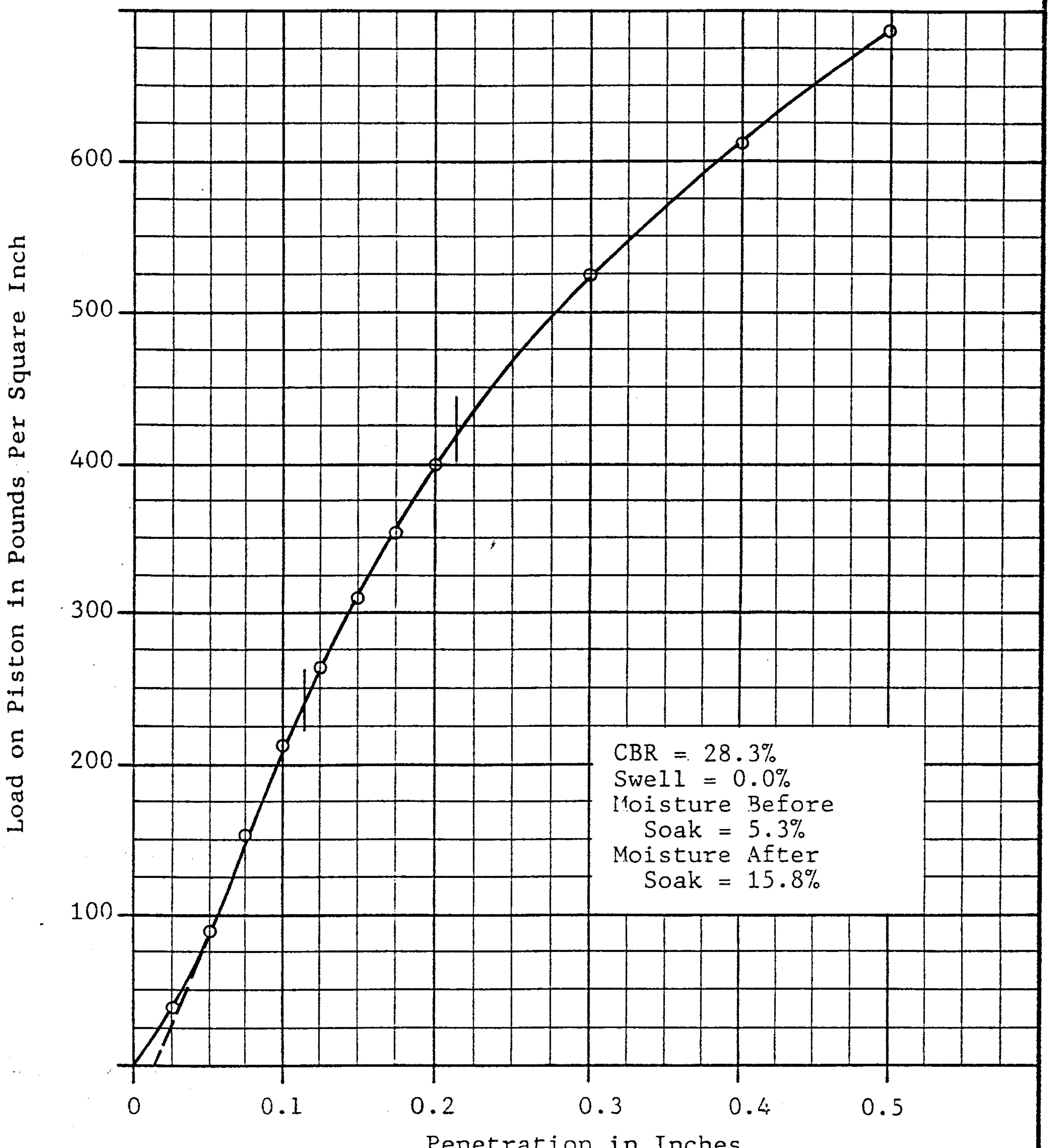
Figure No. 40

CALIFORNIA BEARING RATIO TEST RESULTS



ROLLINS, BROWN & GUNNELL, INC. **CONSULTING ENGINEERS**	PLATEAU MINE PHASE II Sample No. 1 Wattis, Utah	Figure No. 41

CALIFORNIA BEARING RATIO TEST RESULTS



ROLLINS, BROWN & GUNNELL, INC. **CONSULTING ENGINEERS**	PLATEAU MINE PHASE II Sample No. 2 Wattis, Utah	Figure No. 42

TABLE NO. 1 SUMMARY OF TEST DATA

PROJECT Plateau Mine Phase II FEATURE Foundations LOCATION Wattis, 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
1	50-51'	47/5"									34.2	53.1	12.7	SM
	55-56.5'	45									33.5	55.4	11.1	SP,SM
	60-61.5'	27									35.8	51.5	12.7	SM
	65-66.5'	21									17.7	67.4	14.9	SM
	70-71.5'	14									30.1	58.0	11.9	SM
	75-76.5'	11									30.1	41.2	28.7	SM
	80-81.5'	9									23.4	40.5	36.1	SM
	85-86.5'	16									45.5	46.2	8.3	SP,SM
	90-91.5'	41						21.1	16.6	4.5				CL-ML
	95-96.5	34									31.8	36.3	31.9	SM
2	35-36.5'	57									46.2	28.5	25.3	GM
	40-41.5'	9									6.7	37.6	55.7	ML
	45-46.5'	13									25.3	42.1	32.6	SM
	52-53.5'	35									42.3	39.5	18.2	GM

TABLE NO. 1 SUMMARY OF TEST DATA

PROJECT Plateau Mine Phase II FEATURE Foundations LOCATION Wattis, 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
3	60-61.5'	66									33.8	41.0	25.2	SM
	76-77.5'	70									22.9	53.2	23.9	SM
4	55-56.5'	35									37.2	31.7	31.1	GM
	60-61.5'	65									22.8	42.9	34.3	SM
5	5-6.5'	Shelby	107.4	15.4		2768		21.3	16.4	4.9				CL-ML
	10-11'	Shelby	113.2	14.8		*1280		20.6	16.5	4.1				CL-ML
	20-21'	Shelby	113.8	14.8				27.9	16.5	11.4				CL-1
	25-26'	Shelby	103.1	16.1		2422		19.5	17.7	1.8				ML
	30-31'	Shelby	108.8	16.7				24.8	15.2	9.6				CL-1
	35-36.5'	Shelby	102.7	17.0		3838		24.0	16.8	7.2				CL-1
6	0-1.5'										30.1	47.4	22.5	SM
	5-6.5'										42.2	41.4	16.4	GM
	10-11.5'							31.0	22.1	8.9				CL-1
	15-16.5'	75,50/3"						17.3	15.9	1.4				ML
	20-21.5'	30						25.0	15.7	9.3				CL-1

*Torvane Value

TABLE NO. 1 SUMMARY OF TEST DATA

PROJECT Plateau Mine Phase II FEATURE Foundations LOCATION Wattis, Utah

HOLE NO.	DEPTH BELOW GROUND SURFACE	STANDARD PENETRATION BLOWS PER FOOT	IN-PLACE UNIT WEIGHT LB/FT³	IN-PLACE MOISTURE PERCENT	IN-PLACE VOID RATIO	UNCONFINED COMPRESSIVE STRENGTH LB/FT²	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
6	25-26.5'	104									46.1	33.5	20.4	GM
7	3-4.5'	100									43.1	38.0	18.9	GM
	6-7.5'	82									41.1	38.7	20.2	GM
	9-10.5'	62									39.6	39.3	21.1	GM
	12-13.5'	77									47.0	31.6	21.4	GM
	15-16.5'	37									62.4	19.7	17.9	GM
8	3-4.5'	18,47/4"									26.3	28.3	45.4	SM
	9-10.5'	57						18.4	16.7	1.7	13.6	31.8	54.6	ML
	12-12.5'	56/4"									13.8	48.9	37.3	SM
	15-15.5'	56/5"									47.8	30.7	21.5	GM
10	3-4.5'	42									19.7	26.4	53.9	ML
	6-7.5'	26									18.5	35.3	46.2	SM
	9-10.5'	20									8.6	48.9	42.5	SM
	12-13.5'	37									31.6	37.8	30.6	SM
11	3-3.5'	56/4"						21.4	19.8	1.6				ML

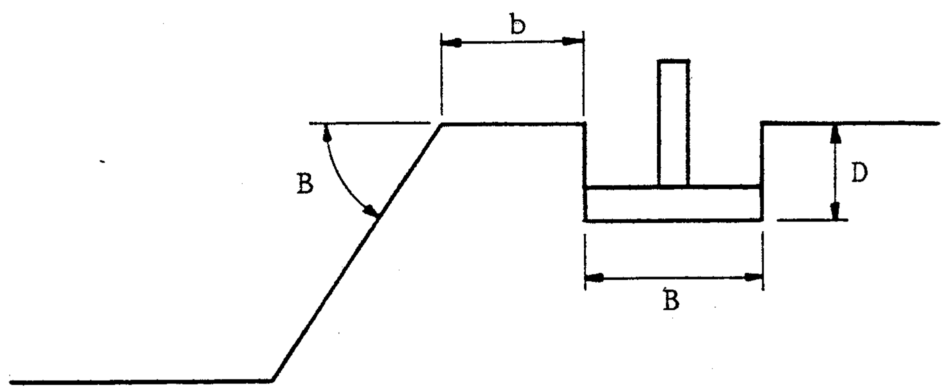
TABLE NO. 1 SUMMARY OF TEST DATA

PROJECT Plateau Mine Phase II FEATURE Foundations LOCATION Wattis, 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
11	6-7.5'	14	122.2	11.6		2369		25.8	16.8	9.0				CL-1
	9-10.5'	19	117.4	15.7				68.1	14.5	53.6				CH
	12-13.5'	86	125.5	10.3		1323		26.7	14.8	11.9				CL-1
	15-16.5'	47/1"	121.8	10.5				29.6	16.9	12.7				CL-1
	20-21'	Core	145.7	4.6		219,168		25.0	15.4	9.6				CL-1
	30-31'	Core	133.7	7.3		8304		26.2	15.8	10.4				CL-1
	40-41'	Core	153.8	3.4		342,864		25.0	14.3	10.7				CL-1
	50-51'	Core	156.0	2.1		748,080		23.6	15.5	8.1				CL-1
	60-61'	Core	156.9	2.6		477,792		23.4	16.8	6.6				CL-ML
	69-70'	Core	157.3	3.3		489,024		24.6	16.0	8.6				CL-1
15	3-3.5'	56/4"									34.3	39.2	26.5	SM
	6-7.5'	38									52.5	27.0	20.5	GM
	9.5-10'	Core	130.1	10.1		3176		25.0	15.2	9.8				CL-1
	15-16'	Core	144.4	2.8		292,134		29.8	17.6	12.2				CL-1
	25-26'	Core	144.9	10.7		1,179,771		24.4	15.3	9.1				CL-1

TABLE NO. 2

Recommended Allowable Bearing Pressures
for Continuous Footings at the Crest of a Slope
(See figure below for identification of symbols)



B(ft)	b(ft)	b/B	D/B	Slope	Allowable Bearing Pressure (kcf)
2	4	2	2.0(D=4')	1.5H:1.0V	2.2
4	4	1.0	1.0(D=4')	1.5H:1.0V	3.7
6	4	0.67	0.67(D=4')	1.5H:1.0V	3.7
8	4	0.5	0.5(D=4')	1.5H:1.0V	3.3