Soil and Foundation Investigation
I-70 at Hidden Valley Interchange
Clear Creek County, Colorado

Prepared for:

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PURPOSE AND SCOPE OF STUDY

This report presents the results of a soil and foundation investigation for cut slopes, bridge widening and retaining walls for the proposed reconstruction and realignment of Interstate 70 from the Hidden Valley interchange to approximately the U.S. 6/ I-70 Interchange east of Hidden Valley. The CDOH project designation is IR 70-3(160). The general project location is shown on Figure 1. This study was conducted in accordance with our proposal dated April 1, 1988. In addition, this report is in conjunction with a previous pavement design analysis submitted on September 17, 1987 under GEC job number 87-203 and its addendum, dated October 13, 1988. The original pavement design report and its addendum are submitted in Appendix A of this report for review. The contents of a preliminary draft soil and foundation investigation for the eastbound on-ramp bridge at Hidden Valley (Structure No. F-15-BK) dated July 25, 1989, (GEC Job No. 87-203B) are incorporated into this report.

A field exploration program was conducted to obtain information on subsurface conditions. Material samples obtained during the subsurface investigation were tested in the laboratory to provide data on the classification and engineering characteristics of the on-site soil and rock. The results of the field and laboratory investigations are presented herein.

This report has been prepared to summarize the data obtained and to present our conclusions and recommendations based on the proposed construction and the subsurface conditions encountered. Design parameters and a discussion of geotechnical engineering considerations related to construction of the proposed roadway and related structures are presented herein.

PROPOSED CONSTRUCTION

Proposed construction at the project site will consist of widening the existing interstate roadway to a 6 lane facility with a 22 foot median, 10 foot shoulders and 12 foot travel lanes. Completing the widening will include relocation of a portion of the Clear Creek channel requiring a series of retaining structures, widening the existing bridge structures over Clear Creek at the Hidden Valley Interchange and excavation into existing slopes or construction of embankments along portions of the alignment. Some details of proposed roadway alignment construction have not yet been determined at the time of preparation of this report.

At the time of our field investigation, the site was occupied with the existing 4-lane interstate highway paralleled by Clear Creek on the south. The existing highway is founded on several feet of embankment fill material which slopes steeply down to Clear Creek. The mountain slopes to the north of I-70 and south of Clear Creek are moderately to very steeply sloping down to the valley floor. Clear

Creek flows down to the east on the order of 4.5% to 5%. Water was flowing in the channel at the time of the investigation.

Site specific details related to each proposed structure are discussed further is subsequent sections of this report.

GEOLOGIC SETTING

The project site is located in the Clear Creek Valley which has been steeply cut into the PreCambrian core complex of the Front Range. The near surface bedrock beneath the site and exposed in the valley walls consists primarily of PreCambrian biotite, hornblende and felsic gneiss. Alluvial gravels, silts and sands with mixed cobbles and boulders overlying the metamorphic bedrock in the channel valley also forms terraces in portions of the project area. Colluvial gravelly soils formed from mechanical and chemical slope weathering overlie both the bedrock and the alluvial soils in areas above the valley floor. The existing topography of the area is mountainous, formed by the erosive action of earlier glacial meltwaters and weathering processes which continue.

Mining of gold in commercial quantities took place in the gravels lying just above the bedrock in the late 1800's and into the 1900's. Hydraulic placer mining methods were used in the meander bar located primarily west and northwest of the bridge structures at the Hidden Valley Interchange. Subsidence of portions of Interstate 70 in this area prompted an investigative drilling program in 1981 by the Colorado Department of Highways. The drilling program revealed several partially filled voids presumably remaining from the previous mining operations. A ground penetrating radar study was conducted during this investigation in an attempt to determine the location and extent of disturbed material or possible voids in the area of known mining activity. Preliminary data and conclusions of the ground penetrating radar survey are presented is a subsequent section of this report. A final report detailing the results and conclusions of the radar study in this area will be submitted when all analysis is completed.

During a geologic review conducted by the Colorado Department of Highways along the project corridor, dated May 5, 1989, the remains of a small precious metal mill site and tailings pond were noted near the meander bar, just northwest of the Hidden Valley Interchange. The remains of the mill and tailings pond were not observed during this investigation. The area of the old mill site has undergone recent and extensive private site grading. The CDOH review also noted that the tailings pond behind the mill contained by-products of precious metal extraction processes. The waste products of cyanidation and heavy metal amalgams (common processing steps in the early 1900's) were noted as potentially hazardous and it was recommended the material not be used for construction purposes unless tests confirm that it is safe.

The western limb of the Floyd Hill fault trends in a northwest/southeast direction and crosses the project area at approximately one-half mile west of the bridges at the U.S. 6/ I-70 Interchange. The fault trace in this area is hidden and has been mapped by Sheridan and Marsh, 1976. The fault should not adversely affect proposed construction in the project area.

FIELD INVESTIGATION

The field investigation for the project was conducted from May to December, 1989. Test pits, test holes, seismic testing and ground penetrating radar were employed in various areas of the corridor. Locations and graphic logs of the test holes and test pits are presented on Figures 1 through 27. Seismic survey data is presented in Appendix C. Ground penetrating radar preliminary results are presented in Appendix B. Locations and elevations of the test holes were determined by referencing features shown on the site plan provided and are summarized on Table 1.

The test holes were advanced through existing pavement structures, embankment materials, overburden soils and into the underlying bedrock with 3 types of drills including: 4-inch diameter continuous flight augers, 7.25 inch O.D. hammer drill and a 6.0-inch O.D. Odex air circulating drill. The test pits were excavated using a Caterpillar 229 track hoe. The test holes and pits were logged by a representative of Ground Engineering Consultants, Inc.

Test holes 1 through 15 were advanced with a Becker Hammer Drill using a 7-1/4-inch O.D. Felcon bit and a Link Belt 180 hammer with a full-stroke capacity of approximately 13,000 ft-lbs. The holes were drilled with compressed air constantly circulating cuttings up to 2.5 inches in diameter to the surface. Test Holes 16 through 18 were drilled with 4-inch diameter continuous flight augers. Test holes O-1, O-2 and O-3 were drilled with a 6.0-inch Odex drill with compressed air constantly circulating bit cuttings to the surface.

Samples of the subsurface materials were taken from drill cuttings and with a 1-3/8 inch I.D. spoon sampler. The sampler was driven into the various strata with blows from a 140-pound hammer falling 30 inches. This test is similar to the standard penetration test described by ASTM Method D-1586. Penetration resistance values, when properly evaluated, indicate the relative density or consistency of the soils. Depths at which the split spoon samples were taken and the penetration resistance value are shown on the logs of the exploratory holes, Figures 8 through 27.

During drilling of test holes 1 through 15, a continuous record of the number of blows per foot required to drive the Becker drill pipe was recorded. The record is shown graphically on the logs of exploratory holes. When evaluating the penetration record it should be noted that, although the Link

Belt hammer generally operated at full stroke and capacity during drilling, some portions of the test holes were drilled at partial capacity (particularly near the surface).

Measurements of the water level were made in the test holes by lowering a weighted plumbline into the open hole shortly after completion of drilling.

The seismic survey was conducted by representatives of Ground Engineering Consultants, Inc. using a portable Bison 1570-C single channel seismograph utilizing a sledge hammer and plate as the source. The seismic equipment was used to obtain information on subsurface conditions and material particle velocities in areas not accessible to conventional drilling methods. Seismic field data is presented in Appendix C.

LABORATORY INVESTIGATION

Samples obtained from our exploratory holes were examined and classified in the laboratory by the project engineer. Laboratory testing included standard property tests, such as natural moisture contents, grain size analyses and liquid and plastic limits. Results of the laboratory testing program are summarized on Table 2. The laboratory testing was conducted in general accordance with applicable ASTM specifications.

SUBSURFACE CONDITIONS AND FOUNDATION RECOMMENDATIONS

Item 1:

Eastbound off-ramp retaining wall, west of Hidden Valley Interchange.

Conclusions: The retaining wall structure may be placed on the natural silty gravel and sand with cobbles. Design and construction recommendations are outlined below.

Proposed Construction: We understand that the proposed retaining wall will measure approximately 600 feet in length and will extend from STA 7+60 E.B. Off-Ramp to STA 13+60 E.B. Off-Ramp. The wall will be constructed to allow widening of the off-ramp. The proposed retaining structure centerline will be located approximately 33 feet right of the Ramp Control Line. If conditions are significantly different from those described above, we should be notified to reevaluate the recommendations contained in this section.

Existing Conditions: At the time of our investigation, the existing slope between the off-ramp and the south frontage road was in good condition with no signs of slope instability observed. The slope ranges in height from approximately 14 feet at the west end of the proposed wall to a maximum of 25 feet near the middle to approximately 8 feet at the east end. The slope was grass covered with occasional cobbles and boulders observed at the surface. Existing slope angles measured in the field ranged from 1.4(H):1(V) to 2.2(H):1(V).

Subsurface Conditions: Test holes 3 and 4 were drilled in the north edge of the south frontage road at the locations shown on Figure 2 to explore subsurface conditions. The test holes were advanced through the existing pavement section and fill material and into the underlying natural sands and gravels. Graphic logs of test holes 3 and 4 are shown on Figures 10 and 11.

A pavement section consisting of 2.5 inches of asphalt was encountered at the surface in test hole 3 and a pavement section consisting of 2 inches of asphalt over 4 inches of base course was encountered at the surface in test hole 4. Embankment fill material was encountered beneath the pavement section in test holes 3 and 4 to depths of 12.5 and 7.5 feet, respectively. The embankment fill material consisted of silty sand and gravel with cobbles and occasional boulders, fine to coarse grained, medium dense to dense, dry to moist and brown to tan.

Natural sands and gravels were encountered from beneath the fill to depths of 40 and 30 feet in test holes 3 and 4, respectively, the maximum depth explored. The natural sands and gravels were mixed with rounded cobbles and occasional boulders, fine to coarse grained, clean to silty, moderately dense to very dense, moist and brown to tan. Free groundwater was not encountered in test holes 3 and 4 at the time of drilling.

Ground Penetrating Radar Study: Two traverses of the ground penetrating radar study were located on the east bound off-ramp. Preliminary radar data analysis coupled with bore hole data from test hole O-3 indicate that the bedrock elevation near the east end of the proposed retaining wall is approximately 7298 feet. Although occasional disturbance of the overburden soils was noted in the radar data near the retaining wall, we do not believe that previous mining activity will adversely affect the founding materials for the retaining wall. The foundation excavation should be carefully observed by the soil engineer to determine if additional excavation and replacement is necessary. Additional information concerning the ground penetrating radar study is discussed later in this report.

Foundation Recommendations: Based on the field and laboratory investigation, the retaining wall at this location may be placed on the undisturbed natural sands and gravels and may be designed for a maximum allowable soil bearing pressure of 4,000 psf. Additional design and construction criteria is outlined under "Spread Footing Design and Construction Criteria".

Cantilevered retaining structures at this site can be expected to deflect sufficiently to mobilize the full active earth pressure condition. Therefore, cantilevered structures may be designed for a lateral earth pressure computed on the basis of a coefficient of active earth pressure (Ka) of 0.3073 and a soil unit weight of 120 pcf.

The retaining structure should be designed for appropriate surcharge pressures such as adjacent traffic. An upward sloping backfill surface also increases the earth pressures on retaining structures. The active and lateral earth pressure coefficients given in this section are for a level backfill surface and must be corrected for upward sloping fills.

The lateral resistance of retaining wall foundations placed on undisturbed natural soils at the site will be developed by a combination of the sliding resistance of the footing on the foundation materials and the passive pressure against the side of the footing. Sliding friction at the bottom of the footing can be taken as 0.4 times the vertical dead load. Passive pressure against the sides of the footing can be calculated using a coefficient of passive earth pressure (Kp) of 3.2546 and a soil unit weight of 120 pcf.

Retaining wall backfill materials and fill placed against the sides of the footings to resist lateral loads should be placed and compacted according to the criteria presented in "Site Grading."

Item 2:

Eastbound On-Ramp Bridge, Structure F-15-BK, at the Hidden Valley Interchange.

<u>Conclusions</u>: The proposed bridge may be founded on end bearing piles driven into the bedrock beneath the site. Design and construction recommendations are outlined below.

<u>Proposed Construction</u>: We understand that the proposed construction for the eastbound on-ramp structure F-15-BK will consist of removal of the existing bridge and construction of a new bridge slightly downstream of the existing location. During construction of the new eastbound and westbound I-70 bridges (F-15-BI/BJ) over Clear Creek, structure F-15-BK will temporarily carry all eastbound I-70 traffic over Clear Creek. Structure loadings will be moderate, typical of bridge structures.

If loadings or conditions are significantly different from those described above, we should be notified to reevaluate the recommendations contained in this report.

<u>Existing Structure</u>: The existing bridge structure is a 3-span concrete bridge with steel beams and concrete abutments and wingwalls. The existing structure carries eastbound on-ramp traffic from the Hidden Valley Interchange over Clear Creek. At the time of this investigation, the water surface in Clear Creek lies approximately 21 feet below the bridge deck. Clear Creek is approximately 4 to 5 feet deep and 50 feet wide at the structure and the drainage channel slopes eastward on the order of 1%.

Vegetation immediately downstream of the existing structure consists of native grasses, shrubs and small deciduous trees on the existing channel banks.

Test holes 1 and 2 were drilled at the locations shown on Figure 3 to explore subsurface conditions. The test holes were advanced through the overburden soils and into the underlying bedrock with the Becker Hammer Drill. Graphic logs of test holes 1 and 2 are shown on Figures 8 and 9.

Test Hole 2, drilled in the shoulder of the on-ramp east of the bridge, encountered 4.75 inches of asphalt at the surface. Test Hole 1 was drilled in the drainage area between the ramp and south frontage road on the west bank and encountered a thin 1-inch layer of topsoil at the surface. Underlying the topsoil and asphalt, embankment fill material was encountered to depths of 12 and 22.5 feet in test holes 1 and 2, respectively. The fill material consisted of slightly silty gravel and sand with cobbles and occasional small boulders, moderately dense to dense, moist to wet and brown. Natural sand and gravel with cobbles and occasional boulders encountered beneath the fill was moderately dense to very dense, very moist to wet and tan to brown.

Metamorphic gneiss was encountered at depths of 22.5 and 33.5 feet in test holes 1 and 2, respectively. The gneiss is crystalline and hard to very hard. A review of drill data collected by the Department of Highways for adjacent structures F-15-BI/BJ and our observations of exposed bedrock indicate the bedrock is foliated with an average orientation striking 70 E and dipping 42 N. The bedrock is foliation jointed with other random normal and diagonal joint sets.

Free groundwater was measured at a depth of 10 feet in test hole 1 and 21 feet in test hole 2 shortly after drilling. The free groundwater level correlates closely with the water level in Clear Creek at the time of drilling.

Foundation Recommendations: Based on the subsurface conditions encountered and the proposed construction, we recommend the bridge structure be supported on end bearing piles. The design and construction details presented below should be observed for a driven pile foundation system. Construction details should be considered when preparing project documents.

(1) The pile should consist of a heavy steel H-section with tip reinforcement.

(2) The maximum allowable pile capacity should not exceed a pile service stress of 9,000 psi for the H-section.

(3) Piles should penetrate through the sand and gravel stratum and bear on the bedrock.

(4) Lateral resistance to horizontal forces can be resisted by battered piles. It is normal to assume a battered pile can resist the same axial load as a vertical pile of the same type and size driven to the same depth. The vertical and horizontal components of the load will depend on the batter inclinations. Batters should not exceed 1 horizontal to 4 vertical.

(5) Piles may be designed to resist lateral loads assuming a modulus of horizontal subgrade reaction in the overburden soils of 100 tcf. The upper 5 feet of pile penetration and all materials above estimated scour depth should be neglected.

(6) Uplift on the piles should be limited to 25% of the indicated vertical load capacities.

(7) Groups of piles required to support concentrated loads will require an appropriate reduction of the estimated bearing capacity based on the effective envelope area of the pile group. This reduction can be avoided by spacing piles a distance of at least 3 diameters center to center. Pile groups spaced less than 3 diameters center to center should be studied on an individual basis to determine the appropriate reduction for both lateral and axial capacities.

(8) All piles should be advanced to virtual refusal. Virtual refusal in defined as 15 blows per inch with an approved pile hammer.

(9) The manufacturer's rated energy output of the hammer should be between 1,800 to 2,000 foot-pounds per square inch of steel section.

(10) The hammer should be operating at the manufacturers recommended stroke and speed when virtual refusal is measured. Embankments in the area of foundation construction should be constructed of materials free of boulders and deleterious soils.

(11) A representative of the soil engineer should observe all pile driving operations.

(12) Predrilling should be specified in areas where embankments are greater than 10 feet in depth. In addition, predrilling may be required to advance piles through dense sand, gravel, boulders and overburden.

(13) Due to the dense sand and gravel layer and large boulders and cobbles encountered in the test borings, difficult driving conditions should be anticipated. These conditions could adversely effect the structural integrity of piles and also result in variables of penetration of subsurface materials. Consequently, we recommend the pile driving

operation be closely monitored. Piles which meet virtual refusal above bedrock (approximate elevation = 7,275) should be assessed in the field on the basis of the driving records.

<u>Abutments and Wingwalls</u>: Cantilevered retaining structures on the bridge site can be expected to deflect sufficiently to mobilize the full active earth pressure condition. Therefore, cantilevered structures may be designed for a lateral earth pressure computed on the basis of an equivalent fluid unit weight of 35 pcf for granular backfill. It should be noted that the equivalent fluid unit weight for backfill material given above is for a level backfill surface and should be corrected for upward sloping fills. All retaining structures should be designed for appropriate hydrostatic and surcharge pressures such as adjacent traffic and construction materials. The abutments and wingwalls should be founded on end-bearing piles designed and constructed the same as the bridge structure.

We recommend granular soils for backfilling abutments and wingwalls because their use results in lower lateral earth pressures. Granular backfill should conform to the Colorado Department of Highways Class I structural fill as defined in the Standard Specifications for Road and Bridge Construction. Granular material should be placed to within 1 foot of the ground surface and to a minimum distance beyond the walls equal to at least one-half the height of the fill. The upper 1 foot of the wall backfill should be a relatively impervious soil or a pavement structure to prevent surface water infiltration into the backfill.

Backfill should be carefully placed in uniform lifts and compacted to at least 95% of the maximum modified Proctor density (ASTM D 1557, AASHTO T-180), within 2% of optimum moisture content. Care should be taken not to overcompact the backfill since this could cause excessive lateral pressure on the walls. Some settlement of deep foundation wall backfills will occur even if the material is placed correctly.

<u>Site Excavations</u>: Temporary cuts in the fill material and overburden soils should not exceed 1.5:1 (horizontal to vertical) slopes. At these slopes some minor sloughing may occur, however major stability problems should not be encountered unless seepage is encountered in the cuts. If seepage is encountered, we should be notified to evaluate the conditions and evaluate if remedial measures are required.

All areas excavated for the proposed bridge construction should be backfilled with an approved material placed under controlled conditions. We recommend that all backfill be placed in uniform lifts not exceeding 8 inches (loose) and compacted to at least 95 percent of the maximum modified Proctor density (ASTM D 1557, AASHTO T-180) within 2% of the optimum moisture content.

Fill for roadway embankments should be placed in uniform, horizontal lifts and compacted as described above. Most of the on-site soils appear to be suitable for use as fill within the roadway embankment. Additional recommendations concerning site grading are outlined under "Site Grading" of this report.

Erosion Potential: The erosion potential of the Clear Creek channel in the vicinity of the proposed bridge construction was evaluated by field observation and laboratory gradation tests of samples obtained from the soil borings and channel bank. Grain size distributions of the materials sampled are presented on Table 2. Results of the soil gradation tests and our visual observation of the existing channel bank material confirm the variability of the soils at the site. Although cobbles and some boulders were observed in the channel, relatively clean, finer grained materials were also observed in the channel banks. It is our opinion that additional scour protection is needed. The type of protection used should be based on the calculated stream velocity.

Item 3:

Retaining wall on north side of Clear Creek, immediately east of Structure F-15-BK.

Conclusions: The retaining wall structure may be placed on the natural silty sands and gravels. Design and construction recommendations are outlined below.

Proposed Construction: We understand that the proposed retaining wall will measure approximately 1,095 feet in length and will extend from approximately STA 226+85 to STA 237+75. The wall will be constructed to allow widening of the eastbound travel lane. The proposed retaining structure centerline will be located approximately 57 to 60 feet right of the I-70 Control Line. If conditions are significantly different from those described above, we should be notified to reevaluate the recommendations contained in this section.

Existing Conditions: At the time of our investigation, the existing slope between east bound I-70 and Clear Creek was in good condition with no signs of major slope instability observed. The existing slope ranges in height from approximately 23 feet near Structure F-15-BK to approximately 11 feet at the east end of the proposed wall. The slope was partially grass covered with cobbles and boulders observed on the slope face. Existing slope angles measured in the field ranged from 1.2(H):1(V) to 1.7(H):1(V). From approximately STA 229+00 to Sta 233+00 grassy banks with deciduous trees and willows extend into Clear Creek from the bottom of the existing slope face. The retaining wall is located east of the ground penetrating radar study area as well as areas of known mining activity.

Subsurface Conditions: Test pits 1 and 2 were excavated at the locations shown on Figure 4 to explore subsurface conditions. The test pits were excavated with a Cat 229 track hoe. In addition, test holes 6 and 7 were drilled at the locations shown on the south side of Clear Creek with the Becker Hammer drill. Test Hole 2, described above under Item 2, was drilled at the west end of the proposed retaining structure (also at the east abutment of Structure F-15-BK).

Test pits 1 and 2 were excavated in the grass bank at the base of the existing slope just above the water level in Clear Creek. Topsoil was encountered at the surface in both test pits measuring 10 and 12 inches thick in test pits 1 and 2, respectively. Beneath the topsoil, natural sand and gravel was encountered to depths of 10 feet in test pit 1 and 7.5 feet in test pit 2, the maximum depths explored. The natural silty sand and gravel mixed with cobbles was also encountered from the surface to depths of 7.5 and 8.5 feet in test holes 6 and 7. Crystalline gneiss bedrock was encountered in test pit 1 at a depth of 10 feet and beneath the natural sand and gravel in test holes 6 and 7. Graphic logs of the test pits and test holes are shown on Figures 13, 14 and 25.

The natural sand and gravel encountered in the test pits was silty to very silty, mixed with cobbles and boulders to 3 feet in diameter, moderately dense to dense, wet, pyritic and red-brown. Free groundwater was encountered in the test pits at a depth of 4.5 feet in test pit 1 and at a depth of 2 feet in test pit 2. Free groundwater was measured in test holes 6 and 7 at a depth of 8 feet in both holes at the time of drilling. The vertical walls of the test pit excavations were unstable due to seepage and caved during or shortly after excavation. Occasional silt lenses were also noted. The gneiss is crystalline and hard to very hard.

Based on the subsurface information from test hole 2 (Item 2) and our observations, existing embankment fill material extends down from the shoulder to within a few feet of the base of the slope. The embankment material encountered in test hole 2 consisted of slightly silty gravel and sand with cobbles and occasional small boulders and was medium dense to dense, moist to wet and brown.

Foundation Recommendations: Based on the field and laboratory investigation and the proposed wall location, the retaining wall may be founded on the existing embankment fill material and may be designed for a maximum allowable soil bearing pressure of 3,500 psf. If the foundation is extended and placed on the undisturbed natural sands and gravels the wall may be designed for a maximum allowable soil bearing pressure of 4,000 psf. It should be noted that footing excavation near or below the water level in Clear Creek will require excavation dewatering during excavation and possibly shoring. Additional design and construction criteria is outlined under "Spread Footing Design and Construction Criteria".

Cantilevered retaining structures at this site can be expected to deflect sufficiently to mobilize the full active earth pressure condition. Therefore, cantilevered structures may be designed for a lateral earth pressure computed on the basis of a coefficient of active earth pressure (Ka) of 0.3073 and a soil unit weight of 120 pcf.

The retaining structure should be designed for appropriate surcharge pressures such as adjacent traffic. An upward sloping backfill surface also increases the earth pressures on retaining structures. The active and lateral earth pressure coefficients given in this section are for a level backfill surface and must be corrected for upward sloping fills.

The lateral resistance of retaining wall foundations placed on undisturbed natural soils at the site will be developed by a combination of the sliding resistance of the footing on the foundation materials and the passive pressure against the side of the footing. Sliding friction at the bottom of the footing can be taken as 0.4 times the vertical dead load. Passive pressure against the sides of the footing can be calculated using a coefficient of passive earth pressure (Kp) of 3.2546 and a soil unit weight of 120 pcf.

Retaining wall backfill materials and fill placed against the sides of the footings to resist lateral loads should be placed and compacted according to the criteria under "Site Grading."

Item 3A:

South frontage road retaining wall at Hidden Valley Interchange.

Conclusions: The retaining wall structure may be founded on the natural silty gravel and sand with cobbles. Design and construction recommendations are outlined below.

Proposed Construction: We understand that the proposed retaining wall at this location will measure approximately 470 feet in length and will extend from STA 12+70 E.B. Off-Ramp to STA 17+40 E.B. Off-Ramp. The proposed retaining structure centerline will be located approximately 18 feet right of the centerline of the South Frontage Road. If conditions are significantly different from those described above, we should be notified to reevaluate the recommendations contained in this section.

Existing Conditions: At the time of our investigation, the existing slope at the proposed wall location was in fair condition with no signs of slope instability observed. An unsurfaced private drive is located at the top of the existing slope. The slope ranges in height from approximately 5 feet at the west end of the proposed wall to a maximum of approximately 20 feet near the middle at the interchange to approximately 10 feet at the east end. The slope was lightly grass covered with occasional shrubs and small pine trees. Cobbles and occasional boulders were observed on the slope face. Existing slope angles measured in the field ranged from 0.9(H):1(V) to 1.6(H):1(V). Surface erosion from water runoff was noted in steeper portion of the existing slope.

Subsurface Conditions: Test holes 4 and 5 were drilled at the locations shown on Figure 4 to explore subsurface conditions. The test holes were advanced through the existing pavement section and fill material and into the underlying natural sands and gravels. Graphic logs of test holes 4 and 5 are shown on Figures 11 and 12.

Test hole 4 was drilled in the north edge of the south frontage road and is described under Item 1. Test hole 5 was drilled near the east end of the proposed retaining wall just south of the frontage road pavement edge. Loose fill material was encountered from the surface in test hole 5 to a depth of 6 feet. The fill material consisted of silty, gravelly sand which was moist and dark brown with occasional cobbles. Below 6 feet to a depth of 17 feet, natural sands and gravels were encountered in test hole 5. The natural sands and gravels were clean to silty, fine to coarse grained with occasional cobbles and small boulders, moderately dense and moist to wet.

Free groundwater was encountered in test hole 5 at a depth of 10 feet at the time of drilling. When the drill pipe was pulled, the test hole caved to a depth of 3 feet. A significant drop in the continuous blow count log was recorded at the approximate free groundwater level from 10 to 12.5 feet in depth.

Ground Penetrating Radar Study: Two traverses of the ground penetrating radar study were located on the south frontage road near the interchange. Occasional disturbance of the overburden soils was noted at approximately STA 14+50 on the south frontage road. In addition, a mining map dated from the late 1800's indicates that small mine shafts were located just south of the proposed wall on the top slope. No evidence of the shafts was observed during our field investigation. Based on the radar data near the proposed retaining wall, we do not believe that previous mining activity will adversely affect the founding materials for the retaining wall. The foundation excavation should, however, be carefully observed by the soil engineer to determine if additional excavation and replacement is necessary. Details of the ground penetrating radar study are discussed in a subsequent section of this report.

Foundation Recommendations: Based on the field and laboratory investigation, the retaining wall at this location may be placed on the undisturbed natural sands and gravels or properly compacted fill and may be designed for a maximum allowable soil bearing pressure of 4,000 psf. Additional design and construction criteria is outlined under "Spread Footing Design and Construction Criteria". The foundation should be extended below existing fill material where possible. The foundation will approach the free groundwater level at the center and east end of the wall. Care should be taken when constructing the wall foundation near the groundwater level as the bearing capacity of the soils can be adversely affected by improper dewatering methods or construction traffic. The recommendations for footing dewatering under "Spread Footing Design and Construction Criteria" should be followed.

Cantilevered retaining structures at this site can be expected to deflect sufficiently to mobilize the full active earth pressure condition. Therefore, cantilevered structures may be designed for a lateral earth pressure computed on the basis of a coefficient of active earth pressure (Ka) of 0.3073 and a soil unit weight of 120 pcf.

The retaining structure should be designed for appropriate surcharge pressures such as adjacent traffic. An upward sloping backfill surface also increases the earth pressures on retaining structures. The active and lateral earth pressure coefficients given in this section are for a level backfill surface and must be corrected for upward sloping fills. The retaining wall should be constructed at the base of the slope cuts such that the top of the wall meets the existing slope face.

The lateral resistance of retaining wall foundations placed on undisturbed natural soils at the site will be developed by a combination of the sliding resistance of the footing on the foundation materials and the passive pressure against the side of the footing. Sliding friction at the bottom of the footing can be taken as 0.4 times the vertical dead load. Passive pressure against the sides of the footing can be calculated using a coefficient of passive earth pressure (Kp) of 3.2546 and a soil unit weight of 120 pcf.

Retaining wall backfill materials and fill placed against the sides of the footings to resist lateral loads should be placed and compacted according to the criteria presented in "Site Grading."

Item 4:

Retaining wall on north side of Clear Creek, from approximately STA 250+00 to STA 267+00.

Although outlined in our original proposal under Item 4, we understand that this retaining wall will not be constructed. Some of the information collected for nearby structures in this investigation can be reviewed to provide foundation information in this area if required in the final design.

Item 5:

Channel change retaining wall on north side of Clear Creek, STA 255+75 to STA 262+75, near east portion of project.

<u>Conclusions</u>: The retaining wall structure may be placed on the natural sands and gravels mixed with cobbles and boulders. Design and construction recommendations are outlined below.

Foundation Recommendations: Based on the field and laboratory investigation and the proposed wall location, the retaining wall may be founded on the undisturbed natural sands and gravels and may be designed for a maximum allowable soil bearing pressure of 4,000 psf. It should be noted that footing excavation near or below the water level in Clear Creek will require excavation dewatering during construction and possibly shoring. Additional design and construction criteria is outlined under "Spread Footing Design and Construction Criteria".

Cantilevered retaining structures at this site can be expected to deflect sufficiently to mobilize the full active earth pressure condition. Therefore, cantilevered structures may be designed for a lateral earth pressure computed on the basis of a coefficient of active earth pressure (Ka) of 0.3073 and a soil unit weight of 120 pcf.

The retaining structure should be designed for appropriate surcharge pressures such as adjacent traffic. An upward sloping backfill surface also increases the earth pressures on retaining structures. The active and lateral earth pressure coefficients given in this section are for a level backfill surface and must be corrected for upward sloping fills.

The lateral resistance of retaining wall foundations placed on undisturbed natural soils at the site will be developed by a combination of the sliding resistance of the footing on the foundation materials and the passive pressure against the side of the footing. Sliding friction at the bottom of the footing can be taken as 0.4 times the vertical dead load. Passive pressure against the sides of the footing can be calculated using a coefficient of passive earth pressure (Kp) of 3.2546 and a soil unit weight of 120 pcf.

Retaining wall backfill materials and fill placed against the sides of the footings to resist lateral loads should be placed and compacted according to the criteria presented in "Site Grading."

Item 6:

Channel change retaining wall on south side of Clear Creek, STA 257+75 to STA 261+75, at the east end of the project.

Conclusions: The retaining wall structure may be placed on the natural sand and gravel materials. Design and construction recommendations are outlined below.

Proposed Construction: We understand that the proposed retaining wall will measure approximately 400 feet in length and will extend from approximately STA 257+75 to STA 261+75. The wall will be constructed to realign the flow of Clear Creek to the south, allowing widening of the I-70 travel lanes. The proposed retaining structure centerline will be located approximately 95 feet right of the I-70 Control Line. The proposed wall location is shown on Figure 5. If conditions are significantly different

Proposed Construction: We understand that the proposed retaining wall will measure approximately 700 feet in length and will extend from approximately STA 255+75 to STA 262+75. The wall will be constructed to realign the flow of Clear Creek to the south, allowing widening of the I-70 travel lanes. The proposed retaining structure centerline will be located approximately 60 to 65 feet right of the I-70 Control Line. The proposed wall location is shown on Figure 5 . If conditions are significantly different from those described above, we should be notified to reevaluate the recommendations contained in this section.

Existing Conditions: The proposed retaining wall will be located near the base of the existing slope between eastbound I-70 and Clear Creek at the west end, will cross the existing Clear Creek channel near the center of the proposed wall, and will be located near the existing south bank of Clear Creek at the east end of the proposed wall. The realigned creek channel will be 30 feet in width. The existing slope between eastbound I-70 and Clear Creek is steep (approximately 1.5(H):1(V)), approximately 17 feet in height and covered with angular boulders. Clear Creek in this portion of the project is swift flowing and the channel bottom is uneven and strewn with boulders. The existing slope south of Clear Creek near the center and east end of the proposed wall is also steep (Approx. 1(H):1(V)), approximately 20-25 feet high and has little vegetation. The proposed retaining wall is located east of known mining activity and the ground penetrating radar study area.

Subsurface Conditions: Test hole 8 was drilled on the south shoulder of east bound I-70 at the west end of the proposed retaining wall. Test holes 10 and 11 were drilled south of Clear Creek on an existing unsurfaced access road. The test holes were drilled to explore subsurface conditions with the Becker Hammer Drill and test hole locations are shown on Figure 5. Graphic logs of the test holes are shown on Figures 15, 17 and 18.

Five inches of asphalt were encountered at the surface in test hole 8 followed by embankment fill material to a depth of 15 feet. The embankment fill material is similar to that encountered in test hole 2 (Item 2). Natural sand and gravel was encountered beneath the embankment material to a depth of 41.5 feet in test hole 8, where crystalline bedrock was encountered. The sand and gravel was medium dense to dense, with boulders and numerous cobbles, clean to silty, moist to wet and tan. The continuous penetration log for the test hole indicates areas of more difficult drilling.

Test hole 10 encountered fill material from the surface to 4 feet in depth. The fill was silty to gravelly sand with cobbles and was moderately dense, moist and brown to dark brown. Natural sand and gravel mixed with cobbles was encountered beneath the fill to 11 feet in depth where crystalline bedrock was noted. Crystalline gneiss bedrock was encountered at a depth of 17.5 feet in test hole 11, beneath the natural gravelly sand and cobble layer.

from those described above, we should be notified to reevaluate the recommendations contained in this section.

Existing Conditions: The proposed retaining wall will be located near the base of the existing slope south of Clear Creek. The realigned creek channel will be 30 feet in width. Clear Creek in this portion of the project is swift flowing and the channel bottom is uneven and strewn with boulders. The existing slope south of Clear Creek near the proposed wall is steep (approximately 1(H):1(V)), approximately 20-25 feet high and has little vegetation. The proposed retaining wall is located east of known mining activity and the ground penetrating radar study area.

Subsurface Conditions: Test holes 9, 10 and 11 were drilled south of Clear Creek on the existing unsurfaced access road. The test holes were drilled to explore subsurface conditions with the Becker Hammer Drill and test hole locations are shown on Figure 5. The subsurface conditions encountered in test holes 10 and 11 are described above under Item 5. Graphic logs of test holes are shown on Figures 16,17 and 18.

Fill material was encountered from the surface to 7 feet in depth in test hole 9. It should be noted that test hole 9 was drilled 10 feet west of an existing CMP drain pipe and the fill encountered near the surface is probably pipe backfill material. The fill material was gravelly sand and was medium dense, moist and brown. Natural colluvial sand and gravel was encountered beneath the fill to approximately 30 feet in depth. The colluvial material is angular with cobble and boulder sized rocks which are flatter than the stream deposited alluvial materials encountered in the majority of the other test holes. Also, the colluvial material is occasionally slightly clayey, medium dense to dense, moist and red brown to brown. Natural sand and gravel mixed with cobbles was encountered from 30 feet to 39 feet in depth where crystalline bedrock was encountered.

Joint orientations in the bedrock were measured in outcrops south and upslope from the test holes. Primary joint set strikes were measured at N 112 E and N 125 E with dips of 41 and 40 degrees, respectively. Occasional normal joint sets were also observed. The bedrock is deeply jointed in some areas.

Foundation Recommendations: Based on the field and laboratory investigation and the proposed wall location, the retaining wall may be founded on the undisturbed natural sands and gravels and may be designed for a maximum allowable soil bearing pressure of 4,000 psf. It should be noted that footing excavation near or below the water level in Clear Creek will require excavation dewatering during construction and possibly shoring. Additional design and construction criteria is outlined under "Spread Footing Design and Construction Criteria".

Cantilevered retaining structures at this site can be expected to deflect sufficiently to mobilize the full active earth pressure condition. Therefore, cantilevered structures may be designed for a lateral earth pressure computed on the basis of a coefficient of active earth pressure (Ka) of 0.3073 and a soil unit weight of 120 pcf.

The retaining structure should be designed for appropriate surcharge pressures such as adjacent traffic. An upward sloping backfill surface also increases the earth pressures on retaining structures. The active and lateral earth pressure coefficients given in this section are for a level backfill surface and must be corrected for upward sloping fills. The retaining wall should be constructed at the base of the slope cuts such that the top of the wall meets the existing slope face.

The lateral resistance of retaining wall foundations placed on undisturbed natural soils at the site will be developed by a combination of the sliding resistance of the footing on the foundation materials and the passive pressure against the side of the footing. Sliding friction at the bottom of the footing can be taken as 0.4 times the vertical dead load. Passive pressure against the sides of the footing can be calculated using a coefficient of passive earth pressure (Kp) of 3.2546 and a soil unit weight of 120 pcf.

Retaining wall backfill materials and fill placed against the sides of the footings to resist lateral loads should be placed and compacted according to the criteria presented in "Site Grading."

Item 7:

Median retaining wall just west of I-70/U.S. 6 Interchange

Conclusions: The retaining wall structure may be placed on the existing embankment fill materials. Design and construction recommendations are outlined below.

Proposed Construction: We understand that the proposed retaining wall will measure approximately 1,350 feet in length and will extend from approximately STA 270+00 to STA 283+50. The wall will be constructed to allow widening of the I-70 alignment and maintain the elevation difference between east bound and west bound lanes. The proposed retaining structure centerline will be located approximately on the I-70 Control Line. The proposed wall location is shown on Figure 7. If conditions are significantly different from those described above, we should be notified to reevaluate the recommendations contained in this section.

Existing Conditions: The proposed retaining wall will be located near the center of the existing grassy median between the west and east bound lanes of I-70. Bedrock is exposed in the steep cut face north of I-70 in this area. The proposed retaining wall is located east of known mining activity and the ground penetrating radar study area.

Subsurface Conditions: Test holes 12 and 13 were drilled in the existing median and test hole 14 was drilled in the left shoulder of the eastbound off-ramp to explore subsurface conditions. The test holes were drilled with the Becker Hammer Drill. Test hole locations are shown on Figure 7. Six inches of topsoil was encountered at the surface in test hole 12 and 5 inches of asphalt was encountered at the surface in test hole 14. Embankment fill material was encountered in all three test holes from near the surface to the depths indicated on the exploratory hole logs. The embankment material consisted of slightly silty gravel and sand with cobbles and occasional small boulders, medium dense to dense, moist and brown. Natural sands and gravels underlying the embankment fill materials was occasionally cobbley with small boulders with occasional silty sand lenses, loose to very dense, moist to wet and brown to tan. The upper 3 feet of natural material in test hole 13 was very sandy, loose and wet. Crystalline gneiss bedrock was encountered in test holes 12, 13 and 14 at depths of 6.5, 32 and 23 feet, respectively. Graphic logs of the test holes are shown on Figures 19, 20 and 21.

Foundation Recommendations: Based on the field and laboratory investigation and the proposed wall location, the retaining wall may be founded on the existing embankment fill materials and may be designed for a maximum allowable soil bearing pressure of 3,500 psf. Additional design and construction criteria is outlined under "Spread Footing Design and Construction Criteria".

Cantilevered retaining structures at this site can be expected to deflect sufficiently to mobilize the full active earth pressure condition. Therefore, cantilevered structures may be designed for a lateral earth pressure computed on the basis of a coefficient of active earth pressure (Ka) of 0.3073 and a soil unit weight of 120 pcf.

The retaining structure should be designed for appropriate surcharge pressures such as adjacent traffic. An upward sloping backfill surface also increases the earth pressures on retaining structures. The active and lateral earth pressure coefficients given in this section are for a level backfill surface and must be corrected for upward sloping fills.

The lateral resistance of retaining wall foundations placed on undisturbed natural soils at the site will be developed by a combination of the sliding resistance of the footing on the foundation materials and the passive pressure against the side of the footing. Sliding friction at the bottom of the footing can be taken as 0.4 times the vertical dead load. Passive pressure against the sides of the footing can be calculated using a coefficient of passive earth pressure (Kp) of 3.2546 and a soil unit weight of 120 pcf.

Retaining wall backfill materials and fill placed against the sides of the footings to resist lateral loads should be placed and compacted according to the criteria presented in "Site Grading."

Item 8:

West bound retaining wall just west of I-70/U.S. 6 Interchange and north of I-70.

Conclusions: The retaining wall structure may be placed on bedrock exposed by excavation into the existing rock slope. Design and construction recommendations are outlined below.

Proposed Construction: We understand that the proposed retaining wall will measure approximately 700 feet in length and will extend from approximately STA 267+75 to STA 274+75. The proposed retaining structure centerline will be located approximately 70 left of the I-70 Control Line. The proposed wall location is shown on Figure 7.. If conditions are significantly different from those described above, we should be notified to reevaluate the recommendations contained in this section.

Existing Conditions: Bedrock is exposed in the steep face north of I-70 in the area of the proposed wall. The existing slope angles measured in this area range from 1.8(H):1(V) to 1(H):1(V). The proposed retaining wall is located east of known mining activity and the ground penetrating radar study area.

Subsurface Conditions: Conventional drilling was not conducted in this area due to inaccessibility. The subsurface conditions were evaluated by field observation regional mapping and information from nearby seismic testing. Foliated PreCambrian biotite gneiss exposed in the area is massive to well layered with primary jointing along foliation planes. Primary jointing and layering planes are oriented with strikes from N 90 E to N 120 E with dips ranging from 40 to 56 degrees north. Other random normal and diagonal joint sets were noted. Occasional loose material and debris was also observed. Seismic velocities in bedrock measured just west of the proposed wall location ranged from 8,500fps to 22,000 fps. Seepage from the slope was not observed in the area of proposed wall construction, however, minor seepage from joints in the bedrock has been observed in the north road cut immediately west of the U.S.6/I-70 Interchange.

Foundation Recommendations: Based on the field investigation and the proposed wall location, construction of the retaining wall will require excavation into the existing bedrock outcrop. The proposed wall may be founded on the exposed bedrock materials and may be designed for a maximum allowable soil bearing pressure of 5,000 psf. It should be noted that excavation into the existing slope will probably require blasting. Additional design and construction criteria is outlined under "Spread Footing Design and Construction Criteria".

Cantilevered retaining structures at this site can be expected to deflect sufficiently to mobilize the full active earth pressure condition. Therefore, cantilevered structures may be designed for a lateral earth pressure computed on the basis of a coefficient of active earth pressure (Ka) of 0.3073 and a soil unit weight of 120 pcf.

The retaining structure should be designed for appropriate surcharge pressures such as adjacent traffic. An upward sloping backfill surface also increases the earth pressures on retaining structures. The active and lateral earth pressure coefficients given in this section are for a level backfill surface and must be corrected for upward sloping fills. The retaining wall should be constructed at the base of the slope cuts such that the top of the wall meets the existing slope face.

The lateral resistance of retaining wall foundations placed on undisturbed natural soils at the site will be developed by a combination of the sliding resistance of the footing on the foundation materials and the passive pressure against the side of the footing. Sliding friction at the bottom of the footing can be taken as 0.4 times the vertical dead load. Passive pressure against the sides of the footing can be calculated using a coefficient of passive earth pressure (Kp) of 3.2546 and a soil unit weight of 120 pcf.

Retaining wall backfill materials and fill placed against the sides of the footings to resist lateral loads should be placed and compacted according to the criteria presented in "Site Grading."

Item 9:

Retaining wall, north side of I-70, west of west bound on ramp at Hidden Valley Interchange.

Conclusions: The retaining wall structure may be placed on spread footings placed on the natural silty gravel and sand with cobbles. Design and construction recommendations are outlined below.

Proposed Construction: We understand that the proposed retaining wall will measure approximately 125 feet in length and will extend from STA 211+50 to STA 212+75. The proposed retaining structure centerline will be located approximately 60 feet left of the I-70 Control Line. The proposed retaining wall location is shown in Figure 2. If conditions are significantly different from those described above, we should be notified to reevaluate the recommendations contained in this section.

Existing Conditions: At the time of our field investigation and test hole drilling, a small existing retaining wall was located north of the highway and the area north of the proposed wall was undergoing private site grading. Recently it was observed that the original ground surface north of the proposed wall had been lowered considerably since the initial investigation and drilling.

Subsurface Conditions: Test hole 15 was drilled at the location shown of Figure 2 to explore subsurface conditions. The test hole was advanced through the existing pavement section and into the underlying fill material and natural sands and gravels with the Becker hammer drill. A graphic log of test hole 15 is shown on Figure 22.

A pavement section consisting of 5 inches of asphalt was encountered at the surface in test hole 15. The fill material encountered beneath the asphalt consisted of silty sand and gravel with cobbles and occasional boulders, fine to coarse grained, medium dense to dense, dry to moist and brown to tan. Natural sand and gravel with mixed cobbles and boulders was encountered at a depth of 4 feet in the test hole. The natural material was also very dense and dry to slightly moist. The test hole was advanced to a depth of 9 feet, where very dense cobbles and boulders mixed with sand were encountered. Due to the recent site grading, however, The proposed retaining wall foundation may now extend below this level. The recommendations below are based on the subsurface conditions encountered in the test hole. Additional drilling or investigation may be required if the proposed wall geometry is significantly changed.

Ground Penetrating Radar Study: Based on the preliminary results of the ground penetrating radar study and our review of information collected by the Department of Highways, the proposed retaining wall falls within a zone of disturbed material as a result of early mining activity. The disturbed zone has a potential of differential settlement and subsequent structure distress. The foundation recommendations below are based on the prior remedial actions described under "Ground Penetrating Radar" in this report.

Foundation Recommendations: Based on the field and laboratory investigation, the retaining wall at this location may be placed on the natural sands and gravels or properly compacted fill and may be designed for a maximum allowable soil bearing pressure of 3,500 psf. Additional design and construction criteria is outlined under "Spread Footing Design and Construction Criteria".

Cantilevered retaining structures at this site can be expected to deflect sufficiently to mobilize the full active earth pressure condition. Therefore, cantilevered structures may be designed for a lateral earth pressure computed on the basis of a coefficient of active earth pressure (Ka) of 0.3073 and a soil unit weight of 120 pcf.

The retaining structure should be designed for appropriate surcharge pressures such as adjacent traffic. An upward sloping backfill surface also increases the earth pressures on retaining structures. The active and lateral earth pressure coefficients given in this section are for a level backfill surface and must be corrected for upward sloping fills.

The lateral resistance of retaining wall foundations placed on undisturbed natural soils at the site will be developed by a combination of the sliding resistance of the footing on the foundation materials and the passive pressure against the side of the footing. Sliding friction at the bottom of the footing can be taken as 0.4 times the vertical dead load. Passive pressure against the sides of the footing can be calculated using a coefficient of passive earth pressure (Kp) of 3.2546 and a soil unit weight of 120 pcf.

Foundation wall backfill materials and fill placed against the sides of the footings to resist lateral loads should be placed and compacted according to the criteria presented in "Site Grading."

SPREAD FOOTING DESIGN AND CONSTRUCTION CRITERIA

The design and construction criteria presented below should be observed for a spread footing foundation system. The construction details should be considered when preparing project documents for the proposed retaining walls.

(1) Footings placed on undisturbed natural soils or properly compacted fill may be designed for the maximum allowable soil bearing pressure described under each item above.

(2) Spread footings placed on granular soils should have a minimum footing dimension of 16 inches.

(3) Exterior footings should be provided with adequate soil cover above their bearing elevation for frost protection.

(5) Continuous foundation walls should be reinforced top and bottom to span an unsupported length of at least 10 feet.

(6) Based on experience, we estimate total settlement for footings designed and constructed as discussed in this section will be approximately one inch.

(7) Areas of loose or soft material or encountered within the foundation excavation should be removed and replaced with nonexpansive granular fill material compacted to 95% of the maximum modified Proctor density within 2 % of the optimum moisture content. New fill should extend down from the edges of the footings at a 1 (horizontal) to 1 (vertical) slope. As an alternate, the soft or loose material may be excavated and footings extended to adequate natural bearing material.

When the proposed wall foundation elevation lies on the existing embankment fill material, the foundation subgrade should be scarified to a depth of 8 inches and recompacted as described above.

(8) All granular footing areas should be compacted with a vibratory plate compactor prior to placement of concrete.

(9) Care should be taken when excavating the foundations to avoid disturbing the supporting materials. Hand excavation or careful backhoe soil removal, may be required in excavating the last few inches.

(10) The natural soils may pump or deform excessively under the wheel loads of heavy construction vehicles as the excavations approach footing levels. Construction equipment should be as light as possible to avoid this difficulty. The use of track-mounted vehicles is recommended since they exert lower contact pressures. The movement of vehicles over proposed foundation areas should be restricted.

(12) The proposed foundation elevations for the channel change retaining and possibly portions of the Item 3 retaining wall appear to be near or slightly above water level. Therefore, it may be necessary to dewater footing excavations during construction. Dewatering should not be conducted by pumping from inside footing limits. This may decrease the supporting capacity of the soils. Shoring may be required during construction of the walls and, if used, should conform to all applicable OSHA safety standards.

(13) A representative of the soil engineer should observe all footing excavations prior to concrete placement.

RETAINING WALL BACKFILL RECOMMENDATIONS

We recommend on-site granular soils for backfilling foundation walls and retaining structures because their use results in lower lateral earth pressures. Granular material should be placed to within one foot of the ground surface and to a minimum distance beyond the walls equal to one half the height of the fill. The granular soil behind foundation and retaining walls should be sloped from the base of the wall at an angle of at least 45 degrees from the vertical. The upper one foot of the wall backfill should be a relatively impervious on-site or imported soil or a pavement structure to prevent surface water infiltration into the backfill.

Backfill should be carefully placed in uniform lifts and compacted to at least 95% of the maximum modified Proctor density, within 2% of optimum moisture content. Care should be taken not to overcompact the backfill since this could cause excessive lateral pressure on the walls. Some settlement of deep foundation wall backfills will occur even if the material is placed correctly.

Compacted fill placed against the sides of footings to resist lateral loads should be a nonexpansive, fine grained material approved by the soil engineer. Fill should be placed and compacted to at least 95% of the maximum modified Proctor density within 2% of the optimum moisture content.

SITE GRADING

Fill Areas: In fill areas in which the embankments will be less than 4 feet in height, all of the topsoil and organic material must be removed and the subgrade materials scarified to a depth of 6 inches and recompacted in accordance with the Colorado Department of Highways (CDOH) Standard Specifications for Road and Bridge Construction. In embankments greater than 4 feet in height, the topsoil and organic materials do not need to be removed, if they are stable and unyielding. Prior to placing any fill, the topsoil and organic materials shall be scarified and recompacted. Whenever the existing roadway surface lies within 3 feet of the proposed subgrade, the existing pavement sections should be ripped and crushed into pieces less than 6 inches in size and recompacted. The existing pavement section may also be removed and the exposed subgrade materials scarified and recompacted.

When embankment fill is to be placed and compacted on hillsides, or when new embankments are to be compacted against existing embankments, or when the embankment is built part width at a time, existing slopes greater than 4:1 when measured at right angles to the roadway shall be continuously benched over those areas where it is required as the work is brought up in layers. Benching shall be well keyed and where practical, a minimum of eight feet wide.

Embankments and backfill areas shall contain no muck, frozen material, roots, sod or other deleterious materials. Rocks, broken pavement, or other solid , bulky materials shall not be placed in embankment areas where piling or drilled piers are to be constructed or placed.

Embankment fill consisting of soil material shall be placed in horizontal layers not exceeding 8 inches (loose measurement) and shall be compacted to at least 95% of the maximum modified Proctor density in accordance with ASTM D1557 or to at least 100% of the maximum standard Proctor in accordance with ASTM D698 and the CDOH specifications. When excavated materials contain more than 25 percent of rock larger than 6 inches in diameter and cannot be placed in layers of the thickness prescribed above without crushing or pulverizing, such materials may be placed on the embankment in layers not exceeding in thickness the approximate average size of the larger rocks, but not greater than 3 feet. Even though the thickness of the layers is limited as provided above, the placing of individual rocks and boulders greater than 3 feet may be permitted provided that when placed, they do not exceed 5 feet in height and provided they are carefully distributed, with the interstices filled with finer material to form a dense and compacted mass. The rock fill areas will be proof rolled with a rubber tired roller to ensure that embankment fill is uniform and well compacted. The density requirements given above for the soil materials will not apply to the rock fill areas.

Embankments consisting predominantly of rock larger than 8 inches in greatest dimension, shall not be constructed above an elevation two feet below the finished subgrade. Where embankments encroach on stream channels, the largest available rock from the excavation shall be placed along the toes of the slopes to protect the embankments against erosion from water action.

Rock shall be excavated to a depth of 6 inches below subgrade within the limits of the roadbed, and the excavation backfilled with material approved by the engineer. Materials other than solid rock shall be scarified to a depth of 6 inches and compacted to the required density and moisture content.

The surficial soils and deeply weathered bedrock materials which have no rocks greater than 8 inches in size and are excavated from the cut areas, shall be conserved and used in the upper two feet of fill in the embankments and/or used as fines for choking off of the voids in the rock fill areas.

Except in solid rock, the tops of all slopes shall be rounded and all loose materials shall be removed to reduce potential rock and debris fall over the face of the cut slopes.

Cut Areas: At the time of preparation of this report, the two major cuts into existing rock slopes will be temporary and will occur during construction of the retaining walls described under Items 6 and 8 of this report. If, during final design, permanent cuts into existing slopes are planned we should be notified to evaluate the stability of the proposed cuts. Also during final design, when final construction geometries are known, rockfall hazard from the new slopes should be analyzed using the Colorado Rockfall Simulation Program. The program is used to generate statistical data on probable rockfall events, aiding in design of side ditches and other measures to minimize the hazard.

Fill Slopes: Fills within the highway embankment up to 25 feet in height can be used if the fill slopes do not exceed 1.5 horizontal to 1 vertical and fills are properly compacted and drained. The fill material should be granular in nature in order to maintain these slopes. Embankment fill should be placed as described under "Site Grading".

Good surface drainage drainage should be provided around all permanent cuts to direct runoff away from the cut face. Cut slopes and other stripped areas should be protected against erosion by revegetation or other methods.

GROUND PENETRATING RADAR STUDY

A ground penetrating radar survey was performed in the area of the Hidden Valley Interchange to locate possible void areas remaining from hydraulic placer mining operations which took place in the late 1800's to early 1900's. The radar survey was conducted by Geo-Recovery Systems, Inc. Control points were marked for each survey line at 50 foot intervals along both the west bound and east bound lanes of I-70 from approximately STA 205 to STA 230. In addition, control points were marked on the on/off ramps on both sides of the interchange and on the south frontage road over the same interval. Antenna of differing wavelengths were used to identify areas of risk for future collapse. A preliminary report and survey map prepared by Geo-Recovery Systems, Inc. is submitted for review in Appendix B of this report. The final ground penetrating radar report and conclusions will be submitted when all analysis is complete.

Three of the potential risk areas targeted by the radar survey were drilled utilizing a 6-inch O.D. Odex drill system using high pressure air as the circulating fluid. Using this method a continuous stream of cuttings up to approximately one inch in diameter was circulated to the surface. In addition, split spoon samples were taken at various locations during drilling. The sampling method is described under the section "Field Investigation" of this report. The test hole locations were determined by correlating radar survey data with the marked control points at the project site. Test Holes O-1, O-2 and O-3 were drilled at the locations shown on Figure 2. Graphic logs of the test holes are shown of Figure 24.

Drill data from the previous investigation performed by the Department of Highways dated August 12, 1981 was also reviewed. The Department of Highways investigation was initiated following several subsidences in the roadway at the study area. Approximately 90 test holes were drilled by the state using a 3-5/8-inch tricone bit with water as the drilling fluid.

<u>Preliminary Results</u>: Based on our review of all data available in the study area at this time, we have, in consultation with Geo-Recovery Systems, Inc., defined an area where the radar data indicates that the subsurface materials have been disturbed, most probably due to the early mining operations. In addition, 4 areas within this boundary were further defined as areas where the mining activity may have been concentrated, as in a drift, and where potential voids below the surface may exist. The preliminary results of the study can be summarized as follows.

(1) Voids were not encountered during dry drilling of three of the potential targets.

(2) Relatively loose, finer grained sand and gravel was encountered above the bedrock in test holes O-1 and O-2.

(3) Areas logged as 'void' during the state drilling program ranged from 0 to 85 percent filled with loose material.

(4) Additional drilling in the area will probably not further define the zone of disturbance.

(5) Differential settlement and subsequent structure distress in the zone is possible with additional settlement in the loose, disturbed or potential void areas.

(6) Bedrock elevations in the test holes drilled in the study area averages 7296 feet.

(7) To effectively eliminate the risk of future structure damage due to differential settlement in the area, the zone of disturbance should be overexcavated to a depth of at least 15 feet from the existing travel lane elevation. Following overexcavation, zones identified by the radar survey to be of highest potential risk should be further excavated with a track hoe. The depth and extent of necessary overexcavation will become evident in the field as overexcavation continues. The excavated materials should be replaced and compacted in accordance with the procedures outlined in "Site Grading" of this report. The soils engineer should observe the removal and replacement operation.

(8) We suggest temporary excavation slopes in the soils be constructed no steeper than 1.5 horizontal to 1 vertical. Some minor surface sloughing may occur adjacent to the slope faces at these angles; however, large-scale slope failures are unlikely. We should be notified to observe excavation as it proceeds. In our opinion, excavation of the overburden soils should be possible with heavy duty conventional excavation equipment.

No formal slope stability analyses were performed to evaluate the slopes recommended above. Published literature and our experience with similar cuts indicate the recommended slopes should have adequate factors of safety. If a detailed slope stability analyses is required, we should be notified.

SEISMIC SURVEY

A seismic survey was conducted on the existing slope in an area north of I-70 from approximately STA 260 to STA 264. Initial proposals called for excavation into this slope to allow for widening of the alignment. We understand that the alignment in this area will now be widened by moving the Clear Creek channel to the south instead of cutting into the slope to the north. The channel change will be accomplished by constructing two retaining walls (Items 5 and 6). Seismic data was also collected south of Clear Creek to obtain preliminary information on the condition of the existing slopes. We understand a bike path or access road is being considered that may be located south of Clear Creek.

The seismic survey was conducted by representatives of Ground Engineering Consultants, Inc. using a portable Bison 1570-C single channel seismograph utilizing a sledge hammer and plate as the source. The seismic information was collected in areas not accessible to conventional drilling methods. Seismic traverse locations and field data are presented in Appendix A of this report. If structures are to be located in these areas, we should be notified to analyze the seismic data as it relates to the proposed structures.

EROSION POTENTIAL

The erosion potential of the Clear Creek channel in the vicinity of the proposed bridge and retaining wall construction was evaluated by field observation and laboratory gradation tests of samples obtained from the soil borings and channel bank. Grain size distributions of the materials sampled are presented on Figures through . Results of the soil gradation tests and our visual observation of the existing channel bank material confirm the variability of the soils at the site. Although cobbles and some boulders were observed in the channel, relatively clean, finer grained materials were also observed in the channel banks. It is our opinion that additional scour protection is needed. The type of protection used should be based on the calculated stream velocity.

PAVEMENT DESIGN

A Subsurface Investigation - Pavement Design, Evaluation and Rehabilitation report under GEC Job No. 87-203 was submitted on September 17, 1987 for the subject project. An addendum to that report was submitted on October 13, 1988. The addendum included updated traffic design information and considered newly adopted pavement design procedures from the American Association of State Highway and Transportation Officials (AASHTO) Guide for Design of Pavement Structures (1986). As a result, recommended pavement sections in the original pavement evaluation and the addendum differ slightly. The recommendations set forth in the addendum letter should be adhered to. Both the addendum and the original evaluation are submitted in Appendix C of this report for review.

LIMITATIONS

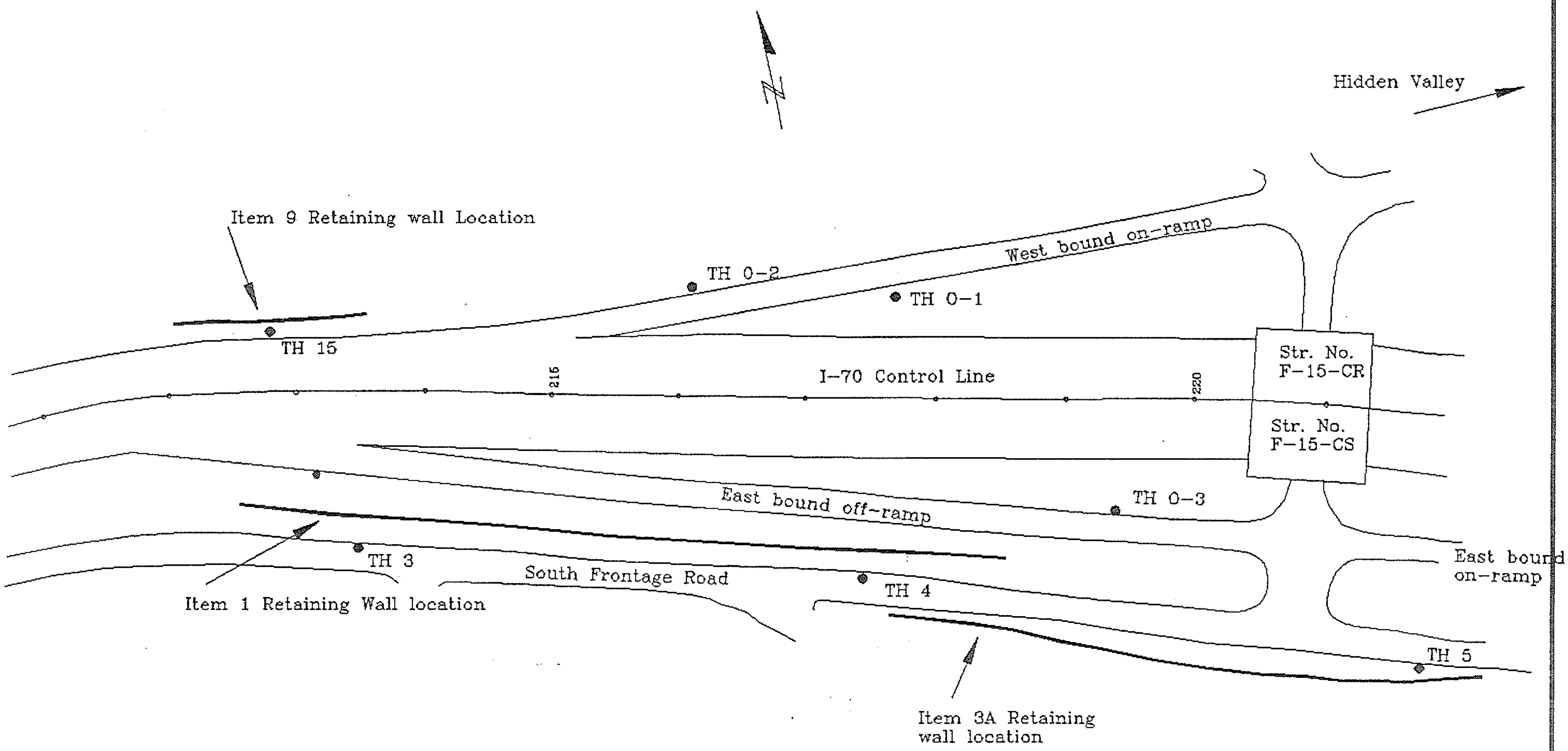
This report has been prepared in accordance with generally accepted soil and foundation engineering practices in this area for use by the client for design purposes. The conclusions and recommendations submitted in this report are based upon the data obtained from the exploratory holes drilled at the locations indicated on the exploratory hole plan and previous investigations performed by Ground Engineering Consultants, Inc.. The nature and extent of variations between the exploratory holes may not become evident until excavation is performed. If during construction, soil, rock and groundwater conditions appear to be different from those described herein, this office should be advised at once so that reevaluation of the recommendations may be made. We recommend on-site observation of excavations and foundation bearing strata by a soil engineer.

Sincerely,

GROUND ENGINEERING CONSULTANTS, INC.

James B. Kowalsky

Reviewed by Richard J. Suedkamp, P.E.



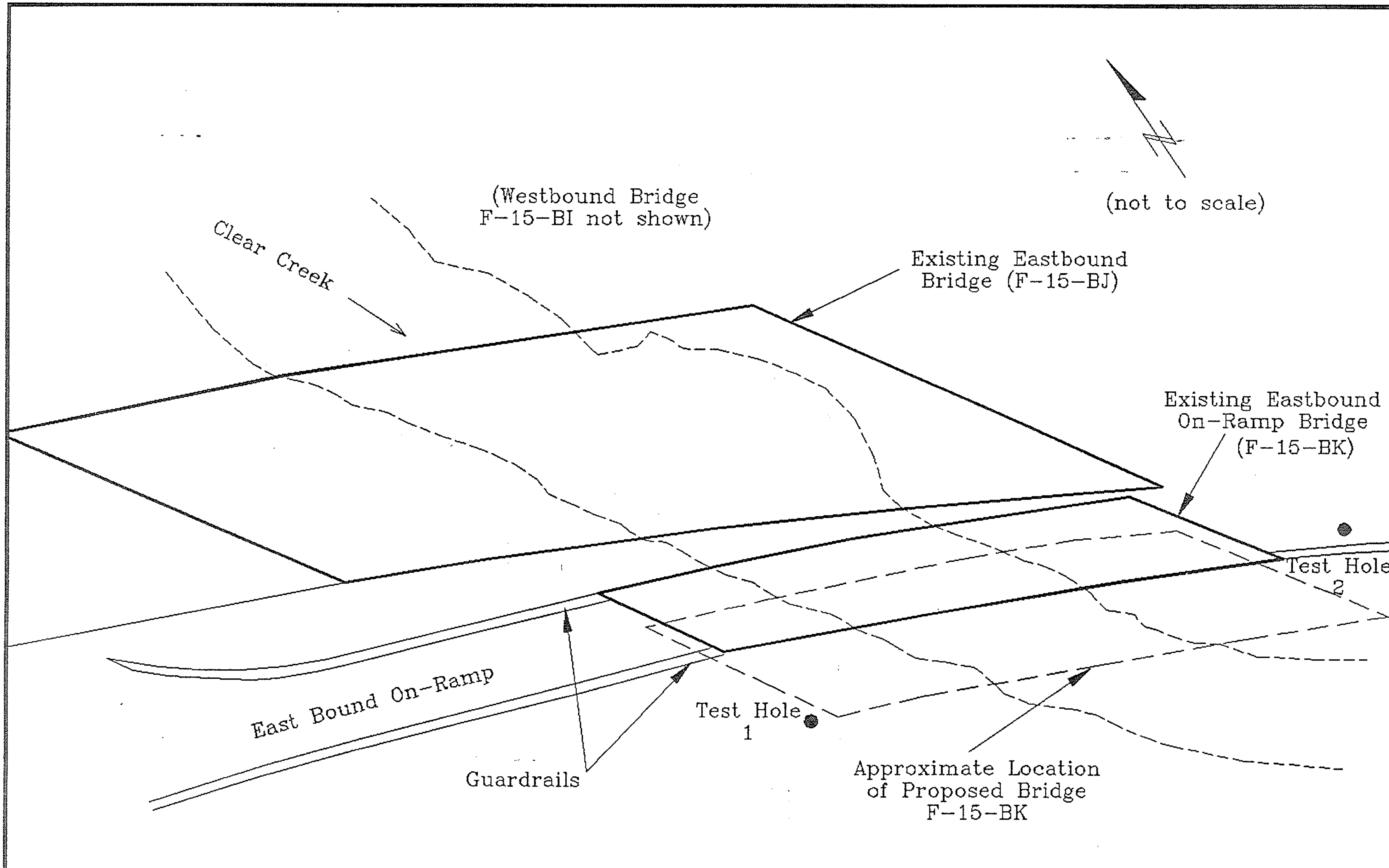
(Not to Scale)

Radar Survey Test Hole locations (0-1, 0-2 and 0-3) also shown.

GROUND ENGINEERING CONSULTANTS, INC.

Test Hole Locations
Items 1, 3A, 9

Job No. 87-203A Figure 2

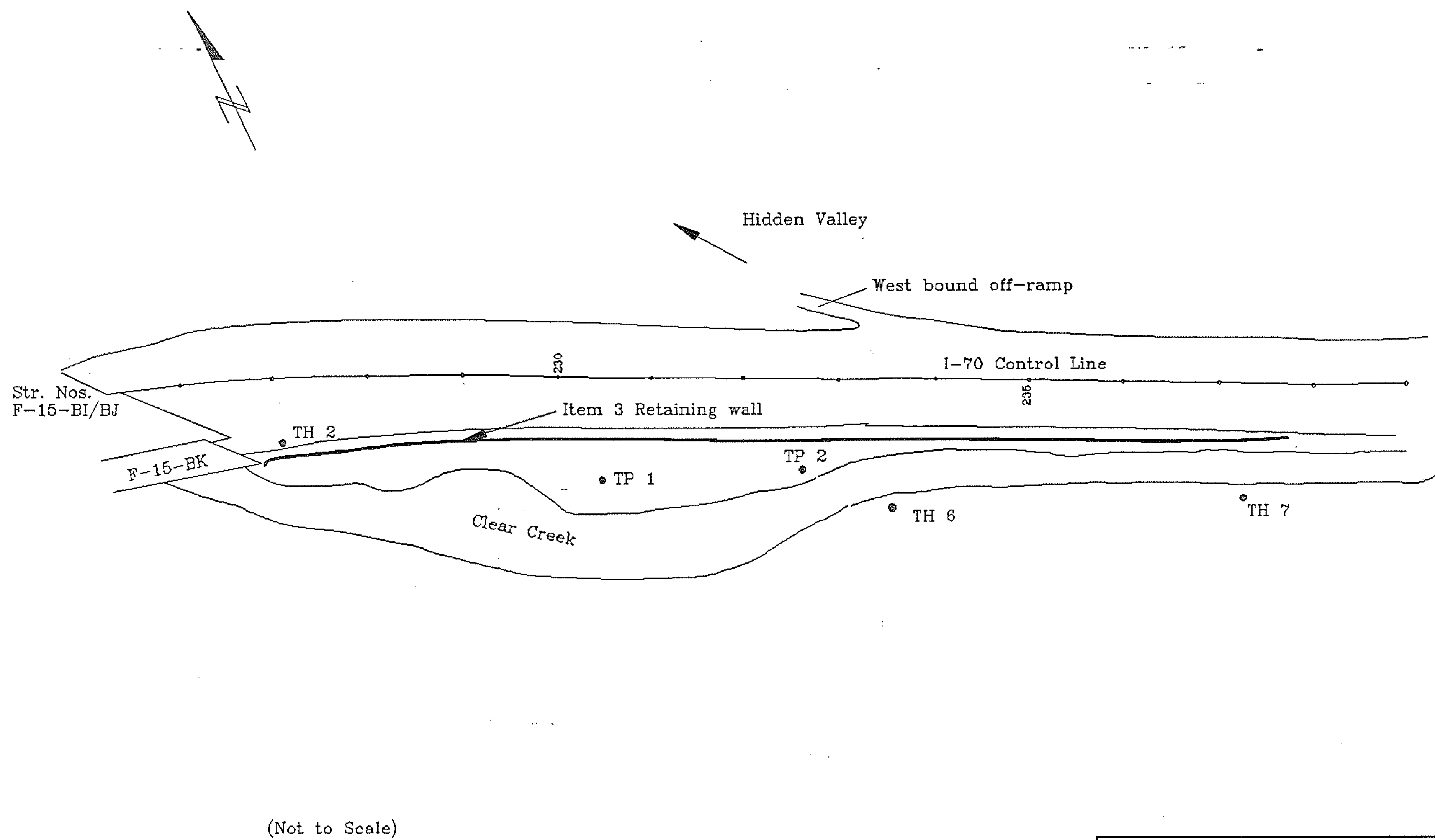


Item 2

GROUND ENGINEERING CONSULTANTS, INC.

Test Hole Locations
Structure F-15-BK

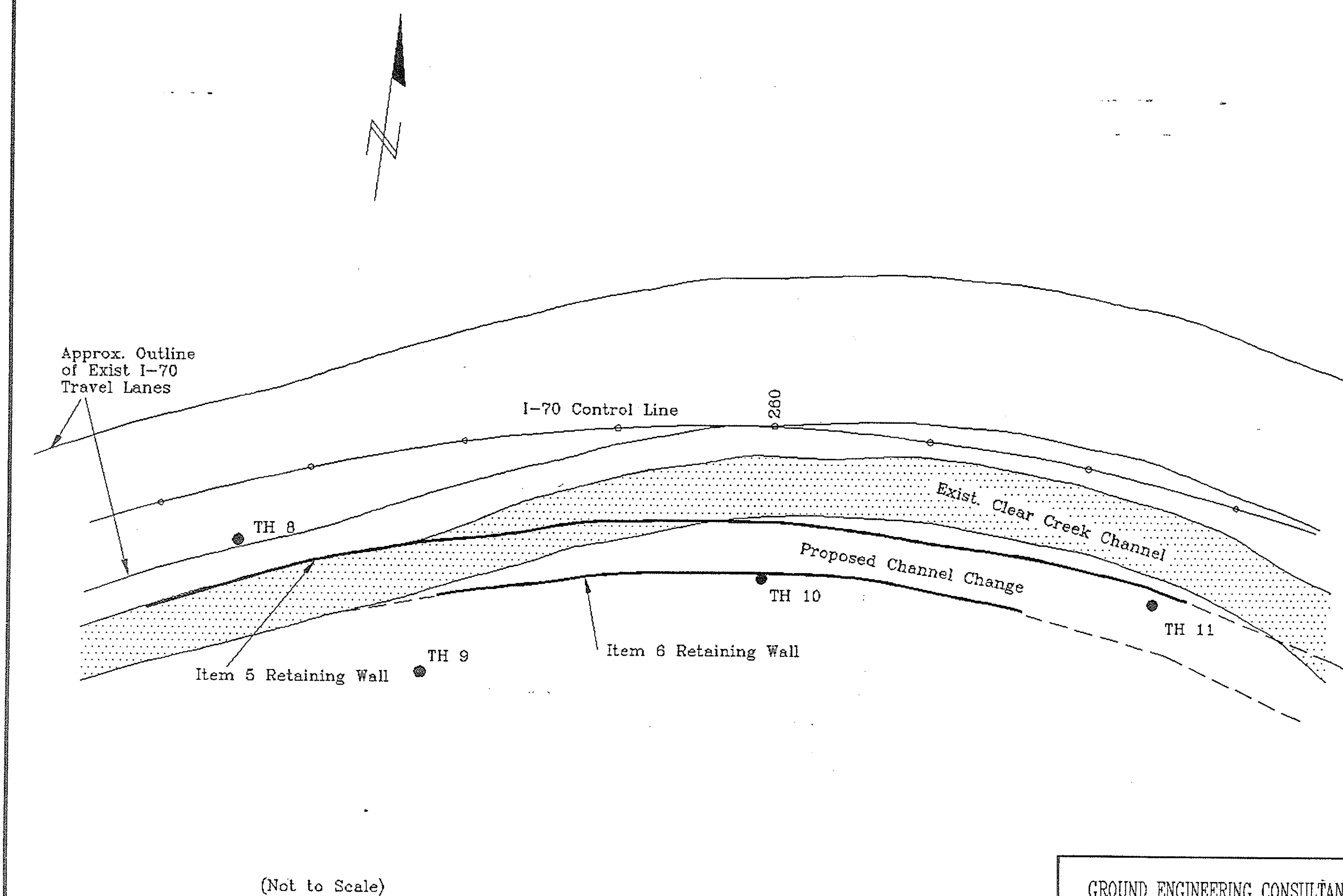
Job No. 87-203B Figure 3



GROUND ENGINEERING CONSULTANTS, INC.

Test Hole Locations
Item 3

Job No. 87-203A Figure 4

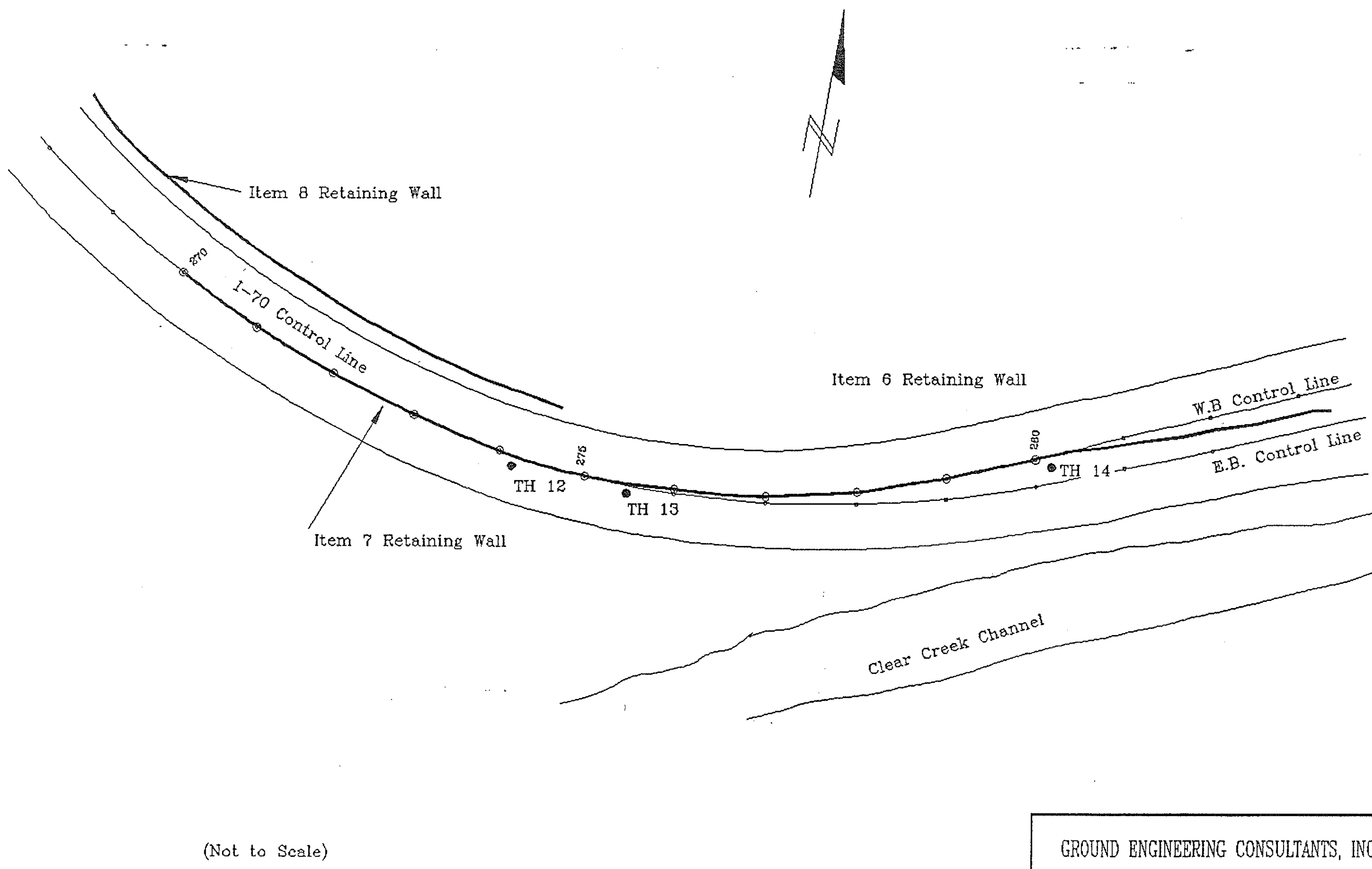


GROUND ENGINEERING CONSULTANTS, INC.

Test Hole Locations

Items 4 and 5

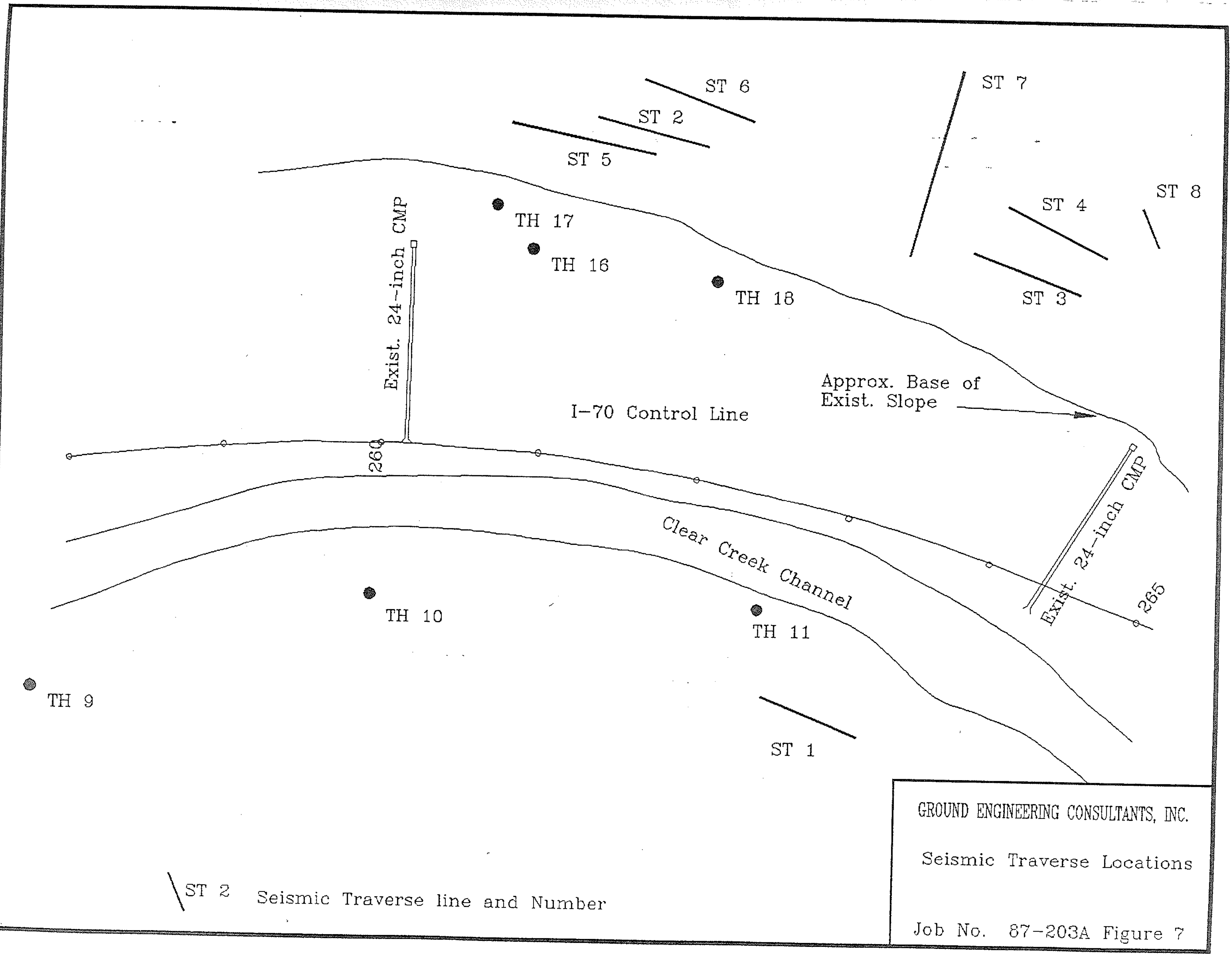
Job No. 87-203A Figure 5



GROUND ENGINEERING CONSULTANTS, INC.

Test Hole Locations
Items 6 and 7

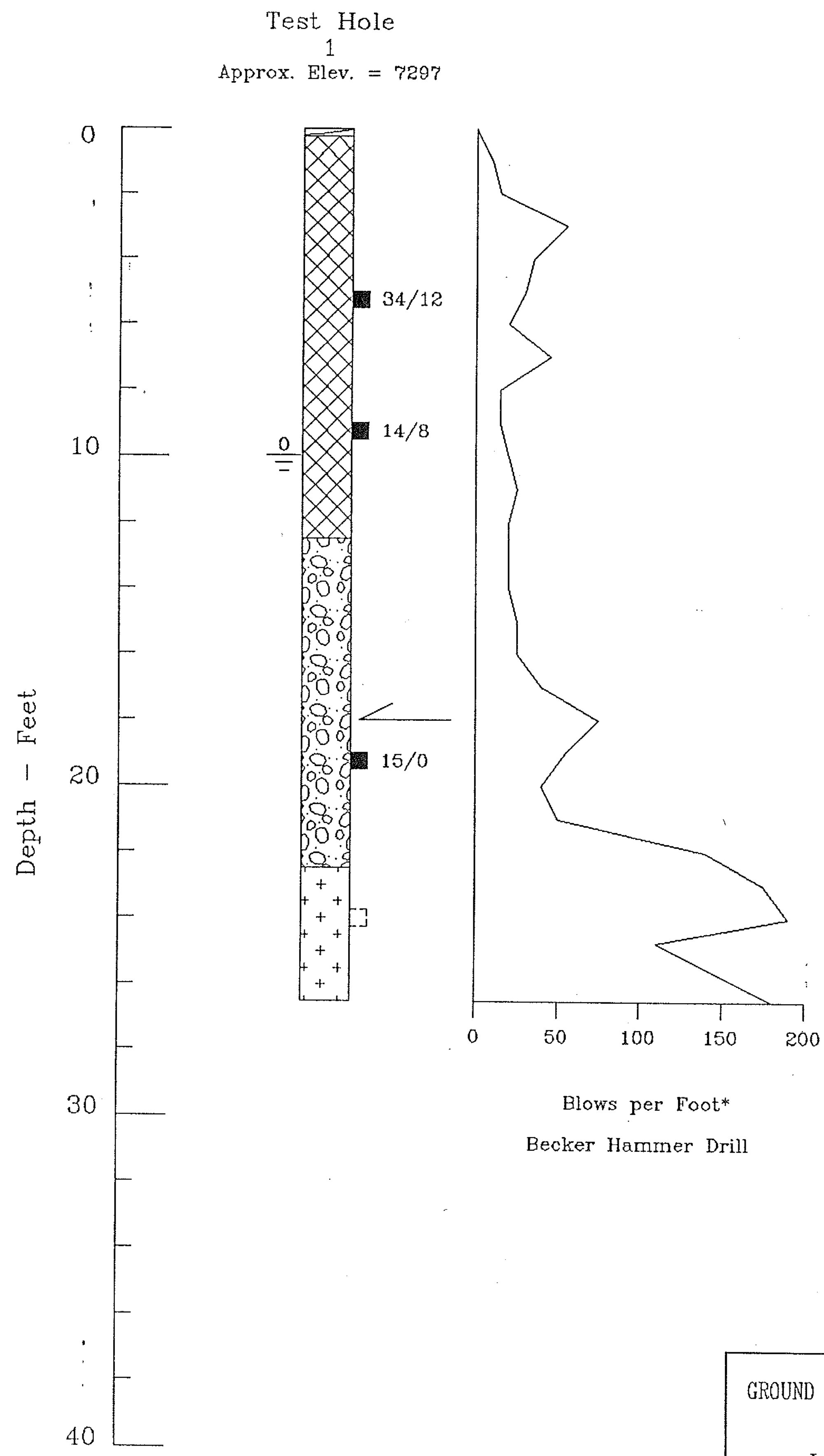
Job No. 87-203A Figure 6



GROUND ENGINEERING CONSULTANTS, INC.

Seismic Traverse Locations

Job No. 87-203A Figure 7

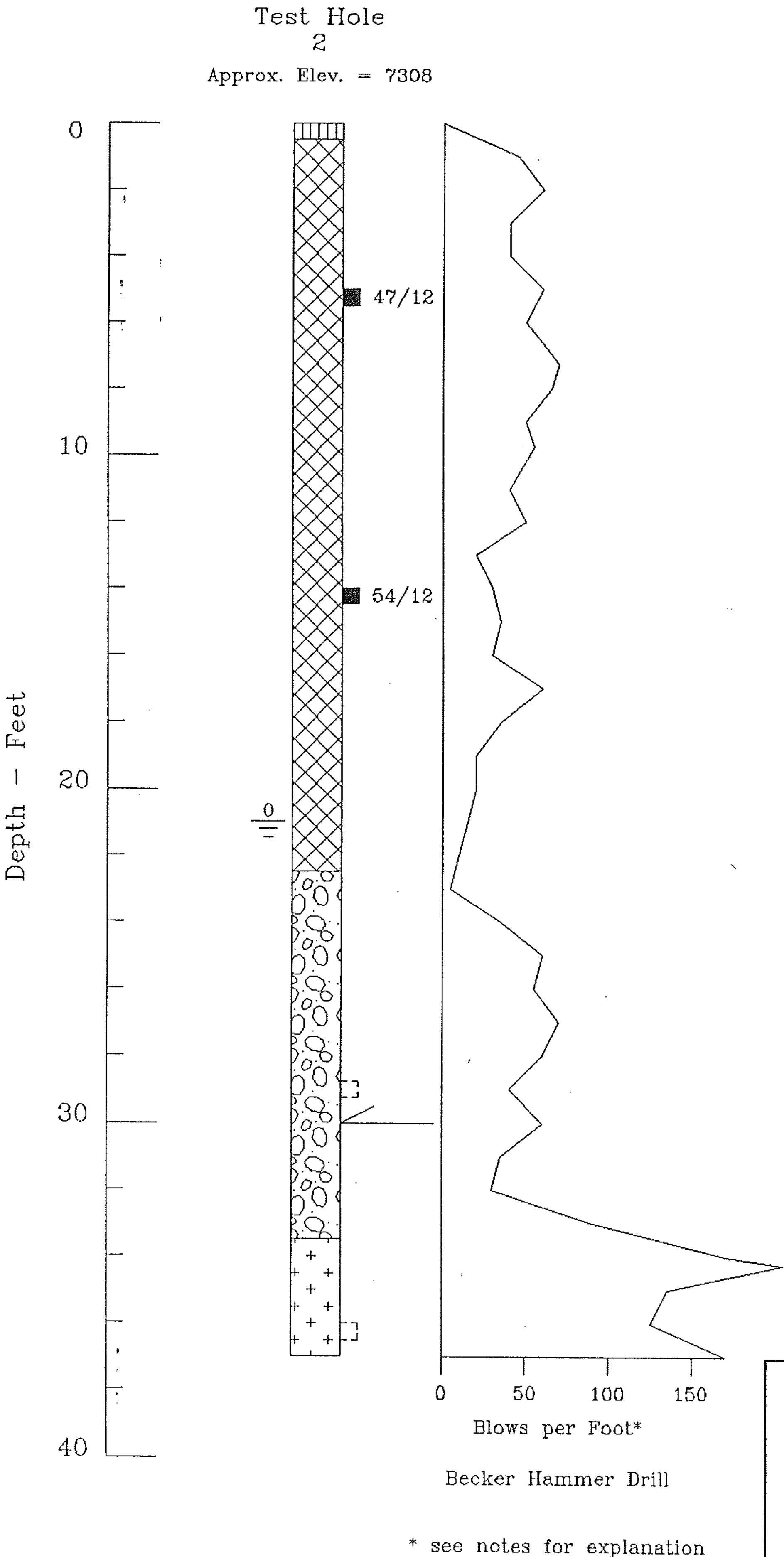


* see notes for explanation

GROUND ENGINEERING CONSULTANTS, INC.

Logs of Test Holes

Job No. 87-203A Figure 8

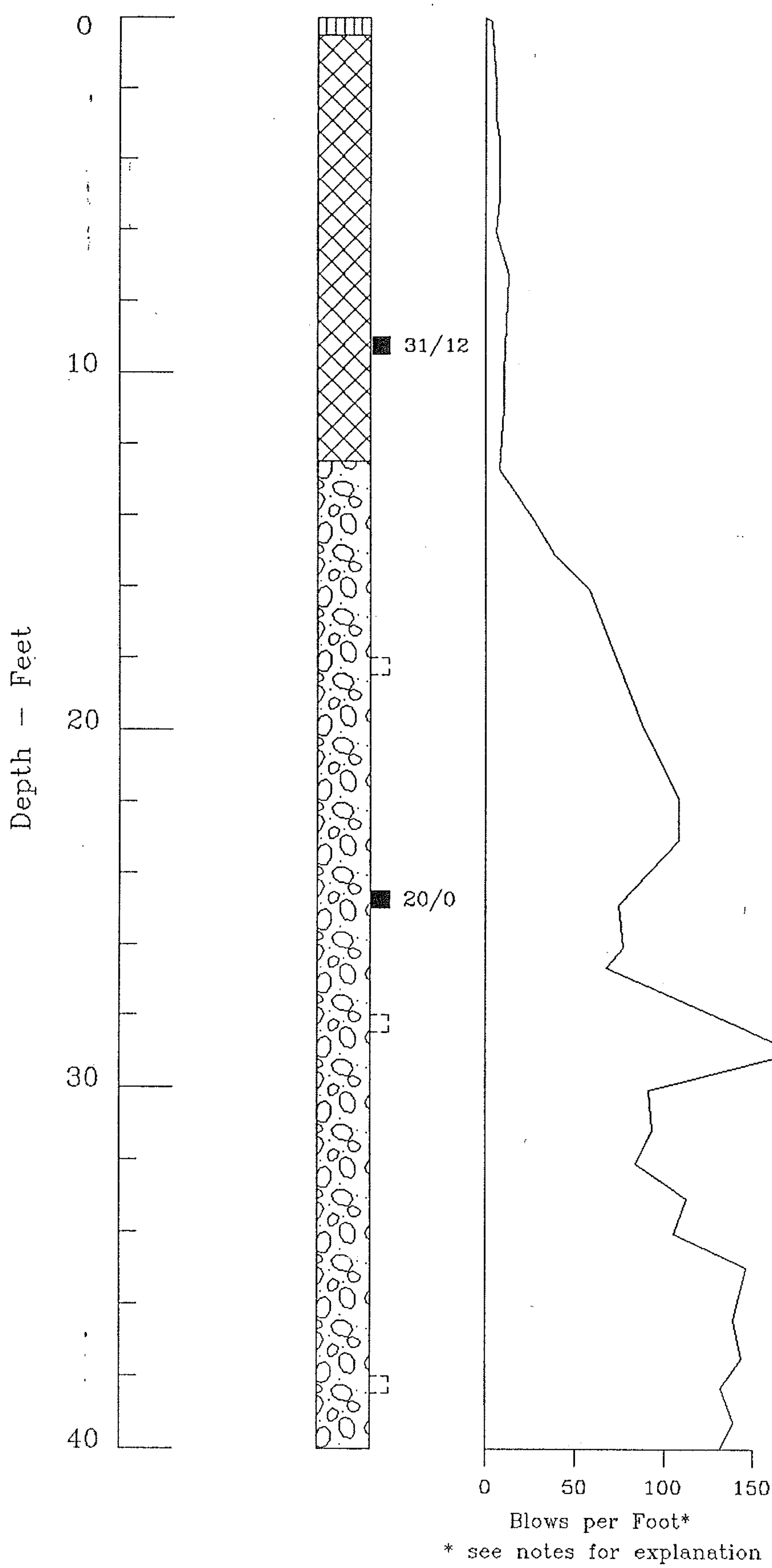


GROUND ENGINEERING CONSULTANTS, INC.

Logs of Test Holes

Job No. 87-203A Figure 9

Test Hole
3
Approx. Elev. = 7349.5

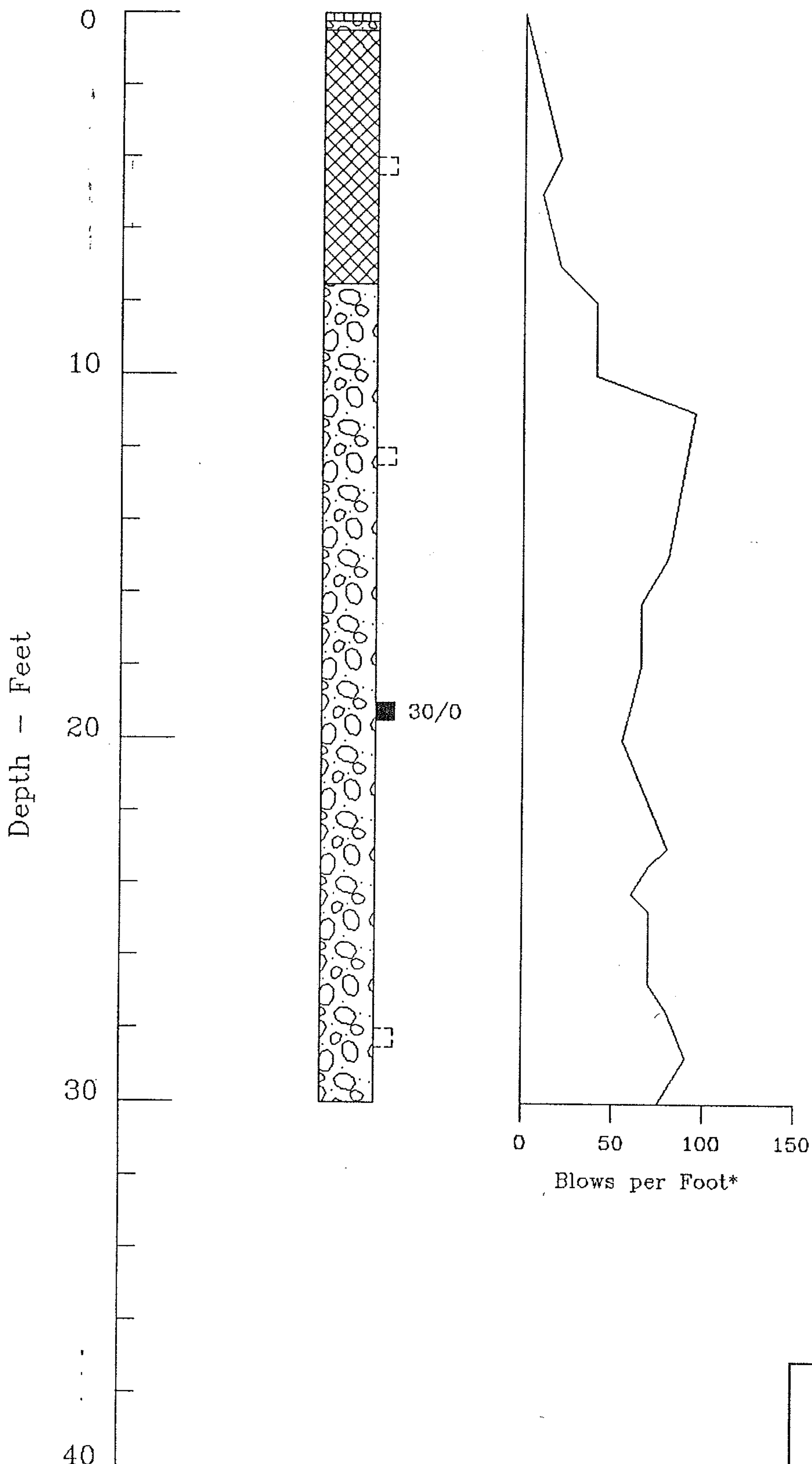


GROUND ENGINEERING CONSULTANTS, INC.

Logs of Test Holes

Job No. 87-203A Figure 10

Test Hole
4
Approx. Elev. = 7331

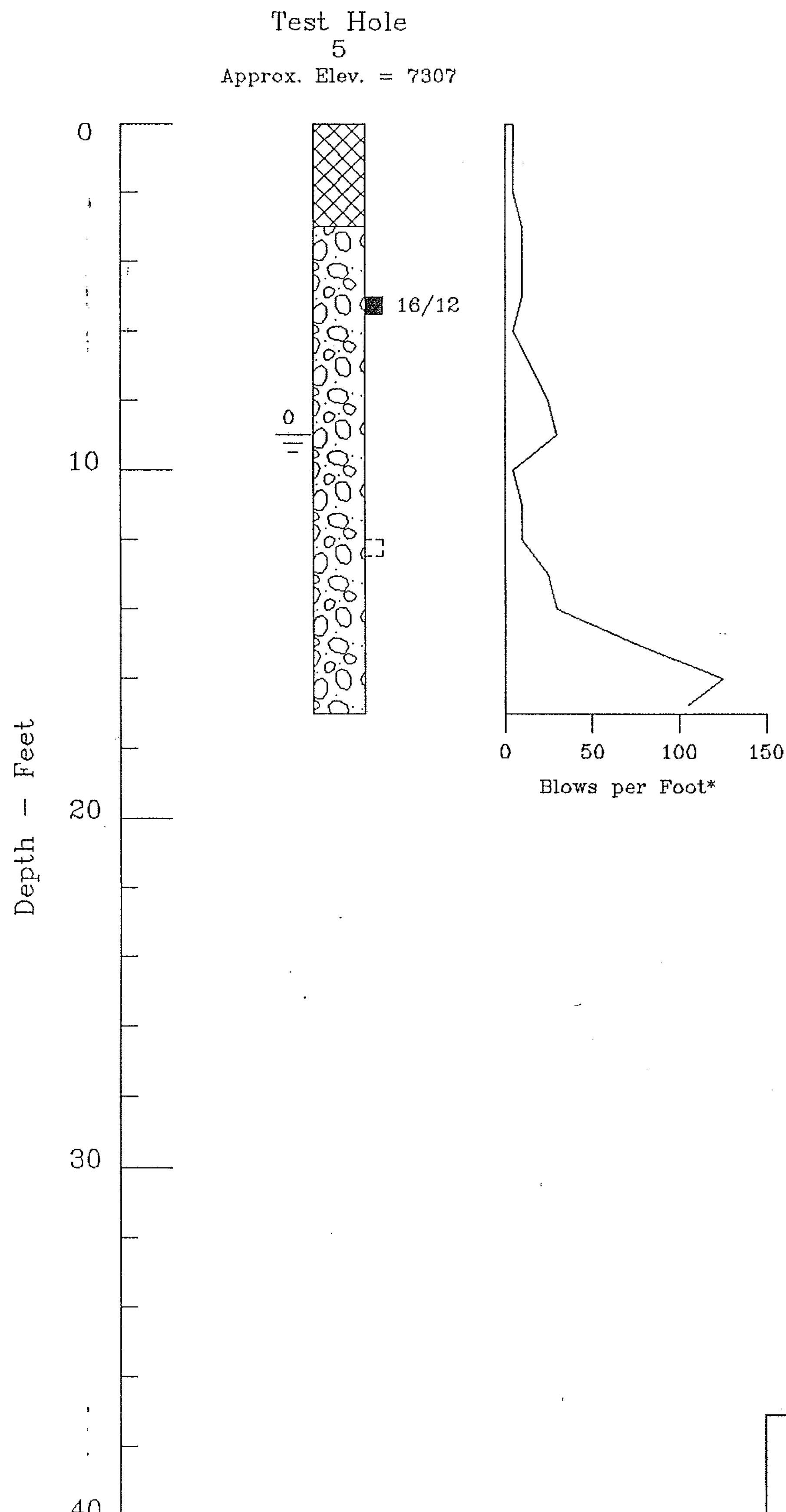


* see notes for explanation

GROUND ENGINEERING CONSULTANTS, INC.

Logs of Test Holes

Job No. 87-203A Figure 11

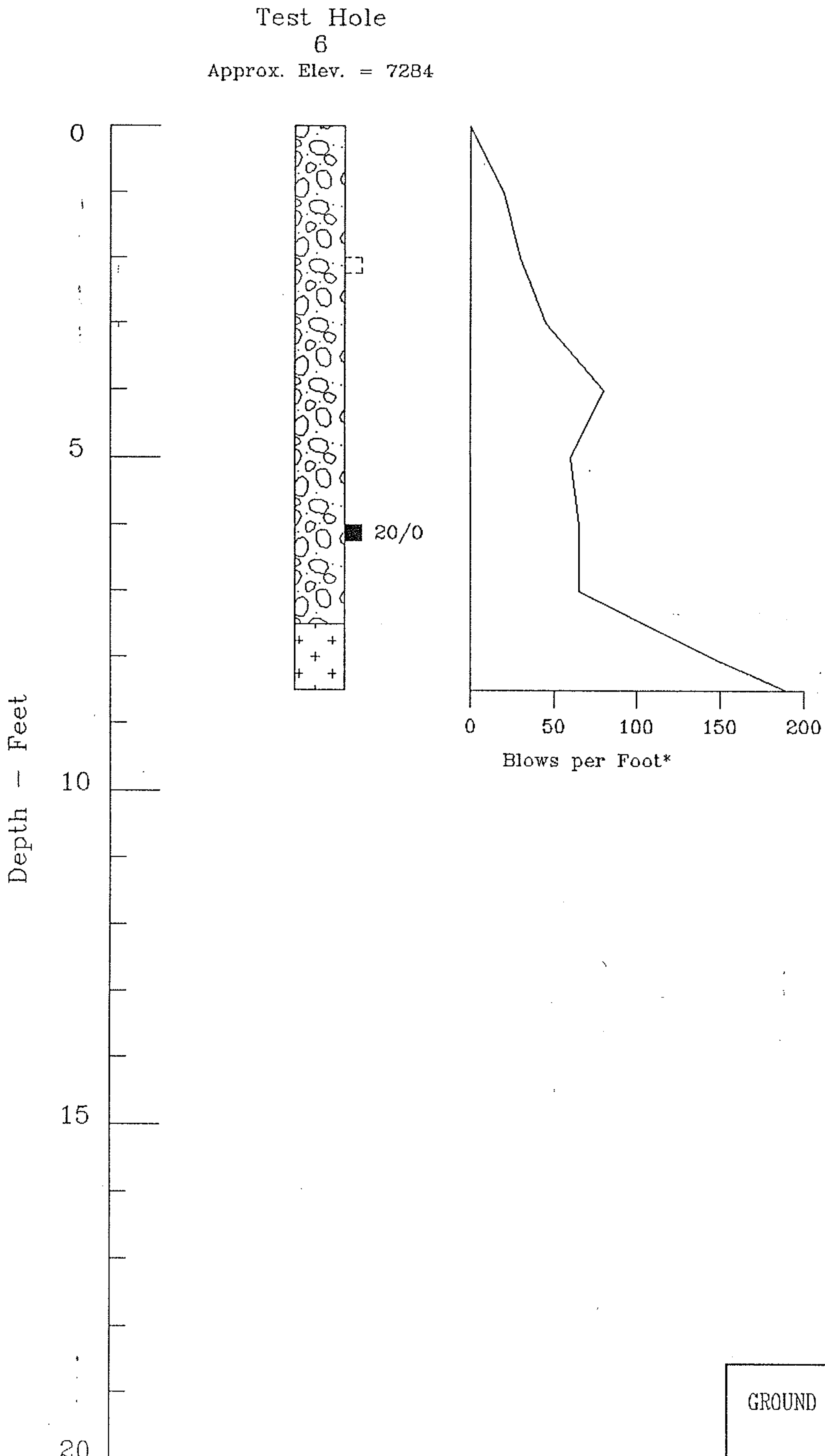


* see notes for explanation

GROUND ENGINEERING CONSULTANTS, INC.

Logs of Test Holes

Job No. 87-203A Figure 12

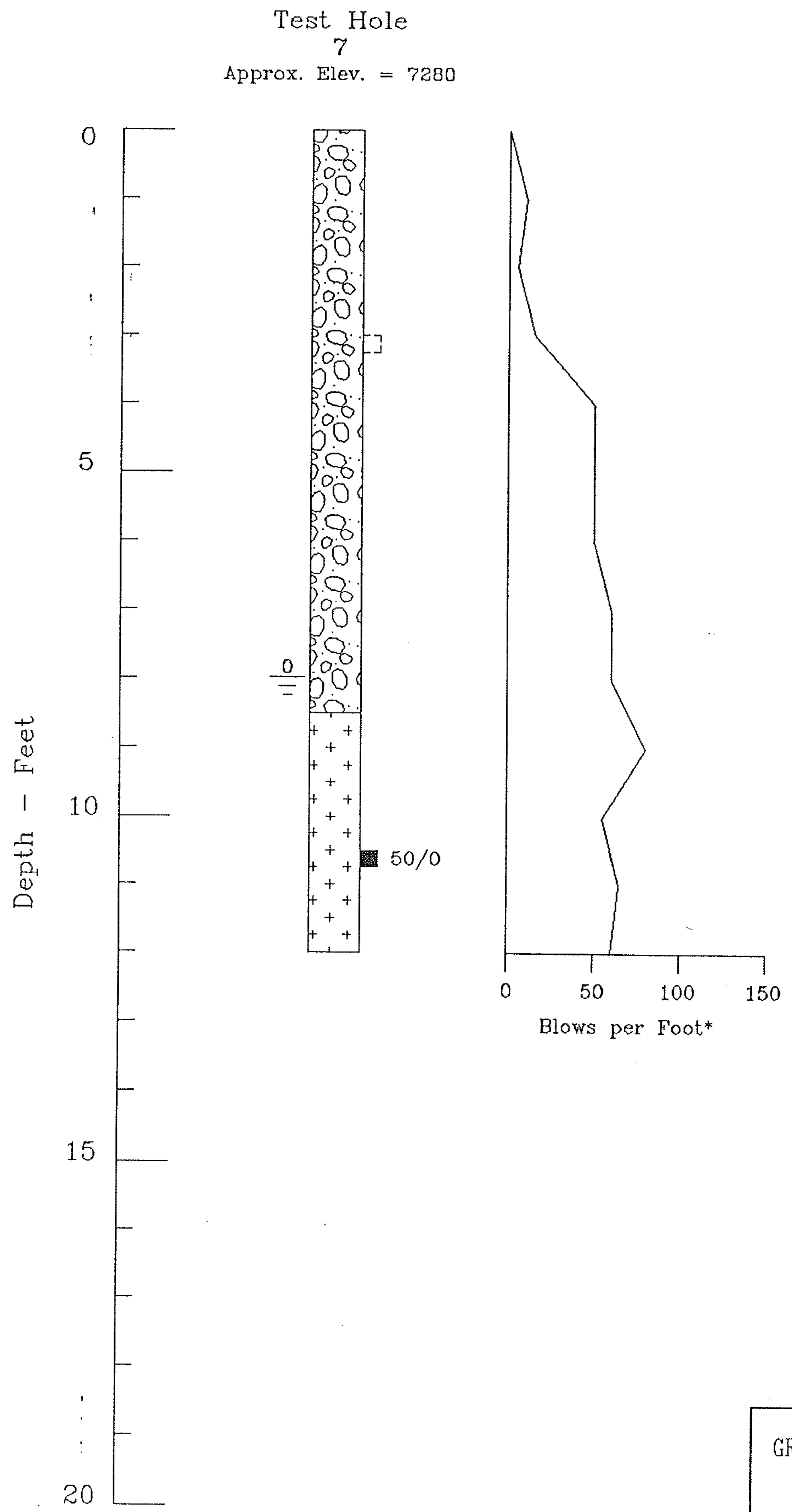


* see notes for explanation

GROUND ENGINEERING CONSULTANTS, INC.

Logs of Test Holes

Job No. 87-203A Figure 13

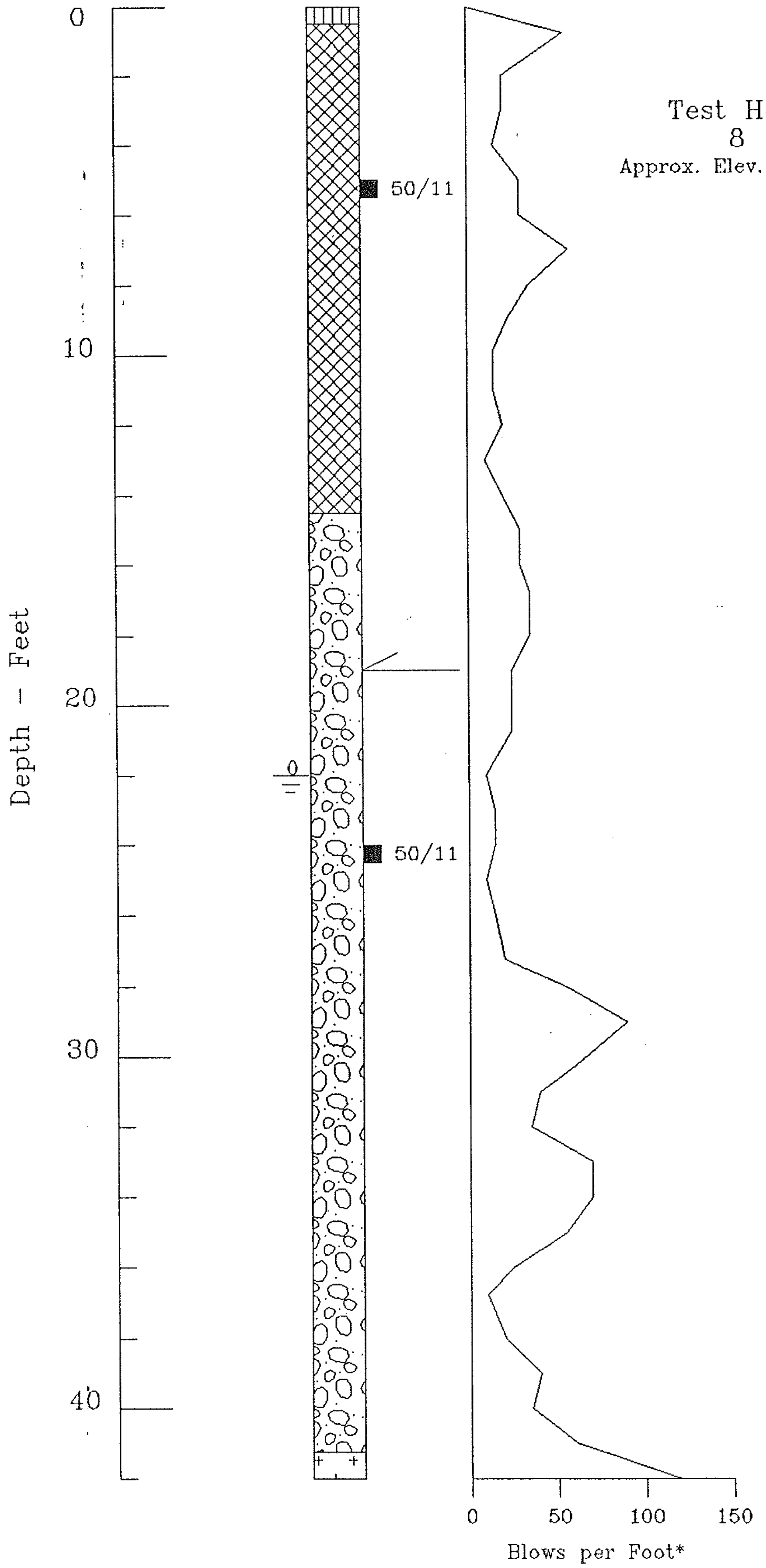


* see notes for explanation

GROUND ENGINEERING CONSULTANTS, INC.

Logs of Test Holes

Job No. 87203A Figure 14



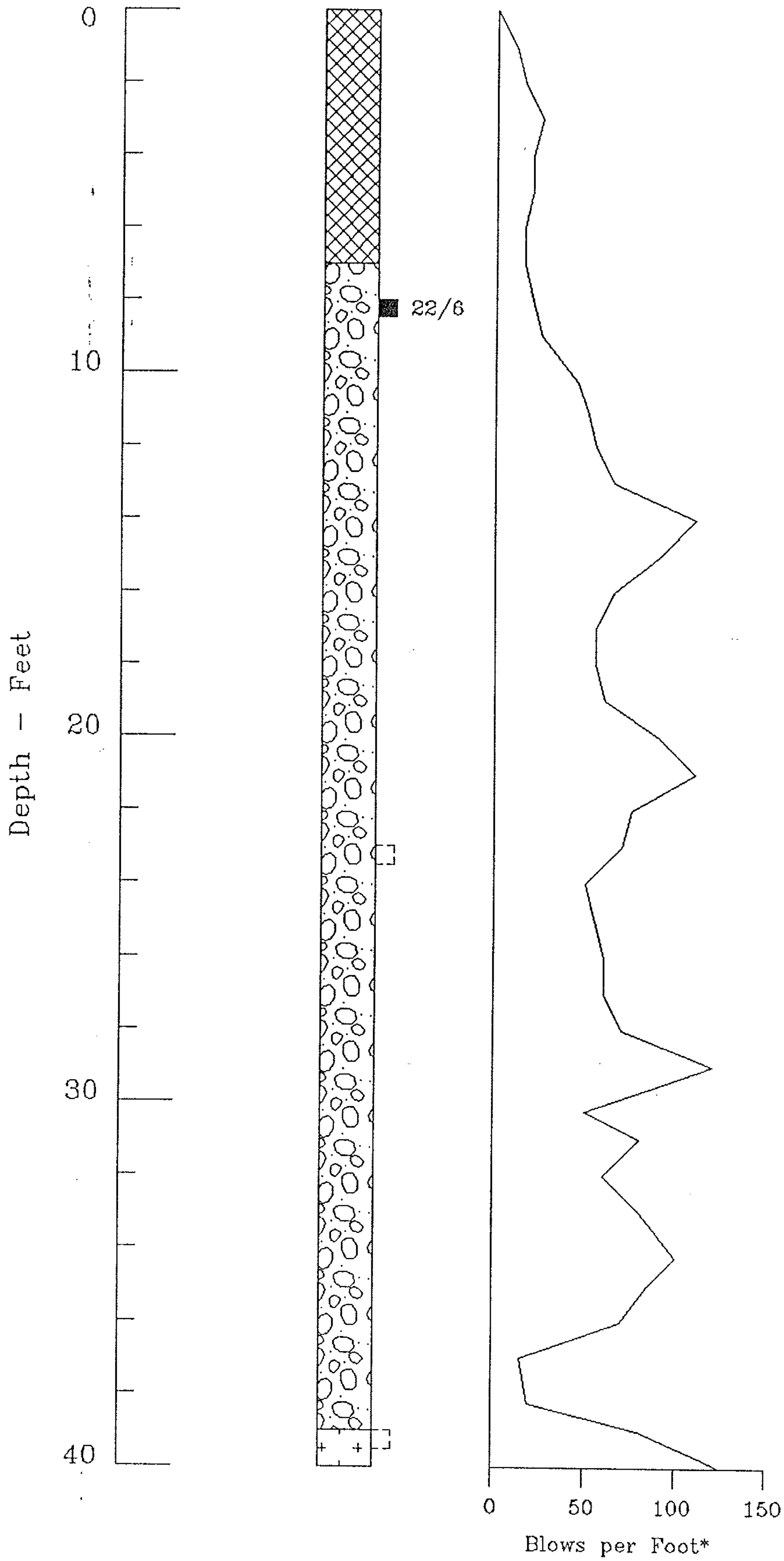
* see notes for explanation

GROUND ENGINEERING CONSULTANTS, INC.

Logs of Test Holes

Job No. 87-203A Figure 15

Test Hole
9
Approx. Elev. = 7288



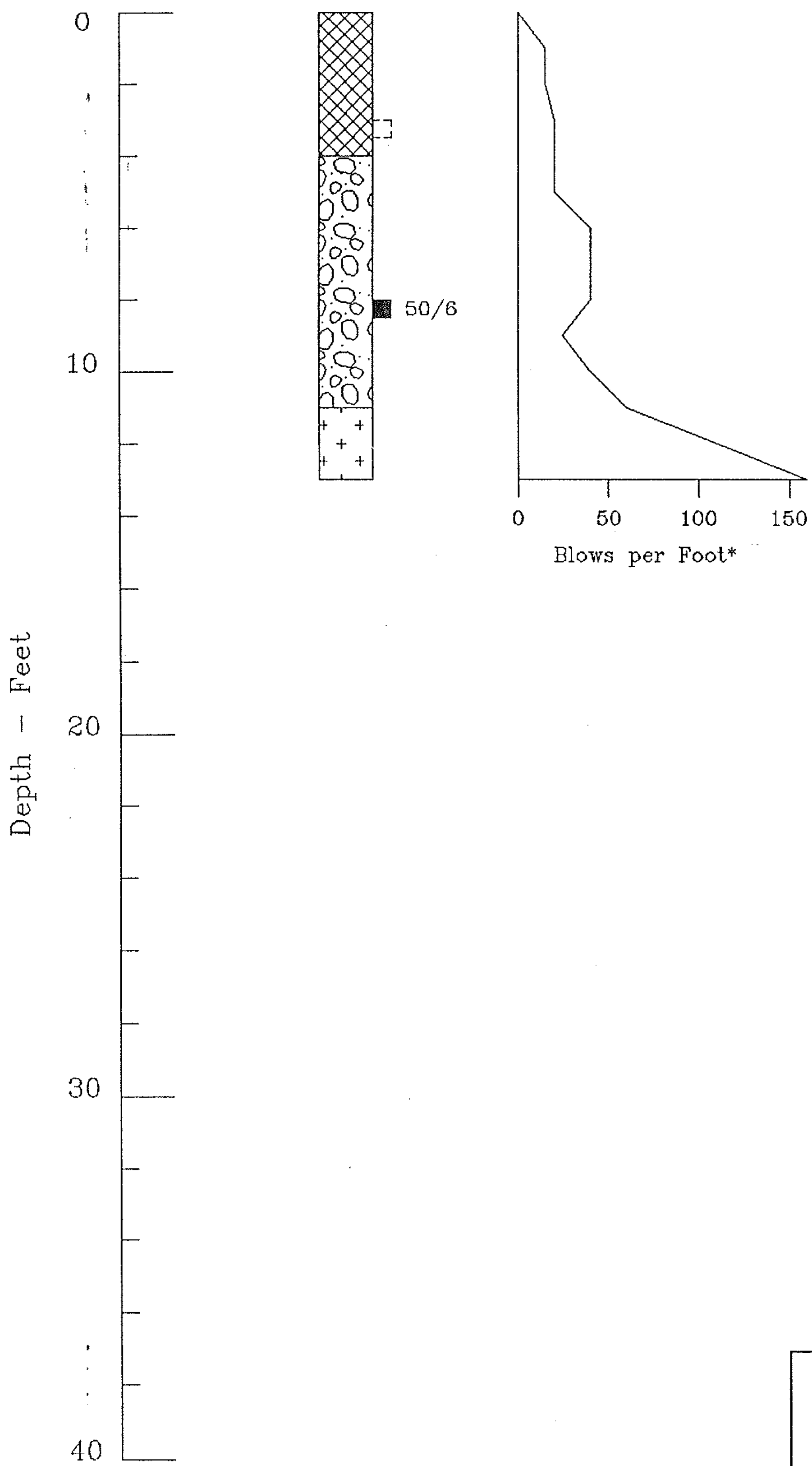
* see notes for explanation

GROUND ENGINEERING CONSULTANTS, INC.

Logs of Test Holes

Job No. 87-203A Figure 16

Test Hole
10
Approx. Elev. = 7267

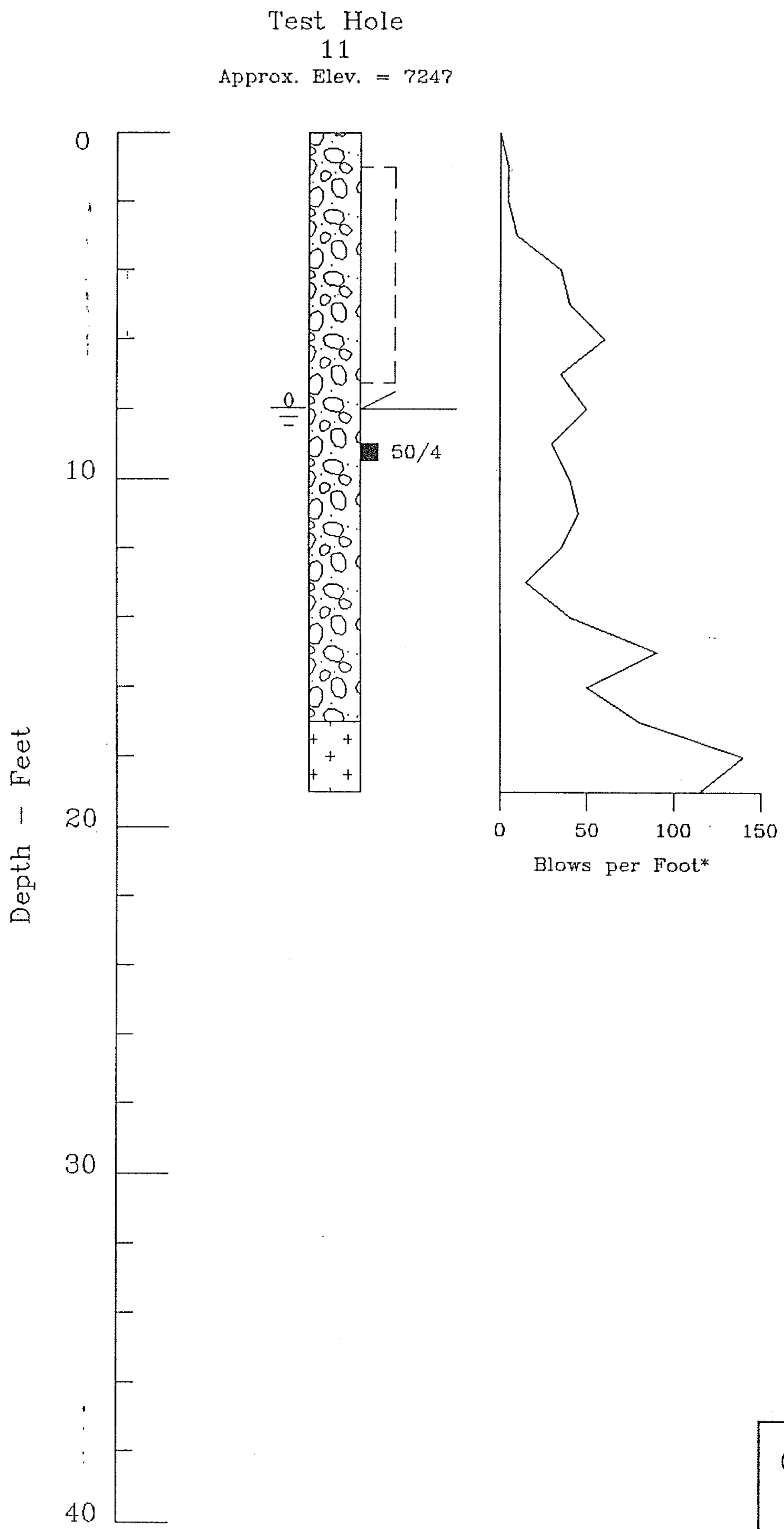


* see notes for explanation

GROUND ENGINEERING CONSULTANTS, INC.

Logs of Test Holes

Job No. 87-203A Figure 17



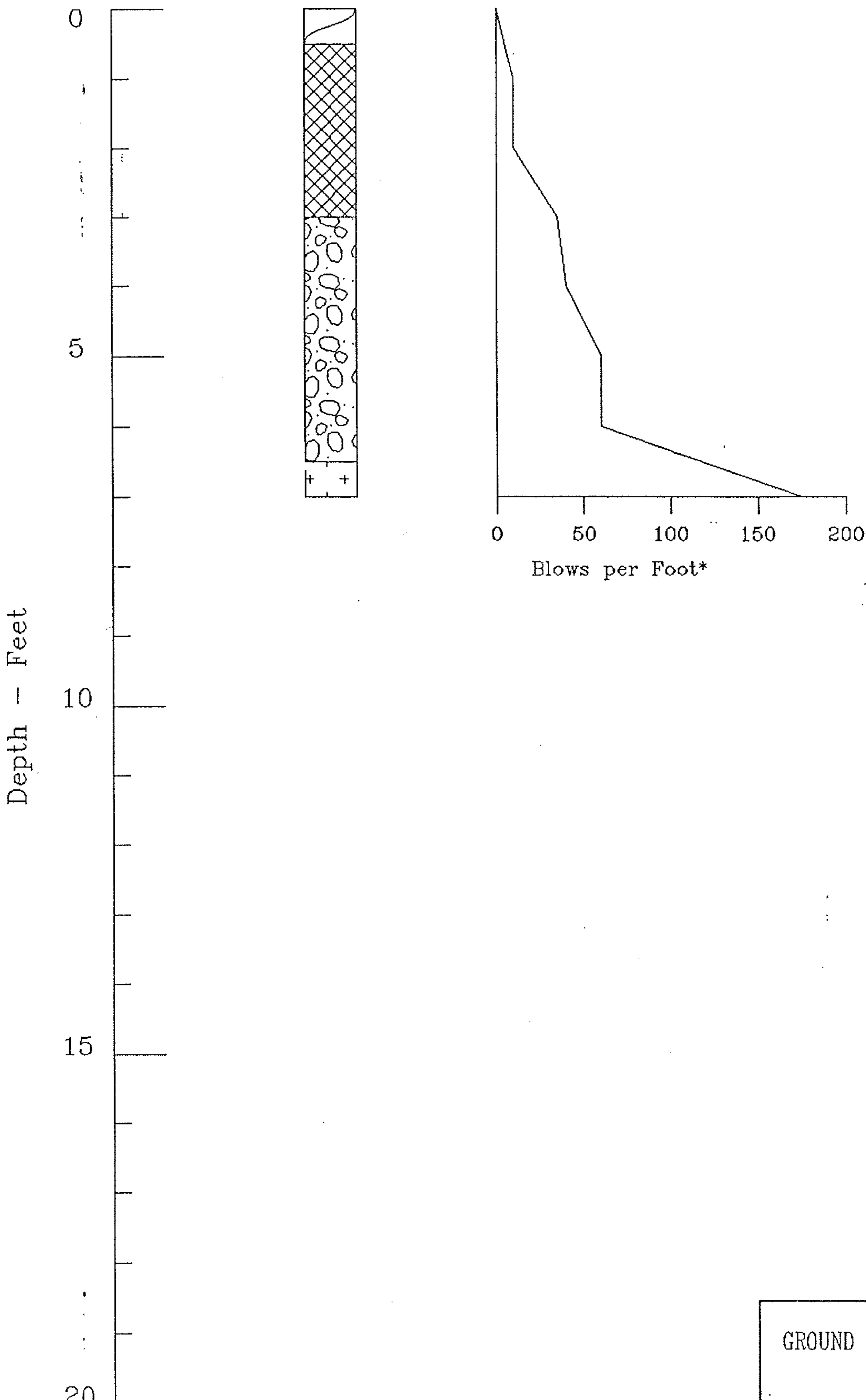
* see notes for explanation

GROUND ENGINEERING CONSULTANTS, INC.

Logs of Test Holes

Job No. 87-203A Figure 18

Test Hole
12
Approx. Elev. = 7260

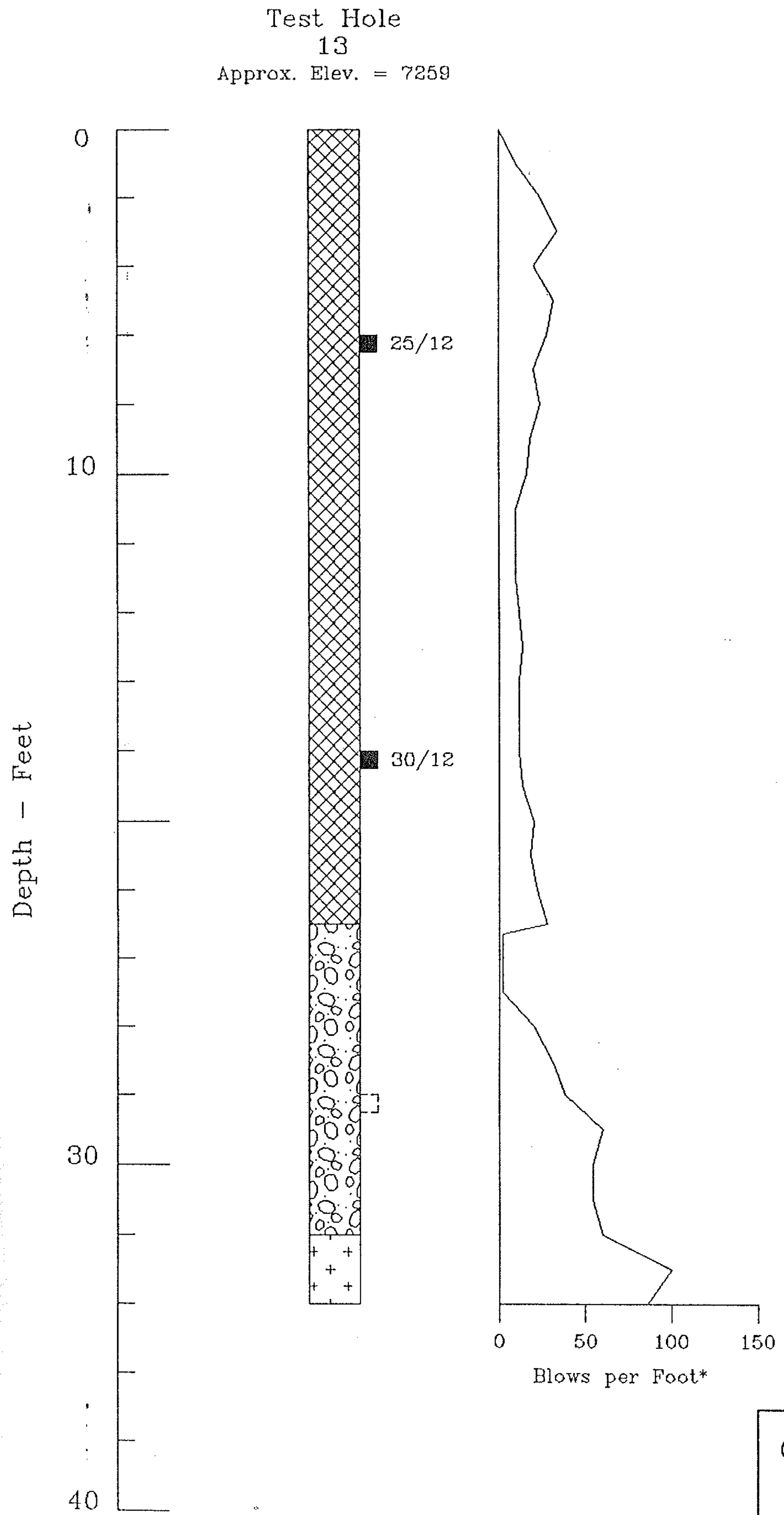


* see notes for explanation

GROUND ENGINEERING CONSULTANTS, INC.

Logs of Test Holes

Job No. 87-203A Figure 19



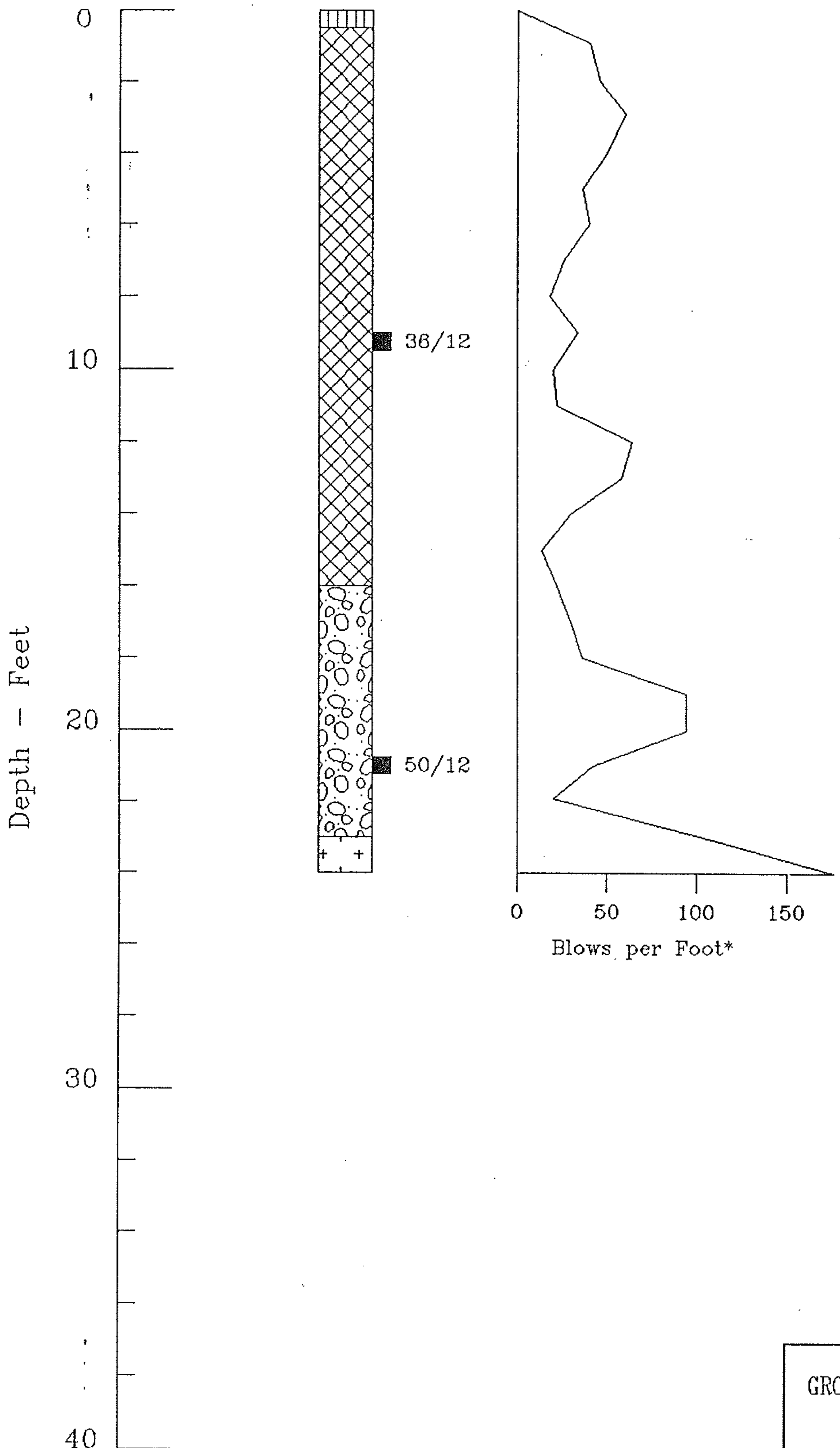
* see notes for explanation

GROUND ENGINEERING CONSULTANTS, INC.

Logs of Test Holes

Job No. 87-203A Figure 20

Test Hole
14
Approx. Elev. = 7247

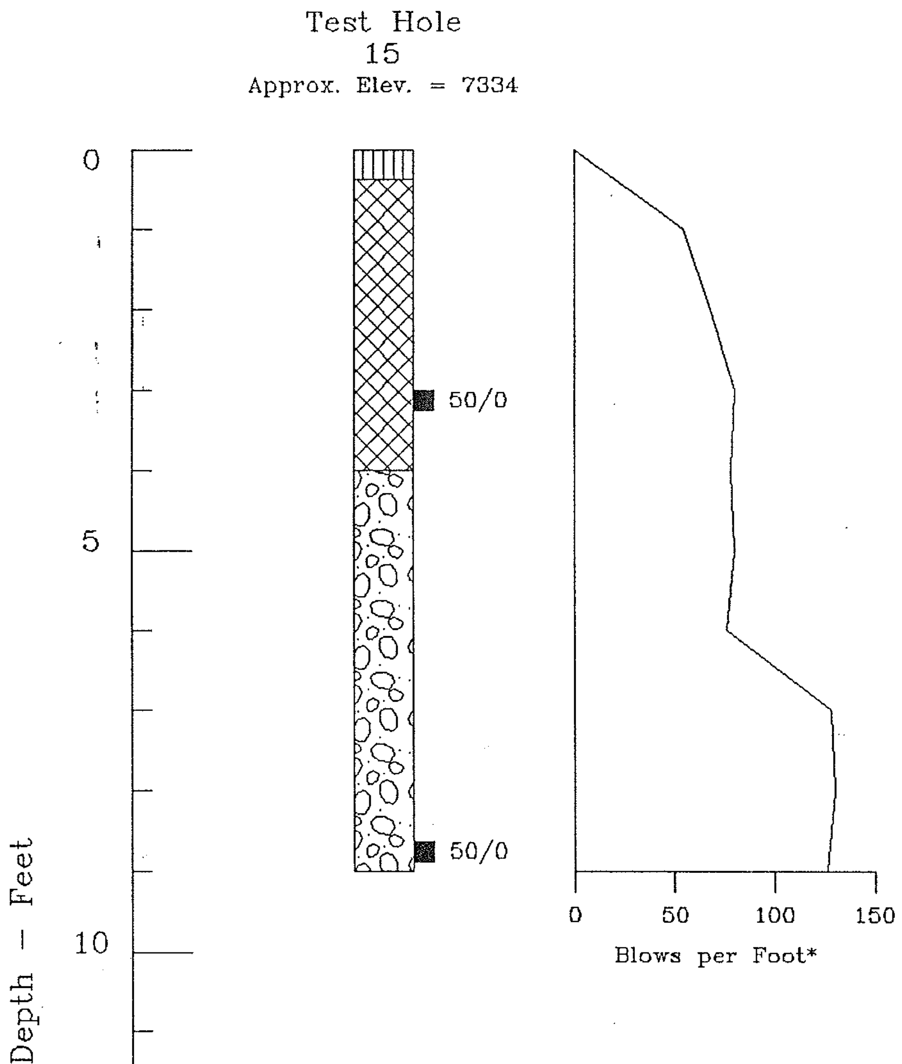


* see notes for explanation

GROUND ENGINEERING CONSULTANTS, INC.

Logs of Test Holes

Job No. 87-203A Figure 21

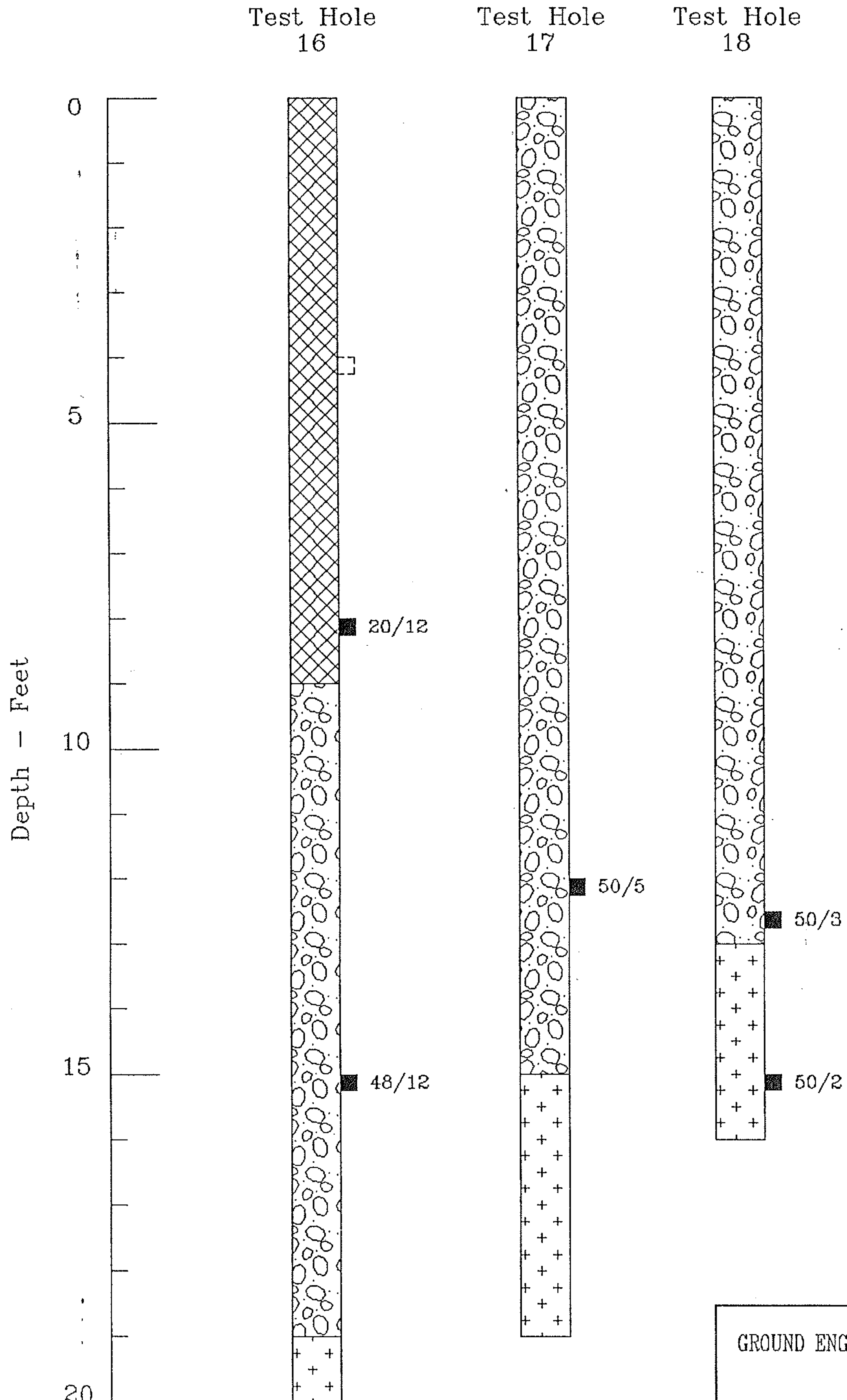


* see notes for explanation

GROUND ENGINEERING CONSULTANTS, INC.

Logs of Test Holes

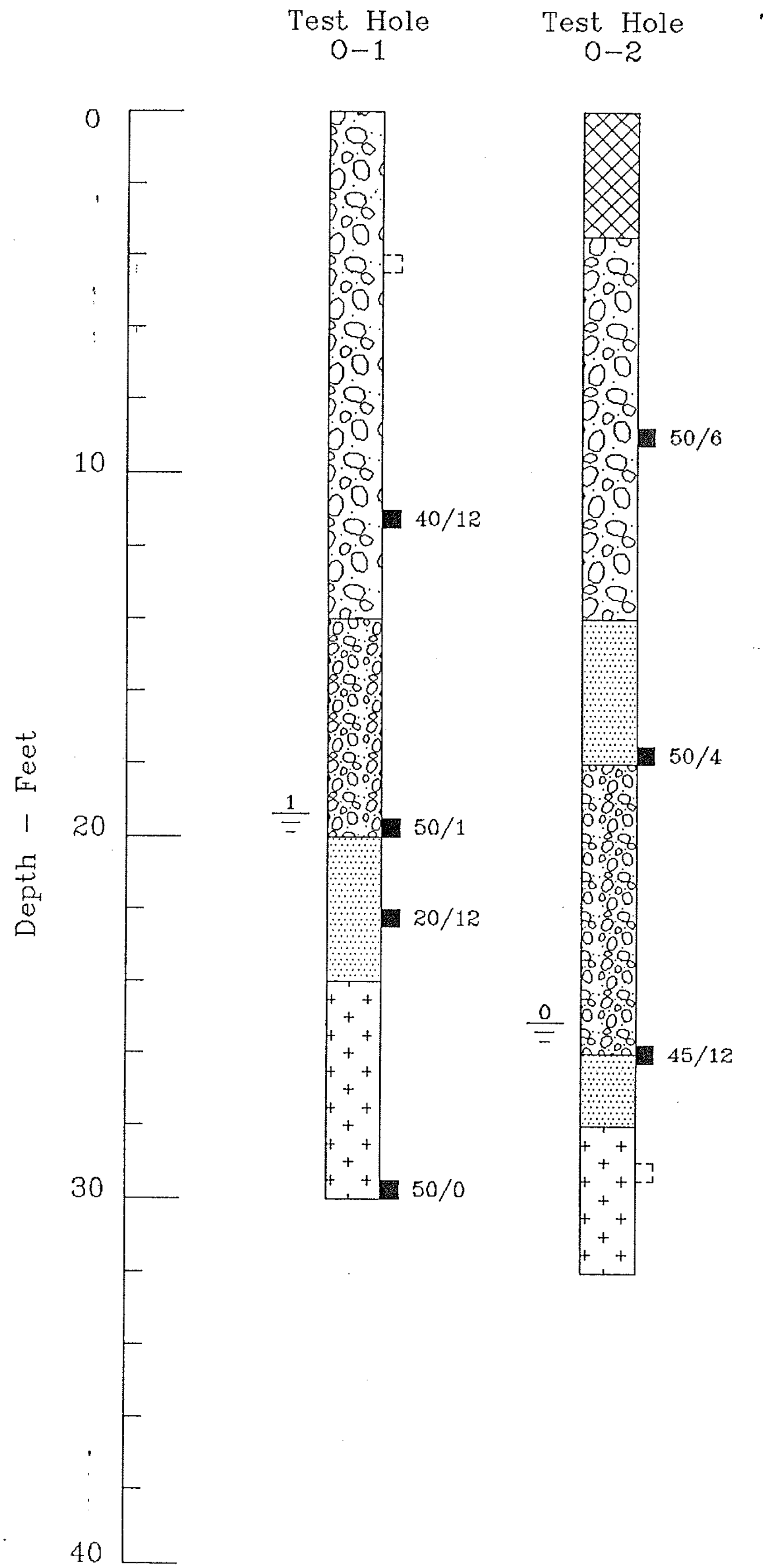
Job No. 87-203A Figure 22



GROUND ENGINEERING CONSULTANTS, INC.

Logs of Test Holes

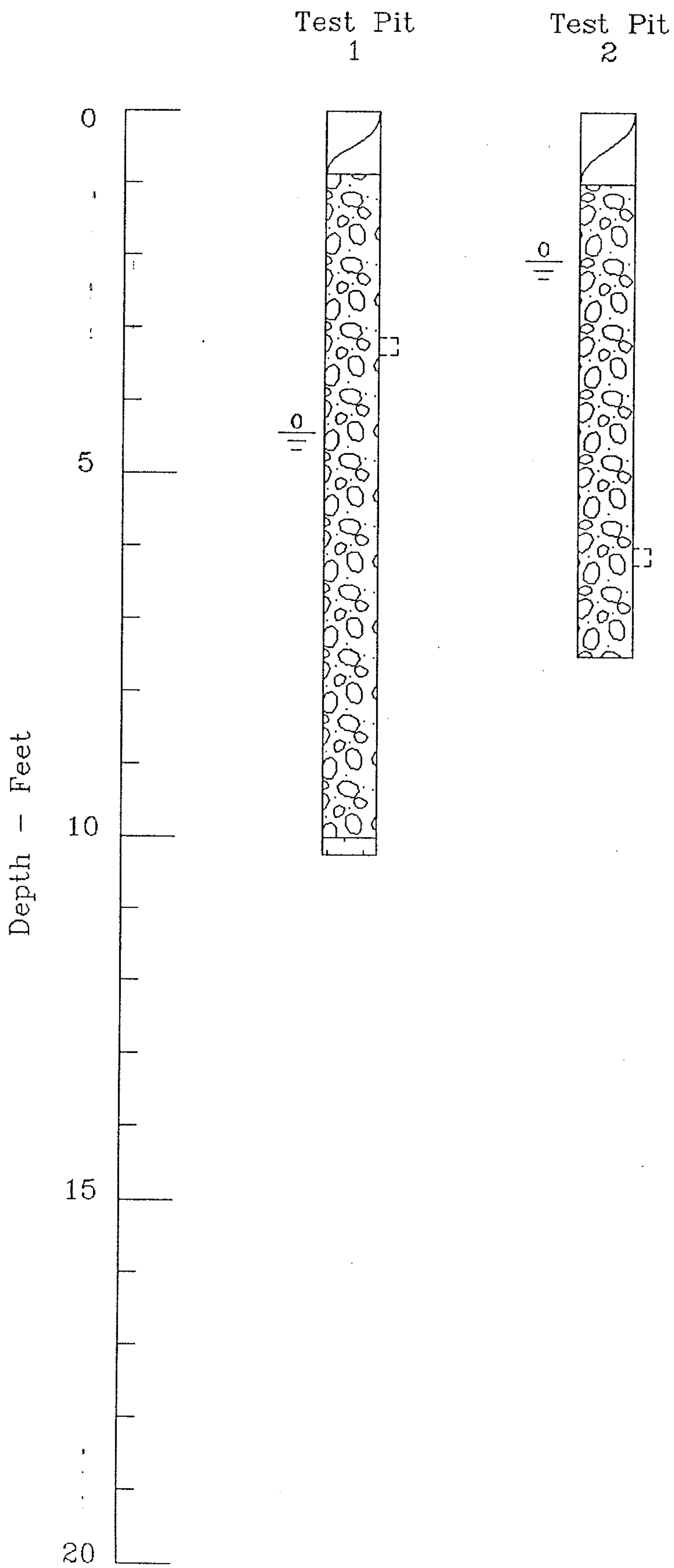
Job No. 87-203A Figure 23



GROUND ENGINEERING CONSULTANTS, INC.

Logs of Test Holes

Job No. 87-203A Figure 24



GROUND ENGINEERING CONSULTANTS, INC.

Logs of Test Pits

Job No. 87-203A Figure 25

LEGEND:

Topsoil

Asphalt

Fill: Slightly silty to silty gravel and sand, cobbles, occasional small boulders, fine to coarse grained, medium dense to dense, moist to wet, brown.

Sand and Gravel: Cobbles and occasional boulders, fine to coarse grained, medium dense to very dense, occasional silt and sand lenses, very moist to wet, tan.

SAnd: fine to coarse grained, occasional gravels, loose to medium dense, very moist to wet, relatively easy drilling, brown.

Sand and Gravel: occasional cobbles and boulders, silty, moderately dense to dense, moist to wet, tan to brown.

Gneiss: Metamorphic, crystalline, hard to very hard, metamorphic, foliated and jointed, hornblende and biotite, blue-gray

Drive sample, 1-3/8 inch I.D. standard sample.

23/12 Drive sample blow count, Indicates 23 blows of a 140-pound hammer falling 30 inches were required to drive the sampler 12 inches.

Small disturbed sample.

0 Depth to water level and number of days after drilling that measurement was taken.

Caved hole.

Continuous blow count log during drilling with Becker Hammer Drill rig. Blow counts shown are for 7-1/4-inch O.D. Felcon bit and Link Belt 180 hammer. Hammer not always at max. stroke.

GROUND ENGINEERING CONSULTANTS, INC.

Legend

Job No. 87-203B Figure 26

NOTES:

1) Test holes were drilled from May to December, 1989 with air–circulating hammer drills, contidrill and a track hoe. Drilling details are provided in the text.

2) Locations of the test holes were determined by referencing features shown on the site plan provided.

3) Elevations of the test holes were determined by refence to contours shown on the site plan provided.

4) The test hole locations and elevations should be considered accurate only to the degree implied by the method used.

5) The lines between materials shown on the test hole logs represent the approximate boundaries between material types and the transitions may be gradual.

6) Water level readings shown on the logs were made at the time and under the conditions indicated. Fluctuations in the water level may occur with time.

GROUND ENGINEERING CONSULTANTS, INC.

Notes

Job No. 87–203B Figure 27

GROUND ENGINEERING CONSULTANTS, INC.

TABLE 1

APPROXIMATE TEST HOLE LOCATIONS/ELEVATIONS

Test Hole *No.*	Location *(From I-70 Control Line)*	Elevation *(feet)*
1	STA 225+50 - 115'RT	7297
2	STA 227+09 - 72'RT	7308
3	STA 213+54 - 124'RT	7349
4	STA 217+44 - 139'RT	7331
5	STA 222+00 - 197'RT	7307
6	STA 233+55 - 140'RT	7284
7	STA 237+27 - 120'RT	7280
8	STA 256+40 - 36'RT	7269
9	STA 257+45 - 141'RT	7288
10	STA 259+93 - 98'RT	7267
11	STA 262+65 - 74'RT	7247
12	STA 274+23 - 10'RT	7260
13	STA 275+50 - 6'RT	7259
14	STA 280+14 - 13'RT	7247
15	STA 212+79 - 45'LT	7334
16	STA 260+85 - 140'LT	7267
17	STA 260+60 - 155'LT	7268
18	STA 261+52 - 135'LT	7265
0-1	STA 217+67 - 82'LT	7319
0-2	STA 216+08 - 72'LT	7326
0-3	STA 219+40 - 90'RT	7307
Test Pit 1	STA 230+40 - 115'RT	7284
Test Pit 2	STA 232+30 - 100'RT	7280

* Locations above were determined by referencing features shown on the plan provided and elevations were determined by interpolating between topographic contours on the plan provided.

GROUND ENGINEERING CONSULTANTS, INC.

TABLE 2

SUMMARY OF LABORATORY TEST RESULTS

Sample Location		Natural Moisture Content	Plasticity Index	GRADATION % Passing Sieve Size or Number											Soil Type
Hole	Depth *(feet)*	*(%)*	*(%)*	3"	1-1/2"	3/4"	3/8"	No. 4	No. 8	No. 16	No. 40	No. 50	No. 100	No. 200	
1	3.5	2.8	NP		100	87	70	58	50	42	33	22	10	3	Sand & Gravel Fill
1	19	4.3	NP			100	70	41	32	28	23	17	9	2	Sandy Gravel
2	5	2.3	NP		100	90	64	45	36	29	22	15	8	5	Sand & Gravel Fill
2	23	7.6	NP		100	87	74	66	60	48	33	16	6	2	Sand & Gravel
3	24	5.0	NP			100	75	39	30	25	18	9	4	1	Sand & Gravel
4	19	3.2	NP		100	91	73	63	51	42	28	17	6	4	Sand & Gravel
5	12	13.1	NP		100	88	60	46	34	24	14	8	3	2	Sandy Gravel
7	6	4.3	NP		100	82	63	51	36	23	20	12	7	4	Sand & Gravel
8	24	10.5	NP		100	87	59	52	43	31	26	14	11	5	Sand & Gravel
9	9	3.1	NP			100	87	63	45	22	13	8	6	2	Gravelly Sand
10	3	4.3	NP		100	84	74	64	57	48	38	28	9	4	Sand & Gravle Fill
11	3'-8'	0.5	NP	100	80	58	44	35	31	26	19	12	5	2	Sandy Gravel
13	18	4.8	NP		100	94	75	62	56	49	40	26	13	5	Sand & Gravel Fill
15	6	3.5	NP		100	74	37	27	22	16	7	6	2	1	Gravel Fill
Test Pit 1	3	9.5	NP	77	72	64	57	52	50	47	42	29	14	7	Sandy Gravel
Test Pit 2	6	11.3	NP	84	70	52	40	33	26	18	10	5	3	2	Sandy Gravel
O-2	18	7.1	NP			100	93	85	76	63	31	18	15	12	Silty Sand

Ground Penetrating Radar Survey

for

Identification of Abandoned Mine Workings
Interstate 70, Hidden Valley, Colorado

Prepared For:

Ground Engineering Consultants
265 S. Harlan Street
Denver, Colorado 80226

Prepared By:

GEO-RECOVERY SYSTEMS, INC.
1200 Seventeenth Street, Suite 2500
Denver, Colorado 80202

(303) 595-0675

January 9, 1990

INTRODUCTION

Geo-Recovery Systems conducted a ground penetrating radar survey within the area of Interstate 70 and Hidden Valley Interchange, Colorado during the month of November, 1989. The intent of the survey was to locate abandoned mine workings believed to exist beneath Interstate 70, associated interchange ramps and frontage roads.

The project site, located some 30 miles west of Denver, lies within the intermountain canyon associated the Clear Creek stream drainage system. Interstate 70, a major four lane east-west highway, occupies much of the area of the canyon. Approximately six (6) miles of ground penetrating radar was collected within the designated project area. Nearly all survey line locations were orientated in an east-west direction, paralleling the highway right-of-way. A total of eleven (11) lines were surveyed having various lengths of 400-3,000 feet.

Equipment Discussion

A GSSI SIR-8 Ground Penetrating Radar System, 300 MH_z and 80 MH_z antennas were employed for data collection purposes. The task requirement governed the selection of instrumentation. Depth of penetration requirements for identification of mine voids, structures and loose fill areas, anticipated to occur at depths of 20-30 feet in sandy gravel geologic materials, warranted selection of the 80 MH_z antenna. For purposes of resolution and antenna shielding requirements, the 300 MH_z antenna was selected to supplement the 80 MH_z data. It was determined during site compatibility testing the 80 MH_z antenna had sufficient penetrative capabilities to detect disturbances attributed to underground mining operations. The 300 MH_z antenna data exhibited less than satisfactory penetrative capabilities. It proved helpful in examining areas where surface feature reflections interfered with the 80 MH_z subsurface data.

Ground Penetrating Radar Methodology

The operating characteristics of a short pulse radar system are dependent upon the partial reflection of radar incident energy at the interface of different subsurface materials. The amplitude of the reflected energy is related to the relative dielectric constants and conductivity of the materials. In the case of identifying utilities generally a metallic object, such as a water line, will produce a strong parabolic shaped reflection as the antenna perpendicularly transects the line. In situations of identifying concrete or other earthen type construction materials in the subsurface, radar signal interpretation becomes more difficult. Due to the often similar dielectric character of construction materials verses those of natural subsurface materials a concrete sewer pipe will be more difficult to detect than a steel pipe.

For purposes of detection of abandoned mine structures, it is often assumed void spaces occur within the structures. In the case of detecting an absolute void, previous experience indicates the vastly dissimilar dielectric properties of the shaft or drift void verses that of the surrounding geologic materials, a strong reflection, derived from the interface, will be observed. In situations where mine workings have been collapsed or backfilled with tailings, areas of inhomogeneity or related disturbances should produce observable reflections, but with much less clarity than those associated with voids. The reason for this observation is often the backfill material retains the same dielectric properties of the surrounding materials but exhibits poorer compaction characteristics, resulting in a somewhat less observable radar reflection disturbance. Within the alluvial subsurface mining operation suspected to be present beneath of survey site, it is anticipated voids will be encountered with much less frequency than areas of backfill disturbances. This situation is anticipated due to the assumed age of the mining operation and the tailings movement practices employed in underground alluvial mining operations.

Survey Location and Collection Parameters

All surface location surveying for ground penetrating radar line orientation was performed by Ground Engineering Consultants. Six east-west lines were located within the project area. All ground location points were spaced at fifty (50) foot intervals. Ground penetrating radar line designations are as follows:

GPR Line	Location
1	West bound frontage road North side of road
2	West bound frontage road South side of road
3	I-70 West bound drive lane
4	I-70 West bound passing lane
5	I-70 East bound passing lane
6	I-70 East bound drive lane
7	East bound off-on ramp North side of ramp
8	East bound off-on ramp South side of ramp
9	East bound frontage road
10 (surveyed by Rolotape)	I-70 Underpass (north to south) West side of road
11 (surveyed by Rolotape)	I-70 Underpass (north to south) East side of road

All 80 MH_z antenna data was collected using a 300 nanosecond range setting. All 300 MH_z antenna data was collected employing a 150 nanosecond range setting. It is estimated by comparison of drill data obtained from Ground Engineering and observation of detected utilities these time ranges correspond to depths of 34 and 17 feet respectively, assuming linear radar velocity constants.

file to profile, are traceable across the area. As to whether they are completely void of material is a debatable question. Their changing radar signature leads to an interpretation of at least a partial backfilling has occurred. It is estimated, by comparison with drill data, the tops of these features occur at depths of 14 feet or greater. Maximum penetration depth of the 80 MH_z radar data is calculated to be 27 feet.

Areas of mining disturbance not associated with drift activities exhibit a more subtle radar signature. These are areas which exhibit disjointed reflection segments not traceable from profile to profile. The relative random nature of these reflection events warranted classification as disturbed areas most likely backfilled during mining operations.

Areas of subsidence previously backfilled and patched by CDOH were also observable on several radar profiles. The radar signature of events occurring beneath these patched areas were used for comparison purposes during the interpretation process.

On the aerial photography all areas of interpreted drifts and mine disturbances are indicated. It is evident that the bulk of the disturbances occur within the I-70 Median locations of 324 to 334. A number of radar profiles from this area are provided in Appendix A for review, examples illustrate interpreted drifts, CDOH patch work and disturbed areas.

One area of particular concern that is recommended for possible near future investigation is a location on Line 6 Eastbound Drive Lane between 129 and 130. This location appears to be a near surface anomaly, (which is assumed not to represent a CDOH patch since no record of it was encountered). It may require rapid remedial attention. See Appendix A, Figure 7.

Other disturbances indicated throughout the survey area are much more random in nature. There are no exactly similar areas exhibiting the continuity of features observed within Median area 324-334. Near the western end of the project area, Median area 308-311, several disturbed areas were observed. These disturbances are assumed to be related to bridge building activities. Some additional geotechnical investigation should be conducted within this area.

APPENDIX A

Ground Penetrating Radar Example Profiles

Figure 1	Line 3	122-197
Figure 2	Line 3	117-121
Figure 3	Line 4	325-329
Figure 4	Line 4	329-333
Figure 5	Line 4	331-335
Figure 6	Line 5	329-332
Figure 7	Line 6	128-132
Figure 8	Line 6	127-173

Power line

122

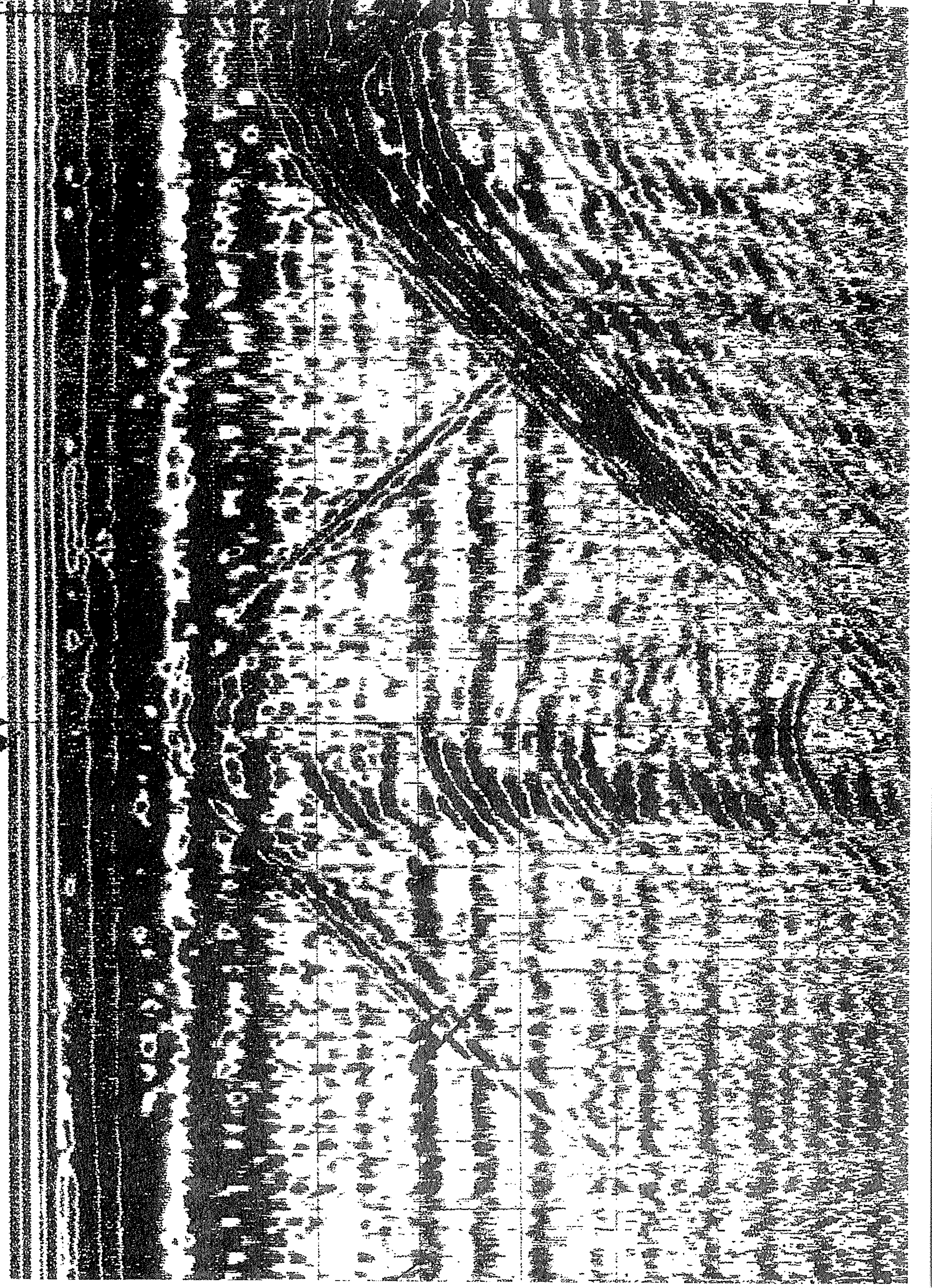
Figure 1

123

169

Drift

198



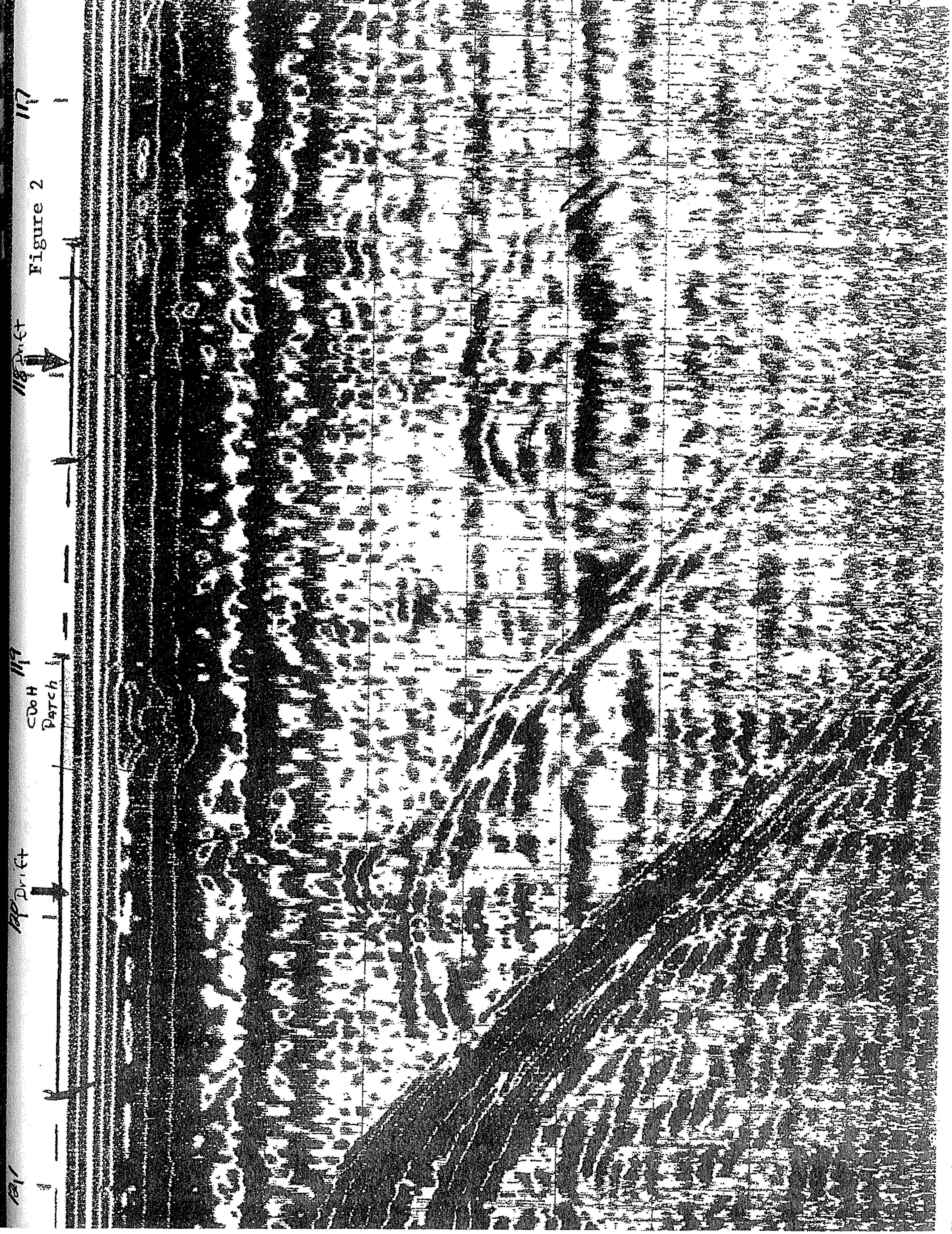


Figure 2

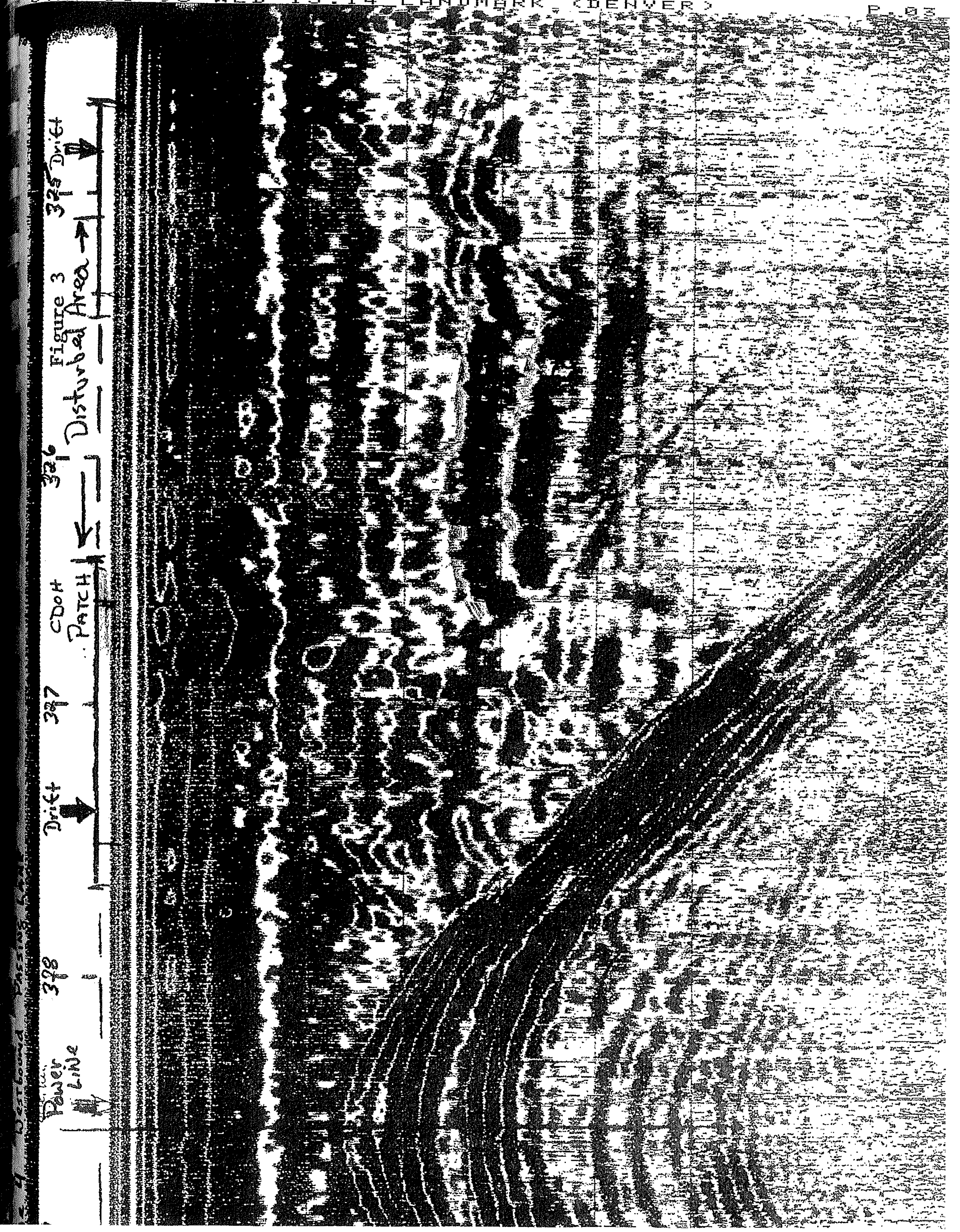


Figure 3
Westbound Passing Lane
Power Line
328
Drift
327
CDOH PATCH
326
Disturbed Area
325
Drift

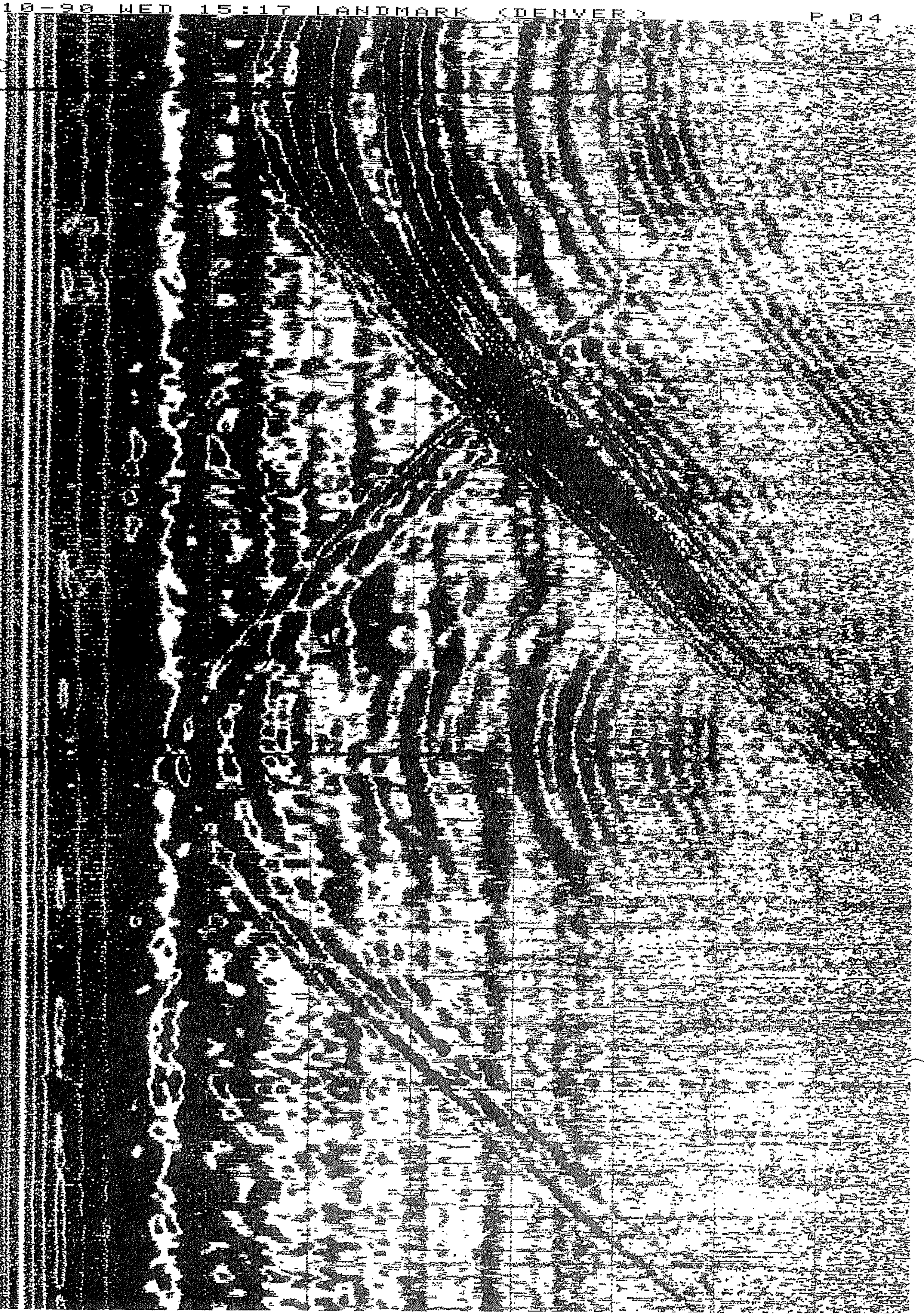


Figure 4

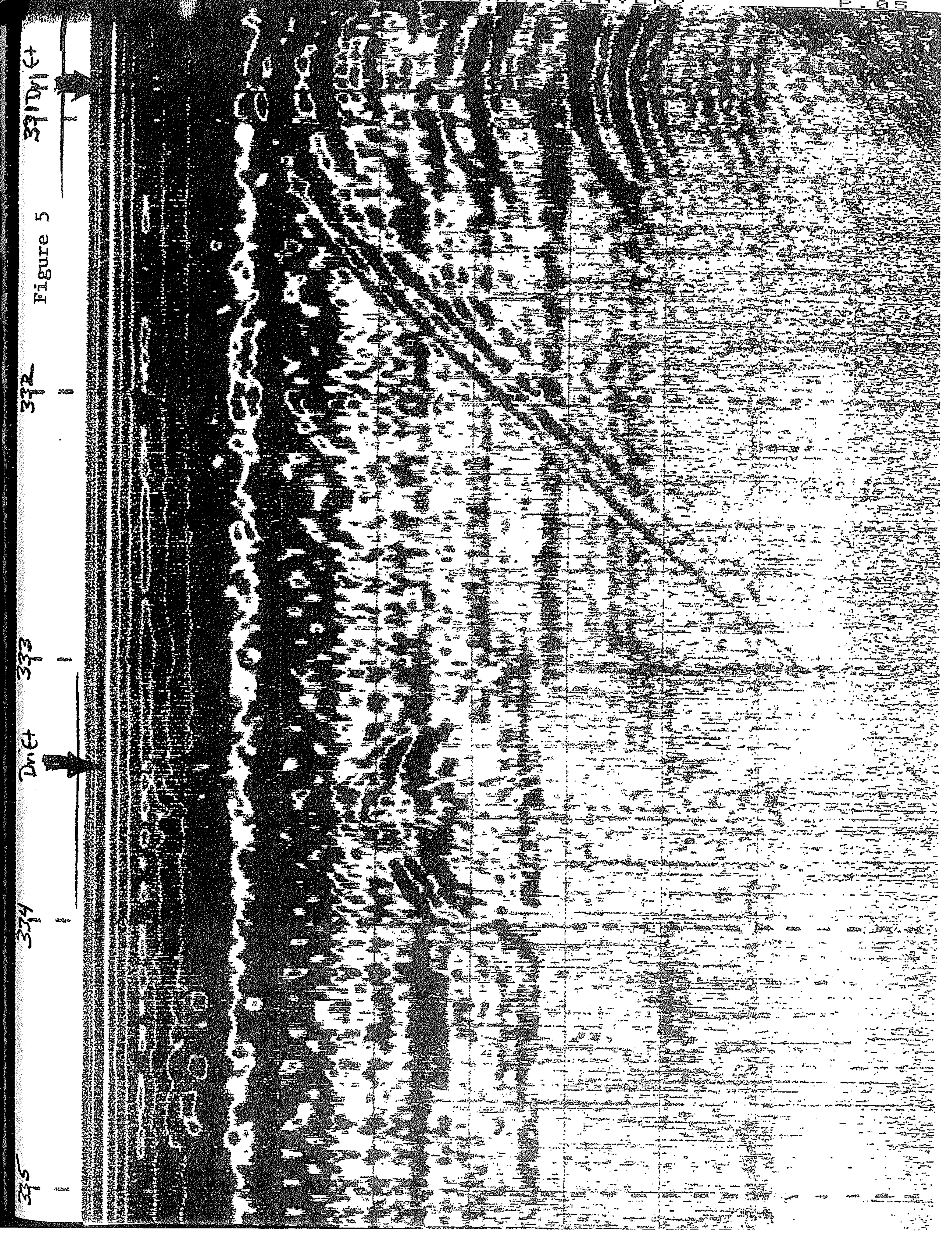


Figure 5

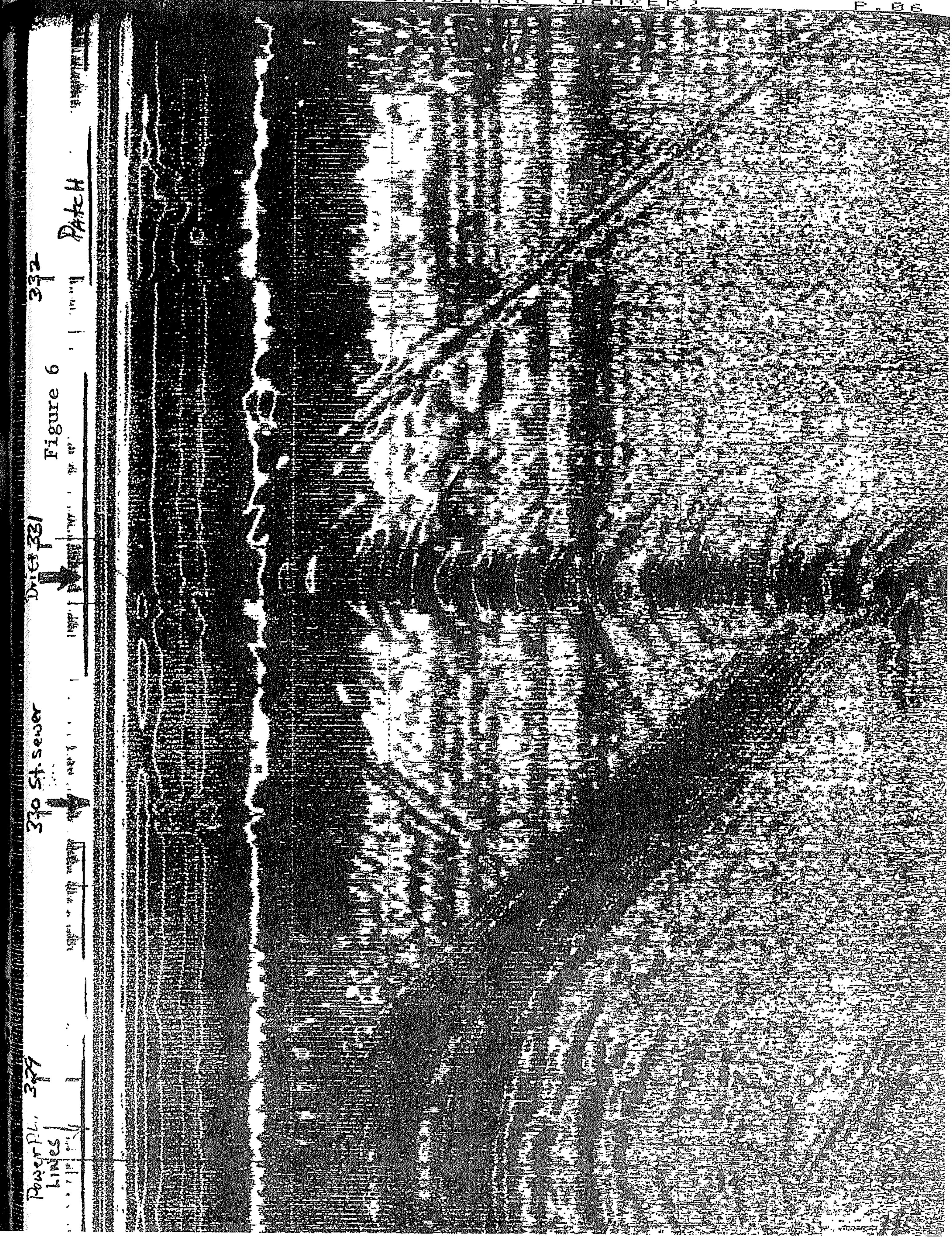
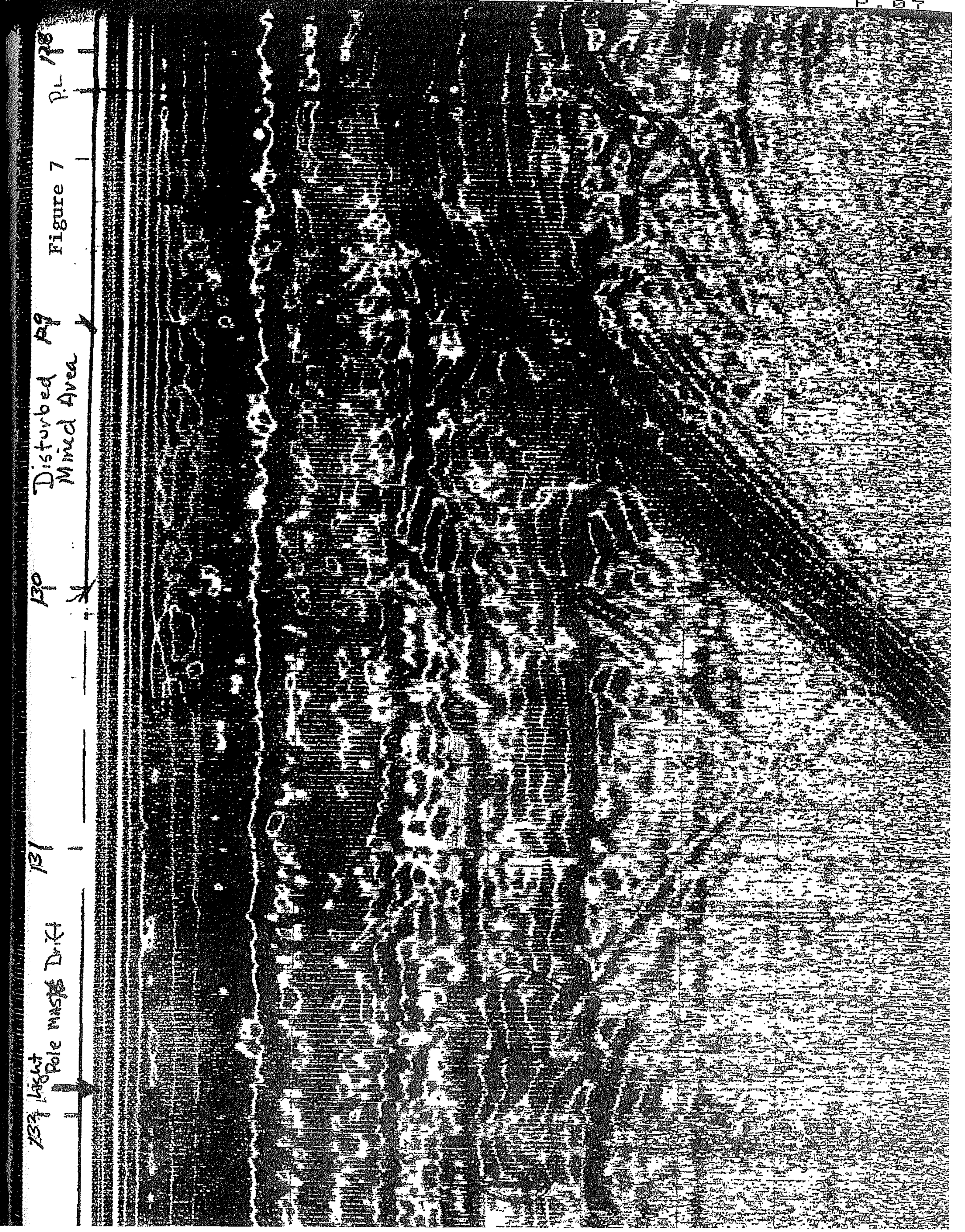


Figure 6


132 Light Pole MAST Drift
131
130
Disturbed Mined Area
129
Figure 7
P.L 128

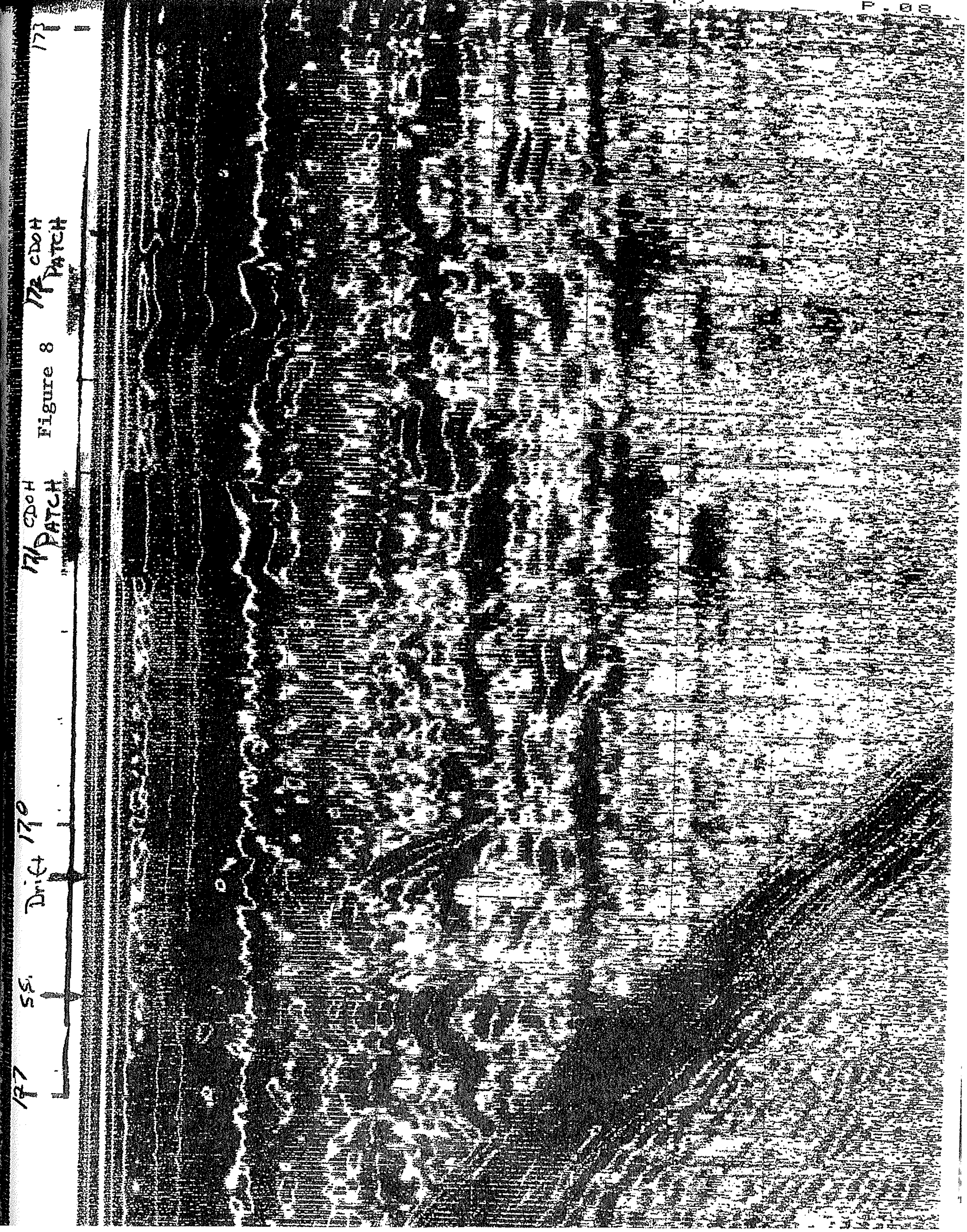


Figure 8