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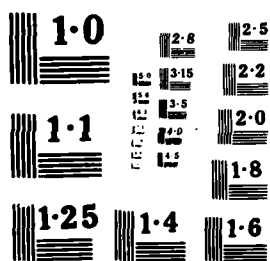
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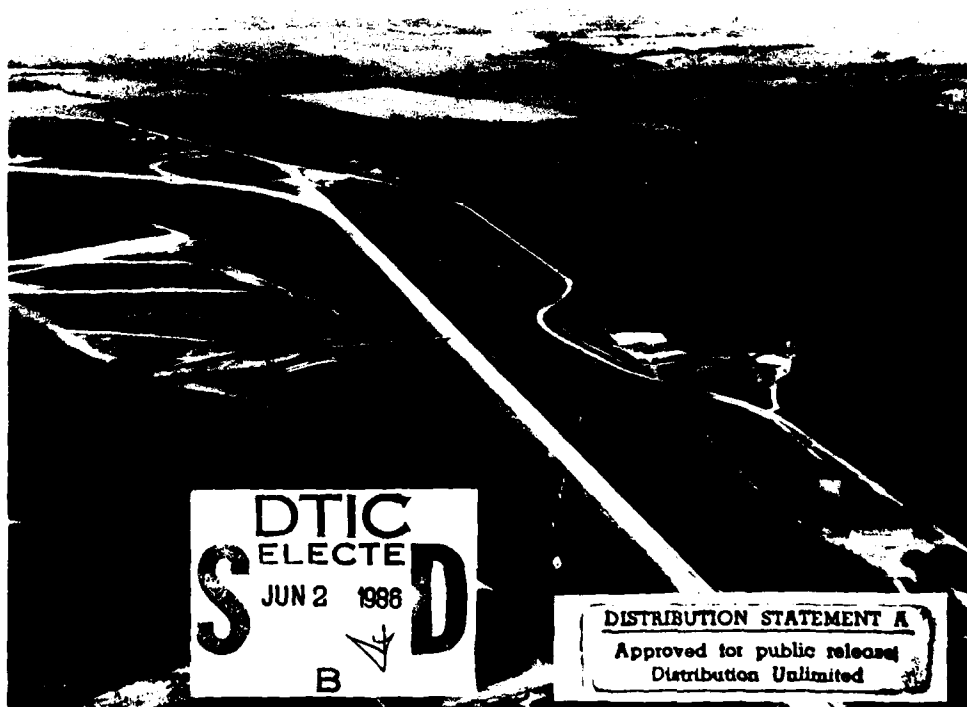
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US Army Corps
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Fort Worth District

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

AQUILLA LAKE AQUILLA CREEK, TEXAS BRAZOS RIVER BASIN



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ATTENTION OF

DEPARTMENT OF THE ARMY
FORT WORTH DISTRICT, CORPS OF ENGINEERS
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25 April 1986

SUBJECT: Aquilla Creek, Texas, Embankment Criteria and Performance Report

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AQUILLA LAKE
AQUILLA CREEK, TEXAS
BRAZOS RIVER BASIN

EMBANKMENT CRITERIA
AND
PERFORMANCE REPORT

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
FORT WORTH, TEXAS

DECEMBER 1985

AQUILLA LAKE
AQUILLA CREEK, TEXAS

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

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AQUILLA LAKE
AQUILLA CREEK, TEXAS
EMBANKMENT CRITERIA AND PERFORMANCE REPORT

SECTION 1 - INTRODUCTION

1-01. Authority. Authority for preparing Embankment Criteria and Performance Reports is contained in ER 1110-2-1901; Subject: Embankment Criteria and Performance Report, dated 31 December 1981.

1-02. Purpose. The purpose of the report is to provide the information needed to, (1) familiarize engineers with the project, (2) re-evaluate the earth embankment and ancillary structural features in the event of unsatisfactory performance and (3) provide guidance for designing comparable future projects.

1-03. Authorization and purpose of the project. The Aquilla Dam and Reservoir project was authorized by the Flood Control Act of 1968; approved August 13, 1968, Public Law 90-483 (82 Stat. 741) 90th Congress. The purpose of the project is flood control, municipal and industrial water supply, fish, and wildlife enhancement, and general recreation.

1-04. Project maintenance. The project is operated and maintained by the Corps of Engineers. Aquilla Dam is inspected annually by the Operations Division and inspected periodically by the Engineering Division in accordance with the Corps of Engineers program of "Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures."

1-05. History of project design. Feature design for the Aquilla Lake embankment, spillway, and outlet works was presented in Design Memorandum No. 7 dated May 1976, prepared by the U.S. Army Corps of Engineers, Fort Worth District. It was reviewed by the Southwestern Division and the Office of Chief of Engineers. Prior to submittal of Design Memorandum No. 7, the overall design of the Aquilla Lake project was presented in General Design Memorandum No. 3 dated July 1975.

SECTION 2 - PROJECT DESCRIPTION

2-01. General. Aquilla Dam is located on Aquilla Creek in Hill County, Texas, approximately 7 miles southwest of the city of Hillsboro, Texas. The Aquilla Creek watershed is in the middle portion of the Brazos River Basin in Central Texas and has a maximum length of about 41 miles and a maximum width of about 16 miles. Aquilla Creek originates near the city of Cleburne, Texas, and flows a distance of about 54 miles in a south to southeasterly direction to its confluence with the Brazos River. Location of the project is shown on Plate 1. Major structures at the project consist of an earthfill embankment, an outlet works, and a spillway, as shown in plan view on Plate 2. Aerial photographs of the embankment, outlet works, and spillway are shown on photographic exhibits 1 and 2. The embankment is a rolled earthfill approximately 11,980 feet long. The limited service spillway consists of an uncontrolled trapezoidal broadcrested weir 1200 feet wide. The outlet works consists of an approach channel, intake structure, and service bridge; a 951 foot long cut and cover conduit, stilling basin, and discharge channel.

2-02. Pertinent data.

a. Embankment.-

- | | |
|------------|---------------|
| (1) Type | - earthfill |
| (2) Length | - 11,980 feet |

(3) Maximum height - 104 feet above streambed

(4) Crest width - 38 feet

(5) Top elevation - 582.5 feet

b. Spillway.-

(1) Type - uncontrolled trapezoidal broadcrested
weir, limited service

(2) Length at crest - 1200 feet

(3) Crest elevation - 564.5

c. Outlet works.-

(1) Type - gated conduit

(2) Conduit diameter - 10 feet

(3) Conduit length - 950.5 feet

(4) Control - two 4.5 x 10-foot sluice-type gates

d. Drainage area. - 252 square miles

e. Reservoir data.-

Feature	Elevation (ft msl)	Area (acres)	Capacity*	
			Acre-feet	Equivalent runoff (inches)
Top of dam	582.5	-	-	-
Maximum design water surface	577.5	14,495	359,900	26.78
Spillway crest	564.5	8,980	213,800	15.91
Top of flood control pool	556.0	7,000	146,000	10.86
Top of conser- vation pool	537.5	3,280	52,400	3.90
Streambed	478.0	-	-	-

*Includes 25,700 acre-feet of storage for estimated 100-year sediment deposition, with 18,800 acre-feet below elevation 537.5 and 6,900 acre-feet between elevations 537.5 and 556.0.

SECTION 3 - CONSTRUCTION HISTORY

3-01. General. The embankment, outlet works, spillway, and appurtenant structures were constructed under two separate contracts. Supervision of the project construction was performed by the Corps of Engineers, Construction Division, Fort Worth District.

3-02. Initial embankment, partial spillway excavation, and outlet works.

a. Under the initial contract the embankment was constructed full height from station 0+00 to station 26+00. The spillway was partially excavated to provide material for the semi-compacted embankment zone, and the outlet works was completely constructed except for the service bridge. Pertinent details of the initial contract are listed below:

- (1) Contract No - DACW63-78-C-0104
- (2) Contractor - Clearwater Constr. Co., Inc., Austin, TX
- (3) Contractor's Bid - \$6,080,122.95
- (4) Notice to Proceed - 5 June 1978
- (5) Actual Completion Date - 20 February 1982
- (6) Total Payment Including Modifications - \$7,172,617.00

b. Construction problems.

(1) The main problems encountered on the initial contract stemmed from the earthwork contractor's inexperience in constructing

an engineered fill using high plasticity clays and in-place moisture requirements. Similar fills had been constructed on other projects with minimal difficulty, but the initial embankment earthwork contractor for the Aquilla Lake project had low fill placement rates and had to reprocess a significant amount of fill in order to produce an engineered fill within the moisture content limits required by the contract documents. The Contractor chose to adjust the moisture content in place on the embankment which proved to be a difficult task without prewetting in the borrow areas.

(2) Moderately severe problems developed during excavation for the outlet works conduit and tower foundation. The conduit and tower were both founded very close to top of rock over most of the total length of these elements. Inadequate control of groundwater and surface water, inadequate grade control, inadequate application and maintenance of shale protection (on the Contractor's part) combined with the inherently troublesome clayshale characteristics including blocky structure, extremely well developed fissility and rapid structural degradation when exposed led to widespread overexcavation and extensive hand cleanup along the outlet works. The overexcavation required a substantial volume of lean concrete backfill to reach structural concrete grade.

3-03. Completion of embankment and spillway and construction of service bridge, access roads, project building, visitor overlook, FM 310, and other appurtenances. Under the completion contract, the embankment was constructed in phases as shown on Plate 4. The service

bridge and project building were also completed, and the excavation of the spillway was completed. Pertinent details of the completion contract are listed below:

- a. Contract No - DACW63-81-C-0035
- b. Contractor - Holloway Constr. Co., and Holloway Sand and Gravel Co., Wixom, MI
- c. Contractor's Bid - \$11,492,320.03
- d. Notice to Proceed - 4 February 1981
- e. Actual Completion Date - 16 May 1983
- f. Total Payment Including Modifications - \$11,823,263.00

The main problems that occurred during the completion contract were related to flooding problems during embankment closure operations. In an effort to save time, the contractor opted to accomplish foundation preparation work in the Aquilla Creek channel area prior to creek diversion and construction of the upstream cofferdam. The Contractor was allowed to do this with the understanding that any flooding damage would be at his own expense. Flooding did occur and this resulted in additional cleanup and foundation preparation cost in the closure area. Also the Contractor experienced flooding problems in low lying areas which restricted his access to higher elevation borrow areas upstream. To minimize access problems the Contractor constructed a large haul road at elevation 500 which contained approximately 100,000

cubic yards of fill. This was supposed to have provided 1 year frequency flooding protection. However, during wet periods the haul road was closed due to inundation several times, thus preventing access to upstream borrow. The decision was made that the limited duration of the flooding did not justify development and reclamation costs of a downstream borrow area. Unlike the initial contract only minor problems were encountered with fill placement rates and fill moisture contents. The problem of nonuniformity in fill moisture content of the highly plastic clay was largely eliminated by prewetting in the borrow areas through the use of spray irrigation equipment.

SECTION 4 - EMBANKMENT DESCRIPTION AND CONSTRUCTION METHODS

4-01. General. The earthfill embankment is essentially symmetrical about its centerline and consists of a compacted central impervious core, compacted random zones adjacent to the core, and semi-compacted berms contiguous to the random zones. A select impervious zone or "cap" was designed at the crest to retard future problem with shallow sliding. Typical embankment sections are presented on Plate 3. The embankment is approximately 11,980 feet long, has a crest width of 38 feet and an approximate fill volume of 7.37 million cubic yards. The embankment height varies from an average of about 60 feet on the right abutment, about 80 feet in the floodplain, and about 40 feet on the left abutment. Maximum height above streambed is 104.5 feet.

4-02. Embankment zoning.

a. Impervious core. The central impervious core was constructed of clay material from on-site borrow. A liquid limit greater than 40 was required. The material was spread in 8-inch maximum loose lifts, processed to bring the after-compaction moisture content between optimum and optimum +3 percent, and compacted with eight passes of a sheepsfoot roller. The acceptability of in-place moisture content was determined using the liquid limit correlation method. The liquid limit correlation method is discussed in Section 8.

b. Random zone. The random zones were constructed of clays and clayey sands from on-site borrow. No restraints on minimum or maximum

liquid limit were used but highly pervious materials were not acceptable. The fill was spread in 8 inch maximum loose lifts, processed to bring the moisture content between -2 to +3 percentage points of optimum, and compacted with eight passes of a sheepsfoot roller. The acceptability of in-place moisture content was determined using the liquid limit correlation method.

c. Semi-compacted zone. The semi-compacted zones were constructed using materials from required excavation. For the initial contract, the main sources were the partial spillway excavation and the outlet works excavation. Semi-compacted fill material was spread in 10-inch maximum loose lifts, processed to bring the moisture content within limits of -2 to +3 percentage points of optimum, and then compacted with two passes of a 50-ton roller or four passes of a sheepsfoot roller. The liquid limit correlation method was used to evaluate the acceptability of in-place moisture content. Moisture limits for semi-compacted fill, however, were specified only for the initial contract. For the completion contract, the moisture control requirements were removed from the semi-compacted fill zones.

d. Select impervious zone. The select impervious zone or "cap" was constructed of CL materials obtained from on-site borrow with liquid limits ranging from 30 to 45. The select impervious cap was designed to minimize the potential for shallow sliding on the 1 vertical on 3 horizontal slopes. The select-impervious cap also provides an improved subgrade for the public roadway located along the crest of the dam. Fill placing, processing, and compaction requirements for select impervious fill were the same as for the random fill.

e. Random rock zone. A random rock zone as shown on Plate 3 was constructed at the downstream toe. The random rock zone was constructed of unprocessed limestone rock from the spillway excavation. The random rock zone contains enough rock fines to fill voids between larger rocks. The rock was placed in loose lifts varying from 12-inches to a thickness equal to the maximum size stone and compacted with four to six passes (depending on lift thickness) of a 50-ton pneumatic roller. The random rock zone was covered with 42 inches of fines from the processing of stone protection materials, and then subsequently topsoiled.

f. Stone protection. A 12-inch thick layer of stone protection material was placed on the upstream 1 vertical on 3 horizontal slope and a 36-inch thick layer of stone protection material was placed on the downstream 1 vertical on 4 horizontal slope as shown on Plate 3. Stone protection materials were produced from limestone from the spillway excavation. Limestone materials were passed over a vibratory bar grizzly with bars spread 4 inches apart. The materials were then passed through a rock crusher with jaws set to crush rocks greater than 12 inches. Lastly, material was passed over another vibrating bar grizzly to remove fines smaller than 2 inches.

4-03. Embankment fill sources. Embankment fill other than that required for the semi-compacted zone came from on-site borrow areas. Borrow areas were investigated with auger borings during the project design phase to determine the type and quantity of overburden soils present. The locations of borrow areas are shown on Plate 1. Borrow

areas were located in cleared fields and pastures. Borrow areas A, B, C, D, and E were located upstream of the embankment in the floodplain and alluvial terraces of Hackberry and Aquilla Creek. Borrow area G was located downstream from the embankment. Borrow area G was intended for use in the event that upstream borrow area became flooded or inaccessible due to flooding. During construction, borrow materials were obtained only from borrow areas A, B, and C. Borrow areas D and E and the downstream borrow area G were not utilized. Fill material for the semi-compacted zone came from required excavation. For construction of the initial embankment, semi-compacted fill material was obtained from the outlet works and partial spillway excavation. For the completion contract the source of semi-compacted fill was from the completion of the spillway excavation.

4-04. Closure plan. Embankment closure was made from station 46+80 to about station 54+50. Plates 4 and 5 show a view of the closure area and construction staging.

a. Diversion channel. Diversion was made through Aquilla Creek during construction of the embankment sections adjacent to the closure section. The creek channel was cleared to provide unobstructed flow along the natural channel alignment.

b. Channel plugs. Three channel plugs were constructed in the closure area as shown on Plate 5. One plug was constructed upstream from the closure section in the Aquilla Creek channel to permit diversion dike construction. The two remaining plugs were constructed

downstream from the closure area in an old and in the existing creek channel. All plugs were constructed to existing channel bank elevation. The plugs had crest widths of 20 feet, 1 vertical on 4 horizontal sideslopes and were constructed from clay. The channel was backfilled to bank elevation with semi-compacted fill materials between the embankment and the channel plugs.

c. Diversion dike. An upstream portion of the permanent embankment in the closure section served as a diversion dike during upstream cofferdam construction. The dike section was constructed to elevation 517.0 with a crest width of 20 feet and symmetrical 1 vertical on 4 horizontal side slopes. The dike was constructed as a semi-compacted fill using clays from borrow.

d. Upstream cofferdam. The upstream cofferdam was constructed as an integral part of the permanent embankment section. The upstream cofferdam, which incorporated the diversion dike, was built to elevation 537.0 with a crest width of 20 feet. The upstream slope was as given for the finished embankment and the downstream slope was 1 vertical on 4 horizontal. The cofferdam was constructed as compacted random fill and semi-compacted fill using clays from borrow. The upstream cofferdam had a projected frequency of overtopping of once every 10 years.

e. Downstream cofferdam. The downstream cofferdam consisted of a semi-compacted fill constructed to elevation 498 to prevent water from backing into the closure area. The cofferdam had a crest width of 20

feet, 1 vertical on 4 horizontal side slopes, and was incorporated into the downstream portion of the embankment.

4-05. Compaction equipment. The specifications governing compaction equipment for the project were based on the Civil Works Construction Guide Specification CW-02212, dated February 1976. Compaction equipment used for the job was primarily sheepsfoot rollers and rubber tired rollers. For all of the embankment zones, except the semi-compacted zone, the material that was compacted consisted mostly of overburden clays and clayey sands. For these types of material, sheepsfoot rollers provide the best results in terms of uniformity of compaction and bonding between lifts. The use of rubber tired rollers was specified as acceptable only in the semi-compacted zones. However, compaction using a sheepsfoot roller was allowed for the semi-compacted fill zone with the stipulation that the number of roller passes be doubled and that the uncompacted lift thickness be reduced from 10-inches to 8-inches. Compaction equipment used during the initial and completion embankment construction is described below:

a. Compaction equipment for initial embankment contract.

(1) Sheepsfoot roller-towed

(a) Ferguson, Model Z32

(b) Two (2) 5 ft diameter X 6 ft long drums

(c) Seven (7) rows of feet, 3 or 4 feet per row and 25 feet per drum; 9.5 in shank and tip, round shape; 9.5 sq in end area.

(d) Total weight empty - 38,457 lbs; No ballast used.
Weight per foot of drum length-3,205 lbs.

(e) Oscillating frame, rigid cleaners, speed not greater than 5 mph.

(2) Sheepsfoot roller - self propelled

(a) Ferguson, Model 120B

(b) Two (2) 5 ft diameter X 5 ft long drums

(c) Thirty (30) rows of feet, 4 feet per row and 120 feet per drum; 10.0 in shank and tip, round shape; 9.5 sq in end area.

(d) Total weight empty - 35,900 lbs; Ballasted with fuel oil; Ballasted weight-38,370 lbs.

(e) Oscillating frame, rigid cleaners, speed not greater than 7.5 mph.

(3) Rubber-tired roller

(a) Ferguson, Model Rt 100-S

(b) One box, rolling width - 9 ft 2 in; length 26 ft 6 in by 9 ft 2 in.

(c) Four 18.00 X 25 tires, 24 ply

(d) Weight empty-20,000 lbs; 100,000 lbs ballasted tire pressure 90 psi, 25,000 lbs per tire

(e) Speed not greater than 5 mph

b. Compaction equipment for completion contract.

(1) Sheepsfoot roller-towed

(a) Southwest triple drum

(b) Three (3) 5 ft diameter X 6 ft long drums

(c) Seventy-two (72) rows of feet per drum; 2 feet per row, 144 feet per drum; 10 in shank and cap, round shape; 9.0 sq in end area

(d) Total weight empty - 61,000 lbs; Ballasted with water, weight-77,000 lbs; Pressure per linear foot of drum - 4,277 lbs

(e) Oscillating frame; speed not greater than 5 mph, spring loaded cleaners

(2) Sheepsfoot roller-self propelled

(a) Ferguson, Model SP-120-D

(b) Two (2) 5 ft diameter X 5 ft long drums

(c) Sixty (60) rows per drum, 9.5 in shank and cap, round shape, 9.5 sq in end area

(d) Total weight empty - 34,000 lbs; Ballasted with diesel fuel; ballasted weight-44,500 lbs; Pressure per lineal foot of drum-4,450 lbs

(e) Oscillating frame, speed not greater than 5 mph,
spring loaded cleaners, front and rear

(3) Rubber-tired roller

(a) American Model 4 BW 50-ton roller

(b) One box, rolling width - 8 ft 11½ in, overall width
9 ft 5½ in, length - 25 ft 8 in.

(c) Four (4) 18.00 X 25 - 24 ply tires

(d) Weight empty - 19,500 lbs; Ballasted with sand;
ballasted weight-100,000 lbs; Tire pressure - 90 psi, 25,000 lbs per
tire.

SECTION 5 - GEOLOGY

5-01. Regional Geology.

a. Physiography. The project area is located in the Eastern Cross Timbers physiographic province. A small portion of the eastern and western limits of the project area extend into the Black Prairie and Grand Prairie provinces, respectively. The area topography generally reflects the eastward dipping strata of the Lower and Upper Cretaceous formations.

b. Stratigraphy. The project area is underlain, in ascending order, by the Georgetown, Del Rio, and Buda Formations of the Lower Cretaceous, Comanche Series, and by the Woodbine Formation and Eagle Ford Group of the Upper Cretaceous Gulf Series. The Georgetown Formation is an argillaceous limestone approximately 190 feet thick that does not crop out in the project area. The Del Rio Formation crops out in the highlands west of the project. It is a massive calcareous clay shale, ranging from 60 to 80 feet in thickness, that contains thin limestone seams. The Buda Formation, not recognized at the project site, is a thin, discontinuous limestone remnant ranging from a few inches to 5 feet in thickness that unconformably overlies the Del Rio Formation. The Woodbine Formation, in turn, unconformably overlies the Buda or Del Rio Formations. The Woodbine consists of interbedded variably cemented, fine-grained sandstones and black, soft, non-calcareous clay shales that together reach a maximum thickness of 125 feet and constitutes bedrock along the entire length of Aquilla Creek.

The Eagle Ford Group unconformably overlies the Woodbine Formation and is composed of shales containing a few thin limestone beds above its contact with the Woodbine. It constitutes the bedrock of the upper eastern slopes of the Aquilla Creek valley at the dam and much of the valley of Hackberry Creek. Its maximum thickness is approximately 220 feet.

c. Structure. The primary structural feature is a regional dip of the bedrock of 35 to 40 feet per mile to the east-southeast, modified by the north-northeast trending Balcones fault system, which is located approximately 9 miles east of the dam. Minor faulting with small displacements have been noted in the Aquilla Creek valley near its confluence with the Brazos River.

5-02. Site geology.

a. Physiography. Aquilla Creek meanders across a fairly broad floodplain embraced by fairly steep valley walls. The valley wall rises abruptly for about 30 feet from the flood plain on the right abutment, then rises again in moderately steep slopes from approximately station 10+60 to the top of the abutment. At the left abutment the valley wall rises more gently and is controlled by a relatively thick, flat terrace remnant that extends for 2,400 feet eastwardly. Beyond this area, the surface rises to a low, knoll-like hill, followed by a narrow saddle, then a gently rising slope to the top of the abutment at the spillway (Plates 6 and 7).

b. Stratigraphy.

(1) **Overburden.** The right abutment is mantled by residual and slope-wash material on its upper and middle slopes and in its tributary drainages. This material consists of sandy clay from 3 to 10 feet thick that is occasionally underlain by clayey sand and sandy, clayey gravel. The floodplain is comprised of alluvial deposits that reach a maximum thickness of 37 feet. These deposits consist of an impervious lean clay blanket of an average thickness of 16 feet underlain by a clayey sand. In some areas the clayey sand is underlain by a basal gravel. A terrace remnant approximately 50 feet thick extends from stations 57+00 to 93+00. This deposit consists of silty sand, sandy clay, and clayey, sandy gravel overlain by an uppermost sandy clay that ranges from 4 to 23 feet in thickness. The left abutment is mantled on its upper slopes by approximately 2 to 8 feet of residual and slope wash material consisting principally of sandy clay.

(2) **Primary strata.** In ascending order, the Del Rio, Woodbine, and Eagle Ford Formations occur at the site and are involved in the structure foundations (Plates 6 through 10).

(a) **Del Rio Formation.** The Del Rio Formation consists of a soft to moderately hard, calcareous, gray to greenish-gray, massively bedded clay shale ranging from 70 to 80 feet thick at the damsite. Scattered thin stringers of very calcareous shale and argillaceous limestone occur through the entire section, but these generally increase in abundance downward through the lower half of the

formation. On the left abutment, where the greatest thickness is present, the Del Rio contains a few thin argillaceous limestone beds in the upper 10 feet.

(b) **Woodbine Formation.** The Woodbine Formation constitutes the primary foundation of the dam and its appurtenant structures. The Woodbine is a soft, non-calcareous, montmorillonite-type clay shale. The upper portion of the formation is characterized by a sandstone unit, while the middle and lower portions are clay shale containing a number of variably sandy shale units, some of which grade laterally into sandstone and a few thin sandstone beds. A 15-foot thick basal member of massive sandstone is present in the right abutment to station 44+00 (Plate 6). From stations 44+00 to 58+00, immediately beneath the present Aquilla Creek channel and its left bank, the Woodbine was deposited as a channel fill in the eroded surface of the Del Rio, which is now sandy shale (Plate 6). East of station 58+00, the base of the Woodbine is characterized by 1 to 3 feet of sandy shale. Overlying these basal members of the Woodbine east and west of Aquilla Creek is an 11-foot thick clay shale interval that is essentially devoid of sand or other modifying constituents. In this stratigraphic unit the shale is soft, slickensided, and has a higher void ratio than most other Woodbine clay shale sections. Above this clay shale interval the Woodbine shale contains soft sandstone seams, seams of clay-ironstone nodules and lenses, a few sandy limestone seams, and thin discontinuous sandy silt laminae and seams scattered throughout the clay shale section. The thick upper sandstone unit of

the Woodbine, which overlies the clay shale, is present only on the left abutment, and varies from 16 to 25 feet in thickness (Plate 7). The sandstone beds of the Woodbine vary widely in hardness ranging from hard to dense sand. Many of the sandstone beds are shaly. A few of the sandstone units, particularly the basal unit, contain sandy limestone which borders on very calcareous sandstone.

(c) **Eagle Ford Group.** The Eagle Ford Group is present only on the left abutment in the area of the spillway. (Plate 10) It is composed of soft calcareous shale with a few persistent, thin limestone beds and a few calcareous, sandstone streaks scattered through the section. Its contact with the underlying Woodbine is at the base of a thin limestone bed.

c. **Structure.** The primary structural feature at the damsite is the regional dip (35 to 40 feet per mile to the east-southeast) of the foundation strata. Although minor faulting was anticipated to be present at the site, no faulting was identified during exploration or construction. A significant number of fractures were noted, primarily in the 11-foot section of soft Woodbine shale. A majority of these fractures are slickensided. They appear to be a product of unloading rather than orogenic movement, and were taken into account during design and construction of the project.

d. **Groundwater, Pre-construction.** Groundwater conditions in the right abutment are essentially controlled by weathering of primary material and by the basal sandstone member of the Woodbine Formation.

These conditions cause two water tables to be present, one being a perched water table near the base of weathering, and the other being the static water table within the basal sandstone (Plate 6). This static water table exists 2 to 8 feet higher than the water table of the flood plain, whose level closely conforms with that of Aquilla Creek. No perched water table conditions exist in the left abutment. There the static water table generally conforms with the topography varying from 11 to 30 feet beneath the ground surface (Plate 7). Groundwater present in thick stream terrace deposits to the east of Aquilla Creek make it highly probable that portions of the left bank of the outlet discharge channel will suffer seepage and soft conditions through the life of the project, possibly causing maintenance problems downstream from the training walls (Plate 9).

e. Groundwater, Post-construction. No groundwater observation wells have as yet been installed at the project to monitor post-construction groundwater conditions.

SECTION 6 - FOUNDATION CONDITIONS

6-01. General. The embankment is founded on residual and alluvial overburden overlying the clay shales of the Woodbine and Del Rio Formations as shown on Plates 11 and 12. The outlet works is founded on clay shales and sandstones of the Woodbine Formation. The spillway was excavated into residual overburden and the primary strata of the Eagle Ford and Woodbine Formations.

6-02. Exploration program. Subsurface investigation at the Aquilla damsite occurred in several stages. The first borings made for site exploration in the general vicinity of the dam were drilled in 1940 and 1964. The first borings drilled at the actual damsite, however, were made in 1972. Four core borings and one auger boring were made along the axis of the dam. Additionally, one core boring was made near the centerline of the spillway location. Most of the 1972 borings were electrically logged for correlation and stratigraphic interpretation. The details of the 1972 borings appear in Design Memorandum No. 3, Phase I - Plan Formulation. During 1973 and 1974 additional borings were made at the damsite as follows: Four Auger borings, 26 core borings, and 1 fishtail boring were made. All the borings were like the earlier borings, "E-logged." This set of borings is detailed in Design Memorandum No. 3, Phase II - Project Design. The final stage of pre-construction field investigation occurred during 1975. The borings of 1975 are detailed in Design Memorandum No. 7, Embankment, Spillway, and Outlet Works. The 1975 borings can be broken down as

follows: 17 were auger borings, 13 were core borings, and 5 were fishtail borings. Electric logs were run in most of the borings. Additional stratigraphic data was obtained during construction from geological maps made of the inspection/cutoff trench under the axis of the dam and also from materials logged during drilling of borings for installation of piezometers and slope indicators.

6-03. Embankment foundation. The embankment foundation can be conveniently divided into four reaches consisting of 1) the right abutment, 2) the floodplain, 3) the left abutment, and 4) the extreme left abutment. The main difference in the reaches is in thickness and type of both overburden and primary strata. The primary strata, consisting of Woodbine and Del Rio clay shales, have different engineering properties. The Woodbine, varies in thickness and material type across the site. In contrast, the Del Rio which underlies the Woodbine to significant depth (average of 70 feet) has relatively uniform material properties.

a. **Right abutment.** The right abutment soil strata from station 0+00 to station 42+80, consist of a relatively thin veneer of overburden overlying primary clay shales (Plate 11). The strata in the reach from station 53+20 to station 60+35 are very similar to the right abutment strata and are included with the right abutment for description and design purposes. The groundwater in the right abutment foundation varied from about 21 to 54 feet below the ground surface at the time the borings were made.

(1) **Overburden.** The overburden ranges in thickness from 1 to 11 feet and varies from clayey gravel and sand to high plasticity clay.

(2) **Primary.** The primary strata immediately underlying the overburden consists predominantly of non-calcareous sandy to waxy Woodbine clay shale varying in thickness from about 33 to 75 feet. These materials are weathered to a depth varying from 5 to 41 feet below top of primary. More significantly, three persistent, slicken-sided zones of waxy clay shale are present at various depths throughout the right abutment foundation. A sandstone layer varying from 9 to 15 feet thick is present near the base of the Woodbine from station 0+00 to station 42+80. The Del Rio formation underlies the Woodbine and consists of intact calcareous shale with shaly limestone zones.

b. **Floodplain.** The floodplain strata (Plate 11) from about station 42+80 to station 53+20, consist of a relatively thick layer of alluvial overburden overlying primary shales. The groundwater table in the floodplain was encountered from 13 to 28 feet below the ground surface.

(1) **Overburden.** The overburden ranges from high plasticity clay and low plasticity sandy clay in the upper 15 feet to interbedded sandy clay, sandy gravel, and clayey sand below 15 feet. The overburden is from 11 to 36 feet thick.

(2) **Primary.** The Woodbine and Del Rio Formations are the primary strata of engineering importance in the floodplain. The Woodbine consists of a relatively thin wedge of unweathered, intact, sandy

clay shale with occasional sandstone and limestone layers. The Woodbine generally varies in thickness from 1 foot to 12 feet, increasing in thickness toward the left abutment. Geologic interpretation of E-log of boring 3F-31 indicates that the Woodbine may increase sharply to a thickness of about 42 feet at station 53+20 as the section approaches the left abutment. The Del Rio underlies the Woodbine, as previously described.

c. Left abutment. Left abutment soil strata, from about station 60+35 to station 93+20, consist of an alluvial terrace deposit overlying shales. The groundwater table varies from 19 to 39 feet below the ground surface in the left abutment.

(1) Overburden. The overburden soils generally range from low to high plasticity sandy clay in the upper 10 to 20 feet to interbedded, clayey sands and gravel deeper in the section. The overburden varies in thickness from 53 feet at station 72+00 to 4 feet at station 93+20.

(2) Primary. The primary strata immediately beneath the overburden consist of weathered to unweathered Woodbine with sandstone and limestone layers. Near the base of the Woodbine there are two waxy clay shale zones which contain scattered low-angle slickensides. Two additional, poorly defined zones of slickensided clay shale are present higher in the section from station 86+00 to station 93+20. Weathering in this reach is limited to a thin layer immediately beneath the overburden, except from about station 83+00 to station 93+20

where it extends to a maximum of about 28 feet below the top of primary.

d. Extreme left abutment. The soil strata on the extreme left abutment, from station 93+20 to about station 118+90, consist of a thin veneer of residual overburden overlying primary strata. The groundwater table in this reach varies from about 7 to 25 feet below the ground surface.

(1) Overburden. The overburden soils consist of 2 to 6 feet of low to high plasticity clays and silty and clayey sands.

(2) Primary. The Woodbine underlies the overburden to depths greater than 90 feet in this reach. A weathered to unweathered Woodbine sandstone layer up to 29 feet thick is present immediately beneath the overburden. The sandstone is underlain by unweathered clay shale containing sandstone and thin limestone layers.

6-04. Outlet works foundation. The outlet works structures is founded on the unweathered clay shales of the Woodbine formation (Plate 11).

a. Intake tower foundation. The intake tower is founded on unweathered Woodbine clay shale at about elevation 499. Two zones of waxy, slickensided clay shale are present below the base of the tower; the first, about 6 feet thick, is located about 4 feet below the tower base, and the second, about 12 feet thick, is located about 18 feet below the tower base. The remaining clay shales are intact and contain sandstone layers of varying thickness and hardness.

b. Conduit. The cut-and-cover conduit was founded in the Woodbine throughout its length.

c. Stilling basin and chute. The stilling basin and chute are founded directly on the Woodbine. The foundation for these structures is similar to that of the outlet works tower except that the waxy slickensided zones are present at a lower elevation at the stilling basin. The chute slope was cut through an upper slickensided zone and the stilling basin slab was founded on a lower slickensided zone.

d. Approach and discharge channels. Excavation for the approach channel cut through a maximum of about 34 feet of overburden and about 7 feet of Woodbine. Excavation for the discharge channel cut through a maximum of 19 feet of overburden materials and about 23 feet of Woodbine. Maximum depth of cut was about 41 feet for the approach and 54 feet for the discharge channel.

6-05. Spillway excavation. The spillway was excavated through overburden and weathered primary strata of the Eagle Ford and Woodbine formations (Plate 12). The overburden materials consisted generally of low to moderately high plasticity sandy clay and clayey sand. The primary materials consisted predominantly of moderately to highly weathered shale and sandstone.

SECTION 7 - EMBANKMENT DESIGN

7-01. Design considerations. Embankment sections, shown on Plate 3, were designed to safely and economically accommodate the previously described foundation reaches. Foundation conditions existing in the right abutment, floodplain, left abutment and extreme left abutment were described in Section VI.

a. Design factors. The following design factors were considered applicable to all embankment reaches:

(1) High excess pore pressures similar to those encountered at Waco Dam were likely to develop in the Woodbine clay shales at Aquilla Dam. These pore pressures might be transmitted undiminished along slickensides, fractures, pervious bedding planes and formational contacts as occurred at Waco Dam. Also, low shear strengths approaching residual conditions were anticipated in the Woodbine clay shale due to its slickensided nature.

(2) A select impervious cap was provided on the embankment crest and the 1 vertical on 3 horizontal slopes. This cap was designed to limit surface cracking due to drying and to prevent surface water infiltration into the upper slopes of the embankment. These properties should limit surface sloughing on the upper slopes during extended wet periods.

(3) The impervious, select impervious, and random fill materials came from borrow areas.

(4) Practically all materials from required excavation were used in the semi-compacted zones.

(5) An inspection trench was provided at the embankment centerline.

b. Right abutment. The right abutment embankment was designed based on the following factors:

(1) Embankment design for the right abutment was controlled by the engineering properties of the Woodbine clay shales. Similar material was encountered at Waco Dam wherein a foundation slide occurred through the Woodbine during embankment construction. Analysis of the Waco Dam slide indicates that the shear strength of the Woodbine clay shale can be low and considerably less than indicated by peak laboratory strengths of intact material.

(2) The right abutment section was designed to resist 80 percent excess pore pressure in the foundation assuming a factor of safety slightly greater than unity for a failure through the slickensided clay shale and assuming the mobilized shear strength was that of low but somewhat higher than residual conditions.

(3) The embankment section shown for the right abutment was also used in the reach from station 53+20 to station 58+35 due to similar foundation conditions.

c. Floodplain and left abutment. Floodplain and left abutment designs were controlled by the shear strength of thick overburden strata overlying the Woodbine clay shale.

d. Outlet works. The embankment slopes over the outlet works were designed to be flatter than the remainder of the left abutment embankment. This was done due to the sensitivity of the outlet works structures to movement, particularly lateral spreading under embankment load and the increased potential for such to develop due to the weak, slickensided clay shale beneath these structures. Additionally, the outlet works conduit was extended well beyond the embankment toes to accommodate flatter slopes in the event that as-designed embankment slopes above the conduit had to be flattened should spreading initiate.

e. Extreme left abutment. The embankment on the extreme left abutment was designed with relatively steep slopes. The embankment averaged about 14 feet high in this reach and was founded on a thin overburden layer overlying sandstone.

f. Embankment design and construction economy. The following design considerations resulting in construction economy were incorporated in the embankment with little or no loss in safety:

(1) Minimize the size of the impervious fill zone and thus minimize the amount of material subject to close moisture control.

(2) Minimize compactive effort in the berms by reducing the specified effort to two passes of a pneumatic roller on a 10-inch lift with the same moisture control as the random fill. On the completion contract the moisture control was totally removed from the semi-compacted fill. The only requirement was that the materials shall neither be sloppy-wet nor crusted-dry.

(3) Essentially all semi-compacted fill used in the embankment and berms was from required excavation. This eliminated any waste of required excavation materials and therefore saved the cost of borrow excavation to obtain materials for semi-compacted fill.

(4) Upstream riprap protection was not used except on the upper 1 vertical on 3 horizontal slope above elevation 574.5 where a 12 inch layer of stone protection was placed. The slope below elevation 564.5 varies from 1 vertical on 8 horizontal to 1 on 12. Experience has shown that flat slopes, particularly in cohesive material, need not be riprapped. Embankment stone protection was obtained on site by selective excavation, stockpiling and processing of limestone ledge rock from required spillway excavation rather than much more costly commercially produced riprap materials. Processing of stockpiled rock from the spillway excavation was described previously in para 4-02 f. Similarly, an erosion resistant random rock zone, consisting of unprocessed excess rock materials from required excavation, was placed in the downstream toe of the floodplain embankment to replace a riprap band required by earlier designs.

(5) Downstream inclined and horizontal drainage blankets fall in the same economy category as does riprap. Experience and theory both show that drainage blankets are not needed in clay fill embankments with long flat berms similar to Aquilla. Consequently, these were not included in the Aquilla embankment.

(6) The diversion dike and cofferdams were incorporated into the closure section, thus eliminating additional fill for these structures.

7-02. Laboratory testing. Laboratory testing was performed during the design stage on samples from the embankment foundation, outlet works foundation, spillway and borrow areas. The overburden materials in the floodplain area, the embankment, and the left abutment consists primarily of sandy clay and clayey sand. Shear strength tests (Q, R, and S triaxial) and consolidation tests were performed on samples from the overburden. Primary materials from the embankment area consist primarily of weathered and unweathered Woodbine clay shales and unweathered Del Rio clay shale. On the weathered and unweathered Woodbine samples Q and R triaxial and standard, residual, and presplit direct shear tests were performed. Presplit and residual tests were performed on originally intact Woodbine shales to determine a likely range of strength for slickensided materials. On the intact Del Rio shale samples, Q triaxial and standard direct shear tests were performed. From the borrow areas, seven sets of bag samples were selected for Standard AASHO compaction, Q and R triaxial compression, S direct shear, and consolidation tests. Q tests were performed on samples compacted to 100 percent of maximum Standard density at optimum moisture content and on samples compacted to 95 percent of maximum Standard density at optimum minus 3 percent, optimum, and optimum plus 3 percent. R tests were performed on samples compacted to 95 percent Standard density at optimum. S tests were performed on samples compacted to 95 and 100 percent Standard density at optimum moisture content. From the spillway excavation area two composite bag samples of weathered Eagle Ford and Woodbine shales were obtained and tested. Compaction, Q and R triaxial compression, direct shear, and controlled expansion tests were performed on the samples.

7-03. Embankment design data. Based on analysis of laboratory test results, the following strengths were adopted for embankment design:

a. Overburden. Design parameters assumed for the right abutment, floodplain, and left abutment overburden strata were as follows:

Moist Unit Weight - 127 pcf
Saturated Unit Weight - 129 pcf

Overburden Shear Strength

Type	c TSF	phi Degrees	Remarks
Q	0.7	5	All reaches except floodplain below El. 485
Q	0.4	3	Floodplain below El. 485
R	0.2	12	All reaches
S	0.0	24	Right abutment
S	0.0 0.0	24 30	(LL > 35) (LL < 35) Left abutment and floodplain

b. Primary. The results of laboratory shear strength tests on intact specimens are not considered representative of field strengths for slickensided clay shales. Presplit and residual test results were used as a guide in estimating the shear strength of the slickensided Woodbine. Values intermediate between residual and peak were used as estimates for shear strength of the weathered Woodbine clay shale and intact unweathered Woodbine. A range of assumed shear strengths and unit weights used for the primary materials follows:

Material Type	c TSF	phi Degrees	Unit Weight - pcf	
			Moist	Saturated
Weathered Woodbine	0	10-14	124	126
Unweathered Slickensided Woodbine	0	5-14	133	135
Unweathered Intact Woodbine	0	25	133	135
Unweathered Sandy Shale and Sandstone (Woodbine)	0	32	133	135

These strengths were used with pore pressure assumptions ranging from 0 to 100 percent in the stability analyses. Design strengths were not assigned to the Del Rio clay shales because experience and laboratory tests have shown them to be stronger than Woodbine materials; therefore, assumed failure planes were more critical in the shallower Woodbine.

c. Embankment fill.

(1) **Borrow.** The impervious zone required clays with a liquid limit greater than 40. All overburden soils except topsoil were considered acceptable for random and semi-compacted zones. The distribution of materials in the borrow areas were such that both the random and impervious zones contain similar materials. Both zones were assumed to have the following design properties:

Liquid Limit - 46
Plasticity Index - 33
Moist Unit Weight - 123 pcf
Saturated Unit Weight - 126 pcf

<u>Shear Strength</u>		
<u>Type</u>	<u>c</u> <u>TSF</u>	<u>phi</u> <u>Degrees</u>
Q	0.9	2
R	0.2	13
S	0.0	24

Design strengths are based on fill compacted to 95 percent Standard density at optimum plus 2 percent moisture content.

(2) **Materials from required excavation.** Materials from outlet works and spillway excavation were used in the semi-compacted zones. The excavated materials consisted mostly of weathered clay shale from the spillway but also contained overburden and unweathered Woodbine from the outlet works area. Due to lack of laboratory testing of this material, design unit weights for required excavation materials were assumed equal to 90 percent of that for compacted borrow, and design shear strengths were assumed equal to 70 percent of the strength assumed for compacted borrow. Subsequent laboratory test results during and after construction indicated that these assumptions were very conservative. There was little actual difference between the density and shear strengths of the semi-compacted, random, and impervious zones of the initial embankment. Semi-compacted fill was not tested during the completion contract.

7-04. **Stability analyses.** Stability analyses were performed during design for the right abutment, floodplain and left abutment embankment sections. The results of these analyses are summarized below:

a. Right abutment.

(1) Failure through slickensided clay shale, El. 499.

Condition	Strength	Method	Remarks	Safety Factor
End of Construction (Downstream slope)	Q,S	Wedge	80% pore pressure, $\phi = 10^\circ$	1.17
End of Construction (Downstream slope)	Q,S	Wedge	80% pore pressure, $\phi = 14^\circ$	1.34
Partial Pool (Upstream slope)	$S, \frac{R+S}{2}$	Wedge	$\phi = 10^\circ$	1.40
Partial Pool (Upstream slope)	$S, \frac{R+S}{2}$	Wedge	$\phi = 14^\circ$	1.75
Steady Seepage (Downstream slope)	$S, \frac{R+S}{2}$	Wedge	$\phi = 10^\circ$	1.15
Steady Seepage (Downstream slope)	$S, \frac{R+S}{2}$	Wedge	$\phi = 14^\circ$	1.46

(2) Failure through slickensided clay shale, El. 503.

Condition	Strength	Method	Remarks	Safety Factor
End of Construction (Downstream slope)	Q,S	Wedge	80% pore pressure, $\phi = 10^\circ$	1.18
End of Construction (Downstream slope)	Q,S	Wedge	80% pore pressure, $\phi = 14^\circ$	1.34
Partial Pool (Upstream slope)	$S, \frac{R+S}{2}$	Wedge	$\phi = 10^\circ$	1.33
Partial Pool (Upstream Slope)	$S, \frac{R+S}{2}$	Wedge	$\phi = 14^\circ$	1.69
Steady Seepage (Downstream slope)	$S, \frac{R+S}{2}$	Wedge	$\phi = 10^\circ$	1.12
Steady Seepage (Downstream slope)	$S, \frac{R+S}{2}$	Wedge	$\phi = 14^\circ$	1.44

A tabulated summary of end-of-construction condition stability analyses and manual computations for the right abutment is shown on Plate 13. Manual computations for partial pool and steady seepage conditions are shown on Plates 14 and 15, respectively.

(3) **Failure through weathered Woodbine.** Failure surfaces assumed through the weathered Woodbine have higher calculated factors of safety than those through the slightly deeper slickensided Woodbine zones.

b. Floodplain.

Condition	Strength	Method	Remarks	Safety Factor
End of Construction (Downstream slope)	Q	Wedge	Failure through overburden El. 473	1.47
Partial Pool (Upstream slope)	$S, \frac{R+S}{2}$	Wedge	Failure through fill at base of embankment	1.95
Steady Seepage (Downstream slope)	$S, \frac{R+S}{2}$	Wedge	Failure through fill at base of embankment	1.68

The locations of critical failure surfaces for the floodplain analyses are shown on Plate 16.

c. Left abutment.

Condition	Strength	Method	Remarks	Safety Factor
End of Construction (Downstream slope)	Q	Wedge	Failure through overburden El. 500	2.44
End of Construction (Downstream slope)	Q	Circular Arc		2.90
Partial Pool (Upstream slope)	$S, \frac{R+S}{2}$	Wedge	Failure through overburden El. 537	1.48
Partial Pool (Upstream slope)	$S, \frac{R+S}{2}$	Circular Arc		1.50
Steady Seepage (Downstream slope)	$S, \frac{R+S}{2}$	Wedge	Failure through fill at base of embankment	1.63
Steady Seepage (Downstream slope)	$S, \frac{R+S}{2}$	Circular Arc		1.73

The locations of critical failure surfaces for the left abutment are shown on Plate 17. For all embankment sections, only downstream slopes were analyzed for end of construction condition since the upstream slopes are less critical.

d. Extreme left abutment. The extreme left abutment embankment section was judged by inspection to be stable and conservative. Stability analyses were not performed. This assumption is still considered appropriate. The embankment height is a maximum of 29 feet in this reach and averages about 14 feet. The foundation consists of a thin veneer of overburden overlying sandstone.

e. Rapid drawdown condition. The embankment is not expected to be subjected to a rapid drawdown condition because the materials com-

prising the upstream slopes are relatively impervious and the expected duration of any high pool will be brief. Therefore, saturation of the embankment is not expected to occur at a high elevation.

f. Computations. Computer programs supplemented with manual computations were used to perform stability analysis computations.

(1) Wedge method with excess pore pressures. Program SSW-039 was used to perform stability computations for the end of construction condition for the right abutment embankment. This program was used to analyze the end of construction condition using S-strengths with excess pore pressure in primary strata and Q-strengths in the embankment and overburden materials. Program execution was via Timesharing to the GE-635 computer at the Waterways Experiment Station. Computational method and accuracy were checked by hand for the right abutment end-of-construction condition (Plate 13).

(2) Wedge method. Program 41-R3-C102 was used to perform stability computations for the end of construction case (using Q-strengths), partial pool case (using S & $\frac{R+S}{2}$ strengths) and steady seepage case (using S & $\frac{R+S}{2}$ strengths). This program was used to analyze the floodplain and left abutment embankment sections for the three conditions cited. The program was also used to analyze the right abutment section for partial pool and steady seepage conditions. Program execution was via Cope 1200 (batch) to the GE-437 computer at Southwestern Division. Computational method and accuracy were checked by hand for the right abutment section for partial pool (Plate 14) and steady seepage (Plate 15) conditions.

(3) Circular arc method. Program WES-104 was used to perform stability computations for the end of construction case (using Q-strengths), the partial pool case (using S & $\frac{R+S}{2}$ strengths) and the steady seepage case. This program uses the modified swedish method of analysis.

g. Evaluation of analyses. In view of the foundation performance and fill conditions encountered during construction, the shear strengths and unit weights indicated by record samples, and the actual excess foundation pore pressures measured during construction, the analyses conducted during design are considered appropriate and sufficient. No additional embankment stability analyses will be conducted.

SECTION 8 - EMBANKMENT FILL CONSTRUCTION CONTROL

8-01. General. Embankment fill construction was monitored through an extensive Government quality assurance field testing program. The testing program was based on a minimum sampling frequency of one set of tests that included an in-place density per 3000 cubic yards of compacted fill. On each sample, the moisture content, liquid limit, and bar linear shrinkage were determined (Note: For the initial contract only, in-place densities were performed on each sample also). On approximately every fifth sample, a grain size analysis was conducted and the plastic limit and in-place density were determined in addition to the above tests. On approximately every tenth sample, a Standard compaction test was run in addition to the other tests. Additional testing of moisture content was performed at the discretion of the Contracting Officer as necessary to assure contract compliance. Unacceptable material, unacceptable in-place moisture content or unacceptable compaction as indicated by the tests, resulted in either reworking of the fill and retesting, or removal of the material in question. Experience on this and other embankment projects have shown for clays that if the materials, lift thickness, uniform moisture content, and compactive effort are in accordance with the specifications, then the density achieved is essentially always greater than 95 percent of maximum Standard density. These items when combined with the controls afforded by the Liquid Limit Correlation Method form a superior fill placement quality assurance program in these type materials. The data from the field tests on random, impervious, and semi-compacted

fill are tabulated on Plates 18 through 20. The average liquid limits, water contents, and percent compaction are as follows:

a. Initial contract:

	<u>Liquid Limit</u>	<u>Water Content</u>	<u>Moisture Variation Frm Optimum, %</u>	<u>% Compaction</u>
Impervious Fill	58.7	25.7	+1.7	107.1
Random Fill	52.8	23.0	+0.8	107.0
Semi-Compacted Fill	45.0	18.7	-0.4	108.2

b. Completion contract:

	<u>Liquid Limit</u>	<u>Water Content</u>	<u>Moisture Variation Frm Optimum, %</u>	<u>% Compaction</u>
Impervious Fill	64.1	26.1	+1.3	108.2
Random Fill	39.6	17.6	-0.2	107.4

During the completion contract, semi-compacted fill placement was not monitored by laboratory testing. The semi-compacted fill berm was needed only for weight in regard to embankment stability.

8-02. Liquid limit correlation method. The primary method of fill placement control was the "Liquid Limit Correlation Method". The Liquid Limit Correlation Method is based on the correlation which can be established between the liquid limits and the other engineering properties of soils. Laboratory tests on the soils used in the embankment are used to establish correlation curves which represent the relationship between the maximum dry density and liquid limit of embankment fill and also between the optimum moisture content and liquid limit of fill materials. Thus, a liquid limit value determined by

testing an embankment sample is used in conjunction with the correlation curves to determine the maximum dry density and the optimum moisture content for that sample. These values are then compared to the in-place density and in-place moisture content to determine compliance of field compaction and moisture to contract specifications or to desired minimum values.

a. Establishment of correlation curves. The specifications for the Aquilla embankment required that the Government laboratory conduct compaction and classification tests to be used in the establishment of correlation curves. Compaction and classification tests were performed on materials representing the entire range of materials from the borrow area and from the required excavations which were used in constructing the embankment. The specifications stipulated the minimum number of tests that would be performed on each type of materials. They included provisions for preparing the clay shale samples by three different methods for liquid limit tests as described in EM 1110-2-1906, Laboratory Soils Testing.

b. Updated correlation curves. Results of all quality assurance tests were furnished to the Chief, Geotechnical Branch, for plotting and continued evaluation of the accuracy of correlation curves to be used in compaction control. Separate correlation curves are sometimes set up for each type of fill material such as overburden and primary materials. However, for the Aquilla embankment construction, a good overall correlation was obtained using only one curve at a time. The correlation curve was furnished to the Resident Engineer prior to placement of fill, and it was updated periodically during construction.

c. Use of correlation curves. The relationship of field moisture content and density to specified or desired values was determined by the Government's laboratory personnel for each embankment control sample. After determination of the liquid limit, the correlation curve of liquid limit versus optimum moisture content was used to obtain the optimum moisture content for comparison with the moisture content of the embankment control sample. For impervious fill material, the moisture content was required to be within the limits of 3 percentage points above optimum and optimum. After compaction, random, and select impervious fill moisture contents were required to be within the limits of 3 percentage points above and 2 percentage points below optimum moisture. For the initial embankment contract, the upper and lower limits of moisture content for the semi-compacted fill were the same as those specified for random material. No moisture control was specified for the semi-compacted zone of the completion embankment. Field density was compared with the estimated maximum laboratory density by using the correlation plot of liquid limit versus 100 percent Standard compaction density. Percent density was obtained by dividing the density of the control sample by the laboratory density for 100 percent Standard compaction. The target or desired minimum density was equal to or greater than 95 percent, although no minimum density was specified. As described earlier, if the specifications concerning material type, lift thickness, moisture and compactive effort are met, the percent density achieved and computed in this manner is always greater than the desired minimum for these type materials. The usual range of compacted values is from 95 percent to 120 percent compaction.

8-03. Construction inspection by geotechnical engineers. Foundation preparation and fill construction was also inspected and evaluated (on a continual basis) by responsible geotechnical design engineers throughout both the initial and completion contracts. All foundation approval was performed by a geotechnical engineer. Construction engineers involved were very cooperative with the design engineers in trying to achieve the intent of the design and in calling any discrepancies to the design engineers attention. This inspection and evaluation during construction by the geotechnical design engineers is routine in the Fort Worth District and is considered necessary for the construction of all massive earth and/or rock fill dams.

SECTION 9 - RECORD SAMPLES

9-01. Record sampling program. A total of 45 record samples were taken from the Aquilla embankment fill for testing. The record sample test data are tabulated on Plate 21. The numbers of record samples taken from each fill type on each contract are as follows:

a. Initial embankment contract:

<u>Fill Type</u>	<u>Numbers of Record Samples</u>
Impervious.....	6
Random.....	8
Semi-Compacted.....	<u>6</u>
Total	20

b. Completion contract:

<u>Fill Type</u>	<u>Numbers of Record Samples</u>
Impervious.....	9
Random.....	<u>16</u>
Total	25

Semi-compacted fill placed during the completion contract was not subjected to record sample testing. Stationing of the record sample locations extends from station 12+25 to station 86+00. Undisturbed and bag samples were recovered from each record sample site and subjected to the following tests:

- 1) Visual Classification.
- 2) Grain Size Analysis (Mechanical and Hydrometer).

- 3) Atterberg Limits.
- 4) Bar Linear Shrinkage.
- 5) Specific Gravity.
- 6) Consolidation.
- 7) Standard Compaction.
- 8) Direct Shear.
- 9) "Q" and "R" Triaxial Shear.

The tests were performed at the Southwest Division Laboratory, Dallas, Texas, with the exception of six of the record samples which were tested at the Missouri River Division Laboratory, Omaha, Nebraska.

9-02. Record sample strength testing. Undisturbed specimens carved from the record samples were subjected to the following strength tests:

- 1) Unconsolidated - Undrained ("Q" - Triaxial Shear); three or four specimens per record sample.
- 2) Consolidated - Undrained ("R" - Triaxial Shear); three or four specimens per record sample.
- 3) Consolidated - Drained ("S" - Direct Shear); three specimens per record sample.

The strength test data are summarized in plots on Plates 22 through 27. The design strength envelopes assumed during design are also

shown on the plots for comparison purposes. A tabulated comparison of the strength test results to the assumed design strength envelopes is shown in Table 9.01.

9-03. Record sample strength testing results. The design strength envelopes were chosen so that approximately two-thirds of the test strengths of the material tested prior to design fell above the strength envelope. It can be observed from Table 9.01 that the composite test results for the initial and completion contract indicate that 72 percent of the "R"-test strengths and 67 percent of the "S"-test strengths are above the design strength envelopes. This compares favorably with the design assumptions. It is also observed that 38 percent of the "Q"-test strengths are above the design envelope. However, since "Q"-strength governs only end of construction stability, the successful topping out of the embankment fill indicates that the "Q"-strengths were obviously adequate. This was further borne out by sensitivity studies of stability analyses which indicated that minor variations in embankment "Q"-strengths resulted in only minor variations in calculated safety factors.

TABLE 9.01

Record Sample Strength Test Result Summary.

a. Initial Contract.

TEST TYPE	FILL TYPE	NUMBER OF TESTS	% WITH STRENGTH EQUAL TO OR ABOVE DESIGN ENVELOPE
Q	Impervious	24	54
	Random	32	56
	Semi-Compacted	24	58
R	Impervious	24	92
	Random	32	87
	Semi-Compacted	22	77
S	Impervious	18	56
	Random	24	67
	Semi-Compacted	18	83

b. Completion Contract. (See Note)

TEST TYPE	FILL TYPE	NUMBER OF TESTS	% WITH STRENGTH EQUAL TO OR ABOVE DESIGN ENVELOPE
Q	Impervious	33	9
	Random	57	30
R	Impervious	33	39
	Random	48	73
S	Impervious	27	44
	Random	48	77

c. Totals - Both Contracts.

TEST TYPE	FILL TYPE	NUMBER OF TESTS	% WITH STRENGTH EQUAL TO OR ABOVE DESIGN ENVELOPE
Q	ALL	170	38
R	ALL	159	72
S	ALL	135	67

NOTE: Semi-compacted fill placed during the completion contract was not subjected to record sample testing.

SECTION 10 - INSTRUMENTATION PROGRAM

10-01. General. Instrumentation for the project consists of piezometers, inclinometers, settlement plates, surface reference marks, and outlet works reference marks. One hundred and six piezometers were installed to measure pore pressures. (Pore pressure in this report is defined as the ratio of increased pore pressure to increased fill load expressed as a percentage.) Eighty of these were open system and 26 were pneumatic type piezometers. Six settlement plates were installed to measure settlement of the foundation. To monitor horizontal deflection of the foundation and outlet works excavation, 16 inclinometers were installed. To monitor surface movement of the embankment fill and outlet works channel side slopes, 31 surface reference marks were installed. Movements of the outlet works conduit and stilling basin walls are monitored by a series of reference marks embedded in the concrete. As a general statement, instrumentation readings have indicated no unusual movement nor any structural distress that would adversely affect stability of the embankment or outlet works. An analysis of instrumentation readings is presented in Section 11. Instrumentation locations are shown in plan view on Plate 28 and a schedule of instrumentation is presented on Plate 29.

10-02. Piezometers. Piezometers for the project were installed with government forces during each of the two separate construction contracts. For the initial contract, 43 Casagrande type open system piezometers were installed: P-1 through P-38 and P-A through P-E. P-1

through P-38 were the porous plastic tube type as manufactured by Slope Indicator Company, Seattle, Washington with $\frac{3}{8}$ - inch diameter PVC risers. P-A through P-E were galvanized steel well point type piezometers. In addition to the open system piezometers, 18 pneumatic piezometers, PP-1 through PP-18, were installed during the initial contract. For the completion contract, 45 additional piezometers were installed as follows: P-38 through P-75 were the porous plastic tube type and PP-19 through PP-26 were the pneumatic type. In general, it should be noted that the open system piezometers functioned adequately while the pneumatic piezometers functioned erratically and were unreliable. Typically the pneumatic piezometers recorded higher pore pressures than the open system piezometers even though both types monitored the same strata. The main reason that pneumatics were selected for use on the project was to minimize potential problems with time lag in pore pressure response in the low permeability clay shale. In actuality even in the open system piezometers time lag proved to be an insignificant problem. Most of the problems encountered with the pneumatic piezometers was due to their complexity. A typical installation schematic is shown on Plate 28. The pneumatics used for the Aquilla project were manufactured by Slope Indicator Company, Seattle Washington. The transducers utilize a flexible diaphragm and a ball check valve. To take a reading, the force of the diaphragm due to water pressure is equalized by gas pressure applied through the input tube. When the gas and water pressure are equalized the ball valve closes and the gas pressure, which registers on a gage, is equal to the pore pressure. One of the disadvantages of this type

of instrument is that the displacement of the check valve that occurs at the instant of reading will artificially increase the pore pressure. This problem is not severe in high permeability soils where increased pressures can quickly bleed off, but in low permeability material such as clay shale, measured pressure will be too high. Other problems experienced with the pneumatic piezometers were gas flow rate problems when long tubes were used and sticking check valves.

10-03. Inclinometers. Sixteen inclinometers were installed at the project. Inclinometers I-1 through I-12 were installed during the initial contract and I-13 through I-16 were installed during the completion contract. The inclinometers were constructed of 3.34-inch diameter grooved PVC casing, the type manufactured by Slope Indicator Company.

10-04. Settlement plates. Six settlement plates were installed at the project. Settlement Plates SP-1 through SP-4 were installed during the initial contract and settlement Plates SP-5 and SP-6 were installed during the completion contract. All the settlement plates consist of 3-foot square, 1/4-inch thick steel plates placed on the embankment foundation with steel riser pipes extended through the fill.

10-05. Surface reference marks. During the initial contract, 31 surface reference marks were installed. No surface reference marks were installed during the completion contract. The surface reference marks consist of sections of 6-inch diameter steel pipe filled with concrete with brass survey monuments installed in the tops.

10-06. Outlet Works Reference Marks. Reference marks were installed along the invert of the conduit, on the intake tower, along the discharge chute, and on the stilling basin walls. The reference marks consist of bronze bolts embedded in concrete.

SECTION 11 - INSTRUMENTATION ANALYSIS

11-01. Piezometers.

a. General. Foundation piezometers were installed for the initial embankment of Aquilla Dam in two general locations, line A and line B. Line A is near the right abutment and extends from station 17+00 to station 21+00. Line B is near the end slope of the initial embankment and extends from station 24+70 to station 26+50. Foundation piezometers were also installed during the completion contract in four lines, designated line C through line F. Instrumentation line C is located at station 41+ on the right abutment of the Aquilla Creek floodplain. Line D is located at station 47+ on the west side of Aquilla Creek and line E is located at station 53+ on the east side of Aquilla Creek. Line F is located at station 70+ on the east side of the outlet works. A plan view of the instrumentation location is shown on Plate 28.

b. Instrumentation line A. Piezometers for instrumentation line A were installed in three cross-sections, A, A₁, and A₂ as shown on Plates 30 through 33. Piezometers for line A were installed in several different foundation strata in both the Woodbine and the Del Rio formations. However, most of the piezometers for line A were installed in the "waxy" clay shale units of the Woodbine formation. These clay shale units were expected to develop the highest pore pressure during construction. At the right abutment there are two waxy units separated by a thick sandstone unit. Also, numerous piezometers were

installed at the contact between the Woodbine and Del Rio formations and one piezometer was installed deeper in the Del Rio. Thirty-seven piezometers were installed along line A, but 20 were subsequently abandoned and are no longer being read. A number of the piezometers were damaged by the Contractor's operations and the remainder were abandoned because of erratic, meaningless readings. However, sufficient piezometers were operational to provide adequate data on pore pressure development in the foundation. Plot of piezometric and fill elevation versus time are shown on Plate 43 through 56 for all functional piezometers.

c. Analysis of piezometric readings for line A. Based on the Waco Dam experience, it was expected that the highest pore pressures would develop in the "waxy" Woodbine clay shale (which is the same geologic unit as the Pepper Shale in the foundation at Waco Dam) or at the Woodbine-Del Rio contact. However, along line A the highest pore pressure development as recorded by open system piezometers, occurred deeper in the Del Rio formation rather than in the "waxy" Woodbine or at the Woodbine-Del Rio contact. Even though the Del Rio formation is lower in elevation, the piezometric elevations in the Del Rio piezometers were almost as high as the elevations recorded in the "waxy" Woodbine units. This behavior occurred despite the fact that the Del Rio clay shale is much stronger than the clay shales of the Woodbine formation and is partially cemented with calcium carbonate where the Woodbine clay shales are non-calcareous. Apparently, the "waxy" units of the Woodbine have significant fracture permeability compared to the

permeability of the Del Rio formation. Pore pressure that developed were able to bleed into a sandstone layer that underlies the "waxy" Woodbine. Water level in the sandstone layer remained the same as static groundwater conditions through out the embankment construction period. The time lag exhibited by piezometer P-12, with its tip in the Del Rio shale reinforces this reasoning that the Del Rio clay shale is much more impervious than the "waxy" Woodbine. To date, the piezometric level of this piezometer has stopped increasing but has not decreased. The highest pore pressure response recorded in P-12 was 72 percent. In the Woodbine P-A and PP-8 showed the highest pore pressures, 64 percent and 76 percent, respectively. Along line A the piezometer readings indicate no lateral transmission of high pore pressures from the centerline toward either the upstream or downstream toes. This was a significant observation since it was lateral transmission of excess pore pressure toward the downstream toe that contributed to the Waco Dam slide. Piezometer data for line A are detailed on Plate 41.

d. Instrumentation line B. Piezometers for instrumentation line B were installed two cross-section, B and B₁, as shown on Plates 34 through 37. Foundation strata at line B are similar to line A and piezometers were installed in the same basic geologic units that were monitored at line A. Most of the piezometers were installed in the "waxy" clay shale units of the Woodbine formation and at the Woodbine-Del Rio contact. Of the 24 piezometers installed along line B, 15 are inoperative and have been abandoned. Plots of piezometric and fill

elevation versus time for the functional piezometers are presented on Plates 43 through 56.

e. Analysis of piezometer readings for line B. Pore pressure response as a result of fill placement along line B was greatest in the Del Rio formation and was similar to Del Rio pore pressure response for line A. The highest piezometric elevation was 560 feet measured in Del Rio piezometer P-28 (Plate 46). Pore pressure response for piezometers in the "waxy" Woodbine formation along line B was less than expected. The highest piezometric elevation was 534 feet measured in the "waxy" Woodbine piezometer P-26 (Plate 46). Piezometric levels at the Woodbine-Del Rio contact were about the same at line B as in line A. Based on the piezometer readings, there is no evidence of any transmission of high pore pressures toward the embankment toes. Most piezometers at or near the toes demonstrated very little pore pressure response due to fill placement and essentially monitored the groundwater level. It is interesting to note that the percent pressure development in the "waxy" Woodbine strata along line B was less than along line A. The primary factor contributing to this difference was the smaller height of fill at line B. The fractured, jointed Woodbine shale along instrumentation lines A and B has significant fracture permeability which may be the reason that pore pressure development was not high. As fill loading increases, the fracture permeability is reduced thus allowing for the development of higher excess pore pressure. Since line B had a smaller fill height, the fracture permeability was less diminished, and pore pressure response was not as great.

f. Instrumentation of line C. Piezometers for instrumentation line C are shown in profile on Plate 38. Both the open system and pneumatic piezometers were installed in a "waxy" Woodbine clay shale unit just above a thick sandstone unit. Plots of piezometric and fill elevation versus time are shown on Plates 43 through 56.

g. Analysis of piezometer readings for line C. Pore pressure response along line C has been low. The maximum piezometric elevation recorded during construction was 532 feet which was measured in piezometers P-42 (Plate 38). Most piezometers essentially monitored the groundwater table or were dry. The lack of pore pressure response along line C is attributed to the presence of fracturing in the Woodbine shale adjacent to the floodplain. The shale at line C comprises the right abutment of the floodplain and has geologically undergone more stress relief both vertically and laterally than the shale under the initial embankment; and, as a result, the shale has greater mass permeability. A perched water table does not exist at line C. Several of the piezometers were installed in unsaturated strata above the permanent water table. Of all the piezometers installed along line C, only open system piezometer P-42 recorded any significant buildup of pore pressure. Piezometer P-42 measured a maximum pore pressure response of 65 percent.

h. Instrumentation line D. Piezometers for instrumentation line D were installed in several different strata as shown on Plate 39. Most of the piezometers were installed at the contact of the Woodbine and Del Rio formations or in a "waxy" unit of the Woodbine just above

the contact. The thick sandstone unit that exists at line C has apparently been removed by erosion and is not present at line D. Also at line D, piezometers were installed in the overburden strata and in the clay shale of the Del Rio formation. Plots of piezometric elevation and fill elevation versus time are shown on Plate 43 through 56.

i. Analysis of piezometer readings for line D. The maximum piezometric elevations recorded during construction for the strata that were monitored and the corresponding percent pore pressure responses are shown on Plate 42. The highest pore pressure response at line D occurred in the Del Rio. Piezometers in the overburden strata developed essentially no excess pore pressure and essentially monitored the groundwater table. The piezometers at the Woodbine-Del Rio contact and those located above the contact in the Woodbine clay shale developed only moderate pore pressures. The percent pore pressure development for the piezometers installed along line D in the Woodbine clay shale was much less than pressures developed along line A. The primary reason for this behavior is that the Woodbine clay shale encountered during the completion contract along instrumentation lines D and E is very sandy and is borderline sandstone. As a result, it has more rigid structure and is much more pervious than the Woodbine clay shale unit monitored for the initial contract. Also, it should be noted from Plate 42 that P-51 which is located at the Woodbine-Del Rio contact indicated less pore pressure development than adjacent piezometers also located at the contact. This is probably a result of pore pressures bleeding off into the overlying overburden strata. Of

the different strata monitored at line D, the greatest pore pressure response occurred in the Del Rio clay shale. A series of piezometers, P-52, P-73, P-74, and P-75 were installed just above the Del Rio correlation bed as shown on Plate 42. All four of the Del Rio piezometers exhibited piezometric levels higher than the embankment fill. The percent pore pressure response for the Del Rio piezometers along line D was similar to that recorded along lines A and B. This was expected since the Del Rio shale at both locations had similar characteristics. It was massive, calcareous, and had high strength compared to Woodbine clay shale which has relatively low strength, and is highly fractured, jointed, and non-calcareous.

j. Instrumentation line E. Piezometers for instrumentation line E were installed in two strata as shown on Plate 40. The upper stratum being monitored was the "waxy" clay shale unit of the Woodbine formation. The lower stratum monitored was at the Del Rio correlation bed. No thick sandstone unit was encountered along this line. Plots of piezometric elevation and fill elevation versus time are shown on Plates 43 through 56.

k. Analysis of piezometer reading line E. For instrumentation line E, all piezometers were installed in Woodbine clay shale except for P-65 which was installed in Del Rio clay shale. Readings in the Woodbine clay shale were all quite low or simply reflected groundwater fluctuations, except for P-67, which registered a maximum piezometric level above the top of fill. This response is due to the characteristics of the Woodbine in this area. The Woodbine shale along line E

abuts the floodplain. The shale in this zone has undergone stress relief due to unloading from erosion. This has produced a fractured and jointed structure in the shale which has increased permeability. Also, the shale along line E is interbedded with numerous sandstone layers. At piezometer P-67, the Woodbine strata is thicker and the piezometer tip was possibly located in a more intact zone with less fracturing. In the Del Rio formation, piezometer P-65, unlike the Del Rio piezometers at line D, exhibited only a moderate pore pressure response. It may be that the tip was not installed deep enough into the Del Rio and pressure may have partially bled off into the more pervious Woodbine formation.

11-02. Inclinometers.

a. General description. During the initial contract, 12 inclinometers were installed, I-1 through I-5 along instrumentation line A, I-6 through I-10 along instrumentation line B, and I-11 and I-12 at the outlet works discharge channel. The inclinometers for the initial contract are shown in plan view on Plate 28. They were bottomed approximately 15 feet into the clay shale of the Del Rio formation. Inclinometers I-1 through I-10 were designed to monitor horizontal movements of the embankment foundation. The instruments were cased through the embankment fill with steel pipes. Inclinometers I-11 and I-12 were designed to monitor channel slope movements during the excavation of the outlet works channel. During the completion contract, four inclinometers, I-13 through I-16, were installed in the embankment closure section at locations shown on Plate 28. These inclino-

meters were designed to monitor horizontal movement of embankment foundation. All embankment foundation monitoring inclinometers were bottomed a minimum of 15 feet into the Del Rio formation.

b. Analysis of inclinometer data. Inclinometers were used to measure horizontal movement in the foundation only. No readings were taken in the embankment fill since foundation shear strength and not fill shear strength controlled the embankment design. Inclinometer readings have indicated only minor horizontal movement in the foundation. Most of this movement has occurred in the overburden and weathered primary strata. Essentially no movement occurred below the weathered zone. Plots of horizontal deflection versus depth are shown on Plates 57 through 61. There is an expected amount of scatter in the data, due to the variations in the different monitoring equipment, probes, cables, and readers used to obtain the readings. However, even with the scatter, the general trend is upstream deflection for the upstream inclinometers and downstream deflection for the downstream inclinometers. It should also be noted that the greatest horizontal movement occurred near the centerline where embankment settlement is greatest. No significant deflection was observed in the unweathered primary including the "waxy" clay shale units.

11-03. Foundation settlement plates and surface reference marks. For the initial contract, foundation settlement Plates, SP-1 through SP-4, and surface reference marks, RM-1 through RM-31, were installed at locations shown on Plate 29. During the course of the initial contract, the bench marks used to obtain surveyed settlement plate eleva-

tions were found to be unstable, probably due to ground surface movement caused by soil moisture changes. As a result, the settlement plate elevation readings were erratic. To remedy this problem, "free standing" deep bench marks were installed. These deep bench marks were designed to be independent of shallow ground movements due to moisture changes. The deep bench marks installed in the summer of 1980 were intended to be the basis of the settlement plate and surface reference marks surveys. However, when the deep bench marks were tied together by survey, the error of closure in the traverse was excessive. It was determined that the PVC casing used to construct the deep bench marks was curved and allowed the free standing steel pipe to rub against the casing. Due to the continued lack of a reliable bench mark, the settlement plate and surface reference mark readings were discontinued on the initial contract. For the completion contract "core tipped" deep bench marks were successfully used to provide a stable bench mark for the settlement plate measurements. This type bench mark is constructed of a 3-inch diameter machined steel cone welded on to a 0.5 inch diameter steel rod. After a bore hole is drilled the cone and attached rod is pushed into the stable founding strata. A protective 2-inch diameter steel pipe is installed over the 0.5 inch diameter steel rod. The hole is then filled with grease except for the top portion which is grouted. For the completion contract, two settlement plates, SP-5 and SP-6, were installed to monitor foundation settlement of the overburden and weathered primary. Both settlement plates were installed on line D, SP-5 was located 90 feet upstream and SP-6 was located 90 feet downstream. Plots of plate ele-

vation and fill elevation versus time are shown on Plate 62. Only moderate settlements were indicated from both instruments.

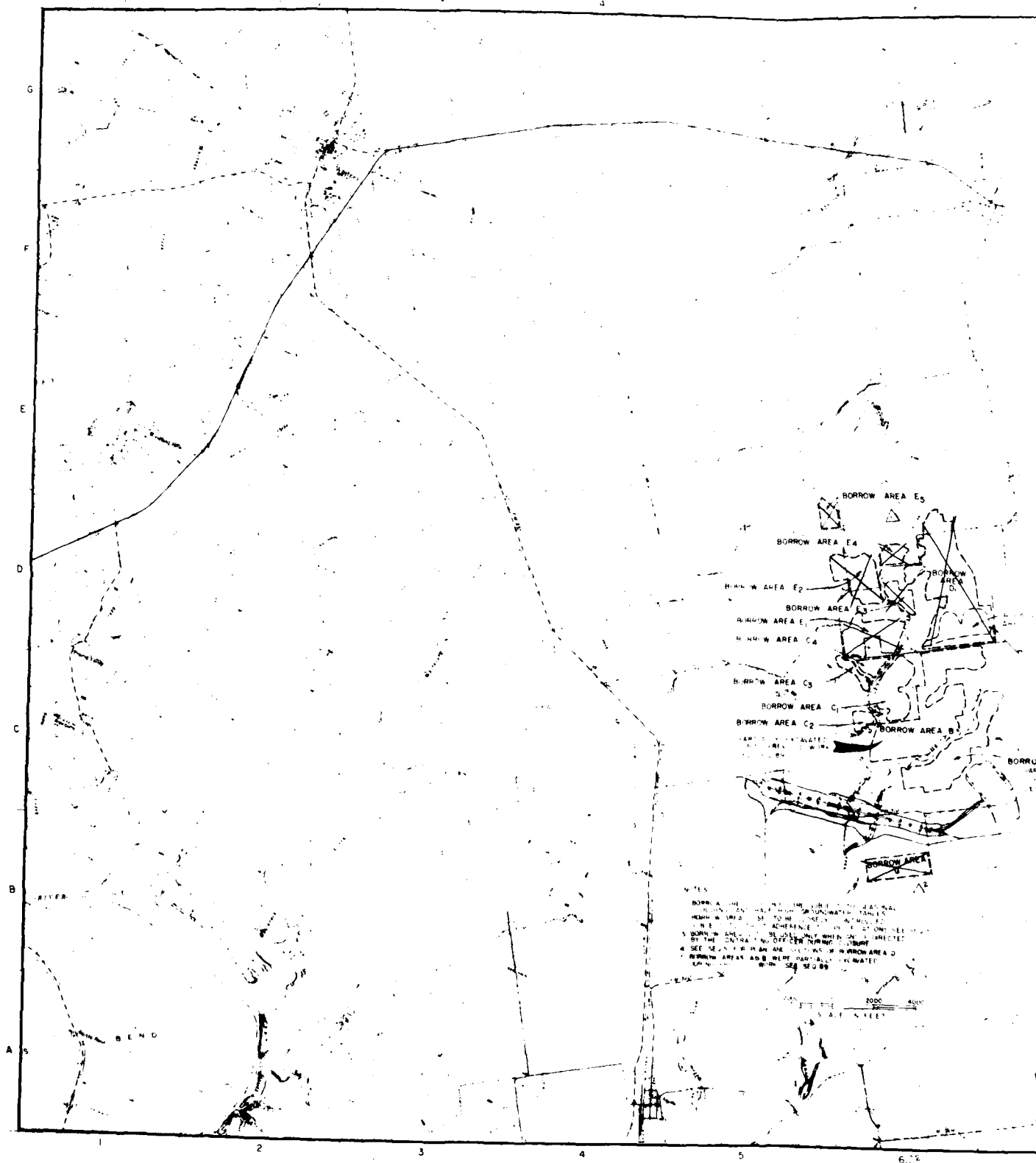
11-04. Outlet works reference marks. Reference marks were installed along the invert of the conduit, on the intake tower, along the discharge chute, and on the stilling basin walls. The reference marks were surveyed after installation in January 1981. A subsequent set of readings was taken in October 1981. Diversion of water through the outlet works during construction was necessarily caused by the inability to close the gates. Electric power required to operate the gates had not been connected. Therefore, no readings were taken until May 1983. Electric power was connected to the control tower in March 1983 making the gates operable. Two additional sets of outlet works reference mark readings were made in May 1983 and March 1984. Plot of the reference mark readings for the outlet works are shown on Plate 63. The initial set of readings taken in January 1981 shows that the outlet works conduit was constructed approximately 0.2 foot higher than the elevation shown on the contract plans. The second set of readings made in October 1981 indicated that only minor vertical movements took place as a result of embankment fill placement over the conduit during the period from January 1981 to October 1981. Readings taken on 12 May 83 indicate end of construction settlement. The maximum settlement recorded was approximately 0.1 foot at the embankment centerline. Readings made in March 1984 indicated no additional displacement and were not plotted. No significant spreading or misalignment of the monolith joints has occurred.

SECTION 12 - INSERVICE EVALUATION

12-01. General. The inservice performance of the Aquilla Lake embankment and appurtenant structures' foundations has shown to be excellent. Deliberate impoundment began on 29 April 1983. The impounded pool had reached elevation 532.4, or about 5 feet below conservation pool, as of January 1985. Surveillance inspections have been conducted subsequent to construction in accordance with the "Reservoir Filling Plan", DM No. 23. No signs of structural distress have been observed.

12-02. Embankment stability. In view of the foundation performance and fill conditions encountered during construction, the shear strengths and unit weights indicated by record samples, and the actual excess pore pressures developed during construction, the analyses conducted during design are considered appropriate and sufficient. No additional embankment stability analyses will be conducted.

12-03. Dam safety. The Fort Worth District has a strong commitment to dam safety. The Aquilla dam embankment has already been subjected to two inspections by teams of geotechnical engineers and geologists as part of the program for Continued Evaluation of Completed Civil Works Projects. Instrumentation is being read and interpreted on a scheduled basis. All data and observations during and subsequent to construction indicate the embankment is and will continue to function as a safe structure as designed.



NOTES

1. BORROW AREA E1 IS THE ONLY ONE THAT WAS
 2. BORROW AREA E2 IS THE ONLY ONE THAT WAS
 3. BORROW AREA E3 IS THE ONLY ONE THAT WAS
 4. BORROW AREA E4 IS THE ONLY ONE THAT WAS
 5. BORROW AREA E5 IS THE ONLY ONE THAT WAS
 6. BORROW AREA C1 IS THE ONLY ONE THAT WAS
 7. BORROW AREA C2 IS THE ONLY ONE THAT WAS
 8. BORROW AREA C3 IS THE ONLY ONE THAT WAS
 9. BORROW AREA C4 IS THE ONLY ONE THAT WAS
 10. BORROW AREA B IS THE ONLY ONE THAT WAS
 11. BORROW AREA A IS THE ONLY ONE THAT WAS

2000 4000
 S. A. F. 1:10,000

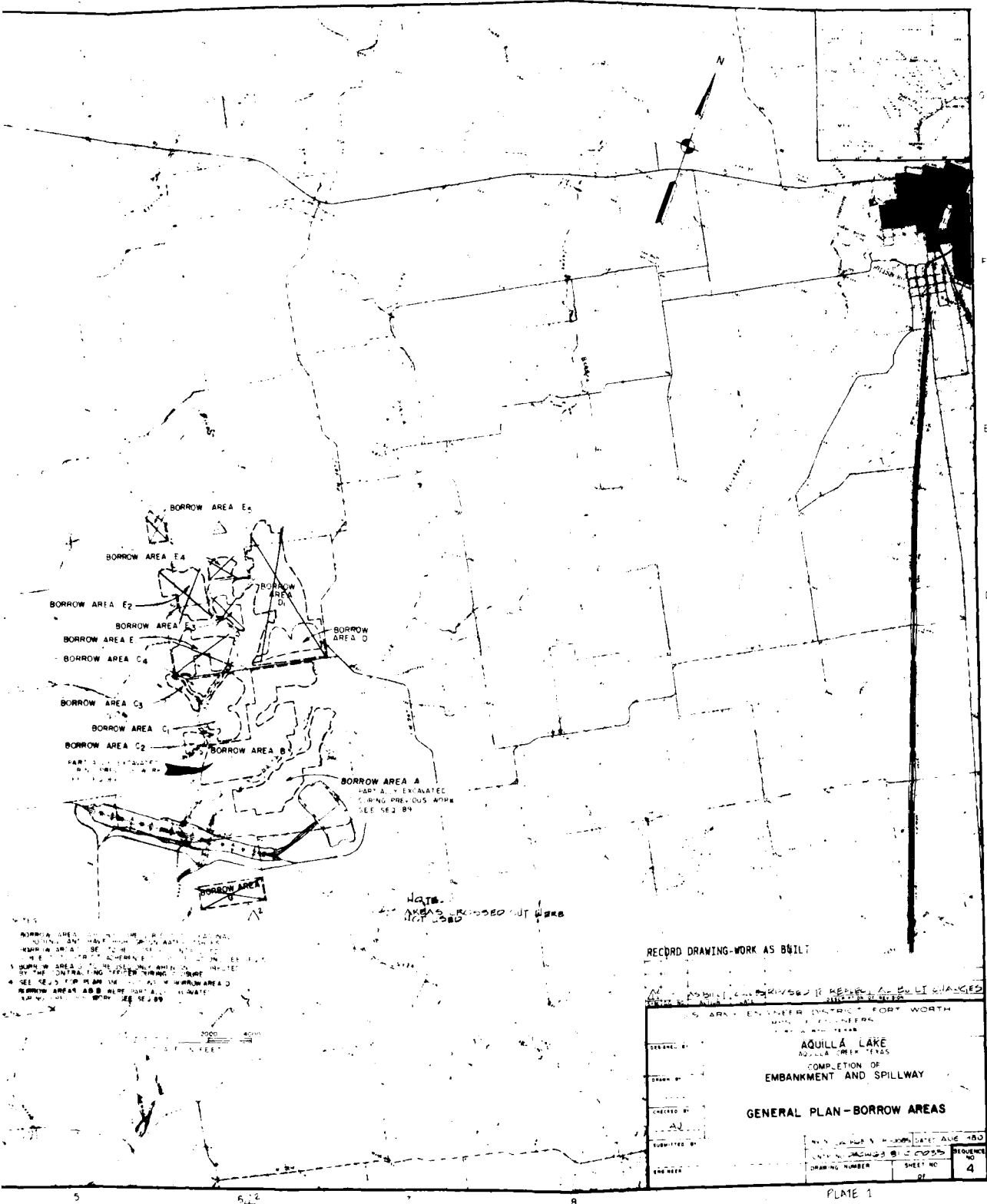
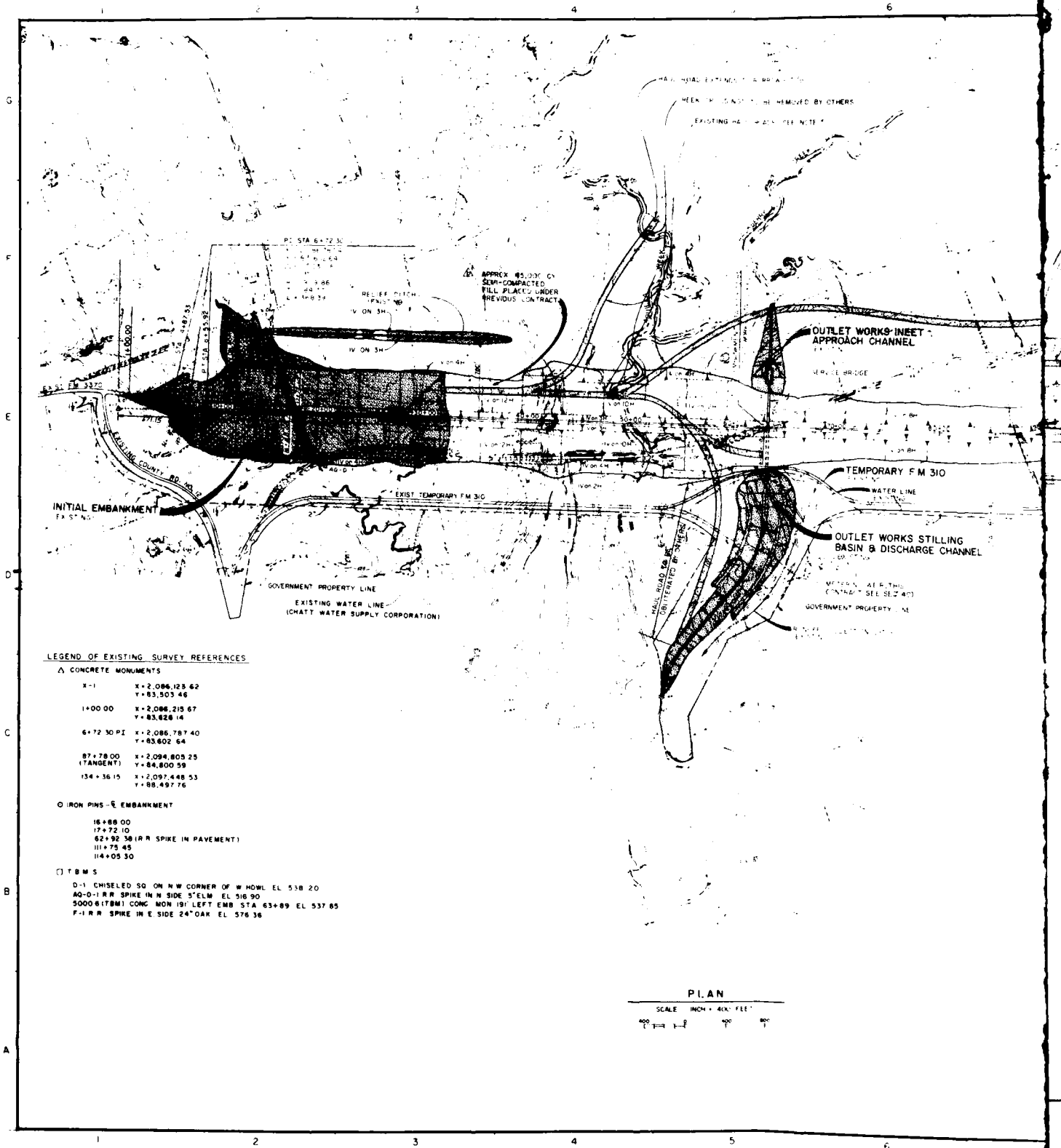
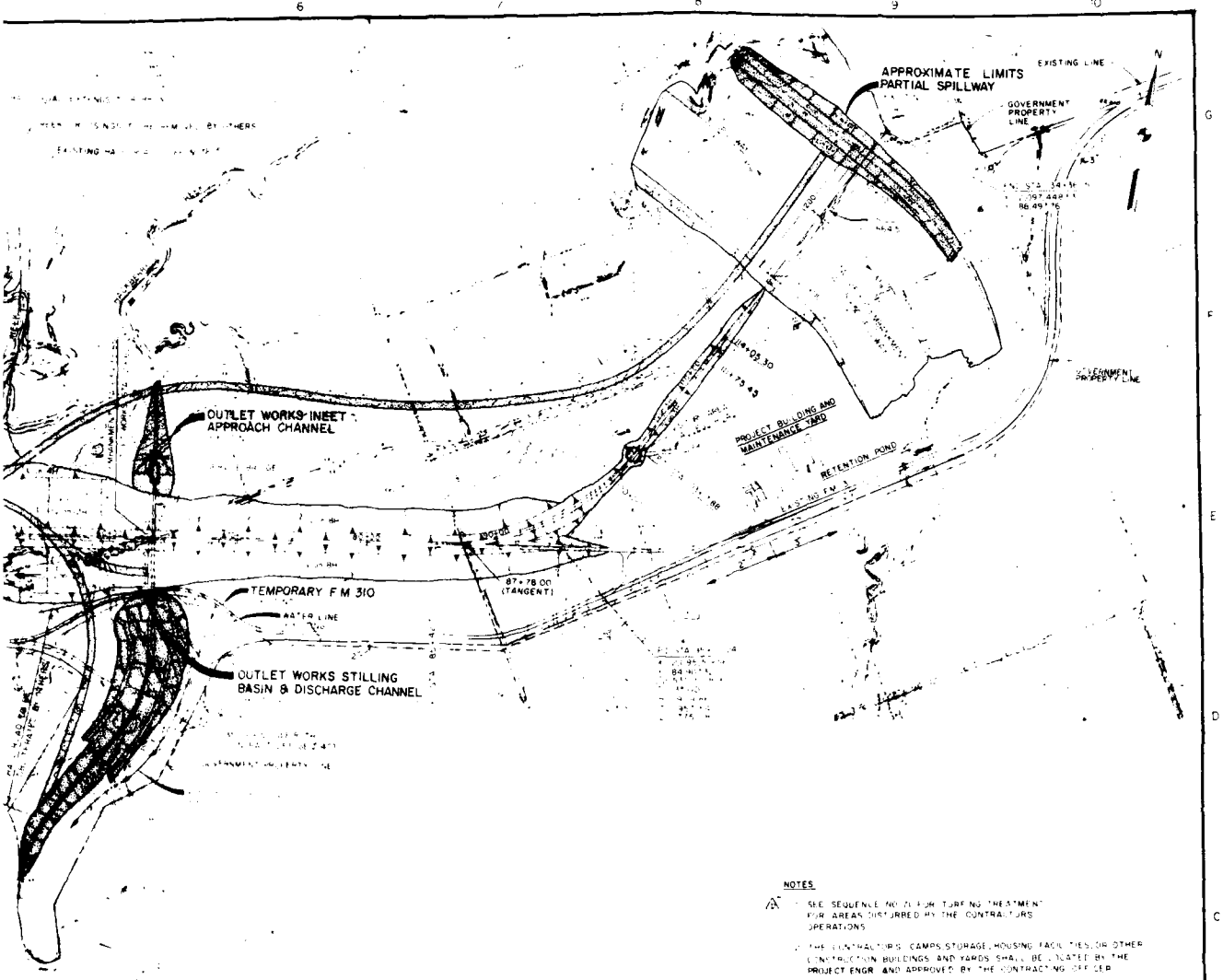


PLATE 1





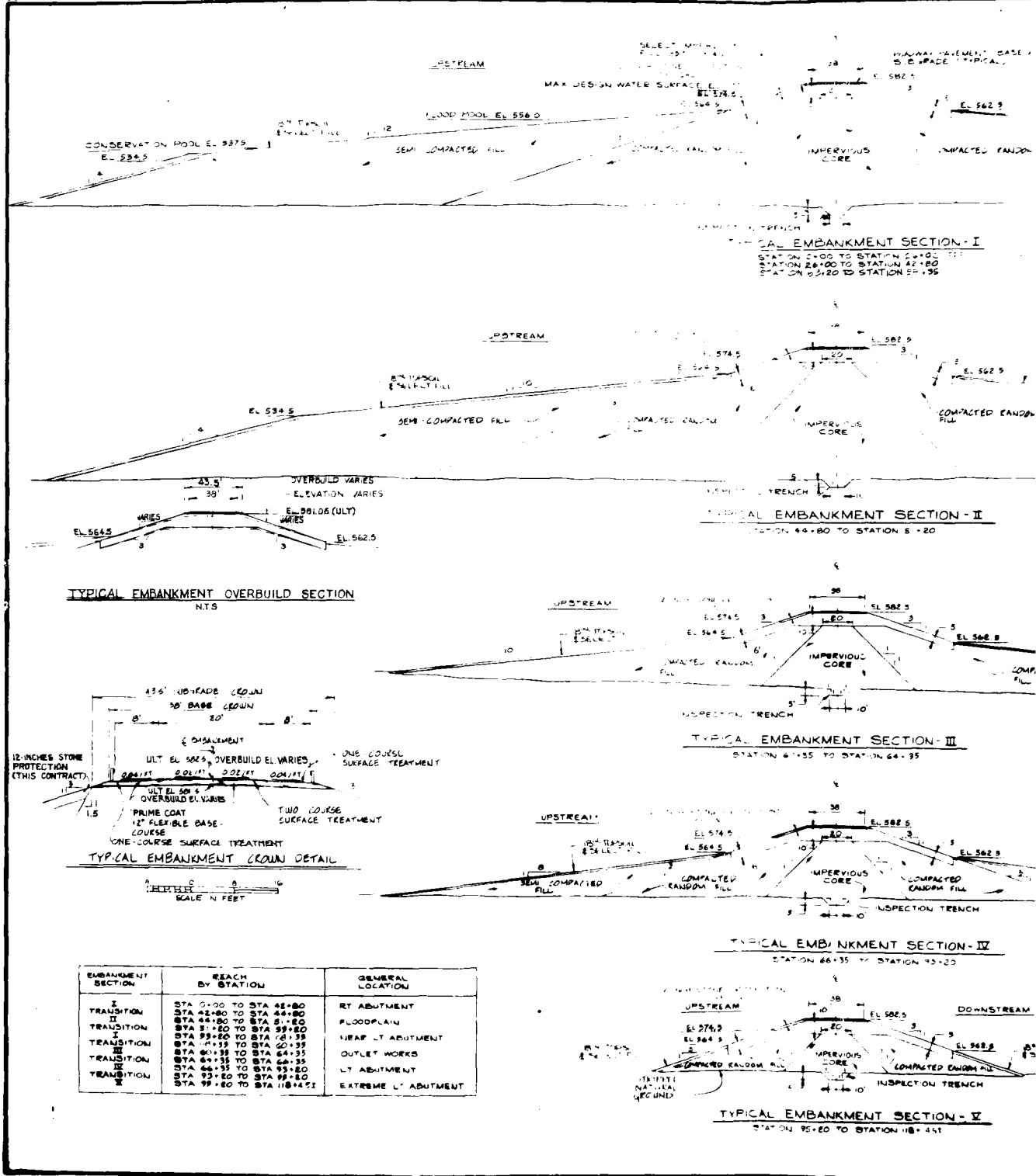
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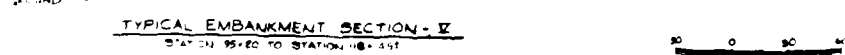
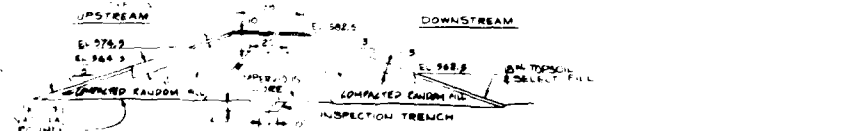
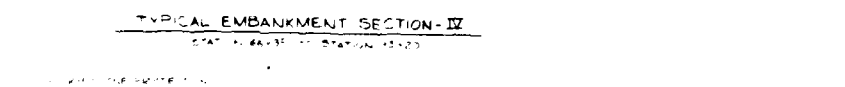
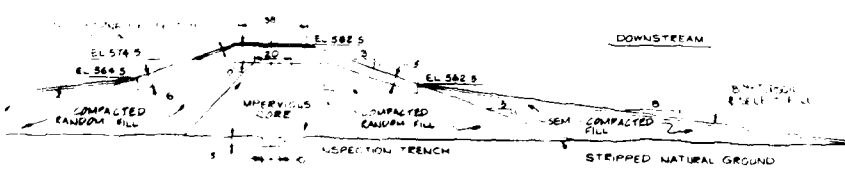
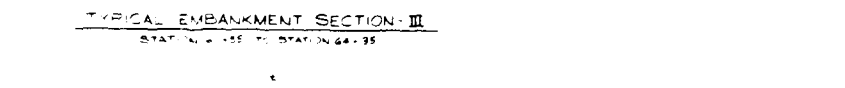
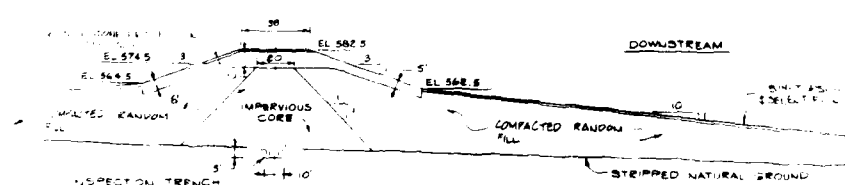
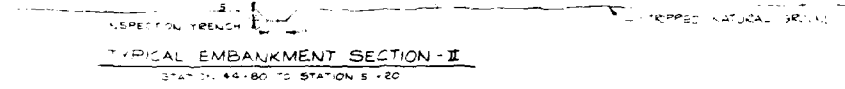
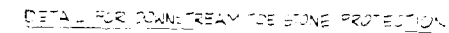
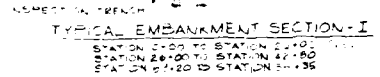
1. SEE SEQUENCE NO. 21 FOR TURNING TREATMENT FOR AREAS DISTURBED BY THE CONTRACTORS OPERATIONS.
2. THE CONTRACTORS CAMPS, STORAGE, HOUSING FACILITIES, OR OTHER UTILITIES, BUILDINGS AND YARDS SHALL BE LOCATED BY THE PROJECT ENGR. AND APPROVED BY THE CONTRACTING OFFICE.
3. CLEARING AND GRUBBING TO BE DONE BEYOND THE LIMITS OF THE EMBANKMENT 10' UPSTREAM AND 20' DOWNSTREAM OF THE DITCHES.
4. LOADING AND GRUBBING TO BE 30' BEYOND LIMITS OF SPILLWAY EXCAVATION.
5. THE CONDITIONS OF THE HAUL ROADS AND THEIR FUTURE USE BY THE CONTRACTOR CANNOT BE ASSURED.
6. APPROXIMATELY 45,000 YDS. OF SEMI-COMPACTED FILL MATERIAL WAS PLACED IN THE UPSTREAM SEMI-COMPACTED FILL FROM STATION 32+00 TO STATION 34+00 UNDER THE PREVIOUS CONTRACT.

RECORD DRAWING-WORK AS BUILT

DESIGNED BY		AQUILLA LAKE	
CHECKED BY		COMPLETION OF	
DRAWN BY		EMBANKMENT AND SPILLWAY	
SUBMITTED BY		SERVICE BRIDGE, ACCESS ROADS, PROJECT BUILDING,	
ENGINEER		VISITORS OVERLOOK, F.M. 310 AND	
		OTHER APPURTENANCES	
GENERAL PLAN			
DRAWING NUMBER		SHEET NO.	
21		3	

PLAN
SCALE: HON + 400 FEET
1" = 400'





GENERAL COMMENTS:
 1. IN THE NEARLY 1000' STATION 42+50 TO 42+00,
 14' TO 16' DEEP, THERE IS A STRATIFIED SAND AND GRAVEL
 DEPOSIT. THE SAND IS WHITE, CLEAN, AND GRAVEL
 IS DARK. THE GRAVEL IS 1/2" TO 1" IN SIZE.

2 ELEVATIONS AT TYPICAL EMBANKMENT SECTIONS ARE
ULTIMATE ELEVATION OVERBUILT WILL BE APPLIED
TO THE GRADE 4 AFTER 15 ON 34 EMBANKMENT SLOPES
IN ACCORDANCE WITH THE FOLLOWING SCHEDULE:

<u>STATION</u>	<u>KNOWN OVERPULL</u>
1000	0.1 FT
1005	0.1 FT
1010	0.1 FT
1015	0.1 FT
1020	0.1 FT
1025	0.1 FT
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1060	0.1 FT
1065	0.1 FT
1070	0.1 FT
1075	0.1 FT
1080	0.1 FT
1085	0.1 FT
1090	0.1 FT
1095	0.1 FT
1100	0.1 FT
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RECORD DRAWING-WORK AS BUILT

AM-9002, REVISED TO REFLECT W.I. CHANGES

U.S. ARMY ENGINEER DISTRICT FORT WORTH
CORPS OF ENGINEERS
FORT WORTH, TEXAS

DESIGNED BY
H. L. HARRIS
A. HARRIS

AQUILLA LAKE
AQUILLA CREEK, TEXAS

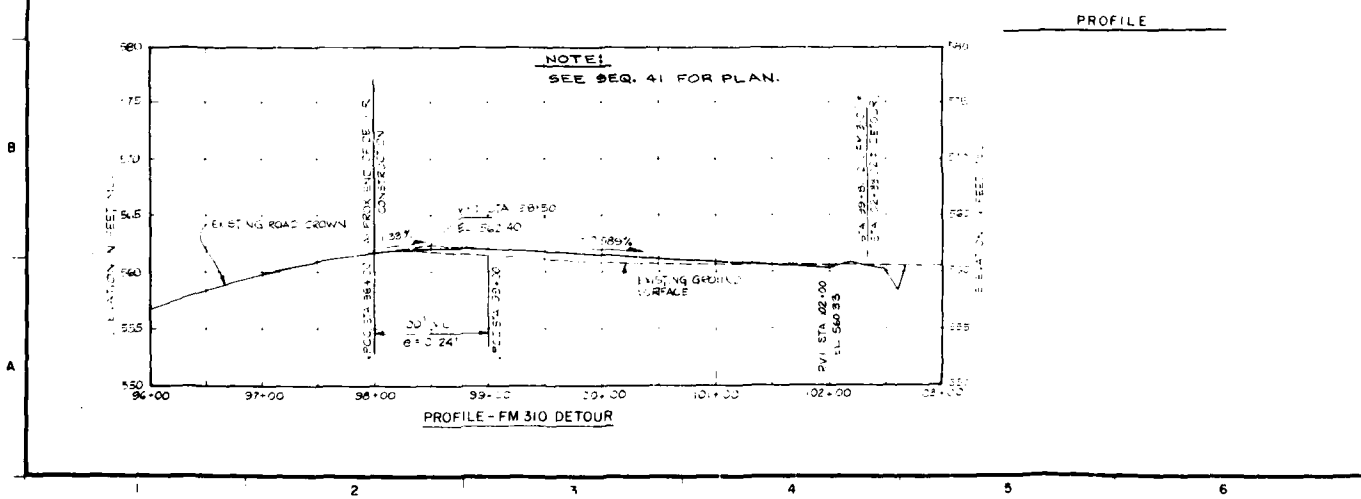
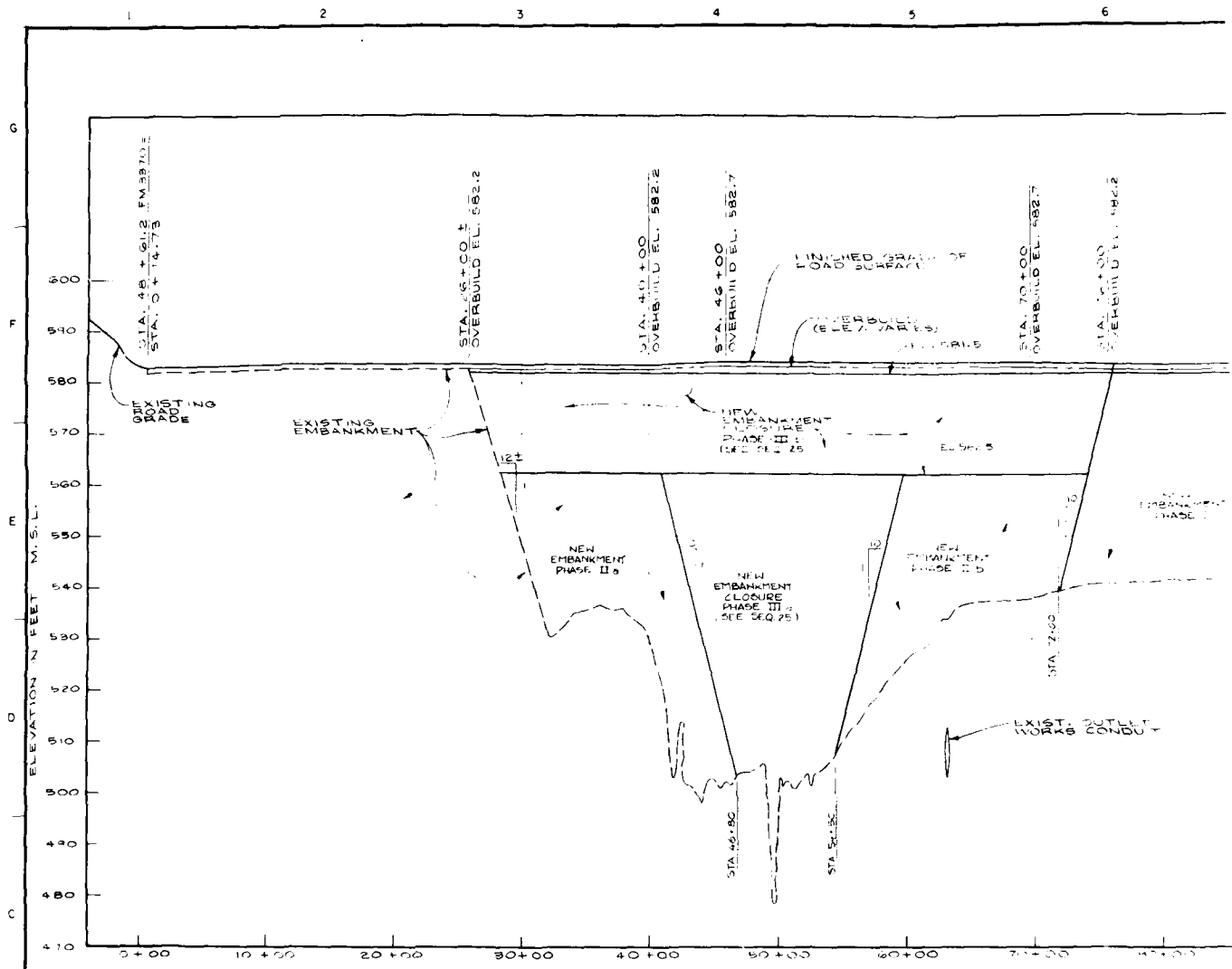
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CHC 880
L 1000
S 1000

TYPICAL EMBANKMENT SECTIONS

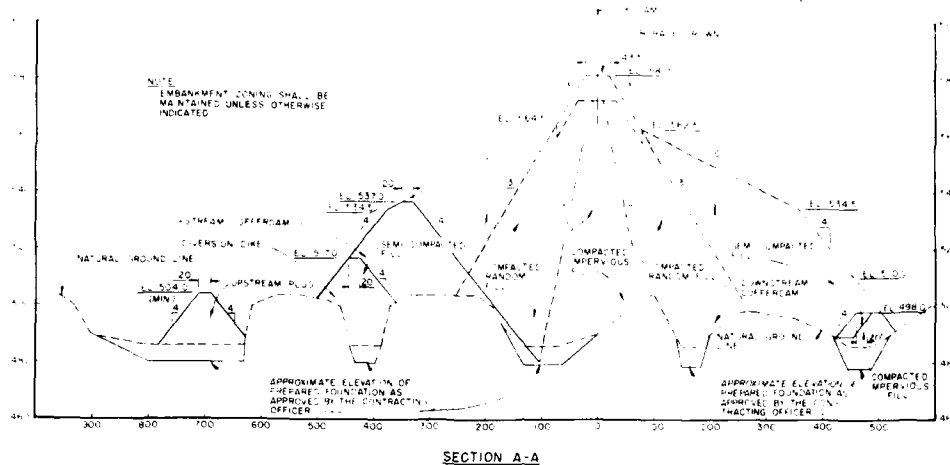
SUBMITTED BY APPROVED	BY: <u>DAE JAB-BI 6-2005</u> DATED: <u>Aug 15 2005</u> CONTR NO: <u>DAE JAB-BI 6-2005</u> DRAWING NUMBER: _____ SHEET NO: <u>?</u>
--------------------------	--

PLATE 3



1. ASSIGNMENT
2. THESE
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NOTE:
 Δ DELETED NOTE 5, ITEM



AQUILLA LAKE - EMBANKMENT COMPLETION EMBANKMENT CLOSURE STAGING NOTES:

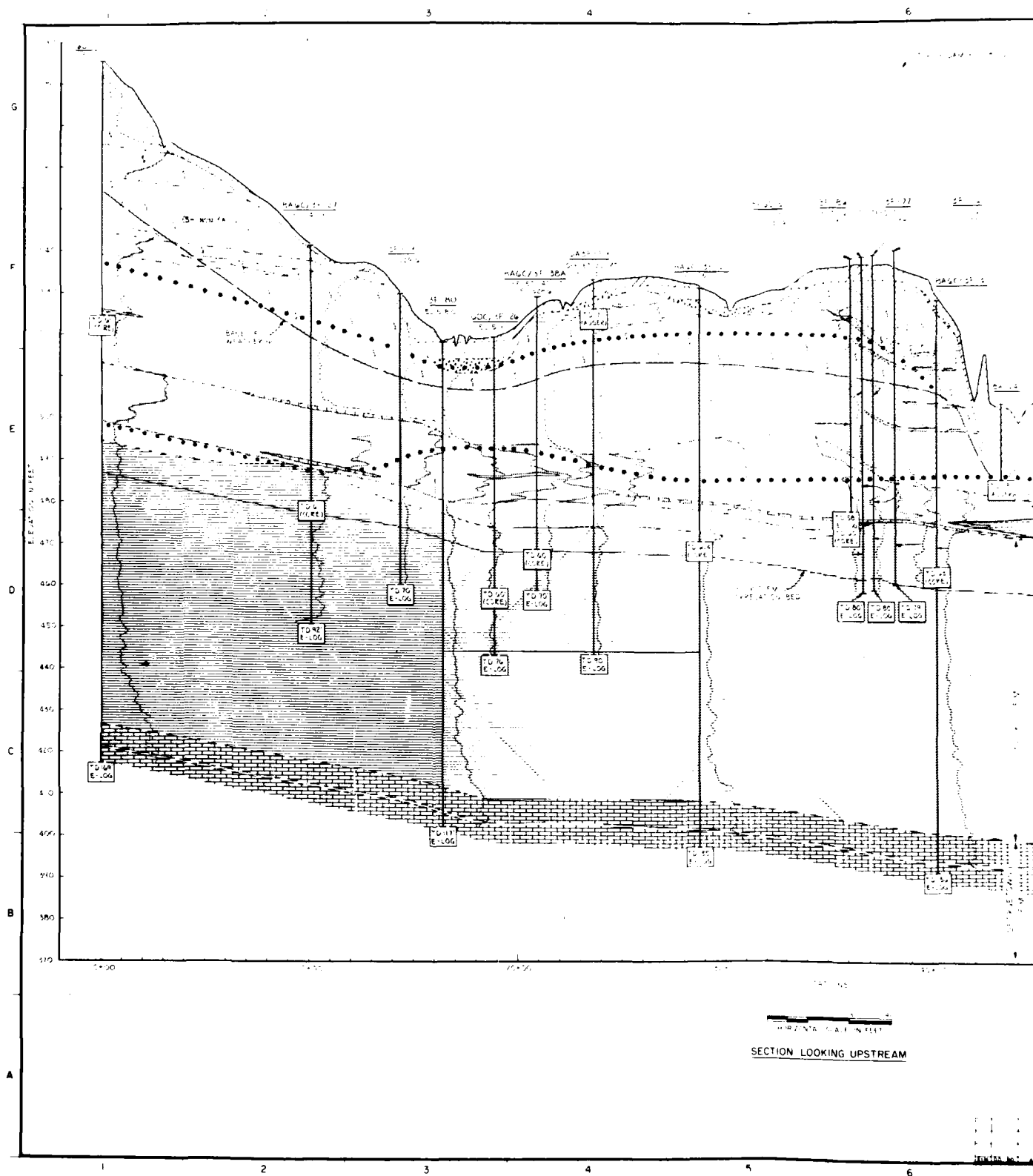
1. SEE SEQUENCE NO. 6 FOR EMBANKMENT CONSTRUCTION STAGING.
2. SPILLWAY EXCAVATION TO PROCEED AT LEAST AT RATE TO SUPPLY MATERIALS FOR SEMI-COMPACTED FILL.
3. CONSTRUCT MAIN EMBANKMENT OUTSIDE OF CLOSURE SECTION (PHASE I) TO ELEVATION 562.5 IN ACCORDANCE WITH EMBANKMENT CONSTRUCTION STAG NO. 6. SEE SEQ. NO. 6; RATE OF PLACEMENT NOT TO EXCEED 4 FEET VERTICALLY EACH WEEK.
4. MAINTAIN OUTLET WORKS GATES IN OPEN POSITION. NOTE: GATE AT ELEVATION 563.5. INITIAL DIVERSION WILL NOT BEGIN UNTIL IMPROVEMENT REACHES INVERT ELEVATION.
5. CONSTRUCT CHANNEL PLUGS.
6. CONSTRUCT DIVERSION DIKE, UPSTREAM AND DOWNSTREAM OFFERDAMS IN THE DRY.
7. CONSTRUCT EMBANKMENT CLOSURE SECTION TO ELEVATION 562.5. RATE OF PLACEMENT NOT TO EXCEED 4 FEET VERTICALLY EACH WEEK. (PHASE II A)
8. CONSTRUCT EMBANKMENT CLOSURE SECTION TO ELEVATION 562.5. RATE OF PLACEMENT NOT TO EXCEED 4 FEET VERTICALLY EACH WEEK.
9. CONSTRUCT EMBANKMENT CLOSURE SECTION TO FULL HEIGHT. RATE OF PLACEMENT NOT TO EXCEED 4 FEET EACH WEEK. (PHASE II B)
10. CONTRACTOR TO BEGIN CLOSURE SEQUENCE ONLY AFTER CONTRACTING OFFICER RECEIVES CONTRACTOR'S WRITTEN STATEMENT OF INTENT AND REQUEST TO BEGIN CLOSURE AT LEAST 60 CALENDAR DAYS IN ADVANCE OF HIS ANTICIPATED START OF CLOSURE SEQUENCE. INITIATION OF CLOSURE SEQUENCE WILL BE ALLOWED ONLY AFTER CONTRACTING OFFICER'S REVIEW AND APPROVAL OF THE SEQUENCE OF THE CONTRACTOR'S WORK WHICH AFFECTS OR INCLUDES CLOSURE WORK ITEMS.
11. ITEMS 2 THRU 4 MAY PROCEED CONCURRENTLY TO THE EXTENT ALLOWED BY STAGING DETAILS. SEE SEQ. NO. 6.
12. ITEMS 5 TO BE UNDERTAKEN ONLY AFTER ITEMS 3 AND 4 ARE ESSENTIALLY COMPLETE.
13. ITEM 6 TO BE CONSTRUCTED ONLY AFTER ITEM 5 IS COMPLETED AND DRAINAGE OF WATER AND FOUNDATION PREPARATION WITHIN THE LIMITS OF CLOSURE SECTION IS COMPLETED AND APPROVED BY THE CONTRACTING OFFICER.
14. ITEMS 8 AND 9 TO BE COMPLETED IN SEQUENCE.
15. ITEMS 1 THRU 7 TO BE COMPLETED PRIOR TO COMMENCEMENT OF ITEM 8.

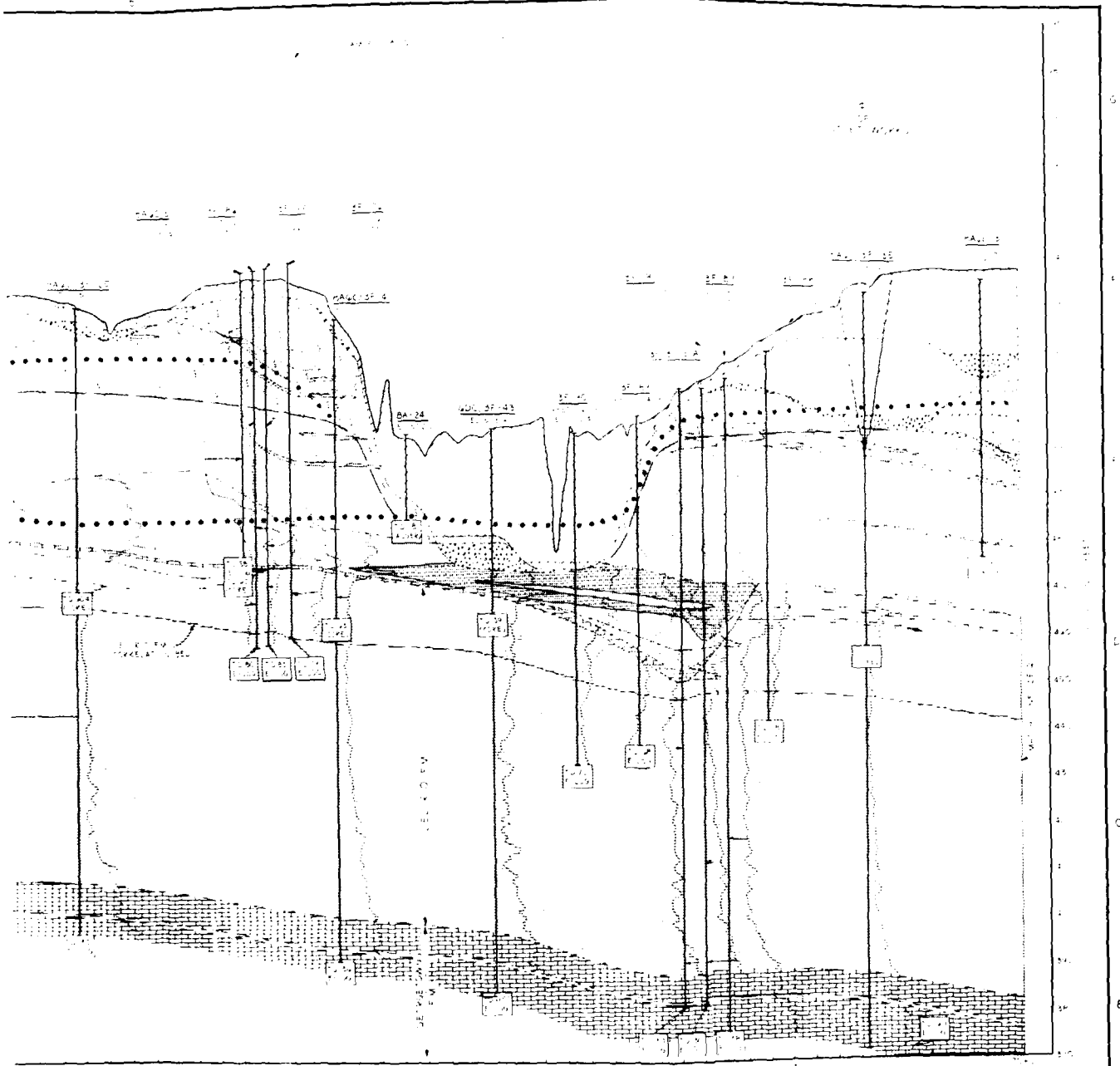
NOTE: DELETED NOTE 15, ITEM 1 AT CONTRACTOR'S REQUEST

RECORD DRAWING-WORK AS BUILT

AS BUILT	REVISIONS	REVISIONS	REVISIONS
AS BUILT	REVISIONS	REVISIONS	REVISIONS
U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS			
DESIGNED BY A. BRANCH	AQUILLA LAKE AQUILLA GREEN, TEXAS		
DRAWN BY	FLOOD PLAIN EMBANKMENT		
CHECKED BY U.S. ARMY ENGINEER	EMBANKMENT CLOSURE PLAN AND SECTION		
SUBMITTED BY	DATE: AUG. 1980		
ENGINEER	DRAWING NUMBER	SHEET NO.	25

PLATE 5

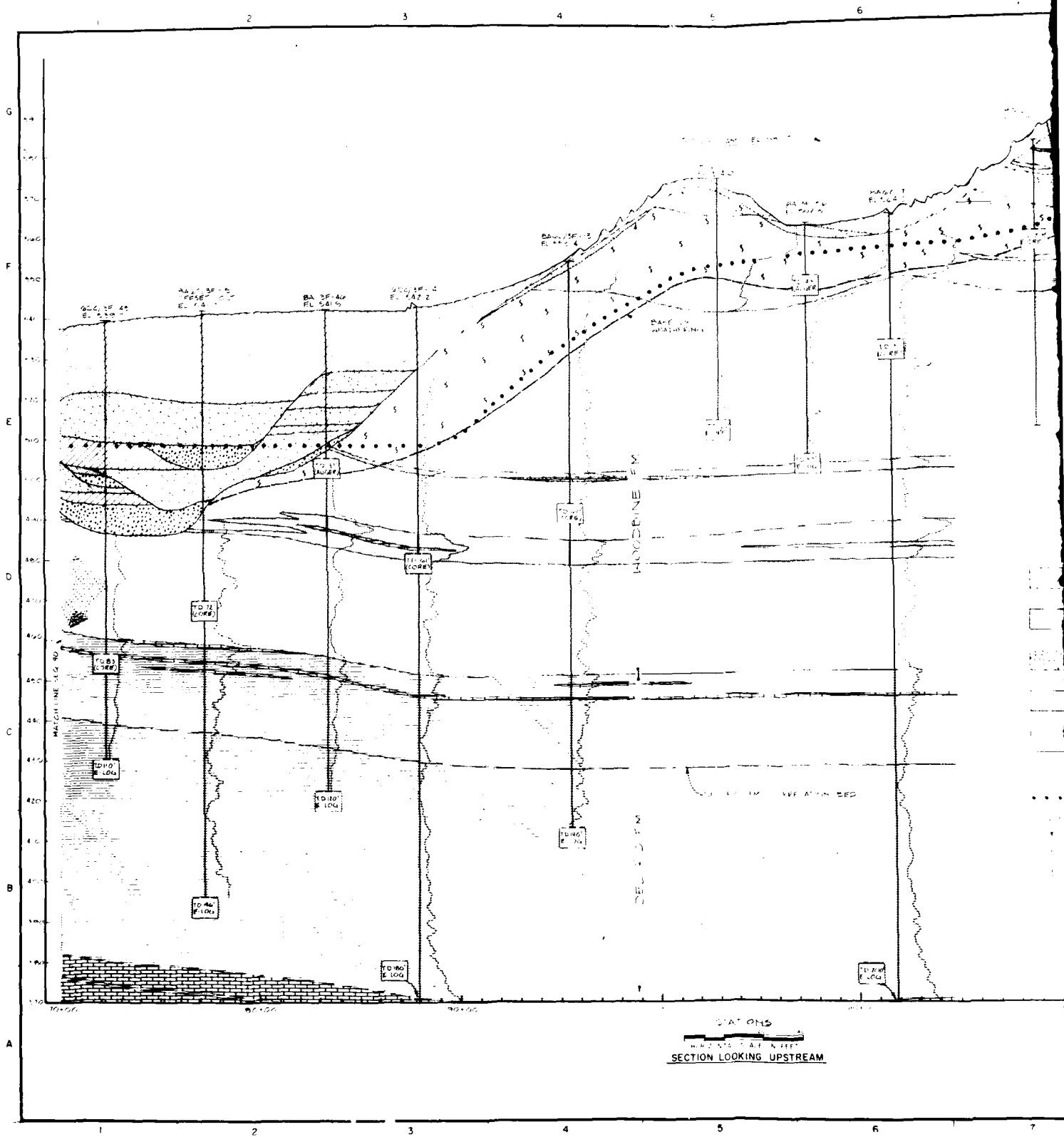


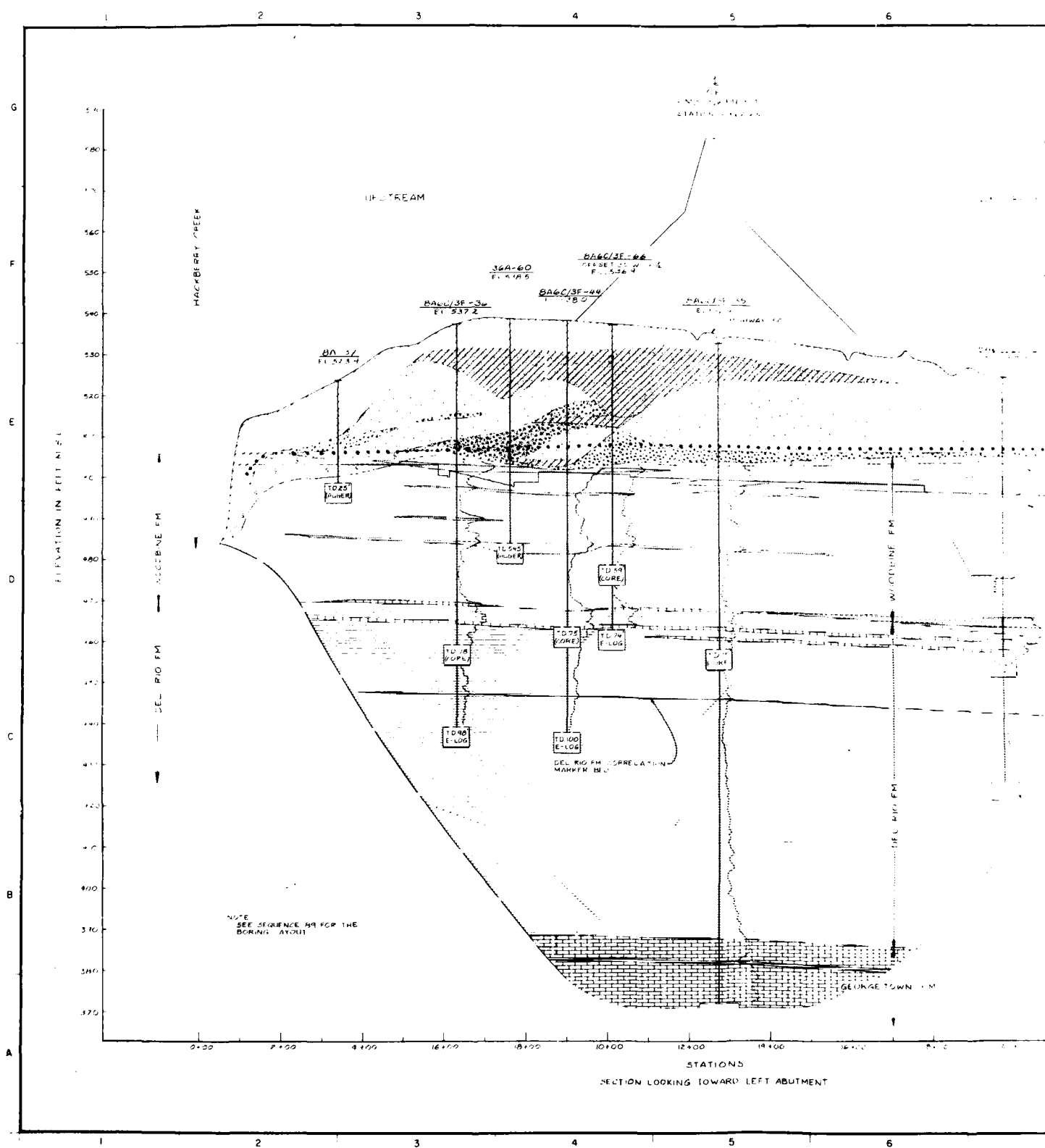


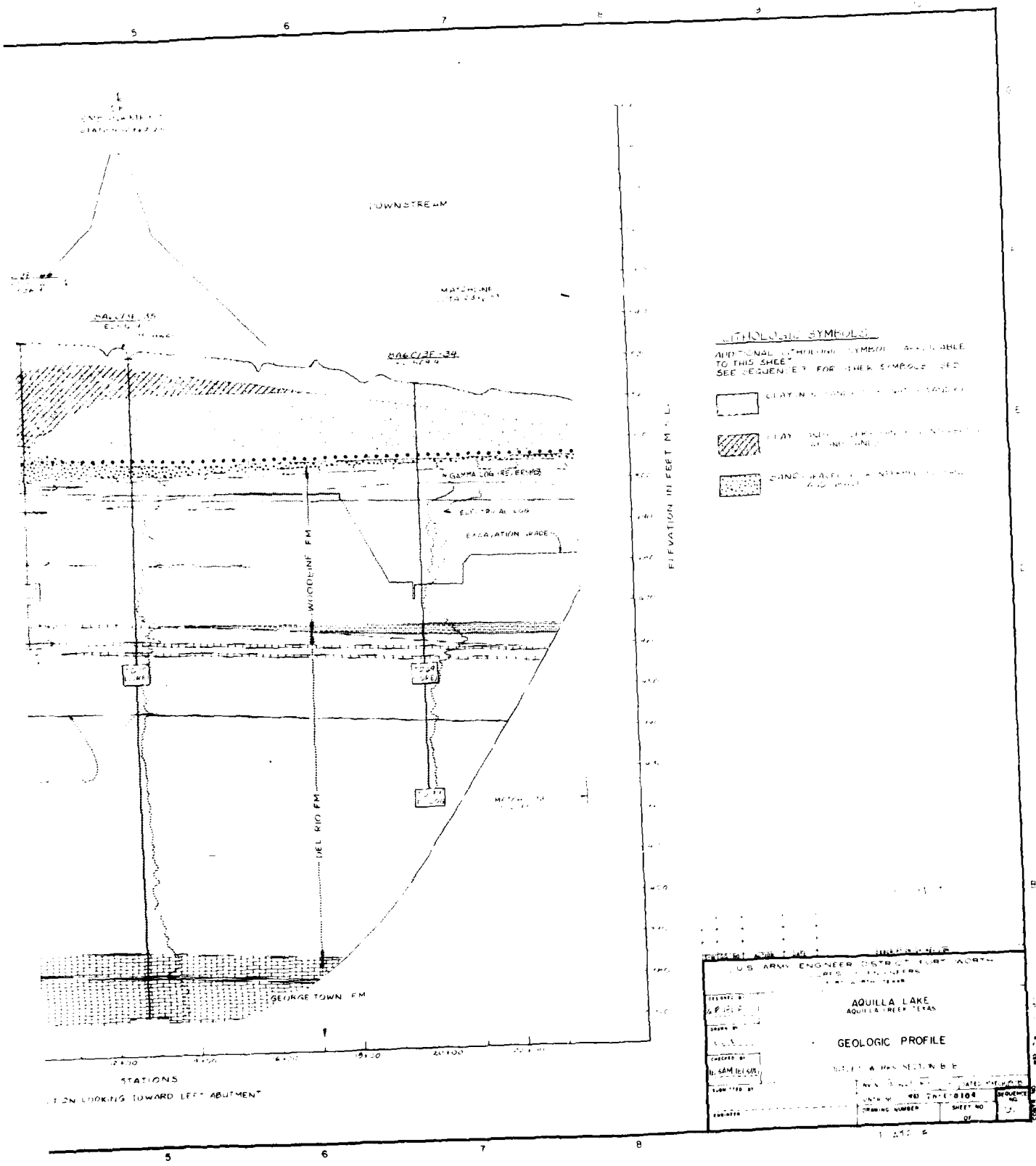
SECTION LOOKING UPSTREAM

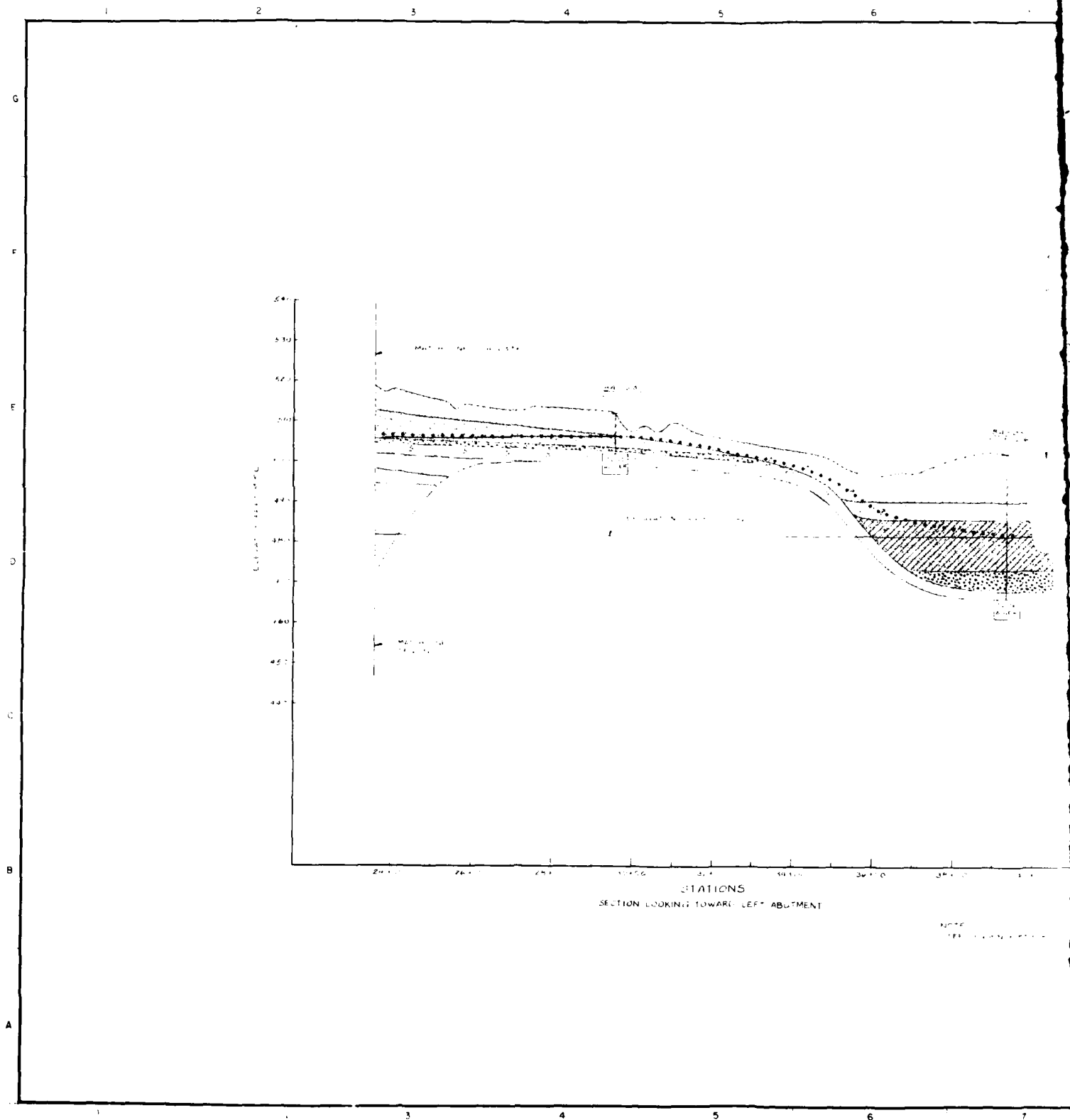
U.S. ARMY ENGINEER DISTRICT, FORT WORTH	
AQUILLA LAKE	
EMBANKMENT	
GEOLOGIC PROFILE	
(EMBANKMENT - SECTION A-A)	
DESIGNED BY	DATE
CHECKED BY	DATE
APPROVED BY	DATE
PROJECT NO.	103
SHEET NO.	103

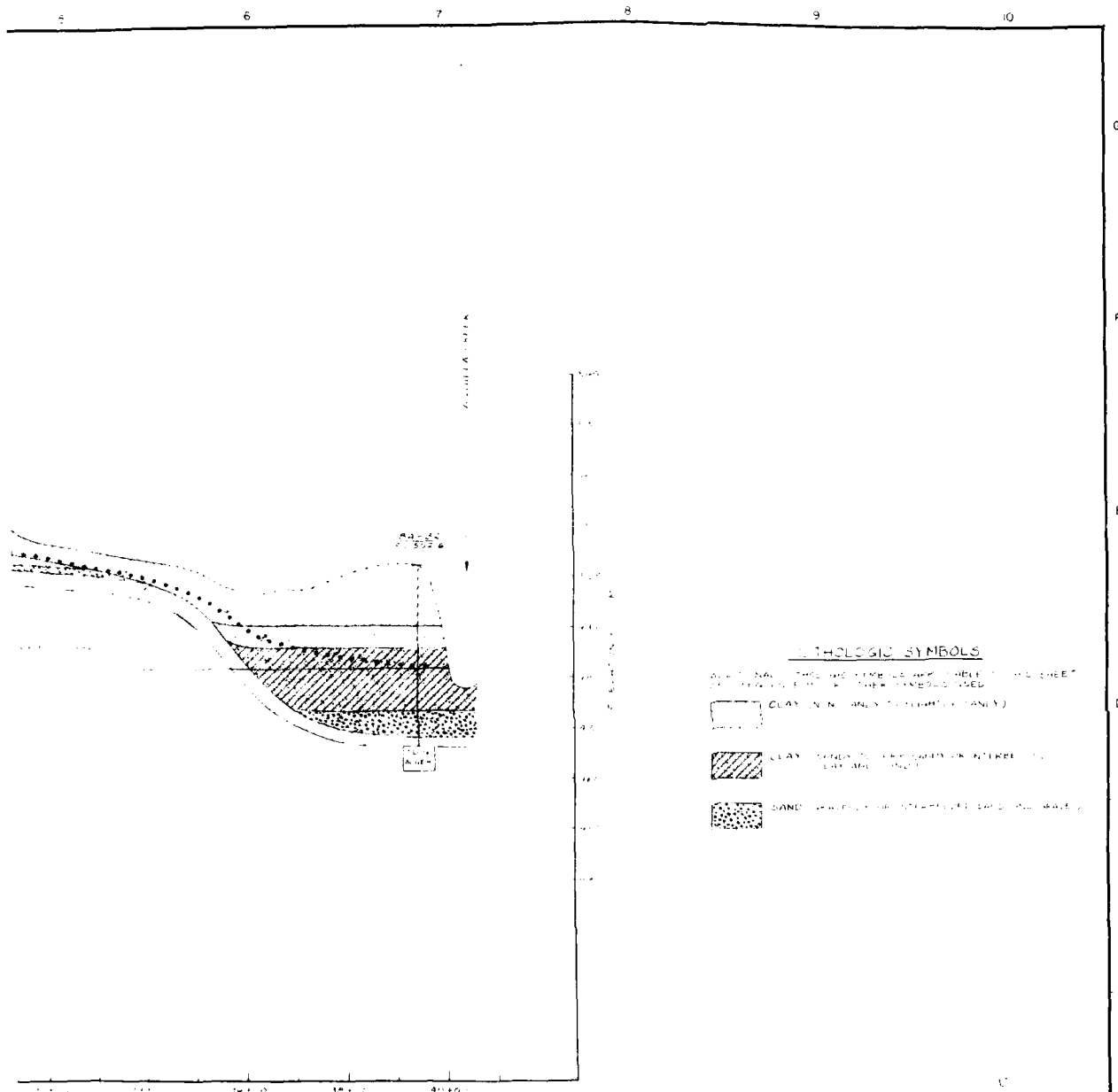
PLATE 6











- GEOLOGIC SYMBOLS**
- SYMBOLS FOR THE GEOLOGIC SYMBOLS ARE SHOWN ON THIS SHEET
 OF THE DRAWING. THE SYMBOLS ARE USED AS FOLLOWS:
- CLAY (IN N. W. CORNER ONLY)
 - CLAY (OTHER THAN IN N. W. CORNER ONLY)
 - SAND (OTHER THAN IN N. W. CORNER ONLY)

NOTE:
 THE PROFILE IS FOR THE PART OF THE OUTLET WORKS SECTION B-B

U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS	
DESIGNED BY A. R. FULTON	PROJECT NO. 100-100-0104
DRAWN BY H. C. LAMM	DATE MAY 1944
CHECKED BY H. C. LAMM	SEQUENCE NO. 93
SUBMITTED BY ENGINEER	DATE MAY 1944

AQUILLA LAKE
AQUILLA CREEK, TEXAS

GEOLOGIC PROFILE

OUTLET WORKS SECTION B-B

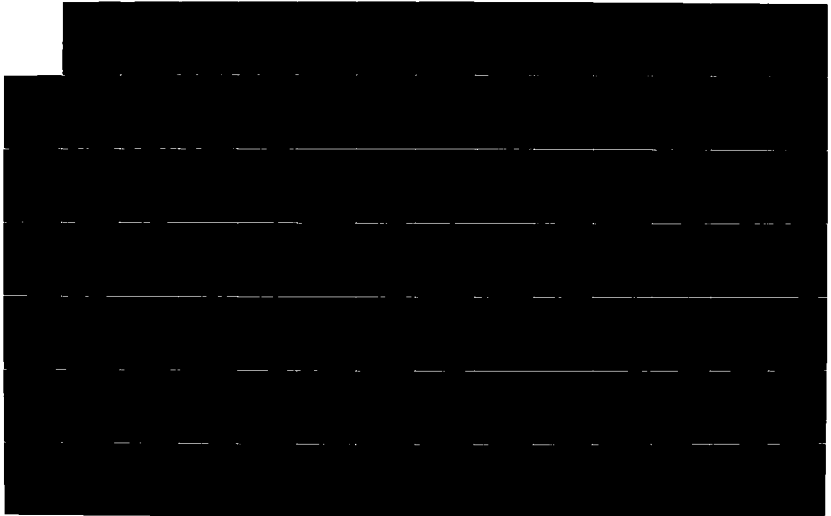
DRAWING NUMBER SHEET NO. 93

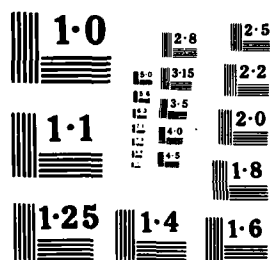
AD-A168 214

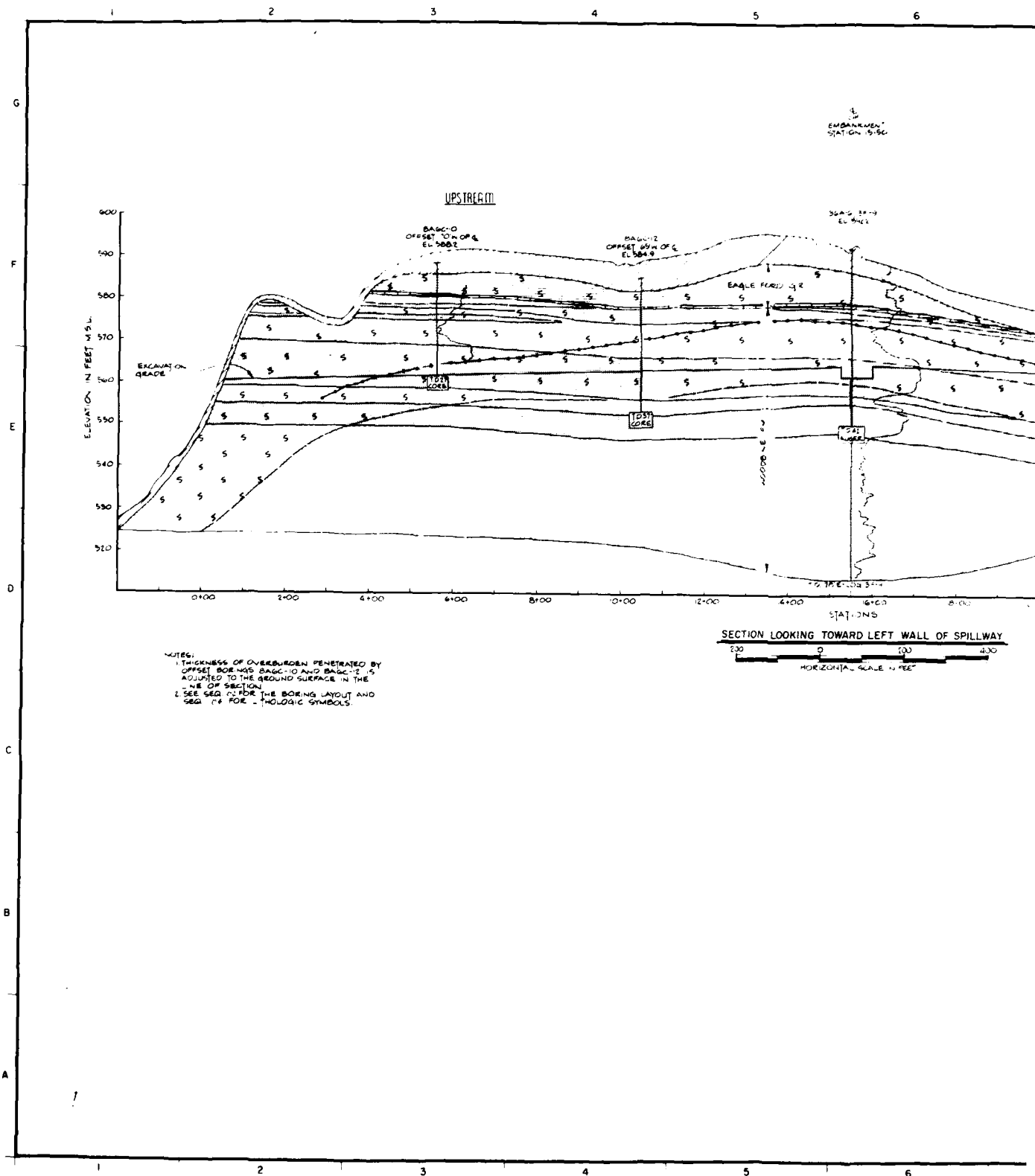
EMBANKMENT CRITERIA AND PERFORMANCE REPORT AQUILLA LAKE 2/8
AQUILLA CREEK TEXAS BRAZOS RIVER BASIN(U) ARMY ENGINEER
DISTRICT FORT WORTH TX DEC 85

UNCLASSIFIED

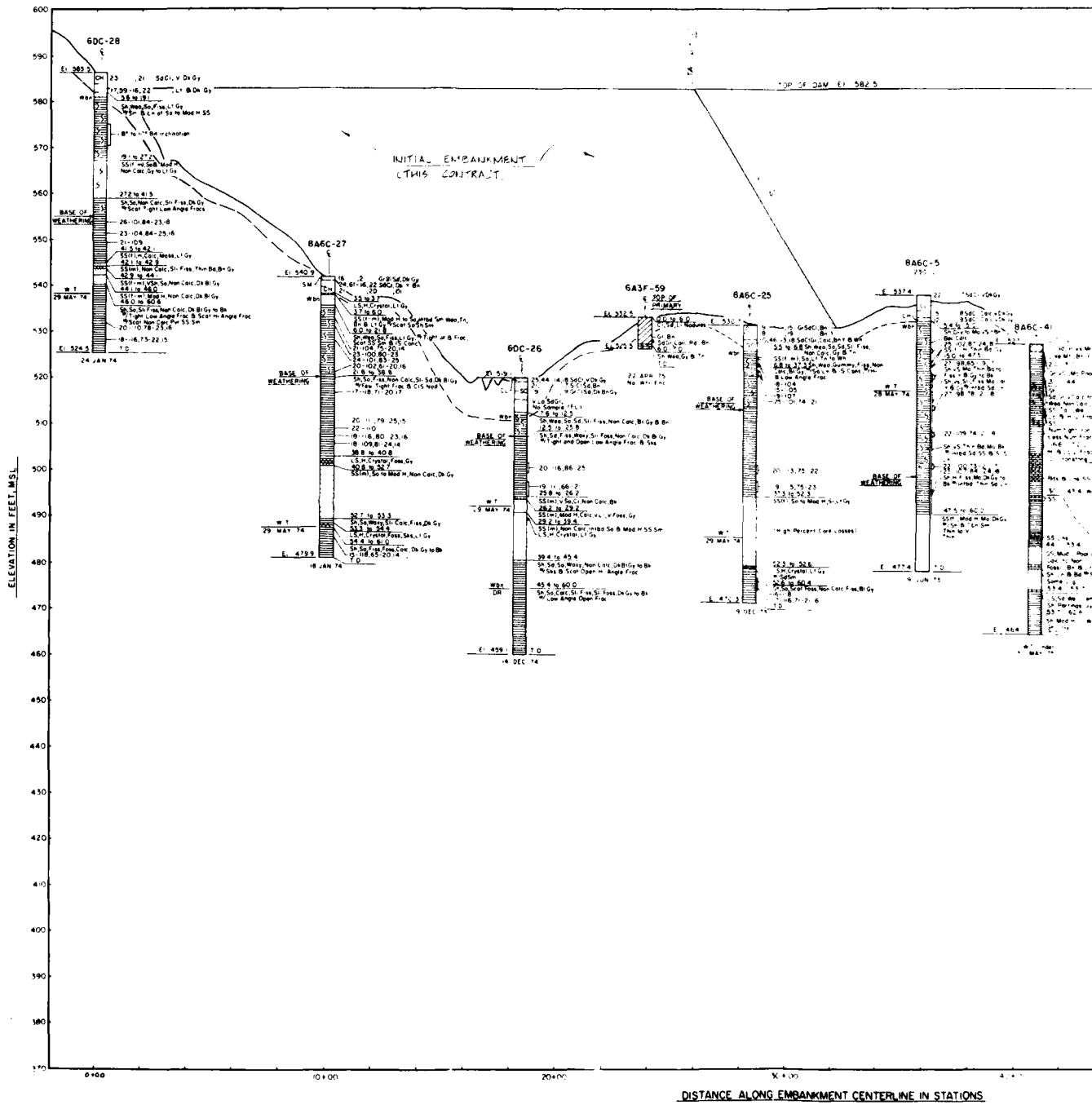
F/G 13/2 NL







CORPS OF ENGINEERS



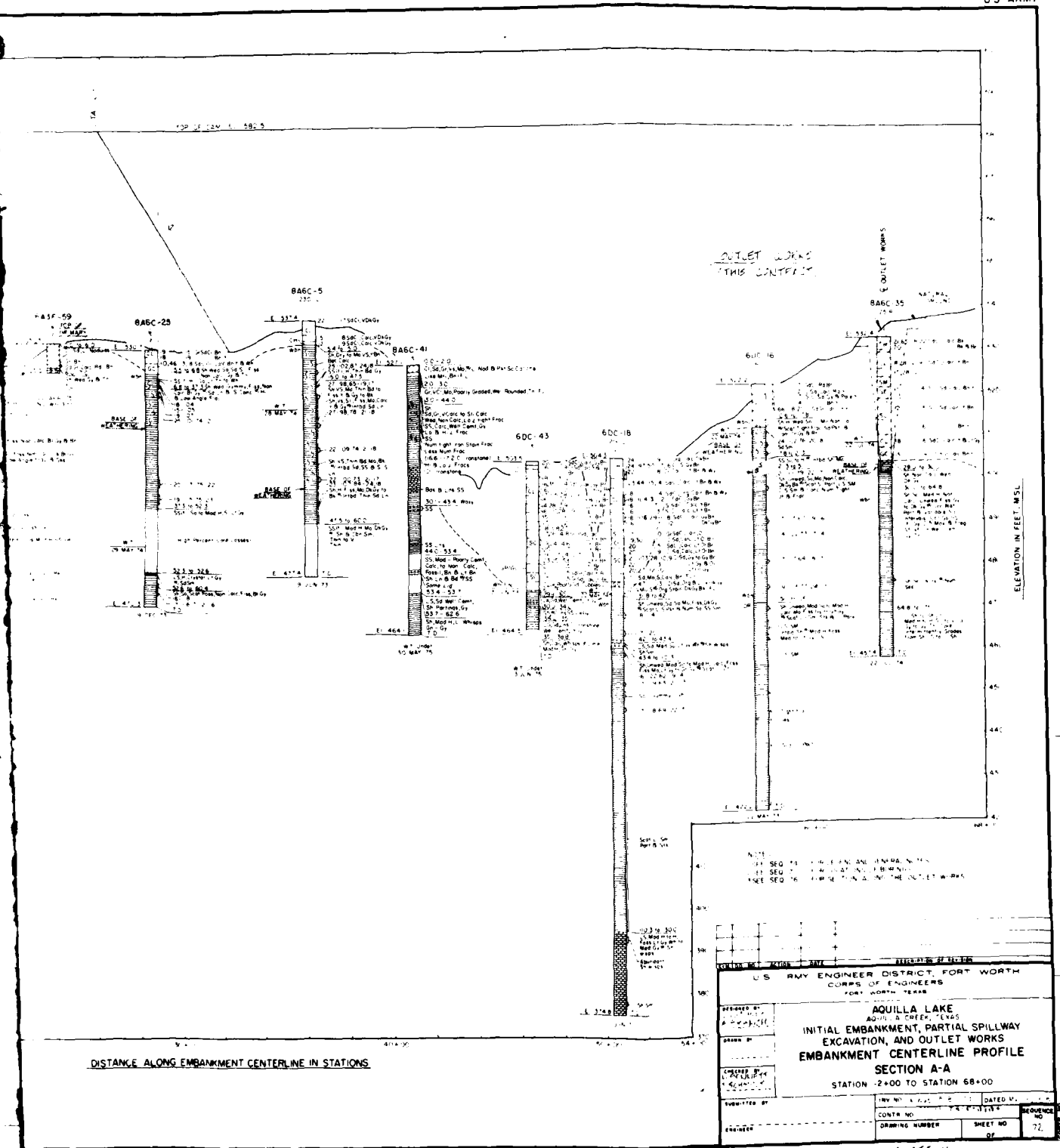


Figure 1 is a graph showing the relationship between the minimum calculated values of shear force and bending moment and the horizontal plane. The vertical axis is labeled "SHEAR FORCE, KNC" and the horizontal axis is labeled "HORIZONTAL PLANE". Two lines are plotted, one labeled "SHEAR FORCE" and the other "BENDING MOMENT". The lines intersect at a point labeled "MINIMUM CALCULATED VALUES OF SHEAR FORCE AND BENDING MOMENT".

SUMMARY OF AVAILABILITY
DETERMINED EARLY FORCES

	R.A. FACTOR		
	1	2	3
SA	100%	100%	100%
FD	50%	50%	50%
LB	50%	50%	50%
DE	50%	50%	50%

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80	81	82	83	84	85	86	87	88	89	90	91	92	93	94	95	96	97	98	99	100
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80	81	82	83	84	85	86	87	88	89	90	91	92	93	94	95	96	97	98	99	100

THE ABOVE INFORMATION IS NOT TO BE RELEASED
TO THE PUBLIC OR TO ANY OTHER PERSONS
EXCEPT BY AUTHORITY OF THE DIRECTOR OF THE
FEDERAL BUREAU OF INVESTIGATION

[illegible]

A BULKY AIR ANALYSIS WERE NOT TESTED DUE TO THE CONSTRUCTION CRITICAL FOR THE
WATER TREATMENT SECTION. LABORATORY HAS
TWO OR THREE DAYS FAILURE SENSITIVITY ANALYSIS
FOR THREE OTHERS HAVE BEEN DONE. A
REMARKABLE OF THE WATER AND AIR ANALYSIS
TREATMENT SYSTEMS. THE TEST SHOULD BE
THE WATER ANALYSIS.

1. THE MARSHALL ISLANDS, MICRONESIA, AND MALDIVAS
2. GUAM, PAGO PAGO, AND AUCKLAND
3. LAYSAN, PALMYRA, AND THE LINE ISLANDS
4. PALAU, THE MARIANAS, AND THE CAROLINES
5. THE MARIANAS, PALAU, AND THE CAROLINES
6. THE MARIANAS, PALAU, AND THE CAROLINES
7. THE MARIANAS, PALAU, AND THE CAROLINES
8. THE MARIANAS, PALAU, AND THE CAROLINES
9. THE MARIANAS, PALAU, AND THE CAROLINES
10. THE MARIANAS, PALAU, AND THE CAROLINES

[illegible]

BRAZOS RIVER BASIN, TEXAS

AQUILLA LAKE

AQUILLA CREEK, TEXAS
STABILITY ANALYSIS

RIGHT ABUTMENT EMBANKMENT SECTION
END-OF-CONSTRUCTION CONDITION
WEDGE METHOD

U S ARMY ENGINEER DISTRICT, FORT WORTH

MAY 1976

TO ACCOMPANY OFFSHORE DESIGN MEMORANDUM NO 7

FILE

NO

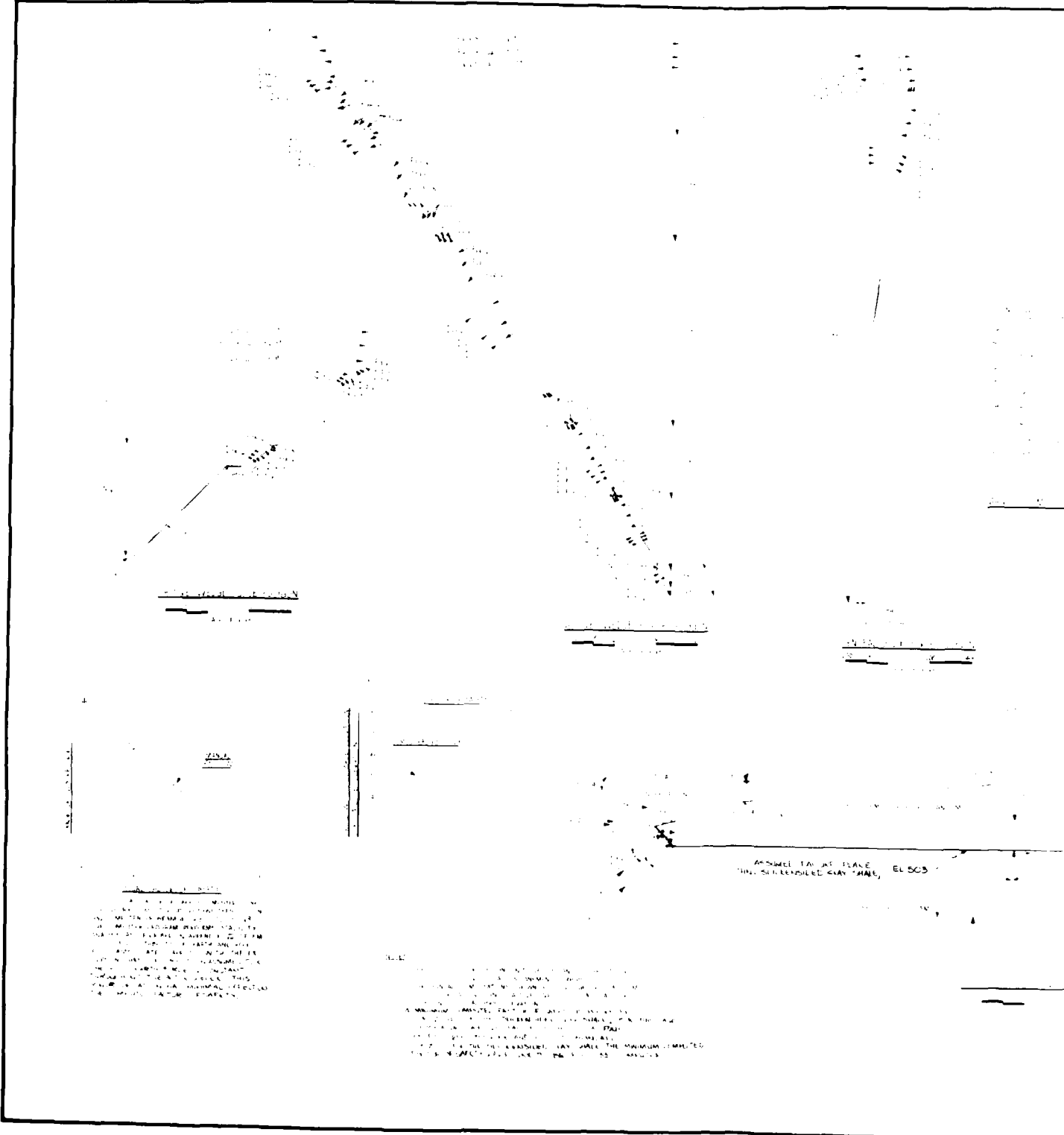
PLATE V-32

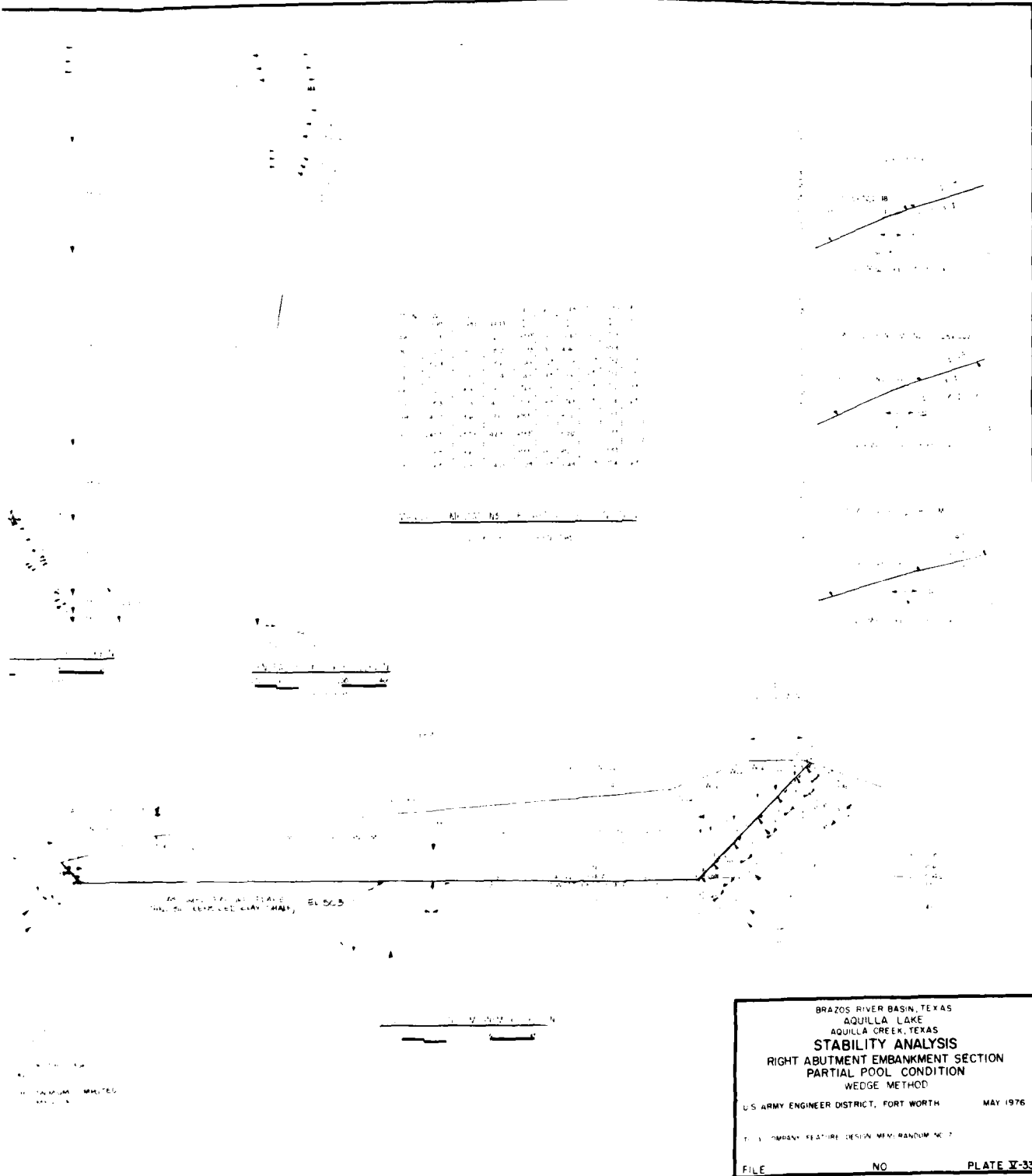
PLATE 13

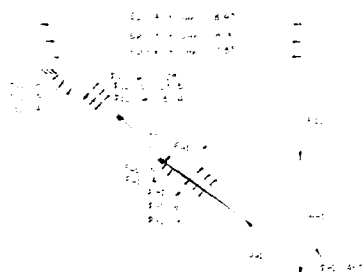
MANUAL COMPUTATIONS
FOR
END OF CONSTRUCTION CONDITION

[illegible]

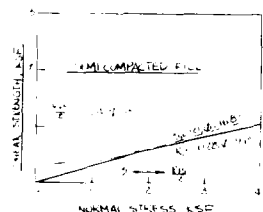
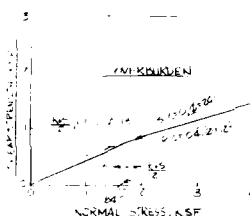
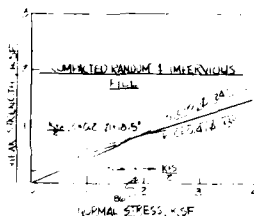
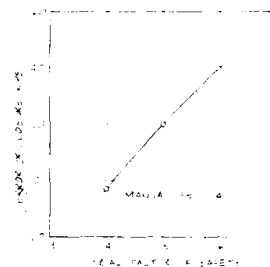
CORPS OF ENGINEERS





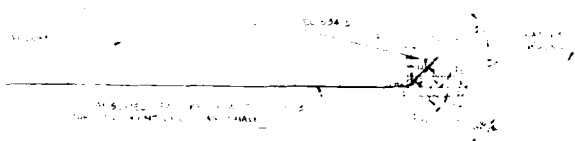


• ALL THE USERS FREE - DOLBY



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BRAZOS RIVER BASIN, TEXAS
AQUILLA LAKE
AQUILLA CREEK, TEXAS
STABILITY ANALYSIS
RIGHT ABUTMENT EMBANKMENT SECTION
STEADY SEEPAGE CONDITION
(SURCHARGE POOL - MAXIMUM WATERSURFACE)
WEDGE METHOD

U.S. ARMY ENGINEER DISTRICT, FORT WORTH MAY 1976

FILE _____ NO _____ PLATE V-34

6-2-2

ASSUMED WATER TABLE
RANGE OF VARIATION

THE FOLLOWING DATA WERE OBTAINED FROM THE
FIELD SURVEY OF THE EMBANKMENT AT
AQUILLA LAKE, TEXAS, IN 1974.
THE EMBANKMENT IS A GRAVELLY SAND
FILL ON A CLAY SHALE FOUNDATION.
THE EMBANKMENT IS 10 FEET HIGH
AND 10 FEET WIDE AT THE TOP.
THE EMBANKMENT IS 10 FEET
WIDE AT THE BASE.

• EMBANKMENT NO. 1

THE EMBANKMENT IS A GRAVELLY SAND
FILL ON A CLAY SHALE FOUNDATION.
THE EMBANKMENT IS 10 FEET HIGH
AND 10 FEET WIDE AT THE TOP.
THE EMBANKMENT IS 10 FEET
WIDE AT THE BASE.

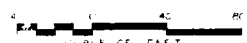
THE EMBANKMENT IS A GRAVELLY SAND
FILL ON A CLAY SHALE FOUNDATION.
THE EMBANKMENT IS 10 FEET HIGH
AND 10 FEET WIDE AT THE TOP.
THE EMBANKMENT IS 10 FEET
WIDE AT THE BASE.

THE EMBANKMENT IS A GRAVELLY SAND
FILL ON A CLAY SHALE FOUNDATION.
THE EMBANKMENT IS 10 FEET HIGH
AND 10 FEET WIDE AT THE TOP.
THE EMBANKMENT IS 10 FEET
WIDE AT THE BASE.

THE EMBANKMENT IS A GRAVELLY SAND
FILL ON A CLAY SHALE FOUNDATION.
THE EMBANKMENT IS 10 FEET HIGH
AND 10 FEET WIDE AT THE TOP.
THE EMBANKMENT IS 10 FEET
WIDE AT THE BASE.

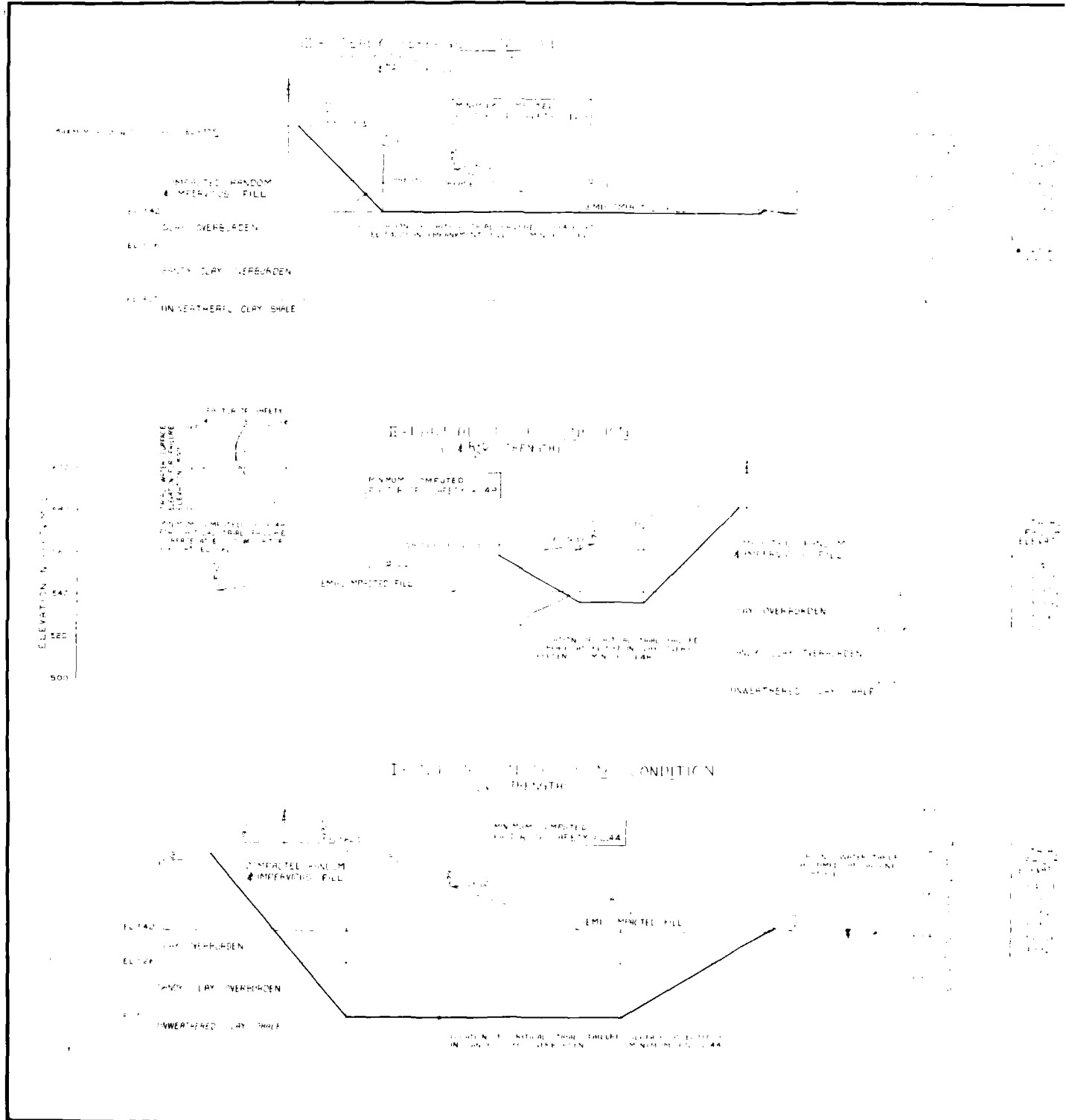
THE FOLLOWING DATA WERE OBTAINED FROM THE
FIELD SURVEY OF THE EMBANKMENT AT
AQUILLA LAKE, TEXAS, IN 1974.
THE EMBANKMENT IS A GRAVELLY SAND
FILL ON A CLAY SHALE FOUNDATION.
THE EMBANKMENT IS 10 FEET HIGH
AND 10 FEET WIDE AT THE TOP.
THE EMBANKMENT IS 10 FEET
WIDE AT THE BASE.

EMBANKMENT NO. 1
LIMIT OF EMBANKMENT
LIMIT OF EMBANKMENT
LIMIT OF EMBANKMENT



BRAZOS RIVER BASIN, TEXAS	
AQUILLA LAKE	
AQUILLA CREEK, TEXAS	
STABILITY ANALYSIS	
FLOODPLAIN EMBANKMENT SECTION	
1. END OF CONSTRUCTION CONDITION	
2. PARTIAL FLOOD CONDITION	
3. STEADY SEEPAGE CONDITION, DOWNHILL FROM	
WEDGE METHOD	
U.S. ARMY ENGINEER DISTRICT, FORT WORTH	MAY 1976
TO ACCOMPANY DRAWING NO. 1	
FILE	NO
PLATE V-35	

CORPS OF ENGINEERS



TOTAL EMBEDDED ELEVATION	MAXIMUM ELEVATION
1.00	1.00
1.00	1.00
1.00	1.00
1.00	1.00
1.00	1.00
1.00	1.00

WEDGE METHOD OF ANALYSIS
FOR EMBANKMENT STABILITY

TOTAL EMBEDDED ELEVATION	MAXIMUM ELEVATION
1.00	1.00
1.00	1.00
1.00	1.00
1.00	1.00
1.00	1.00
1.00	1.00

TOTAL EMBEDDED ELEVATION	MAXIMUM ELEVATION
1.00	1.00
1.00	1.00
1.00	1.00
1.00	1.00
1.00	1.00
1.00	1.00

BRAZOS RIVER BASIN, TEXAS
AQUILLA LAKE
AQUILLA CREEK, TEXAS
STABILITY ANALYSIS
LEFT ABUTMENT EMBANKMENT SECTION
I. END OF CONSTRUCTION CONDITION
II. PARTIAL POOL CONDITION
III. STEADY SEEPAGE CONDITION SURCHARGE POOL
WEDGE METHOD
U.S. ARMY ENGINEER DISTRICT, FORT WORTH MAY 1976
ACCOMPANY FEATURE DM NO 7
FILE NO PLATE V-36

IMPERVIOUS FILL

TABLE 1 OF 1

[illegible]

RANDOM FILE

TABLE 1 OF 2

SAMPLE IDENTIFICATION		CLASSIFICATION		FIELD COMPONENT CONTROL										LABORATORY CONFIRMATION			
				SPLIT TYPE	ATTEMBERG LIMITS		FIELD TESTS		LIQUID LIMIT CORRELATION		MOISTURE CONTENT AND DENSITY		GOV'T PROJECT		RECORD SAMPLE		
													LAB TESTS		SND LAB TESTS		
FIELD NO.	STATION	OFFSET	ELEV.	CH	LL	PL	SH	WT	WATER	CONTENT	MAX DRY OPT	MAX DRY OPT	PERCENT	MAX DRY OPT	MAX DRY OPT	MAX DRY OPT	MAX DRY OPT
RD-3	14-15	14-15	53.1	CH	68.8	87.8	97.1				92.4	21.7	102.0				
RD-10	26-40	15-05	53.5	CH	50.6	81.8	86.1	75.1			94.2	23.4	101.7	102.0	91.7	25.7	
RD-15	19-15	15-05	53.2	CH	58.9		81.2	13.1			105.6	17.4	109.3				
RD-25	14-15	15-05	53.2	CH	67.2		102.7	18.5			103.0	20.3	109.7	101.9	22.2		
RD-25A	11-15	15-05	53.6	CH	50.0	30.9		13.1			92.4	21.7	102.0				
RD-100A	16-10	17-15	52.4	CH	66.6	36.4		27.2	19.8		130.0	20.5	109.7	99.9	18.7	102.7	20.9
RD-13	18-15	15-05	52.5	CH	51.6		89.2	23.8			96.5	22.5	104.4	102.8			
RD-100C	14-15	15-05	52.3	CH	60.6		90.9	23.1			92.2	21.7	101.6	104.1	19.2	103.2	19.2
RD-15	26-15	81-05	53.3	CH	58.1	38.4		108.9	21.0		96.5	22.5	104.4	102.8			
RD-25	22-15	73-05	53.8	CH	50.0		97.5	21.6			92.4	21.7	102.0				
RD-60	22-15	73-05	53.8	CH	59.7	32.6		23.2	21.0		97.8	21.6	101.6	102.8			
RD-65	23-15	73-05	53.2	CH	52.6	38.7		103.3	21.2		97.6	22.6	101.1	108.7			
RD-75	28-15	73-05	53.2	CH	58.9		92.0	23.4			93.5	23.8	101.6	106.7			
RD-75	15-10	12-15	53.0	CH	50.0	61.8		130.6	21.5		97.6	22.5	101.6	102.8			
RD-80	14-10	15-05	53.5	CH	52.6	38.6		105.6	21.6		95.8	22.4	101.6	102.7	17.3		
RD-85	18-10	12-15	53.0	CH	58.7	32.6		101.2	22.9		98.6	21.2	101.7	102.9			
RD-90	15-15	73-05	52.4	CH	51.8	34.1		100.0	22.6		96.6	22.1	100.9	104.1	24.1	22.8	102.1
RD-95	15-10	50-05	52.3	CH	60.0	64.2		128.0	22.6		91.8	24.8	101.2	106.8			
RD-100	17-10	50-05	52.8	CH	60.1	64.1		98.0	22.6		92.2	25.8	101.1	106.8			
RD-105	21-10	50-05	53.7	CH	56.6	39.4		102.0	20.9		93.7	23.9	101.6	108.9			
RD-110	22-10	73-05	53.6	CH	61.1	62.0		101.2	24.2		91.8	25.1	100.9	101.7	37.3	23.3	99.6
RD-115	22-10	50-05	53.8	CH	60.0	62.9		98.9	22.6		91.9	25.0	101.6	106.7			
RD-120A	19-15	12-15	53.7	CH	53.9	17.7		101.2	21.4		93.1	22.8	100.3	102.0	97.7	20.3	103.0
RD-125A	19-15	50-05	53.6	CH	50.0		97.5	21.6			92.4	21.7	102.0				
RD-100P	17-10	15-05	53.3	CH	69.8	35.9		103.0	21.3		97.8	21.4	101.9	105.3	105.3	18.1	102.4
RD-135	19-10	15-05	52.4	CH	64.0	27.6		101.2	20.3		102.1	19.6	100.7	105.1			
RD-140	19-10	50-05	53.3	CH	58.2	38.1		101.1	24.3		92.5	22.5	101.8	106.3	95.7	22.4	
RD-145	19-10	110-05	53.3	CH	62.6	13.3		102.7	21.2		100.0	21.4	101.3	102.7			
RD-150A	12-15	15-05	53.3	CH	53.8	40.0		95.3	23.7		93.2	21.8	101.7		94.5	22.0	

RANDOM FILL

TABLE 2 OF 2

[illegible]
$$N(\mathbf{S}) = \{ \mathbf{S} \in \mathcal{S} : \mathbf{S} \text{ is a normal matrix} \}$$
[illegible]

SEMI-COMPACTED FILLS

[illegible]

SEM COMPACT MODEL

[illegible]

SUBJECTS

TABLE 1 OF 3

[illegible]

SEMI-COMPACTED FILL

TABLE 2 OF 3

[illegible]

SEMI-COMPACTED FILL

TABLE 2 OF 3

TEST DATA		FIELD COMPACTION CONTROL				LABORATORY COMPACTION DATA			
TEST NO.	DATE	TEST LOCATION	FIELD DATA		MOISTURE CONTENT		LAB. TESTS	REFERENCE	
			DENSITY	WATER	WET	WET			
1	10/1/54	100	100	100	100	100	100		
2	10/1/54	100	100	100	100	100	100		
3	10/1/54	100	100	100	100	100	100		
4	10/1/54	100	100	100	100	100	100		
5	10/1/54	100	100	100	100	100	100		
6	10/1/54	100	100	100	100	100	100		
7	10/1/54	100	100	100	100	100	100		
8	10/1/54	100	100	100	100	100	100		
9	10/1/54	100	100	100	100	100	100		
10	10/1/54	100	100	100	100	100	100		
11	10/1/54	100	100	100	100	100	100		
12	10/1/54	100	100	100	100	100	100		
13	10/1/54	100	100	100	100	100	100		
14	10/1/54	100	100	100	100	100	100		
15	10/1/54	100	100	100	100	100	100		
16	10/1/54	100	100	100	100	100	100		
17	10/1/54	100	100	100	100	100	100		
18	10/1/54	100	100	100	100	100	100		
19	10/1/54	100	100	100	100	100	100		
20	10/1/54	100	100	100	100	100	100		
21	10/1/54	100	100	100	100	100	100		
22	10/1/54	100	100	100	100	100	100		
23	10/1/54	100	100	100	100	100	100		
24	10/1/54	100	100	100	100	100	100		
25	10/1/54	100	100	100	100	100	100		
26	10/1/54	100	100	100	100	100	100		
27	10/1/54	100	100	100	100	100	100		
28	10/1/54	100	100	100	100	100	100		
29	10/1/54	100	100	100	100	100	100		
30	10/1/54	100	100	100	100	100	100		
31	10/1/54	100	100	100	100	100	100		
32	10/1/54	100	100	100	100	100	100		
33	10/1/54	100	100	100	100	100	100		
34	10/1/54	100	100	100	100	100	100		
35	10/1/54	100	100	100	100	100	100		
36	10/1/54	100	100	100	100	100	100		
37	10/1/54	100	100	100	100	100	100		
38	10/1/54	100	100	100	100	100	100		
39	10/1/54	100	100	100	100	100	100		
40	10/1/54	100	100	100	100	100	100		
41	10/1/54	100	100	100	100	100	100		
42	10/1/54	100	100	100	100	100	100		
43	10/1/54	100	100	100	100	100	100		

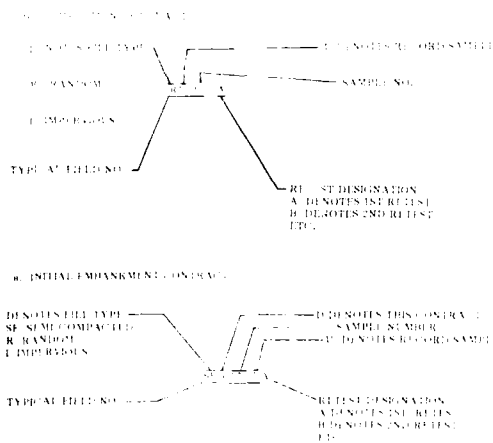
NOTES ON COMPACTLY $\mathcal{N}$ -ORTHOGONAL $\mathcal{N}$ -MOD $\mathcal{R}$ [illegible]

TABLE 1 OF 3

[illegible]

RESULTS

TYPICAL FIELD NO. 7

NOTES ON COMPACTAIR CONTROL FIELD NUMBER

DE NOTES FILE TYPE
 DE NOTES THIS CONTRA
 DE NOTES REL. OLD SAMPL
 REFUSE DESIGNATION
 AD NOTES INC. REFUSE
 DE NOTES AND REFUSE
 ID

Diagram illustrating the components of a typical field number:

- RC**: Denotes Fill Type
 - R**: Random
 - C**: ImperVIOUS
- 210**: Typical Field No.
- A**: Retest Designation
 - A**: Denotes 1st Retest
 - B**: Denotes 2nd Retest
 - ETC.**: Etc.

Additional labels in the diagram include:

- U**: Denotes Record Sample
- S**: Sample No.

REPORT NO. <u>1000</u> DATE <u>10/1/54</u>		REVISION NO. <u>NONE</u>	
DESIGNED BY <u>W. PAULIETT</u> <u>K. MORRILL</u>		U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS	
DRAWN BY <u>W. PAULIETT</u>		ADJUTANT GENERAL	
CHECKED BY <u>W. PAULIETT</u>		RECOMMENDATION: <u>FOR THE DISTRICT</u> <u>FOR THE DISTRICT</u> <u>FOR THE DISTRICT</u>	
SUBMITTED BY <u>W. PAULIETT</u>		(COMPLETION) <u>10/1/54</u>	
APPROVED BY <u>W. PAULIETT</u>		INVENTORY NO. <u>1000</u> DATED <u>10/1/54</u>	
ENGINEER		COUNTY NO. <u>1000</u>	
DRAWING NUMBER <u>1000</u>		SHEET NO. <u>1000</u>	
OF <u>1000</u>		INVENTORY NO. <u>1000</u>	

[illegible]

TIME

SAMPLE IDENTIFICATION				CLASSIFICATION	
				SOIL TYPE	ATTEMPTED LIMITS
FIELD NO.	STATION	OFFSET	ELEV.	LL	PI
1	100	0	100		
2	100	0	100		

a. INITIAL, 1.00ANKM; NT CONTRACT

TYPE: FILE NO. _____ EPOCH: DISC NO. _____
 A 100 N 0718 180 100 100
 B 100 N 0718 180 100 100
 C 100 N 0718 180 100 100

100

END OF 2

SAMPLE IDENTIFICATION				CLASSIFICATION		FIELD COMPACTION CONTROL						LABORATORY COMPACTION TESTS			
				SOIL TYPE	ATTERBERG LIMITS	FIELD TESTS		LIQUID LIMIT CORRELATION		MOISTURE CONTENT AND DENSITY		GOV'T PROJECT LAB TESTS		RECORD SAMPLES SUB LAB TESTS	
FIELD NO	STATION	OFFSET	ELEV.	LL	PI	DRY DENS.	WATER CONTENT	MAX DRY DENSITY	OPT WAT	% OF OPT	PERCENT	MAX DRY DENSITY	OPT WAT	MAX DRY DENSITY	OPT WAT
1															
2															
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4															
5															
6															
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1. *Abstract* 10

[illegible]

NOTES ON COMPA 2006-0518 (continued) M. J. R.

$$u_1 = \frac{1}{2} \left(\frac{1}{\sqrt{2}} \begin{pmatrix} 1 \\ 1 \end{pmatrix} + \frac{1}{\sqrt{2}} \begin{pmatrix} 1 \\ -1 \end{pmatrix} \right) = \frac{1}{2} \begin{pmatrix} 1+1 \\ 1-1 \end{pmatrix} = \begin{pmatrix} 1 \\ 0 \end{pmatrix}.$$

1. DATE OF RECEIPT
 2. NAME OF COMPANY
 3. NAME OF PERSON
 4. ADDRESS
 5. CITY
 6. STATE
 7. ZIP CODE
 8. PHONE NUMBER
 9. FAX NUMBER
 10. E-MAIL ADDRESS
 11. NAME OF PROJECT
 12. NAME OF PERSON
 13. ADDRESS
 14. CITY
 15. STATE
 16. ZIP CODE
 17. PHONE NUMBER
 18. FAX NUMBER
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 93. NAME OF PERSON
 94. ADDRESS
 95. CITY
 96. STATE
 97. ZIP CODE
 98. PHONE NUMBER
 99. FAX NUMBER
 100. E-MAIL ADDRESS

TYPICAL

REFLECT DISSEMINATION
A DUNCAN'S INC. REFLECT
DUNCAN'S AND REFLECT
Etc.

1.1. *Background*

121 N. 1st St., 1st Fl., New York, N.Y.

R. R. N. 1997

1. INTRODUCTION

TYPICAL FIELD NO.

— *Journal of the American Statistical Association*, 1997, 92, 1, 11–22.

— 122 —

— **RETEST POSITIONING**
 A. 10 NOTES IN; RETEST
 B. 10 NOTES IN; OR TEST
 1.

SUBMITTER'S NAME		SUBMITTER'S TITLE		SUBMITTER'S ORGANIZATION	
RECORD NO. _____ DRAWING NO. _____ SHEET NO. _____		U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS			
RECEIVED BY _____ DATE _____ APPROVED BY _____ DATE _____		1627 1627 1627			
SUBMITTED BY _____ CONTRACT NO. _____ DRAWING NUMBER _____ SHEET NO. _____ OF _____		INVO NO. _____ CONTRACT NO. _____ DRAWING NUMBER _____ SHEET NO. _____ OF _____		QUANTITY NO. _____	

RECORD SAMPLE TEST DATA: COMPLETION CONTRACT

RANDOM FILLS

[illegible]

TABLE 2. (continued)

[illegible]

RANDOM VIII.

[illegible]

TABLE 2 (continued)

[illegible]

IMPROVED FILLS

[illegible]

TABLE 1 OF

Time	Altitude	Latitude	Longitude	Temperature	Humidity	Wind Speed	Wind Direction	Clouds	Visibility	Remarks
01:00	5000	30°N	100°E	10°C	60%	10	090	0	10	Clear
02:00	5000	30°N	100°E	10°C	60%	10	090	0	10	Clear
03:00	5000	30°N	100°E	10°C	60%	10	090	0	10	Clear
04:00	5000	30°N	100°E	10°C	60%	10	090	0	10	Clear
05:00	5000	30°N	100°E	10°C	60%	10	090	0	10	Clear
06:00	5000	30°N	100°E	10°C	60%	10	090	0	10	Clear
07:00	5000	30°N	100°E	10°C	60%	10	090	0	10	Clear
08:00	5000	30°N	100°E	10°C	60%	10	090	0	10	Clear
09:00	5000	30°N	100°E	10°C	60%	10	090	0	10	Clear
10:00	5000	30°N	100°E	10°C	60%	10	090	0	10	Clear
11:00	5000	30°N	100°E	10°C	60%	10	090	0	10	Clear
12:00	5000	30°N	100°E	10°C	60%	10	090	0	10	Clear
13:00	5000	30°N	100°E	10°C	60%	10	090	0	10	Clear
14:00	5000	30°N	100°E	10°C	60%	10	090	0	10	Clear
15:00	5000	30°N	100°E	10°C	60%	10	090	0	10	Clear
16:00	5000	30°N	100°E	10°C	60%	10	090	0	10	Clear
17:00	5000	30°N	100°E	10°C	60%	10	090	0	10	Clear
18:00	5000	30°N	100°E	10°C	60%	10	090	0	10	Clear
19:00	5000	30°N	100°E	10°C	60%	10	090	0	10	Clear
20:00	5000	30°N	100°E	10°C	60%	10	090	0	10	Clear
21:00	5000	30°N	100°E	10°C	60%	10	090	0	10	Clear
22:00	5000	30°N	100°E	10°C	60%	10	090	0	10	Clear
23:00	5000	30°N	100°E	10°C	60%	10	090	0	10	Clear

NOTES ON COMPACTION CONTROL FIELD SUMBLER

b. INITIAL EMBANKMENT CONTRACT

90. COTTELL, J. G. 1968. ANTHRA-1.

DENOTES FILL TYPE
 SF SEMI-COMPACTED
 R RANDOM
 I IMPERVIOUS

D DENOTES THIS CONTRACT
 SAMPLE NUMBER
 U DENOTES RECORD SAMPLE

DEFINES FILE TYPE

R RANDOM

TYPICAL FIELD NO.

TYPICAL FIELD SIZE

RETEST DESIGNATION
A DENOTES 1ST RETEST
B DENOTES 2ND RETEST
ETC.

RECORD SAMPLE TIME DATA: INITIAL, 0.000000.

PREVIOUS FILE

CONTRACT				NO.		DATE		TIME		PLACE		REMARKS	
DATE	NAME	AGE	SEX	NO.	DATE	TIME	PLACE	REMARKS	DATE	NAME	AGE	SEX	NO.
1900	John	25	M	1	1900	10:00	1000	1000	1900	John	25	M	1
1901	John	25	M	2	1901	10:00	1000	1000	1901	John	25	M	2
1902	John	25	M	3	1902	10:00	1000	1000	1902	John	25	M	3
1903	John	25	M	4	1903	10:00	1000	1000	1903	John	25	M	4
1904	John	25	M	5	1904	10:00	1000	1000	1904	John	25	M	5
1905	John	25	M	6	1905	10:00	1000	1000	1905	John	25	M	6
1906	John	25	M	7	1906	10:00	1000	1000	1906	John	25	M	7
1907	John	25	M	8	1907	10:00	1000	1000	1907	John	25	M	8
1908	John	25	M	9	1908	10:00	1000	1000	1908	John	25	M	9
1909	John	25	M	10	1909	10:00	1000	1000	1909	John	25	M	10
1910	John	25	M	11	1910	10:00	1000	1000	1910	John	25	M	11
1911	John	25	M	12	1911	10:00	1000	1000	1911	John	25	M	12
1912	John	25	M	13	1912	10:00	1000	1000	1912	John	25	M	13
1913	John	25	M	14	1913	10:00	1000	1000	1913	John	25	M	14
1914	John	25	M	15	1914	10:00	1000	1000	1914	John	25	M	15
1915	John	25	M	16	1915	10:00	1000	1000	1915	John	25	M	16
1916	John	25	M	17	1916	10:00	1000	1000	1916	John	25	M	17
1917	John	25	M	18	1917	10:00	1000	1000	1917	John	25	M	18
1918	John	25	M	19	1918	10:00	1000	1000	1918	John	25	M	19
1919	John	25	M	20	1919	10:00	1000	1000	1919	John	25	M	20
1920	John	25	M	21	1920	10:00	1000	1000	1920	John	25	M	21
1921	John	25	M	22	1921	10:00	1000	1000	1921	John	25	M	22
1922	John	25	M	23	1922	10:00	1000	1000	1922	John	25	M	23
1923	John	25	M	24	1923	10:00	1000	1000	1923	John	25	M	24
1924	John	25	M	25	1924	10:00	1000	1000	1924	John	25	M	25
1925	John	25	M	26	1925	10:00	1000	1000	1925	John	25	M	26
1926	John	25	M	27	1926	10:00	1000	1000	1926	John	25	M	27
1927	John	25	M	28	1927	10:00	1000	1000	1927	John	25	M	28
1928	John	25	M	29	1928	10:00	1000	1000	1928	John	25	M	29
1929	John	25	M	30	1929	10:00	1000	1000	1929	John	25	M	30
1930	John	25	M	31	1930	10:00	1000	1000	1930	John	25	M	31
1931													

TABLE 1 OF 1

SEMI-COMPACTED FILL.

[illegible]

[APPL. 1, 95]

PYTHON

A-1										A-2										A-3										A-4										A-5										A-6										A-7										A-8										A-9										A-10										A-11										A-12										A-13										A-14										A-15										A-16										A-17										A-18										A-19										A-20										A-21										A-22										A-23										A-24										A-25										A-26										A-27										A-28										A-29										A-30										A-31										A-32										A-33										A-34										A-35										A-36										A-37										A-38										A-39										A-40										A-41										A-42										A-43										A-44										A-45										A-46										A-47										A-48										A-49										A-50										A-51										A-52										A-53										A-54										A-55										A-56										A-57										A-58										A-59										A-60										A-61										A-62										A-63										A-64										A-65										A-66										A-67										A-68										A-69										A-70										A-71										A-72										A-73										A-74										A-75										A-76										A-77										A-78										A-79										A-80										A-81										A-82										A-83										A-84										A-85										A-86										A-87										A-88										A-89										A-90										A-91										A-92										A-93										A-94										A-95										A-96										A-97										A-98										A-99										A-100									
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TABLE 1 OF 1

NOTES ON COMPACTIFICATION OF THE CONTROL SYSTEM NUMBER

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1. **INSTRUMENT TYPE**

12. $\{1, 2, 3, \dots, 100\}$

1. **EXPERIMENTAL DESIGN**

TYPE A: FIELD NO.

5. *Journal of the American Statistical Association*, 93(443):1089-1092, 1998.

$$\begin{aligned} \text{Fe} &= 0.11 \text{ g Fe} \cdot 100 \text{ g Fe}_2\text{O}_3 \cdot 100\% \\ \text{Al} &= 0.11 \text{ g Al} \cdot 100 \text{ g Al}_2\text{O}_3 \cdot 100\% \\ \text{Si} &= 0.11 \text{ g Si} \cdot 100 \text{ g SiO}_2 \cdot 100\% \end{aligned}$$

DEPARTMENT OF AGRICULTURE
WASHINGTON, D. C.
OFFICE OF THE SECRETARY

DESIGNATION
CLASS RETS
CLASS RETS

DRAWING NO.		DATE	
UNITED STATES OF AMERICA			
U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS			
DESIGNED BY: J. M. H. H. H.			
CHECKED BY:			
DRAWN BY:			
IN CHARGE:			
REVIEWED BY:			
APPROVED BY:			
SUBMITTED BY:			
	REV NO.	DATED	
	CONTR NO.		
ENGINEER	DRAWING NUMBER	SHEET NO.	EVIDENCE NO.
		OF	

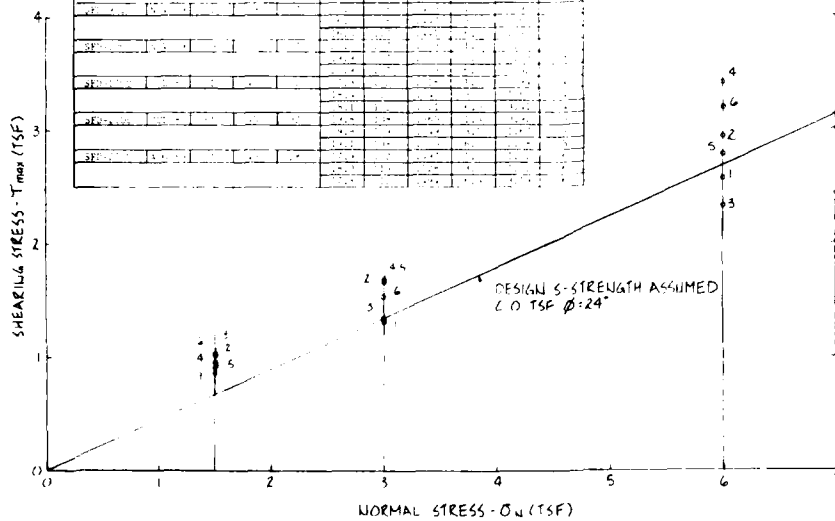
Diagram illustrating the Mohr's circle for a soil sample under stress. The vertical axis represents Shear Stress (τ) in KSE, and the horizontal axis represents Normal Stress (σ_n) in KSE.

The failure envelope is a straight line passing through the origin (0,0) and the point (3, 1.5). The Mohr's circle is drawn tangent to this line, with its center at $\sigma_c = 1.5$ KSE on the horizontal axis. The circle passes through point A at $(\sigma_1 = 3, \tau = 1.5)$ and point B at $(\sigma_3 = 0, \tau = 0)$.

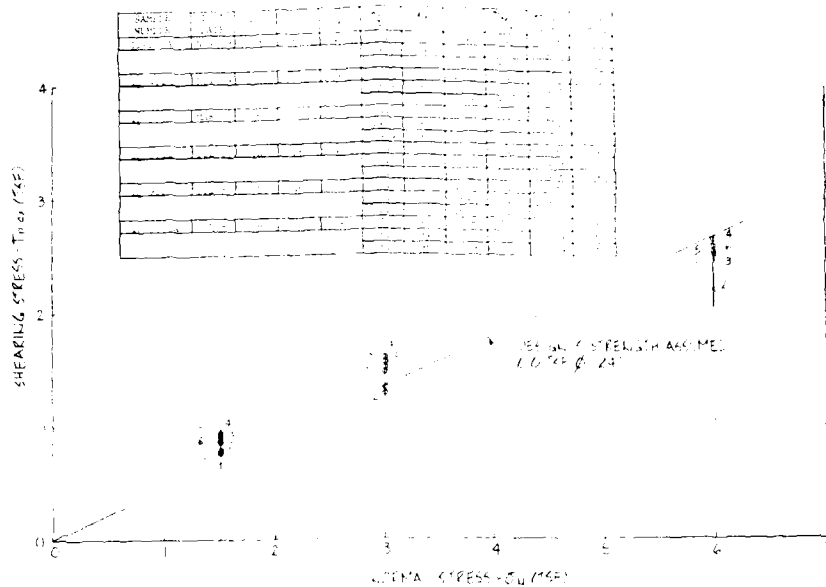
The maximum shear stress τ_{max} is indicated as 1.5 KSE.

Inset Table:

Principal Stress	Value (KSE)
σ_1	3
σ_3	0
τ_{max}	1.5

[illegible]

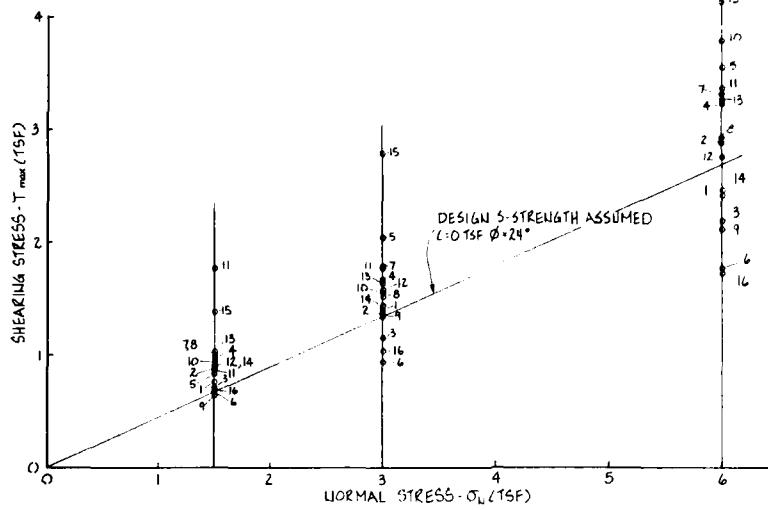
(CONSOLIDATED-DRAINED (S) DIRECT SHEAR TESTS ON SEMI-COMPACTED FILL



CONSOLIDATED GRADE (1) DIRECT SHEAR TESTS (2) IMPERVIOUS FILL

1. The test results show that the soil is a fine-grained soil with a high plasticity index. The soil is classified as a clayey silt or silty clay. The soil is highly compressible and has a low shear strength. The soil is not suitable for use as a foundation material for heavy structures. The soil is suitable for use as a foundation material for light structures.

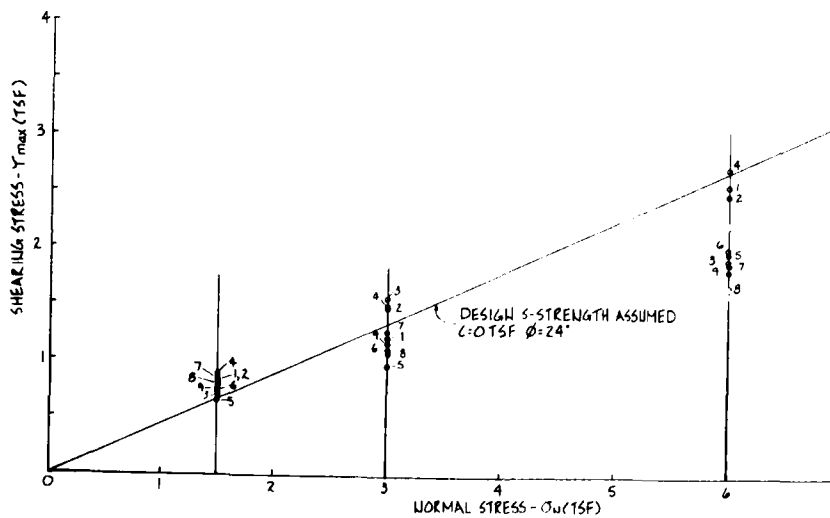
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PROJECT LAITY: [Laity]	
PROJECT MONKS: [Monks]	
PROJECT NUNS: [Nuns]	
PROJECT PRIESTS: [Priests]	
PROJECT BISHOPS: [Bishops]	
PROJECT CARDINALS: [Cardinals]	
PROJECT POPES: [Popes]	
PROJECT PATRIARCHS: [Patriarchs]	
PROJECT METROPOLITANS: [Metropolitans]	
PROJECT ARCHBISHOPS: [Archbishops]	
PROJECT BISHOPS: [Bishops]	
PROJECT PRIESTS: [Priests]	
PROJECT DEACONS: [Deacons]	
PROJECT MONKS: [Monks]	
PROJECT NUNS: [Nuns]	
PROJECT CLERGY: [Clergy]	
PROJECT LAITY: [Laity]	
PROJECT MON	



CONSOLIDATED-DRAINED (S) DIRECT SHEAR TESTS ON RANDOM FILL

Consolidated-Drained (S) Direct Shear Tests on Random Fill

SAMPLE NUMBER	TEST DATE	CLASS.	LL	P1	W _c (%)	#
RC-100	JAN 81	CL	42	22	25.0	1
RC-100	JAN 81	CL	42	22	25.0	2
RC-100	JAN 81	CL	42	22	25.0	3
RC-100	JAN 81	CL	42	22	25.0	4
RC-100	JAN 81	CL	42	22	25.0	5
RC-100	JAN 81	CL	42	22	25.0	6
RC-100	JAN 81	CL	42	22	25.0	7
RC-100	JAN 81	CL	42	22	25.0	8
RC-100	JAN 81	CL	42	22	25.0	9
RC-100	JAN 81	CL	42	22	25.0	10
RC-100	JAN 81	CL	42	22	25.0	11
RC-100	JAN 81	CL	42	22	25.0	12
RC-100	JAN 81	CL	42	22	25.0	13
RC-100	JAN 81	CL	42	22	25.0	14
RC-100	JAN 81	CL	42	22	25.0	15
RC-100	JAN 81	CL	42	22	25.0	16



CONSOLIDATED-DRAINED (S) DIRECT SHEAR TESTS ON IMPERVIOUS FILL

Consolidated-Drained (S) Direct Shear Tests on Impervious Fill

SAMPLE NUMBER	TEST DATE	CLASS.	LL	P1	W _c (%)	#
IC-120A	DEC 81	CH	58	22	25.0	1
IC-140A	DEC 81	CL	42	22	25.0	2
IC-160B	JAN 82	CH	62	22	25.0	3
IC-270	MAY 81	CL	32	22	25.0	4
IC-320	MAY 81	CH	62	22	25.0	5
IC-350	MAY 81	CH	62	22	25.0	6
IC-360A	APR 81	CH	72	22	25.0	7
IC-380	APR 81	CH	72	22	25.0	8
IC-390A	APR 81	CL	42	22	25.0	9

Consolidated-Drained (S) Direct Shear Tests on Random Fill (Completion Contract)

SAMPLE NUMBER	TEST DATE	CLASS.	LL	PI	w_c (%)	V_d (pcf)	e_o	σ_v (tsf)	τ_{max} (tsf)	PLOT SYMBOL
RL-200	DEC 81	CL	22	24	18.2	109	.555	3.0	0.75	1
					18.2	109	.507	3.0	1.14	2
					18.2	110	.499	6.0	2.40	3
RL-210	JAN 82	CL	22	30	16.5	114	.455	3.0	0.81	1
					17.8	113	.478	3.0	1.15	2
					17.5	114	.459	6.0	2.66	3
RL-240	JAN 82	CL	22	28	18.0	111	.515	3.0	0.70	1
					18.3	111	.509	3.0	1.15	2
					18.5	110	.528	6.0	2.16	3
RL-250	MAR 82	CL	22	29	17.0	115	.463	3.0	0.97	1
					15.2	118	.428	3.0	1.66	2
					12.2	119	.390	6.0	1.71	3
RL-260	MAR 82	CL	22	28	15.8	118	.423	3.0	0.84	1
					14.7	119	.398	3.0	2.02	2
					14.6	119	.403	6.0	1.53	3
RL-270	MAY 82	CH	26	27	26.3	97	.695	1.5	0.86	1
					27.3	96	.722	3.0	0.93	2
					26.7	97	.701	6.0	1.77	3
RL-280	MAY 82	CH	27	27	14.7	120	.393	1.5	0.98	1
					15.3	119	.408	3.0	1.28	2
					15.5	120	.398	6.0	1.28	3
RL-290	MAY 82	CH	27	24	14.0	110	.516	1.5	0.98	1
					18.0	110	.522	3.0	1.51	2
					18.7	110	.522	6.0	2.40	3
RL-300	JUN 82	CL	25	30	26.2	99	.711	1.5	0.85	1
					26.3	98	.686	3.0	1.16	2
					26.2	96	.705	6.0	2.09	3
RL-310	JUN 82	CH	27	14	15.8	117	.402	1.5	0.92	1
					15.2	115	.396	3.0	1.73	2
					15.0	117	.410	6.0	1.75	3
RL-320	MAY 82	CH	25	22	18.7	108	.540	1.5	0.81	1
					14.3	109	.526	3.0	1.76	2
					12.5	108	.527	6.0	1.11	3
RL-330	MAY 82	CH	26	25	14.0	112	.475	1.5	0.89	1
					15.3	111	.487	3.0	1.55	2
					17.9	112	.475	6.0	2.73	3
RL-340	MAY 82	CH	26	29	12.7	112	.416	1.5	0.92	1
					14.4	112	.412	3.0	1.63	2
					14.5	118	.408	6.0	1.24	3
RL-350	JUN 82	CH	27	27	20.2	110.7	.510	1.5	0.85	1
					20.0	111.6	.510	3.0	1.43	2
					20.0	108.2	.500	6.0	2.51	3
RL-360	JUN 82	CH	26	12	12.3	122.1	.370	1.5	1.37	1
					11.8	123.2	.360	3.0	2.77	2
					13.7	117.5	.420	6.0	4.10	3
RL-370	JUN 82	CH	26	29	25.7	92.4	.850	1.5	0.69	1
					24.9	96.0	.790	3.0	1.03	2
					28.0	93.4	.800	6.0	1.71	3

LEGEND

CLASS SAMPLE CLASSIFICATION ACCORDING TO UNITED SOIL CLASSIFICATION SYSTEM

LL LIQUID LIMIT
PI PLASTICITY INDEX
 w_c MOISTURE CONTENT
 e_o DRY DENSITY
 σ_v INITIAL VOID RATIO
 τ_{max} NORMAL STRESS
SHEAR STRESS AT FAILURE

Consolidated-Drained (S) Direct Shear Tests on Impervious Fill (Completion Contract)

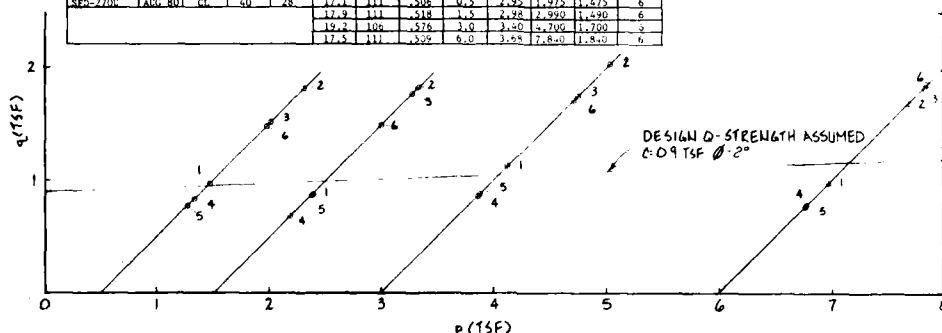
SAMPLE NUMBER	TEST DATE	CLASS.	LL	PI	w_c (%)	V_d (pcf)	e_o	σ_v (tsf)	τ_{max} (tsf)	PLOT SYMBOL
RL-100A	DEC 81	CH	26	19	25.6	97	.680	1.5	0.82	1
					25.0	98	.686	3.0	1.21	2
					25.2	97	.688	6.0	2.57	3
RL-100B	DEC 81	CH	24	16	23.2	102	.618	1.5	0.82	1
					24.6	99	.665	3.0	1.48	2
					23.4	101	.633	6.0	2.48	3
RL-100C	JAN 82	CH	24	24	27.2	94	.799	1.5	0.70	1
					28.7	92	.741	3.0	1.06	2
					24.4	93	.805	6.0	1.91	3
RL-100D	MAY 82	CH	24	24	21.0	106	.557	1.5	0.81	1
					21.2	106	.563	3.0	1.49	2
					20.2	108	.540	6.0	2.72	3
RL-100E	MAY 82	CH	26	25	26.9	96	.708	1.5	0.86	1
					26.1	95	.711	3.0	0.97	2
					25.9	98	.680	6.0	1.97	3
RL-100F	MAY 82	CH	26	23	21.4	98	.685	1.5	0.74	1
					26.4	96	.713	3.0	1.11	2
					22.5	98	.685	6.0	2.09	3
RL-100G	APR 82	CH	27	22	26.7	98.2	.710	1.5	0.84	1
					26.4	98.0	.727	3.0	1.24	2
					27.5	95.1	.790	6.0	1.89	3
RL-100H	APR 82	CH	23	24	25.2	99.2	.740	1.5	0.82	1
					28.0	91.2	.820	3.0	1.05	2
					27.2	95.0	.720	6.0	1.66	3
RL-100I	APR 82	CH	27	13	27.9	96.9	.780	1.5	0.75	1
					29.4	95.0	.740	3.0	1.17	2
					28.6	92.7	.790	6.0	1.82	3

NOTE:
NO RECORD SAMPLE TESTS PERFORMED ON
SEMI-COMPACTED FILL PLACED DURING
COMPLETION CONTRACT.

UNITED STATES OF AMERICA		SECTION OF RECORD	
U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS			
AQUILLA LAKE EMBANKMENT RECORD SAMPLE TESTS CONSOLIDATED-DRAINED (S) DIRECT SHEAR TESTS ON RANDOM & IMPERVIOUS FILL (COMPLETION CONTRACT)			
DESIGNED BY H.E. KARBS	DATE 12/81	REVISED BY	DATE
CHECKED BY	DATE	CONTRACT NO.	SEQUENCE NO.
DRAWING NUMBER	SHEET NO.	OF	

Unconsolidated-Undrained (q) Tests on Semi-Compacted Fill (Initial Embankment Contract)

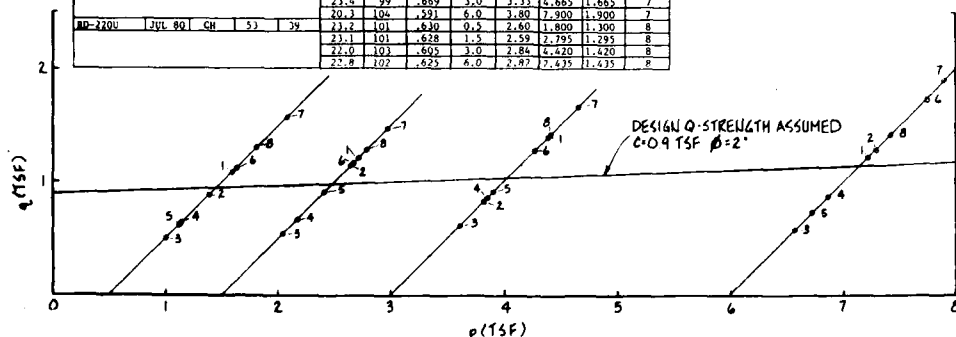
SAMPLE NUMBER	TEST DATE	CLASS.	LL	PI	w _c (%)	S _d (pcf)	e _s	σ _h (TSF)	σ _v -σ _h (TSF)	p (TSF)	q (TSF)	PLOT SYMBOL
SFD-80U	NOV 79	CL	48	14	15.0	98	.699	0.5	1.93	1.445	0.885	1
					15.7	97	.717	1.5	1.78	2.390	0.890	1
					15.9	95	.753	3.0	2.24	4.120	1.120	1
SFD-90U	NOV 79	CL	39	27	23.6	99	.691	6.0	1.92	6.960	0.960	1
					15.1	117	.436	0.5	1.62	2.310	1.810	2
					15.8	116	.451	1.5	3.60	3.330	1.830	2
SFD-100U	NOV 79	CH	51	37	15.5	116	.452	3.0	4.05	5.025	2.025	2
					15.3	117	.437	6.0	3.37	7.685	1.685	2
					20.3	108	.576	0.5	3.01	2.005	1.505	3
SFD-170U	JUL 80	CL	50	27	18.9	107	.527	1.5	3.82	3.285	1.765	3
					19.2	108	.535	3.0	3.49	5.745	1.745	3
					19.1	107	.552	6.0	3.62	7.810	1.810	3
SFD-210U	JUL 80	CL	45	32	22.6	102	.663	0.5	1.67	1.335	0.835	4
					19.1	99	.702	1.5	1.37	2.182	0.882	4
					19.1	107	.589	3.0	1.70	1.850	0.850	4
SFD-270U	AUG 80	CL	40	28	21.8	103	.652	6.0	1.57	6.785	0.785	4
					18.6	110	.538	0.5	1.34	1.270	0.770	5
					19.5	108	.558	1.5	1.73	2.375	0.875	5
SFD-270U	AUG 80	CL	40	28	19.7	107	.580	1.0	1.73	1.865	0.865	5
					18.8	110	.531	6.0	1.53	6.765	0.765	5
					17.7	111	.506	0.5	2.95	1.275	1.675	6
SFD-270U	AUG 80	CL	40	28	17.2	110	.518	1.5	2.28	2.990	1.490	6
					17.5	111	.525	6.0	3.58	7.840	1.840	6



UNCONSOLIDATED-UNDRAINED (q) TESTS ON SEMI-COMPACTED FILL

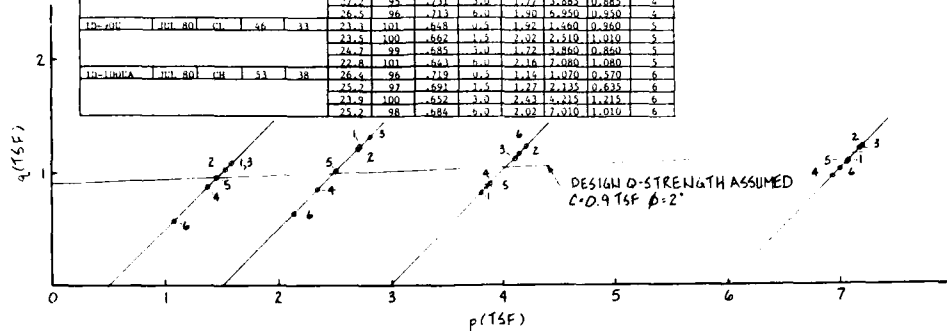
Unconsolidated-Undrained (q) Triaxial Shear Tests on Random Fill (Initial Embankment Contract)

SAMPLE NUMBER	TEST DATE	CLASS.	LL	PI	w _c (%)	S _d (pcf)	e _s	σ _h (TSF)	σ _v -σ _h (TSF)	p (TSF)	q (TSF)	PLOT SYMBOL
RD-40U	JUN 79	CL	47	34	20.3	106	.561	0.5	2.18	1.590	1.090	1
					19.7	107	.545	1.5	2.44	2.720	1.220	1
					19.2	109	.525	3.0	2.80	4.500	1.400	1
RD-30UA	NOV 79	CL	41	27	19.2	109	.525	6.0	2.43	4.500	1.400	1
					19.5	109	.528	0.5	1.78	1.390	0.890	2
					19.2	109	.535	1.5	2.30	2.650	1.150	2
RD-120CA	JAN 80	CL	44	31	20.7	107	.561	3.0	1.65	3.825	0.825	2
					19.1	109	.523	6.0	2.58	7.290	1.290	2
					26.0	96	.739	0.5	1.01	1.005	0.505	3
RD-110U	JAN 80	CL	47	34	25.9	97	.712	1.5	1.09	2.045	0.545	3
					25.9	97	.715	3.0	1.23	2.615	0.615	3
					25.5	98	.697	6.0	1.15	2.572	0.572	3
RD-90U	FEB 80	CH	54	38	23.4	100	.662	0.5	1.29	1.145	0.645	4
					23.0	101	.652	1.5	1.35	2.175	0.675	4
					21.7	101	.617	3.0	1.21	1.855	0.855	4
RD-110U	FEB 80	CH	60	42	22.0	101	.620	6.0	1.76	6.870	0.870	4
					27.8	94	.753	0.5	1.25	1.120	0.620	5
					27.2	95	.740	1.5	1.83	2.415	0.915	5
RD-180U	AUG 80	CL	43	28	26.5	96	.717	3.0	1.81	3.905	0.905	5
					27.6	98	.726	6.0	1.67	6.735	0.735	5
					26.4	96	.625	0.5	2.26	1.630	1.130	6
RD-220U	JUL 80	CH	53	39	25.7	97	.663	1.5	2.31	2.655	1.155	6
					26.1	96	.679	3.0	2.56	4.280	1.280	6
					24.2	99	.633	6.0	3.48	7.740	1.740	6
RD-180U	AUG 80	CL	43	28	20.7	101	.645	0.5	1.17	2.085	1.085	7
					24.7	96	.725	1.5	2.36	2.980	1.480	7
					23.4	99	.669	3.0	3.33	6.665	1.665	7
RD-220U	JUL 80	CH	53	39	20.3	104	.529	6.0	3.80	7.900	1.900	7
					23.1	101	.628	0.5	2.80	1.800	1.300	8
					23.1	101	.628	1.5	2.59	2.795	1.295	8
RD-220U	JUL 80	CH	53	39	22.0	103	.605	3.0	2.84	4.420	1.420	8
					22.8	102	.625	6.0	2.87	7.435	1.435	8

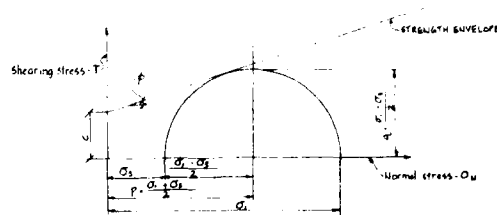


UNCONSOLIDATED-UNDRAINED (q) TRIAXIAL SHEAR TESTS ON RANDOM FILL (INITIAL EMBANKMENT CONTRACT)

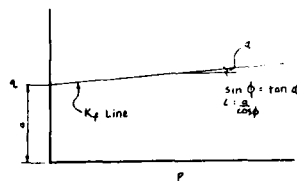
SAMPLE NUMBER	TEST DATE	CLASS.	LL	P1	W (%)	Std (FCF)	σ _o	σ _o (TSF)	σ _o + σ _o (TSF)	σ _o - σ _o (TSF)	σ _o / σ _o (TSF)	σ _o / σ _o (TSF)	σ _o / σ _o (TSF)
12-10-6A	NOV 79	CH	62	2	27.1	95	1.6	1.6	1.6	1.6	1.6	1.6	1.6
					25.3	97	2.0	1.9	2.0	2.0	2.0	2.0	2.0
					23.9	100	2.4	2.3	2.4	2.4	2.4	2.4	2.4
					22.6	95	2.4	2.4	2.4	2.4	2.4	2.4	2.4
12-10-6B	NOV 79	CH	62	36	25.0	100	0.8	0.8	0.8	0.8	0.8	0.8	0.8
					23.1	101	0.8	0.8	0.8	0.8	0.8	0.8	0.8
					22.8	104	0.7	0.7	0.7	0.7	0.7	0.7	0.7
12-10-6C	FEB 80	CH	62	37	22.4	99	0.8	0.8	0.8	0.8	0.8	0.8	0.8
					22.7	99	0.8	0.8	0.8	0.8	0.8	0.8	0.8
					21.9	98	0.7	0.7	0.7	0.7	0.7	0.7	0.7
					21.9	99	0.7	0.7	0.7	0.7	0.7	0.7	0.7
12-10-6D	JUL 80	CH	56	40	26.3	96	2.1	2.1	2.1	2.1	2.1	2.1	2.1
					27.6	95	2.1	2.1	2.1	2.1	2.1	2.1	2.1
					26.2	95	2.1	2.1	2.1	2.1	2.1	2.1	2.1
12-10-6E	JUL 80	CL	66	33	23.3	101	0.8	0.8	0.8	0.8	0.8	0.8	0.8
					23.5	100	0.8	0.8	0.8	0.8	0.8	0.8	0.8
					26.2	99	0.8	0.8	0.8	0.8	0.8	0.8	0.8
					23.0	101	0.8	0.8	0.8	0.8	0.8	0.8	0.8
12-10-6F	JUL 80	CH	56	38	26.4	96	2.1	2.1	2.1	2.1	2.1	2.1	2.1
					25.2	97	0.9	0.9	0.9	0.9	0.9	0.9	0.9
					23.9	100	0.9	0.9	0.9	0.9	0.9	0.9	0.9
					25.5	98	0.9	0.9	0.9	0.9	0.9	0.9	0.9



UNCONSOLIDATED-UNDRAINED (Q) TRIAXIAL SHEAR TESTS ON IMPERVIOUS FILL



DETERMINATION OF p^* AND q^* FROM
TRIAxIAL TEST USING MOHR'S DIAGRAM

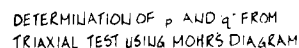
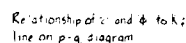


Relationship of γ and ϕ to K_f line on p - q diagram

UNIT NO.			SECTION OF SECTION		
U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS					
AQUILLA LAKE					
EMBANKMENT RECORD SAMPLE TESTS UNCONSOLIDATED-UNDRAINED (Q) TESTS ON RANDOM, IMPERVIOUS, & SEMI-COMPACTED FILL (INITIAL EMBANKMENT CONTRACT)					
SUBMITTED BY HE KARBS					
DESIGNED BY R. HOWELL		DATE OF DE LA MAR		CHECKED BY TCHUMIT	
DRAWN BY		CONTRACT NO.		REVISION NO.	
ENGINEER		ORDERING NUMBER		SHEET NO. OF	

SAMPLE NO	TEST DATE	CLASS.	LL	PI	N _c (3)	μ _{sf} (PSF)	c _c (TSF)	σ _c (TSF)	σ _{sf} (TSF)	τ _{sf} (TSF)	τ _c (TSF)	PLANT SYMBOL
11-300A	MAR 81	CH	26	39	22.9	98	665	5	1.84	1,420	9,920	5
					22.0	98	670	1.5	1.85	1,425	9,925	1
					22.1	97	680	3.0	1.72	1,380	9,880	1
					21.5	96	667	8.0	1.28	1,490	9,920	1
11-300A	MAR 81	CL	49	36	23.4	100	650	0.5	1.70	1,350	9,850	1
					23.2	98	692	1.5	1.60	1,400	9,800	1
					22.1	102	617	3.0	2.17	1,085	1,085	2
					22.2	102	620	6.0	2.13	1,085	1,085	2
11-300B	JAN 82	CH	68	49	29.2	90	359	0.5	1.87	1,435	9,915	1
					29.5	91	358	1.5	2.09	1,565	1,045	1
					29.5	91	362	3.0	2.16	1,580	1,040	1
					29.6	90	368	6.0	2.12	1,585	1,082	1
11-300B	MAY 82	CL	41	29	22.9	102	579	0.5	1.65	1,225	9,725	1
					23.0	102	555	1.5	1.96	1,480	9,980	1
					23.3	102	592	3.0	1.70	1,350	9,850	1
11-300C	MAY 82	CH	92	53	29.2	106	366	6.0	1.94	1,670	9,770	1
					29.3	99	370	1.5	2.10	1,610	9,810	1
					29.3	96	375	1.5	1.57	1,285	9,785	1
					29.5	95	378	3.0	1.52	1,260	9,780	1
					29.0	96	376	6.0	1.52	1,260	9,760	1
11-300C	MAY 82	CL	90	32	29.6	97	396	0.5	1.65	1,225	9,725	1
					29.6	98	392	1.5	1.62	1,210	9,725	1
					29.8	96	393	3.0	1.39	1,695	9,695	6
					29.7	97	697	6.0	1.68	1,840	9,840	5
11-300A	APR 82	CH	22	22	29.1	94	61	1.5	1.61	1,315	9,815	1
					29.1	94.5	6	3.0	1.78	1,890	9,890	1
					29.8	94.5	0	6.0	1.90	2,030	9,930	2
11-300C	APR 82	CH	73	56	29.3	92.5	9	1.5	1.74	2,270	9,870	9
					29.3	92.5	91	3.0	1.82	1,910	9,910	8
					29.6	94.5	81	6.0	1.56	1,780	9,780	8
11-300A	APR 82	CL	47	33	29.9	92.5	63	1.5	1.28	2,140	9,640	9
					30.4	91.8	85	3.0	1.27	1,635	9,635	9
					30.2	92.1	84	6.0	1.32	1,660	9,660	9

Note: Second test gave 11-50, PI-3A

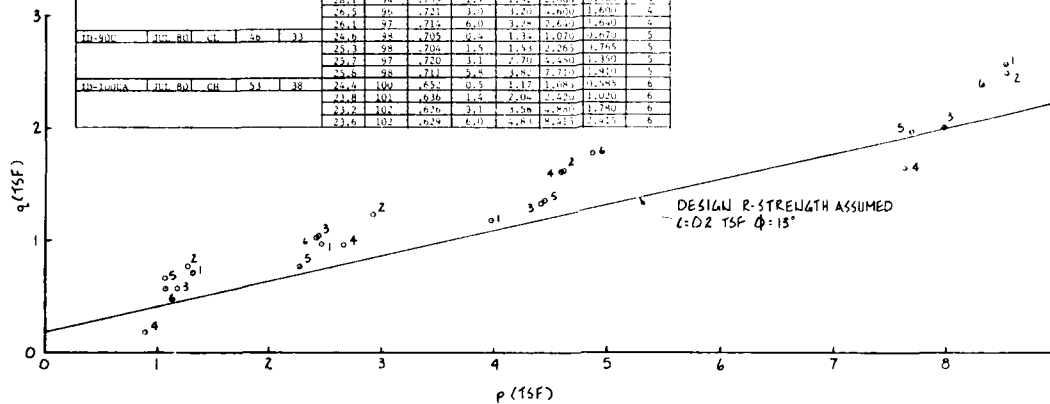


NOTE:
NO RECORD SAMPLE TESTS PERFORMED ON
SEMI-COMPACTED FILL PLACED DURING
COMPLETION CONTRACT.

UNCLASSIFIED		DATE		CLASSIFICATION BY	
DESIGNED BY K. POWELL K. HOWELL		U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS			
DRAWN BY J. GELNARE		AQUILLA LAKE			
RE-TESTED BY T. CHAMDS		EMBANKMENT RECORD SAMPLE TESTS UNCONSOLIDATED UNDRAINED (G) TESTS ON SAND (IMPERVIOUS FILL) (COMPLETION CONTRACT)			
SUBMITTED BY H.E. KARBS ENGINEER		INV NO CONTR NO DRAWING NUMBER		DATED SHEET NO OF	
				PROJECT NO DRAWING NO	

Consolidated-Undrained (R) Triaxial Shear Tests on Impervious Fill (Initial Embankment Control)

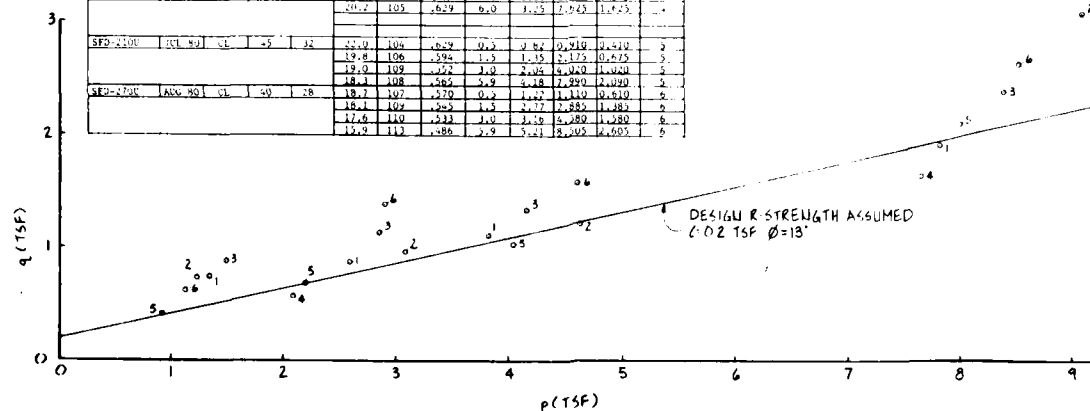
SAMPLE NUMBER	TEST DATE	CLASS.	LL	PI	w_c (%)	d (PCF)	e_0	σ'_v (TSF)	$\sigma'_v - \sigma'_v$ (TSF)	σ'_v (TSF)	q (TSF)	TEST SYMBOL
1D-900A	NOV 79	CH	80	34	19.5	107	0.70	0.6	1.47	1.135	0.715	1
					21.7	103	0.77	1.7	1.74	1.270	0.870	2
					20.9	105	0.68	2.2	2.20	1.400	1.100	3
					18.5	109	0.57	5.9	3.80	2.200	1.200	4
					16.5	114	0.49	0.6	1.14	1.420	0.720	5
					16.1	115	0.49	1.6	2.41	1.615	1.255	6
					15.2	115	0.48	5.9	2.41	2.905	1.705	7
					15.2	115	0.43	6.0	6.16	3.080	3.040	8
					18.6	110	0.56	0.6	1.75	1.425	0.875	9
					19.1	107	0.68	1.7	2.26	2.295	1.110	10
					18.7	107	0.65	5.8	5.57	2.110	1.255	11
					18.7	110	0.56	6.0	5.77	2.185	1.285	12
					19.5	106	0.60	1.5	1.13	2.062	0.765	13
					20.2	105	0.69	6.0	1.15	1.945	1.425	14
					22.0	104	0.69	0.5	0.82	0.910	0.510	15
					19.8	106	0.59	1.5	1.15	2.175	0.875	16
					19.0	109	0.64	1.0	2.18	2.020	1.130	17
					18.3	108	0.66	5.9	5.68	2.290	1.090	18
					18.7	107	0.70	0.2	1.62	1.110	0.510	19
					18.1	109	0.55	1.5	2.77	2.882	1.482	20
					17.6	110	0.53	1.0	2.10	2.280	1.280	21
					17.2	112	0.58	2.9	5.41	2.405	2.405	22



CONSOLIDATED-UNDRAINED (R) TRIAXIAL SHEAR TESTS ON IMPERVIOUS FILL

Consolidated-Undrained (R) Triaxial Shear Tests on Semi-Compacted Fill (Initial Embankment Control)

SAMPLE NUMBER	TEST DATE	CLASS.	LL	PI	w_c (%)	d (PCF)	e_0	σ'_v (TSF)	$\sigma'_v - \sigma'_v$ (TSF)	σ'_v (TSF)	q (TSF)	TEST SYMBOL
SFD-900	NOV 79	CL	80	34	19.5	107	0.70	0.6	1.47	1.135	0.715	1
					21.7	103	0.77	1.7	1.74	1.270	0.870	2
					20.9	105	0.68	2.2	2.20	1.400	1.100	3
					18.5	109	0.57	5.9	3.80	2.200	1.200	4
					16.5	114	0.49	0.6	1.14	1.420	0.720	5
					16.1	115	0.49	1.6	2.41	1.615	1.255	6
					15.2	115	0.48	5.9	2.41	2.905	1.705	7
					15.2	115	0.43	6.0	6.16	3.080	3.040	8
					18.6	110	0.56	0.6	1.75	1.425	0.875	9
					19.1	107	0.68	1.7	2.26	2.295	1.110	10
					18.7	107	0.65	5.8	5.57	2.110	1.255	11
					18.7	110	0.56	6.0	5.77	2.185	1.285	12
					19.5	106	0.60	1.5	1.13	2.062	0.765	13
					20.2	105	0.69	6.0	1.15	1.945	1.425	14
					22.0	104	0.69	0.5	0.82	0.910	0.510	15
					19.8	106	0.59	1.5	1.15	2.175	0.875	16
					19.0	109	0.64	1.0	2.18	2.020	1.130	17
					18.3	108	0.66	5.9	5.68	2.290	1.090	18
					18.7	107	0.70	0.2	1.62	1.110	0.510	19
					18.1	109	0.55	1.5	2.77	2.882	1.482	20
					17.6	110	0.53	1.0	2.10	2.280	1.280	21
					17.2	112	0.58	2.9	5.41	2.405	2.405	22



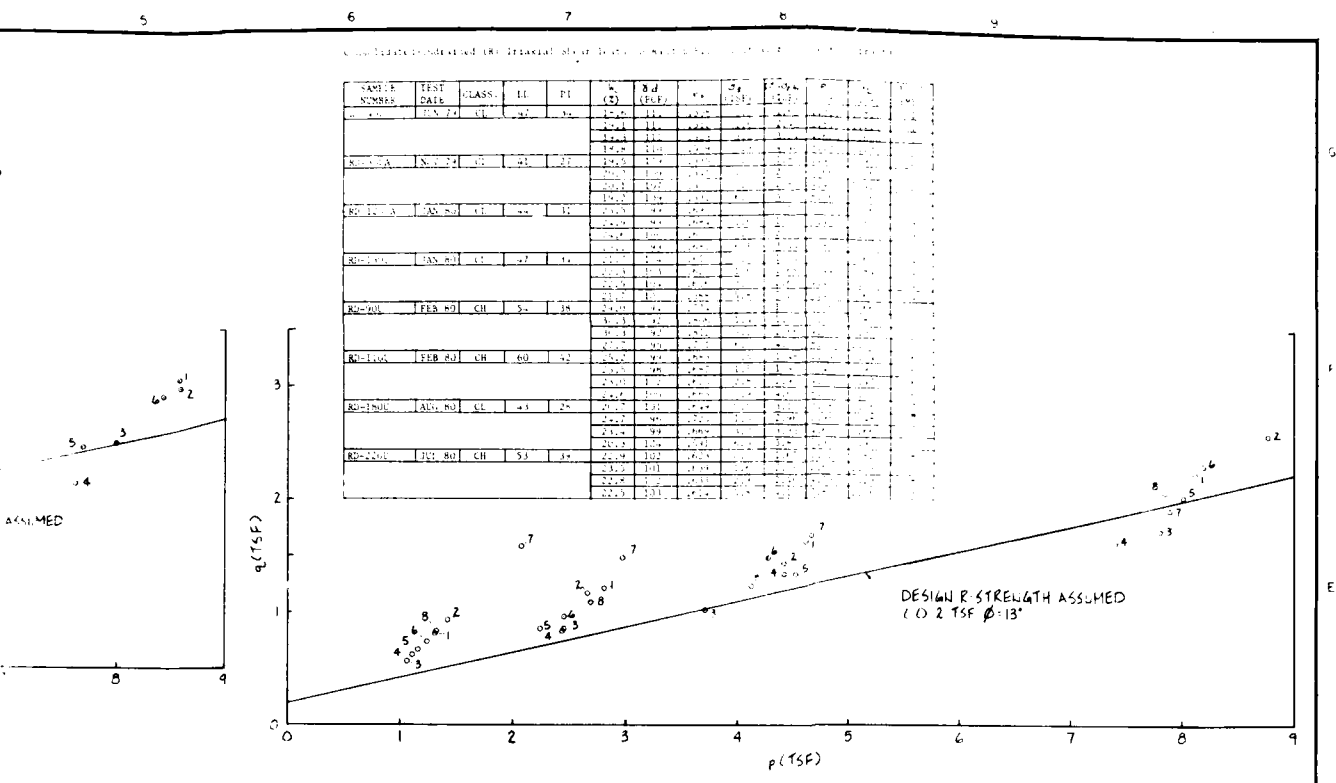
CONSOLIDATED-UNDRAINED (R) TRIAXIAL SHEAR TESTS ON SEMI-COMPACTED FILL

Consolidated-Undrained (R) Triaxial Shear Tests on Impervious Fill (Initial Embankment Control)

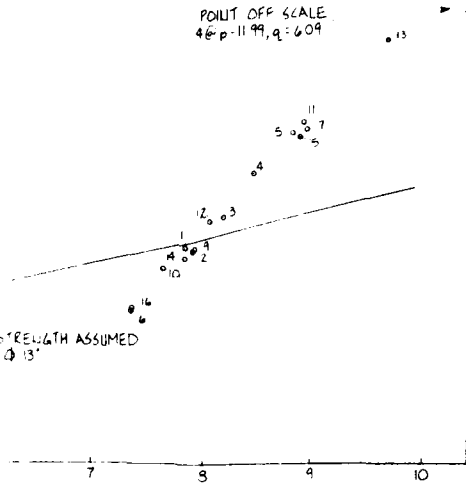
SAMPLE NUMBER	TEST DATE	CLASS.	LL	PI	w_c (%)	d (PCF)	e_0	σ'_v (TSF)	$\sigma'_v - \sigma'_v$ (TSF)	σ'_v (TSF)	q (TSF)	TEST SYMBOL
1D-900A	NOV 79	CH	80	34	19.5	107	0.70	0.6	1.47	1.135	0.715	1
					21.7	103	0.77	1.7	1.74	1.270	0.870	2
					20.9	105	0.68	2.2	2.20	1.400	1.100	3
					18.5	109	0.57	5.9	3.80	2.200	1.200	4
					16.5	114	0.49	0.6	1.14	1.420	0.720	5
					16.1	115	0.49	1.6	2.41	1.615	1.255	6
					15.2	115	0.48	5.9	2.41	2.905	1.705	7
					15.2	115	0.43	6.0	6.16	3.080	3.040	8
					18.6	110	0.56	0.6	1.75	1.425	0.875	9
					19.1	107	0.68	1.7	2.26	2.295	1.110	10
					18.7	107	0.65	5.8	5.57	2.110	1.255	11
					18.7	110	0.56	6.0	5.77	2.185	1.285	12
					19.5	106	0.60	1.5	1.13	2.062	0.765	13
					20.2	105	0.69	6.0	1.15	1.945	1.425	14
					22.0	104	0.69	0.5	0.82	0.910	0.510	15
					19.8	106	0.59	1.5	1.15	2.175	0.875	16
					19.0	109	0.64	1.0	2.18	2.020	1.130	17
					18.3	108	0.66	5.9	5.68	2.290	1.090	18
					18.7	107	0.70	0.2	1.62	1.110	0.510	19
					18.1	109	0.55	1.5	2.77	2.882	1.482	20
					17.6	110	0.53	1.0	2.10	2.280	1.280	21
					17.2	112	0.58	2.9	5.41	2.405	2.405	22

CONSOLIDATED-UNDRAINED (R) TRIAXIAL SHEAR TESTS ON SEMI-COMPACTED FILL

Revised
line on p. 4



POINT OFF SCALE
46-1199, q: 609



STRENGTH ASSUMED
13

13 ON RANDOM FILL

LEGEND

CLASS	SAMPLE CLASSIFICATION ACCORDING TO UNIFIED SOIL CLASSIFICATION SYSTEM
LL	LIQUID LIMIT
PI	PLASTICITY INDEX
W	WATER CONTENT
Ed	DRY DENSITY
e ₀	INITIAL VOID RATIO
σ ₁	MAJOR PRINCIPAL STRESS
σ ₃	MINOR PRINCIPAL STRESS
(σ ₁ - σ ₃) _m	DEVIATOR STRESS AT FAILURE
P	$\frac{\sigma_1 + \sigma_3}{2}$
q	$\frac{\sigma_1 - \sigma_3}{2}$

STRENGTH ASSUMED
13

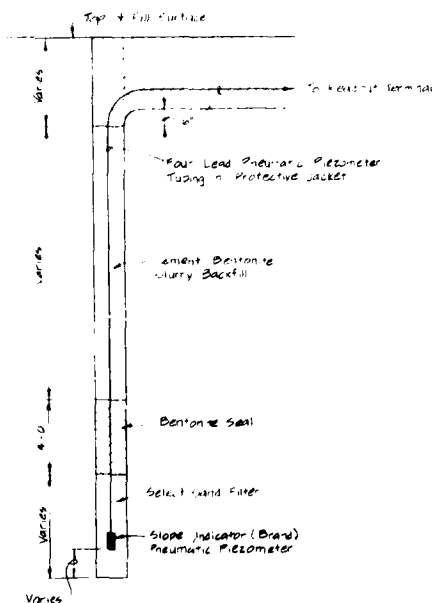
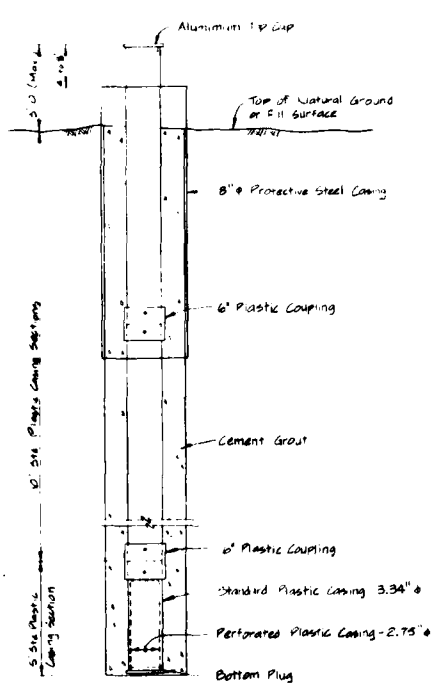
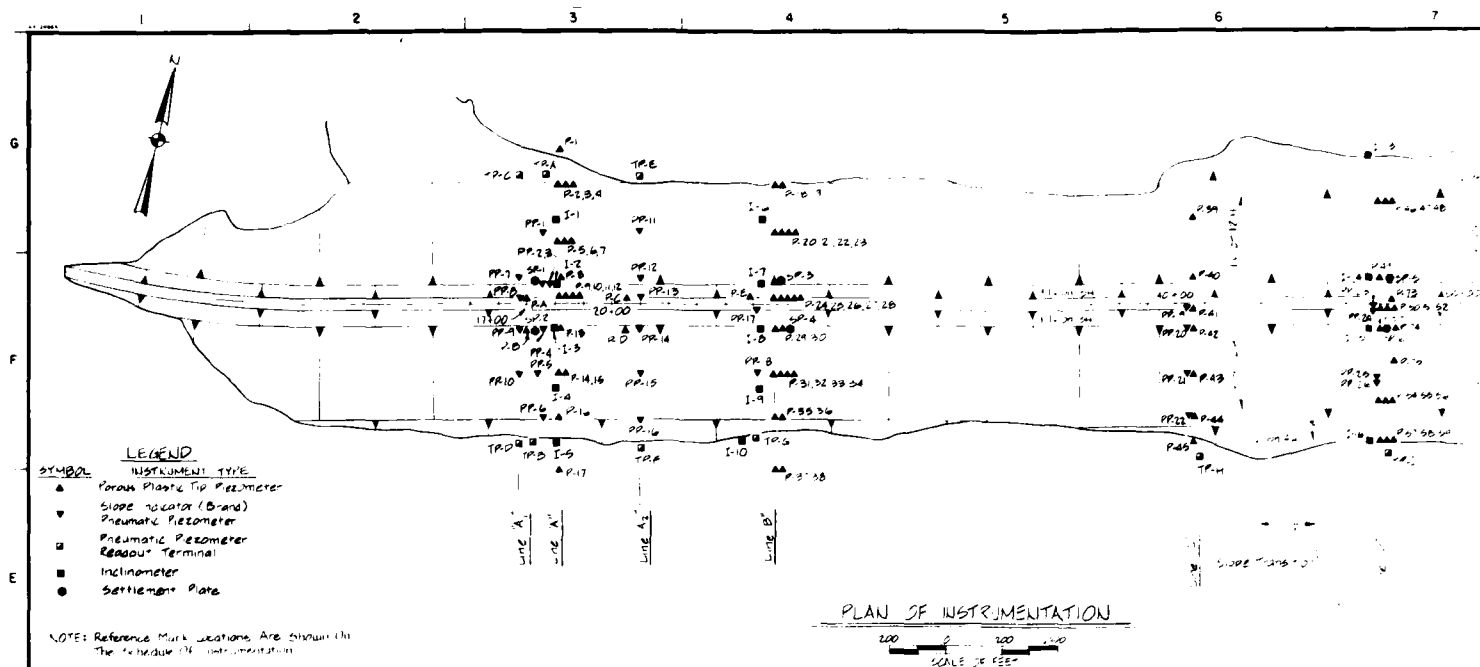
Consolidated-Undrained (R) Triaxial Shear Tests on Imperious Fill (Completion Contract)

SAMPLE NUMBER	TEST DATE	CLASS.	LL	PI	W _c (%)	Ed (PCF)	e ₀	σ ₁ (TSF)	σ ₃ (TSF)	(σ ₁ - σ ₃) _m (TSF)	P (TSF)	q (TSF)	PLIOT SYMBOL
IL-300A	DEC 81	CH	56	39	25.9	97	.705	0.5	1.09	1.045	0.545	0.545	1
IL-300A	DEC 81	CH	56	39	26.5	96	.710	1.3	1.57	2.085	0.785	0.785	2
IL-300A	DEC 81	CH	56	39	26.6	97	.708	3.2	2.22	2.316	1.116	1.116	3
IL-300A	DEC 81	CH	56	39	26.9	99	.698	5.9	3.91	7.825	1.925	1.925	4
IL-300A	DEC 81	CH	56	39	23.6	101	.637	0.5	1.10	1.050	0.550	0.550	5
IL-300A	DEC 81	CH	56	39	23.1	102	.626	1.4	1.63	2.215	0.815	0.815	6
IL-300A	DEC 81	CH	56	39	23.4	101	.621	3.0	2.95	4.675	1.475	1.475	7
IL-300A	DEC 81	CH	56	39	24.7	101	.615	6.0	3.63	7.815	1.815	1.815	8
IL-300A	JAN 82	CH	58	49	29.3	93	.827	0.6	0.96	1.080	0.480	0.480	9
IL-300A	JAN 82	CH	58	49	30.3	92	.843	1.6	1.68	2.350	0.750	0.750	10
IL-300A	JAN 82	CH	58	49	28.9	93	.813	3.1	1.85	4.020	0.920	0.920	11
IL-300A	JAN 82	CH	58	49	26.4	95	.789	6.0	3.10	7.350	1.350	1.350	12
IL-300A	MAY 81	CH	64	29	23.8	101	.647	0.5	1.08	0.940	0.340	0.340	13
IL-300A	MAY 81	CH	64	29	21.2	102	.681	1.3	1.60	2.190	0.890	0.890	14
IL-300A	MAY 81	CH	64	29	20.7	106	.667	3.3	2.51	4.255	1.255	1.255	15
IL-300A	MAY 81	CH	64	29	19.4	102	.654	6.0	3.84	5.324	1.814	1.814	16
IL-300A	MAY 81	CH	60	43	27.0	96	.741	0.6	0.96	0.925	0.325	0.325	17
IL-300A	MAY 81	CH	60	43	28.0	95	.718	1.3	0.92	1.760	0.460	0.460	18
IL-300A	MAY 81	CH	60	43	26.9	96	.710	3.3	1.74	3.170	0.870	0.870	19
IL-300A	MAY 81	CH	60	43	26.3	98	.715	6.0	2.92	7.380	1.560	1.560	20
IL-300A	MAY 81	CH	60	43	26.1	99	.684	0.5	0.76	0.880	0.380	0.380	21
IL-300A	MAY 81	CH	60	43	26.3	101	.640	1.4	1.15	1.972	0.572	0.572	22
IL-300A	MAY 81	CH	60	43	26.6	98	.702	3.0	1.86	3.930	0.930	0.930	23
IL-300A	MAY 81	CH	60	43	23.8	99	.681	2.8	3.03	7.315	1.515	1.515	24
IL-300A	MAY 81	CH	60	43	26.3	99.1	.790	1.5	1.63	2.315	0.815	0.815	25
IL-300A	MAY 81	CH	60	43	29.2	93.9	.820	3.0	2.02	4.010	1.010	1.010	26
IL-300A	MAY 81	CH	60	43	28.2	95.6	.780	6.0	3.03	7.515	1.515	1.515	27
IL-300A	APR 83	CH	73	34	27.0	97.4	.760	1.5	1.55	2.275	0.775	0.775	28
IL-300A	APR 83	CH	73	34	27.6	96.3	.780	3.0	2.01	4.005	1.005	1.005	29
IL-300A	APR 83	CH	73	34	28.2	95.9	.790	6.0	3.03	7.515	1.515	1.515	30
IL-300A	APR 83	CH	73	34	29.9	93.0	.820	1.5	1.37	2.185	0.885	0.885	31
IL-300A	APR 83	CH	73	34	29.1	94.0	.800	3.0	1.95	3.980	0.980	0.980	32
IL-300A	APR 83	CH	73	34	29.2	94.2	.800	6.0	2.91	7.555	1.555	1.555	33

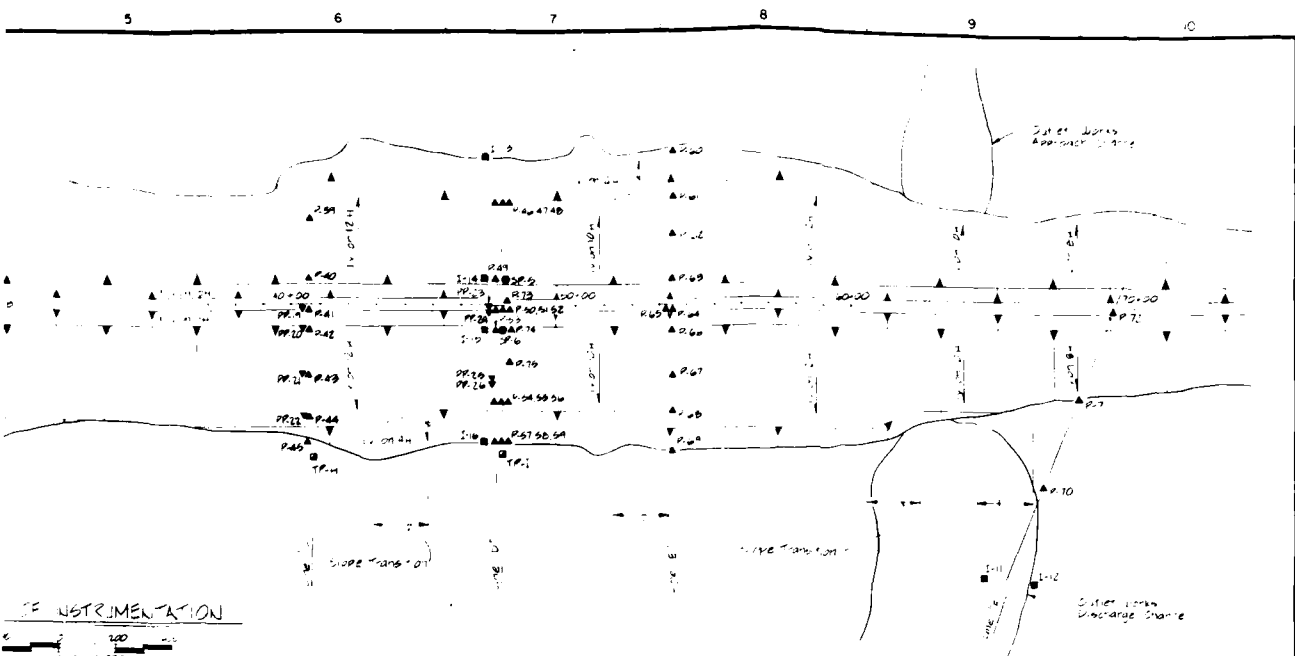
NOTE:
NO RECORD SAMPLE TESTS PERFORMED ON
SEMI-COMPACTED FILL PLACED DURING
COMPLETION CONTRACT.

U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS	
AQUILLA LAKE	
EMBANKMENT RECORD SAMPLE TESTS CONSOLIDATED-UNDRAINED (R) TRIAXIAL SHEAR TESTS ON RANDOM IMPERVIOUS FILL (COMPLETION CONTRACT)	
DESIGNED BY E. H. KARBS	DRAWN BY L. DELAMARE
CHECKED BY T. SCHMIDT	SUBMITTED BY H. E. KARBS
REVISION NO.	DATED
CONTRACT NO.	SHEET NO.
DRAWING NUMBER	SEQUENCE NO.

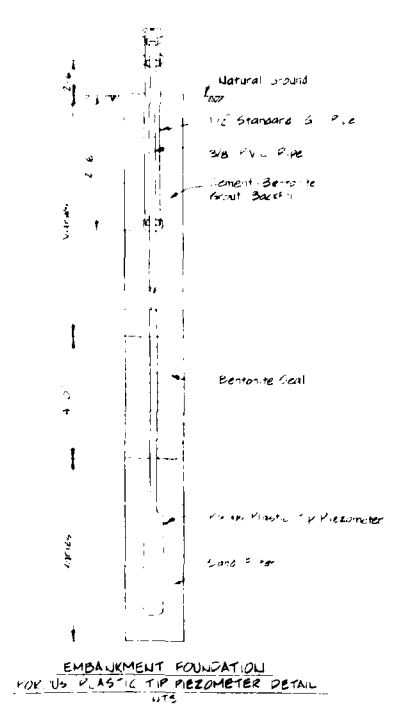
PLATE 27



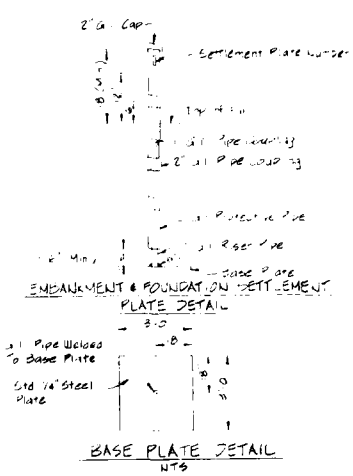
EMBANKMENT FOUNDATION
FOR US P. 150 TIP PIEZOMETER



INSTRUMENTATION
SCALE 1" = 100'



EMBANKMENT FOUNDATION
FOR US PLASTIC TIP PIEZOMETER DETAIL
NTS



DESIGNED BY H. PENNETT		CHECKED BY H. PENNETT	
DRAWN BY E. REED		DATE 10/1/54	
REVIEWED BY T. SCHMITZ		DATE 10/1/54	
SUBMITTED BY H. B. KAKES		DATE 10/1/54	
ENGINEER		DRAWING NUMBER 100-100-100	
		SHEET NO. 100-100-100	
		SEQUENCE NO. 100-100-100	

REFERENCE MARKS				
LINE	NO	STATION	OFFSET	REMARKS
A	RM-1	8100	450 US	
	RM-2	8100	350 US	
	RM-3	8100	420 US	
	RM-4	8100	300 US	
	RM-5	8100	200 US	
	RM-6	8100	10 US	
	RM-7	8100	10 DS	
	RM-8	8100	300 DS	
	RM-9	8100	300 DS	
	RM-10	8100	405 DS	
B	RM-11	8100	480 DS	
	RM-12	8100	510 DS	
	RM-13	8100	610 DS	
	RM-14	25150	340 US	
	RM-15	25150	440 US	
	RM-16	25150	300 US	
	RM-17	25150	220 US	
	RM-18	25150	10 US	
	RM-19	25150	10 DS	
	RM-20	25150	200 DS	
OUTLET DORIES CHANNEL	RM-21	25150	300 DS	
	RM-22	25150	480 DS	
	RM-23	25150	510 DS	
	RM-24	25150	610 DS	
	RM-25	25150	340 US	
	RM-26	25150	440 US	
	RM-27	25150	300 US	
	RM-28	25150	220 US	
	RM-29	25150	10 US	
	RM-30	25150	10 DS	
DEEP REFERENCE MARKS	DEM-1	1100	140 DS	DEEP REFERENCE MARKS INSTALLED
	DEM-2	1100	140 DS	DEEP REFERENCE MARKS INSTALLED
	DEM-3	1100	140 DS	DEEP REFERENCE MARKS INSTALLED
	DEM-4	1100	140 DS	DEEP REFERENCE MARKS INSTALLED
	DEM-5	1100	140 DS	DEEP REFERENCE MARKS INSTALLED
	DEM-6	1100	140 DS	DEEP REFERENCE MARKS INSTALLED
	DEM-7	1100	140 DS	DEEP REFERENCE MARKS INSTALLED
	DEM-8	1100	140 DS	DEEP REFERENCE MARKS INSTALLED
	DEM-9	1100	140 DS	DEEP REFERENCE MARKS INSTALLED
	DEM-10	1100	140 DS	DEEP REFERENCE MARKS INSTALLED
NOT INSTALLED	DEM-11	1100	140 DS	NOT INSTALLED
	DEM-12	1100	140 DS	NOT INSTALLED
	DEM-13	1100	140 DS	NOT INSTALLED
	DEM-14	1100	140 DS	NOT INSTALLED
	DEM-15	1100	140 DS	NOT INSTALLED
	DEM-16	1100	140 DS	NOT INSTALLED
	DEM-17	1100	140 DS	NOT INSTALLED
	DEM-18	1100	140 DS	NOT INSTALLED
	DEM-19	1100	140 DS	NOT INSTALLED
	DEM-20	1100	140 DS	NOT INSTALLED

POROUS PLASTIC TIP PIZOMETERS				
LINE	NO	STATION	OFFSET	REMARKS
A	P-1	8100	350 US	ABANDONED
	P-2	8100	420 US	ABANDONED
	P-3	8100	420 US	ABANDONED
	P-4	8100	420 US	ABANDONED
	P-5	8100	420 US	ABANDONED
	P-6	8100	420 US	ABANDONED
	P-7	8100	420 US	ABANDONED
	P-8	8100	420 US	ABANDONED
	P-9	8100	420 US	ABANDONED
	P-10	8100	420 US	ABANDONED
B	P-11	8100	420 US	ABANDONED
	P-12	8100	420 US	ABANDONED
	P-13	8100	420 US	ABANDONED
	P-14	8100	420 US	ABANDONED
	P-15	8100	420 US	ABANDONED
	P-16	8100	420 US	ABANDONED
	P-17	8100	420 US	ABANDONED
	P-18	8100	420 US	ABANDONED
	P-19	8100	420 US	ABANDONED
	P-20	8100	420 US	ABANDONED

POROUS PLASTIC TIP PIZOMETERS				
LINE	NO	STATION	OFFSET	REMARKS
C	P-31	40140	300 US	ABANDONED
	P-32	40140	300 US	ABANDONED
	P-33	40140	300 US	ABANDONED
	P-34	40140	300 US	ABANDONED
	P-35	40140	300 US	ABANDONED
	P-36	40140	300 US	ABANDONED
	P-37	40140	300 US	ABANDONED
	P-38	40140	300 US	ABANDONED
	P-39	40140	300 US	ABANDONED
	P-40	40140	300 US	ABANDONED
D	P-41	40140	300 US	ABANDONED
	P-42	40140	300 US	ABANDONED
	P-43	40140	300 US	ABANDONED
	P-44	40140	300 US	ABANDONED
	P-45	40140	300 US	ABANDONED
	P-46	40140	300 US	ABANDONED
	P-47	40140	300 US	ABANDONED
	P-48	40140	300 US	ABANDONED
	P-49	40140	300 US	ABANDONED
	P-50	40140	300 US	ABANDONED

SETTLEMENT PLATES				
LINE	NO	STATION	OFFSET	REMARKS
A	SP-1	17140	40 US	SETTLEMENT PLATE
	SP-2	17140	40 US	SETTLEMENT PLATE
B	SP-3	24170	40 US	SETTLEMENT PLATE
	SP-4	24170	40 US	SETTLEMENT PLATE
D	SP-5	47150	40 US	SETTLEMENT PLATE
	SP-6	47150	40 US	SETTLEMENT PLATE

SETTLEMENT PLATES				
LINE	NO	STATION	OFFSET	REMARKS
A	SP-1	17140	40 US	SETTLEMENT PLATE
	SP-2	17140	40 US	SETTLEMENT PLATE
B	SP-3	24170	40 US	SETTLEMENT PLATE
	SP-4	24170	40 US	SETTLEMENT PLATE
D	SP-5	47150	40 US	SETTLEMENT PLATE
	SP-6	47150	40 US	SETTLEMENT PLATE

NOTE: OPEN SYSTEM PIZOMETERS WERE USED FOR WELL MONITORING RATHER THAN POROUS PLASTIC TIP PIZOMETERS.

PNEUMATIC PIEZOMETERS					
LINE NO.	STATION	DEPTH FT	LOCATED	REMARKS	
P-34	47+100	350 DS	WBN	ABANDONED	
P-40	47+100	40 DS	WBN	ABANDONED	
P-41	47+100	20 DS	WBN	ABANDONED	
P-42	47+100	40 DS	WBN	ABANDONED	
P-43	47+100	25 DS	WBN	ABANDONED	
P-44	47+100	40 DS	WBN	ABANDONED	
P-45	47+100	40 DS	WBN	ABANDONED	
P-46	47+100	350 DS	WBN	ABANDONED	
P-47	47+100	350 DS	WBN	ABANDONED	
P-48	47+100	350 DS	WBN	ABANDONED	
P-49	47+100	350 DS	WBN	ABANDONED	
P-50	47+100	350 DS	WBN	ABANDONED	
P-51	47+100	350 DS	WBN	ABANDONED	
P-52	47+100	350 DS	WBN	ABANDONED	
P-53	47+100	350 DS	WBN	ABANDONED	
P-54	47+100	350 DS	WBN	ABANDONED	
P-55	47+100	350 DS	WBN	ABANDONED	
P-56	47+100	350 DS	WBN	ABANDONED	
P-57	47+100	350 DS	WBN	ABANDONED	
P-58	47+100	350 DS	WBN	ABANDONED	
P-59	47+100	350 DS	WBN	ABANDONED	
P-60	47+100	350 DS	WBN	ABANDONED	
P-61	47+100	350 DS	WBN	ABANDONED	
P-62	47+100	350 DS	WBN	ABANDONED	
P-63	47+100	350 DS	WBN	ABANDONED	
P-64	47+100	350 DS	WBN	ABANDONED	
P-65	47+100	350 DS	WBN	ABANDONED	
P-66	47+100	350 DS	WBN	ABANDONED	
P-67	47+100	350 DS	WBN	ABANDONED	
P-68	47+100	350 DS	WBN	ABANDONED	
P-69	47+100	350 DS	WBN	ABANDONED	
P-70	47+100	350 DS	WBN	ABANDONED	
P-71	47+100	350 DS	WBN	ABANDONED	
P-72	47+100	350 DS	WBN	ABANDONED	
P-73	47+100	350 DS	WBN	ABANDONED	
P-74	47+100	350 DS	WBN	ABANDONED	
P-75	47+100	350 DS	WBN	ABANDONED	
P-76	47+100	350 DS	WBN	ABANDONED	
P-77	47+100	350 DS	WBN	ABANDONED	
P-78	47+100	350 DS	WBN	ABANDONED	
P-79	47+100	350 DS	WBN	ABANDONED	
P-80	47+100	350 DS	WBN	ABANDONED	
P-81	47+100	350 DS	WBN	ABANDONED	
P-82	47+100	350 DS	WBN	ABANDONED	
P-83	47+100	350 DS	WBN	ABANDONED	
P-84	47+100	350 DS	WBN	ABANDONED	
P-85	47+100	350 DS	WBN	ABANDONED	
P-86	47+100	350 DS	WBN	ABANDONED	
P-87	47+100	350 DS	WBN	ABANDONED	
P-88	47+100	350 DS	WBN	ABANDONED	
P-89	47+100	350 DS	WBN	ABANDONED	
P-90	47+100	350 DS	WBN	ABANDONED	
P-91	47+100	350 DS	WBN	ABANDONED	
P-92	47+100	350 DS	WBN	ABANDONED	
P-93	47+100	350 DS	WBN	ABANDONED	
P-94	47+100	350 DS	WBN	ABANDONED	
P-95	47+100	350 DS	WBN	ABANDONED	
P-96	47+100	350 DS	WBN	ABANDONED	
P-97	47+100	350 DS	WBN	ABANDONED	
P-98	47+100	350 DS	WBN	ABANDONED	
P-99	47+100	350 DS	WBN	ABANDONED	
P-100	47+100	350 DS	WBN	ABANDONED	

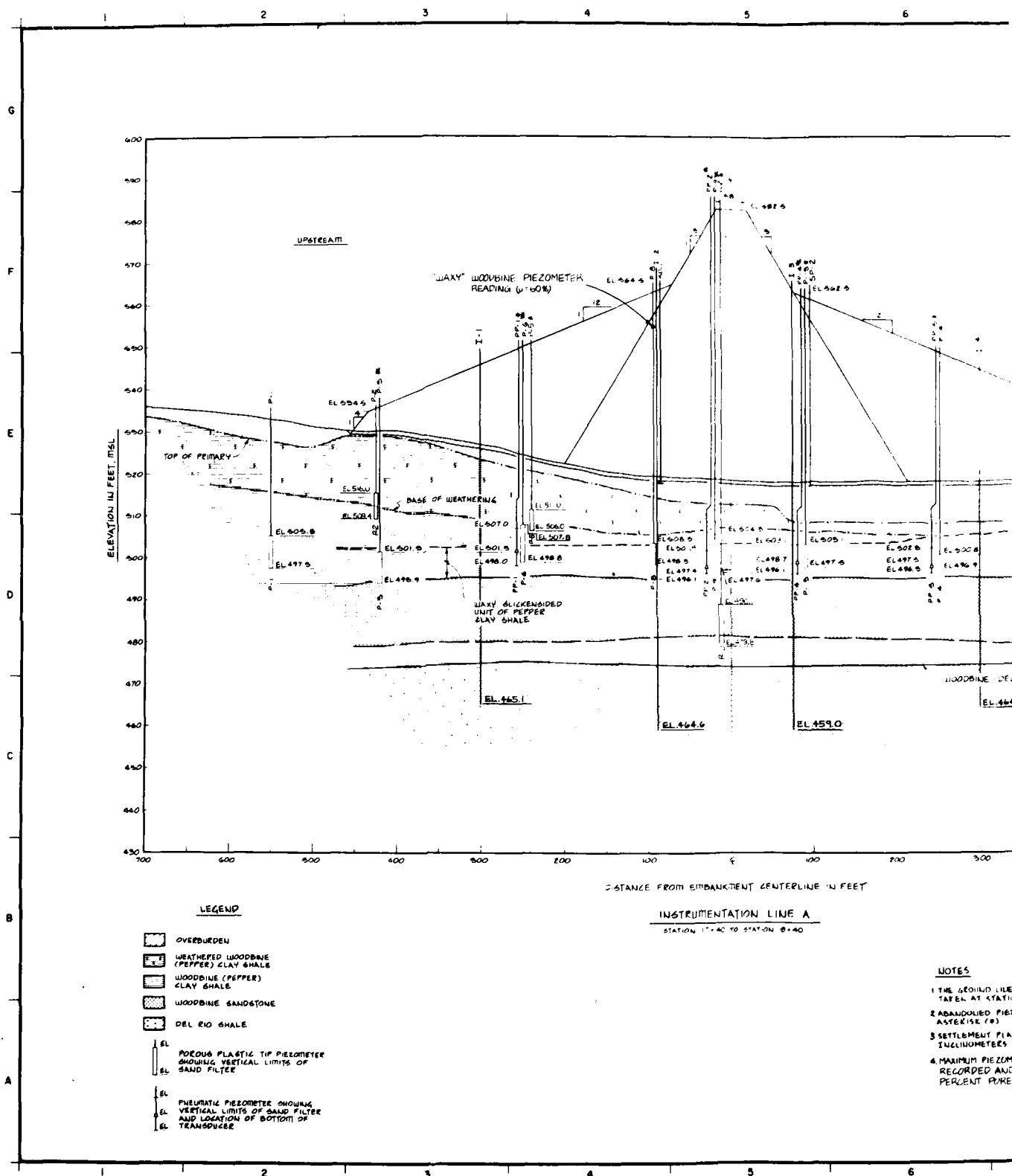
NOTE: PNEUMATIC PIEZOMETERS PAT-RO DE HAVE METAL
CELL POINTS RATHER THAN POROUS PLASTIC TIPS

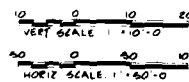
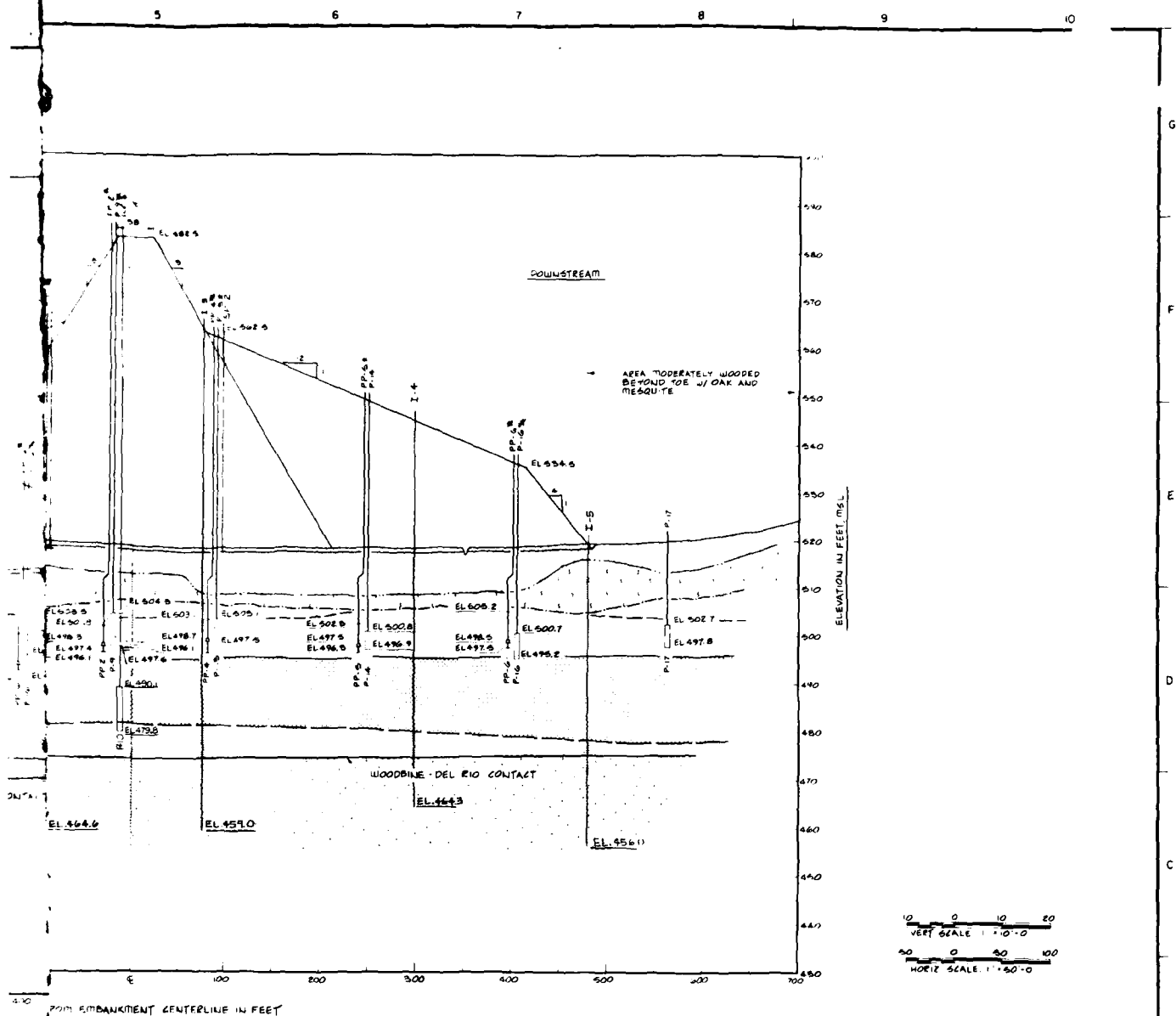
PNEUMATIC PIEZOMETERS					
LINE NO.	STATION	DEPTH FT	LOCATED	REMARKS	
P-1	47+100	350 DS	WBN	ABANDONED	
P-2	47+100	350 DS	WBN	ABANDONED	
P-3	47+100	350 DS	WBN	ABANDONED	
P-4	47+100	350 DS	WBN	ABANDONED	
P-5	47+100	350 DS	WBN	ABANDONED	
P-6	47+100	350 DS	WBN	ABANDONED	
P-7	47+100	350 DS	WBN	ABANDONED	
P-8	47+100	350 DS	WBN	ABANDONED	
P-9	47+100	350 DS	WBN	ABANDONED	
P-10	47+100	350 DS	WBN	ABANDONED	
P-11	47+100	350 DS	WBN	ABANDONED	
P-12	47+100	350 DS	WBN	ABANDONED	
P-13	47+100	350 DS	WBN	ABANDONED	
P-14	47+100	350 DS	WBN	ABANDONED	
P-15	47+100	350 DS	WBN	ABANDONED	
P-16	47+100	350 DS	WBN	ABANDONED	
P-17	47+100	350 DS	WBN	ABANDONED	
P-18	47+100	350 DS	WBN	ABANDONED	
P-19	47+100	350 DS	WBN	ABANDONED	
P-20	47+100	350 DS	WBN	ABANDONED	
P-21	47+100	350 DS	WBN	ABANDONED	
P-22	47+100	350 DS	WBN	ABANDONED	
P-23	47+100	350 DS	WBN	ABANDONED	
P-24	47+100	350 DS	WBN	ABANDONED	
P-25	47+100	350 DS	WBN	ABANDONED	
P-26	47+100	350 DS	WBN	ABANDONED	

INCLINOMETERS					
LINE NO.	STATION	DEPTH FT	LOCATED	REMARKS	
I-1	47+100	350 DS	WBN	ABANDONED	
I-2	47+100	350 DS	WBN	ABANDONED	
I-3	47+100	350 DS	WBN	ABANDONED	
I-4	47+100	350 DS	WBN	ABANDONED	
I-5	47+100	350 DS	WBN	ABANDONED	
I-6	47+100	350 DS	WBN	ABANDONED	
I-7	47+100	350 DS	WBN	ABANDONED	
I-8	47+100	350 DS	WBN	ABANDONED	
I-9	47+100	350 DS	WBN	ABANDONED	
I-10	47+100	350 DS	WBN	ABANDONED	
I-11	47+100	350 DS	WBN	ABANDONED	
I-12	47+100	350 DS	WBN	ABANDONED	
I-13	47+100	350 DS	WBN	ABANDONED	
I-14	47+100	350 DS	WBN	ABANDONED	
I-15	47+100	350 DS	WBN	ABANDONED	
I-16	47+100	350 DS	WBN	ABANDONED	

NOTE:
WBN - WOODBINE FORMATION
DE - DEL RIO FORMATION
WBN-DE CONTACT BETWEEN WOODBINE
AND DEL RIO FORMATION
OVB - OVERBURDEN
SD - SAND

U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS	
DESIGNED BY P. J. HARRIS	AGUILLA LAKE AGUILLA CREEK, TEXAS
DRAWN BY E. J. HARRIS	SCHEDULE OF INSTRUMENTATION
REVIEWED BY T. J. HARRIS	
SUBMITTED BY E. J. HARRIS	DATE
ENGINEER	DATE
INSTRUMENT NO.	SHEET NO.
SEQUENCE NO.	OF





70M EMBANKMENT CENTERLINE IN FEET

INSTRUMENTATION LINE A

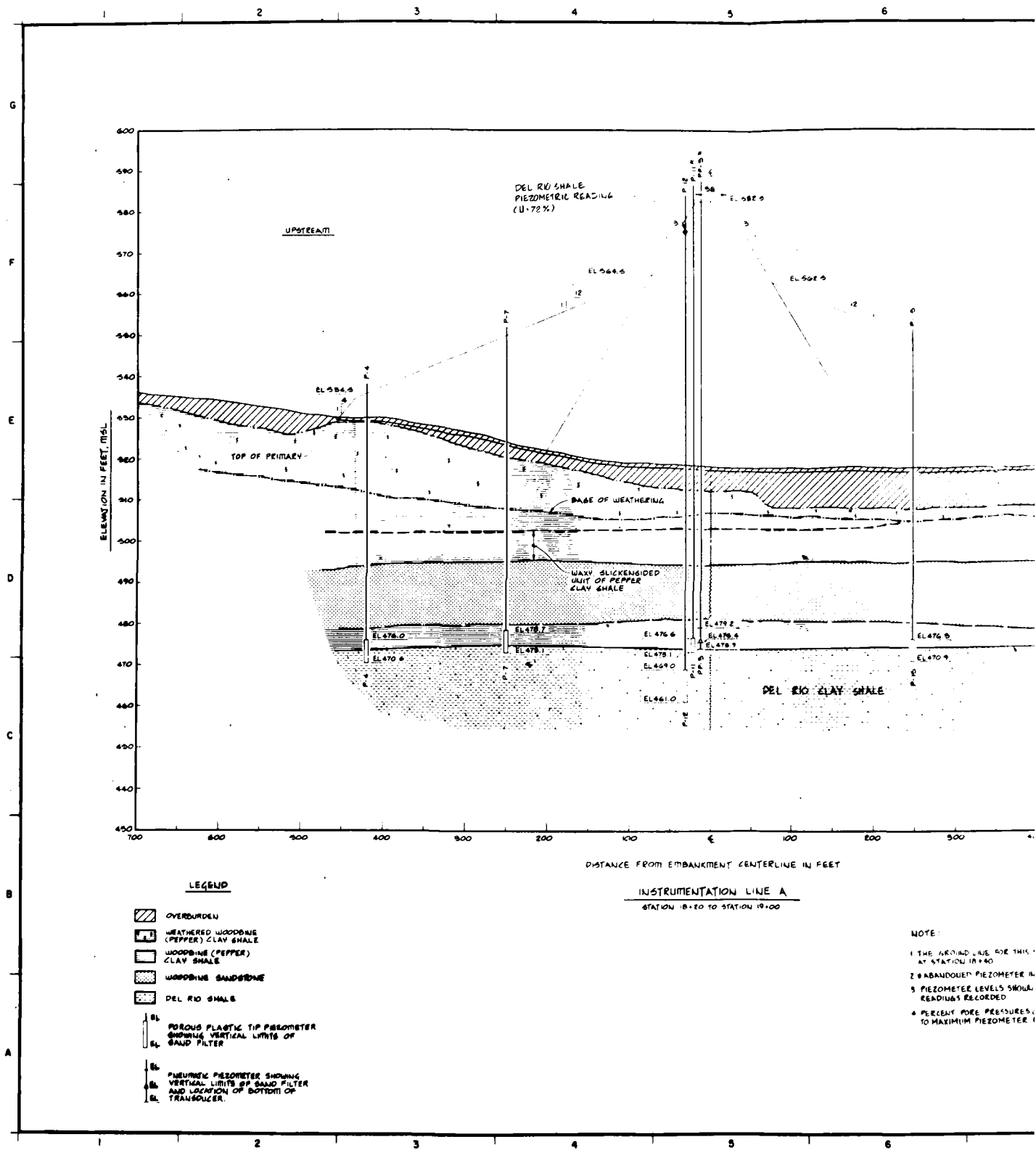
FROM 17+40 TO STATION 0+40

NOTES:

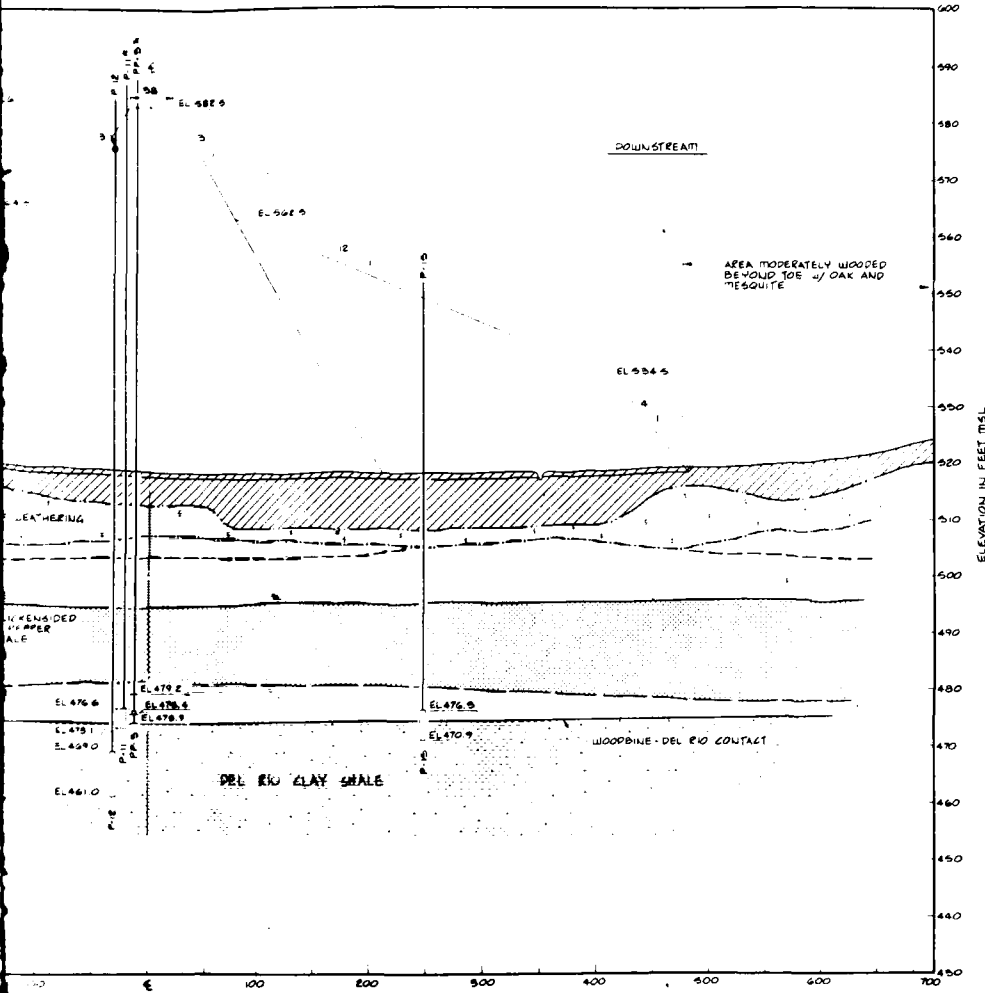
1. THE GROUND LINE FOR THIS SECTION WAS TAKEN AT STATION 19+40
2. ABANDONED PIEZOMETERS INDICATED BY ASTERISK (*)
3. SETTLEMENT PLATES 1, 2 ARE AT 17+40. INCLINOMETERS 1, 2, 3, 4, 5 ARE AT 17+80
4. MAXIMUM PIEZOMETRIC ELEVATION RECORDED AND CORRESPONDING PERCENT PORE PRESSURE (U = %)

U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS	
DESIGNED BY W. H. WILSON & B. H. WILSON	BRAXTON CREEK DAM, TEXAS AGUILA CREEK, TEXAS
DRAWN BY E. L. HILL	INITIAL EMBANKMENT INSTRUMENTATION LINE A
CHECKED BY W. H. WILSON	
DATE 1954	
DRAWING NUMBER H. E. KARBS	SHEET NO. 32

PLATE 30



5 6 7 8 9 10



ELEVATION IN FEET MSL

VERT SCALE 1" = 10' 0"
HORIZ SCALE 1" = 50' 0"

FEET FROM EMBANKMENT CENTERLINE IN FEET

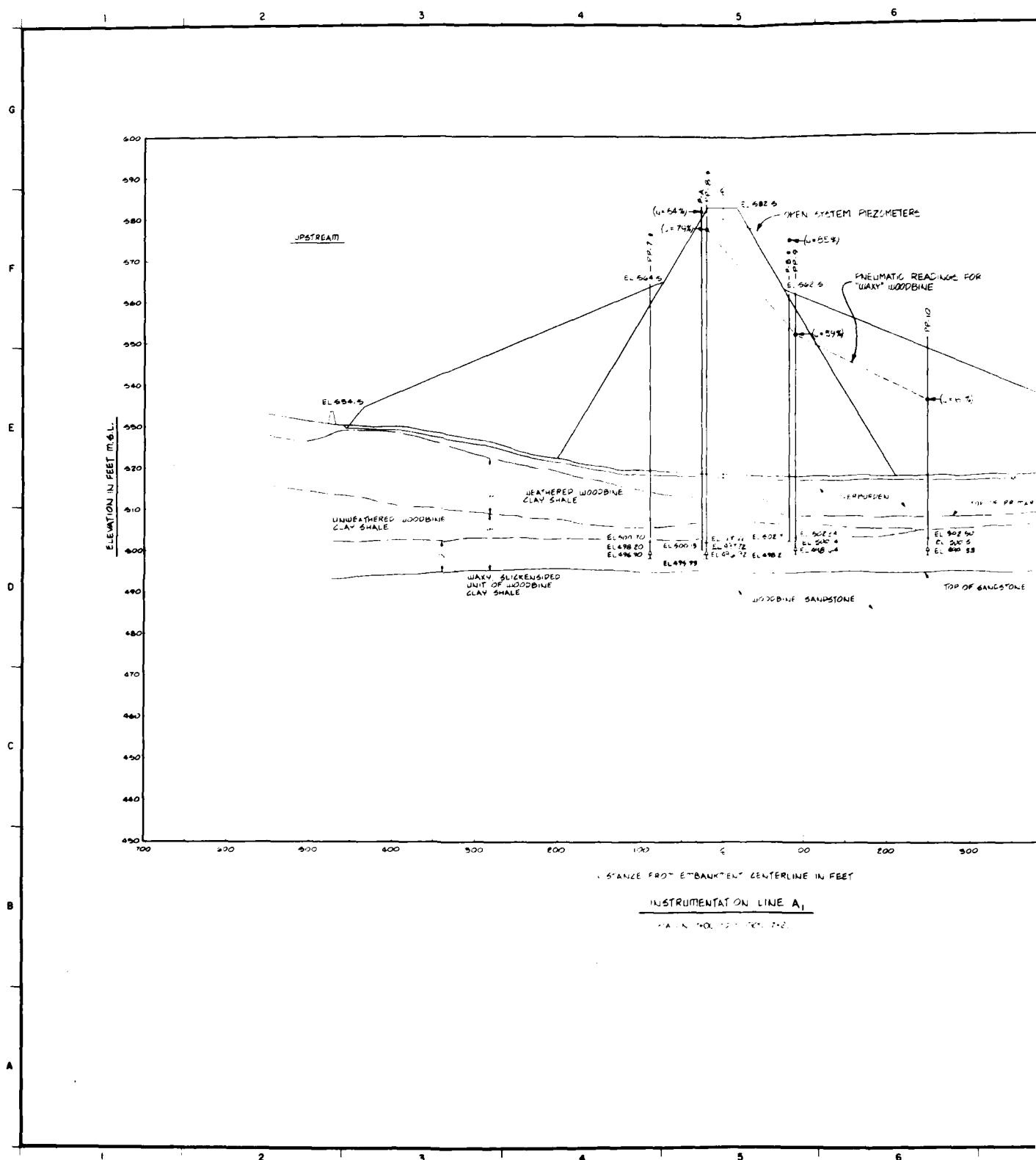
INSTRUMENTATION LINE A
STATION 8+20 TO STATION 19+00

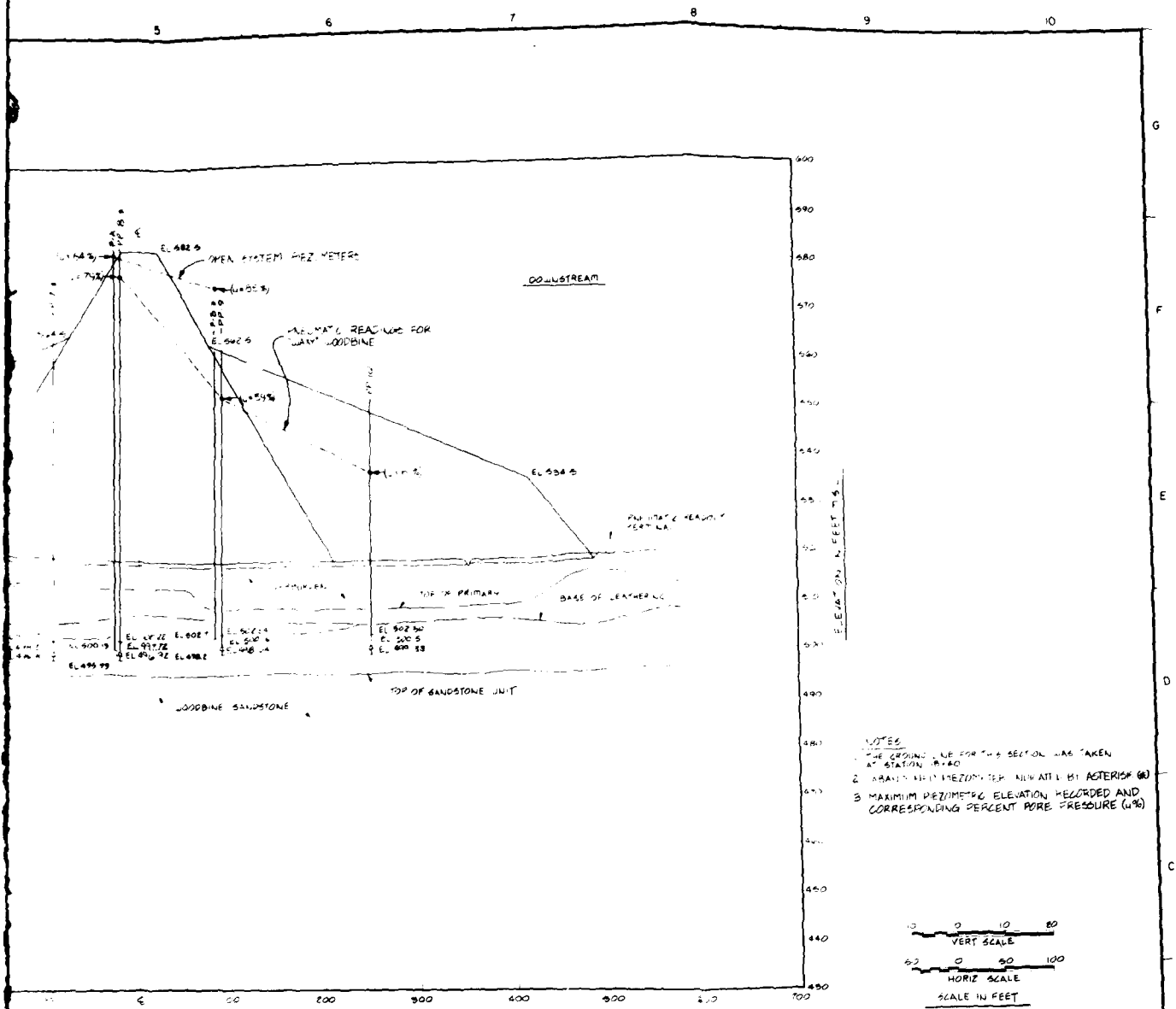
NOTE:
1 THE GROUND LINE FOR THIS SECTION WAS TAKEN AT STATION 18+40
2 *ABANDONED PIEZOMETER INDICATED BY ASTERISK (*)
3 PIEZOMETER LEVELS SHOWN ARE FOR THE MAXIMUM READINGS RECORDED
4 PERCENT PORE PRESSURES (U%) SHOWN CORRESPOND TO MAXIMUM PIEZOMETER READING.

U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS			
DESIGNED BY W. WICKS A. REAGAN	BRAZOS RIVER BASIN, TEXAS AQUILA LAKE AQUILA CREEK, TEXAS		
DRAWN BY H. L. HILL	INITIAL EMBANKMENT PIEZOMETER SECTION		
CHECKED BY T. ALMENDY H. DELUETT	LINE A		
H. E. KARBS		REV NO. DATED DRAWING NUMBER	SEQUENCE NO. SHEET NO. 21

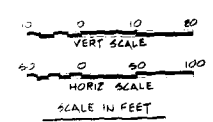
PLATE 31

5 6 7 8





- NOTES
1. THE GROUND LINE FOR THIS SECTION WAS TAKEN AT STATION 10+00.
 2. MAXIMUM PIZOMETER ELEVATION RECORDED BY ASTERISK (*)
 3. MAXIMUM PIZOMETER ELEVATION RECORDED AND CORRESPONDING PERCENT PORE PRESSURE (U%)



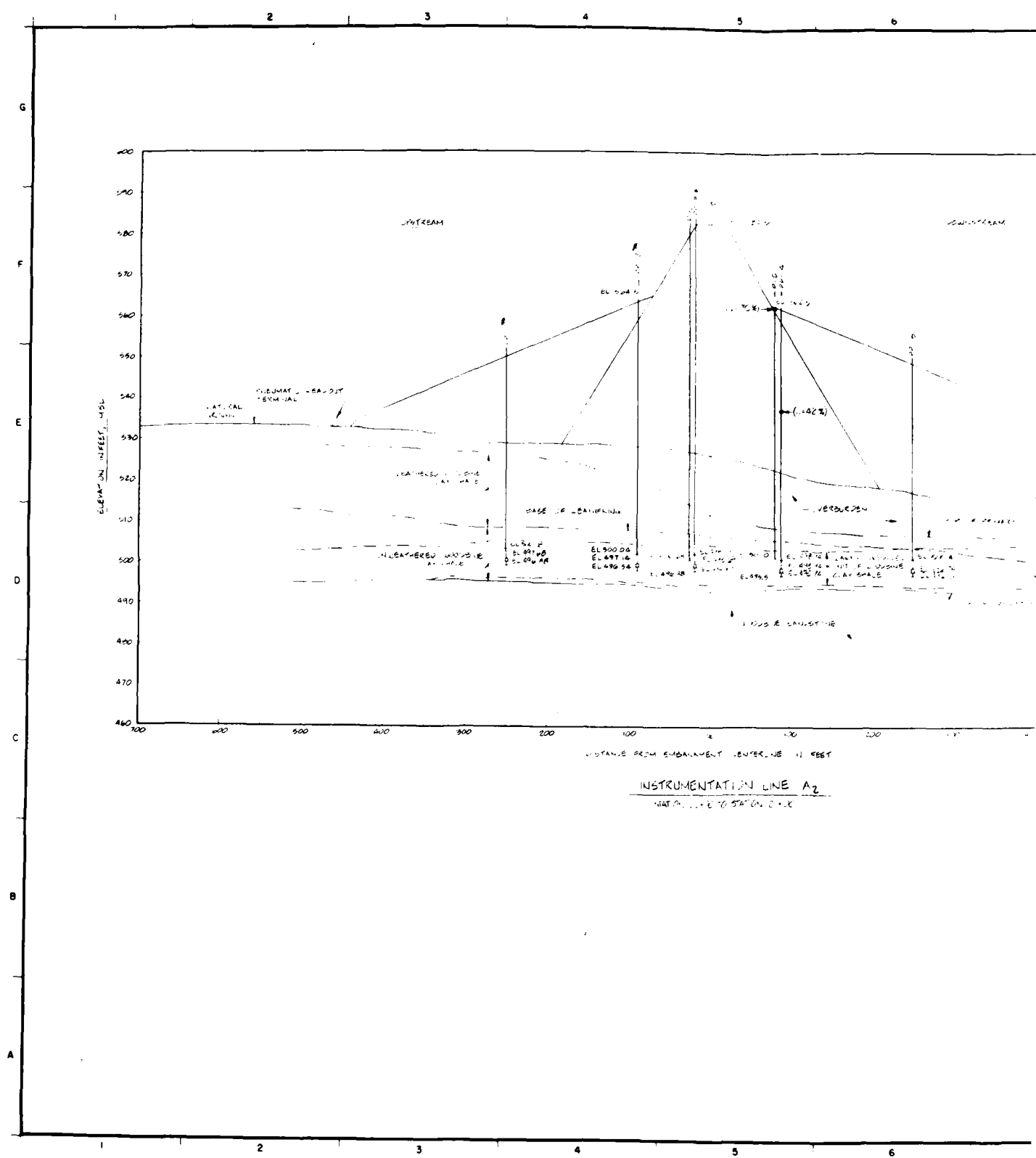
INSTRUMENTATION LINE A1

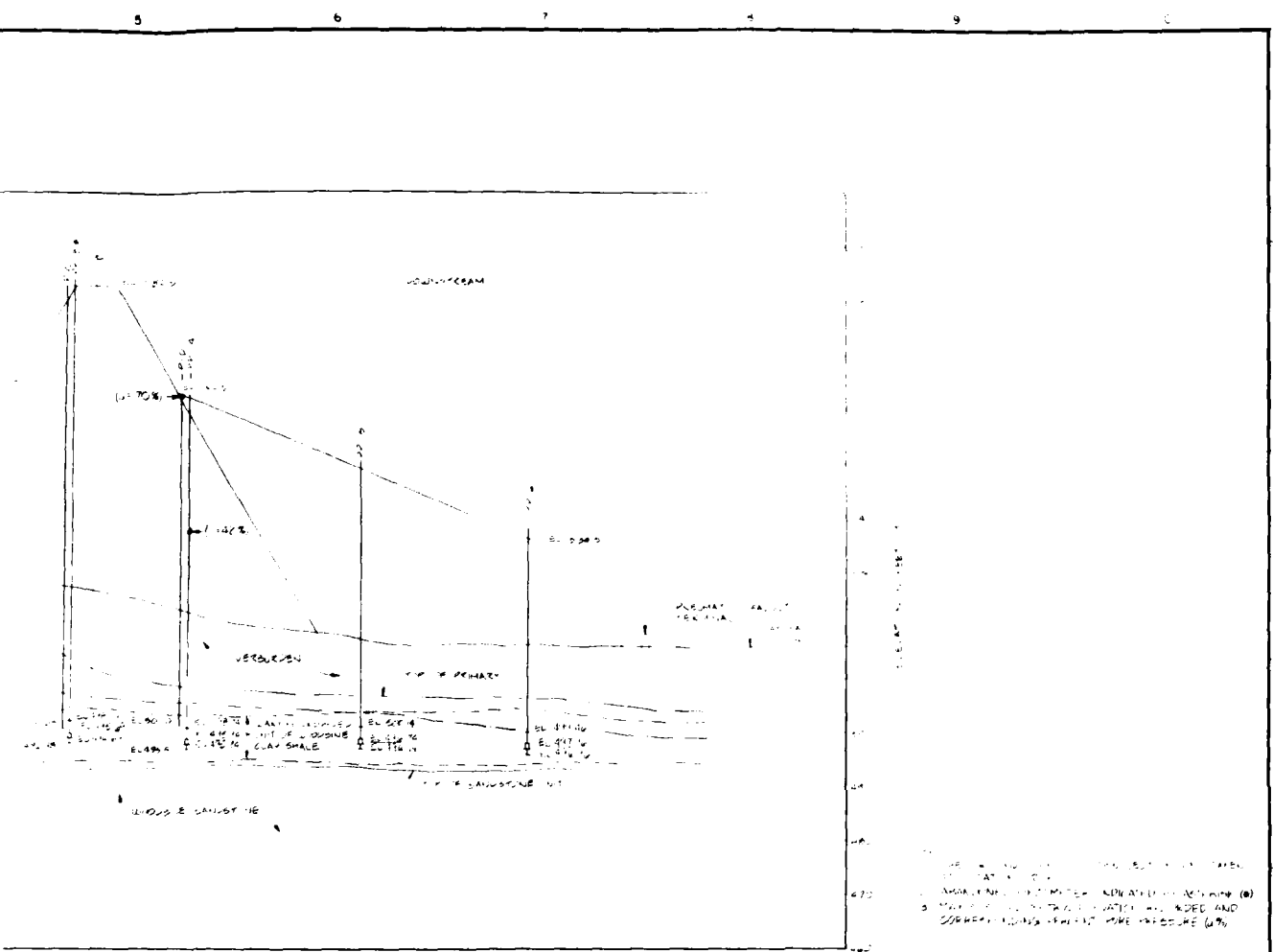
1000' EMBANKMENT CENTERLINE IN FEET

TOP OF FILTER ELEVATION
 TRANSPIER ELEVATION
 BOTTOM OF FILTER ELEVATION
 PNEUMATIC PIZOMETER

U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS			
DESIGNED BY W. H. KIRBY	APPROVED BY A. C. LAYNE ADJUTANT GENERAL FORT WORTH, TEXAS		
DRAWN BY R. H. KIRBY	CHECKED BY J. H. KIRBY		
DATE 10/1/50	PROJECT NO. 1000' EMBANKMENT CENTERLINE IN FEET		
DRAWING NUMBER 1000' EMBANKMENT CENTERLINE IN FEET	SHEET NO. 1000' EMBANKMENT CENTERLINE IN FEET	PLATE NO. 1000' EMBANKMENT CENTERLINE IN FEET	

PLATE 32





DOCUMENTATION LINE A2

1:1 TO 5:1

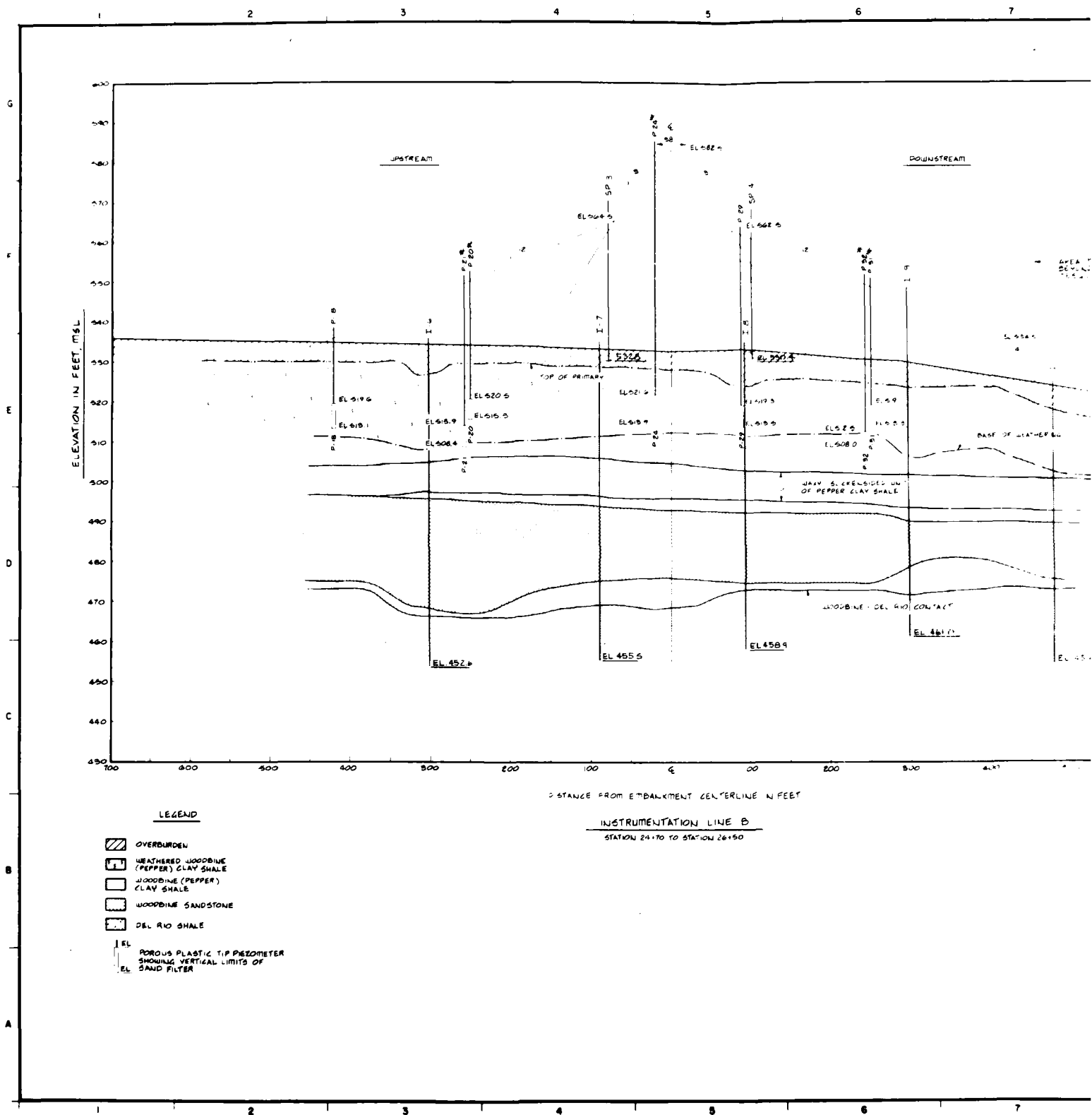
TOP FILTER EL.
TRANSDUCER EL.
BOTTOM OF FILTER EL.

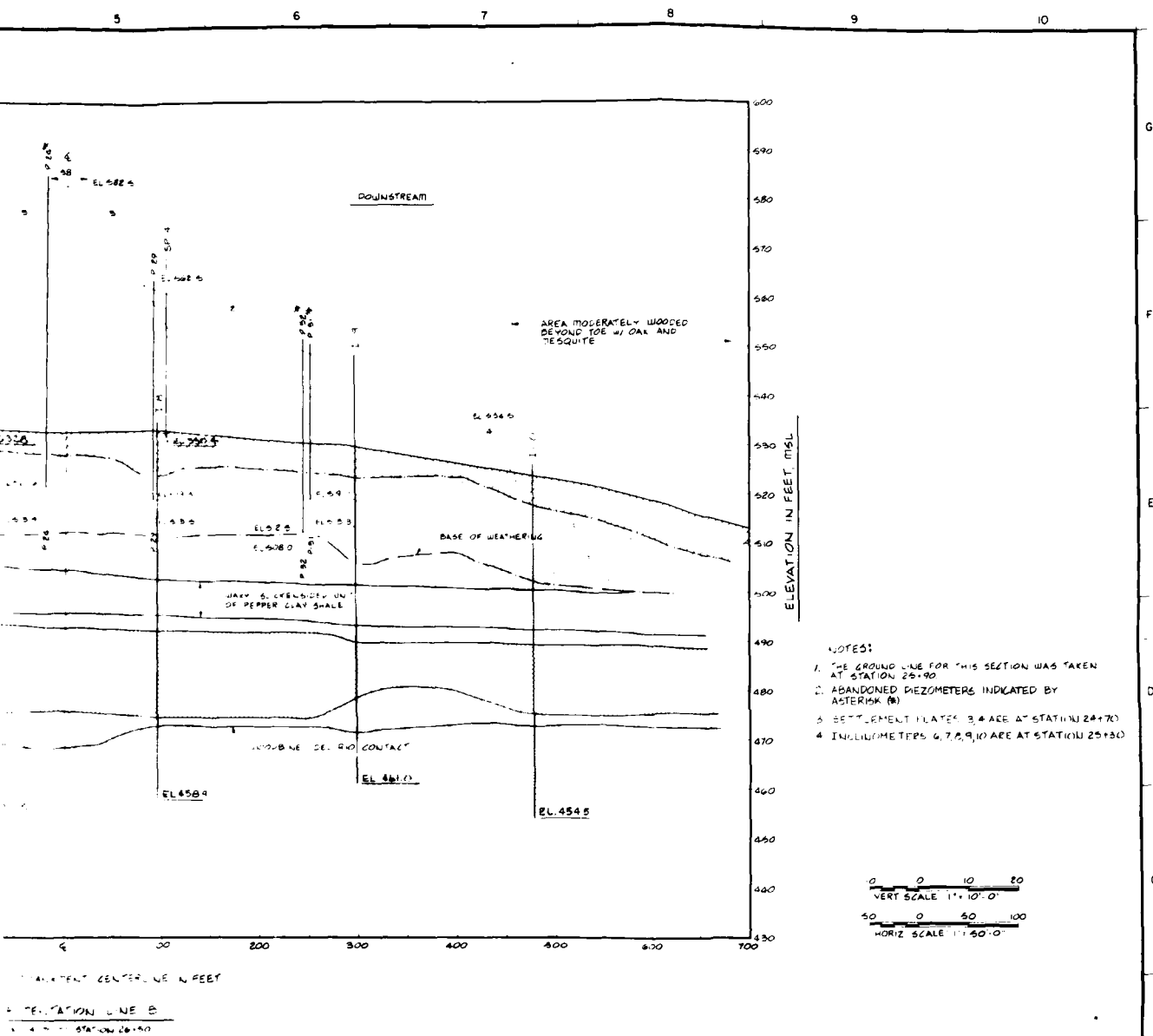
PNEUMATIC
PIEZOMETER

VERT. SCALE 1" = 10'
HORIZ. SCALE 1" = 100'

DESIGNED BY A. J. K. K. K.		DRAWN BY A. J. K. K. K.		CHECKED BY A. J. K. K. K.	
DATE 10/1/50		SHEET NO. 3 OF 4		SEQUENCE NO. 3	
U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS INITIAL EMBANKMENT PIEZOMETER SECTION D. H. K.					

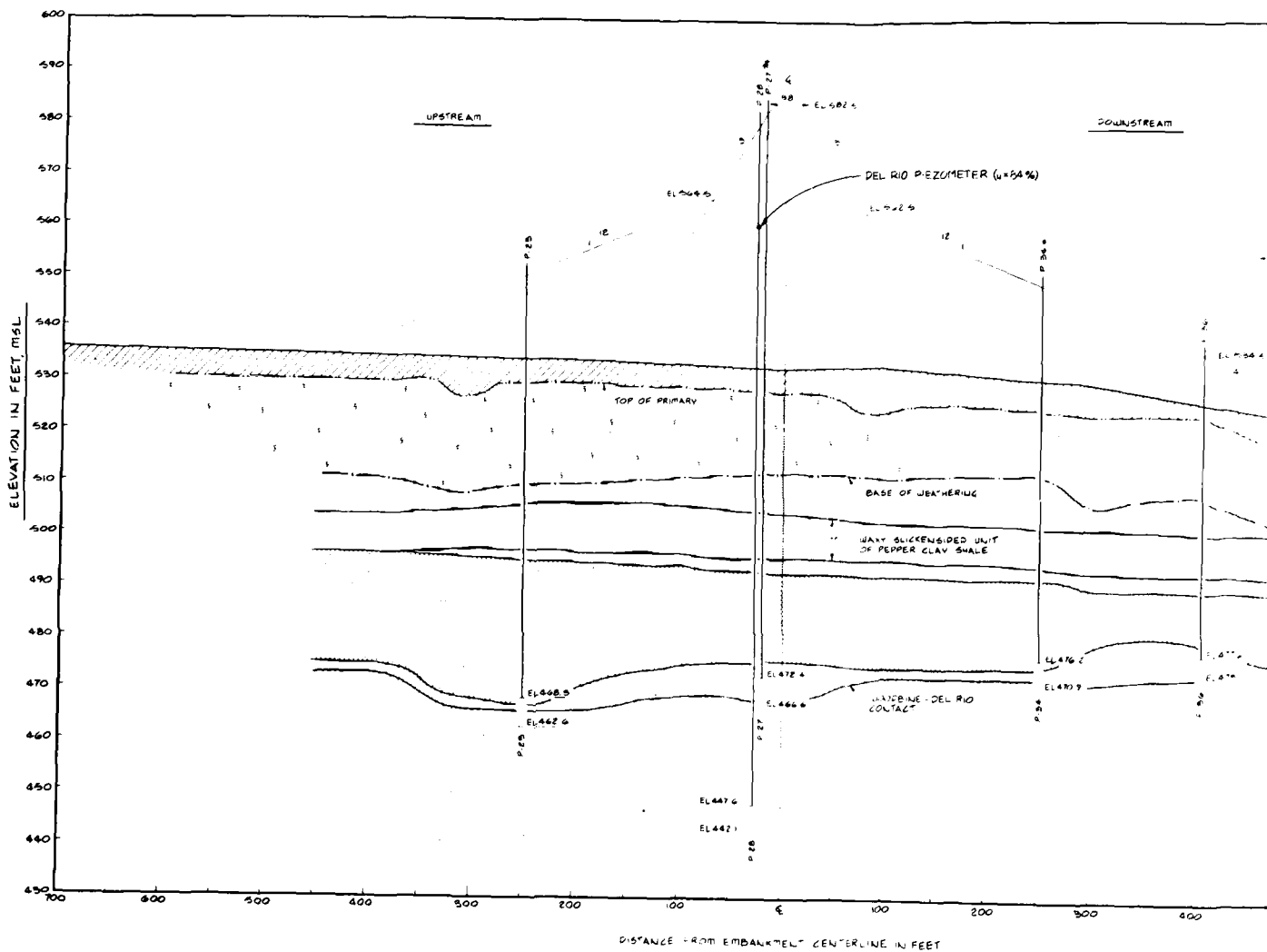
PLATE 33





U.S. ARM ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS	
DESIGNED BY W. L. WILSON A. BEAUCH	PRATON RIVER BASIN, TEXAS AGUILA LAKE AGUILA CREEK, TEXAS INITIAL EMBANKMENT
DRAWN BY R. W. WILSON	INSTRUMENTATION LINE B
CHECKED BY W. L. WILSON F. SCHMIDT	
DATE 10/1/50	PROJECT NO. 1000
DRAWING NUMBER 1000	SHEET NO. OF
W. E. KAPBS	

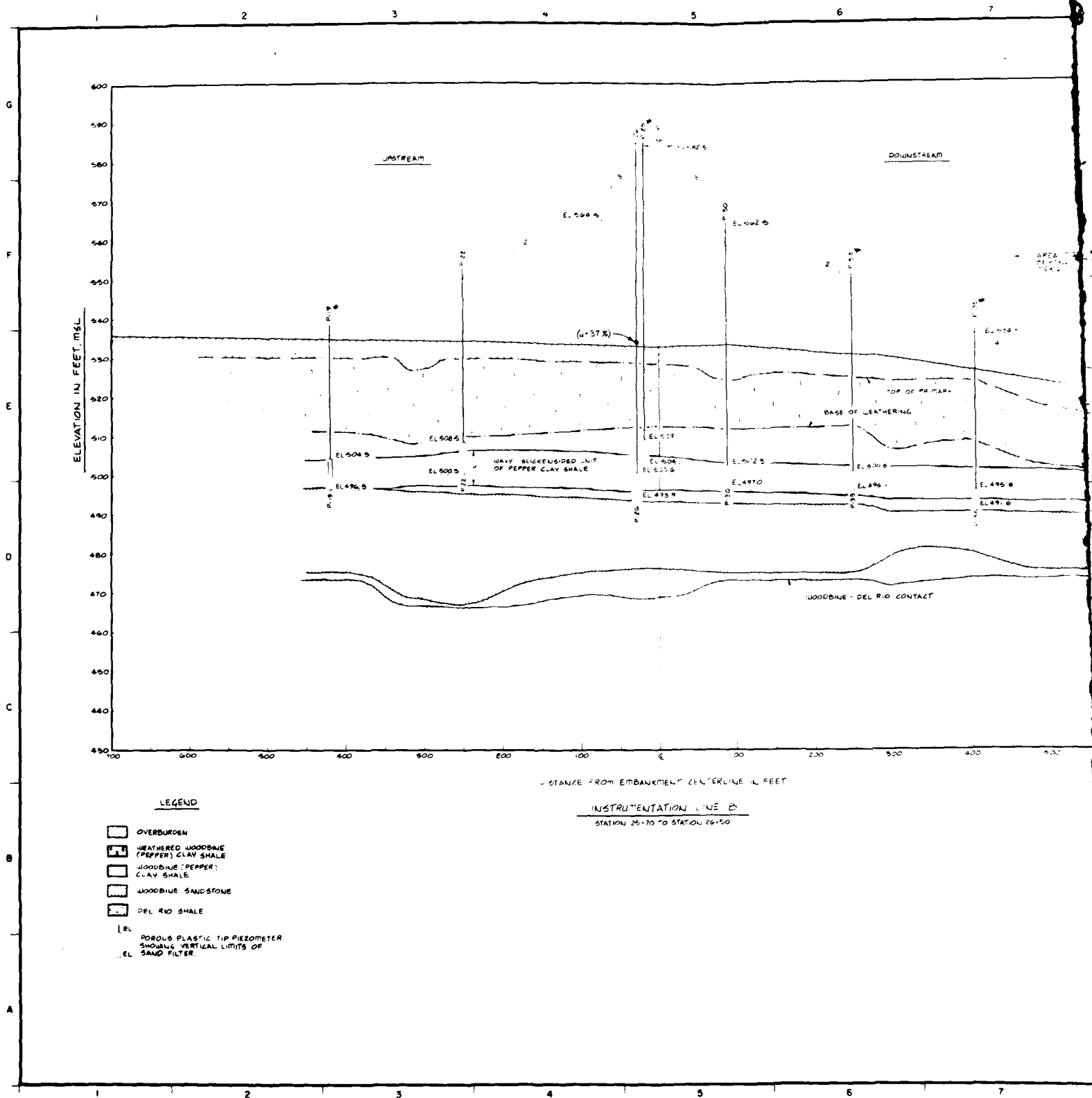
PLATE 34

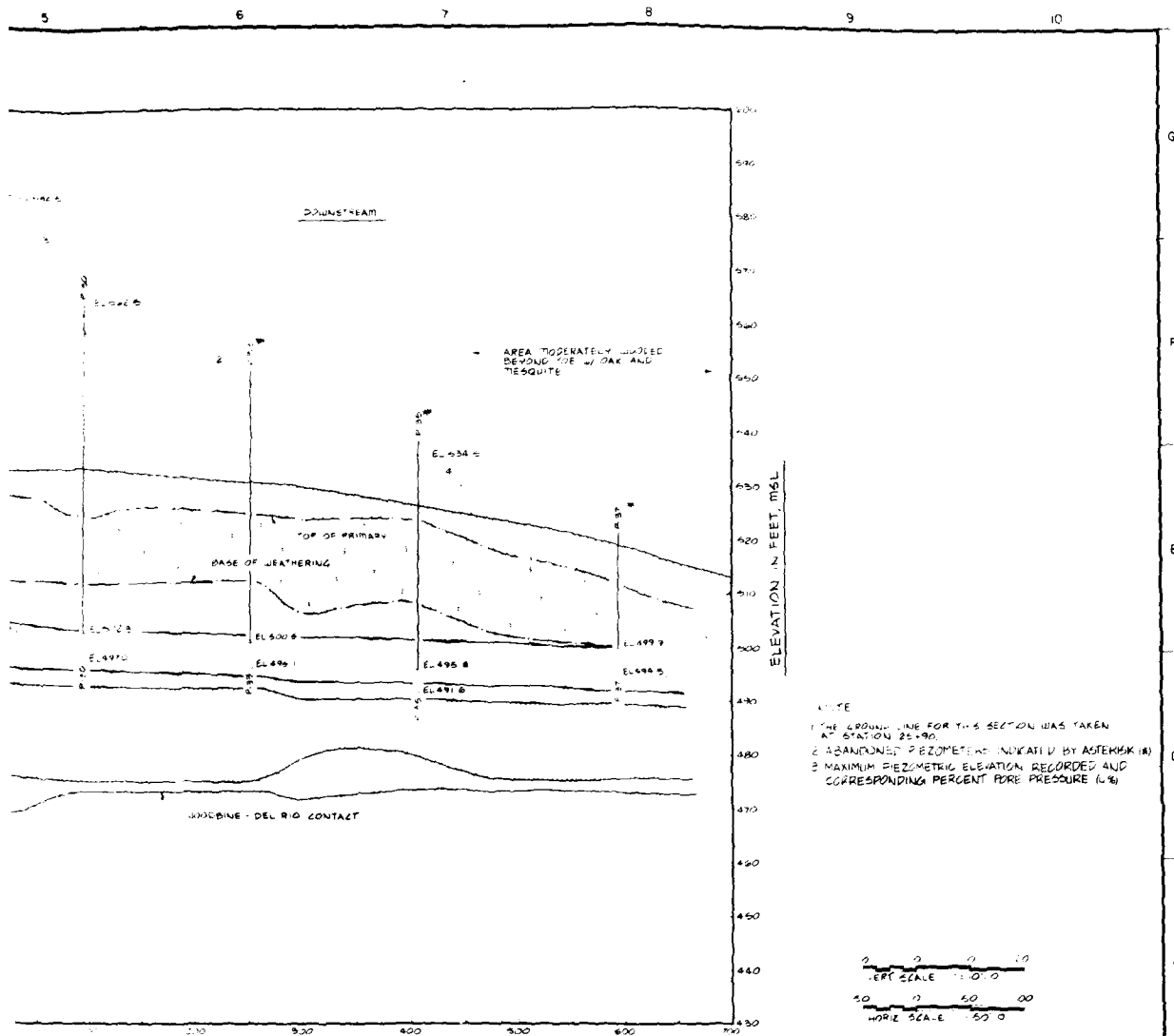


LEGEND

- OVERBURDEN
- WEATHERED WOODBINE (PEPPER) CLAY SHALE
- WOODBINE (PEPPER) CLAY SHALE
- WOODBINE SANDSTONE
- DEL RIO SHALE
- POROUS PLASTIC TIP PEZOMETER SHOWING VERTICAL LIMITS OF SAND FILTER

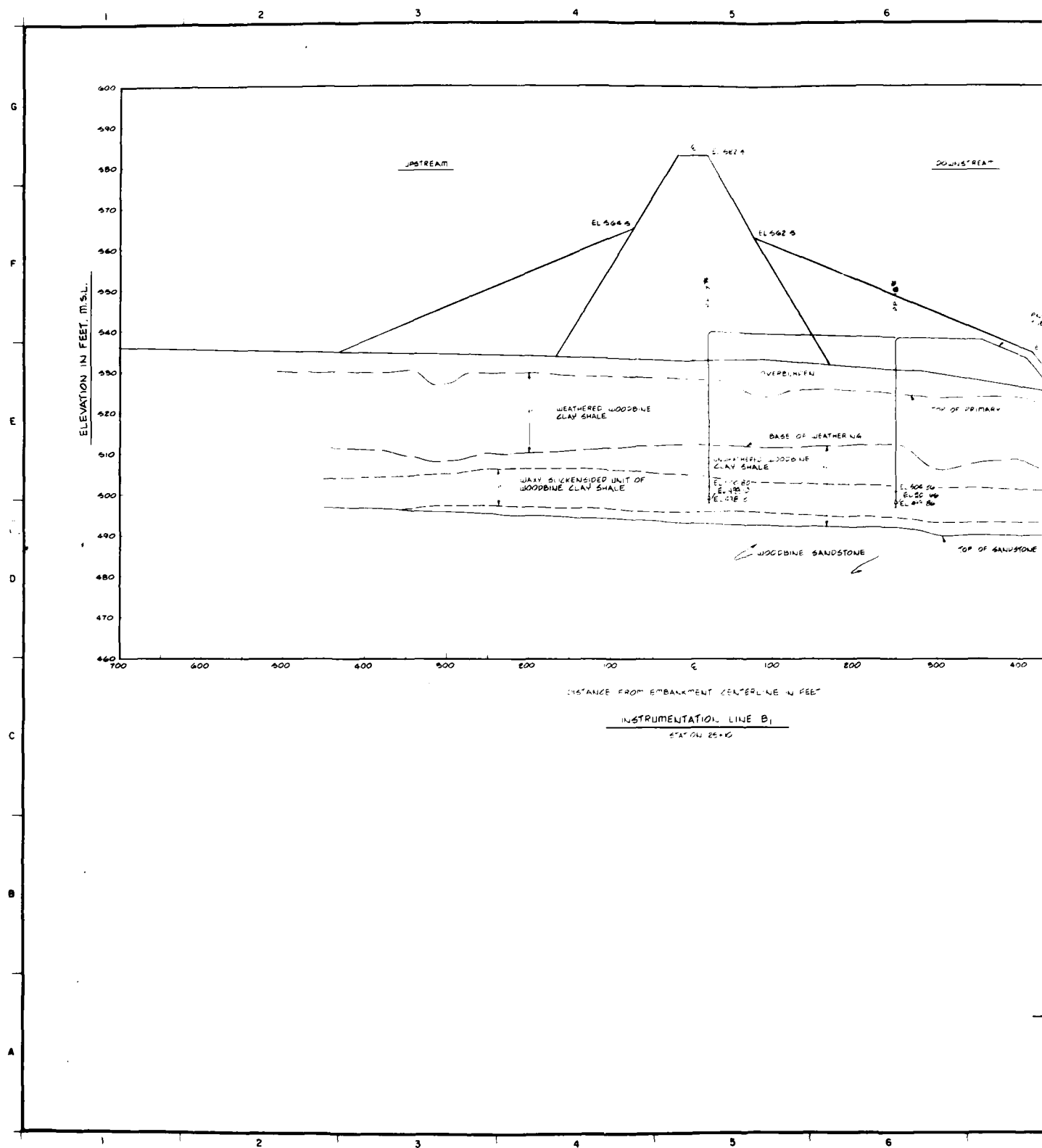
INSTRUMENTATION - NE B
STATION 25+70 TO STATION 26+50

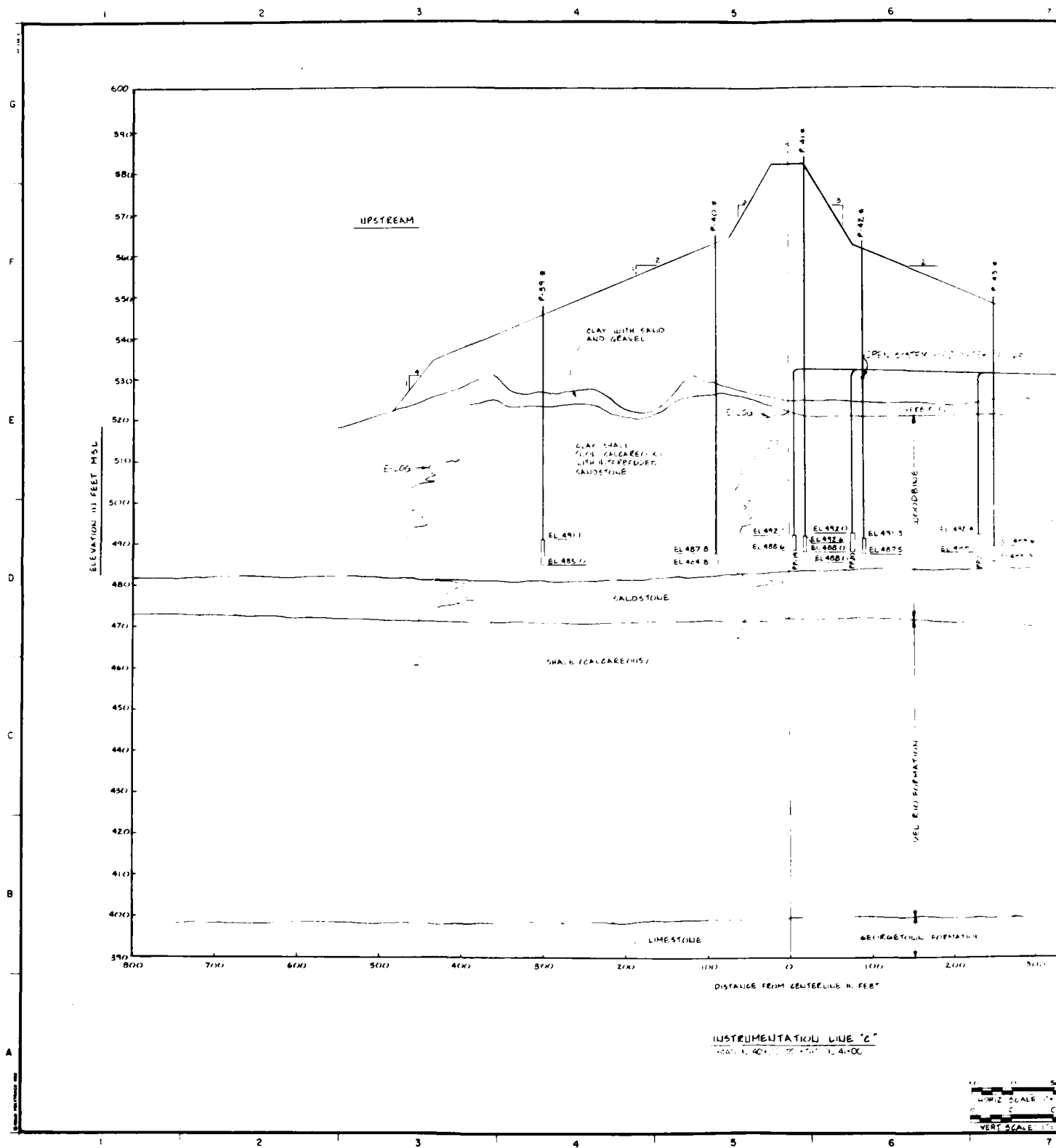


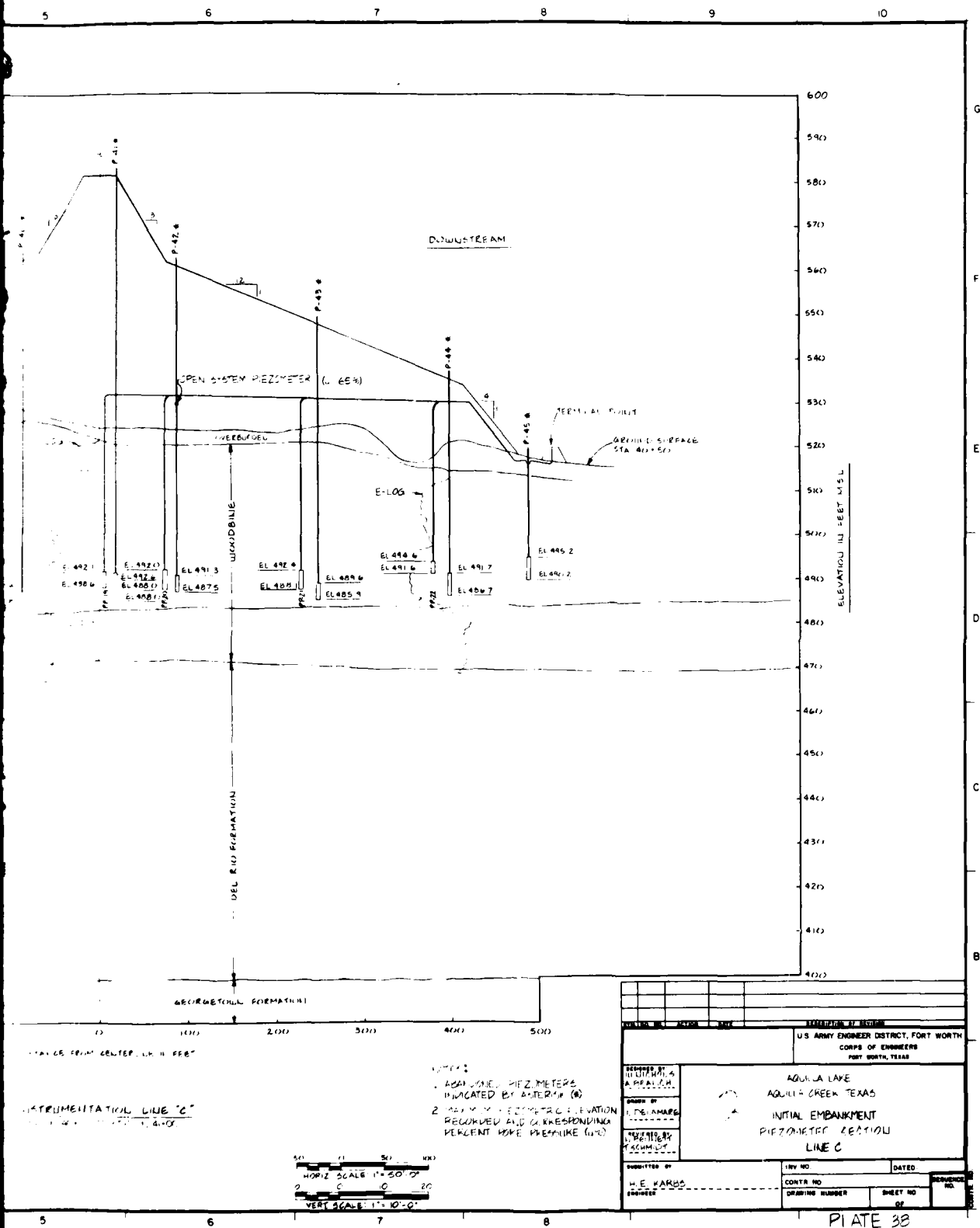


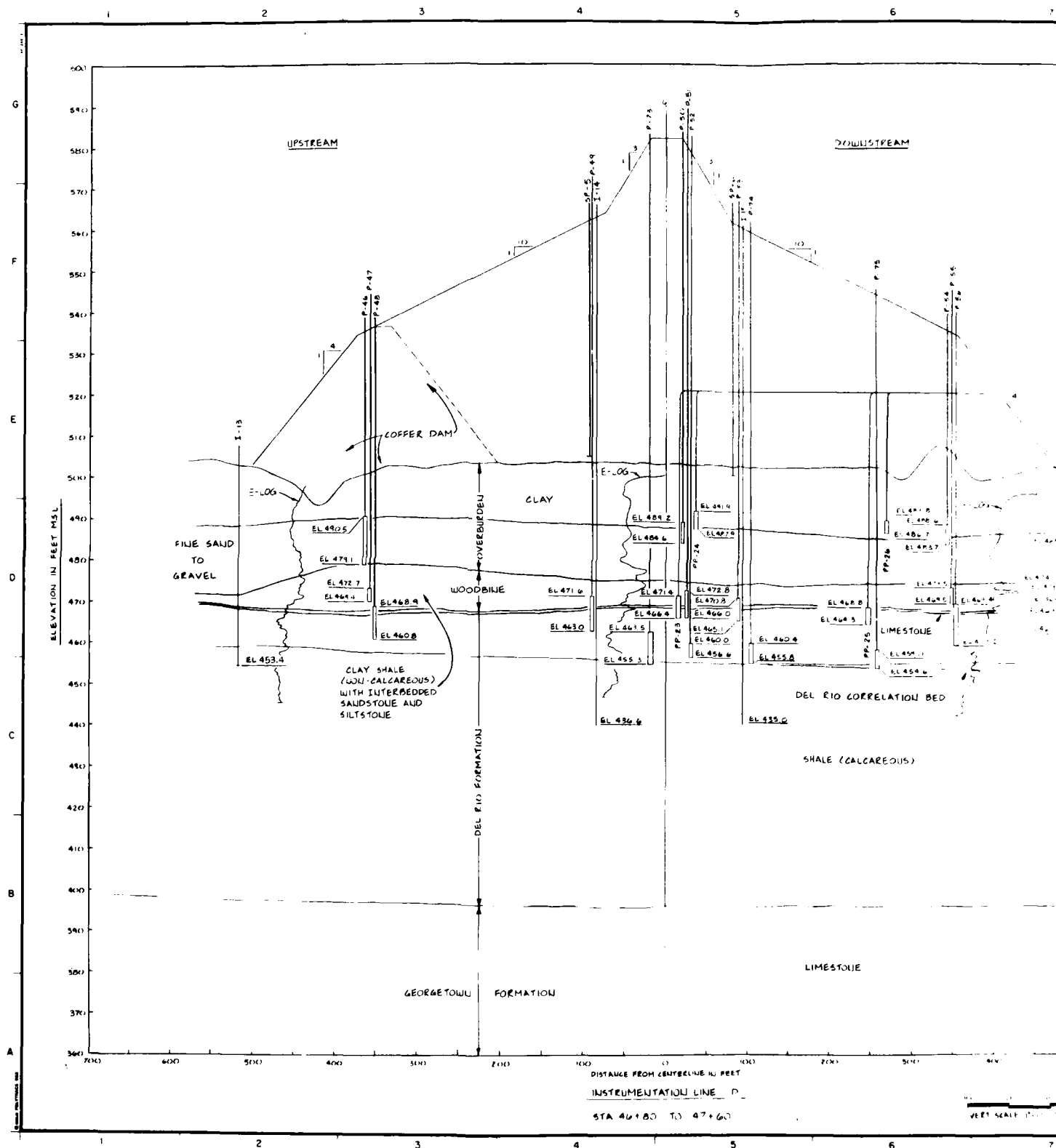
U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS			
DESIGNED BY W. L. WILKINS A. BRANCH	PROJECT: ALCOA DAM, TEXAS ALCOA LAKE ALCOA CREEK, TEXAS		
DRAWN BY R. L. WILKINS	INITIAL EMBANKMENT PIEZOMETER SECTION		
CHECKED BY L. BRUNNEN A. SCHMIDT	LITTLE P.		
H.E. KARBS	DATE DRAWING NUMBER	SHEET NO. 22	REVISIONS NO.

PLATE 36









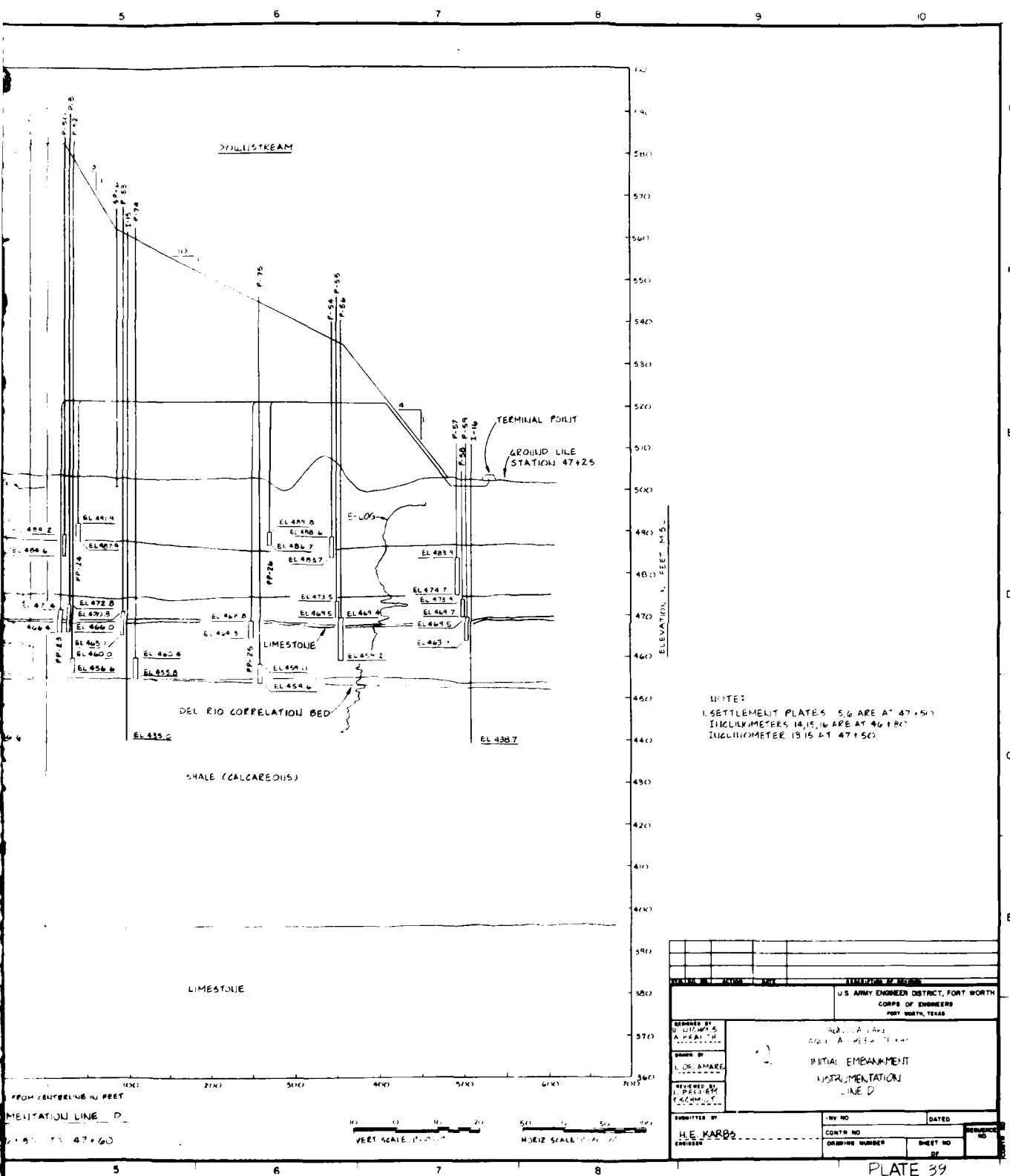
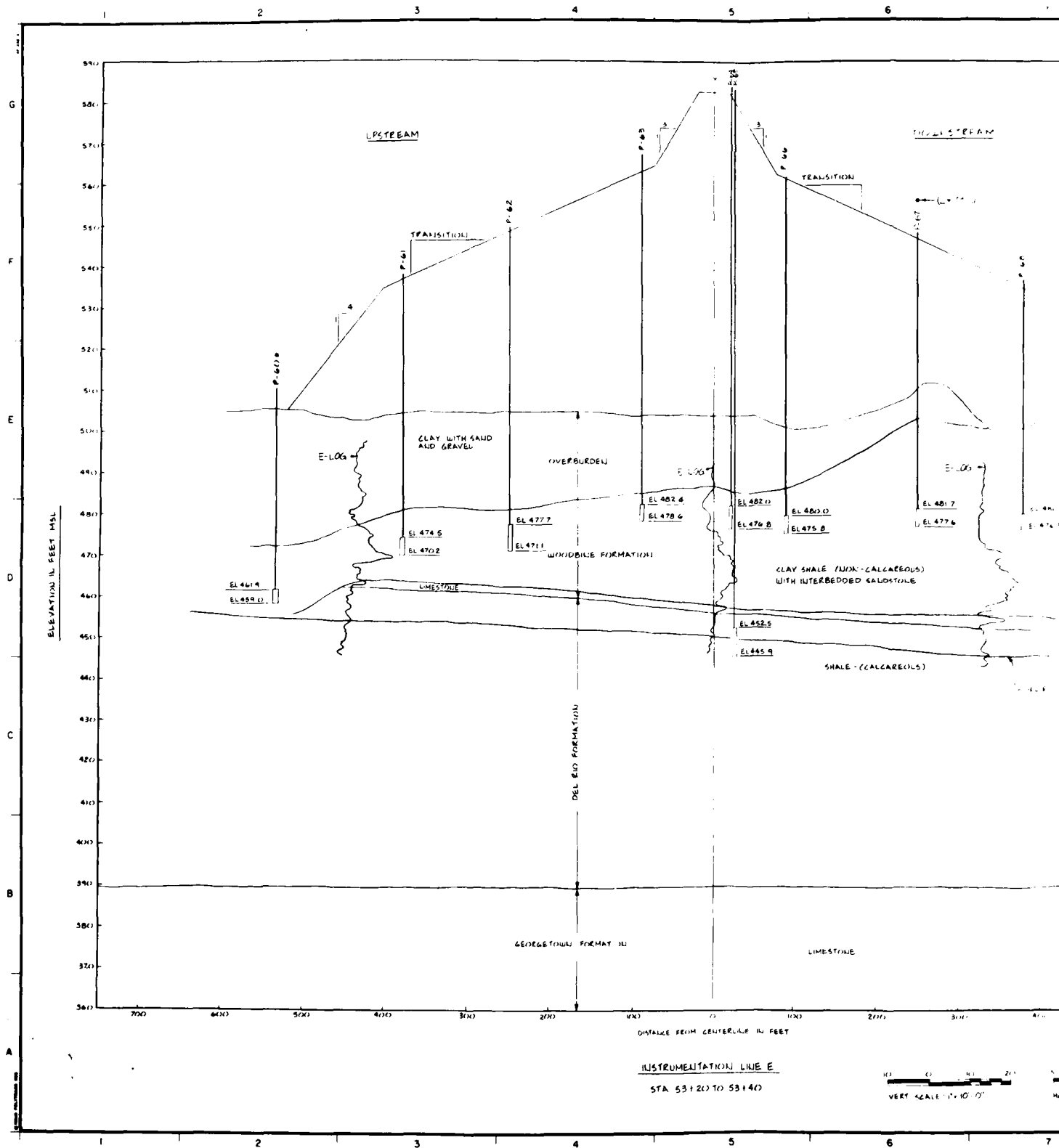
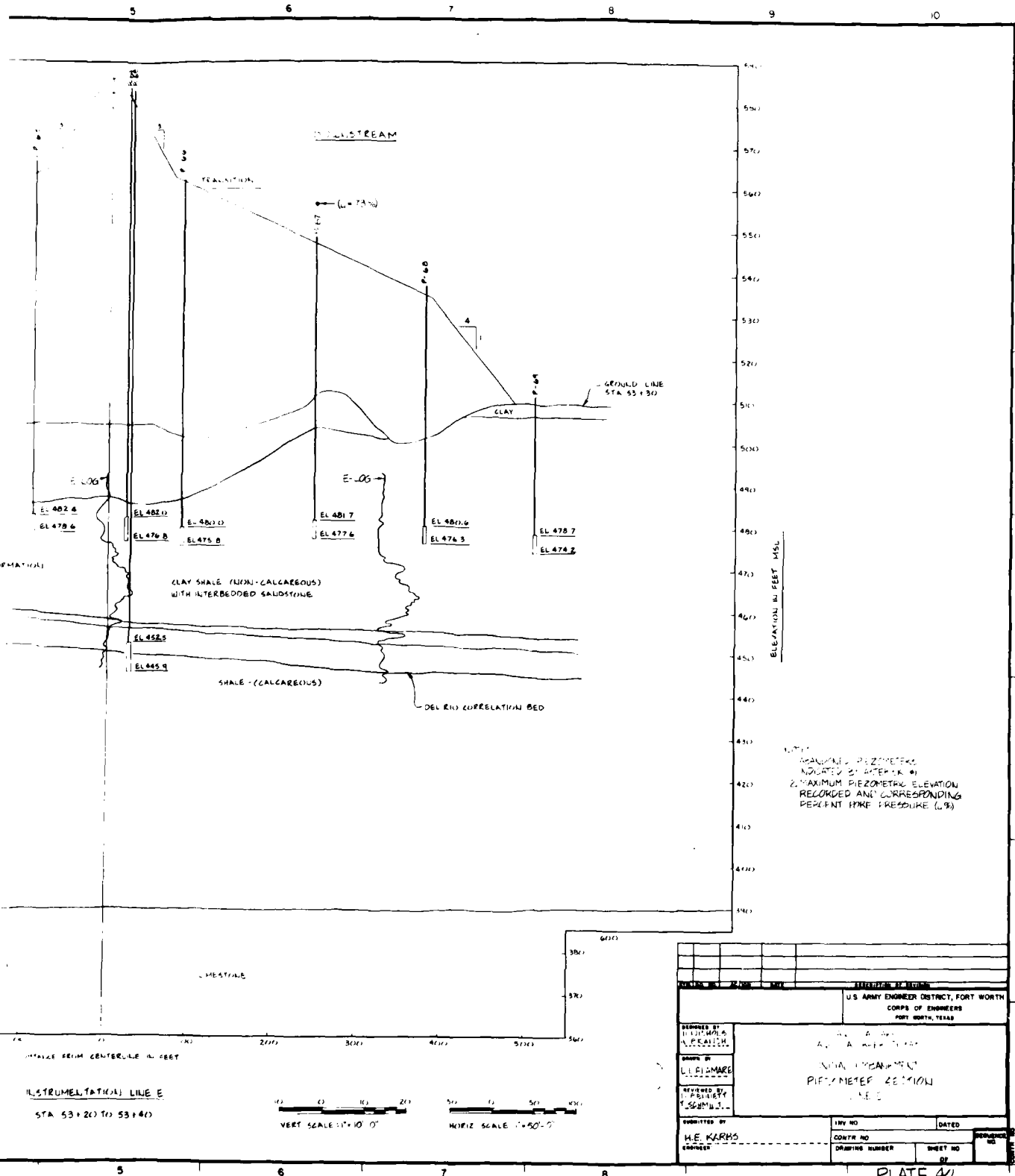
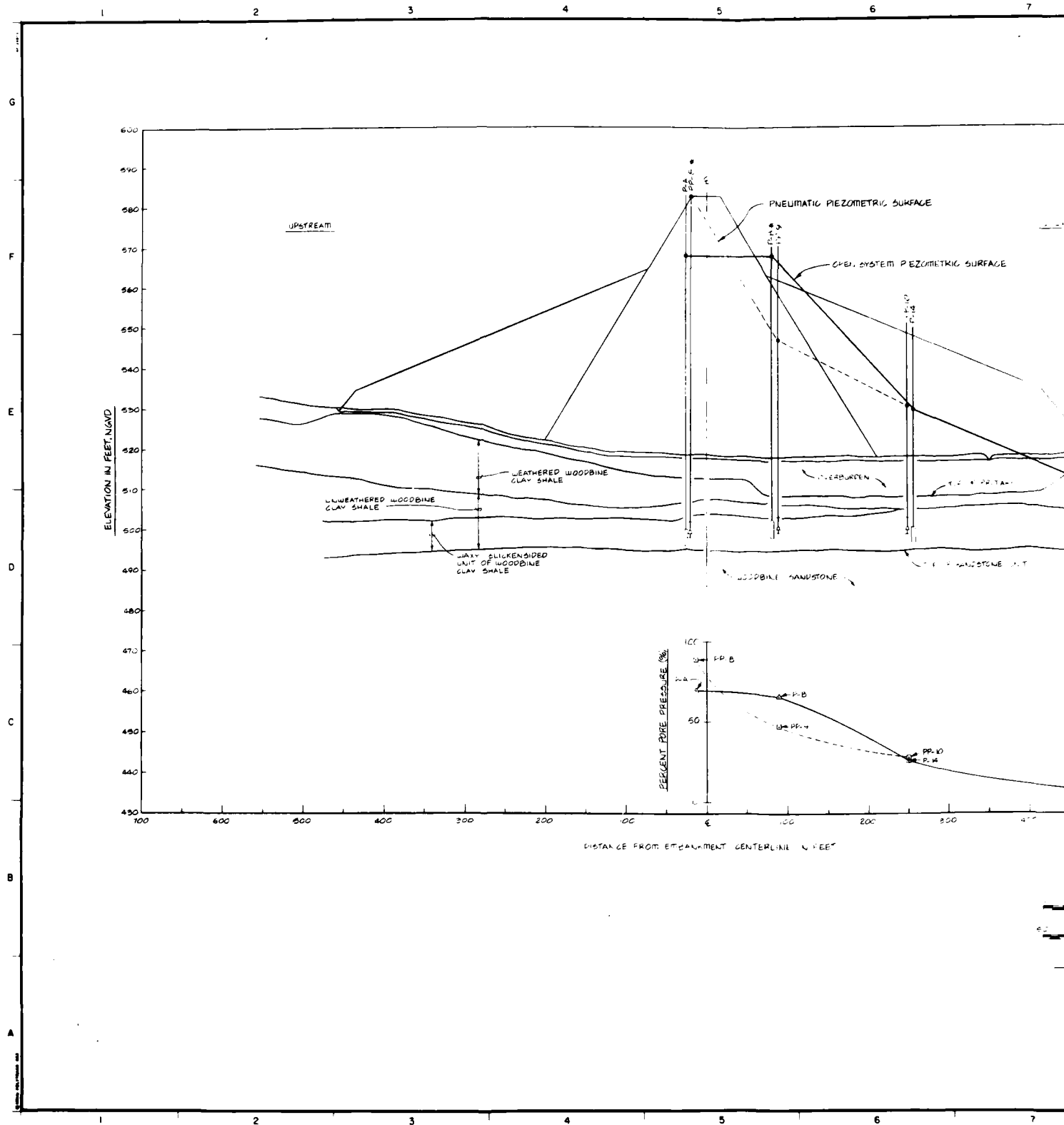
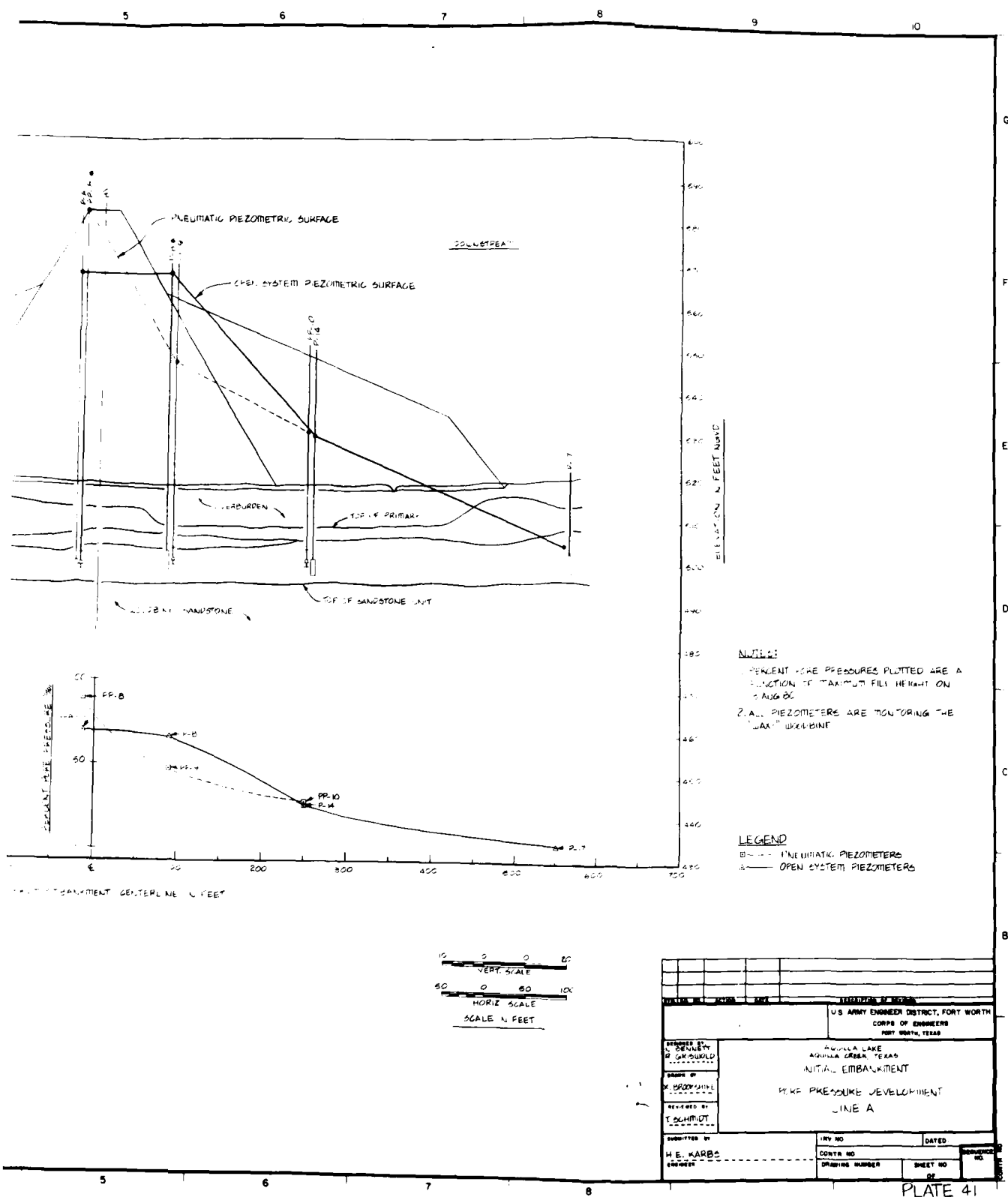


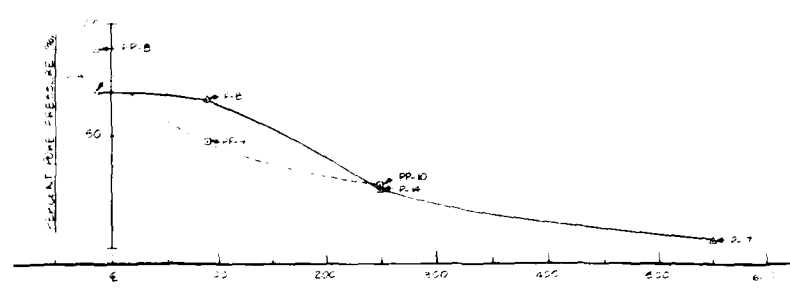
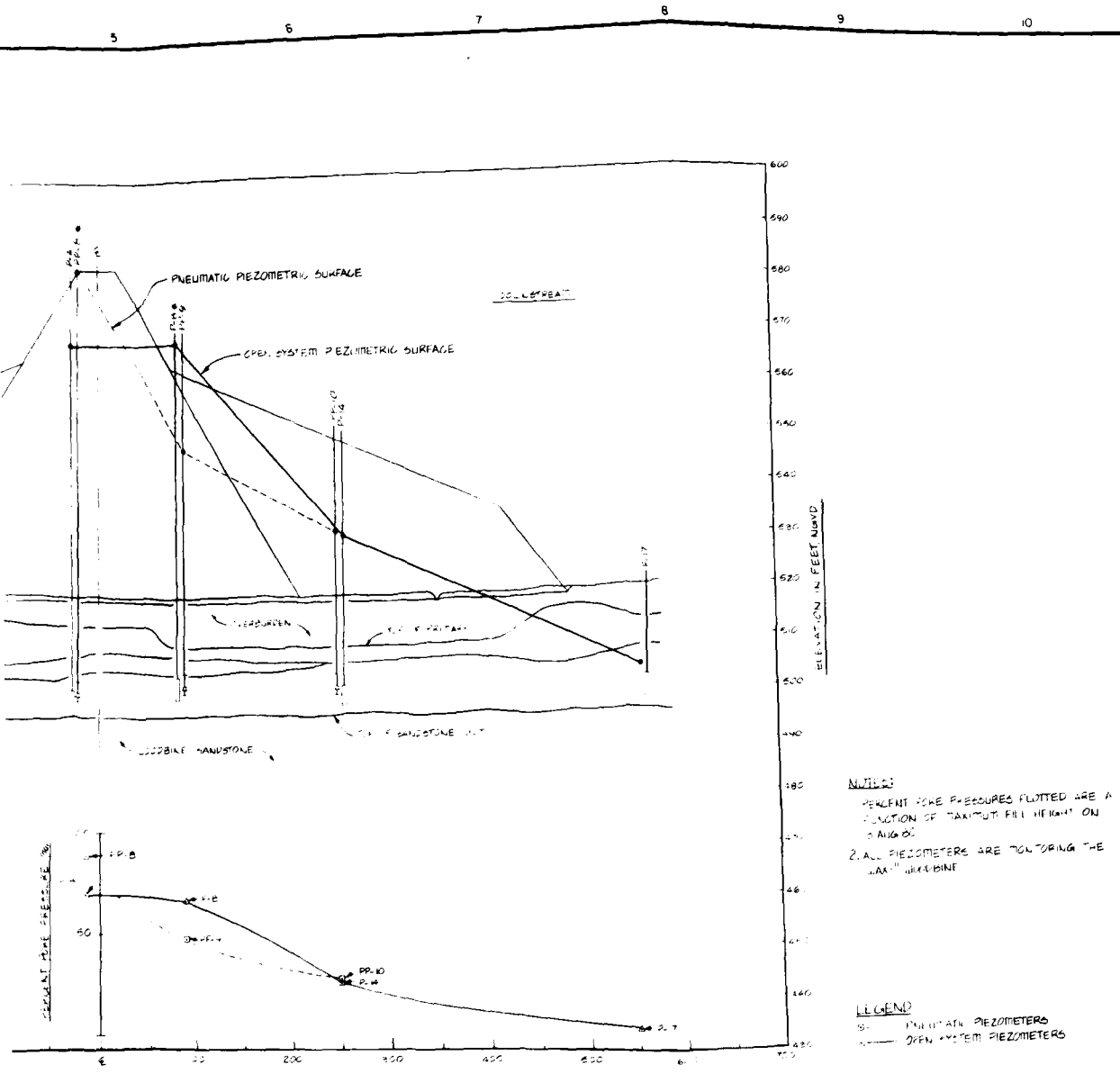
PLATE 39





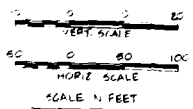






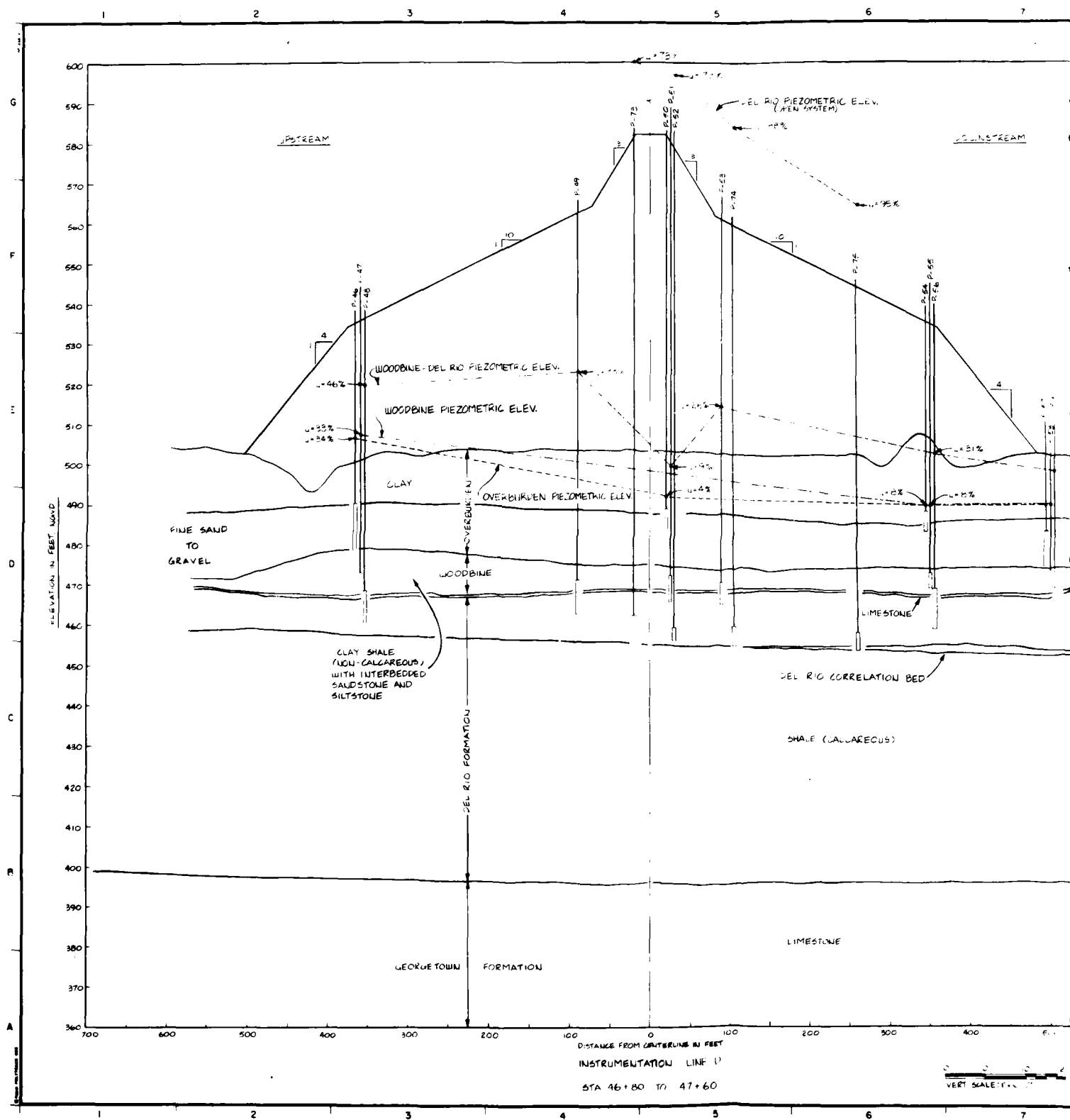
NOTES:
 1. PERCENT PORE PRESSURES PLOTTED ARE A FUNCTION OF MAXIMUM PORE PRESSURE ON 1 AUGUST 1964.
 2. ALL PIEZOMETERS ARE MONITORING THE "A" WATERBINE.

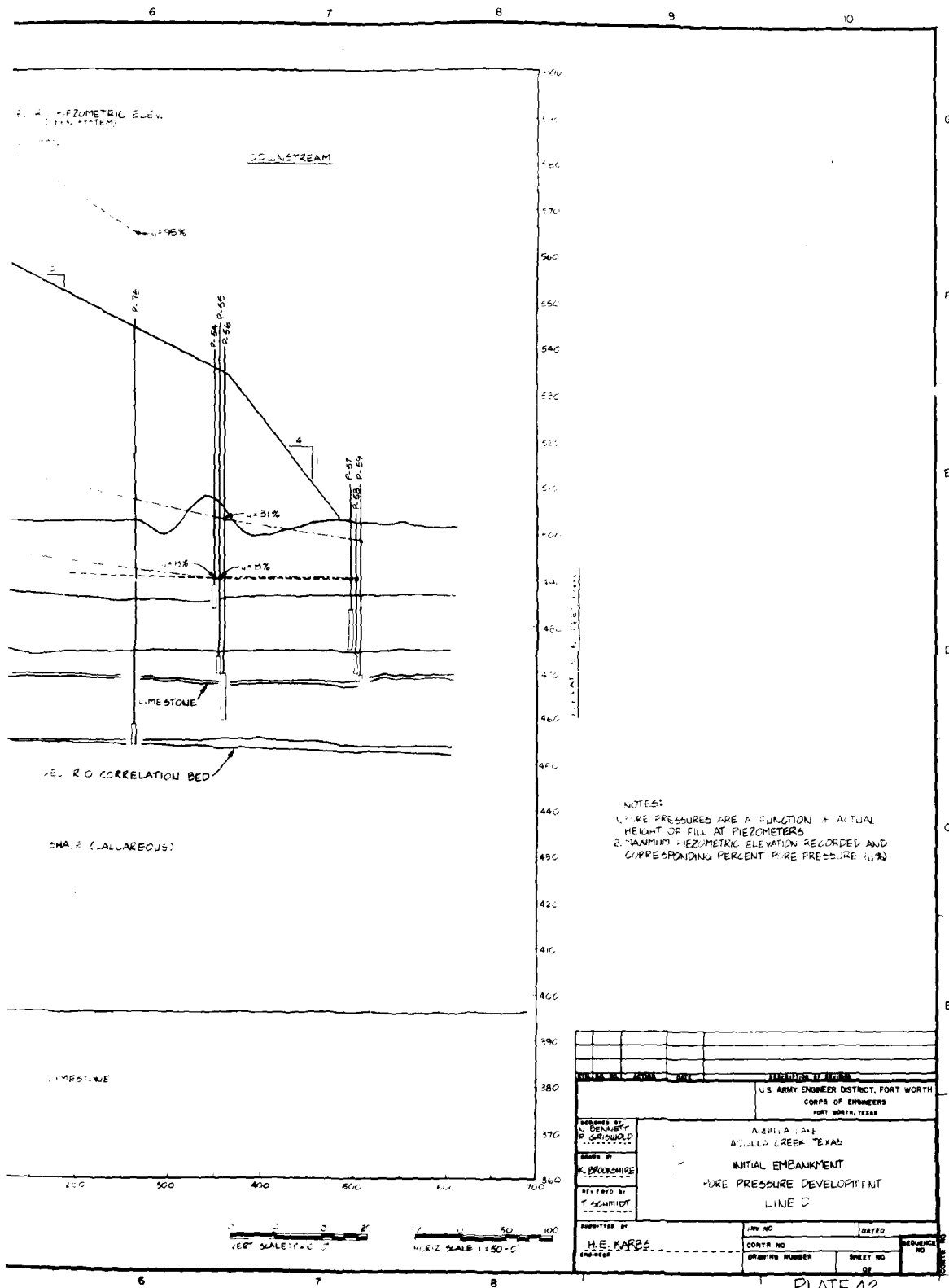
LEGEND:
 PP- PNEUMATIC PIEZOMETERS
 O- OPEN SYSTEM PIEZOMETERS



PROJECT TITLE DOUGLASS LAKE AUGUSTA CREEK, TEXAS INITIAL IMPROVEMENT WATER PRESSURE DEVELOPMENT LINE A	
SUBMITTED BY H. E. KARBO ENGINEER	DATED 1964 SHEET NO. 41
CONTRACT NO. 100-1000-100	PROJECT NO. 100-1000-100

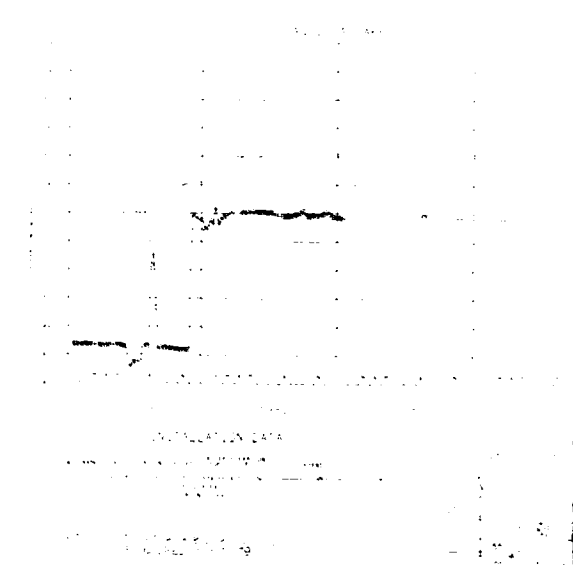
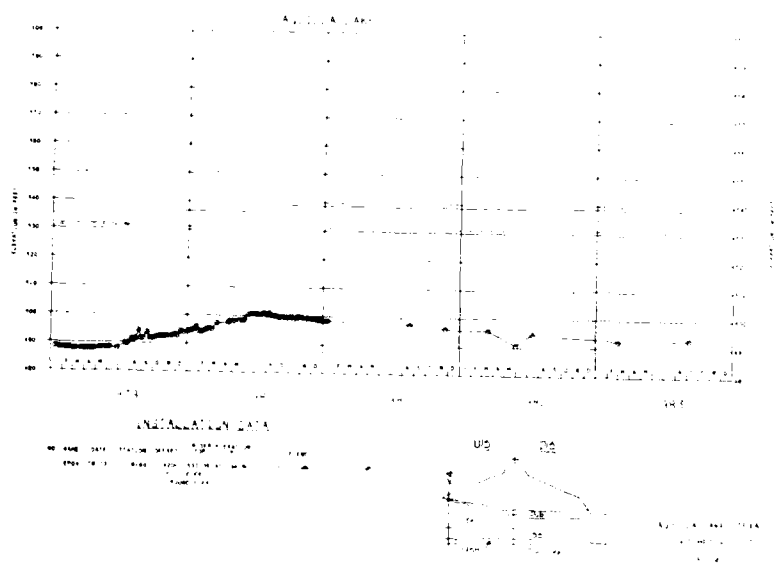
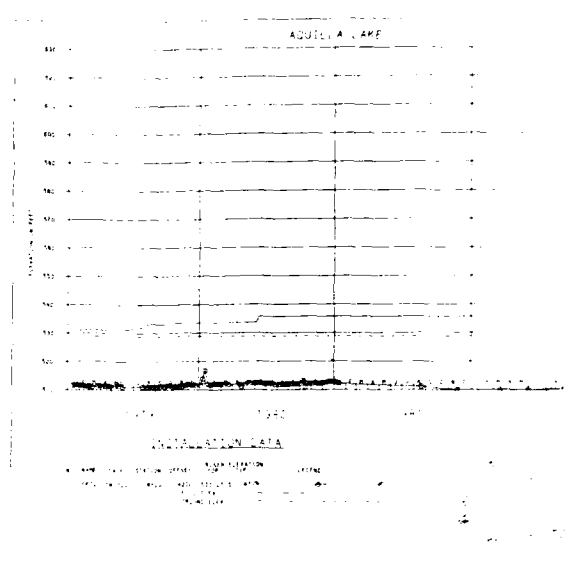
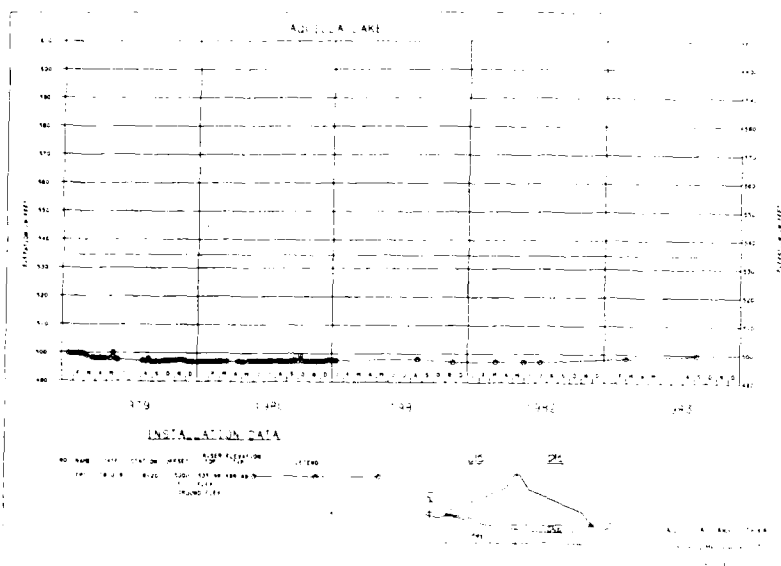
PLATE 41



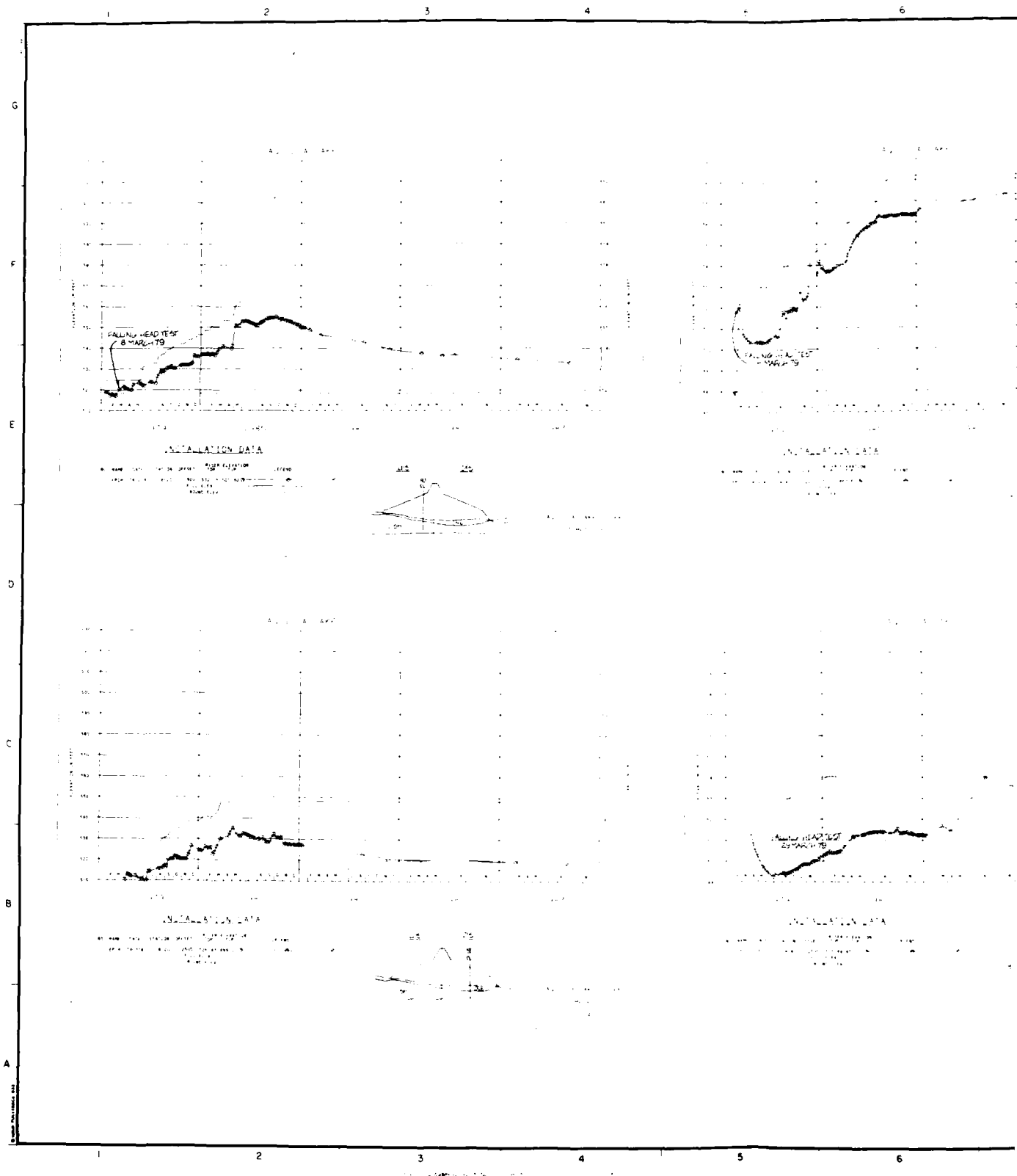


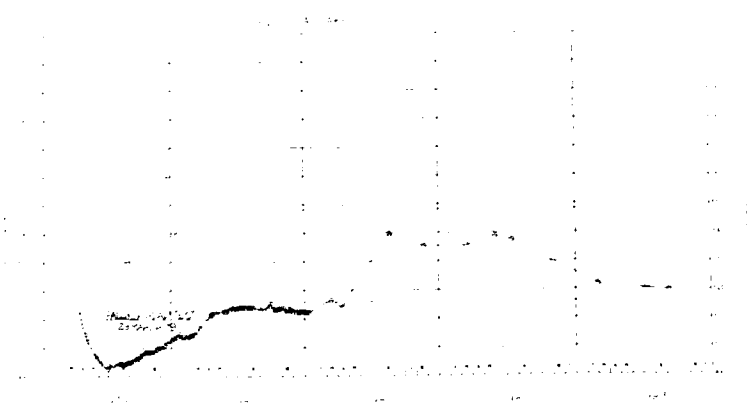
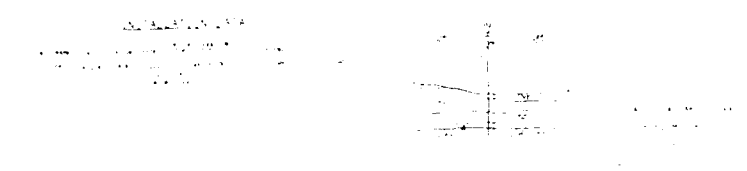
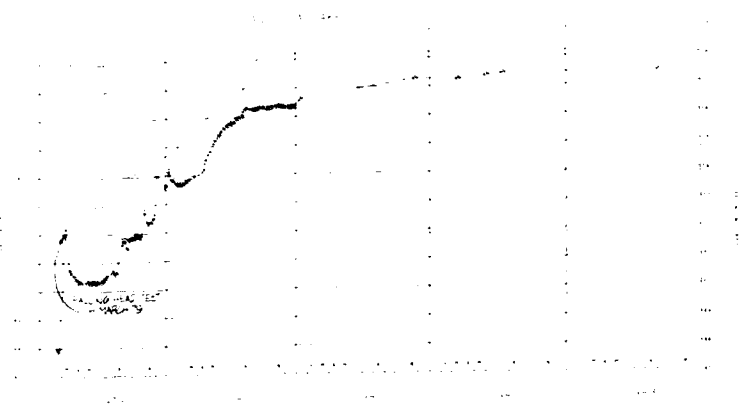
1 2 3 4 5 6 7

G
F
E
D
C
B
A



1 2 3 4 5 6 7



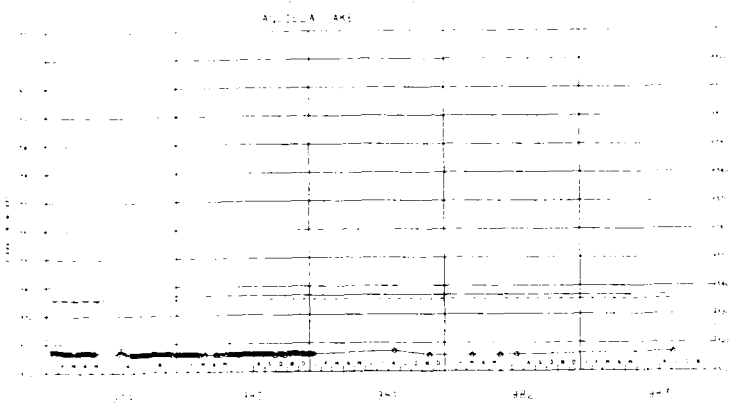


NOTE: RISE TO ELEVATION 4 FOR
BEGINNING OF READING.

LEGEND
 R12 - PIEZOMETER
 R15 - PIEZOMETER
 FILL - FILL ELEVATION

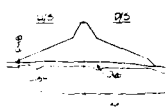
DESIGNED BY M. E. KARBS		CHECKED BY A. W. WILSON	
DRAWN BY A. W. WILSON		APPROVED BY T. C. HINCH	
AQUILLA LAKE PIEZOMETERS R12, R15 PIEZOMETER AND FILL ELEVATION VS TIME			
SUBMITTED BY M. E. KARBS		DATE NOV 10	SEQUENCE NO 1
CONTR NO		DRAWING NUMBER	SHEET NO OF 1

PLATE 44

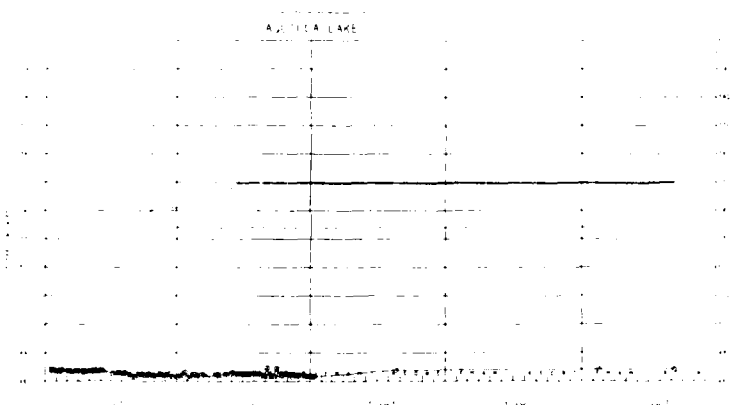


PIEZOMETER DATA

PIEZOMETER NO. P-17, P-18, P-22, P-23
 DATE OF READING
 TIME OF DAY



PIEZOMETER NO. P-17, P-18, P-22, P-23
 DATE OF READING
 TIME OF DAY



PIEZOMETER DATA

PIEZOMETER NO. P-17, P-18, P-22, P-23
 DATE OF READING
 TIME OF DAY

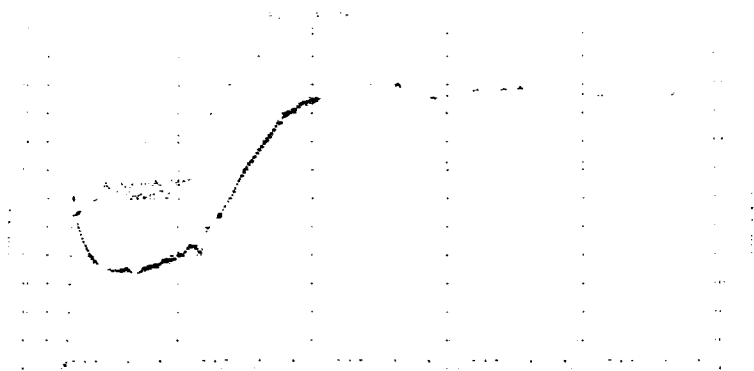


PIEZOMETER NO. P-17, P-18, P-22, P-23
 DATE OF READING
 TIME OF DAY

NOTE: PIER TOP ELEVATION 14 FOR
 BEGINNING OF READING.

- LEGEND
- - WATER LEVEL
 - - GRAVEL
 - △ - SAND
 - ◇ - CLAY
 - ◇ - LIMESTONE
 - ◇ - BED ROCK

DESIGNED BY U. BENNETT P. GARDNER		U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS	
DRAWN BY H. MARSHALL		AQUILLA LAKE PIEZOMETERS P-17, P-18, P-22, P-23 PIEZOMETER AND FILL ELEVATION VS TIME	
CHECKED BY J. SHAWLIN		REV NO. _____ DATED _____ CONTR NO. _____ DRAWING NUMBER _____ SHEET NO. _____ OF _____	
SUBMITTED BY H. E. KARBS ENGINEER		PLATE 45	



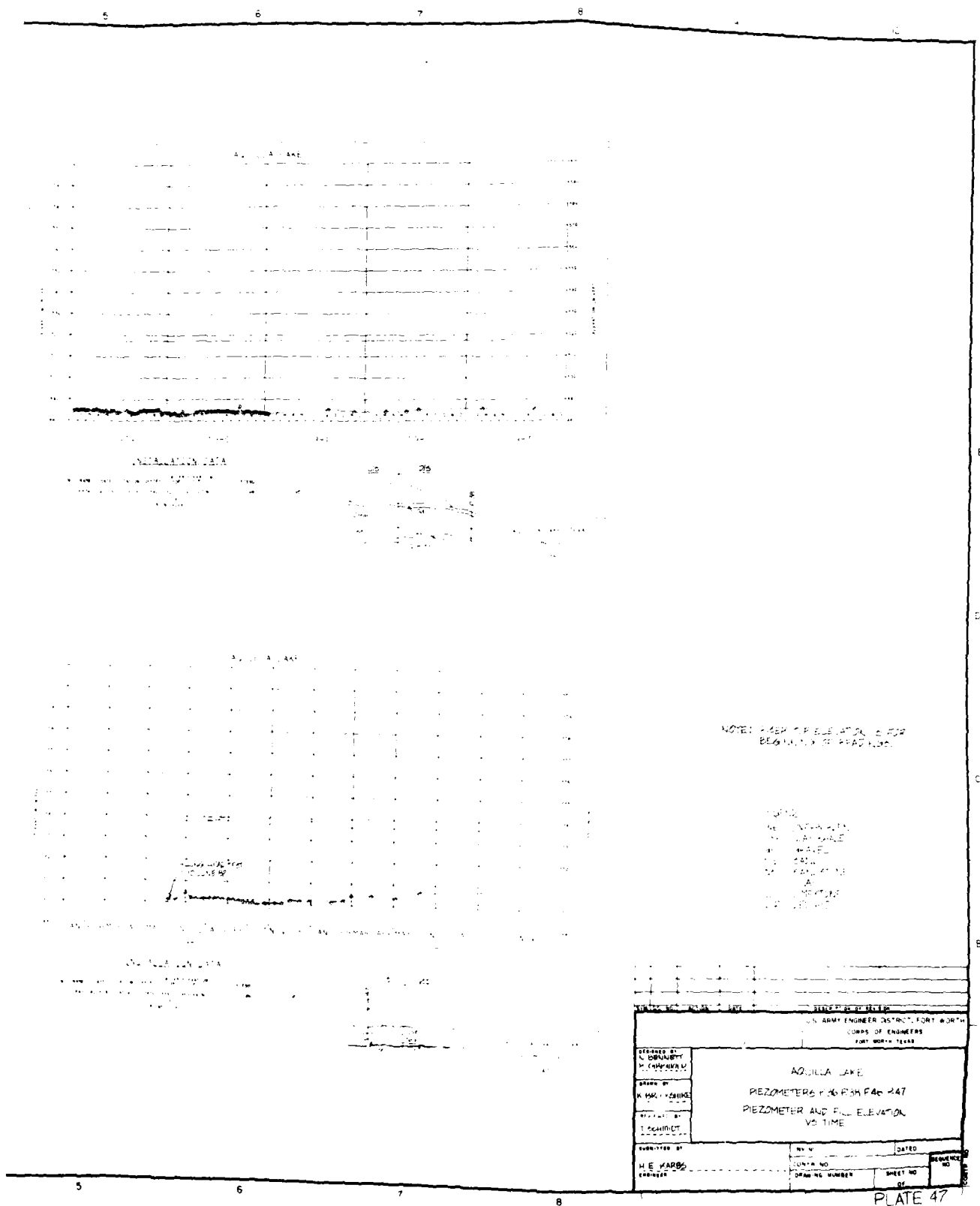
NOTE: PIER TOP ELEVATION 5 FOR
BEGINNING OF READINGS.

LEGEND
 O- OVERBANK
 G- GRAY SHALE
 S- SAND
 L- LIMESTONE
 C- CLAY
 D- DEL RIO

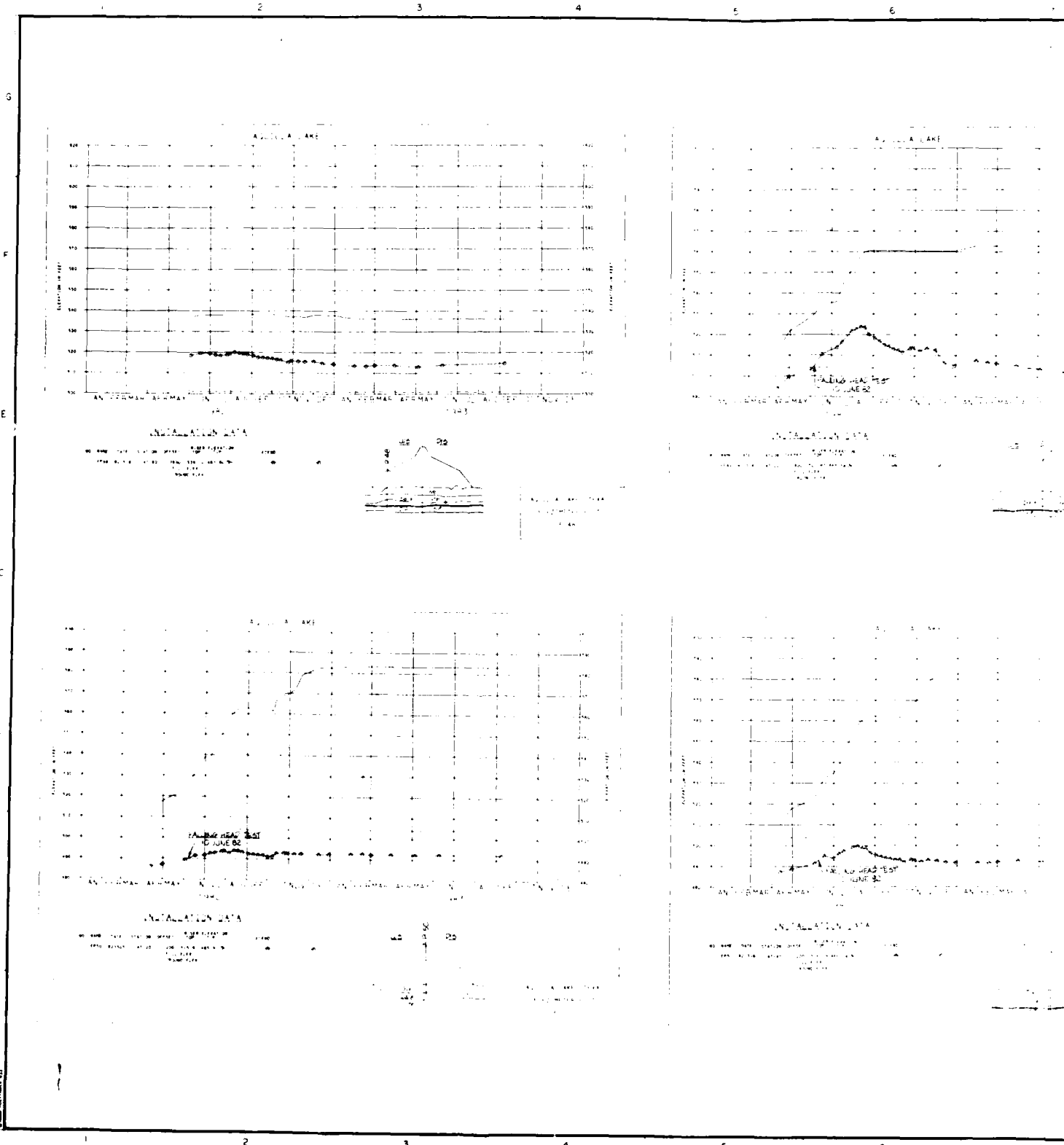
DRAWN BY H. H. HARRIS		CHECKED BY H. H. HARRIS	
APPROVED BY H. H. HARRIS		DATE 10/1/54	
SUBMITTED BY H. H. HARRIS			
ENGINEER			
CONTRACT NO.		DRAWING NUMBER	
SHEET NO.		OF	
SEQUENCE NO.		OF	

AQUILLA LAKE
 PIEZOMETERS P-26, P-28, P-29, P-30
 PIEZOMETER AND FILL ELEVATION
 VS TIME

PLATE 46



U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS	
AQUILLA LAKE PIEZOMETER AND FILL ELEVATION VS TIME	
DESIGNED BY A. D. DUNN	DATE
DRAWN BY A. D. DUNN	DATE
CHECKED BY A. D. DUNN	DATE
APPROVED BY A. D. DUNN	DATE
SUBMITTED BY A. D. DUNN	DATE
ENGINEER	ENGINEER
NO. 1	NO. 2
NO. 3	NO. 4
NO. 5	NO. 6
NO. 7	NO. 8
NO. 9	NO. 10
NO. 11	NO. 12
NO. 13	NO. 14
NO. 15	NO. 16
NO. 17	NO. 18
NO. 19	NO. 20
NO. 21	NO. 22
NO. 23	NO. 24
NO. 25	NO. 26
NO. 27	NO. 28
NO. 29	NO. 30
NO. 31	NO. 32
NO. 33	NO. 34
NO. 35	NO. 36
NO. 37	NO. 38
NO. 39	NO. 40
NO. 41	NO. 42
NO. 43	NO. 44
NO. 45	NO. 46
NO. 47	NO. 48
NO. 49	NO. 50
NO. 51	NO. 52
NO. 53	NO. 54
NO. 55	NO. 56
NO. 57	NO. 58
NO. 59	NO. 60
NO. 61	NO. 62
NO. 63	NO. 64
NO. 65	NO. 66
NO. 67	NO. 68
NO. 69	NO. 70
NO. 71	NO. 72
NO. 73	NO. 74
NO. 75	NO. 76
NO. 77	NO. 78
NO. 79	NO. 80
NO. 81	NO. 82
NO. 83	NO. 84
NO. 85	NO. 86
NO. 87	NO. 88
NO. 89	NO. 90
NO. 91	NO. 92
NO. 93	NO. 94
NO. 95	NO. 96
NO. 97	NO. 98
NO. 99	NO. 100



5

6

7

8

9

10

G

F

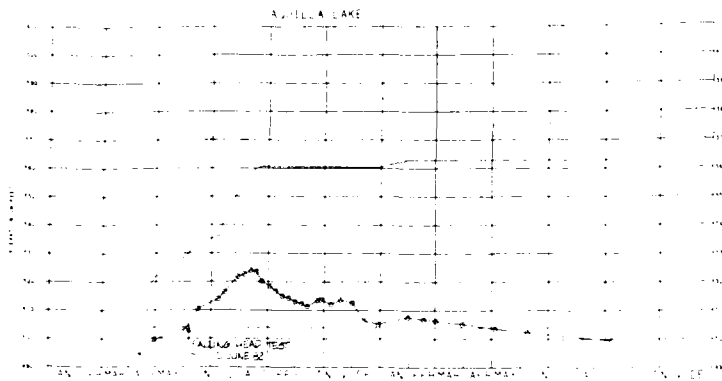
E

D

C

B

AGUILA LAKE



INSTALLATION DATA

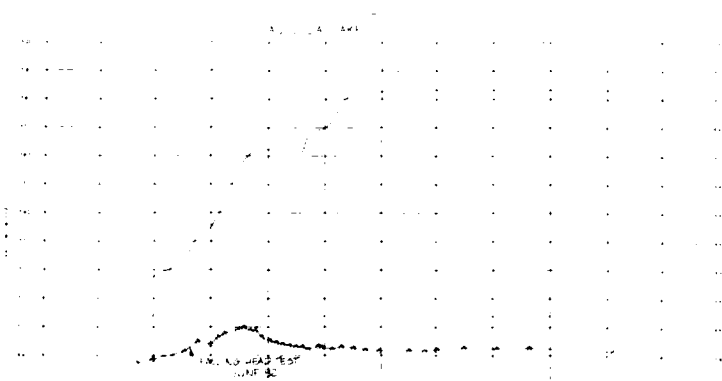
PIEZOMETER NO. 1
PIEZOMETER NO. 2



NOTE: PIER TOP ELEVATION 6 FOR
BEGINNING OF READINGS.

LEGEND

NEUTRAL POINT
PIEZOMETER NO. 1
PIEZOMETER NO. 2
PIEZOMETER NO. 3
PIEZOMETER NO. 4
PIEZOMETER NO. 5
PIEZOMETER NO. 6
PIEZOMETER NO. 7
PIEZOMETER NO. 8
PIEZOMETER NO. 9
PIEZOMETER NO. 10



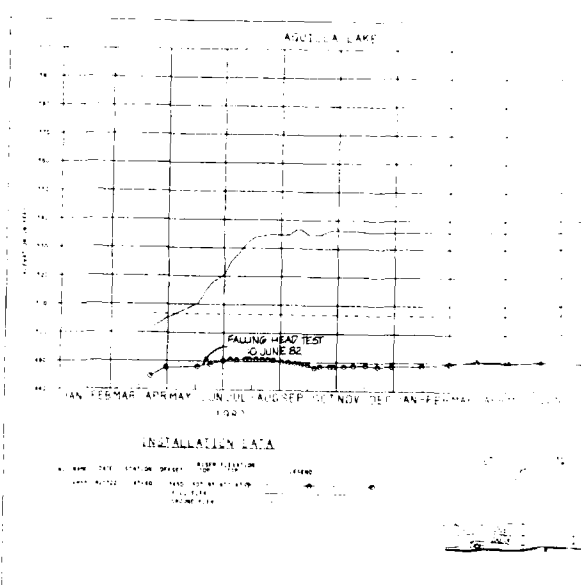
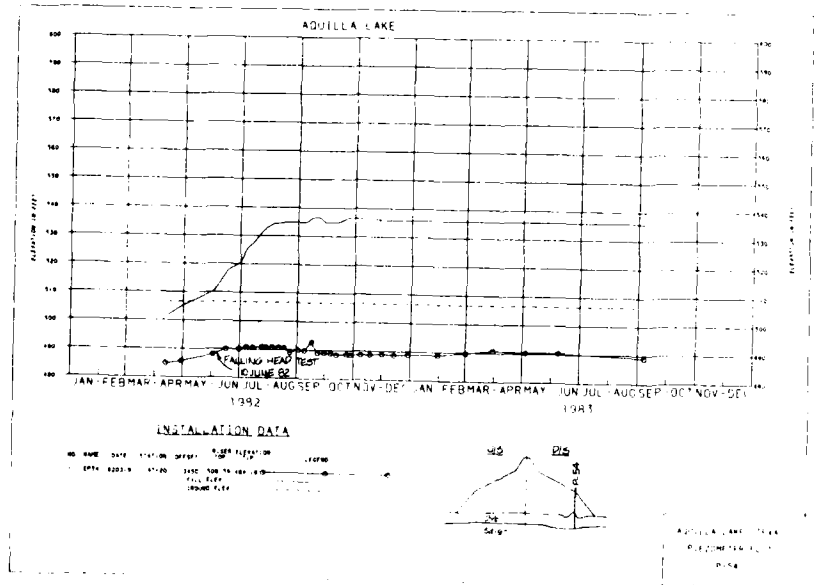
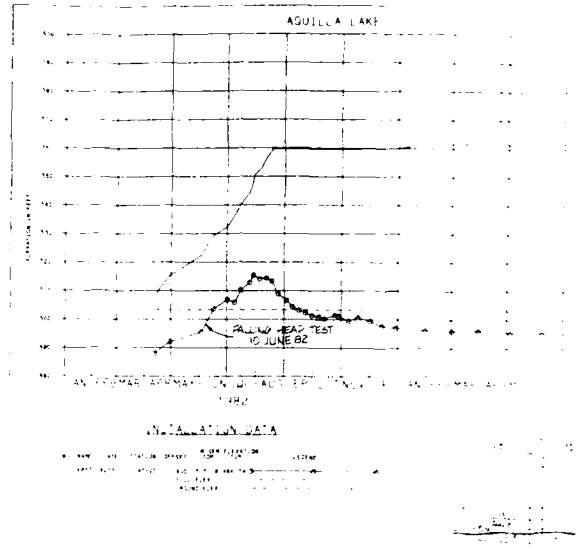
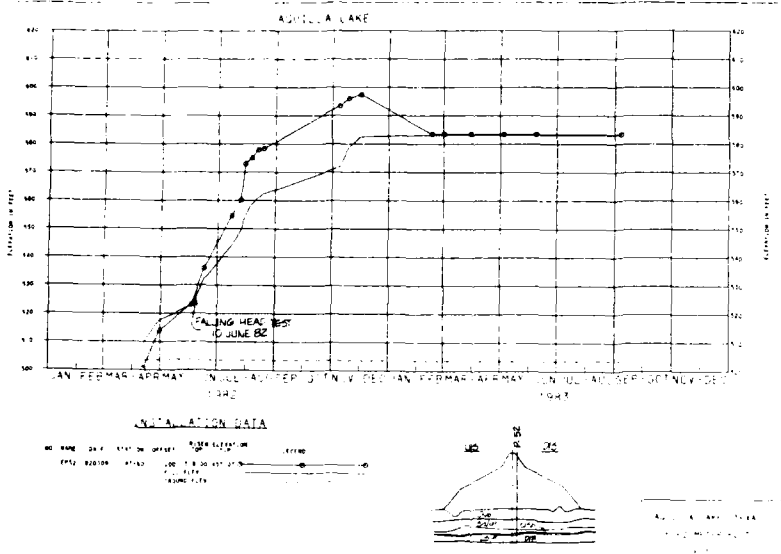
INSTALLATION DATA

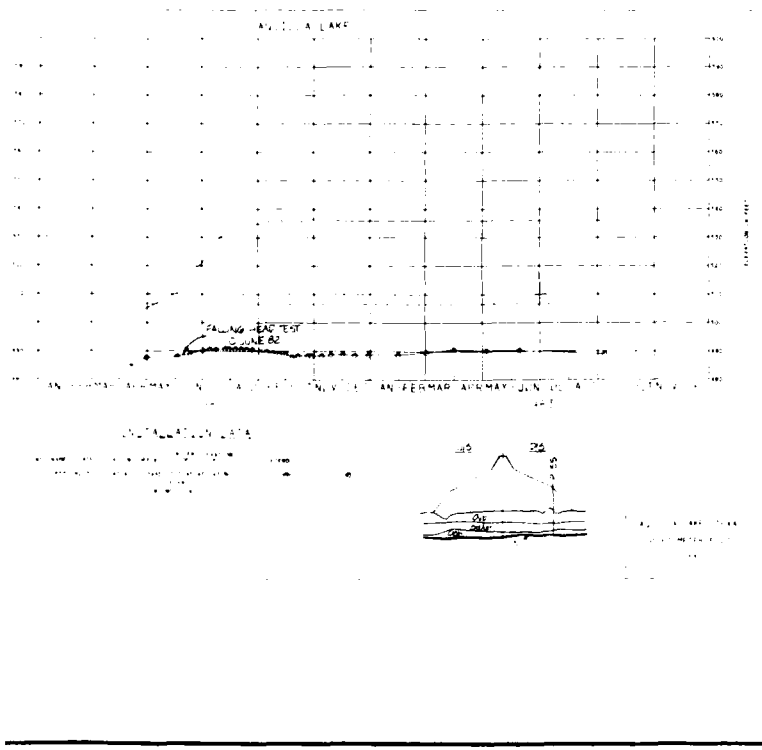
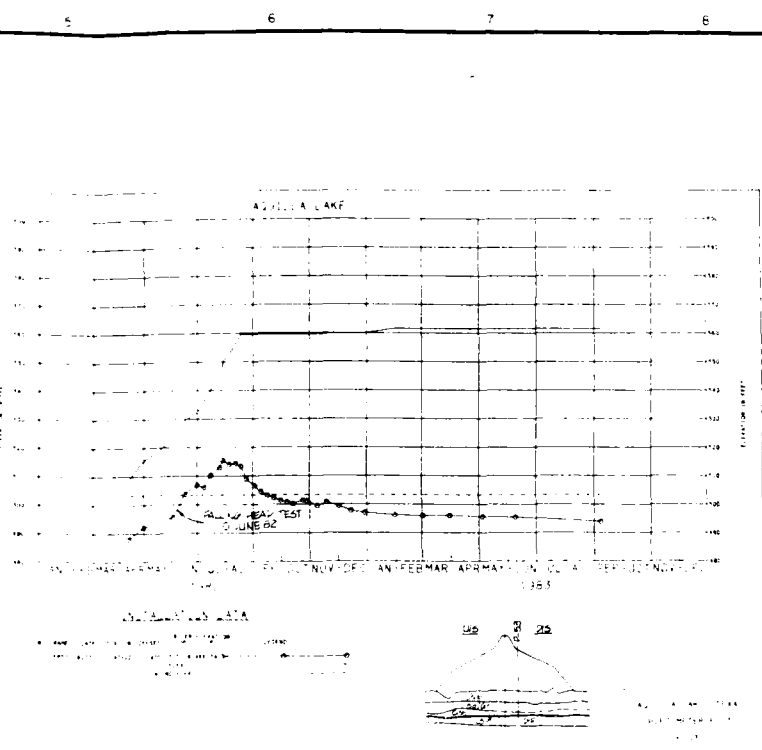
PIEZOMETER NO. 1
PIEZOMETER NO. 2



U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS			
AGUILA LAKE PIEZOMETERS NO. 1, 2, 3, 4, 5, 6, 7, 8, 9, 10 PIEZOMETER AND FILL ELEVATION VS TIME			
DESIGNED BY P. H. KARBOS	DATE 10/1/51	SEQUENCE NO. 1	
DRAWN BY P. H. KARBOS	COUNT NO.	SHEET NO.	OF
ENGINEER	DRAWING NUMBER	SHEET NO.	OF

PLATE 43



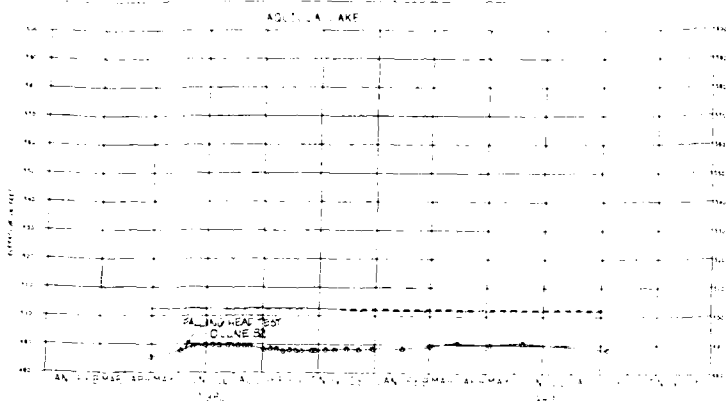


NOTE: PIER TOP ELEVATION IS FOR BEGINNING OF READINGS.

- LEGEND
- ONE OVERB R.S.
 - CLAY SHALE
 - GRAVEL
 - SAND
 - SANDSTONE
 - CLAY
 - LIMESTONE
 - REF. RD.

DESIGNED BY H. PENNETT R. BROWN		U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS	
DRAWN BY H. HARRIS		AQUILLA LAKE PIEZOMETERS P-52, P-53, P-54, P-55 PIEZOMETER AND FILL ELEVATION VS TIME	
CHECKED BY H. HARRIS		INT. NO.	DATED
ENGINEER		CONT. NO.	SEQUENCE NO.
		DRAWING NUMBER	SHEET NO.
			52





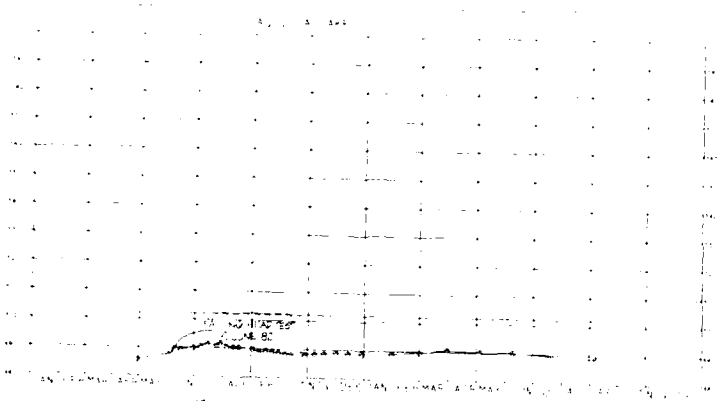
INSTALLATION DATA

PIEZOMETER NO. 25
 DATE OF INSTALLATION
 LOCATION OF PIEZOMETER



NOTE: FOR THE PURPOSE OF THIS REPORT, THE DATA IS BEING PRESENTED AS IS.

PIEZOMETER NO. 25
 DATE OF INSTALLATION
 LOCATION OF PIEZOMETER



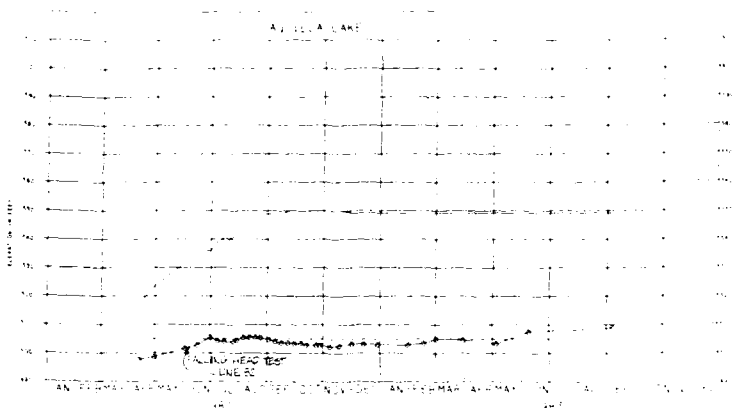
INSTALLATION DATA

PIEZOMETER NO. 25
 DATE OF INSTALLATION
 LOCATION OF PIEZOMETER



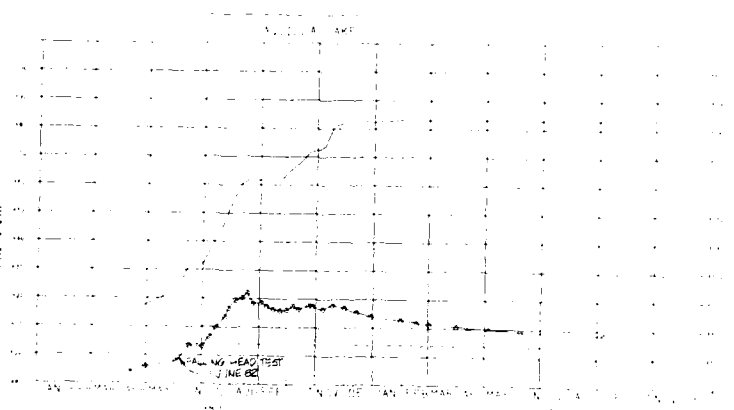
DESIGNED BY E. BENNETT CHECKED BY R. W. WILKINSON DATE 1/1/41 DRAWN BY J. C. HUBBARD		AGUILA LAKE PIEZOMETERS FOR R27 P58, R59 PIEZOMETER AND FILL ELEVATION VS TIME	
SUBMITTED BY H. E. HARBO ENGINEER	DATE 1/1/41	SHEET NO. 27	SEQUENCE NO. 1

PLATE 50



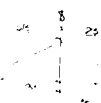
ANALYSIS DATA

1. NAME OF PROJECT: AGUILA LAKE
2. LOCATION: ...
3. DATE: ...



ANALYSIS DATA

1. NAME OF PROJECT: AGUILA LAKE
2. LOCATION: ...
3. DATE: ...



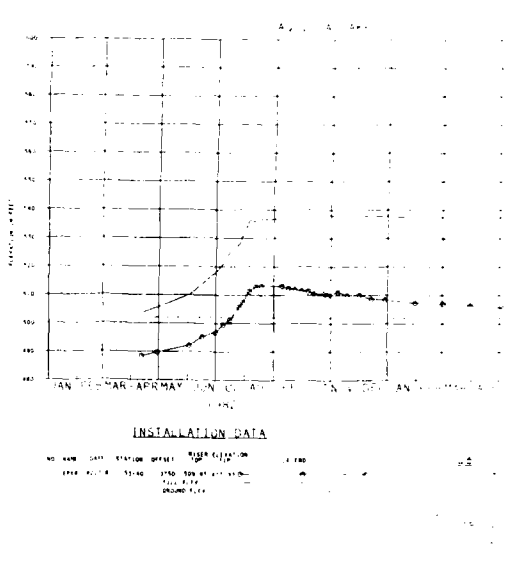
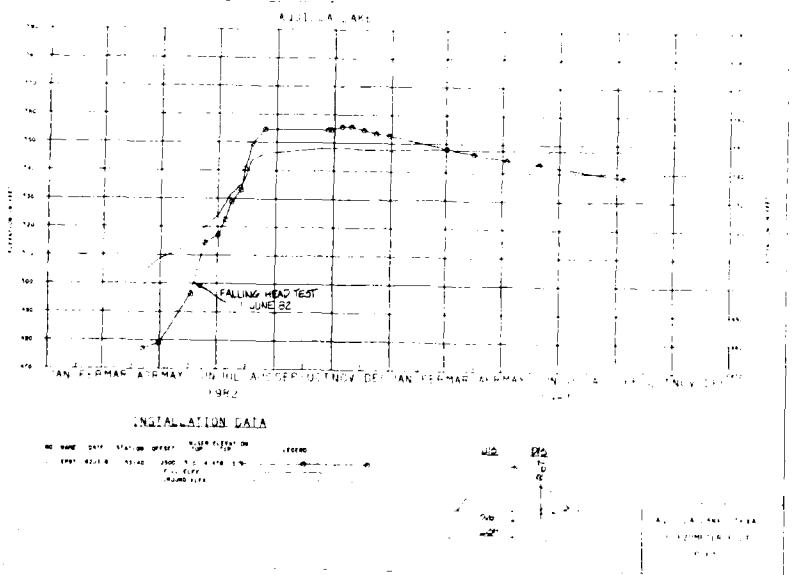
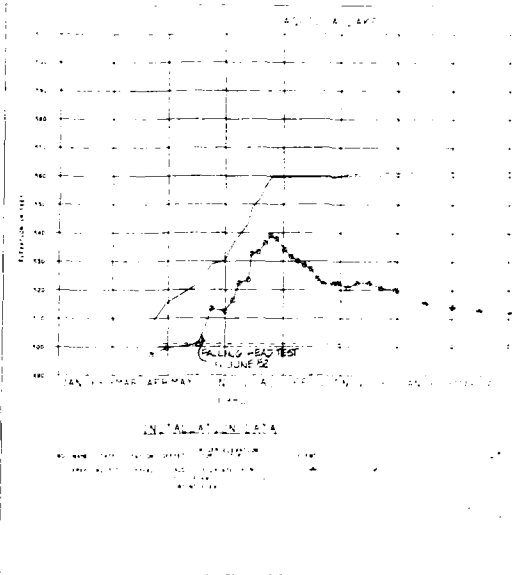
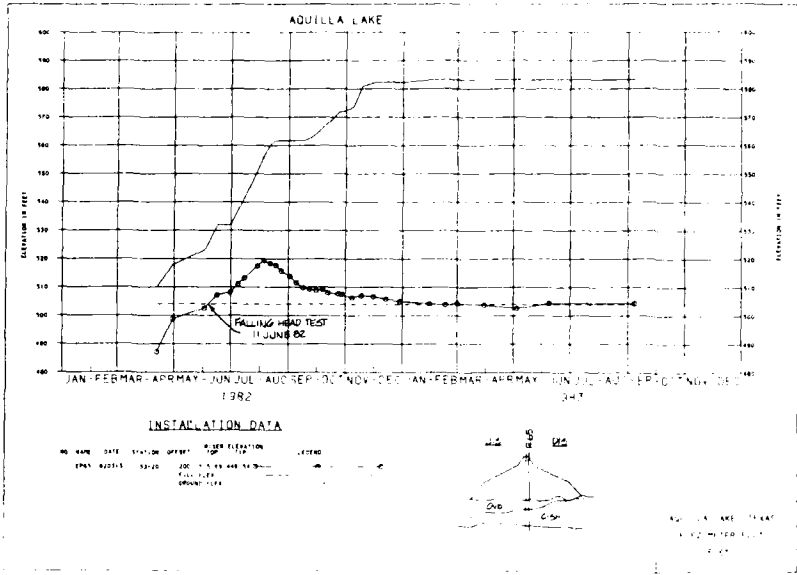
NOTES: WATER TABLE ELEVATION FOR
BEGINNING OF HEAD TEST

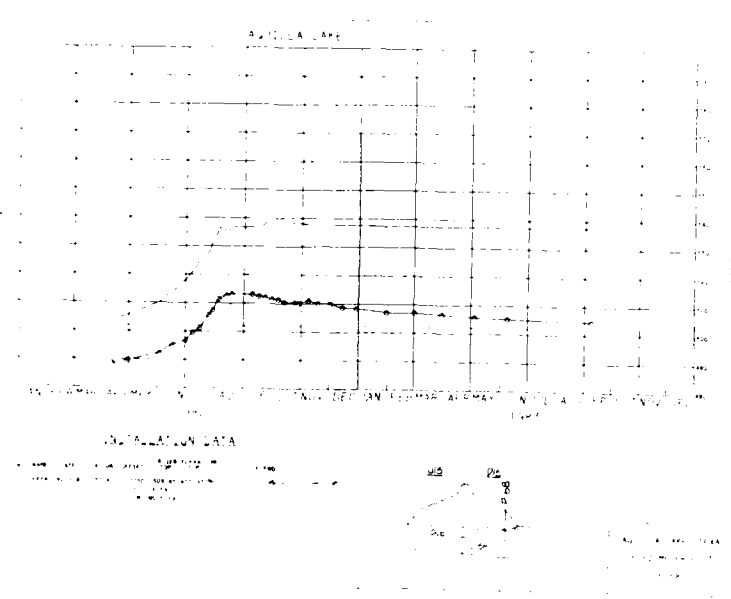
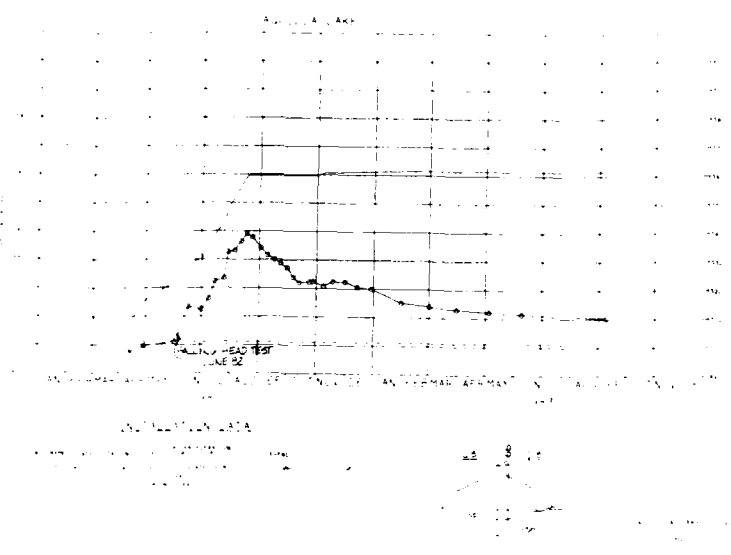
1. NAME OF PROJECT: AGUILA LAKE
2. LOCATION: ...
3. DATE: ...

DESIGNED BY N. BENNETT		DRAWN BY P. P. P. P. P.		CHECKED BY T. SCHULTZ	
AGUILA LAKE PIEZOMETERS R61, R62, R63, R64 PIEZOMETER AND FILL ELEVATION VS TIME					
SUBMITTED BY H. E. KARDS		DATE ...		SEQUENCE NO. ...	
ENGINEER		DRAWING NUMBER		SHEET NO. 51	

PLATE 51

G
F
E
D
C
B
A



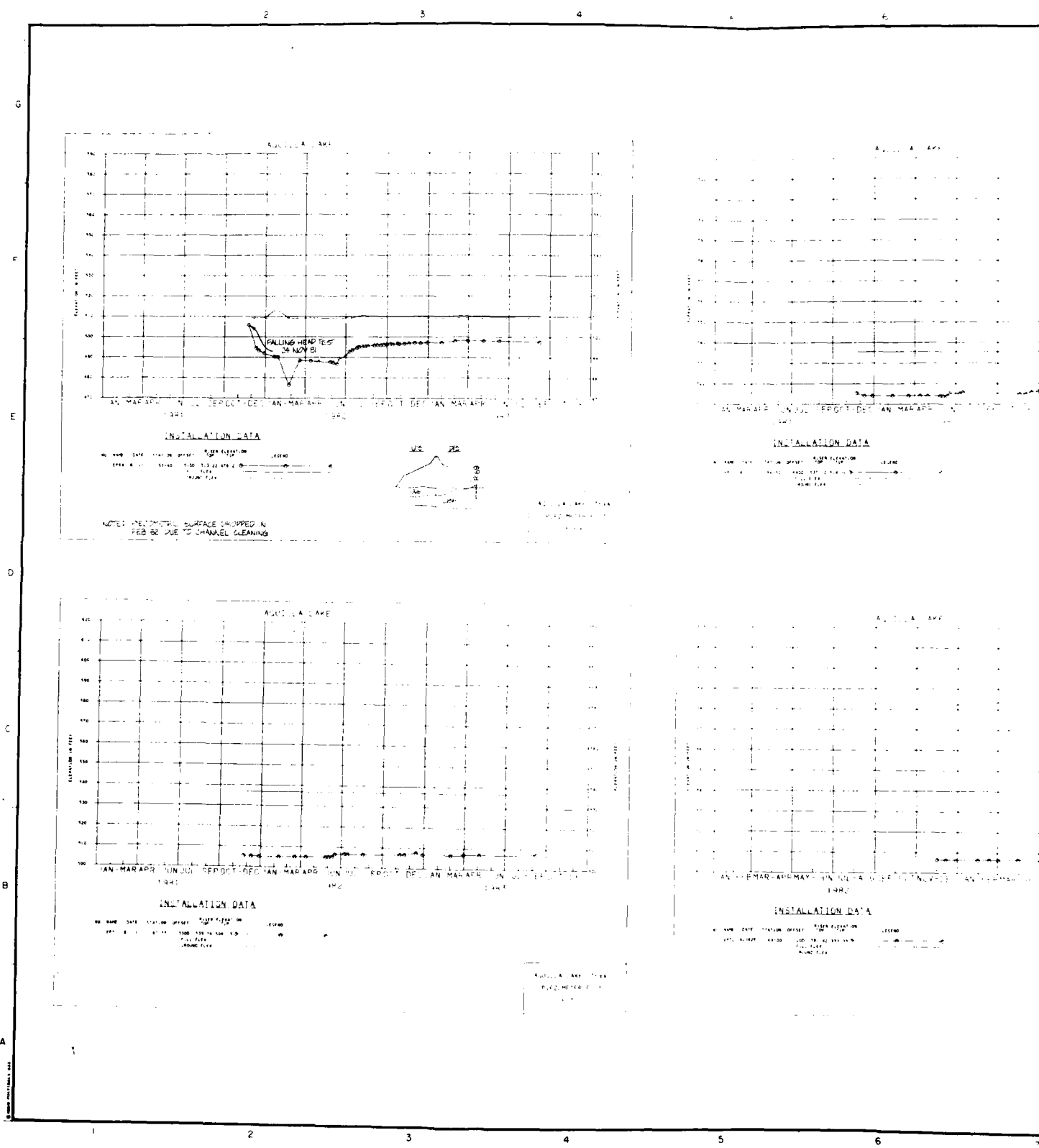


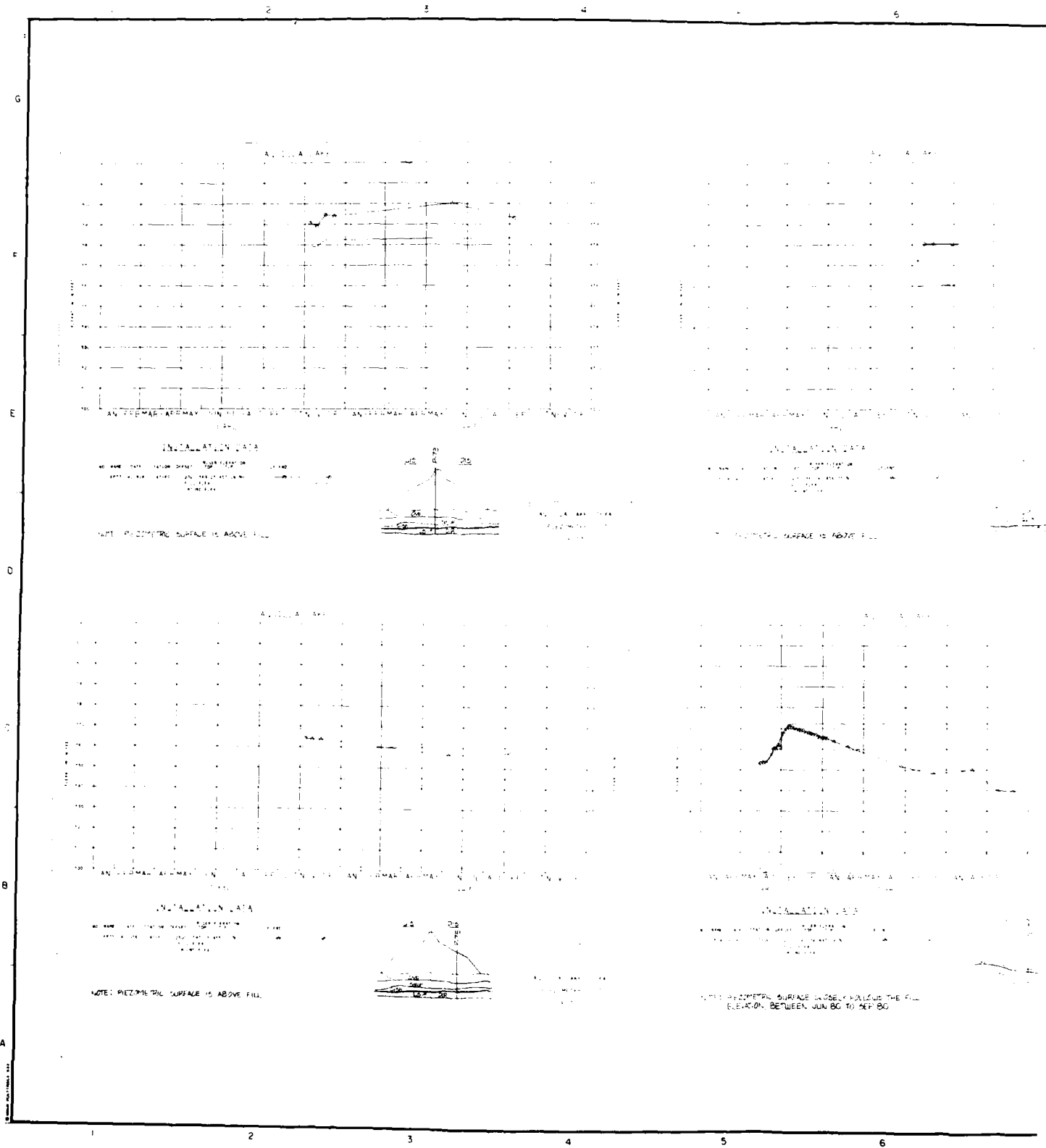
NOTE: Riser top elevation is for beginning of readings.

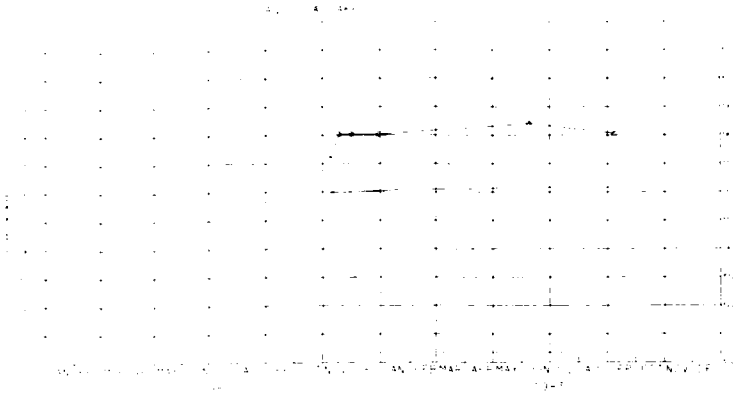
- LEGEND
- Ob - OVERB. FOR.
 - Ca - CLAY SHALE
 - Gr - GRAVEL
 - Ss - SAND
 - St - SANDSTONE
 - Cl - CLAY
 - Ls - LIMESTONE
 - DR - DEL. RD

DESIGNED BY H. E. KARBO		CHECKED BY H. E. KARBO	
DRAWN BY H. E. KARBO		PIECED BY H. E. KARBO	
REVIEWED BY T. SCHINDT		DATE JUN 52	
SUBMITTED BY H. E. KARBO		ENGINEER	
CONTRACT NO.		DRAWING NUMBER	
SHEET NO.		SEQUENCE NO.	

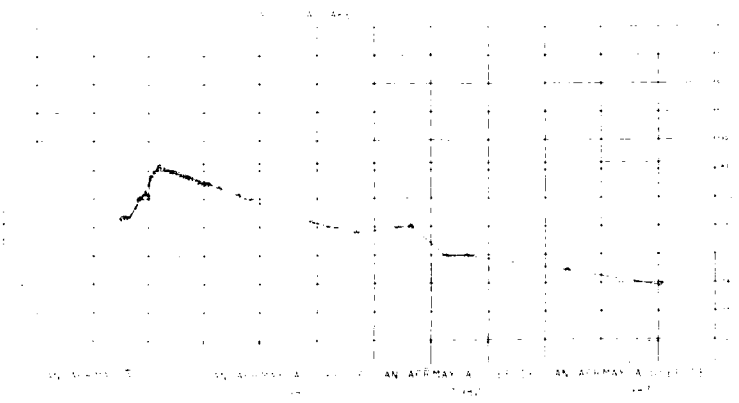
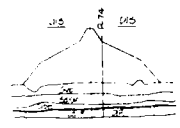
AQUILLA LAKE
PIEZOMETERS P65, P66, P67, P68
PIEZOMETER AND FILL ELEVATION
VS TIME



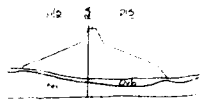




PIEZOMETER R73
WATER LEVEL
TIME



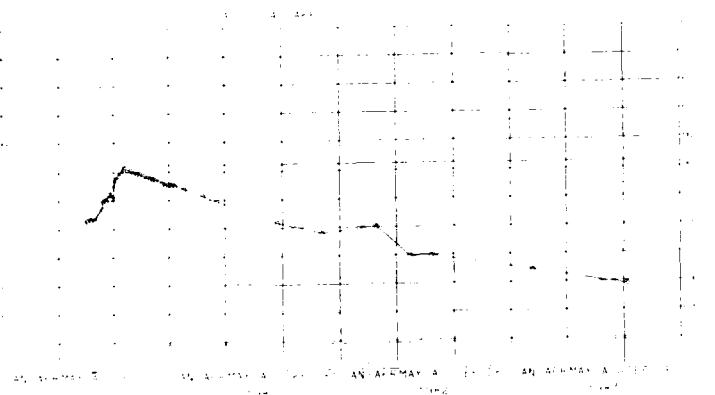
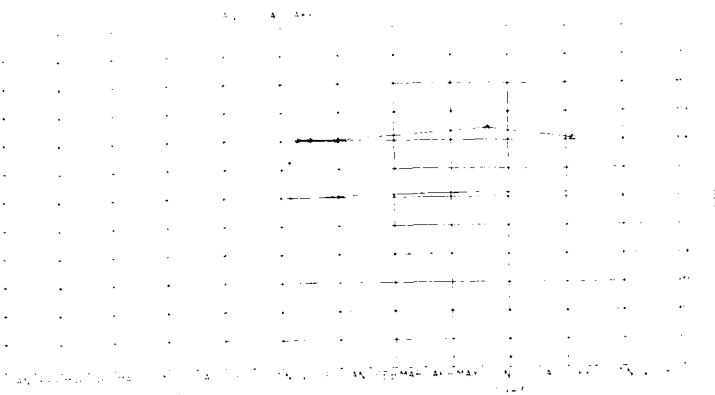
PIEZOMETER R75
WATER LEVEL
TIME



NOTE: WSER TOP ELEVATION 5.42
BEGINNING OF READINGS.

- LEGEND
- OWB - OVERBORDEN
 - CLSH - CLAY SHALE
 - GR - GRAVEL
 - SD - SAND
 - SD - SANDSTONE
 - CL - CLAY
 - LS - LIMESTONE
 - DR - DEL RIO

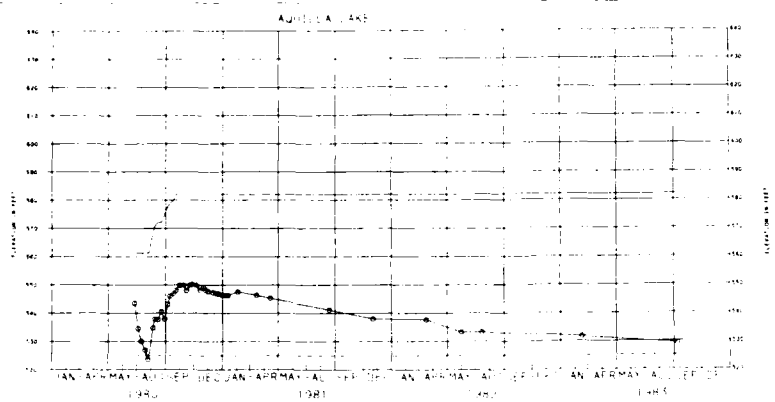
SUBMITTED BY H.E. KARDO ENGINEER		DATE 10/1/54	
CONTRACT NO. 100-100-100		SHEET NO. OF	
DRAWING NUMBER 100-100-100			
SEQUENCE NO. 1			
U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS AQUILLA LAKE PIEZOMETERS R73, R74, R75, R76 PIEZOMETER AND FILL ELEVATION VS TIME			



NOTE: PIER TOP ELEVATION 5 FOR BEGINNING OF READING.

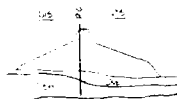
- LEGEND
- OB - OVERBURDEN
 - CS - CLAY SHALE
 - ST - GRAVEL
 - SI - SAND
 - SS - SANDSTONE
 - CL - CLAY
 - LS - LIMESTONE
 - DR - DEL RIO

DESIGNED BY H. E. KARBO		CHECKED BY H. E. KARBO	
DRAWN BY H. E. KARBO		PIEZOMETER R73, R74, P75, P76	
REVIEWED BY H. E. KARBO		PIEZOMETER AND FILL ELEVATION VS TIME	
SUBMITTED BY H. E. KARBO		DATE 10/10/40	REVISION NO
CONTRACT NO.		DRAWING NUMBER	SHEET NO.
		OF	



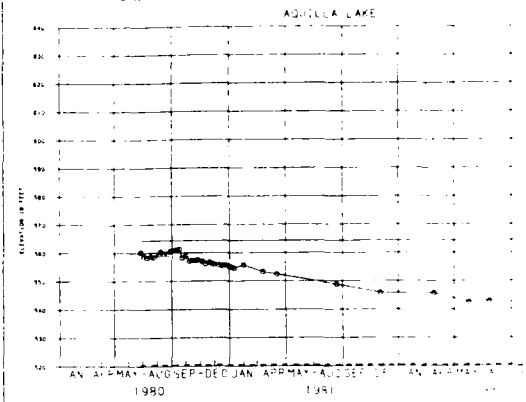
INSTALLATION DATA

NO. NAME DATE STATION OFFSET WATER ELEVATION
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 1000 1000 1000 1000 1000 1000
 1000 1000 1000 1000 1000 1000



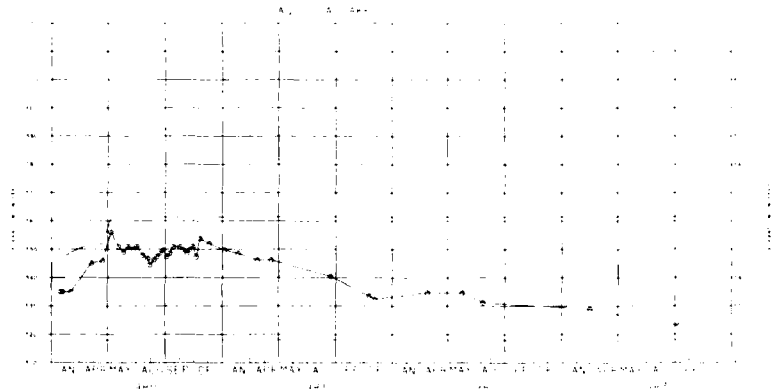
ALL DATA ARE FROM
 1000 FEET ABOVE
 1000

NOTE: INITIAL DECLINING SLOPE DUE TO EXCESS
 WATER IN BOREHOLE



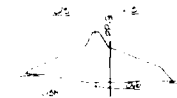
INSTALLATION DATA

NO. NAME DATE STATION OFFSET WATER ELEVATION
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 1000 1000 1000 1000 1000 1000

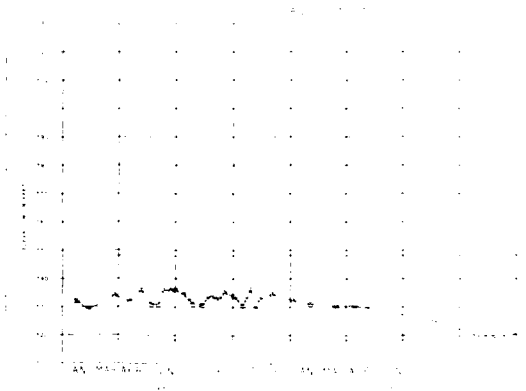


INSTALLATION DATA

NO. NAME DATE STATION OFFSET WATER ELEVATION
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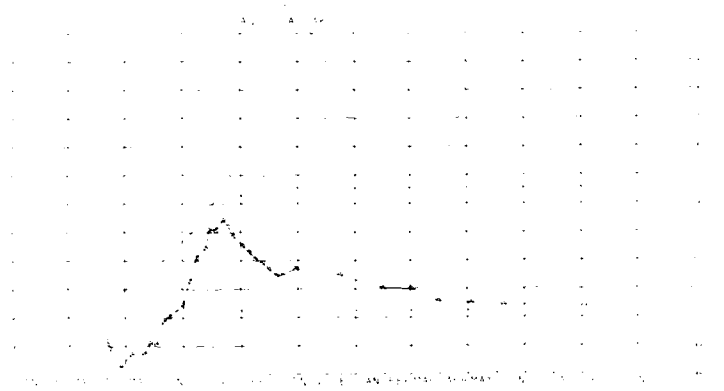
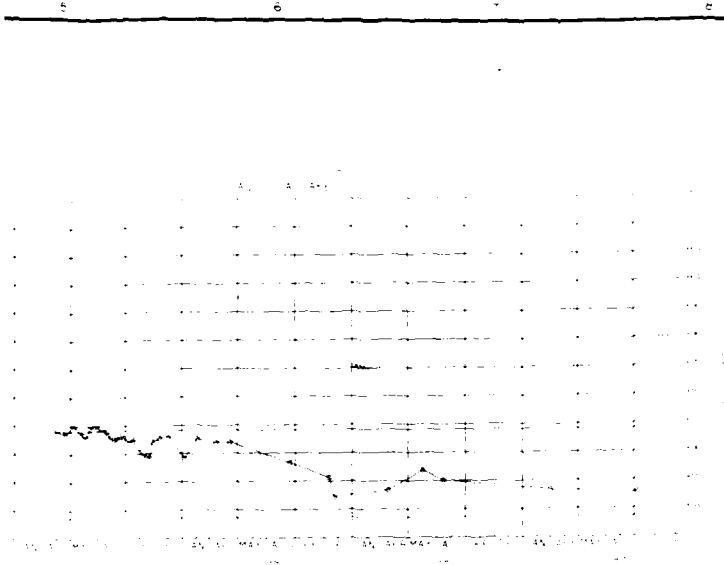
ALL DATA ARE FROM
 1000 FEET ABOVE
 1000



INSTALLATION DATA

NO. NAME DATE STATION OFFSET WATER ELEVATION
 1000 1000 1000 1000 1000 1000
 1000 1000 1000 1000 1000 1000
 1000 1000 1000 1000 1000 1000





NOTE: PIER TOP ELEVATION 4 FOR BEGINNING OF READING.

- LEGEND
- Q - LIVE & DEAD
 - CL - CLAY SHALE
 - SL - GRAVEL
 - SD - SAND
 - DO - SANDSTONE
 - C - CLAY
 - LS - LIMESTONE
 - DR - DEL RIO

U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS	
AQUILLA LAKE PIEZOMETERS PR-4, PR-15, PR-16, PR-25 PIEZOMETER AND FILL ELEVATION VS TIME	
DESIGNED BY DRAWN BY CHECKED BY SUBMITTED BY E. KAREB	DATE DRAWING NO. SHEET NO. SEQUENCE NO.

AD-A168 214

EMBANKMENT CRITERIA AND PERFORMANCE REPORT AQUILLA LAKE
AQUILLA CREEK TEXAS BRAZOS RIVER BASIN (U) ARMY ENGINEER
DISTRICT FORT WORTH TX DEC 85

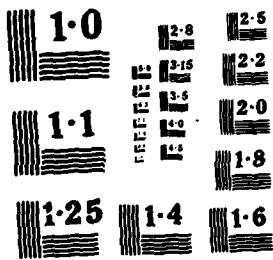
3/9

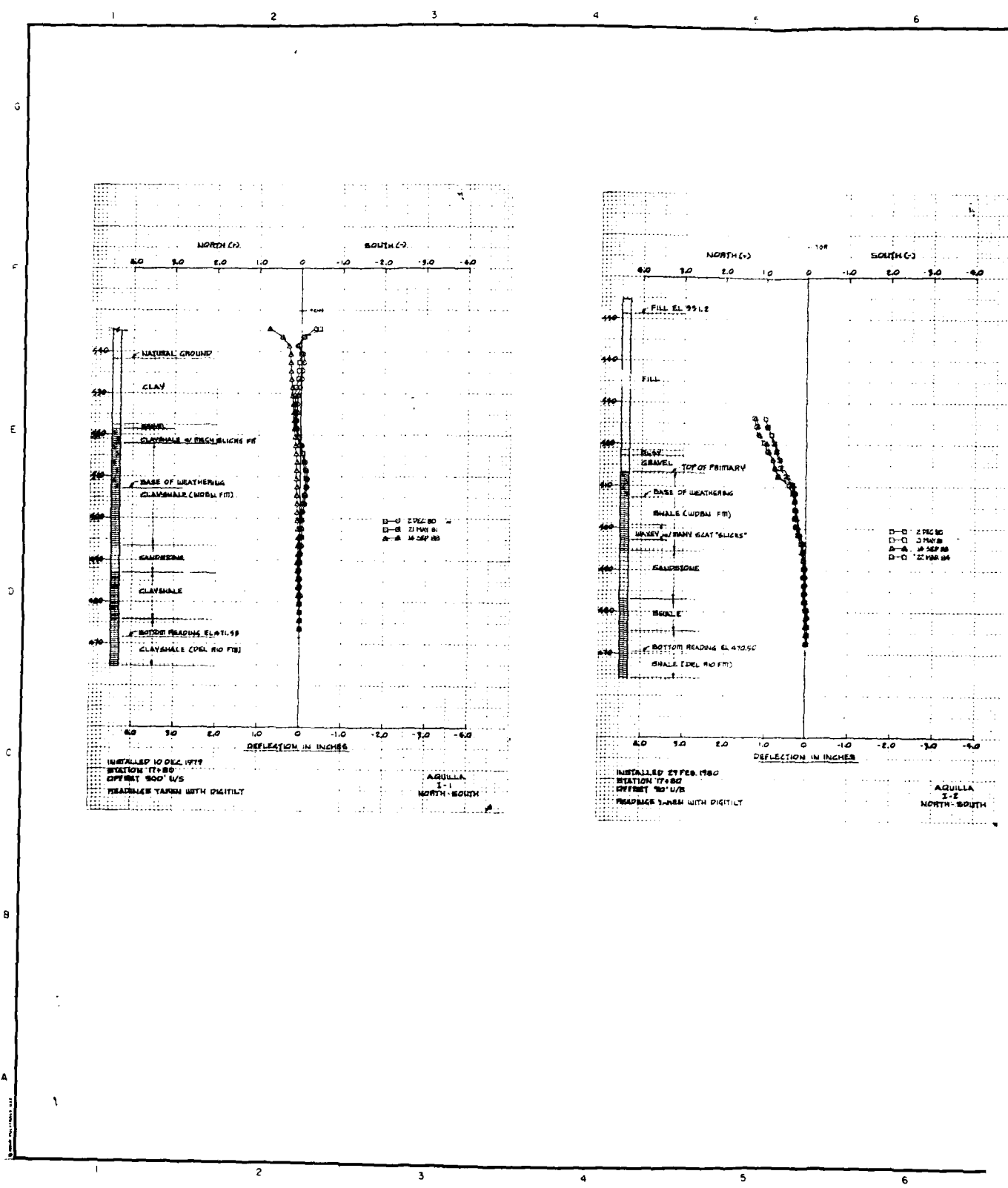
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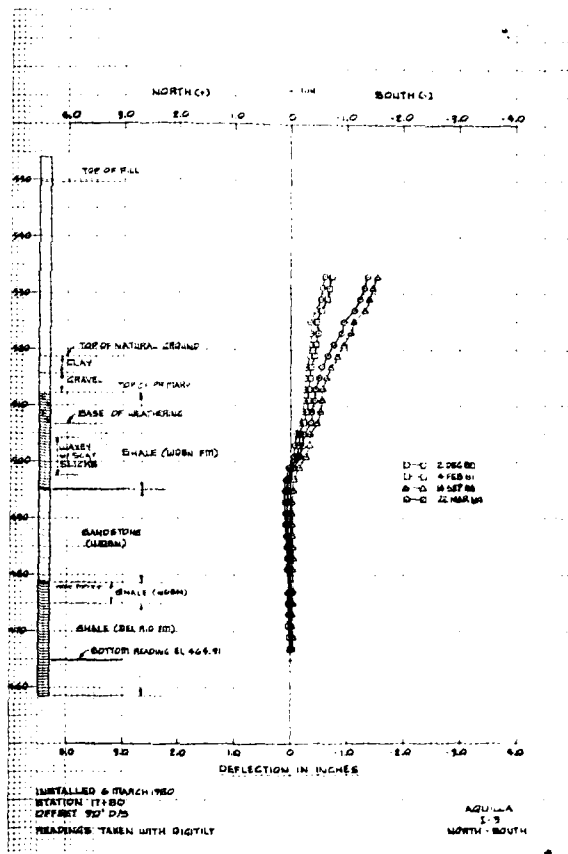
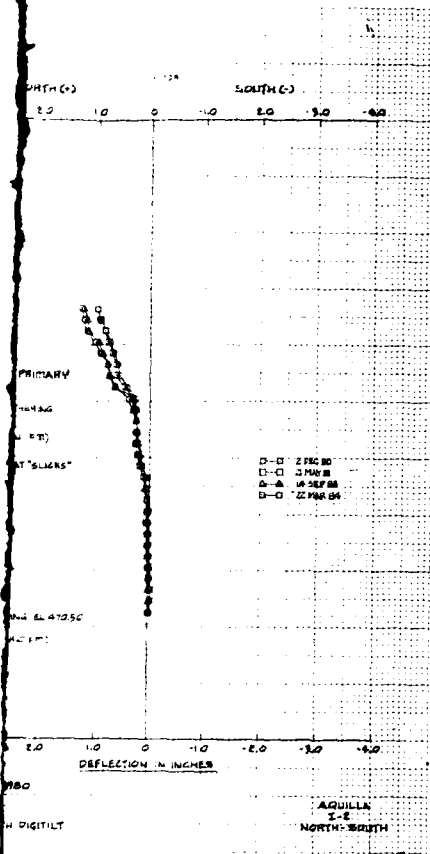
F/G 13/2

NL



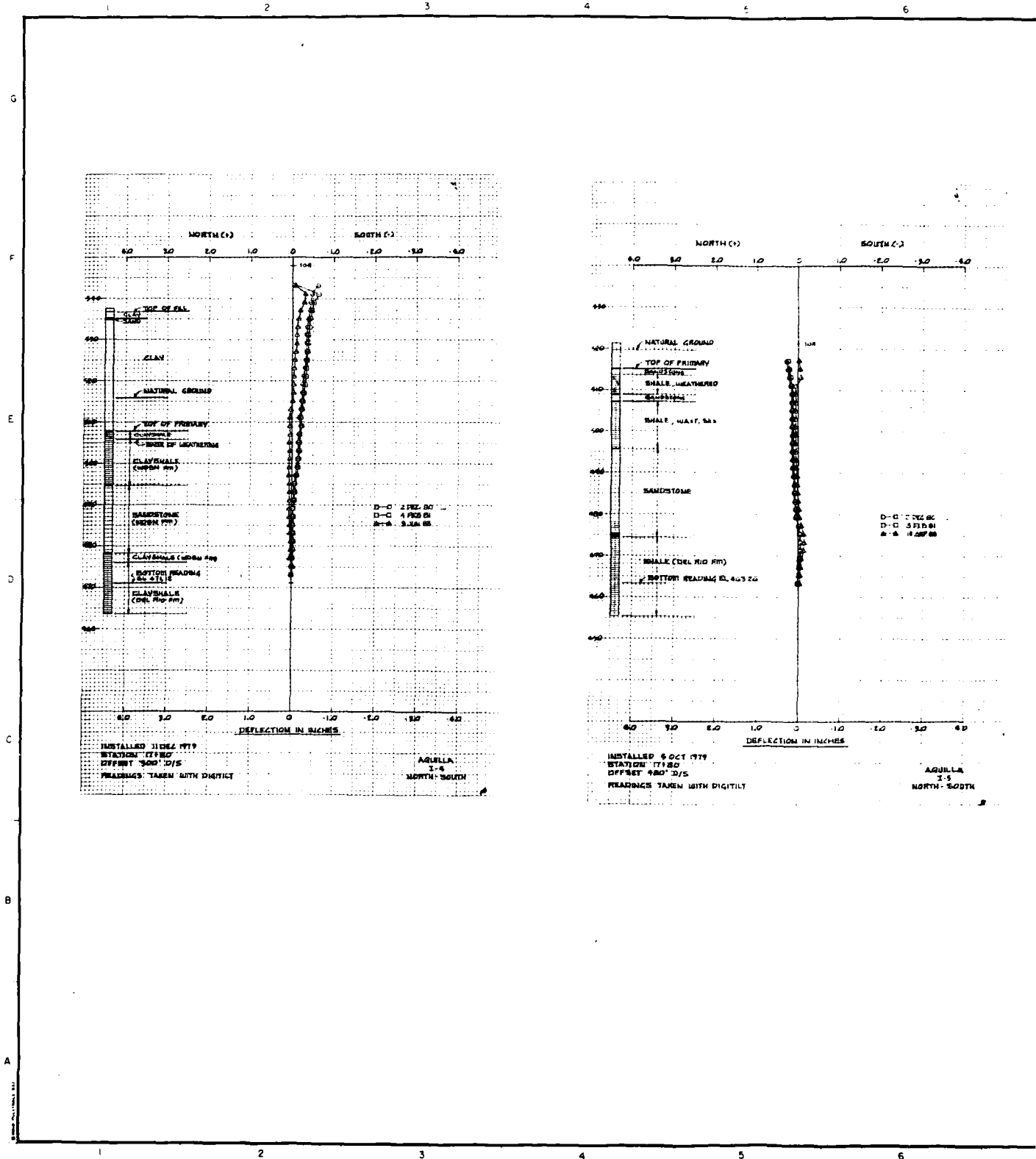


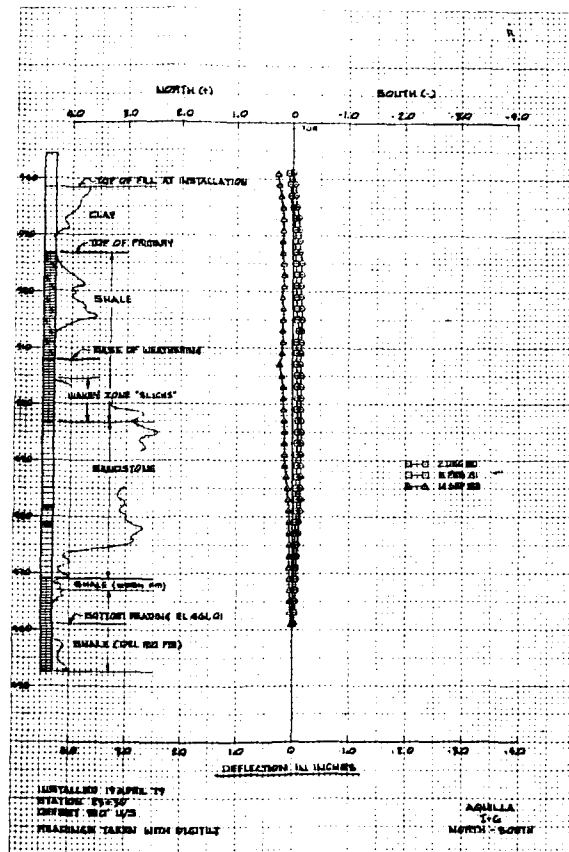
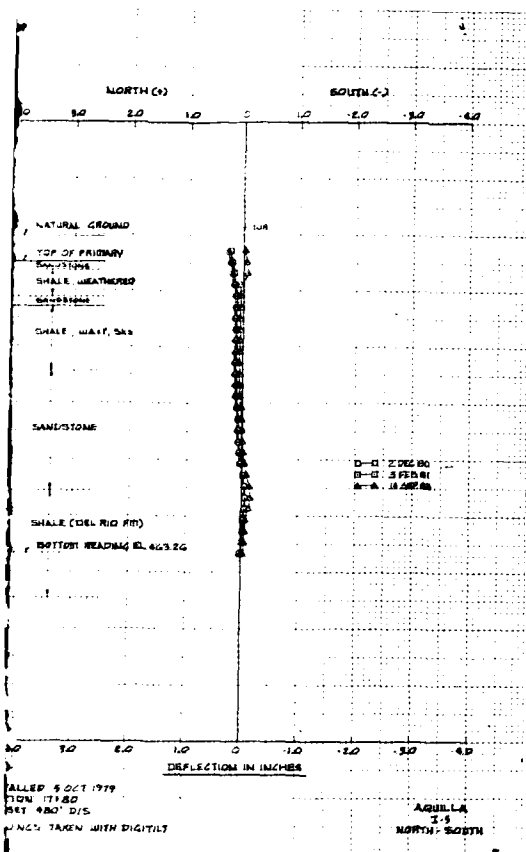




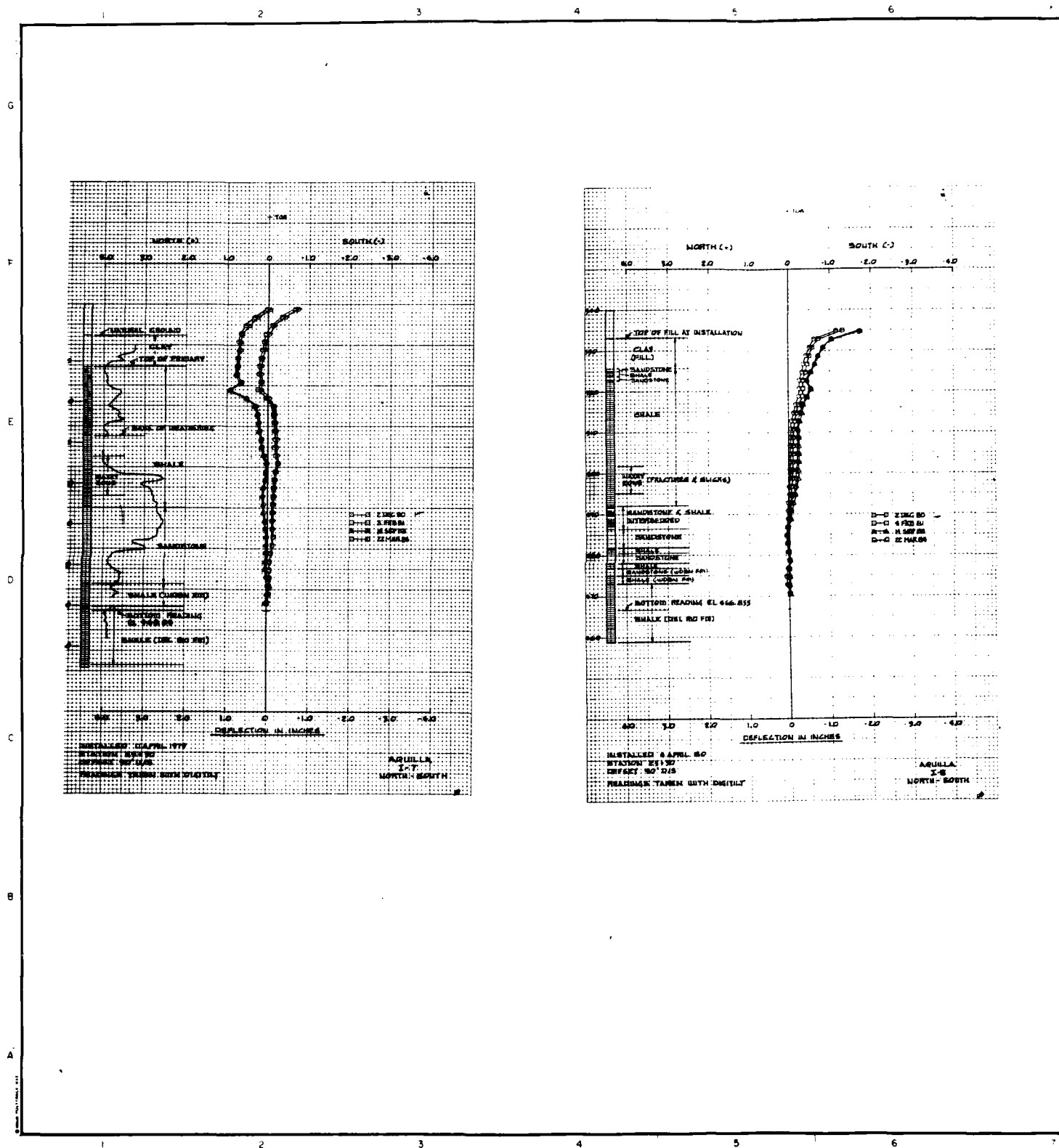
DESIGNED BY J. H. HARRIS		CHECKED BY J. H. HARRIS	
DRAWN BY J. H. HARRIS		DATE 10/1/50	
U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS			
AQUILLA LAKE AQUILLA CREEK, TEXAS			
INCLINOMETER NO. I-1, I-2, I-3			
SUBMITTED BY J. H. HARRIS		DATE 10/1/50	
CONTR. NO. 11-80		SEQUENCE NO. 1	
DRAWING NUMBER 11-80		SHEET NO. 1	

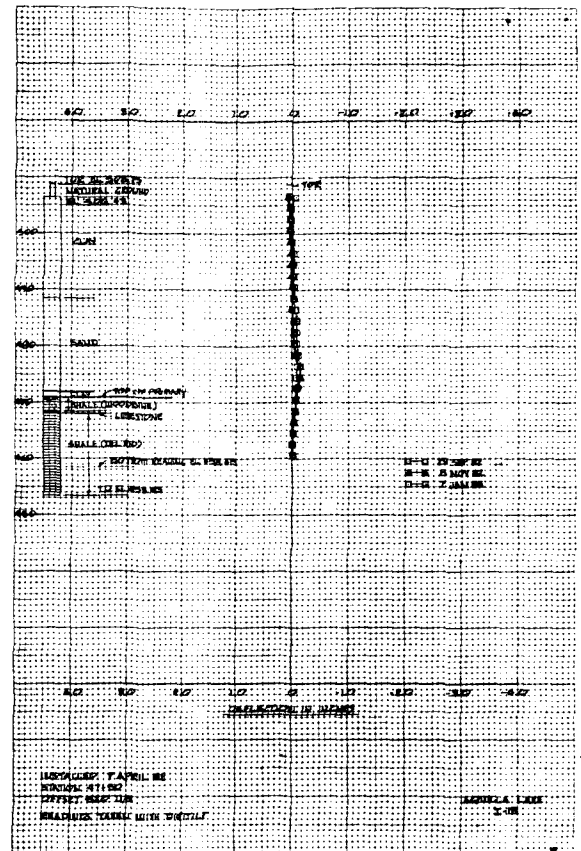
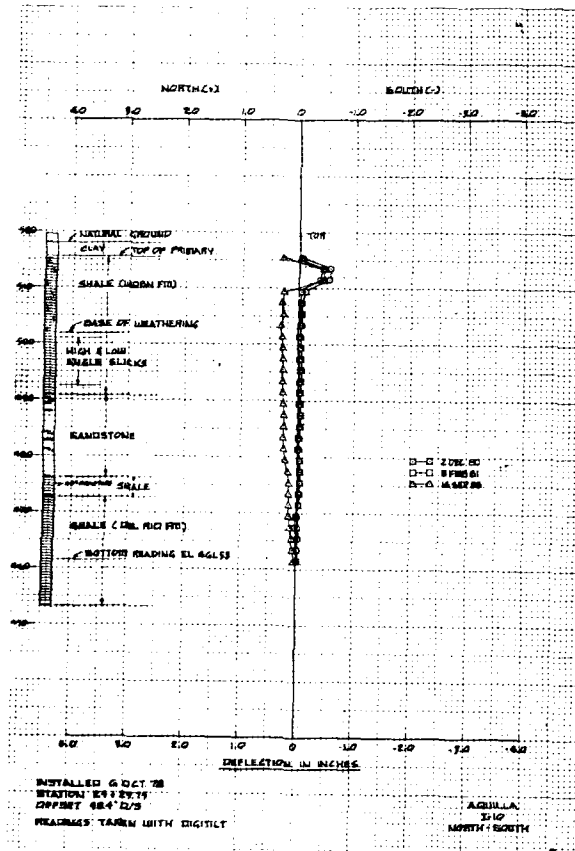
PLATE 57

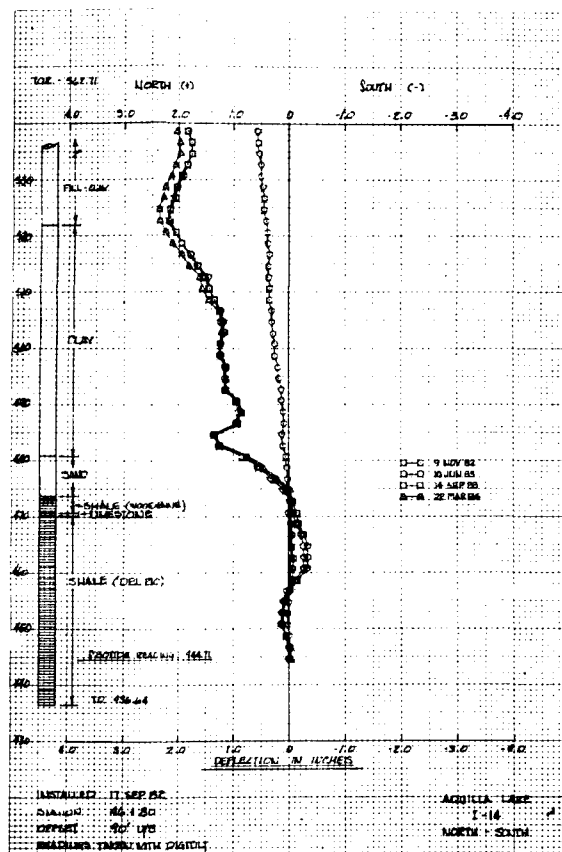
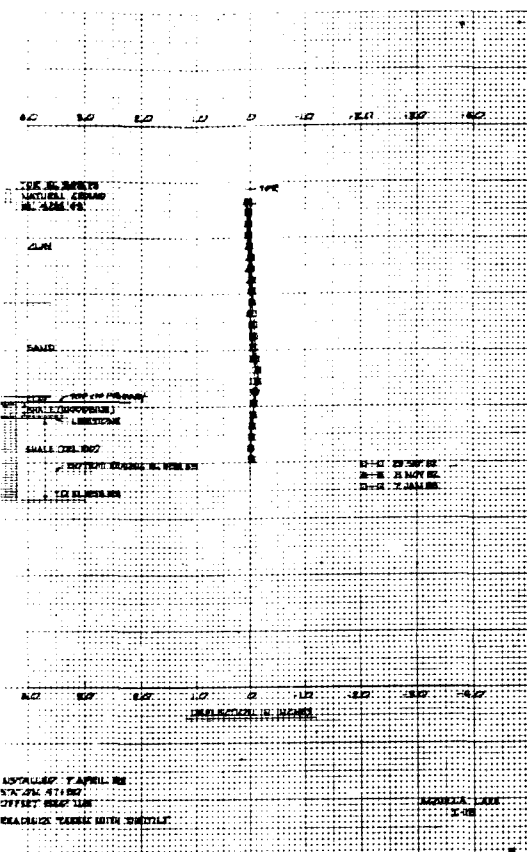




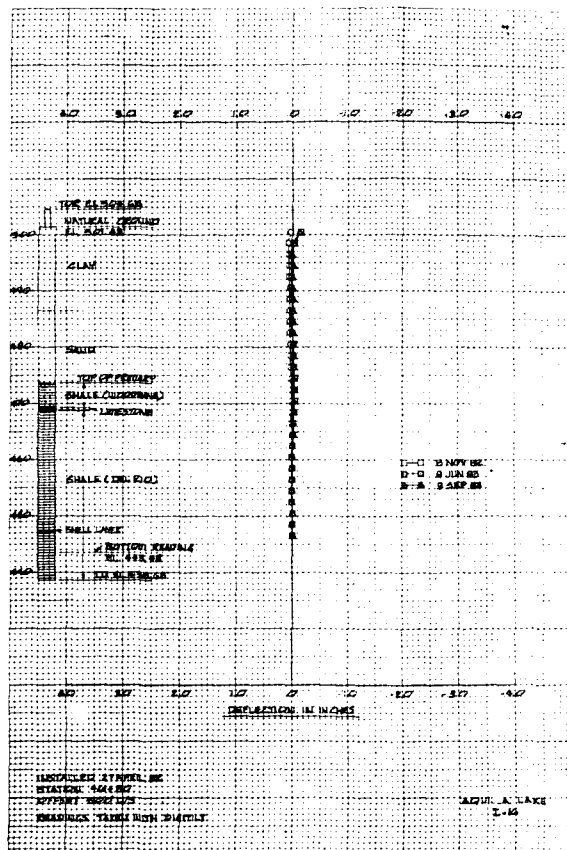
DRAWN BY H. KADIN		CHECKED BY H. KADIN	
DESIGNED BY H. KADIN		U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS	
SUBMITTED BY H. KADIN		AQUILLA LAKE AQUILLA CREEK TEXAS HILLINGMETH RS I-4, I-5, I-6	
ENGINEER		INV NO.	DATED
CONTR NO.		SHEET NO.	REVISION NO.
DRAWING NUMBER		DATE 1979	





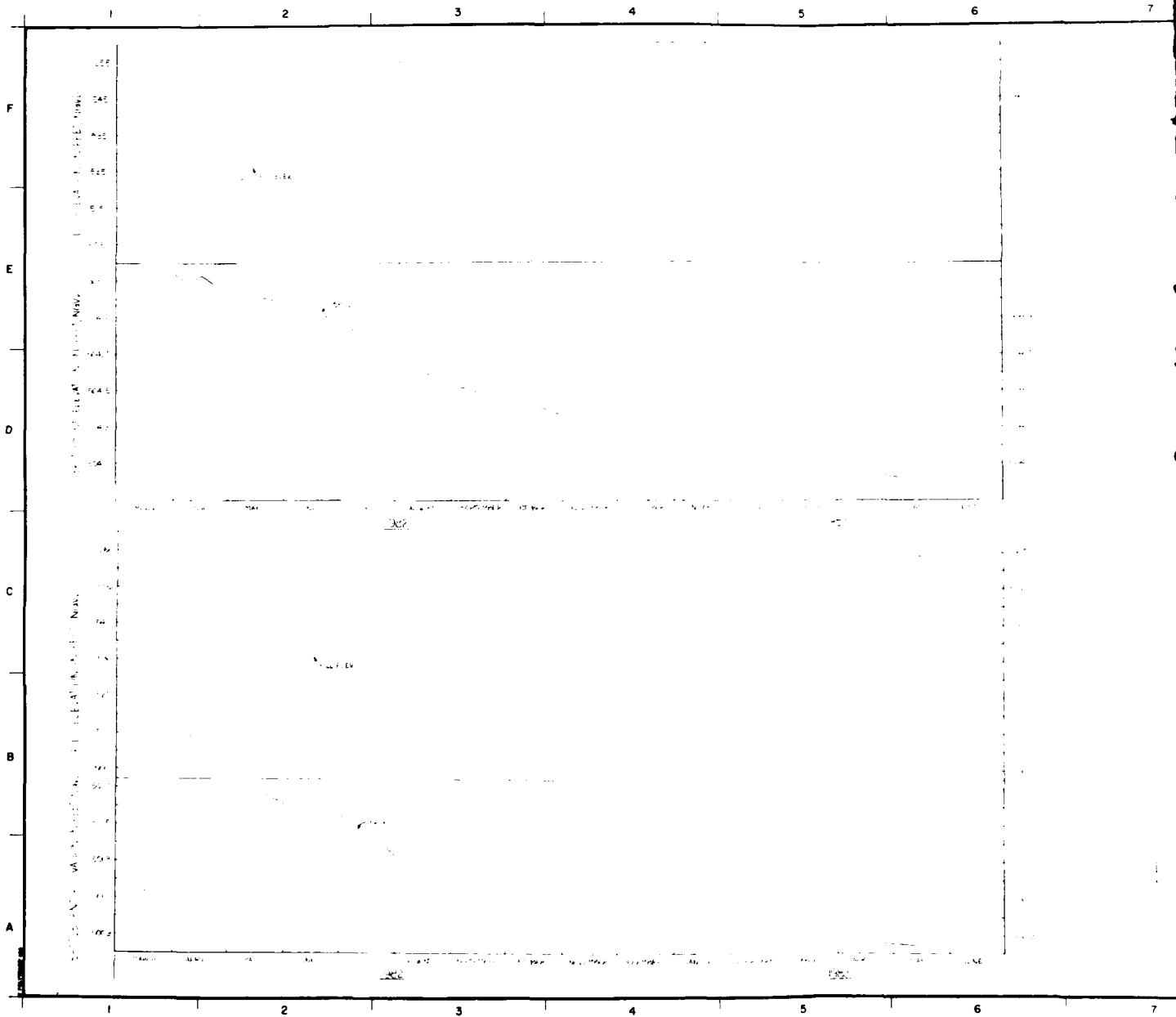


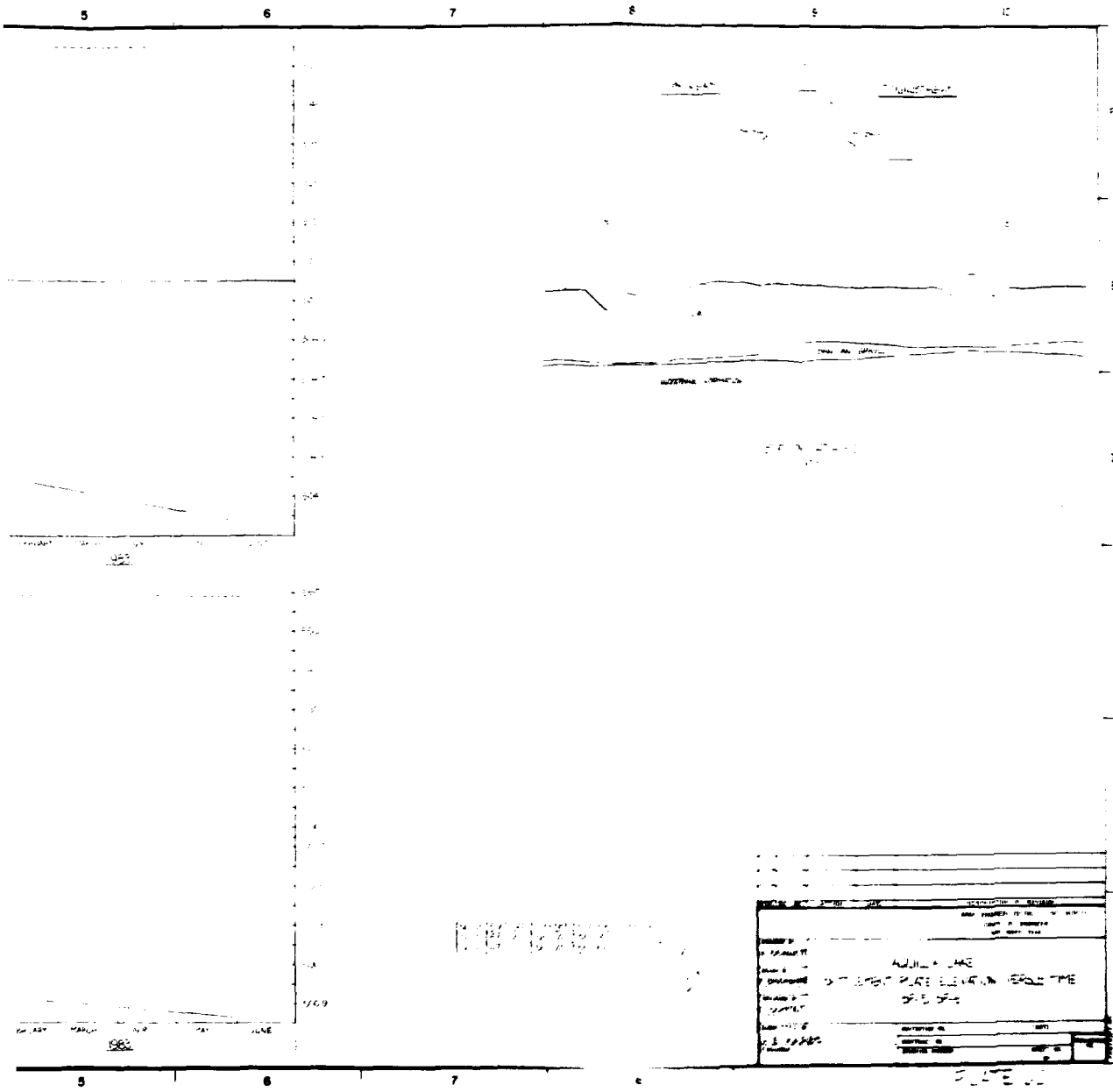
DESIGNED BY W. BERNET P. G. HARRIS		APPROVED BY J. B. G. HARRIS	
DRAWN BY J. B. G. HARRIS		CHECKED BY T. G. HARRIS	
SUBMITTED BY W. E. HARRIS		DATE 1-10-13	
COUNTY NO. DRAWING NUMBER		SHEET NO. OF	
PROJECT NO. 1-10-13		SEQUENCE NO. 1	



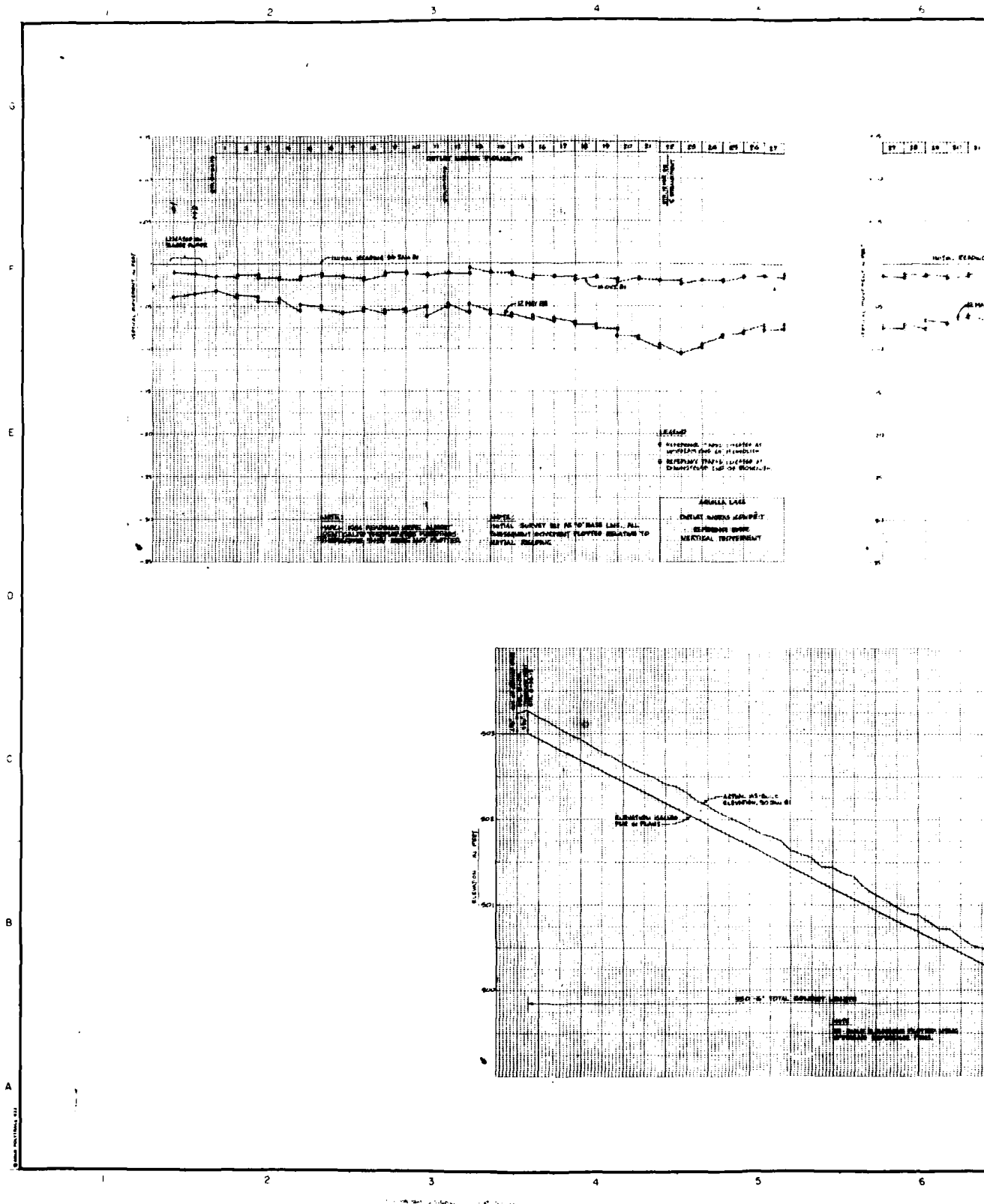
NO. 100-50000-10100		DATE		SHEET NO. OF 10000	
U. S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS					
DESIGNED BY W. H. HARRIS DRAWN BY W. H. HARRIS CHECKED BY W. H. HARRIS APPROVED BY W. H. HARRIS					
AQUILLA LAKE AQUILLA CREEK TEXAS HIGLINGS METERS I-5, I-16					
SUBMITTED BY		NO. 100		DATED	
W. H. HARRIS		COUNT NO.		SEQUENCE NO.	
ENGINEER		DRAWING NUMBER		SHEET NO. OF	

PLATE 21





SECTION 10, T. 10 N., R. 10 E., S. 10 E.	
COUNTY OF SHERBORN, MASS.	
TOWN OF SHERBORN, MASS.	
ADJUTANT GENERAL	
EXTENDING PLATE ELEVATION FROM THE	
OF 100	
DATE 1910	
BY 100	
100	



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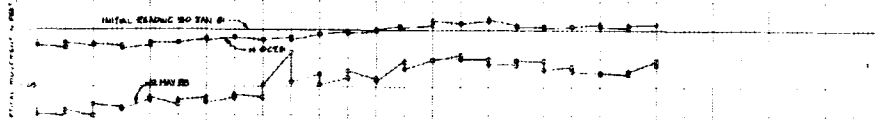
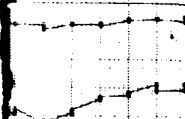
D

C

B

27 28 29 30 31 32

27 28 29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48



LEGEND
 * REFERENCE MARK LOCATED AT
 UPSTREAM END OF CONDUIT
 * REFERENCE MARK LOCATED AT
 DOWNSTREAM END OF CONDUIT

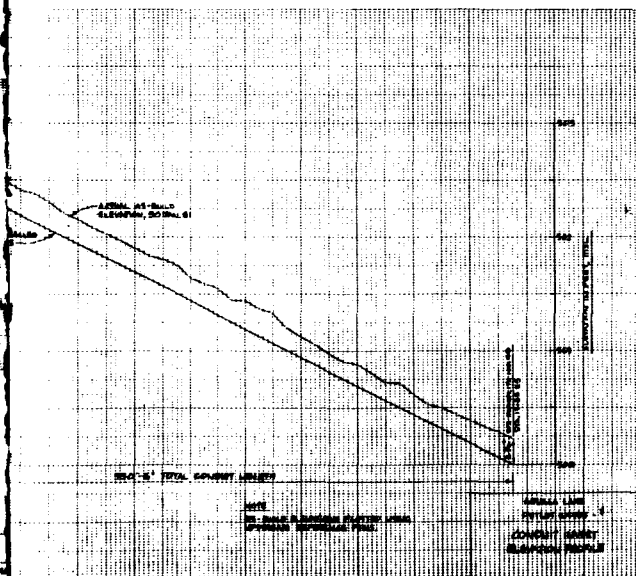
NOTE
 * REFERENCE MARK LOCATED AT
 UPSTREAM END OF CONDUIT
 * REFERENCE MARK LOCATED AT
 DOWNSTREAM END OF CONDUIT

AQUILA LAKE
 OUTLET WORKS CONDUIT
 REFERENCE MARKS
 VERTICAL MOVEMENT

NOTE
 MARKS FOR REMOVED MARKS ALREADY
 LOCATED AT THE END OF CONDUIT
 THEREFORE, THEY WERE NOT PLOTTED

NOTE
 INITIAL SURVEY SET AT 10' BASE LINE. ALL
 SUBSEQUENT RECONNAISSANCE PLANNING RELATIVE TO
 INITIAL SURVEY

AQUILA LAKE
 OUTLET WORKS CONDUIT
 REFERENCE MARKS
 VERTICAL MOVEMENT



DESIGNED BY N. BENNETT D. GOSWOLD		DRAWN BY K. BAZZANIRE		CHECKED BY T. SCHMIDT	
SUBMITTED BY H. E. KAMM		CONTRACT NO. DRAWING NUMBER		SHEET NO. OF	
DATE		DATE		SEQUENCE NO.	
U.S. ARMY ENGINEER DISTRICT, FORT WORTH CORPS OF ENGINEERS FORT WORTH, TEXAS AQUILA LAKE AQUILA CREEK, TEXAS OUTLET WORKS CONDUIT REFERENCE MARKS AND ELEVATION PROFILE					

PLATE 63



Photo 1 July 1983
Aquilla Dam, looking west along upstream side of embankment.



Photo 2
Completed spillway, looking upstream

December 1982



Photo 3
Outlet works looking upstream

December 1983

Aquilla Dam
Embankment Criteria and Performance Report

EXHIBIT



Photo 4
Mucking-out operation at Aquilla Creek channel

January 1982



Photo 5
Foundation preparation and backfilling in Aquilla Creek channel

February 1982

Aquilla Dam
Embankment Criteria and Performance Report

EXHIBIT 3



Photo 6 February 1982
Backfilling in Aquilla Creek area looking downstream



Photo 7 March 1980
Fill placement operations at right abutment of Aquilla Dam

Aquilla Dam
Embankment Criteria and Performance Report

EXHIBIT 4

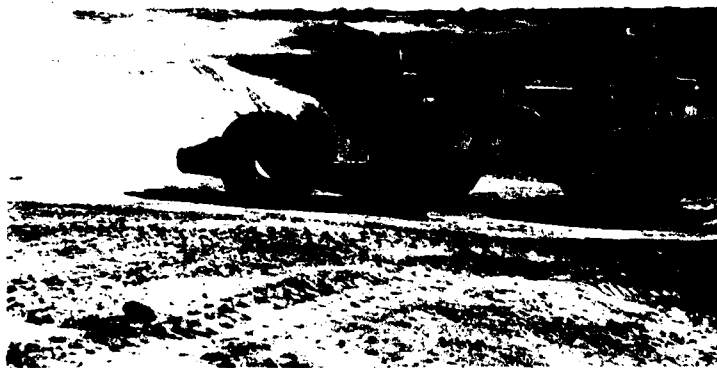


Photo 8 February 1982
Fill placement operations at right floodplain abutment of
Aquilla Creek.



Photo 9 November 1981
Fill placement in random and semi-compacted zones for Aquilla
Creek floodplain embankment.



Photo 10
Fill placement operations in closure area

April 1982



Photo 11
Sheepfoot rollers processing material in random fill zone
of dam

August 1981



Photo 12
Pre-watering in borrow area

September 1981



Photo 13
Loading of material removed from spillway excavations

August 1981

Aquilla Dam
Embankment Criteria and Performance Report

EXHIBIT 7

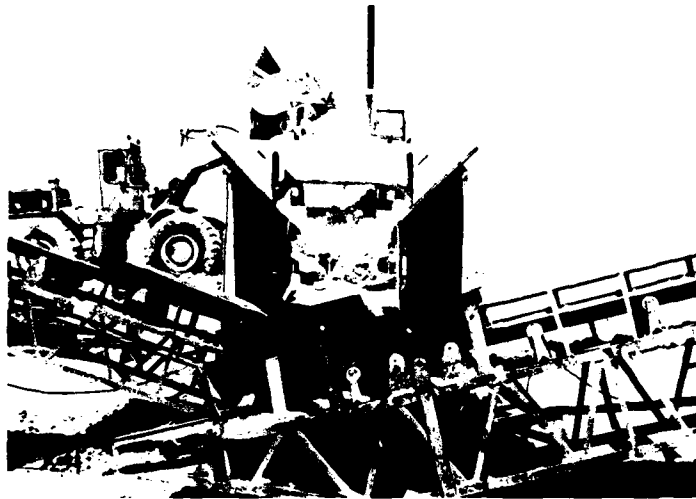


Photo 14
Processing of 12" stone protection materials

March 1982



Photo 15
Sand cone density test in inspection trench

August 1981



Photo 16 April 1983
Upstream slope protection on initial embankment



Photo 17
Display of materials for an open system piezometer installation.
Notice the 2-foot long porous plastic tip attached to the 3/8-inch
diameter PVC riser pipe, the bag of filter sand, and bentonite
pellets for the seal.

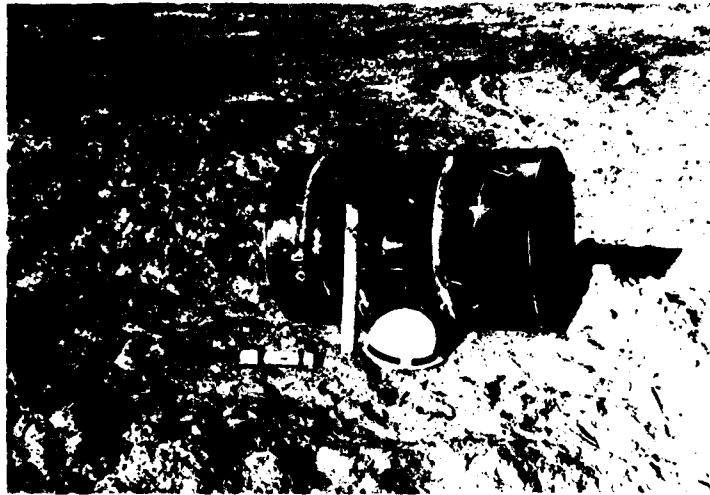


Photo 18

View of an open system piezometer after installation. Notice the inner 3/8-inch PVC riser pipe with vented cap and the outer 1 1/2-inch diameter steel protective pipe.



Photo 19

View of piezometer showing temporary instrumentation protection, consisting of an earth mound and painted barrel.



Photo 20

View of an open system piezometer being read by probing with an electrical water level indicator.



Photo 21

View showing a pneumatic piezometer transducer being maintained in a saturated condition with de-aired water prior to installation.



Photo 22

View of pneumatic piezometer transducer prior to being lowered into drill hole. The white portion is the high air entry filter.

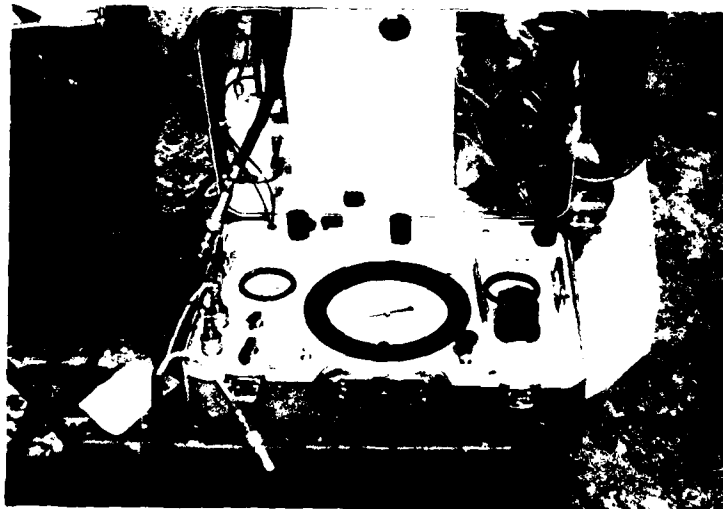


Photo 23

Portable pneumatic piezometer reader. Note the 4 tube leads encased in protective plastic that go from transducer in bottom of hole to reader at surface.



Photo 24

View showing a phase of the inclinometer installation. Air hose shown is connected to vibrator used to densify sand backfill around the blue PVC inclinometer casing shown in the photo.



Photo 25

Preparation to read an inclinometer. The sensor probe that attaches to the cable is not shown. Note protective fencing and protection steel casing.

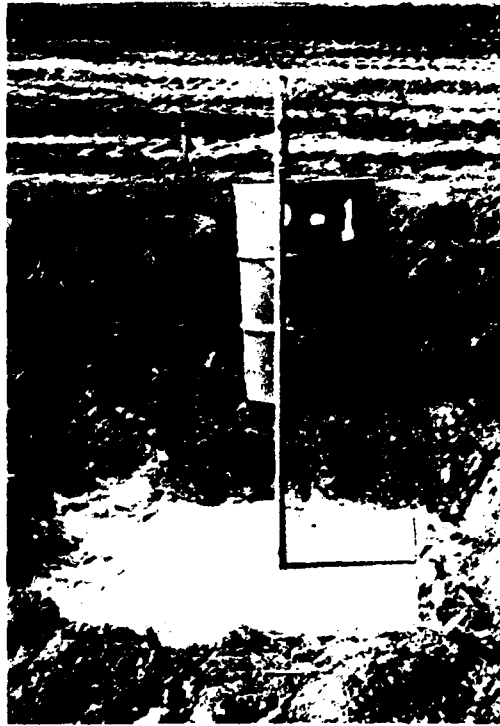


Photo 26

Settlement plate on prepared embankment foundation. Note the 3'x3' steel base plate and 1-inch diameter galvanized steel riser pipe. Not shown is the outer 2-inch diameter galvanized steel protective pipe.

Aquilla Dam
Embankment Criteria and Performance Report

EXHIBIT 14



Photo 27

View of cone-tipped rod and outer protective pipe used to construct the deep bench marks for the completion contract.



Photo 28

View of deep bench mark after installation of cone-tipped rod and outer protective pipe. Elevations are established by measuring top of inner rod which does not move.

END

DATE
FILMED

7-86