US Army Corps
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GRAND PRAIRIE REGION AND BAYOU METO BASIN, ARKANSAS PROJECT

BAYOU METO BASIN, ARKANSAS

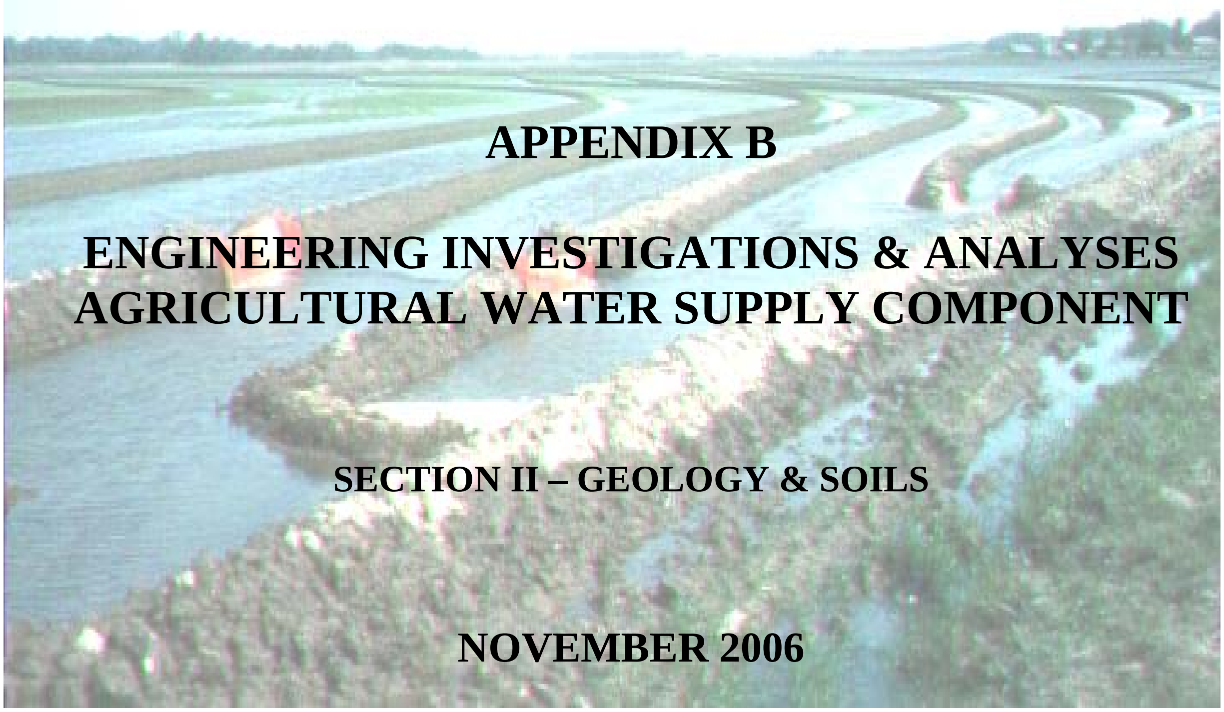
GENERAL REEVALUATION REPORT

VOLUME 4

APPENDIX B

ENGINEERING INVESTIGATIONS & ANALYSES AGRICULTURAL WATER SUPPLY COMPONENT

SECTION II – GEOLOGY & SOILS



NOVEMBER 2006

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Report Prepared By:

Cory Williams
Geotechnical Engineering Branch
Memphis District
June 2002

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SECTION II - GEOLOGY AND SOILS

2.01 GENERAL

This report includes the Geotechnical analyses related to the irrigation portion of the Bayou Meto Comprehensive Study. The project is located in the Bayou Meto Basin in the vicinity of Lonoke, Arkansas. The project consists of a large water distribution system to provide supplemental agricultural water supply to the depleted areas of the Bayou Meto Basin while preserving the groundwater resource. The project area includes portions of Prairie, Arkansas, Lonoke, Jefferson and Pulaski counties in eastern Arkansas. The general area begins about 10 miles east of Little Rock, Arkansas and extends approximately 60 miles to the southeast. See Plate II-1 for a general location map. The plan of improvement for the Bayou Meto area consists of a major pumping station located at the Arkansas River, 3 other pumping stations and a network of new canals, existing channels, pipelines, and associated channel structures to accomplish interbasin transfer of surface water to the water depleted areas. This report will address the major features of the project: a 1750 cubic feet per second (cfs) pumping station (Pumping Station #1), a 625 cubic feet per second (cfs) pumping station (Pumping Station #2), a 260 cubic feet per second (cfs) pumping station (Pumping Station #3), a 125 cubic feet per second (cfs) pumping station (Pumping Station #4), 12 gated control structures located in the main distribution canals, 3 pumping station reservoirs, general stability of the canals and levee embankment system and the estimated seepage loss throughout the network of the water distribution system. There are a significant number of other smaller structures or features that will not be addressed in this report due to the magnitude of this project. These features include gated conduit structures, gated and lateral canal turnouts, bridges, retaining walls, inverted box siphons, inverted siphons (natural drainage), inverted pipe siphons (road crossings and natural drainage), wasteways, small horsepower pumps, and weirs on existing streams. More detail on these structures can be found in Section IV - Structural, Mechanical & Electrical. A location map of the project limits, new canal alignment, some pipeline alignment are shown on Plate II-6. Corps of Engineers design criteria and standards were used for the Geotechnical design presented in this study.

The major pumping station (Pumping Station #1) will be located on the Arkansas River southeast of Little Rock, Arkansas in Pulaski County. Water will be provided from the Arkansas River to the pumping station by an approximate 4800-foot long inlet channel with a 100-foot bottom width that slopes from El. 220.0 feet NGVD at the Arkansas River confluence to an inlet El. of 217.0 feet NGVD at the station. The pumping station consists of six pumps capable of producing a total of 1750 cfs of flow to the distribution canals. Four of the pumps are rated at 375 cfs with two smaller pumps rated at 125 cfs. These pumps will discharge into two 4-foot inside diameter (ID) and four 6.5-foot ID discharge pipes that will supply water to the 35 acre reservoir. The canal system then will be supplied by a control structure located in the reservoir embankment. Other features of the major pumping station include inlet retaining walls, an earthen access road, earthen reservoir embankment, an earthen cofferdam for protection during construction and a dewatering system to lower the ground water during construction. A final site plan for the major pumping station #1 is shown on Plate II-23.

Pumping station #2 is a smaller 625 cfs lift station that is located approximately 2 miles south of Lonoke, Arkansas. This station will pump water from canal 2000 across the existing Bayou Meto channel into a second reservoir located on the north side of stream. Water will be transported across the channel through a system of five elevated 4-foot diameter pipes supported by concrete pilings. Reservoir #2 will supply water for the canals and pipelines with the 2500 and 3000 series designation located north and east of pumping station #2 as shown on Plate II-6. The station will lift the water around 17 feet in elevation utilizing five pumps rated at 125 cfs each. Other features include an outlet structure, five 48-inch diameter concrete discharge pipes, a pile supported bridge structure to support the discharge pipes, fill placed around the station, earthen reservoir embankments, and a reservoir control structure. A general site plan of Pumping station #2 is shown on Plate II-37 with sections through the station shown on Plates II-38 & II-39.

Pumping station #3 is a smaller 260 cfs lift station that is located approximately 3 miles north of Lonoke, Arkansas. This station will pump water from canal 3000 into a third reservoir located just east of the pumping station. Reservoir #3 will supply water for the canals and pipelines with the 4000 series designation located to the north and east of pumping station #3 as shown on Plate II-6. The station will lift the water around 17 feet in elevation utilizing three pumps rated at 87 cfs each. Other features include an inlet structure, inlet retaining structure, three 60-inch diameter concrete inlet pipes, fill placed around the station, earthen reservoir embankments, and a reservoir control structure. A general site plan of Pumping station #3 is shown on Plate II-47 with sections through the station shown on Plates II-48 and II-49.

Pumping station #4 is a smaller 125 cfs lift station that is located approximately 23 miles southeast of Lonoke, Arkansas. This station will pump water from canal 2120 to canal 2140. No reservoir will be constructed in association with the station. Pumping Station #4 will supply water for the canals and pipelines with the 2140 and 2160 series designation located to the south of the station as shown on Plate II-6. The station will lift the water around 8 feet in elevation utilizing three pumps rated at 42 cfs each. Other features include an inlet structure, inlet retaining structure, three 42-inch diameter concrete inlet pipes and fill placed around the station. A general site plan of Pumping station #4 is shown on Plate II-60 with sections through the station shown on Plates II-61 and II-62.

There are 12 major gated control structures located in the main distribution canals. Overflow weirs are located on each side of the gated structures that will limit the maximum upstream height of the water surface.

2.02 GEOLOGY

a. General Geology. This project consists of two distinctive geological areas located within the project boundaries. The first geological area encompasses the alluvial floodplain of the Arkansas River where the majority of the irrigation project is located. The other geological area is the Grand Prairie region located in the uplands northeast of the town of Lonoke. Each of these geological areas is described in detail in the following paragraphs.

b. Geology of the Grand Prairie Region. The Grand Prairie region of Arkansas is in a subdivision of the Coastal Plain province known as the Mississippi Alluvial Plain. Geological maps indicate deposits of Quaternary age are continuous throughout the northeast areas of the project limits. The Quaternary deposits are composed of sediments from the Pleistocene and Recent series. No differentiation has been made between the Pleistocene and Recent deposits. Two major zones have been identified within the Quaternary deposits. They are a substratum zone of sand or sand and gravel and an overlying zone of silt and clay. Particle sizes of the substratum sands vary from very fine to coarse size. The substratum sands are also interbedded with thin clay and silt lenses. Within the Grand Prairie region the substratum sands generally vary in thickness from 25 to 140 feet and are underlain by Tertiary deposits. Geological maps indicate the Tertiary deposits to be of the Jackson or Claiborne Group. Review of selected drillers' logs for deep wells indicates the top of the Tertiary deposits within the Grand Prairie region to be composed of a dark-bluish-gray clay. Tertiary deposits were generally encountered between Elevation 50 and Elevation 100 feet NGVD.

The surface deposits, which is the upper zone identified within the Quaternary age deposits, are composed of very dense and relatively impervious layers of clays and silts. These deposits are also referred to as the Loessial Plains. The thickness of the surface deposits ranges from 5 to 60 feet throughout the study area. These deposits are also noted in geological literature for being remarkably continuous and very nearly impervious over much of the Grand Prairie region. See Plates II-2, II-3 and II-4 for geologic maps of the general area and of the individual pumping station sites.

c. Geology of the Arkansas River Alluvial Floodplain. Based on a review of geological maps, the project is located predominately on point bar deposits of the Mississippi-Ohio River complex. The alluvial deposits resulting from the meandering process tend to be highly variable in short distances in both areal extent and with depth. The more predominate forms of deposition within the study are point bar, backswamp and abandoned channels. Point bar deposits tend to consist of alternating sand silt and clay, whereas the backswamp deposits formed by the frequent inundation of the area during several periods are generally composed of finer grained materials. The abandoned channels are old river courses and are generally composed of highly plastic clays that can extend to a depth of 100 feet. Also, the point bar deposits frequently contain clay swales, which are similar in composition to the abandoned channel deposits. However, two primary differences between the deposits are that the swales tend to be less plastic due to the presence of more silty and sandy clays and they do not extend to a depth of the abandoned channel deposits. The surface deposits are underlain by substratum sands which range from a fine through a gravely sand. The Undifferentiated Tertiary deposits underlie the substratum sands. See Plates II-2, II-3 and II-4 for geologic maps of the area and of the individual pumping station sites.

2.03 SUBSURFACE INVESTIGATION.

a. General. The level of the subsurface investigation performed for this report was selected in an attempt to provide an accurate yet cost efficient characterization of the subsurface. The investigation consisted of taking both soil borings and cone penetrometer testing. The soil borings were obtained using standard drilling equipment, methods and procedures. A total of 66 borings were taken. Most of these borings were taken at a major canal structure and/or canal junction locations. Cone penetrometer tests were performed at road crossings near existing streams. An additional 7 shallow borings (10 to 18.5 foot depth range) were also taken for this project for determining permeability of different soil types or strata. These holes were drilled at previous boring locations.

b. Borings. The undisturbed borings were advanced and sampled with a 5-inch auger, 5-inch thin wall tubes and split spoon samplers. The general borings were advanced and sampled with a 5-inch auger and split spoon samplers. In general, the borings indicated typical stratification expected after review of the geological maps. The boring locations are shown on Plates II-6, II-7A, II-7B and II-7C. The boring logs are shown on Plate II-8 through II-18. The standard boring legend for the Unified Soil Classification System is presented as Plate II-19.

c. Cone Penetrometer Tests. Cone penetrometer tests were taken throughout the Bayou Meto Basin to supplement soil-boring information. A total of 317 cone penetrometer tests were taken at road crossings near existing streams throughout the Bayou Meto Basin. Elevations of the cone penetrometer locations were estimated from surface contours shown on topographic maps. The cone penetrometer test locations are also presented on Plates II-6, II-7A, II-7B and II-7C.

Cone penetrometer tests were performed by pushing a 60° cone tip into the ground using drill rods. Electronic sensing units were mounted on the cone to measure the resistance at the tip of the cone and the frictional resistance on the drill rods above the tip. The ratio between the tip and frictional resistance measurements was generally correlated to the soil type and consistency. Empirical correlations were used to relate cone resistance data to N-values. Because the tip and friction resistance data was measured directly as the cone was advanced, soil shear strength data was recorded in small increments as a function of depth. It is noted that the soil classifications indicated on the logs were based on empirical relationships from penetrometer tip and frictional resistance ratios. These classifications should be considered an estimate and may vary from what is encountered in the field. These cone penetrometer logs are on file in the Geotechnical Engineering Branch in Memphis, TN.

2.04 LABORATORY TESTING.

The Engineer Research and Development Center (ERDC) in Vicksburg, Mississippi (formerly Waterways Experiment Station - WES) and the Memphis District laboratories performed laboratory testing of the boring samples. Four Q-triaxial compression tests, 11 direct shear tests and 4 consolidation tests were performed by the ERDC. Classification, natural moisture contents, natural densities, and Atterberg limits were performed on representative samples by both laboratories. Unconfined compression tests and mechanical analyses on coarse-grained samples were performed by the Memphis District Laboratory. Test results for the project are located on file in the Memphis District Geotechnical Engineering Branch. The specific strata where WES tests were conducted are shown on the boring logs.

2.05 SOILS AND FOUNDATION ANALYSES.

a. General. The boring logs along with the corresponding test results were examined to determine appropriate soil stratification and shear strength parameters for the major structures foundation design. Once stratifications and shear strength values were assigned, a variety of foundation analyses were performed as applicable to determine liquefaction potential, channel slope stability, structural excavation slope stability, structural sliding and overturning stability, bearing capacity, settlement, uplift analyses, and dewatering requirements.

b. Design Shear Strengths. (1). Clays. The selection of the design values for the Q and R conditions was based on consistencies indicated by the boring logs, natural densities, moisture contents, Atterberg limits, unconfined compression tests and Q triaxial tests. Design values for the S condition were based on the Plasticity Index (PI) verses ϕ' relationship developed for LMVD and contained in WES Technical Report No. 31-604, June 1962. Direct Shear tests were also reviewed for use in developing design S strengths. However, since only a limited number of samples were tested and the results were generally higher than the LMVD design curve, the standard curve was used in design.

(2). Silts. The design shear strengths for silts were selected conservatively using consistencies indicated by the boring logs, natural moisture contents, past experience with similar soils, and values suggested in DIVR I 110- 1-400, Section 5, Part 4 Item 1, dated March 1973. Design values of $\phi=20^\circ$, c=300 psf were selected for the Q and R conditions. Design values of $\phi=28^\circ$, c=0 psf were chosen for the S case strengths.

(3). Sands. Design values for the coarse grained soils were selected based on past experience with similar soils, suggested values in the above DIVR, and correlations between standard penetration tests (N-values) and the angle of internal friction (ϕ). Cohesive values were taken as zero for all loading conditions.

2.06 STABILITY ANALYSES.

a. Slope Stability. Slope stability analyses were performed to determine the required slopes for the canals and embankments of the distribution system, for excavation slopes at pumping stations and the gated check structures, and for the inlet and outlet channel slopes at pumping stations. Slope stability analyses were conducted in accordance with guidelines and criteria presented in D1VR 1110-1-400, Section 5, Part 4, Item 1, dated March 1973, for Type A projects. Long-term stability analyses for the canals used Type B criteria or a factor of safety greater than 1.0. All other stability analyses were performed for the following loading cases with respective minimum allowable factors of safety for Type A projects:

Loading Case	Minimum Factor of Safely
After Construction (AC)	1.30
Long- Term (LT)	1.25
Sudden Drawdown (SD)	1.20
Partial Pool (PP)	1.30

Stability analyses for the appropriate loading conditions were performed using the microcomputer software package GEOSLOPE that is available through the GEOCOMP Corporation. The program's capability to analyze both circular and non-circular shaped failure surfaces was used in the analysis of all slopes. The analysis of circular failure surfaces was performed using the Modified Bishop Method of Slices. For non-circular failure surfaces or wedge failures, the Janbu Method of Generalized Slices was utilized. Manual checks of the minimum factors of safety for the stability analyses are included for designed slopes at major structures only. Only the critical factors of safety are presented for each analysis. A legend for the slope stability analyses is presented as Plate II- 20.

b. Structural Stability Analyses. (1). Sliding. Sliding stability analyses were performed for each pumping station structure, related inlet structures and gated control structures in accordance with procedures presented in ETL 1110-2-256 and EM 1110-2-2502. Sliding stability analyses were performed to ensure the stability against sliding at the base of the structure or through any soil layer below the base. Program CSLIDE from the WES library was used to design and analyze the structure. CSLIDE is a computer program for assessing the sliding stability of concrete structures using the Limit Equilibrium Method described in ETL 1110-2-256, "Sliding Stability For Concrete Structures". A minimum allowable factor of safety of 1.50 was used for normal loading conditions and 1.33 for unusual loading conditions. Since the project is located near the boundary of Seismic Risk Zones 1 and 2, stability of each structure was analyzed for earthquake-induced loadings. A horizontal acceleration coefficient of 0.10 g was used to determine the lateral earthquake forces acting on the structure. This acceleration coefficient was developed using the equation (kh = 0.5*PGA*Site Coefficient) found in “Geotechnical Earthquake Engineering” by Steven Kramer. (Page 436) A standard building code site coefficient of 1.5 was also used in determining the horizontal acceleration. The Peak Ground Acceleration (PGA) was estimated from the USGS Seismic Hazard Mapping website using the site coordinates. This acceleration was developed from USGS recommendations and corresponds to an event with a 5% probability of exceedance in a 50-year period with a 0.2 sec spectral acceleration. This ground motion is equivalent to an approximate 1000-year return period ground motions, or ground motions likely to occur once during a 1000-years period. The vertical earthquake acceleration was neglected. The minimum allowable factor of safety for sliding for earthquake loading was 1.10. Manual checks are included for Pumping Station #1 & #2 to clarify the analysis methodology. Due to the preliminary nature of this report, manual checks were performed but are not presented for other major structures.

(2). Overturning Analyses. Overturning stability analyses were performed for the major pumping station structures, related inlet structures, and gated check structures. These analyses were conducted to ensure the location of the resultant force occurred within the middle third of the base for the after construction (AC) loading condition (Case I) and the middle half of the base for the sudden drawdown (SD) loading condition (Case III). The method of analysis used for overturning along with the required minimum resultant locations are presented in EM 1110-2-2502. The active soil, passive soil and water pressures were applied to the structure using at rest earth pressure coefficients. Overturning analyses were also performed for earthquake loading to determine the resultant location with seismic forces applied. A horizontal acceleration coefficient of 0.10 g was used to determine the lateral earthquake forces. Guidance presented in EM 1110-2-2502 requires only that the resultant force be located within the base for earthquake conditions.

(3). Bearing Capacity. Bearing capacity computations were based on principles and methods presented in EM 1110-1-1905. Bearing capacity was determined by Meyerhof's Equation, which is a modification of the general bearing capacity equation to account for effects of embedment, overburden pressure, foundation shape, and inclination of loading. Earthquake loading using a horizontal acceleration coefficient of 0.10 g was considered a viable case. Analyses included determining a factor of safety by dividing the ultimate bearing capacity of the foundation soil by the soil pressure determined from the net vertical load and the eccentricity loading on each structure. A minimum allowable factor of safety of 3.0 is required for normal loading conditions, 2.0 for unusual loading conditions and foundations of retaining walls founded on sand, and 1.0 for earthquake loading as presented in EM 1110-1-1905 and EM 1110-2-2502. Pumping stations #1 and #4, and some of the gated check structures will be founded on sand. Pumping stations #2 and #3 will be founded on clay, but may be over-excavated to a predetermined depth due to soft clays encountered in borings performed in the vicinity of the structures. The other gated check structure foundations could vary depending on their final location.

(4). Settlement. Since the pumping stations #1 & #4 will be founded on sand, settlement was considered negligible and no analyses were performed. However, settlement was a concern for Pumping stations #2 and #3. Settlement analyses were conducted to estimate the magnitude of the settlement. The estimates were based on limited consolidation test data taken for this project, empirical relationships developed for clay strata, and experience from previous projects.

(5). Uplift. Uplift analyses were performed for the pumping stations. ETL 1110-2-307, Flotation Stability Criteria for Concrete Hydraulic Structures, was used for the analysis for the pumping stations. As recommended in the above referenced ETL, no side friction was used in the uplift analysis. The line of creep method was used for determining seepage forces under the structures. A factor of safety of 1.50 was used in the design.

2.07 CANAL STABILITY.

A preliminary canal design was performed to determine the critical section required for the water distribution system. This critical section was based on the highest combination of both canal embankment and canal excavation required for water distribution and included freeboard above maximum pool and over excavation of the canal bottom for additional material needed for construction of the levee embankments in select reaches. It was assumed that additional material would be excavated at the same continuous slope as selected for the combined canal and embankment slope. This review resulted in a critical section with a maximum height of 33 feet as shown on Plate II-21. Stability analyses on a 20 foot high embankment (conservative) as shown on Plate II-21 were conducted utilizing the after construction loading case, partial pool loading case and long term stability case. The sudden drawdown condition was not analyzed due to the upper banks consisting either of clay or clayey silt for the borings taken throughout the project limits. The AC analyses with a slope of 1V on 3.0H resulted in a minimum factor of safety of 1.31 utilizing very conservative shear strengths of either $\phi=0$, c=750 psf for clay or $\phi=20^{o}$, c=300 psf for the silt, or a combination of the two materials. Shear strengths in the water depleted areas of the Bayou Meto basin generally ranged in excess of ($\phi=0$, c=1500 psf for these upper bank clays.

Eleven direct shear "S" tests were analyzed by WES. These tests were conducted on insitu samples. The results are plotted on Plate II-22 along with the standard Plasticity Index (PI) verses ϕ\` relationship developed for LMVD and contained in WES Technical Report No. 3 1-604, June 1962. The results of the Bayou Meto test data generally plotted above than the standard curve, although four of the eleven data points plotted below the curve. However, it was decided to use the standard LMVD curve for determining ϕ\` and long-term factors of safety mainly because of limited test data for this study and the curve generally resulted in more conservative results. The recommended geometry for the canal system is listed below.

CANALS

SLOPES: 1V: 3.0H CANALSIDE
CROWN WIDTH: 12 FEET MINIMUM FOR BOTH SIDES OF CANAL
BERM WIDTH: 15 FEET MINIMUM FOR BOTH SIDES OF CANAL

SECONDARY CANALS

SLOPES: 1V:3.0H CANALSIDE
IV:3.5H LAND SIDE FOR HEIGHTS > 10 FEET ABOVE NATURAL GROUND
1V:3.0H LANDSIDE FOR HEIGHTS < 10 FEET ABOVE NATURAL GROUND

2.08 CANAL SEEPAGE

a. General. The seepage loss was calculated using the method of inversion by Vedernikov as presented in Section 9.3 of Groundwater and Seepage by M. E. Harr published by McGraw-Hill Book Company in 1962. The seepage loss for a trapezoidal channel was calculated using the formula, q=k(B+AH), where:

q	=	Seepage Loss
k	=	Permeability of Soil in Channel Bottom
B	=	BW + 2MH
BW	=	Channel Bottom Width
M	=	Channel Slope (IV on 3H)
H	=	Head from Projected Water Surface to Channel Grade (See Tables II-2 through II-3)

The type of soil in the bottom of each seepage canal was estimated using relevant borings and the cone penetrometer tests. The analysis assumed that the least permeable layer below canal excavation would control the soil permeability in the canal. For example, if a canal is excavated to silt and a clay layer exists immediately below the silt layer, a clay permeability was assumed in the analysis.

b. Soil Permeability Assumptions. The permeabilities of insitu soils were estimated using field permeability tests of the type performed by the U. S. Bureau of Reclamation, Designation E-14, using open-end casings. The permeabilities were calculated using the relationship, k = q/5.5rh, where:

Q	=	Constant Rate of Flow into the Hole
r	=	Inside Radius of the Casing
h	=	Differential Head of Water Used in Maintaining a Steady Rate

Sixteen field tests were performed in 1999 resulting in the following permeabilities:

Soil Permeability Estimates

Soil Type	Permeability (cm/sec)
Sand	$2x10^{-4}$
Silty Sand	$7x10^{-5}$
Silt	$1.5x10^{-5}$
Clay	$1x10^{-7}$

c. Initial Seepage Estimate (1999). Initial seepage loss estimates used in sizing the pumping station were made in 1999 during initial design based on assumed channel geometry and permeabilities obtained from the field permeability tests described above. During this analysis, three different canal geometries were assumed for the initial estimate. The main canals were assumed have a 75 foot bottom width, 30 foot water depth, and 1V:3H side slopes. Intermediate canals were assumed have a 50 foot bottom width, 20 foot water depth, and 1V:3H side slopes. Permeabilities were based on data from the field permeability tests as discussed above. Seepage from existing channels was estimated assuming a total of approximately 670 miles of channel length, a bottom width of 20 feet, a water depth of 8 feet, an average side slope of 1V:2H and foundation permeability of 1×10^{-5} cm/sec. The results indicated an estimated total seepage loss of approximately 200 cfs (120 cfs from the new canals, 80 cfs for existing channels). This estimate was used in the Hydraulic Model to determine actual cfs required for the major pumping station.

d. Final Seepage Analysis (2002). A more detailed seepage analysis was performed to incorporate cone penetrometer test data and account for changes made in the water distribution system design since the initial estimate. During the final seepage estimate, canal geometries were taken from the balanced cut/fill designs from the Memphis District Civil Design Branch. Geometries varied throughout the system as some canals will be over-excavated to build levees. The total estimated seepage losses for the system was approximately 40 cfs initially that will decrease to approximately 10 cfs as siltation occurs.

The difference between the 40 cfs estimate and the initially assumed 200 cfs estimate is due to major changes in the distribution system from 1999 to 2002. The initial seepage estimate in 1999 assumed the distribution canals would be primarily gravity controlled. This design resulted in high seepage losses, as the majority of the canals in the system would require deeper cuts and would have been excavated into foundation sands. Additionally, many of the manmade canals proposed in 1999 were replaced with buried pipelines. These changes resulted in a significant decrease in seepage losses from the 1999 estimate to the 2002 estimate. A summary of the seepage analysis is provided in Table II-2 and II-3. A more detailed breakdown of the seepage loss estimate is provided in Tables II-4 through II-30.

2.09 PUMPING STATION #1.

a. Introduction. Pumping station #1 will consist of a 84-foot by 126.5-foot concrete structure approximately 82 feet in height with an inlet slab grade of elevation 217.0 feet NGVD. The station consists of six pump bays and one service bay. The substructure consists of an operating floor, sump floors, intake and trash racks. Two borings (44-BMU-99 and 45-BMU-99) were originally taken in 1999 in the general proximity of the proposed site to determine the most favorable location. However, due to a change in pumping station design considerations, (design change from all gravity system to a pumped system) a reservoir was introduced and the pumping station was designed. An additional boring (62-BMU-01) was performed at the current pumping station location. Pumping station #1 will operate by pumping from the inlet channel (Canal 500) supplied by the Arkansas River into to reservoir #1. The reservoir will regulate the water in the main canal using a control structure located on the east side of the reservoir.

b. Structural Stability Loading Cases. Pumping station #1 was analyzed for structural stability. The following loading cases were established. Factors of safety for sliding, overturning, and bearing capacity were determined based on these loading conditions. The results of the analyses are discussed in the paragraphs below.

Case I	Construction case; backfill in place; no hydrostatic forces applied.
Case II	Drawdown condition; water in backfill at El. 246 feet NGVD; water in the inlet channel at El. 241 feet NGVD; uplift determined by creep path method.
Case III	Partial pool case; water in backfill same as in the inlet channel and varied. (Case presented water at El. 233 NGVD)
Case IV	Earthquake condition. Water in inlet channel same as backfill at El. 233 feet NGVD with earthquake forces applied. Horizontal acceleration coefficient of 0.10 was used.

c. Sliding Stability Analyses. Sliding stability analyses were performed for a one-foot section of the 126.5-foot-wide pumping station structure as shown on Plates II-24 and II-25. A pervious backfill will be placed behind the structure. Design shear strengths of $\phi = 30^{\circ}$, c=0 psf were selected for the backfill material. Passive soil resistance was conservatively ignored. The results of the sliding stability analysis as shown on Plates II-33 and II-34 indicate sliding stability for the pumping station structure is adequate for all loading cases.

d. Overturning Analyses. Overturning stability analyses were performed for a one-foot section of the pumping station structure. A lateral at-rest earth pressure coefficient of $K_o = 0.5$ was selected based on the value of $\phi = 30^{\circ}$ for a horizontal backfill. Results of the overturning analyses as shown on Plates II-36 and indicate a resultant force location within the allowed structural base to meet the minimum requirements for all cases analyzed.

e. Bearing Capacity. Using the results of the overturning analyses, bearing capacity analyses were then performed to determine the maximum foundation pressures and the corresponding factors of safety for bearing. Since the pumping station will be founded on sand, the factors of safety against bearing capacity failure were very high for all three cases analyzed. The results are presented on Plate II-35.

f. Uplift Analysis. An uplift analysis of the major pumping station was performed to determine if the proposed station has sufficient weight to prevent the structure from floating or moving due to uplift pressures associated with high river stages. This analysis assumed only the weight of the concrete in the structure plus additional water loading on the inlet slab for the resisting forces. The uplift forces were based on the maximum historical elevation on the Arkansas River or elevation 243 feet NGVD. The bottom of the pumping station slab is at elevation 213.5 feet NGVD resulting in a net uplift head of 29.5 feet. ETL 1110-2-307 was used for determining the factors of safety against uplift for the applicable loading conditions that include normal operation, unusual operation, and extreme maintenance or the pumping station. The analysis is presented on Plate II-35 and indicates the structure is safe against uplift.

g. Settlement. Since the major pumping station structure will be founded on sand, settlement was considered negligible and no analyses were performed.

h. Seismic Risk Study. An analysis of the proposed site of Pumping Station #1 (1750 cfs) was undertaken to determine the annual risk factor (R) for an earthquake to cause liquefaction of the underlying foundation sands. This analysis is shown on Plates II-26 and II-27. The analysis was controlled by the sand layer at Elevation 184.0 feet NGVD (58 feet from the surface) of Boring 62-BMU-01. This risk factor was determined to be equal to 0.000188 for existing conditions.

Several alternatives were considered for improving the foundation conditions at the pumping station site. Because of the depth of the controlling layer, excavation of the controlling layer was not considered a viable option for foundation improvement. It was assumed that an in-place ground modification method would be the most economical method of densifying the controlling sand layer. This method assumes that a vibroflotation method will be used through the loose sand layer and reduce the liquefaction risk.

The cost effectiveness of the vibroflotation method for densifying the foundation sands was determined by comparing the present worth value of the annual damages eliminated by densifying the foundation sands to the cost of densifying the foundation sands. The damages from liquifaction were estimated by assuming that the structure remained as a unit but sustained excessive settlement severing all outside connections. The total estimated cost required to repair the pumping station assumed dewatering, pressure grouting the foundation, removing, and replacing the inlet retaining walls, and repairing the structural, mechanical and electrical damage. This cost was estimated at $7,000,000. The benefit to cost ratio for densifying the sands was 0.08. Because the risk of liquefaction was relatively low for the subject site, the benefit to cost ratios for in-place densification was significantly below 1.0. Therefore, ground modification by densification is not an economic solution to mitigate seismic risk due to liquefaction.

i. Excavation and Cofferdam Stability Analyses. The structural excavation will be protected from flood stages on the Arkansas River by a small earthen cofferdam. The cofferdam will consist of a semi-compacted embankment with side slopes of 1V on 3H minimum, a minimum crown width of 10 feet and a crown elevation of 247 feet NGVD. This crown elevation is based on the maximum flood plus an additional 4 feet to provide adequate freeboard and account for settlement. The location of the cofferdam will require a minimum 30-foot berm. The excavation slopes will begin at natural ground (elevation 242.0 feet NGVD) and extend to elevation 209.0 feet NGVD on a minimum slope of 1V to 2.5H.

Stability analyses were conducted for the cofferdam, excavation, and the combined cofferdam and excavation together with the above slopes. The results are presented on Plate II-31. The analysis resulted in a factor of safety of 1.75 excavation a, a factor of safety greater than 3 for the cofferdam with an overall factor of safety of approximately 1.95. The analysis was performed using soil data from Boring 62-BMU-01. Due to the size of the pumping station and assumed length of construction, long-term stability analyses were also performed for both the excavation and the cofferdam. The long-term stability resulted in a factor of safety of 1.18 in the upper fine-grained soils. However, the lower factor of safety was considered adequate due to the temporary loading for this condition. These analyses assumed that the dewatering system will maintain the ground water below the excavation bottom and side slopes.

j. Dewatering. A dewatering analysis was performed to determine the required number and layout of wells for the dewatering cost estimate. Dewatering requirements were based on procedures and guidelines presented in TM 5-818-5. The dewatering system was designed to lower the water table to elevation 206 feet NGVD which is 3 feet below the bottom of the deepest excavation. It was assumed that the Arkansas River would act as a line source at a distance of approximately 4000 feet from the centerline of the excavation. The analysis was performed assuming a headwater elevation of 243 feet NGVD which is the maximum historic stage on the Arkansas River and assuming artesian flow conditions would prevail. The Tertiary deposits were conservatively assumed to be located at elevation 154 feet NGVD based on Boring 62-BMU-01 resulting in an aquifer thickness of 80 feet. A horizontal permeability of 800 x 10^{-4} cm/sec was selected based on the D_{10} grain size of the foundation sands taken from boring samples in the area. The analysis as presented on Plate II-30 indicates that a dewatering system consisting of four 14-inch-diameter fully penetrating deep wells is required. As noted before, the dewatering analysis is presented for cost estimating purposes only since the actual dewatering system will be Contractor designed, installed and operated.

k. Inlet Channel Stability. The inlet channel for the major pumping station consists of a 4800-foot long, 100-foot bottom width channel with an inlet invert Elevation of 217 feet NGVD. The borings performed in the vicinity of the inlet channel indicate a very thin stratum of fine-grained deposits overlying the subsurface sands. Stability analyses were performed using boring data taken from borings 61-BMU-01 and 62-BMU-01. See Plate II-32 for the boring stratification and channel stability analysis. The analysis resulted in adequate factors of safety using both Q and S strengths with a slope of 1V on 3H for the inlet channel. A minimum factor of safety of 1.28 was calculated from this analysis as shown on Plate II-32.

l. Reservoir #1 Embankment Stability. Slope stability analyses for the reservoir embankment was performed based on criteria set forth in EM 1110-2-1902. All slopes were analyzed for possible wedge and arc failures using the microcomputer software package GEOSLOPE that is available through the GEOCOMP Corporation. The embankment slopes were analyzed for each of the loading cases as required by the referenced EM. An after drawdown pool elevation of 235 feet NGVD was used in the slope stability analysis. This elevation is based on the invert elevation of the outlet structure. A before draw down water elevation of 248 feet NGVD was used for the maximum pool elevation and the crest elevation of 252 feet NGVD was used for the surcharge pool elevation. A horizontal acceleration of 0.1 was used for case VII, Earthquake Loading. This acceleration was developed from USGS recommendations and corresponds to an event with a 5% probability of exceedance in a 50-year period with a 0.2 sec spectral acceleration. This ground motion is equivalent to an approximate 1000-year return period ground motions, or ground motions likely to occur once during a 1000-years period. A standard building code site coefficient of 1.5 was also used in determining the horizontal acceleration. The analysis indicated that a slope of 1V to 4H is required for stability of the pumping station #1 reservoir embankment. The earthquake loading case controlled the design. Soil strengths utilized for the slope stability analysis and the loading cases with the respective minimum and calculated factors of safety for each case are presented in Plate II-28.

m. Reservoir Seepage Analysis. Borings performed in the vicinity of the proposed reservoir indicates that sand may exist at the surface of the reservoir. A seepage analysis was performed for the proposed reservoir to estimate the seepage losses. A flow net was developed to model the reservoir embankment and foundation sands. For the analysis, the thin clay overburden was conservatively neglected. A horizontal permeability of 800×10^{-4} cm/sec was selected based on the D_{10} grain size of the foundation sands taken from boring 62-BMU-01 samples. The analysis was performed assuming a one foot cross section through the embankment. A flow net was developed using equipotential lines and drops. Considering the uncertainties in this type of seepage analysis, a sensitivity analysis was performed on the variables. The sensitivity analysis was performed by selecting the most likely high and low values of each of the variables in the seepage estimate, specifically, the number of equipotential lines (Nf), drops (Nd), Head (H) and the permeability of the aquifer (Kh). The flow from the reservoir was calculated using the high and low values of each variable. The analysis resulted in estimated seepage losses ranging between 35 cfs and 69 cfs with an average of approximately 55 cfs. The average estimated seepage loss is approximately 3% of the pumping station capacity of 1750 cfs. This loss is considered to be within acceptable limits and may be conservative, as the presence of thin clay layers and gradual siltation will reduce the losses from the reservoir. No clay cap is proposed for the reservoir at this time. Additional borings and more detailed analyses will be performed prior to plans and specifications. A summary of the average seepage loss estimate is presented on Plate II-29.

2.10 PUMPING STATION #2.

a. Introduction. Pumping station #2 (625 cfs) will consist of a concrete structure with dimensions of 47 feet by 56 feet at the base and a height of 39 feet. A site plan, general layout and section of the station are shown on Plates II-37 through II-39. Boring 59-BMU-00 was taken at the pumping station site and was used for all analyses performed for the structure. Considering the depth of clay at the site and the fact that groundwater has been lowered in the Bayou Meto region, dewatering will not be required. Boring 59-BMU-00 indicates that soft clay layer exists from approximate elevation 208 feet to 201 feet NGVD. To reduce the potential for excessive settlement, the foundation will be over-excavated to an elevation of 201 feet and replaced with select granular backfill. The inlet channel will consist of a 56-foot wide canal excavated into natural ground with an approximate channel bottom elevation of 217 feet NGVD. The water will be pumped through five lines of 54-inch diameter reinforced concrete pipe approximately 500 feet in length, across the Bayou Meto channel and into reservoir #2. Because of right of way issues, no borings were taken across the Bayou Meto Channel near reservoir #2. It is noted that the bottom slab of the pumping station slopes from elevation 217.2 to elevation 213. Because of this irregular shape, the bottom slab was assumed to be flat to simplify the analysis.

b. Structural Stability Loading Cases. Pumping Station #2 was analyzed for structural stability. The following loading cases were established. The loading cases used for the stability analyses of the pumping station are also defined on Plate II-42. Factors of safety for sliding, overturning, and bearing capacity were determined based on these loading conditions. The results of the analyses are discussed in the paragraphs below.

Loading	
Case I	Construction case; backfill in place; no hydrostatic forces applied.
Case II	Drawdown condition; water in backfill at El. 230 feet NGVD; water in the inlet channel at El. 225 feet NGVD (normal pool); uplift determined by creep path method.
Case III	Partial pool case; water in backfill same as in the inlet channel and varied. (Case presented water at El. 225 NGVD)
Case IV	Earthquake condition. Water in inlet channel same as backfill at El. 225 feet NGVD with earthquake forces applied. Horizontal acceleration coefficient of 0.10 was used.

c. Sliding Stability. Sliding stability analyses were performed for a one-foot section through the center of the station. A full view section of the station and results of the analyses are shown on Plates II-42 and II-43. Because the foundation will be over-excavated and replaced with 10 feet of granular backfill, all loading cases were analyzed using sand under the base of the structure with a sand backfill. The sliding stability analysis was performed using the WES computer program "CSLIDE" and was also checked manually. Calculations indicate a marginal factor of safety for all loading cases when neglecting side friction on the structure. Side friction was conservatively neglected in sliding stability analyses for other structures in the Bayou Meto Irrigation Project.

However, forces developed due to side resistance was included in the analysis for Pumping station #2 because these forces were necessary to meet all factor of safety requirements. The side friction was calculated using the normal force on the wall due to the backfill multiplied by tangent of the shear strength ($\phi^o = 30^o$). Final calculations shown on Plate II-42 include the influence of side friction (7.31-kips/foot) on the structure. The results indicate Pumping station #2 has an adequate factor of safety against sliding for all loading cases analyzed.

d. Overturning Stability. Overturning stability analyses were performed for a one-foot section of Pumping station #2. A lateral earth pressure coefficient of $K_o = 0.50$ was selected based on a horizontal granular backfill of $\phi = 30^o$. Results of the analyses as shown on Plates II-44 indicate that the resultant force is located within the allowable structural base for all loading cases.

e. Bearing Capacity. Using the results of the overturning analyses, bearing capacity analyses were then performed to determine the maximum foundation pressures and the corresponding factors of safety for bearing. Since the foundation will be over-excavated approximately 10 feet, the bearing capacity analysis was performed assuming the structure was founded on granular backfill of $\phi = 30^o$. The analysis is presented on Plate II-45. Bearing capacity is adequate for all loading cases.

f. Uplift Analysis. An uplift analysis of pumping station #2 was performed to determine if the proposed station has sufficient weight to prevent the structure from floating or moving due to uplift pressures associated with high groundwater levels. Considering the depth of clay and natural groundwater in the vicinity of the pumping station, uplift is not likely to be a problem. However, the analysis assumes that the inlet canal and the neighboring Bayou Meto channel may create high groundwater levels within the granular backfill around the structure. The uplift analysis assumed only the weight of the concrete in the structure plus additional water loading on the inlet slab for the resisting forces. The uplift forces were based on an assumed groundwater elevations ranging from 215 feet to 230 feet NGVD in the inlet channel and backfill. Factors of safety were calculated assuming the water in the backfill was approximately the same as the water in the canal and one of the pumping bays may be dewatered. The pumping station slab elevation is approximately 211 feet NGVD resulting in a net uplift head of 19 feet. ETL 1110-2-307 was used for determining the factors of safety against uplift. The analysis is presented on Plate II-45 and indicates the structure is safe against uplift assuming that only one of the five bays is dewatered at any time.

g. Settlement. Settlement for Pumping station #2 was estimated using Boussinesq Coefficients. The fill height varies around the structure, with a maximum fill placement of 10 feet above natural ground. As discussed above, soft clay layers encountered at an elevation of 208 feet to 201 feet will be removed and replaced with compacted granular backfill to reduce the potential of excessive settlement. No consolidation tests were conducted on the soil samples taken from Boring 59-BMU-00, therefore foundation consolidation parameters were developed from empirical relationships derived from liquid limit data. Stresses induced on foundation were estimated using "Tables of

Bousinesq Coefficients for Vertical Stress Induction" published in 1969 by the U.S. Army Corps of Engineers New Orleans District. Preconsolidation stresses of the underlying deposits were estimated using equation (1-2) in EM 1110-1-1904. Because the structure weighs the same as the soil that will be excavated, settlement will be induced from the earth fill placed around three sides of the pumping station. Soil indices and calculated stresses suggest that underlying material is preconsolidated. Considering this information, it was assumed that the induced loading would fall on the recompression portions of the consolidation curve. Therefore, recompression indices (c_r) were used in the settlement calculations. This assumption resulted in an estimated settlement of less than one inch. This settlement would be in the tolerable limits for the structure. The results of these analyses are presented on Plates II-46. Additional soil data will be obtained at the pumping station site and consolidation tests will be conducted to determine the preconsolidation history of the clay layer. Depending on the test results, additional analyses will be performed to insure that an adequate foundation meeting minimum settlement criteria is incorporated into the design of the station.

h. Dewatering. Boring 59-BMU-00 indicates that clay extends to a depth of approximately 48 feet from the existing ground surface (Elevation 182). The structural excavation will to extend to elevation 201. Considering the depth of clay at the site, dewatering is anticipated for construction. However, the need for dewatering will be evaluated closer during future studies.

i. Stability Analyses – Excavation and Inlet Canals. As shown by Boring 59-BMU-00, the upper bank material consists of a silt and silty clay material. Due to the lean nature of upper clays, long-term shear strengths did not govern design. Slope stability analysis was governed by a wedge type failure surface through a soft clay layer at an elevation of approximately 201 feet. No sudden drawdown analysis was performed for the inlet channel since the excavation was in fine-grained material. Due to the short-term nature of the excavation no long-term analysis was considered for the excavation. Slopes of 1V:3H and 1V:2.5H are recommended for the inlet channel and the excavation respectively. No cofferdam was assumed in this analysis because no hydraulic data exists for this area of Bayou Meto. When more data is available, the use of a cofferdam will be evaluated. The results of the stability analyses for the inlet canal and structural excavation are presented on Plates II-40 and II-41 respectively.

j. Reservoir #2 Embankment/Outlet Structure Stability. Because of problems gaining right of entry on to the property where Reservoir #2 will be located, no soil borings were taken in the area of the reservoir or near the outlet structure during this study. Consequently, no slope stability analyses were performed for reservoir #2. However, reservoir slopes of 1V: 4H were assumed for the purposes of cost and quantity estimating for this feasibility study. Slopes of 1V:4H are consistent with the result of the stability analyses for Reservoirs #1 and #3 and are considered adequate for this stage of the study. The outlet structure dimensions are generally similar to other structures throughout the project study. Therefore the structure is assumed to be stable. More analysis will be performed after the location of the reservoir is finalized and permission is granted to take soil borings.

2.11 PUMPING STATION #3.

a. Introduction. Pumping station #3 (260 cfs) will consist of a concrete structure with dimensions of 30.67 feet by 29 feet at the base and a height of 32 feet. A general layout and section of the station are shown on Plates II-47 through II-49. The structure will be placed on a lean clay layer by excavating to elevation 215 feet NGVD and backfilling with three feet of compacted pervious material to support the base slab of the structure at elevation 218.0 feet NGVD. The inlet channel will consist of a 30-foot bottom width canal excavated into natural ground with a channel bottom elevation of 222 feet NGVD. The flow will be carried horizontally from the inlet canal to the inlet pump bays through a concrete inlet structure and three lines of 60-inch diameter reinforced concrete pipe approximately 90 feet in length. Three pumps at the station will lift the water approximately 17 feet in height to supply Reservoir #3. The bottom of the reservoir will conform to the natural ground elevation of approximately 238 feet at the site and a reservoir embankment will be constructed to a crest elevation of 250 feet. Boring 68-BMU-01 was taken at the pumping station site and was used for all analyses performed for the structure. Boring 37-BMU-99 was also taken in the vicinity of the pumping station site and was used in performing and verifying the inlet and reservoir stability analyses. The loading cases used for the stability analyses of the pumping station are defined on Plate II-53. Since the structure is located in the Bayou Meto region, dewatering is not expected since the water table is substantially below the pumping station foundation.

b. Structural Stability Loading Cases. Pumping Station #3 was analyzed for structural stability. Factors of safety for sliding, overturning, and bearing capacity were determined based on these loading conditions. The results of the analyses are discussed in the paragraphs below. The following loading cases were used in the analysis of the structures. The loading cases are also defined on Plate II-53.

Loading	
Case I	Construction case; backfill in place; no hydrostatic forces applied.
Case II	Drawdown condition; water in backfill at El. 247 feet NGVD (normal pool); water in the reservoir at El. 242 feet NGVD; uplift determined by creep path method.
Case III	Partial pool case; water in backfill same as in the inlet channel and varied. (Case presented water at El. 247 NGVD)
Case IV	Earthquake condition. Water in backfill at El. 242 feet NGVD and water in the reservoir at El. 247 with earthquake forces applied. Horizontal acceleration coefficient of 0.10 was used.
Foundation	
Case A	Structure founded on Sand ($\phi = 30^o$)
Case B	Structure founded on Clay (Q case c=1500 psf)
Case C	Structure founded on Clay (S case $\phi' = 24^o$)

d. Sliding Stability. Sliding stability analyses were performed for a one-foot section through the center of the station. A full view section of the station and results of the analyses are shown on Plate II-53. Because the foundation will be over-excavated and replaced with 3 feet of granular backfill, all loading cases were analyzed using sand (Case A) and clay with Q & S strengths (Cases B & C) under the base of the structure with a sand backfill. The sliding stability analysis was performed using the WES computer program "CSLIDE" with critical cases checked manually. All cases resulted in factors of safety above 2.0 for all loading cases. The results indicate Pumping station #3 has an adequate factor of safety against sliding for all loading cases analyzed.

c. Overturning Stability. Overturning stability analyses were performed for a one-foot section of Pumping station #3. A lateral earth pressure coefficient of $K_o = 0.50$ was selected based on a horizontal granular backfill of $\phi = 30^o$. Results of the analyses as shown on Plates II-54 indicate that the resultant force is located within the allowable structural base for all loading cases.

d. Bearing Capacity. Using the results of the overturning analyses, bearing capacity analyses were then performed to determine the maximum foundation pressures and the corresponding factors of safety for bearing. Since the foundation will be over-excavated approximately 3 feet, the bearing capacity analysis was performed assuming the structure was founded on granular backfill of $\phi = 30^o$ (Case A). However, the bearing capacity was also calculated assuming the structure was founded on clay using both Q and S strengths (Cases B & C). For this case, the bearing capacity analysis was performed using the full foundation pressures calculated from the overturning analysis. This assumption is considered conservative as a significant reduction in foundation pressure is expected with increasing depth. The results of the bearing capacity analysis for cases B & C are presented on Plate II-55. Bearing capacity is adequate for all loading cases.

e. Uplift Analysis. An uplift analysis of pumping station #3 was performed to determine if the proposed station has sufficient weight to prevent the structure from floating or moving due to uplift pressures associated with high groundwater levels. Considering the depth of clay and natural groundwater in the vicinity of the pumping station, uplift is not likely to be a problem. However, the analysis assumes that the inlet canal and the reservoir may create high groundwater levels within the granular backfill around the structure. The uplift analysis assumed only the weight of the concrete in the structure plus additional water loading on the inlet slab for the resisting forces. The uplift forces were based on an assumed groundwater elevations ranging from 220 feet to 247 feet NGVD in the backfill. Factors of safety were calculated assuming the water in the backfill was approximately the same as the water in the canal and one of the pumping bays may be dewatered. Water in the inlet channel and in the pumping station was assumed to be at elevation 232 feet NGVD. The pumping station slab elevation is approximately 220 feet NGVD resulting in a net uplift head of 15 feet. ETL 1110-2-307 was used for determining the factors of safety against uplift. The analysis is presented on Plate II-55 and indicates the structure is safe against uplift.

f. Settlement. Settlement for Pumping station #3 was estimated using Boussinesq Coefficients. The fill height varies around the structure, with a maximum fill placement of 12 feet above natural ground (El. 250). As discussed above, soft clay layers encountered at an elevation of 217.5 feet to 215 feet will be removed and replaced with compacted granular backfill to reduce the potential of excessive settlement. No consolidation tests were conducted on the soil samples taken from Boring 68-BMU-01, therefore foundation consolidation parameters were developed from empirical relationships derived from liquid limit data. Stresses induced on foundation were estimated using "Tables of Bousinesq Coefficients for Vertical Stress Induction" published in 1969 by the U.S. Army Corps of Engineers New Orleans District. Preconsolidation stresses of the underlying deposits were estimated using equation (1-2) in EM 1110-1-1904. Because the structure weighs the same as the soil that will be excavated, settlement will be induced from the earth fill placed around three sides of the pumping station. Soil indices and calculated stresses suggest that underlying material is preconsolidated. Considering this information, it was assumed that the induced loading would fall on the recompression portions of the consolidation curve. Therefore, recompression indices (c_r) were used in the settlement calculations. This assumption resulted in an estimated settlement of 1.9 inches. Additional soil data will be obtained at the pumping station site and consolidation tests will be conducted to determine the preconsolidation history of the clay layer. Depending on the test results, additional analyses will be performed to insure that an adequate foundation meeting minimum settlement criteria is incorporated into the design of the station. The results of these analyses are presented on Plates II-56

g. Dewatering. Borings 37-BMU-99 and 68-BMU-01 indicates that clay extends to a depth of approximately 65 to 75 feet from the existing ground surface (Elevation 173 to 163). The structural excavation will to extend to elevation 215. Considering the depth of clay and groundwater at the site, dewatering is not anticipated for construction.

h. Stability Analyses – Excavation and Inlet Canals. As shown by Borings 68-BMU-01 and 37-BMU-99, the upper bank material consists of highly plastic clay. Due to the high plasticity of upper clays long-term shear strengths governed the inlet slope design. Due to the presence of a soft clay layer, the excavation stability analysis was governed by a wedge type failure surface at an elevation of approximately 201 feet. No sudden drawdown analysis was performed for the inlet channel since the excavation was in fine-grained material. Due to the short-term nature of the excavation no long-term analysis was considered for the excavation. Slopes of 1V:3H and 1V:2.5H are recommended for the inlet channel and the excavation respectively. The results of the stability analyses for the inlet canals and the structural excavation are presented on Plates II-51 and II-52.

i. Reservoir #3 Embankment Stability. Slope stability analyses for the reservoir embankment were performed based on criteria set forth in EM 1110-2-1902. All slopes were analyzed for possible wedge and arc failures using the microcomputer software package GEOSLOPE that is available through the GEOCOMP Corporation. The embankment slopes were analyzed for each of the loading cases as required by the referenced EM. An after drawdown pool elevation of 238 feet NGVD was used in the slope stability analysis. This elevation is based on the invert elevation of the outlet structure. A before draw down water elevation of 247 feet NGVD was used for the maximum pool elevation and the crest elevation of 249 feet NGVD was used for the surcharge pool elevation. A horizontal acceleration of 0.1 was used for case VII, Earthquake Loading. This acceleration was developed from USGS recommendations and corresponds to an event with a 5% probability of exceedance in a 50-year period with a 0.2 second spectral acceleration. This ground motion is equivalent to an approximate 1000-year return period ground motions, or ground motions likely to occur once during a 1000-years period. A standard building code site coefficient of 1.5 was also used in determining the horizontal acceleration. The analysis indicated that a slope of 1V to 4H is required for stability of the pumping station #3 reservoir embankment. The earthquake loading case controlled the design. Factors of safety for Case III (sudden drawdown from top of gates) were slightly lower than allowable. However, this analysis is considered conservative as critical circular failure was governed by conservative shear strength values in the embankment fill material. Additionally, the sudden draw down condition performed for this analysis is severe and would likely only occur due to catastrophic failure of the water control structure. Soil strengths utilized for the slope stability analysis and the loading cases with the respective minimum and calculated factors of safety for each case are presented in Plate II-50.

2.12 PUMPING STATION #3 INLET STRUCTURE.

a. Introduction. Associated with Pumping station #3 is a concrete inlet structure with dimensions of 30.67 feet by 21.5 feet at the base and a height of 12.5 feet. A general layout and section of the station are shown on Plates II-57. The structure will be placed on a clay layer by excavating to elevation 220 feet NGVD and backfilled with two feet of compacted pervious material to support the base slab of the structure at elevation 222.0 feet NGVD. Boring 68-BMU-01 was taken at the pumping station site and was used for all analyses performed for the structure. The loading cases used for the stability analyses of the pumping station are defined on Plate II-53.

b. Structural Stability Loading Cases. The Pumping Station #3 Inlet Structure was analyzed for structural stability. Factors of safety for sliding, overturning, and bearing capacity were determined based on these loading conditions. The results of the analyses are discussed in the paragraphs below. The following loading cases were used in the analysis of the structures. The loading cases are also defined on Plate II-53.

Loading	
Case I	Construction case; backfill in place; no hydrostatic forces applied.
Case II	Drawdown condition; water in backfill at El. 232 feet NGVD (normal pool); water in the inlet channel at El. 227 feet NGVD; uplift determined by creep path method.
Case III	Partial pool case; water in backfill same as in the inlet channel and varied. (Case presented water at El. 227 NGVD)
Case IV	Earthquake condition. Water in backfill at El. 227 feet NGVD with earthquake forces applied. Horizontal acceleration coefficient of 0.10 was used.
Foundation	
Case A	Structure founded on Sand ($\phi = 30^{o}$)
Case B	Structure founded on Clay (Q case c=750 psf)
Case C	Structure founded on Clay (S case $\phi' = 21^{o}$)

c. Sliding Stability. Sliding stability analyses were performed for a one-foot section through the center of the station. A full view section of the station and results of the analyses are shown on Plate II-53. Because the foundation will be over-excavated and replaced with 2 feet of granular backfill, all loading cases were analyzed using sand (Case A) and clay with Q & S strengths (Cases B & C) under the base of the structure with a sand backfill. The sliding stability analysis was performed using the WES computer program "CSLIDE" with critical cases checked manually. A side friction resisting force of 2.76 kips/ft was included in the analysis for Pumping station #3 inlet structure because these forces were necessary to meet all factor of safety requirements. The side friction was calculated using the normal force on the wall due to the backfill multiplied by tangent of the shear strength ($\phi^{o} = 30^{o}$). The results indicate the Pumping station #3 Inlet structure has an adequate factor of safety against sliding for all loading cases analyzed.

d. Overturning Stability. Overturning stability analyses were performed for a one-foot section of the Pumping station #3 Inlet Structure. A lateral earth pressure coefficient of $K_o = 0.50$ was selected based on a horizontal granular backfill of $\phi = 30^{o}$. Results of the analyses as shown on Plate II-58 indicate that the resultant force is located within the allowable structural base for all loading cases.

e. Bearing Capacity. Using the results of the overturning analyses, bearing capacity analyses was then performed to determine the maximum foundation pressures and the corresponding factors of safety for bearing. Since the foundation will be over-excavated approximately 10 feet, the bearing capacity analysis was performed assuming the structure was founded on granular backfill of $\phi = 30^{o}$ (Case A). However, the bearing capacity was also calculated assuming the structure was founded on clay using both Q and S strengths (Cases B & C). For this case, the bearing capacity analysis was performed using the full foundation pressures calculated from the overturning analysis. This assumption is considered conservative as a reduction in foundation pressure is expected with increasing depth. The results of the bearing capacity analysis for cases B & C are presented on Plate II-59. Bearing capacity is adequate for all loading cases.

f. Settlement. The inlet structure generally weighs less than the material being excavated. Therefore, little to no settlement is expected due to the influence of the inlet structure. However, some settlement is expected due to the fill around Pumping station #3. Settlement calculations for Pumping station #3 are summarized on Plate II-56.

2.13 PUMPING STATION #4.

a. Introduction. Pumping station #4 (260 cfs) will consist of a concrete structure with dimensions of 26.67 feet by 23.33 feet at the base and a height of 21.5 feet. A general layout and section of the station are shown on Plates II-60 through II-62. The structure will be placed on sand by over-excavating to elevation 177 feet NGVD and backfilling with six feet of compacted pervious material to support the base slab of the structure at elevation 182.5 feet NGVD. The inlet channel will consist of an approximate 27-foot bottom width canal excavated into natural ground with a channel bottom elevation of 188 feet NGVD. The flow will be carried horizontally from the inlet canal to the inlet pump bays through a concrete inlet structure and three lines of 42-inch diameter reinforced concrete pipe approximately 37 feet in length. Three pumps at the station will lift the water approximately 8 feet in height to supply Canal 2140. Boring 24-BMU-99 was taken at the pumping station site and was used for all analyses performed for the structure. The loading cases used for the stability analyses of the pumping station are defined on Plate II-65. Since the structure is located in the Bayou Meto region, dewatering is not expected since the water table is substantially below the pumping station foundation.

b. Structural Stability Loading Cases. Pumping Station #4 was analyzed for structural stability. Factors of safety for sliding, overturning, and bearing capacity were determined based on these loading conditions. The results of the analyses are discussed in the paragraphs below. The following loading cases were used in the analysis of the structures. The loading cases are also defined on Plate II-65.

Loading	
Case I	Construction case; backfill in place; no hydrostatic forces applied.
Case II	Drawdown condition; water in backfill at El. 202 feet NGVD (normal pool); water in the canal at El. 197 feet NGVD; uplift determined by creep path method.
Case III	Partial pool case; water in backfill same as in Canal 2140 and varied. (Case presented water at El. 192 NGVD)
Case IV	Earthquake condition. Water in backfill at El. 195.5 feet NGVD and water in the canal at El. 200 with earthquake forces applied. Horizontal acceleration coefficient of 0.10 was used.
Foundation	
Case A	Structure founded on Sand ($\phi = 30^{\circ}$)

c. Sliding Stability. Sliding stability analyses were performed for a one-foot section through the center of the station. A full view section of the station and results of the analyses are shown on Plate II-65. Because the foundation will be over-excavated and replaced with 5.5 feet of granular backfill, all loading cases were analyzed using sand (Case A) under the base of the structure with a sand backfill. The sliding stability analysis was performed using the WES computer program "CSLIDE" with critical cases checked manually. The results indicate Pumping station #4 has an adequate factor of safety against sliding for all loading cases analyzed.

d. Overturning Stability. Overturning stability analyses were performed for a one-foot section of Pumping station #4. A lateral earth pressure coefficient of $K_o = 0.50$ was selected based on a horizontal granular backfill of $\phi = 30^o$. Results of the analyses as shown on Plate II-66 indicate that the resultant force is located within the allowable structural base for all loading cases.

e. Bearing Capacity. Using the results of the overturning analyses, bearing capacity analyses were then performed to determine the maximum foundation pressures and the corresponding factors of safety for bearing. Since the foundation will be over-excavated approximately 5.5 feet, the bearing capacity analysis was performed assuming the structure was founded on granular backfill of $\phi = 30^o$ (Case A). The results of the bearing capacity analysis are presented on Plate II-67. Bearing capacity is adequate for all loading cases.

f. Uplift Analysis. An uplift analysis of pumping station #4 was performed to determine if the proposed station has sufficient weight to prevent the structure from floating or moving due to uplift pressures associated with high groundwater levels. Considering the depth of natural groundwater in the vicinity of the pumping station, uplift is not likely to be a problem. However, the analysis assumes that the inlet canal may create high groundwater levels within the granular backfill around the structure. The uplift analysis assumed only the weight of the concrete in the structure plus additional water loading on the inlet slab for the resisting forces. The uplift forces were based on an assumed groundwater elevations ranging from 182.5 feet to 204 feet NGVD in the backfill. Factors of safety were calculated assuming the water in the backfill was approximately the same as the water in the canal and one of the pumping bays may be dewatered. Water in the inlet channel and in the pumping station was assumed to be at elevation 192 feet NGVD. The pumping station slab elevation is approximately 184.5 feet NGVD resulting in a net uplift head of 12 feet. ETL 1110-2-307 was used for determining the factors of safety against uplift. The analysis is presented on Plate II-67 and indicates the structure is safe against uplift.

g. Settlement. Since pumping station #4 structure will be founded on sand, settlement was considered negligible and no analyses were performed.

h. Dewatering. Borings 24-BMU-98 indicates that fine-grained soils extend to a depth of approximately 23 feet from the existing ground surface (Elevation 177). To reduce the potential for differential settlement, the structural excavation will extend to elevation 177 feet. Considering the depth of groundwater at the site, dewatering is not anticipated for construction.

i. Stability Analyses – Excavation and Inlet Canals. As shown by boring 24-BMU-98, the upper bank material consists of highly plastic clay. Due to the high plasticity and relatively high shear strengths of upper clays, long-term shear strengths governed the inlet slope design. The excavation stability analysis was governed by an arc type failure surface. No sudden drawdown analysis was performed for the inlet channel since the excavation was in fine-grained material. Due to the short-term nature of the excavation no long-term analysis was considered for the excavation. Slopes of 1V on 3H and 1V on 2.5H are recommended for the inlet channel and the excavation respectively. The results of the stability analyses for the inlet canals and structural excavation and are presented on Plates II-63 and II-64.

2.14 PUMPING STATION #4 INLET STRUCTURE.

a. Introduction. Associated with Pumping station #4 is a concrete inlet structure with dimensions of 26.67 feet by 23.33 feet at the base and a height of 13.5 feet. A general layout and section of the station are shown on Plates II-68. The structure will be placed on sand by excavating to elevation 177 feet NGVD and backfilled with 9 feet of compacted pervious material to support the base slab of the structure at elevation 186.0 feet NGVD. Boring 24-BMU-98 was taken at the pumping station site and was used for all analyses performed for the structure. The loading cases used for the stability analyses of the pumping station are defined on Plate II-65.

b. Structural Stability Loading Cases. The Pumping Station #4 Inlet Structure was analyzed for structural stability. Factors of safety for sliding, overturning, and bearing capacity were determined based on these loading conditions. The results of the analyses are discussed in the paragraphs below. The following loading cases were used in the analysis of the structures. The loading cases are also defined on Plate II-65.

Loading	
Case I	Construction case; backfill in place; no hydrostatic forces applied.
Case II	Drawdown condition; water in backfill at El. 232 feet NGVD (normal pool); water in the inlet channel at El. 227 feet NGVD; uplift determined by creep path method.
Case III	Partial pool case; water in backfill same as in the inlet channel and varied. (Case presented water at El. 227 NGVD)
Case IV	Earthquake condition. Water in backfill at El. 227 feet NGVD with earthquake forces applied. Horizontal acceleration coefficient of 0.10 was used.
Foundation	
Case A	Structure founded on Sand ($\phi = 30^{o}$)

d. Sliding Stability. Sliding stability analyses were performed for a one-foot section through the center of the station. A full view section of the station and results of the analyses are shown on Plate II-65. Because the foundation will be excavated to natural sand and replaced with 9 feet of granular backfill, all loading cases were analyzed using sand (Case A) under the base of the structure with a sand backfill. The sliding stability analysis was performed using the WES computer program "CSLIDE" with critical cases checked manually. Unlike the inlet structure for Pumping station #3, side friction was conservatively neglected in the analysis for Pumping station #4 inlet. The results indicate the Pumping station #3 Inlet structure has an adequate factor of safety against sliding for all loading cases analyzed.

c. Overturning Stability. Overturning stability analyses were performed for a one-foot section of the Pumping station #4 Inlet Structure. A lateral earth pressure coefficient of $K_o = 0.50$ was selected based on a horizontal granular backfill of $\phi = 30^o$. Results of the analyses as shown on Plate II-69 indicate that the resultant force is located within the allowable structural base for all loading cases.

e. Bearing Capacity. Using the results of the overturning analyses, bearing capacity analyses were then performed to determine the maximum foundation pressures and the corresponding factors of safety for bearing. Since the foundation will be over-excavated approximately 9 feet, the bearing capacity analysis was performed assuming the structure was founded on granular backfill of $\phi = 30^o$ (Case A). The results of the bearing capacity analysis are presented on Plate II-70. Bearing capacity is adequate for all loading cases.

f. Settlement. The inlet structure generally weighs less than the material being excavated. Additionally, the inlet structure will be over excavated to natural sands. Therefore, little to no settlement is expected due to the influence of the inlet structure.

2.15 TYPICAL CONTROL STRUCTURES.

a. Introduction. The canal distribution system has 12 control structures for controlling pool elevations and passing water downstream. Three of these structures control the flow of water out of Reservoirs #1, #2 and #3. The following table lists the control structures in the distribution system and provides basic gate dimensions.

Table II-1
Control Structures

Structure No.	Canal No.	Number of Gates	Gate Width (ft.)	Gate Height (ft.)	Upstream Water Elevation (ft)	Downstream Water Elevation (ft)	Top of Slab Elevation (ft)
C-1000R	1000	3	14	16	248.00	245.00	235.00
C-1000.1	1000	3	14	16	243.00	240.30	229.30
C-1000.2	1000	3	10	14	238.00	234.60	225.60
C-2000.1	2000	2	10	14	232.00	227.34	219.35
C-2000.1	2100	2	10	14	232.00	226.00	219.35
C-2110	2110	2	10	10	223.00	220.87	213.87
C-2500.R	2500	2	10	10	242.00	239.00	233.00
C-2500.1	2500	2	10	7.5	238.00	236.20	231.20
C-2500.2	2500	2	10	7	235.00	233.60	228.60
C-3000	3000	2	10	10	236.00	232.42	226.42
C-4000R	4000	2	10	7	247.00	246.00	241.00
C-4111.1	4111	2	4	5.5	232.00	231.25	227.25

These structures also provide overflow weirs on each side of the structure along the canal slopes. These weirs will limit the maximum upstream pool elevation at each structure. Site plans and typical sections of the proposed gated check structures are presented on Plates II-71 and II-73. Normal head differentials of approximately 3 feet will exist between upstream and downstream water pools during normal operation of the system. However, each structure will be required to hold either the upstream or downstream water surface while the other side is dewatered for maintenance purposes. Control structure C-1000R (the largest structure) was analyzed as the typical control structure. Analyses for sliding, overturning, bearing, settlement and uplift were performed on the typical structure to insure adequate stability of the proposed design. There were no site-specific borings at the gated check structures. Analyses were performed using conservative shear strength values assuming that the structure is founded on clay, silt or sand. Critical loadings along with the critical soil conditions were considered in each analysis. However, site-specific borings will be required at each structure to verify soil conditions and completion of the design for plans and specs. Slope stability has already been addressed in this report. Excavation slopes of IV on 2.5H will be required during construction and a final canal slope of 1V on 3.0H will be required for each structure.

b. Structural Stability Loading Cases. The typical control structure was analyzed for structural stability. Factors of safety for sliding, overturning, and bearing capacity were determined based on these loading conditions. The results of the analyses are discussed in the paragraphs below. The following loading cases were used in the analysis of the structures. The loading cases are also defined on Plate II-74.

Loading	
Case I	Construction case; No water in inlet or outlet channel. No Load on Structure. **Not Applicable**
Case II	Drawdown condition; water in inlet channel/Reservoir at normal pool; water in the outlet channel at channel bottom; uplift determined by creep path method.
Case III	Partial pool case; water in inlet channel same as in the outlet channel. No load on structure. **Not Applicable**
Case IV	Earthquake condition. Water in inlet channel/reservoir at normal pool with water in outlet channel 4 feet below inlet channel. Earthquake forces applied. Horizontal acceleration coefficient of 0.10 was used.
Case V	Same as Case II using S strengths for clay and silt.
Case VI	Same as Case IV using S strengths for clay and silt.

Foundation Strength Assumptions

SOIL	Q-Case		S-Case	
Clay	$\phi = 0^{o}$	C = 750 psf	$\phi` = 18^{o}$	C = 0 psf
Silt	$\phi = 20^{o}$	C = 300 psf	$\phi` = 28^{o}$	C = 0 psf
Sand	$\phi = 30^{o}$	C = 0 psf	$\phi` = 30^{o}$	C = 0 psf

c. Sliding Stability. Sliding Stability analyses were performed for a one-foot section of the gated check structure as shown on Plate II-74. Clay, silt and sand foundation conditions were all considered in the sliding analyses. The results of the analyses indicate that sliding stability is adequate for all loading cases. The structure is basically symmetrical from upstream to downstream resulting in a centroid very near the center of the structure. Therefore, these analyses would also be representative of reverse loading simulating upstream canal dewatering with a normal pool elevation downstream.

d. Overturning Stability and Bearing Capacity. Overturning stability analyses were performed for a one-foot section of the control structure for Cases II and IV. Both cases resulted in resultant locations within the allowable portion of the structure base for each respective loading condition. The results of the overturning analysis are presented on Plate II-75. The results of the overturning analyses were then used to determine the maximum foundation pressures and the corresponding factors of safety for bearing capacity. Again, all three possible soil foundations were considered in the analyses. The results of the bearing analyses are presented on Plate II-76 and indicate that the bearing capacity is adequate for all loading cases.

Table II-2
Manmade Canals
Seepage Loss Estimate Summary

Canal #	Length Length (ft)	Initial Average Permeability (cm/sec)	Initial Seepage Loss (cfs)	After Siltation Average Permeability (cm/sec)	After Siltation Seepage Loss (cfs)
1000	66000	4.605E-05	13.543	6.162E-06	1.886
1400 / 1410	40600	2.545E-05	6.281	3.623E-06	0.761
2000	17100	1.112E-05	0.913	2.461E-06	0.201
2100/2110	36000	2.625E-06	0.428	2.625E-06	0.428
2140/2160	77200	1.000E-07	0.011	1.000E-07	0.011
2220	9000	1.000E-07	0.003	1.000E-07	0.003
2260	21100	1.905E-05	3.564	1.512E-06	0.274
2280	28700	2.726E-05	1.793	8.666E-06	0.499
2500	33400	1.000E-07	0.008	1.000E-07	0.008
2510	6000	1.000E-07	0.001	1.000E-07	0.001
2520	13600	1.000E-07	0.002	1.000E-07	0.002
2531	12000	1.000E-07	0.002	1.000E-07	0.002
2533	3500	1.000E-07	0.001	1.000E-07	0.001
3000	49500	9.127E-07	0.104	9.127E-07	0.104
4000 / 4100	15100	1.000E-07	0.004	1.000E-07	0.004
4111	54500	2.561E-06	0.472	2.561E-06	0.472
4112	30000	4.371E-06	0.316	4.371E-06	0.316
4113	5300	1.000E-07	0.001	1.000E-07	0.001
TOTALS	**518600 ft**		**27.45 cfs**		**4.97 cfs**

Table II-3
Existing Channels
Seepage Loss Estimate Summary

Canal #	Length Length (ft)	Initial Average Permeability (cm/sec)	Initial Seepage Loss (cfs)	After Siltation Average Permeability (cm/sec)	After Siltation Seepage Loss (cfs)
1500	117000	3.07E-05	5.401	6.40E-06	1.165
1530	49200	8.87E-07	0.096	8.84E-07	0.096
2120	63500	2.30E-05	3.451	7.05E-06	1.082
2511	37000	1.00E-07	0.012	1.00E-07	0.012
2521	17300	1.00E-07	0.002	1.00E-07	0.002
2530	43200	1.71E-05	1.115	9.05E-06	0.591
2532	27500	8.06E-06	0.632	8.06E-06	0.632
2540	38300	4.61E-06	0.390	4.61E-06	0.390
4110	30000	2.21E-05	1.904	1.34E-05	1.184
TOTALS	**423000 ft**		**13.00 cfs**		**5.15 cfs**

LIST OF PLATES

Bayou Meto
Seepage Loss Estimate

Table 4 - Canal 1000

** Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
45-BMU-01	0	20	2000	234.00	245.00	CH	1.00E-07	3.28E-09	3	11.00	40	106	9.64	2.37	4.33E-07	0.0009
35-BMU	20	74	5400	224.00	244.50	CH	1.00E-07	3.28E-09	3	20.50	0	123	6.00	1.7	5.18E-07	0.0028
34-BMU	74	152	7750	231.00	243.00	SP	2.00E-04	6.56E-06	3	12.00	40	112	9.33	2.35	9.20E-04	7.1296
32-BMU	152	170	1850	231.00	243.00	CL	1.00E-07	3.28E-09	3	12.00	40	112	9.33	2.35	4.60E-07	0.0009
32-BMU	170	200	3000	225.00	243.00	ML	1.50E-05	4.92E-07	3	18.00	0	108	6.00	1.7	6.82E-05	0.2046
32-BMU	200	235	3500	218.00	243.00	ML	1.50E-05	4.92E-07	3	25.00	0	150	6.00	1.7	9.47E-05	0.3316
	235	235	0	229.00	240.30				3	11.30					0.00E+00	0.0000
33-BMU	235	276	4100	229.00	239.50	CH	1.00E-07	3.28E-09	3	10.50	40	103	9.81	2.39	4.20E-07	0.0017
3-BMU	276	338	6200	227.50	239.00	CH	1.00E-07	3.28E-09	3	11.50	40	109	9.48	2.36	4.47E-07	0.0028
3-BMU	338	350	1200	220.00	239.00	ML	1.50E-05	4.92E-07	3	19.00	0	114	6.00	1.7	7.20E-05	0.0864
7-BMU	350	375	2500	227.00	235.00	CH	1.00E-07	3.28E-09	3	8.00	40	88	11.00	2.45	3.53E-07	0.0009
7-BMU	375	400	2500	224.00	235.00	CL	1.00E-07	3.28E-09	3	11.00	40	106	9.64	2.37	4.33E-07	0.0011
7-BMU	400	438	3800	215.00	235.00	CH	1.00E-07	3.28E-09	3	20.00	0	120	6.00	1.7	5.05E-07	0.0019
	438	438	0						3	0.00					0.00E+00	0.0000
4-BMU	438	515	7700	215.00	234.60	CH	1.00E-07	3.28E-09	3	19.60	0	117.6	6.00	1.7	4.95E-07	0.0038
8-BMU	515	585	7000	222.00	232.00	SM	7.00E-05	2.30E-06	3	10.00	40	100	10.00	2.4	2.85E-04	1.9934
8-BMU	585	629	4400	215.00	232.00	SP	2.00E-04	6.56E-06	3	17.00	0	102	6.00	1.7	8.59E-04	3.7793
5-BMU	629	660	3100	221.00	232.00	CL	1.00E-07	3.28E-09	3	11.00	40	106	9.64	2.37	4.33E-07	0.0013
			66000				**4.61E-05**									**13.5429**

Bayou Meto

Seepage Loss Estimate

Table 5 - Canal 1400/ 1410

** Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V: _H)	Channel Head H (ft)	Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
7-BMU	0	30	3000	218.00	236.00	CH	1.00E-07	3.28E-09	3	18.00	20	128	7.11	1.9	5.32E-07	0.0016
9-BMU	30	50	2000	226.00	235.00	ML	1.50E-05	4.92E-07	3	9.00	20	74	8.22	2.1	4.57E-05	0.0914
10-BMU	50	210	16000	228.00	235.00	CH	1.00E-07	3.28E-09	3	7.00	20	62	8.86	2.2	2.54E-07	0.0041
	210	210	0				0.00E+00	0.00E+00	3							
BM 55.c99	0	90	9000	227.00	232.00	CH	1.00E-07	3.28E-09	3	5.00	20	50	10.00	2.4	2.03E-07	0.0018
BM 50.c99	90	120	3000	227.00	231.00	CH	1.00E-07	3.28E-09	3	4.00	20	44	11.00	2.5	1.77E-07	0.0005
BM 50.c99	120	160	4000	214.00	231.00	SM	7.00E-05	2.30E-06	3	17.00	20	122	7.18	1.9	3.54E-04	1.4175
BM 50.c99	160	196	3600	208.00	231.00	SP	2.00E-04	6.56E-06	3	23.00	20	158	6.87	1.9	1.32E-03	4.7646
Totals			**40600**				**2.54E-05**									**6.2815**

Table 6 - Canal 1500 (Indian Bayou)

Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V: _H)	Channel Head H (ft)	Assumed Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
4-BMU	0	94	9350	225.00	228.00	CL	1.00E-07	3.28E-09	3	3.00	20	38	12.67	2.6	1.50E-07	0.0014
BM 78.c99	94	236	14250	215.00	218.00	CH	1.00E-07	3.28E-09	3	3.00	20	38	12.67	2.6	1.50E-07	0.0021
BM 77.c99	236	379	14250	214.00	217.00	SM	7.00E-05	2.30E-06	3	3.00	20	38	12.67	2.6	1.05E-04	1.4989
BM 129.c99	379	603	22400	212.50	215.50	sm	7.00E-05	2.30E-06	3	3.00	20	38	12.67	2.6	1.05E-04	2.3561
BM 136.c99	603	782	17900	210.00	213.00	CL	1.00E-07	3.28E-09	3	3.00	20	38	12.67	2.6	1.50E-07	0.0027
BM 137.c99	782	859	7750	209.00	212.00	CL	1.00E-07	3.28E-09	3	3.00	20	38	12.67	2.6	1.50E-07	0.0012
BM 138.c99	859	896	3650	207.50	210.50	CH	1.00E-07	3.28E-09	3	3.00	20	38	12.67	2.6	1.50E-07	0.0005
BM 314.c99	896	1024	12850	207.00	210.00	CH	1.00E-07	3.28E-09	3	3.00	20	38	12.67	2.6	1.50E-07	0.0019
BM 171.c99	1024	1170	14600	205.00	208.00	SM	7.00E-05	2.30E-06	3	3.00	20	38	12.67	2.6	1.05E-04	1.5357
			117000				**3.07E-05**									**5.4005**

Bayou Meto

Seepage Loss Estimate

Table 7 - Canal 1530 (Main Ditch)

Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Assumed Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
52-BMU	0	26	2600	201.30	204.30	ML	1.50E-05	4.92E-07	3	3.00	40	58	19.33	3.1	3.31E-05	0.0861
BM 167.c99	26	71	4500	200.26	203.26	CH	1.00E-07	3.28E-09	3	3.00	40	58	19.33	3.1	2.21E-07	0.0010
BM 168.c99	71	94	2300	199.50	202.50	CH	1.00E-07	3.28E-09	3	3.00	40	58	19.33	3.1	2.21E-07	0.0005
	94	94	0	199.50	202.50		1.00E-07	3.28E-09	3	3.00	40	58	19.33	3.1	2.21E-07	0.0000
BM 179.c99	94	321	22700	197.56	200.56	CL	1.00E-07	3.28E-09	3	3.00	40	58	19.33	3.1	2.21E-07	0.0050
	321	321	0	196.25	199.25		1.00E-07	3.28E-09	3	3.00	40	58	19.33	3.1	2.21E-07	0.0000
BM 307.c99	321	476	15500	192.10	195.10	CH	1.00E-07	3.28E-09	3	3.00	40	58	19.33	3.1	2.21E-07	0.0034
BM 306.c99	476	492	1600	191.46	194.46	CH	1.00E-07	3.28E-09	3	3.00	40	58	19.33	3.1	2.21E-07	0.0004
			49200				**8.87E-07**									**0.0964**

Table 8 - Canal 2000

** Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
5-BMU	0	25	2500	220.50	232.00	CH	1.00E-07	3.28E-09	3	11.50	35	104	9.04	2.1	4.20E-07	0.0011
5-BMU	25	60	3500	209.00	232.00	CH	1.00E-07	3.28E-09	3	23.00	0	138	6.00	1.7	5.81E-07	0.0020
	60	60	0				0.00E+00	0.00E+00	3							
65-BMU	60	87	2700	208.00	227.00	SM	7.00E-05	2.30E-06	3	19.00	0	114	6.00	1.7	3.36E-04	0.9072
6-BMU	87	134	4700	210.00	227.00	CL	1.00E-07	3.28E-09	3	17.00	0	102	6.00	1.7	4.29E-07	0.0020
59-BMU	134	160	2600	206.00	227.00	CL	1.00E-07	3.28E-09	3	21.00	0	126	6.00	1.7	5.31E-07	0.0014
59-BMU	160	171	1100	218.00	227.00	CL	1.00E-07	3.28E-09	3	9.00	35	89	9.89	2.39	3.63E-07	0.0004
Totals			**17100**				**1.11E-05**									**0.9130**

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Table 9 - Canal 2100 / 2110

** Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
65-BMU	0	61	6100	212.00	226.00	ML	1.50E-05	4.92E-07	3	14.00	25	109	7.79	2.1	6.81E-05	0.4155
31-BMU	61	85	2400	206.00	225.00	CH	1.00E-07	3.28E-09	3	19.00	0	114	6.00	1.7	4.80E-07	0.0012
31-BMU	85	105	2000	192.00	225.50	CH	1.00E-07	3.28E-09	3	33.50	0	201	6.00	1.7	8.46E-07	0.0017
31-BMU	105	135	3000	205.00	224.00	CH	1.00E-07	3.28E-09	3	19.00	0	114	6.00	1.7	4.80E-07	0.0014
31-BMU	135	155	2000	200.00	223.00	CH	1.00E-07	3.28E-09	3	23.00	0	138	6.00	1.7	5.81E-07	0.0012
30-BMU	155	175	2000	207.00	223.00	CH	1.00E-07	3.28E-09	3	16.00	0	96	6.00	1.7	4.04E-07	0.0008
30-BMU	175	201	2600	214.00	223.00	CH	1.00E-07	3.28E-09	3	9.00	25	79	8.78	2.2	3.24E-07	0.0008
30-BMU	201	275	7400	214.00	218.00	CH	1.00E-07	3.28E-09	3	4.00	20	44	11.00	2.5	1.77E-07	0.0013
30-BMU	0	20	2000	214.00	220.00	CH	1.00E-07	3.28E-09	3	6.00	20	56	9.33	2.3	2.29E-07	0.0005
30-BMU	20	30	1000	200.00	220.00	CH	1.00E-07	3.28E-09	3	20.00	0	120	6.00	1.7	5.05E-07	0.0005
90.C99	30	85	5500	194.00	220.00	CH	1.00E-07	3.28E-09	3	26.00	0	156	6.00	1.7	6.57E-07	0.0036
Totals			**36000**				**2.62E-06**									**0.4285**

Table 10 - Canal 2120 (Caney Creek)

Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
	0	35	3500	218.00	221.00	CH	1.00E-07	3.28E-09	3	3.00	30	48	16.00	2.9	1.86E-07	0.0007
BM 89.c99	35	111	7550	211.00	214.00	CH	1.00E-07	3.28E-09	3	3.00	35	53	17.67	3	2.03E-07	0.0015
BM 73.c99	111	213	10200	206.00	209.00	CL	1.00E-07	3.28E-09	3	3.00	50	68	22.67	3.2	2.55E-07	0.0026
BM 144.c99	213	328	11550	200.00	203.00	SM	7.00E-05	2.30E-06	3	3.00	40	58	19.33	3.1	1.55E-04	1.7852
BM 143.c99	328	440	11150	196.00	199.00	ML	1.50E-05	4.92E-07	3	3.00	50	68	22.67	3.2	3.82E-05	0.4258
BM 155.c99	440	566	12650	191.00	194.00	CH	1.00E-07	3.28E-09	3	3.00	100	118	39.33	3.2	4.19E-07	0.0053
BM 154.c99	566	635	6900	189.00	192.00	SM	7.00E-05	2.30E-06	3	3.00	50	68	22.67	3.2	1.78E-04	1.2297
Totals			**63500**				**2.30E-05**									**3.4507**

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Table 11 - Canal 2140 / 2160

** Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Assumed Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
24-BMU	0	77	7700	196.00	200.50	CH	1.00E-07	3.28E-09	3	4.50	20	47	10.44	2.45	1.90E-07	0.0015
47-BMU	77	234	15700	194.00	198.50	CL	1.00E-07	3.28E-09	3	4.50	20	47	10.44	2.45	1.90E-07	0.0030
48-BMU	234	360	12600	192.50	196.50	CH	1.00E-07	3.28E-09	3	4.00	20	44	11.00	2.5	1.77E-07	0.0022
29-BMU	360	495	13500	191.00	194.00	CH	1.00E-07	3.28E-09	3	3.00	10	28	9.33	2.3	1.15E-07	0.0015
293.c99	495	659	16350	190.00	193.00	CH	1.00E-07	3.28E-09	3	3.00	10	28	9.33	2.3	1.15E-07	0.0019
53-BMU	659	753	9450	189.00	191.50	CH	1.00E-07	3.28E-09	3	2.50	5	20	8.00	2.1	8.28E-08	0.0008
184.c99	753	772	1900	188.30	191.50	CH	1.00E-07	3.28E-09	3	3.20	5	24.2	7.56	2	1.00E-07	0.0002
			77200				**1.00E-07**									**0.0111**

Table 12 - Canal 2220

** Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Assumed Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
147.c99	0	45	4500	197.50	208.00	CH	1.00E-07	3.28E-09	3	10.50	15	78	7.43	2.35	3.37E-07	0.0015
118.c99	45	65	2000	200.00	207.50	CH	1.00E-07	3.28E-09	3	7.50	15	60	8.00	2.33	2.54E-07	0.0005
118.c99	65	90	2500	197.50	207.50	CH	1.00E-07	3.28E-09	3	10.00	15	75	7.50	2.33	3.23E-07	0.0008
			9000				**1.00E-07**									**0.0028**

Table 13 - Canal 2260

** Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Assumed Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
220.c99	0	30	3000	186.00	198.00	CL	1.00E-07	3.28E-09	3	12.00	10	82	6.83	1.85	3.42E-07	0.0010
219.c99	30	140	11000	188.00	196.50	CL	1.00E-07	3.28E-09	3	8.50	10	61	7.18	1.9	2.53E-07	0.0028
219.c99	140	180	4000	176.00	196.50	CL	1.00E-07	3.28E-09	3	20.50	50	173	8.44	2.2	7.16E-07	0.0029
219.c99	180	200	2000	175.00	196.50	SP	2.00E-04	6.56E-06	3	21.50	90	219	10.19	2.42	1.78E-03	3.5568
219.c99	200	211	1100	178.00	196.50	CL	1.00E-07	3.28E-09	3	18.50	110	221	11.95	2.6	8.83E-07	0.0010
			21100				**1.90E-05**									**3.5645**

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Table 14 - Canal 2280

** Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Assumed Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
220.C99	0	42	4200	191.00	195.00	CL	1.00E-07	3.28E-09	3	4.00	10	34	8.50	2.2	1.40E-07	0.0006
217.C99	42	110	6800	190.00	194.00	ML	1.50E-05	4.92E-07	3	4.00	10	34	8.50	2.2	2.11E-05	0.1432
215.C99	110	190	8000	186.00	193.50	CL	1.00E-07	3.28E-09	3	7.50	10	55	7.33	1.95	2.28E-07	0.0018
214.C99	190	215	2500	182.00	193.00	SM	7.00E-05	2.30E-06	3	11.00	10	76	6.91	1.9	2.23E-04	0.5563
214.C99	215	287	7200	186.00	193.00	SM	7.00E-05	2.30E-06	3	7.00	10	52	7.43	2	1.52E-04	1.0913
			28700				**2.73E-05**									**1.7933**

Table 15 - Canal 2500

Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
36-BMU	0	90	9000	231.50	239.00	CH	1.00E-07	3.28E-09	3	7.50	20	65	8.67	2.2	2.67E-07	0.0024
36-BMU	90	150	6000	230.00	236.20	CH	1.00E-07	3.28E-09	3	6.20	20	57.2	9.23	2.3	2.34E-07	0.0014
46-BMU	150	220	7000	228.00	235.00	CL	1.00E-07	3.28E-09	3	7.00	20	62	8.86	2.1	2.52E-07	0.0018
46-BMU	220	259	3850	228.00	233.60	CL	1.00E-07	3.28E-09	3	5.60	20	53.6	9.57	2.35	2.19E-07	0.0008
	259	259	0													
39-BMU	259	334	7550	227.00	232.50	CL	1.00E-07	3.28E-09	3	5.50	20	53	9.64	2.35	2.16E-07	0.0016
Totals			**33400**				**1.00E-07**									**0.0081**

Table 16 - Canal 2510

Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
36-BMU	0	60	6000	230.00	236.50	CH	1.00E-07	3.28E-09	3	6.50	8	47	7.23	2.1	1.99E-07	0.0012
			6000				**1.00E-07**									**0.0012**

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Table 17 - 2511 (Oak Branch)

Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
BM 58.c99	55	425	37000	206.70	210.70	CH	1.00E-07	3.28E-09	3	4.00	60	84	21.00	3.2	3.18E-07	0.0118
Totals			**37000**				**1.00E-07**									**0.0118**

Table 18 - Canal 2520

Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
46-BMU	0	68	6800	226.00	229.50	CH	1.00E-07	3.28E-09	3	3.50	8	29	8.29	2.1	1.19E-07	0.0008
BM 102.c98	68	136	6800	220.00	228.00	CH	1.00E-07	3.28E-09	3	8.00	8	56	7.00	2.1	2.39E-07	0.0016
Totals			**13600**				**1.00E-07**									**0.0024**

Table 19 - 2521 (Fishtrap Slough)

Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
BM 101.c98	136	249	11300	203.20	206.70	CH	1.00E-07	3.28E-09	3	3.50	8	29	8.29	2.1	1.19E-07	0.0013
BM 98.c98	249	309	6000	219.82	223.32	CH	1.00E-07	3.28E-09	3	3.50	8	29	8.29	2.1	1.19E-07	0.0007
Totals			**17300**				**1.00E-07**									**0.0021**

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Table 20 - Canal 2530 (Skinner BR)

Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
39-BMU	0	12	1150	225.20	228.20	CH	1.00E-07	3.28E-09	3	3.00	20	38	12.67	2.7	1.51E-07	0.0002
BM 111.c99	12	47	3500	223.40	226.40	CL	1.00E-07	3.28E-09	3	3.00	20	38	12.67	2.7	1.51E-07	0.0005
BM 110.c99	47	82	3550	218.80	221.80	CH	1.00E-07	3.28E-09	3	3.00	20	38	12.67	2.7	1.51E-07	0.0005
BM 113.c99	82	128	4550	215.10	218.10	ML	1.50E-05	4.92E-07	3	3.00	20	38	12.67	2.7	2.27E-05	0.1032
BM 114.c99	128	191	6300	209.70	212.70	SM	7.00E-05	2.30E-06	3	3.00	20	38	12.67	2.7	1.06E-04	0.6670
BM 115.c99	191	251	6000	203.60	206.60	ML	1.50E-05	4.92E-07	3	3.00	20	38	12.67	2.7	2.27E-05	0.1361
BM 116.c99	251	342	9100	199.10	202.10	ML	1.50E-05	4.92E-07	3	3.00	20	38	12.67	2.7	2.27E-05	0.2065
BM 125.c99	342	417	7550	194.00	197.00	CH	1.00E-07	3.28E-09	3	3.00	20	38	12.67	2.7	1.51E-07	0.0011
BM 124.c99	417	432	1500	193.00	196.00	CH	1.00E-07	3.28E-09	3	3.00	20	38	12.67	2.7	1.51E-07	0.0002
Totals			**43200**				**1.71E-05**									**1.1154**

Table 21 - Canal 2531

Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
BM 115.C99	0	60	6000	211.00	214.00	CL	1.00E-07	3.28E-09	3	3.00	10	28	9.33	2.3	1.15E-07	0.0007
BM 105.c98	60	120	6000	210.00	214.00	CL	1.00E-07	3.28E-09	3	4.00	10	34	8.50	2.2	1.40E-07	0.0008
Totals			**12000**				**1.00E-07**									**0.0015**

Table 22 - Canal 2532 (BluePoint Ditch)

Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
BM 121.c99	0	70	7000	192.00	207.00	ML	1.50E-05	4.92E-07	3	15.00		90	6.00		4.43E-05	0.3100
BM 123.c99	70	117	4700	193.00	208.00	CL	1.00E-07	3.28E-09	3	15.00		90	6.00		2.95E-07	0.0014
BM 107.c99	117	194	7700	196.00	210.00	ML	1.50E-05	4.92E-07	3	14.00		84	6.00		4.13E-05	0.3183
BM 106.c99	194	260	6600	200.00	212.00	CH	1.00E-07	3.28E-09	3	12.00		72	6.00		2.36E-07	0.0016
BM 105.c99	260	275	1500	200.50	214.00	CH	1.00E-07	3.28E-09	3	13.50		81	6.00		2.66E-07	0.0004
			27500				**8.06E-06**									**0.6317**

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Table 23 - Canal 2533

Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
BM 123.C99	0	17	1700	200.00	208.00	CH	1.00E-07	3.28E-09	3	8.00	15	63	7.88	2.05	2.60E-07	0.0004
41-bmu	17	35	1800	202.00	208.00	CH	1.00E-07	3.28E-09	3	6.00	15	51	8.50	2.2	2.11E-07	0.0004
Totals			**3500**				**1.00E-07**									**0.0008**

Table 24 - Canal 2540 (Shumaker BR)

Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
39-BMU	0	24	2350	232.50	235.50	CL	1.00E-07	3.28E-09	3	3.00	40	58	19.33	3.1	2.21E-07	0.0005
BM 41.c99	24	80	5650	226.38	229.38	ML	1.50E-05	4.92E-07	3	3.00	40	58	19.33	3.1	3.31E-05	0.1871
BM 40.c99	80	140	5950	218.50	221.50	ML	1.50E-05	4.92E-07	3	3.00	40	58	19.33	3.1	3.31E-05	0.1971
BM 39.c99	140	215	7500	217.50	220.50	CH	1.00E-07	3.28E-09	3	3.00	40	58	19.33	3.1	2.21E-07	0.0017
BM 117.c99	215	383	16850	211.50	214.50	CL	1.00E-07	3.28E-09	3	3.00	40	58	19.33	3.1	2.21E-07	0.0037
Totals			**38300**				**4.61E-06**									**0.3901**

Table 25 - Canal 3000

Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
bm82.C99	0	27	2700	232.00	238.00	ML	1.50E-05	4.92E-07	3	6.00	20	56	9.33	2.3	3.44E-05	0.0927
BM 49.C99	27	100	7300	230.00	236.00	CH	1.00E-07	3.28E-09	3	6.00	20	56	9.33	2.3	2.29E-07	0.0017
BM 47.C99	100	175	7500	229.00	235.00	CL	1.00E-07	3.28E-09	3	6.00	20	56	9.33	2.3	2.29E-07	0.0017
BM 46.C99	175	220	4500	228.00	234.00	CH	1.00E-07	3.28E-09	3	6.00	20	56	9.33	2.3	2.29E-07	0.0010
12-BMU	220	295	7500	227.00	233.00	CL	1.00E-07	3.28E-09	3	6.00	20	56	9.33	2.3	2.29E-07	0.0017
11-BMU	295	410	11500	224.00	231.00	CL	1.00E-07	3.28E-09	3	7.00	20	62	8.86	2.25	2.55E-07	0.0029
37-BMU	410	495	8500	222.00	231.00	CH	1.00E-07	3.28E-09	3	9.00	20	74	8.22	2.15	3.06E-07	0.0026
Totals			**49500**				**9.13E-07**									**0.1044**

Bayou Meto
Seepage Loss Estimate

Table 26 - Canal 4000 / 4100

Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
68-BMU	0	50	5000	240.00	246.00	CH	1.00E-07	3.28E-09	3	6.00	20	56	9.33	2.3	2.29E-07	0.0011
14-BMU	50	75	2500	240.00	245.00	CH	1.00E-07	3.28E-09	3	5.00	20	50	10.00	2.4	2.03E-07	0.0005
14-BMU	75	113	3800	230.00	244.00	CH	1.00E-07	3.28E-09	3	14.00	0	84	6.00	1.7	3.54E-07	0.0013
	113	113	0				0.00E+00	0.00E+00	3	0.00		0	#####		0.00E+00	0.0000
BM 28.c99	113	151	3800	237.00	244.00	CH	1.00E-07	3.28E-09	3	7.00	18	60	8.57	2.2	2.47E-07	0.0009
			15100				**1.00E-07**									**0.0039**

Table 27 - Canal 4110 (Ricky Br)

Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
BM 28.c99	0	33	3250	236.00	241.00	CH	1.00E-07	3.28E-09	3	5.00	40	70	14.00	2.8	2.76E-07	0.0009
BM 26.c99	33	77	4400	236.00	241.00	ML	1.50E-05	4.92E-07	3	5.00	40	70	14.00	2.8	4.13E-05	0.1819
BM 24.c99	77	124	4750	235.50	240.50	SM	7.00E-05	2.30E-06	3	5.00	40	70	14.00	2.8	1.93E-04	0.9163
BM 25.c99	124	174	5000	234.00	239.00	ML	1.50E-05	4.92E-07	3	5.00	40	70	14.00	2.8	4.13E-05	0.2067
BM 29.c99	174	226	5150	233.50	238.50	ML	1.50E-05	4.92E-07	3	5.00	40	70	14.00	2.8	4.13E-05	0.2129
BM 30.c99	226	282	5600	233.00	238.00	ML	1.50E-05	4.92E-07	3	5.00	60	90	18.00	3	5.17E-05	0.2894
BM 31.c99	282	300	1850	233.00	238.00	ML	1.50E-05	4.92E-07	3	5.00	60	90	18.00	3	5.17E-05	0.0956
			30000				**2.21E-05**									**1.9037**

Bayou Meto
Seepage Loss Estimate

Table 28 - Canal 4111

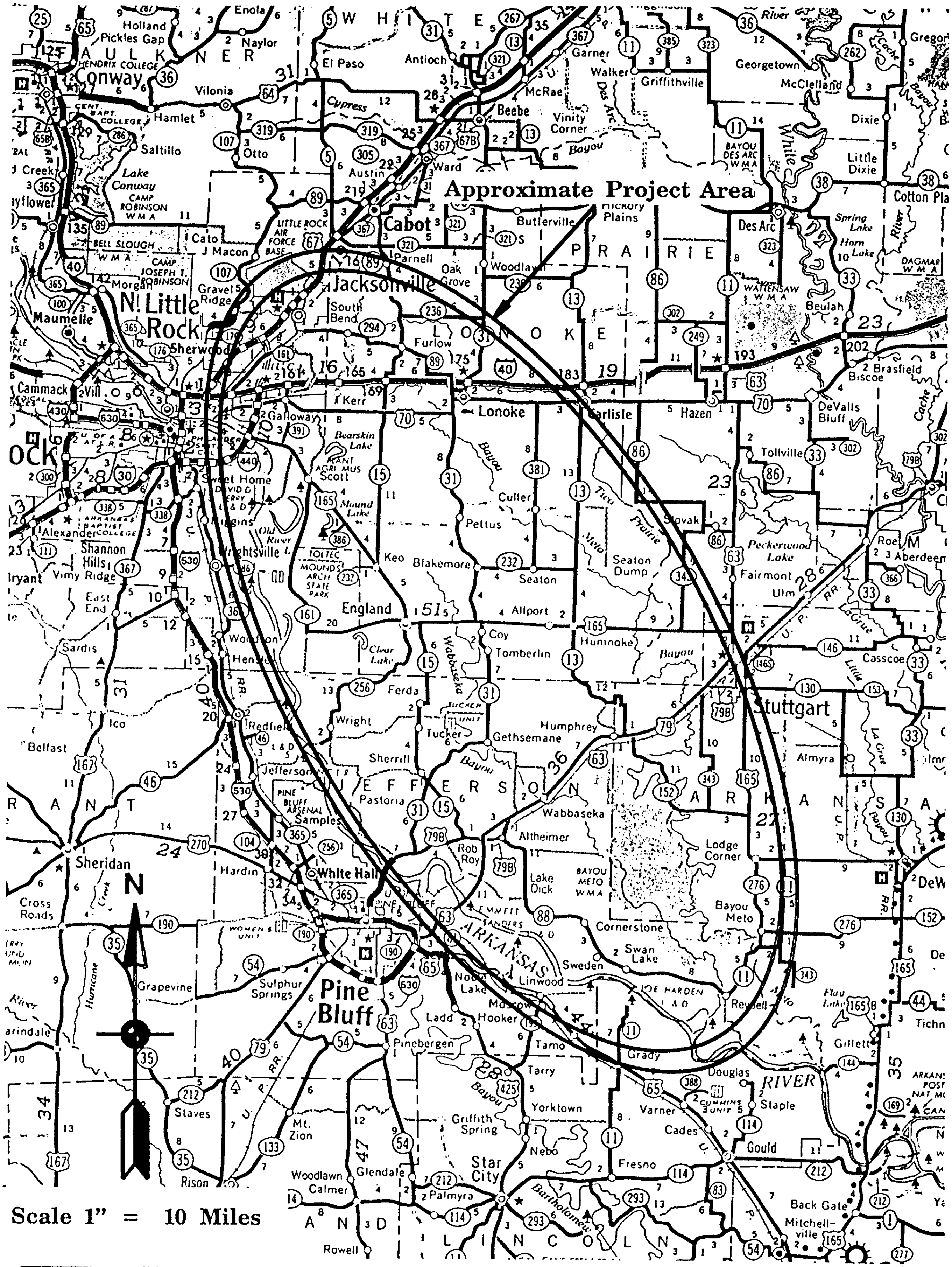
Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
BM 30.c99	0	25	2500	230.00	234.00	ML	1.50E-05	4.92E-07	3	4.00	10	34	8.50	2.2	2.11E-05	0.0527
BM 60.c99	25	60	3500	225.00	233.00	CH	1.00E-07	3.28E-09	3	8.00	10	58	7.25	1.95	2.41E-07	0.0008
BM 37.c99	60	200	14000	222.00	233.00	CL	1.00E-07	3.28E-09	3	11.00	10	76	6.91	1.9	3.18E-07	0.0045
BM 10.c99	200	300	10000	220.00	232.00	CL	1.00E-07	3.28E-09	3	12.00	10	82	6.83	1.9	3.44E-07	0.0034
BM 10.c99	300	380	8000	224.00	225.00	CL	1.00E-07	3.28E-09	3	1.00	10	16	16.00	2.9	6.20E-08	0.0005
BM 13.c99	380	480	10000	222.00	225.00	CH	1.00E-07	3.28E-09	3	3.00	10	28	9.33	2.3	1.15E-07	0.0011
BM 13.c99	480	545	6500	210.00	225.00	ML	1.50E-05	4.92E-07	3	15.00	10	100	6.67	1.85	6.29E-05	0.4086
	545	545	0				0.00E+00	0.00E+00	3	0.00		0	#####		0.00E+00	0.0000
			54500				**2.56E-06**									**0.4717**

Table 29 - Canal 4112

Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
BM 31.c99	0	41	4100	233.00	238.00	CL	1.00E-07	3.28E-09	3	5.00	20	50	10.00	2.4	2.03E-07	0.0008
27-BMU	41	137	9600	232.00	237.00	CH	1.00E-07	3.28E-09	3	5.00	20	50	10.00	2.4	2.03E-07	0.0020
28-BMU	137	214	7700	231.50	236.50	CL	1.00E-07	3.28E-09	3	5.00	20	50	10.00	2.4	2.03E-07	0.0016
BM 35.c99	214	300	8600	230.00	236.50	ML	1.50E-05	4.92E-07	3	6.50	20	59	9.08	2.25	3.62E-05	0.3116
			30000				**4.37E-06**									**0.3160**

Table 30 - Canal 4113

Boring #	Sta. # From x100 ft	Sta. # To x100 ft	Length L ft	AVERAGE BOTTOM ELEV. ft	Water Elevation ft	Bottom Soil Type	Permeability Canal Bottom k (cm/sec)	Permeability Canal Bottom k (ft/sec)	Channel Slope m (1V:_H)	Channel Head H (ft)	Bottom Width BW (ft)	Wetted Perimeter BW+2mH B (ft)	B/H	Harr Fig 9-3 A	Seepage Loss /ft q=k*(B+AH) q (cfs / ft)	Seepage Loss (Total) Q=q*L Q (cfs)
28-BMU	0	53	5300	231.50	235.00	CL	1.00E-07	3.28E-09	3	3.50	8	29	8.29		9.51E-08	0.0005
			5300				**1.00E-07**									**0.0005**



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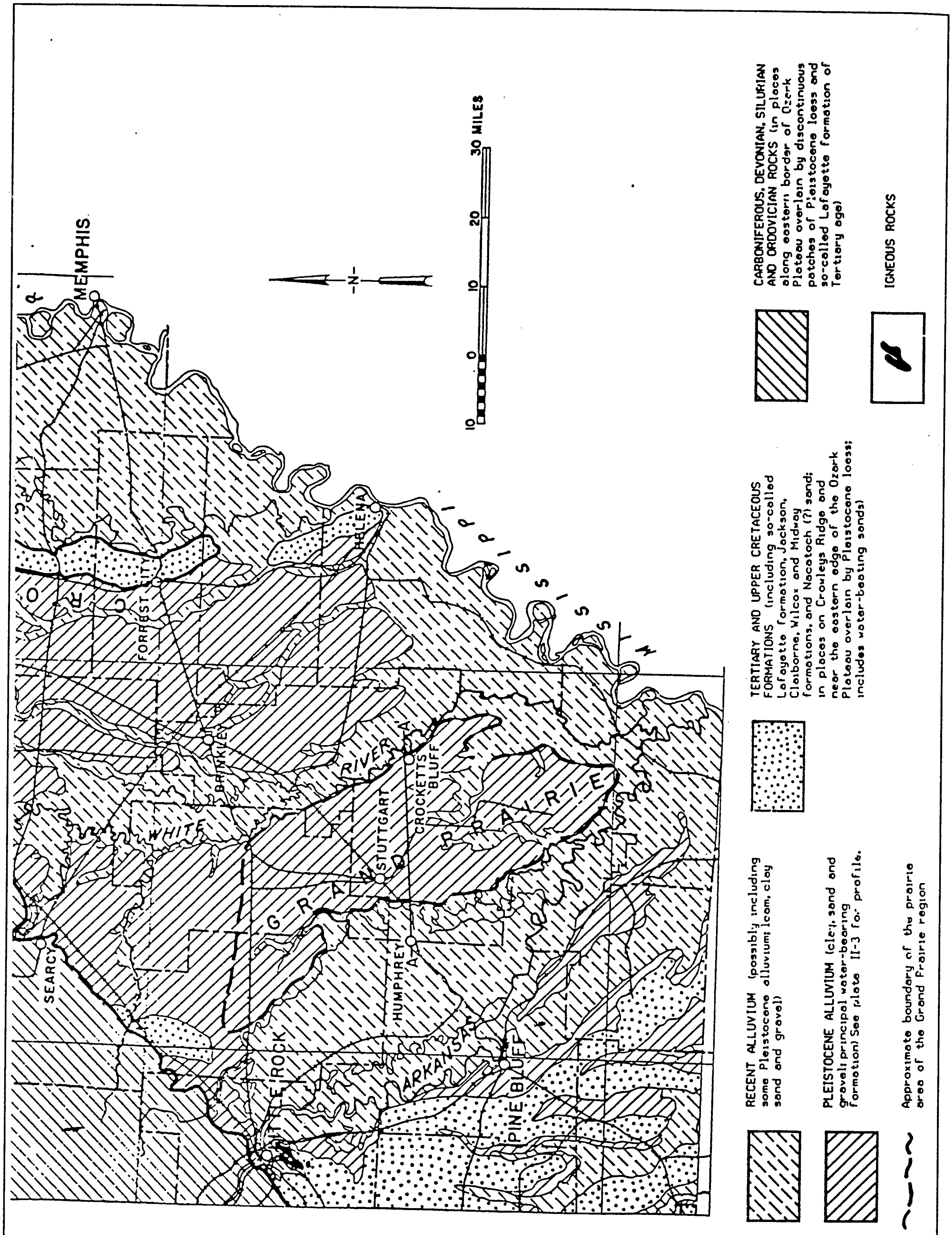
Project Location Map

PLATE

II-1



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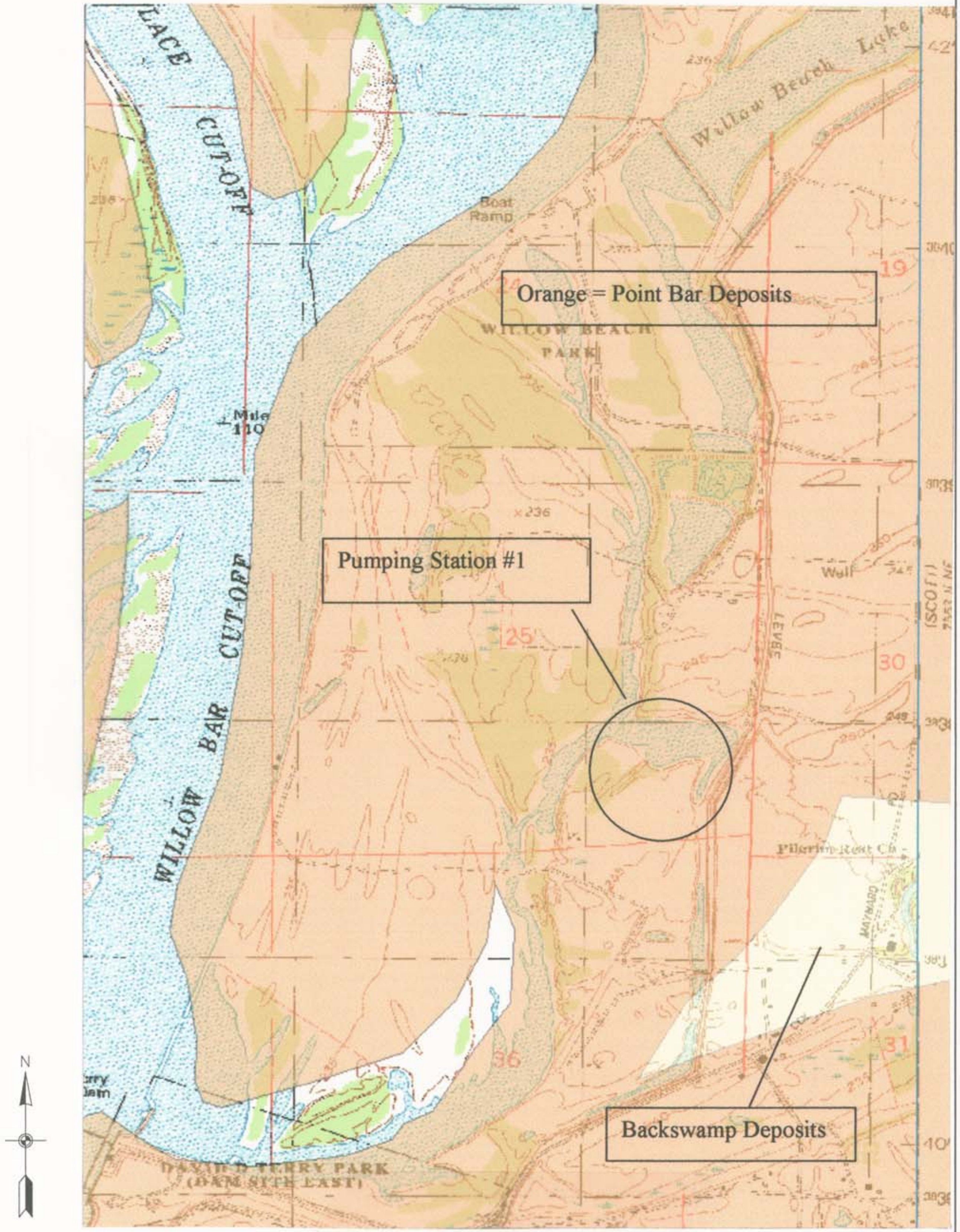
General Geology Map

PLATE

II–2



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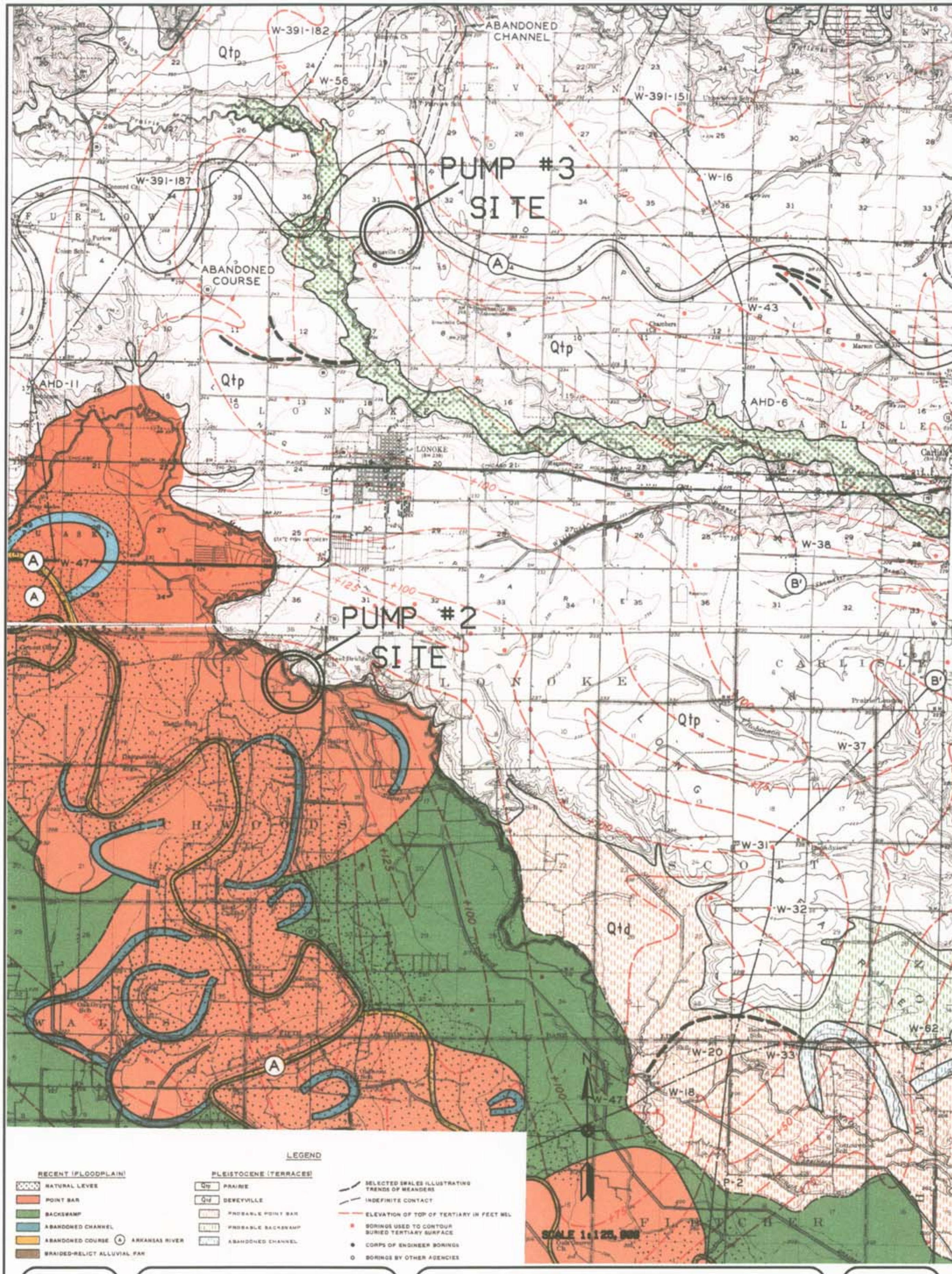
PUMPING STATION #1
GEOLOGIC MAP

PLATE
II-3



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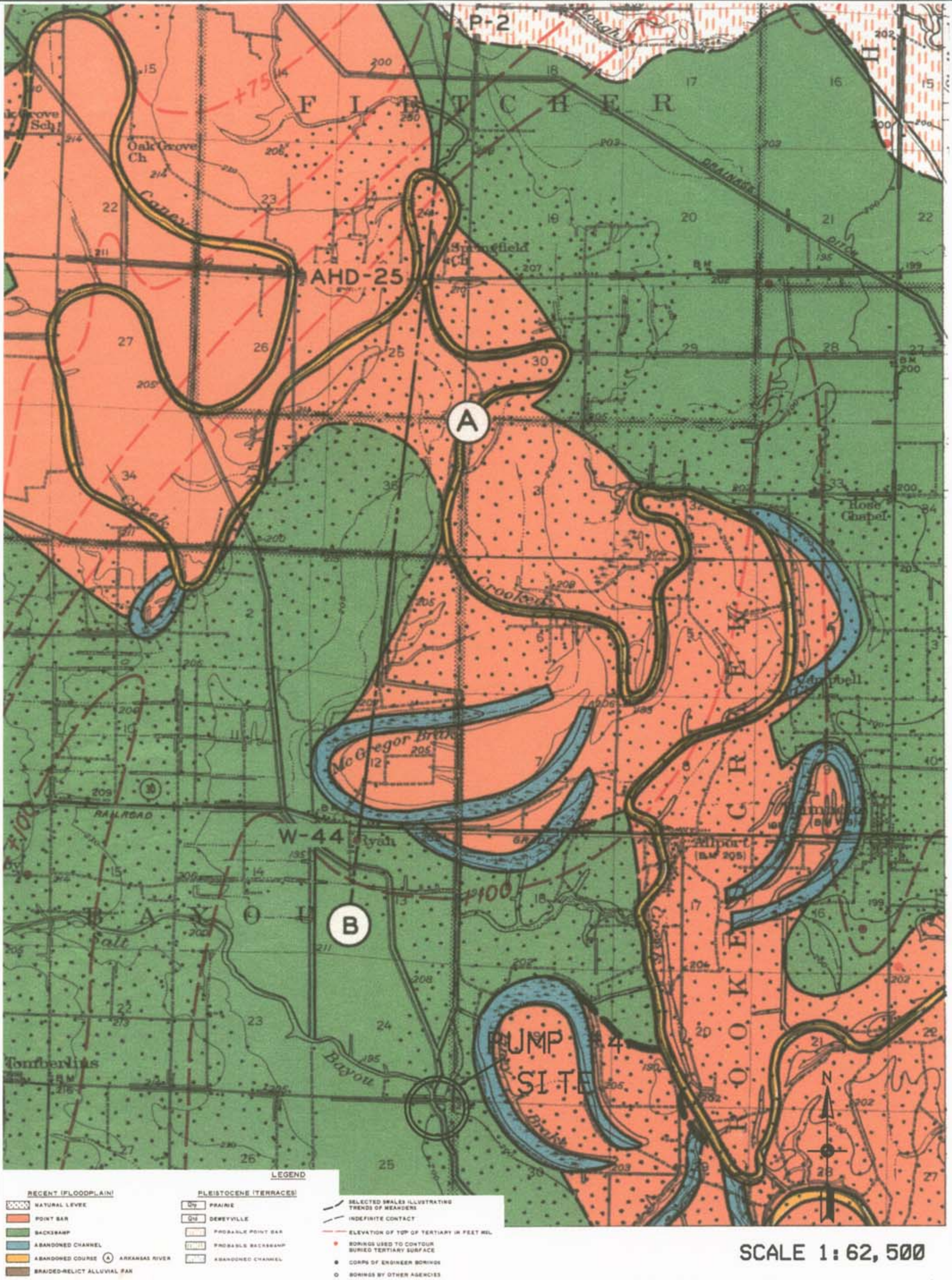
PUMP STATION #2 & #3 GEOLOGIC MAP

PLATE II-4



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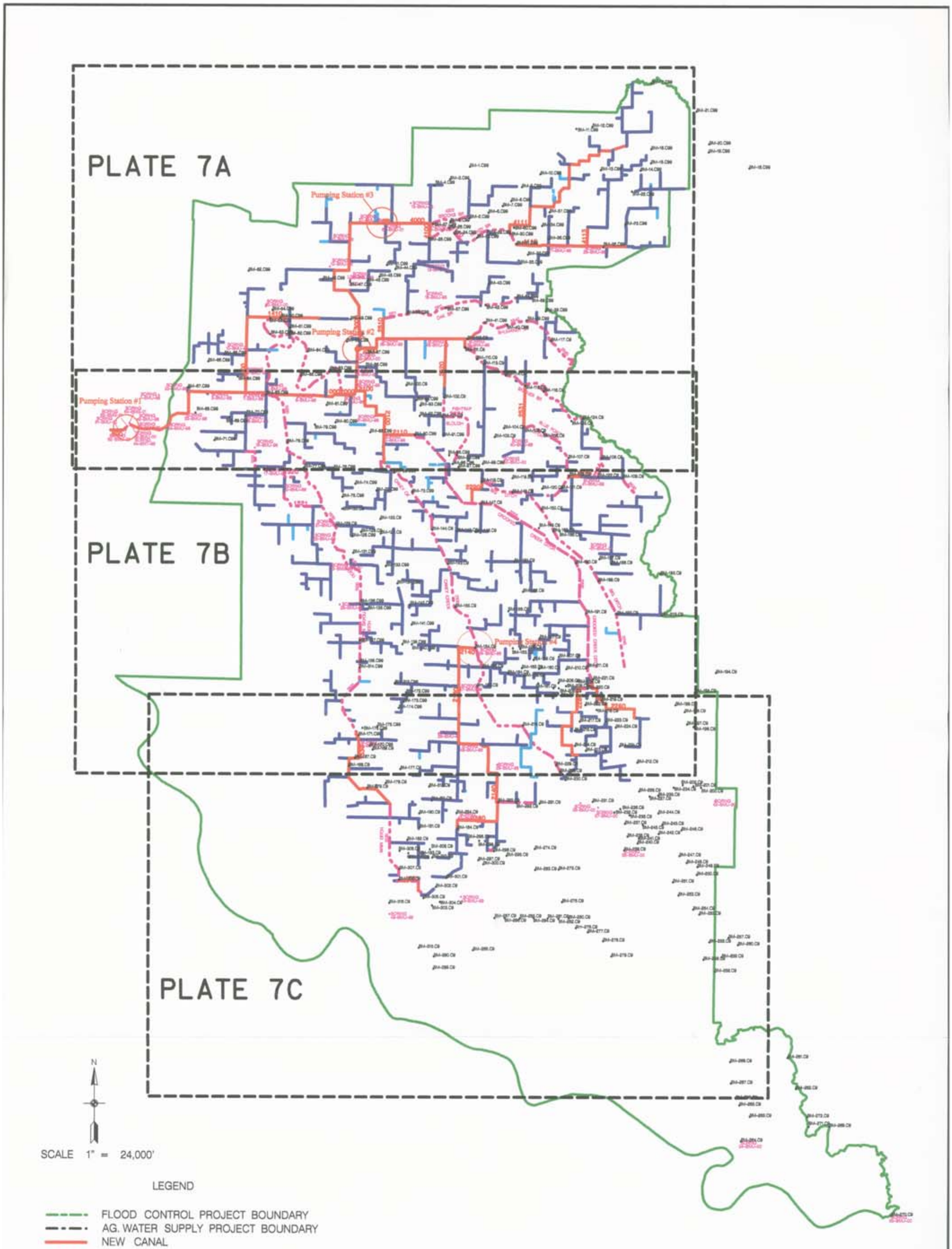
PUMP STATION #4 GEOLOGIC MAP

PLATE

II–5



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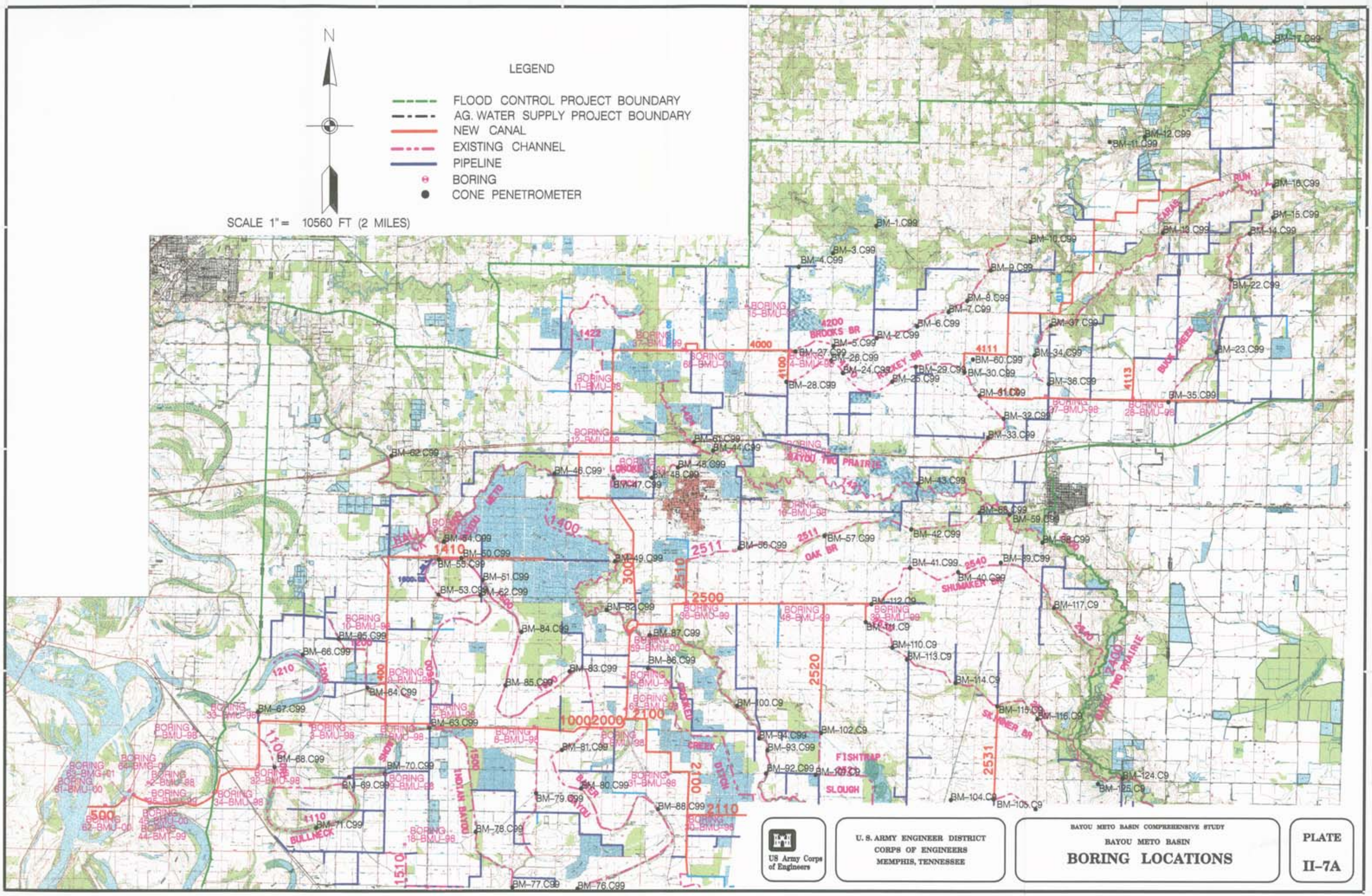
BORING LOCATIONS INDEX

PLATE II-6


N
LEGEND
FLOOD CONTROL PROJECT BOUNDARY
AG. WATER SUPPLY PROJECT BOUNDARY
NEW CANAL
EXISTING CHANNEL
PIPELINE
BORING
CONE PENETROMETER
SCALE 1" = 10560 FT (2 MILES)
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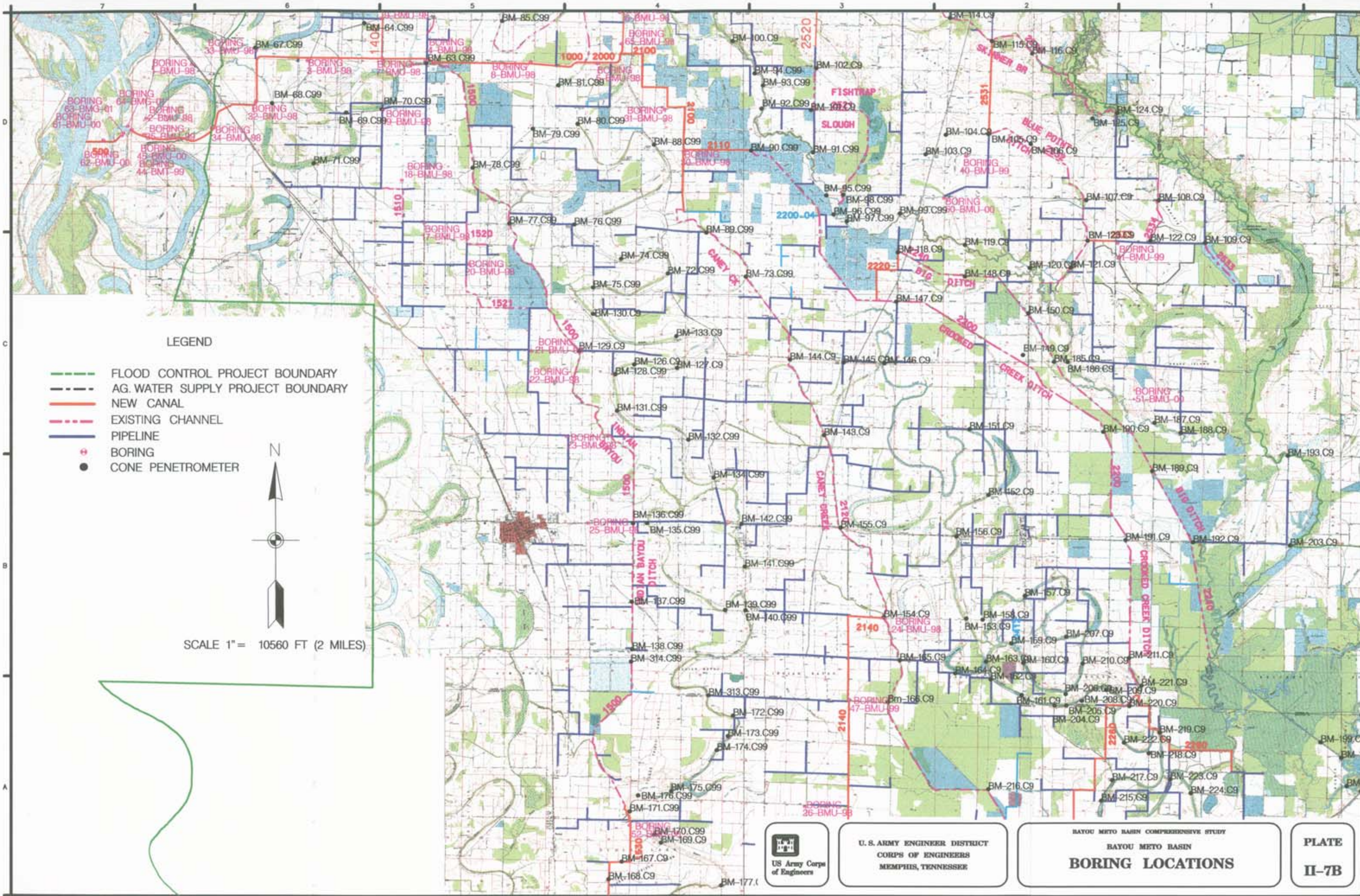
BAYOU METO BASIN COMPREHENSIVE STUDY
BAYOU METO BASIN
BORING LOCATIONS
PLATE
II-7A


LEGEND
FLOOD CONTROL PROJECT BOUNDARY
AG. WATER SUPPLY PROJECT BOUNDARY
NEW CANAL
EXISTING CHANNEL
PIPELINE
BORING
CONE PENETROMETER
N
SCALE 1" = 10560 FT (2 MILES)
FISHTRAP
SLOUGH
SKINNER BR
BLUE POINT DITCH
BIG DITCH
CROOKED CREEK DITCH
CANEY CREEK
CANEY CK
INDIAN BAYOU
INDIAN BAYOU DITCH
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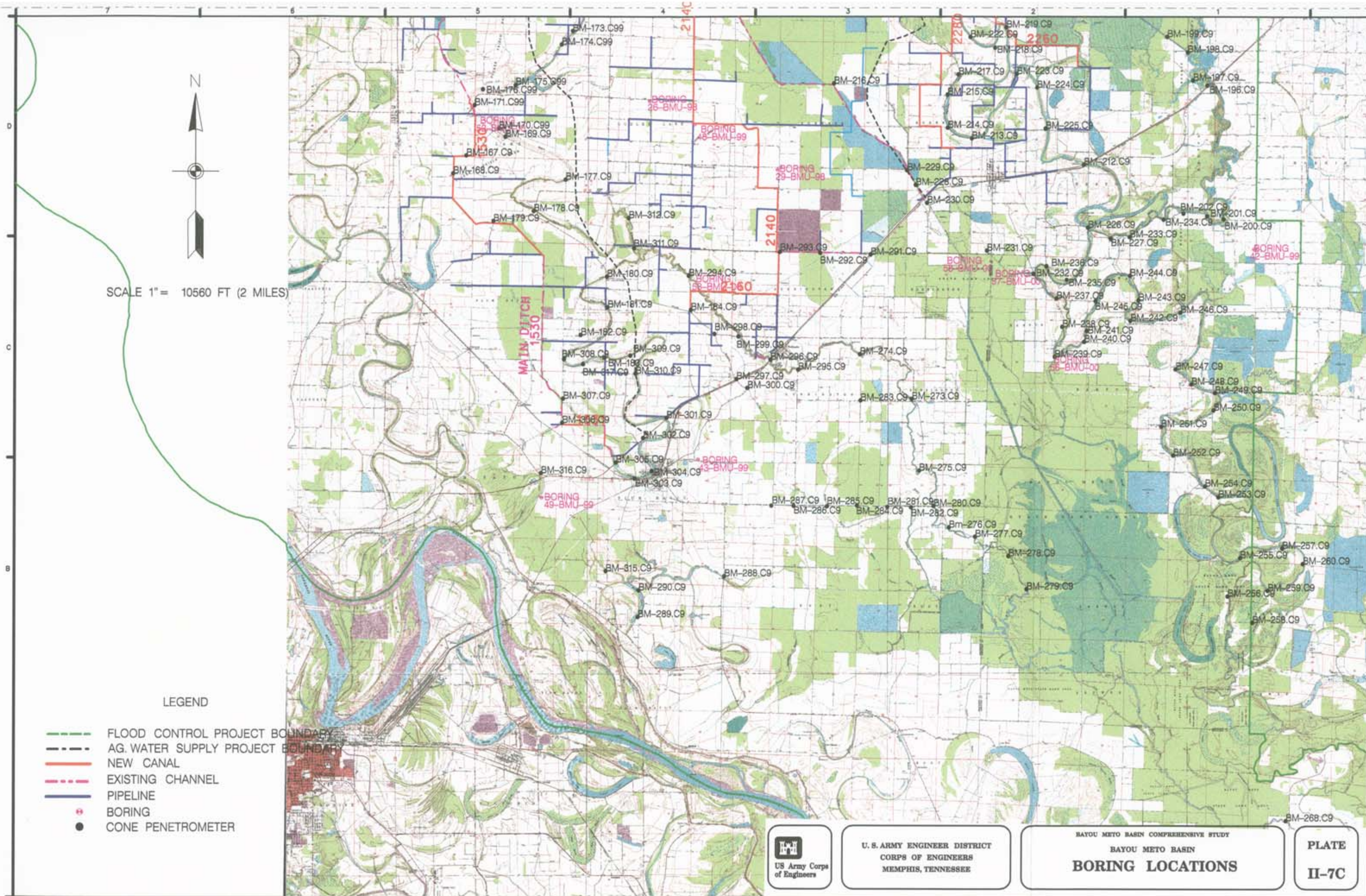
BAYOU METO BASIN
BORING LOCATIONS
PLATE
II-7B



N
SCALE 1" = 10560 FT (2 MILES)
LEGEND
FLOOD CONTROL PROJECT BOUNDARY
AG. WATER SUPPLY PROJECT BOUNDARY
NEW CANAL
EXISTING CHANNEL
PIPELINE
BORING
CONE PENETROMETER
MAIN DITCH
1530
2140
2160
2260
2280
BORING 26-BMU-98
BORING 48-BMU-99
BORING 29-BMU-98
BORING 43-BMU-99
BORING 49-BMU-99
BORING 42-BMU-99
BORING 37-BMU-00
BM-173.C99
BM-174.C99
BM-175.C99
BM-176.C99
BM-171.C99
BM-170.C99
BM-169.C9
BM-167.C9
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BM-258.C9
BM-268.C9
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BORING LOCATIONS
PLATE
II-7C

BOR. 1-BMU-98
LAT:34°42.0'15.3"N;
LON:92°06'16.5"W
BOR. TYPE-AUG, 5" TUBE & SS
GROUND WATER EL. 233.2
2 SEP 98

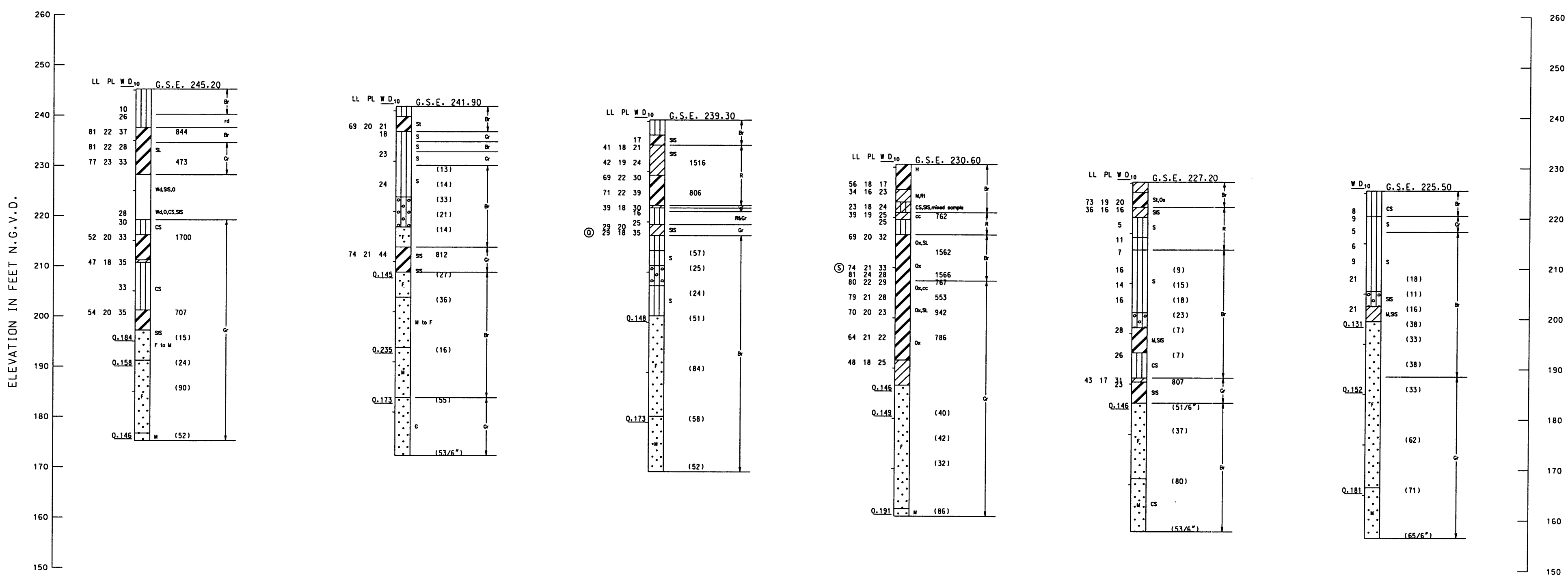
BOR. 2-BMU-98
LAT:34°41'06.3"N;
LON:92°07'30.0"W
BOR. TYPE-AUG, 5" TUBE & SS
GROUND WATER EL. 233.3
1 SEP 98

BOR. 3-BMU-98
LAT:34°42'21.5"N;
LON:92°03'23.8"W
BOR. TYPE-AUG, 5" TUBE & SS
GROUND WATER EL. 216.5
3 SEP 98

BOR. 4-BMU-98
LAT:34°42'19.8"N;
LON:92°00'28.7"W
BOR. TYPE-AUG, 5" TUBE & SS
GROUND WATER EL. 222.3
4 SEP 98

BOR. 5-BMU-98
LAT:34°42'16.4"N;
LON:91°56'07.5"W
BOR. TYPE-AUG, 5" TUBE & SS
9 SEP 98

BOR. 6-BMU-98
LAT:34°43'20.1"N;
LON:91°55'33.2"W
BOR. TYPE-AUG, 5" TUBE & SS
9 SEP 98



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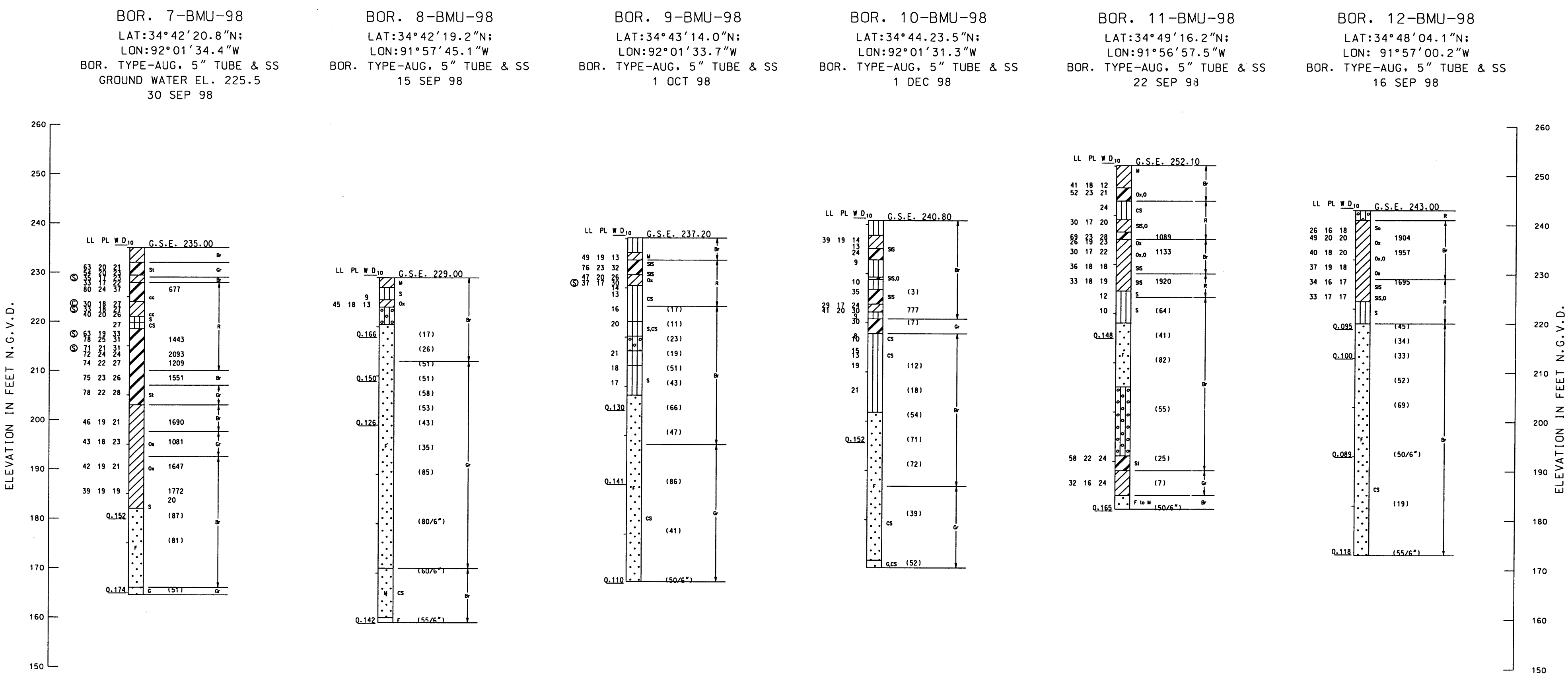
Boring Logs

PLATE
II-8



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Boring Logs

PLATE
II-9

BOR. 13-BMU-98
LAT:34°48'10.8"N;
LON:91°51'36.9"W
BOR. TYPE-AUG, 5" TUBE & SS
25 SEP 98

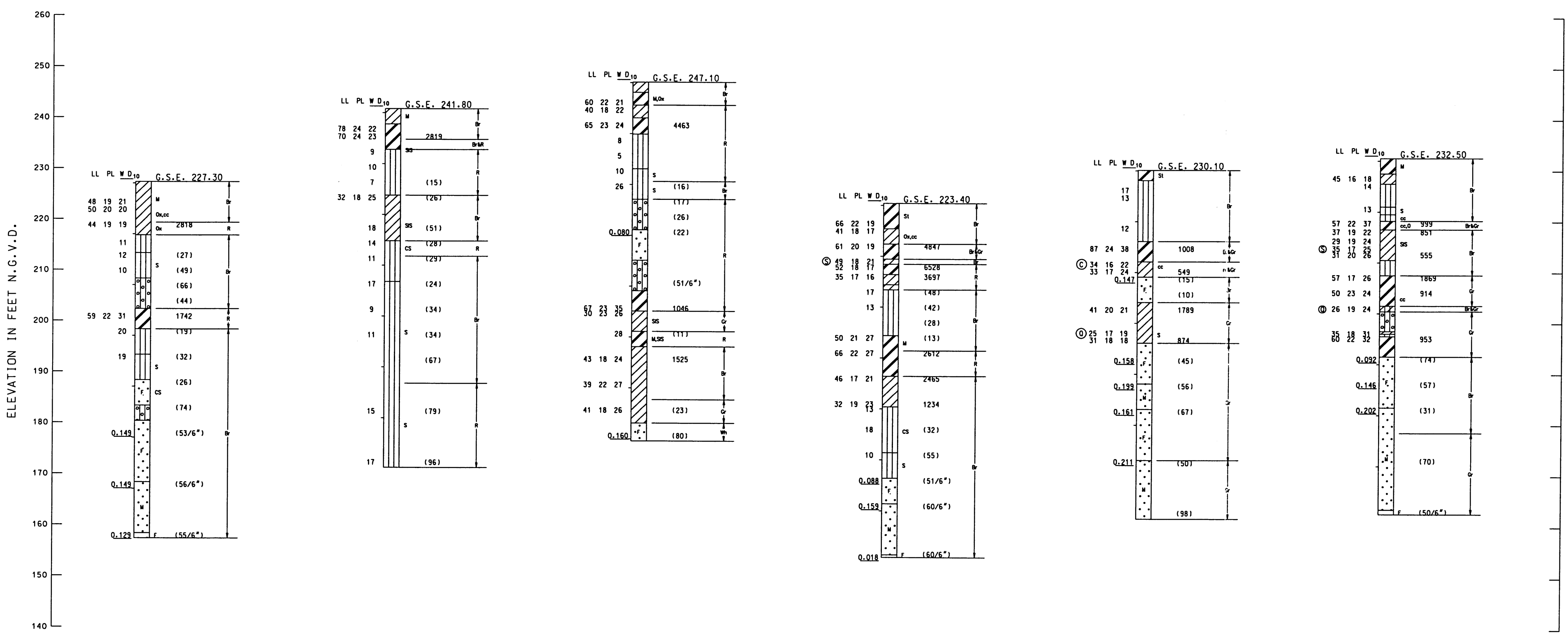
BOR. 14-BMU-98
LAT:34°50'06.6"N;
LON:91°51'28.9"W
BOR. TYPE-AUG, 5" TUBE & SS
24 SEP 98

BOR. 15-BMU-98
LAT:34°50'59.5"N;
LON:91°52'31.4"W
BOR. TYPE-AUG, 5" TUBE & SS
22 SEP 98

BOR. 16-BMU-98
LAT:34°47'00.6"N;
LON:91°51'36.9"W
BOR. TYPE-AUG, 5" TUBE & SS
29 SEP 98

BOR. 17-BMU-98
LAT:34°39'00.52"N;
LON:92°00'29.85"W
BOR. TYPE-AUG, 5" TUBE & SS
19 OCT 98

BOR. 18-BMU-98
LAT:34°39'41.74"N;
LON:92°01'04.26"W
BOR. TYPE-AUG, 5" TUBE & SS
GROUND WATER EL. 215.3
29 OCT 98



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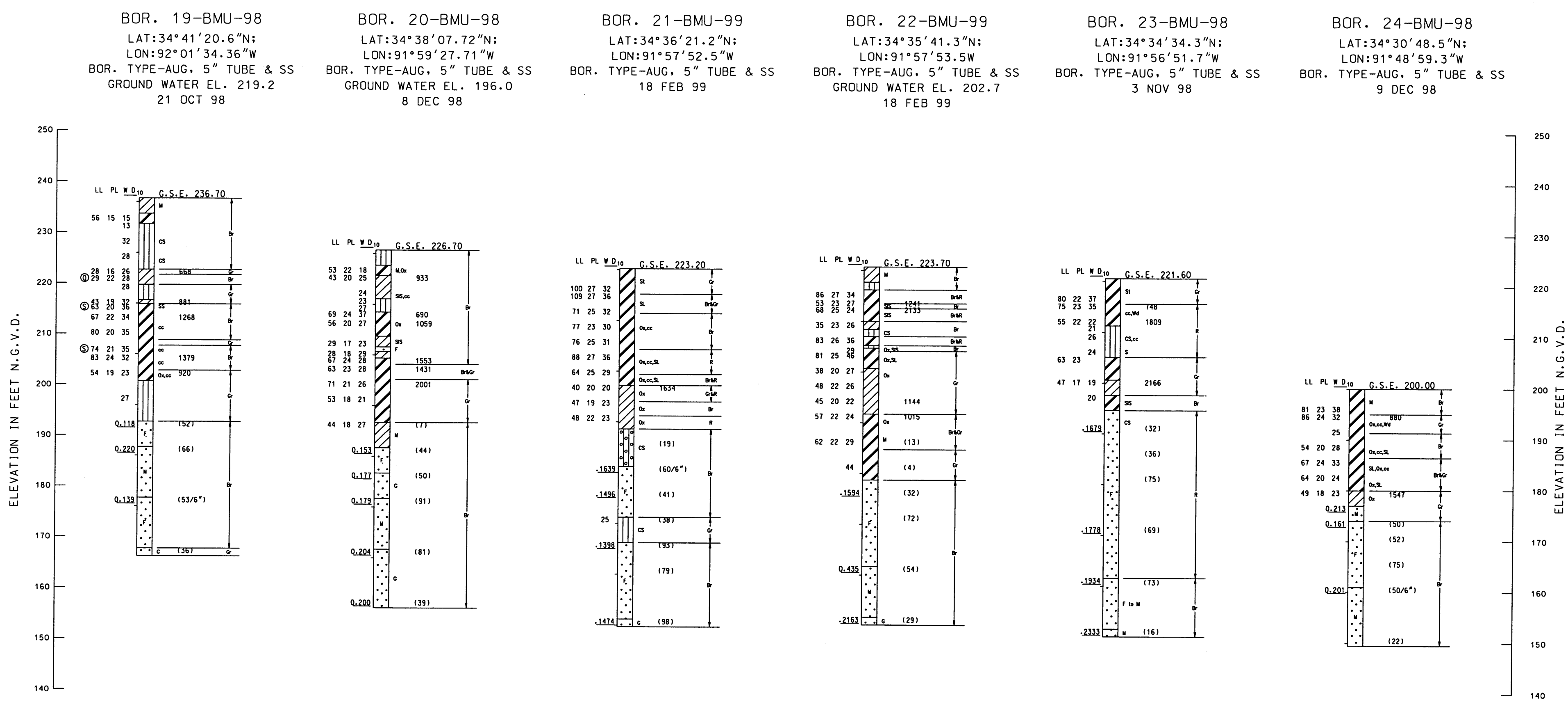
Boring Logs

PLATE
II-10



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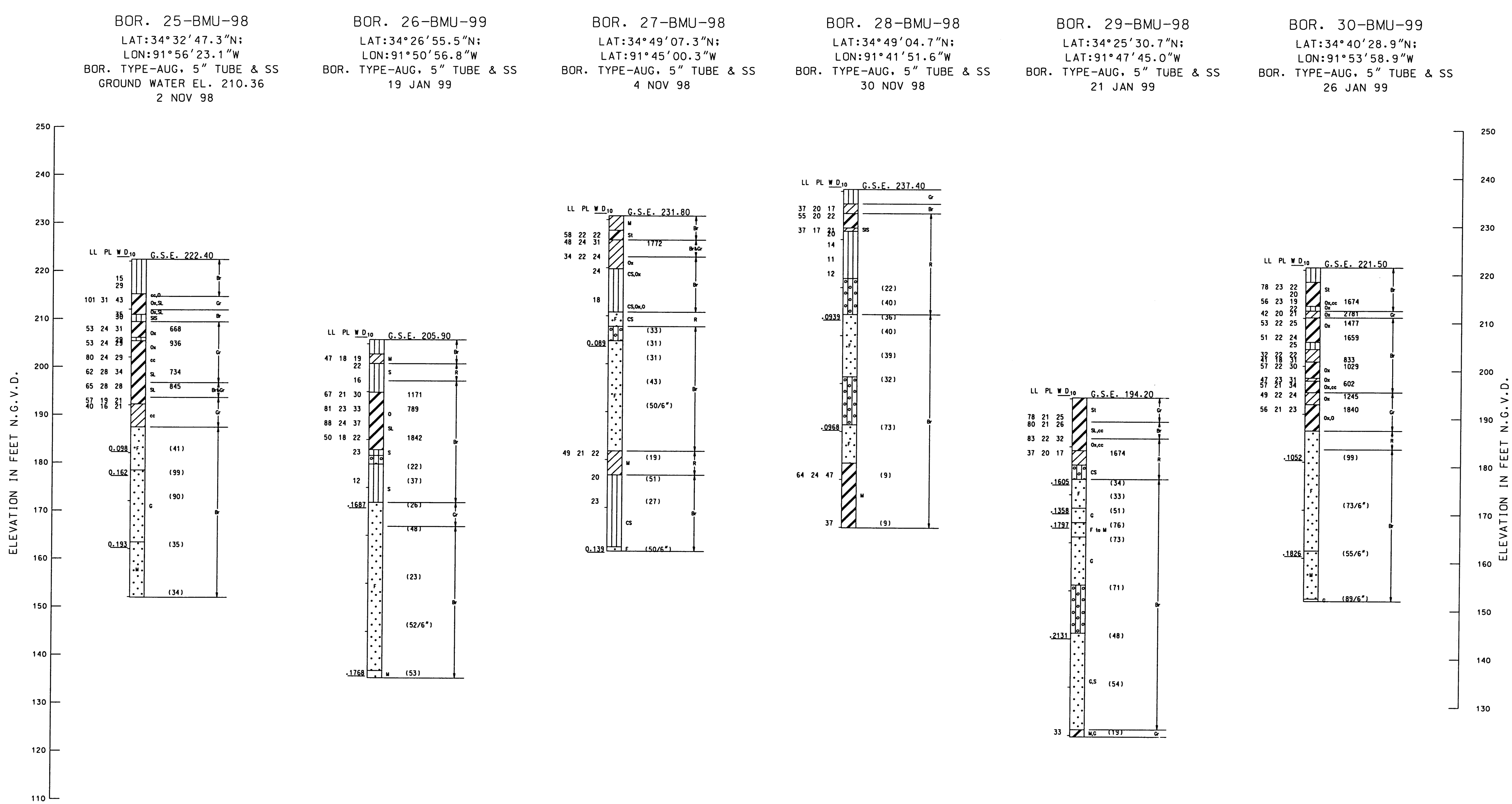
Boring Logs

PLATE
II-11



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BAYOU METO BASIN COMPREHENSIVE STUDY
BAYOU METO BASIN
Boring Logs

PLATE
II-12

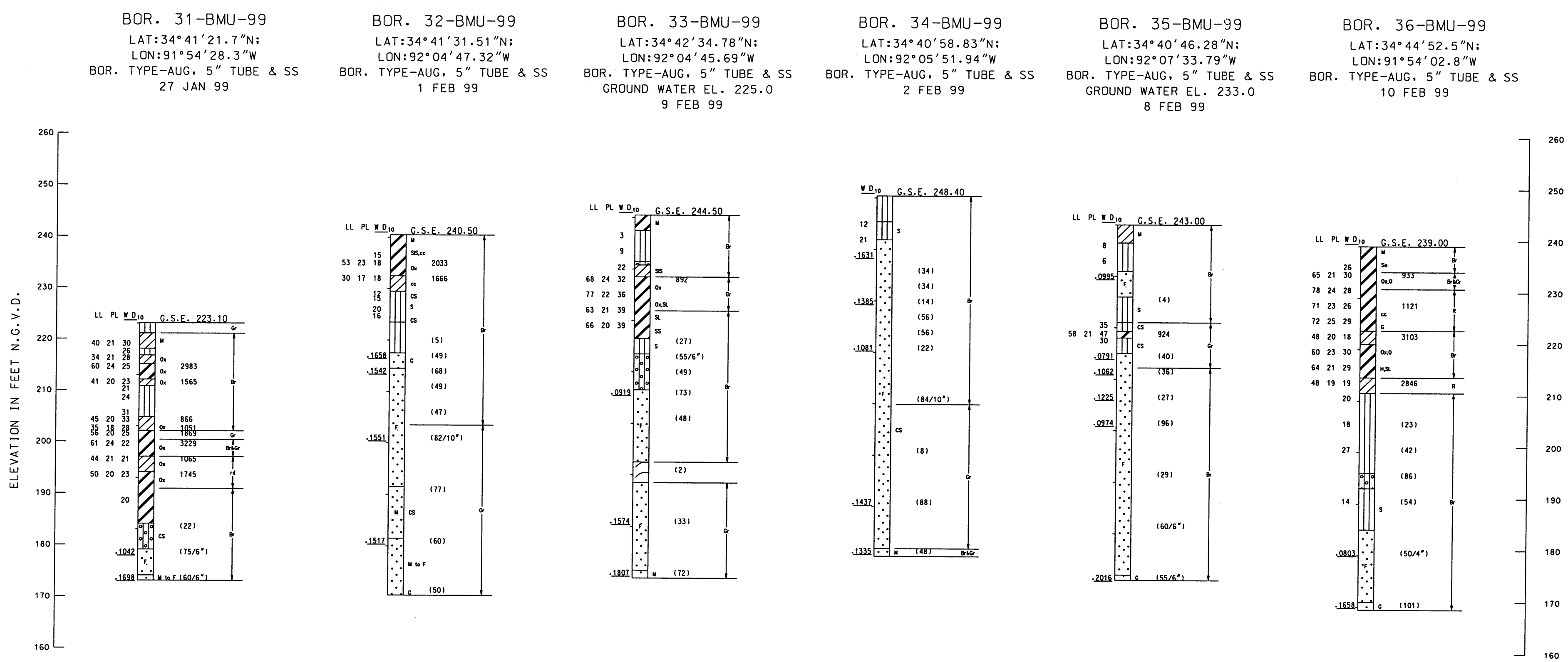

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PLATE
II-13

BOR. 37-BMU-99
LAT:34°50'07.1"N;
LON:91°54'10.0"W
BOR. TYPE-AUG, 5" TUBE & SS
GROUND WATER EL. 220.0
24 FEB 99

BOR. 38-BMU-99
LAT:34°47'31.6"N;
LON:91°55'52.4W
BOR. TYPE-AUG, 5" TUBE & SS
23 FEB 99

BOR. 39-BMU-99
LAT:34°44'51.7"N;
LON:91°49'31.1"W
BOR. TYPE-AUG, 5" TUBE & SS
2 MAR 99

BOR. 40-BMU-99
LAT:34°40'22.2"N;
LON:91°46'54.4"W
BOR. TYPE-AUG, 5" TUBE & SS
3 MAR 99

BOR. 41-BMU-99
LAT:34°38'35.3"N;
LON:91°43'07.9"W
BOR. TYPE-AUG, 5" TUBE & SS
10 MAR 99

BOR. 42-BMU-99
LAT:34°23'47.4"N;
LON:91°35'47.7"W
BOR. TYPE-AUG, 5" TUBE & SS
30 MAR 99

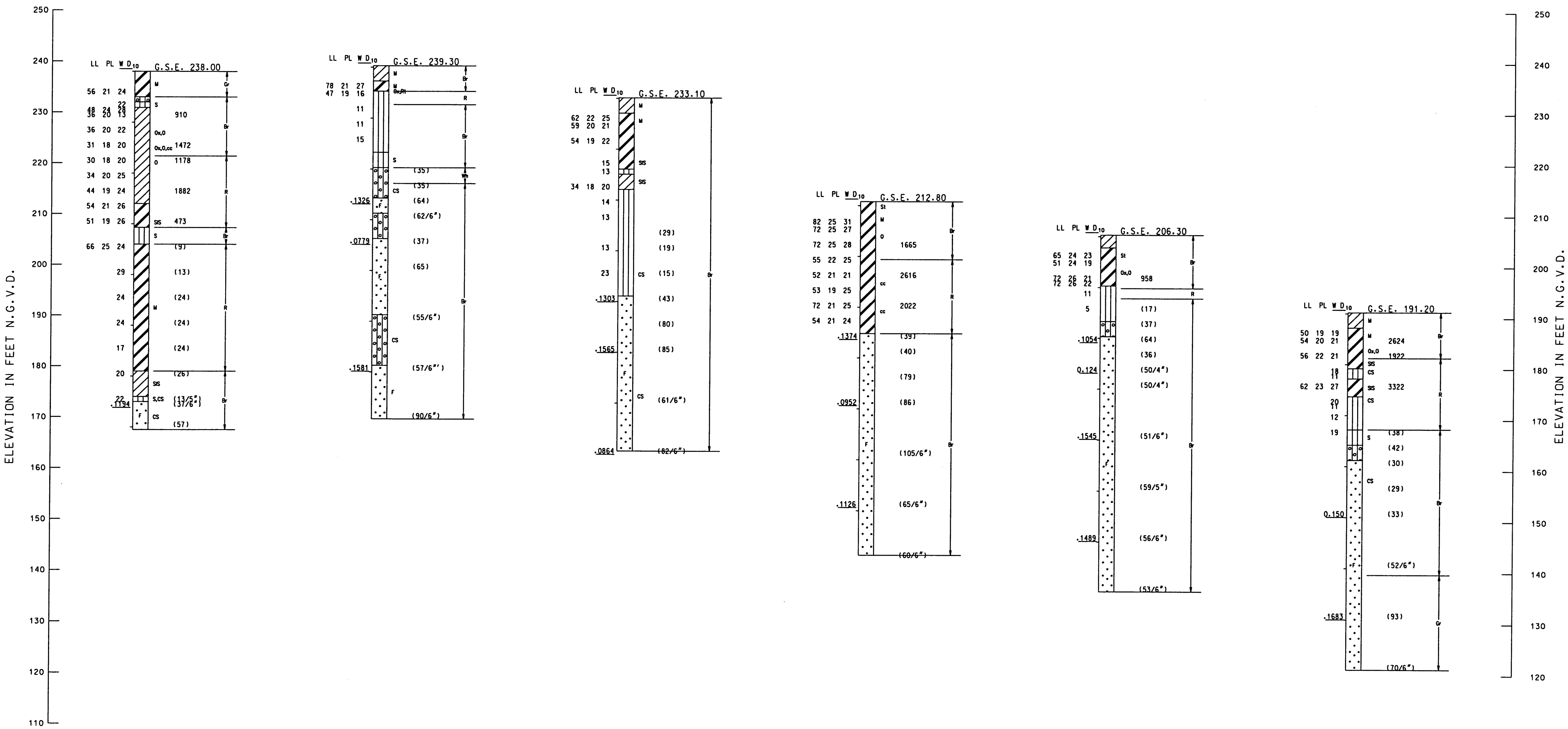

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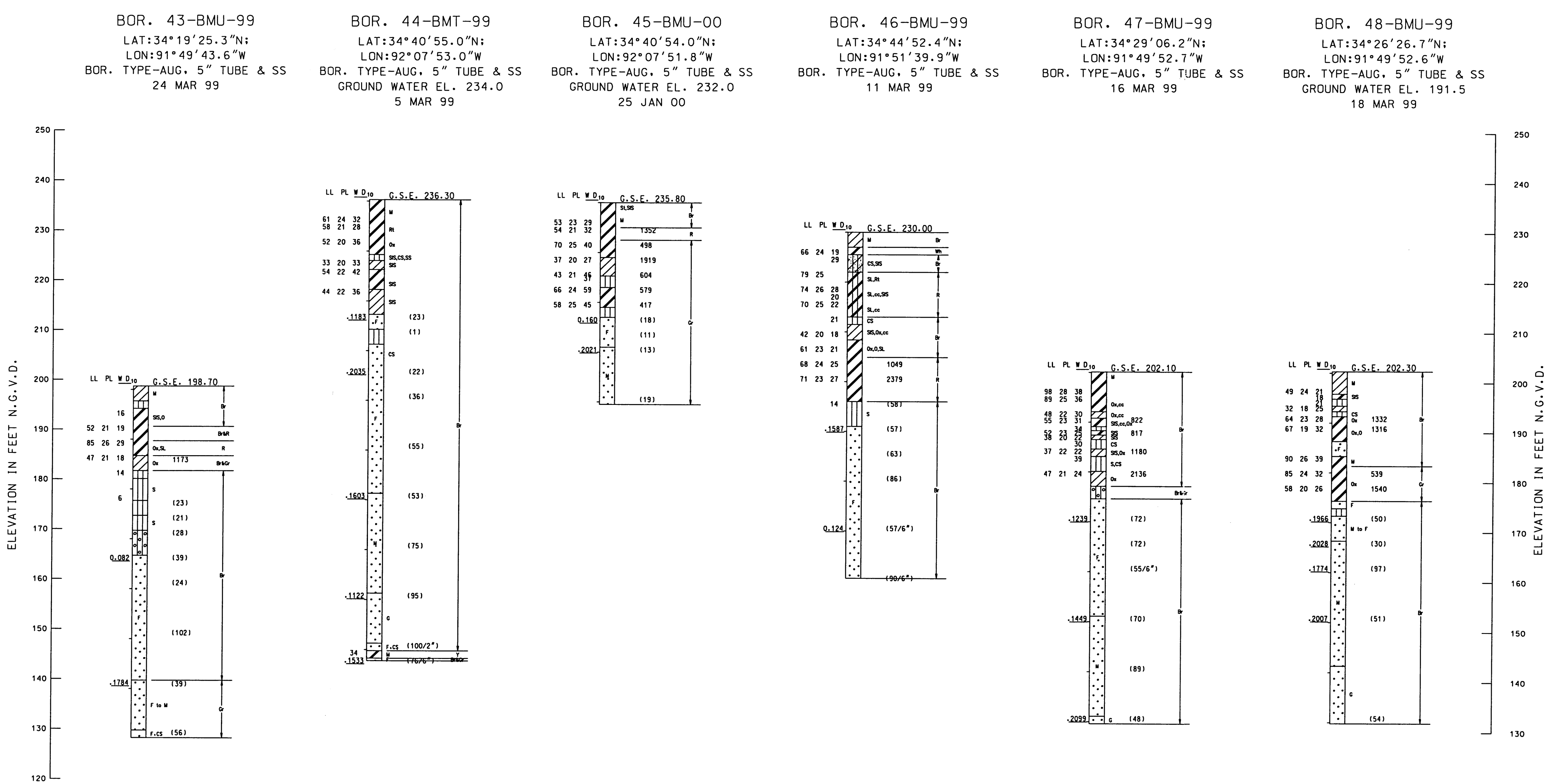
PLATE
II-14



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BAYOU METO BASIN
Boring Logs

PLATE
II-15

BOR. 49-BMU-99
LAT:34°18'40.7"N;
LON:91°53'35.9"W
BOR. TYPE-AUG, 5" TUBE & SS
GROUND WATER EL. 204.8
22 MAR 99

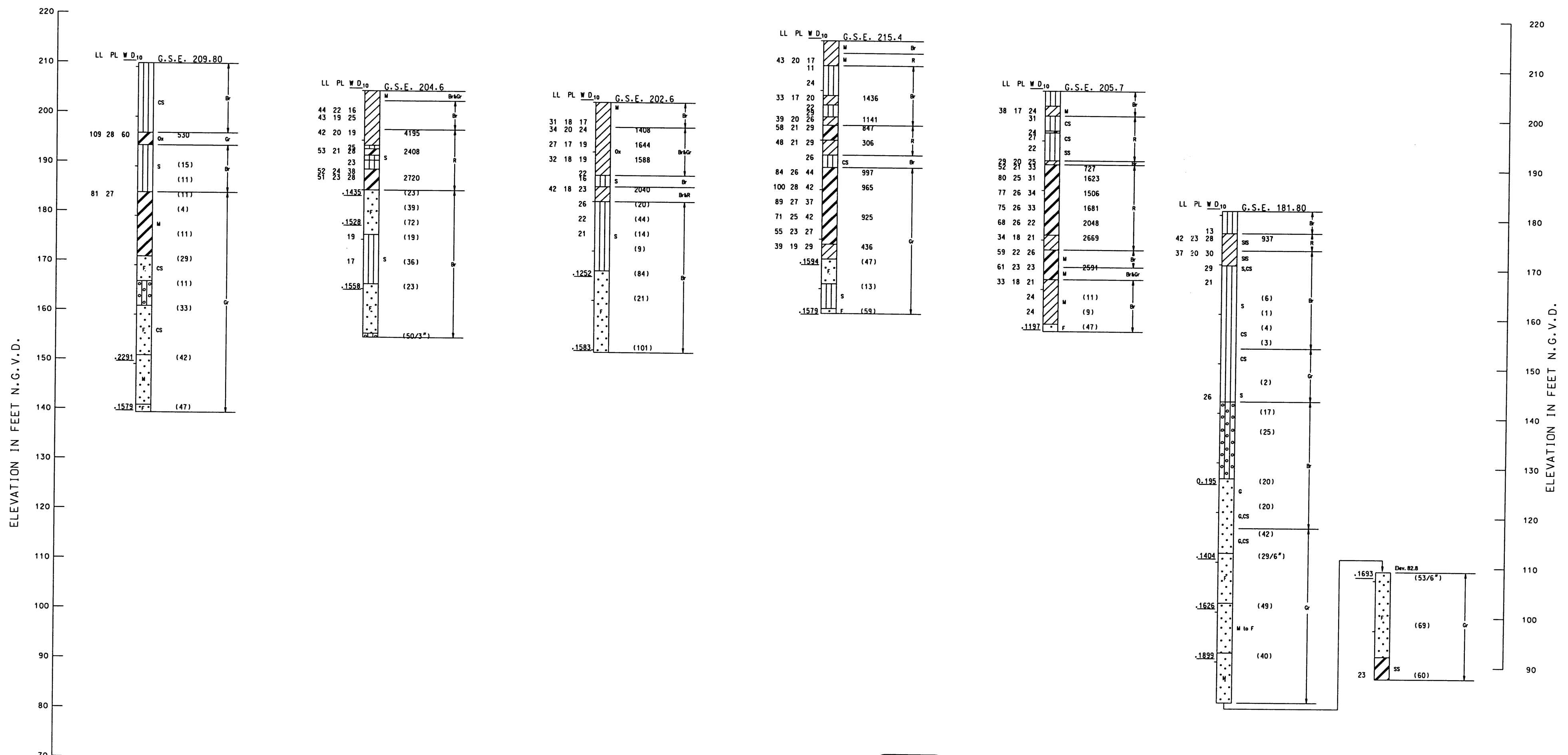
BOR. 50-BMU-00
LAT:34°39'29.24"N;
LON:91°47'27.93"W
BOR. TYPE-AUG, 5" TUBE & SS
25 JAN 00

BOR. 51-BMU-00
LAT:34°35'19.12"N;
LON:91°42'41.47"W
BOR. TYPE-AUG, 5" TUBE & SS
31 JAN 00

BOR. 52-BMU-00
LAT:34°26'31.16"N;
LON:91°55'21.92"W
BOR. TYPE-AUG, 5" TUBE & SS
GROUND WATER EL. 196.4
1 FEB 00

BOR. 53-BMU-00
LAT:34°22'56.43"N;
LON:91°49'56.68"W
BOR. TYPE-AUG, 5" TUBE & SS
GROUND WATER EL. 198.1
4 FEB 00

BOR. 54-BMU-00
LAT:34°8'17.53"N;
LON:91°34'25.40"W
BOR. TYPE-AUG, 5" TUBE & SS
11 FEB 00



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Boring Logs

PLATE
II-16

BOR. 55-BMU-00
LAT:34°04'53.99"N;
LON:91°26'14.69"W
BOR. TYPE-AUG, 5" TUBE & SS
GROUND WATER EL. 162.6
8 FEB 00

BOR. 56-BMU-00
LAT:34°21'33.99"N;
LON:91°40'48.42"W
BOR. TYPE-AUG, 5" TUBE & SS
23 FEB 00

BOR. 57-BMU-00
LAT:34°23'30.96"N;
LON:91°42'14.80"W
BOR. TYPE-AUG, 5" TUBE & SS
16 FEB 0

BOR. 58-BMU-00
LAT:34°23'42.96"N;
LON:91°43'12.47"W
BOR. TYPE-AUG, 5" TUBE & SS
18 FEB 00

BOR. 59-BMU-00
LAT:34°44'13.68"N;
LON:91°55'28.24"W
BOR. TYPE-AUG, 5" TUBE & SS
GROUND WATER EL. 216.7
29 FEB 00

BOR. 60-BMU-00
LAT:34°46'20.16"N;
LON:92°00'28.83"W
BOR. TYPE-AUG, 5" TUBE & SS
24 FEB 00

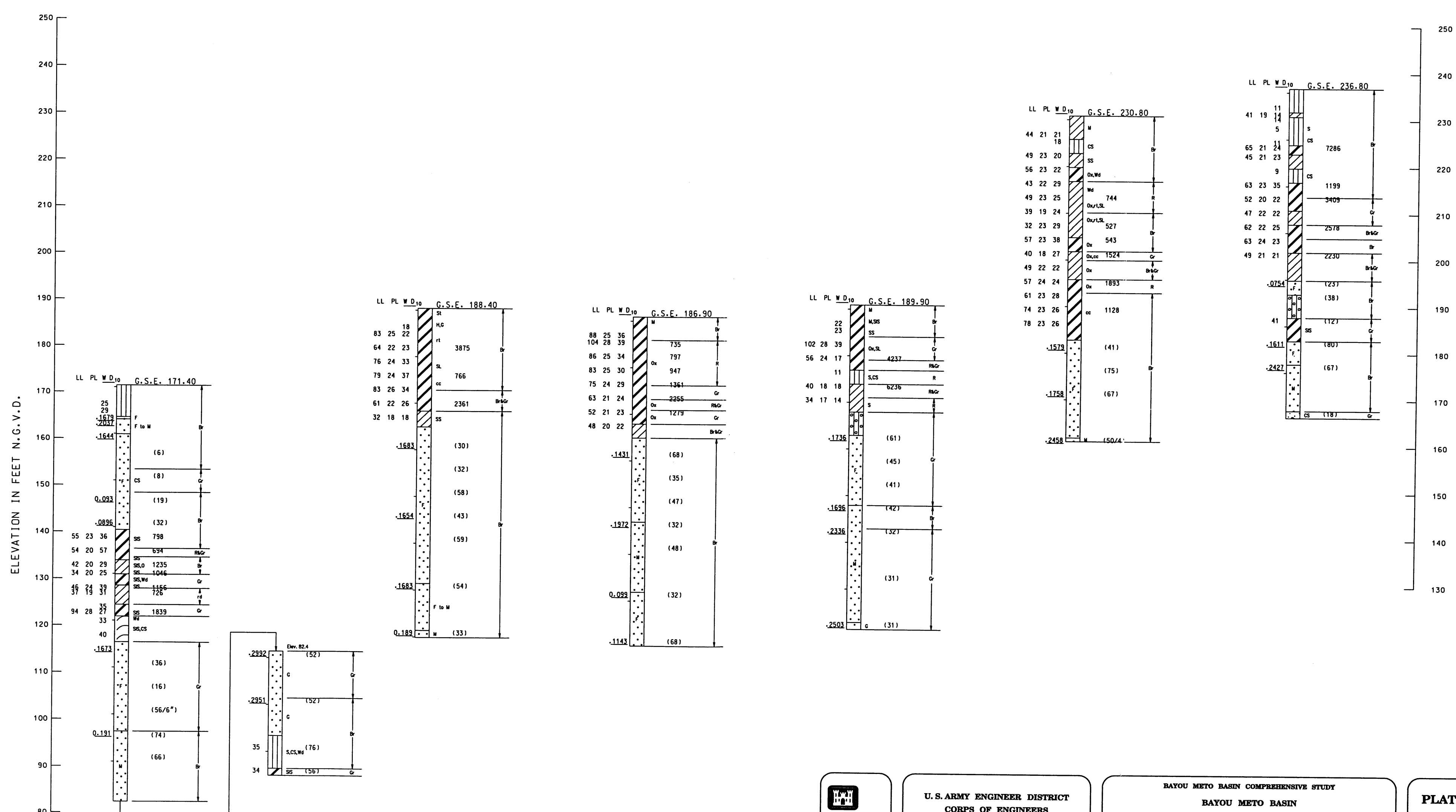

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PLATE
II-17

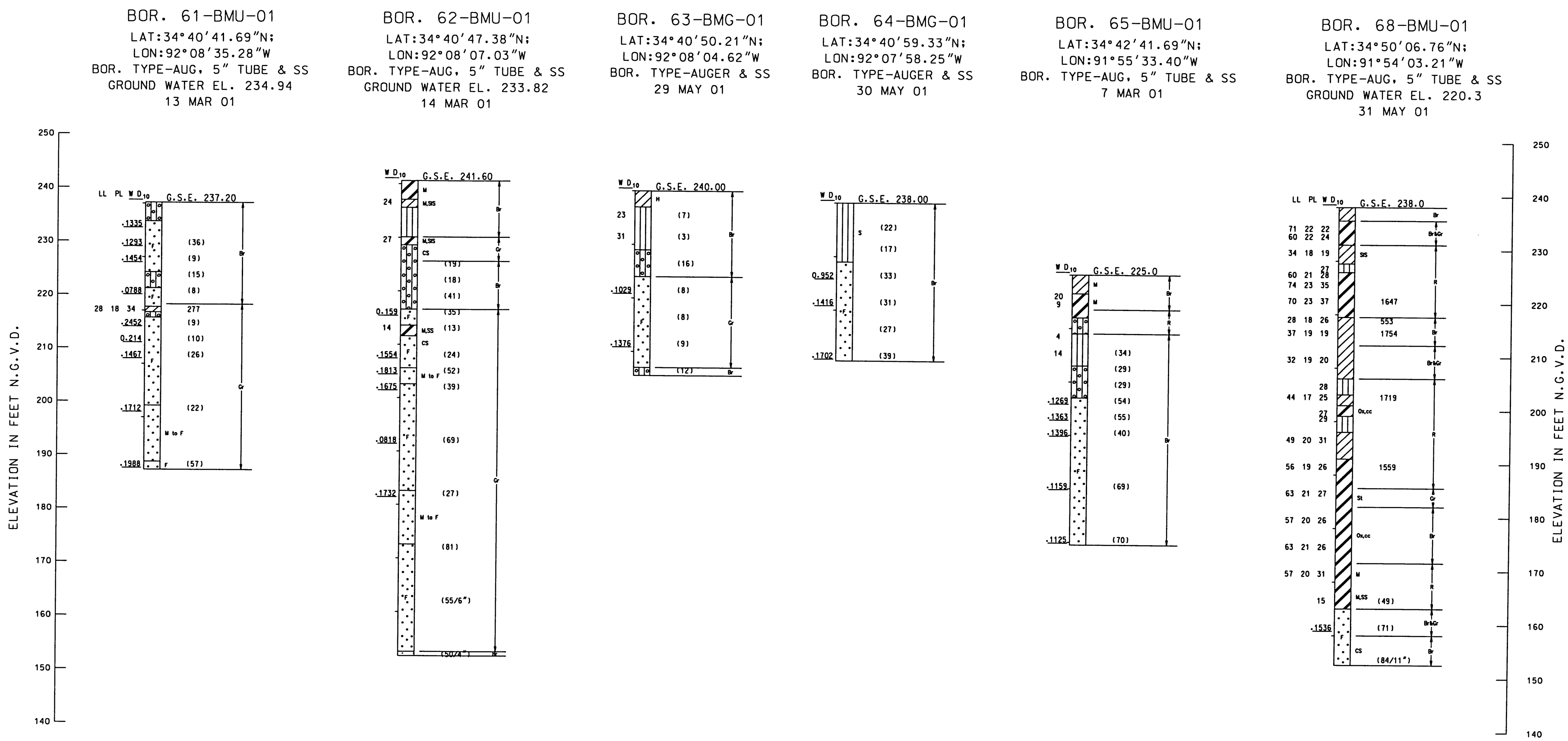

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PLATE
II-18

UNIFIED SOIL CLASSIFICATION

MAJOR DIVISION		TYPE	LETTER SYMBOL	SYMBOL	TYPICAL NAMES
COARSE GRAINED SOILS More than half the material is larger than No. 200 sieve size	GRAVEL More than half of coarse fraction is larger than No. 4 sieve size	CLEAN GRAVEL (Little or No Fines)	GW		GRAVEL, Well Graded, gravel-sand mixtures, little or no fines
			GP		GRAVEL, Poorly Graded, gravel-sand mixtures, little or no fines
		GRAVEL WITH FINES (Appreciable Amount of Fines)	GM		SILTY GRAVEL, gravel-sand-silt mixtures
			GC		CLAYEY GRAVEL, gravel-sand-clay mixtures
	SAND More than half of coarse fraction is smaller than No. 4 sieve size	CLEAN SAND (Little or No Fines)	SW		SAND, Well Graded, gravelly sands
			SP		SAND, Poorly Graded, gravelly sands
		SAND WITH FINES (Appreciable Amount of Fines)	SM		SILTY SAND, sand-silt mixtures
			SC		CLAYEY SAND, sand-clay mixtures
FINE GRAINED SOILS	More than half the material is smaller than No. 200 sieve size	SILTS AND CLAYS (Liquid Limit < 50)	ML		SILT & very fine sand, silty or clayey fine sand or clayey silt with slight plasticity
			CL		LEAN CLAY; Sandy Clay; Silty Clay; of low to medium plasticity
			OL		ORGANIC SILTS, and organic silty clays of low plasticity
		SILTS AND CLAYS (Liquid Limit > 50)	MH		SILT, fine sandy or silty soil with high plasticity
			CH		FAT CLAY, inorganic clay of high plasticity
			OH		ORGANIC CLAYS of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			Pt		PEAT, and other highly organic soil
WOOD			Wd		WOOD
MIXED SAMPLE			VM		Variable mixed silts, clays and sands
NO SAMPLE					

NOTE: Soils possessing characteristics of two groups are designated by combinations of group symbols.

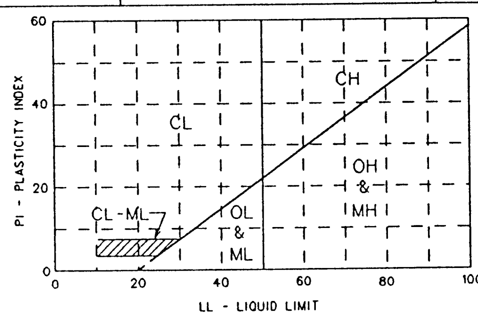
DESCRIPTIVE SYMBOL

COLOR

COLOR	SYMBOL
TAN	T
YELLOW	Y
RED	R
BLACK	BK
GRAY	Gr
LIGHT GRAY	lGr
DARK GRAY	dGr
BROWN	Br
LIGHT BROWN	lBr
DARK BROWN	dBr
BROWNISH - GRAY	brGr
GRAYISH - BROWN	grBr
GREENISH - GRAY	gnGr
GRAYISH - GREEN	grGn
GREEN	Gn
BLUE	Bl
BLUE - GREEN	BlGn
WHITE	Wh
MOTTLED	Mot

CONSISTENCY FOR COHESIVE SOILS

CONSISTENCY	COHESION IN LBS./SQ. FT. FROM UNCONFINED COMPRESSION TEST	SYMBOL
VERY SOFT	< 250	vSo
SOFT	250 - 500	So
MEDIUM	500 - 1000	M
STIFF	1000 - 2000	St
VERY STIFF	2000 - 4000	vSt
HARD	> 4000	H



PLASTICITY CHART
For classification of fine - grained soils

MODIFICATIONS

MODIFICATION	SYMBOL
Traces	Tr -
Fine	F
Medium	M
Coarse	C
Concretions	cc
Rootlets	rt
Lignite fragments	lg
Shale fragments	sh
Sandstone fragments	sds
Shell fragments	slf
Organic matter	O
Clay strata or lenses	CS
Silt strata or lenses	SIS
Sand strata or lenses	SS
Sandy	S
Gravelly	G
Boulders	B
Slickensides	SL
Wood	Wd
Oxidized	Ox
Saturated	sat
Lumps of Clay	Clp

NOTES:

FIGURES TO THE LEFT OF BORING UNDER COLUMN "W OR D_{10}"

Are natural water contents in percent dry weight

When underlined denotes D_{10} size in mm *

FIGURES TO THE LEFT OF BORING UNDER COLUMNS "LL" AND "PL"

Are liquid and plastic limits, respectively

SYMBOLS TO THE LEFT OF BORING

▽ Ground water surface and date observed

(C) Denotes location of consolidation test * *

(S) Denotes location of consolidation - drained direct shear test * *

(R) Denotes location of consolidation - undrained triaxial compression test * *

(Q) Denotes location of unconsolidated - undrained triaxial compression test * *

(T) Denotes location of sample subjected to consolidation test and each of the above three types of shear tests * *

FW Denotes free water encountered in boring or sample

o— Denotes channel grade

FIGURES TO THE RIGHT OF BORING

Are values of cohesion in lbs/sq. ft. from unconfined compression tests

In parenthesis are driving resistances in blows per foot determined with a standard split spoon sampler (1 3/8" I.D. 2" O.D.) and a 140 lb. driving hammer with a 30" drop

Where underlined with a solid line denotes laboratory permeability in centimeters per second of undisturbed sample

Where underlined with a dashed line denotes laboratory permeability in centimeters per second of sample remoulded to the estimated natural void ratio

* The D_{10} size of a soil is the grain diameter in millimeters of which 10% of the soil is finer, and 90% coarser than size D_{10}.

* * Results of these tests are available for inspection in the U. S. Army Engineer District Office if these symbols appear beside the boring logs on the drawings.

GENERAL NOTES:

While the borings are representative of subsurface conditions at their respective locations and for their respective vertical reaches, local variations characteristic of subsurface materials of the region are anticipated and if encountered, such variations will not be considered as differing materially within purview of the contract clause entitled, "Differing Site Conditions".

Ground water elevations shown on boring logs represent ground water surfaces encountered in such borings on the dates shown. Absence of water surface data on certain borings indicates that no ground water data are available from the borings but does not necessarily mean that ground water will not be encountered at the locations or within the vertical reaches of such borings.

Consistency of cohesive soils shown on the boring logs is based on driller's log and visual examination and is approximate, except within those vertical reaches of the borings where shear strengths from unconfined compression tests are shown.

STANDARD BORING LEGEND

DEPARTMENT OF THE ARMY
MEMPHIS DISTRICT, CORPS OF ENGINEERS

LEGEND
FOR
SLOPE STABILITY PRESENTATION

SOIL PROFILE

EL.	Elevation
FT.	Feet
N.G.V.D.	National Geodetic Vertical Datum
L.W.R.P.	Low Water Reference Plane
BOR	Boring
X_1	Neutral Block Left Base Coordinate
X_2	Neutral Block Right Base Coordinate
Y	Neutral Block Base Elevation
∇	Water Surface
BD	Before Drawdown
AD	After Drawdown
CP	Critical Partial Pool

SLOPE STABILITY LOADING CASES

AC	After Construction (End of Construction)
SD	Sudden Drawdown
PP	Partial Pool
SS	Steady Seepage
EQ	Earthquake
LT	Long Term

SOIL SHEAR STRENGTHS

C	Soil Cohesion - PSF
ϕ	Soil Friction Angle - Degrees
Q	Unconsolidated-Undrained Shear Strengths
R	Consolidated-Undrained Shear Strengths
S	Consolidated-Drained Shear Strengths

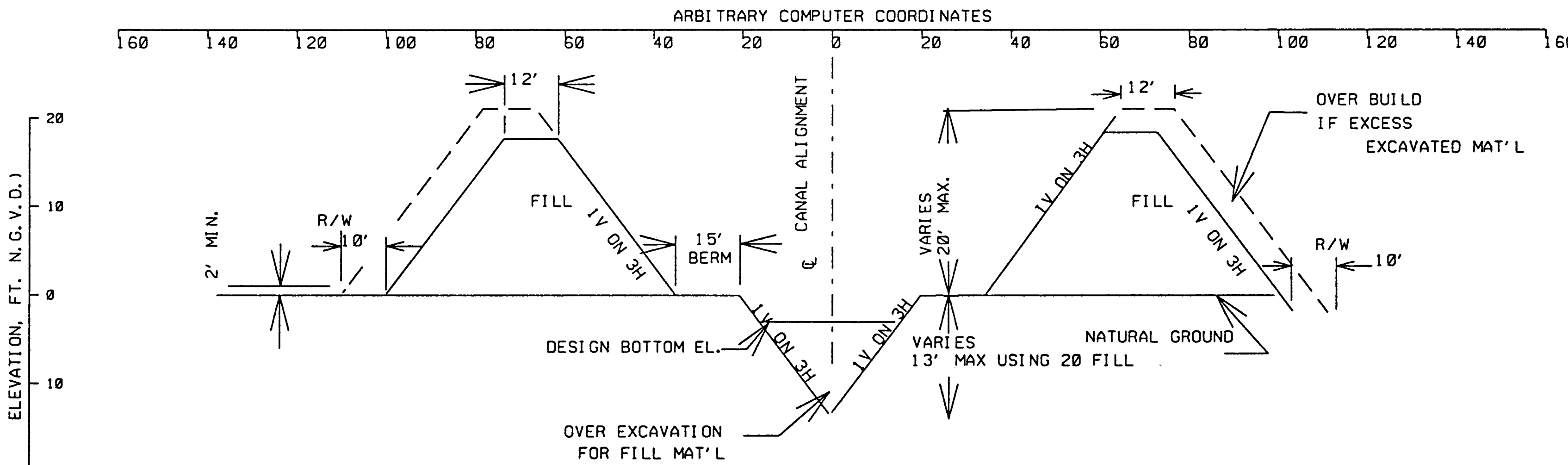
SLOPE STABILITY COMPUTER PROGRAMS

Program Name	Contact Office	Program Number	Reference Document
SSAØØ3	WES*	741-F3-RØØØ3	M.P. K-73-2
SSWØ28	WES	741-F3-RØØ28	M.P. K-76-3
CHANELS	Memphis District		M.P. GL-82-1
SSWT28**	WES		M.P. K-76-3
GEOSLOPE***	GEOCOMP CORP.		

*Waterways Experiment Station
**Modified Version of SSWØ28
***Geoslope is a microstation software package available through the GEOCOMP Corporation

LEGEND FOR SLOPE STABILITY PRESENTATION

DEPARTMENT OF THE ARMY
MEMPHIS DISTRICT, CORPS OF ENGINEERS



TYPICAL EXCAVATION SECTION

ASSUMPTIONS

1. THE MAXIMUM ELEVATION DIFFERENCE BETWEEN THE TOP OF THE EMBANKMENT AND CHANNEL EXCAVATION BOTTOM FOR THE WATER DISTRIBUTION CHANNELS OF BAYOU METO WAS 33 FEET.
2. CRITICAL ANALYSES ASSUMING A CHANNEL SLOPE HEIGHT OF 33 FEET, 1V:3H SLOPES, AND BOTH CLAY AND/OR SILT STRENGTHS RESULTED IN A MINIMUM FACTOR OF SAFETY OF 1.31. CONSERVATIVE SHEAR STRENGTHS OF $\phi = 20°$ AND C = 300 PSF WERE UTILIZED FOR THE SILT AND $\phi = 0°$ AND C = 750 PSF FOR THE CLAY (CLAY CONTROLLED).
3. LONG TERM STABILITY CONTROLLED FOR THE CHANNEL SLOPE DESIGN. AN AVERAGE 'S' STRENGTH OF $\phi' = 20°$ WAS USED FOR THE ANALYSES. THIS AVERAGE VALUE WAS BASED ON TESTING OF RANDOM SOIL SAMPLES AND EXISTING SLOPES IN THE PROPOSED PROJECT AREA.
4. FOR EMBANKMENT HEIGHTS LESS THAN 10' ABOVE NATURAL GROUND, SIDE SLOPES OF 1V:3H ARE RECOMMENDED. FOR EMBANKMENT HEIGHTS GREATER THAN 10', A SLOPE OF 1V:3.5H IS RECOMMENDED FOR THE LANDSIDE SLOPE FOR MAINTENANCE CONSIDERATIONS. SECTIONS ARE SHOWN ABOVE.
5. MINIMUM EMBANKMENT HEIGHT IS 2 FEET.

LONG TERM STABILITY

$$FS_{LT} = \frac{TAN\ \phi}{TAN\ l} = M\ TAN\ \phi'$$

$$FS_{LT} = M\ TAN\ 20° = M(0.3640)$$

FOR M = 3.0, FS = 1.09 (CONTROLS)

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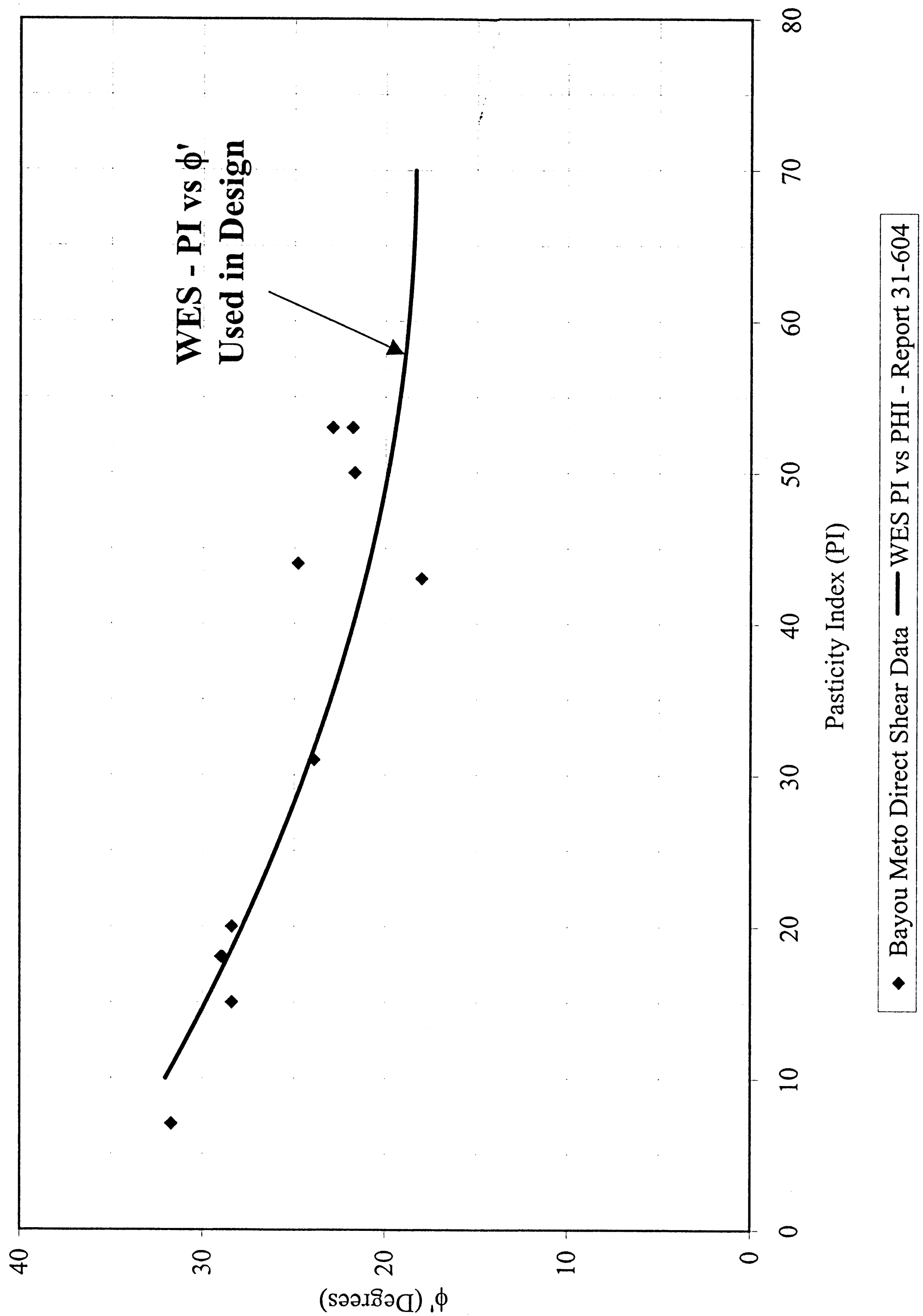
GENERAL SLOPE STABILITY

PLATE II-21



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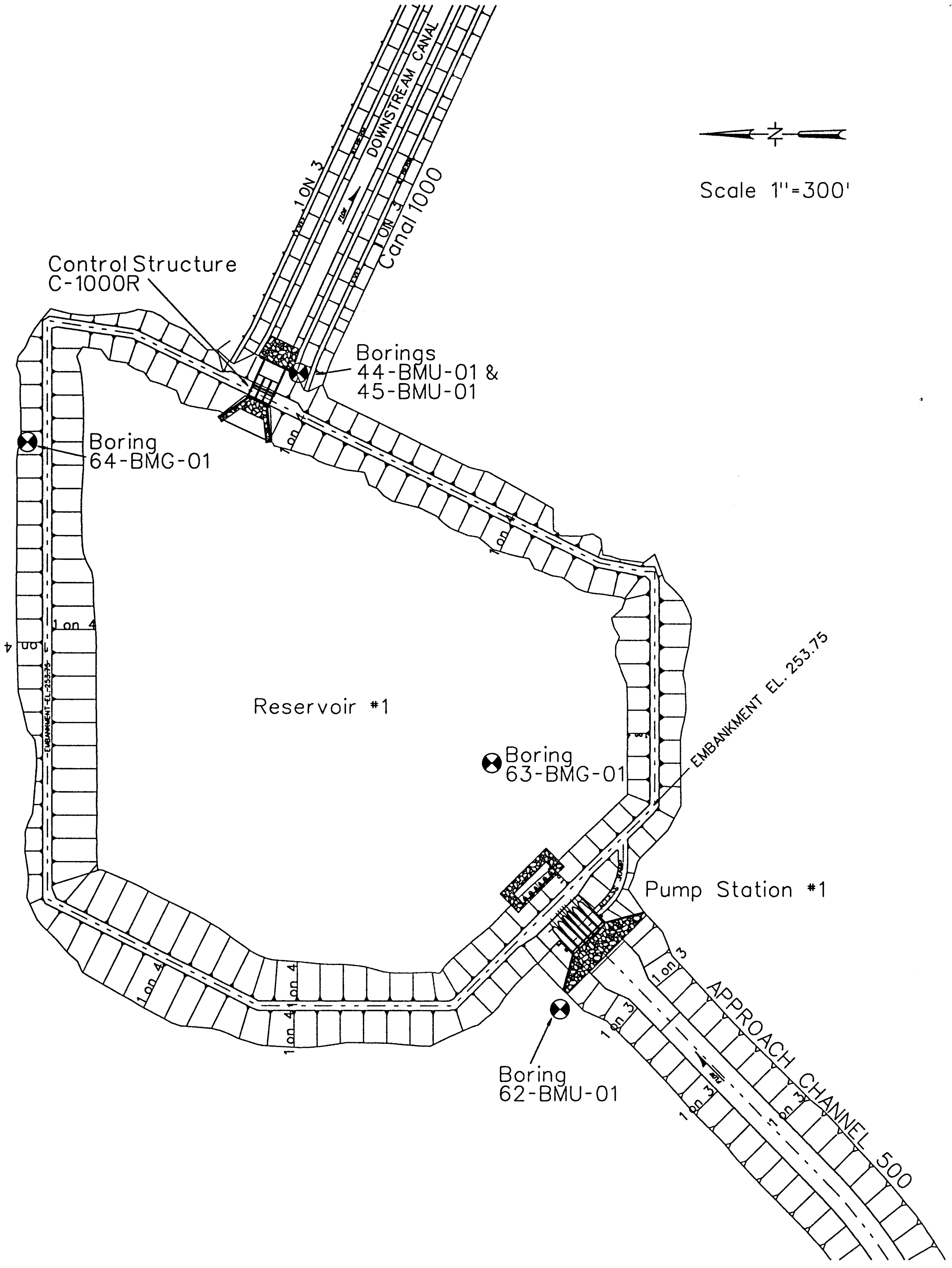
PI vs ϕ' RELATIONSHIP

PLATE

II–22



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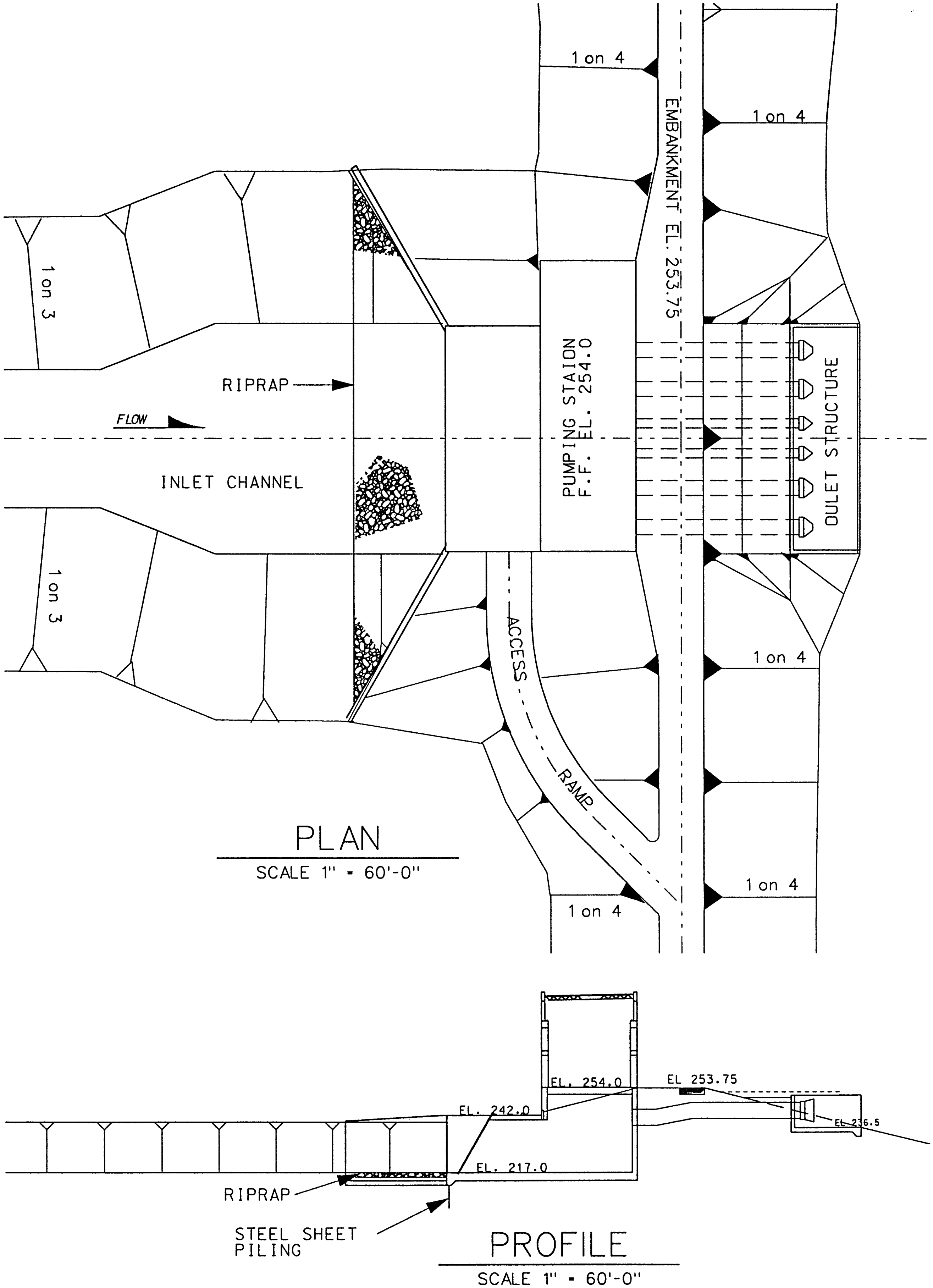
**PUMPING STATION #1
SITE PLAN**

PLATE

II-23



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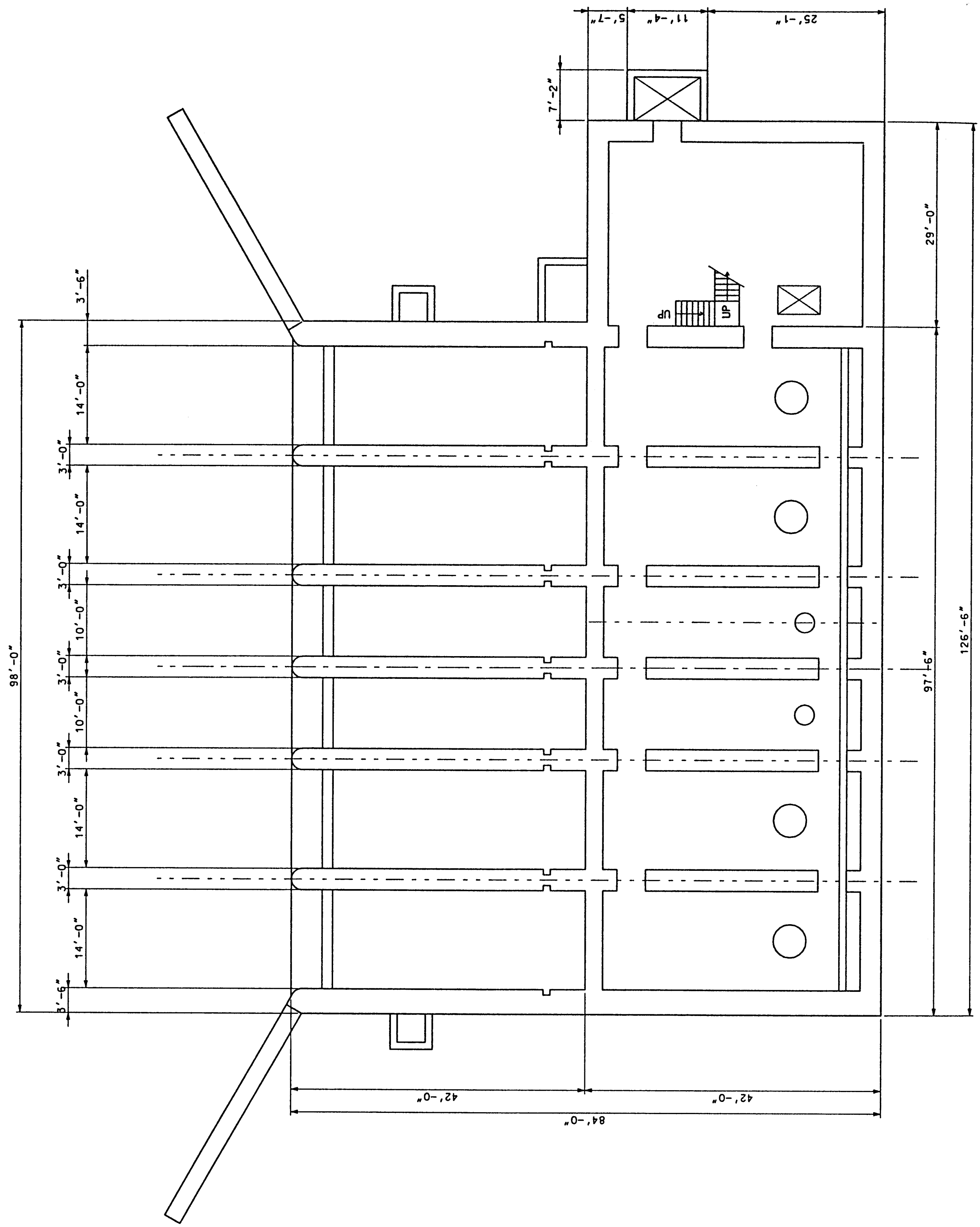
PUMPING STATION #1
PLAN & PROFILE

PLATE

II-24



EL. 237.0 PLAN

SCALE 1"20'

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PUMPING STATION #1
PLAN VIEW @ EL. 237'

PLATE

II-25

RECURRENCE EQUATIONS - CENTRAL UNITED STATES[1]

REGION	SEISMIC AREA "A" (km^2)	a	M_bMAX
WITCHITA-QUACHITA	261,829	2.79	6.3
NEW MADRID "B"	27,506	2.99	6.5
NEW MADRID "A"	22,506	3.90	7.5
OZARK UPLIFT	36,557	3.19	6.7
RESIDUAL	6,185,019	1.83	5.3

[1] BOUNDARIES OF SEISMIC SOURCE ZONES AND COEFFICIENTS FOR MAGNITUDE-RECURRENCE EQUATIONS OBTAINED FROM WES MP S-73-1, "STATE-OF-ART FOR ASSESSING EARTHQUAKE HAZARDS IN THE UNITED STATES, REPORT 12, CREDIBLE EARTHQUAKES FOR THE CENTRAL UNITED STATES", BY NUTTLI & HERRMANN, 1978.

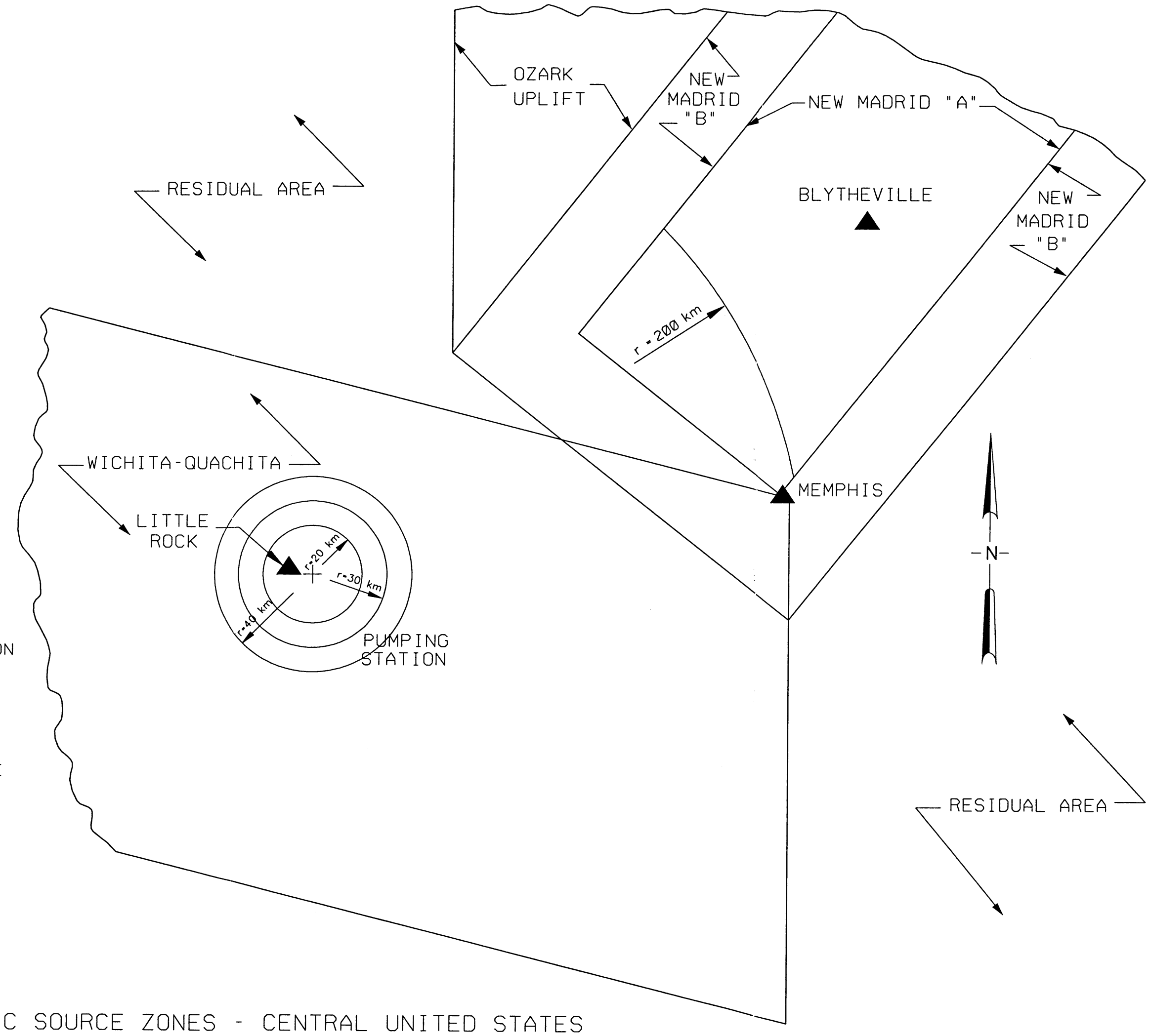
REFERENCES:

(1) NUTTLI, O.W. AND HERRMANN, R.B., "STATE-OF-THE-ART FOR ASSESSING EARTHQUAKE HAZARDS IN THE UNITED STATES, CREDIBLE EARTHQUAKE FOR THE CENTRAL UNITED STATES", MISCELLANEOUS PAPERS S-73-1, REPORT 12, DECEMBER 1978, U.S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, CE, VICKSBURG, MS.

(2) NUTTLI, O. W. AND HERRMANN, R. B., "GROUND MOTION OF MISSISSIPPI VALLEY EARTHQUAKES", JOURNAL OF TECHNICAL TOPICS IN CIVIL ENGINEERING, ASCE, VOL. 110. NO. 1, MAY, 1984, PP. 54-68.

(3) SEED, H.B. AND IDRISS, I.M., "EVALUATION OF LIQUEFACTION POTENTIAL USING FIELD PERFORMANCE DATA", JOURNAL OF GEOTECHNICAL ENGINEERING, ASCE, VOL. 109, NO. 3, MARCH, 1983, PP. 459-482.

(4) SEED, H.B. AND IDRISS, I.M., "SIMPLIFIED PROCEDURE FOR EVALUATING SOIL LIQUEFACTION POTENTIAL", JOURNAL OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION, ASCE, VOL. 97, NO. SMO, SEPT., 1971, PAGES 1255 AND 1265.



SEISMIC SOURCE ZONES - CENTRAL UNITED STATES

SCALE: 1" = 50 KM

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SEISMIC ANALYSIS
SOURCE ZONES

PLATE
II-26

I. EVALUATE LIQUEFACTION POTENTIAL AT SITE OF PUMPING STATION. ASSUME WATER AT ELEVATION 230 FT:
γ_{clay} = 115 PCF, γ'_{clay} = 53 PCF. γ_{silt} = 120 PCF, γ'_{silt} = 57 PCF. γ_{sand} = 125 PCF, γ'_{sand} = 63 PCF.

SEMI-EMPIRICAL PROCEDURE TO EVALUATE LIQUEFACTION POTENTIAL

BORING 62-BMU-01

DEPTH	ELEV	SOIL TYPE	N_f	σ_o (ksf)	σ'_o (ksf)	N_1	C_N	N_2	τ/σ'_o	r_d	a/g
(ABOVE 35' TO BE REPLACED BY STRUCTURE)											
35	207	SP	52	4.26	2.83	34	0.88	29	0.32	0.89	0.367
38	204	SP	39	4.64	3.02	27	0.84	23	0.24	0.87	0.276
48	194	SP	69	5.89	3.64	42	0.79	33	0.4	0.76	0.501
58	184	SP	27	7.14	4.27	21	0.72	15	0.16	0.7	0.210*
68	174	SP	81	8.39	4.89	48	0.69	33	0.4	0.6	0.598
78	164	SP	110	9.64	5.52	63	0.62	39	0.4	0.55	0.641
88	154	SP	100	10.89	6.15	58	0.6	35	0.4	0.52	0.668

* CONTROLS (FIRST TO LIQUEFY)

II. UTILIZING THE ATTENUATION RELATIONSHIP CURVES FOR THE MISSISSIPPI VALLEY (SEE FIG. 6, REF. 2) AND THE CHART EVALUATING LIQUEFACTION POTENTIAL FOR DIFFERENT MAGNITUDE (M_b) EARTHQUAKES (SEE FIG. 13, REF. 3) THEN THE MAXIMUM DISTANCE (r) FROM THE EPICENTER TO THE SITE THAT WOULD PRODUCE LIQUEFACTION CAN BE DETERMINED FOR DIFFERENT MAGNITUDES OF EARTHQUAKES. FURTHERMORE, THE TOTAL ANNUAL RISK FACTOR (R_T) FOR AN EARTHQUAKE TO CAUSE LIQUEFACTION OF THE CONTROLLING SAND STRATUM CAN BE DETERMINED AS THE SUM OF THE INDIVIDUAL RISKS FOR ALL SEISMIC ZONES.

CALCULATION OF ANNUAL RISK FACTOR OF LIQUEFACTION

BORING 62-BMU-01, STRATUM AT 58' DEPTH, N_2 = 15, σ_o = 7.14, σ'_o = 4.27

EXISTING CONDITIONS

M_b	N_c	I	τ/σ'_o	a/g	r(km)	P_A	P_{AVG}	RISK
WICHITA-QUACHITA SEISMIC ZONE; a = 2.79; M_b (MAX) = 6.3								
5.25	0.00912		0.24	0.315	5	0.000785		
		0.007258					0.006676	0.000049
6	0.001862		0.21	0.276	20	0.012566		
		0.000643					0.02042	0.000013
6.2	0.001219		0.18	0.237	30	0.028274		
		0.000233					0.03927	0.000009
6.3	0.000986		0.16	0.210	40	0.050265		
								Σ = 0.000071
NEW MADRID 'A' SEISMIC ZONE; a = 3.90; M_b (MAX) = 7.5								
7.1	0.00233		0.14	0.184	140	(NO RISK TO PUMP STATION)		
		0.001333					0.087976	0.000117
7.5	0.001		0.13	0.171	200	0.175953		
								Σ = 0.000117
NEW MADRID 'B' SEISMIC ZONE; a = 2.99; M_b (MAX) = 6.5								
6.5	0.00102		0.163	0.214	55	(NO RISK TO PUMP STATION)		
								≈ 0.
OZARK UPLIFT SEISMIC ZONE; a = 3.19; M_b (MAX) = 6.7								
6.7	0.00106		0.155	0.204	75	(NO RISK TO PUMP STATION)		
								≈ 0.
RESIDUAL EVENTS SEISMIC ZONE; a = 1.83; M_b (MAX) = 5.3								
5.3	0.0009		0.230	0.302	5	(NO RISK TO PUMP STATION)		
								≈ 0.
								Σ RISKS = 0.000188

III. IF THE CONTROLLING STRATUM AT A DEPTH OF 58 FT WAS DENSIFIED THE RISK OF OF LIQUEFACTION COULD BE REDUCED. ASSUMING DENSIFICATION OF THE EXISTING MATERIALS TO A RELATIVE DENSITY OF 80% WOULD RESULT IN A FIELD N VALUE OF N_f = 39 (SEE FIG. 13, REF. 4) WHICH TRANSLATES TO N_2 = 28. THE RISK OF LIQUEFACTION ASSOCIATED WITH EACH N VALUE IS COMPUTED BELOW.

CALCULATION OF ANNUAL RISK FACTOR OF LIQUEFACTION

BORING 62-BMU-01, STRATUM AT 58' DEPTH, σ_o = 5.35, σ'_o = 3.60

IMPROVED CONDITIONS

M_b	N_c	I	τ/σ'_o	a/g	r(km)	P_A	P_{AVG}	RISK
NEW MADRID 'A' SEISMIC ZONE; a = 3.90; M_b (MAX) = 7.5; N_2 = 28								
7.5	0.001		0.24	0.32	120	NO RISK TO PUMP STATION		
WITCHATA-QUACHITA SEISMIC ZONE; a = 2.79; M_b (MAX) = 6.3; N_2 = 28								
6	0.00186			0.00		(NO RISK TO PUMP STATION)		
		0.00064					0.00353	0.000002
6.2	0.00122		0.355	0.47	15	0.007069		
		0.00023					0.00981	0.000002
6.3	0.00099		0.34	0.45	20	0.012566		
								Σ = 0.000005

IV. DETERMINE COST EFFECTIVENESS FOR METHODS OF GROUND MODIFICATION TO REDUCE THE RISK OF LIQUEFACTION. ASSUME DAMAGES DUE TO LIQUEFACTION EQUALS $7,000,000. (DAMAGES BASED ON COSTS TO PRESSURE GROUT THE FOUNDATION, MINOR DAMAGE INSIDE STRUCTURE, RECONNECTING PIPES AND UTILITIES AND REPLACING INLET WALLS). EFFECTIVENESS OF VARIOUS METHODS IS MEASURED AS SAVINGS IN ANNUAL DAMAGE ASSOCIATED WITH EACH METHOD. ANNUAL DAMAGES ARE CONVERTED TO PRESENT WORTH BASED ON 100-YEAR LIFE OF STRUCTURE AND AN INTEREST RATE EQUAL TO 7.375%.

EXISTING FOUNDATION SANDS DENSIFIED

BENEFITS:
SAVINGS IN ANNUAL DAMAGES = (0.000188 - 0.000005)($7,000,000) = $1281
PRESENT WORTH (n = 100 YEARS, i = 7.375%, A = 1281) = $17400

COSTS:
ASSUME VIBROFLOTATION WILL BE USED TO DENSIFY SANDS FROM ELEVATON 242' TO 180'
AREA = (90' X 130')
ASSUME 5 FT. ON CENTER W/ $50,000 MOB.
8840 L.F. @ $20/L.F. = $226,800

BENEFITS/COST:
B/C = 17,400/226,800 = 0.08

V. IN SUMMARY IT IS NOT COST EFFECTIVE TO PERFORM GROUND MODIFICATIONS FOR THE PURPOSE OF REDUCING OR ELIMINATING THE RISKS OF EARTHQUAKE INDUCED LIQUEFACTION

DEFINITIONS:

$1/N_c$ = RECURRENCE PERIOD IN YEARS.
LOG N_c = a - 0.92 M_b (PARA. 50, REF. 1)
a = COEFFICIENT INDICATIVE OF THE NUMBER OF EVENTS PER SEISMIC AREA 'A' UP TO 100,000 km²
M_b = BODY WAVE MAGNITUDE.
N_f = FIELD RESULTS OF STANDARD PENETRATION TEST (SPT) BLOWS/FT.
N_1 = RESULTS OF SPT CORRECTED FOR PRESENCE OF FINES.

$$N_1 = 15 + \frac{(N - 15)}{2}$$

C_n = CORRECTION FACTOR FOR EFFECTIVE OVERBURDEN PRESSURE (FIG. 4, REF. 3)
N_2 = N_1 CORRECTED FOR EFFECTIVE OVERBURDEN PRESSURE.

$$N_2 = C_n N_1$$

σ_o = TOTAL OVERBURDEN PRESSURE (ksf)
σ'_o = EFFECTIVE OVERBURDEN PRESSURE (ksf)
τ/σ'_o = CYCLIC STRESS RATIO CAUSING LIQUEFACTION AT σ'_o = 1tsf (FIG. 9, REF. 3)
r_d = STRESS REDUCTION COEFFICIENT (FIG. 4, REF. 4)
a/g = ACCELERATION/GRAVITY AT WHICH LIQUEFACTION WILL OCCUR

$$a/g = \left(\frac{\tau}{\sigma'_o}\right)\left(\frac{\sigma'_o}{\sigma_o}\right)\left(\frac{1}{0.65\ r_d}\right) \quad \text{(EQ. 1, REF. 3)}$$

r = MAXIMUM RADIUS FROM EPICENTER AT WHICH LIQUEFACTION WILL OCCUR
I = INCREMENTAL RECURRENCE ($N_{C(n)} - N_{C(n+1)}$)

P_A = AREA OF SEISMIC SOURCE ZONE ENCOMPASSED BY RADIUS "r" DIVIDED BY SEISMIC AREA 'A' FOR 'A' ≤ 100,000 km²

$$P_{AVG} = \frac{(P_{A(n)} + P_{A(n+1)})}{2}$$

RISK = I X P_{AVG} = INCREMENTAL ANNUAL RISK FACTOR FOR AN EARTHQUAKE OF M_b = X TO CAUSE LIQUEFACTION OF THE CONTROLLING SAND STRATUM.

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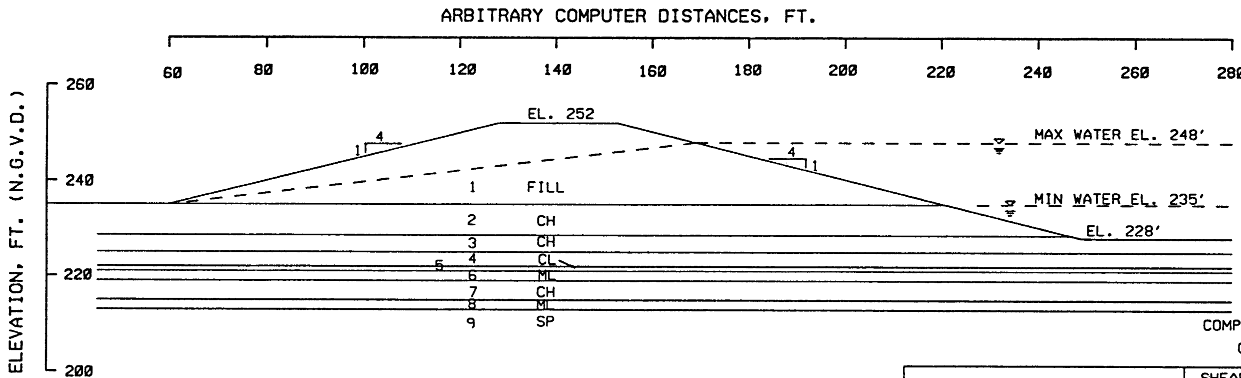
SEISMIC ANALYSIS

PLATE
II-27



STRATIFICATION FROM 45-BMU-00

COMPUTER RESULTS
GEOSLOPE

CASE ANALYZED	SHEAR STRENGTH USED	MIN REQUIRED FS	45-BMU-01		62-BMU-01	
			ARC	WEDGE	ARC	WEDGE
CASE I-AC	Q	1.3	1.931	1.894	2.675	2.950
CASE I-AC	S	1.3	1.718	1.759	1.571	1.968
CASE II-SD (MAX POOL)	R,S	1.0	1.531	1.366	1.305	1.353
CASE III-SD (TOP OF GATES)	R,S	1.2	1.531	1.366	1.305	1.353
CASE IV-PP (W/STD SEEPAGE)	$\frac{R+S}{2}$ FOR R<S; S FOR R>S	1.5	1.721	1.871	1.942	1.889
CASE V-SS (W/ MAX POOL)		1.5	1.826	1.632	1.791	1.783
CASE VI-SS (W/SURCHARGE POOL)		1.4	1.503	1.486	1.492	1.579
CASE VII-EQ (CASE I)	Q	1.0	1.292	1.114	1.814	2.057
CASE VII-EQ (CASE I)	S	1.0	1.164	1.150	1.090	1.347
CASE VII-EQ (CASE IV)	$\frac{R+S}{2}$ FOR R<S	1.0	1.161	1.171	1.251	1.220
CASE VII-EQ (CASE V)	S FOR R>S	1.0	1.194	1.093	1.223	1.157

SOIL SHEAR STRENGTHS - BORING 45-BMU-00

SOIL NO.	SOIL TYPE	ELEVATION (N.G.V.D.)		UNIT WT. (PCF)	Q		R		S	
					Φ ( )°	C (PSF)	Φ ( )°	C (PSF)	Φ ( )°	C (PSF)
1	FILL	252.0	235.8	115	0	750	13.0	375	26.0	0
2	CH	235.8	228.3	115	0	750	11.75	375	23.5	0
3	CH	228.3	224.8	112	0	1000	10.5	500	21	0
4	CL	224.8	222.8	120	0	1000	14	500	28	0
5	CL	222.8	220.8	112	0	600	13.5	300	27	0
6	ML	220.8	218.8	120	20	300	20	300	28	0
7	CH	218.8	214.8	107	0	500	11	250	22	0
8	ML	214.8	212.8	120	20	300	20	300	28	0
9	SP	212.8	195.8	125	31	0	31	0	31	0

SOIL SHEAR STRENGTHS - BORING 62-BMU-01

SOIL NO.	SOIL TYPE	ELEVATION (N.G.V.D.)		UNIT WT. (PCF)	Q		R		S	
					Φ ( )°	C (PSF)	Φ ( )°	C (PSF)	Φ ( )°	C (PSF)
1	FILL	252	242	115	0	750	13.0	375	26.0	0
2	CH	242	237	115	0	750	12	375	24	0
3	ML	237	231.5	120	20	300	20	300	28	0
4	CH	231.5	230	115	0	750	10	375	20	0
5	SM	230	218	125	32	0	32	0	32	0
6	SP	218	215	125	34	0	34	0	34	0
7	CH	215	213	120	0	1000	10	500	20	0
8	SP	213	189	125	33	0	33	0	33	0
9	SP	189	179	126	32	0	32	0	32	0
10	SP	179	153	127	36	0	36	0	36	0

ASSUMPTIONS:

1) FILL MATERIAL USED FOR THE EMBANKMENTS WILL BE LEAN CLAY WITH Q STRENGTHS OF C=750 PSF Φ=0° AND S STRENGTHS OF C=0 PSF AND Φ=26°.

2) THE RESERVIOR POOL WILL OPERATE AT A MAXIMUM POOL OF ELEVATION 248 FT. AND A MINIMUM POOL OF 235 FT.

3) STABILITY ANALYSIS WAS BASED ON CRITERIA SET FORTH IN EM 1110-2-1902 'ENGINEERING AND DESIGN STABILITY OF EARTH AND ROCK-FILL DAMS'. THE CASES ANALYIZED FOLLOW CRITERIA ON TABLE PAGE 25 OF ABOVE REFERENCED CRITERIA. REQUIREMENTS OF CASES III AND VI ARE MET BY CASES II AND V RESPECTIVELY

4) CASE VII WAS ANALYIZED USING A PSEUDOSTATIC APPROACH USING A HORIZONTAL COEFFICIENT OF K =0.1. THE COEFFICIENT K_H WAS DETERMINED BY THE FOLLOWING EQUATION...

$$K_H = 0.5 \times PGA \times S$$

WHERE:
K_H = DIMENSIONLESS HORIZONTAL PSEUDOSTATIC COEFFICIENT
PGA = PEAK GROUND ACCELERATION (1000 YEAR RETURN PERIOD)
(5% PROBABILITY OF EXCEEDANCE IN 50 YEARS WITH 0.2 SEC SPECTRAL ACCELERATION)
S = SITE COEFFICIENT FROM STANDARD BUILDING CODE (S = 1.5)

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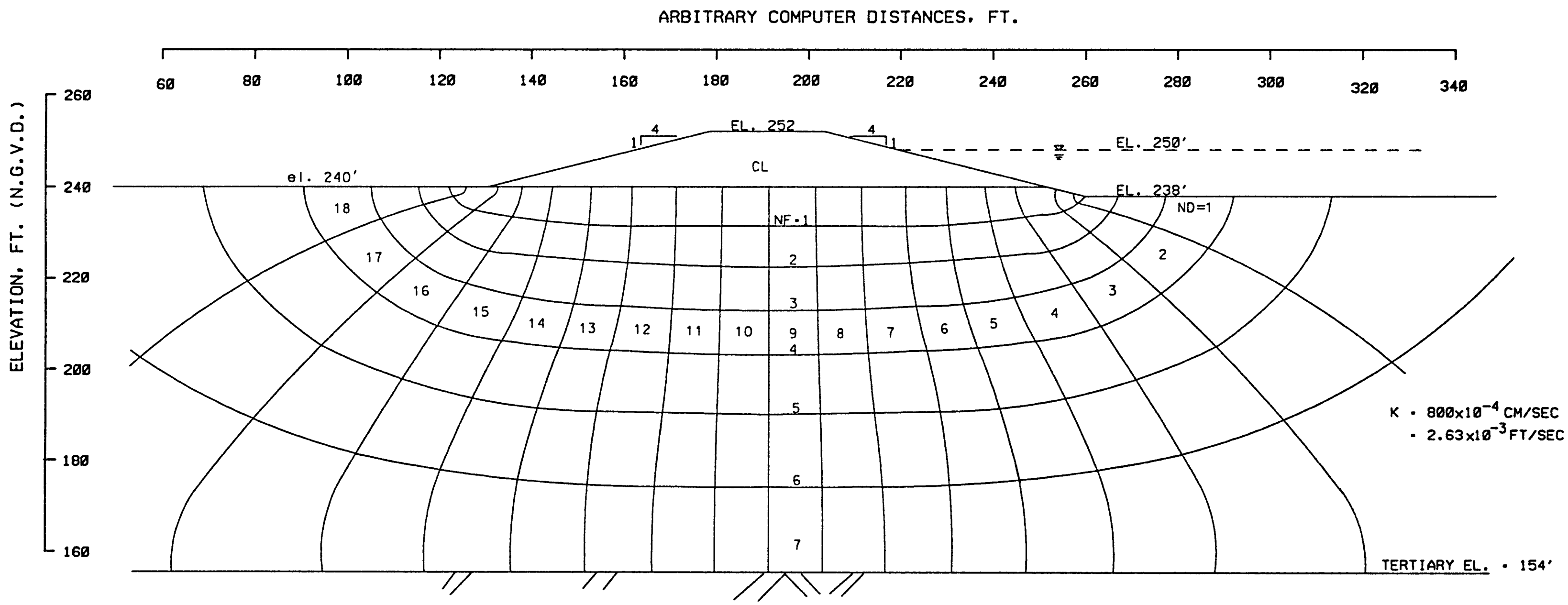
PUMPING STATION #1 RESERVIOR STABILITY

PLATE
II-28



CALCULATE SEEPAGE LOSSES FROM RESERVIOR

$Q = (\frac{NF}{ND}) \cdot K \cdot H$

WHERE NF = NUMBER OF FLOW CHANNELS

ND = NUMBER OF EQUIPOTENTIAL DROPS

K = PERMEABILITY OF FOUNDATION

H = HYDRAULIC HEAD

$Q = (\frac{7}{18}) \cdot 0.00263 = 12'$

$= 0.0122\ FT^3/SEC/\ FT$ LEVEE

$= 1058.2\ FT^3/DAY/\ FT$ LEVEE

ASSUME APPROXIMATELY 4500 FT OF LEVEE

$Q = 0.0122\ (FT^3/SEC/FT) \cdot 4500$ FT

$= 55.11\ (FT^3/SEC)$

PUMPING STATION CAPACITY = 1750 CFS

% LOSSES IN RESERVIOR = Q / Q

% LOSS = $\frac{55.1}{1750}$ = 3% OF PUMP CACPACITY

CONCLUSION: LOSSES WITHIN ACCEPTABLE LIMITS
NO REMEDIATION REQUIRED

ASSUMPTIONS:

1) THE QUANTITY OF FLOW DUE TO LEVEE THROUGH SEEPAGE WILL BE SMALL COMPARED TO THE QUANTITY DUE TO UNDERSEEPAGE.

2) THE RESERVIOR POOL WILL OPERATE AT A MAXIMUM POOL OF ELEVATION 250 FT. AND A MINIMUM POOL OF 238 FT.

3) THE FOUNDATION PERMEABILITY WAS ESTIMATED BASED ON D10 TESTS FROM 62-BMU-01. THE THIN CLAY OVERBURDEN FOUND IN BORING 62-BMU-01 WAS CONSERVATIVELY NEGLECTED IN THE ANALYSIS.

4) THE GROUNDWATER LANDSIDE WAS ASSUMED TO BE AT EL. 240 FEET.

5) THE CONDITIONS SHOWN ARE CONSERVATIVE ESTIMATE OF FIELD CONDITIONS AFTER CONSTRUCTION. LOSSES WILL REDUCE AS SILTATION OCCURS IN RESERVIOR.

6) THE CROSS SECTION SHOWN ABOVE WAS TAKEN AS A CONSERVATIVE ESTIMATE OF CONDTIONS NEAR THE PROPOSED PUMPING STATION #1.

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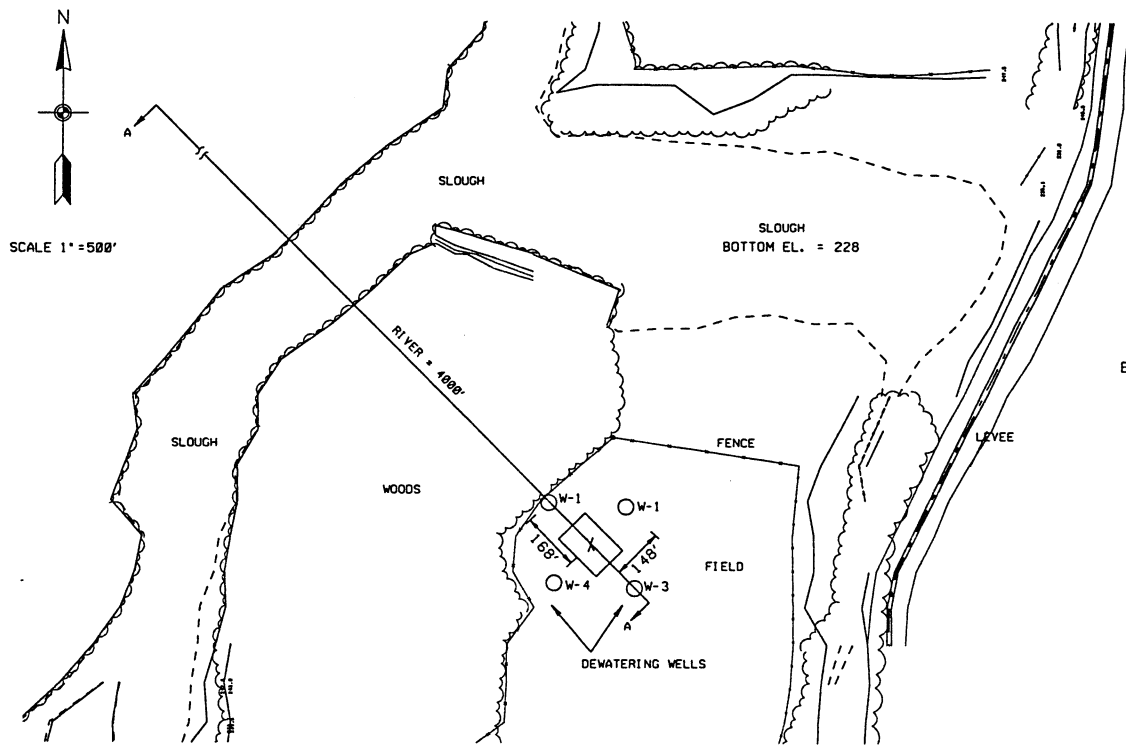
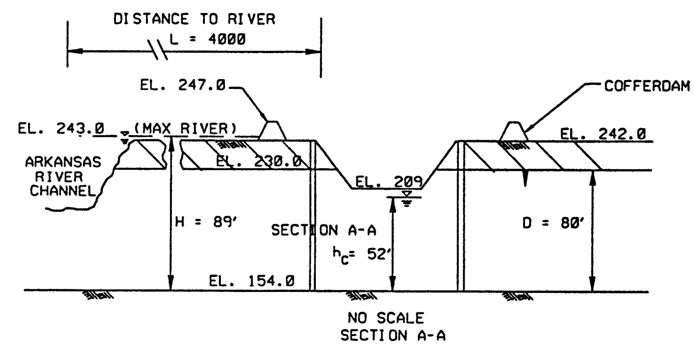
PUMPING STATION #1 RESERVIOR SEEPAGE

PLATE II-29





ASSUMPTIONS

1. TERTIARY ELEVATION WAS CONSERVATIVELY BASED ON BORING 62-BMU-01.
2. THE DEWATERING SYSTEM WILL BE REQUIRED TO LOWER THE WATER TABLE TO EL. 206.0 OR THREE FEET BELOW THE EXCAVATION.
3. HEAD WATER ELEVATION OF 243.0 IS BASED ON THE MAXIMUM ARKANSAS RIVER FLOOD.
4. THE ARKANSAS RIVER CHANNEL IS LOCATED APPROXIMATELY 4000 FEET AWAY FROM THE EXCAVATION. CIRCULAR SOURCE ASSUMED TO CONTROL.
5. PERMEABILITY OF AQUIFER, $K_h = 800 \times 10^{-4}$ CM/SEC = 0.158 FT/MIN.
6. DEPTH OF AQUIFER = 80 FT.
7. A SECTION PERPENDICULAR TO THE INLET CHANNEL IS SHOWN. (NOT TO SCALE)
8. THE SLOUGH LOCATED NEARBY IS RELATIVELY SHALLOW AND WILL NOT ACT AS LINE SOURCE FOR SEEPAGE

DETERMINE EQUIVALENT RADIUS A_E OF WELL SYSTEM
ASSUME BASICALLY SQUARE

$A_E = (4/\pi)\sqrt{(295/2)(335/2)} \simeq 200.0$ FT $= r_w$

COMPUTE RADIUS OF INFLUENCE (R):

$D_{10} = 0.16$MM, $K \simeq 800 \times 10^{-4}$ CM/SEC ≈ 0.158 FT/MIN

$R = C(H - h_c)\sqrt{K}$

$R = 3(243 - 206)\sqrt{800} = 3140$ FT

COMPARE 2L VS R :

$2L = 2(4000) = 8000'$ $R = 3140'$

$\therefore$ SINCE 2L IS GREATER THAN R, THE EQUATIONS FOR A CIRCULAR SOURCE OF SEEPAGE ARE APPLICABLE.

ESTIMATE TOTAL FLOW USING RADIUS A_E OF AN EQUIVALENT LARGE DIAMETER WELL FOR ARTESIAN FLOW CONDITIONS.

$$Q_T = \frac{2\pi KD(H-h_c)}{\ln(R/r_w)}$$

$$Q_T = \frac{2\pi(0.158)(80)(89-52)}{\ln(3140/200)} = 1067.1 \text{ CFM} = 7982.7 \text{ GPM}$$

ASSUME 14" DIAMETER FULLY PENTRATING DEEP WELLS WILL BE USED. MAXIMUM FLOW FOR 14" WELLS IS 2150 GPM.

NO. OF WELLS = 7982.7 GPM/2150 GPM = 3.7 OR 4 WELLS

FLOW PER WELL $7983/4 \simeq 2000$ GPM $\simeq 267$ CFM $= Q_w$

HEAD AT POINT C

WELL	R_i	r_i	$\ln R_i/r_i$
1	3140	168	2.93
2	3140	148	3.05
3	3140	168	2.93
4	3140	148	3.05
			11.96

$F'_c = 11.96(267) = 3193$

$H - h_c = F'_c / 2\pi KD = 3193/[2\pi(0.158)(80)] = 40.2'$

$H - h_c = 40.2$

$h_c = 89 - 40.2 = 48.8' < 52'$ O.K.

REFERENCES: 1. TM 5-818-5, "DEWATERING AND GROUNDWATER FOR DEEP EXCAVATIONS"

2. BORING 62-BMU-01

NOTE: DEWATERING ANALYSIS AND LAYOUT ARE FOR ESTIMATING PURPOSES ONLY.

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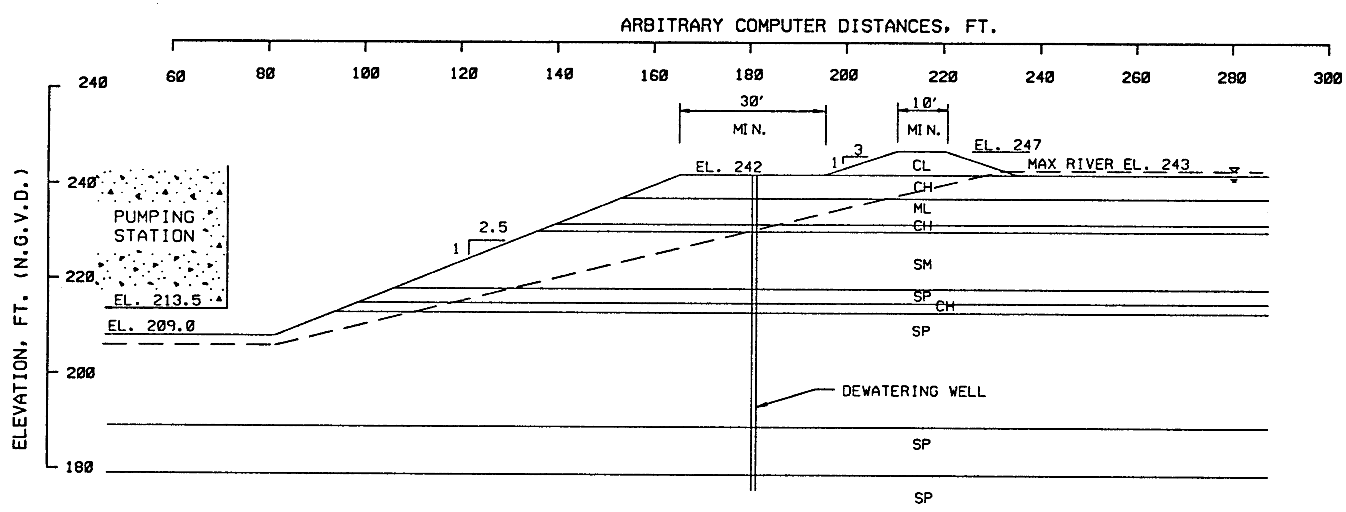
PUMPING STATION 1
DEWATERING ANALYSIS

PLATE
II-30



SOIL SHEAR STRENGTHS - BORING 62-BMU-01

SOIL NO.	SOIL TYPE	ELEVATION (N.G.V.D.)		UNIT WT. (PCF)	Q		R		S	
					ϕ ( )°	C (PSF)	ϕ ( )°	C (PSF)	ϕ ( )°	C (PSF)
1	CH	242	237	115	0	750	12	375	24	0
2	ML	237	231.5	120	20	300	20	300	28	0
3	CH	231.5	230	115	0	750	10	375	20	0
4	SM	230	218	125	32	0	32	0	32	0
5	SP	218	215	125	34	0	34	0	34	0
6	CH	215	213	120	0	1000	10	500	20	0
7	SP	213	189	125	33	0	33	0	33	0
8	SP	189	179	125	32	0	32	0	32	0
9	SP	179	153	125	36	0	36	0	36	0

ASSUMPTIONS

1. ANALYSES ASSUMED THAT THE DEWATERING SYSTEM SHALL BE CAPABLE OF MAINTAINING THE GROUND WATER AT OR BELOW EL. 206.0 FOR THE EXCAVATION.
2. THE PHREATIC PROFILE OUTSIDE OF THE WELLS VARIED FROM EL. 230.0 (NORMAL GROUND WATER) TO EL. 243.0. (MAX RIVER EL.).
3. Q STRENGTHS WERE USED IN THE ANALYSES.
4. BOTH CIRCULAR AND WEDGE TYPE METHODS WERE UTILIZED
5. THE FACTORS OF SAFETY FOR THE ANALYSIS ABOVE ARE AS FOLLOWS:

	Q
MIN. FS FOR EXCAVATION	1.75
MIN. FS FOR COFFERDAM	> 3.0

LONG TERM STABILITY (EXCAVATION)

$$FS_{LT} = \frac{\tan \phi}{\tan l} = M \tan \phi'$$

$$FS_{LT} = M\left[\frac{5 \tan 24° + 5.5 \tan 28° + 1.5 \tan 20° + 12 \tan 32° + 3 \tan 34° + 2 \tan 20° + 4 \tan 33°}{33}\right]$$

FS_{LT} @ EL. 209.0 = M(0.5619) FOR M = 2.5, FS = 1.40

FS_{LT} @ EL. 230.0 = M(0.4747) FOR M = 2.5, FS = 1.18 (CONTROLS)

LONG TERM STABILITY (COFFERDAM)

$$FS_{LT} = \frac{\tan \phi}{\tan l} = M \tan \phi'$$

S STRENGTH OF 26° WAS SELECTED FOR THE FILL MATERIAL PLACED IN THE COFFERDAM

FS_{LT} = M(TAN 26°) = M(0.4877) FOR M = 3.0, FS = 1.46

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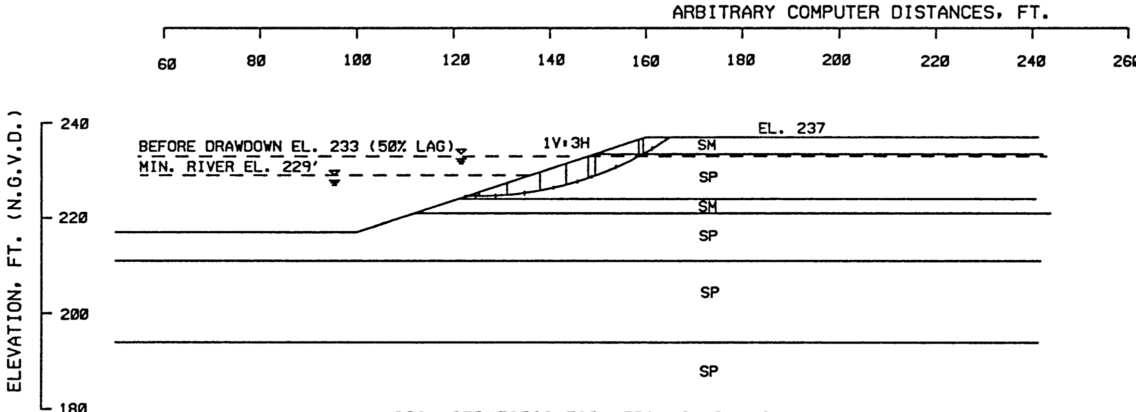
PUMPING STATION #1
EXCAVATION STABILITY

PLATE
II-31



SOIL STRATIFICATION FROM 61-BMU-01

LONG TERM STABILITY

$$FS_{LT} = \frac{\tan \phi}{\tan I} = M \tan \phi$$

$$FS_{LT} = M \; \frac{(17 \cdot \tan 30) + (3 \cdot \tan 33)}{20}$$

$$FS_{LT} = M(0.5882)$$

FOR M = 3 FS = 1.76

COMPUTER RESULTS
GEOSLOPE

	ARC METHOD		WEDGE METHOD	
BORING #	A.C.	S.D.	A.C.	S.D.
61-BMU-01	1.603	1.283*	1.604	1.375
62-BMU-01	2.205	1.803	2.211	1.899

* CASE PRESENTED

THE INLET CHANNEL STABILITY ANALYSIS WAS PERFORMED USING BORINGS 61-BMU-01AND 62-BMU-01. THE ANALYSIS RESULTING IN THE LOWEST FACTOR OF SAFETY IS PRESENTED ON THIS PLATE. THE ANALYSIS WAS PERFORMED USING GEOSLOPE COMPUTER SOFTWARE.

SOIL SHEAR STRENGTHS - BORING 61-BMU-01

SOIL NO.	SOIL TYPE	ELEVATION (N.G.V.D.)		UNIT WT. (PCF)	Q		R		S	
					φ ()°	C (PSF)	φ ()°	C (PSF)	φ ()°	C (PSF)
1	SM	237	233.5	125	30	0	30	0	30	0
2	SP	233.5	224	125	30	0	30	0	30	0
3	SM	224	221	125	33	0	33	0	33	0
4	SP	221	211	125	30	0	30	0	30	0
5	SP	211	194	125	32	0	32	0	32	0
6	SP	194	187	125	36	0	36	0	36	0

SOIL SHEAR STRENGTHS - BORING 62-BMU-01

SOIL NO.	SOIL TYPE	ELEVATION (N.G.V.D.)		UNIT WT. (PCF)	Q		R		S	
					φ ()°	C (PSF)	φ ()°	C (PSF)	φ ()°	C (PSF)
1	CH	242	237	115	0	750	12	375	24	0
2	ML	237	231.5	120	20	300	20	300	28	0
3	CH	231.5	230	115	0	750	10	375	20	0
4	SM	230	218	125	32	0	32	0	32	0
5	SP	218	215	125	34	0	34	0	34	0
6	CH	215	213	120	0	1000	10	500	20	0
7	SP	213	189	125	33	0	33	0	33	0
8	SP	189	179	125	32	0	32	0	32	0
9	SP	179	153	125	36	0	36	0	36	0

MANUAL SLOPE STABILITY CALCUALTIONS (61-BMU-01)

SLICE	DRIVE WT. KIPS	RESIST WT. KIPS	ALPHA DEGRESS	SIN ALPHA	COS ALPHA	NORMAL FORCE KIPS	ARC LENGTH FEET	NORMAL * TAN (PHI)	TAN FORCE KIPS
1	1.27	1.27	59.10	0.86	0.51	1.09	6.70	0.63	0.65
2	0.39	0.39	61.32	0.88	0.48	0.34	1.00	0.20	0.19
3	4.74	3.51	66.05	0.91	0.41	3.21	10.00	1.85	1.92
4	0.83	0.45	70.51	0.94	0.33	0.43	1.50	0.25	0.28
5	2.22	1.30	73.12	0.96	0.29	1.24	4.50	0.72	0.64
6	1.98	1.44	77.33	0.98	0.22	1.41	5.60	0.81	0.43
7	1.42	1.38	82.29	0.99	0.13	1.37	6.80	0.79	0.19
8	0.61	0.61	87.26	1.00	0.05	0.61	5.70	0.35	0.03
9	0.06	0.06	90.63	1.00	-0.01	0.06	2.30	0.03	0.00

SUM 5.63 4.34

$$FS = \frac{5.63}{4.34} = 1.30^{*}$$

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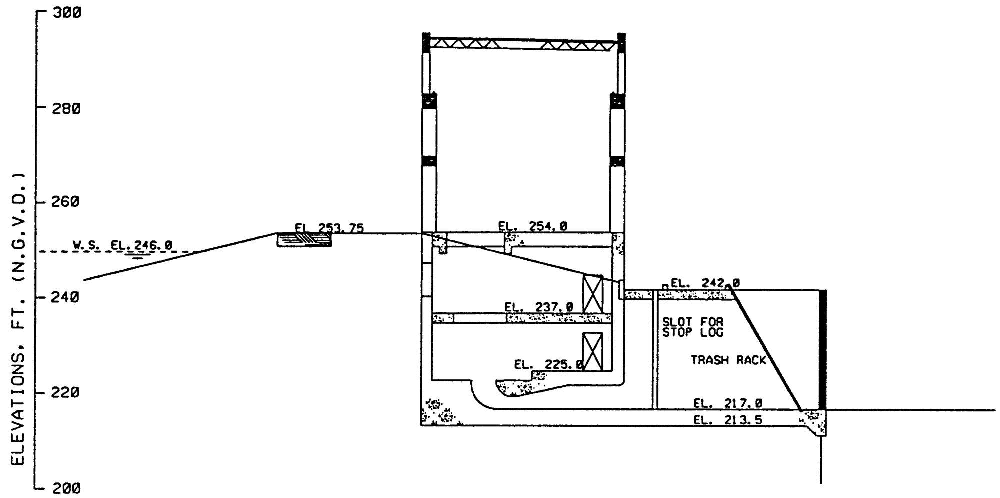
PUMPING STATION #1
INLET CHANNEL STABILITY

PLATE II-32



SLIDING ANALYSIS (CASE II)

**PASSIVE RESISTANCE WAS CONSERVATIVELY NEGLECTED IN THIS ANALYSIS

CASES ANALYZED

CASE I - AFTER CONSTRUCTION CONDITION. NO WATER IN OUTLET CHANNEL.

CASE II - SUDDEN DRAWDOWN CONDITION. WATER IN BACKFILL AT EL. 246.0, WATER IN OUTLET CHANNEL AT 241.0.

CASE III - PARTIAL POOL CONDITION. WATER ELEVATION IN THE BACKFILL SAME AS WATER ELEVATION IN OUTLET CHANNEL, WITH VARYING POOL ELEVATIONS.

CASE IV - EARTHQUAKE CONDITION. WATER IN BACKFILL AND OUTLET CHANNEL AT EL. 233.0.

FACTORS OF SAFETY

FAILURE PLANE	CASE A Q-STRENGTH	REQUIRED MIN. F.S.
CASE I	2.01	1.50
CASE II	1.54*	1.33
CASE III	1.74	1.50
CASE IV	1.27	1.10

*CASE PRESENTED

THE ABOVE FACTORS OF SAFETY WERE CALCULATED USING $\phi = 30^\circ$ FOR THE BACKFILL.

SLIDING STABILITY CALCULATIONS

(DEVELOPED TAN ϕ METHOD)
(CASE II - SUDDEN DRAWDOWN)

FS = 1.0

ϕ = TAN 30°/1.0 = 0.5774° ϕ = 30.0 °

$(45° - \phi/2)$ = 30.0 ° $(45° + \phi/2)$ = 60.0 °

CREEP PATH = 32.5 + 84 = 116.5 FT

PRESSURE AT HEEL = [(246-213.5)-(32.5/116.5)*5](0.0624) = 1.94 KSF

PRESSURE AT TOE = [(246-213.5)-(116.5/116.5)*5](0.0624) = 1.72 KSF

PRESSURE AT GROUND SURFACE = 27.5(0.0625) = 1.72 KSF

WT. OF ACTIVE WEDGE = $[0.5(40.25)^2(\text{TAN } 30^\circ)]\ 0.125 = 58.46^K$

WT. OF STRUCTURE = 263.5

WT. OF PASSIVE WEDGE = 0^K

RESISTING WATER = $0.5(27.5)^2(0.0624) = 23.6^K$

UPLIFT OF ACTIVE WEDGE = $0.5(1.94)(37.5) = 36.42^K$

UPLIFT OF STRUCTURE = $0.5(1.94 + 1.72)(84) = 153.59^K$

UPLIFT OF PASSIVE WEDGE = 0^K

$D_A = W\ \text{TAN}\ (45° + \phi/2) = 58.46\ \text{TAN}\ 60° = 101.25^K$

$D_P = W\ \text{TAN}\ (45° - \phi/2) = 0^K$

$R_A = 2[W - U_{SIN}(45° - \phi/2)]\ \text{TAN}\ \phi$
$= 2[58.46 - 36.42\ \text{SIN}\ 30°]\ \text{TAN}\ 30° = 46.47^K$

$R_B = (W-U)\ \text{TAN}\ \phi = (263.5 - 153.59)\ \text{TAN}\ 30° = 63.46^K$

$R_P = 2[W - U_{COS}(45° - \phi/2)]\ \text{TAN}\ \phi = 0^K$

$\Sigma R = 46.47 + 63.46 + 0 = 109.93^K$

$\Sigma D = 101.25 - 0 - 23.6 = 77.66^K$

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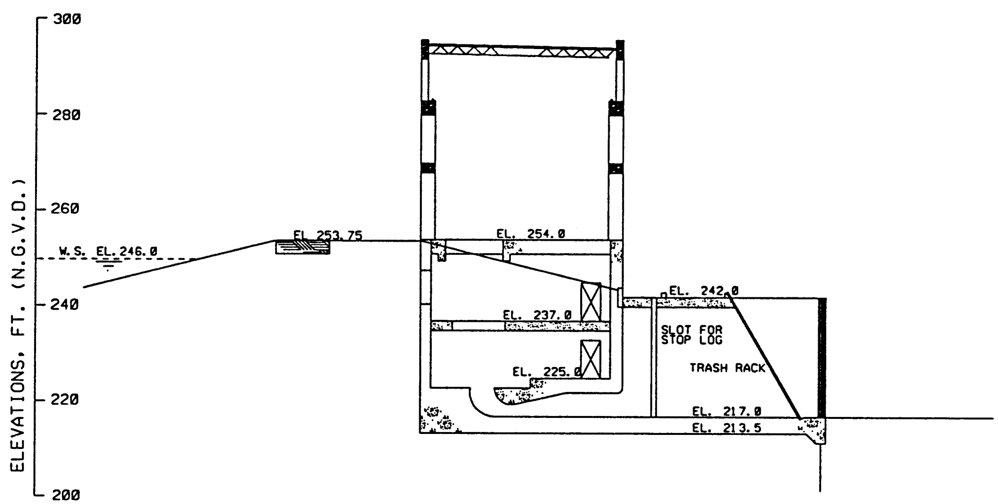
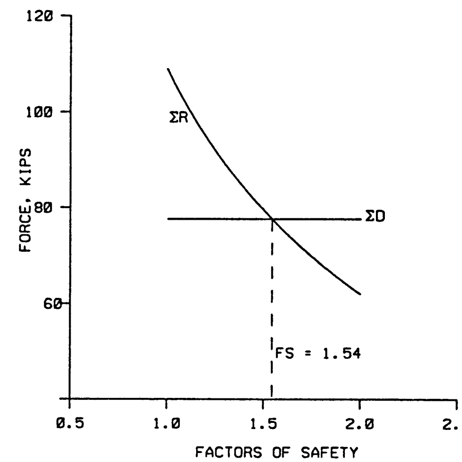
PUMPING STATION #1 SLIDING STABILITY ANALYSIS

PLATE II-33





SLIDING STABILITY CALCULATIONS

(DEVELOPED TAN ϕ METHOD)
(CASE II - SUDDEN DRAWDOWN)

FS = 1.54

ϕ = TAN 30°/1.54 = 0.374 ° ϕ = 20.55°

(45° - ϕ/2) = 34.72°   (45° + ϕ/2) = 55.28°

CREEP PATH = 32.5 + 84 = 116.5 FT

PRESSURE AT HEEL = [(246-213.5)-(32.5/116.5)•5](0.0624) = 1.94 KSF

PRESSURE AT TOE = [(246-213.5)-(116.5/116.5)•5](0.0624) = 1.72 KSF

PRESSURE AT GROUND SURFACE = 27.5(0.0625) = 1.72 KSF

WT. OF ACTIVE WEDGE = [0.5(40.25)^2 (TAN 34.72°)] 0.125 = 70.18^K

WT. OF STRUCTURE = 263.5

WT. OF PASSIVE WEDGE = 0^K

RESISTING WATER = 0.5(27.5)^2(0.0624) = 23.6^K

UPLIFT OF ACTIVE WEDGE = 0.5(1.94)(39.54) = 38.38^K

UPLIFT OF STRUCTURE = 0.5(1.94 + 1.72)(84) = 153.59^K

UPLIFT OF PASSIVE WEDGE = 0^K

D_A = W TAN (45° + ϕ/2) = 70.18 TAN 55.28° = 101.25^K

D_P = W TAN (45° - ϕ/2) = 0^K

R_A = 2[W-U_{SIN}(45° - ϕ/2)]TAN ϕ
= 2[70.18-38.38•SIN 34.72°]TAN 34.72° = 36.23^K

R_B = (W-U) TAN ϕ = (263.5 - 153.59) TAN 20.55° = 41.20^K

R_P = 2[W-U_{COS}(45° - ϕ/2)] TAN ϕ = 0^K

ΣR = 36.22 + 41.20 + 0 = 77.43 ^{K}

ΣD = 101.25 - 0 - 23.6 = 77.66^K

FS = 2.0

ϕ = TAN 30°/2.0 = 0.2887 ° ϕ = 16.10°

(45° - ϕ/2) = 36.95°   (45° + ϕ/2) = 53.05°

CREEP PATH = 32.5 + 84 = 116.5 FT

PRESSURE AT HEEL = [(246-213.5)-(32.5/116.5)•5](0.0624) = 1.94 KSF

PRESSURE AT TOE = [(246-213.5)-(116.5/116.5)•5](0.0624) = 1.72 KSF

PRESSURE AT GROUND SURFACE = 27.5(0.0625) = 1.72 KSF

WT. OF ACTIVE WEDGE = [0.5(40.25)^2 (TAN 36.95°)] 0.125 = 76.16^K

WT. OF STRUCTURE = 263.5

WT. OF PASSIVE WEDGE = 0^K

RESISTING WATER = 0.5(27.5)^2(0.0624) = 23.6^K

UPLIFT OF ACTIVE WEDGE = 0.5(1.94)(40.66) = 39.47^K

UPLIFT OF STRUCTURE = 0.5(1.94 + 1.72)(84) = 153.59^K

UPLIFT OF PASSIVE WEDGE = 0^K

D_A = W TAN (45° + ϕ/2) = 76.16 TAN 53.05° = 101.25^K

D_P = W TAN (45° - ϕ/2) = 0^K

R_A = 2[W-U_{SIN}(45° - ϕ/2)]TAN ϕ
= 2[76.16-39.47 SIN 36.95°]TAN 36.95° = 30.27^K

R_B = (W-U) TAN ϕ = (263.5 - 153.59) TAN 16.12° = 31.72^K

R_P = 2[W-U_{COS}(45° - ϕ/2)] TAN ϕ = 0^K

ΣR = 30.27 + 31.73 + 0 = 62.0 ^{K}

ΣD = 101.25 - 0 - 23.6 = 77.66^K

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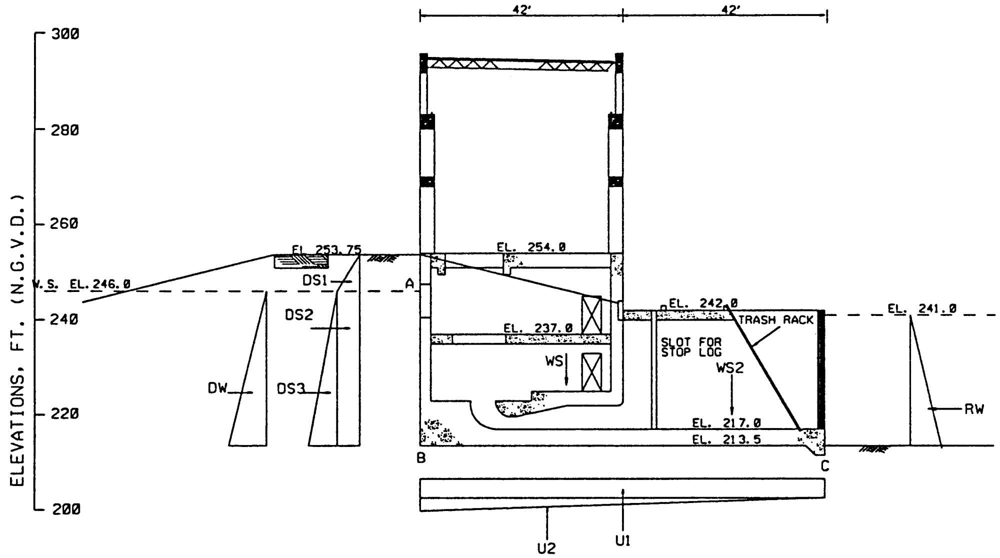
PUMPING STATION #1
SLIDING STABILITY ANALYSIS

PLATE
II-34



UPLIFT

BASE WIDTH = (84' • 98') + (42• 28.5') = 9429 S.F.

PRESSURE AT EL. 213.5 = ($\frac{(246-241)}{2}$ - 213.5)(0.0625) = 1.875 KSF

UPLIFT FORCE = 9429(1.875) ≈ 17,679 KIPS

RESISTING UPLIFT

WEIGHT OF SUBSTRUCTURE (CONCRETE ONLY) = 22792 KIPS
(NO LIVE LOAD OR EQUIPMENT WEIGHT)

WEIGHT OF WATER ON PUMPING STATION
BELOW EL. 242.0
(35' WIDE X 76' LONG X 25' HEIGHT) 0.0624 = 4150 KIPS

ABOVE EL. 242.0
(35' WIDE X 98' LONG X 1' HEIGHT) 0.0624 = 214.0 KIPS

WEIGHT OF WATER = 4150 + 214 = 4364 KIPS

TOTAL WEIGHT = 22792 + 4364 ≅ 27,200 KIPS

FS AGAINST UPLIFT: $FS = \frac{\text{TOTAL WIEGHT}}{\text{UPLIFT}}$

$FS = \frac{27,200}{17,679}$

FS = 1.54 ∴ O.K.

BEARING CAPACITY

THE EXCAVATION WILL EXTEND TO EL. 209.5. FOUR FEET OF COMPACTED PERVIOUS BACKFILL WILL BE PLACED UNDER THE STRUCTURE TO EL. 213.5. THE STRUCTURE WILL THEN BE CONSTRUCTED ON GRADE.

STRUCTURE = B = 126.5' AND L = 84'; L/6 = 84/6 = 14'

B' = B - 2e

CASE I (AC) - e = 1.27'* B' = 81.45' V = 200.6
CASE II (SD) - e = 10.55' B' = 62.89' V = 109.91
CASE III (PP) - e = 4.18' B' = 75.64' V = 140.32
CASE IV (EQ) - e = 9.14' B' = 65.73' V = 140.32
FOR ALL CASES - e < L/6 .. FULL FOOTING
* e IS FROM OVERTURNING ANALYSES

$$Q = \frac{V}{BL}\left(1 + \frac{6e}{L}\right)^{**}$$

$$Q_{max} = \frac{200.6}{84}\left[1 + \frac{6(1.27)}{84}\right] = 2.61 \text{ K/S.F.} \quad \text{CASE I}$$

$$Q_{max} = \frac{109.91}{84}\left[1 + \frac{6(10.55)}{84}\right] = 2.29 \text{ K/S.F.} \quad \text{CASE II}$$

$$Q_{max} = \frac{140.32}{84}\left[1 + \frac{6(4.18)}{84}\right] = 2.17 \text{ K/S.F.} \quad \text{CASE III}$$

$$Q_{max} = \frac{140.32}{84}\left[1 + \frac{6(9.14)}{84}\right] = 2.76 \text{ K/S.F.} \quad \text{CASE IV}$$

$$q_{ult} = \cancel{cN_cS_cd_cI_cg_cb_c}^{\,0} + \bar{q}N_qS_qd_qI_qg_qb_q + 0.5\gamma B N_\gamma S_\gamma d_\gamma I_\gamma g_\gamma b_\gamma R_\gamma{}^{**}$$

ASSUME ϕ = 30; N_c = 30.14; N_q = 18.4; N_γ = 15.67

γ' = 0.0625 D = 4' B = 84' L = 126.5'

$\bar{q} = \gamma' D$ = 0.0625(4) = 0.25

$S_q = 1 + 0.1 \cdot N_\phi (B'/L')$ = 1 + 0.1•3 (81.45/126.5) = 1.19
$S_\gamma = S_q$ = 1.19

$d_q = 1 + 0.1 \cdot \sqrt{N_\phi}\, D/B$ = 1 + 0.1•1.73•(4/84) = 1.01
$d_\gamma = d_q$ = 1.01

$R_\gamma = 1 - 0.25 \cdot \text{LOG}\left(\frac{B}{6}\right)$ WHERE B> 6 FT.
$i_q = g_q = b_q = d_\gamma = i_\gamma = g_\gamma = b_\gamma = 1$

Q CASE = S CASE

CASE I q_{ult} = 0.25(18.4)(1.19)(1.01)(1) + 0.5(0.0625)(81.45)(15.67)(1.19)(1.01)(0.71)(1)
= 39.8 KSF

CASE II q_{ult} = 30.8 KSF

CASE III q_{ult} = 37.0 KSF

CASE IV q_{ult} = 32.2 KSF

CASE I F.S. = $\frac{q_{ult}}{Q_{max}} = \frac{39.8}{2.61}$ = 15.29

CASE II F.S. = $\frac{30.8}{2.29}$ = 13.44

CASE III F.S. = $\frac{37.0}{2.17}$ = 17.04

CASE IV F.S. = $\frac{32.2}{2.76}$ = 11.66 SAFE, ALL CASES > 3.0

** FROM EM 1110-1-1905 'BEARING CAPACITY OF SOILS' PAGES 4-4 THROUGH 4-14.

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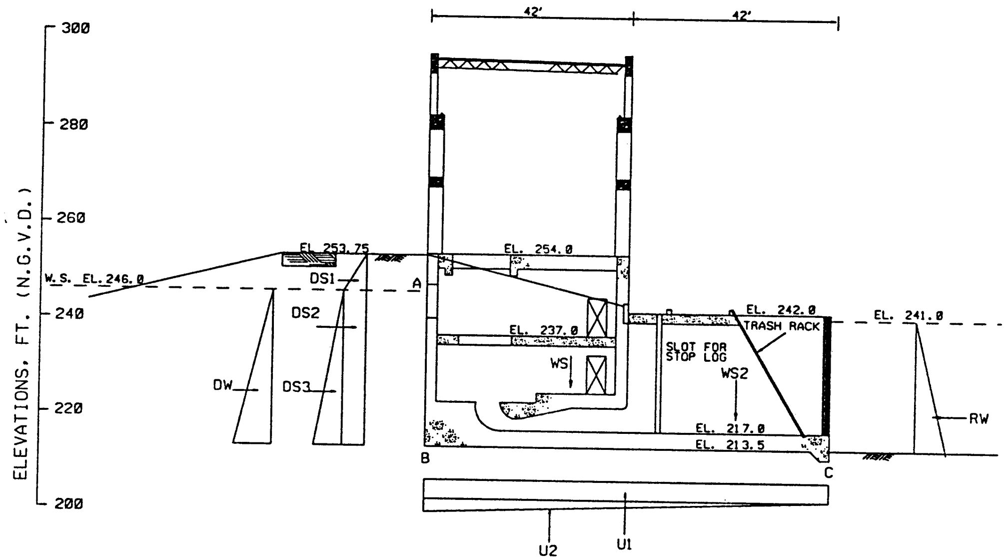
PUMPING STATION #1 BEARING & UPLIFT ANALYSIS

PLATE
II-35



NOTE: PASSIVE RESISTANCE OF THE SOIL WAS IGNORED IN THIS ANALYSIS.

ITEM	FORCES (KIPS) VERTICAL ↓	VERTICAL ↑	HORIZONTAL →	HORIZONTAL ←	MOMENT ARM (FEET)	MOMENTS (KIP-FEET) OVERTURNING	RESISTING
WS	200.6				46.67		9361.40
WS2	62.9				21.0		1320.90
D_{S1}			1.9		35.08	65.85	
D_{S2}			15.8		16.25	255.94	
D_{S3}			16.5		10.83	179.02	
D_W			33.0		10.83	356.96	
U_1		144.1			42.0	6053.88	
U_2		9.4			56.0	526.85	
R_{W1}				23.6	9.17		216.34
TOTALS	263.5	153.5	67.1	23.6		7438.5	10898.6

OVERTURNING ANALYSIS

(LOADING DIAGRAM - CASE II)
CASE II - SUDDEN DRAWDOWN CONDITION - WATER IN BACKFILL AT EL. 246.0; WATER IN THE OUTLET CHANNEL AT EL. 241.0.

$K_0 = (1 - \sin \phi)(1 + \sin B)$
$K_0 = (1 - \sin 30°)(1 + \sin 0)°$
$K_0 = 0.50$

CREEP PATH = 32.5 + 84 + 0 = 116.5 FT
PRESSURE AT B = [(246-213.5)-(32.5/116.5)5](0.0624) = 1.94 KSF
PRESSURE AT C = [(246-213.5)-(116.5/116.5)5](0.0624) = 1.72 KSF

$D_{S1} = 0.5(7.75)^2(0.125)(0.50) = 1.88^K$
$D_{S2} = 0.125(7.75)(32.5)(0.5) = 15.74^K$
$D_{S3} = 0.5(32.5)^2(0.0626)(0.5) = 16.53^K$
$D_W = 0.5(32.5)^2(0.0624) = 32.95^K$
$U_1 = 1.72(86) = 144.1^K$
$U_2 = 0.5(1.940 - 1.72)(86) = 9.45^K$
$R_W = 0.5(27.5)^2(0.0624) = 23.59^K$

STRUCTURE - TOTAL WEIGHT (STRUCTURE WITHOUT PUMPS) = 25378.5^K
25378.5 / 126.5'(WIDTH) = 200.6 K/FT. WIDTH
WATER - (42)(24)(0.0624) = 62.90 K/FT. WIDTH
TOTAL WEIGHT = 263.5 K/FT. WIDTH

$\bar{y}$ = RESULTANT LOCATION = $\frac{\Sigma M}{\Sigma V} = \frac{10898.6 - 7438.5}{263.5 - 153.55} = 31.47'$ FROM C

KERN 28' TO 56' FROM C

THEREFORE, RESULTANT FORCE IS LOCATED WITHIN THE KERN AND THE STRUCTURE IS SAFE AGAINST OVERTURNING.

CASE I (AFTER CONSTRUCTION) RESULTED IN A $\bar{y}$ = 43.27'
CASE III (PARTIAL POOL) RESULTED IN A $\bar{y}$ = 37.82'
CASE IV (EARTHQUAKE) RESULTED IN A $\bar{y}$ = 32.86'

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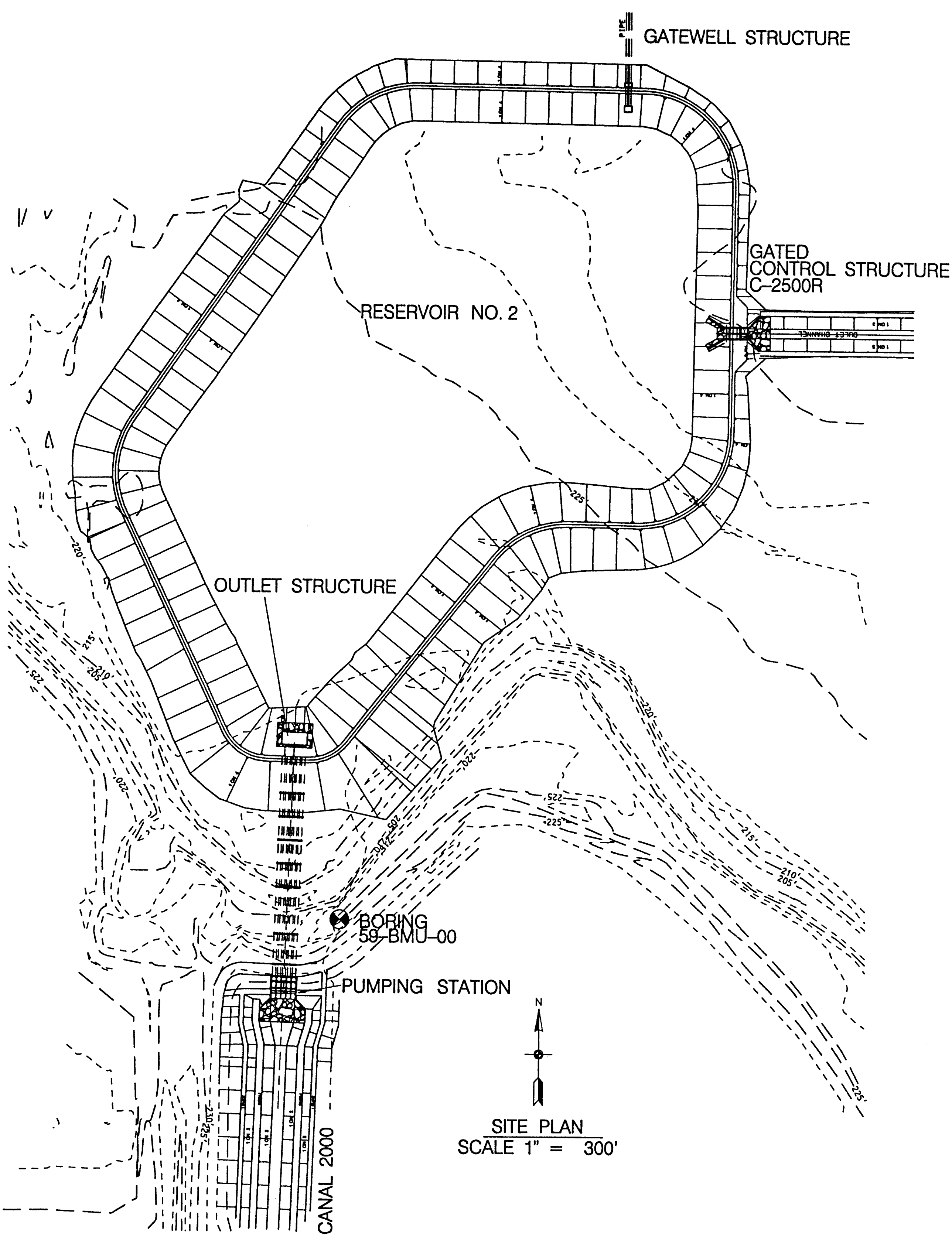
PUMPING STATION #1 OVERTURNING ANALYSIS

PLATE
II-36



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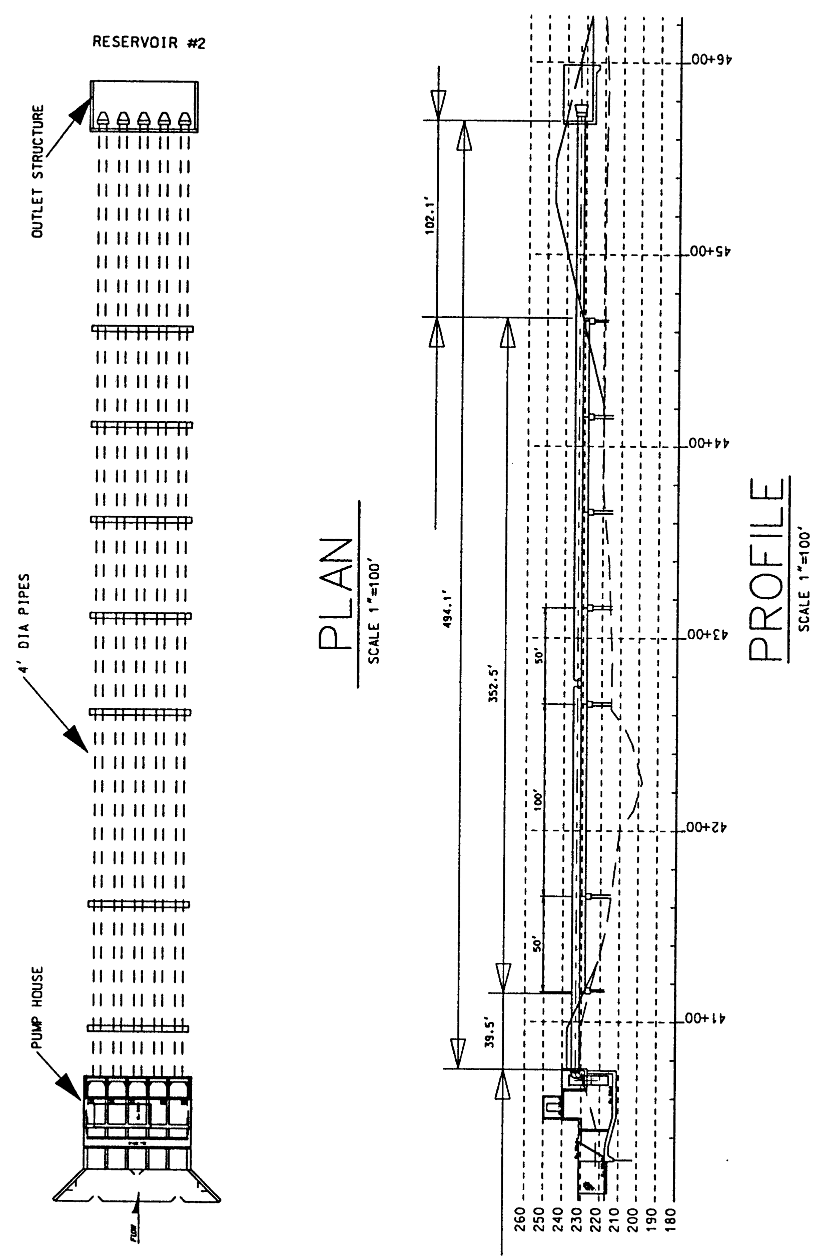
PUMPING STATION #2
SITE PLAN

PLATE

II-37



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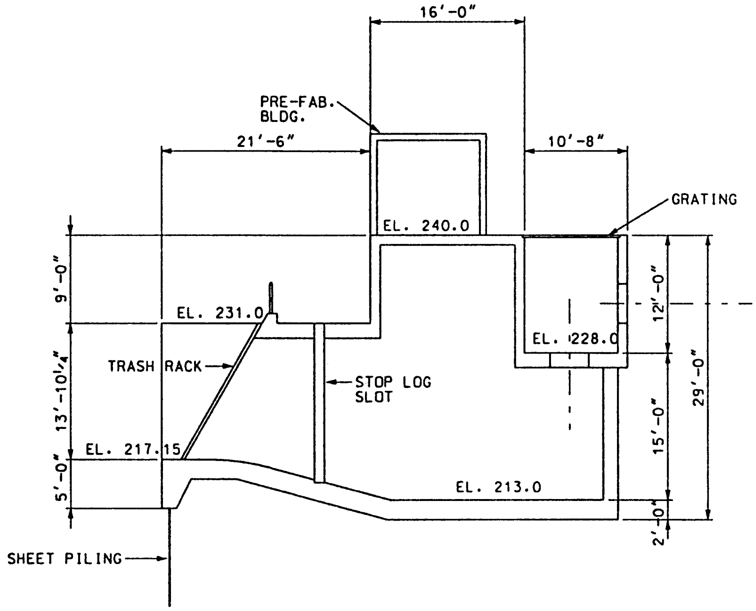
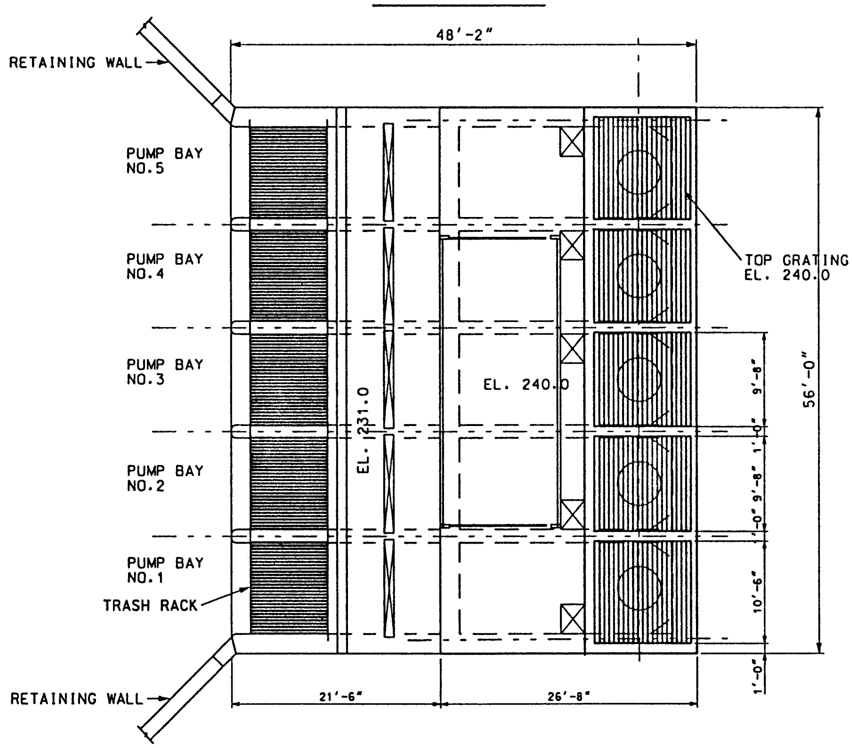
PUMPING STATION #2
PLAN & PROFILE

PLATE

II-38



SECTION



PLAN @ EL. 240/231

SCALE 1"=20'

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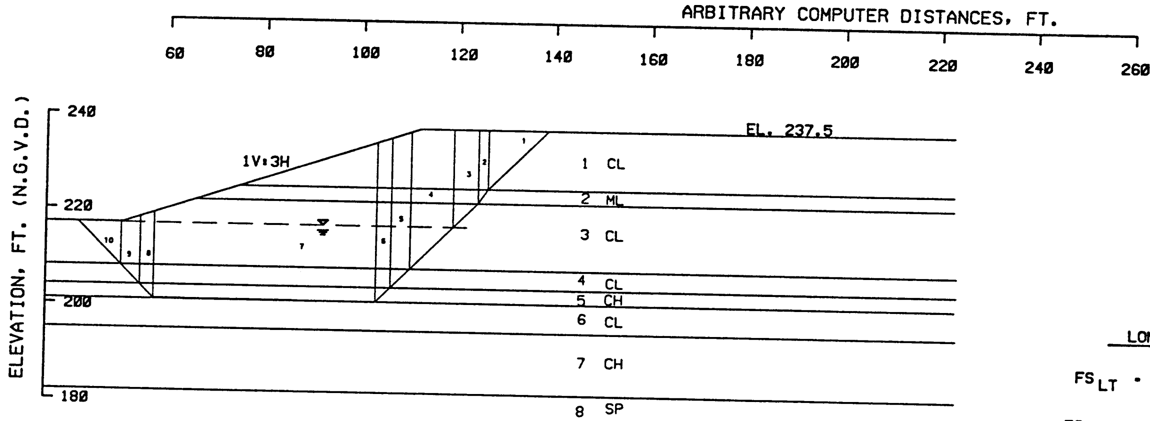
PUMPING STATION #2
PLAN & SECTION

PLATE

II-39



SOIL STRATIFICATION FROM 59-BMU-00

COMPUTER RESULTS
GEOSLOPE

	ARC METHOD		WEDGE METHOD	
BORING #	A.C.	S.D.	A.C.	S.D.
59-BMU-00	1.571	NA	1.423*	NA

* CASE PRESENTED

THE INLET CHANNEL STABILITY ANALYSIS WAS PERFORMED USING BORING 59-BMU-00 . THE ANALYSIS RESULTING IN THE LOWEST FACTOR OF SAFETY IS PRESENTED ON THIS PLATE. THE ANALYSIS WAS COMPUTED USING GEOSLOPE COMPUTER SOFTWARE.

SOIL SHEAR STRENGTHS - BORING 59-BMU-00

SOIL NO.	SOIL TYPE	ELEVATION (N.G.V.D.)		UNIT WT. (PCF)	Q		R		S	
					ϕ (°)	C (PSF)	ϕ (°)	C (PSF)	ϕ (°)	C (PSF)
1	CL	230	225	120	0	500	13.25	250	26.5	0
2	ML	225	222	120	20	300	20	300	28	0
3	CL	222	208	120	0	750	12.5	375	25	0
4	CL	208	204	120	0	500	14	250	28	0
5	CH	204	201	120	0	500	11.5	250	23	0
6	CL	201	195	120	0	1200	13	600	26	0
7	CH	195	182	120	0	1100	10.5	550	21	0
8	SP	182	160	125	34	0	34	0	34	0

LONG TERM STABILITY

$$FS_{LT} = \frac{\text{TAN } \phi}{\text{TAN I}} = \text{M TAN } \phi$$

$$FS_{LT} = \text{M} \; \frac{(12.5 \cdot \text{TAN } 26.5) + (3 \cdot \text{TAN } 28) + (5 \cdot \text{TAN } 25)}{20.5}$$

FS_{LT} = M(0.4956)

FOR M = 3 FS = 1.48

MANUAL COMPUTATIONS

D_A = W TAN (45° + ϕ/2)
D_A = 9.38 + (3.52•TAN 55) + 11.07 + 26.64 + 13.98 + 10.59
D_A = 76.69 K

R_A = 2[W TAN ϕ + CH TAN (45° - ϕ/2)]
R_A = 2[.5(12.5) + (3.52•TAN 20 + .3(3) TAN35) + .75(5) + .75(9) + .5(4) + .5(2.97)]
R_A = 44.30 K

D_B = 0 K

R_B = W TAN ϕ + CL
R_B = 0.5(45.96)
R_B = 22.98 K

D_P = W TAN (45° - ϕ/2)
D_P = 4.56 + 5.05 + 4.86
D_P = 14.47 K

R_P = 2[W TAN ϕ + CH TAN (45° + ϕ/2)]
R_P = 2[.5(2.85) + .75(4) + .5(9)]

R_P = 20.35 K

$$FS = \frac{R_A + R_B + R_P}{D_A - D_P} = \frac{44.30 + 22.98 + 20.35}{76.69 - 14.47}$$

FS = 1.408

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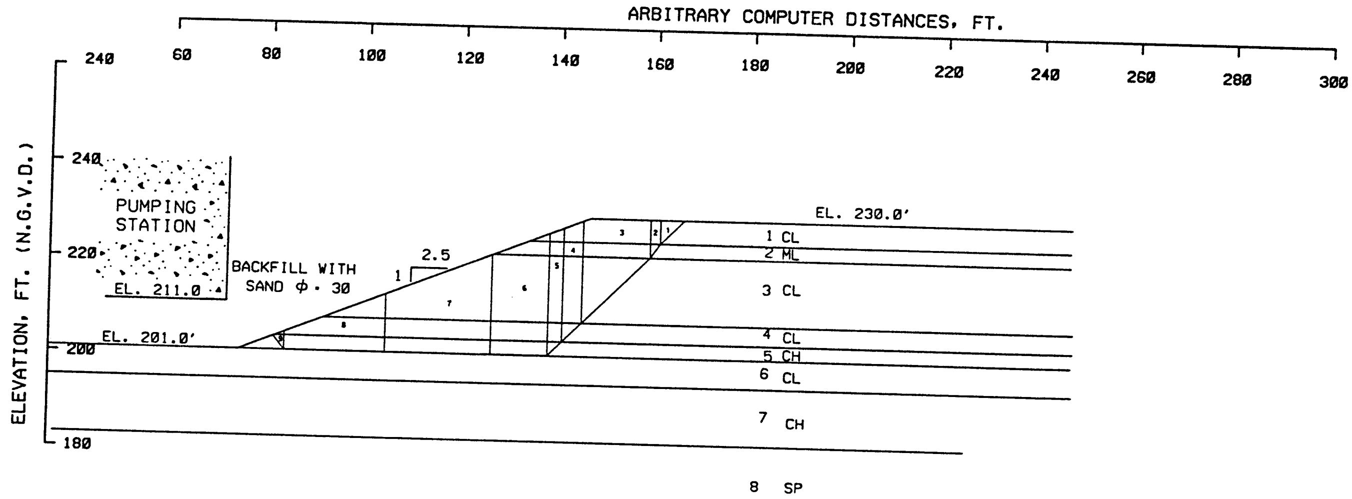
PUMPING STATION #2
INLET CHANNEL STABILITY

PLATE
II-40



•DUE TO THE TEMPORARY NATURE OF THE EXCAVATION, NO LONG TERM ANALYSIS WAS PERFORMED

COMPUTER RESULTS
GEOSLOPE

	ARC METHOD		WEDGE METHOD	
BORING •	A.C.	S.D.	A.C.	S.D.
59-BMU-00	1.480	NA	1.367*	NA

* CASE PRESENTED

THE PUMPING STATION •2 STABILITY ANALYSIS WAS PERFORMED USING BORING 59-BMU-00 . THE ANALYSIS RESULTING IN THE LOWEST FACTOR OF SAFETY IS PRESENTED ON THIS PLATE. THE ANALYSIS WAS COMPUTED USING GEOSLOPE COMPUTER SOFTWARE.

SOIL SHEAR STRENGTHS - BORING 59-BMU-00

SOIL NO.	SOIL TYPE	ELEVATION (N.G.V.D.)		UNIT WT. (PCF)	Q		R		S	
					φ (°)	C (PSF)	φ (°)	C (PSF)	φ (°)	C (PSF)
1	CL	230	225	120	0	500	13.25	250	26.5	0
2	ML	225	222	120	20	300	20	300	28	0
3	CL	222	208	120	0	750	12.5	375	25	0
4	CL	208	204	120	0	500	14	250	28	0
5	CH	204	201	120	0	500	11.5	250	23	0
6	CL	201	195	120	0	1200	13	600	26	0
7	CH	195	182	120	0	1100	10.5	550	21	0
8	SP	182	160	125	34	0	34	0	34	0

MANUAL COMPUTATIONS

D_A = W TAN (45° + φ/2)
D_A = 1.5 + (1.64•TAN 55) + 25.15 + 10.85 + 8.65
D_A = 48.48 K

R_A = 2[W TAN φ + CH TAN (45° - φ/2)]
R_A = 2[.5(5) + (.3(3)•TAN 35 +1.64•TAN 20) + .75(14) + .5(4) + .5(2.91)]

R_A = 35.36 K

D_B = 0 K

R_B = W TAN φ + CL
R_B = 0.5(54.59)
R_B = 27.30 K

D_P = W TAN (45° - φ/2)
D_P = 0.54 K

R_P = 2[W TAN φ + CH TAN (45° + φ/2)]
R_P = 2[.5(2.85)]

R_P = 2.85 K

$$FS = \frac{R_A + R_B + R_P}{D_A - D_P} = \frac{35.36 + 27.30 + 2.85}{48.48 - 0.54}$$

FS = 1.366

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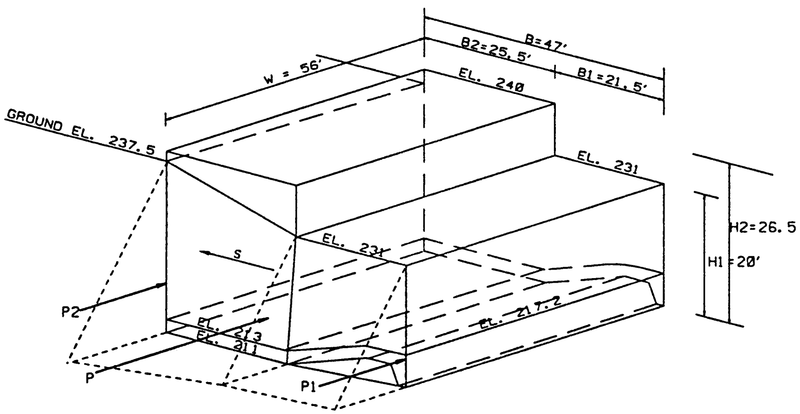
PUMPING STATION #2 EXCAVATION STABILITY

PLATE
II-41

PUMPING STATION #2



CASES ANALYZED

CASE I - AFTER CONSTRUCTION CONDITION. NO WATER IN OUTLET CHANNEL.

CASE II - SUDDEN DRAWDOWN CONDITION. WATER IN BACKFILL AT EL. 230.0. WATER IN OUTLET CHANNEL AT 225.0.

CASE III - PARTIAL POOL CONDITION. WATER ELEVATION IN THE BACKFILL SAME AS WATER ELEVATION IN OUTLET CHANNEL, WITH VARYING POOL ELEVATIONS.

CASE IV - EARTHQUAKE CONDITION. WATER IN BACKFILL AND OUTLET CHANNEL AT EL. 225.0.

WALL SIDE FRICTION CALCULATIONS

PRESSURE ON WALL $P = 0.5 H^2 \gamma K_0$

$P1 = 0.5 \cdot (20.0) \cdot 0.0626 \cdot 0.5 = 6.26$ KIPS/FT
$P2 = 0.5 \cdot (26.5) \cdot 0.0626 \cdot 0.5 = 10.99$ KIPS/FT

$$\text{TOTAL PRESSURE} = P = (P1 \cdot B1) + \left(\frac{P1 + P2}{2}\right) \cdot B2$$

$$P = (6.26 \cdot 21.5) + \left(\frac{6.26 + 10.99}{2}\right) \cdot 25.5$$

P = 354.53 KIPS

SIDE FRICTION DEVELOPED ALONG WALL

$S = P \cdot \tan\phi$
$S = 354.53 \cdot \tan 30$
S = 204.68 KIPS
FOR TWO SIDES $S = 204.68 \cdot 2 = 409.37$ KIPS
S = 409.37 KIPS / 56 FT = 7.31 KIPS / FT. OF STRUCTURE

SLIDING STABILITY
FACTORS OF SAFETY

FAILURE PLANE	CALC. F.S SLIDING WITH SIDE FRICTION (7.31 KIPS)	CALC. F.S SLIDING NO SIDE FRICTION	REQUIRED MIN. F.S.
CASE I	1.96	1.58	1.50
CASE II	1.66*	1.26	1.33
CASE III	1.96	1.50	1.50
CASE IV	1.33	1.09	1.10

*CASE PRESENTED

THE ABOVE FACTORS OF SAFETY WERE CALCULATED USING $\phi = 30°$ FOR THE BACKFILL USING THE WES PROGRAM CSLIDE (X0075).

SIDE FRICTION WAS USED IN THE CALCULATION TO MEET THE REQUIRED FACTOR OF SAFETY. THE TABLE ABOVE INDICATES THE RESULTING FACTORS WITH AND WITHOUT THE INFLUENCE OF SIDE FRICTION. THE BASE SLAB MAY BE EXTENDED BY 1 FOOT AROUND THE STRUCTURE TO MEET THE MINIMUM FACTOR OF SAFETY REQUIREMENTS.

SLIDING ANALYSIS (CASE II)

SLIDING STABILITY CALCULATIONS

(DEVELOPED TAN ϕ METHOD)
(CASE II - SUDDEN DRAWDOWN)

FS = 1.0

$\phi = \tan 30°/1.0 = \underline{0.5774}$ $\quad \phi = \underline{30.0}°$

$(45° - \phi/2) = \underline{30.0}°$ $\quad (45° + \phi/2) = \underline{60.0}°$

CREEP PATH = 19 + 47.2 + 6 = 72.2 FT

PRESSURE AT HEEL = [(230-211)-(19/72.2)*5](0.0624) = 1.103 KSF

PRESSURE AT TOE = [(230-211)-(66.2/72.2)*5](0.0624) = 0.9 KSF

PRESSURE AT GROUND SURFACE = (225-217) * (0.0624) = 0.5 KSF

WT. OF ACTIVE WEDGE = $[0.5(26.5)^2(\tan 30°)]\ 0.125 = \underline{25.34}^K$

WT. OF STRUCTURE = $\underline{81.22}^K$

WT. OF PASSIVE WEDGE = $[0.5(6)^2(\tan 60°)]\ 0.125 + 5.18 = \underline{9.09}^K$

RESISTING WATER = $0.5(8)^2(0.0624) = \underline{1.997}^K$

UPLIFT OF ACTIVE WEDGE = $0.5(1.103)(21.94) = \underline{12.105}^K$

UPLIFT OF STRUCTURE = $(0.9 \cdot 47.2) + (.5 \cdot (1.103-0.9) \cdot 47.2) = \underline{47.27}^K$

UPLIFT OF PASSIVE WEDGE = $0.5(0.9+0.5)(12) = \underline{8.40}^K$

$D_A = W \tan(45° + \phi/2) = 25.34 \tan 60° = \underline{43.89}^K$

$D_P = W \tan(45° - \phi/2) = 9.09 \cdot \tan 30 = \underline{5.245}^K$

$R_A = 2[W - U\sin(45° - \phi/2)]\tan\phi$
$= 2[25.34 - 12.105 \sin 30°]\tan 30° = \underline{22.27}^K$

$R_B = (W-U)\tan\phi = (81.22 - 47.27)\tan 30° = \underline{19.60}^K$

$R_P = 2[W - U\cos(45° - \phi/2)]\tan\phi$
$= 2[9.085 - 8.40 \cos 30°]\tan 30° = \underline{2.10}^K$

$\Sigma R = 22.27 + 19.60 + 2.10 + 7.31 = \underline{51.28}^K$

$\Sigma D = 43.89 - 5.245 - 1.997 = \underline{36.65}^K$

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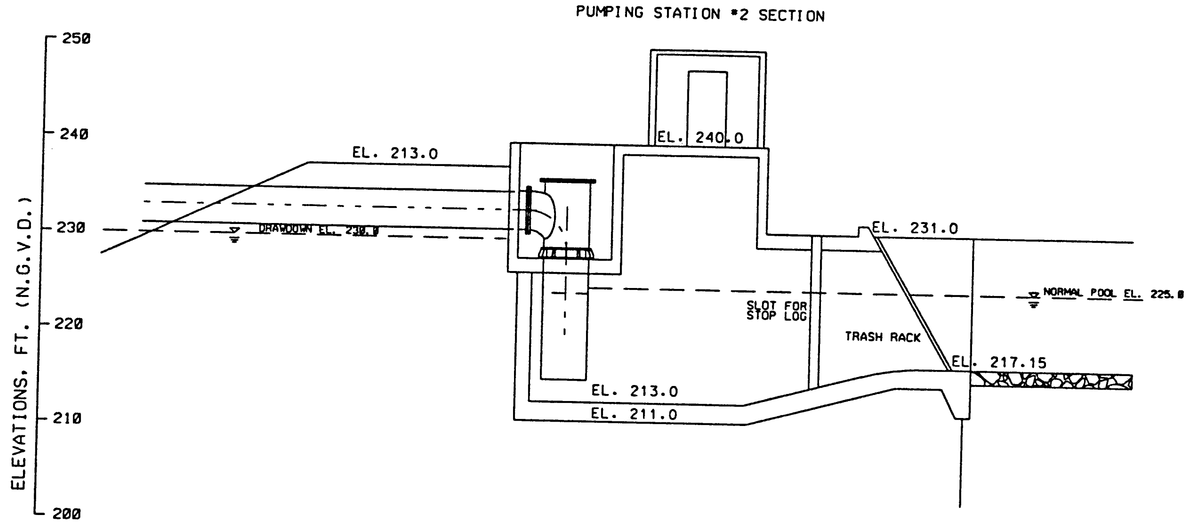
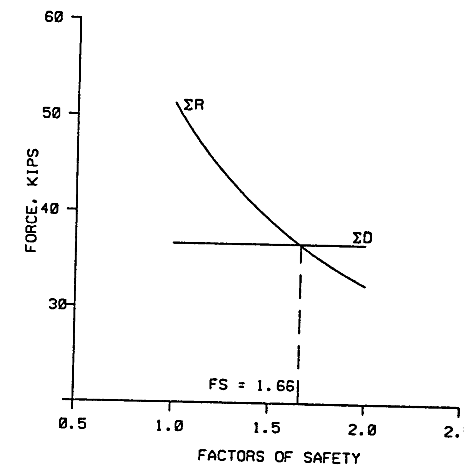
PUMPING STATION #2
SLIDING STABILITY ANALYSIS





SLIDING STABILITY CALCULATIONS

(DEVELOPED TAN ϕ METHOD)
(CASE II - SUDDEN DRAWDOWN)

FS = 1.66

ϕ = TAN 30°/1.661 = 0.3475 $\quad\phi$ = 19.16°

(45° - ϕ/2) = 35.42° $\quad$ (45° + ϕ/2) = 54.58°

CREEP PATH = 19 + 47.2 + 6 = 72.2 FT

PRESSURE AT HEEL = [(230-211)-(19/72.2)•5](0.0624) = 1.103 KSF

PRESSURE AT TOE = [(230-211)-(66.2/72.2)•5](0.0624) = 0.9 KSF

PRESSURE AT GROUND SURFACE = (225-217) • (0.0624) = 0.5 KSF

WT. OF ACTIVE WEDGE = [0.5(26.5)^2(TAN 35.42°)] 0.125 = 31.21^K

WT. OF STRUCTURE = 81.22^K

WT. OF PASSIVE WEDGE = [0.5(6)^2(TAN 54.58°)] 0.125 + 4.21 = 7.376^K

RESISTING WATER = 0.5(8)^2(0.0624) = 1.997^K

UPLIFT OF ACTIVE WEDGE = 0.5(1.103)(19/COS 35.42) = 12.86^K

UPLIFT OF STRUCTURE = (0.9•47.2) + (.5•(1.103-0.9)•47.2) = 47.27^K

UPLIFT OF PASSIVE WEDGE = 0.5(0.9+0.5)(6/COS 54.58) = 7.24^K

D_A = W TAN (45° + ϕ/2) = 31.21 TAN 54.58° = 43.89^K

D_P = W TAN (45° - ϕ/2) = 7.376 • TAN 35.42° = 5.245^K

R_A = 2[W-U SIN(45° - ϕ/2)] TAN ϕ
= 2[31.21-12.86 SIN 35.42°] TAN 19.17° = 16.52^K

R_B = (W-U) TAN ϕ = (81.22 - 47.27) TAN 19.17° = 11.80^K

R_P = 2[W-U COS(45° - ϕ/2)] TAN ϕ
= 2[7.38-7.24 COS 35.42°] TAN 19.17° = 1.026^K

ΣR = 16.52 + 11.80 + 1.026 + 7.31 = 36.65^K

ΣD = 43.89 - 5.25 - 1.997 = 36.65^K

FS = 2.0

ϕ = TAN 30°/2.0 = 0.2887 $\quad\phi$ = 16.10°

(45° - ϕ/2) = 36.95° $\quad$ (45° + ϕ/2) = 53.05°

CREEP PATH = 19 + 47.2 + 6 = 72.2 FT

PRESSURE AT HEEL = [(230-211)-(19/72.2)•5](0.0624) = 1.103 KSF

PRESSURE AT TOE = [(230-211)-(66.2/72.2)•5](0.0624) = 0.9 KSF

PRESSURE AT GROUND SURFACE = (225-217) • (0.0624) = 0.5 KSF

WT. OF ACTIVE WEDGE = [0.5(26.5)^2(TAN 36.95°)] 0.125 = 33.01^K

WT. OF STRUCTURE = 81.22^K

WT. OF PASSIVE WEDGE = [0.5(6)^2(TAN 53.05°)] 0.125 + 3.98 = 6.97^K

RESISTING WATER = 0.5(8)^2(0.0624) = 1.997^K

UPLIFT OF ACTIVE WEDGE = 0.5(1.103)(19/COS 36.95) = 13.12^K

UPLIFT OF STRUCTURE = (0.9•47.2) + (.5•(1.103-0.9)•47.2) = 47.27^K

UPLIFT OF PASSIVE WEDGE = 0.5(0.9+0.5)(6/COS 53.05) = 6.98^K

D_A = W TAN (45° + ϕ/2) = 33.01 TAN 53.05° = 43.89^K

D_P = W TAN (45° - ϕ/2) = 6.97 • TAN 36.95° = 5.25^K

R_A = 2[W-U SIN(45° - ϕ/2)] TAN ϕ
= 2[33.01-13.12 SIN 36.95°] TAN 16.10° = 14.51^K

R_B = (W-U) TAN ϕ = (81.22 - 13.12) TAN 16.10° = 9.80^K

R_P = 2[W-U COS(45° - ϕ/2)] TAN ϕ
= 2[6.97-6.98 COS 36.95°] TAN 16.10° = 0.81^K

ΣR = 14.51 + 9.80 + 0.81 + 7.31 = 32.42^K

ΣD = 43.89 - 5.25 - 1.997 = 36.65^K

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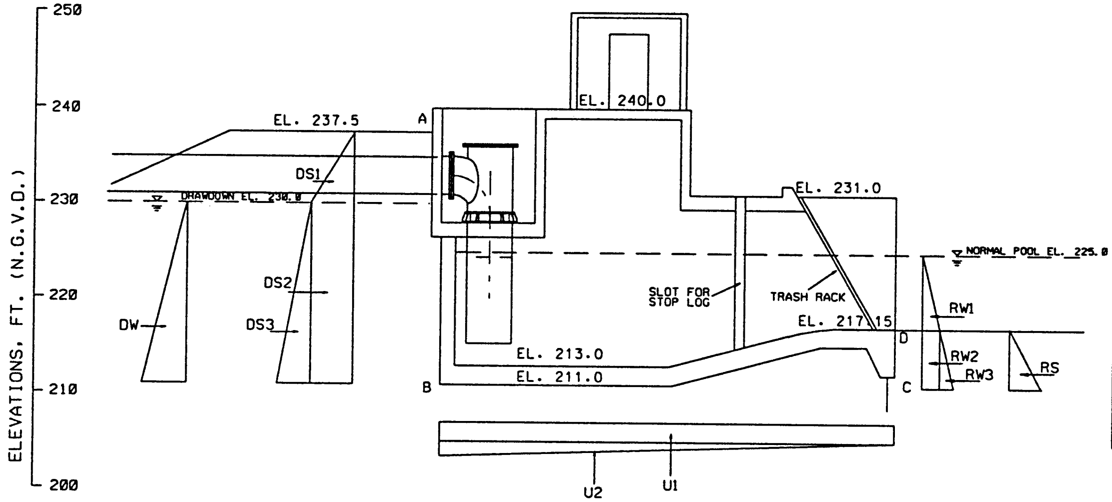
PUMPING STATION #2 SLIDING STABILITY ANALYSIS

PLATE

II-43



ITEM	FORCES (KIPS) VERTICAL ↓	FORCES (KIPS) VERTICAL ↑	FORCES (KIPS) HORIZONTAL →	FORCES (KIPS) HORIZONTAL ←	MOMENT ARM (FEET)	MOMENTS (KIP-FEET) OVERTURNING	MOMENTS (KIP-FEET) RESISTING
WS	58.13				27.32		1588.11
WS2	15.00				31.30		469.50
D_{s_1}			1.76		21.50	37.79	
D_{s_2}			8.91		9.50	84.61	
D_{s_3}			5.65		6.33	35.78	
D_W			11.26		6.33	71.32	
U_1		42.32			23.50	994.48	
U_2		4.77			31.33	149.55	
R_{W_1}				1.9	8.8		16.7
R_{W_2}				3.02	3.10		9.35
R_{W_3}				1.20	2.07		2.48
R_S				0.60	2.07		1.24
TOTALS	73.13	47.09	6.7	27.6		1373.5	2087.40

OVERTURNING ANALYSIS

(LOADING DIAGRAM - CASE II)
CASE II - SUDDEN DRAWDOWN CONDITION - WATER IN BACKFILL AT EL. 230.0; WATER IN THE OUTLET CHANNEL AT EL. 225.0.

$K_O = (1 - \sin \phi)(1 + \sin B)$
$K_O = (1 - \sin 30°)(1 + \sin 0°)$
$K_O = 0.50$

CREEP PATH = 19 + 47 + 6.2 = 72.2 FT

PRESSURE AT B = [(230-211)-(19/72.2)5](0.0624) = 1.1 KSF
PRESSURE AT C = [(230-211)-(66/72.2)5](0.0624) = 0.9 KSF

$D_{s_1} = 0.5(7.50)^2(0.125)(0.50) = 1.76^K$
$D_{s_2} = 0.125(7.50)(19.0)(0.5) = 8.91^K$
$D_{s_3} = 0.5(19.0)^2(0.0626)(0.5) = 5.65^K$
$D_W = 0.5(19.0)^2(0.0624) = 32.95^K$
$U_1 = 0.90(47) = 42.32^K$
$U_2 = 0.5(1.1 - 0.90)(47) = 4.77^K$
$R_{W_1} = 0.5(7.8)^2(0.0624) = 1.9^K$
$R_{W_2} = (7.8)(6.2)(0.0624) = 3.02^K$
$R_{W_3} = 0.5(6.2)^2(0.0624) = 1.2^K$
$R_S = 0.5(6.2)(0.0626) = 0.6^K$

STRUCTURE - TOTAL WEIGHT (STRUCTURE WITHOUT PUMPS) = 3255.3^K
$3255.3^K / 56'$(WIDTH) = 58.13 K/FT. WIDTH
WATER = 15.0 K/FT. WIDTH
TOTAL WEIGHT = 73.13 K/FT. WIDTH

$\bar{y}$ = RESULTANT LOCATION = $\frac{\Sigma M}{\Sigma V} = \frac{2087.39 - 1373.55}{73.13 - 47.09} = 27.41'$ FROM C

KERN 15.67' TO 31.33' FROM C

THEREFORE, RESULTANT FORCE IS LOCATED WITHIN THE KERN AND THE STRUCTURE IS SAFE AGAINST OVERTURNING.

CASE I (AFTER CONSTRUCTION) RESULTED IN A $\bar{y}$ = 24.03'
CASE III (PARTIAL POOL) RESULTED IN A $\bar{y}$ = 28.51'
CASE IV (EARTHQUAKE) RESULTED IN A $\bar{y}$ = 24.15'

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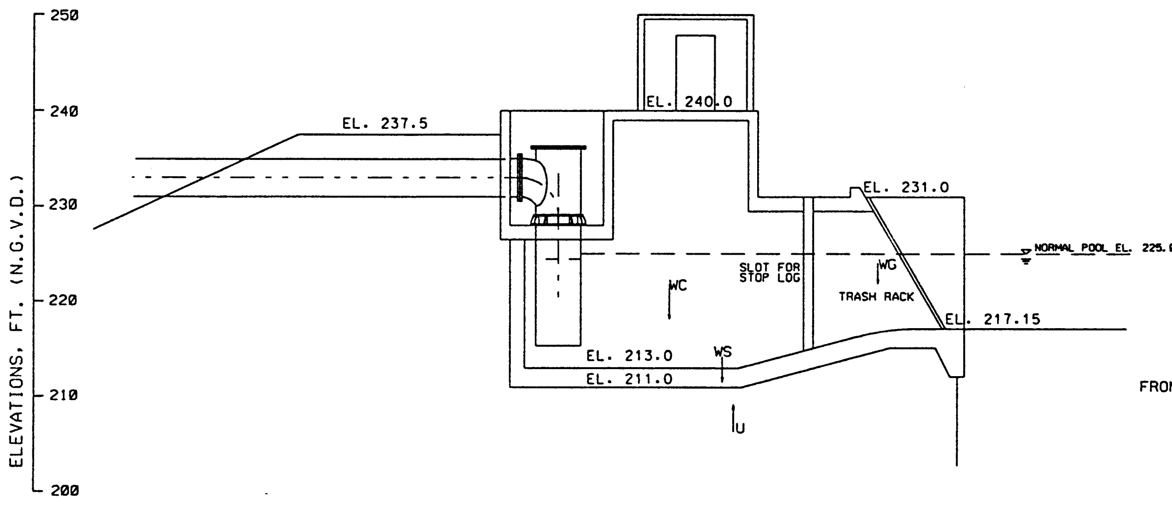
PUMPING STATION #2 OVERTURNING ANALYSIS

PLATE II-44



UPLIFT

BASE WIDTH = (47' • 56') = 2632 S.F.

PRESSURE AT EL. 211.0 = $\left(\frac{230+225}{2} - 211\right)(0.0625) = 1.03$ KSF

UPLIFT FORCE = U = 2632(1.03) ≅ 2714 KIPS

RESISTING UPLIFT

WEIGHT OF SUBSTRUCTURE (CONCRETE ONLY) = W_S = 2955 KIPS
(NO LIVE LOAD OR EQUIPMENT WEIGHT)

WEIGHT OF WATER ON PUMPING STATION
WATER IN PUMPING STATION - EL. 225 - 4 BAYS FULL
W_C = (39.5' WIDE X 28.4' LONG X 12' HEIGHT) 0.0624 = 840 KIPS

WATER IN FRONT OF STOP LOGS
W_G = (56' WIDE X 16' LONG X 8' HEIGHT) 0.0624 = 447 KIPS

FROM ETL 1110-2-307 (PAGE 1)

$$FS = \frac{W_S + W_C}{U - W_G}$$

$$FS = \frac{2955 + 840}{2714 - 447}$$

FS = 1.67 ∴ O.K.

BEARING CAPACITY

THE EXCAVATION WILL EXTEND TO EL. 201.0. TEN FEET OF COMPACTED PERVIOUS BACKFILL WILL BE PLACED UNDER THE STRUCTURE TO EL. 211.0. THE STRUCTURE WILL THEN BE CONSTRUCTED ON GRADE.

STRUCTURE = B = 47' AND W = 56'; B/6 = 47/6 = 7.83'

B' = B - 2e

CASE I (AC) - e = 0.53'* B' = 45.94' V = 58.13
CASE II (SD) - e = 3.91' B' = 39.17' V = 26.04
CASE III (PP) - e = 5.01' B' = 36.98' V = 32.07
CASE IV (EQ) - e = 0.65' B' = 45.7' V = 32.07
FOR ALL CASES - e < B/6 ∴ FULL FOOTING

* e IS FROM OVERTURNING ANALYSES

$$Q = \frac{V}{BL}\left(1 + \frac{6e}{B}\right)^{**}$$

$$Q_{max} = \frac{58.13}{47}\left[1 + \frac{6(0.53)}{47}\right] = 1.32 \text{ K/S.F. CASE I}$$

$$Q_{max} = \frac{26.04}{47}\left[1 + \frac{6(3.91)}{47}\right] = 0.83 \text{ K/S.F. CASE II}$$

$$Q_{max} = \frac{32.07}{47}\left[1 + \frac{6(5.01)}{47}\right] = 1.12 \text{ K/S.F. CASE III}$$

$$Q_{max} = \frac{32.07}{47}\left[1 + \frac{6(0.65)}{47}\right] = 0.74 \text{ K/S.F. CASE IV}$$

$$q_{ult} = cN_cS_c\cancel{d_c}^{0}i_cg_cb_c + \bar{q}N_qS_qd_qi_qg_qb_q + 0.5\gamma BN_\gamma S_\gamma d_\gamma i_\gamma g_\gamma b_\gamma R_\gamma^{**}$$

ASSUME $\phi = 30$; $N_c = 30.14$; $N_q = 18.4$; $N_\gamma = 15.67$

$\gamma' = 0.0625$ D = 6' B = 47' W = 56'

$\bar{q} = \gamma' D = 0.0625(6) = 0.38$

$S_q = 1 + 0.1 \cdot N_\phi (B'/W') = 1 + 0.1 \cdot 3\,(45.94/56) = 1.25$
$S_\gamma = S_q = 1.25$

$d_q = 1 + 0.1 \cdot \sqrt{N_\phi}\, D/B = 1 + 0.1 \cdot 1.73 \cdot (6/47) = 1.02$
$d_\gamma = d_q = 1.02$

$R_\gamma = 1 - 0.25 \cdot \text{LOG}\left(\frac{B}{6}\right)$ WHERE B > 6 FT.
$i_q = g_q = b_q = d_\gamma = i_\gamma = g_\gamma = b_\gamma = 1$

Q CASE = S CASE

CASE I q_{ult} = 0.38(18.4)(1.25)(1.02)(1) + 0.5(0.0625)(45.94)(15.67)(1.25)(1.02)(0.78)(1)
= 31.09 KSF

CASE II q_{ult} = 26.99 KSF

CASE III q_{ult} = 25.71 KSF

CASE IV q_{ult} = 30.94 KSF

CASE I F.S. = $\frac{q_{ult}}{Q_{max}} = \frac{31.09}{1.32} = 23.55$

CASE II F.S. = $\frac{26.99}{0.83} = 32.49$

CASE III F.S. = $\frac{25.71}{1.12} = 22.98$

CASE IV F.S. = $\frac{30.94}{0.74} = 41.88$ SAFE, ALL CASES > 3.0

** FROM EM 1110-1-1905 'BEARING CAPACITY OF SOILS' PAGES 4-4 THROUGH 4-14.

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PUMPING STATION #2 BEARING & UPLIFT ANALYSIS

PLATE
II-45

ASSUMPTIONS

1. WEIGHT OF PUMPING STATION (3255 KIPS) + WATER (840 KIPS) = 4095 KIPS

   WEIGHT OF SOIL EXCAVATED

   19' DEPTH X (47' X 56') AREA = 50,000 FT^3

   50,000 X 0.120 KCF = 6000 KIPS

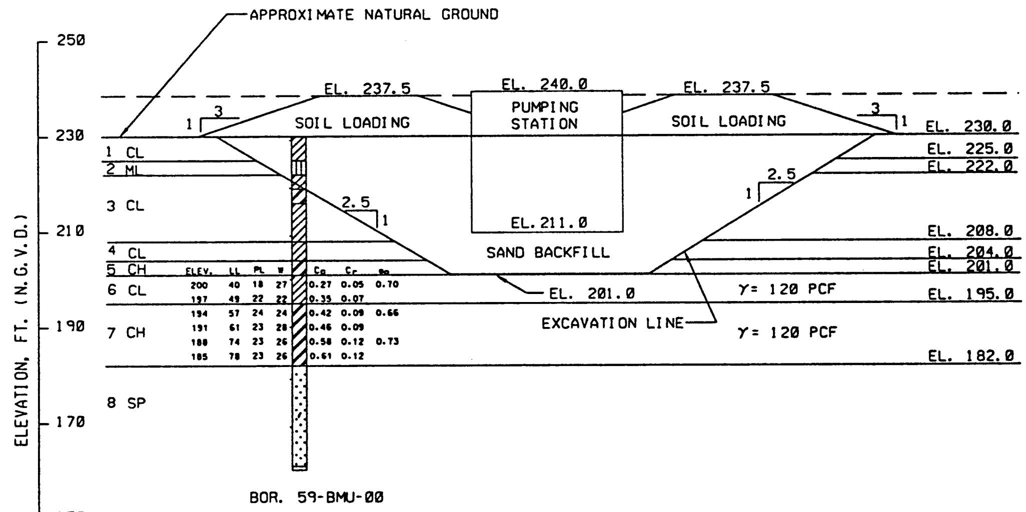
   WEIGHT IS NEGLIGIBLE FOR THE PUMPING STATION AS THE STATION WEIGHS LESS THAN THE SOIL DISPLACED. THE ONLY ADDITIONAL LOAD WOULD BE THE EARTH FILL AROUND THREE SIDES OF THE PUMPING STATION.

2. NO CONSOLIDATION TESTS WERE PERFORMED. THE VALUES SELECTED FOR THE COMPRESSION INDEX, C_c, WERE BASED ON THE EMPERICAL RELATIONSHIP C_c = 0.009 (LL-10). ALL OTHER LAYERS WERE ASSUMED INCOMPRESSIBLE.

3. THE TOTAL SETTLEMENT WAS ESTIMATED ASSUMING THE FILL LOAD MIMICED A LEVEE TYPE LOADING DISTRIBUTED OVER THE STATION AREA. THE INCREASE IN STRESS WAS ESTIMATED USING BOSSINESQ COEFFICIENTS FROM PAGE 43 OF 'TABLES OF BOSSINESQ COEFFICIENTS FOR VERTICAL STRESS INDUCTION' DATED MARCH 1969 THIS ANALYSIS IS SHOWN BELOW.

4. BASED ON THE FACT THAT THE WATER TABLE HAS BEEN DEPLETED (BY PUMPING), ATTERBERG LIMIT VALUES AND THE HIGH UNDRAINED SHEAR STRENGTH OF THE CLAY LAYER, IT APPEARS THAT THE CLAY MAY HAVE PRECONSOLIDATION STRESS (P_o') GREATER THAN THAT OF THE EXISTING VERTICAL STRESS PLUS THE SURCHARGE OF THE PROPOSED PUMPING STATION FILL (P_o + ΔP).

5. THE ANALYSIS RESULTED IN A NET SETTLEMENT OF LESS THAN 1 INCH UNDER THE STRUCTURE IF THE ASSUMPTION THAT THE ADDITIONAL LOADING WOULD FALL ALONG THE RECOMPRESSION PORTIONS OF THE CONSOLIDATION CURVE RATHER THAN THE VIRGIN COMPRESSION PORTION. THIS BEING THE CASE, IT WAS DETERMINED THAT REASONABLE VALUES OF C_r ARE APPROXIMATELY 20% OF C_c. THIS CONCLUSION IS BASED ON THE RESULTS OF NUMEROUS CONSOLIDATION TESTS ON OTHER PROJECTS WITHIN THE MEMPHIS DISTRICT.



SECTION PERPENDICULAR TO INLET & OUTLET CHANNELS THROUGH CENTER OF THE STATION

CONCLUSION

ADDITIONAL SOIL DATA WILL BE TAKEN AT THE PUMPING STATION LOCATION. CONSOLIDATION TESTS WILL BE CONDUCTED ON THE DEEPER CLAY LAYER TO DETERMINE P_o' (PREVIOUS PRESSURE OR MAXIMUM LOADING IN THE FIELD AND C_c AND C_r VALUES). A FINAL ANALYSIS WILL BE CONDUCTED BEFORE PLANS AND SPECS ARE COMPLETED.

LAYER	ELEVATION	DEPTH TO MIDPOINT	P_o (KSF)	ΔP (KSF)	P + ΔP (KSF)	PRECONSOLIDATION PRESSURE (P_o')	H (FT)	C_c	C_r	e_o	$S = \frac{C_r H}{1 + e_o} \log \frac{P_o + \Delta P}{P_o}$
6	201.0-198.0	30.5	3.66	0.863	4.523	5.225	3.0	0.27	0.05	0.70	0.009
6	198.0-195.0	33.5	4.02	0.855	4.875	7.146	3.0	0.35	0.07	0.70	0.010
7	195.0-192.0	36.5	4.38	0.847	5.227	7.755	3.0	0.42	0.09	0.66	0.012
7	192.0-189.0	39.5	4.74	0.837	5.577	5.986	3.0	0.46	0.09	0.66	0.012
7	189.0-186.0	42.5	5.10	0.828	5.928	3.683	3.0	0.58	0.12	0.73	0.013
7	186.0-182.0	47.0	5.64	0.810	6.450	3.509	4.0	0.61	0.12	0.73	0.017

0.070 FT = 0.87 INCH

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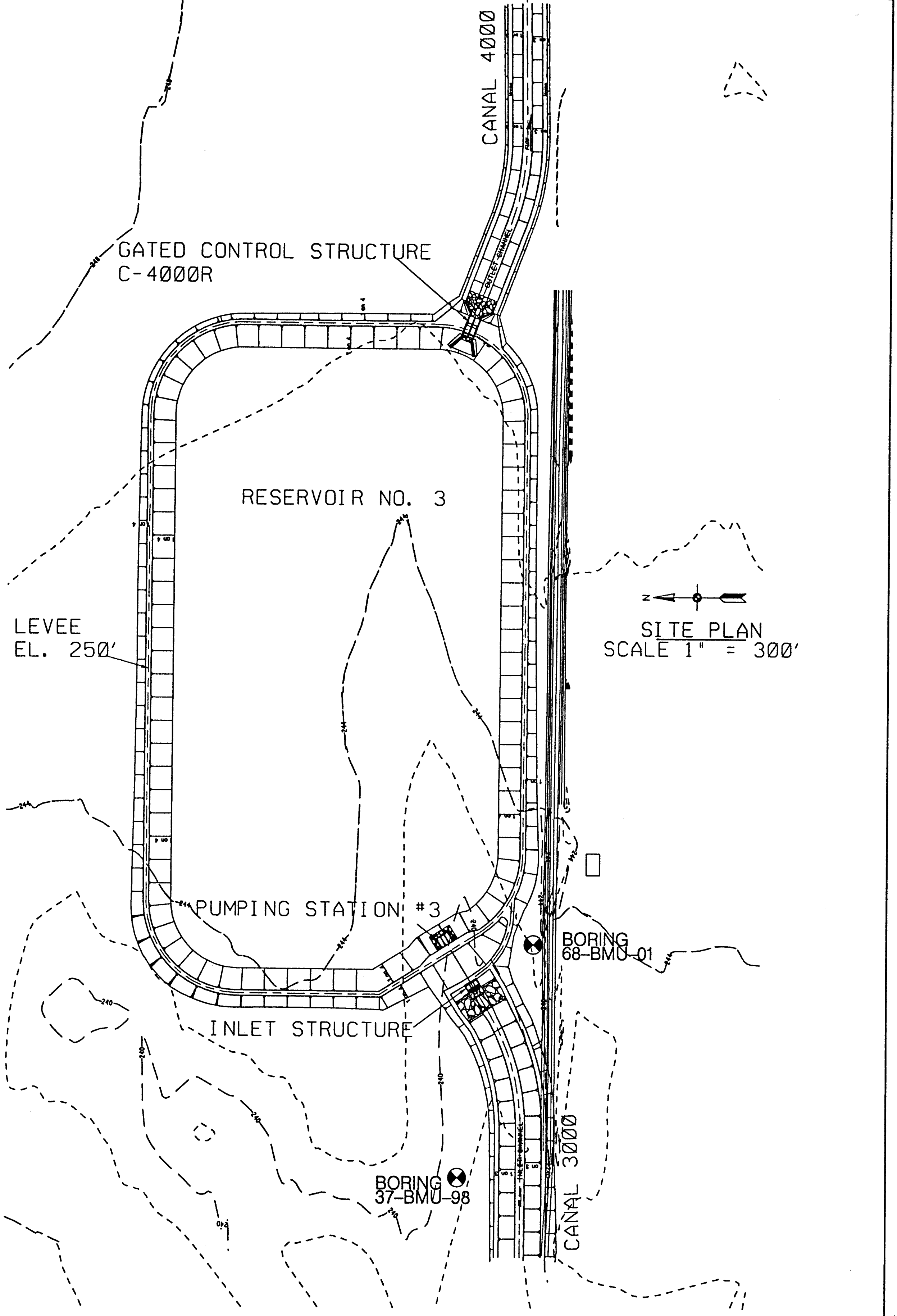
PUMPING STATION #2 SETTLEMENT ANALYSIS

PLATE II-46



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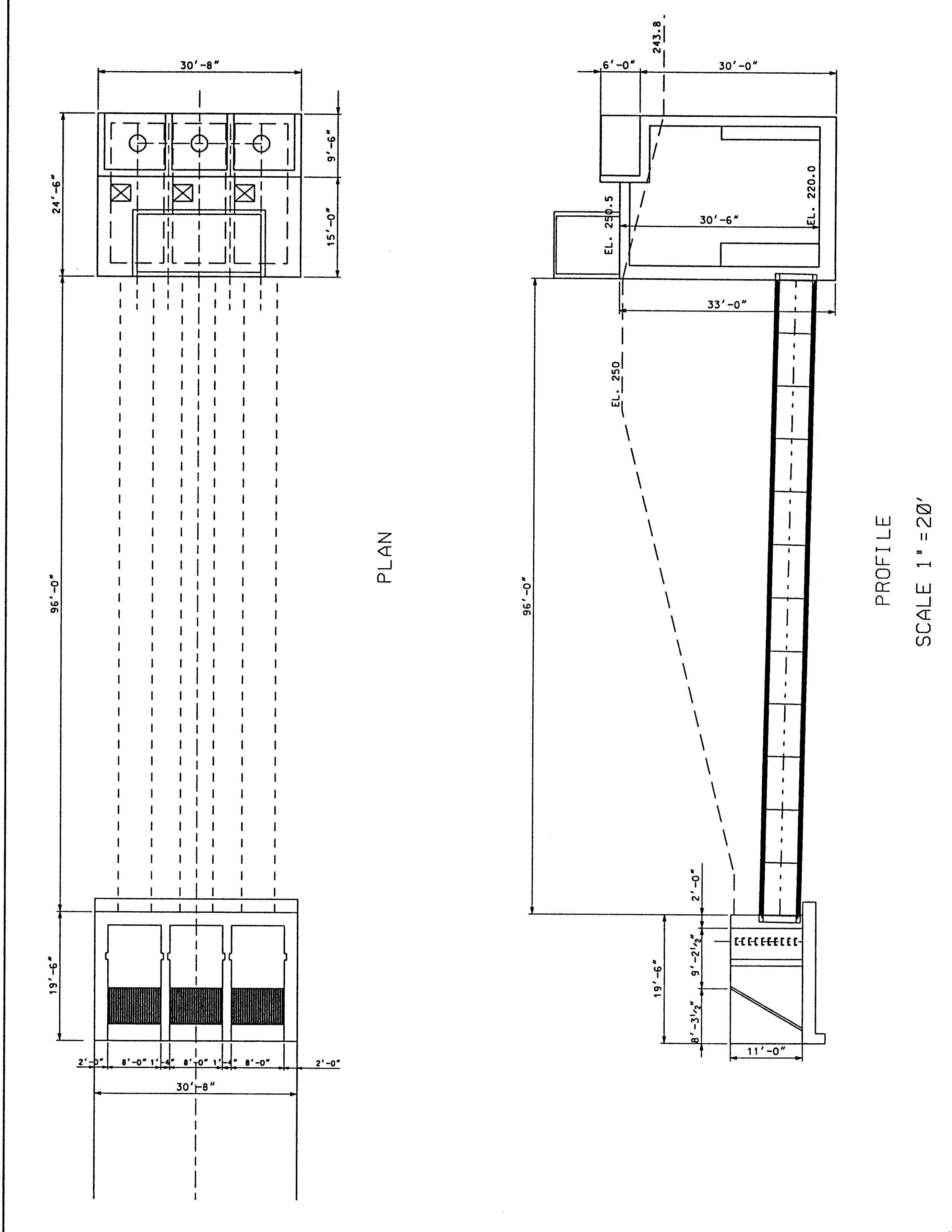
PUMPING STATION #3
SITE PLAN

PLATE

II-47



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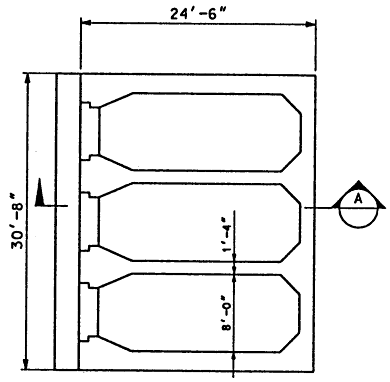
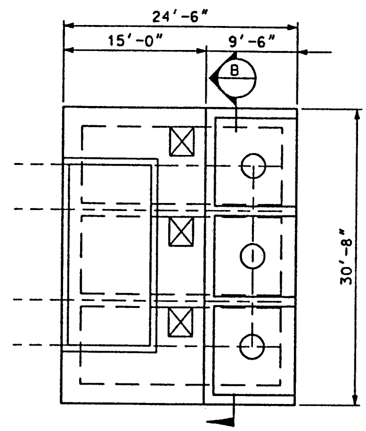
PUMPING STATION #3
PLAN & PROFILE

PLATE

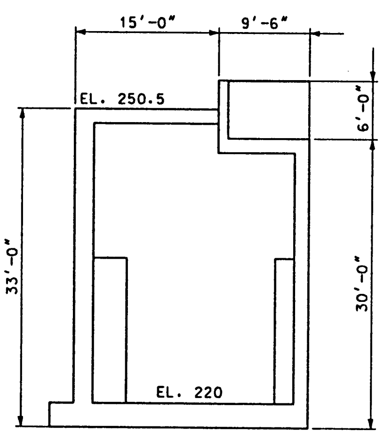
II-48



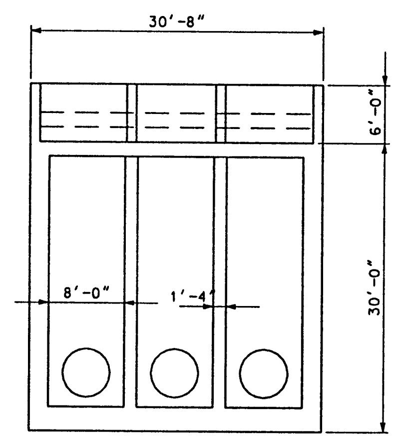
PLAN - EL. 220.0



PLAN - EL. 250.5



SECTION - A



SECTION - B

SCALE 1" =20'

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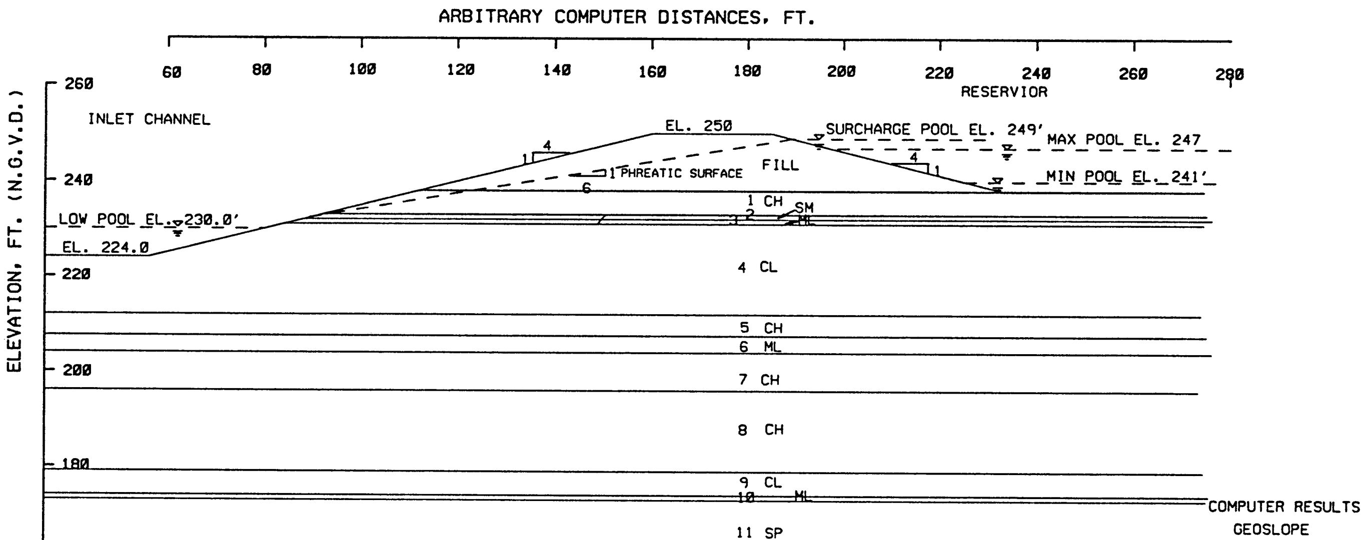
PUMPING STATION #3
PLAN & SECTION

PLATE

II-49



STRATIFICATION FROM 37-BMU-99

SOIL SHEAR STRENGTHS - BORING 37-BMU-99

SOIL NO.	SOIL TYPE	ELEVATION (N.G.V.D.)		UNIT WT. (PCF)	Q		R		S	
					φ (°)	C (PSF)	φ (°)	C (PSF)	φ (°)	C (PSF)
	FILL	250	238.0	120	0	750	13	375	26	0
1	CH	238	233	120	0	750	11.5	375	23	0
2	SM	233	232	125	30	0	30	0	30	0
3	ML	232	231	120	20	300	20	300	28	0
4	CL	231	212	120	0	1000	13.5	500	27	0
5	CH	212	207.5	120	0	500	11.5	250	23	0
6	ML	207.5	204	120	20	300	20	300	28	0
7	CH	204	196	120	0	1000	10.5	500	21	0
8	CH	196	179	120	0	1500	10.5	750	21	0
9	CL	179	174	120	0	1500	12.5	750	25	0
10	ML	174	173	120	20	300	20	300	28	0
11	SP	173	168	125	32	0	32	0	32	0

SOIL SHEAR STRENGTHS - BORING 68-BMU-01

SOIL NO.	SOIL TYPE	ELEVATION (N.G.V.D.)		UNIT WT. (PCF)	Q		R		S	
					φ (°)	C (PSF)	φ (°)	C (PSF)	φ (°)	C (PSF)
	FILL	250	238.0	120	0	750	13	375	26	0
1	CH	238	235.5	120	0	750	14	375	28	0
2	CH	235.5	231	120	0	750	10.5	375	21	0
3	CH	231	227.5	120	0	750	14	375	28	0
4	ML	227.5	226	120	20	300	20	300	28	0
5	CH	226	217.5	115	0	1500	10.5	750	21	0
6	CL	217.5	215	120	0	500	14	250	28	0
7	CL	215	206	120	0	1500	14	750	28	0
8	ML	206	203	120	20	300	20	300	28	0
9	CL	203	199	120	0	1500	12	750	24	0
10	ML	199	196	120	20	300	20	300	28	0
11	CL	196	191	120	0	1500	12	750	24	0
12	CH	191	163	120	0	1500	11	750	22	0
13	SP	163	153	125	34	0	34	0	34	0

CASE ANALYZED	SHEAR STRENGTH USED	MIN REQUIRED FS	37-BMU-99		68-BMU-01	
			ARC	WEDGE	ARC	WEDGE
CASE I-AC	Q	1.3	1.98	1.71	2.58	1.82
CASE I-AC	S	1.3	1.78	1.79	1.65	1.76
CASE II-SD (MAX POOL)	R,S	1.0	1.12	1.28	1.10	1.20
CASE III-SD (TOP OF GATES)	R,S	1.2	1.12*	1.28	1.10*	1.20
CASE IV-PP (W/STD SEEPAGE)	$\frac{R+S}{2}$ FOR R<S, S FOR R>S	1.5	1.63	1.68	1.72	1.60
CASE V-SS (W/ MAX POOL)		1.5	1.63	1.50	1.67	1.59
CASE VI-SS (W/SURCHARGE POOL)		1.4	1.55	1.42	1.59	1.50
CASE VII-EQ (CASE I)	Q	1.0	1.30	1.12	1.72	1.23
CASE VII-EQ (CASE I)	S	1.0	1.19	1.17	1.18	1.20
CASE VII-EQ (CASE IV)	$\frac{R+S}{2}$ FOR R<S	1.0	1.04	1.09	1.09	1.06
CASE VII-EQ (CASE V)	S FOR R>S	1.0	1.07	1.00	1.13	1.33

* FS SLIGHTLY BELOW ALLOWABLE. CRITICAL CASE GOVERNED BY ASSUMED S-CASE EMBANKMENT FILL SHEAR STRENGTH.

ASSUMPTIONS:

1) FILL MATERIAL USED FOR THE EMBANKMENTS WILL BE LEAN CLAY WITH Q STRENGTHS OF C=750 PSF φ=0° AND S STRENGTHS OF C=0 PSF AND φ=26°.

2) THE RESERVIOR POOL WILL OPERATE AT A MAXIMUM POOL OF ELEVATION 247 FT. AND A MINIMUM POOL OF 241 FT.

3) STABILITY ANALYSIS WAS BASED ON CRITERIA SET FORTH IN EM 1110-2-1902 "ENGINEERING AND DESIGN STABILITY OF EARTH AND ROCK-FILL DAMS". THE CASES ANALYIZED FOLLOW CRITERIA ON TABLE PAGE 25 OF ABOVE REFERENCED CRITERIA. REQUIREMENTS OF CASE III ARE MET BY CASE II.

4) CASE VII WAS ANALYIZED USING A PSEUDOSTATIC APPROACH USING A HORIZONTAL COEFFICIENT OF K =0.1. THE COEFFICIENT K_H WAS DETERMINED BY THE FOLLOWING EQUATION...

$$K_H = 0.5 * PGA * S$$

WHERE:
K_H = DIMENSIONLESS HORIZONTAL PSEUDOSTATIC COEFFICIENT
PGA = PEAK GROUND ACCELERATION (1000 YEAR RETURN PERIOD)
(5% PROBABILITY OF EXCEEDANCE IN 50 YEARS WITH 0.2 SEC SPECTRAL ACCELERATION)
S = SITE COEFFICIENT FROM STANDARD BUILDING CODE (S = 1.5)

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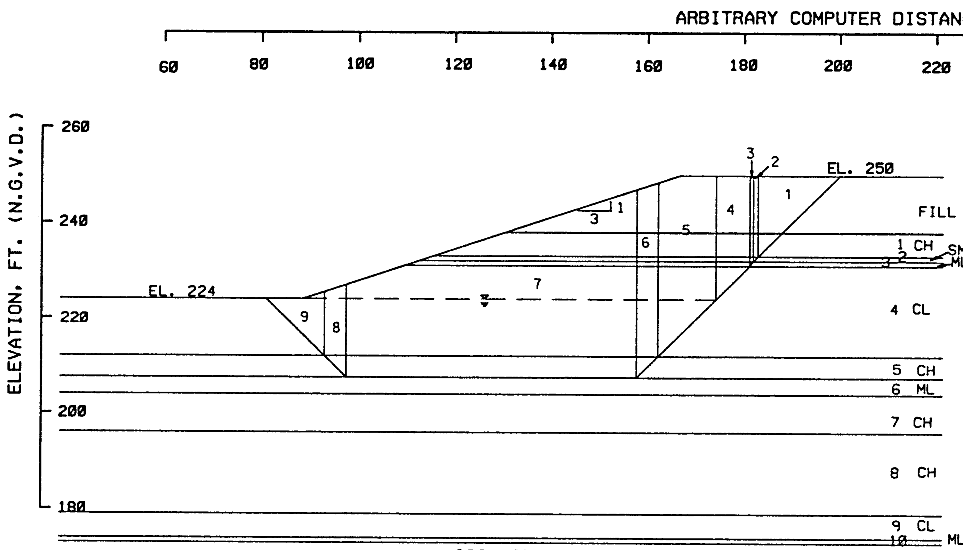
PUMPING STATION #3 RESERVOIR STABILITY

PLATE

II-50



SOIL STRATIFICATION FROM 37-BMU-99

COMPUTER RESULTS
GEOSLOPE

	ARC METHOD		WEDGE METHOD	
BORING *	A.C.	S.D.	A.C.	S.D.
37-BMU-99	1.653	NA	1.447*	NA
68-BMU-01	2.306	NA	1.790	NA

*CASE PRESENTED

THE INLET CHANNEL STABILITY ANALYSIS WAS PERFORMED USING BORING 37-BMU-99 AND 68-BMU-01. THE ANALYSIS RESULTING IN THE LOWEST FACTOR OF SAFETY IS PRESENTED ON THIS PLATE. THE ANALYSIS WAS COMPUTED USING GEOSLOPE COMPUTER SOFTWARE.

LONG TERM STABILITY

$$FS_{LT} = \frac{\tan\phi}{\tan I} = M \tan\phi$$

$$FS_{LT} = M \frac{(12 \cdot \tan 26) + (5 \cdot \tan 23) + (1 \cdot \tan 30) + (1 \cdot \tan 28) + (7 \cdot \tan 27)}{26}$$

$$FS_{LT} = M(0.5250)$$

FOR M = 3 FS = 1.58

SOIL SHEAR STRENGTHS - BORING 37-BMU-99

SOIL NO.	SOIL TYPE	ELEVATION (N.G.V.D.)		UNIT WT. (PCF)	Q		R		S	
					φ ( )°	C (PSF)	φ ( )°	C (PSF)	φ ( )°	C (PSF)
	FILL	250	238.0	120	0	750	13	375	26	0
1	CH	238	233	120	0	750	11.5	375	23	0
2	SM	233	232	125	30	0	30	0	30	0
3	ML	232	231	120	20	300	20	300	28	0
4	CL	231	212	120	0	1000	13.5	500	27	0
5	CH	212	207.5	120	0	500	11.5	250	23	0
6	ML	207.5	204	120	20	300	20	300	28	0
7	CH	204	196	120	0	1000	10.5	500	21	0
8	CH	196	179	120	0	1500	10.5	750	21	0
9	CL	179	174	120	0	1500	12.5	750	25	0
10	ML	174	173	120	20	300	20	300	28	0
11	SP	173	168	125	32	0	32	0	32	0

SOIL SHEAR STRENGTHS - BORING 68-BMU-01

SOIL NO.	SOIL TYPE	ELEVATION (N.G.V.D.)		UNIT WT. (PCF)	Q		R		S	
					φ ( )°	C (PSF)	φ ( )°	C (PSF)	φ ( )°	C (PSF)
	FILL	250	238.0	120	0	750	13	375	26	0
1	CH	238	235.5	120	0	750	14	375	28	0
2	CH	235.5	231	120	0	750	10.5	375	21	0
3	CH	231	227.5	120	0	750	14	375	28	0
4	ML	227.5	226	120	20	300	20	300	28	0
5	CH	226	217.5	115	0	1500	10.5	750	21	0
6	CL	217.5	215	120	0	500	14	250	28	0
7	CL	215	206	120	0	1500	14	750	28	0
8	ML	206	203	120	20	300	20	300	28	0
9	CL	203	199	120	0	1500	12	750	24	0
10	ML	199	196	120	20	300	20	300	28	0
11	CL	196	191	120	0	1500	12	750	24	0
12	CH	191	163	120	0	1500	11	750	22	0
13	SP	163	153	125	34	0	34	0	34	0

MANUAL COMPUTATIONS

$D_A = W \tan(45° + \phi/2)$
$D_A = 17.34 + (2.10 \cdot \tan 60) + (1.56 \cdot \tan 55) + 18.94 + 41.27 + 16.2$
$D_A = \underline{99.62}^K$

$R_A = 2[W \tan\phi + CH \tan(45° - \phi/2)]$
$R_A = 2[\ .75(17) + (2.1 \cdot \tan 30) + (.3(1) \tan 35 + 1.56 \cdot \tan 20) + 1.0(7) + 1.0(12) + 0.5(4.5)\ ]$
$R_A = \underline{71.98}^K$

$D_B = \underline{0}^K$

$R_B = W \tan\phi + CL$
$R_B = 0.5(60.34)$
$R_B = \underline{30.17}^K$

$D_P = W \tan(45° - \phi/2)$
$D_P = 4.84 + 4.54$
$D_P = \underline{9.39}^K$

$R_P = 2[W \tan\phi + CH \tan(45° + \phi/2)]$
$R_P = 2[\ 0.5(4.45) + 1.0(12)\ ]$
$R_P = \underline{28.45}^K$

$$FS = \frac{R_A + R_B + R_P}{D_A - D_P} = \frac{71.98 + 30.17 + 28.45}{99.62 - 9.39}$$

$FS = 1.447$

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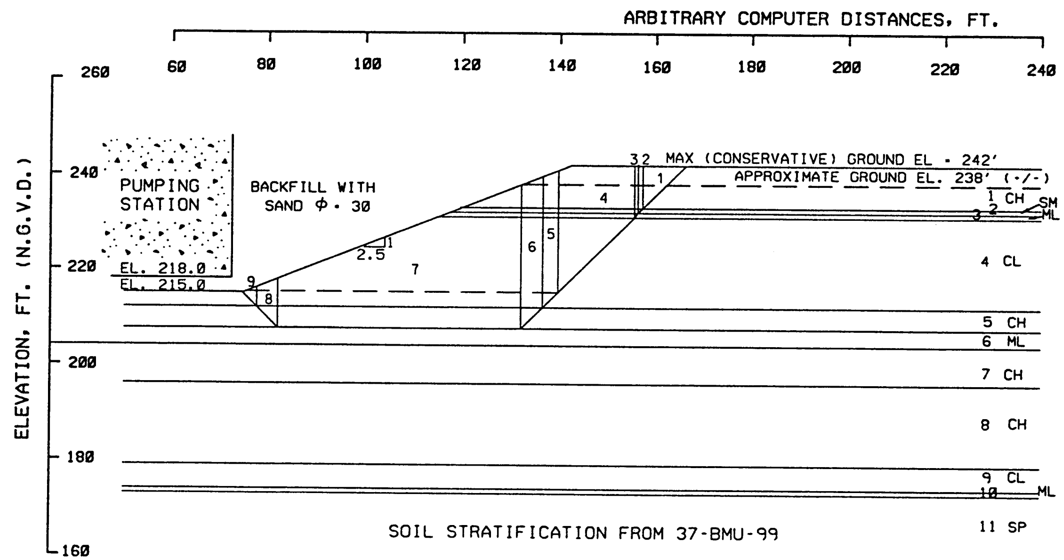
PUMPING STATION #3 INLET CHANNEL STABILITY

PLATE

II-51



COMPUTER RESULTS
GEOSLOPE

	ARC METHOD		WEDGE METHOD	
BORING •	A.C.	S.D.	A.C.	S.D.
37-BMU-99	1.643	NA	1.439*	NA
68-BMU-01	2.180	NA	1.932	NA

*CASE PRESENTED

THE PUMP #3 EXCAVATION STABILITY ANALYSIS WAS PERFORMED USING BORING 37-BMU-99 AND 68-BMU-01. THE ANALYSIS RESULTING IN THE LOWEST FACTOR OF SAFETY IS PRESENTED ON THIS PLATE. THE ANALYSIS WAS COMPUTED USING GEOSLOPE COMPUTER SOFTWARE.

•DUE TO THE TEMPORARY NATURE OF THE EXCAVATION. NO LONG TERM ANALYSIS WAS PERFORMED

SOIL SHEAR STRENGTHS - BORING 37-BMU-99

SOIL NO.	SOIL TYPE	ELEVATION (N.G.V.D.)		UNIT WT. (PCF)	Q		R		S	
					φ ()°	C (PSF)	φ ()°	C (PSF)	φ ()°	C (PSF)
	FILL	250	238.0	120	0	750	13	375	26	0
1	CH	238	233	120	0	750	11.5	375	23	0
2	SM	233	232	125	30	0	30	0	30	0
3	ML	232	231	120	20	300	20	300	28	0
4	CL	231	212	120	0	1000	13.5	500	27	0
5	CH	212	207.5	120	0	500	11.5	250	23	0
6	ML	207.5	204	120	20	300	20	300	28	0
7	CH	204	196	120	0	1000	10.5	500	21	0
8	CH	196	179	120	0	1500	10.5	750	21	0
9	CL	179	174	120	0	1500	12.5	750	25	0
10	ML	174	173	120	20	300	20	300	28	0
11	SP	173	168	125	32	0	32	0	32	0

SOIL SHEAR STRENGTHS - BORING 68-BMU-01

SOIL NO.	SOIL TYPE	ELEVATION (N.G.V.D.)		UNIT WT. (PCF)	Q		R		S	
					φ ()°	C (PSF)	φ ()°	C (PSF)	φ ()°	C (PSF)
	FILL	250	238.0	120	0	750	13	375	26	0
1	CH	238	235.5	120	0	750	14	375	28	0
2	CH	235.5	231	120	0	750	10.5	375	21	0
3	CH	231	227.5	120	0	750	14	375	28	0
4	ML	227.5	226	120	20	300	20	300	28	0
5	CH	226	217.5	115	0	1500	10.5	750	21	0
6	CL	217.5	215	120	0	500	14	250	28	0
7	CL	215	206	120	0	1500	14	750	28	0
8	ML	206	203	120	20	300	20	300	28	0
9	CL	203	199	120	0	1500	12	750	24	0
10	ML	199	196	120	20	300	20	300	28	0
11	CL	196	191	120	0	1500	12	750	24	0
12	CH	191	163	120	0	1500	11	750	22	0
13	SP	163	153	125	34	0	34	0	34	0

MANUAL COMPUTATIONS

$D_A = W \tan(45^\circ + \phi/2)$
$D_A = 4.86 + (1.13 \cdot \tan 60) + (0.91 \cdot \tan 55) + 35.80 + 9.87 + 14.14$
$D_A = \underline{67.92}^{K}$

$R_A = 2[W \tan \phi + CH \tan(45^\circ - \phi/2)]$
$R_A = 2[\ .75(9) + (1.13 \cdot \tan 30) + (.3(1) \tan 35 + 0.91 \cdot \tan 20) + 1.0(15.8) + 1.0(3.17) + 0.5(4.5)\ ]$
$R_A = \underline{58.33}^{K}$

$D_B = \underline{0}^{K}$

$R_B = W \tan \phi + CL$
$R_B = 0.5(50.02)$
$R_B = \underline{25.01}^{K}$

$D_P = W \tan(45^\circ - \phi/2)$
$D_P = 2.30 + 0.41$
$D_P = \underline{2.71}^{K}$

$R_P = 2[W \tan \phi + CH \tan(45^\circ + \phi/2)]$
$R_P = 2[\ 0.5(4.5) + 1.0(3)\ ]$

$R_P = \underline{10.5}^{K}$

$$FS = \frac{R_A + R_B + R_P}{D_A - D_P} = \frac{58.33 + 25.01 + 10.5}{67.92 - 2.71}$$

$FS = 1.439$

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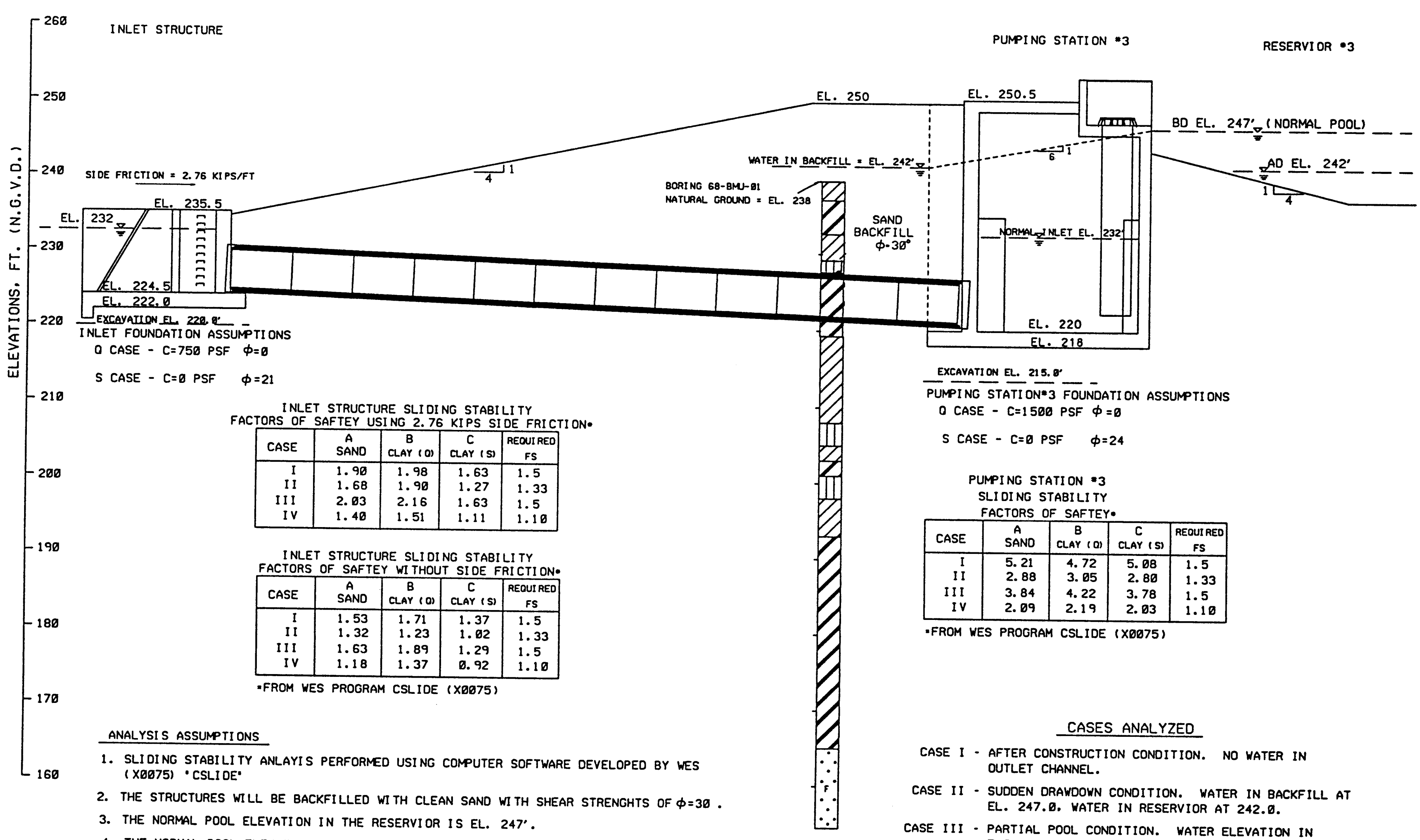
PUMPING STATION #3 EXCAVATION STABILITY

PLATE
II-52

PUMPING STATION #3 SECTION



INLET FOUNDATION ASSUMPTIONS

Q CASE - C=750 PSF $\phi=0$

S CASE - C=0 PSF $\phi=21$

PUMPING STATION#3 FOUNDATION ASSUMPTIONS

Q CASE - C=1500 PSF $\phi=0$

S CASE - C=0 PSF $\phi=24$

INLET STRUCTURE SLIDING STABILITY
FACTORS OF SAFTEY USING 2.76 KIPS SIDE FRICTION*

CASE	A SAND	B CLAY (Q)	C CLAY (S)	REQUIRED FS
I	1.90	1.98	1.63	1.5
II	1.68	1.90	1.27	1.33
III	2.03	2.16	1.63	1.5
IV	1.40	1.51	1.11	1.10

INLET STRUCTURE SLIDING STABILITY
FACTORS OF SAFTEY WITHOUT SIDE FRICTION*

CASE	A SAND	B CLAY (Q)	C CLAY (S)	REQUIRED FS
I	1.53	1.71	1.37	1.5
II	1.32	1.23	1.02	1.33
III	1.63	1.89	1.29	1.5
IV	1.18	1.37	0.92	1.10

*FROM WES PROGRAM CSLIDE (X0075)

PUMPING STATION #3
SLIDING STABILITY
FACTORS OF SAFTEY*

CASE	A SAND	B CLAY (Q)	C CLAY (S)	REQUIRED FS
I	5.21	4.72	5.08	1.5
II	2.88	3.05	2.80	1.33
III	3.84	4.22	3.78	1.5
IV	2.09	2.19	2.03	1.10

*FROM WES PROGRAM CSLIDE (X0075)

ANALYSIS ASSUMPTIONS

1. SLIDING STABILITY ANLAYIS PERFORMED USING COMPUTER SOFTWARE DEVELOPED BY WES (X0075) "CSLIDE"
2. THE STRUCTURES WILL BE BACKFILLED WITH CLEAN SAND WITH SHEAR STRENGHTS OF $\phi=30$.
3. THE NORMAL POOL ELEVATION IN THE RESERVIOR IS EL. 247'.
4. THE NORNAL POOL ELEVATION IN THE INLET CHANNEL IS EL. 232'.
5. CASES II, III & IV ASSUME THAT THE WATER ELEVATION IN THE PUMPING STATION IS EQUAL TO THE NORMAL INLET CHANNEL WATER ELEVATION (EL. 232).
6. THE CALCULATED WEIGHT OF THE PUMPING STATION ASSUMES THAT ONE OF THE THREE PUMPING STATION BAYS WILL BE DEWATERED AT ANY TIME.
7. RESERVIOR TO BE BUILT WITH CLAY LAYER TO LIMIT WATER IN BACKFILL. WATER IN BACKFILL (EL. 242) WAS ESTIMATED BY SLOPING PHREATIC SURFACE ON 1V:6H FROM NORMAL POOL EL. 247 TO BACK WALL.
8. SIDE FRICTION FORCE OF 2.76 KIPS WAS USED FOR THE INLET STRUCTURE SLIDING STABILITY ANALYSIS
9. SIDE FRICTION WAS USED IN THE INLET CALCULATIONS TO MEET THE REQUIRED FACTOR OF SAFETY. THE TABLES ABOVE INDICATES THE RESULTING FACTORS WITH AND WITHOUT THE INFLUENCE OF SIDE FRICITON. THE BASE SLAB MAY BE EXTENDED BY 5 FOOT AROUND THE STRUCTURE TO MEET THE MINIMUM FACTOR OF SAFETY REQUIREMENTS.

CASES ANALYZED

CASE I - AFTER CONSTRUCTION CONDITION. NO WATER IN OUTLET CHANNEL.

CASE II - SUDDEN DRAWDOWN CONDITION. WATER IN BACKFILL AT EL. 247.0. WATER IN RESERVIOR AT 242.0.

CASE III - PARTIAL POOL CONDITION. WATER ELEVATION IN THE BACKFILL SAME AS WATER ELEVATION IN RESERVIOR (EL. 247), WITH VARYING POOL ELEVATIONS.

CASE IV - EARTHQUAKE CONDITION. WATER IN BACKFILL AT EL. 242 AND WATER IN RESERVIOR AT EL. 247. HORIZONTAL EARTHQUAKE ACCELERATION COEFFICIENT KH-0.1.

CASE A - STRUCTURE FOUNDED ON SAND $\phi=30$.

CASE B - STRUCTURE FOUNDED ON CLAY C=1500 PSF (Q-CASE).

CASE C - STRUCTURE FOUNDED ON CLAY $\phi=24$ (S-CASE).

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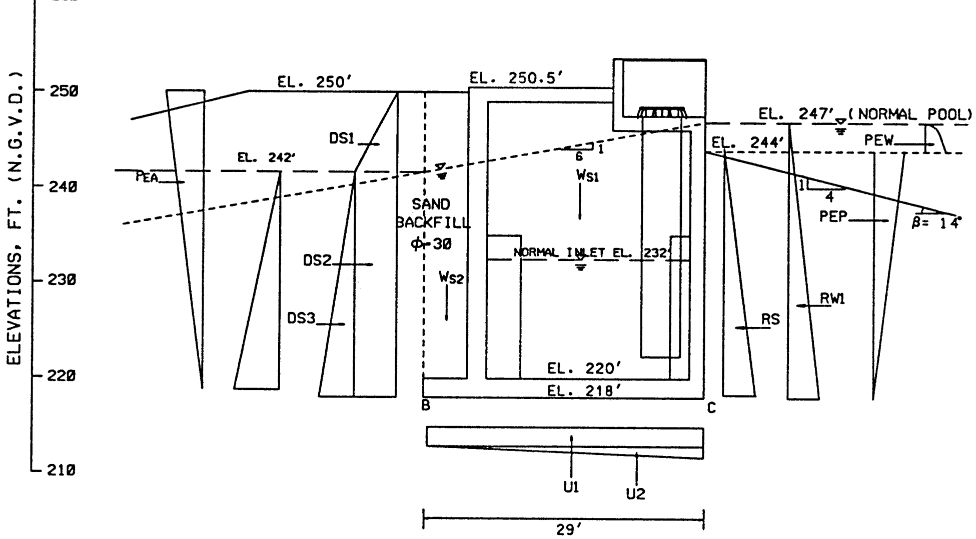
PUMPING STATION #3 SLIDING STABILITY ANALYSIS

PLATE

II-53

PUMPING STATION #3



OVERTURNING SUMMARY (CASE IV)

ITEM	FORCES (KIPS) VERTICAL ↓	VERTICAL ↑	HORIZONTAL →	HORIZONTAL ←	MOMENT ARM (FEET)	MOMENTS (KIP-FEET) OVERTURNING	RESISTING
WS	63.20				12.69		802.01
WS2	16.88				26.75		451.41
D_{S1}			2.0		26.67	53.33	
D_{S2}			12.0		12.0	144.00	
D_{S3}			9.01		8.0	72.12	
D_W			17.97		8.0	143.77	
U_1		46.18			14.5	717.76	
U_2		1.66			9.67	16.05	
R_{W1}				0.28	27.0		7.58
R_{W2}				4.87	13.0		63.27
R_{W3}				21.09	8.67		182.79
R_S				8.02	8.67		69.51
P_{EPS}			8.01		15.65	125.32	
P_{EA}			4.10		20.16	82.64	
P_{EW}			0.03		27.2	0.83	
P_{EP}			3.40		18.57	63.11	
TOTALS	80.08	47.84	56.52	34.26		1370.77	1576.57

OVERTURNING ANALYSIS

(LOADING DIAGRAM - CASE IV)
CASE IV - EARTHQUAKE CASE - WATER IN BACKFILL AT EL. 242.0, WATER IN THE RESERVIOR AT EL. 247.0

DRIVING SIDE

$K_O = (1 - \sin\phi')(1 + \sin\beta)$
$K_O = (1 - \sin 30°)(1 + \sin 0°)$
$K_O = 0.50$

RESISTING SIDE

$K_O = (1 - \sin\phi')(1 + \sin\beta)$
$K_O = (1 - \sin 30°)(1 + \sin(-14°))$
$K_O = 0.38$

CREEP PATH = 24 + 29 + 26 = 79 FT
PRESSURE AT B = [(242-218)-(24/79)(-5)](0.0624) = 1.59 KSF
PRESSURE AT C = [(242-218)-(53/79)(-5)](0.0624) = 1.71 KSF

$D_{S1} = 0.5(8.0)^2(0.125)(0.50) = 2.0^K$
$D_{S2} = 0.125(8.0)(24.0)(0.5) = 12.0^K$
$D_{S3} = 0.5(24.0)^2(0.0626)(0.5) = 9.01^K$
$D_W = 0.5(24.0)^2(0.0624) = 17.97^K$
$U_1 = 1.59(29) = 46.11^K$
$U_2 = 0.5(1.71-1.59)(29) = 1.66^K$
$R_{W1} = 0.5(3.0)^2(0.0624) = 0.28^K$
$R_{W2} = (3)(26.0)(0.0624) = 4.87^K$
$R_{W3} = 0.5(26.0)^2(0.0624) = 21.09^K$
$R_S = 0.5(26.0)^2(0.0626)(0.38) = 8.02^K$

DYNAMIC FORCES

$P_{EA} = (0.1)(2.0+12.0+9.01+17.97) = 4.1^K$
$P_{EP} = (0.1)(8.02+4.87+21.09) = 3.43^K$
$P_{EW} = (2/3)(0.051)(0.1)(247-244)^2 = 0.03^K$
$P_{EPS} = (0.1)(63.2+16.88) = 8.01^K$

STRUCTURE - STRUCTURE WEIGHT = 1699.5^K
1699.5^K/ 30.67'(WIDTH) = 55.4 K/FT. WIDTH
WATER = 237.7/30.67 = 7.75 K/FT. WIDTH (1 BAY DEWATERED)
TOTAL WEIGHT = 63.2 K/FT. WIDTH

CASE IV (EARTHQUAKE)

$\bar{y}$ = RESULTANT LOCATION = $\frac{\Sigma M}{\Sigma V} = \frac{1576.57 - 1370.77}{80.08 - 47.84}$ = 6.38' FROM C (OK LOCATED WITHIN BASE OF SLAB)

CASE I (AFTER CONSTRUCTION) RESULTED IN A $\bar{y}$ = 13.22' (OK LOCATED WITHIN KERN)
CASE II (SUDDEN DRAWDOWN) RESULTED IN A $\bar{y}$ = 10.08' (OK LOCATED WITHIN KERN)
CASE III (PARTIAL POOL) RESULTED IN A $\bar{y}$ = 12.59' (OK LOCATED WITHIN KERN)
KERN 9.66' TO 19.33' FROM C

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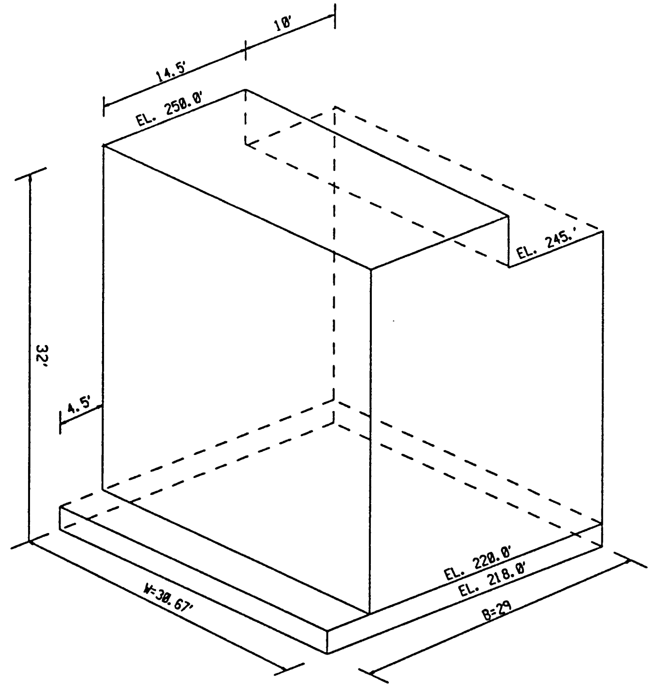
PUMPING STATION #3 OVERTURNING ANALYSIS

PLATE
II-54



UPLIFT (NORMAL OPERATION - WATER IN BACKFILL TO 242)

BASE WIDTH = (29' • 30.67') = 889.43 S.F.

PRESSURE AT EL. 218.0 = $(\frac{247+242}{2} - 218)(0.0624)$ = 1.65 KSF

UPLIFT FORCE = U = 889.43 (1.65) = 1470 KIPS

RESISTING UPLIFT

WEIGHT OF STRUCTURE = W_S = STRUCTURE WT. + SOIL WT.

STRUCTURE WT = 1699.53 KIPS

SOIL WT = [(250-242)•4.5•30.67•0.125] + [(242-218)•4.5•30.67•0.0626)] = 328.09 KIPS

W_S = 1699.53 + 328.09 = 2113.74 KIPS (NO LIVE LOAD)

WEIGHT OF WATER ON PUMPING STATION

WATER IN PUMPING STATION - EL. 232 - 3 BAYS FULL

W_C = (22.6' WIDE X 21' LONG X 12' HEIGHT) 0.0624 = 356.6 KIPS

WATER ON/OUTSIDE PUMP STATION

W_G = 0 KIPS

FROM ETL 1110-2-307 (PAGE 1)

$$FS = \frac{W_S + W_C}{U - W_G}$$

$$FS = \frac{2113.74 + 356.6}{1470 - 0}$$

FS = 1.67 ∴O.K.

BEARING CAPACITY

STRUCTURE = B = 29' AND W = 30.67'; B/6 = 29/6 = 4.83'

B' = B - 2e

CASE I (AC)	- e = 1.28'*	B' = 26.43'	V = 72.28^K
CASE II (SD)	- e = 4.42'	B' = 20.15'	V = 32.4 ^{K}
CASE III (PP)	- e = 1.91'	B' = 25.18	V = 27.60^K
CASE IV (EQ)	- e = 8.12'	B' = 12.77'	V = 32.24^K

* e IS FROM OVERTURNING ANALYSES

$$Q = \frac{V}{BL}\left(1 + \frac{6e}{B}\right)^{**}$$

$$Q_{max} = \frac{72.28}{29}\left[1 + \frac{6(1.28)}{29}\right] = 3.15 \text{ K/S.F. CASE I}$$

$$Q_{max} = \frac{32.40}{29}\left[1 + \frac{6(4.42)}{29}\right] = 2.14 \text{ K/S.F. CASE II}$$

$$Q_{max} = \frac{27.60}{29}\left[1 + \frac{6(1.91)}{29}\right] = 1.33 \text{ K/S.F. CASE III}$$

$$Q_{max} = \frac{32.24}{29}\left[1 + \frac{6(8.12)}{29}\right] = 3.37 \text{ K/S.F. CASE IV}$$

γ' = 0.0625 D = 26' B = 29' W = 30.67'

$\bar{q} = \gamma'$ D = 0.0626(26) = 1.63

Q CASE

$S_c = 1 + 0.1 \cdot N_\phi (B'/W') = 1 + 0.1 \cdot 1.0\,(26.43/30.67) = 1.09$

$S_\gamma = S_q = 1.08$

$d_c = 1 + 0.2 \cdot \sqrt{N_\phi}\ D/B = 1 + 0.2 \cdot 1.0 \cdot (26/29) = 1.18$

$d_q = 1 + 0.1 \cdot \sqrt{N_\phi}\ D/B = 1 + 0.1 \cdot 1.0 \cdot (26/29) = 1.09$

$d_\gamma = d_q = 1.09$

$R_\gamma = 1 - 0.25 \cdot \text{LOG}\left(\frac{B}{6}\right)$ WHERE B> 6 FT. R_γ = 0.83

$I_q = g_q = b_q = d_\gamma = I_\gamma = g_\gamma = b_\gamma = 1$

Q CASE

C=1500 PSF; ϕ = 0

$q_{ult} = cN_cS_cd_cI_cg_cb_c + \bar{q}N_qS_qd_qI_qg_qb_q + \cancel{0.5\gamma BN_\gamma S_\gamma d_\gamma I_\gamma g_\gamma b_\gamma R_\gamma}^{\,0}$ **

CASE I q_{ult} = (1.5)(5.14)(1.09)(1.18)(1) + (1.75)(1)(1.09)(1.09)(1) + 0 = 11.8 KSF

CASE I F.S. = $\frac{q_{ult}}{Q_{max}} = \frac{11.8}{3.15}$ = 3.74 > 3.0 OK

CASE II F.S. = $\frac{11.57}{2.14}$ = 5.41 > 2.0 OK

CASE III F.S. = $\frac{11.74}{1.33}$ = 8.86 > 3.0 OK

CASE IV F.S. = $\frac{11.24}{3.37}$ = 3.36 > 1.0 OK

S CASE

C=0 PSF; ϕ = 24

$q_{ult} = \cancel{cN_cS_cd_c}^{\,0}I_cg_cb_c + \bar{q}N_qS_qd_qI_qg_qb_q + 0.5\gamma BN_\gamma S_\gamma d_\gamma I_\gamma g_\gamma b_\gamma R_\gamma$ **

CASE I q_{ult} = 0 + (1.63)(9.6)(1.2)(1.14)(1) + (0.5)(0.0626)(26.4)(5.72)(1.2)(1.14)(1)(0.83) = 26.79 KSF

CASE I F.S. = $\frac{q_{ult}}{Q_{max}} = \frac{26.79}{3.15}$ = 8.49 > 3.0 OK

CASE II F.S. = $\frac{26.31}{2.14}$ = 13.30 > 2.0 OK

CASE III F.S. = $\frac{26.33}{1.33}$ = 19.83 > 3.0 OK

CASE IV F.S. = $\frac{21.91}{3.37}$ = 6.51 > 1.0 OK

** FROM EM 1110-1-1905 'BEARING CAPACITY OF SOILS' PAGES 4-4 THROUGH 4-14.

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PUMPING STATION #3 BEARING & UPLIFT ANALYSIS

PLATE II-55

ASSUMPTIONS

1. WEIGHT OF PUMPING STATION 1,950 KIPS

   WEIGHT OF SOIL EXCAVATED

   20' DEPTH X (30.67' X 24.5') AREA = 15,028 FT^3

   15,028 X 0.120 KCF = 1,803 KIPS

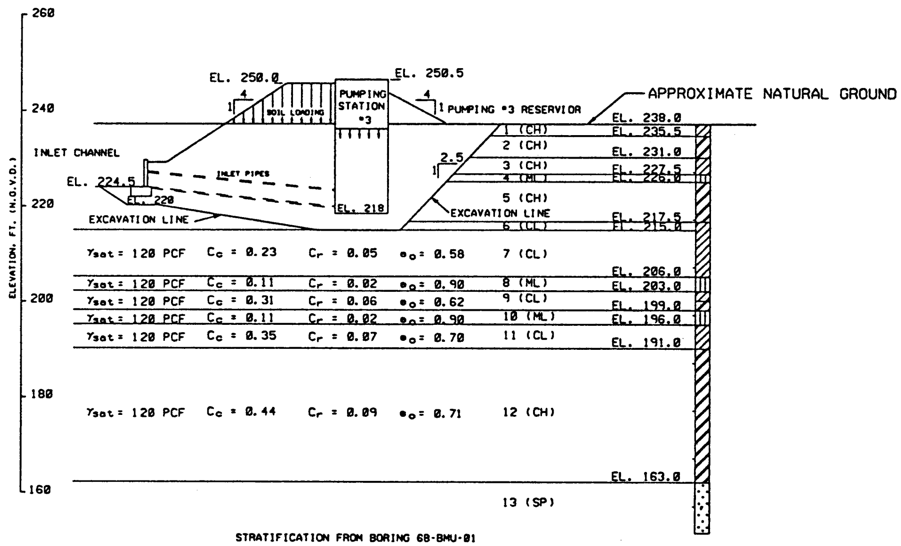
   WEIGHT IS NEGLIGIBLE FOR THE PUMPING STATION AS THE STATION WEIGHS LESS THAN THE SOIL DISPLACED. THE ONLY ADDITIONAL LOAD WOULD BE THE EARTH FILL AROUND THREE SIDES OF THE PUMPING STATION.

2. NO CONSOLIDATION TESTS WERE PERFORMED. THE VALUES SELECTED FOR THE COMPRESSION INDEX, C_c, WERE BASED ON THE EMPERICAL RELATIONSHIP C_c = 0.009 (LL-10). ALL OTHER LAYERS WERE ASSUMED INCOMPRESSIBLE.

3. THE TOTAL SETTLEMENT WAS ESTIMATED ASSUMING THE FILL LOAD MIMICED A LEVEE TYPE LOADING DISTRIBUTED OVER THE STATION AREA. THE INCREASE IN STRESS WAS ESTIMATED USING BOSSINESQ COEFFICIENTS FROM PAGE 43 OF 'TABLES OF BOSSINESQ COEFFICIENTS FOR VERTICAL STRESS INDUCTION' DATED MARCH 1969 THIS ANALYSIS IS SHOWN BELOW.

4. BASED ON THE FACT THAT THE WATER TABLE HAS BEEN DEPLETED (BY PUMPING), ATTERBERG LIMIT VALUES AND THE HIGH UNDRAINED SHEAR STRENGTH OF THE CLAY LAYER, IT APPEARS THAT THE CLAY MAY HAVE PRECONSOLIDATED STRESSES (P_o') GREATER THAN THAT OF THE EXISTING VERTICAL STRESS PLUS THE SURCHARGE OF THE PROPOSED PUMPING STATION FILL ($P_o + \Delta P$).

5. THE ANALYSIS RESULTED IN A NET SETTLEMENT OF APPROXIMATELY 2 INCHES UNDER THE STRUCTURE IF THE ASSUMPTION THAT THE ADDITIONAL LOADING WOULD FALL ALONG THE RECOMPRESSION PORTIONS OF THE CONSOLIDATION CURVE RATHER THAN THE VIRGIN COMPRESSION PORTION. THIS BEING THE CASE, IT WAS DETERMINED THAT REASONABLE VALUES OF C_R ARE APPROXIMATELY 20% OF C_c. THIS CONCLUSION IS BASED ON THE RESULTS OF NUMEROUS CONSOLIDATION TESTS ON OTHER PROJECTS WITHIN THE MEMPHIS DISTRICT.



STRATIFICATION FROM BORING 68-BMU-01

NOTE: SECTION IS PARALLEL WITH INLET AND OUTLET CHANNELS THROUGH THE CENTER OF THE STRUCTURE.

CONCLUSION

ADDITIONAL SOIL DATA WILL BE TAKEN AT THE PUMPING STATION LOCATION. CONSOLIDATION TESTS WILL BE CONDUCTED ON THE DEEPER CLAY LAYER TO DETERMINE P_o' (PREVIOUS PRESSURE OR MAXIMUM LOADING IN THE FIELD AND C_c AND C_r VALUES). A FINAL ANALYSIS WILL BE CONDUCTED BEFORE PLANS AND SPECS ARE COMPLETED.

LAYER	ELEVATION	DEPTH TO MIDPOINT	P_o (KSF)	ΔP (KSF)	$P_o + \Delta P$ (KSF)	H (FT)	C_c	C_r	e_o	$S = \frac{C_r H}{1 + e_o} \log \frac{P_o + \Delta P}{P_o}$
7	215.0-206.0	27.5	3.30	1.484	4.784	9.0	0.23	0.05	0.58	0.041
8	206.0-203.0	33.5	4.02	1.427	5.447	3.0	0.11	0.02	0.90	0.005
9	203.0-199.0	37.0	4.44	1.369	5.809	4.0	0.31	0.06	0.62	0.018
10	199.0-196.0	40.5	4.86	1.340	6.200	3.0	0.11	0.02	0.90	0.004
11	196.0-191.0	44.5	5.34	1.281	6.621	5.0	0.35	0.07	0.70	0.019
12	191.0-163.0	61.0	7.32	1.120	8.440	28.0	0.44	0.09	0.72	0.089
										0.18 FT = 2.1 INCH

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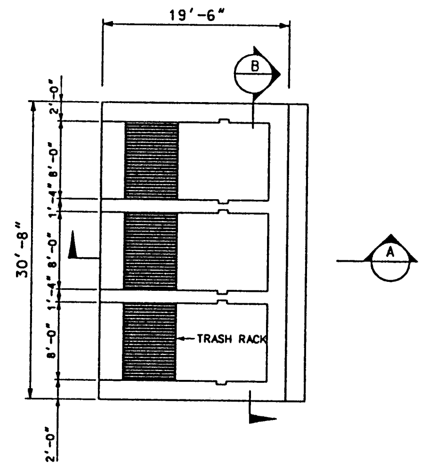
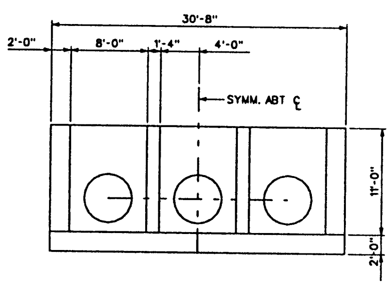
PUMPING STATION #3 SETTLEMENT ANALYSIS

PLATE

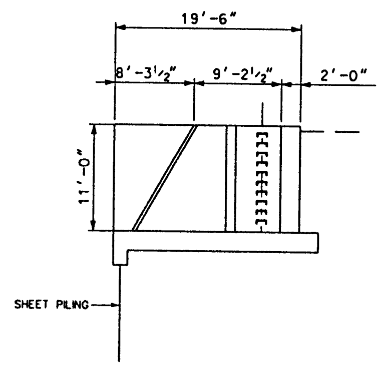
II-56



PLAN



SECTION - B



SECTION - A

SCALE 1" = 20'

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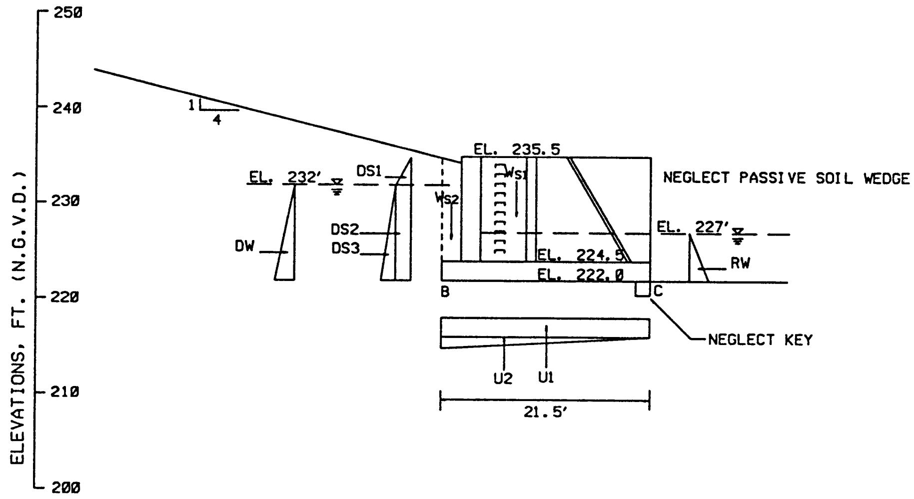
PUMPING STATION #3 INLET PLAN & SECTION

PLATE

II-57



OVERTURNING SUMMARY (CASE II)

ITEM	FORCES (KIPS)				MOMENT ARM	MOMENTS (KIP-FEET)	
	VERTICAL		HORIZONTAL			OVERTURNING	RESISTING
	↓	↑	→	←	(FEET)		
WS	18.93				10.83		205.01
WS2	2.69				20.5		55.09
D_{s1}			0.48		11.0	5.23	
D_{s2}			2.72		5.0	13.58	
D_{s3}			1.94		3.3	6.48	
D_w			3.12		3.3	10.4	
U_1		6.71			10.75	72.11	
U_2		2.29			14.33	32.81	
R_w				0.78	1.67		1.3
TOTALS	21.62	9.0	8.26	0.78		140.61	261.40

OVERTURNING ANALYSIS

(LOADING DIAGRAM - CASE II-SD)
CASE IV - SUDDEN DRAWDOWN CASE - WATER IN BACKFILL AT EL. 232.0; WATER IN THE INLET CHANNEL AT EL. 227.0

DRIVING SIDE

$K_o = (1 - \sin \phi')(1 + \sin \beta)$
$K_o = (1 - \sin 30°)(1 + \sin 14°)$
$K_o = 0.62$

RESISTING SIDE

$K_o = (1 - \sin \phi')(1 + \sin \beta)$
$K_o = (1 - \sin 30°)(1 + \sin 0°)$
$K_o = 0.5$

CREEP PATH = 10 + 21.5 + 0 = 31.5 FT
PRESSURE AT B = [(232-222)-(10/31.5)5](0.0624) = 0.52 KSF
PRESSURE AT C = [(232-222)-(31.5/31.5)5](0.0624) = 0.31 KSF

$D_{s1} = 0.5(3.5)^2(0.125)(0.62) = 0.48^K$
$D_{s2} = 0.125(3.5)(10.0)(0.62) = 2.72^K$
$D_{s3} = 0.5(10.0)^2(0.0626)(0.62) = 1.94^K$
$D_w = 0.5(10.0)^2(0.0624) = 3.12^K$
$U_1 = 0.31(21.5) = 6.71^K$
$U_2 = 0.5(0.52-0.31)(21.5) = 2.29^K$
$R_w = 0.5(5.0)^2(0.0624) = 0.78^K$

STRUCTURE - STRUCTURE WEIGHT = 580.4 K
$580.4^K / 30.67'$(WIDTH) = 18.93 K/FT. WIDTH

CASE I (AFTER CONSTRUCTION) RESULTED IN A $\bar{y}$ = 10.61' (OK LOCATED WITHIN KERN)

CASE II (SUDDEN DRAWDOWN)

$\bar{y}$ = RESULTANT LOCATION = $\frac{\Sigma M}{\Sigma V} = \frac{261.4 - 140.6}{21.62 - 9.0}$ = 9.57' FROM C (OK LOCATED WITHIN KERN)

CASE III (PARTIAL POOL) RESULTED IN A $\bar{y}$ = 11.05' (OK LOCATED WITHIN KERN)
CASE IV (EARTHQUAKE) RESULTED IN A $\bar{y}$ = 8.83' (OK LOCATED WITHIN KERN)

KERN 7.16' TO 14.33' FROM C

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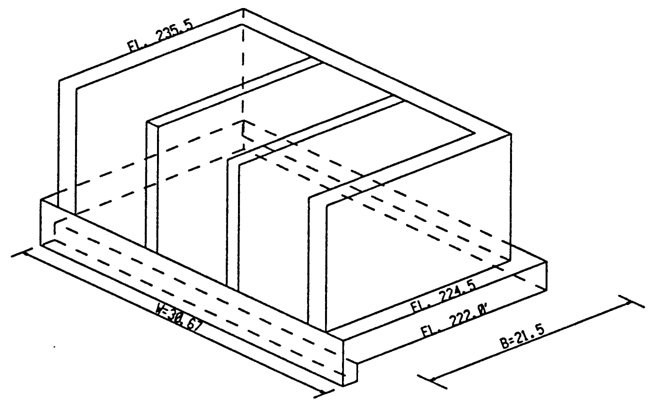
PUMPING STATION #3 - INLET OVERTURNING ANALYSIS

PLATE

II-58



BEARING CAPACITY

STRUCTURE = B = 21.5' AND W = 30.67'; B/6 = 21.5/6 = 3.58'

B' = B - 2e

CASE I (AC)	- e = 0.14'*	B' = 21.23'	V = 21.62 K
CASE II (SD)	- e = 1.18'	B' = 19.14'	V = 12.62 K
CASE III (PP)	- e = 0.30'	B' = 20.91'	V = 8.20 K
CASE IV (EQ)	- e = 1.92'	B' = 17.66'	V = 8.20 K

* e IS FROM OVERTURNING ANALYSES

$$Q = \frac{V}{BL}\left(1 + \frac{6e}{B}\right)^{**}$$

$$Q_{max} = \frac{21.62}{21.5}\left[1 + \frac{6(0.14)}{21.5}\right] = 1.04 \text{ K/S.F.} \quad \text{CASE I}$$

$$Q_{max} = \frac{12.62}{21.5}\left[1 + \frac{6(1.18)}{21.5}\right] = 0.78 \text{ K/S.F.} \quad \text{CASE II}$$

$$Q_{max} = \frac{8.20}{21.5}\left[1 + \frac{6(0.30)}{21.5}\right] = 0.41 \text{ K/S.F.} \quad \text{CASE III}$$

$$Q_{max} = \frac{8.20}{21.5}\left[1 + \frac{6(1.92)}{21.5}\right] = 0.59 \text{ K/S.F.} \quad \text{CASE IV}$$

$$q_{ult} = cN_c S_c d_c I_c g_c b_c + \bar{q}N_q S_q d_q I_q g_q b_q + 0.5\gamma B N_\gamma S_\gamma d_\gamma I_\gamma g_\gamma b_\gamma R_\gamma^{**}$$

Q CASE - C= 750 PSF; ϕ = 0; N_c=5.14; N_q=1.0; N_γ=0.0

S CASE - C=0 PSF; ϕ = 21; N_c=15.85; N_q=7.11; N_γ=3.47

γ' = 0.0625 D = 2' B = 21.5' W = 30.67'

$\bar{q} = \gamma'$ D = 0.0626(2) = 0.13

CASE I (Q)

$S_c = 1 + 0.1 \cdot N_\phi (B'/W') = 1 + 0.1 \cdot 1.0\ (21.23/30.67) = 1.07$

$S_\gamma = S_q = 1.07$

$d_c = 1 + 0.2 \cdot \sqrt{N_\phi}\ D/B = 1 + 0.2 \cdot 1.0 \cdot (2/21.5) = 1.02$

$d_q = 1 + 0.1 \cdot \sqrt{N_\phi}\ D/B = 1 + 0.1 \cdot 1.0 \cdot (2/21.5) = 1.01$

$d_\gamma = d_q = 1.01$

$R_\gamma = 1 - 0.25 \cdot \text{LOG}\left(\frac{B}{6}\right)$ WHERE B> 6 FT. R_γ= 0.86

$I_q = g_q = b_q = d_\gamma = I_\gamma = g_\gamma = b_\gamma = 1$

Q CASE

C=750 PSF; ϕ= 0

$$q_{ult} = cN_c S_c d_c I_c g_c b_c + \bar{q}N_q S_q d_q I_q g_q b_q + \cancel{0.5\gamma B N_\gamma} S_\gamma d_\gamma I_\gamma g_\gamma b_\gamma R_\gamma^{**} \quad 0$$

CASE I q_{ult} = (0.75)(5.14)(1.07)(1.02)(1) + (0.13)(1)(1.07)(1.09)(1.01) + 0
= 4.33 KSF

CASE I F.S. = $\frac{q_{ult}}{Q_{max}} = \frac{4.33}{1.04}$ = 4.15 > 3.0 OK

CASE II F.S. = $\frac{4.31}{0.78}$ = 5.52 > 2.0 OK

CASE III F.S. = $\frac{4.33}{0.41}$ = 10.48 > 3.0 OK

CASE IV F.S. = $\frac{4.29}{0.59}$ = 7.32 > 1.0 OK

S CASE

C=0 PSF; ϕ= 21

$$q_{ult} = \cancel{cN_c S_c d_c} I_c g_c b_c + \bar{q}N_q S_q d_q I_q g_q b_q + 0.5\gamma B N_\gamma S_\gamma d_\gamma I_\gamma g_\gamma b_\gamma R_\gamma^{**} \quad 0$$

CASE I q_{ult} = 0 + (0.13)(7.11)(1.15)(1.01)(1) + (0.5)(0.0626)(21.5)(3.47)(1.15)(1.01)(1)(0.86)
= 3.16 KSF

CASE I F.S. = $\frac{q_{ult}}{Q_{max}} = \frac{3.16}{1.04}$ = 3.03 > 3.0 OK

CASE II F.S. = $\frac{2.91}{0.78}$ = 3.73 > 2.0 OK

CASE III F.S. = $\frac{3.12}{0.41}$ = 7.55 > 3.0 OK

CASE IV F.S. = $\frac{2.74}{0.59}$ = 4.68 > 1.0 OK

** FROM EM 1110-1-1905 'BEARING CAPACITY OF SOILS' PAGES 4-4 THROUGH 4-14.

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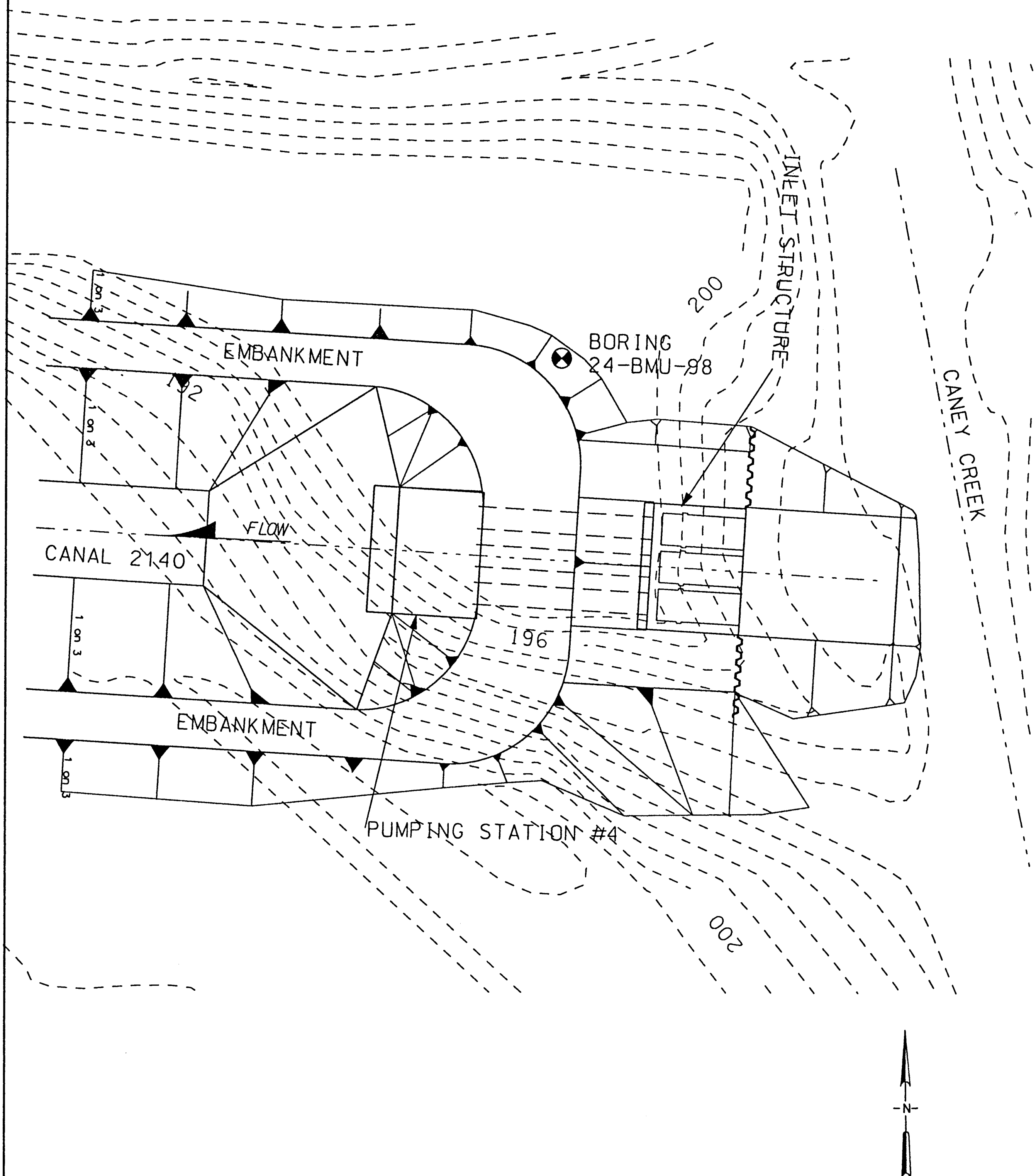
PUMPING STATION #3 INLET BEARING ANALYSIS

PLATE

II-59



SCALE 1"=30'

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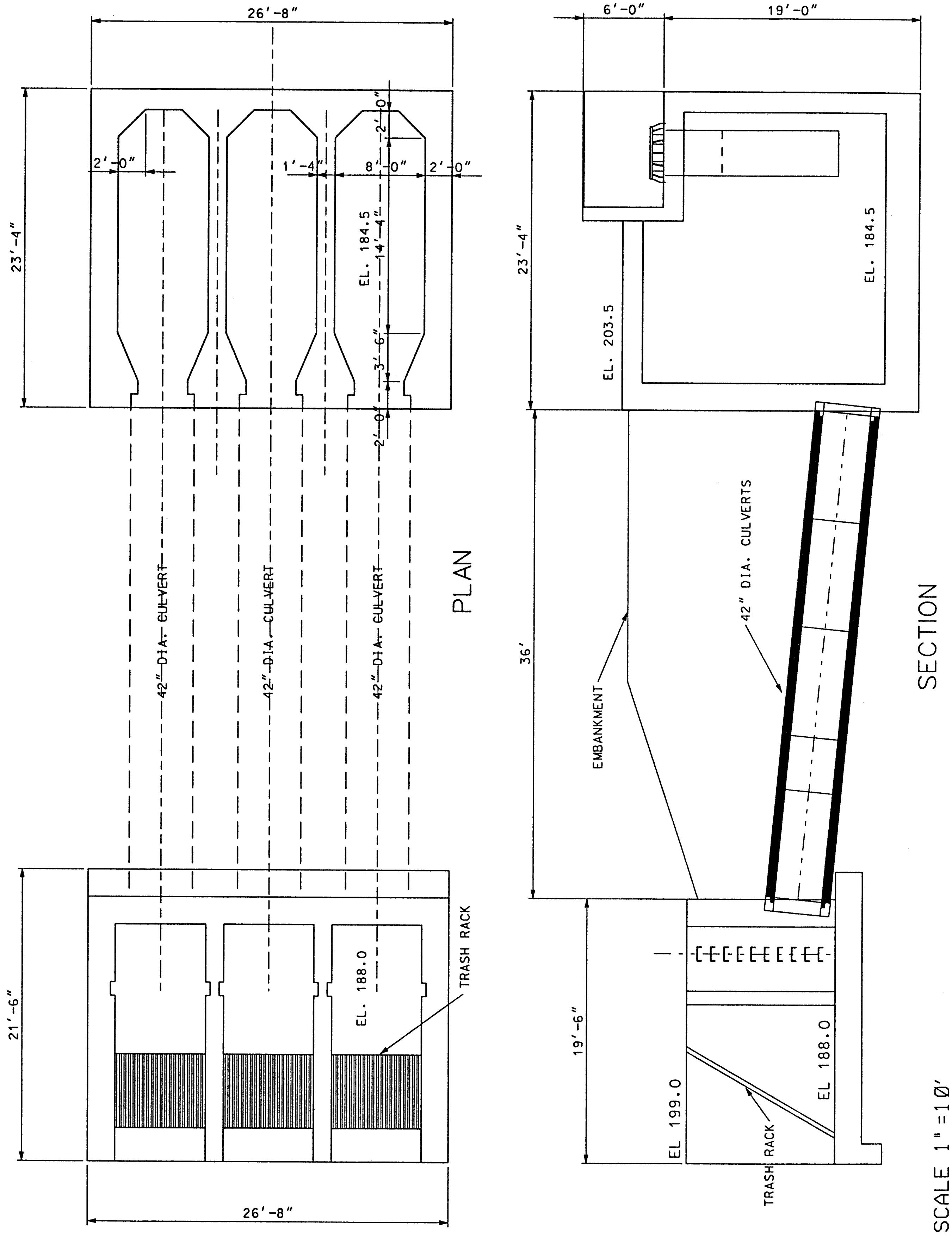
PUMPING STATION #4
SITE PLAN

PLATE

II-60



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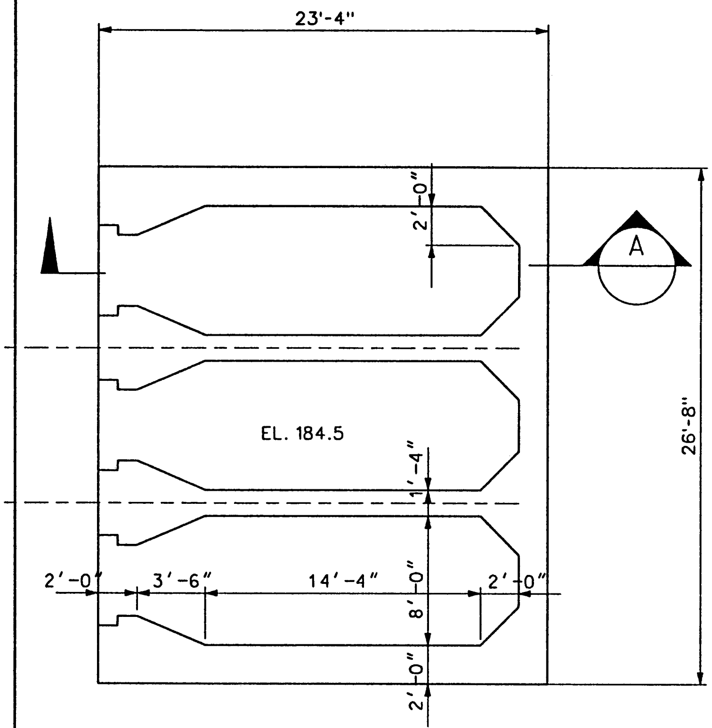
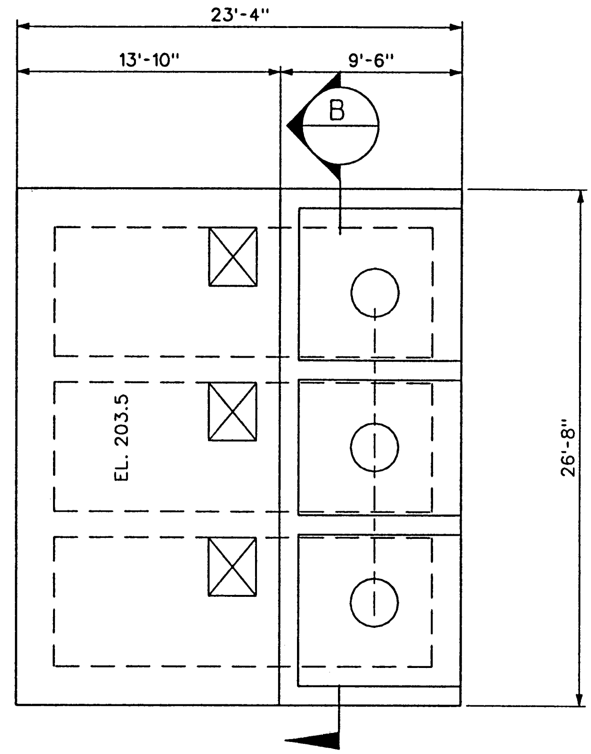
PUMPING STATION #4
PLAN & PROFILE

PLATE

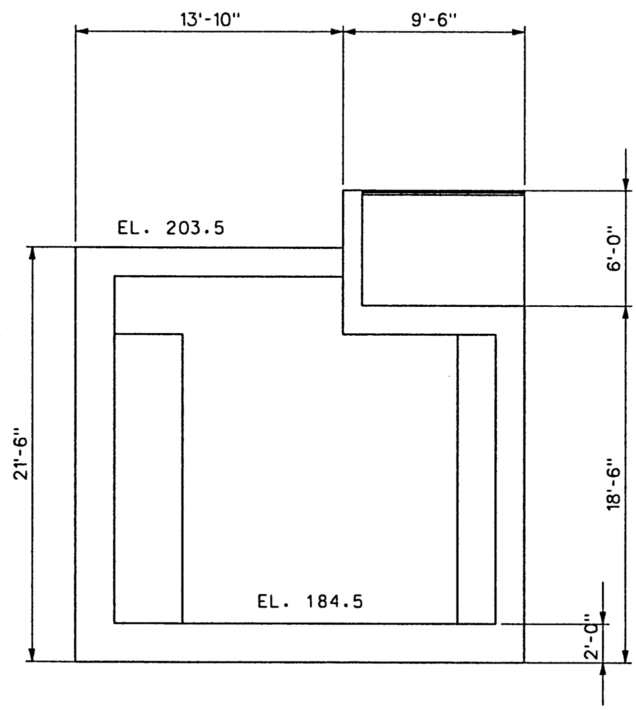
II-61



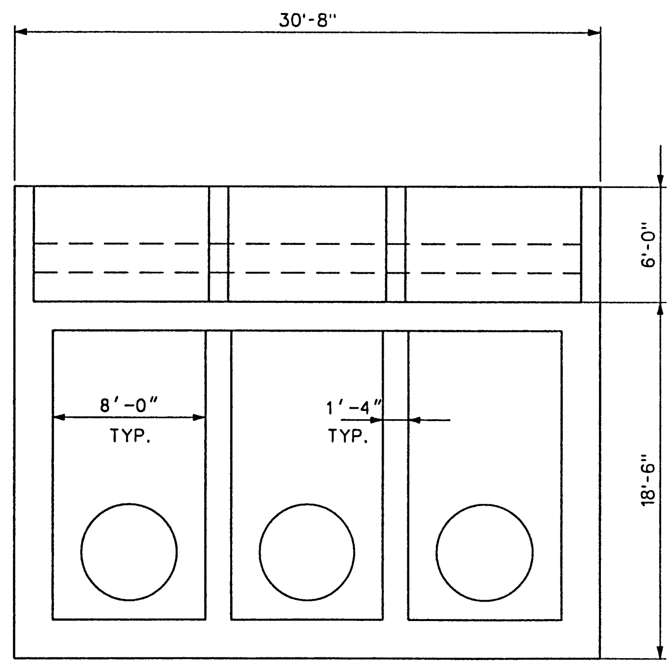
PLAN - EL. 184.5



PLAN - EL. 203.5



SECTION - A



SECTION - B

SCALE 1" = 10'

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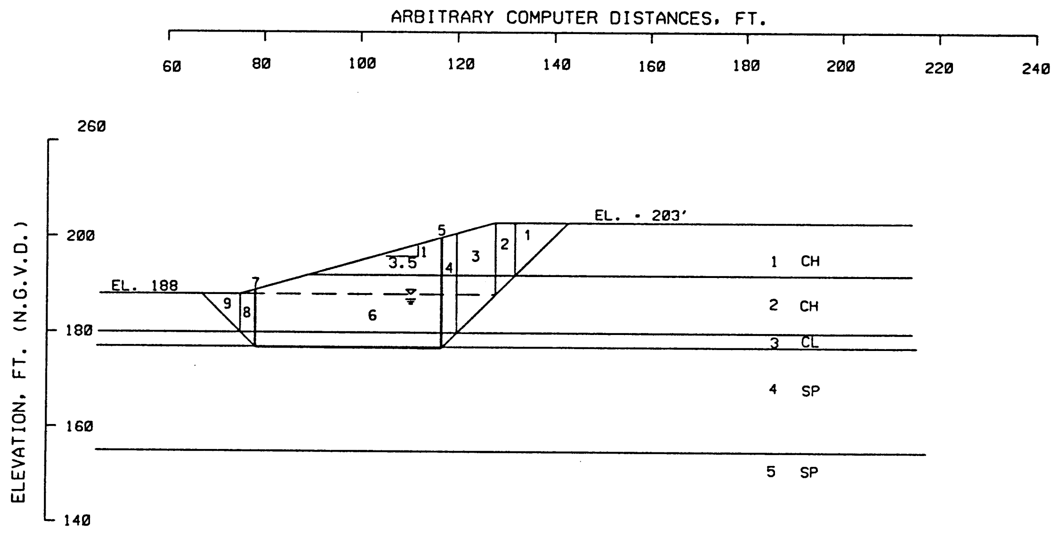
PUMPING STATION #4
PLAN & SECTION

PLATE
II-62



SOIL STRATIFICATION FROM 24-BMU-98

COMPUTER RESULTS
GEOSLOPE

	ARC METHOD		WEDGE METHOD	
BORING •	A.C.	S.D.	A.C.	S.D.
24-BMU-98	3.792	NA	3.547*	NA

THE INLET CHANNEL STABILITY ANALYSIS WAS PERFORMED USING BORING 24-BMU-98. THE ANALYSIS RESULTING IN THE LOWEST FACTOR OF SAFETY IS PRESENTED ON THIS PLATE. THE ANALYSIS WAS COMPUTED USING GEOSLOPE COMPUTER SOFTWARE.

LONG TERM STABILITY

$$FS_{LT} = \frac{\text{TAN } \phi}{\text{TAN } I} = M \text{ TAN } \phi$$

$$FS_{LT} = M \; \frac{(11 \cdot \text{TAN } 19) - (4 \cdot \text{TAN } 21.5)}{15}$$

$$FS_{LT} = M(0.3575)$$

FOR M = 3.5 FS = 1.25 = GOVERNS

SOIL SHEAR STRENGTHS - BORING 24-BMU-98

SOIL NO.	SOIL TYPE	ELEVATION (N.G.V.D.)		UNIT WT. (PCF)	Q		R		S	
					φ (°)	C (PSF)	φ (°)	C (PSF)	φ (°)	C (PSF)
1	CH	203	192	120	0	750	9.5	375	19	0
2	CH	192	180	120	0	1000	10.75	500	21.5	0
3	CL	180	177	120	0	1500	12	750	24	0
4	SP	177	155	125	34	0	34	0	34	0
5	SP	155	150	125	31	0	31	0	31	0

MANUAL COMPUTATIONS

DA = W TAN (45° + φ/2)
DA = (7.26•TAN 45) +(6.24•TAN 45) +(15.19•TAN 45) +(6.08•TAN 45) +(0.64•TAN 62)
DA = 35.97 KIPS

RA = 2[W TANφ + CH TAN (45° - φ/2)]
RA = 2[(0.75•11•TAN 45) +(1•4•TAN 45) +(1•8•TAN 45) +(1.5•3•TAN 45) +(0.64•TAN 34)
RA = 50.36 KIPS

DB = 0*

RB = W TANφ + CL
RB = (54.38•TAN 34)
RB = 36.68 KIPS

DP = W TAN (45° - φ/2)
DP = (0.18•TAN 28) + (2.02•TAN 45) + (1.96•TAN 45)
DP = 4.07 KIPS

RP = 2[W TANφ + CH TAN (45° + φ/2)]
RP = 2[(0.18•TAN 34) + (1.5•3•TAN 45) + (1•8•TAN 45)
RP = 25.24 KIPS

$$FS = \frac{R_A + R_B + R_P}{D_A - D_P} = \frac{50.36 + 36.68 + 25.24}{35.97 - 4.07}$$

FS = 3.52

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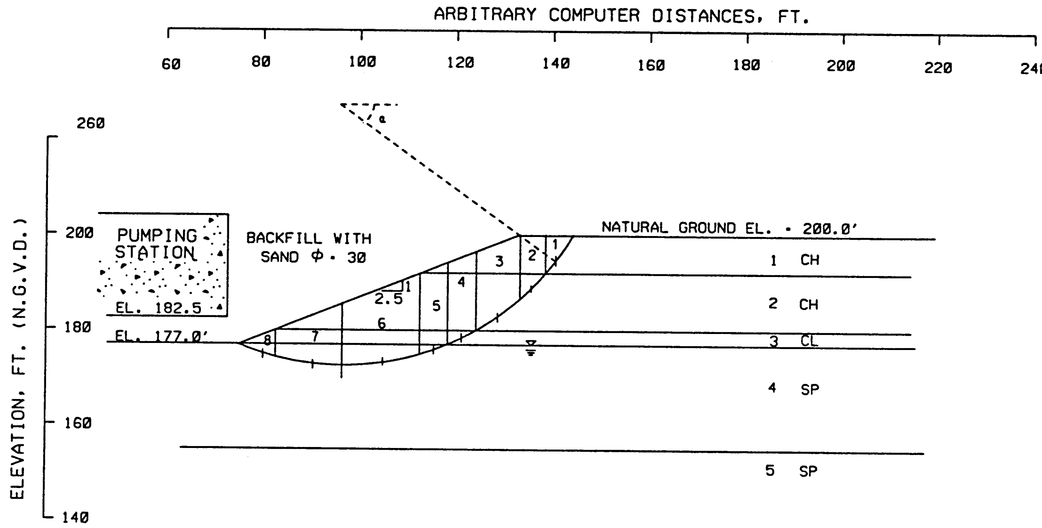
PUMPING STATION #4
INLET CHANNEL STABILITY

PLATE
II-63



SOIL STRATIFICATION FROM 24-BMU-98

COMPUTER RESULTS
GEOSLOPE

	ARC METHOD		WEDGE METHOD	
BORING #	A.C.	S.D.	A.C.	S.D.
24-BMU-98	2.539*	NA	2.830	NA

*CASE PRESENTED

THE PUMP #4 EXCAVATION STABILITY ANALYSIS WAS PERFORMED USING BORING 24-BMU-98. THE ANALYSIS RESULTING IN THE LOWEST FACTOR OF SAFETY IS PRESENTED ON THIS PLATE. THE ANALYSIS WAS COMPUTED USING GEOSLOPE COMPUTER SOFTWARE.

•DUE TO THE TEMPORARY NATURE OF THE EXCAVATION, NO LONG TERM ANALYSIS WAS PERFORMED

SOIL SHEAR STRENGTHS - BORING 24-BMU-98

SOIL NO.	SOIL TYPE	ELEVATION (N.G.V.D.)		UNIT WT. (PCF)	Q		R		S	
					Φ (°)	C (PSF)	Φ (°)	C (PSF)	Φ (°)	C (PSF)
1	CH	200	192	120	0	750	9.5	375	19	0
2	CH	192	180	120	0	1000	10.75	500	21.5	0
3	CL	180	177	120	0	1500	12	750	24	0
4	SP	177	155	125	34	0	34	0	34	0
5	SP	155	150	125	31	0	31	0	31	0

MANUAL CALCULATIONS

SLICE	SOIL COHESION (C) PSF	SOIL FRICTION (Φ)°	DRIVE WT. KIPS	RESIST WT. KIPS	ALPHA (α) DEGRESS	SIN ALPHA	COS ALPHA	NORMAL FORCE (ΔN) KIPS	ARC LENGTH (L) FEET	(C·L) + (ΔN·TAN Φ) KIPS	TAN FORCE KIPS
1	750		2.89	2.89	36.91	0.60	0.80	1.73	9.80	7.35	2.31
2	1000		6.90	6.90	45.06	0.71	0.71	4.89	7.70	7.70	4.88
3	1000		16.66	16.66	54.42	0.81	0.58	13.55	11.30	11.30	9.69
4	1500		11.96	11.96	63.41	0.89	0.45	10.69	6.70	10.05	5.35
5		34	11.49	11.49	70.04	0.94	0.34	10.80	6.30	7.29	3.92
6		34	25.76	25.76	81.27	0.99	0.15	25.46	16.20	17.17	3.91
7		34	12.53	12.53	96.25	0.99	-0.11	12.45	13.90	8.40	-1.36
8		34	2.01	2.01	107.08	0.96	-0.29	1.93	7.92	1.30	-0.59
									SUM	70.57	28.10

$$F.S. = \frac{NORM \cdot TAN\ (PHI)}{TAN\ F}$$

$$F.S. = \frac{70.57}{28.11} = 2.51$$

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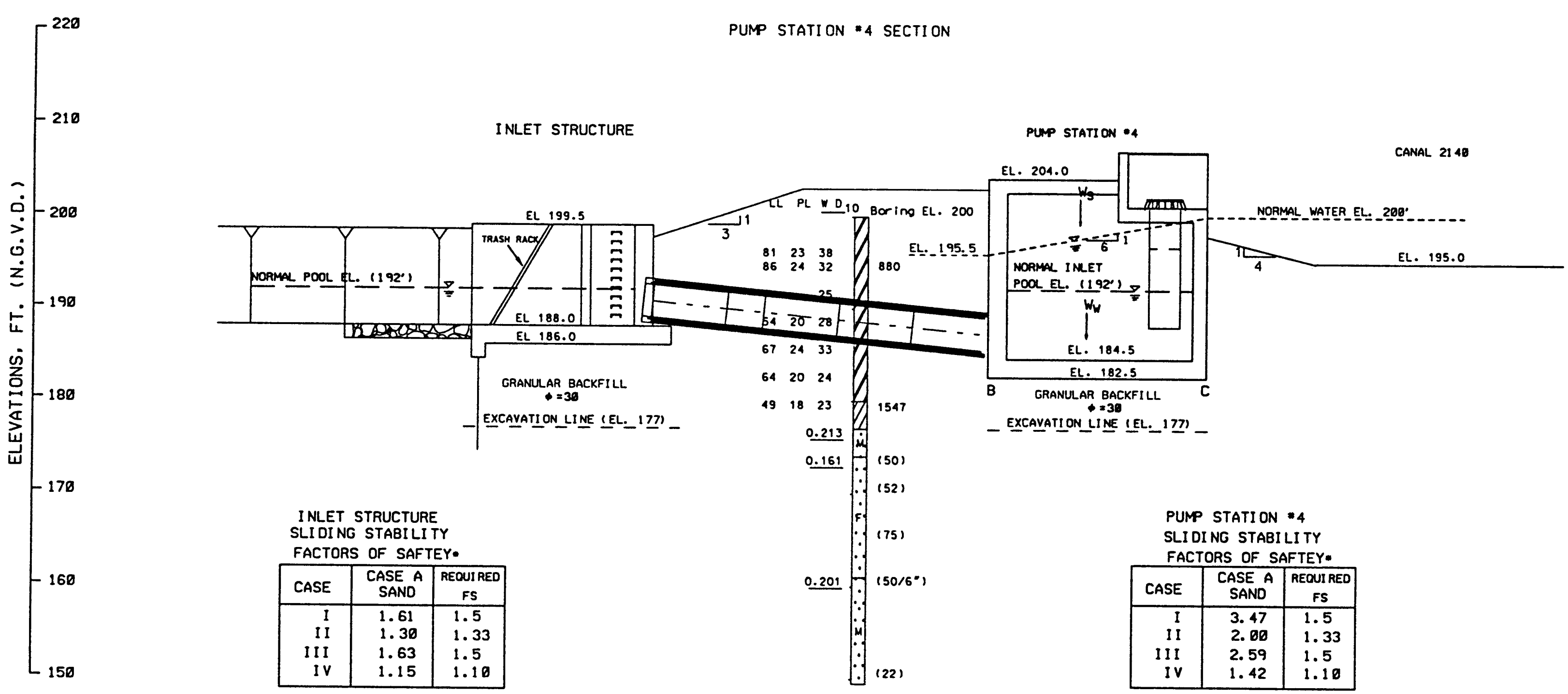
PUMPING STATION #4 EXCAVATION STABILITY

PLATE

II-64



INLET STRUCTURE
SLIDING STABILITY
FACTORS OF SAFTEY*

CASE	CASE A SAND	REQUIRED FS
I	1.61	1.5
II	1.30	1.33
III	1.63	1.5
IV	1.15	1.10

*FROM WES PROGRAM CSLIDE (X0075)

PUMP STATION #4
SLIDING STABILITY
FACTORS OF SAFTEY*

CASE	CASE A SAND	REQUIRED FS
I	3.47	1.5
II	2.00	1.33
III	2.59	1.5
IV	1.42	1.10

*FROM WES PROGRAM CSLIDE (X0075)

ANALYSIS ASSUMPTIONS

1. SLIDING STABILITY ANLAYIS PERFORMED USING COMPUTER SOFTWARE DEVELOPED BY WES (X0075) 'CSLIDE'
2. THE STRUCTURES WILL BE BACKFILLED WITH CLEAN SAND WITH SHEAR STRENGHTS OF ϕ=30 .
3. THE NORMAL POOL ELEVATION IN CANAL 2140 IS EL. 200'.
4. THE NORNAL POOL ELEVATION IN THE INLET CHANNEL IS EL. 192'.
5. CASES II, III & IV ASSUME THAT THE WATER ELEVATION IN THE PUMP STATION IS EQUAL TO THE NORMAL POOL ELEVATION IN THE INLET CHANNEL (EL. 192).
6. THE CALCULATED WEIGHT OF THE PUMP STATION ASSUMES THAT ONE OF THE THREE PUMP STATION BAYS WILL BE DEWATERED AT ANY TIME.
7. FOR CASE IV, THE WATER IN BACKFILL (EL. 195.5) WAS ESTIMATED BY SLOPING PHREATIC SURFACE ON 1V:6H FROM NORMAL CANAL POOL (EL. 200) TO BACK WALL.
8. DUE TO THE HIGH PLASTICITY AND LOW LONG-TERM STRENGTH OF THE FOUNDATION MATERIAL BELOW BOTH STRUCTURES, THE FOUNDATION MATERIALS WILL BE EXCAVATED TO SAND (APPROX. EL. 177) AND REPLACED WITH GRANULAR BACKFILL.

CASES ANALYZED

CASE I - AFTER CONSTRUCTION CONDITION. NO WATER IN CHANNEL.

CASE II - SUDDEN DRAWDOWN CONDITION. WATER IN BACKFILL AT EL. 202.0, WATER IN CANAL AT 197.0.
(INLET STRUCTURE DRAWDOWN CONDITIONS - EL. 195 TO EL. 190)

CASE III - PARTIAL POOL CONDITION. WATER ELEVATION IN THE BACKFILL SAME AS WATER ELEVATION IN CANAL 2140 (EL. 200), WITH VARYING POOL ELEVATIONS.
(INLET NORMAL POOL - EL. 192)

CASE IV - EARTHQUAKE CONDITION. WATER IN BACKFILL AT EL. 195.5 AND WATER IN RESERVIOR AT EL. 200.
(INLET NORMAL POOL - EL. 192)
HORIZONTAL EARTHQUAKE ACCELERATION COEFFICIENT KH=0.1.

CASE A - STRUCTURE FOUNDED ON SAND ϕ=30 .

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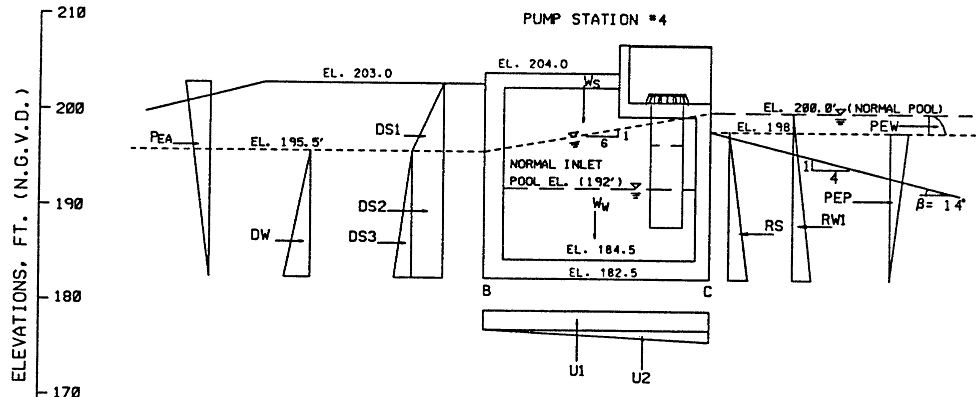
PUMPING STATION #4 SLIDING STABILITY ANALYSIS

PLATE
II-65



OVERTURNING SUMMARY (CASE IV)

ITEM	FORCES (KIPS) VERTICAL ↓	FORCES (KIPS) VERTICAL ↑	HORIZONTAL →	HORIZONTAL ←	MOMENT ARM (FEET)	MOMENTS (KIP-FEET) OVERTURNING	MOMENTS (KIP-FEET) RESISTING
WS	42.10				11.60		488.36
WW	4.32				11.67		50.41
D_{S1}			1.76		15.5	27.25	
D_{S2}			6.09		6.5	39.61	
D_{S3}			2.64		4.33	11.46	
D_W			5.27		4.33	22.85	
U_1		20.57			11.67	239.93	
U_2		1.47			7.78	11.47	
R_{W1}				0.12	0.67		0.08
R_{W2}				1.93	7.75		14.99
R_{W3}				7.50	5.17		38.73
R_S				2.85	5.17		14.73
P_{EPS}			1.23		15.56	72.23	
P_{EA}			1.58		12.83	20.23	
P_{EV}			0.01		16.3	0.16	
P_{EP}			4.64		11.15	13.69	
TOTALS	46.42	22.04	23.22	12.40		458.73	607.30

OVERTURNING ANALYSIS

(LOADING DIAGRAM - CASE IV)
CASE IV - EARTHQUAKE CASE - WATER IN BACKFILL AT EL. 195.5;
WATER IN CANAL 2140 AT EL. 200.0

DRIVING SIDE

$K_0 = (1 - \sin \phi')(1 + \sin \beta)$
$K_0 = (1 - \sin 30°)(1 + \sin 0°)$
$K_0 = 0.50$

RESISTING SIDE

$K_0 = (1 - \sin \phi')(1 + \sin \beta)$
$K_0 = (1 - \sin 30°)(1 + \sin(-14°))$
$K_0 = 0.38$

CREEP PATH = 13 + 23.33 + 15.5 = 51.83 FT
PRESSURE AT B = [(195.5-182.5)-(13/51.83)(-4.5)](0.0624) = 0.88 KSF
PRESSURE AT C = [(195.5-182.5)-(36.33/51.83)(-4.5)](0.0624) = 1.01 KSF

$D_{S1} = 0.5(7.5)^2(0.125)(0.50) = 1.76^K$
$D_{S2} = 0.125(7.5)(13.0)(0.5) = 6.09^K$
$D_{S3} = 0.5(13.0)^2(0.0626)(0.5) = 2.64^K$
$D_W = 0.5(13.0)^2(0.0624) = 5.27^K$
$U_1 = 0.88(23.33) = 20.57^K$
$U_2 = 0.5(1.01-0.88)(23.33) = 1.47^K$
$R_{W1} = 0.5(2.0)^2(0.0624) = 0.12^K$
$R_{W2} = 0.5(2.0)(15.5)(0.0624) = 1.93^K$
$R_{W3} = 0.5(15.5)^2(0.0624) = 7.5^K$
$R_S = 0.5(15.5)^2(0.0626)(0.38) = 2.85^K$

DYNAMIC FORCES

$P_{EA} = (0.1)(1.76+6.09+2.64+5.27) = 1.58^K$
$P_{EP} = (0.1)(2.85+1.93+7.5) = 1.23^K$
$P_{EW} = (2/3)(0.051)(0.1)(200-198)^2 = 0.01^K$
$P_{EPS} = (0.1)(42.01+4.32) = 4.64^K$

STRUCTURE - STRUCTURE WEIGHT = 1122.96 K
1122.96K/ 26.67'(WIDTH) = 42.10 K/FT.WIDTH
WATER = 115.1/26.67 = 4.31 K/FT. WIDTH (1 BAY DEWATERED)
TOTAL WEIGHT = 46.41 K/FT. WIDTH

CASE IV (EARTHQUAKE)

$\bar{y}$ = RESULTANT LOCATION = $\frac{\Sigma M}{\Sigma V} = \frac{607.3 - 458.7}{46.42 - 22.04}$ = 6.1' FROM C (OK LOCATED WITHIN BASE OF SLAB)

CASE I (AFTER CONSTRUCTION) RESULTED IN A $\bar{y}$ = 10.17' (OK LOCATED WITHIN KERN)
CASE II (SUDDEN DRAWDOWN) RESULTED IN A $\bar{y}$ = 7.76' (OK WITHIN 75% OF BASE)
CASE III (PARTIAL POOL) RESULTED IN A $\bar{y}$ = 9.19' (OK LOCATED WITHIN KERN)
KERN 7.78' TO 15.55' FROM C

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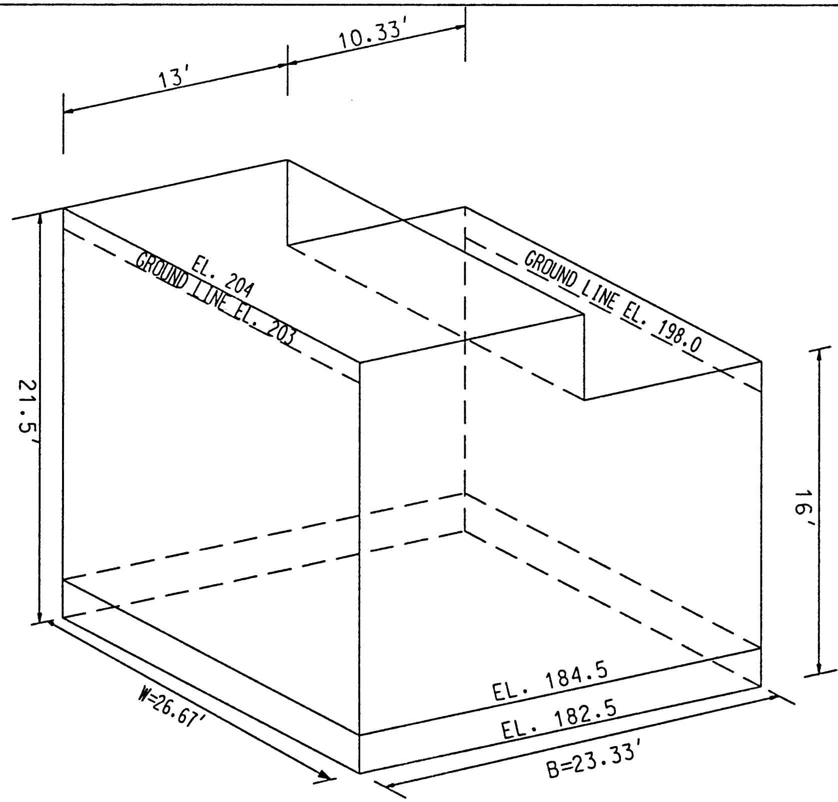
OVERTURNING ANALYSIS

PLATE
II-66



UPLIFT (NORMAL OPERATION - WATER IN BACKFILL TO 196)

BASE WIDTH = (23.33' • 26.67') = 622.21 S.F.

PRESSURE AT EL. 182.5 = $(\frac{200 + 195}{2} - 182.5)(0.0624) = 0.936$ KSF

UPLIFT FORCE = U = 622.21 (0.94) = 582.39 KIPS

RESISTING UPLIFT

WEIGHT OF STRUCTURE = W_S = STRUCTURE WT. + EQUIPTMENT WT.
STRUCTURE WT = 1122.96 KIPS (NO LIVE LOAD)

WEIGHT OF WATER
WATER IN PUMPING STATION - EL. 192 - 3 BAYS FULL
W_C = (13.6' WIDE X 21.3' LONG X 9.5' HEIGHT) 0.0624 = 172.6 K

WATER ON/OUTSIDE PUMP STATION
W_G = 0 KIPS

FROM ETL 1110-2-307 (PAGE 1)

$$FS = \frac{W_S + W_C}{U - W_G}$$

$$FS = \frac{1122.96 + 172.6}{582.39 - 0}$$

WATER EL. 200 FS = 2.22 ∴ O.K.

BEARING CAPACITY

STRUCTURE = B = 23.33' AND W = 26.67'; B/6 = 23.3/6 = 3.89'

B' = B - 2e

CASE I (AC)	- e = 1.5' •	B' = 20.33'	V = 42.1 K
CASE II (SD)	- e = 3.90'	B' = 15.52'	V = 21.99 K
CASE III (PP)	- e = 2.47'	B' = 18.38'	V = 20.94 K
CASE IV (EQ)	- e = 5.57'	B' = 12.19'	V = 24.38 K

• e IS FROM OVERTURNING ANALYSES

$$Q = \frac{V}{BL}\left(1 + \frac{6e}{B}\right)^{**}$$

$$Q_{max} = \frac{42.1}{23.33}\left[1 + \frac{6(1.50)}{23.33}\right] = 2.50 \text{ K/S.F. CASE I}$$

$$Q_{max} = \frac{21.99}{23.33}\left[1 + \frac{6(3.90)}{23.33}\right] = 1.89 \text{ K/S.F. CASE II}$$

$$Q_{max} = \frac{20.94}{23.33}\left[1 + \frac{6(2.47)}{23.33}\right] = 1.47 \text{ K/S.F. CASE III}$$

$$Q_{max} = \frac{24.38}{23.33}\left[1 + \frac{6(5.57)}{23.33}\right] = 2.67 \text{ K/S.F. CASE IV}$$

SAND FOUNDATION (CASE I -AC)

SAND - C=0 PSF; ϕ = 30; N_c=30.14; N_q=18.4; N_γ=15.67

γ' = 0.0625 D = 15.5' B = 23.33' W = 26.67'

$\bar{q} = \gamma' D = 0.0626(15.5) = 0.97$

$S_c = 1 + 0.1 \cdot N_\phi (B'/W') = 1 + 0.1 \cdot 3.0\,(20.33/26.67) = 1.21$

$S_\gamma = S_q = 1.21$

$d_c = 1 + 0.2 \cdot \sqrt{N_\phi}\, D/B = 1 + 0.2 \cdot 1.73 \cdot (23.3/26.67) = 1.23$

$d_q = 1 + 0.1 \cdot \sqrt{N_\phi}\, D/B = 1 + 0.1 \cdot 1.73 \cdot (23.3/26.67) = 1.12$

$d_\gamma = d_q = 1.12$

$R_\gamma = 1 - 0.25 \cdot \text{LOG}\left(\frac{B}{6}\right)$ WHERE B> 6 FT. R_γ= 0.85

$I_q = g_q = b_q = d_\gamma = I_\gamma = g_\gamma = b_\gamma = 1$

$q_{ult} = \cancel{cN_c S_c d_c I_c g_c b_c}^{\,0} + \bar{q} N_q S_q d_q I_q g_q b_q + 0.5 \gamma B N_\gamma S_\gamma d_\gamma I_\gamma g_\gamma b_\gamma R_\gamma$ **

CASE I q_{ult} = 0 + (0.97)(30.14)(1.21)(1.12)(1) + (0.5)(0.0626)(18.38)(15.67)(1.21)(1.12)(1)(0.85)
= 36.11 KSF

CASE I F.S. = $\frac{q_{ult}}{Q_{max}} = \frac{36.11}{2.5}$ = 14.45 > 3.0 OK

CASE II F.S. = $\frac{31.89}{1.89}$ = 16.88 > 2.0 OK

CASE III F.S. = $\frac{34.37}{1.47}$ = 23.4 > 3.0 OK

CASE IV F.S. = $\frac{29.1}{2.67}$ = 10.91 > 1.0 OK

** FROM EM 1110-1-1905 'BEARING CAPACITY OF SOILS'
PAGES 4-4 THROUGH 4-14.

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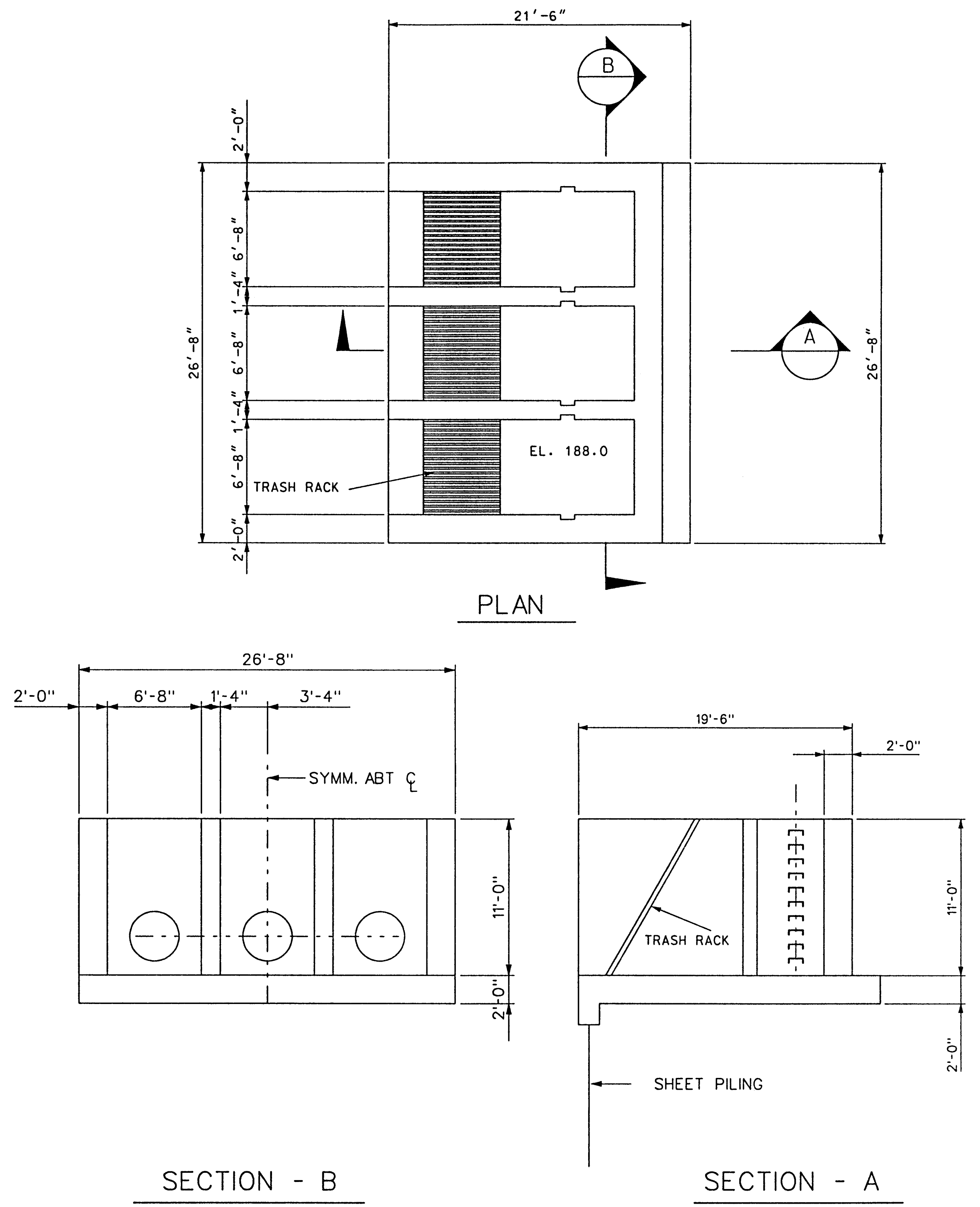
PUMPING STATION #4 BEARING & UPLIFT ANALYSIS

PLATE II-67



SCALE 1" = 10'

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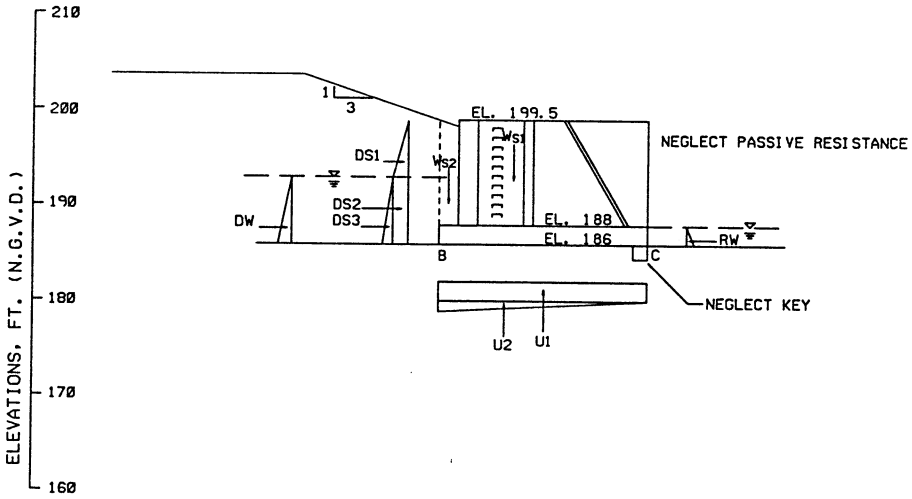
PUMPING STATION #4 INLET
PLAN & SECTION

PLATE

II-68



OVERTURNING SUMMARY (CASE II-SD)

ITEM	FORCES (KIPS)				MOMENT ARM (FEET)	MOMENTS (KIP-FEET)	
	VERTICAL		HORIZONTAL			OVERTURNING	RESISTING
	↓	↑	→	←			
WS	17.53				10.66		186.87
WS2	2.71				20.5		55.52
D_{S1}			1.56		8.83	13.82	
D_{S2}			3.55		3.5	12.43	
D_{S3}			1.01		2.33	2.36	
D_W			1.53		2.33	3.57	
U_1		2.68			10.75	28.84	
U_2		2.53			14.33	36.27	
R_W				0.12	0.67		0.08
TOTALS	20.24	5.21	7.65	0.12		97.28	242.47

OVERTURNING ANALYSIS

(LOADING DIAGRAM - CASE II)
CASE II - SUDDEN DRAWDOWN CASE - WATER IN BACKFILL AT EL. 193.0; WATER IN CANAL AT EL. 188.0

DRIVING SIDE

$K_0 = (1 - \sin\phi')(1 + \sin\beta)$
$K_0 = (1 - \sin 30°)(1 + \sin 18.4°)$
$K_0 = 0.66$

RESISTING SIDE

$K_0 = (1 - \sin\phi')(1 + \sin\beta)$
$K_0 = (1 - \sin 30°)(1 + \sin 0°)$
$K_0 = 0.5$

CREEP PATH = 7 + 21.5 + 0 = 28.5 FT
PRESSURE AT B = [(193-186)-(7/28.5)5](0.0624) = 0.36 KSF
PRESSURE AT C = [(193-186)-(28.5/28.5)5](0.0624) = 0.12 KSF

$D_{S1} = 0.5(6.2)^2(0.125)(0.66) = 1.56^K$
$D_{S2} = 0.125(6.2)(7.0)(0.66) = 3.55^K$
$D_{S3} = 0.5(7.0)^2(0.0626)(0.66) = 1.01^K$
$D_W = 0.5(7.0)^2(0.0624) = 1.53^K$
$U_1 = 0.12(21.5) = 2.68^K$
$U_2 = 0.5(0.36-0.12)(21.5) = 2.53^K$
$R_{W1} = 0.5(7.0)^2(0.0624) = 0.12^K$

STRUCTURE - STRUCTURE WEIGHT = 467.62^K
$467.62^K / 26.67'$(WIDTH) = 17.53 K/FT.WIDTH

CASE I (AFTER CONSTRUCTION) RESULTED IN A $\bar{y} = 10.51'$ (OK LOCATED WITHIN KERN)

CASE II (SUDDEN DRAWDOWN)

$$\bar{y} = \text{RESULTANT LOCATION} = \frac{\Sigma M}{\Sigma V} = \frac{242.47 - 97.28}{20.24 - 5.21} = 9.66' \text{ FROM C (OK LOCATED KERN)}$$

CASE III (PARTIAL POOL) RESULTED IN A $\bar{y} = 10.38'$ (OK LOCATED WITHIN KERN)
CASE IV (EARTHQUAKE) RESULTED IN A $\bar{y} = 9.01'$ (OK LOCATED WITHIN KERN)

KERN 7.17' TO 14.33' FROM C

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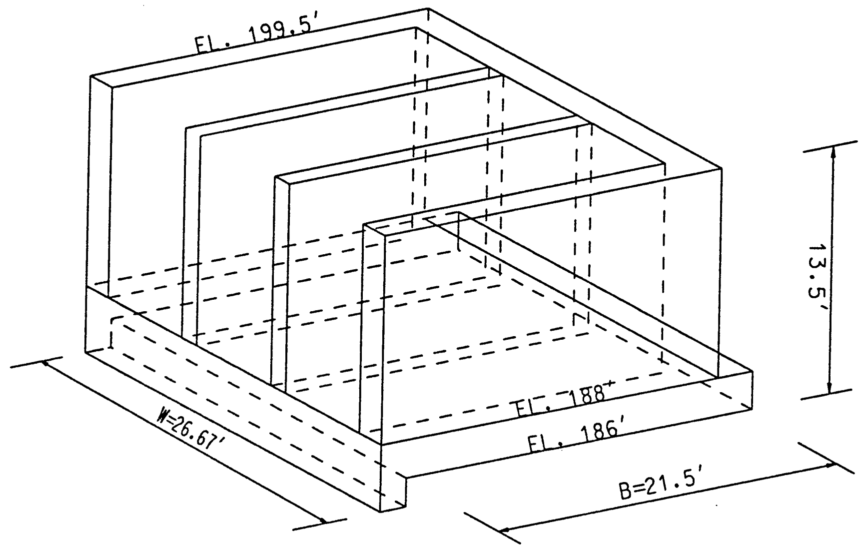
PUMP STATION #4 - INLET OVERTURNING ANALYSIS

PLATE
II-69



BEARING CAPACITY

STRUCTURE = B = 21.5' AND W = 26.67'; B/6 = 21.5/6 = 3.58'

$B' = B - 2e$

CASE I (AC)	- e = 0.24'*	B' = 21.02'	V = 20.24^K
CASE II (SD)	- e = 1.09'	B' = 19.33'	V = 15.02^K
CASE III (PP)	- e = 0.37'	B' = 20.76'	V = 12.19^K
CASE IV (EQ)	- e = 1.74'	B' = 18.01'	V = 12.19^K

* e IS FROM OVERTURNING ANALYSES

$$Q = \frac{V}{BL}\left(1 + \frac{6e}{B}\right)^{**}$$

$$Q_{max} = \frac{20.24}{21.5}\left[1 + \frac{6(0.24)}{21.5}\right] = 1.0 \text{ K/S.F.} \quad \text{CASE I}$$

$$Q_{max} = \frac{15.02}{21.5}\left[1 + \frac{6(1.09)}{21.5}\right] = 0.91 \text{ K/S.F.} \quad \text{CASE II}$$

$$Q_{max} = \frac{12.19}{21.5}\left[1 + \frac{6(0.37)}{21.5}\right] = 0.63 \text{ K/S.F.} \quad \text{CASE III}$$

$$Q_{max} = \frac{12.19}{21.5}\left[1 + \frac{6(1.74)}{21.5}\right] = 0.84 \text{ K/S.F.} \quad \text{CASE IV}$$

$$q_{ult} = cN_c S_c d_c I_c g_c b_c + \bar{q} N_q S_q d_q I_q g_q b_q + 0.5 \gamma B N_\gamma S_\gamma d_\gamma I_\gamma g_\gamma b_\gamma R_\gamma^{**}$$

SAND - C=0 PSF; ϕ = 30; N_c=30.14; N_q=18.4; N_γ=15.67

γ' = 0.0625 D = 0' B = 21.5' W = 26.67'

$\bar{q} = \gamma' D = 0.0626(0) = 0$

CASE I-AC

$S_c = 1 + 0.1 \cdot N_\phi (B'/W') = 1 + 0.1 \cdot 3.0\,(21.02/26.67) = 1.24$

$S_\gamma = S_q = 1.24$

$d_c = 1 + 0.2 \cdot \sqrt{N_\phi}\ D/B = 1 + 0.2 \cdot 1.73 \cdot (0/21.5) = 1.0$

$d_q = 1 + 0.1 \cdot \sqrt{N_\phi}\ D/B = 1 + 0.1 \cdot 1.0 \cdot (0/21.5) = 1.0$

$d_\gamma = d_q = 1.0$

$R_\gamma = 1 - 0.25 \cdot \mathrm{LOG}\left(\frac{B}{6}\right)$ WHERE B> 6 FT. $R_\gamma = 0.86$

$I_q = g_q = b_q = d_\gamma = I_\gamma = g_\gamma = b_\gamma = 1$

SAND FOUNDATION

C=0 PSF; ϕ= 30

$$q_{ult} = \cancel{cN_c S_c} d_c I_c g_c b_c + \cancel{\bar{q} N_q S_q} d_q I_q g_q b_q + 0.5 \gamma B N_\gamma S_\gamma d_\gamma I_\gamma g_\gamma b_\gamma R_\gamma^{**}$$

CASE I $q_{ult} = 0 + 0 + (0.5)(0.0626)(21.02)(15.67)(1.24)(1)(1)(0.86)$
$= 10.98$ KSF

CASE I F.S. $= \frac{q_{ult}}{Q_{max}} = \frac{10.98}{1.0} = 10.93 > 3.0$ OK

CASE II F.S. $= \frac{9.94}{0.91} = 10.92 > 2.0$ OK

CASE III F.S. $= \frac{10.82}{0.63} = 17.3 > 3.0$ OK

CASE IV F.S. $= \frac{9.15}{0.84} = 10.86 > 1.0$ OK

** FROM EM 1110-1-1905 'BEARING CAPACITY OF SOILS' PAGES 4-4 THROUGH 4-14.

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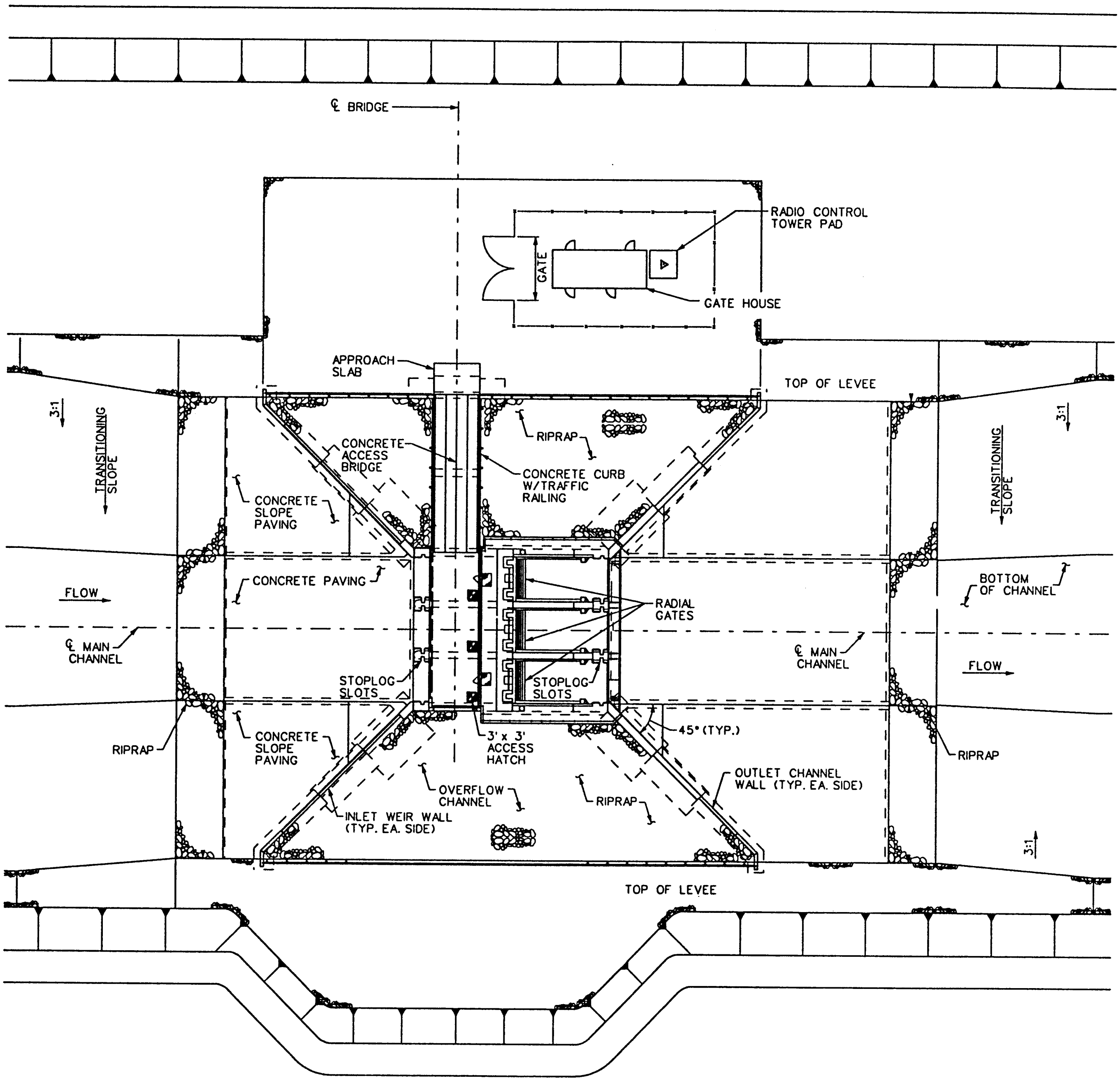
PUMPING STATION #4 INLET BEARING ANALYSIS

PLATE II-70



TYPICAL

NO SCALE

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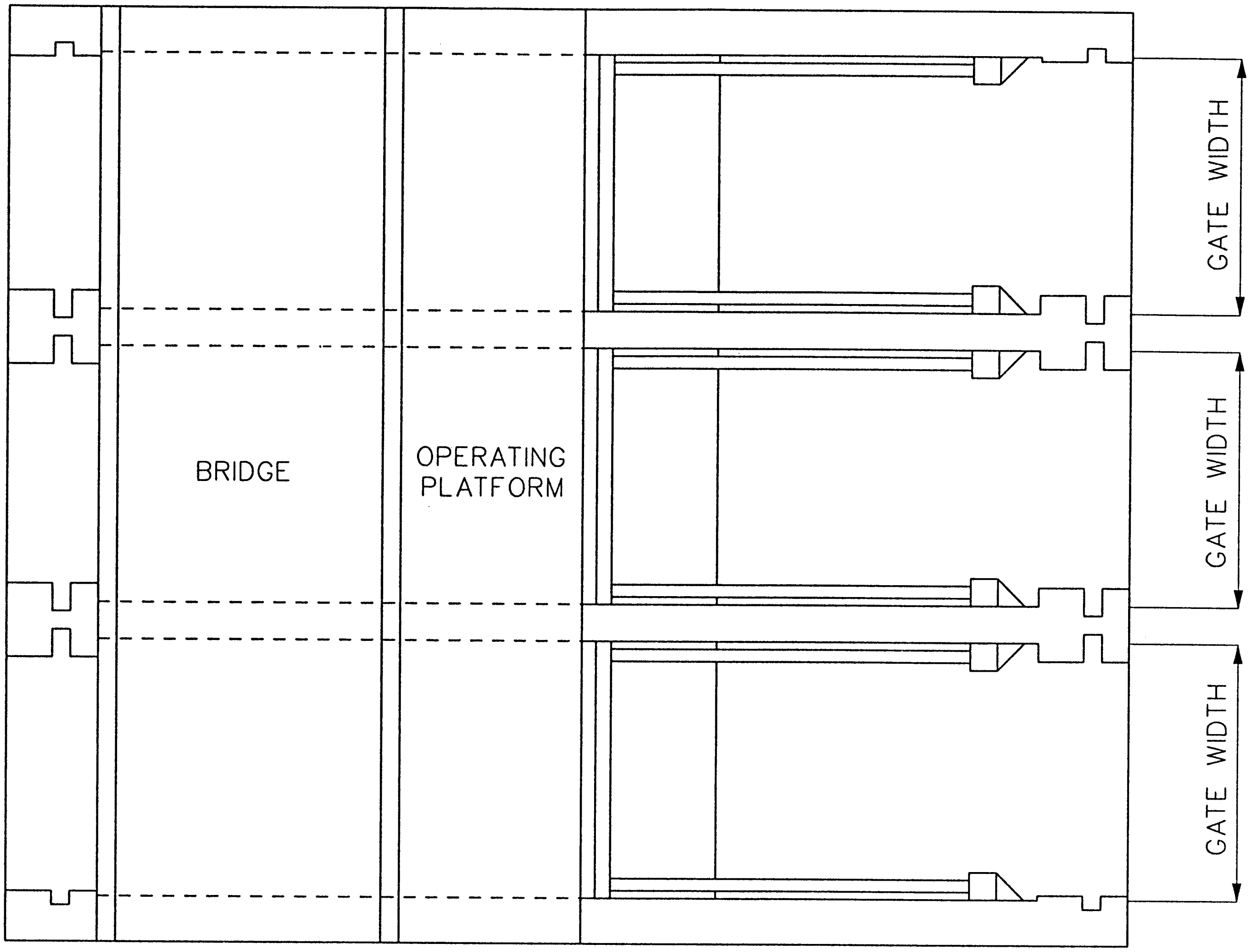
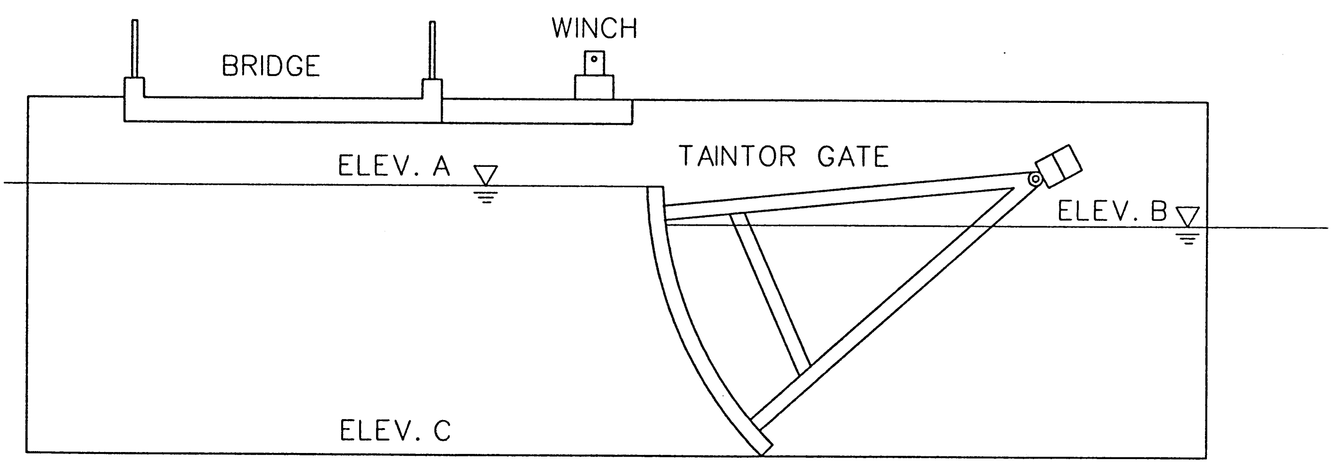
TYPICAL CONTROL STRUCTURE SITE PLAN

PLATE

II-71



PLAN OF THREE GATED CHECK STRUCTURE



LONGITUDINAL SECTION

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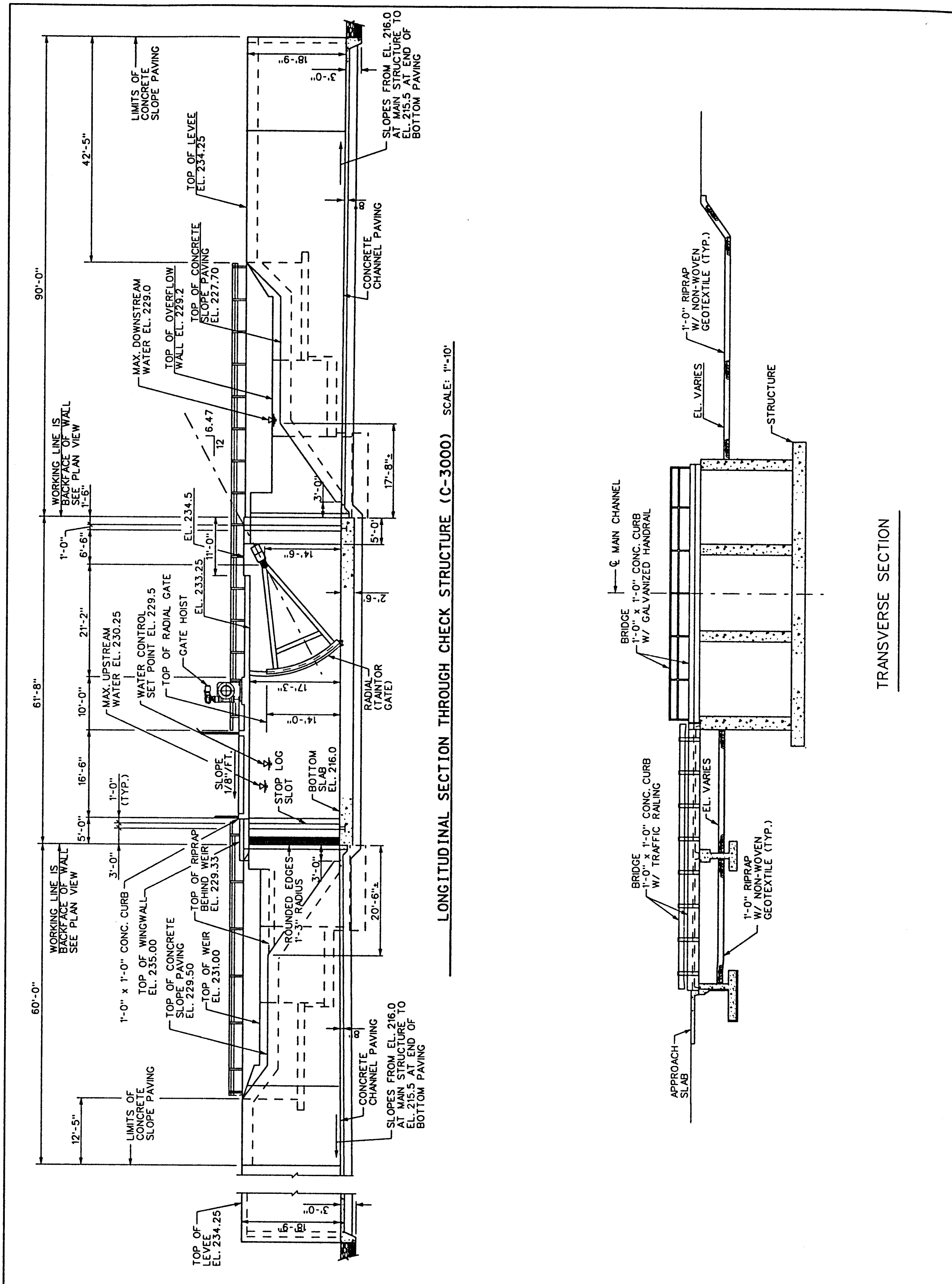
TYPICAL CONTROL STRUCTURE SECTIONS

PLATE

II–72



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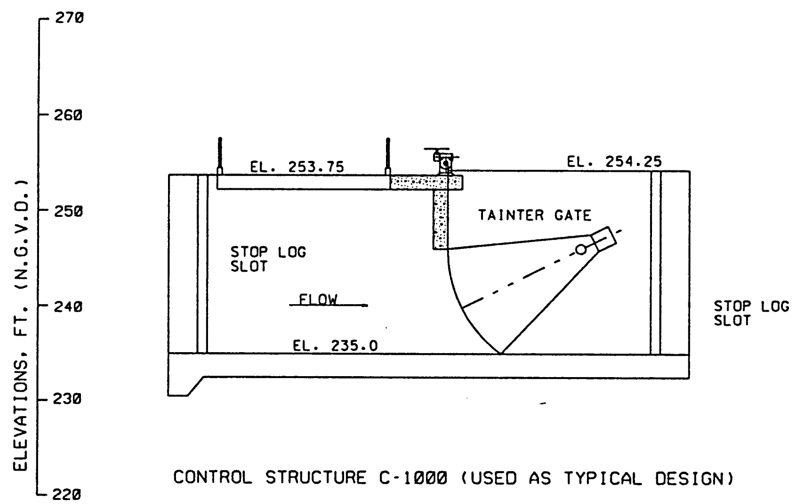
TYPICAL CONTROL STRUCTURE PLAN & PROFILE

PLATE

II-73



CONTROL STRUCTURE C-1000 (USED AS TYPICAL DESIGN)

SLIDING ANALYSIS

NOTE: SLIDING ANALYSES WERE COMPUTED FOR THE CONTROL STRUCTURE (C-1000) FOR CASES II, IV AND V. THESE WERE THE ONLY CASES RESULTING IN HORIZONTAL LOADING. ALL THREE SOIL CONDITIONS FOR POSSIBLE STRUCTURE PLACEMENT WERE CONSIDERED: CLAY, SILT OR SAND. CONSERVATIVE SHEAR STRENGTHS AS SHOWN BELOW WERE USED:

	Q CASE	S CASE
CLAY	$\phi = 0°$, C = 750 PSF,	$\phi' = 18°$ C=0 PSF
SILT	$\phi = 20°$, C = 300 PSF,	$\phi' = 28°$ C=0 PSF
SAND	$\phi = 30°$, C = 0	

CASES ANALYZED

CASE I - AFTER CONSTRUCTION CONDITION. NO WATER IN INLET CHANNEL OR OUTLET CHANNEL. N/A (NO LOAD)

CASE II - SUDDEN DRAWDOWN CONDITION. WATER IN THE RESERVIOR CHANNEL AT EL. 248.0, (13.0 HEAD). WATER IN OUTLET CHANNEL AT THE CHANNEL BOTTOM (EL. 235.0).

CASE III - PARTIAL POOL (N/A)

CASE IV - EARTHQUAKE CONDITION. WATER IN THE INLET CHANNEL AT NORMAL POOL, EL. 248.0. WATER IN OUTLET CHANNEL AT LOW POOL, EL. 244.0.

CASE V - SAME AS CASE II USING S STRENGTH FOR CLAY AND SILT ANALYSES.

CASE VI - SAME AS CASE IV USING S STRENGTH FOR CLAY AND SILT ANALYSES.

FACTORS OF SAFETY

FAILURE PLANE	CLAY	SILT	SAND
CASE II	5.61	4.66	3.84
CASE IV	3.56	3.29	2.97
CASE V	2.44	3.54	N/A
CASE VI	1.88	2.74	N/A

SLIDING ANALYSES WERE COMPUTED USING WES COMPUTER PROGRAM CSLIDE (X0075) THESE ANALYSES WERE COMPUTED ASSUMING A SAND BACKFILL AROUND THE STRUCTURE AND UTILIZED THE WEIGHT OF CONCRETE IN THE STRUCTURE, GATES AND WATER UPSTREAM OF TAINER GATES. THE STRUCTURE IS SAFE AGAINST SLIDING.

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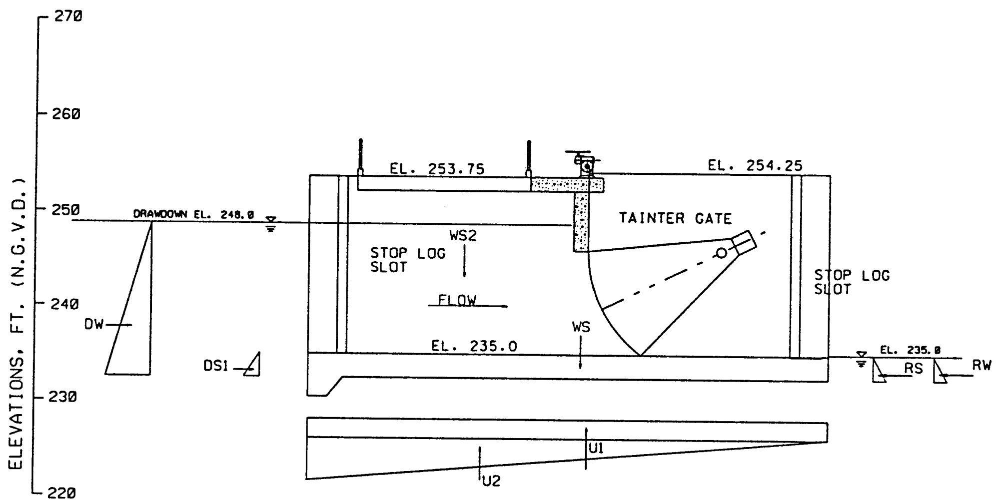
TYPICAL CONTROL STRUCTURE SLIDING STABILITY ANALYSIS

PLATE II-74



CONTROL STRUCTURE C-1000 (USED AS TYPICAL DESIGN)

ITEM	FORCES (KIPS) VERTICAL ↓	FORCES (KIPS) VERTICAL ↑	FORCES (KIPS) HORIZONTAL →	FORCES (KIPS) HORIZONTAL ←	MOMENT ARM (FEET)	MOMENTS (KIP-FEET) OVERTURNING	MOMENTS (KIP-FEET) RESISTING
WS	56.02				28.1		1574.16
WS2	17.69				40.25		712.02
D_{S1}			0.1		0.83	0.08	
D_W			7.5		5.17	38.73	
U_1		10.28			27.0	277.56	
U_2		20.05			36.0	721.67	
R_{W1}				0.2	0.83		0.16
R_S				0.1	0.83		0.08
TOTALS	73.71	30.33	7.6	0.3		1038.04	2286.43

OVERTURNING ANALYSIS

(LOADING DIAGRAM - CASE II)

CASE II - SUDDEN DRAWDOWN CONDITION - WATER IN RESEREVIOR AT EL. 248.0; WATER IN THE OUTLET CHANNEL AT EL. 235.0.

$K_0 = (1 - \sin \phi')(1 + \sin B)$

$K_0 = (1 - \sin 30°)(1 + \sin 0°)$

$K_0 = 0.50$

CREEP PATH = 2.5 + 54 + 2.5 = 59 FT

PRESSURE AT B = [(248-232.5)-(2.5/59)5](0.0624) = 0.93 KSF

PRESSURE AT C = [(248-232.5)-(56.5/59)5](0.0624) = 0.192 KSF

$D_{S1} = 0.5\,(2.5)^2(0.125)(0.50) = 0.1^K$

$D_W = 0.5(15.5)^2(0.0624) = 7.5^K$

$U_1 = 0.90(54) = 10.28^K$

$U_2 = 0.5(0.93 - 0.19)(54) = 20.05^K$

$R_{W1} = 0.5(2.5)^2(0.0624) = 0.2^K$

$R_S = 0.5(2.5)^2(0.0626) = 0.1^K$

STRUCTURE - TOTAL WEIGHT = 2913.02^K

$2913.02^K/52'$(WIDTH) = 56.02 K/FT.WIDTH

WATER = 17.69 K/FT. WIDTH

TOTAL WEIGHT = 73.71 K/FT. WIDTH

$\bar{y}$ = RESULTANT LOCATION = $\frac{\Sigma M}{\Sigma V} = \frac{2286.43 - 1038.04}{73.71 - 30.33} = 28.78'$ FROM C

KERN 18' TO 36' FROM C

THEREFORE, RESULTANT FORCE IS LOCATED WITHIN THE KERN AND THE STRUCTURE IS SAFE AGAINST OVERTURNING.

CASE I (AFTER CONSTRUCTION); NA

CASE III (PARTIAL POOL) NA

CASE IV (EARTHQUAKE) RESULTED IN A $\bar{y} = 29.2'$

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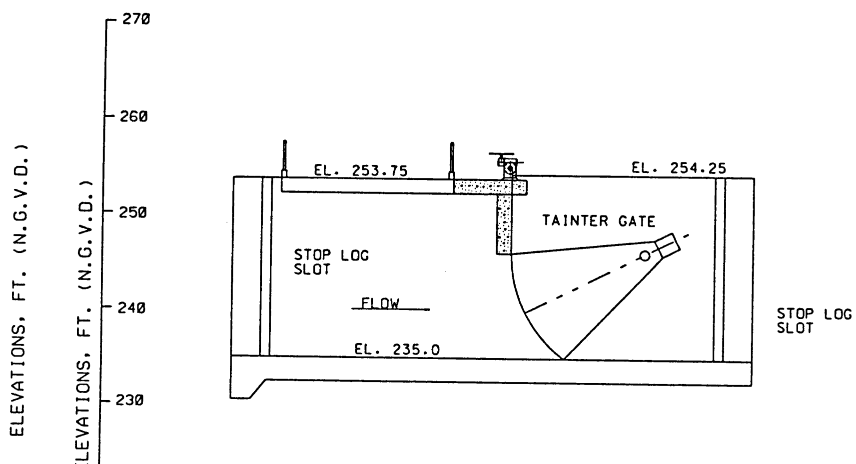
TYPICAL CONTROL STRUCTURE OVERTURNING ANALYSIS

PLATE II-75



CONTROL STRUCTURE C-1000 (USED AS TYPICAL DESIGN)

ELEVATIONS, FT. (N.G.V.D.)

BEARING ANALYSIS

WEIGHT - 2913.02 KIPS

DIMENSIONS B = 54' W = 52' B/6 = 54/6 = 9

B' = B - 2e

CASE II (SD): e = 1.78' B' = 40.45' e < B/6 V = 43.38^K
CASE IV (EQ): e = 2.2' B' = 49.61' e < B/6 V = 38.87^K
*e FROM OVERTURNING ANALYSES

FOR e < B/6, $q = \frac{V}{BL}\left(1 + \frac{6e}{B}\right)$ FOR e > B/6, $q = \frac{2V}{3L(B/2 - e)}$

CASE II $q = \frac{43.38}{54}\left[1 + \frac{6(1.78)}{54}\right] = 0.96$ K/S.F.

CASE IV $q = \frac{38.87}{54}\left[1 + \frac{6(2.2)}{54}\right] = 0.90$ K/S.F.

$q_{ult} = cN_c S_c d_c i_c g_c b_c + \bar{q} N_q S_q d_q i_q g_q b_q + 0.5 \gamma B N_\gamma S_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$

NOTE: THREE SUBGRADES WERE ANALYZED TO FIND CRITICAL CASE

SILT	$\phi = 20°$, C = 300	$N_c = 14.83$,	$N_q = 6.4$,	$N_\gamma = 2.9$
SAND	$\phi = 30°$, C = 0	$N_c = 30.14$,	$N_q = 18.4$,	$N_\gamma = 15.1$
CLAY	$\phi = 0°$, C = 750	$N_c = 5.14$,	$N_q = 1$,	$N_\gamma = 0$

B = 54' B'(CASE II) = 50.45' $\gamma' = 0.0624$ W = 52' D/B = 2.5/54 = 0.046

$q = \gamma' D = 0.0626(2.5) = 0.1565$

$S_c = 1 + 0.1\ N_\phi B'/W = 1 + (0.1)2.04(50.45)/52 = 1.2$ SILT; FOR CLAY, $S_c = 1.1$
$S_q = 1 + 0.1\ N_\phi (B'/L) = 1 + .1(2.04)(50.45/52) = 1.2$; FOR $\phi = 30°$, $S_q = 1.29$
$S_\gamma = S_q = 1.2$ FOR CLAY $S_q = 1.1$

$d_q = 1 + 0.1 \cdot \sqrt{N_\phi}\ D/B = 1 + 0.1 \cdot 2.04 \cdot (2.5/54) = 1.01$ SILT, D = 1.01 CLAY
$d_\gamma = d_\gamma = 1.01$

FOR SILT

$q_{ult} = 0.3(14.83)(1.2)(1.01) + 0.61(6.4)(1.2)(1.01)$
$+ 0.5(0.0626)(1.01)(2.87)(0.76) = 10.44$ KSF

FOR SAND

$q_{ult} = 0.16(18.4)(1.29)(1.01) + 0.5(0.0626)(54)(15.1)(1.29)(1.01)(0.76) = 28.27$ KSF

FOR CLAY

$q_{ult} = 1.0(5.14)(1.1)(1.01) + 0.16(1)(1.1)(1) = 4.44$ KSF

CLAY SUBGRADE RESULTED IN THE MOST CRITICAL ULTIMATE BEARING CAPACITY AND WAS USED FOR DETERMINING FACTORS OF SAFETY. MOST OF THE STRUCTURES WILL BE PLACED ON EITHER A SANDY SILT OR SAND FOUNDATION. ALSO, THE CLAY STRENGTH OF C = 750 PSF IS VERY CONSERVATIVE.

CASE II F.S. = $\frac{4.44}{0.96} = 4.62$ OK > 2

CASE IV F.S. = $\frac{4.38}{0.38} = 11.59$ OK > 1

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TYPICAL CONTROL STRUCTURE BEARING ANALYSIS

PLATE II-76