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May 6, 1985
ST-HL-AE-124C
File No.: G4.2, G9.17, C13.5.3

Mr. George W. Knighton, Chief
Licensing Branch No. 3
Division of Licensing
U. S. Nuclear Regulatory Commission
Washington, DC 20555

South Texas Project
Units 1 & 2
Docket Nos. STN 50-498, STN 50-499
Main Cooling Reservoir Meeting

Dear Mr. Knighton:

As you requested (NRC letter dated March 13, 1985) we met with the NRC staff on April 2, 1985. As noted in your letter the intent of this meeting was to address staff concerns relative to "erosion effects due to dike failure". This topic has been the subject of several meetings with the staff over the past two years and has further been identified to the NRC in the past as potentially reportable under 10CFR50.55(e).

During the meeting HL&P provided details on the studies, analyses, and modifications that have taken place relative to the Main Cooling Reservoir (MCR) embankment adjacent to the power block. These studies and analyses demonstrate that an adequate margin of safety for the MCR embankment can be maintained for all credible failure mechanisms. Therefore, all non- mechanistic effects (such as scour and erosion) associated with a postulated failure of the embankment need not be included in the design basis of the plant.

Details of the various studies and analyses are provided in Attachment 1. Slides and charts used in the presentation are provided in Attachment 2.

The NRC staff concurred that erosion and scour associated with a postulated embankment breach need not be considered in the design basis of the power block. This concurrence is contingent upon successful completion of the following items.

1. HL&P is to provide additional test/boring data as recommended by Dr. Seed in the area just east of Embankment Station 40+00. This will be provided by July 1, 1985.

2. HL&P is to include figures in the FSAR of the wave barriers used in the area where the circulating water piping crosses over the embankment. This will be provided by July 1, 1985.

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3. The NRC will review the operating procedures for the MCR. A draft of the operating procedures will be provided by July 1, 1985.

   Included in the procedures should be:

   ° Monitoring program (frequencies, instrumentation, etc.)

   ° Appropriate actions to be taken in case of problems.

4. HL&P is to provide to the NRC the evaluation performed with reservoir level held at elevation 35' to ensure that conditions are stable and acceptable. This evaluation will be provided by September 30, 1986 or when the reservoir embankment conditions are stable enough to permit an evaluation.

HL&P indicated that each of the above items will be performed and the required information will be submitted in a timely manner.

If you have any questions on the content of this letter, please contact M. E. Powell at (713) 993-1328.

Very truly yours,

M. R. Wisenburg
Manager, Nuclear Licensing

SMH/yd
Attachment: 1. Main Cooling Reservoir Embankment Summary Report
2. Presentation Slides

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W2/NRC2/e

Attachment 1

SOUTH TEXAS PROJECT

MAIN COOLING RESERVOIR

TABLE OF CONTENTS

0.0 EXECUTIVE SUMMARY

The Main Cooling Reservoir of the South Texas Project Nuclear Plant is impounded by an above ground embankment. Governing flood elevations for the design of safety related structures were determined by postulation of a non-mechanistic breach of this embankment. The mechanistic effects of the breach (i.e. scour and erosion) were not included in the plant design basis.

During a Bechtel review performed as part of Bechtel's assumption of Architect/Engineer responsibilities for the South Texas Project, the following technical issues were studied:

1. Embankment and foundation stability
2. Seepage and internal erosion
3. Embankment overtopping during floods
4. Erosion potential at penetrations

The above issues have been fully addressed by analytical studies, field test programs, additional instrumentation to monitor embankment performance and, where necessary, by structural modifications.

These studies and tests performed on the as-built embankment demonstrated that an adequate margin of safety can be maintained for all credible failure mechanisms for the section of the reservoir embankment opposite the power block. Therefore, mechanistic effects (i.e. scour and erosion) associated with a postulated breach of the embankment were not included in the design basis of the plant.

1.0 BACKGROUND

South Texas Project Nuclear Plant receives cooling water for non-safety related purposes from an onsite, above ground, man-made reservoir called the Main Cooling Reservoir (MCR). The reservoir impoundment consists of a 12.4 mile long compacted earth embankment about 40 to 50 feet high. The ultimate heat sink is the Essential Cooling Pond (ECP), which is a completely separate structure.

In order to satisfy the requirements of Regulatory Guide 1.59, Rev. 0, which was the governing regulatory document at the time the STP Construction Permit was docketed, an instantaneous non-mechanistic breach of the embankment was postulated to determine flood levels. The size of the breach was determined by a parametric study to be that resulting in the highest flood levels. Issuance of Rev. 2 of R.G.1.59 (1977), and the change from Appendix A of the guide to ANSI N170 for guidelines on postulation of events producing the Design Basis Flood, necessitated further evaluation of the effects of the postulated breach, or alternately an evaluation of the integrity of the segment of the MCR embankment facing the safety-related plant structures.

Reviews and studies from 1982 through 1985 were undertaken to evaluate the design basis of the as-constructed embankment. Emphasis was placed on the section between stations 640 + 00 to 655 + 00 and 0 + 00 to 40 + 00 due to its proximity to the safety related structures. To assist in this review a board of eminent geotechnical engineers (Consulting Review Board - CRB) was retained in early 1983. The concerns raised by Bechtel and the CRB were addressed fully by analytical studies, field test programs, and where necessary, design modifications. Throughout these developments, HL&P has been in contact with the NRC in an effort to fully clarify the licensing position for STP (Ref. 1-1). The major portion of the studies were completed in 1984. The conclusions of some of these studies were communicated to the NRC in the "Final Report Concerning the Design Basis Flood for the STP" (Ref. 1-2). Further conclusions were discussed with the NRC during a site visit to examine, among other plant features, the embankment (Reference 1-3). Physical modifications to the embankment pipe penetrations are presently underway. The studies and analyses performed demonstrate that an adequate margin of safety for the portion of the MCR embankment facing the power block can be maintained against all credible failure mechanisms. Therefore, mechanistic effects (i.e. scour and erosion) associated with a postulated breach of the embankment have been excluded from the design basis of the plant.

This document summarizes the studies, analyses, tests and modifications undertaken in the post-construction period.

2.0 PHYSICAL DESIGN FEATURES OF THE SYSTEM

The STP site is located about 12 miles south of Bay City, Texas. The cooling water supply for the power plant is provided by the MCR which is an above ground reservoir, located to the south of the power block (Figure 1). The MCR has approximately 7000 acres of surface area and 200,000 acre-feet storage capacity. The rolled earth embankment has a crest elevation which varies between 65.00 and 66.25 feet Mean Sea Level (MSL). The normal maximum operating level is at elevation 49 feet MSL.

The embankment consists of a homogeneous section of highly plastic clay with an upstream slope of 2.5H:1V and a downstream slope of 3H:1V (Figure 2). A downstream berm and, where required for stability during construction, an upstream berm of varying width are provided. The upstream slope of the embankment is protected by a soil cement layer averaging 2.5 feet in thickness.

For purposes of analysis and evaluation the foundation soils are generally grouped into four units. The strata are arranged from the uppermost downwards as follows; Clay Layer 1, Sand Layer 2, Clay Layer 3, and Sand Layer 4. The clays are stiff to very stiff and sands are medium dense to very dense. Sand Layer 2 is present beneath most of the MCR area. Where Sand Layer 2 is nonexistent or irregular, (about 10 percent of the embankment length) a horizontal sand drainage blanket with a toe drain system is provided.

Borrow areas inside the reservoir were controlled to limit the exposure of Sand Layer 2. The minimum distance from the upstream toe for sand borrow was 1200 ft and for clay borrow 800 ft.

Underseepage control is provided by over 650 relief wells extending into Sand Layer 2 near the toe of the downstream berm (Figure 6). The embankment section includes an internal sand core extending to the uppermost foundation clay stratum. Proper functioning of the underseepage control measures is ensured by periodic monitoring of the piezometers installed for this purpose.

There are a number of pipe and structural penetrations through the embankment: The Circulating Water Intake and Discharge Pipes (eight 96" diameter precast, prestressed concrete-embedded steel pipes for the intake which merge into four 138" diameter pipes near the toe berm and four 138" diameter pipes of the same type for the discharge), Make-up Water Discharge Structure, and Spillway structure and Blowdown Pipe (Figure 1). The make-up water pipes consist of two 108" diameter prestressed concrete pipes. The spillway structure includes an intake chamber for the 78" diameter prestressed concrete blowdown pipe. The section of the blowdown pipe parallel to the spillway structure is embedded in concrete. Only the Circulating Water Intake and Discharge Pipes are located in the section of the embankment opposite the power block.

An instrumentation and monitoring system consisting of piezometers, settlement plates and inclinometers has been installed to monitor the performance of the MCR embankment. Data from these instruments is gathered and evaluated periodically as part of the inspection-monitoring program discussed in Section 7.0.

3.0 EMBANKMENT EVALUATION

Bechtel was retained as Architect/Engineer in late 1981. During the project transition phase in late 1981 and early 1982, Bechtel performed an evaluation of pertinent drawings, specifications and reports and identified certain technical concerns related to the MCR embankment.

To assist in the evaluation and resolution of these concerns, Bechtel initially retained Dr. A. J. Hendron, Jr. and Dr. R. E. Olson to perform a preliminary review. Later, an expanded Consulting Review Board (CRB) was formed, consisting of:

Mr. S. D. Wilson
Dr. H. B. Seed
Dr. A. J. Hendron, Jr.
Dr. R. E. Olson
Mr. K. V. Taylor (Deceased)

The CRB has met four times since its inception in early 1983 and has provided guidance in the studies. The Board has also reviewed the results of various evaluations by Bechtel and other consultants including the resulting modifications.

General areas of concern identified by Bechtel and its consultants fall into the following four categories:

1. Stability of the as-built embankment under the design loading conditions.

2. Dispersive potential of clays and the effectiveness of the sand core and drainage blanket as they affect embankment stability.

3. Pipe and structure penetrations through the embankment.

4. Underseepage control measures and effectiveness of the relief well system to maintain uplift pressures at an acceptable level for stability.

To address Item 1, the embankment was analyzed for stability under the following loading conditions:

a) Steady State Seepage

b) Rapid Drawdown

c) Earthquake (seismic stability of the embankment as well as the underlying foundation)

In addition, the potential for embankment overtopping during a Probable Maximum Flood (PMF) and erosion potential during an external flood event were also investigated.

The stability analyses were performed based on strength parameters established for the initial design. Additional investigations and testing performed to verify the design parameters for the stability analyses and the soil dispersion tests to address Item 2 above are discussed in Section 4.0. The analytical studies performed are discussed in detail in Section 5.0. Pipe and structure penetration concerns and the proposed remedial work are discussed in Section 6.0.

The original relief well system designer McClelland Engineers, Inc. (MEI) was retained to address the underseepage control issue and the concerns about uplift pressures. MEI cleaned and refurbished the wells in the portions of the embankment not affected by power block dewatering. MEI has also developed recommendations for further renovation of that portion of the relief well system.

Essential to this type of underseepage control system is the ongoing monitoring and evaluation of the relief well system adequacy. The embankment will be closely monitored during filling which is scheduled to resume August 1985, and modifications to the system, if required, will be implemented to ensure satisfactory performance. Details of the monitoring system are discussed in Section 7.0.

In addition to the design aspects, the important issue of construction quality of the as-built embankment has also been addressed. Recently, Harza and Bechtel performed a review of construction records and identified the documentation corresponding to the embankment section opposite the power block. Reviews of this documentation revealed that thirty three Standard Proctor Tests were conducted and the average standard proctor density was 98% for the section in question. In-place density tests averaged one per one thousand cubic yard of fill placed in the portion of the embankment facing the power block.

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Five percent of the construction reports were sampled to determine disposition and action taken if a field test did not meet specification. On the basis of this sampling, 7.5% of the field tests failed to meet the specification requirements. In all cases reviewed, records show action taken for rework of the fill and subsequent re-test until acceptable results were achieved.

This review showed that the construction was carried out in accordance with the specification, the strength of the as-built embankment matched or exceeded the values used in design, and the tests and inspections performed during the construction are adequately documented.

4.0 CONFIRMATORY TESTS

An extensive investigation and testing program with special emphasis on the embankment section facing the power block was implemented by Harza Engineering Company to verify the parameters used in the stability analyses (Reference 4-1). A total of 48 borings were drilled for a cumulative length of about 2,300 linear feet (Figure 3). 644 calibrated standard penetration tests (SPT) were performed at 5 foot intervals in the embankment and foundation. Where low blow counts were encountered, the intervals were reduced to 2.5 feet. About one hundred undisturbed samples were collected from borings during the field investigations. Twelve laboratory consolidation tests and 106 shear strength tests (69 triaxial and 37 direct shear) were conducted. These tests confirmed that the strength parameters used in the embankment stability analyses are conservative.

In addition, laboratory tests were conducted to evaluate the dispersive potential of the embankment clays. The results of this testing, have shown that the clay has low to moderate dispersive potential. However, because of the extremely low permeability of the material and the resulting low amount of seepage through the embankment, internal erosion is not a concern. Furthermore, the inspection and monitoring program discussed in Section 7.0 will identify, in a timely manner, any potential for seepage pressure exceeding design pressures.

5.0 ANALYTICAL STUDIES

5.1 STABILITY ANALYSES

Harza Engineering Company was retained to address the concerns identified during the initial reviews (Section 3.0). Harza studied the stability of the embankment for the following conditions:

a) Steady state seepage
b) Rapid drawdown
c) Seismic event

A brief summary of the studies and the conclusions are given below:

a) Steady state seepage

The MCR embankment was analyzed for steady-state seepage conditions with reservoir water surface at normal maximum pool elevation of 49 ft.

Soil parameters for this analysis were derived from the results of field and laboratory tests conducted by Harza Engineering Company. Effective, or drained shear strength parameters were used in the analyses. The Section at Station 20 + 00 was taken as representative of the length of embankment opposite the power block. The phreatic surface was assumed to follow a straight line from normal reservoir level on the upstream slope to the intersection of the downstream slope and the top of the toe berm. This resulted in the conservative assumption of a saturated toe berm.

Computed factors of safety for the upstream and downstream slopes at Station 20+00 are 1.82 and 1.72 respectively. Both values are well above the normally accepted minimum value of 1.5. (Reference 4-1).

b) Rapid drawdown

Rapid drawdown on the upstream face at Station 20+00 was analyzed by Harza as representative of the section facing the power block. The water level was assumed to drop from normal pool at El. 49 ft. to the crest of the spillway at El. 39 ft. A combined "R" and "S" shear strength envelope was used for selecting appropriate strength parameters as recommended by the Corps of Engineers. For the expected range of effective overburden pressure within the embankment and the foundation, it was found that the drained strength parameters were lower than the undrained strengths as would be expected for overconsolidated clays under low confining pressure. While rapid drawdown involves undrained shear, drained strength parameters were used for conservatism in accordance with the Corps of Engineers combined "R" and "S" envelope procedures. The change in pore water pressure was assumed equal to the change in total pressure in the upstream face after drawdown. Laboratory tests confirmed the validity of the pore water pressure assumption used in the analysis. The computed factor of safety for rapid drawdown of the upstream face at Station 20+00 was found to be 1.50 for drawdown from El. 49 ft. to El. 39 ft. This value is well above the minimum factor of safety of 1.2 normally used in embankment design (Reference 4-1).

Rapid drawdown conditions on the downstream face are not possible because the short duration of external floods will not allow the saturation of the embankment clays. However, assuming drawdown-type conditions might develop on the downstream toe, the maximum drawdown could only be 6 ft. (maximum external flood level El. 34 ft. to ground El. 28 ft). There is a 48 ft. wide toe berm on the downstream side with 6H:1V slope. Comparing these conditions to the conditions used for the steeper upstream face rapid drawdown study it is clear that rapid drawdown on the downstream face would not be as severe a loading condition as on the upstream face.

c) Seismic stability

The embankment section adjoining the power block (from Station 640+00 to Station 655+00 and from Station 0+00 to Station 40+00) was analyzed for a Safe Shutdown Earthquake (SSE) of 0.1 g in combination with a 25-year recurrent MCR water level which was conservatively taken as El. 50.00 feet. The use of the SSE acceleration of 0.1 g is conservative since the maximum vibratory ground acceleration determined for the STP site is 0.07 g, as described in FSAR Section 2.5.2.6.

The results of the stability analysis show that the embankment section in the vicinity of the power block and the essential cooling pond can withstand SSE accelerations with no permanent deformation (Reference 4-1). These results are conservative because the analysis was based on drained strength parameters for overconsolidated clays in the foundation and a conservatively high phreatic surface within the embankment.

The results of the seismic analysis were also reviewed in relation to the behavior of similar dams which have been subjected to earthquake motions in the past, based on case history evaluation. It is concluded that clay embankments on clay or dense silty sand foundations have successfully withstood earthquake accelerations much higher than the SSE acceleration for the STP with no significant effects (Seed, Makdisi, DeAlba, Reference 5-1).

The seismic stability analysis results, coupled with the case history evaluation of the seismic behavior of similar dams, lead to the conclusion that the section of the embankment facing the STP Category I structures, from Station 640+00 to Station 655+00 and Station 0+00 to Station 40+00, will safely withstand SSE accelerations of 0.1 g with no permanent deformation.

5.2 EMBANKMENT SAFETY DURING FLOOD EVENTS

Possible mechanisms of failure were analyzed to determine the capability of the embankment to withstand the effect of floods. The failure mechanisms considered are:

a) Embankment overtopping during a Probable Maximum Flood (PMF) event.

b) Embankment slope erosion during flood events on Colorado River Basin.

c) Embankment slope stability during flood events on Colorado River Basin.

5.2.1 EMBANKMENT OVERTOPPING DURING A PMF EVENT

Bechtel performed an analysis to evaluate the potential for embankment overtopping during a PMF event. This analysis considered combinations of flood events that would produce the most critical water elevations including the wind setup and wave run-up.

The as-built MCR embankment crest corrected for settlement ranges from El. 65.00 feet to El. 66.25 feet. The normal maximum operating water level in the reservoir is at El. 49.00 feet. Thus the as-built embankment provides a minimum freeboard of 16.00 feet above the normal maximum operating water level in the reservoir.

The estimated total rise in water level above the normal maximum operating level due to a PMF event, in combination with a conservatively calculated wind setup and wave runup on the north section of the embankment, is approximately 12.74 feet. The embankment height provided above the PMF event water level is approximately 3.26 feet, which is more than adequate to prevent embankment overtopping. In addition, the wind setup and wave runup associated with a Probable Maximum Hurricane (PMH) event was considered. This evaluation showed that, even under a PMH event, the embankment height provided above the significant wave runup is approximately 1.4 feet, which is adequate to prevent embankment overtopping (Ref. 5-2).

5.2.2 EMBANKMENT SLOPE EROSION DURING FLOOD EVENTS ON COLORADO RIVER BASIN

Embankment erosion due to a flood event may occur through two distinct mechanisms. The first is the result of flow past the embankment at velocities large enough to cause erosion, and the second is the result of sustained wave action.

Possible erosion of the exterior slope of MCR embankment in the vicinity of the power block was considered. Water levels above the plant grade could occur only for PMF or failure of upstream dam events. Although the maximum discharge at the site is large for either event, the area of the flooded section is sufficiently large to prevent flow velocities from approaching values required for erosion of compacted soils.

Action of waves on the exterior embankment was considered coincident with the flood events. Under the influence of a 50 MPH (2-yr.) wind, waves are essentially depth limited at the toe of embankment, with heights decreasing with decreasing water level. Considering the limited duration of the event, the well compacted clay of the embankment and the protective grass cover, it is concluded that wind wave erosion potential is negligible.

5.2.3 EMBANKMENT SLOPE STABILITY DURING FLOOD EVENTS ON COLORADO RIVER BASIN

As discussed in Section 5.1b, rapid drawdown on the downstream face is not a critical loading condition.

6.0 PHYSICAL MODIFICATIONS

6.1 POTENTIAL FOR INTERNAL SOIL EROSION IN THE EMBANKMENT FACING THE POWER BLOCK

The circulating water system intake and discharge pipes penetrate the embankment section adjoining the power block (Figure 4). During the initial evaluations of the MCR embankment, a concern relative to potential joint leakage and embankment erosion was identified.

The pipes that penetrate the embankment are precast, prestressed concrete - embedded steel cylinder pipes with bell and spigot joints with rubber gaskets. Some of the joints are welded to provide thrust restraint at bends while some are harnessed to permit joint rotation. The pipes are embedded in sand bedding surrounded by embankment material.

The major concern identified by the CRB relates to the watertightness of joints in the pipe sections penetrating the embankment under embankment settlement and hydraulically-induced longitudinal movement. The pipe joints under these loads could open causing leakage which could lead to embankment failure by erosion or by static overstress due to high pore pressures.

As part of their evaluation, Harza studied the pipe penetrations and possible remedial measures to eliminate the erosion concern (Reference 6-1). Their recommendations for the circulating water intake and discharge pipes are summarized below (Figure 5):

- o Expose the circulating water pipes at embankment crest and slopes to provide inspectability of joints.

- o Use new concrete pipes supported in concrete chutes to provide erosion protection.

- o Install additional piezometers along the existing pipes at the toe berm to monitor the performance.

These modifications are currently in progress.

In addition, Harza has recommended improvements such as individual joint liners for makeup and blowdown pipes and chemical grout barriers for the makeup structure and spillway. Although these structures are located further away such that their failure does not constitute a concern for the power block, these recommendations are being implemented to improve the reliability of the reservoir embankment.

The above discussed modifications to pipe and structural penetrations eliminate the possibility of fill erosion due to pipe leakage in the embankment and improve the safety of the plant.

7.0 INSPECTION AND MONITORING PROGRAM

An inspection and monitoring program has been developed for filling of the MCR. The instrumentation that is currently in place includes 274 piezometers, 27 deep and 24 shallow settlement points, and 12 inclinometers. Typical locations of these instruments are shown in Figure 6. 36 additional piezometers have been installed in the toe berm in the area facing the power block. The inspection and monitoring program will follow the guidelines of R.G. 1.127 and will monitor the embankment performance and foundations during reservoir filling. The data gathered will also be used to evaluate uplift pressures in the foundation and determine where additional relief wells may be required. The monitoring program during reservoir filling will provide the basis for long-term monitoring during normal plant operation. The detailed procedures and frequencies for inspection, monitoring and evaluation are included in the project specification.

8.0 SUMMARY AND CONCLUSIONS

As discussed in the previous sections, the portion of the MCR embankment facing the power block has been analyzed for the following credible modes of failure:

1. Embankment stability

    a) Steady state seepage

b) Rapid drawdown

c) Seismic stability (including foundation stability)

2. Seepage and piping

   a) Underseepage

   b) Embankment seepage

3. Embankment overtopping during floods

4. Erosion potential at penetrations.

The results of these analyses indicate that the as-built MCR embankment will be stable and it will perform satisfactorily under the loading conditions analyzed.

The potential for erosion due to pipe leakage has been eliminated by exposing the circulating water pipes penetrating the embankment for inspectability and the provision of erosion resistant concrete chutes around the pipes. Piezometers installed in the toe berm will serve to monitor the embankment performance.

Review of the construction records established the existence of adequate documentation to show that the embankment construction was carried out in accordance with the specifications and deviations were dispositioned satisfactorily. The results of post-construction boring and testing program support the conclusions of this review.

The inspection and monitoring program discussed in Section 7.0 is adequate to identify potential problems in a timely manner so that corrective actions can be implemented if required.

Since it is demonstrated that an adequate safety margin can be maintained for the MCR embankment for all credible failure mechanisms, the mechanistic effects (i.e., scour and erosion) of a postulated breach of the embankment can be excluded from the design basis of the plant.

9.0 REFERENCES:

1-1 NRC to HL&P Letter, ST-AE-HL-90321, dated February 27, 1984.

1-2 HL&P to NRC Letter, ST-HL-AE-1093, dated May 25, 1984.

1-3 NRC to HL&P Letter, ST-AE-HL-90562, dated March 13, 1985.

4-1 Report on Evaluation of Strength Parameters and Stability, MCR Embankment, Harza Engineering Company, September 1984.

5-1 Seed, H. B., F. I. Makdisi, and P. De Alba, "Performance of Earth Dams During Earthquakes," Journal of the Geotechnical Engineering Division, ASCE, Vol. 104, GT7, July 1978, pp. 967-994.

6-1 Report on Evaluation of Pipe and Structural Penetrations, MCR Embankment, Harza Engineering Company, December 1983.

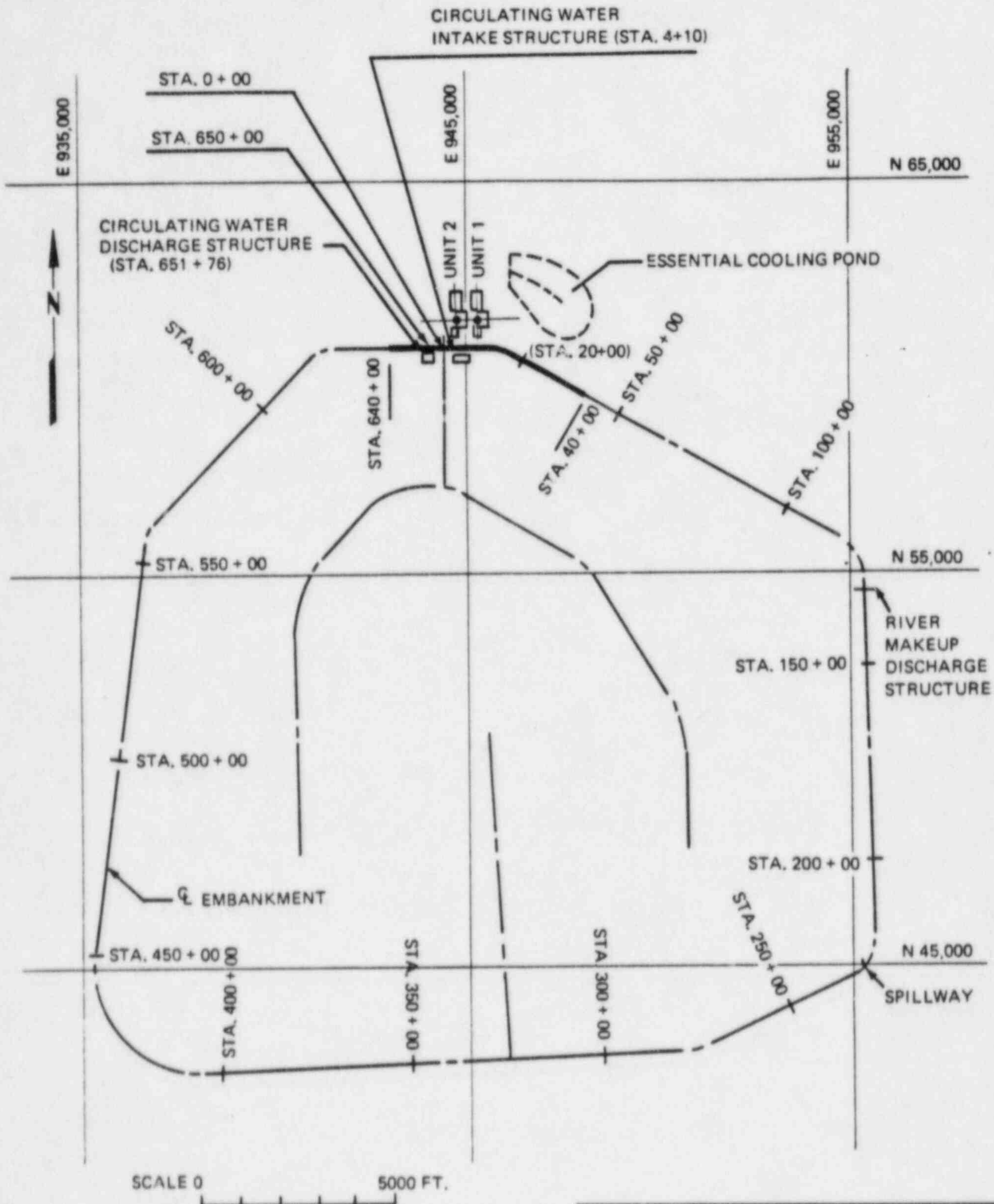


FIGURE 1

MAIN COOLING
RESERVOIR
KEY PLAN

MATERIAL	Ø	C.psf
EMBANKMENT	20°	300
SAND CORE	35°	0
TYPE 1	17°	350
TYPE 2	35°	0
TYPE 3	17°	350
TYPE 4	35°	0

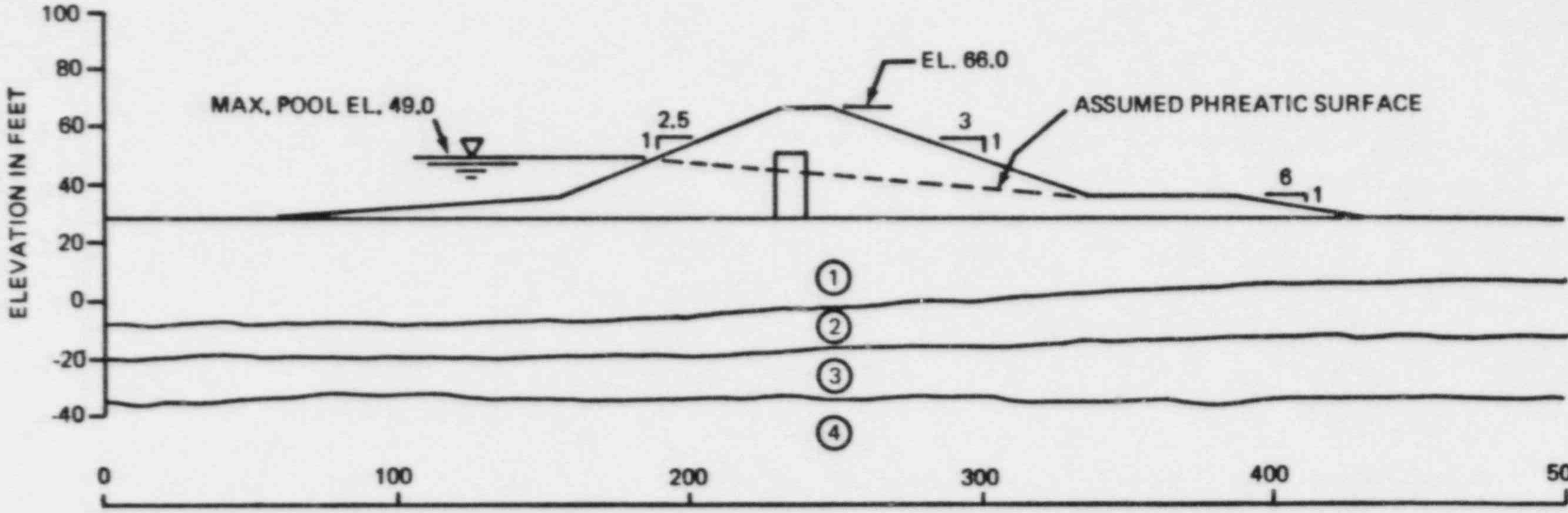
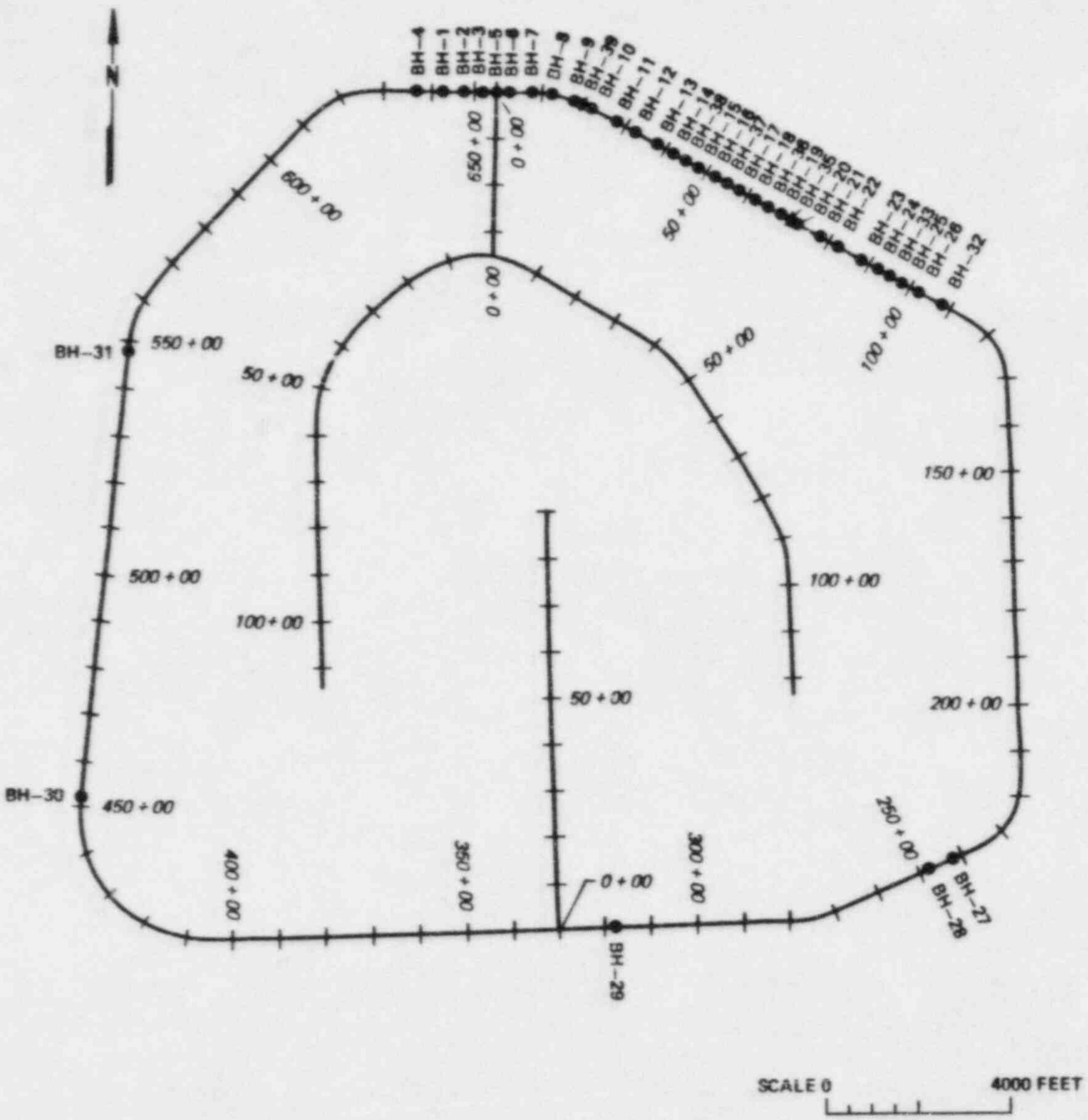


FIGURE 2

TYPICAL CROSS-SECTION FOR ANALYSIS



BOREHOLE No.	STATION	LOCATION
BH–1	642 + 50	D/S
BH–2	647 + 50	CREST
BH–3	652 + 50	U/S
BH–4	637 + 50	D/S
BH–5	0 + 00	D/S
BH–6	2 + 50	U/S
BH–7	7 + 50	CREST
BH–8	12 + 50	D/S
BH–9	17 + 50	U/S
BH–10	22 + 50	D/S
BH–11	27 + 50	CREST
BH–12	32 + 50	U/S
BH–13	37 + 50	D/S
BH–14	42 + 50	U/S
BH–15	47 + 50	D/S
BH–16	52 + 50	U/S
BH–17	57 + 50	D/S
BH–18	62 + 50	U/S
BH–19	67 + 50	D/S
BH–20	72 + 50	U/S
BH–21	77 + 50	D/S
BH–22	82 + 50	U/S
BH–23	87 + 50	D/S
BH–24	92 + 50	U/S
BH–25	97 + 50	D/S
BH–26	102 + 50	U/S
BH–27	242 + 50	CREST
BH–28	247 + 50	D/S
BH–29	317 + 50	D/S
BH–30	452 + 50	D/S
BH–31	547 + 50	CREST
BH–32	107 + 50	U/S
BH–33	96 + 25	D/S
BH–35	71 + 25	U/S
BH–36	66 + 25	D/S
BH–37	56 + 25	U/S
BH–38	46 + 75	D/S
BH–39	20 + 00	CREST

FIGURE 3

CONFIRMATORY PROGRAM BOREHOLE LOCATION PLAN

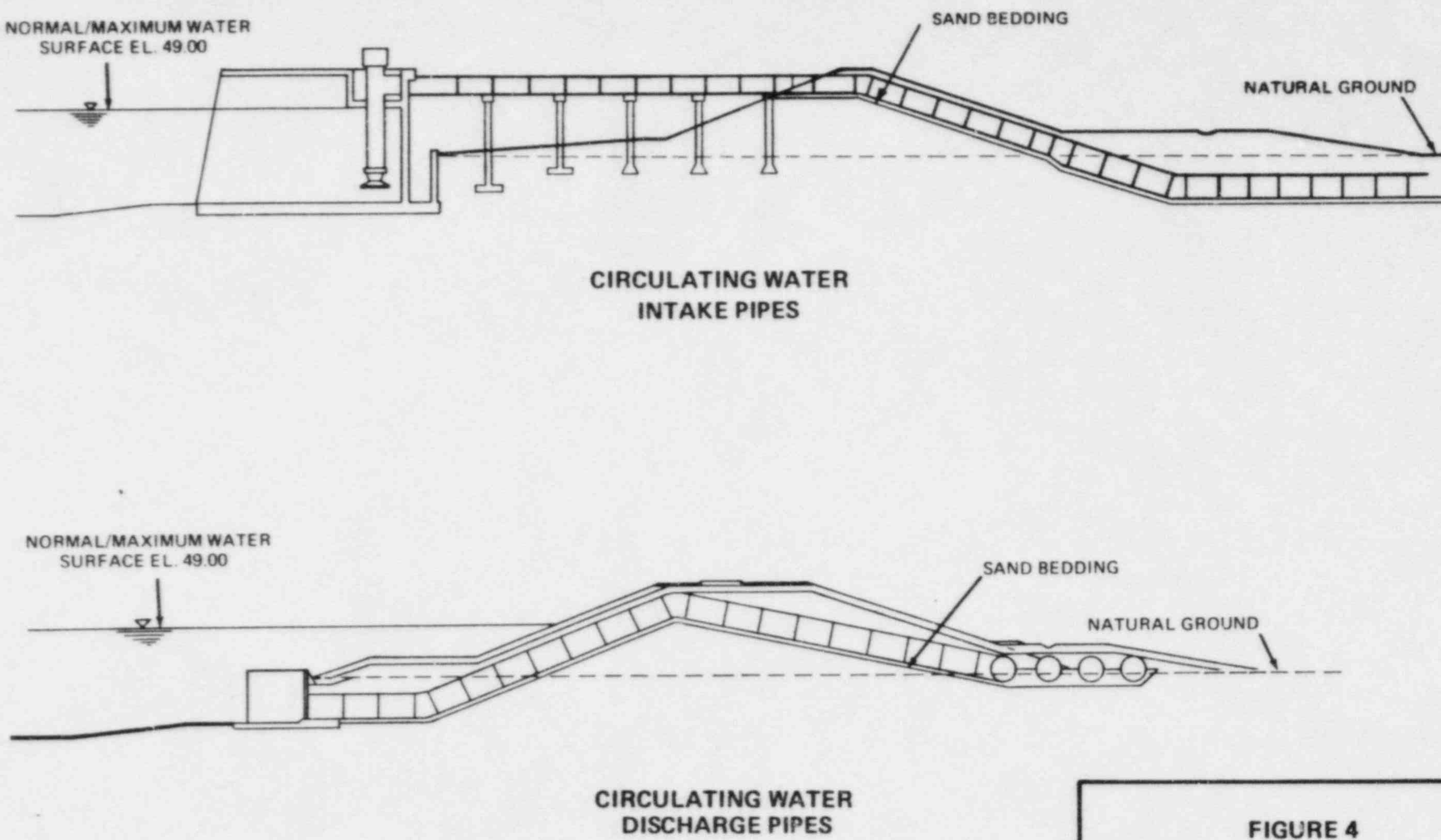
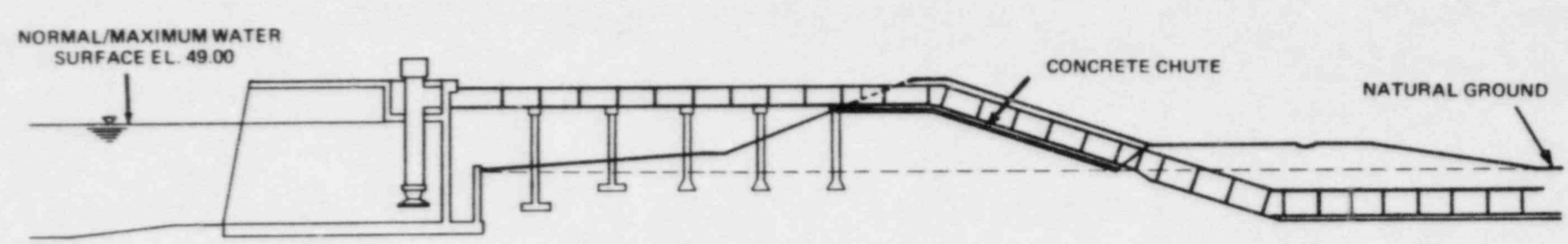
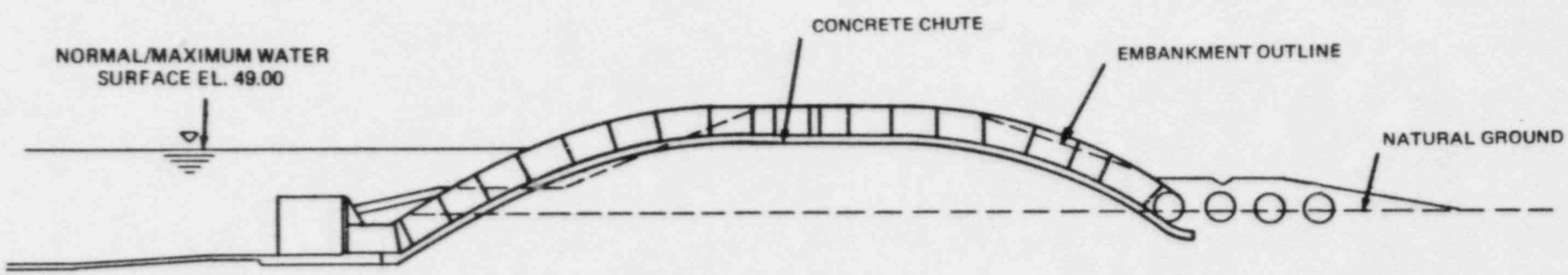


FIGURE 4

CIRCULATING WATER INTAKE & DISCHARGE PIPES BEFORE MODIFICATIONS



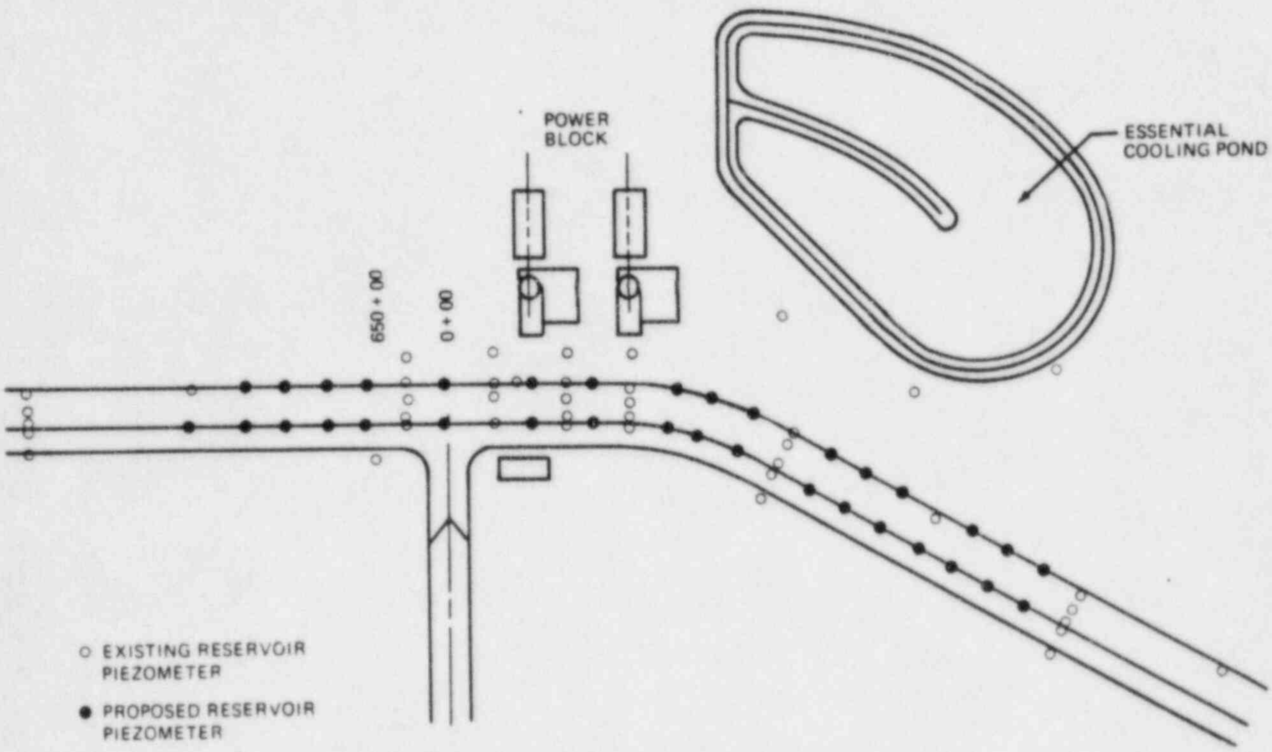
CIRCULATING WATER
INTAKE PIPES



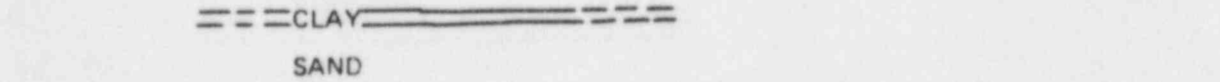
CIRCULATING WATER
DISCHARGE PIPES

FIGURE 5

CIRCULATING WATER
INTAKE & DISCHARGE
PIPES AFTER
MODIFICATIONS



PLAN
(NEAR POWER BLOCK)



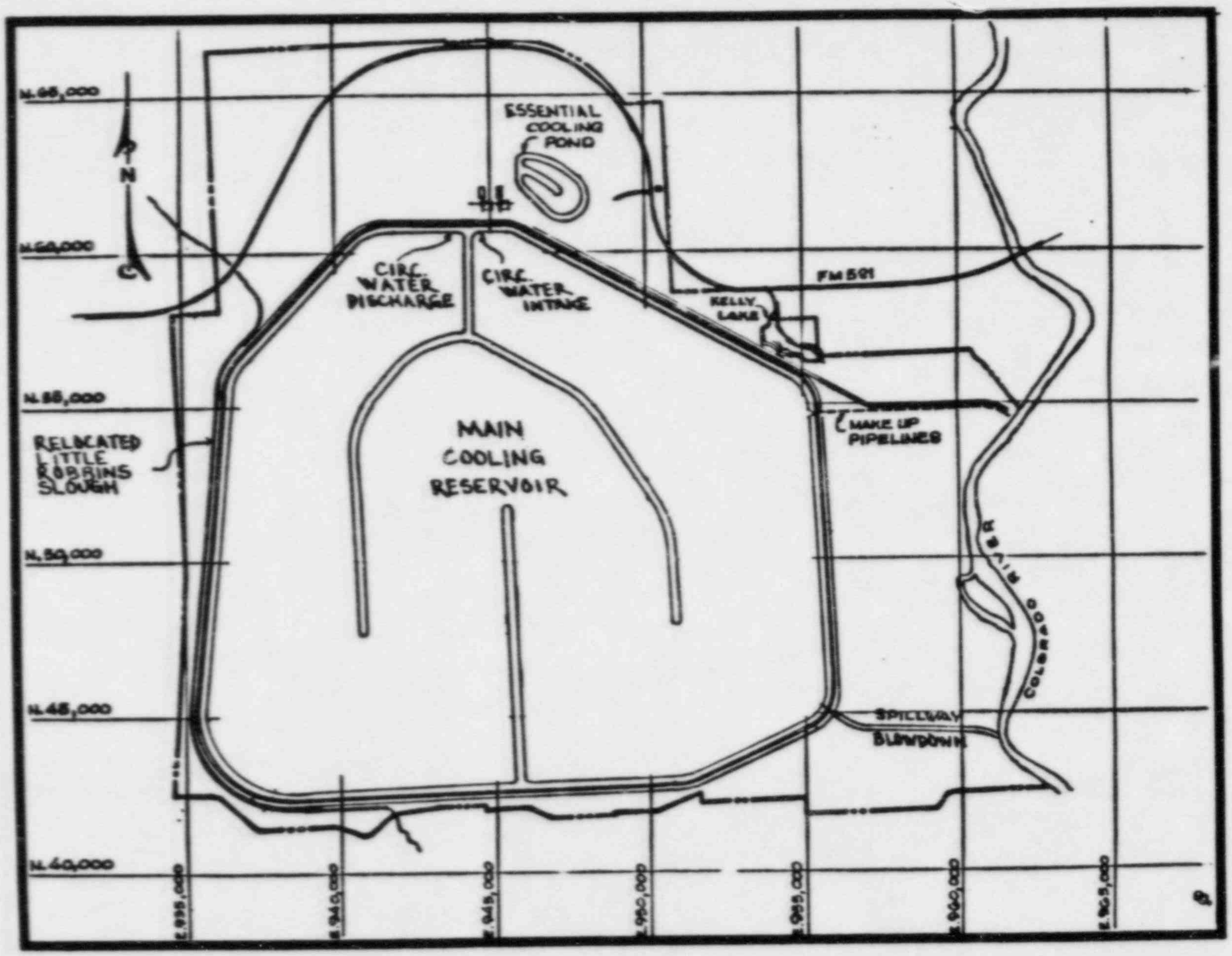
TYPICAL PIEZOMETER ARRAY

FIGURE 6

INSTRUMENT LOCATIONS

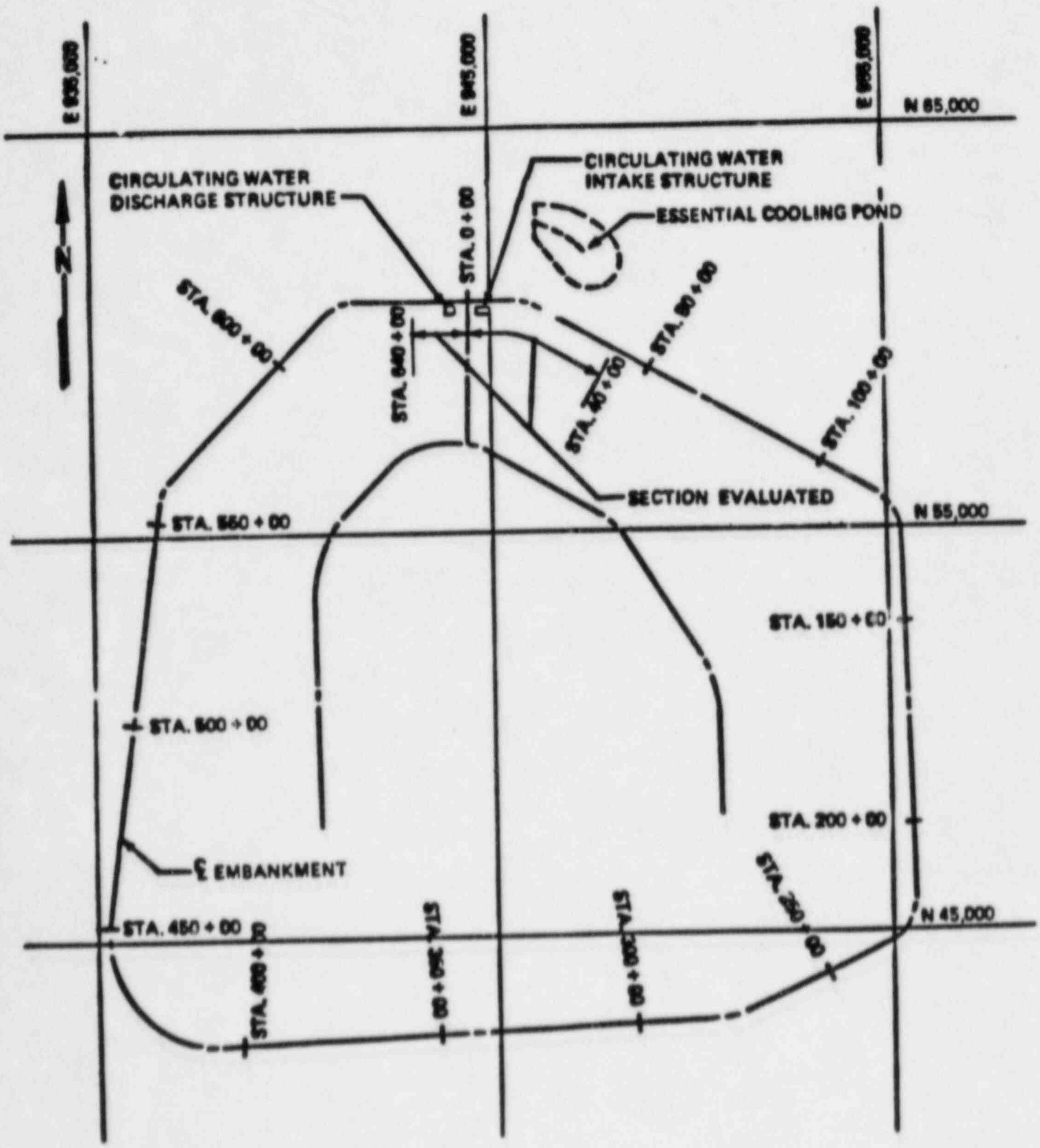
Attachment 2

STP SITE

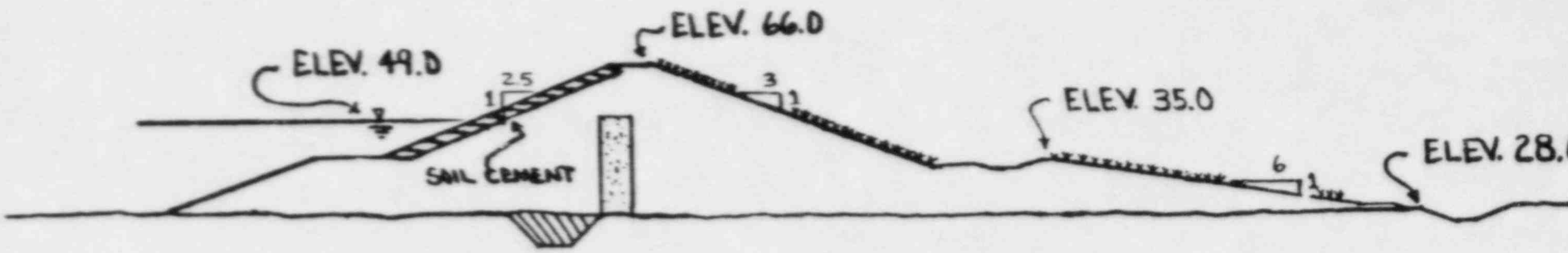


N
N.65,000
N.60,000
N.55,000
N.50,000
N.45,000
N.40,000
E.935,000
E.940,000
E.945,000
E.950,000
E.955,000
E.960,000
E.965,000
ESSENTIAL COOLING POND
CIRC. WATER DISCHARGE
CIRC. WATER INTAKE
FM 521
KELLY LAKE
MAKE UP PIPELINES
MAIN COOLING RESERVOIR
RELOCATED LITTLE ROBBINS SLOUGH
COLORADO RIVER
SPILLWAY
BLOWDOWN

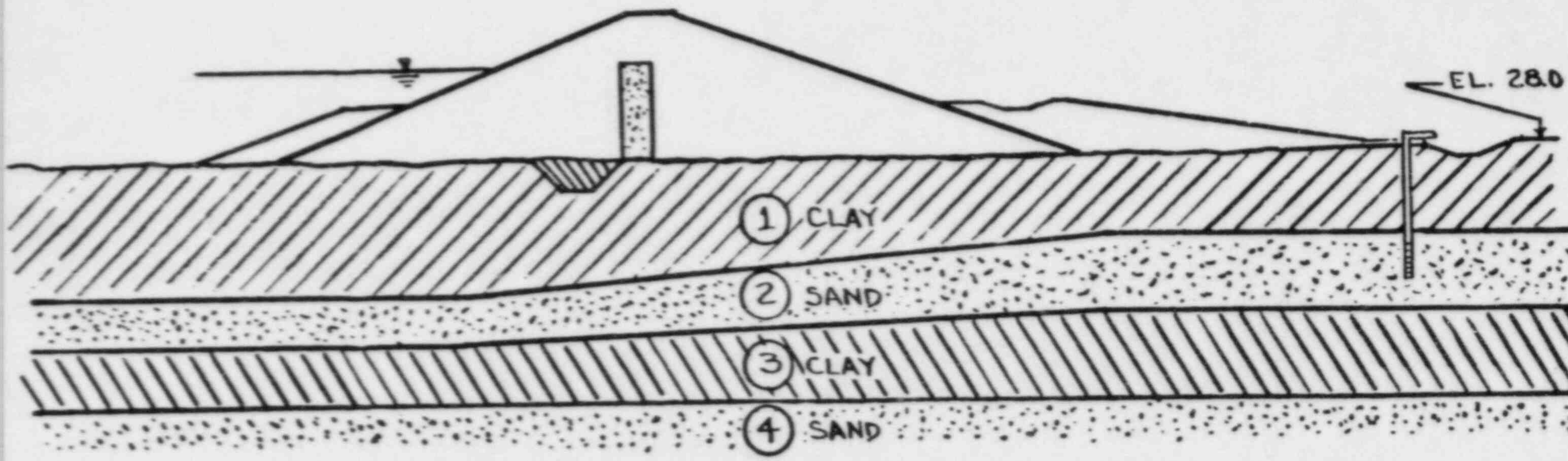
EMBANKMENT CENTERLINE STATIONING



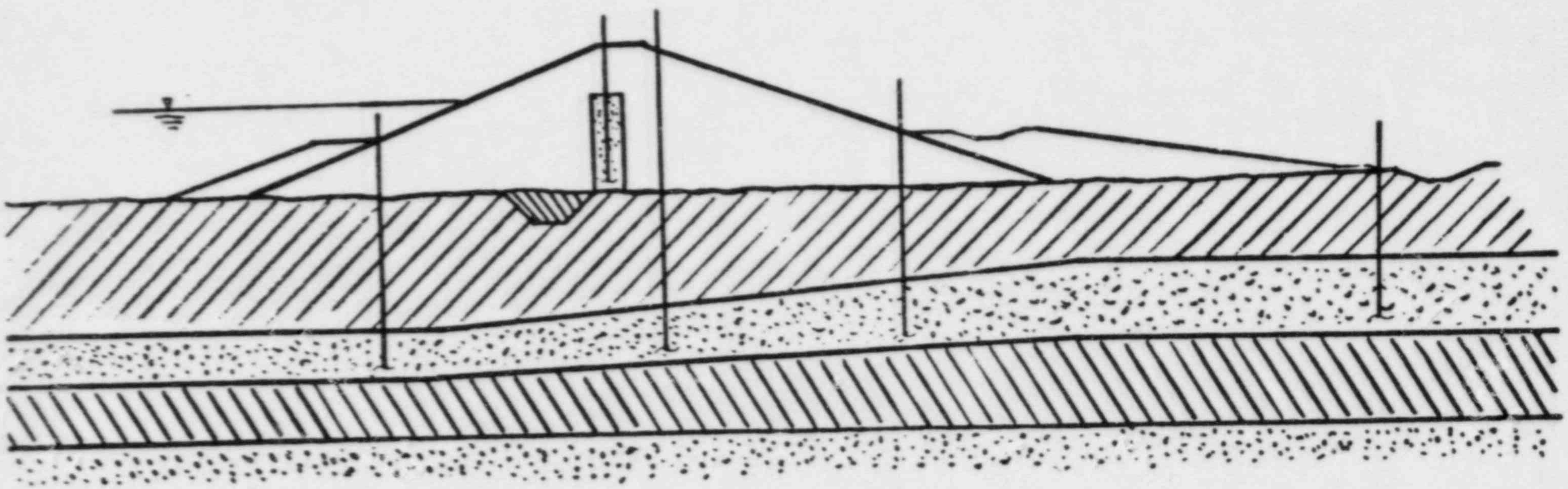
CROSS-SECTION MCR EMBANKMENT



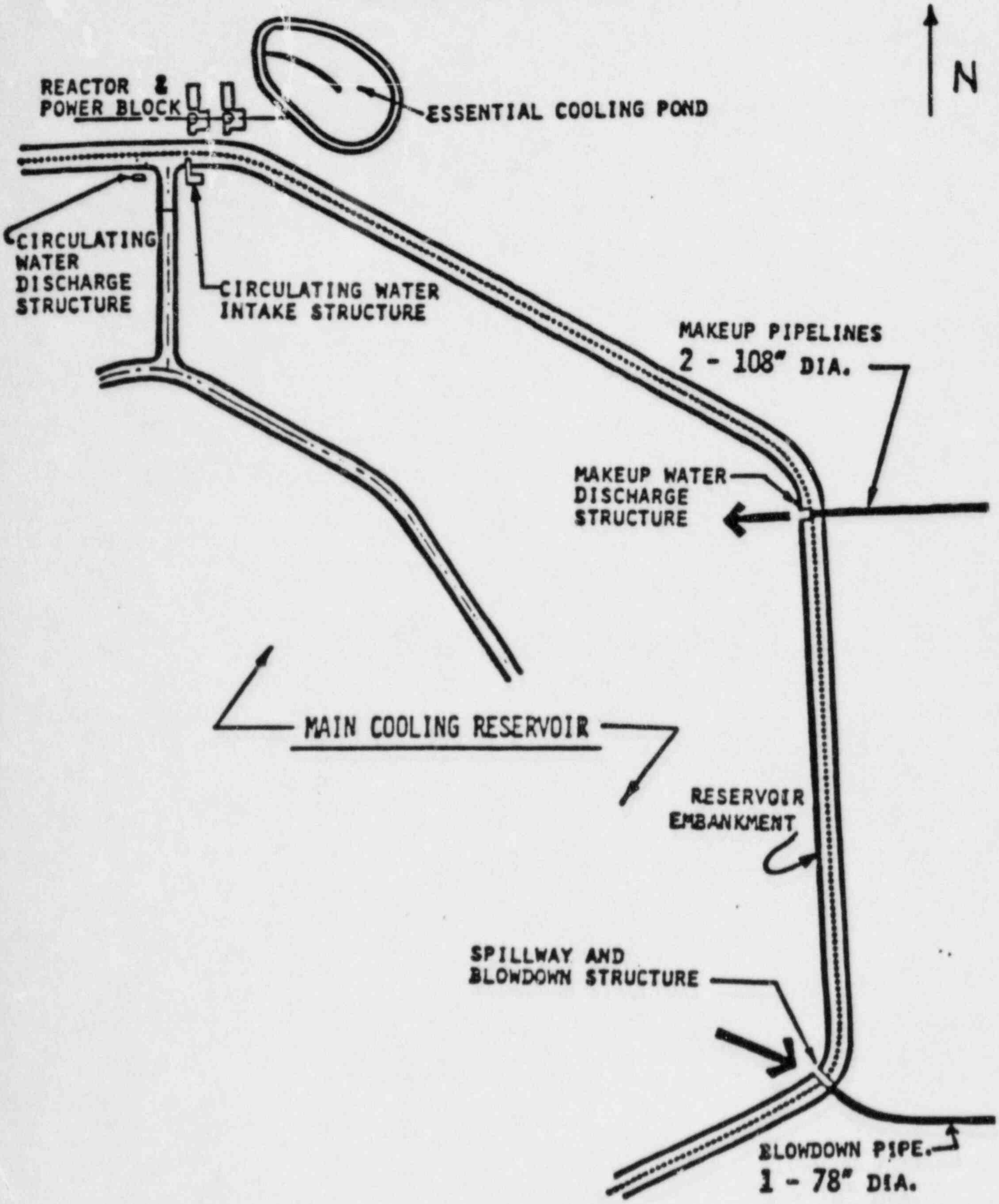
CROSS-SECTION MCR EMBANKMENT AND FOUNDATION



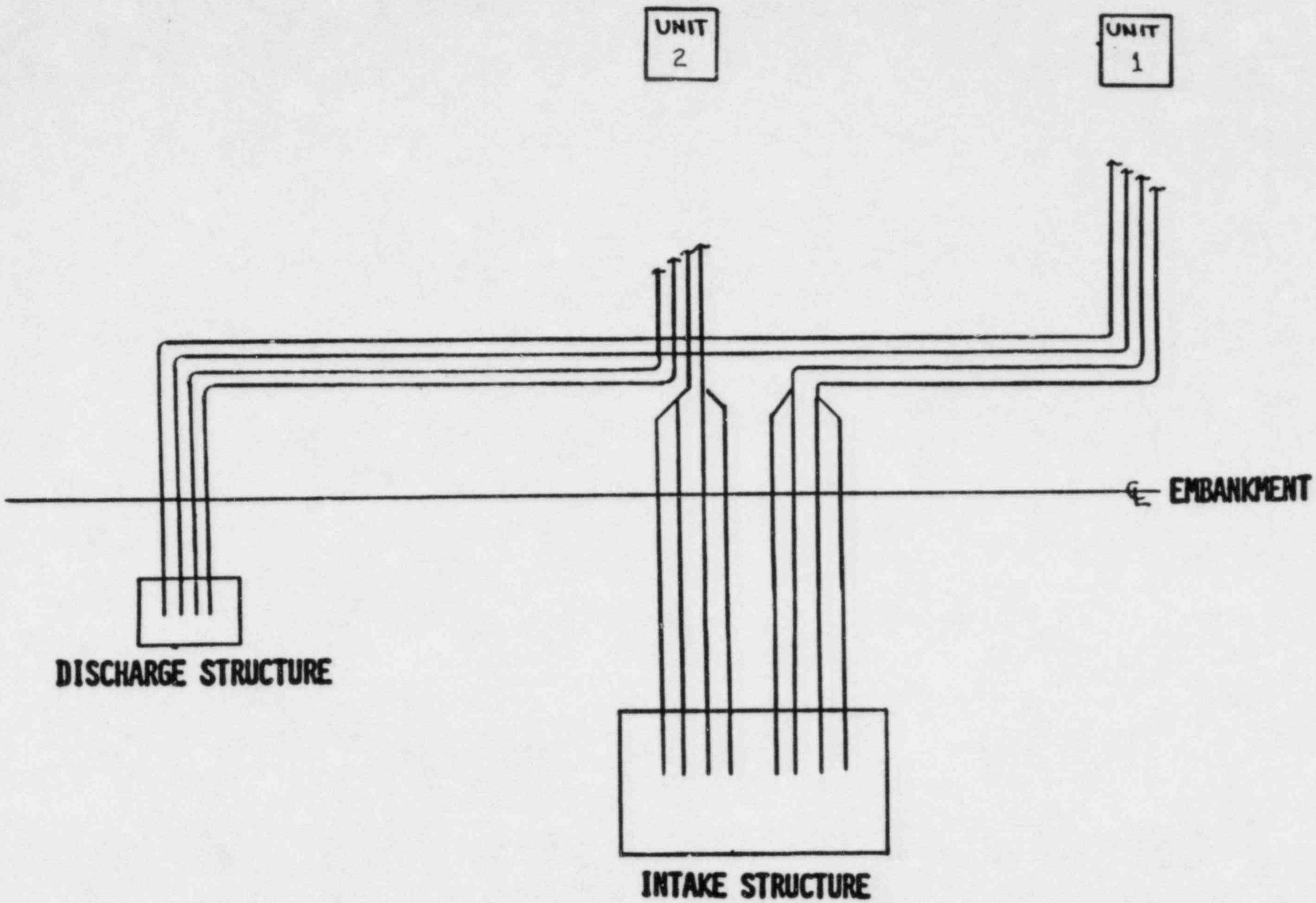
PIEZOMETER PLACEMENT



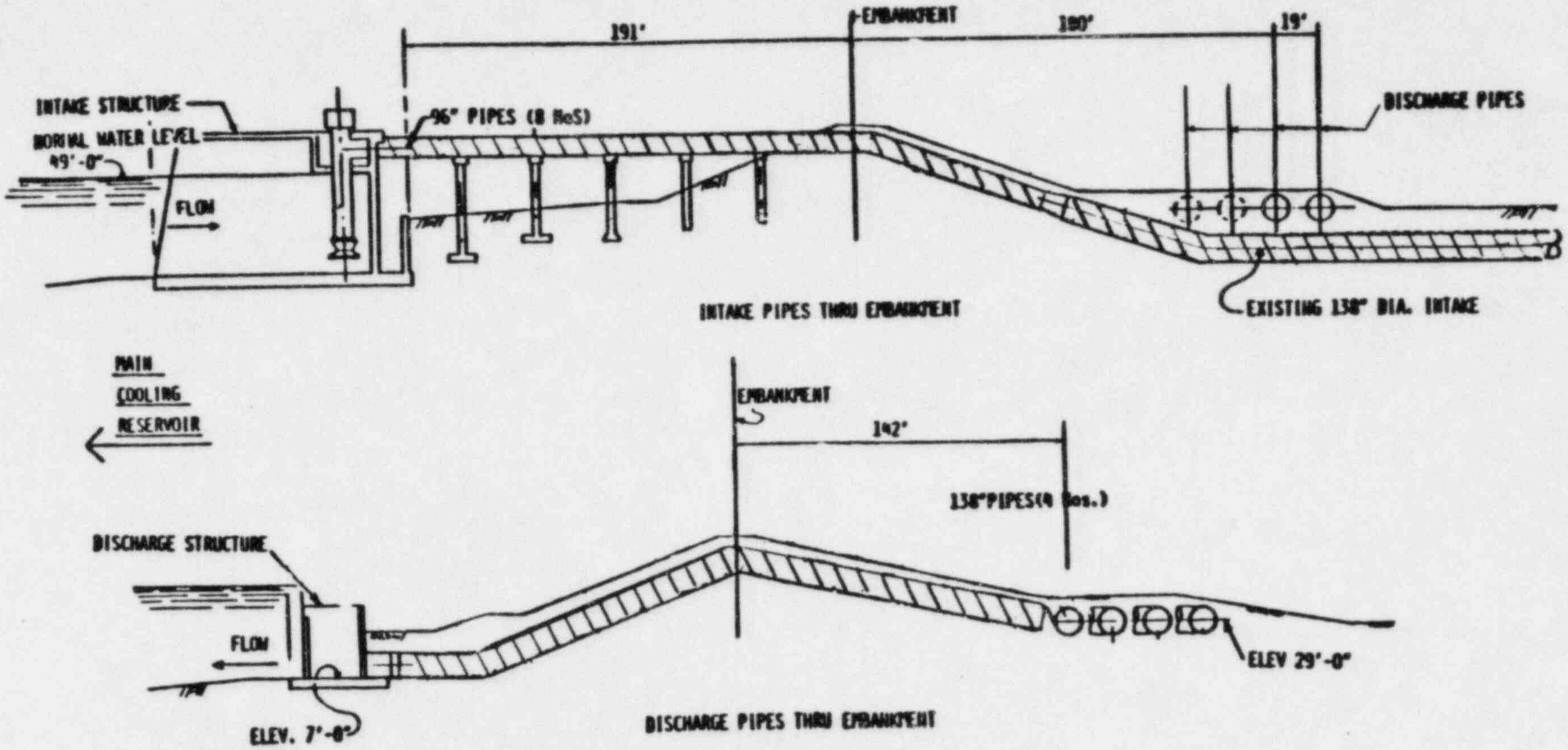
GENERAL LAYOUT



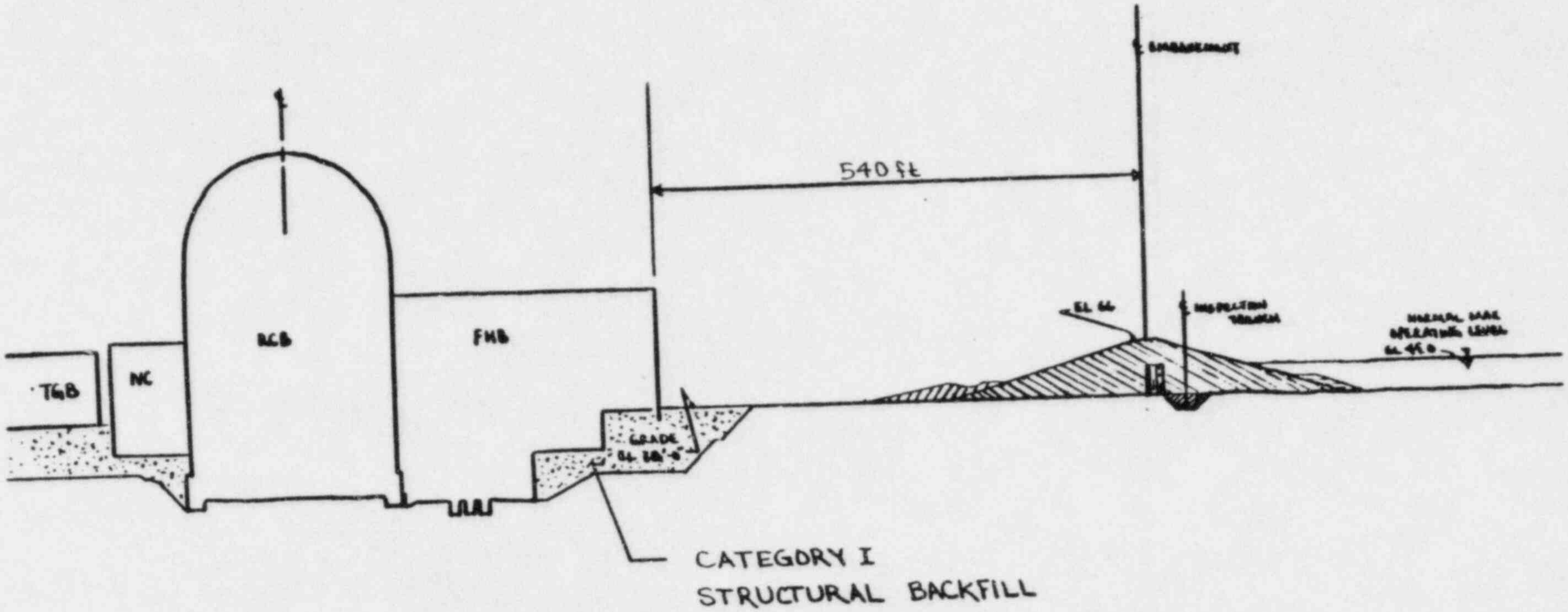
CIRCULATING WATER INTAKE AND DISCHARGE PIPE PENETRATION THRU EMBANKMENT



CROSS SECTION OF INTAKE AND DISCHARGE PIPE PENETRATION THRU EBANKMENT



CROSS-SECTION OF EMBANKMENT AND POWER BLOCK



Walter Ferris Slides

DESIGN VERIFICATION

- EMBANKMENT PENETRATIONS
- STABILITY ANALYSES
- POTENTIAL DISPERSION OF CLAYS
- UNDERSEEPAGE CONTROL

TECHNICAL ISSUES

- EMBANKMENT PENETRATIONS
- EMBANKMENT STABILITY
  - STEADY STATE SEEPAGE
  - SEISMIC STABILITY
  - RAPID DRAWDOWN
- FOUNDATION STABILITY
- UNDERSEEPAGE CONTROL
- EMBANKMENT FREEBOARD
- CONSTRUCTION QUALITY

SOUTH TEXAS PROJECT

PIPE PENETRATIONS

Handouts by Harza at 4/2/85 MCR Mtg.

MCR EMBANKMENT POWER BLOCK AREA
CIRCULATING WATER PIPE PENETRATIONS

GENERAL FEATURES:

- AWWA C-301 PRESTRESSED CONCRETE EMBEDDED STEEL CYLINDER PIPE
- BELL AND SPIGOT JOINTS WITH GASKETS
- TIED JOINTS FOR THRUST RESTRAINT
  - NO THRUST BLOCKS
  - WELDED OR HARNESSED JOINTS MAKE PIPE ITSELF A VIRTUAL ANCHOR FOR THRUST RESTRAINT AT BENDS

PENETRATION CONFIGURATIONS

TWO GENERAL AREAS:

- PIPE PENETRATIONS THROUGH HIGH PART OF EMBANKMENT
- PIPES BURIED IN OR BENEATH EMBANKMENT TOE BERM

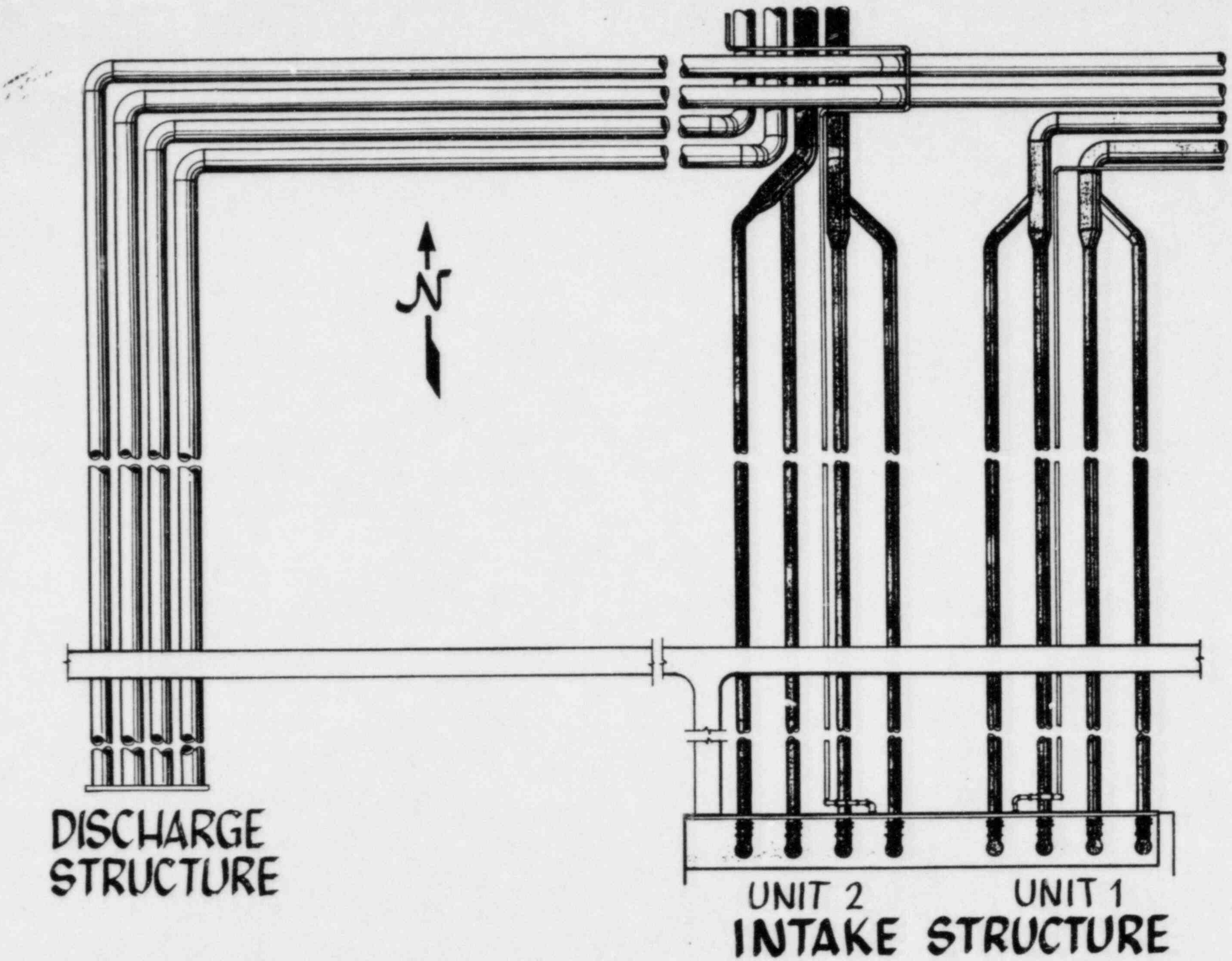
SPECIFIC FEATURES

INTAKE FEATURES

- 4 PAIRS 96-INCH DIAMETER PIPES
- PAIRS MERGE @ TOE TO FORM 138-INCH DIAMETER
- WYE AND INCREASER @ TOE IS ALL WELDED

DISCHARGE

- 4 - 138-INCH DIAMETER PIPES


N
DISCHARGE STRUCTURE
UNIT 2
UNIT 1
INTAKE STRUCTURE

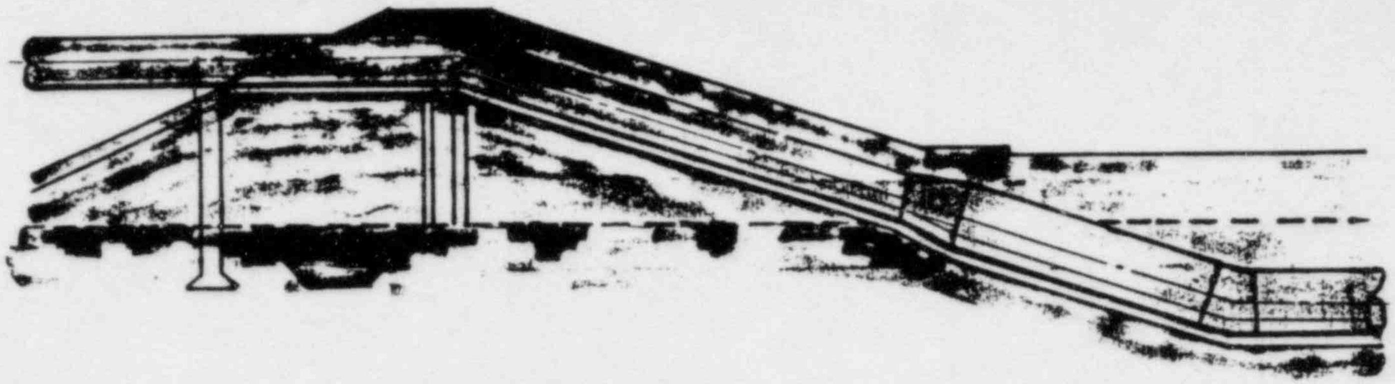
DESIGN CONDITIONS

- SEEPAGE FROM RESERVOIR ALONG SAND PIPE BEDDING
- JOINT OPENING DUE TO:
  - EMBANKMENT SETTLEMENT
  - HYDRAULIC THRUST
- LACK OF INSPECTABILITY - PIPES ARE BURIED

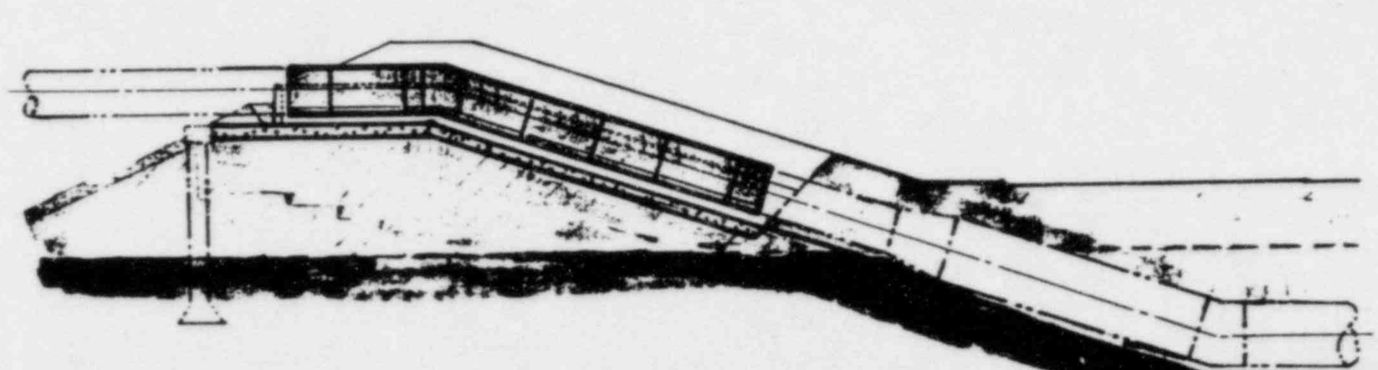
RESULTS OF DESIGN REVIEW

- JOINT OPENING DUE TO SETTLEMENT OR HYDRAULIC FORCES UNLIKELY
- WATERTIGHTNESS NOT GUARANTEED
- LACK OF INSPECTABILITY-MAJOR PROBLEM
- INCREMENTAL FLEXIBILITY OF EXISTING PIPES POSITIVE DESIGN IF INSPECTABLE

INTAKE STRUCTURE

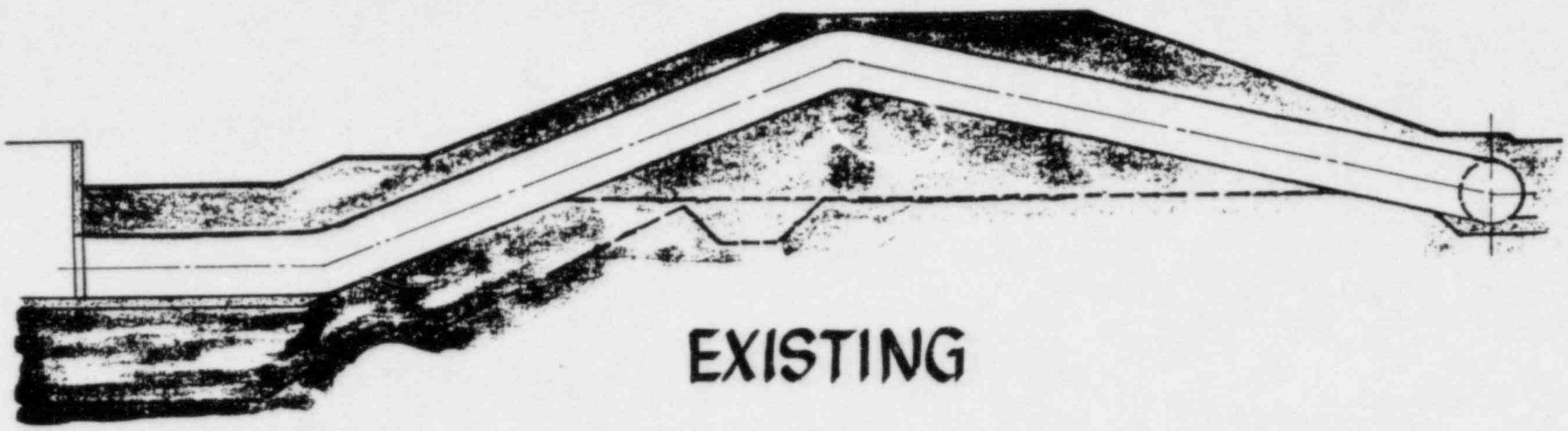


EXISTING

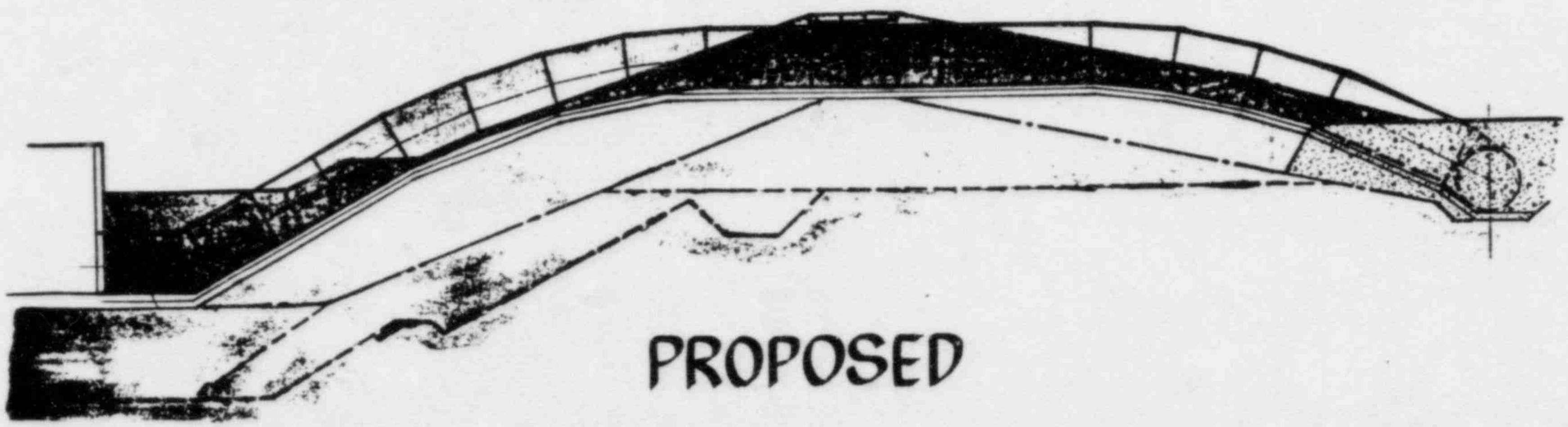


PROPOSED

DISCHARGE STRUCTURE



EXISTING



PROPOSED

PIPES BURIED BENEATH TOE

- INITIAL OVERALL SYSTEM HYDROTESTS COMPLETE
- INDIVIDUAL JOINTS TO BE REPAIRED BY:
  - WELDING
  - INDIVIDUAL INTERNAL ELASTOMERIC LINERS
- PIEZOMETERS INSTALLED INTO FILL AROUND PIPES

MCR EMBANKMENT - POWER BLOCK AREA

EMBANKMENT STABILITY

INITIAL HARZA EFFORTS :

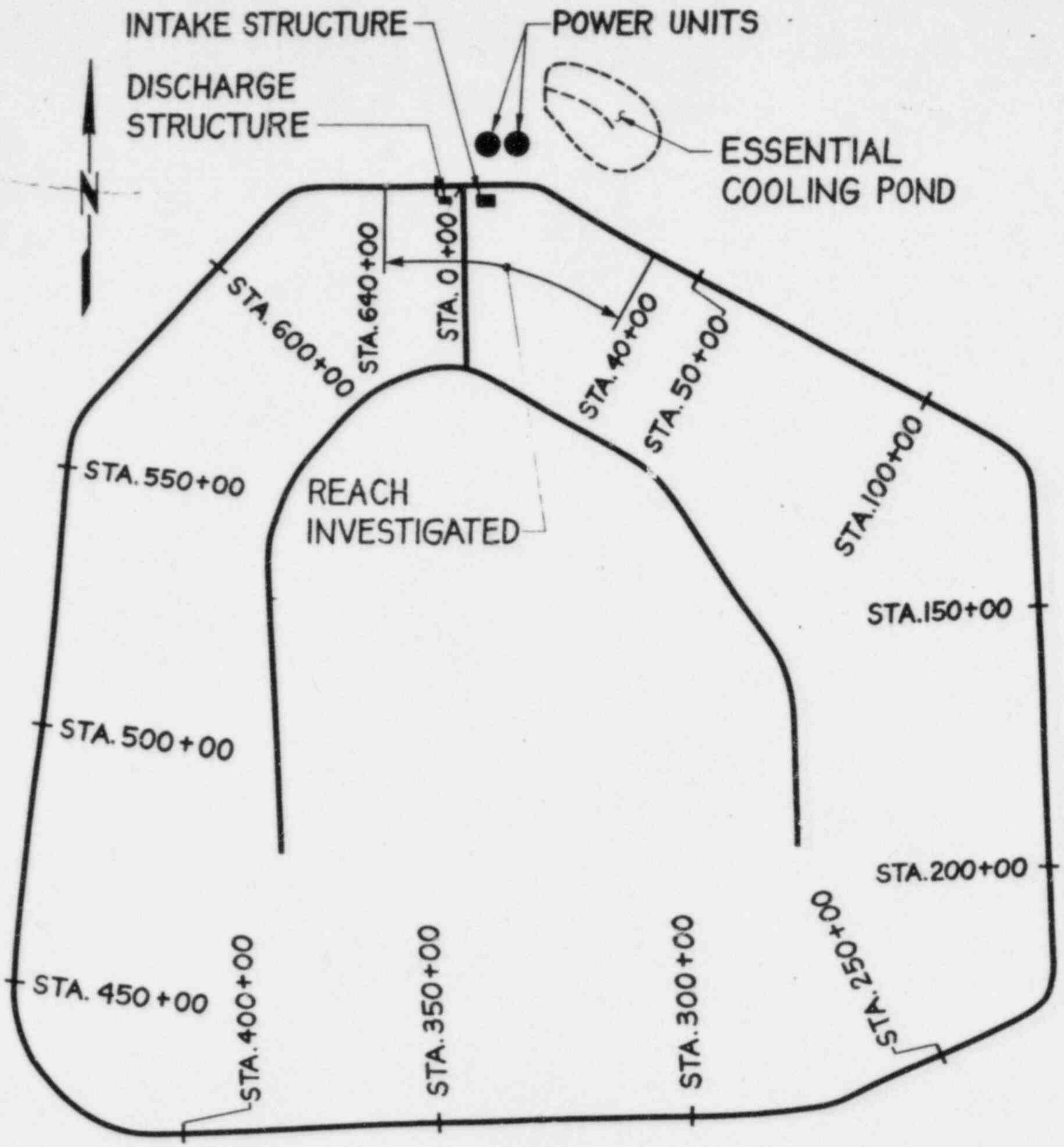
- CHECK STABILITY USING SOILS PARAMETERS RECOMMENDED BY CONSULTANTS

- MAKE CERTAIN PENETRATION RECONFIGURATION DOES NOT COMPROMISE STABILITY

EXPANDED SCOPE :

- CHECK STABILITY OF AREA OPPOSITE POWER BLOCK

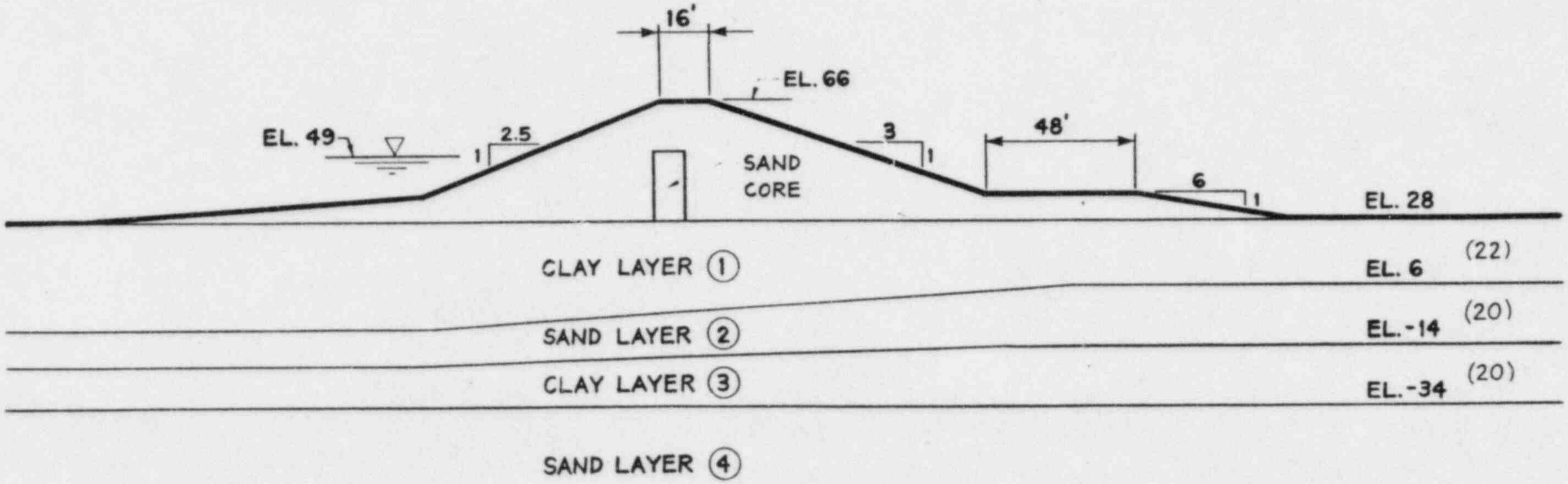
- DOCUMENT SOILS CONDITIONS AND STRENGTH PARAMETERS WITH DRILLING AND TESTING PROGRAM



BOREHOLE NO.	STATION	LOCATION
BH-1, 1A, 1B, 1C, 1D, 1E, 1F	642+50	D/S
BH-2	647+50	CREST
BH-3, 3A	652+50	U/S
BH-4	637+50	D/S
BH-5, 5A, 5B	0+00	D/S
BH-6	2+50	U/S
BH-7, 7A	7+50	CREST
BH-8	12+50	D/S
BH-9	17+50	U/S
BH-10, 10A, 10B	22+50	D/S
BH-11	27+50	CREST
BH-12	32+50	U/S
BH-13	37+50	D/S
BH-14	42+50	U/S
BH-39	20+00	CREST

TOTAL NO HOLES DRILLED = 27

GENERALIZED STRATIGRAPHY - POWER BLOCK AREA
(STA. 20+00 - SECTION FOR ANALYSES)



SUMMARY OF SOILS TESTING

CALIBRATED SPT TESTS	199
UNDISTURBED SAMPLES	90
CONSOLIDATION TESTS	
Clay Layer 1	8
Embankment	2
DRAINED DIRECT SHEAR	
Clay Layer 1	26
Embankment	9
TRIAXIAL SHEAR TESTS ($\overline{CIU}$)	
Clay Layer 1	30
Embankment	13
TRIAXIAL SHEAR TESTS (CID)	
Clay Layer 1	9
Embankment	—

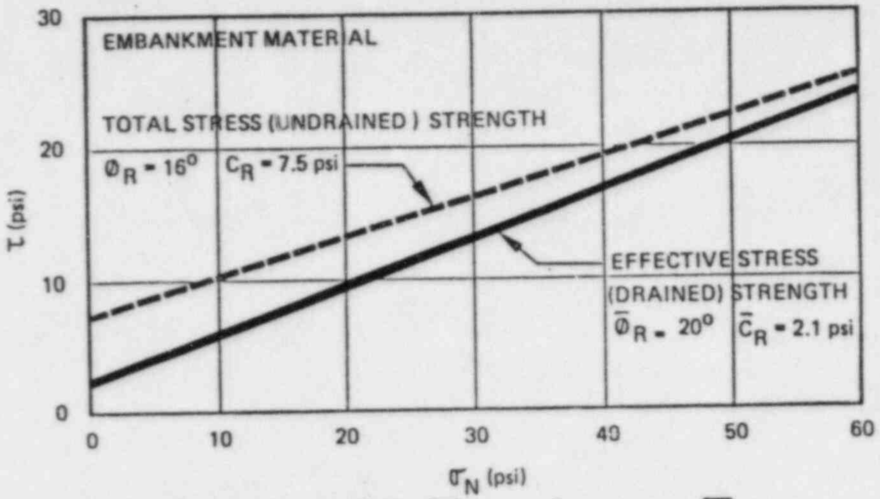
LOADING CONDITIONS

- STEADY STATE SEEPAGE
- RAPID DRAWDOWN (EL.49 TO SPILLWAY CREST @ EL.39
- SAFE SHUTDOWN EARTHQUAKE (a_{max} = 0.1 g)

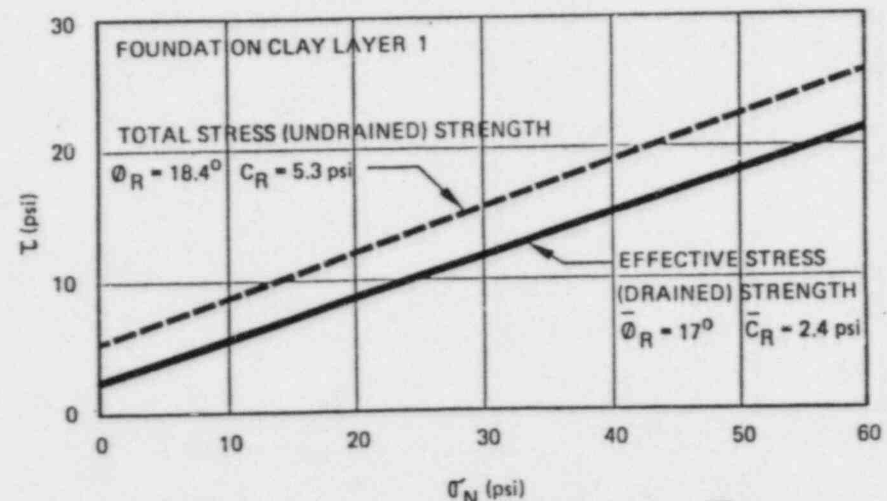
TEST RESULTS - RECOMMENDED PARAMETERS FOR USE IN ANALYSES
(STEADY STATE, RAPID DRAWDOWN, SEISMIC)

	$\bar{\phi}$, deg	$\bar{c}$, psf
Embankment	20	300
Sand Core	35	0
Clay Layer 1	17	350
Sand Layer 2	35	0
Clay Layer 3	17	350

COMPARISON EFFECTIVE STRESS & TOTAL STRESS STRENGTHS



Strength Envelopes For Embankment Material

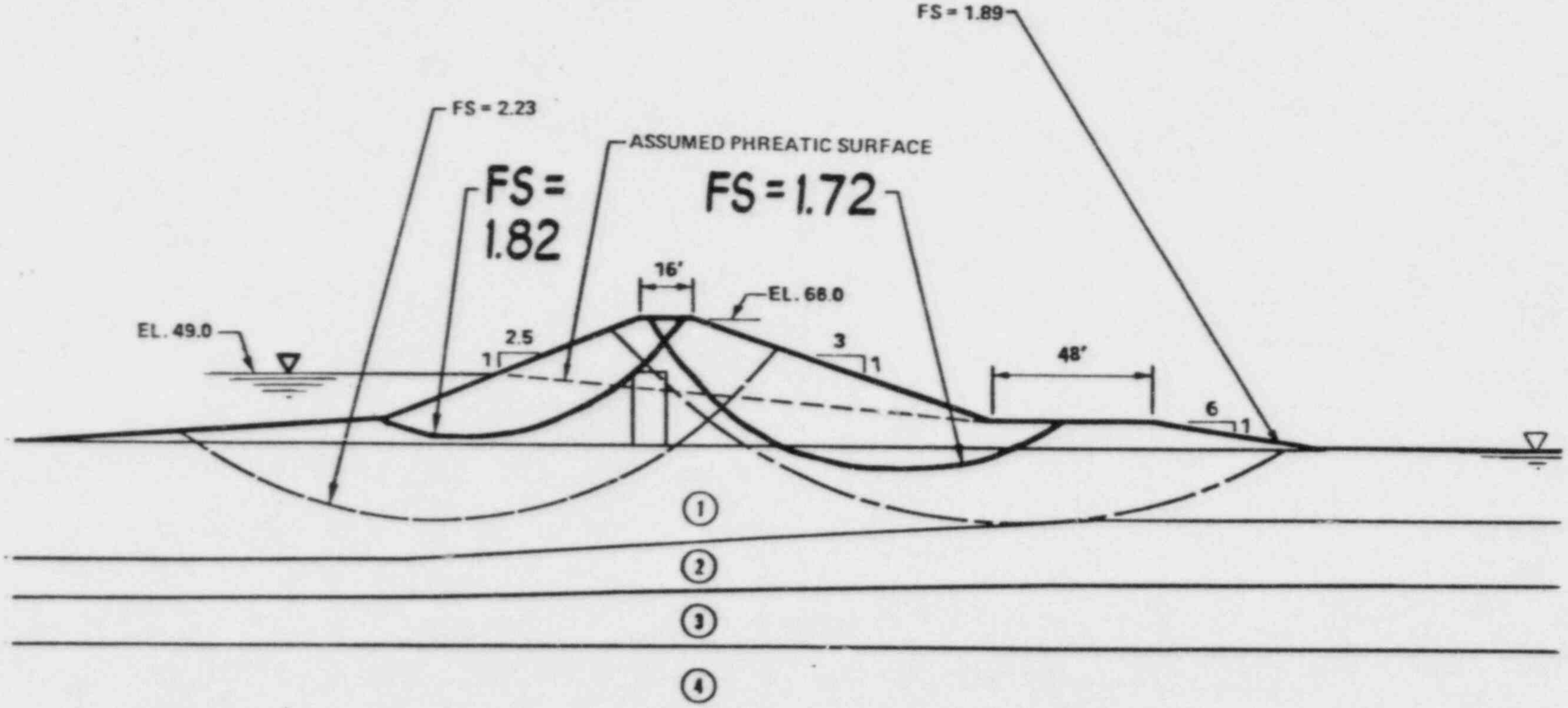


Strength Envelopes For Foundation Clay Layer 1

OTHER PHYSICAL PROPERTIES

	$\overline{W}_n$,%	$\overline{W}_l$,%	$\overline{W}_p$,%	$\overline{\gamma}_d$,PCF	-#$\overline{200}$,%	OCR	K ×10^{-7} ft/min
Embankment	21	58	20	105	92	4	6.5-35
Clay Layer 1	25	56	20	99	92	1.7-9.4	0.05
Sand Layer 2	23	0-23	NP-20	-	63	-	-
Clay Layer 3	28	55	22	96	85	-	-

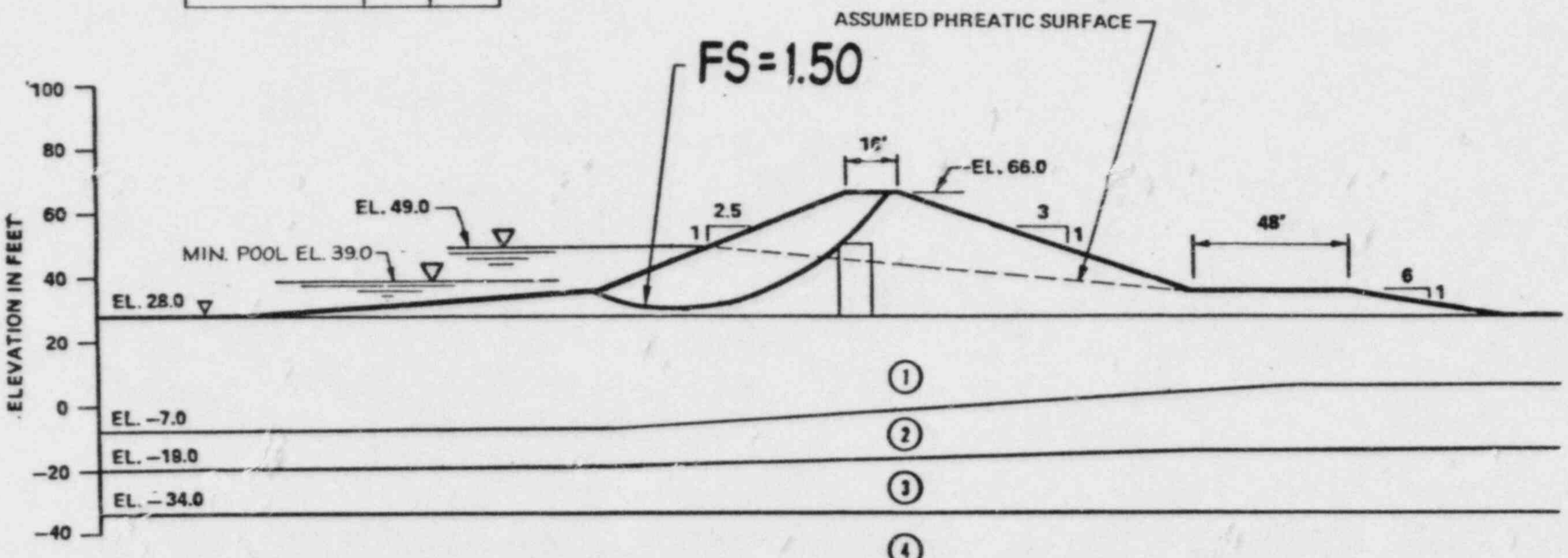
SUMMARY OF STATIC STABILITY ANALYSES POWER BLOCK AREA



DRAWDOWN CASE

POOL EL. 49 TO 39

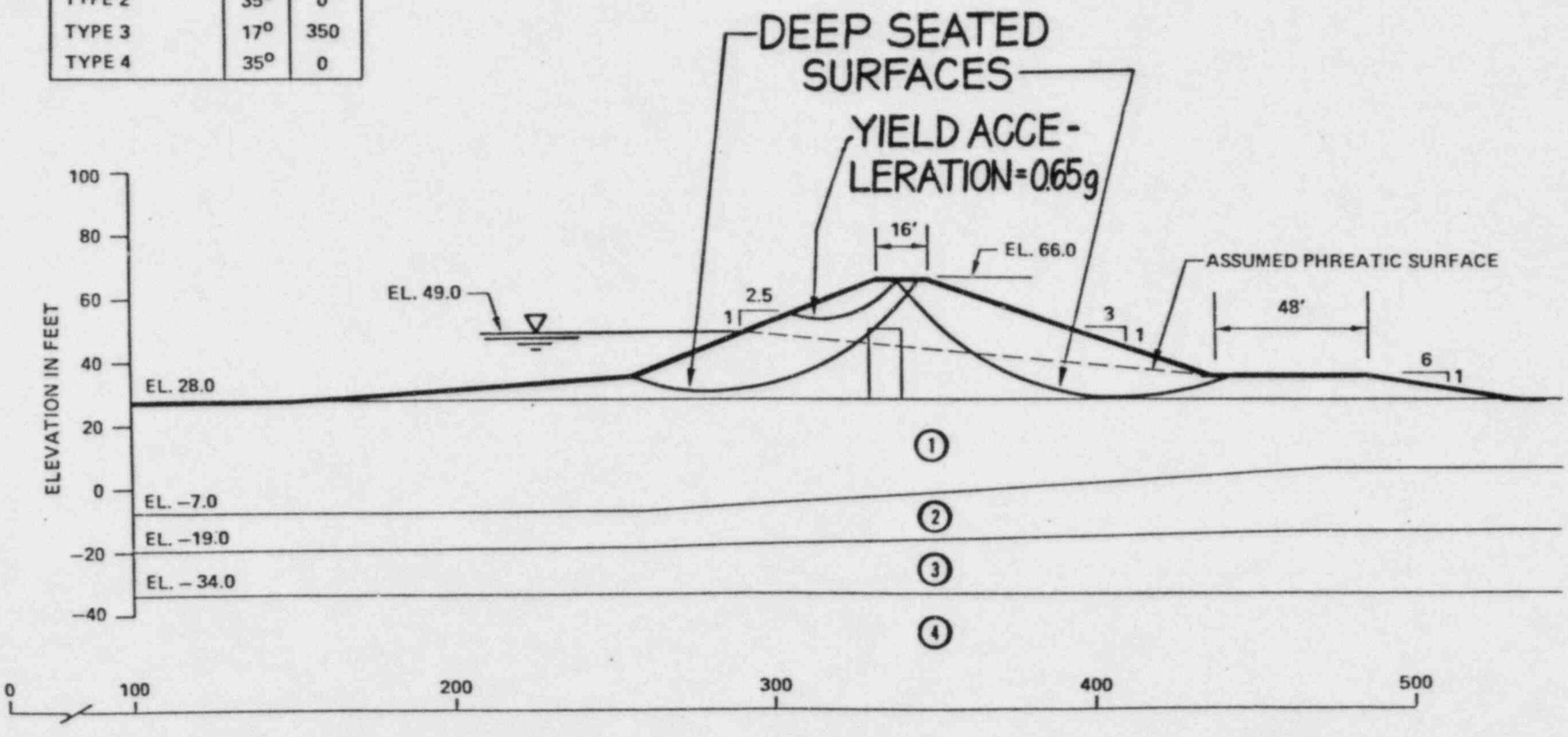
MATERIAL	$\bar{\phi}$	$\bar{C}$, psf
EMBANKMENT	20°	300
SAND CORE	35°	0
TYPE 1	17°	350
TYPE 2	35°	0
TYPE 3	17°	350
TYPE 4	35°	0



Note that critical failure plane for rapid drawdown (F.S. = 1.50) is wholly within embankment

SEISMIC STABILITY EVALUATION

MATERIAL	$\bar{\phi}$	$\bar{C}$, psf
EMBANKMENT	20°	300
SAND CORE	35°	0
TYPE 1	17°	350
TYPE 2	35°	0
TYPE 3	17°	350
TYPE 4	35°	0



DISPLACEMENT ANALYSIS FOR SSE EVENT ($a_{max} = 0.1g$) POWER BLOCK AREA

DEEP SEATED SURFACE

	Yield Acceleration, g	
Section	Upstream	Downstream
Sta. 20+00	0.22	0.25
Intake Structure	0.25	0.25
Discharge Structure	0.17	0.11

No deformation

SURFACE HIGHER IN THE EMBANKMENT

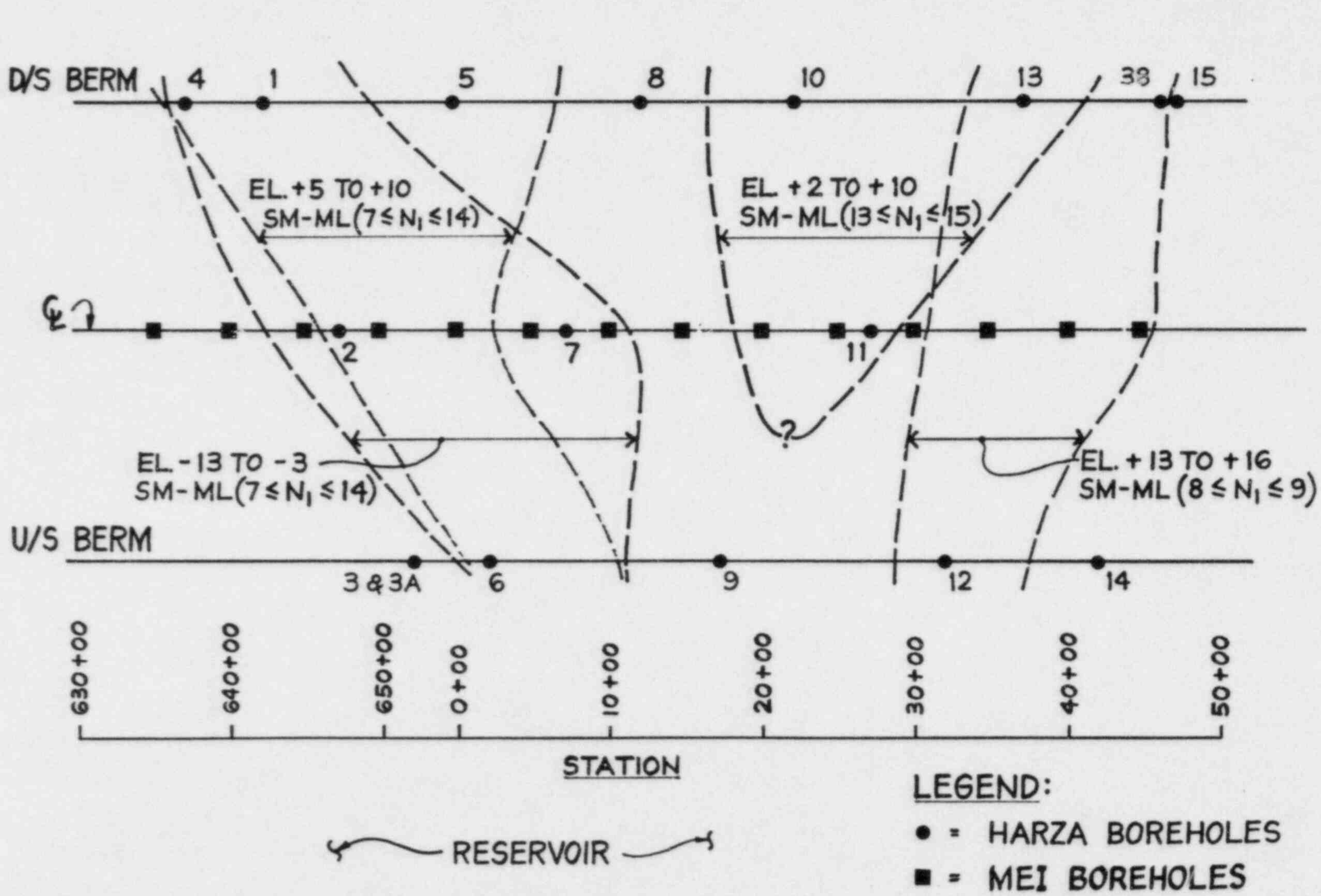
Critical Sliding Mass near Crest

Yield Acceleration = 0.65 g.

Max. Avg. Acceleration = 0.34

No Deformation

IDENTIFICATION OF LOW BLOWCOUNT MATERIAL AND INTERPRETATION OF DATA



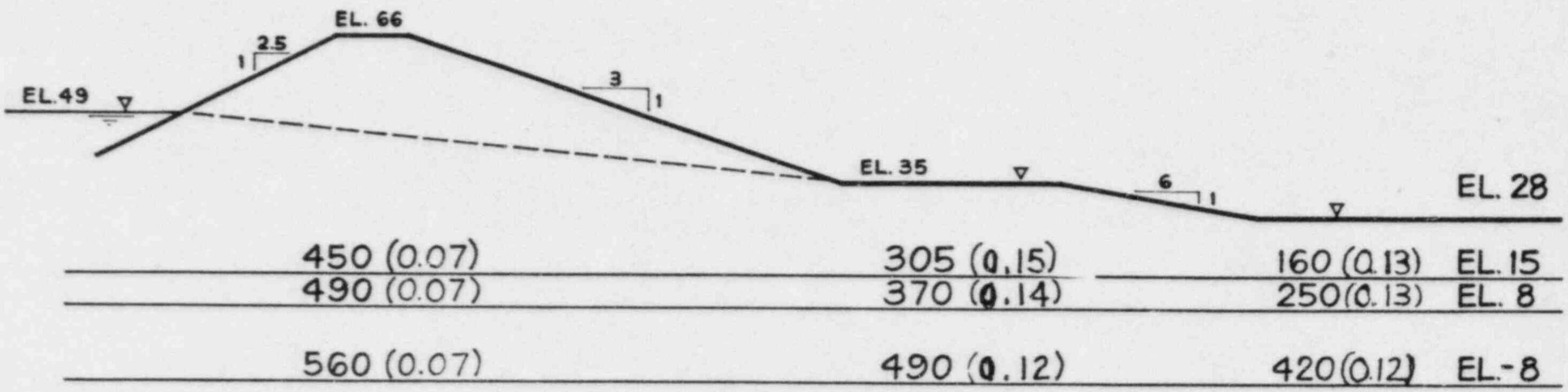
CYCLIC STRENGTH

Station	Elevation	Material Type	-#200 Sieve	$\bar{N}_{1(60)} - \frac{1}{2}\sigma$	Cyclic Strength* (τ_{ave}/σ_v')	
					15% Fines	35% Fines
637± to 10±	-13 to -3	SM - ML	90.0	9.8	0.21	0.25
637± to 10±	+5 to +10	SM-ML	58.0	9.3	0.20	0.24
16± to 40±	+2 to +10	SM-ML	53.0	13.9	0.26	0.32
30± to 45±	+13 to +16	SM-ML	36.0	8.1	0.18	0.22

* Based on figure published in UCB/EERC Report No. 84/15 October 1984 (Seed, Tokimatsu, Harder & Chung)

CYCLIC STRESSES

- DYNAMIC RESPONSE ANALYSIS BY SHAKE PROGRAM



LEGEND:

Max. Cyclic Shear Stress $\tau_{max.}$ (psf) — 450 (0.07) — Corrected Uniform Cyclic Stress Ratio $\frac{\tau_{av}}{\sigma_v'} \cdot \frac{1}{C_{\sigma'}} \cdot \frac{1}{C_{\alpha}}$

Max. Shear Stress Under D/S Berm - Interpolated

FACTOR OF SAFETY AGAINST LIQUEFACTION

Station	Elevation	$F.S. = \frac{\text{Cyclic Strength}}{\text{Cyclic Stress}}$		
		Crest	D/S Berm	Free Field
		Fines 15% - 35%	Fines 15% - 35%	Fines 15% - 35%
637± to 10±	-13 to -3	3.0 - 3.6	1.8 - 2.1	1.8 - 2.1
637± to 10±	+ 5 to +10	2.9 - 3.4	1.4 - 1.7	1.5 - 1.8
16± to 40±	+ 2 to +10	3.7 - 4.6	1.9 - 2.3	2.0 - 2.5
30± to 45±	+13 to +16	2.6 - 3.1	1.2 - 1.5	1.4 - 1.7

CONSTRUCTION QUALITY

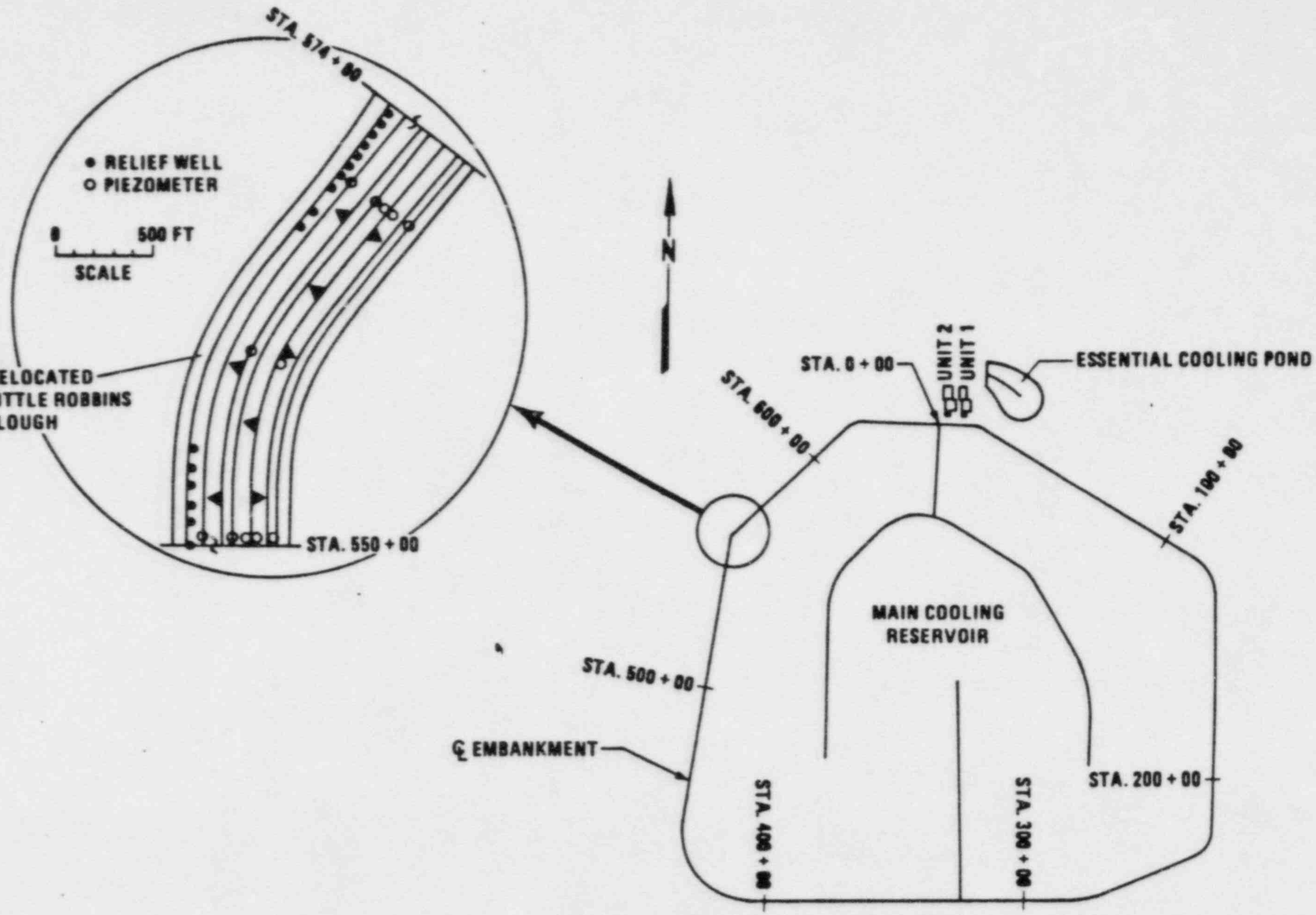
CONSTRUCTION QUALITY REVIEW

- VAULT RECORDS (BROWN & ROOT)
- SPECIFICATIONS
- TESTING PROCEDURES
- SAMPLED 5% OF RECORDS TO FOLLOW DOCUMENTATION

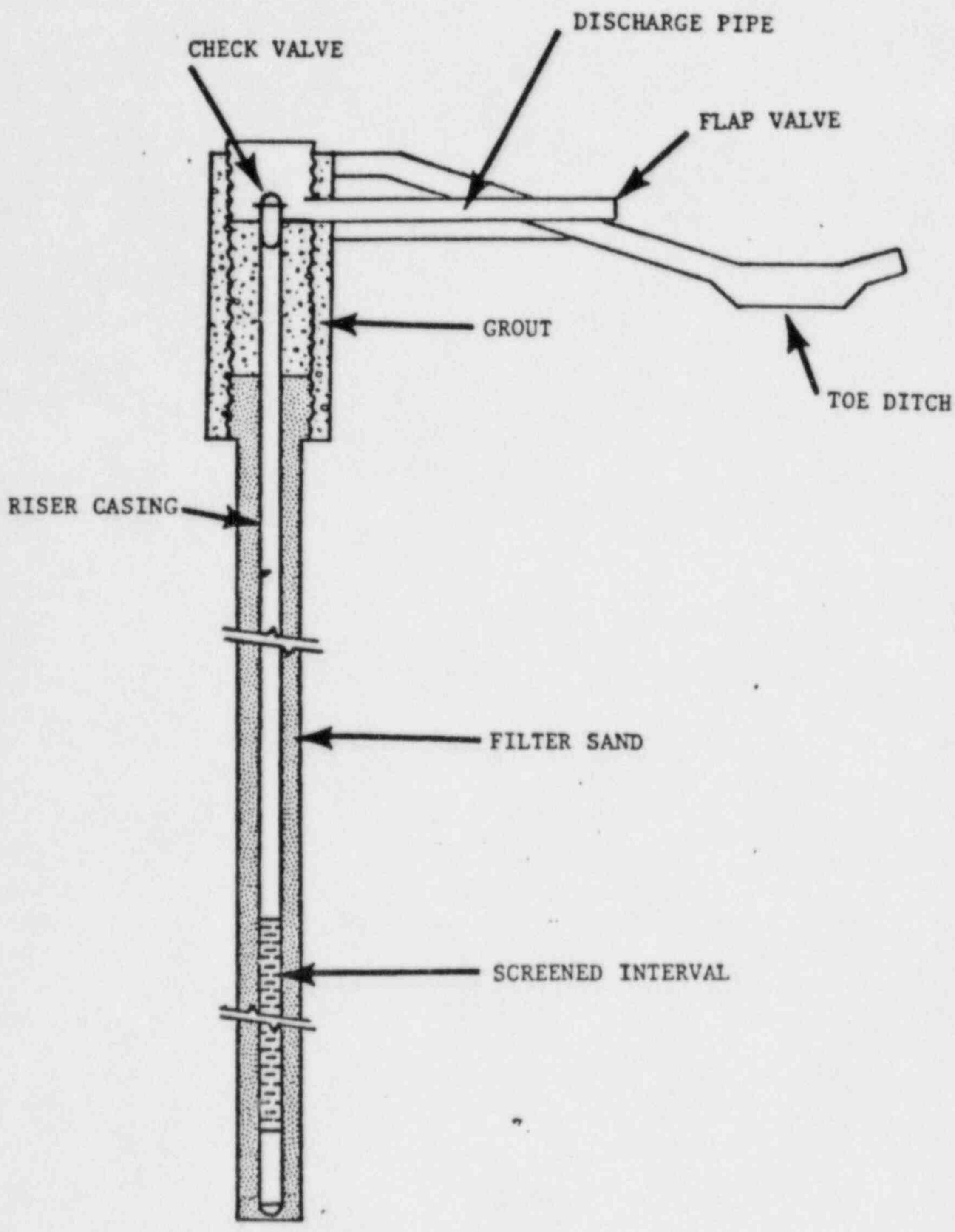
REVIEW RESULTS

- 1100 FIELD DENSITY TESTS (AVG. 1 PER 1000 C.Y.)

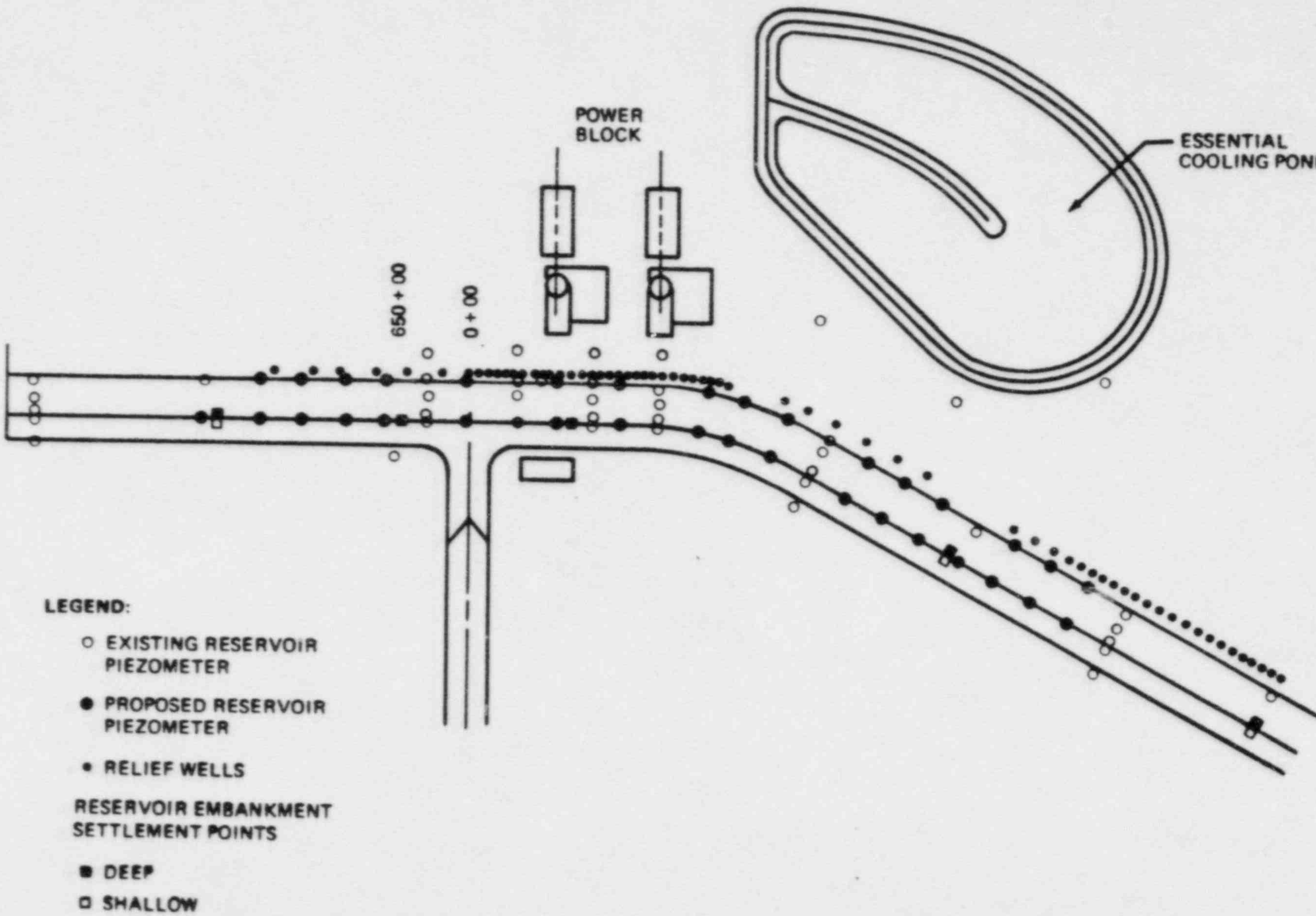
- 33 PROCTOR DENSITY TESTS
- AVERAGE COMPACTION: 98% STANDARD PROCTOR
- 7.5% OF FIELD TESTS FAILED; AREAS WERE REWORKED AND RETESTED



Handouts by R. Talmage + D. Hollingshead



TYPICAL RELIEF WELL

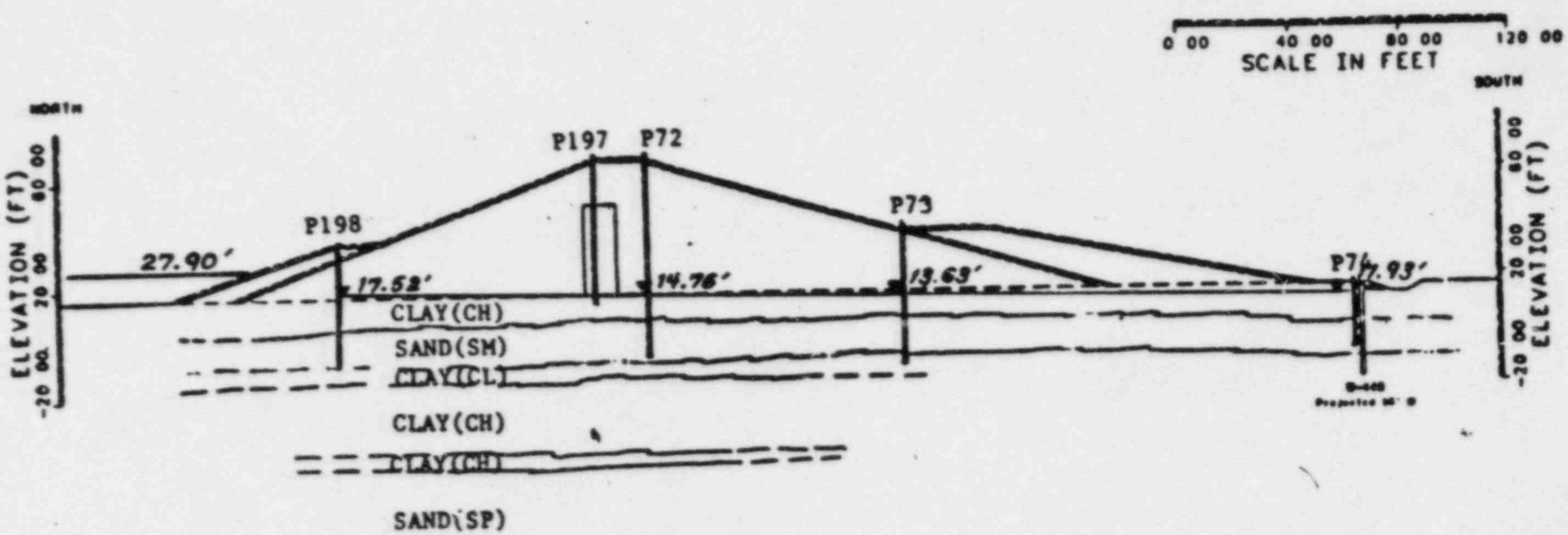

POWER BLOCK
ESSENTIAL COOLING POND
650 + 00
0 + 00
LEGEND:
EXISTING RESERVOIR PIEZOMETER
PROPOSED RESERVOIR PIEZOMETER
RELIEF WELLS
RESERVOIR EMBANKMENT SETTLEMENT POINTS
DEEP
SHALLOW

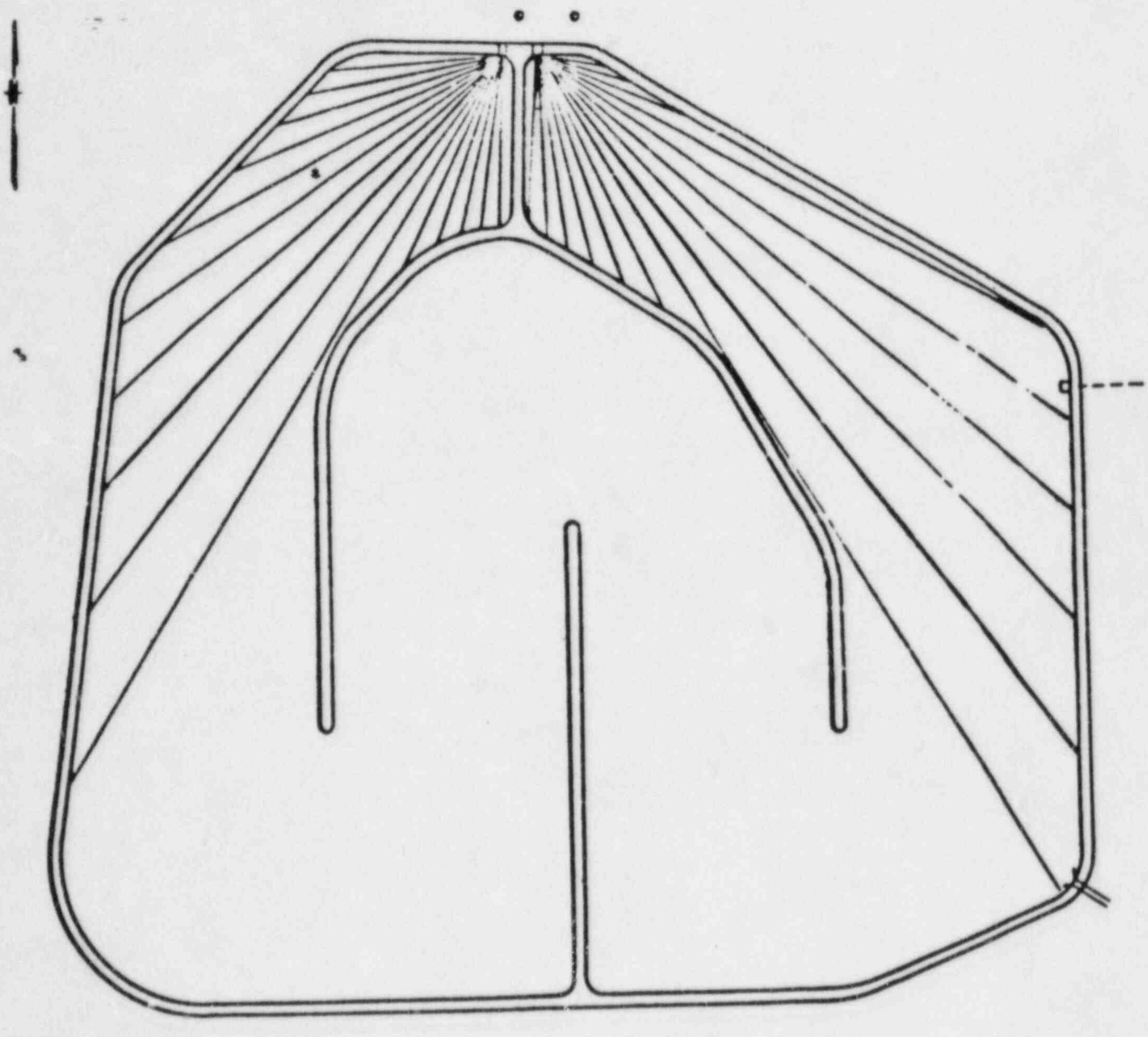
PROFILE 18

EMBANKMENT STATION 320+00

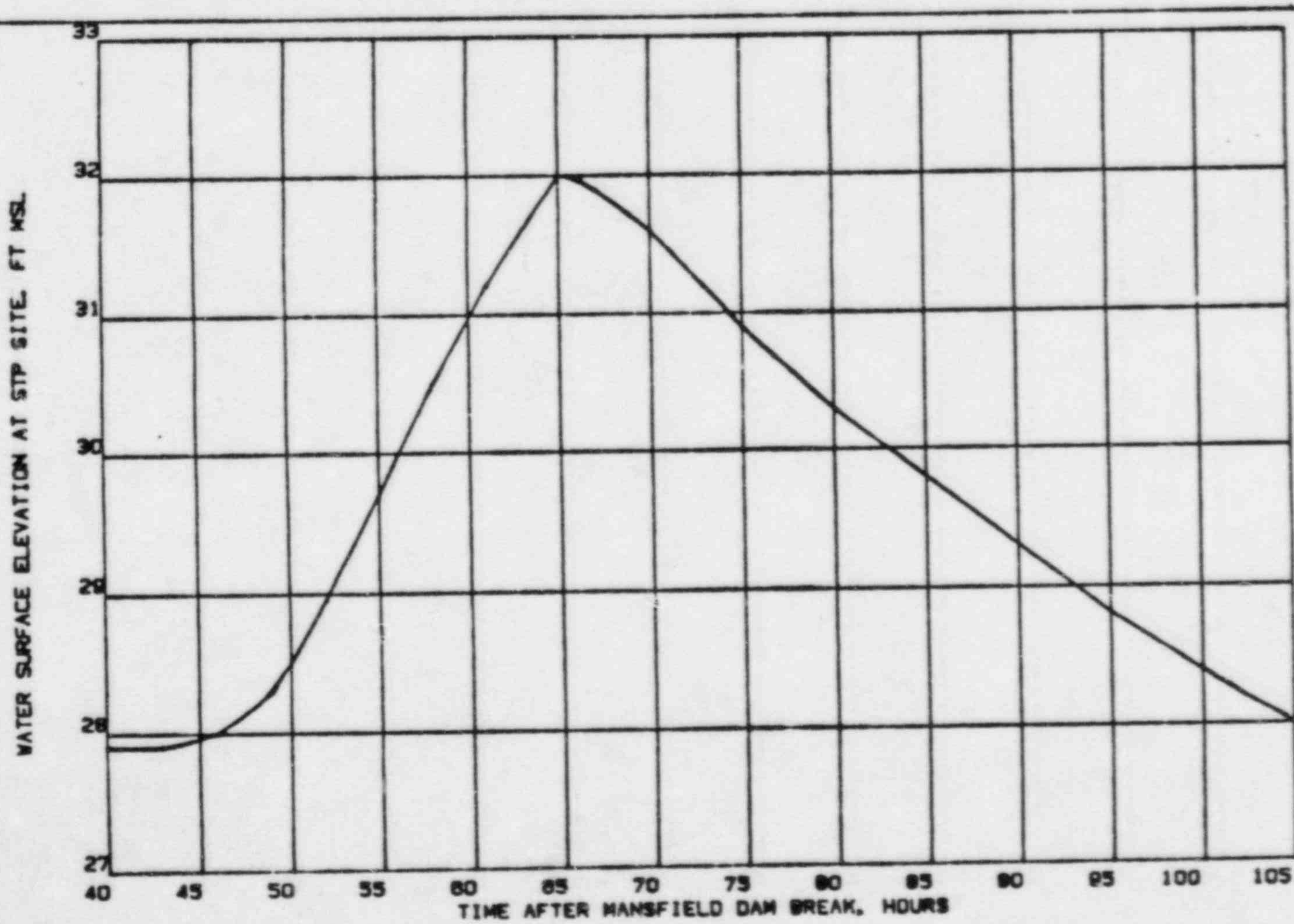
SOUTH TEXAS PROJECT

14926-001





SOUTH TEXAS PROJECT
MAIN COOLING RESERVOIR
EFFECTIVE FETCH DETERMINATIONS
INTAKE: 1.52 mi   DISCHARGE: 1.28 mi



STAGE HYDROGRAPH AT STP SITE FOLLOWING UPRIVER DAM BREAKS