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Date: 4/16/2007 6:18:10 PM
Subject: RAI Letter #6 Hydrology Response 2 of 4

<<AR-07-0639_2 of 4.pdf>>

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From: "Davis, James T." <JTDAVIS@southernco.com>

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LIST OF FIGURES, continued

LIST OF TABLES

FINAL REPORT ON DEWATERING AND REPAIR OF EROSION IN CATEGORY I BACKFILL IN POWER BLOCK AREA

I. INTRODUCTION AND PURPOSE

Heavy rainfall in early November, 1979, resulted in erosion of Category I backfill and caused a re-evaluation of groundwater controls. On November 14, 1979, it was reported to the Nuclear Regulatory Commission (NCR) that a potential reportable item under 10CFR50.55(e) existed at Plant Vogtle concerning dewatering and erosion of backfill. Subsequent communications to the Nuclear Regulatory Commission culminated in a summary submittal (Reference 1) on January 8, 1980, and a presentation of the summary to the Nuclear Regulatory Commission on January 9, 1980, in Bethesda, Maryland.

The report outlined steps that had been initiated subsequent to the erosion to repair the affected areas and to facilitate resumption of backfilling operations in the power block area. Also included in the report were a preliminary engineering evaluation of the affected and adjacent areas and recommended methods of repair. Following submission of the report to the Nuclear Regulatory Commission and concurrence by that agency with the proposed measures, backfill repair work was accomplished in all areas subjected to erosion. Implementation of the backfill repair procedures was started toward the end of January, 1980, and completed in August, 1980. During the period of the backfill repair operation, a Bechtel Power Corporation geotechnical engineer was on site to provide surveillance of the overall erosion repair and groundwater program. He also assisted in the interpretation of field test data and repair procedures. In addition, Bechtel engineering personnel and a Bechtel consultant made periodic site visits to review the repair work.

This document is written to describe the actual repair work, the associated testing, and the final engineering evaluation of the integrity of the adjacent structures. Existing and future erosion and groundwater control measures are also described.

II. EVALUATION OF TESTING AND REPAIR

A. General

All erosion areas identified in the power block were repaired in accordance with the procedures specified in Reference 1, except where noted in Section II.C. In each case of variation from Reference 1, a description of the variation and technical justification for it is presented. Prior to backfilling, field and laboratory testing was performed in each area which provided the basis for determining the depth of disturbed zone and depth to competent existing backfill.

B. Field and Laboratory Testing

Field testing included the proving ring penetrometer, dynamic cone penetrometer, and sand cone density tests (ASTM D-1556). Laboratory testing consisted of the Modified Proctor compaction test (ASTM D-1557). All tests were performed in accordance with the procedures described in the Appendix to this Report.

Prior to testing, the dynamic cone penetrometer was calibrated against the Standard Penetration Test (SPT) for Category I backfill materials. A total of six SPT test borings were drilled in undisturbed Category I backfill to a maximum depth of 5-feet. SPT tests were performed continuously from the surface down to 5-feet in accordance with ASTM D-1586. Adjacent to the SPT test borings, a total of ten dynamic cone penetrometer tests were made at 6-inch intervals in holes drilled down to a maximum depth of 4-feet. The results of these tests are summarized in Table 1. Test results are shown in Figures 2 and 3. Based on these tests, the calibration ratio of the SPT resistance to the Dynamic cone penetrometer resistance is roughly 1 for the range of blowcounts recorded. No correlation tests were made for the proving ring penetrometer. The use of proving ring and dynamic cone penetrometers was limited only to a qualitative evaluation of the backfill compaction. These tests were used only to determine the depth of competent fill and were not intended to determine the percent compaction. Final control testing was done using the sand cone test method in conjunction with the laboratory Modified Proctor compaction test. However, based on the experience obtained from the use of the proving ring penetrometer, a reading of 2 or greater indicated that the sand cone test method would show a degree of compaction greater than 97 percent. This criterion was used to determine the depth of disturbed zone in Category I backfill slopes where it was not possible to perform sand cone density tests.

C. Evaluation of Specific Areas

1. Area between Control Building Electrical Shafts Units 1 and 2 and Turbine Building:

   Erosion in this vicinity was identified as Areas 1, 2, 3, 15, 16 and 18 respectively (Figure 1). Areas 1, 2, 3, 15 and 16 referred to erosion areas along the Turbine Building south slope; Area 18 referred to the area between the toe of the Turbine Building south slope and the edge of the Control Building shafts' mudslab. All these areas were repaired in accordance with the procedures specified in Reference 1.

   The Turbine Building slope was reworked to a minimum of 1.5 horizontal to 1.0 vertical and then gunited for erosion protection (see Section IV). This involved removal of a portion of the Turbine Building mudslab and some Turbine Building base slab steel reinforcement bars. After reshaping the slope, the minimum distance from the top of the slope to the nearest edge of the existing Turbine Building base mat was apprxoimately 19-feet. This was consistent with the minimum distance specified in Reference 1. Figure 4 shows a typical section of the reworked slope.

   In Area 18, the depth of disturbed zone, as determined by proving ring penetrometer and sand cone tests, was approximately 2-feet. Sand cone density tests were performed every 20-feet along the perimeter in this area. Test results are summarized in Table 2. A typical cross-section through Area 18, showing the extent of disturbed material removed, is presented in Figure 5.

2. Area between Unit 1 Containment Tendon Gallery and Unit 1 Electrical Tunnel:

   Erosion areas for repair in this area were identified as Areas 4, 5 and 6 respectively (Figure 1).

   Areas 4 and 6 refer to erosion along the slope adjacent to the Unit 1 Electrical Tunnel east wall mudslab. Area 5 refers to erosion in the backfill between the tunnel east wall and the Unit 1 Tendon Gallery.

   Along the Unit 1 Electrical Tunnel east wall, dynamic cone penetrometer tests were performed to a maximum depth of 4-feet below the bottom of the mudslab. Prior to the tests, the mudslab was core-cut at the test locations approximately 2-feet from the edge of the wall. The locations of these tests are shown on Figure 6 and the results plotted in Figure 7. Data relating to these dynamic cone penetrometer tests are presented in Table 3. The data indicate that with the exception of Test Locations 3A and 5A, high resistances were obtained in the backfill adjacent to the tunnel wall. In addition, these resistances were observed to generally increase with depth.

In order to confirm the low driving resistances encountered at Test Locations 3A and 5A, additional tests were run a few feet north and south of Test Locations 3A and 5A. These tests are designated as 3B, 3C, 5B and 5C respectively. It appeared from these results that a zone of material of questionable compaction could exist in the vicinity of Test Location 3A at elevation 149.5' to 150.0'. In order to evaluate the percent compaction in this area on a quantitative basis, four sand cone density tests were performed at the elevation in question. These tests were run after removal of the east Electrical Tunnel mudslab to within a foot of the base slab. For each sand cone density test, a laboratory Modified Proctor compaction test was run on material obtained at the test location. The results of these tests are shown in Table 2. The data showed values of relative compaction of 104.8, 102.2, 102.8 and 96.0 percent, respectively. Thus, it can be seen that the lower penetrometer resistances encountered at Test Location 3A were not indicative of an average degree of compaction less than 97 percent.

Sand cone density tests were performed a few feet from the east wall at approximately those locations where dynamic cone penetrometer tests were performed. In addition, four tests were conducted in the area between the Electrical Tunnel and Unit 1 Tendon Gallery bounded by coordinates N80+35 and N81+50. Two tests were performed in the area between coordinates N79+85 and N80+35. The results of these tests are shown in Table 2. A typical section showing extent of disturbed material removed in the area between the Electrical Tunnel and the Containment is shown in Figure 8.

The procedure used to backfill against the east wall was in compliance with the repair procedure specified in Reference 1, with the exception of the variation which is explained below.

The approved repair procedure specified hand-excavation to remove existing gunite and loose materials near the toe of the slope to a maximum height of 1.5-feet from the backfill surface. After repairing the exposed portion of the slope, the area was to be backfilled to correlation purposes. The sampling was attempted in accordance with the procedure described in the Appendix. Owing to the very dense condition of the underlying backfill, it was not possible to obtain undisturbed samples. The height of sample recovery ranged from 4 to 6-inches. Unit weights determined from these samples were abnormally low indicating sample disturbance. Therefore, these data were not considered representative of the in situ density of the backfill. Shelby tube sampling was discontinued after it was established that the small size of the sample, the manner in which it was extracted and the deformations and sample disturbance occurring as a consequence, rendered the results unreliable.

A total of 33 sand cone density tests were performed along the inside perimeter of the Tendon Gallery mudslab. These tests were made on the backfill surface after the mudslab had been removed to within 3-feet of the base slab. Additionally, some sand cone density tests were made in the area between the Tendon Gallery and the Reactor Cavity. The results of these tests are summarized in Table 2. Test results were satisfactory in all areas except for two isolated areas (approximately 10-feet by 12-feet) north and south of the Reactor Cavity. These areas were excavated down to the existing lean concrete fill and backfilled.

Dewatering of the backfill was achieved by a series of vacuum type wellpoints installed around the inside perimeter of the Containment Tendon Gallery. Five short-term piezometers were installed to monitor the water table inside this area. At the time backfilling operations were resumed in this area, the water table, as indicated by the piezometers, was at least 5-feet below the existing backfill surface.

Some typical cross-sections of the Containment area showing the extent of loose material removed are shown in Figure 11.

4. Unit 2 Containment Area:

Erosion in the Unit 2 Containment area was designated as Areas 14 and 17 (Figure 1). Area 14 referred to erosion below the Tendon Gallery mudslab on the west side. However, the construction of the Tendon Gallery had not begun on this section of the mudslab. Erosion in Area 14 was quite limited in extent. Repairs in this area involved removal of the mudslab over the eroded area, excavation to undisturbed material and then backfilling the excavation. Area 17 pertained to erosion below the mudslab of the partially built a maximum depth of 1-foot. The procedure specified that all further stages of slope repair work and backfilling be done at height and depth increments of 1.5-feet and 1.0-foot respectively. Subsequent to the erosion last year, the undisturbed Electrical Tunnel slope surface was protected by polyethylene sheeting, on which a layer of loose fill was placed. The entire slope was then gunited. Apparently, no bond existed between the existing loose fill and gunite with Category I backfill because of the polyethylene sheeting. Consequently, the protection system became unstable when the lower section was removed, necessitating removal of the full height rather than in 1.5-foot increments.

The intent of the specified repair procedure was to prevent long-term exposure of the undisturbed fill slope prior to backfilling. This was satisfied, since backfilling was accomplished expeditiously in the east-west direction in slope lengths not exceeding 10-feet. This involved removing the gunite and loose fill to a height dictated by practical considerations but restricting the working slope to a segment 10-feet long, thus limiting the area exposed to possible erosion during the repair work.

Heavy compaction equipment was not permitted near the slope during the remedial work. It was used only after the adjacent 30-foot width of backfill had been raised to the same elevation as the top of the slope by the use of hand-compaction equipment.

In the other areas east and south of the slope, where erosion had taken place, all disturbed material was removed prior to backfilling. The piezometer readings in the area indicated the water table to be at least 2-feet below the existing backfill surface. Backfilling was accomplished in accordance with the approved procedures.

3. Unit 1 Containment Area:

Erosion outside the Unit 1 Containment area was identified as Areas 7, 8, 9, 19 and 20 respectively (Figure 1). Area 7 had been repaired earlier in November, 1979 (Reference 1). Areas 8, 9 and 19 were repaired in accordance with specified procedures. The depth of the disturbed zone was determined by proving ring penetrometer probing. The disturbed fill was excavated to competent fill material and backfilled. At least one sand cone density test was made in each of the above areas prior to fill placement. Area 20, which delineated a washout in the backfill below the expansion joint opening between the Tendon Gallery Unit 1 and the Auxiliary Building north wall, was backfilled by pumping grout into the void. This work was done in accordance with the approved procedures and the grouting pressure was maintained below 5 psi.

For the inside area between the Tendon Gallery and the Reactor Cavity, no specific erosion areas were identified in Reference 1. However, it was stated in Reference 1 that all disturbed fill in the area would be excavated and removed by using field density testing and probing procedures. A minimum of three sand cone density tests were specified at equidistant locations around the inside perimeter of the Tendon Gallery mudslab.

The NRC, in a letter to Georgia Power Company (GPC), directed that for the Unit 1 Tendon Gallery an investigative approach similar to that proposed by GPC for Unit 2 be followed to determine the extent of any erosion around the Tendon Gallery foundation (Reference 2). For Unit 2 Containment, a number of dynamic cone penetrometer and sand cone density tests were proposed around the inside perimeter of the Tendon Gallery mudslab. Accordingly, a program of in situ density testing around the inside perimeter of the Unit 1 Tendon Gallery mudslab was developed by Bechtel for the purpose of verifying the competency of the backfill. Dynamic cone penetrometer tests taken at seventeen locations shown in Figure 9 were performed below the mudslab after core-cutting through it. These tests were made to a maximum depth of 3-feet. A summary of the test results is in Table 4. Figure 10 represents a plot of the penetrometer blowcounts with depth.

The test data indicate that high blowcounts were obtained at all the test locations. These blowcounts ranged from 14 to 77 blows for 1-3/4 inches penetration and increased with depth except in a few locations. Sand cone testing, as discussed below, was done in this area and the results confirmed that the fill meets the compaction criteria even though lower cone penetration resistance with depth was recorded in a few locations. Based on the correlation ratio obtained between the dynamic cone penetrometer and standard penetration resistances (Section II.B.), the data indicated that high Standard Penetration Test resistances could be expected below the mudslab.

Attempts were made to extract Shelby tube samples from the penetrometer test holes, so that the in situ density of backfill below the mudslab could be determined for

Tendon Gallery on the inside of the Containment area. Extensive testing was performed in this area around the perimeter of the partial Tendon Gallery to ascertain whether the base slab had been undermined.

Dynamic cone, proving ring penetrometer, and sand done density tests were carried out as specified in Reference 1. No Shelby tube samples were attempted for the reasons stated in Section II.C.4.

Dynamic cone penetrometer tests were performed below the mudslab at a distance of approximately 1.5-feet from the edge of the Tendon Gallery. These tests were run at 10-foot centers along the perimeter to a maximum depth of 3-feet. Test locations are shown on Figure 12. The results of these tests are summarized in Table 5 and shown plotted in Figure 13. As in Unit 1, the cone penetrometer resistances in Unit 2 were consistently high and increased with depth. The data indicate that the backfill immediately adjacent to the Tendon Gallery base slab was dense and, therefore, had not been subjected to erosion.

The Tendon Gallery mudslab extended to approximately 3.5-feet from the edge of the base slab and was removed to within 2-feet of the base slab. By means of the proving ring penetrometer, it was determined that disturbed material extended (horizontally) to a maximum of 4-inches under the sawed-off edge of the mudslab. After the mudslab was removed, thirteen sand cone density tests were made immediately at what was previously the interface between the mudslab and the backfill. Results of these tests are summarized in Table 2. Values of relative compaction ranging from 102.1 to 107.4 percent were obtained; these values confirmed the results yielded by cone penetrometer tests.

Immediately after the tests were completed, minor additional erosion occurred as a result of a rainstorm. The area was retested and repaired in accordance with approved procedures. The maximum extent of disturbed backfill under the mudslab was increased to about 10-inches. This situation was remedied by the procedure illustrated in Figure 14 and outlined below.

a. All loose material was removed from below the mudslab and 1-foot away from it. Proving ring penetrometer tests were made to assure that all disturbed material was removed.

b. A form was placed 1-foot away from the edge of the mudslab.

From: **Melba F. Kicklighter**

To: Gloria Kelly

Please replace pages in backfill erosion report ACPM-G-3-X2AP01 dated 8/15/80 with the attached pages.

Memo

DATE	9-15-80
FROM	Gloria Kelley
TO	File
TO	
TO	
TO	

- [ ] NOTE AND FILE
- [ ] NOTE AND RETURN TO ME
- [ ] RETURN WITH MORE DETAILS
- [ ] NOTE AND SEE ME ABOUT THIS
- [ ] PLEASE ANSWER
- [ ] FOR YOUR APPROVAL
- [ ] PREPARE REPLY FOR MY SIGNATURE
- [ ] TAKE APPROPRIATE ACTION
- [ ] PER YOUR REQUEST
- [ ] SIGNATURE
- [ ] FOR YOUR INFORMATION
- [ ] INVESTIGATE AND REPORT

COMMENTS

Void Pages
see attached memo

700218A

c. Concrete was placed to within 2 to 3-inches of the bottom of the mudslab.

d. The remaining 2 to 3-inches, as stated in "c" above, was drypacked to assure that no voids remained under the mudslab.

Dewatering of the backfill in Unit 2 Containment was achieved by a series of eductor type wellpoints that were extended from a line of wellpoints north of the Auxiliary Building. The water table in the backfill was monitored by means of three short-term piezometers. At the time backfilling operations were resumed in the area, the water table had been effectively lowered to at least 6-feet below the fill surface.

5. Area between Unit 2 Containment Tendon Gallery and Electrical Tunnel:

Erosion in this area was identified as Areas 10,11, 12, and 13 (Figure 1). Areas 10 and 11 were repaired in late 1979, as described in Reference 1. Areas 12 and 13 were repaired in February, 1980, in accordance with approved procedures.

Heavy rains on Saturday, March 8, 1980, caused additional erosion along the west wall of Unit 2 Electrical Tunnel which was repaired as described in Reference 3.

6. Electrical Tunnel, Unit 2, East Side:

An additional erosion area occurred below the mudslab of the Electrical Tunnel, Unit 2, in July, 1980. This erosion, was caused by construction water due to a hose failure. The maximum depth of erosion below the basemat was 0.8-feet and it extended approximately 1.8-feet below the tunnel base slab for a distance of approximately 0.8-feet. (See Figure 15). The area was repaired in accordance with approved procedures.

page corrected (see next page) 9/15/80

c. Concrete was placed to within 2 to 3-inches of the bottom of the mudslab.

d. The remaining 2 to 3-inches, as stated in "c" above, was drypacked to assure that no voids remained under the mudslab.

Dewatering of the backfill in Unit 2 Containment was achieved by a series of eductor type wellpoints that were extended from a line of wellpoints north of the Auxiliary Building. The water table in the backfill was monitored by means of three short-term piezometers. At the time backfilling operations were resumed in the area, the water table had been effectively lowered to at least 6-feet below the fill surface.

5. Area between Unit 2 Containment Tendon Gallery and Electrical Tunnel:

Erosion in this area was identified as Areas 10, 11, 12 and 13 (Figure 1). Areas 10 and 11 were repaired in late 1979, as described in Reference 1. Areas 12 and 13 were repaired in February, 1980, in accordance with approved procedures.

Heavy rains on Saturday, March 8, 1980, caused additional erosion along the west wall of Unit 2 Electrical Tunnel which was repaired as described in Reference 3.

6. Electrical Tunnel, Unit 2, East Side:

An additional erosion area occurred below the mudslab of the Electrical Tunnel, Unit 2, in July, 1980. This erosion, which was caused by construction water, extended approximately 1.8-feet below the tunnel base slab for a distance of approximately 0.8-feet. The area was repaired in accordance with approved procedures.

III. FINAL ENGINEERING EVALUATION OF STRUCTURE FOUNDATIONS

A preliminary evaluation of the effects of the backfill erosion on the structural integrity of each structure in the power block area was submitted in Reference 1. It was concluded that no undermining of Category I foundations had occurred as a result of the erosion caused by the rainfall of early November, 1979. This applied to all structures except for the Containment Unit 2 Tendon Gallery, where additional information was required for an evaluation of its structural integrity.

During the period of erosion repairs, additional information was developed to support the preliminary conclusions arrived at in Reference 1 and to evaluate the structural integrity of Containment Unit 2 Tendon Gallery. This information consisted of settlement data, field test data, and visual inspection of backfill surface following removal of mudslab. Based on these data, it has been concluded that no undermining of Category I foundations had occurred as a result of the erosion caused by the rainfall of early November, 1979, including the Containment Unit 2 Tendon Gallery.

A final evaluation of the integrity of the foundation of each structure is presented below.

A. Containment Unit 1

Inside the Containment area along the inside perimeter of the Tendon Gallery foundation, extensive field testing revealed that the backfill adjacent to the foundation was in a very dense condition. The relative compaction of the backfill as obtained from sand cone density tests ranged from 96.9 to 106.8 percent (Table 2). Dynamic cone penetrometer tests indicated high resistance, and these resistances increased with depth (Table 4, Figure 10). These test results were supported by visual inspection of the backfill surface beneath the Tendon Gallery foundation mudslab. After the mudslab had been removed to within 3-feet of the foundation base slab, inspection revealed no evidence of any erosion features in the fill. The fill surface and slope against the mudslab were devoid of any erosion channels, nor was there any evidence of loss of density. It has been concluded that no piping of fines occurred below the Tendon Gallery foundation. If piping had occurred, it would have manifested itself in the form of erosion adjacent to the Tendon Gallery foundation mudslab.

Two settlement markers were installed to monitor settlement of the Tendon Gallery foundation. These markers, designated as Nos. 323 and 324, were located as shown on Figure 16. A plot of settlement versus time for the period January 1 through July 1, 1980, is shown on

Figure 16-1. The plot indicates that the observed settlements to date are small. The maximum settlement recorded is on the order of 0.26 inch, which is reasonable considering the current loading and the limits of the survey accuracy.

The effect of the erosion on the outside of the Containment area on the integrity of the Containment structure was evaluated in Reference 1. All these were localized areas and were repaired as described in Section II.C. As stated in Reference 1, no damage was caused to the Tendon Gallery foundation as a result of erosion in these localized areas.

In summary, the Unit 1 Tendon Gallery wall foundation was not jeopardized by the heavy rainfall of early November, 1979. It has been concluded from field test data and visual observations that no erosion occurred below the Tendon Gallery base slab.

B. <u>Turbine Building Units 1 and 2</u>

The Turbine Building foundation base slab was not subjected to any erosion. The erosion that occurred was confined to the south slope, off the south side of the Turbine Building mudslabs. Erosion gulleys extending to a maximum of 4-feet below the mudslab caused cracking to occur in the mudslab. During repair all cracked sections of the mudslab were removed and the erosion gulleys were cut back to sound material at a slope of 1.5 horizontal to 1.0 vertical.

All other sections of the Turbine Building south slope that were steeper than 1.5 horizontal to 1.0 vertical were reworked to 1.5 horizontal to 1.0 vertical and then protected from erosion by guniting. The minimum setback distance from the top of a 1.5 horizontal to 1.0 vertical slope to the edge of the existing Turbine Building base slab was determined by a slope stability analysis to be approximately 20-feet (Reference 1). This requirement was met even though the nonconforming slope had to be cut back substantially to satisfy the design criteria for temporary Category I fill slopes.

Settlement of the Turbine Building base slab was monitored by two settlement markers, Nos. 308 and 310 (Figure 16). Readings were taken on a weekly basis during the period January 1 through July 1, 1980. These readings are shown plotted on Figure 16-2. The maximum observed settlement is on the order of 0.16 inch, which is reasonable considering the current loading condition and the limits of the survey accuracy.

In summary, the Turbine Building base slab was not undermined by erosion. The affected sections of the mudslab have been removed and the slope reworked to conform to the specifications.

C. Control Building Shafts Units 1 and 2

Erosion of backfill in the Control Building shafts area occurred at least 2-feet away from the permanent foundations. Visual inspection showed that the foundations were not affected by erosion. All disturbed areas in the proximity of the Control Building shafts were repaired in accordance with the specified procedures. Settlement in these areas is discussed under Items "D" and "E" below.

D. Electrical Tunnel Unit 1

Along the Unit 1 Electrical Tunnel east wall, the data obtained from cone penetrometer and sand cone density tests indicated that the backfill adjacent to the tunnel foundation was in sound condition. The disturbed material in the two erosion areas along the slope adjacent to the foundation was carefully removed by hand excavation and the areas backfilled in accordance with the procedure described in Section II.C.2. A visual inspection made prior to backfill revealed that the zone of erosion in both areas did not extend to below the tunnel foundation.

Based on a slope stability analysis done earlier for the Unit 1 Electrical Tunnel foundation, it was determined that there was no potential for a deep-seated slope failure in the backfill (Reference 1). Minor surface ravelling could have occurred in areas where the slope protection system had been removed. It was further determined that even if minor sliding should occur close to the foundation, the integrity of the existing tunnel would not be affected because of the rigidity of the foundation slab. Visual inspection showed no evidence of ravelling of undisturbed Category I backfill in areas where gunite protection had been removed. Any potential for sloughing or ravelling of the slope was precluded by expeditiously backfilling to the top of the slope.

Prior to backfilling against the slope, two additional settlement markers (423-1-A and 423-1-B) were installed along the east wall approximately 30 and 60-feet north of an existing marker No. 423-1 (Figure 16). These two markers were read on a daily basis from the time the slope protection system was removed until backfilling to the top of the slope was completed. In addition, settlement markers 423-1 and 420-1 were read on a weekly basis from

January, 1980, onward. Plots of settlement versus time for the markers are shown on Figures 16-4A and 16-4B. The maximum recorded settlement was on the order of 0.2 inch, which is reasonable considering the current loading and the limits of the survey accuracy.

In summary, both field test data and visual observations indicate that the Unit 1 Electrical Tunnel foundation was not affected by erosion adjacent to the foundation. The erosion was outside the limits of the existing foundation and was successfully repaired to conform to the specifications.

E. Electrical Tunnel Unit 2

The effect of the four erosion areas along the Unit 2 Electrical Tunnel west wall (Figure 1) on the tunnel foundation was evaluated in Reference 1. The erosion was limited to the tunnel foundation mudslab except in one instance (that which occurred in September, 1979) where it extended about a foot below the foundation itself. The erosion was subsequently repaired in accordance with the specified repair procedures.

The additional erosion that occurred along the west wall in March, 1980, was evaluated and repaired as described in Reference 3.

The erosion along the east wall which occurred in July, 1980, was evaluated and repaired in accordance with approved procedures.

A plot of settlement versus time for the Unit 2 Electrical Tunnel foundation is shown on Figure 16-3. Small settlements, on the order of 0.2 inch, were recorded, which are reasonable considering the current loading condition and the limits of the survey accuracy.

It was concluded that the erosion had not affected the permanent foundation.

F. Containment Unit 2 - Partial Tendon Gallery

There were two specific areas of erosion in the Containment Unit 2 area. Area 14 was at least 50-feet away from the west end of the partially built Tendon Gallery wall (Figure 1). This area was repaired as described in Section II.C.4.

Area 17 pertained to the area surrounding the completed segment of the Tendon Gallery wall foundation. Extensive testing was performed in the area adjacent to the Tendon Gallery foundation. The test data obtained showed that the backfill adjacent to the foundation was dense. Visual inspection revealed that some erosion had occurred at the edge of the mudslab along a few sections of the inside perimeter. A portion of the mudmat was removed and by means of the proving ring penetrometer it was established that the erosion extended to approximately 18-inches from the edge of the foundation. It was concluded that this erosion was caused by run-off flowing along the periphery of the Tendon Gallery wall and flowing away toward the Auxiliary Building. The fill surface and slope against the mudslab were devoid of any erosion channels, nor was there any evidence of loss of density. It has been concluded that no piping of fines occurred below the Tendon Gallery foundation. If piping had occurred, it would have manifested itself in the form of erosion adjacent to the Tendon Gallery foundation mudslab.

Minor additional erosion occurred below the mudmat due to rainfall that occurred immediately after the evaluation tests were complete. However, the zone of disturbed material was at least 1-foot away from the Gallery foundation. The disturbed material was excavated, and the area was backfilled following approved repair procedures.

Three settlement markers had been installed to monitor settlement of the Tendon Gallery foundation. These markers, designated as Nos. 425, 426 and 427, were located as shown on Figure 16. A plot of settlement versus time for the period January 1, 1980, through July 1, 1980, is shown on Figure 16-5. The data indicate that a maximum settlement of 0.17 inch was recorded, which is considered reasonable for the current loading condition and the limits of the survey accuracy. It was concluded from field test data and visual observations that the Unit 2 Containment Tendon Gallery was not affected by erosion adjacent to the foundation.

G. <u>Auxiliary Building and NSCW Towers</u>

The Auxiliary Building and NSCW Towers were founded on the marl formation. The Auxiliary Building base mat is approximately 22-feet below the top of the marl. The NSCW Towers are founded approximately 3-feet below the marl surface. Therefore, none of these structures were affected by the erosion in the backfill.

IV. SURFACE WATER CONTROL

Several steps have been taken to prevent the recurrence of significant erosion due to rainfall. These steps include increasing the protection against externally generated storm run-off entering the power block excavation, preventing the uncontrolled flow of storm run-off within the power block excavation by use of temporary ditches and berms, increasing the use of slope protection, and increasing the capacity for pumping storm run-off out of the power block excavation. As backfill progresses, the pumping scheme and capacities will be altered to meet any new requirements caused by the changing configuration of the backfill.

A. External Run-Off Control

The effective height of the berm surrounding the top of the power block excavation, including the crests of ramps entering the excavation, has been raised approximately 2-1/2 feet. This has effectively precluded the entrance of externally generated storm run-off into the excavation.

B. Control of Storm Run-Off Within the Power Block Excavation

All backfill surfaces are sloped so that run-off flows away from fill slopes and away from buildings to swales which flow to sumps. Run-off collected in the sumps is pumped out of the excavation to existing discharge piping and discharge channels which flow away from the excavation. An 18-inch berm is provided at the top of the fill slope south of the Turbine Building to prevent run-off from flowing to lower elevations.

C. Slope Protection

Gunite has been applied to all long-term exposed slopes in an extensive program to prevent erosion in the event of heavy rainfall. Short-term slopes are protected with plastic sheeting.

D. Pumping Capacity

Run-off is removed from the power block excavation at three primary locations. Water collected in the Turbine Building area is pumped from a sump in the northeast corner of the excavation. Isolated areas which cannot drain around the Turbine Building are pumped to this sump. Run-off collected in the southeast corner area is pumped from this area. The remaining areas, which constitute a majority of the total area, drain to and are pumped from several sumps in the southwest area of the power block.

Figure 17, Surface Water Control, shows the location of the sumps along with pumping capacity. The pumping system in the northeast corner is capable of pumping 2000 gpm. Five pumping systems located in the southwest area of the power block have a total capacity of 6575 gpm. Two systems located in the southeast area have a total capacity of 2625 gpm. The total capacity of all systems is 11,200 gpm. The pump capacities shown on Figure 17 are as-built conditions and may be increased.

Rainfall:-

12hr - 4.5" of rain
24hr - 10" of rain
4500 gpm gw

Calculations were made based on 5-inches of rainfall to determine the amount of water that would collect in the power block and the length of time necessary to remove this run-off from the power block. A 10-year storm with a duration of 12-hours would produce 4.5-inches of rainfall; a 50-year storm with a duration of 24-hours would provide 10-inches of rainfall. Figure 17 shows the amount of rainfall and the length of time needed to remove the run-off from each area. These figures are based on having approximately 4500 gpm of groundwater entering the power block and show that the existing system can adequately handle both the 10-year, 12-hour storm and the 50-year, 24-hour storm. Several areas of the power block may also be utilized to store rainfall for later removal. The northeast sump has a capacity of approximately 450,000 gallons, the southwest area has a storage capacity of approximately 1.7-million gallons, and the Auxiliary Building and its sumps may store 200,000 gallons without causing any harm to equipment.

E. Construction Water

The amount and use of construction water is controlled. Excess water is directed to common collection points and removed from the power block excavation by the surface water pumping system.

Page incorrect (see next page) 9/15/80

also, see note preceding p. 9

Figure 17, Surface Water Control, shows the location of the sumps along with pumping capacity. The pumping system in the northeast corner is capable of pumping 2000 gpm. Five pumping systems located in the southwest area of the power block have a total capacity of 6575 gpm. Two systems located in the southeast area have a total capacity of 2625 gpm. The total capacity of all systems if 11,200 gpm. The pump capacities shown on Figure 17 are as-built conditions and may be further optimized.

Calculations were made based on 5-inches of rainfall to determine the amount of water that would collect in the power block and the length of time necessary to remove this run-off from the power block. A 10-year storm with a duration of 12-hours would produce 4.5-inches of rainfall; a 50-year storm with a duration of 24-hours would provide 10-inches of rainfall. Figure 17 shows the amount of rainfall and the length of time needed to remove the run-off from each area. These figures are based on having approximately 4500 gpm of groundwater entering the power block and show that the existing system can adequately handle both the 10-year, 12-hour storm and the 50-year, 24-hour storm. Several areas of the power block may also be utilized to store rainfall for later removal. The northeast sump has a capacity of approximately 450,000 gallons, the southwest area has a storage capacity of approximately 1.7-million gallons, and the Auxiliary Building and its sumps may store 200,000 gallons without causing any harm to equipment.

V. SUBSURFACE WATER CONTROL

A. Monitoring

1. Backfill Piezometers

Continuous monitoring of subsurface water conditions has been performed both inside and outside the power block excavation. In addition to the previously existing piezometer network located outside the excavation, a number of new piezometers were placed in the Category I backfill. These consisted of long-term piezometers extending through the backfill to the marl and short-term piezometers which extended a few feet into the backfill in critical areas. These piezometers were monitored to insure that the water table was located sufficiently below the backfill surface to conform to the specifications during backfill operations.

The groundwater elevations read in these piezometers indicated sources influencing the groundwater inside the excavation. Gradients and corresponding directions of flow obtained from the piezometer data indicated that groundwater inside the excavation originated from rainfall, and that there was no external groundwater entering the power block past the perimeter filter blanket and dewatering system. Piezometer locations are shown in Figure 20.

2. Wellpoint Piezometers

Wellpoint piezometers were installed along the wellpoint lines in order to monitor the performance of the wellpoint system, as well as to provide additional water level data. These piezometers were installed in the same manner as the wellpoints except that the eductor was not installed. The performance of the wellpoints is discussed in Section V.B., Dewatering Systems.

3. Wellpoint Discharge

During the operational periods of the various wellpoint systems, the discharge water was monitored to insure that no significant amount of sand-size particles was being pumped out of the backfill. The testing of discharge samples was done in accordance with the procedure described in Reference 1.

Samples were first visually examined as specified in Reference 1. Samples failing to meet the visual criteria were tested in accordance with ASTM D-1888 using a 40 to 60 micron filter to determine the amount of sand particles and a 0.45 micron filter for total suspended solids.

The criteria used limited the amount of sand particles in the discharge water to 5 ppm and total suspended solids to 50 ppm. Frequent visual and laboratory testing on wellpoint discharge water indicated that the criteria for sand particles and total suspended solids were satisfied.

B. <u>Dewatering Systems</u>

1. Types

There are basically three types of dewatering systems utilized to control groundwater in the power block excavation. The three types are eductor wellpoint systems, a vacuum wellpoint system, and trench drain systems. The eductor (also called ejector) systems were used for dewatering the following areas: (1) the area along the north wall of the Auxiliary Building and later extension to Containment Unit 2, (2) slopes east of Containment Unit 1, and (3) slopes adjacent to Containment Unit 2. The eductor type system was chosen for these areas because of its ability to pump from depths exceeding that of the conventional vacuum wellpoint installation (18'$\pm$). The eductor system utilizes a double manifold, one a supply and the other a return line, which circulates water through eductors which are connected to the wellpoint. This results in the development of a vacuum at the wellpoint elevation rather than at the ground surface. Eductor wellpoints were installed in maximum 10-inch diameter holes drilled with rotary equipment using Revert. Appropriately graded filter material was installed.

A vacuum wellpoint system was installed inside the Containment Unit 1 area to lower the groundwater in the backfill. This type of system is applicable where the depth of water does not exceed 18'$\pm$, since it employs the use of a conventional vacuum wellpoint pump which applies the vacuum at the header manifold level. Installation of the wellpoints was similar to that used for the eductor systems.

Trench drains were installed in the marl in areas where backfill had not yet been placed. Their function is to control future groundwater build-up in the backfill due to rainfall. Trench drains were installed southeast of the Auxiliary Building and are presently being planned for installation southwest of it. Attempts to install a trench drain along the toe of the slope directly east of Containment Unit 1 were abandoned in favor of the eductor wellpoint method due to the difficulty caused by wet conditions along the toe of the slope. A typical detail of the trench drains used is shown on Figure 18.

2. Specific Locations

Approximately 30-feet north of the north wall of the Auxiliary Building an eductor system, consisting of 51 eductor wellpoints on 5-foot centers, was installed to dewater the area for backfill operations. This system was later extended into Containment Unit 2 by the addition of 47 eductor wellpoints on 5-foot centers.

Along the inside perimeter of Containment Unit 1 a vacuum wellpoint system, consisting of 52 wellpoints on 5-foot centers, was installed. This system satisfactorily lowered the water level to permit backfill to proceed in this area.

Along the top of the slope east of Containment Unit 1 and along the top of the slope west of Containment Unit 2, two additional eductor systems were installed. These systems consisted of 50 eductor wellpoints on 5-foot centers on the east side and 82 eductor wellpoints on 5-foot centers on the west side. These wellpoints satisfactorily dewatered the east and west slopes to permit backfilling against the slopes.

At the southeast corner of the Auxiliary Building a trench drain was installed at the toe of the new backfill slope. This trench drain will minimize future seepage from the toe of the slope, so that backfill operations may continue when needed.

At the southwest corner of the Auxiliary Building another trench drain is planned. The toe of the future slope will be placed over the trench. This will permit backfilling against this slope at a later date.

The locations of the above dewatering systems are shown on Figure 19.

3. System Performance

Discharge rates from the various wellpoint installations, both eductor and vacuum types, were quite low, generally less than 5 gpm from a system. This was due mainly to the relatively low permeability of the backfill. Even though discharge rates were significantly less than originally anticipated, prolonged pumping produced noticeable drawdown in the vicinity of the wellpoints.

Permeability of Backfill - A preliminary estimate of backfill permeability based on a consideration of grain size was about 0.01 ft./min. Pumping rates based on this permeability were estimated to range from 36 gpm initially down to 13 gpm after prolonged pumping (Reference 1). Actual pumping rates of the various installations were significantly less than these amounts, apparently due to the backfill having a lower permeability than estimated. Later field permeability testing, using falling head tests on previously installed piezometers, indicated typical backfill permeabilities to range from about 3×10^{-4} to 7×10^{-4} ft./min. The most reasonable explanation for these relatively low permeabilities is the high degree of compaction of the backfill, notwithstanding that the backfill is generally quite clean (less than 10% passing a #200 sieve).

Drawdown Influence - Due to the relatively low permeability of the backfill material, the drawdown effected by the wellpoint dewatering systems was restricted to the immediate vicinity of the wellpoints. Maximum drawdown along a line of wellpoints, based on observations made on wellpoint piezometers, was about 10-feet decreasing rapidly with distance from the wellpoints. It is doubtful that any drawdown was exerted beyond about 50-feet away from a line of wellpoints. Figure 21 illustrates groundwater elevations, with approximate contours, for 12/27/79, 2/5/80 and 5/5/80.

VI. SUMMARY AND CONCLUSIONS

All erosion in the power block backfill was satisfactorily repaired according to procedures submitted to the Nuclear Regulatory Commission by Georgia Power Company, with the exception of minor deviations that were necessitated by practical considerations.

Extensive field and laboratory tests were performed to verify the extent of disturbed material in the eroded areas. These tests were used to verify the competency of the backfill adjacent to the foundations of various Category I structures. The evaluation of the effect of erosion on Category I structure foundations was based on data developed during testing, settlement readings and visual observations made during the entire period of repair.

The field testing and evaluations described in this Report provided adequate data which defined the disturbed zones in the Category I backfill. All erosion was successfully repaired. This evaluation has established that there is no detrimental effect on the existing structures as a result of the heavy rainfalls of early November, 1979.

References:

1. Letter, with attachments, from D. E. Dutton to J. P, O'Reilly of the NRC, dated January 8, 1980.

2. Letter from J. P. O'Reilly of the NRC to J. H. Miller, Jr. of GPC, dated February 8, 1980.

3. Letter, with attachment, from D. E. Dutton to J. P. O'Reilly of the NRC, dated April 30, 1980.

APPENDIX

A. FIELD TESTING AND SAMPLING PROCEDURES

1. Procedure for Dynamic Cone Penetrometer Test

In order to perform dynamic cone penetrometer tests, the mudslab was first core-cut at the test locations. A hand auger was then used to auger to a depth of 1-foot, at which depth the cone penetrometer device was lowered into the hole. The cone was driven at least 2-inches into the hole to insure that it was properly seated. The number of blows required to seat the cone was recorded. After seating, the cone was driven a further 1-3/4 inches into the hole and the number of blows recorded as the penetrometer resistance value. Driving was accomplished by means of a 15-pound steel ring weight dropping a height of 20-inches on an E-rod slide drive (see attached sketch). The hole was then augered down to depths of 2, 3 and 4-feet and the test repeated at each depth. All tests were run above the water table to insure that the test results were not influenced by inflow and soil softening inside the bore hole.

All dynamic cone penetrometer tests were performed by GPC Quality Control personnel.

2. Procedure for Proving Ring Penetrometer Test

Proving ring penetrometer tests were performed at specified locations to determine the depths of disturbed zone in the backfill. The tests were performed at depth intervals of 6-inches as required to reach competent material. Testing was accomplished by pushing the penetrometer into the soil perpendicular to the surface at a uniform rate until the top of the penetrometer cone was reached. At this point the proving ring dial was read. If the reading indicated a disturbed zone, the testing was continued to greater depths. This was done by shovelling away the disturbed material and testing at approximately 6-inch depth intervals until competent material was reached. At this point the penetrometer was moved to another specified test location.

All proving ring penetrometer tests were performed by GPC Quality Control personnel.

3. Procedure for Sand Cone Density Tests

All sand cone density tests were performed by GPC Quality Control personnel in accordance with ASTM D-1556. Moisture content determinations, as part of the sand cone density test, were made in accordance with ASTM D-2216.

4. Method of Shelby Tube Sampling

As part of the backfill testing program for the Unit 1 Containment Building Tendon Gallery foundations, Shelby tube samples were taken at selected locations along the inside perimeter of the Unit 1 Tendon Gallery. These samples were extracted from holes that were hand augered to a total depth of approximately 3-feet below top of mudslab for the purpose of performing dynamic cone penetrometer tests.

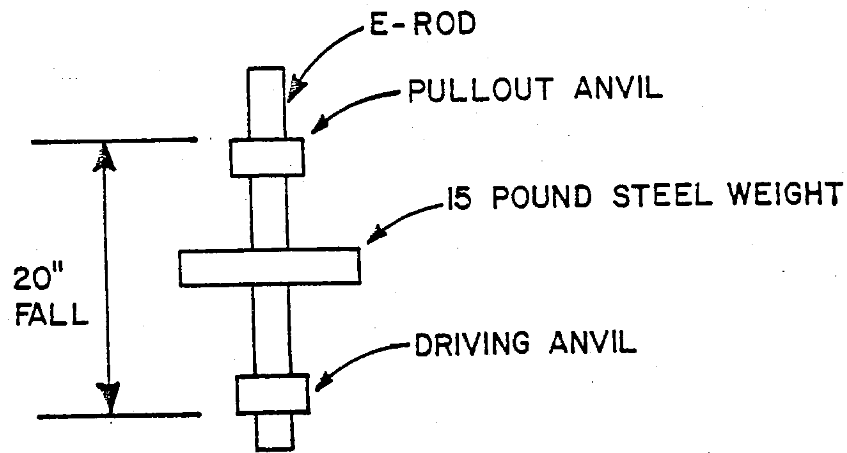
A sketch showing the Shelby tube sampler used in sample extraction is included in the Appendix. A 3-inch diameter, 30-inch long Shelby tube was attached to a 2-foot length of pipe by means of a heavy adaptor. The driving head was then screwed into the pipe. A flat plate was welded on top of the driving head. The entire assembly was then lowered into the hole and driven by means of a 10-pound sledge hammer.

Immediately following completion of the first dynamic cone penetrometer test (at depth of 12-inches), the hole was augered down a further 6-inches. No drilling mud was used. The Shelby tube was then seated in the hole and driven by successive blows of the sledge hammer. A total of four Shelby tube samples were attempted at a depth of approximately 2-feet. Samples were recovered in three of the four attempts that were made. The height of recovery ranged from 4 to 6-inches. Following extraction, the samples were transported to the laboratory, where density determination was made by the following procedure:

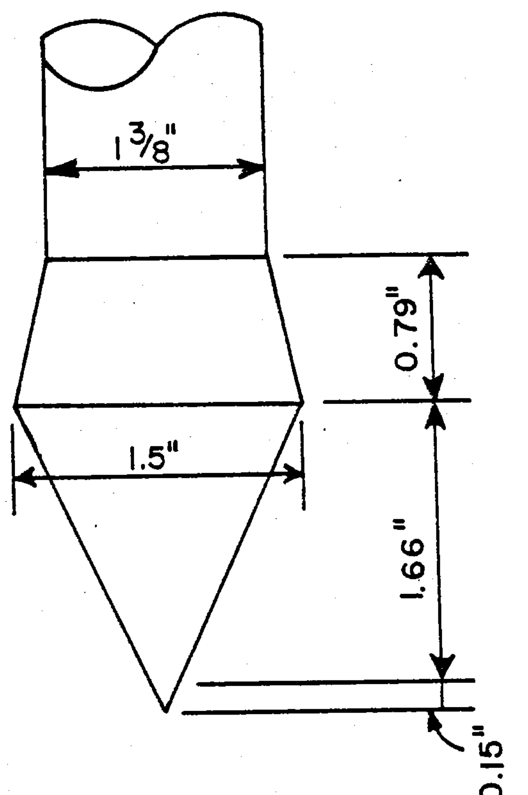
The volume of the sample inside the tube was determined by first measuring the distances inside the tube from the top of the sample to the top of the tube and the bottom of the sample from the bottom of the tube. These distances were subtracted from the total length of the tube sampler and then multiplied by the cross-sectional area of the tube. With the volume of sample thus obtained, the sample was pushed out of the tube and weighed. A moisture content determination was made on the sample. The dry density of the sample was then computed.

B. LABORATORY TESTING PROCEDURES

The Modified Proctor Compaction Test was the only type of laboratory compaction test performed during the period of backfill erosion repairs. This test was performed by GPC Quality Control personnel in the field soils laboratory. Moisture content determinations, as part of the Modified Proctor Compaction Test, were made in accordance with ASTM D-2216.

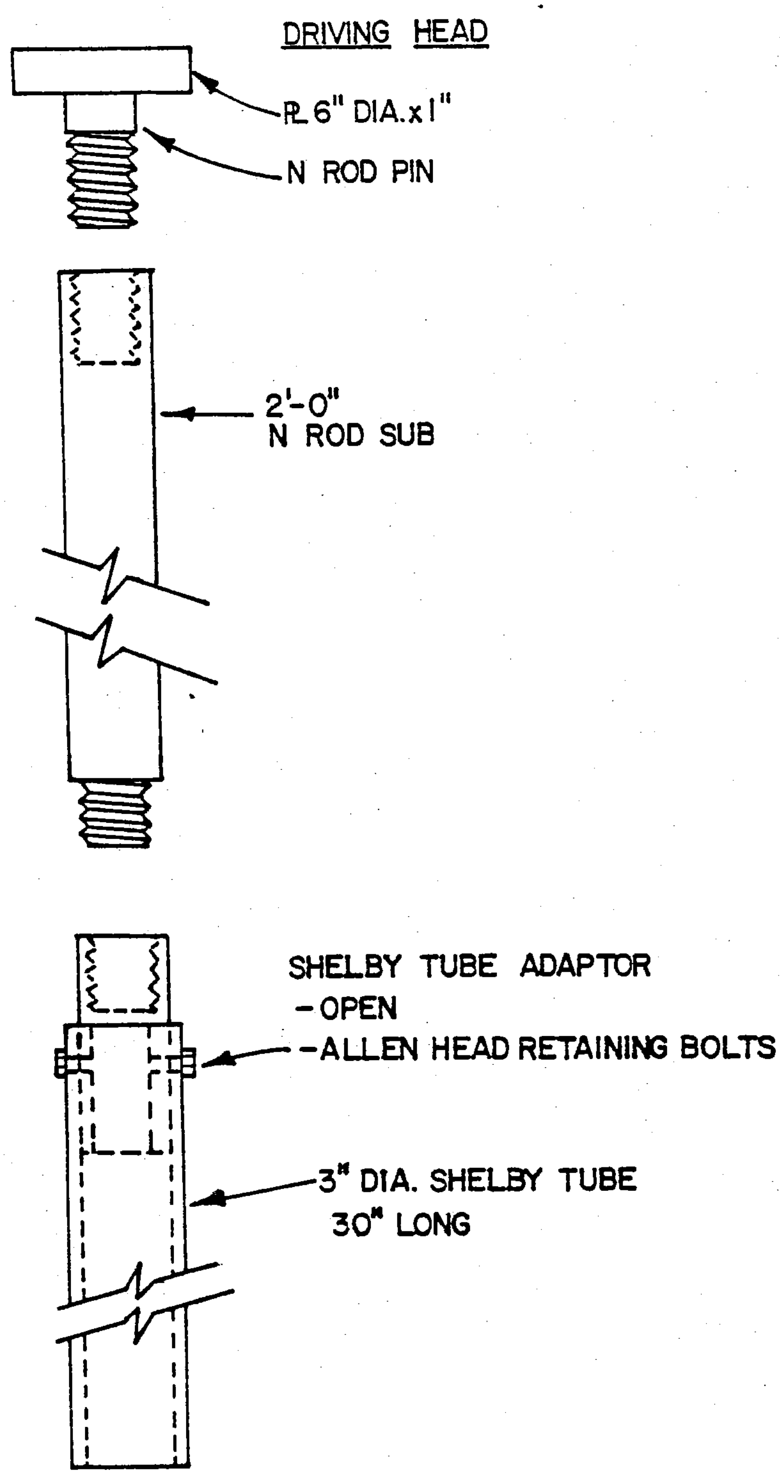


SLIDING DRIVE HAMMER

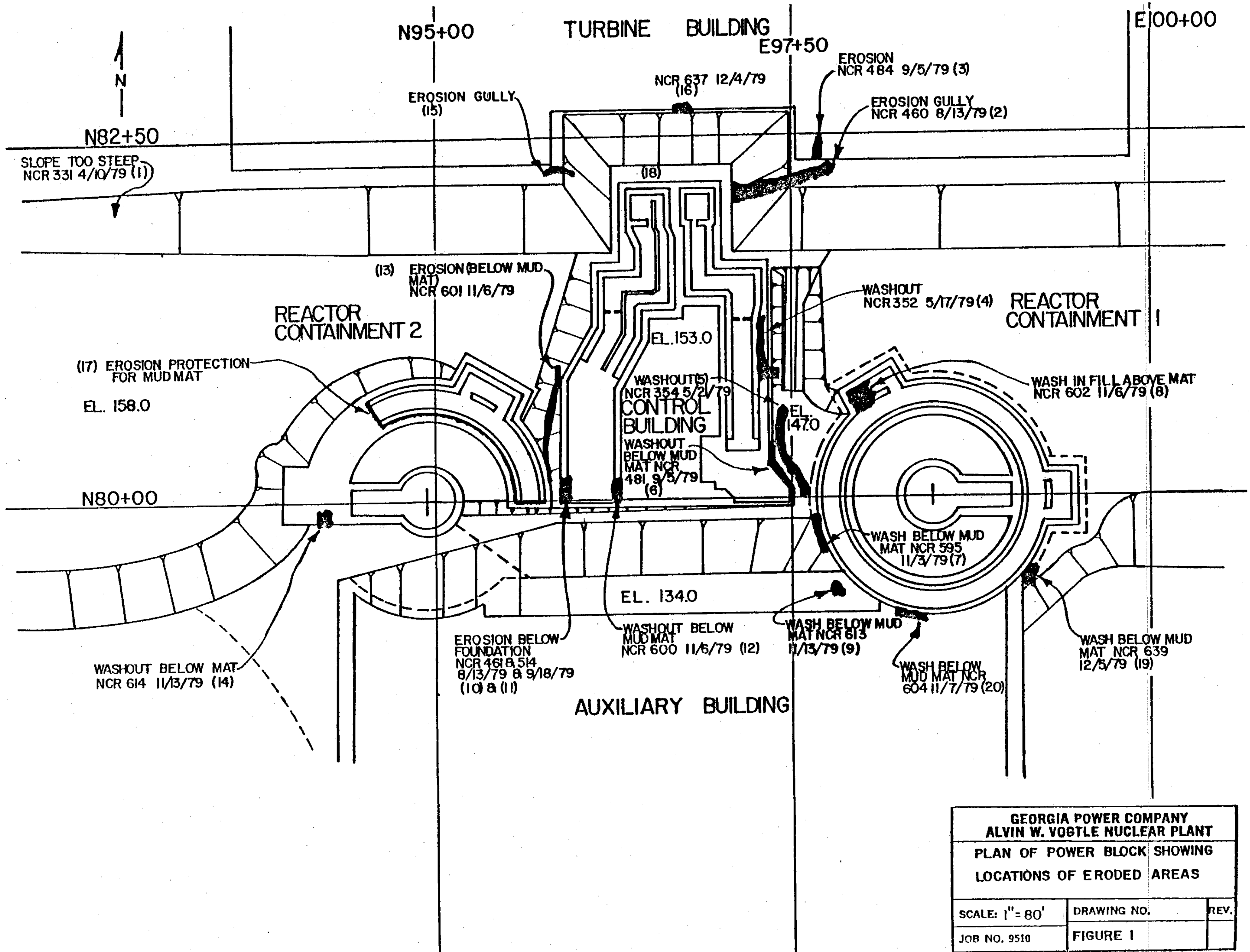


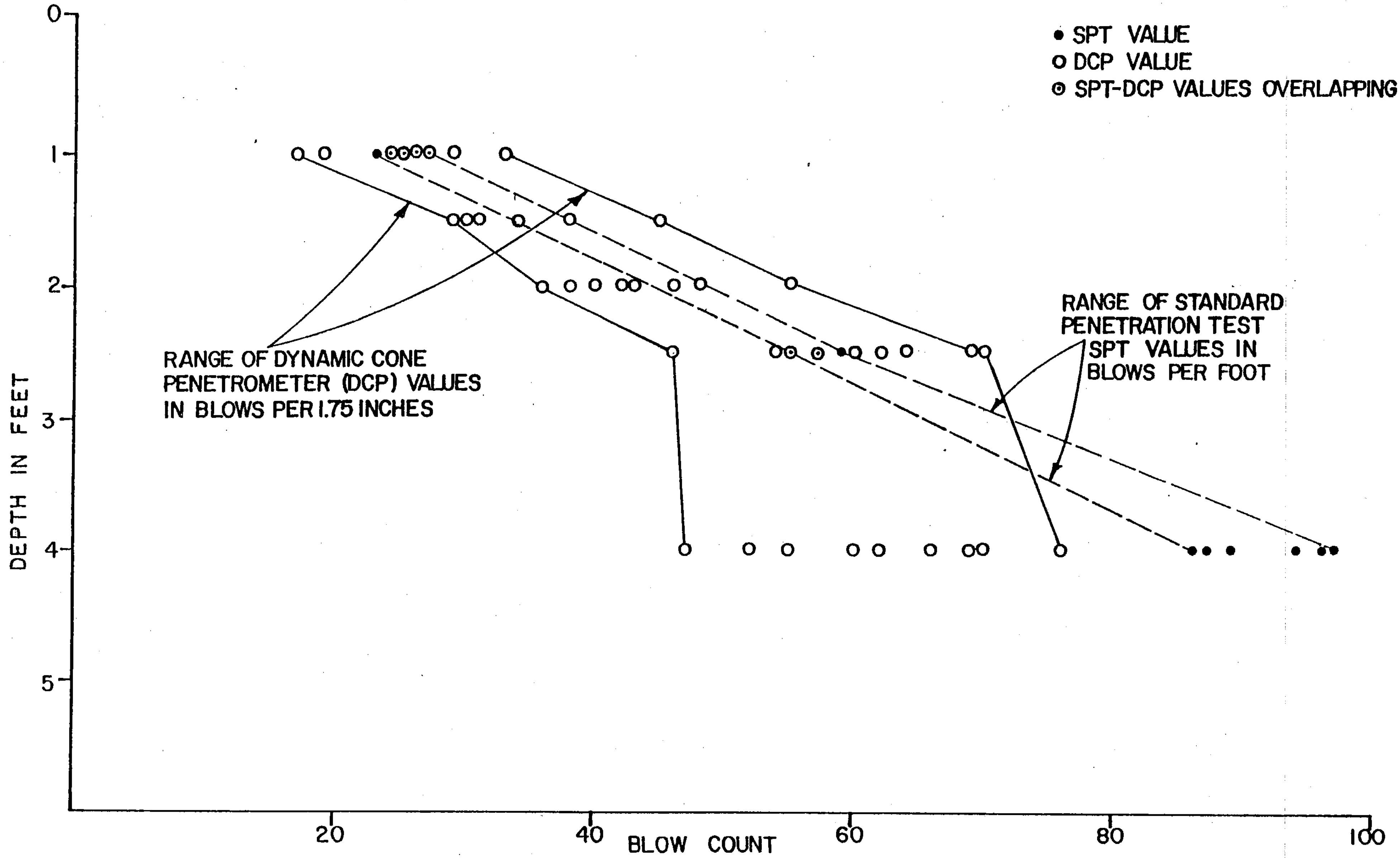
CONE POINT

GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
SKETCH SHOWING DYNAMIC CONE PENETROMETER		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	A - 1	

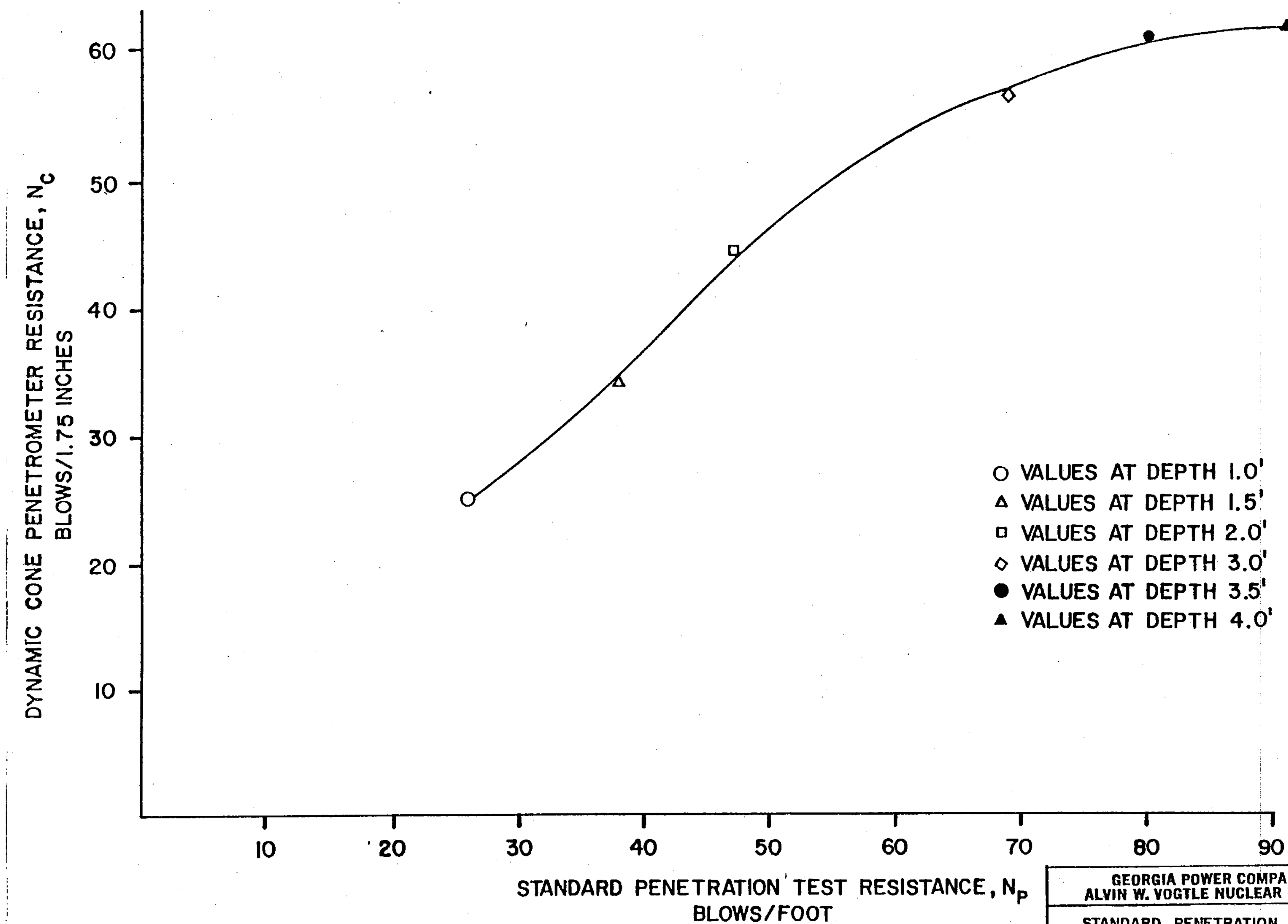


GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
SHELBY TUBE SAMPLER		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	A - 2	

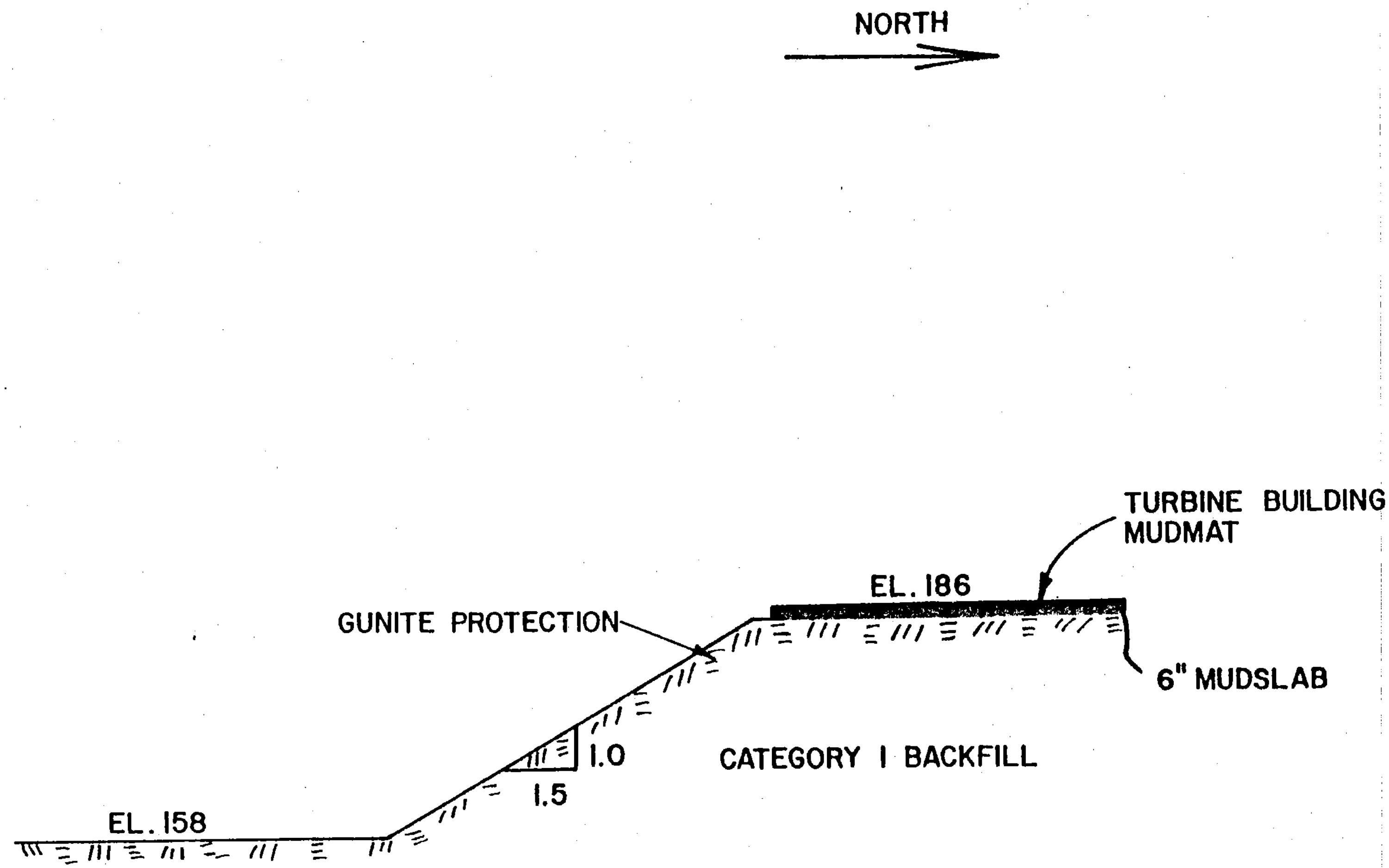

TURBINE BUILDING
N95+00
E97+50
E00+00
N82+50
N80+00
N
SLOPE TOO STEEP NCR 331 4/10/79 (1)
EROSION GULLY (15)
NCR 637 12/4/79 (16)
EROSION NCR 484 9/5/79 (3)
EROSION GULLY NCR 460 8/13/79 (2)
(18)
(13) EROSION (BELOW MUD MAT) NCR 601 11/6/79
REACTOR CONTAINMENT 2
REACTOR CONTAINMENT 1
WASHOUT NCR 352 5/17/79 (4)
EL. 153.0
(17) EROSION PROTECTION FOR MUD MAT
EL. 158.0
WASHOUT(5) NCR 354 5/21/79
CONTROL BUILDING
EL. 147.0
WASH IN FILL ABOVE MAT NCR 602 11/6/79 (8)
WASHOUT BELOW MUD MAT NCR 481 9/5/79 (6)
WASH BELOW MUD MAT NCR 595 11/3/79 (7)
EL. 134.0
EROSION BELOW FOUNDATION NCR 461 & 514 8/13/79 & 9/18/79 (10) & (11)
WASHOUT BELOW MUD MAT NCR 600 11/6/79 (12)
WASH BELOW MUD MAT NCR 613 11/13/79 (9)
WASH BELOW MUD MAT NCR 604 11/7/79 (20)
WASH BELOW MUD MAT NCR 639 12/5/79 (19)
WASHOUT BELOW MAT NCR 614 11/13/79 (14)
AUXILIARY BUILDING
GEORGIA POWER COMPANY
ALVIN W. VOGTLE NUCLEAR PLANT
PLAN OF POWER BLOCK SHOWING
LOCATIONS OF ERODED AREAS
SCALE: 1" = 80'
DRAWING NO.
REV.
JOB NO. 9510
FIGURE 1



GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
STANDARD PENETRATION TEST AND DYNAMIC CONE PENETROMETER TEST BLOWCOUNTS VS. DEPTH		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 2	

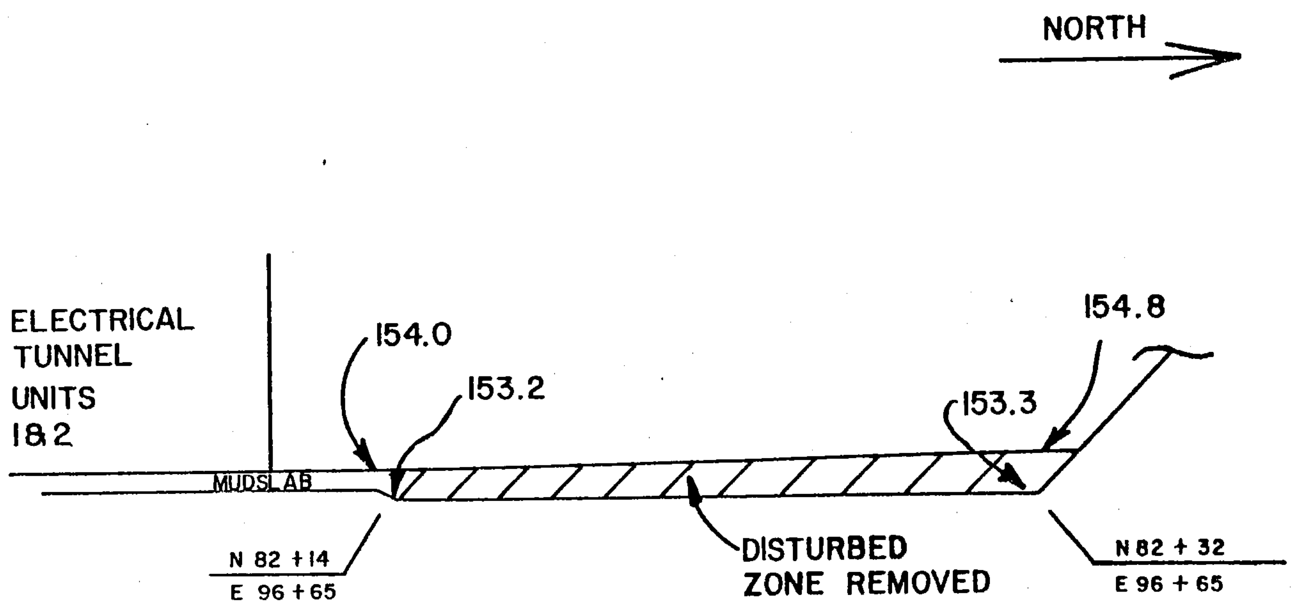


GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
STANDARD PENETRATION TEST AND DYNAMIC CONE PENETROMETER TEST CALIBRATION CURVE		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 3	



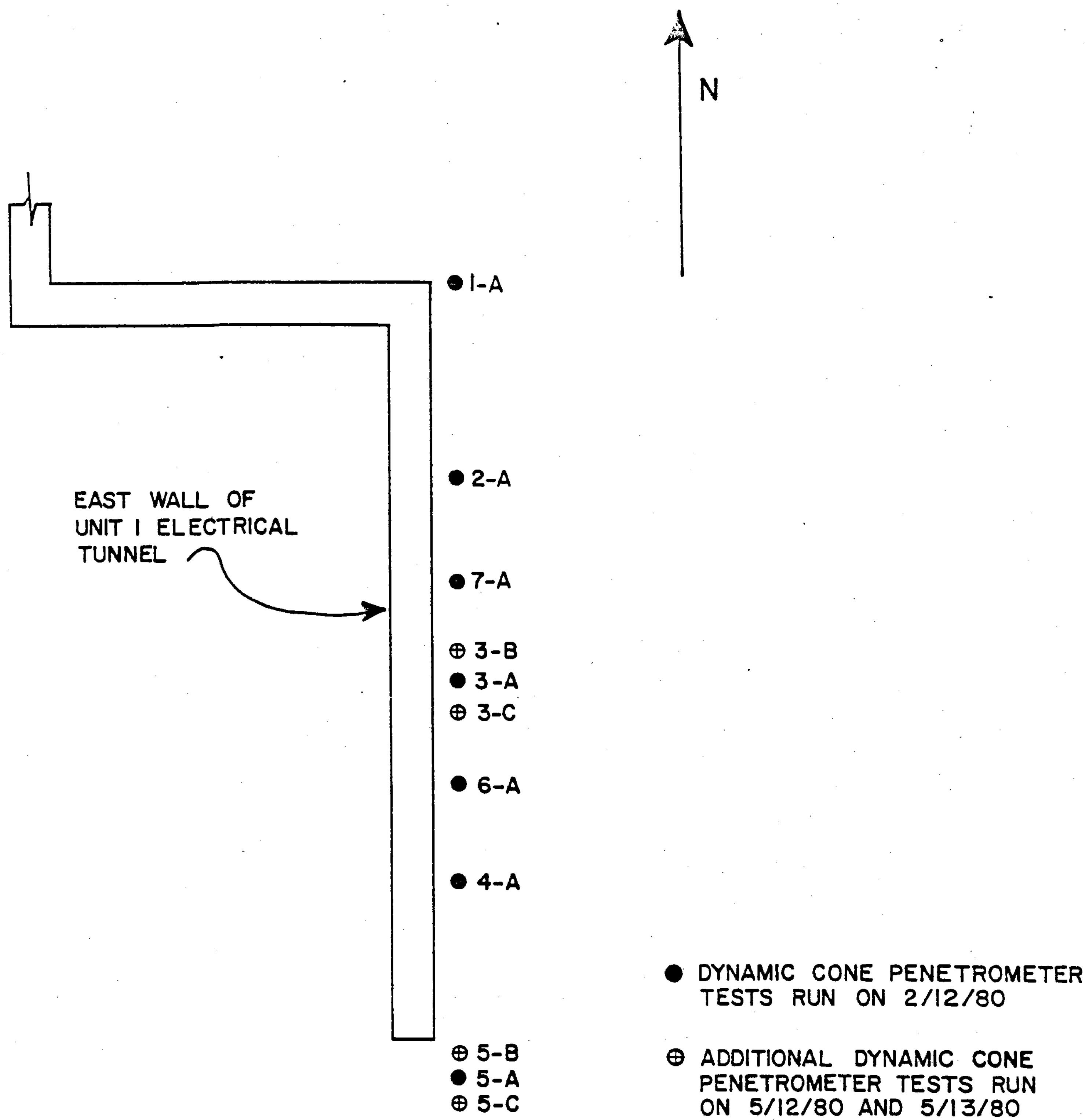
NOT TO SCALE

GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
TYPICAL REWORKED SECTION OF TURBINE BUILDING SOUTH SLOPE		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 4	



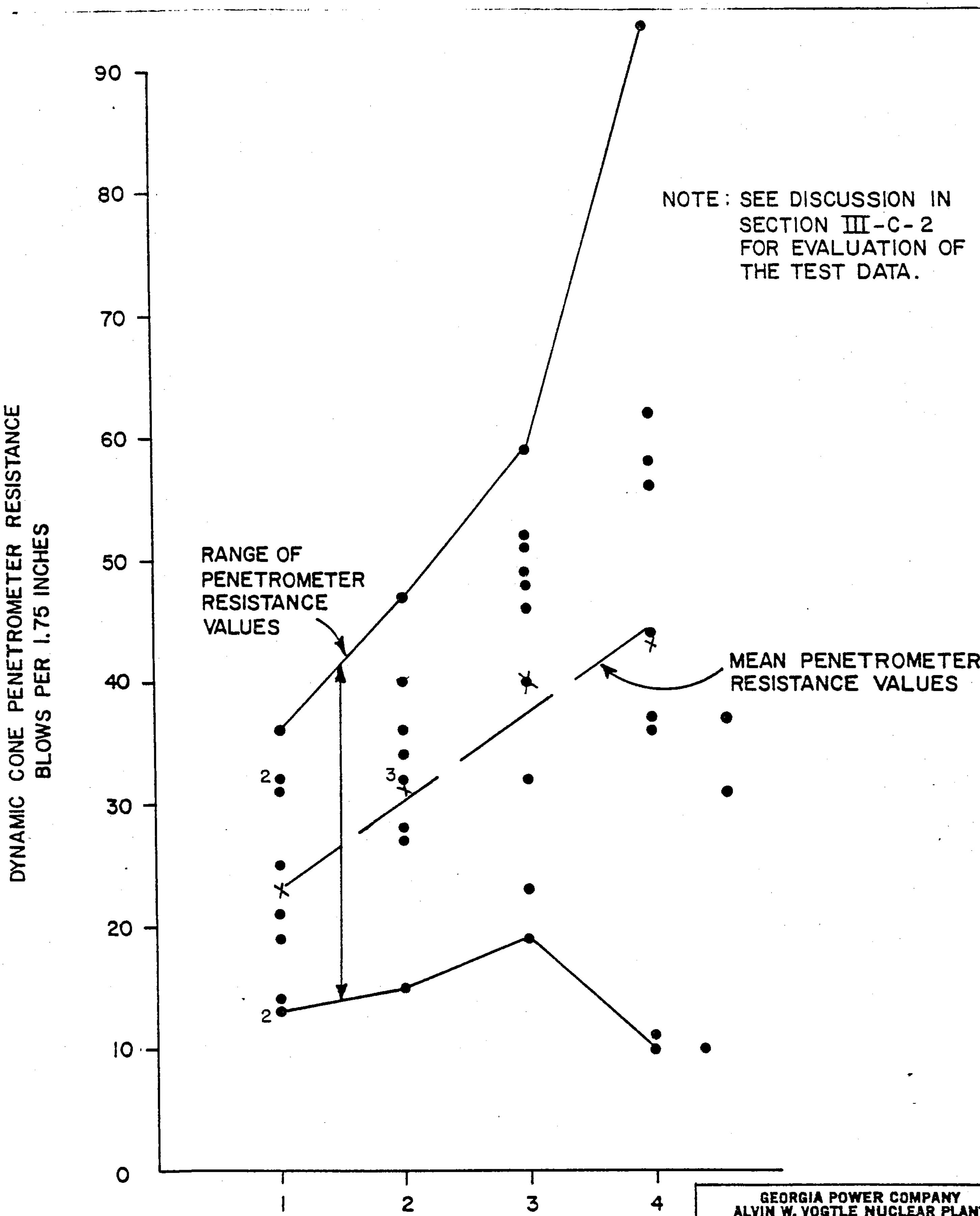
NOT TO SCALE

GEORGIA POWER COMPANY **ALVIN W. VOGTLE NUCLEAR PLANT**		
TYPICAL SECTION SHOWING EXTENT OF DISTURBED ZONE REMOVED IN CONTROL BLDG. SHAFT AREA		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 5	



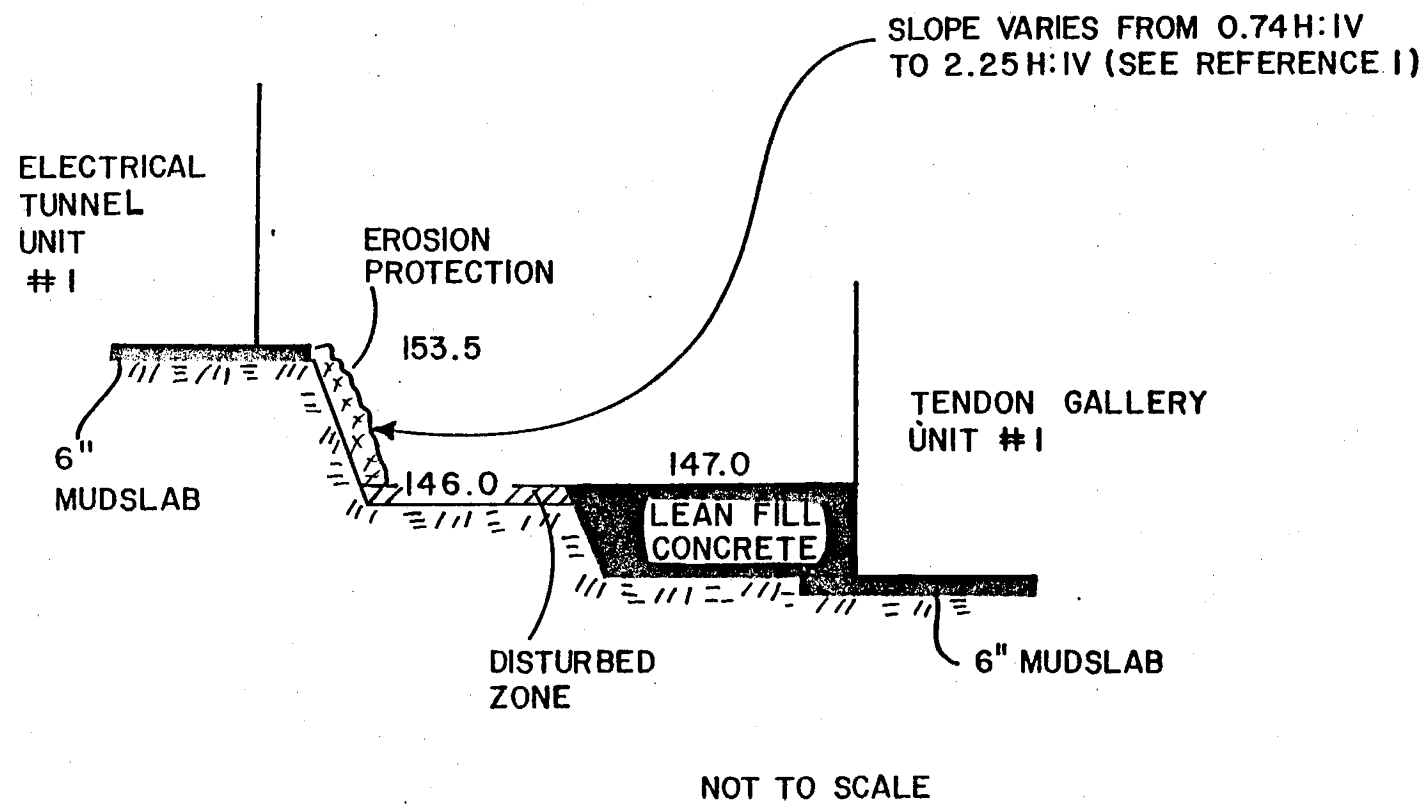
PLAN SHOWING LOCATIONS OF DYNAMIC CONE PENETROMETER TESTS ADJACENT TO ELECTRICAL TUNNEL EAST WALL.

GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
ELECTRICAL TUNNEL EAST WALL DCP TEST LOCATIONS		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 6	



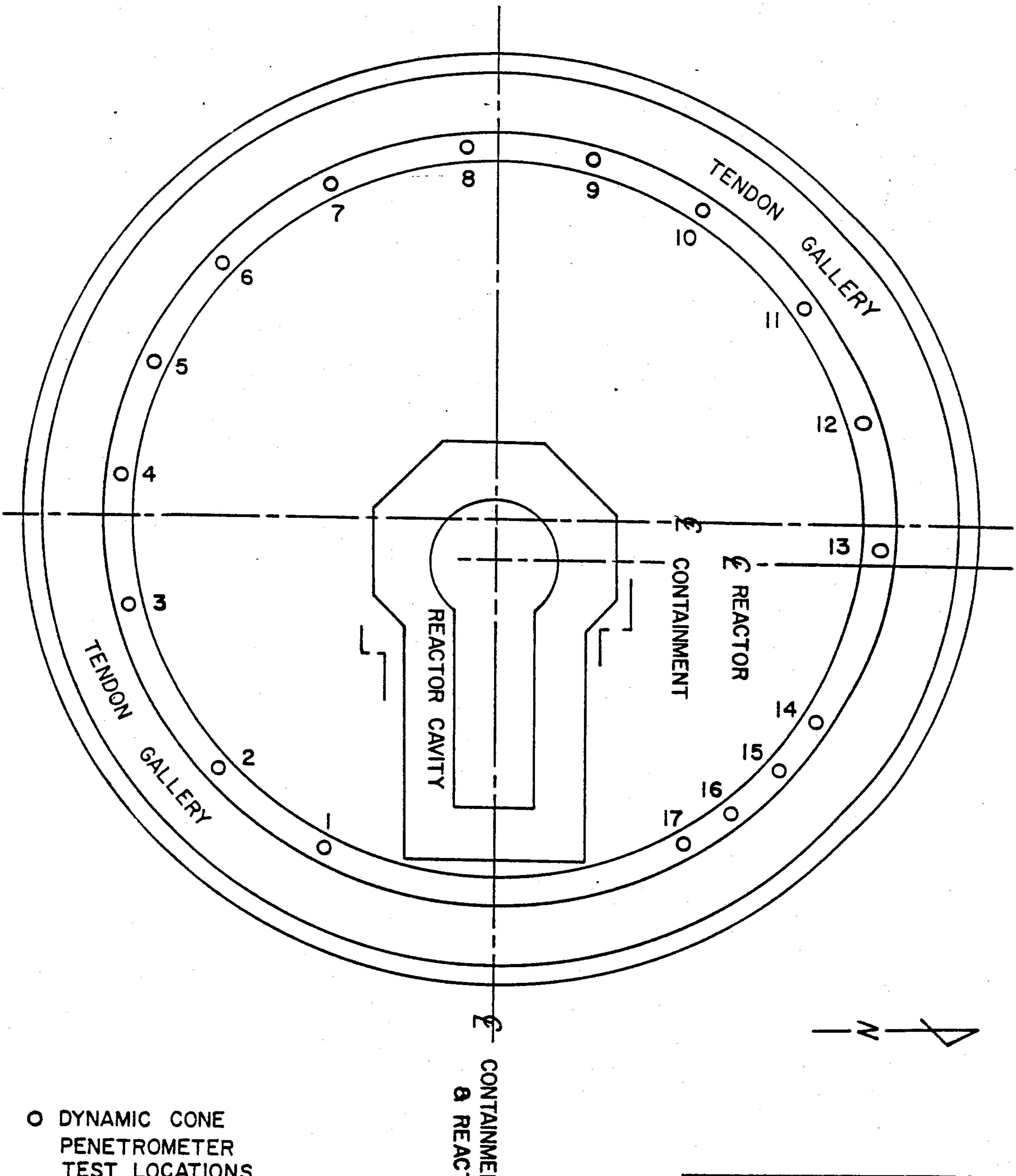
DYNAMIC CONE
PENETROMETER RESISTANCE
VERSUS DEPTH ALONG
ELECTRICAL TUNNEL EAST WALL

GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
ELECTRICAL TUNNEL EAST WALL DCP RESISTANCE VERSUS DEPTH		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 7	



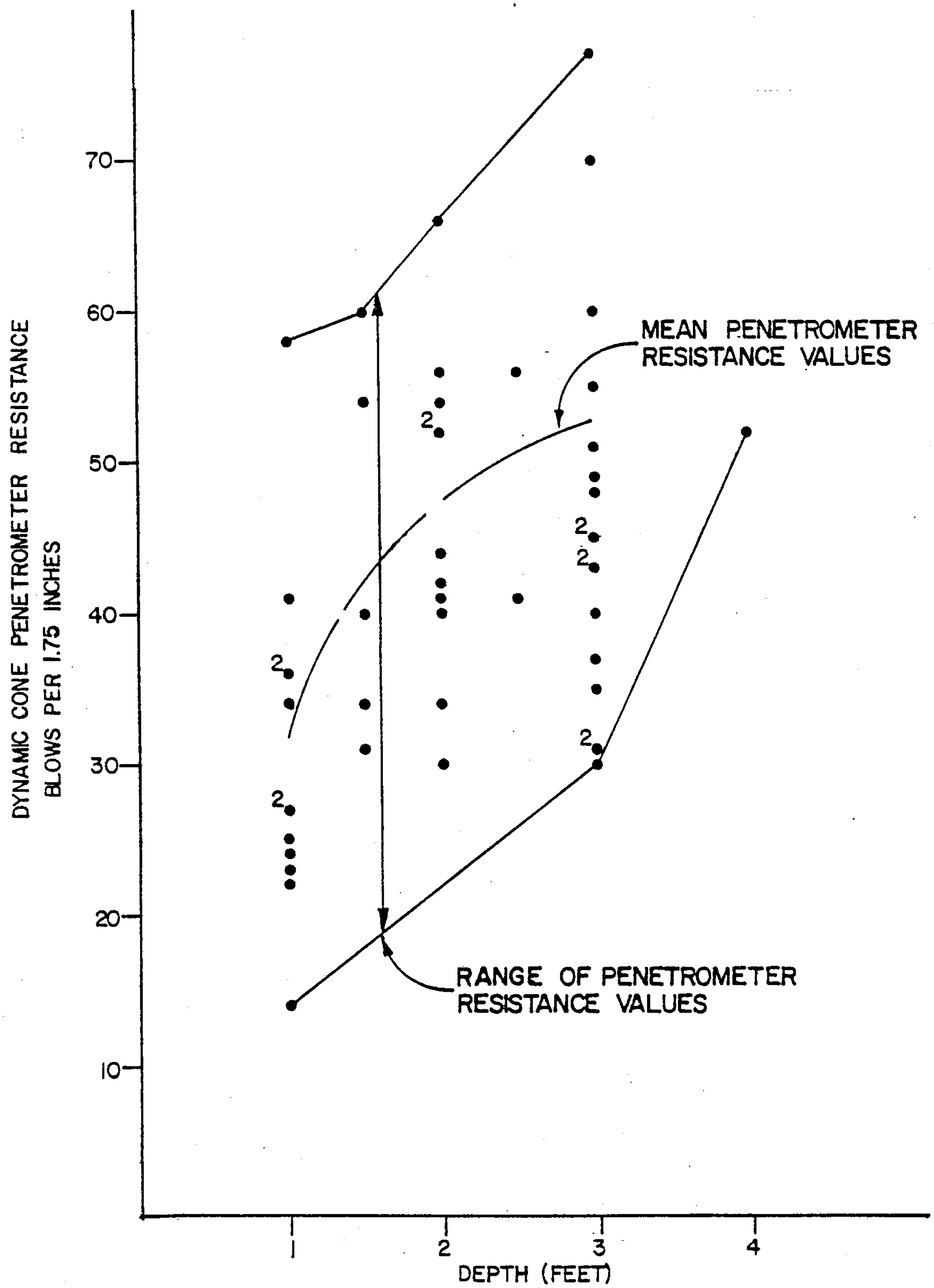
TYPICAL SECTION SHOWING EXTENT OF DISTURBED ZONE REMOVED IN AREA BETWEEN UNIT I ELECTRICAL TUNNEL AND CONTAINMENT.

GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
TYPICAL SECTION OF REMOVED DISTURBED ZONE BETWEEN UNIT #I CONTAINMENT & UNIT #I ELEC. TUNNEL		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 8	

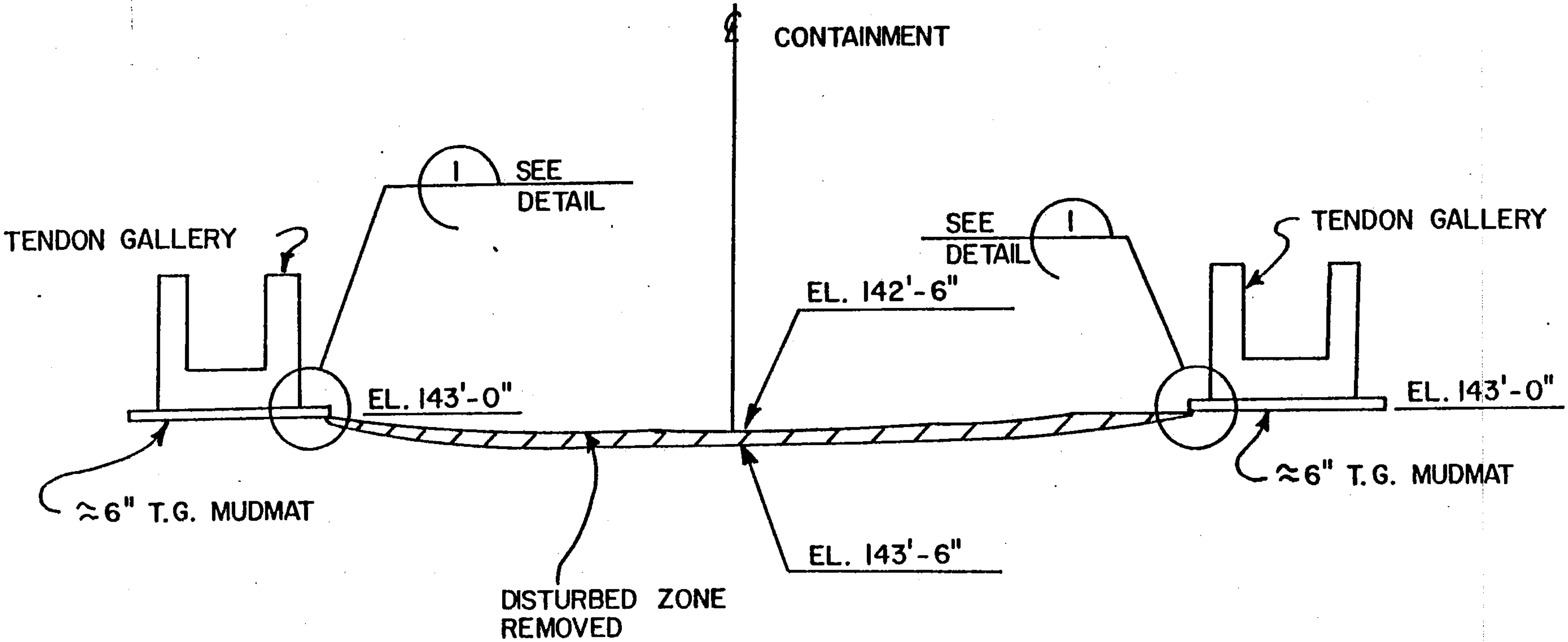


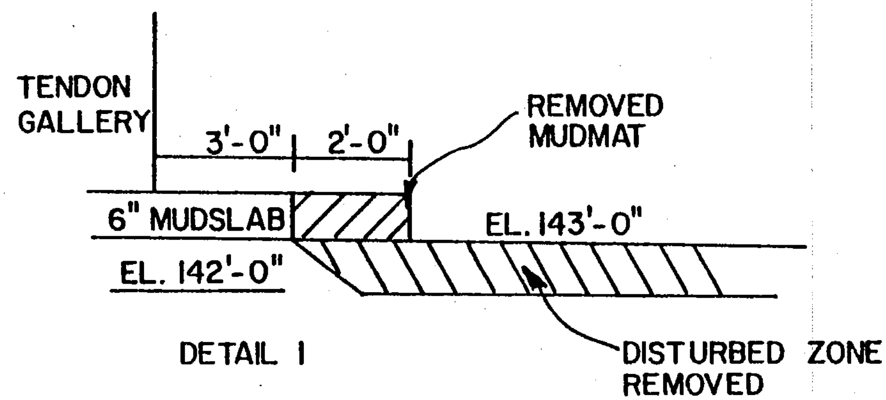
O DYNAMIC CONE PENETROMETER TEST LOCATIONS

GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
DCP TEST LOCATIONS ALONG UNIT I TENDON GALLERY		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 9	



GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
DYNAMIC CONE PENETRATOR RESISTANCE VERSUS DEPTH FOR UNIT I TENDON GALLERY		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 10	

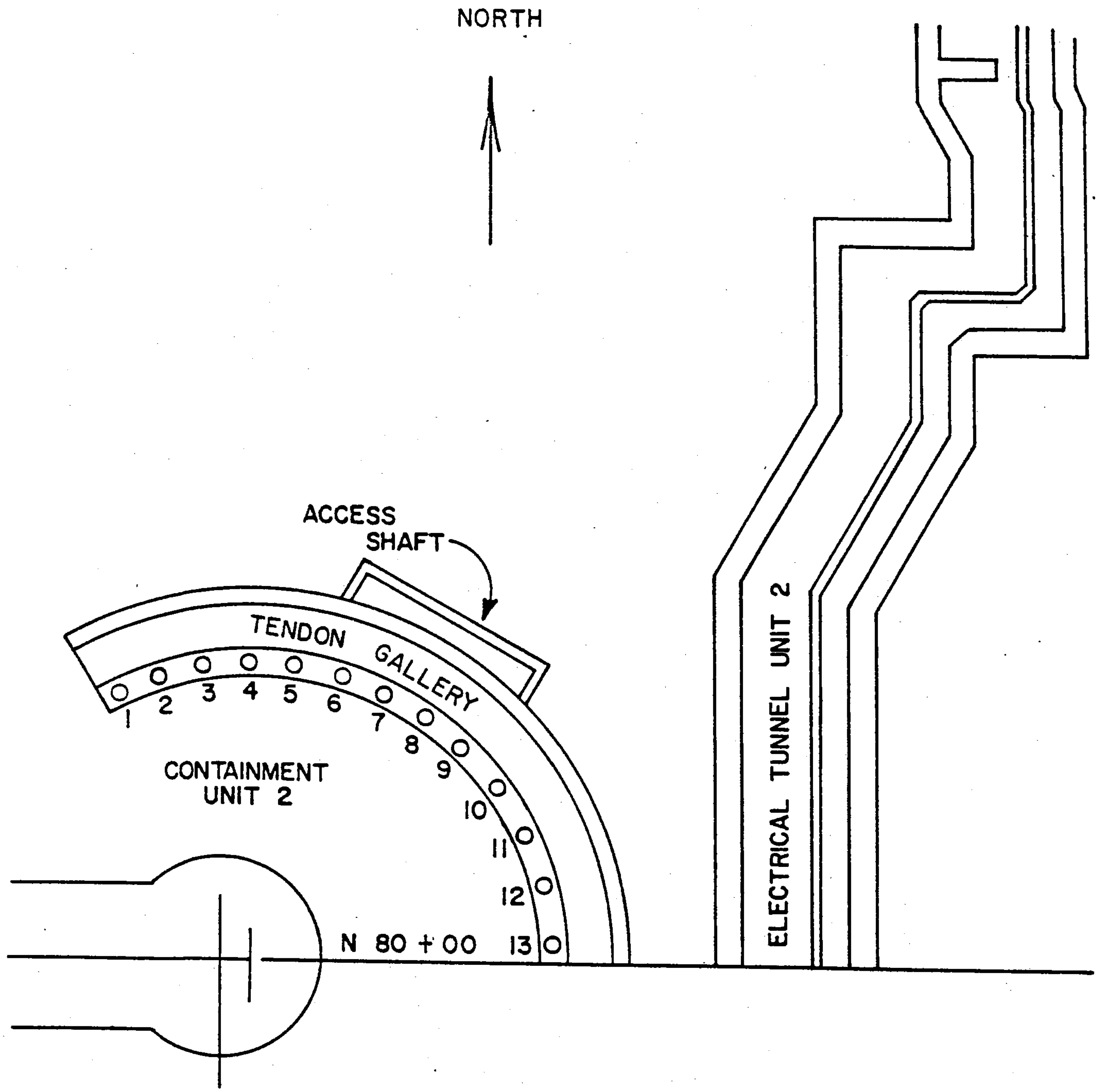




DETAIL 1

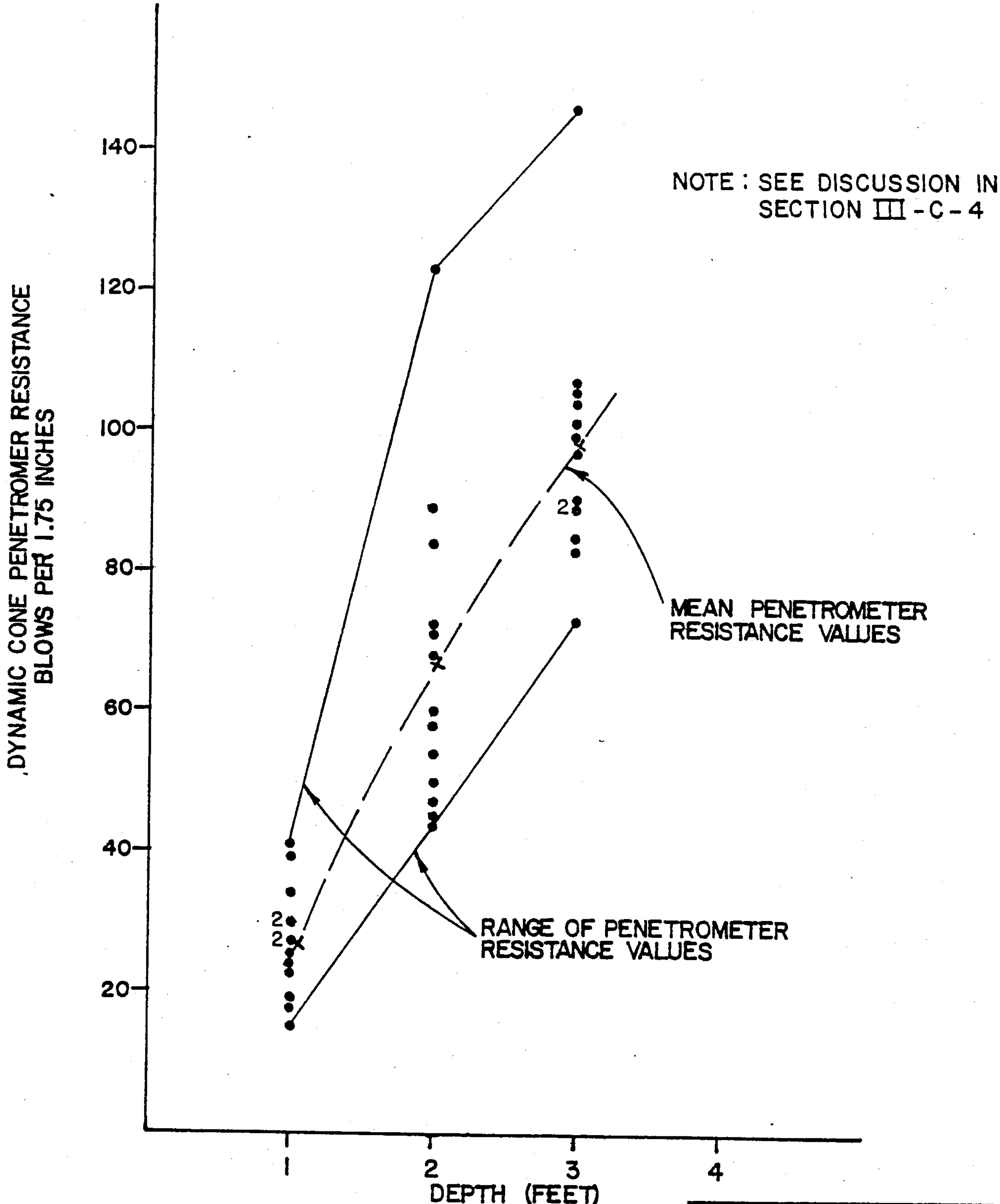
NOT TO SCALE

GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
TYPICAL CROSS SECTION IN UNIT 1 CONTAINMENT AREA SHOWING EXTENT OF DISTURBED ZONE REMOVED		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 11	

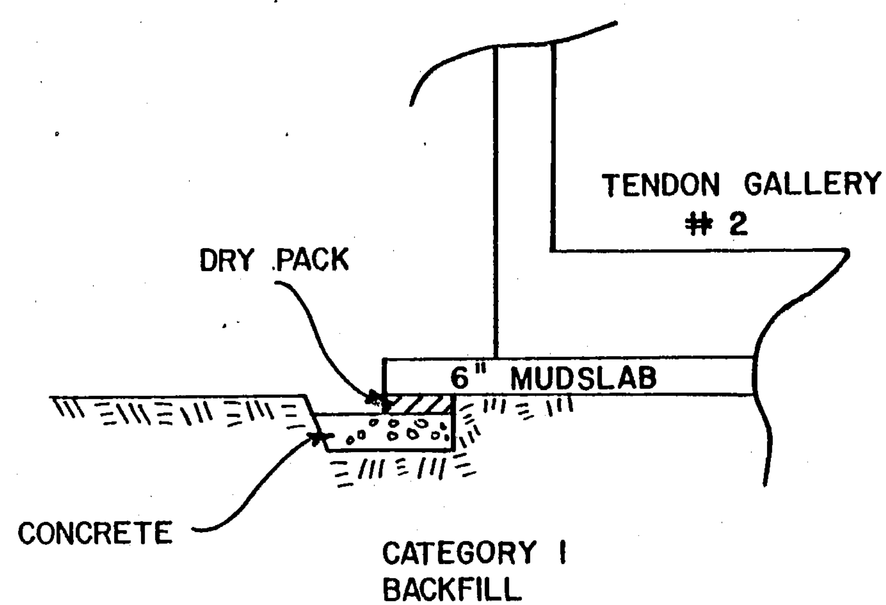


O DYNAMIC CONE PENETROMETER TEST LOCATIONS

GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
DCP TEST LOCATIONS ALONG UNIT 2 TENDON GALLERY		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 12	

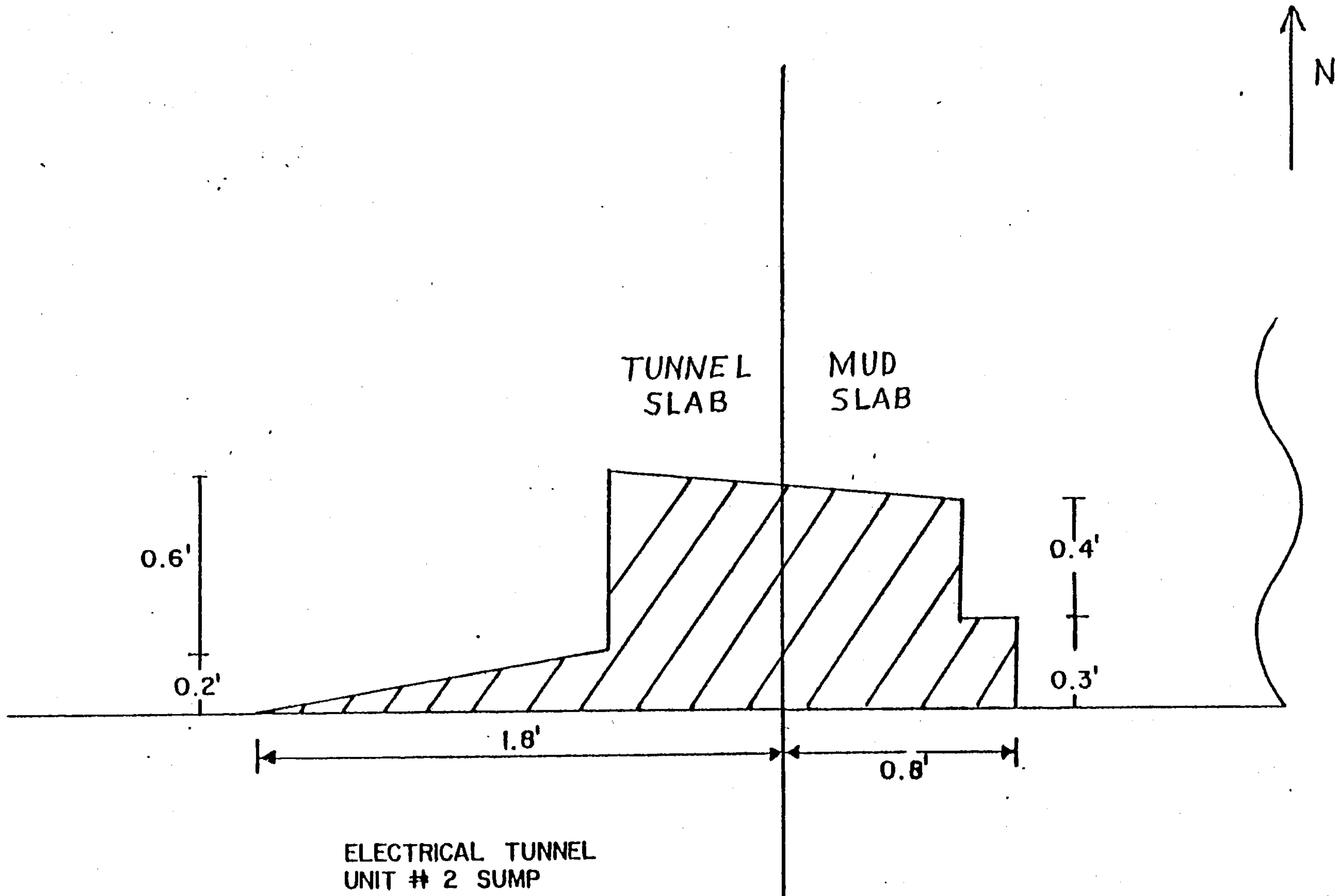


GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
DYNAMIC CONE PENETROMETER RESISTANCE VERSUS DEPTH FOR UNIT 2 TENDON GALLERY		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 13	

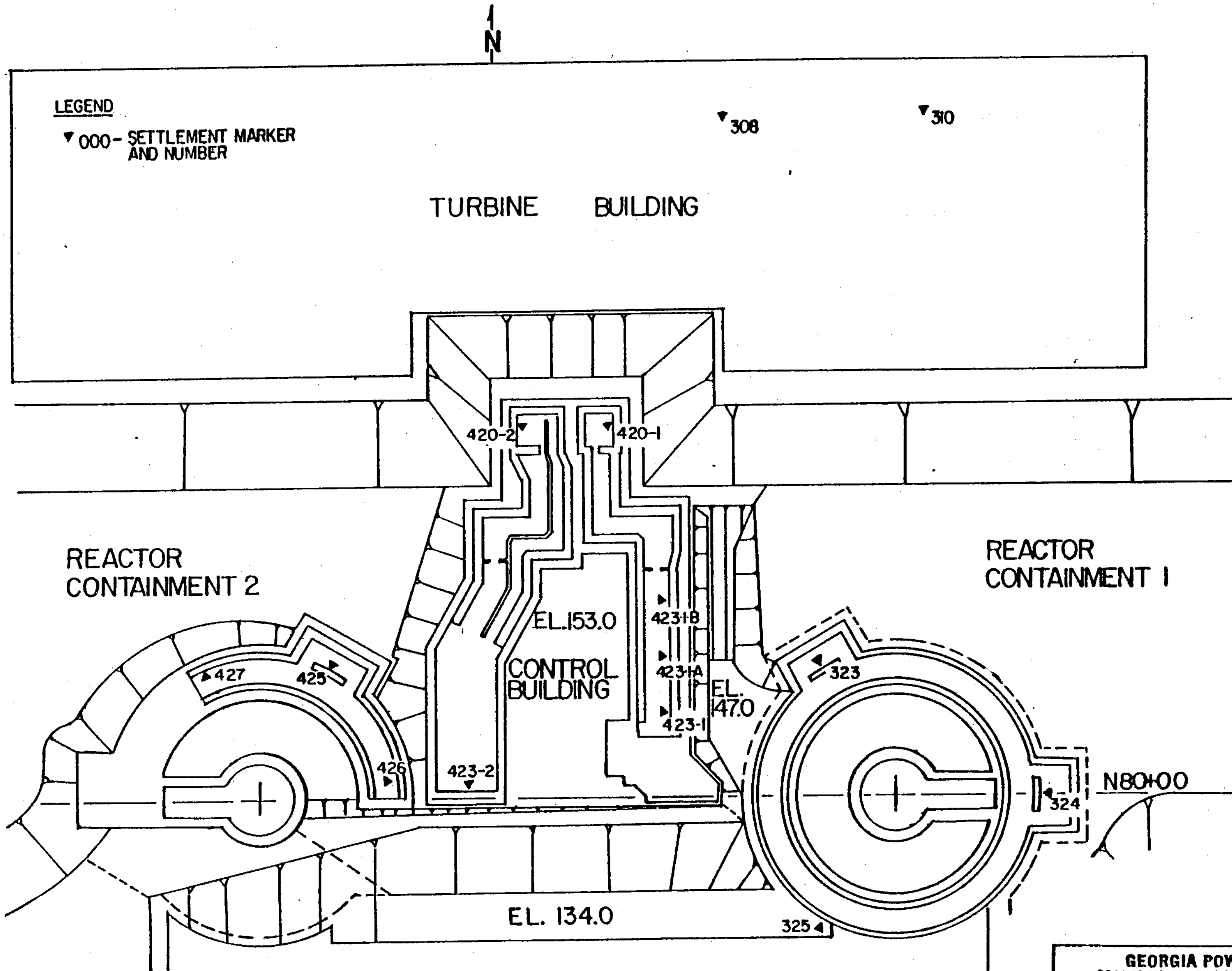


NOT TO SCALE

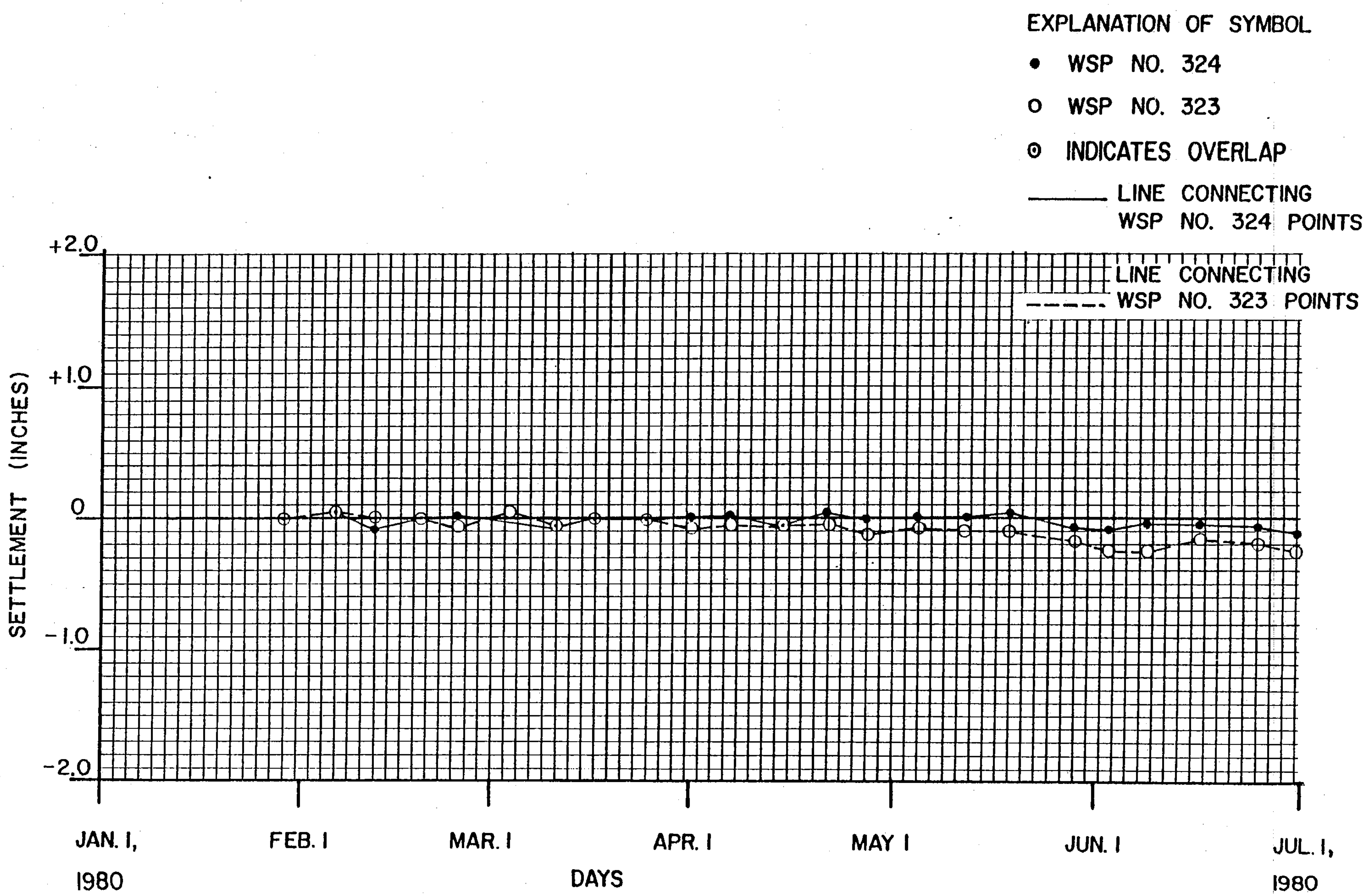
GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
SKETCH SHOWING PROCEDURE USED TO REPAIR UNIT 2 TENDON GALLERY MUDSLAB		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 14	



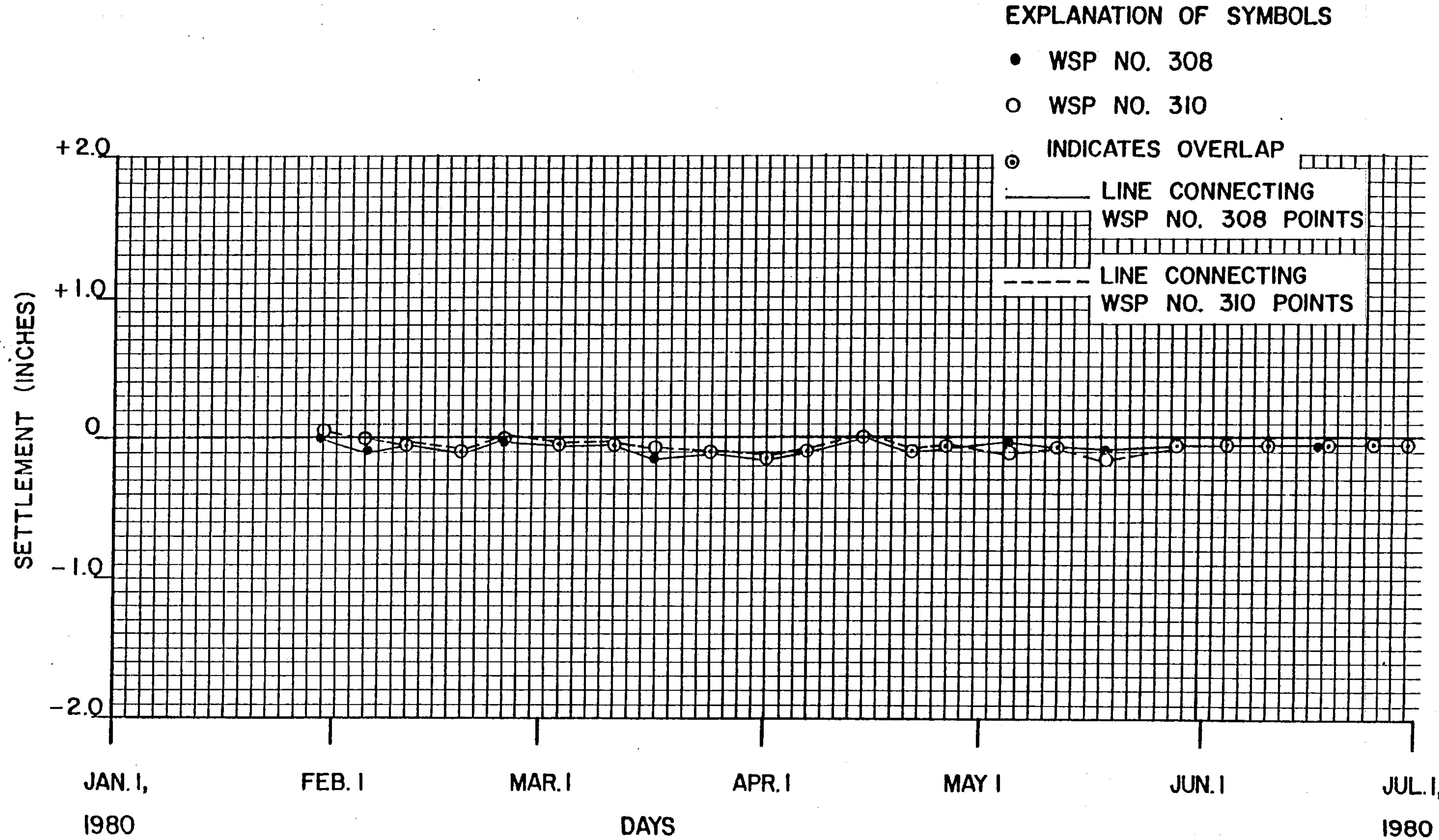
GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
PLAN SHOWING EROSION IN JULY 1980 UNIT 2 ELECTRICAL TUNNEL MUDSLAB		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 15	



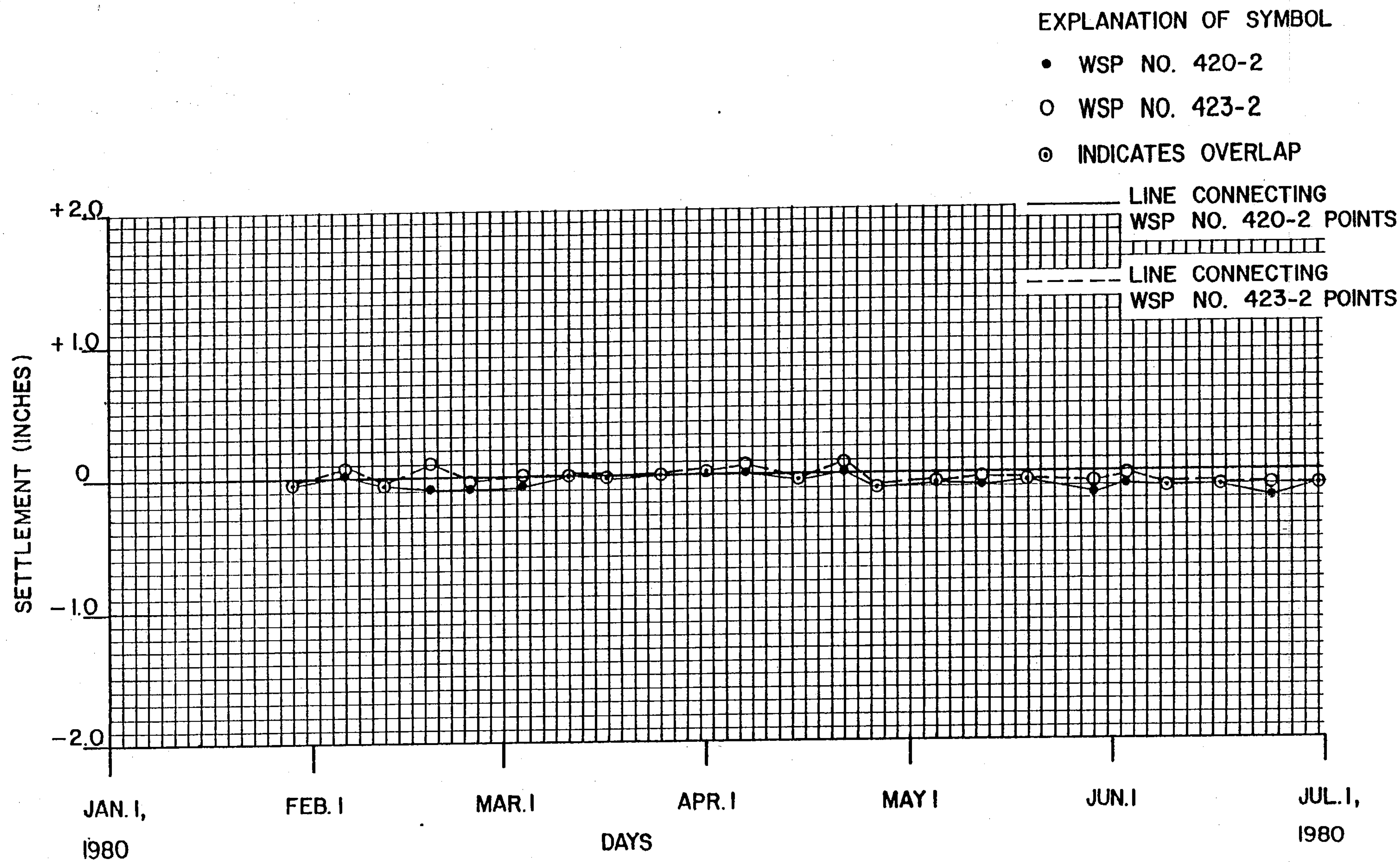
GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
PLAN SHOWING LOCATIONS OF SETTLEMENT MARKER POINTS		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 16	



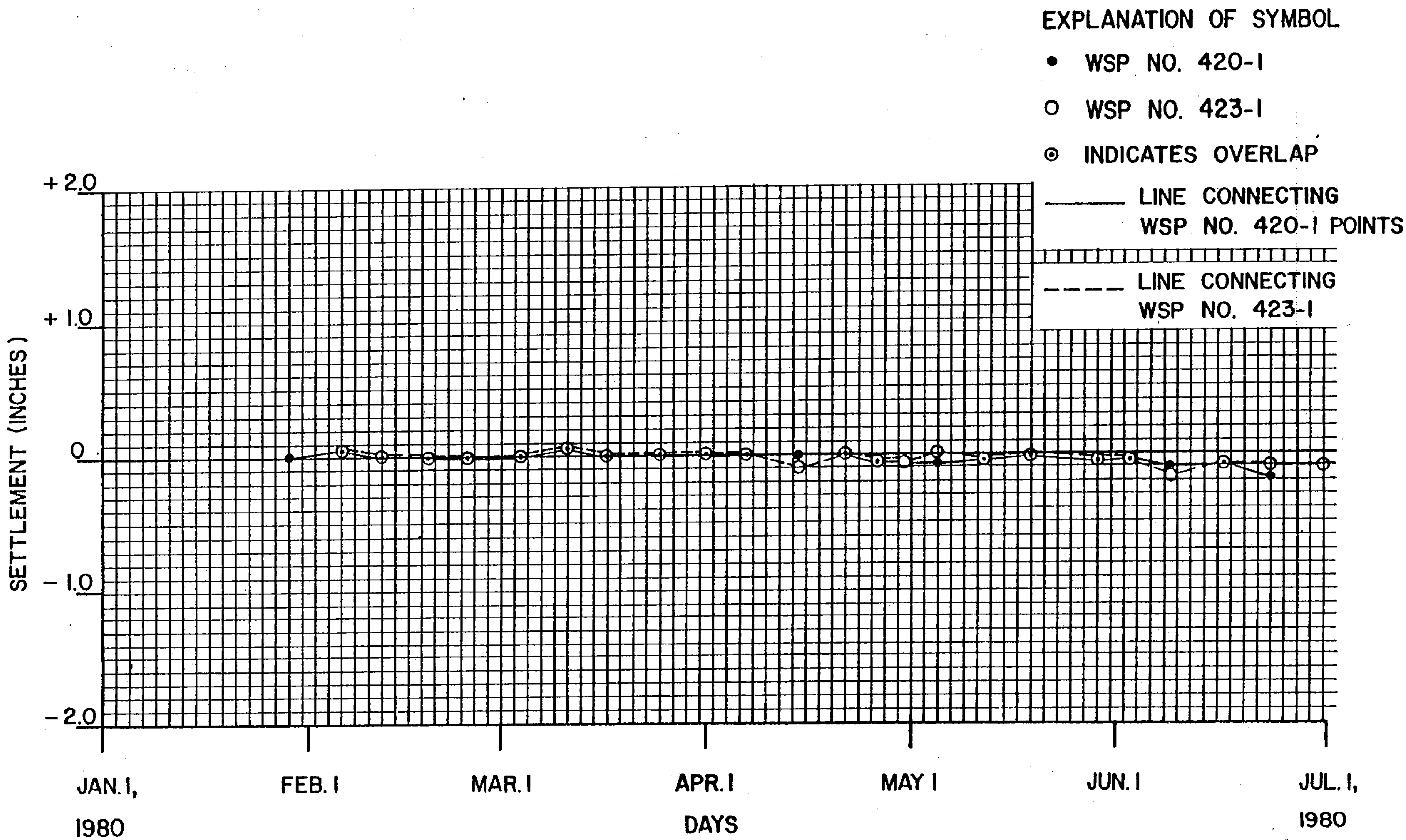
GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
SETTLEMENT VERSUS TIME FOR CONTAINMENT UNIT 1		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 16-1	



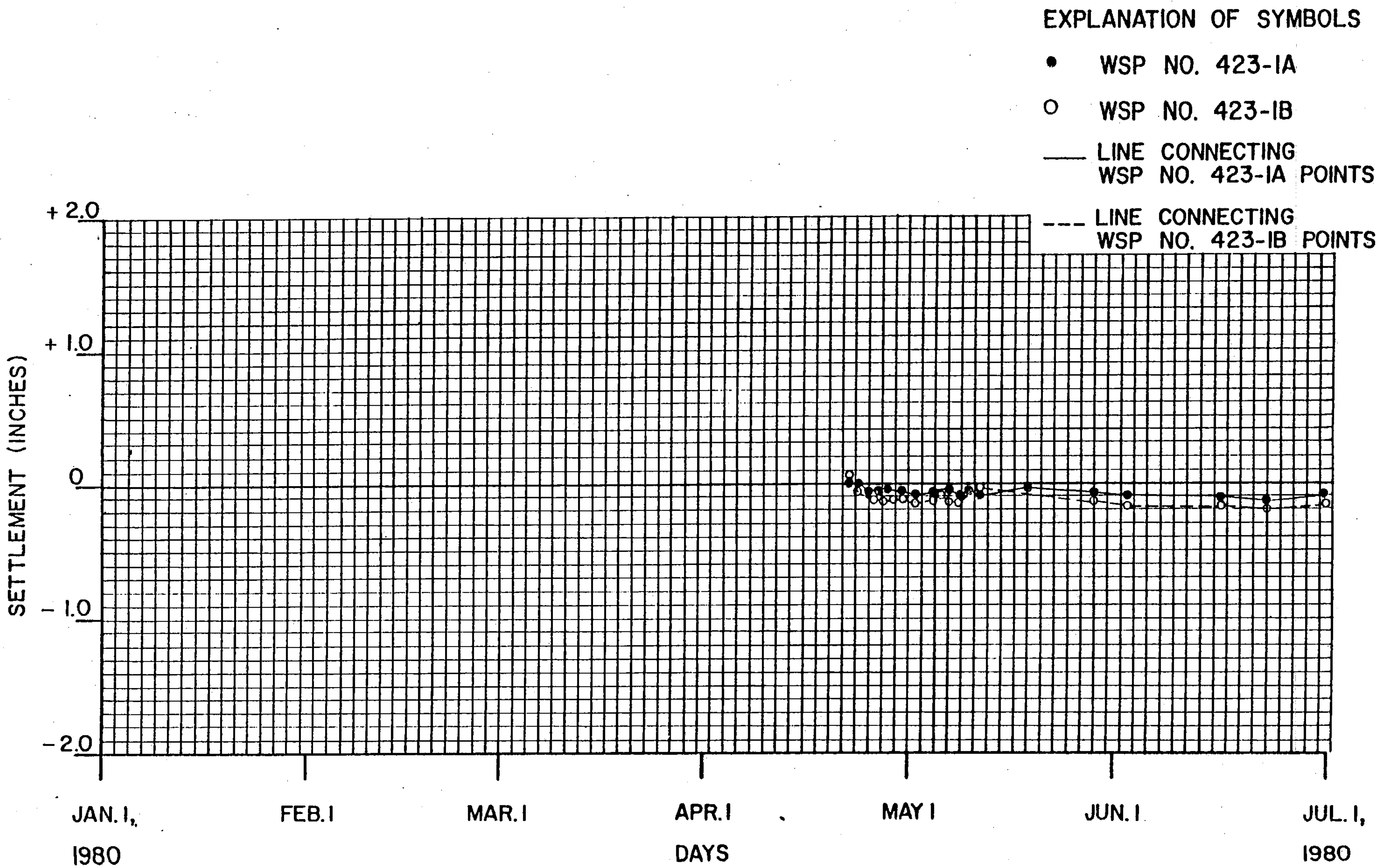
GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
SETTLEMENT VERSUS TIME FOR TURBINE BUILDING UNITS 1 & 2		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 16-2	



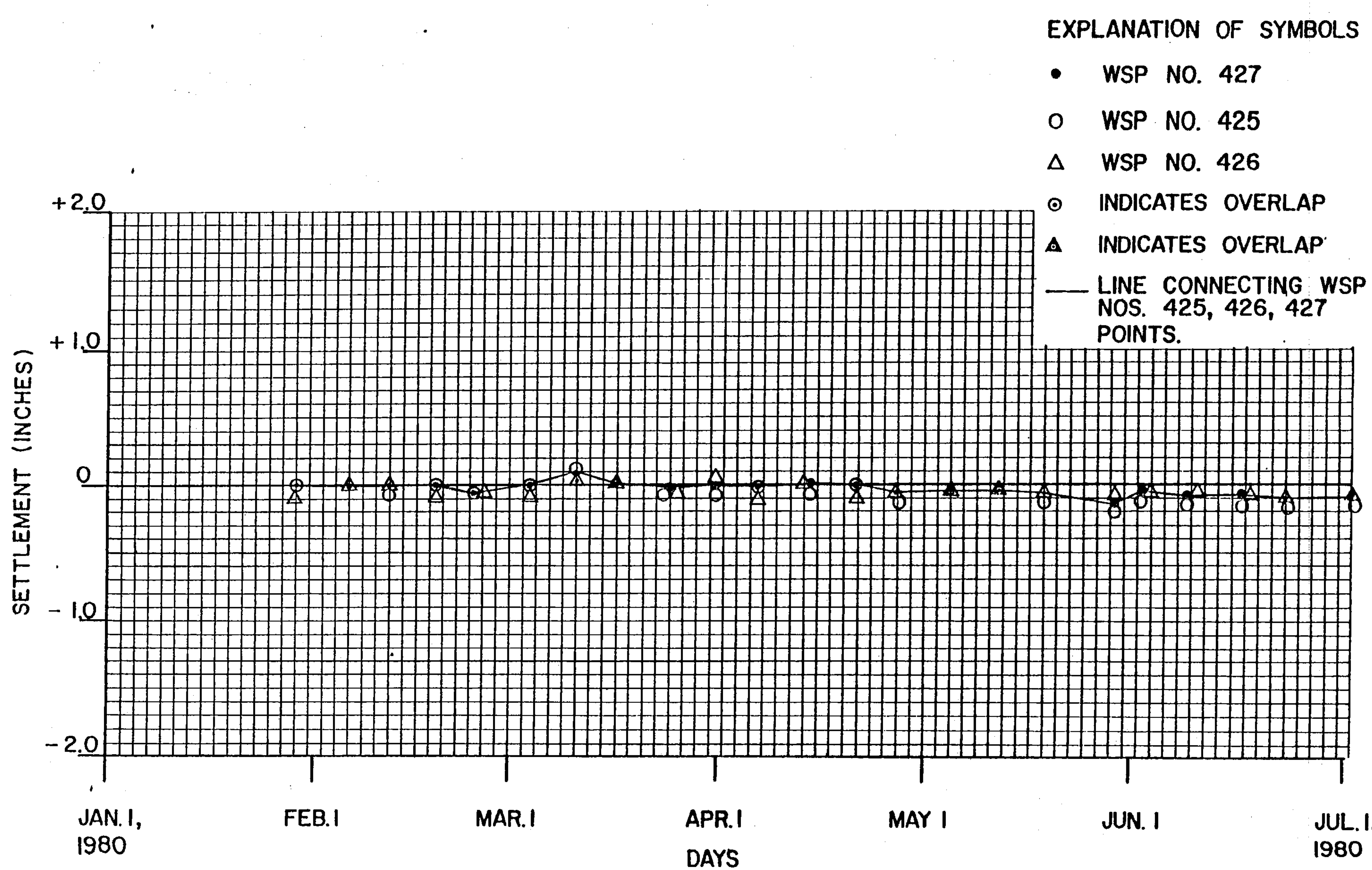
GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
SETTLEMENT VERSUS TIME FOR ELECTRICAL TUNNEL UNIT 2		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 16-3	



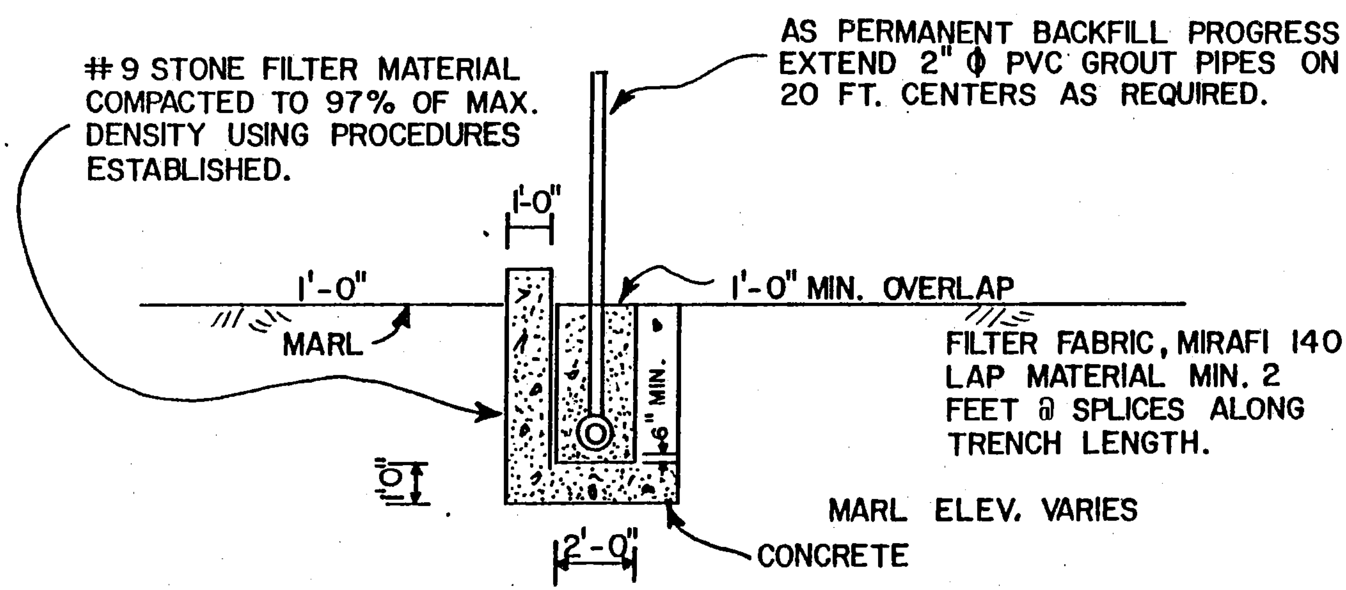
GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT	
SETTLEMENT VERSUS TIME FOR ELECTRICAL TUNNEL UNIT 1	
SCALE:	DRAWING NO. / REV.
JOB NO. 9510	FIGURE 16-4A



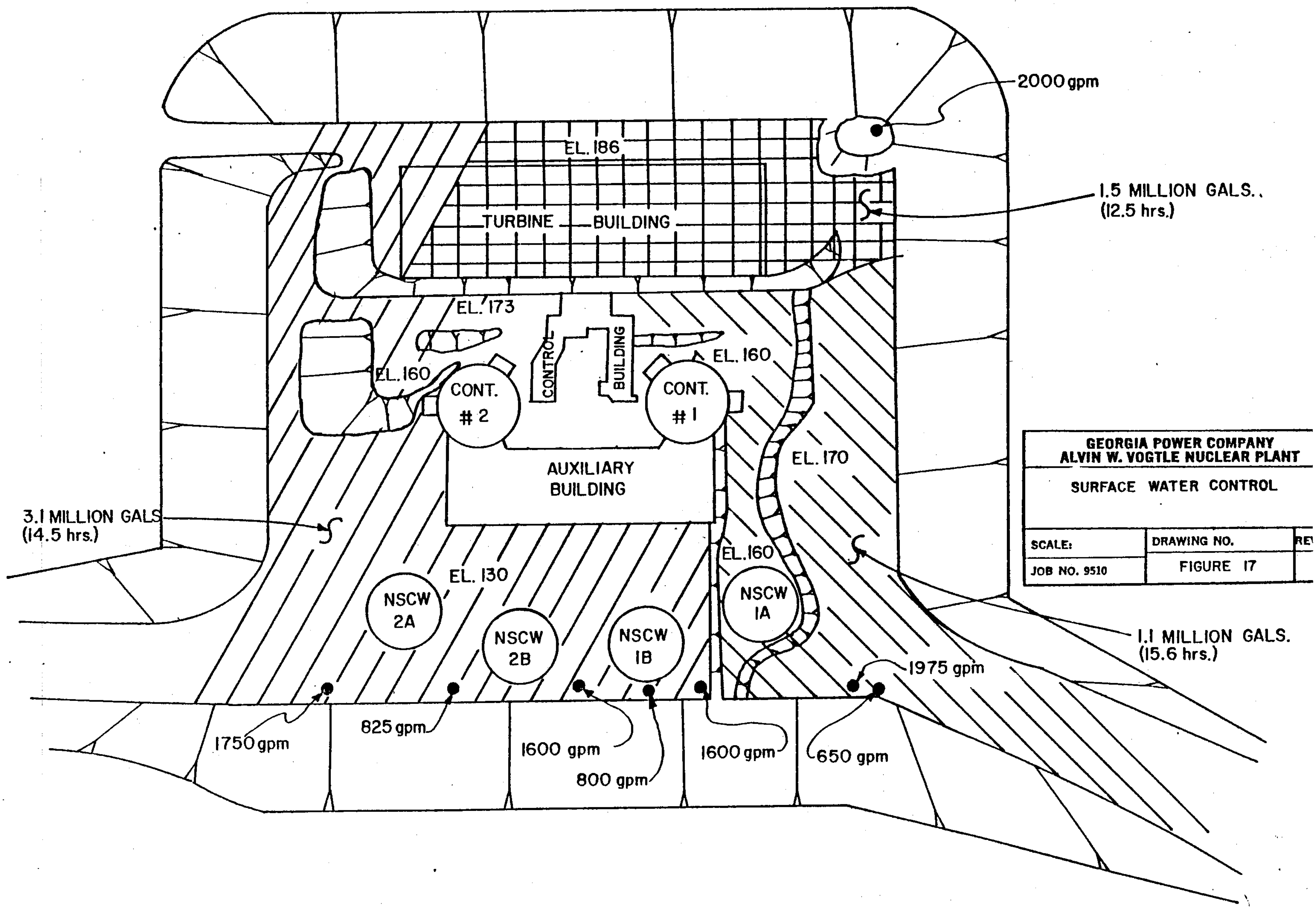
GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
SETTLEMENT VERSUS TIME FOR ELECTRICAL TUNNEL UNIT 1		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 16-4B	



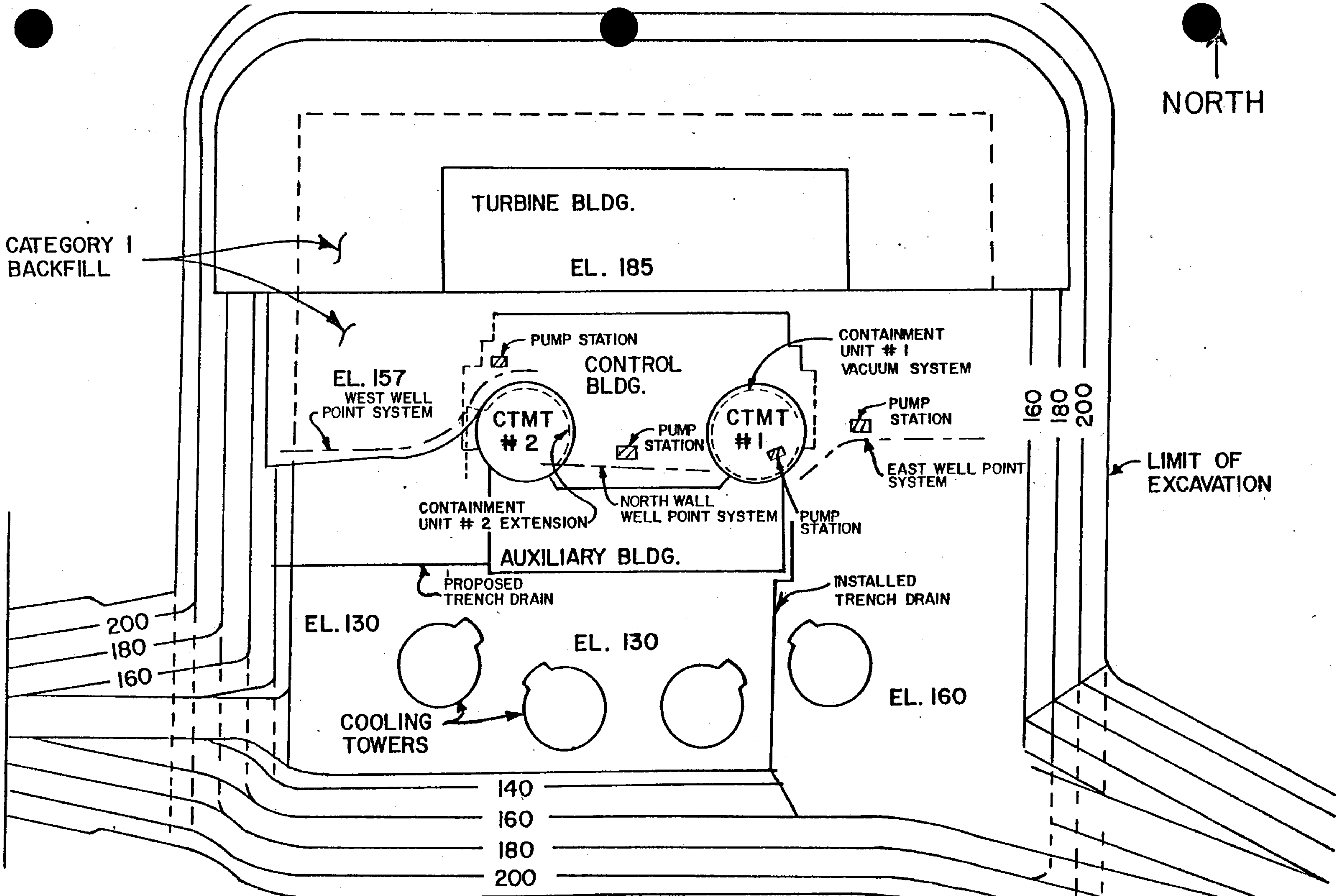
GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
SETTLEMENT VERSUS TIME FOR CONTAINMENT UNIT 2		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 16-5	



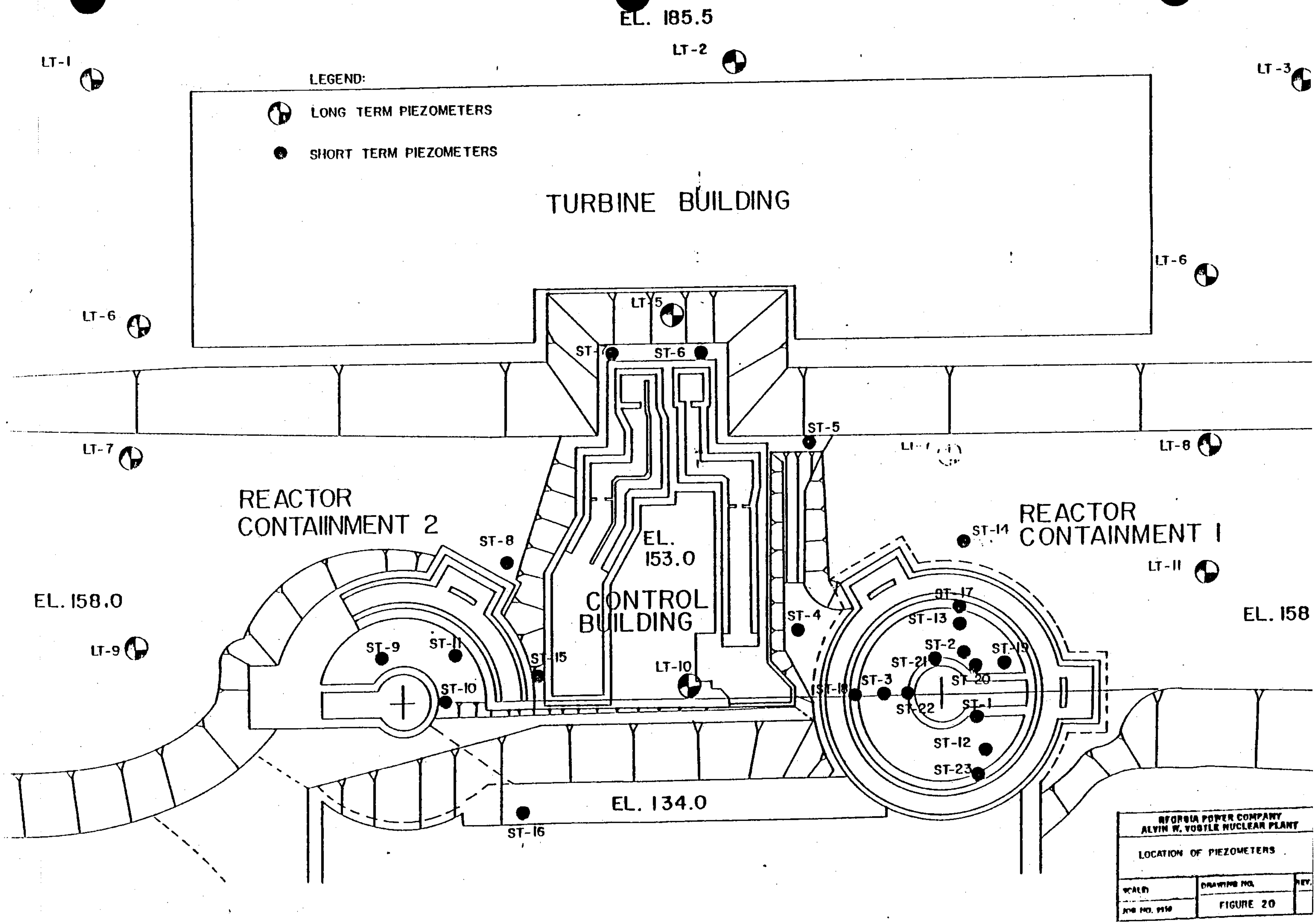
GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
TRENCH DRAIN TYPICAL SECTION		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 18	

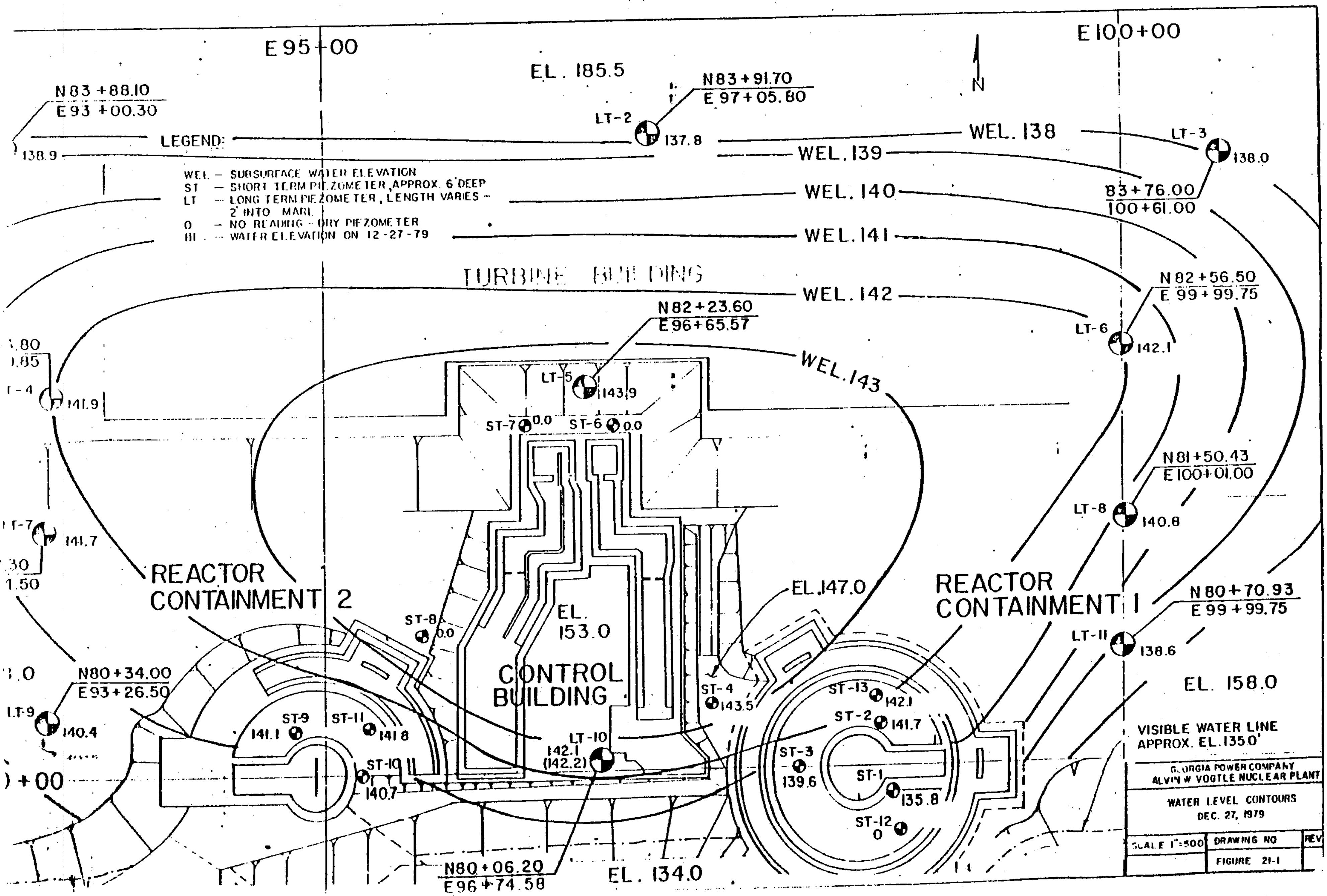


GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
SURFACE WATER CONTROL		
SCALE:	DRAWING NO.	REV
JOB NO. 9510	FIGURE 17	

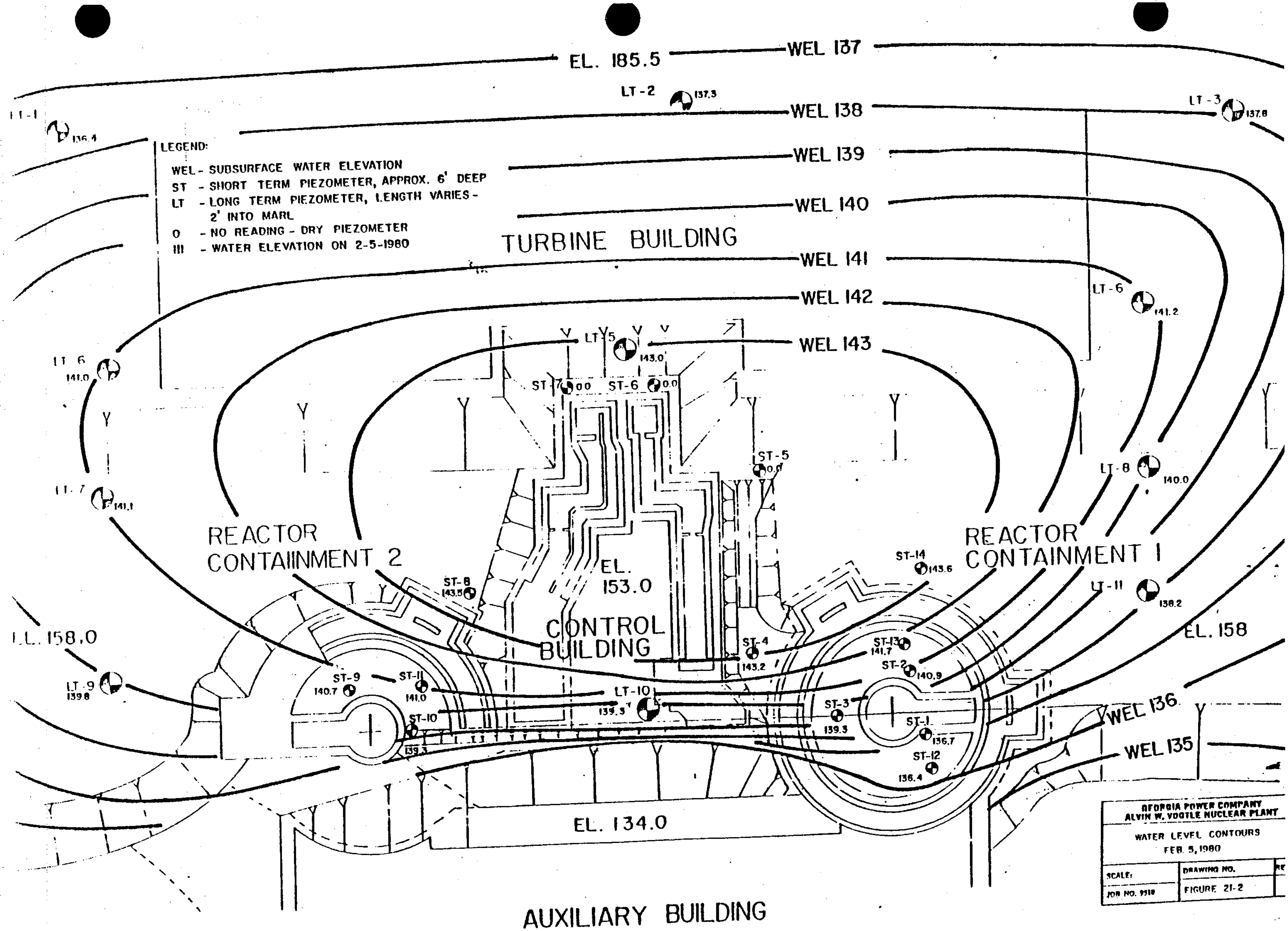


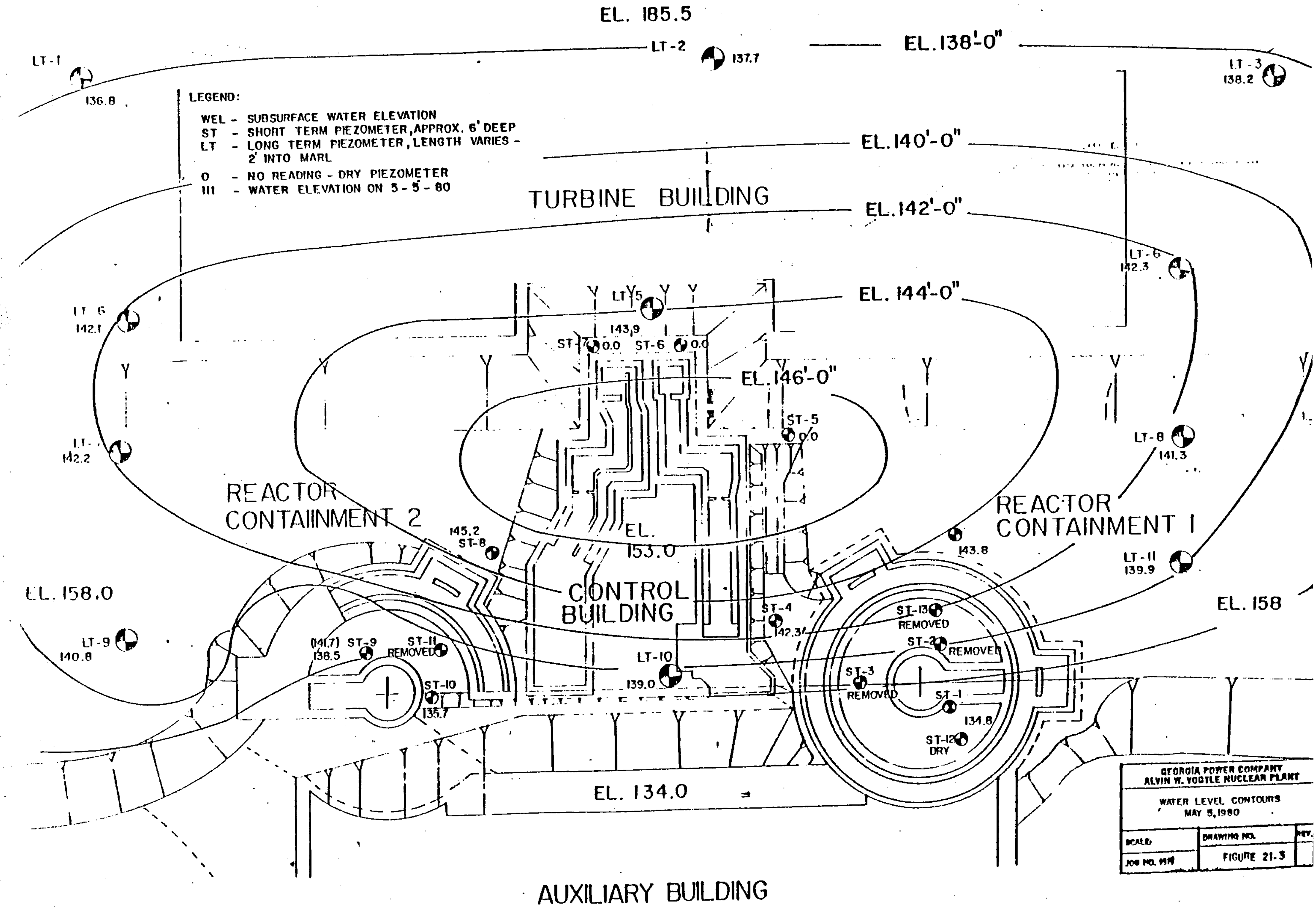
GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
LOCATION OF DEWATERING SYSTEMS		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 19	


EL. 185.5
LT-1
LT-2
LT-3
LEGEND:
LONG TERM PIEZOMETERS
SHORT TERM PIEZOMETERS
TURBINE BUILDING
LT-6
LT-6
LT-5
ST-7
ST-6
ST-5
LT-7
LT-8
REACTOR CONTAINMENT 2
ST-8
EL. 153.0
CONTROL BUILDING
ST-14
REACTOR CONTAINMENT 1
LT-11
EL. 158.0
EL. 158
ST-17
ST-13
ST-4
ST-2
ST-19
ST-21
ST-20
ST-3
ST-18
ST-22
ST-1
ST-12
ST-23
LT-9
ST-9
ST-11
ST-15
ST-10
LT-10
EL. 134.0
ST-16
GEORGIA POWER COMPANY
ALVIN W. VOGTLE NUCLEAR PLANT
LOCATION OF PIEZOMETERS
FIGURE 20



E 95+00
E 100+00
EL. 185.5
N
N83 +88.10
E93 +00.30
138.9
LEGEND:
WEL. – SUBSURFACE WATER ELEVATION
ST – SHORT TERM PIEZOMETER, APPROX. 6' DEEP
LT – LONG TERM PIEZOMETER, LENGTH VARIES – 2' INTO MARL
0 – NO READING – DRY PIEZOMETER
III – WATER ELEVATION ON 12-27-79
LT-2
137.8
N83+91.70
E 97+05.80
WEL. 138
WEL. 139
WEL. 140
WEL. 141
WEL. 142
WEL. 143
LT-3
138.0
83+76.00
100+61.00
TURBINE BUILDING
N82+56.50
E 99+99.75
LT-6
142.1
N82+23.60
E96+65.57
LT-5
143.9
ST-7
0.0
ST-6
0.0
LT-4
141.9
LT-7
141.7
N81+50.43
E100+01.00
LT-8
140.8
REACTOR CONTAINMENT 2
REACTOR CONTAINMENT 1
EL. 147.0
ST-8
0.0
EL. 153.0
CONTROL BUILDING
N80+70.93
E 99+99.75
LT-11
138.6
EL. 158.0
N80+34.00
E93+26.50
LT-9
140.4
ST-9
141.1
ST-11
141.8
ST-10
140.7
LT-10
142.1
(142.2)
ST-4
143.5
ST-13
142.1
ST-2
141.7
ST-3
139.6
ST-1
135.8
ST-12
0
VISIBLE WATER LINE APPROX. EL. 135.0'
N80+06.20
E96+74.58
EL. 134.0
GEORGIA POWER COMPANY
ALVIN W. VOGTLE NUCLEAR PLANT
WATER LEVEL CONTOURS
DEC. 27, 1979
SCALE 1"=500
DRAWING NO
FIGURE 21-1
REV


EL. 185.5
WEL 137
WEL 138
WEL 139
WEL 140
WEL 141
WEL 142
WEL 143
WEL 136
WEL 135
LT-1 136.4
LT-2 137.3
LT-3 137.8
LEGEND:
WEL - SUBSURFACE WATER ELEVATION
ST - SHORT TERM PIEZOMETER, APPROX. 6' DEEP
LT - LONG TERM PIEZOMETER, LENGTH VARIES - 2' INTO MARL
0 - NO READING - DRY PIEZOMETER
III - WATER ELEVATION ON 2-5-1980
TURBINE BUILDING
LT-6 141.2
LT 6 141.0
LT-5 143.0
ST-7 0.0
ST-6 0.0
ST-5 0.0
LT-7 141.1
LT-8 140.0
REACTOR CONTAINMENT 2
REACTOR CONTAINMENT 1
EL. 153.0
CONTROL BUILDING
ST-8 143.5
ST-14 143.6
LT-11 138.2
EL. 158.0
EL. 158
LT-9 139.8
ST-9 140.7
ST-11 141.0
ST-10 139.3
LT-10 139.3
ST-4 143.2
ST-13 141.7
ST-2 140.9
ST-3 139.3
ST-1 136.7
ST-12 136.4
EL. 134.0
AUXILIARY BUILDING
GEORGIA POWER COMPANY
ALVIN W. VOGTLE NUCLEAR PLANT
WATER LEVEL CONTOURS
FEB. 5, 1980
SCALE:
JOB NO. 9510
DRAWING NO.
FIGURE 21-2



EL. 185.5
LT-1
136.8
LT-2
137.7
EL.138'-0"
LT-3
138.2
LEGEND:
WEL - SUBSURFACE WATER ELEVATION
ST - SHORT TERM PIEZOMETER, APPROX. 6' DEEP
LT - LONG TERM PIEZOMETER, LENGTH VARIES - 2' INTO MARL
0 - NO READING - DRY PIEZOMETER
111 - WATER ELEVATION ON 5-5-80
EL.140'-0"
TURBINE BUILDING
EL.142'-0"
LT-6
142.3
LT 6
142.1
LT-5
EL. 144'-0"
143.9
ST-7
0.0
ST-6
0.0
EL.146'-0"
ST-5
0.0
LT-
142.2
LT-8
141.3
REACTOR CONTAINMENT 2
REACTOR CONTAINMENT 1
145.2
ST-8
EL. 153.0
143.8
LT-11
139.9
EL. 158.0
CONTROL BUILDING
ST-4
142.3
ST-13
REMOVED
EL. 158
LT-9
140.8
(141.7)
ST-9
138.5
ST-11
REMOVED
ST-2
REMOVED
LT-10
139.0
ST-3
REMOVED
ST-10
135.7
ST-1
134.8
ST-12
DRY
EL. 134.0
GEORGIA POWER COMPANY
ALVIN W. VOGTLE NUCLEAR PLANT
WATER LEVEL CONTOURS
MAY 5, 1980
SCALE
DRAWING NO.
REV.
JOB NO.
FIGURE 21-3
AUXILIARY BUILDING

TABLE 1

STANDARD PENETRATION TEST, DYNAMIC CONE PENETROMETER TEST, CALIBRATION DATA

a) Summary of Dynamic Cone Penetrometer Test Data

Depth (ft.)	Test Designation									
	CP-1	CP-2	CP-3	CP-4	CP-5	CP-6	CP-7	CP-8	CP-9	CP-10
1.0	26	26	27	29	25	24	24	33	17	19
1.5	31	31	34	34	30	38	31	45	29	--
2.0	40	38	40	36	55	42	46	46	48	43
3.0	56	58	62	51	57	49	46	57	54	69
3.5	62	54	70	55	60	64	--	--	--	--
4.0	62	70	62	55	60	69	47	52	66	76

b) Summary of Standard Penetration Test Data

Depth (ft.)	Test Designation					
	SPT-1	SPT-2	SPT-3	SPT-4	SPT-5	SPT-6
0.5-1 (set)	6	5	5	7	6	7
1.0	24	26	25	26	27	26
2-2.5 (set)	6	15	14	16	14	15
2.5	59	55	55	57	57	57
3.5-4 (set)	20	21	21	25	21	22
4.0	86	97	96	94	89	87

c) Correlation Curve Values

Depth (ft.)	Average SPT Values, Blows/Ft., Np	Average DCP Values, Blows/1.75 Inches Nc	Remarks
1.0	26	25	Values Plotted in Figure 3
1.5	38*	34	
2.0	47*	44	
3.0	69*	56	
3.5	80*	60	
4.0	92	62	

*interpolated values

γ_w = Wet Density
W = Moisture Content
γ_d = Dry Density
γ_d(max) = Maximum Proctor Dry Density
OMC = Optimum Moisture Content

TABLE 2

SUMMARY OF SAND CONE DENSITY TEST DATA

Test No.	Elev. (Ft.)	Coordinates N	Coordinates E	Field Test γ_w (pcf)	Field Test W (%)	Field Test γ_d (pcf)	Laboratory Test γ_d (max) (pcf)	Laboratory Test OMC (%)	Percent Compaction	Remarks
UNIT 1 CONTAINMENT										
1644	141.8	79+79	98+74	120.7	11.2	108.5	108.9	12.2	99.6	Test Nos. 1644 through 1658, 1722 through 1731, 1734 through 1739, 1744 and 1774 were performed adjacent to the Unit 1 Tendon Gallery foundation below the mudslab. Test Nos. 1659, 1682 and 1684 were performed north of Reactor Cavity to determine extent of disturbed zone. Test Nos. 1680 and 1683 were performed south of the Reactor Cavity. Areas represented by Test Nos. 1659 and 1683 were excavated down to lean concrete fill and then backfilled.
1645	141.6	79+71	98+60	120.3	8.8	110.6	107.0	13.0	103.4	
1646	141.4	79+69	98+42	122.6	10.1	111.4	105.2	13.0	105.9	
1647	141.6	79+60	98+26	121.1	9.3	110.8	106.7	12.5	103.8	
1648	142.1	79+55	98+12	125.2	10.1	113.7	108.3	11.2	105.0	
1649	142.5	79+66	97+96	123.1	11.8	110.1	105.7	12.8	104.2	
1650	142.9	79+80	97+90	127.3	10.7	114.9	109.2	12.3	105.2	
1651	142.1	79+96	97+81	126.5	13.6	111.9	105.6	13.1	106.0	
1652	142.5	80+13	97+82	124.1	16.5	106.5	109.9	11.8	96.9	
1653	142.4	80+30	97+90	127.1	15.1	110.4	107.5	14.5	102.7	
1654	142.8	80+38	98+05	125.2	15.2	108.9	105.3	11.2	103.4	
1655	142.4	80+50	98+18	123.7	15.0	107.6	105.7	13.9	101.8	
1656	142.6	80+50	98+53	126.6	13.5	111.5	107.0	12.8	104.2	
1657	142.7	80+53	98+35	124.7	16.0	107.5	105.0	13.5	102.4	
1658	142.0	80+41	98+67	126.5	16.2	108.9	108.0	12.8	100.8	
1659	141.8	80+29	98+80	114.4	13.2	101.1	107.3	12.4	94.2	
1680	137.1	79+81	98+57	128.3	14.6	112.0	106.2	13.8	105.5	
1682	138.8	80+21	98+58	116.7	10.8	105.3	106.3	11.5	99.1	
1683	137.5	79+81	98+79	121.0	17.1	103.3	108.2	12.0	95.5	
1684	139.4	80+23	98+37	124.1	11.3	111.5	106.5	13.0	104.7	
1722	141.9	79+69	97+88	126.2	12.8	111.9	105.6	13.4	106.0	
1723	141.6	80+05	97+79	123.8	17.6	105.3	103.3	13.5	101.9	
1724	142.2	79+86	97+81	120.9	14.4	105.7	106.2	14.1	99.5	
1725	142.0	79+48	98+17	125.7	10.3	114.0	107.8	13.3	105.8	
1726	142.1	79+56	98+01	124.9	11.0	112.5	108.6	14.9	103.6	
1727	142.1	79+44	98+35	122.1	10.1	110.9	105.9	11.9	104.7	
1728	142.0	79+48	98+53	122.1	9.0	112.0	104.9	14.1	106.8	
1729	141.8	80+54	98+44	123.9	10.4	112.2	106.0	13.0	105.8	
1730	142.0	80+23	97+84	125.5	14.7	109.4	107.0	14.1	102.2	

...continued...

TABLE 2, continued

Summary of Sand Cone Density Test Data

Test No.	Elev. (Ft.)	Coordinates N	Coordinates E	Field Test γ_w (pcf)	Field Test W (%)	Field Test γ_d (pcf)	Laboratory Test γ_d (max) (pcf)	Laboratory Test OMC (%)	Percent Compaction	Remarks
1731	141.9	79+70	98+82	123.6	10.9	111.5	106.8	13.8	104.4	
1734	141.7	80+38	97+94	122.9	17.0	105.0	103.1	14.5	101.8	
1735	142.0	80+50	98+08	126.7	11.8	113.3	108.8	10.8	104.1	
1736	141.9	80+55	98+26	124.1	10.0	112.8	106.4	14.2	106.0	
1737	141.8	80+49	98+62	126.2	14.2	110.5	106.5	12.4	103.8	
1738	141.9	80+38	98+77	126.7	13.7	111.4	106.5	13.0	104.6	
1739	141.7	80+22	98+87	125.8	12.6	111.7	106.2	14.3	105.2	
1744	142.3	79+99	97+79	120.0	17.1	102.5	103.8	12.5	98.7	
1774	141.2	79+54	98+68	120.3	9.8	109.6	104.9	14.0	104.5	
UNIT 2 CONTAINMENT										
2095	141.4	80+47	94+69	125.5	17.2	107.1	103.9	15.0	103.1	Test Nos. 2095 through 2098, 2101 through 2110, and 2112 were performed adjacent to the Unit 2 Tendon Gallery foundation below the mudslab. Test 2074 was performed north of the Reactor Cavity to verify existing fill compaction.
2096	142.1	80+51	94+77	127.7	16.6	110.0	104.5	12.0	105.3	
2097	142.1	80+55	94+85	125.9	18.3	106.4	102.5	10.5	103.8	
2098	142.5	80+05	95+51	120.2	11.4	107.9	104.4	13.5	103.3	
2101	142.1	80+55	94+95	126.8	16.8	108.6	103.5	11.5	104.9	
2102	142.2	80+56	95+04	127.3	14.5	111.2	103.5	12.0	107.4	
2105	142.3	80+52	95+13	124.3	13.7	109.3	104.3	9.0	104.8	
2106	142.4	80+49	95+21	122.8	14.5	107.2	104.5	12.3	102.6	
2107	142.3	80+45	95+29	122.1	13.2	107.9	105.2	12.3	102.6	
2108	142.3	80+38	95+36	124.3	13.2	109.8	106.3	12.2	103.3	
2109	141.8	80+31	95+42	124.1	12.5	110.3	108.0	10.1	102.1	
2110	141.8	80+23	95+46	127.9	12.9	113.3	110.1	9.5	102.9	
2112	142.3	80+14	95+50	128.5	14.3	112.4	108.3	10.3	103.8	
2074	137.9	80+02	95+30	135.3	11.5	121.3	106.9	12.2	113.5	
NORTH OF CONTROL BUILDING SHAFTS UNITS 1 AND 2										
1542	151.7	82+27	96+38	129.0	16.2	111.0	107.8	13.5	103.0	Area represented by Test Nos. 1544, 1545 and 1546 was excavated down to competent material and
1543	151.5	82+25	96+59	125.0	17.9	106.0	104.7	14.5	101.2	
1544	152.2	82+07	96+24	124.4	13.7	109.4	112.2	10.5	97.5	
1545	152.2	81+88	96+24	127.0	17.6	108.0	112.2	10.5	96.3	

...continued...

TABLE 2, continued

Summary of Sand Cone Density Test Data

Test No.	Elev. (Ft.)	Coordinates N	Coordinates E	Field Test γ_w (pcf)	Field Test W (%)	Field Test γ_d (pcf)	Laboratory Test γ_d (max) (pcf)	Laboratory Test OMC (%)	Percent Compaction	Remarks
1546	151.8	81+68	96+24	123.4	19.3	103.4	104.7	14.5	98.8	retested as designated by Test Nos. 1547, 1548 and 1549 respectively.
1547	151.0	82+07	96+24	132.6	12.4	118.0	113.0	13.5	104.4	
1548	151.4	81+88	96+24	128.8	20.0	107.3	108.8	10.5	98.6	
1549	151.2	81+68	96+24	127.5	17.6	108.4	108.8	10.5	99.6	
1572	156.3	82+23	96+80	118.0	16.0	108.9	107.0	14.0	101.8	
1560	152.9	81+65	96+96	114.8	11.5	103.0	96.1	12.5	106.2	
1561	153.1	82+01	96+96	122.7	15.7	106.1	96.1	13.0	110.4	
WEST OF UNIT 2 ELECTRICAL TUNNEL										
1605	153.0	80+99	95+97	123.4	11.9	110.3	104.3	11.0	105.8	
1606	153.1	81+22	96+07	116.4	11.5	104.4	103.7	12.5	100.7	
1617	147.6	80+43	95+70	119.9	9.2	109.8	105.8	13.0	103.8	
1618	147.7	80+18	95+74	121.6	10.8	109.7	105.8	13.0	103.7	
1668	154.7	81+66	95+83	121.8	11.9	108.8	106.3	12.8	102.4	
1669	154.6	81+60	96+22	121.2	15.3	105.1	104.7	13.6	100.4	
1699	146.3	80+12	95+86	121.4	10.0	110.4	104.6	13.9	105.5	
EAST OF UNIT 1 ELECTRICAL TUNNEL										
1997	152.1	80+22	97+26	117.8	8.1	109.0	107.5	14.4	101.4	Test Nos. 2018, 2019, 2020 and 1986 were run adjacent to mudslab to determine if a zone of low compaction existed at the dynamic cone penetrometer test locations. All other tests were performed adjacent to the mudslab and in the area between the east wall of the Unit 1 Electrical Tunnel and West of Unit 1 Tendon Gallery.
1998	152.3	80+52	97+25	116.4	9.4	106.4	106.2	13.0	100.2	
1999	152.4	80+82	97+25	117.7	8.7	108.3	106.9	13.0	101.3	
2000	152.1	81+12	97+25	124.3	15.7	107.4	98.1	13.0	109.5	
2001	152.5	81+42	97+27	123.8	15.2	107.5	106.3	15.1	101.1	
2018	149.8	80+92	97+27	123.5	11.4	110.9	105.8	14.4	104.8	
2019	149.6	80+98	97+27	111.5	8.6	102.7	100.5	17.6	102.2	
2020	149.6	80+95	97+27	110.6	8.4	102.0	99.2	17.3	102.8	
2021	152.3	80+35	97+27	119.5	8.4	110.2	106.2	12.7	103.8	
1986	150.0	80+83	97+26	103.7	9.9	94.4	98.3	17.0	96.0	
1797	146.3	80+57	97+37	128.9	16.2	110.9	106.0	11.2	104.6	
1824	146.6	80+77	97+36	123.2	13.0	109.0	107.7	13.5	101.2	
1836	148.4	80+80	97+76	126.8	14.2	111.0	106.6	13.1	104.1	
1822	146.6	80+89	97+36	122.9	11.3	110.4	104.6	14.4	105.5	
1841	145.8	80+92	97+53	127.8	14.6	111.5	106.9	11.5	104.3	

<u>TABLE 3</u>

SUMMARY OF DYNAMIC CONE PENETROMETER TEST DATA
ADJACENT TO UNIT 1 ELECTRICAL TUNNEL EAST WALL

Test Designation	Depth (Feet)	Blows to Seat 2 Inches	Blows to Drive 1-3/4 Inches	Remarks
1-A	1.0 2.0 3.0 4.0	-- -- -- --	36 40 52 95	Test performed on 2/12/80. Blows to seat not recorded.
2-A	1.0 2.0 3.0 4.0	-- -- -- --	25 32 59 56	Test performed on 2/12/80. Blows to seat not recorded.
3-A	1.0 2.0 3.0 4.0	-- -- -- --	13 15 19 10	Test performed on 2/12/80. Blows to seat not recorded.
3-B	1.0 2.0 3.0 4.0	16 16 17 10	32 47 49 36	Located approximately 5 feet north of DCP Hole No. 3-A. Test performed on 5/12/80.
3-C	1.0 2.0 3.0 4.0 4.4	12 15 15 7 5	21 27 23 11 10	Test performed on 5/12/80. Located approximately 3 feet south of DCP Hole No. 3-A.
4-A	1.0 2.0 3.0 4.0	-- -- -- --	31 32 46 58	Test performed on 2/13/80. Blows to seat not recorded.
5-A	1.0 2.0 3.0 4.0	-- -- -- --	14 18 17 24	Test performed on 2/13/80. Blows to seat not recorded.
5-B	1.0 2.0 3.0 4.0 4.6	13 21 21 14 12	19 34 48 37 37	Test performed on 5/13/80. Located approximately 3 feet north of DCP Hole No. 5-A.

...continued...

TABLE 3, continued

Summary of Dynamic Cone Penetrometer Test Data
Adjacent to Unit 1 Electrical Tunnel East Wall

Test Designation	Depth (Feet)	Blows to Seat 2 Inches	Blows to Drive 1-3/4 Inches	Remarks
5-C	1.0	14	19	Test performed on 5/13/80. Located approximately 3 feet south of DCP Hole No. 5-A.
	2.0	16	32	
	3.0	16	32	
	4.0	18	44	
	4.6	8	31	
6-A	1.0	--	32	Test performed on 2/13/80. Blows to seat not recorded.
	2.0	--	36	
	3.0	--	40	
	4.0	--	62	
7-A	1.0	--	13	Test performed on 2/13/80. Blows to seat not recorded.
	2.0	--	28	
	3.0	--	51	

NOTE: See discussion in Section III.C.2 for evaluation and details of repair work done at locations where low penetration resistance was recorded.

TABLE 4

SUMMARY OF DYNAMIC CONE PENETROMETER
TEST DATA FOR UNIT 1 TENDON GALLERY

Test Designation	Depth (Feet)	Blows to Seat 2 Inches	Blows to Drive 1-3/4 Inches	Remarks
1	1.0	16	34	
	2.0	30	56	
	3.0	28	55	
2	1.0	14	27	
	2.0	28	54	
	3.0	26	60	
3	1.0	15	24	
	2.0	22	34	
	3.0	20	43	
4	1.0	14	27	
	2.0	18	41	
	3.0	22	45	
5	1.0	21	58	
	2.0	41	66	
	3.0	37	70	
6	1.0	24	41	
	2.0	21	52	
	3.0	39	51	
7	1.0	20	36	
	2.0	29	52	
	3.0	18	48	
8	1.5	17	31	
	2.5	34	56	
	3.0	19	30	
9	1.5	29	60	
	2.0	--	--	Shelby tube sample attempted
	3.0	26	49	
10	1.5	22	54	
	2.0	--	--	Shelby tube sample attempted
	3.0	28	45	
11	1.5	19	40	
	2.0	--	--	Shelby tube sample attempted
	2.5	14	41	
	3.0	19	40	

...continued...

TABLE 4, continued

Summary of Dynamic Cone Penetrometer
Test Data for Unit 1 Tendon Gallery

Test Designation	Depth (Feet)	Blows to Seat 2 Inches	Blows to Drive 1-3/4 Inches	Remarks
12	1.0	3	23	
	2.0	--	--	Shelby tube sample attempted
	3.0	17	37	
	4.0	15	52	
13	1.5	18	34	
	2.0	--	--	Shelby tube sample attempted
	3.0	29	77	
14	1.0	18	36	
	2.0	20	44	
	3.0	20	43	
15	1.0	11	25	
	2.0	24	40	
	3.0	25	31	
16	1.0	10	14	
	2.0	21	30	
	3.0	23	35	
17	1.0	13	22	
	2.0	25	42	
	3.0	16	31	

NOTE: See discussion in Section III.C.3

TABLE 5

SUMMARY OF DYNAMIC CONE PENETROMETER
TEST DATA FOR UNIT 2 TENDON GALLERY

Test Designation	Depth (Feet)	Blows to Seat 2 Inches	Blows to Drive 1-3/4 Inches	Remarks
1	1.0	19	39	
	2.0	28	58	
	3.0	33	85	
2	1.0	22	30	
	2.0	35	50	
	3.0	30	73	
3	1.0	11	15	
	2.0	29	45	
	3.0	25	89	
4	1.0	13	19	
	2.0	29	44	
	3.0	33	83	
5	1.0	16	24	
	2.0	26	54	
	3.0	45	97	
6	1.0	17	30	
	2.0	27	68	
	3.0	43	107	
7	1.0	12	23	
	2.0	27	60	
	3.0	40	104	
8	1.0	11	18	
	2.0	27	71	
	3.0	40	90	
9	1.0	17	27	
	2.0	28	47	
	3.0	46	99	
10	1.0	15	34	
	2.0	36	72	
	3.0	34	101	
11	1.0	12	25	
	2.0	44	89	
	3.0	37	106	

...continued...

TABLE 5, continued

Summary of Dynamic Cone Penetrometer Test Data for Unit 2 Tendon Gallery

Test Designation	Depth (Feet)	Blows to Seat 2 Inches	Blows to Drive 1-3/4 Inches	Remarks
12	1.0	19	27	
	2.0	53	123	
	3.0	77	146	
13	1.0	19	41	
	2.0	39	84	
	3.0	47	89	

ANALYSIS OF DEWATERING

PUMP

PERFORMANCE

Prepared by: D.H. Day
Mechanical Section - Instrumentation

I. SCOPE

Provide calculations to establish a more precise value for the flow rate capablility for dewatering sump pumps located in the power block.

Plant Vogtle Final Report on Dewatering and Repair of Erosion In Category I Backfill In Power Block Area, Section IV, Pumping Capacity makes an assumption for total pumping capacity to be 10,400 GPM. This analysis will show the 10,400 GPM to be realistic....

II. METHOD

Each pump with its associated system head requirements will be individually analyzed.

III. ASSUMPTIONS AND CONSIDERATIONS

- All pumps will be considered to be open-ended since the pumps feed both open channels and a gravity drained, partially-filled 24" Ø corrugated collection header....
- Water flowing temperature is calculated at 60^{O}F though actual temperature range is about 35-100^{O} F....
- Fittings are not totally accounted for in the K_{total} computation....
- All piping to be considered as clean commercial grade steel pipe, including rubber hose, for pipe friction factor purposes. However, rubber hose may exhibit higher actual friction factors than steel pipe. Consideration of rubber hose as commercial steel pipe for friction factor purposes is justified by the fact that hose runs are less than 30 feet in the attached piping systems and the error introduced here will be insignificant. (Except pumps F & H)
- Symbology and units follow the convention established in Reference 1. (See figure 2)
- For identification purposes, pumps are designated in accordance with Figure 1.

IV. ACCURACY

Accuracies based on assumptions, considerations and inherent errors are estimated to be ± 5% of predicted flow rates.

V. SUMMATION

A. Predicted flows are as follows:

1. Pump A - Not installed
2. Pump B - 1750 GPM
3. Pump C - 825 GPM
4. Pump D - 1600 GPM
5. Pump E - 1600 GPM
6. Pump F - 650 GPM
7. Pump G - 1975 GPM
8. Pump H - 1200 GPM
9. Pump J - 800 GPM

Total existing flow rates capability

$$\sum = 1750 + 825 + 1600 + 1600 + 650 + 1975 + 1200 + 800 = 10,400 \text{ GPM}$$

B. Tabulation of Performance Data

PUMP DESIGNATION	PUMP MAKE/MODEL	PUMP RATED FLOW(GPM)/HEAD(FT)	ACTUAL AS-BUILT FLOW(GPM)/HEAD(FT)	REMARKS
A	Gorman-Rupp S8A1-1	2600/55	None	Pump not installed
B	Gorman-Rupp S8A1-1	2600/55	1750/122	Improper pump selection-insufficient head, reasonable system performance based on pump utilized.
C	Gorman-Rupp S8A1-1	2600/55	825/153	Improper pump selection-insufficient head. Incorrect piping size and layout.
D	Gorman-Rupp T10A3	2750/90	1600/107	Improper pump selection-insufficient head. Loss of about 50 GPM by using 8" line instead of 10".
E	Gorman-Rupp T10A3	2750/90	1600/107	Improper pump selection-insufficient head. Loss of about 50 GPM by using 8" line instead of 10".
F	Gorman-Rupp S8A1-1	2600/55	650/158	Improper pump selection-insufficient head. Incorrect piping size (4").
G	Gorman-Rupp S8A1-1	2600/55	2000/110	6" line installed on 8" pump. 8" line would increase flow 500 GPM.
H	Gorman-Rupp S8A1-1	2600/55	1975/113	Reasonable performance considering pump selection and length of piping required.
J	Gorman-Rupp S4B1	900/90	800/108	Oversized 8" line on 4" pump provides small pipe losses & reasonably good flow rate for this pump.

C. Accumulation versus Pumping Capability

Figure 2, prepared by Civil/Duncan, details the power block ponding areas and the water accumulation following a postulated 5-inch rain. The following calculations provide the pumping time required per area for total drawoff: (<u>Does not include existing groundwater</u>)

1. Northeast Corner -

$$\frac{\text{Quantity accumulated (gallons)}}{\text{Pumping capacity (gallons/hour)}} = \frac{1.5 \times 10^6 \text{ Gal}}{(2000)(60) \text{ Gal/hr}} =$$

$$\frac{1.5 \times 10^6}{1.2 \times 10^5} = 12.5 \text{ hrs.}$$

2. Southwest Corner -

$$\frac{\text{Quantity accumulated (gallons)}}{\text{Pumping capacity (gallons/hour)}} = \frac{3.1 \times 10^6 \text{Gal}}{(6575)(60) \text{ Gal/hr}} =$$

$$\frac{3.1 \times 10^6}{3.945 \times 10^5 \text{ Gal/hr.}} = 7.86 \text{ hrs.}$$

3. Southeast Corner

$$\frac{\text{Quantity accumulated (gallons)}}{\text{Pumping capacity (gallons/hour)}} = \frac{1.1 \times 10^6 \text{Gal}}{} =$$

$$\frac{1.1 \times 10^6 \text{Gal.}}{157.5 \times 10^3 \text{ Gal/hr}} = 6.98 \text{ hrs.}$$

VI. RECOMMENDATIONS

1. System pumping requirements should be determined prior to pump procurement...

2. Submersible pumps need to be restrained within the sump to prevent "burrowing" into sand and mud as a result of start-up torque. Pumps should also be elevated above sand and mud level...

3. Sufficiently rated starters should be provided on all pumps. Specifically, pump C's electrical circuit should be checked...

4. Change 4" rubber hose to 8" on pump C...Major improvement...

5. Change 6" piping to 8" on pump G...

6. Install check valves on all lines. Remove homemade check valve from pump J line...

7. Design and install Cippoletti weir at groundwater/rainwater effluent line discharge to determine exact rate of flow...

8. Change 4" piping to 8" on Pump F....Major improvement...

9. Remove kinks from suction line hoses on diesel pumps. Replace 8" hose section on pump E with 8" hard piping.

VII. REFERENCES

1. Flow of Fluids Through Valves, Fittings, and Pipe, Technical Paper No. 410, Crane.

2. Pump Handbook, McGraw Hill

3. Hydraulic Institute Standards, Thirteenth Edition, Hydraulic Institute

4. Principles and Practices of Flow Meter Engineering, Foxboro, L.K. Spink

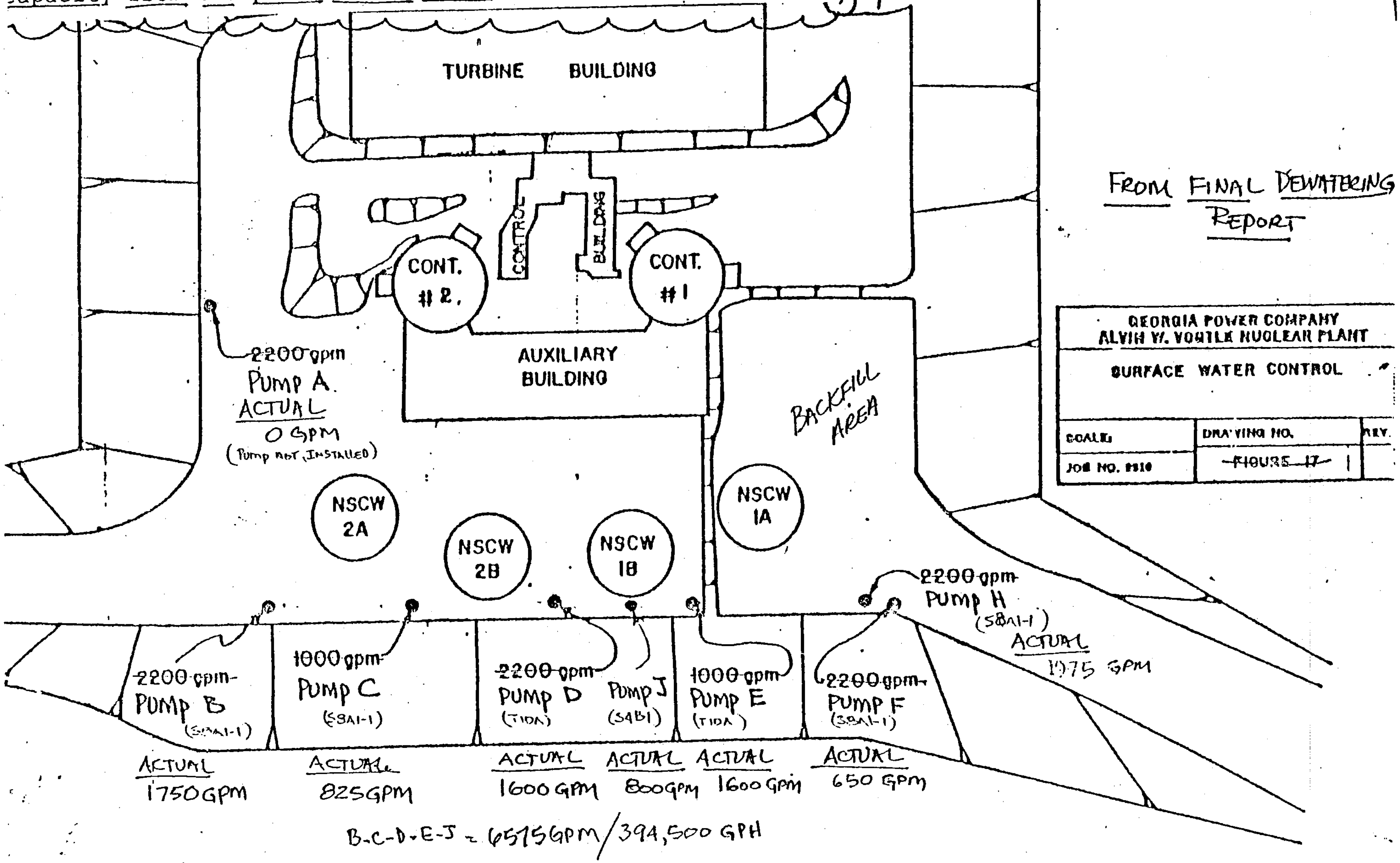
VIII. ATTACHMENTS

Figure 1 - Surface Water Control - Plan View
Figure 2 - Nomenclature
Figure 3 - General Energy Equation - Bernoulli's Theorem
Figure 4 - Ponding Areas and Anticipated Collection of Rainfall Following Postulated 5-inch Rain
Figure 5 - Weir Construction Details

Appendix A - Dewatering Pump Performance - Pump A
Appendix B - " " " - Pump B
Appendix C - " " " - Pump C
Appendix D - " " " - Pump D
Appendix E - " " " - Pump E
Appendix F - " " " - Pump F
Appendix G - " " " - Pump G
Appendix H - " " " - Pump H
Appendix J - " " " - Pump J
Appendix K - Comparative pipe losses for 6"-8"-12" NPS

umping capacity

Figure 17 Surface Water Control shows the locations of the sumps, pumps and the northeast impoundment. The pump located in the northeast impoundment is rated at 2600 gpm at 45' of head. The remaining seven pumps located in the main power block have a total rating of 13,000 gpm at 90' of head. Five of the pumps are each rated at 2200 gpm at 90' of head, the other two pumps are rated at 1000 gpm at 90' of head each. The total pumping capacity from the power block during a rainfall is 15,600 gpm.



FROM FINAL DEWATERING REPORT

GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
SURFACE WATER CONTROL		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	~~FIGURE 17~~	

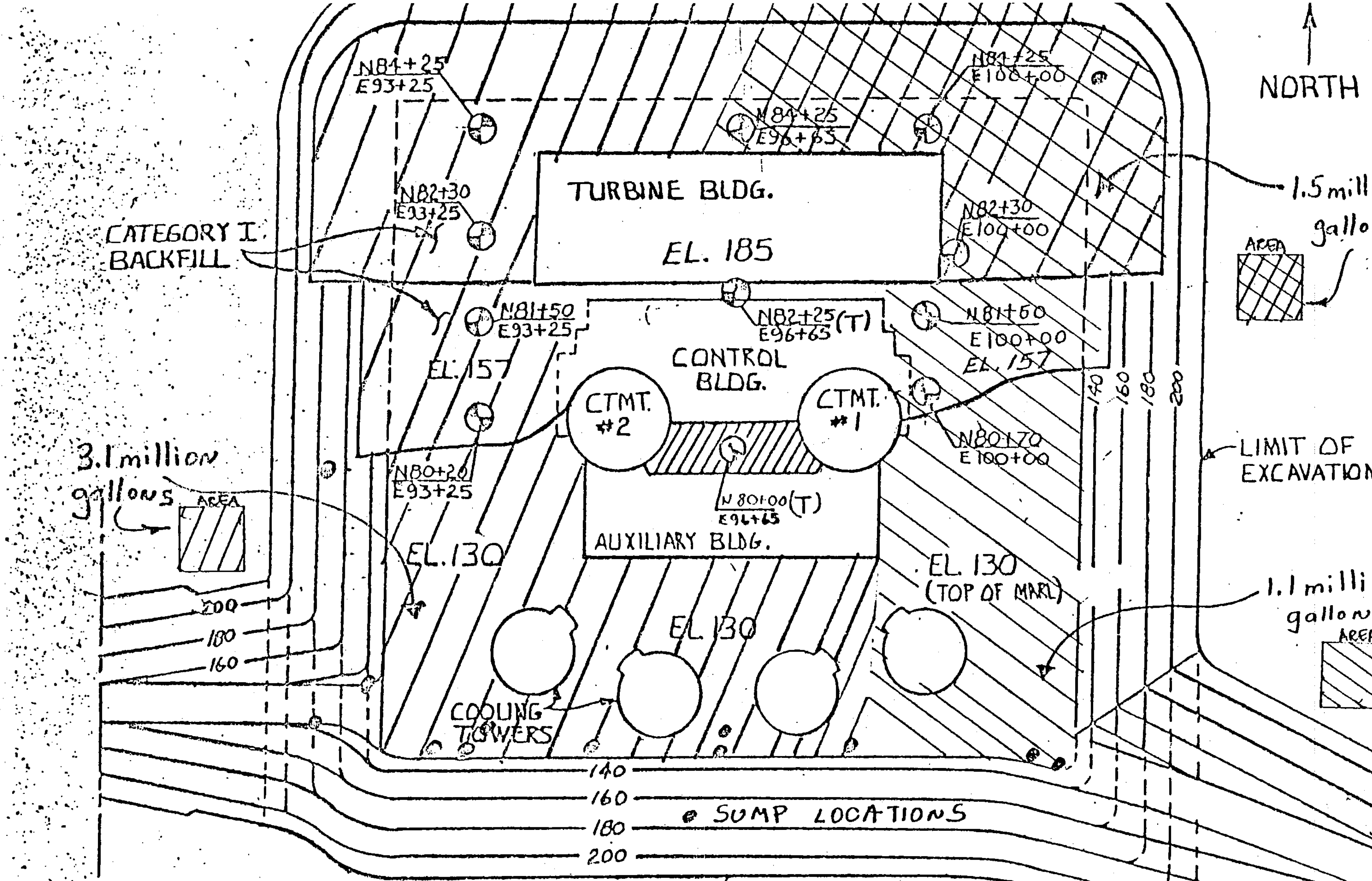


FIGURE 2

NOTES: 1. (piezometer symbol) INDICATES PIEZOMETER.

2. PIEZOMETER SHALL BE LOCATED WITHIN 15 FEET OF THAT SHOWN.

3. (T) INDICATES TEMPORARY (UNDER FUTURE BUILDING)

FIGURE 2a

PONDING AREAS AND ANTICIPATED COLLECTION OF RAINFALL Following POSTULATED 5" RAIN....

Nomenclature

Unless otherwise stated, all symbols used in this book are defined as follows:

A = cross sectional area of pipe or orifice, in square feet

a = cross sectional area of pipe or orifice, or flow area in valve, in square inches

B = rate of flow in barrels (42 gallons) per hour

C = flow coefficient for orifices and nozzles = discharge coefficient corrected for velocity of approach = $C_d / \sqrt{1-\beta^4}$.

C_d = discharge coefficient for orifices and nozzles

C_v = flow coefficient for valves; expresses flow rate in gallons per minute of 60 F water with 1.0 psi pressure drop across valve

D = internal diameter of pipe, in feet

d = internal diameter of pipe, in inches

e = base of natural logarithm = 2.718

f = friction factor in formula $h_L = fLv^2/D2g$

f_T = friction factor in zone of complete turbulence

g = acceleration of gravity = 32.2 feet per second per second

H = total head, in feet of fluid

h = static pressure head existing at a point, in feet of fluid

h_g = total heat of steam, in Btu per pound

h_L = loss of static pressure head due to fluid flow, in feet of fluid

h_w = static pressure head, in inches of water

K = resistance coefficient or velocity head loss in the formula, $h_L = Kv^2/2g$

k = ratio of specific heat at constant pressure to specific heat at constant volume = c_p/c_v

L = length of pipe, in feet

L/D = equivalent length of a resistance to flow, in pipe diameters

L_m = length of pipe, in miles

M = molecular weight

MR = universal gas constant = 1544

n = exponent in equation for polytropic change ($p'V_a^n$ = constant)

P = pressure, in pounds per square inch gauge

P' = pressure, pounds per square inch absolute

(see page 1-5 for diagram showing relationship between gauge and absolute pressure)

p' = pressure, in pounds per square foot absolute

Q = rate of flow, in gallons per minute

q = rate of flow, in cubic feet per second at flowing conditions

q' = rate of flow, in cubic feet per second at standard conditions (14.7 psia and 60F)

q'_d = rate of flow, in millions of standard cubic feet per day, MMscfd

q'_h = rate of flow, in cubic feet per hour at standard conditions (14.7 psia and 60F), scfh

q_m = rate of flow, in cubic feet per minute at flowing conditions

q'_m = rate of flow, in cubic feet per minute at std. conditions (14.7 psia and 60F), scfm

R = individual gas constant = MR/M = $1544/M$

R_e = Reynolds number

R_H = hydraulic radius, in feet

r_c = critical pressure ratio for compressible flow

S = specific gravity of liquids at specified temperature relative to water at standard temperature (60 F)

S_g = specific gravity of a gas relative to air = the ratio of the molecular weight of the gas to that of air

T = absolute temperature, in degrees Rankine (460 + t)

t = temperature, in degrees Fahrenheit

$\bar{V}$ = specific volume of fluid, in cubic feet per pound

V = mean velocity of flow, in feet per minute

V_a = volume, in cubic feet

v = mean velocity of flow, in feet per second

v_s = sonic (or critical) velocity of flow of a gas, in feet per second

W = rate of flow, in pounds per hour

w = rate of flow, in pounds per second

w_a = weight, in pounds

x = percent quality of steam = 100 minus per cent of moisture

Y = net expansion factor for compressible flow through orifices, nozzles, or pipe

Z = potential head or elevation above reference level, in feet

Greek Letters

Beta

β = ratio of small to large diameter in orifices and nozzles, and contractions or enlargements in pipes

Delta

Δ = differential between two points

Epsilon

ϵ = absolute roughness or effective height of pipe wall irregularities, in feet

Mu

μ = absolute (dynamic) viscosity, in centipoise

μ_e = absolute viscosity, in pound mass per foot second or poundal seconds per sq foot

μ'_e = absolute viscosity, in slugs per foot second or pound force seconds per square foot

Nu

ν = kinematic viscosity, in centistokes

ν' = kinematic viscosity, square feet per second

Rho

ρ = weight density of fluid, pounds per cubic ft

ρ' = density of fluid, grams per cubic centimeter

Theta

θ = angle of convergence or divergence in enlargements or contractions in pipes

Subscripts for Diameter

(1)...defines smaller diameter
(2)...defines larger diameter

Subscripts for Fluid Property

(1)...defines inlet (upstream) condition
(2)...defines outlet (downstream) condition

FIGURE 3

General Energy Equation
Bernoulli's Theorem

The Bernoulli theorem is a means of expressing the application of the law of conservation of energy to the flow of fluids in a conduit. The total energy at any particular point, above some arbitrary horizontal datum plane, is equal to the sum of the elevation head, the pressure head, and the velocity head, as follows:

$$Z + \frac{144\,P}{\rho} + \frac{v^2}{2g} = H$$

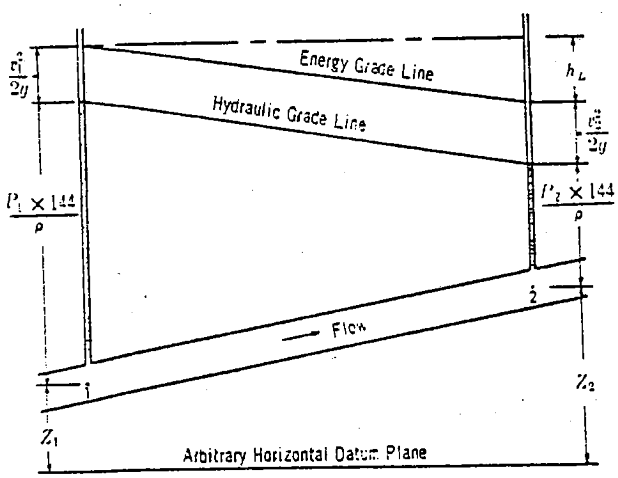


Figure 1-4
Energy Balance for Two Points in a Fluid

By permission, from *Fluid Mechanics*[1*] by R. A. Dodge and M. J. Thompson. Copyright 1937; McGraw-Hill Book Company, Inc.

If friction losses are neglected and no energy is added to, or taken from, a piping system (i.e., pumps or turbines), the total head, H, in the above equation will be a constant for any point in the fluid. However, in actual practice, losses or energy increases or decreases are encountered and must be included in the Bernoulli equation. Thus, an energy balance may be written for two points in a fluid, as shown in the example in Figure 1-4.

Note the pipe friction loss from point 1 to point 2 is h_L foot pounds per pound of flowing fluid; this is sometimes referred to as the head loss in feet of fluid. The equation may be written as follows:

Equation 1-3

$$Z_1 + \frac{144P_1}{\rho_1} + \frac{v_1^2}{2g} = Z_2 + \frac{144P_2}{\rho_2} + \frac{v_2^2}{2g} + h_L$$

All practical formulas for the flow of fluids are derived from Bernoulli's theorem, with modifications to account for losses due to friction.

1. $h_l = \frac{0.00259\,KQ^2}{d^4}$
2. $R_e = 123.9\,\frac{dv\rho}{\mu}$
3. $V = \frac{0.408\,Q}{d^2}$
4. $d_1^2 V_1 = d_2^2 V_2$
5. $bhp = \frac{QH\rho}{247000\,e\rho}$

FIGURE 4

GEORGIA POWER COMPANY

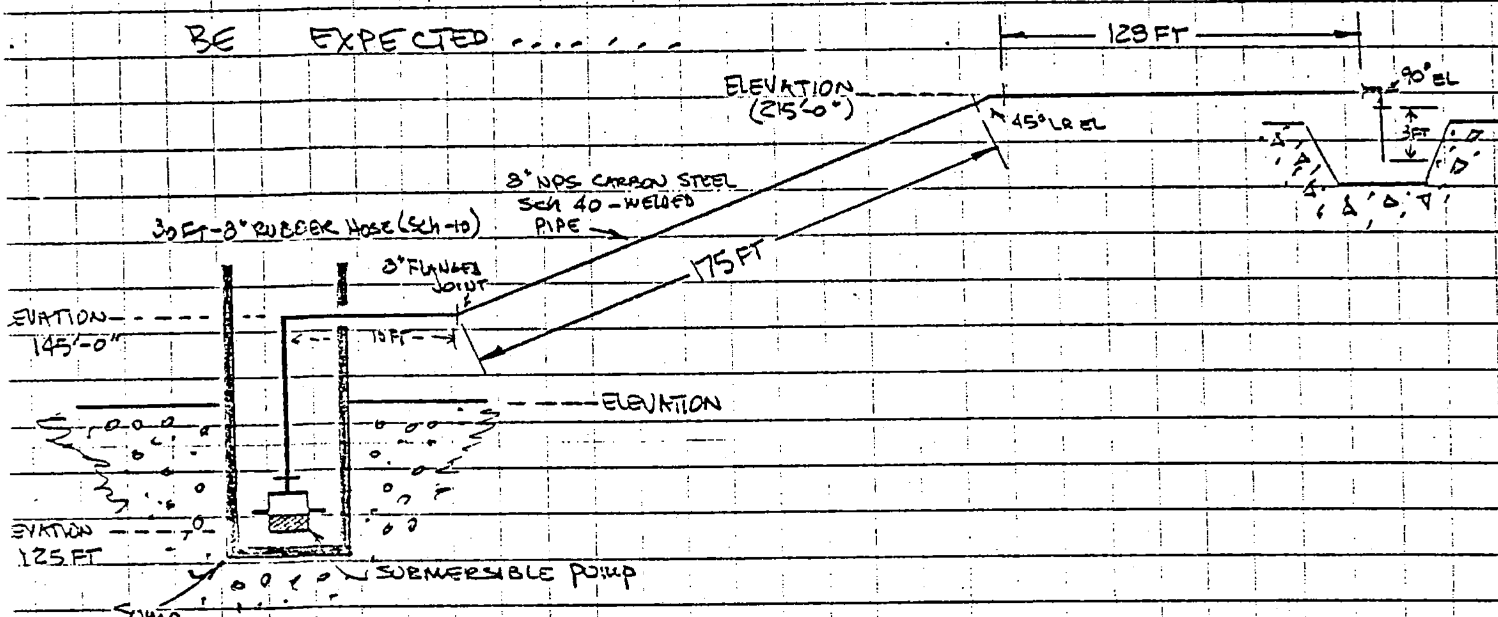
Sheet ____ of ____

Subject: DEWATERING PUMP PERFORMANCE
POWER BLOCK WEST SLOPE – PUMP A

Project: VOGTLE

Computed by: D. DAY Date: 8-7-80 Checked by: ____ Date: ____

GIVEN: WATER AT (60°F) IS FLOWING THROUGH THE PIPING SYSTEM AS DETAILED BELOW. DEVELOP SYSTEM CURVE AND PUMP CURVE TO DETERMINE MAXIMUM RATE OF FLOW THAT CAN BE EXPECTED ....



PIPING SYSTEM CONFIGURATION

PUMP SPECS: (SEE DATA SHEET ATTACHED)
GORMAN-RUPP MODEL
RATED

$d = 7.981_{CS.P}$ & 8.329_{HOSE}

PUMP NOT INSTALLED

APPENDIX A-1