J. A. "Buzz" Miller
Senior Vice President
Nuclear Development

Southern Nuclear Operating Company, Inc.
40 Inverness Center Parkway
Post Office Box 1295
Birmingham, Alabama 35201

Tel 205.992.5754
Fax 205.992.6165

SOUTHERN COMPANY
Energy to Serve Your World℠

APR 16 2007

Docket No.: 52-011

AR-07-0639

U.S. Nuclear Regulatory Commission
Document Control Desk
Washington, DC 20555-0001

Southern Nuclear Operating Company
Vogtle Early Site Permit Application
Response to Requests for Additional Information Letter No. 6 Involving Hydrology

Ladies and Gentlemen:

By letter dated March 15, 2007, the U.S. Nuclear Regulatory Commission (NRC) provided Southern Nuclear Operating Company (SNC) with Request for Additional Information (RAI) Letter No. 6 on the Vogtle Early Site Permit (ESP) Application. The RAIs in that letter pertain to ESP application Part 2, Site Safety Analysis Report (SSAR), Section 2.4, *Hydrologic Engineering*, and Section 2.5, *Geology, Seismology and Geotechnical Engineering*. SNC's response to the RAIs pertaining to SSAR Section 2.4 is provided in the following Enclosures to this letter. SNC's response to the RAIs pertaining to SSAR Section 2.5 is provided in AR-07-0801.

The SNC contact for this RAI response letter is J. T. Davis at (205) 992-7692.

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Mr. J. A. (Buzz) Miller states he is a Senior Vice President of Southern Nuclear Operating Company, is authorized to execute this oath on behalf of Southern Nuclear Operating Company and to the best of his knowledge and belief, the facts set forth in this letter are true.

Respectfully submitted,

SOUTHERN NUCLEAR OPERATING COMPANY

Joseph A. (Buzz) Miller

Sworn to and subscripted before me this 16 *day of* April, *2007*

Notary Public

My commission expires: 05/06/08

JAM/BJS/dmw

Enclosures:

1. Response to March 15, 2007 RAI Letter No. 6 for the Vogtle ESP Application Involving SSAR Section 2.4 Hydrologic Engineering
2. Proposed Revision to SSAR Section 2.4.13
3. Requested Reference Documents

cc: Southern Nuclear Operating Company
Mr. J. B. Beasley, Jr., President and CEO (w/o enclosures)
Mr. J. T. Gasser, Executive Vice President, Nuclear Operations (w/o enclosures)
Mr. T. E. Tynan, Vice President - Vogtle (w/o enclosures)
Mr. D. M. Lloyd, Vogtle Deployment Director (w/o enclosures)
Mr. C. R. Pierce, Vogtle Development Licensing Manager (w/o enclosures)
Document Services RTYPE: AR01
File AR.01.01.06

Nuclear Regulatory Commission
Mr. R. W. Borchardt, Director of Office of Nuclear Regulation (w/o enclosures)
Mr. W. D. Travers, Region II Administrator (w/o enclosures)
Mr. D. B. Matthews, Director of New Reactors (w/o enclosures)
Ms. S. M. Coffin, AP1000 Manager of New Reactors (w/o enclosures)
Mr. C. J. Araguas, Project Manager of New Reactors
Mr. H. B. Clayton, Branch Chief – Environmental Technical Support (w/o enclosures)
Mr. M. D. Notich, Environmental Project Manager
Mr. G. J. McCoy, Senior Resident Inspector of VEGP (w/o enclosures)

Georgia Power Company
Mr. O. C. Harper, Vice President, Resource Planning and Nuclear Development (w/o enclosure)

Oglethorpe Power Corporation
Mr. M. W. Price, Chief Operating Officer (w/o enclosure)

Municipal Electric Authority of Georgia
Mr. C. B. Manning, Senior Vice President and Chief Operating Officer (w/o enclosure)

Dalton Utilities
Mr. D. Cope, President and Chief Executive Officer (w/o enclosure)

Bechtel Power Corporation
Mr. J. S. Prebula, Project Engineer (w/o enclosures)
Mr. R. W. Prunty, Licensing Engineer (w/o enclosures)

Tetra Tech NUS, Inc.
Ms. K. K. Patterson, Project Manager (w/o enclosures)

Southern Nuclear Operating Company

AR-07-0639

Enclosure 1

Response to March 15, 2007 RAI Letter No. 6

for the Vogtle ESP Application

Involving

SSAR Section 2.4 Hydrologic Engineering

Section 2.4, Hydrologic Engineering

2.4.1-1 Please revise the SSAR by incorporating Environmental Report (ER) Section 2.3.2, and Tables 2.3.2-12 and 2.3.2-13, which report the rates of total water demand. The values for the total water demand should be included in the SSAR.

Response:

The rates for total water demand, as requested by this RAI, have been added to a new SSAR Section 2.4.1.2.6, Water Consumption, which references a new SSAR Table 2.4.1-11, Plant Water Use. Both the discussion section and the summary table reflect the plant water use/water consumption discussion that is contained in Section 3.3 of the Environmental Report (ESP Application Part 3). The following proposed new water consumption section and associated water use table will be integrated in the next revision to the ESP application.

2.4.1.2.6 Water Consumption

The new AP1000 units require water for both plant cooling and operational uses. The Savannah River provides make-up water for the circulating water system (CWS) to replace the water lost to evaporation, drift, and blowdown. On-site wells provide groundwater make-up for the service water system (SWS). The wells also provide water for other plant systems, including the fire protection system, the plant demineralized water supply system, and the potable water system. Surface water consumptive use for the two AP1000 units' normal operation is 27,924 gpm, with a maximum of 28,904 gpm. Groundwater consumptive use is 752 gpm on average, with a maximum of 3,140 gpm. During normal operation, approximately 305 gpm of groundwater is returned as surface water to the Savannah River. Table 2.4.1-11 identifies the normal and maximum water demand and effluent streams for the AP1000 units.

The CWS and SWS cooling towers lose water from evaporation and drift. Evaporation and drift from the CWS cooling towers is estimated at 27,924 gpm during normal operations. Evaporation and drift for the SWS cooling tower is estimated at 403 gpm. These values are based on site characteristics and AP1000 design parameters for the cooling.

Table 2.4.1-11 also provides the water release estimates for wastewater and blowdown discharged to the Savannah River. These include estimates for all wastewater flows from the site, including radiological effluent releases, sanitary waste, miscellaneous drains, and demineralizer discharges. The normal values listed are the expected values for normal plant operation with two new units in operation. The maximum values are those expected for upset or abnormal conditions with two new units in operation.

Table 2.4.1-11 Plant Water Use

Stream Description	Normal Case[a] gpm	Maximum Case[a,b] gpm	Comments
Groundwater (Well) Streams:			
Plant Well Water Demand	752	3,140	
Well Water for Service Water System Makeup	537	2,353	
• Service Water System Consumptive Use	403	1,177	
- Service Water System Evaporation	402	1,176	
- Service Water System Drift	1	1	c
• Service Water System Blowdown	134	1,176	d
Well Water for Power Plant Make-up/Use	215	787	
• Demineralized Water System Feed	150	600	
- Plant System Make-up/Processes	109	519	
- Misc. Consumptive Use	41	81	
• Potable Water Feed	42	140	
• Fire Water System	10	12	
• Misc. Well Water Users	13	35	
Surface Water (Savannah River) Streams			
River Water for Circulating Water / Turbine Plant Cooling Water System Make-up	37,224	57,784	
• Circulating Water / Turbine Plant Cooling Water System Consumptive Use	27,924	28,904	
- Circulating Water / Turbine Plant Cooling Water System Evaporation	27,900	28,880	
- Circulating Water / Turbine Plant Cooling Water System Drift	24	24	c
• Circulating Water / Turbine Plant Cooling Water System Blowdown	9,300	28,880	d
Plant Effluent Streams			
Final Effluent Discharge to River	9,608	30,761	
• Blowdown Sump Discharge	9,605	30,561	
- Wastewater Retention Basin Discharge	171	505	
o Miscellaneous Low Volume Waste	129	365	
o Treated Sanitary Waste	42	140	
- Service Water System Blowdown	134	1,176	d
- Circulating Water / Turbine Plant Cooling Water System Blowdown	9,300	28,880	d
- Start-up Pond Discharge	0	0	e
• Treated Liquid Radwaste	3	200	f

NOTES:

a The flow rate values are for two AP1000 units.

b These flows are not necessarily concurrent.

c The cooling tower drifts are 0.002% of the tower circulating water flow.

d For the normal case, the cooling towers are assumed operating at four cycles of concentration. For the service water cooling tower (maximum case), both unit towers are assumed operating at two cycles of concentration. For the main condenser / turbine auxiliary cooling water tower (maximum case), both towers are assumed operating at two cycles of concentration.

e Start-up flushes and start-up pond discharge would occur only during the initial plant start-up phase and potentially after unit outages when system flushes are required.

f The short-term liquid waste discharge flow rate may be up to 200 gpm. However, given the waste liquid activity level, the discharge rate must be controlled to be compatible with the available dilution (cooling tower blowdown) flow.

2.4.4-1 During the January site safety audit (1/10-11/2007) the NRC staff requested and the applicant provided a narrative (AR-07-0302, 2/13/2007) describing the process used to compute the maximum stage due to a cascade failure of upstream dams, including the sensitivity of the initial water surface elevations in each reservoir, and showed how the calculations provide the bounding case. The narrative included a summary of all dam breach analysis parameters. Please update the site safety analysis report (SSAR) to incorporate the information and data contained in the narrative.

Response:

The next revision to the ESP application will revise SSAR Section 2.4.4, to include the narrative (AR-07-0302, 2/13/2007) as requested.

2.4.7-1 Please revise the SSAR by providing a reference in SSAR Sections 2.3.1.3.4 through 2.4.10 to clarify the method for determining the intensity of short term rainfall for roof drainage and probable maximum winter precipitation that combines with the snow accumulation for roof loading of all safety-related structures.

Response:

SSAR Section 2.4.10 contains a discussion of flood protection requirements, including the roofs of safety-related structures. A reference will be added to SSAR Section 2.4.10 directing reader to SSAR Section 2.3.1.3.4, Precipitation Extremes, which discusses the design basis combination of 100-year return period ground-level snow pack and 48-hour PMWP (probable maximum winter precipitation) as applied to safety-related roofs. As noted in Section 2.3.1.3.4, application of these two climate-related components of design basis snow load would be described in the COL Application.

2.4.12-1 (a) Discuss the reasons why OW-1001 and OW-1001A present questionable results regarding water table elevations for the unconfined or Water Table aquifer. Are there alternate interpretations that suggest either (1) they are invalid data and the Water Table aquifer does not see any impact from the fractured and faulted Blue Bluff Marl above the Pen Branch fault, or (2) they are valid data revealing a perhaps local hydraulic connection between the Water Table aquifer and the Tertiary aquifer?

NOTE: Prior to responding to this request, it should be noted that some minor typographic and transcription errors were identified in ER Tables 2.3.1-18 and -19 and SSAR Tables 2.4.12-1 and -2. Because the magnitudes of these errors are small, the interpretation and conclusions regarding the hydrogeologic site characteristics reported in the ESP application are not affected. The corrected monthly groundwater level elevations in the Water Table and Tertiary aquifers are shown in the following RAI Tables 2.4.12-1.1 and 2.4.12-1.2, respectively. Monthly groundwater elevation data for observation wells OW-1001 and OW-1001A are considered invalid and will be omitted from these tables in the next revision of the ESP application. The reasons for omitting these data are discussed below.

Response:

The following response is divided into three parts. The first and second parts discuss the validity of the anomalous groundwater level elevation data for observation wells OW-1001 and OW-1001A, while the third part discusses the validity of anomalous groundwater levels in these wells with respect to the Pen Branch fault.

Observation Well OW-1001

RAI Table 2.4.12-1.1 shows that monthly groundwater level elevations in the Water Table aquifer (excluding the data for OW-1001 and OW-1001A) for the period from June 2005 to November 2006 range from about 133 to 165 ft msl with seasonal fluctuations averaging about 1.0 ft. RAI Table 2.4.12-1.2 shows that for the same monitoring period, groundwater level elevations in the Tertiary aquifer range from about 82 to 128 ft msl with seasonal fluctuations averaging about 7.6 ft. The groundwater levels measured in OW-1001, installed in the Water Table aquifer, range from about 114 to 118 ft msl with a seasonal fluctuation of about 4.4 ft. These groundwater levels and seasonal fluctuations are not consistent with the groundwater levels and seasonal fluctuation of groundwater levels in the Water Table aquifer and suggest that the screened portion of the well is not in good hydraulic communication with the Water Table aquifer. Review of the boring log, daily field log, well development log and in situ hydraulic conductivity test results for the well indicate that either the formation material adjacent to the well was adversely impacted by well construction or that the well was inadvertently installed in the confining unit underlying the formation material. Both of these hypotheses are discussed below.

Formation material adjacent to the well was adversely impacted by well construction.

The construction log for OW-1001, contained in SSAR Appendix 2.4A (Appendix F report), indicates that the screened interval of the well ranges in elevation from 110 to 101 ft msl. The boring log for OW-1001, contained in SSAR Appendix 2.4A (Appendix E report), indicates that the bottom of the screen is about 5 ft above the top of the Blue Bluff Marl (BBM) which was encountered at elevation 96 ft msl. The boring log reports that 1,500 gallons of water were lost during cleaning of the hole upon its completion. In addition, the daily field log for June 6, 2005, contained in SSAR Appendix 2.4A (Appendix A report), reports that significant grout loss occurred during backfilling of the well annulus above the screened interval. The well development log for OW-1001, contained in SSAR Appendix 2.4A (Appendix G report), indicates that the well was dry after the removal of two well volumes of water and that the recovery of water into the well was very slow (less than 1-ft over a 12-hour period). The log also indicates that the water removed from the well during development was gray in color suggesting that grout may have been within close proximity to the well screen. The results of the in situ hydraulic conductivity test for OW-1001, contained in SSAR Appendix 2.5A (Appendix D report), show almost no measurable water inflow during the test and report a hydraulic conductivity value of 2.7×10^{-7} cm/s (7.6×10^{-4} ft/day). This value is about three to four orders of magnitude less than the hydraulic conductivity values of 0.12 to 2.7 ft/day reported for the Water Table aquifer (SSAR Table 2.4.12-3).

The well was inadvertently installed in the confining unit underlying the formation material.

The boring log for OW-1001 indicates that the bottom of the well screen is about 5 ft above the top of the confining unit, the BBM, which was encountered at a depth of about 135 ft below the ground surface at an elevation of approximately 96 ft msl. RAI Figure 2.4.12-1.1 shows the contours of the top of the BBM underlying the VEGP site. RAI Figure 2.4.12-1.1 was developed primarily from boring information obtained from the subsurface Investigation program for the ESP application and preliminary boring information obtained from the current subsurface investigation for the COL application. The boring logs prepared for the ESP application are contained in SSAR Appendix 2.5A. The boring logs prepared for the COL application are currently in preliminary form. RAI Figure 2.4.12-1.1 shows that in the vicinity of OW-1001 the top of the BBM is at an elevation of between 120 to 125 ft msl, which is approximately 25 to 30 ft higher than the surface elevation of the BBM reported in OW-1001 (96 ft msl).

Groundwater level elevations similar to those observed in OW-1001 were reported in a series of observation wells installed in the BBM in 1985. Two clusters of observation wells were installed in the BBM at the VEGP site in 1985. Well cluster "A" consisted of three wells, 900, 901 and 902 and well cluster "B" consisted of wells 903, 904B and 905. The wells were installed at opposite corners of the VEGP Units 1 and 2 power block area to provide detail on the pore pressure distribution within the marl. The well construction and installation details are provided in the Geotechnical Verification Work Report of Results for the VEGP Units 1 and 2 (Bechtel Geology Group, 1985). Groundwater level elevations for the wells for the period from July 1985 to January 1987 are provided in the Piezometer Weekly Readings Report for VEGP Units 1 and 2 (Georgia Power, 1987) and the Observation Well Reading reports for VEGP Units 1 and 2 (Georgia Power, 1986 and 1987) and are summarized in Table 2.4.12-1.3. A hydrograph plot of each of the wells is shown in RAI Figure 2.4.12-1.2.

RAI Figure 2.4.12-1.2 shows groundwater level elevations in the BBM for the July 1985 to January 1987 period to range from about 104 to 127 ft msl, with a seasonal fluctuation averaging about 4.5 ft. These groundwater level elevations are similar to the groundwater levels reported for OW-1001, which range from about 114 to 118 ft msl. The average seasonal fluctuation in groundwater levels measured in OW-1001 of 4.4 ft is also of similar magnitude to the seasonal fluctuation in groundwater levels measured in the marl. While these seasonal fluctuations correlate more with the Tertiary aquifer than the Water Table aquifer, this is considered to be due to a pressure response in the marl and not a reflection of movement of groundwater into or out of the marl.

For the reasons presented above, the groundwater level elevation data for observation well OW-1001 are considered invalid.

Observation Well OW-1001A

As a result of groundwater levels reported in OW-1001 that were not consistent with the groundwater levels reported in the other observation wells open to the Water Table aquifer, a new observation well, OW-1001A, was installed in the Water Table aquifer approximately 70 feet from OW-1001. The well was installed on October 11, 2005 during the geotechnical investigation performed for the ESP application. The construction log for OW-1001A, contained in SSAR Appendix 2.5A (Appendix D report), indicates that the screened portion of the well ranges in elevation from 146.13 to 136.13 ft msl. Groundwater level elevations in OW-1001A are summarized in Table 2.4.12-1.1. For the period from October 2005 to November 2006 groundwater level elevations range from 135.91 to 135.99 ft msl. It is apparent that groundwater levels in the well are close to or below the bottom of the screened interval of the well, indicating no hydraulic communication with the aquifer.

For this reason, the groundwater level data for observation well OW-1001A are considered invalid.

Pen Branch Fault and Associated Faulting and Fracturing of the Blue Bluff Marl in the Vicinity of OW-1001.

There is no evidence to suggest that the BBM, the confining unit between the Water Table and Tertiary aquifers, is faulted or fractured in the vicinity of observation well OW-1001, or indeed within the vicinity of the VEGP site, such that it would provide communication between the two aquifers. SSAR Section 2.5.1.2.4 describes previous investigations of the Pen Branch fault and the site subsurface investigation of the fault that was conducted for the ESP application. Results of the ESP investigation, which included seismic reflection and refraction surveys, clearly document that the Pen Branch fault strikes northeast and dips southeast beneath the VEGP site. SSAR Figure 2.5.1-42 shows the vertical projection of the fault from the top of basement rock in relation to VEGP Units 3 and 4. The plan projection of the intersection of the Pen Branch fault with the top of basement rock is located beneath or slightly southeast of the antiformal hinge at the top of the monocline in the BBM. Because of its spatial association with the Pen Branch fault, it is likely that this monoclinal feature is the result of reverse or reverse-oblique slip on the Pen Branch fault. The results further indicate that the fault terminates in Upper Cretaceous age sediments. Overlying Tertiary age sediments including those comprising the Tertiary sand aquifer, the BBM, and the Water Table aquifer are therefore not affected by the Pen Branch fault. The results of the seismic survey conducted for the ESP application are presented in the Geologic Interpretation of Seismic Reflection Data at Vogtle Plant Site (Bechtel Power Corporation, 2006). The location of the Pen Branch fault at the top of the basement rock and the monoclinal fold in the BBM are shown on RAI Figure 2.4.12-1.3.

The comprehensive exploration and testing programs that have been conducted at the VEGP site demonstrate that the BBM is an extensive and persistent unit at the site. Over 3,000 feet of BBM has been penetrated at the site by drilling, coring, Standard Penetration Testing and undisturbed sampling performed for the ESP and COL applications. Contours of the top and bottom of the marl, as shown in the following RAI Figures 2.4.12-1.1 and 2.4.12-1.4, show the marl to be a continuous unit that generally ranges in thickness from about 60 to 70 ft. None of the borings completed to date encountered fracture zones within the marl and at no time during drilling was any abnormal drill rod drop observed in the BBM, indicative of the presence of solution cavities, etc. A number of the borings reported loss of drilling fluid during drilling, but this was typically at the contact between the marl and overlying Utley limestone. This is considered to be due to the presence of localized, high permeability zones within the Utley limestone. Visual inspection and logging of split-spoon samples of the marl retrieved from the borings produced no indication of voids or fracture zones.

The results of in situ hydraulic conductivity tests performed in the BBM for the VEGP Units 1 and 2 indicate that the marl is relatively impermeable. UFSAR Section 2.4.12.2.4.2 reports that in situ hydraulic conductivity tests were performed in 28 exploratory holes and that in ninety percent of the intervals tested there was no measurable water inflow. Laboratory hydraulic conductivity tests conducted on ten samples of the BBM also confirm the relatively impermeable nature of the marl. The in situ and laboratory hydraulic conductivity tests yielded hydraulic conductivity values ranging from 5.2×10^{-3} ft/yr (1.4×10^{-5} ft/day) to 51 ft/yr (1.4×10^{-1} ft/day).

References:

Bechtel Geology Group, 1985, Geotechnical Verification Work Report of Results, Vogtle Electric Generating Plant, August.

Bechtel Power Corporation, 2006, Geologic Interpretation of Seismic Reflection Data at Vogtle Plant Site, Report Number 25144-006-V14-CY06-00008-001, August.

Georgia Power, 1987, Piezometer Weekly Readings Report Nos: 79, 80, 81, 82, and 83 Vogtle Electric Generating Plant – Units 1 and 2, February.

Georgia Power, 1986, Observation Well Readings, July – December 1985, Vogtle Electric Generating Plant – Units 1 and 2, January.

Georgia Power, 1986, Observation Well Readings, January – June 1986, Vogtle Electric Generating Plant - Units 1 and 2, June.

Georgia Power, 1987, Observation Well Readings, July – December 1986, Vogtle Electric Generating Plant – Units 1 and 2, January.

2.4.12-1 (b) Provide an explanation in the SSAR regarding the nomenclature used to denote an abandoned well, especially OW-1001A, which is denoted as abandoned in Appendix 2.4A, and OW-1001A, which is apparently denoted as a functioning well in Appendix 2.5A.

Response:

The following information will be incorporated in the next revision of the ESP application under existing SSAR Section 2.4.12.1.3 Observation Well Data.

The only "A" well installed at the site for the ESP application was observation well OW-1001A (none of the observation wells installed at the site have been abandoned to date). The confusion arises because the boring or drill logs contained in SSAR Appendix 2.4A (Appendix E report) are labeled "OW" as opposed to "B" or "D". Therefore, references to OW-A borings in this appendix are different from references to OW-A wells in SSAR Section 2.4.12. A summary of the holes drilled at the site to accommodate installation of the observation wells is provided in RAI Table 2.4.12-1.4.

The hydrogeological investigation contractor drilled twenty one borings between May 24 and June 14, 2005 as shown in RAI Table 2.4.12-1.4. Boring logs for all of these holes, with the exception of OW-1001A and OW-1003, are contained in SSAR Appendix 2.4A (Appendix E report). Boring logs were not prepared for OW-1001A and OW-1003 as no soil samples were retrieved from these holes (*Note: Boring log OW-1003 should read OW-1003A, as described in the footnote to RAI Table 2.4.12-1.4*).

Of the twenty one borings drilled at the site, six were designated as "A" holes. These were: OW-1001A, OW-1002A, OW-1003A, OW-1005A, OW-1006A and OW-1008A. Four of these borings (OW-1001A, OW-1002A, OW-1003A, and OW-1005A) were abandoned because the diameter of the hole was too small to accommodate proper installation of the observation wells. Boring OW-1006A was abandoned because of a shortage in 4.25-in ID hollow-stem auger flights to advance the hole. The hole abandonment records for these borings are contained in SSAR Appendix 2.4A (Appendix F report). Boring OW-1008A is the upper portion of boring OW-1008 and was not abandoned. The "A" is designated to show that the upper portion of this boring was drilled using 3.25-in hollow-stem augers while the lower portion was drilled using the rotosonic drilling method.

2.4.12-1 (c) In the formulation of alternate conceptual models, as well as the design of monitoring programs, describe how the OW-1001 and OW-1001A data and the remarks of Summerour et al. (1998) are taken into account regarding the potential for communication between the Water Table aquifer and the Tertiary aquifer in the vicinity of fractures and faulting in the confining unit separating these two aquifers. The authors (Summerour et al. (1998)) state, "It is unclear whether the fractures also cut the Gordon aquitard. The large number of fractures and the fact that they appear to cut most of the aquitards in the stratigraphic sequence suggests that there may be leakage between aquifers near the Pen Branch fault. Therefore, both the Pen Branch fault and the associated fracture system may provide pathways for the movement of tritium from the Upper Three Runs aquifer into deeper, normally confined aquifers."

Response:

Summerour et al. (1998) do not present evidence in their discussion on the seismic reflection data collected and interpreted by Waddell et al. (1995) that would indicate that the anomalous groundwater levels observed in observation well OW-1001 could be attributed to communication between Water Table and Tertiary aquifers through fractures or faults in the BBM associated with the Pen Branch fault.

As part of an investigation of tritium in the Gordon (Tertiary aquifer) and other aquifers in Burke County, Georgia, Summerour et al. (1998) reported seismic reflection data collected and interpreted by Waddell et al. (1995). The seismic reflection survey extended over 7,000 ft in the vicinity of Hancock Landing and was intended to trace the extension of the Pen Branch fault into Georgia. The results of the survey identified three fault zones that cut the basement rock and extended into the lower Dublin aquifer, the upper Midville aquitard, the lower Midville aquitard, and the basal Appleton aquitard. However, there is no evidence to suggest that the fault zones extended into the Gordon aquitard (BBM). Summerour et al. state the following: "Whether the Pen Branch fault cuts the Gordon aquitard in the study area, remains uncertain". In addition, Waddell et al. identify a large number of short fractures within the Cretaceous and Tertiary age sediments associated with these fault zones. These short fractures are interpreted to cut the Dublin aquitard, upper Midville aquitard, the lower Midville aquitard and possibly the upper Dublin and Millers Pond aquitards. However, there is no evidence to suggest that these short fractures extend into the Gordon aquitard (BBM). Summerour et al. state the following: "It is unclear whether the fractures also cut the Gordon aquitard".

The validity of the interpretation of the seismic profile by Waddell et al. is drawn into question by the apparent misinterpretation of a series of depositional anomalies identified on the seismic profile. Waddell et al. interpret the depositional anomalies to be unconformities or channel features stacked vertically on top of one another. However, soil core retrieved from a boring drilled over the deepest part of one of these channels revealed a normal stratigraphic sequence without any evidence of channel scour or fill. Summerour et al. state the following: "The disparity between the seismic line and the core data remains unresolved. The existence of the channel features (and their effects on local groundwater flow patterns) remains unresolved".

Finally, the seismic reflection and refraction data collected at the VEGP site as part of the ESP application subsurface investigation program and reported by Bechtel Power Corporation (2006) projects the location of the Pen Branch fault at the top of basement rock further to the south than Waddell et al.'s (1995) projected location. As a result, the seismic reflection data collected by Waddell et al. do not traverse the

Pen Branch fault. The location of Waddell et al.'s seismic reflection survey is shown on SSAR Figure 2.5.1-34.

In addition, based on the information presented in response to RAI question 2.4.12-1(a), the groundwater level elevation data for observation wells OW-1001 and OW-1001A are considered invalid. For this reason, SSAR Tables 2.4.12-1 and -2 and ER Tables 2.3.1-18 and -19 will be replaced with the following RAI Tables 2.4.12-1.1 and 2.4.12-1.2 in the next revision of the ESP application. SSAR Figures 2.4.12-7 to 2.4.12-11 and Figures 2.4.12-14 to 2.4.12-18 and ER Figures 2.3.1-16 to 2.3.1-20 and 2.3.1-23 to 2.3.1-27 will also be revised in the next revision of the ESP application.

References:

Bechtel Power Corporation, 2006, Geologic Interpretation of Seismic Reflection Data at Vogtle Plant Site, Report Number 25144-006-V14-CY06-00008-001, August.

Summerour, J.H., Shapiro, E.A., and Huddlestun, P.F., 1998, An Investigation of Tritium in the Gordon and Other Aquifers in Burke County, Georgia, Phase II: Georgia Geologic Survey Information Circular 102, 72 p.

Waddell, M.G., Keith, J.F., and Domoracki, W. J., 1995, High resolution seismic characterization GGS-1, Burk county, GA; University of South Carolina Project Report to Georgia Geologic Survey, ESRI Technical Report 95-F129-1, 20 p., 2pl.

RAI Table 2.4.12-1.1 Monthly Groundwater Level Elevations in the Water Table Aquifer

	Groundwater Level Elevation (ft msl)																	
Well No.	**Jun-05**	**Jul-05**	**Aug-05**	**Sep-05**	**Oct-05**	**Nov-05**	**Dec-05**	**Jan-06**	**Feb-06**	**Mar-06**	**Apr-06**	**May-06**	**Jun-06**	**Jul-06**	**Aug-06**	**Sep-06**	**Oct-06**	**Nov-06**
142	154.37	154.38	154.49	154.64	154.75	154.69	154.60	154.71	154.78	154.71	154.63	154.55	154.48	154.41	154.36	0.00	0.00	154.16
179	147.42	148.40	148.42	148.72	148.69	148.75	148.52	148.61	148.64	148.72	148.66	148.76	148.78	148.56	148.75	0.00	0.00	148.79
802A	157.88	157.86	158.07	158.23	158.29	158.34	158.28	158.28	158.39	158.23	158.17	158.09	157.99	157.91	157.89	0.00	0.00	157.56
803A	159.98	159.91	160.15	160.32	160.39	160.48	160.39	160.37	160.48	160.45	160.30	160.20	160.12	159.96	159.88	0.00	0.00	159.64
804	163.73	163.62	163.92	164.10	164.21	164.23	164.05	164.08	164.23	164.30	164.11	163.99	163.88	163.69	163.69	0.00	0.00	162.84
805A	158.53	158.57	158.84	158.98	159.09	159.09	159.05	158.94	158.92	158.98	158.82	158.82	158.63	158.53	158.45	0.00	0.00	158.19
806B	155.62	155.65	155.78	155.90	155.96	155.98	155.88	155.97	155.98	156.03	155.85	155.78	155.73	155.68	155.62	0.00	0.00	155.42
808	158.88	159.14	159.42	159.55	159.49	159.37	159.15	159.04	159.19	159.15	158.99	158.53	158.80	158.72	158.65	0.00	0.00	158.40
809	152.78	152.70	152.75	152.89	152.98	152.97	152.98	153.10	153.22	153.18	153.05	153.02	153.00	152.88	152.86	0.00	0.00	152.71
LT-1B	154.92	154.82	155.01	155.16	155.18	155.22	155.06	155.18	155.52	155.28	155.18	155.15	154.95	154.95	154.95	0.00	0.00	154.78
LT-7A	154.39	154.15	154.33	154.46	154.48	154.46	154.31	154.57	154.83	154.59	154.57	154.50	154.41	154.30	154.34	0.00	0.00	154.25
LT-12	158.21	157.90	158.07	158.22	158.31	158.28	158.21	158.53	158.66	158.48	158.54	158.48	158.23	158.19	158.18	0.00	0.00	158.11
LT-13	156.10	155.92	156.13	156.30	156.32	156.37	156.23	156.36	156.66	156.35	156.32	156.32	156.23	156.08	156.14	0.00	0.00	155.93
OW-1001*	113.98	118.03	118.36	117.95	117.69	116.54	116.56	116.93	117.22	117.34	117.56	117.13	116.63	116.06	115.56	0.00	0.00	114.49
OW-1001A*					135.95	135.91	135.97	135.99	135.98	135.96	135.97	135.96	135.96	135.96	135.96	0.00	0.00	135.96
OW-1003	155.94	155.89	156.06	156.29	156.24	156.36	156.26	156.34	156.37	156.43	156.32	157.24	156.16	156.03	155.98	0.00	0.00	155.90
OW-1005	132.95	132.73	132.88	133.01	132.67	132.65	132.53	132.74	133.04	133.12	133.14	133.20	133.12	132.94	132.84	0.00	0.00	132.50
OW-1006	147.66	147.48	147.57	147.60	147.49	147.20	147.18	147.41	147.40	147.37	147.35	147.12	147.05	146.88	146.80	0.00	0.00	146.47
OW-1007	151.82	151.72	151.78	151.63	151.45	151.15	151.05	151.41	151.49	151.45	151.22	151.11	150.99	150.76	150.53	0.00	0.00	150.08
OW-1009	162.38	162.40	162.71	162.90	163.01	163.03	162.87	162.93	163.01	163.01	162.89	162.79	162.65	162.50	162.44	0.00	0.00	162.17
OW-1010	163.06	163.26	163.59	163.77	163.81	163.78	163.62	163.60	163.63	163.57	163.44	163.29	163.09	162.91	162.84	0.00	0.00	162.51
OW-1012	161.83	161.93	162.07	162.06	161.98	161.80	161.71	161.82	161.86	161.80	161.68	161.53	161.37	161.22	161.00	0.00	0.00	160.49
OW-1013	164.95	165.00	165.29	165.47	165.48	165.42	165.21	165.29	165.46	165.31	165.23	165.11	164.96	164.79	164.68	0.00	0.00	164.25
OW-1015	159.63	159.58	159.78	159.90	159.96	159.96	159.82	159.81	159.79	159.89	159.75	159.66	159.58	159.45	159.35	0.00	0.00	159.06

Note:

* Groundwater level elevations for OW-1001 and OW-1001A will be omitted in the next revision of the ESP application. Groundwater level elevations are shown in the above table for discussion purposes only.

RAI Table 2.4.12-1.2 Monthly Groundwater Level Elevations in the Tertiary Aquifer

	Groundwater Level Elevation (ft msl)																	
Well No.	**Jun-05**	**Jul-05**	**Aug-05**	**Sep-05**	**Oct-05**	**Nov-05**	**Dec-05**	**Jan-06**	**Feb-06**	**Mar-06**	**Apr-06**	**May-06**	**Jun-06**	**Jul-06**	**Aug-06**	**Sep-06**	**Oct-06**	**Nov-06**
27	91.50	89.96	91.63	83.96	82.13	88.24	82.57	84.62	85.77	84.49	83.42	83.08	83.03	84.54	84.73	0.00	0.00	81.50
29	98.88	97.80	98.33	93.17	91.86	91.89	92.59	93.97	94.19	93.63	93.05	92.16	91.76	91.86	91.44	0.00	0.00	89.97
850A	105.27	104.68	104.76	101.04	100.03	99.91	100.70	101.86	101.69	101.48	101.14	100.07	99.63	99.23	98.57	0.00	0.00	97.56
851A	114.54	114.40	114.02	111.59	111.38	110.60	112.34	112.32	112.43	112.42	112.23	111.08	110.36	109.31	108.00	0.00	0.00	107.77
852	114.71	114.49	114.00	111.88	111.09	111.21	111.88	113.06	113.51	113.14	112.82	111.74	110.38	108.78	107.20	0.00	0.00	108.35
853	108.60	108.17	107.98	104.51	103.64	103.45	104.18	105.32	105.14	104.97	104.65	103.58	103.15	102.57	101.86	0.00	0.00	101.13
854	107.06	106.88	106.65	103.37	102.38	102.23	102.38	104.13	103.85	103.73	103.45	102.31	101.86	101.31	100.57	0.00	0.00	99.87
855	102.63	101.74	102.00	97.22	96.08	96.21	96.85	98.43	98.48	98.15	97.53	96.75	95.93	95.85	94.96	0.00	0.00	94.12
856	114.07	113.94	113.49	111.37	110.57	110.63	111.31	112.52	112.46	112.39	112.07	111.21	109.94	108.36	106.75	0.00	0.00	107.75
OW-1002	120.76	120.61	120.04	118.65	117.81	117.71	118.44	119.36	119.63	119.64	119.43	118.37	117.65	116.45	114.48	0.00	0.00	114.77
OW-1004	108.27	108.14	108.01	105.06	104.05	103.75	104.51	105.56	105.38	105.28	105.12	103.88	103.54	102.81	102.06	0.00	0.00	101.26
OW-1008	126.06	127.99	125.09	124.24	123.49	123.51	124.19	125.10	125.46	125.54	125.21	124.33	123.42	122.18	119.64	0.00	0.00	120.42
OW-1011	122.50	122.38	121.49	120.37	119.59	119.73	120.46	121.41	121.64	121.70	121.48	120.47	119.37	117.67	115.35	0.00	0.00	116.59
OW-1014	111.18	111.00	110.74	108.34	107.34	107.11	107.81	108.87	108.73	108.75	108.66	107.41	106.94	105.98	104.86	0.00	0.00	104.44

RAI Table 2.4.12-1.3 Groundwater Level Elevations in the Blue Bluff Marl

Date	Observation Well and Water Level Elevation (ft msl)					
	900	**901**	**902**	**903**	**904B**	**905**
16-Jul-85	117.2	119.18	126.26	107.12	122.43	110.5
23-Jul-85	117.38	119.42	126.63	106.73	121.31	109.93
31-Jul-85	116.84	118.67	125.39	106.63	121.6	
7-Aug-85	116.69	118.57	125.18	106.46	121.31	109.36
14-Aug-85	117.09	119.01	125.27	106.34	121.31	109.28
21-Aug-85	116.35	118.24	124.72	106.49	121.31	109.58
28-Aug-85	116.21	118.07	124.36	106.33	121.46	104.45
4-Sep-85	116.16	118.04	124.25	106.28	121.3	109.35
11-Sep-85	116.19	118.03	124.19	106.29	121.29	109.37
18-Sep-85	115.76	117.5	123.58	105.96	121.29	108.94
25-Sep-85	115.55	117.35	123.4	105.74	121.29	108.74
6-Oct-85	115.23	117.04	123.06	105.56	121.29	108.66
9-Oct-85	115.39	117.07	123	106.05	121.29	108.61
16-Oct-85	115.68	117.4	123.38	106.08	121.29	108.84
23-Oct-85	115.65	117.36	123.33	106.04	121.29	108.81
30-Oct-85	115.52	117.27	123.18	105.75	121.29	108.58
6-Nov-85	115.51	117.18	123.06	105.89	121.29	108.6
13-Nov-85	115.4	117.08	122.93	105.79	121.29	108.55
20-Nov-85	115.36	117.12	123.08	105.75	121.31	108.58
27-Nov-85	115.42	117.27	123.11	105.81	121.4	108.75
4-Dec-85	115.7	117.29	122.99	106.49	121.22	108.97
11-Dec-85	115.81	117.39	122.97	106.66	121.3	109.17
28-Dec-85	115.75	117.4	123.04	106.21	121.3	108.84
2-Jan-86	115.53	117.23	123.06	106.29	121.4	108.85
10-Jan-86	115.79	117.4	123.08	106.31	121.4	108.87
15-Jan-86	115.85	117.45	123.08	106.51	121.39	108.94
22-Jan-86	116.09	117.63	123.16	106.44	121.4	108.96
29-Jan-86	116.09	117.63	123.12	107.16	121.31	109.6
5-Feb-86	116.61	118.33	123.82	107.04	121.31	109.57
12-Feb-86	116.67	118.37	123.95	106.88	121.25	109.3
19-Feb-86	116.84	118.56	124.11	107.24	121.27	109.57
26-Feb-86	116.82	118.53	124.06	107.16	121.25	109.77
5-Mar-86	116.59	118.3	123.91	106.99	121.36	109.57
15-Mar-86	116.52	118.26	123.86	106.76	121.35	109.49
19-Mar-86	116.54	118.24	123.98	106.86	121.2	109.47
26-Mar-86	116.16	117.75	123.44	107.41	121.57	109.77
2-Apr-86	116.25	117.9	123.51	107.25	121.33	109.8
9-Apr-86	116.46	118.25	123.71	107.13	121.27	109.9
16-Apr-86	116.16	117.75	123.42	107.41	121.27	109.77
23-Apr-86	115.82	117.58	123.41	106.89	121.72	109.07
30-Apr-86	115.84	117.61	123.48	106.09	121.22	109.06
7-May-86	115.87	117.61	123.43	105.71	121.3	108.74
14-May-86	115.14	116.89	122.83	105.37	121.26	108.36

RAI Table 2.4.12-1.3 Groundwater Level Elevations in the Blue Bluff Marl

Date	Observation Well and Water Level Elevation (ft msl)					
	900	**901**	**902**	**903**	**904B**	**905**
21-May-86	115.11	116.86	122.81	105.46	121.25	108.38
28-May-86	114.69	116.35	122.35	105.11	121.27	107.85
4-Jun-86	114.91	116.58	122.49	105.35	121.27	108.24
11-Jun-86	114.8	116.55	122.41	106.05	121.27	107.75
18-Jun-86	115.04	116.85	122.61	106.01	121.29	107.79
25-Jun-86	115.03	116.77	122.66	106.04	121.29	107.79
2-Jul-86	115.04	116.75	122.67	106.07	121.29	107.76
9-Jul-86	114.85	116.66	122.58	105.86	121.29	107.63
16-Jul-86	114.73	116.57	122.45	105.13	121.29	107.48
23-Jul-86	114.71	116.47	122.34	104.79	121.3	107.46
30-Jul-86	114.68	116.48	122.32	104.83	121.3	107.49
6-Aug-86	114.61	116.34	122.18	105.33	121.26	108.09
13-Aug-86	114.5	116.23	122.05	105.56	121.33	108.71
20-Aug-86	114.42	116.18	121.89	105.62	121.57	108.57
27-Aug-86	114.47	116.11	121.88	105.69	121.31	108.64
3-Sep-86	114.53	116.1	121.86	105.73	121.28	108.68
10-Sep-86	114.51	116.09	121.84	105.71	121.3	108.65
17-Sep-86	114.46	116.15	121.91	105.42	121.29	108.37
24-Sep-86	114.33	116.06	121.83	105.15	121.27	108.14
1-Oct-86	114.28	115.99	121.89	105.01	121.31	107.94
11-Oct-86	114.25	115.93	121.81	104.73	121.3	107.59
15-Oct-86	114.09	115.86	121.7	104.37	121.27	107.15
22-Oct-86	114	115.67	121.46	104.12	121.25	106.94
29-Oct-86	114.05	115.73	121.42	104.09	121.28	106.86
5-Nov-86	113.7	115.42	121.4	104.12	121.24	109.92
12-Nov-86	113.65	115.45	121.37	104.13	121.28	106.91
19-Nov-86	113.8	115.51	121.29	104.15	121.26	106.87
26-Nov-86	113.77	115.55	121.32	104.11	121.28	106.87
3-Dec-86	113.81	115.57	121.28	104.18	121.27	106.82
31-Dec-86	114.31	115.95	121.56	105.31	121.27	107.8
10-Jan-87	114.5	116.05	121.61	105.81	121.28	108.19
14-Jan-87	114.67	116.33	121.84	105.77	121.26	108.18
21-Jan-87	114.7	116.23	121.77	105.69	121.31	108.16
28-Jan-87	115.16	116.58	121.83	106.94	121.28	108.86

Notes:
1) Well data for 15 July 1985 to 11 December 1985 contained in the Observation Well Readings report for VEGP Units 1 and 2, July - December 1985 (Georgia Power 1986).
2) Well data for 28 Dec 1985 to 18 June 1986 contained in the Observation Well Readings report for VEGP Units 1 and 2, January - June 1986 (Georgia Power 1986).
3) Well data for 25 June 1986 to 31 December 1986 contained in the Observation Well Readings report for VEGP Units 1 and 2, July – December (Georgia Power 1987).
4) Well data for 10 Jan 1987 to 28 Jan 1987 contained in the Piezometer Weekly Readings report (Georgia Power 1987).

RAI Table 2.4.12-1.4 Summary of Holes Drilled at the Site for the Installation of Observation Wells

Boring / Drill Log No.	Drilling Method	Drill Dates		Sampled Depth		Drilled Depth Below the GS	Boring "Abandoned" or "Well" Installed
		Start	End	From (ft)	To (ft)	(ft)	
OW-1001A	3.25" HSA	25-May	25-May	No sampling		100	Abandoned
OW-1001	4.25" HSA	24-May	29-May	113.5	140	140	Well
OW-1002A	3.25" HSA	24-May	25-May	0	108.5	108.5	Abandoned
OW-1002	Rotosonic	2-Jun	6-Jun	87	237	237	Well
OW-1003A	3.25" HSA	24-May	24-May	0	90	90	Abandoned
OW-1003	4.25" HSA	25-May	25-May	No sampling		90.5	Well
OW-1004	Rotosonic	3-Jun	11-Jun	87	187	187	Well
OW-1005A	3.25" HSA	31-May	31-May	0	75	75	Abandoned
OW-1005	4.25" HSA	2-Jun	7-Jun	68.5	170	170	Well
OW-1006A	4.25" HSA	3-Jun	4-Jun	0	125	125	Abandoned
OW-1006	4.25" HSA	9-Jun	14-Jun	118.5	135	135	Well
OW-1007	4.25" HSA	4-Jun	7-Jun	98.5	122	122	Well
OW-1008A	3.25" HSA	26-May	26-May	0	107.5	105	Well OW-1008
OW-1008	Rotosonic	31 May	1-Jun	108	247	247	Well
OW-1009	4.25" HSA	24-May	27-May	0	100	100	Well
OW-1010	4.25" HSA	1-Jun	1-Jun	0	93.5	93.5	Well
OW-1011	Rotosonic	11-Jun	12-Jun	87	217	217	Well
OW-1012	4.25" HSA	31-May	1-Jun	0	93.6	93.6	Well
OW-1013	4.25" HSA	9-Jun	10-Jun	0	103.5	103.5	Well
OW-1014	Rotosonic	11-Jun	11-Jun	97	197.4	197.4	Well
OW-1015	4.25" HSA	30-May	3-Jun	0	120	120	Well

Notes:

1) Borings OW-1001A, OW-1002A, OW-1003A, and OW-1005A were abandoned due to the use of 3.25-in hollow stem auger, which would not adequately accommodate well installation.

2) Boring OW-1006A was abandoned due to the of shortage hollow stem auger flights.

3) Boring log OW-1003 contained in SSAR Appendix 2.4A (Appendix E report) should read OW-1003A.

4) The drilling method for boring OW-1006 is assumed to be 4.25" HSA (not described in SSAR Appendix 2.4A (Appendix E report)).

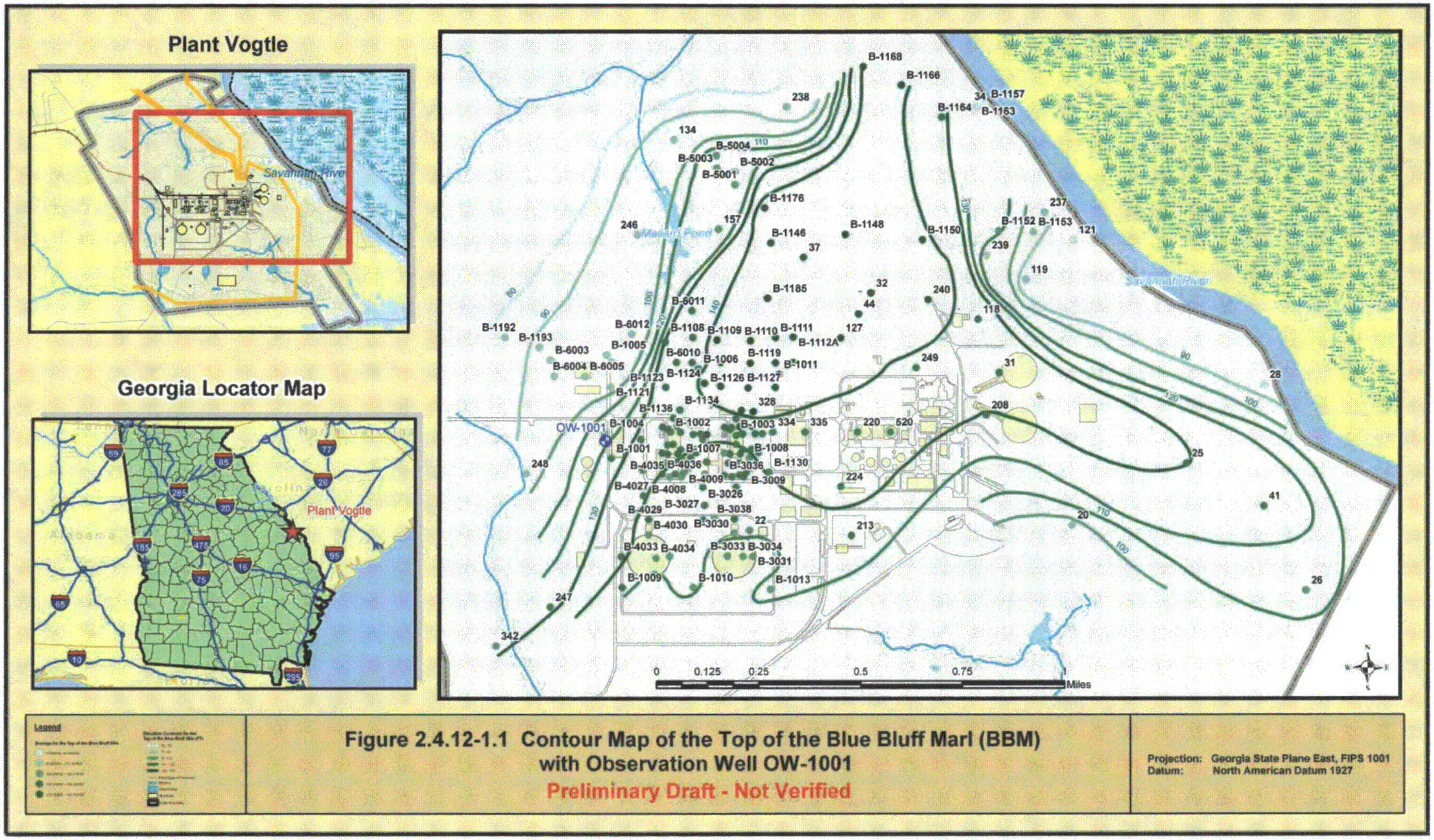
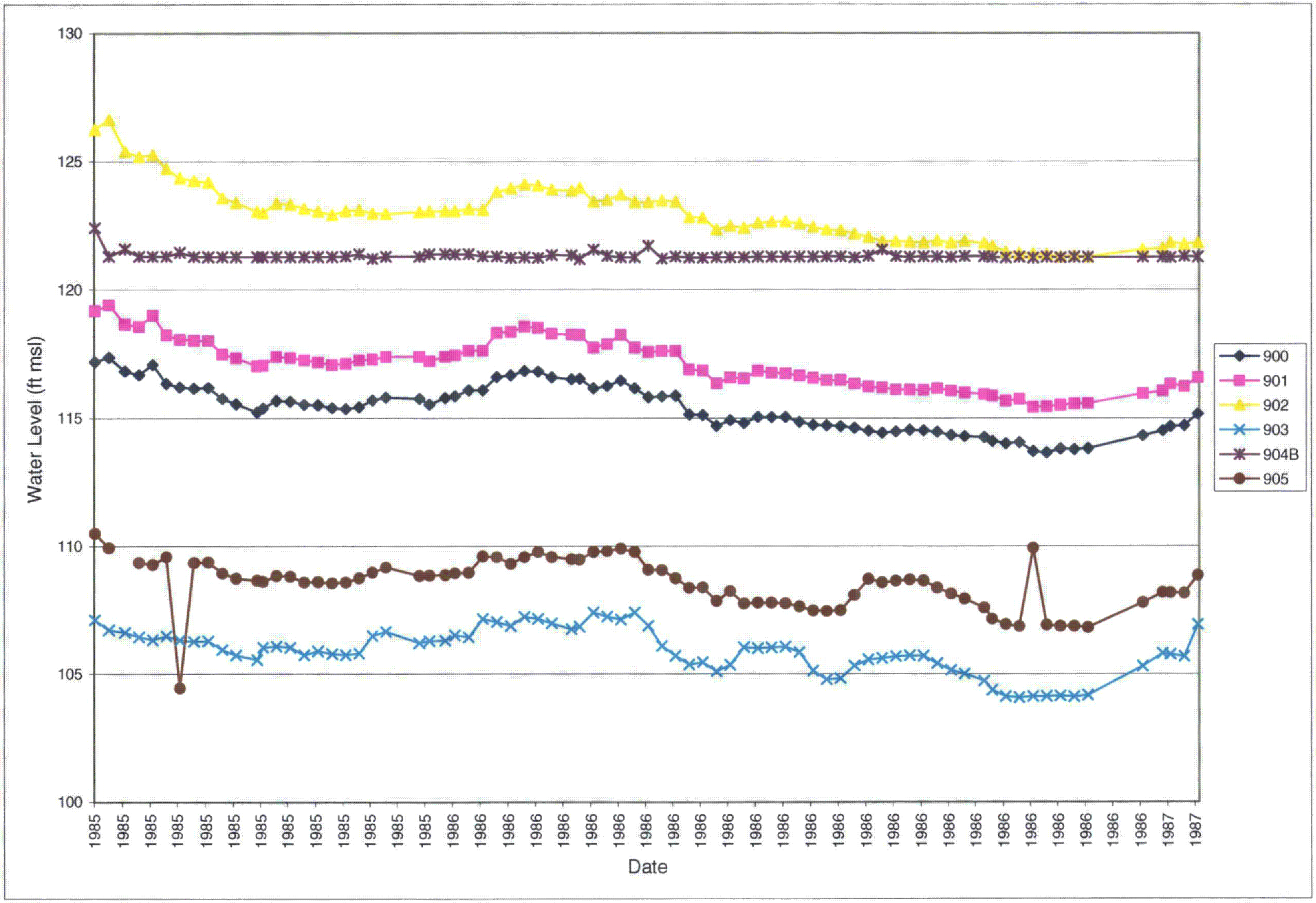


Figure 2.4.12-1.1 Contour Map of the Top of the Blue Bluff Marl (BBM) with Observation Well OW-1001

Preliminary Draft - Not Verified

RAI Figure 2.4.12-1.1



RAI Figure 2.4.12-1.2 Marl Aquitard Hydrographs for the Period 1985 to 1987

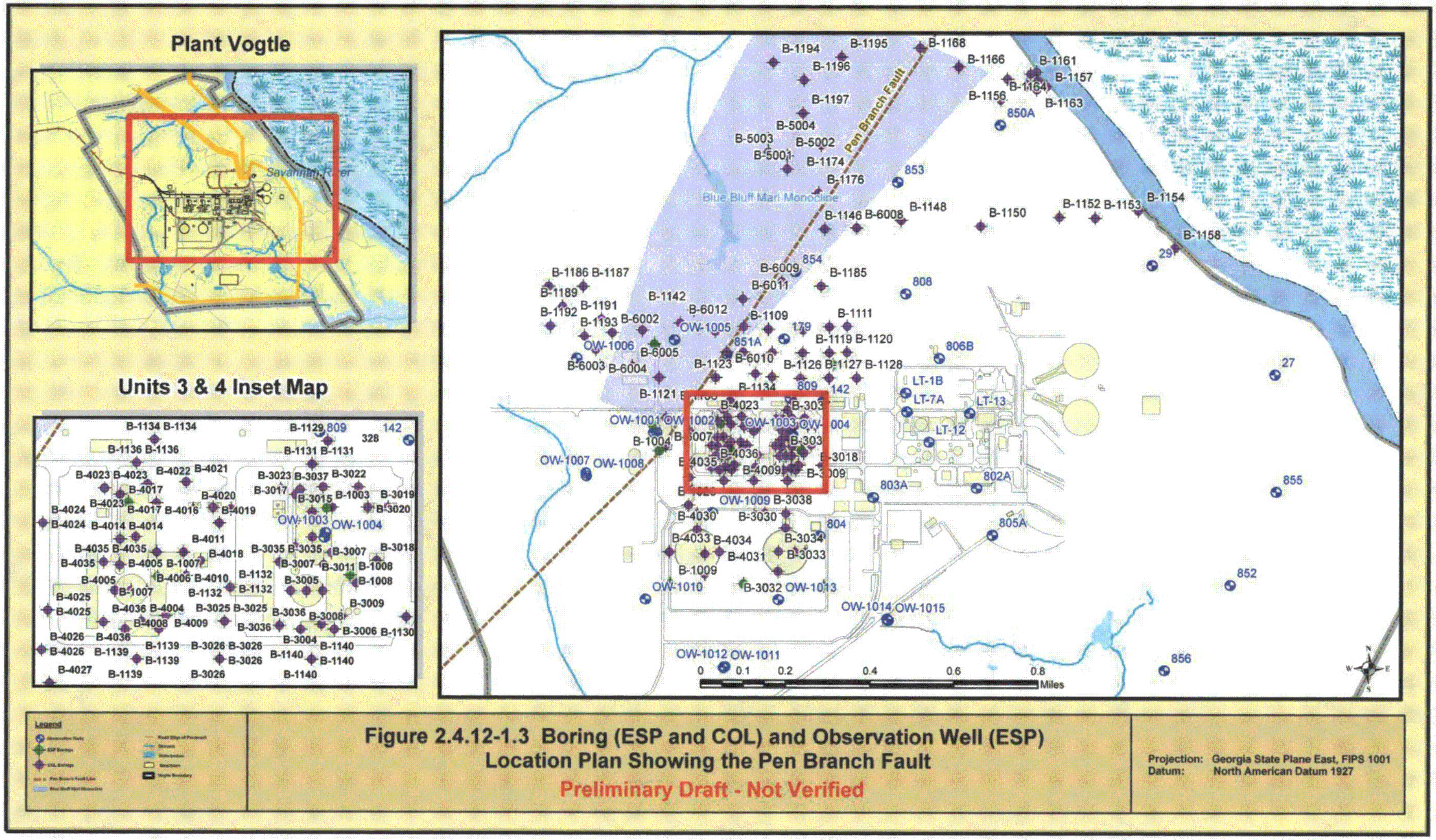
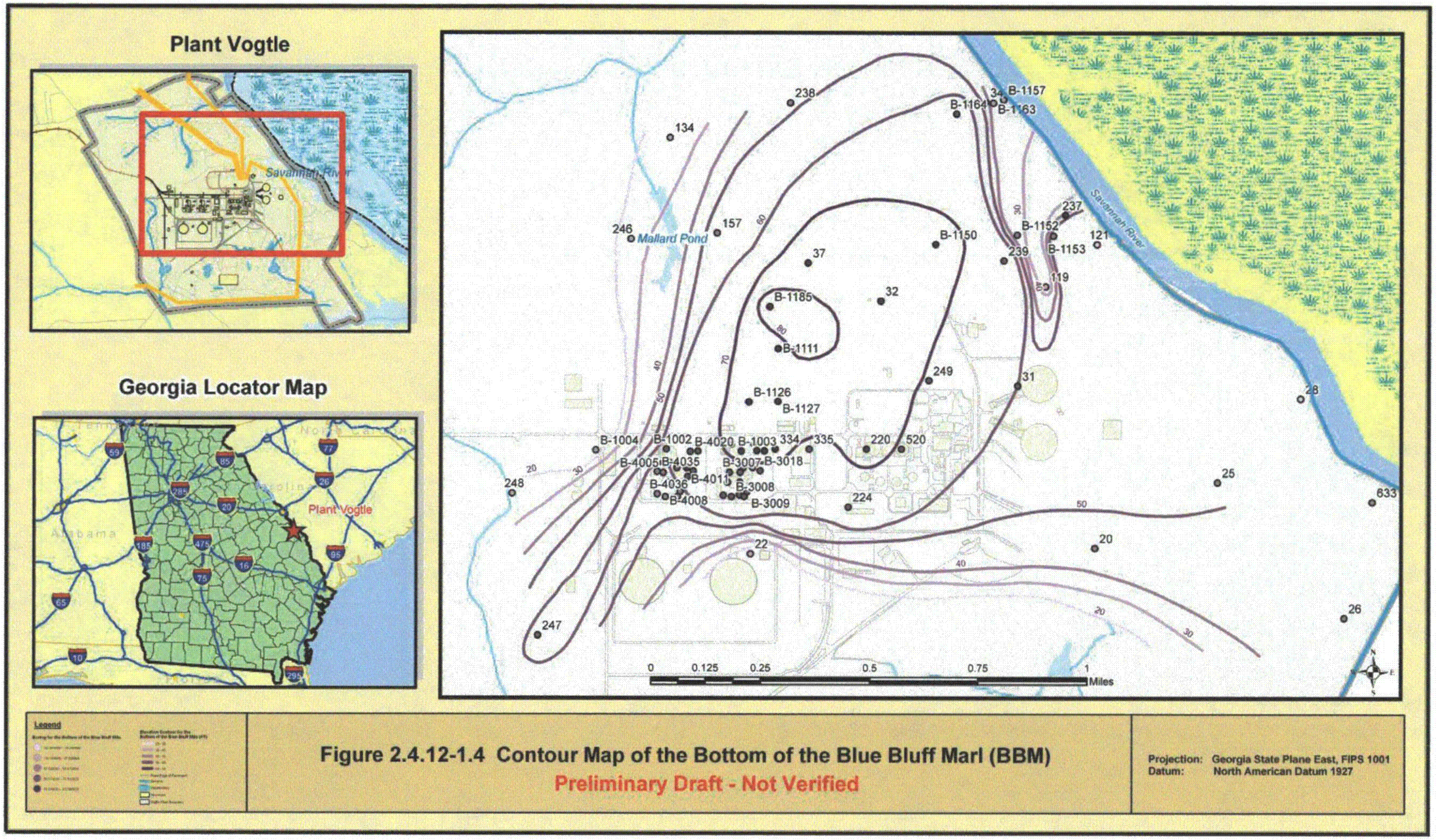


Figure 2.4.12-1.3 Boring (ESP and COL) and Observation Well (ESP) Location Plan Showing the Pen Branch Fault

RAI Figure 2.4.12-1.3



RAI Figure 2.4.12-1.4

2.4.12-2 (a) **Figure 2.4.12-4 indicates that Water Table Aquifer recovery to an asymptotic value following cessation of dewatering requires 1.5 to 2 years. However, there is no record of the period preceding the dewatering activity in the figure. Also Figures 2.4.12-4, -5 & -6 show that the groundwater levels in the Water Table Aquifer vary significantly from year to year or even from season to season for some periods of time and not for others. These facts indicate that the Water Table Aquifer is able to undergo substantial change in water table elevation (with a corresponding movement of water) while not undergoing substantial local stress (pumping), and being isolated from the underlying confined Tertiary aquifer that is stressed by Unit 1 & 2 operations. It is essential that the underlying conceptual model of the unconfined aquifer and key parameters describing the aquifer should be in agreement with this known system behavior, (e.g., that the hydraulic conductivity, storativity, and porosity, should be consistent with the ability of the unconfined aquifer to respond to dewatering, severe drought, and the return to a normal precipitation level) in order to identify potential contamination pathways in the ground and to estimate corresponding travel times.**

Note: 2.4.12-2 items 1-4 were included to clarify RAI 2.4.12-2 (a) needed information. These clarification were included as a result of a conference call and by an email transmittal from Christian Araguas, dated, Thursday, March 22, 2007.

(1) Provide information on dewatering data on rate and amount of water pumped out with corresponding dates and time from the units 1&2 construction activities.

Response:

The following references contain information on dewatering for Units 1 and 2 and are provided in Enclosure 3:

Bechtel Power Corporation and Georgia Power Company, 1980, Final Report on Dewatering and Repair of Erosion in Category I Backfill in Power Block Area, Vogtle 5.16.

Bechtel Power Corporation, 1972, Aquifer Tests for Construction Dewatering, Vogtle 8.7.1

(2) Provide hydraulic conductivity maps for the Barnwell Aquifer and the Utley Limestone Aquifer drawn based on all available existing information, including those from recent site exploration work for COL application.

Response:

At the VEGP site, the Water Table aquifer is found in the Barnwell sands and Utley limestone. For the ESP application, the hydraulic conductivity values for the Water Table aquifer were determined from in situ hydraulic conductivity (slug) tests performed in the groundwater observation wells installed at the site. The results of these tests are presented in SSAR Table 2.4.12-2 and summarized in the following RAI Table 2.4.12-2.1. RAI Table 2.4.12-2.1 shows that the wells are screened in portions of the Barnwell sands and Utley limestone with hydraulic conductivity values ranging from 0.12 to 2.65 ft/day.

For the VEGP Units 1 and 2, hydraulic conductivity tests were performed in the Barnwell sands and the Utley limestone, as described in Section 2.4.12.2.4.3 of the VEGP Updated Final Safety Analysis Report (UFSAR). The hydraulic conductivity tests performed in the Barnwell sands consisted of two in situ constant head tests and three laboratory tests on undisturbed samples of the Barnwell sands. The results are presented in UFSAR Table 2.4.12-12 and are summarized in the following RAI Table 2.4.12-2.2. RAI Table 2.4.12-2.2 shows the hydraulic conductivity values ranging from 9.8 ft/yr (0.03 ft/day) to 302 ft/yr (0.8 ft/day). Hydraulic conductivity tests performed in the Utley limestone consisted of two pumping tests, seven falling head and four constant head tests. The results are presented in UFSAR Table 2.4.12-13 and are summarized in the following RAI Table 2.4.12-2.2. The results of one pumping test indicate the possibility of localized, highly permeable zones in the Utley limestone based on hydraulic conductivity values ranging from 14,100 ft/yr (39 ft/day) to 125,400 ft/yr (343 ft/day). A second pumping test was performed in a less permeable zone of the limestone resulting in an estimated hydraulic conductivity of 3,250 ft/yr (9 ft/day). The falling head and constant head tests yielded hydraulic conductivity values ranging from 96 ft/yr (0.3 ft/day) to 5,800 ft/yr (16 ft/day).

It is not possible to construct hydraulic conductivity maps for the Barnwell sands and Utley limestone because the locations of the in situ hydraulic conductivity tests for the VEGP Units 1 and 2 are unavailable.

For the current subsurface investigation program for the COL application, no in situ hydraulic conductivity tests are being performed.

(3) Provide top and bottom elevation contours for the Barnwell Aquifer and the Utley Limestone Aquifer drawn from all available existing information – this was requested in 2.4.13-3(a).

Response.

Contours of the top of the Barnwell sands underlying the VEGP site are shown on the following RAI Figure 2.4.12-2.1. As the Barnwell sands were typically encountered at the ground surface, RAI Figure 2.4.12-2.1 is based upon surface topography.

The thickness of the Utley limestone in the vicinity of the VEGP site is shown on following RAI Figure 2.4.12-2.2. RAI Figure 2.4.12-2.2 was developed primarily from boring information obtained from the subsurface investigation program for the ESP application and preliminary boring information obtained from the current subsurface investigation for the COL application. The boring logs prepared for the ESP application are contained in SSAR Appendix 2.5A. The boring logs for the COL application are currently in preliminary form.

(4) Provide historical groundwater level data at the Surficial Aquifer from 1995 to 2004 at the monitoring wells that are not covered by SSAR Figure 2.4.12-5.

NOTE: RAI Table 2.4.12-2.3, following this response, summarizes all available historical groundwater level elevations for the Water Table aquifer for the period between 1971 to 2004 for the following observation wells: 142, 179, 802A, 803A, 804, 805A, 806B, 808, 809, LT-1B, LT-7A, LT-12 and LT-13. RAI Figure 2.4.12-2.3 shows the hydrographs for each well for the period between 1979 and 2004.

Response:

Historical groundwater level elevations for the 1995 to 2004 period for the Water Table aquifer are not available for the following observation wells: 142, 179, 803A, 804 and 809.

In the next revision of the ESP application, SSAR Figures 2.4.12-4 and -5 and ER Figures 2.3.1-13 and -14 will be combined and replaced with RAI Figure 2.4.12-2.3.

2.4.12-2 (b) Provide and incorporate into the SSAR a discussion of the process used to develop the site hydrologic conceptual model. Discuss the various conceptual models considered in developing the final conceptual model, and how your model contrasts with the conceptual models of the Vogtle Electric Generating Plant (VEGP) Updated Final Safety Analysis Report, and United States Geological Survey (USGS) studies (Clark and West 1997; Cherry 2006). Describe the data sets and rationale used to establish the final conceptual model. This discussion of the conceptual model should cover the continuity or discontinuity of the hydrogeologic units, and their connectivity to the other surface water features, and then to the Savannah River.

Response.

Conceptual Model Description

The conceptual hydrogeological model for the VEGP site was developed using site-specific data acquired to support the ESP application, information and data included in the VEGP Updated Final Safety Analysis Report, U.S Geological Survey studies, and Georgia Geologic Survey studies.

The VEGP site is located in the Coastal Plain physiographic province. Coastal Plain sediments comprise three aquifer systems consisting of seven aquifers that are separated hydraulically by confining units. As discussed by Clarke and West (1997), the aquifer systems are, in descending order: (1) the Floridan aquifer system, which consists of the Upper Three Runs and Gordon aquifers in sediments of Eocene age; (2) the Dublin aquifer system, consisting of the Millers Pond, upper Dublin, and lower Dublin of Paleocene-Late Cretaceous age; and (3) the Midville aquifer system, consisting of the upper Midville and lower Midville aquifers in sediments of Late Cretaceous age. Note that nomenclature used by the U.S. Geological Survey (Clarke and West, 1997) for geologic and hydrogeologic units differs from that used in the ESP application. In the ESP application, the Water Table aquifer comprises the Upper Three Runs aquifer, the Tertiary sand aquifer comprises the Gordon aquifer, and the Cretaceous aquifer comprises the Dublin and Midville aquifer systems. (A figure showing the schematic hydrostratigraphic classification at the VEGP site will be included in the next revision of the ESP application).

The Upper Three Runs aquifer is the shallowest aquifer and is unconfined to semi-confined throughout most of the area. Groundwater levels in the Upper Three Runs aquifer respond to a local flow system and are affected mostly by topography and climate. Groundwater flow in the deeper, Gordon aquifer and Dublin and Midville aquifer systems is characterized by local flow near outcrop areas to the northwest, changing to intermediate flow and then regional flow downdip (southeastward) as the aquifers become more deeply buried. Water levels in these deeper aquifers show a pronounced response to topography and climate in the vicinity of outcrops that diminishes southeastward where the aquifer is more deeply buried. Stream stage and pumpage affect groundwater levels in these deeper aquifers to varying degrees throughout the area. (Clarke and West 1997)

The geologic characteristics of the Savannah River alluvial valley substantially control the configuration of potentiometric surfaces, groundwater flow directions, and stream-aquifer relations. Data from 18 shallow borings indicate incision into each aquifer by the paleo Savannah River and subsequent infill by permeable alluvium have resulted in direct hydraulic connection between the aquifers and the Savannah River along various parts of its reach. This hydraulic connection may be the cause of large groundwater discharge to the river near Jackson, South Carolina (located approximately 8 miles northeast of the VEGP site) as evidenced by stream baseflow and potentiometric measurements, where the Gordon aquifer is in contact with Savannah River alluvium, and also the cause of lows or depressions in potentiometric surfaces of confined aquifers that are in contact with the alluvium. Groundwater in these aquifers flows toward the depressions. The influence of the river diminishes downstream where the aquifers become deeply buried beneath the river channel, and where upstream and downstream groundwater flow is possibly separated by a water divide or "saddle". Water-level data indicate that saddle features probably exist in the Gordon aquifer and Dublin aquifer system, with the groundwater divide occurring just downstream of the VEGP site, and also might be present in the Midville aquifer system, as shown on Plate 1 produced by Clarke and West (1997)

Basin-wide potentiometric-surface maps for the unconfined Upper Three Runs aquifer and confined Gordon, Dublin and Midville aquifer systems have been prepared using historical data (Clarke and West 1997) and numerical simulation (Cherry 2006). Detailed discussions of these maps are provided in the cited references. Data from observation wells installed and monitored for one year at the VEGP site have also been used to develop potentiometric-surface maps on a more highly resolved, site-specific basis. The groundwater flow directions inferred from the maps are generally consistent with the larger-scale maps produced by Clarke and West (1997) and Cherry (2006), i.e., groundwater flow in the Upper Three Runs (Water Table) aquifer generally conforms with surface topography, while that in the confined Gordon (Tertiary) aquifer is towards the Savannah River.

Recharge to the Upper Three Runs (Water Table) aquifer is almost exclusively by precipitation, while discharge is primarily to local drainages. Recharge to the confined Gordon, Dublin, and Midville (Tertiary and Cretaceous) aquifers occurs primarily by direct infiltration of rainfall in their outcrop areas northwest of the VEGP site and generally parallel to the Fall Line (the boundary between the Coastal Plain and Piedmont physiographic provinces). Because the permeable alluvium of the Savannah River valley allows for direct hydraulic connection between aquifers and the Savannah River, the river serves as the major discharge area for the confined aquifers in hydraulic connection with the river valley alluvium. Potentiometric maps presented by Clarke and West (1997) indicate groundwater discharge from the confined Gordon, Dublin, and Midville aquifers to the Savannah River. For the shallower Gordon confined aquifer, groundwater flow directions are generally perpendicular to the river reach. In the case of the deeper Dublin and Midville aquifers, there are upriver components to the groundwater flow directions that depend on where the paleo river channel has breached confining units. Clarke and West (1997) provide a detailed discussion of this phenomenon.

Although a water budget for the VEGP site has not been quantified, recharge and discharge rates have been estimated on a basin-wide basis by other investigators. Clarke and West (1997) estimated groundwater discharge to the Savannah River based on the net gain in stream discharge for local, intermediate, and regional groundwater flow systems and for different hydrologic conditions. Groundwater discharge ranged from 910 ft^3/s during a drought year (1941), to 1,670 ft^3/s during a wet year (1949), and averaged 1,220 ft^3/s. Of the average discharge, the local flow system contributed an estimated 560 ft^3/s and the intermediate and regional flow systems contributed an estimated 660 ft^3/s. Clarke and West (1997) approximated the long-term average recharge by weighting these values according to drainage area, and estimated the average groundwater recharge in the Savannah River basin to be 14.5 inches, of which 6.8 inches is to the local flow system, 5.8 inches is to the intermediate flow system, and 1.9 inches is to the regional flow system. Mean-annual precipitation in the basin ranges from 44 to 48 inches. Cherry (2006) presents simulated water budgets for different hydrologic conditions using a numerical model for groundwater flow near the Savannah River Site, Georgia and South Carolina. Estimates of inflow or outflow across lateral boundaries, recharge, discharge, groundwater pumpage, and vertical flow upward and downward across confining units are obtained from the numerical model.

Near-Field Subsurface Conceptual Model Description

As described in SSAR Section 2.5.4.5, construction of the new units will require a substantial amount of excavation and backfill. The excavation will be necessary to completely remove the Upper Sand Stratum (Barnwell Group and Utley limestone). Total excavation depth to the Blue Bluff Marl bearing stratum is expected to range from approximately 80 to 90 ft below existing grade. Backfilling will be performed from the top of the Blue Bluff Marl to the bottom of the containment and auxiliary buildings at a depth of about 40 ft below final grade. Filling will continue up around these structures to final grade. The fill will primarily consist of granular materials, selected from portions of the excavated Upper Sand Stratum and from other available borrow sources. Following the guidelines used during construction of VEGP Units 1 and 2, structural fill will be a sandy or silty sand material with no more than 25 percent of the particle sizes smaller than the No. 200 sieve. This structural fill will be compacted to an average of 97 percent of the maximum dry density.

Excavating existing soils and replacing these soils with structural fill will alter the hydrogeologic characteristics of the subsurface materials within the footprint of VEGP Units 3 and 4. In situ hydraulic testing of fill material for VEGP Units 1 and 2 indicates a hydraulic conductivity range of 480 ft/yr (1.3 ft/day) to 1,220 ft/yr (3.3 ft/day) based data included in Table 2.4.12-15 of the UFSAR. Values for Units 3 and 4 are expected to be similar because the borrow sources and compaction criteria for the fill will be the same. Compared to the hydraulic conductivities for the Water Table aquifer (ER Table 2.3.1-20), it can be seen that the hydraulic conductivity of the fill is generally higher than that of the in situ soils.

Development of VEGP Units 3 and 4 will also increase the impervious area across the VEGP site where power generation and associated facilities are constructed. Storm-water management facilities (e.g., catch basins, storm sewers) will be used to convey runoff from precipitation offsite. The increased impervious area and use of storm-water management facilities will tend to reduce the recharge to the Water Table aquifer in areas affected by Units 3 and 4 construction.

Construction of VEGP Units 3 and 4 will entail the placement of relatively large and impermeable structures below grade. The base elevations of the major structures (containment and auxiliary buildings) will be at about El. 180 ft msl. This elevation is at least 15 ft above the water table. Because these structures will not extend below the water table, they would not affect the hydrogeologic characteristics of the underlying saturated zone.

<u>Continuity of the Utley Limestone</u>

As noted in ER Section 2.3.1.2.2, the Utley limestone consists of sand, clay, and silt with carbonate-rich layers. The stratum is discontinuous across the VEGP site and was not encountered in several of the ESP borings. To assess its degree of discontinuity, borings logged for the hydrogeological and geotechnical investigations have been examined for the presence/absence of the Utley limestone. Logs for these borings are included in SSAR Appendices 2.4A and 2.5A. In completing this assessment, effort was made to eliminate spatial bias. Therefore, only one boring log was considered when there were adjacent borings from OW-series well pairs, or adjacent B- and OW-series borings.

Spatial trends in the presence/absence of the Utley limestone indicate that the unit tends to be present in the power block area for VEGP Units 3 and 4 and the area to the north towards Mallard Pond. The Utley limestone tends to be absent in the cooling tower area for VEGP Units 3 and 4 and the area to the south. These results are consistent with the Utley limestone isopachs presented in the UFSAR for VEGP Units 1 and 2. These isopachs indicate that the limestone increases in thickness to a maximum of about 80 ft and then decreases in thickness to 10 ft or less along a profile extending from the power block to Mallard Pond, with the long axis of this limestone unit trending in a northeast-southwest direction.

These results along with water table contour maps indicate that groundwater flow from the power block area to the north and towards Mallard Pond will occur in the Utley limestone, as the data suggest that the limestone is continuous along this pathway.

Continuity of the Blue Bluff Marl

Section 2.5.1.2.2.2.1.1 of the UFSAR for VEGP Units 1 and 2 indicates that the Blue Bluff marl is a distinct unit that is relatively constant in thickness over many square miles, although variable in lithology. Contours of the upper and lower surfaces as well as an isopach map of the marl in the vicinity of the plant are shown on UFSAR drawings AX6DD352, AX6DD371, and AX6DD372. These drawings indicate the Blue Bluff Marl to be continuous over the entire VEGP site. On the VEGP site, the ESP application subsurface investigation (SSAR Appendix 2.5A) determined that the Blue Bluff Marl ranges in thickness from 63 to 95 ft at three locations where the stratum was fully penetrated, with an average thickness of 76 ft and a median thickness of 69 ft.

With respect to data from well OW-1001 (screened within the Water Table aquifer, but with measured hydraulic head values appearing to be more consistent with the Tertiary aquifer), further review of boring logs, well construction logs, and water levels for both wells indicates that water levels recorded in this wells are invalid. Given these results and considering that the Blue Bluff Marl was encountered in deeper borings in the vicinity of wells OW-1001, there is no evidence suggesting that the Blue Bluff Marl is absent or discontinuous at this location.

Isolation of Tertiary and Cretaceous Aquifers

Summerour et al. (1998) and SSAR Section 2.5.1.2.4 present evidence indicating that the Tertiary and Cretaceous aquifers are isolated from the Water Table aquifer. Seismic data acquired at the VEGP site indicate that the fault terminates in the Cretaceous deposits and does not extend into the Tertiary-age Gordon aquitard (Blue Bluff Marl) isolating the unconfined and confined aquifers. Additional discussion is provided below under "Location and Role of the Pen Branch Fault."

Hydraulic Connection of Hydrologic Units to the Savannah River Through River Alluvium

Clarke and West (1997) have documented the direct hydraulic connection between aquifers and the Savannah River along parts of its reach. This connection occurs due to incision into each aquifer by the paleo Savannah River and the subsequent deposition of permeable alluvium. Additional discussion of this hydraulic connection is given in the conceptual model description provided above. Clarke and West (1997) provide detailed discussion and further analysis.

Location and Role of the Pen Branch Fault

SSAR Section 2.5.1.2.4 describes previous investigations of the Pen Branch fault and the site subsurface investigation of the fault that was conducted for the ESP application. Results of the ESP investigation, which included seismic reflection and refraction surveys, clearly document that the Pen Branch fault strikes northeast and dips southeast beneath the VEGP site. SSAR Figure 2.5.1-42 shows the vertical projection of the Pen Branch fault from the top of basement rock in relation to VEGP Units 3 and 4. The plan projection of the intersection of the Pen Branch fault with the top of basement rock is located beneath or slightly southeast of the antiformal hinge at the top of the monocline in the Blue Bluff Marl (SSAR Figure 2.5.1-39). Because of its spatial association with the Pen Branch fault, it is likely that this monocline feature is the result of reverse or reverse-oblique slip on the Pen Branch fault. The seismic survey data further indicate that the fault terminates in the Cretaceous Coastal Plain deposits and does not extend into the overlying Tertiary deposits, including those comprising the Gordon (Tertiary sand) aquifer, Gordon aquitard (Blue Bluff Marl), and Upper Three Runs (Water Table) aquifer, are therefore not affected by the Pen Branch fault. This result is consistent with that of Summerour et al. (1998), who reported that none of the faults identified in their seismic surveys appear to have disturbed the Gordon aquitard (Blue Bluff Marl), which isolates the unconfined aquifer from underlying confined aquifers.

Based on the results and discussion presented above, the Pen Branch fault has not affected the Tertiary deposits at the VEGP site and would be neither a barrier nor conduit for transport in these deposits. Insufficient data are available to determine if the fault would be a barrier or conduit in the deeper, Cretaceous deposits that have been affected by the fault.

The next revision to the ESP application will address as appropriate the information provided in this response.

References:

Cherry, G.S., 2006, Simulation and Particle-Tracking Analysis of Ground-Water Flow Near the Savannah River Site, Georgia and South Carolina, 2002, and for Selected Water-Management Scenarios, 2002 and 2020: U.S Geological Survey Scientific Investigations Report 2006-5195, 156 p.

Clarke, J.S., and West, C.T., 1997, Ground-Water Levels, Predevelopment Ground-Water Flow, and Stream-Aquifer Relations in the Vicinity of the Savannah River Site, Georgia and South Carolina: U.S Geological Survey Water-Resources Investigations Report 97-4197, 120 p.

Clarke, J.S., and West, C.T., 1998, Simulation of Ground-Water Flow and Stream-Aquifer Relations in the Vicinity of the Savannah River Site, Georgia and South Carolina: U.S Geological Survey Water-Resources Investigations Report 98-4062, 134 p.

Georgia Department of Natural Resources, 2004, Environmental Radiation Surveillance Report, 2000-2002, Environmental Protection Division, March.

Georgia Power, 1985, Ground Water Supplement, Vogtle Electric Plant Unit 1 and Unit 2, March.

Summerour, J.H., Lineback, J.A, Huddlestun, P.F., and Hughes, A.C., 1994, An Investigation of Tritium in the Gordon and Other Aquifers in Burke County, Georgia: Georgia Geologic Survey Information Circular 95, 93 p.

Summerour, J.H., Shapiro, E.A., and Huddlestun, P.F., 1998, An Investigation of Tritium in the Gordon and Other Aquifers in Burke County, Georgia, Phase II: Georgia Geologic Survey Information Circular 102, 72 p.

2.4.12-2 (c) Discuss the reasons why the temporal variability of the water levels in the Water Table Aquifer during the period from 2005 to 2006 (Figure 2.4.12-6) were reduced substantially compared to those before 2005 (Figures 2.4.12-4&5).

NOTE on 2.4.12-2 (c) related Figures and Tables:
The following RAI Table 2.4.12-2.3, summarizes all available historical groundwater level elevations for the Water Table aquifer for the period between 1971 to 2004 for the following observation wells: 142, 179, 802A, 803A, 804, 805A, 806B, 808, 809, LT-1B, LT-7A, LT-12 and LT-13. In addition, the following RAI Figure 2.4.12-2.3, plots the hydrographs for each well for the period between 1979 and 2004. In addition, minor typographic and transcription errors were identified in ER Tables 2.3.1-18 and SSAR Tables 2.4.12-1. The corrected monthly groundwater level elevations in the Water Table aquifer for the 2005 to 2006 period are shown in the following RAI Table 2.4.12-2.4 and the following RAI Figure 2.4.12-2.4 plots the hydrographs for each well for this time period.

Response:

Groundwater levels in the Water Table aquifer exhibit very little variability over the 17 month monitoring period between June 2005 and November 2006 because the recharge during this period was evidently relatively constant. Comparison of historical groundwater level elevations to precipitation events and other meteorological indices over a longer time period suggest that persistent and significant wet weather is required to elicit any significant water table response. A discussion of the data supporting this conclusion is provided below.

RAI Figure 2.4.12-2.3 shows the groundwater level data for the Water Table aquifer available for the 1979 through 2006 period. Also shown on this figure is annual precipitation measured at three climate stations close to the VEGP site, which includes the Augusta WSO Airport, Waynesboro 2 NE, and Milen 4N climate stations. Precipitation data were obtained from the South Carolina Department of Natural Resources website, Southeast Regional Climate Center (**http://www.dnr.sc.gov/climate/sercc/index.html**). In addition, the Palmer Drought Severity Index (PDSI) and Palmer Hydrological Drought Index (PHDI) are plotted on RAI Figure 2.4.12-2.5 for the same period. The PDSI attempts to measure the duration and intensity of the long-term cumulative meteorological drought and wet conditions. The PDHI is another long-term drought index intended to measure the hydrological impacts of drought (e.g., reservoir levels, groundwater levels, etc.). PDSI and PHDI data were obtained from the National Oceanic and Atmospheric Administration website (**ftp://ftp.ncdc.noaa.gov/pub/data/cirs/**). These indices provide an indication of the severity of a wet or dry spell. The indices generally range from +6 to -6 with negative values denoting dry spells and positive values denoting wet spells. Values of +0.5 to -0.5 indicate normal conditions.

RAI Figure 2.4.12-2.3 shows that between the period 1979 to 1984, groundwater level elevations in the Water Table aquifer were impacted (lowered) by construction dewatering of the power block excavation for VEGP Units 1 and 2 that was in effect from June 1976 to March 1983. The hydrographs suggest that groundwater elevations at distances of about 1,000 ft or more from the excavation were relatively unaffected by dewatering (observation well 804) and that it took about one year for the groundwater to recover after dewatering activities were completed.

Groundwater level data for subsequent years exhibit variability in response to meteorological conditions. The magnitude of the variability can be estimated using data from the wells having the longest period of record, which include wells 802A, 805A, 808, LT-7A, LT-12, and LT-13. Table 2.4.12-2.5 summarizes the minimum and maximum water levels recorded at each of these wells. These results indicate a 5 to 8 ft range in water levels over the 15 year period of record for these wells.

Inspection of the long-term hydrographs for wells 802A, 805A, 808, LT-7A, LT-12, and LT-13 in conjunction with the drought severity indices for the same period indicates that groundwater levels in the Water Table aquifer generally correlate with the PDSI and PDHI. Water levels tend to remain unchanged when the drought severity indices remain near normal (±1). During drought periods when the PDSI or PDHI index falls to -2 or below, groundwater levels tend to decline. Conversely, during wet periods when the PDSI or PDHI increases to +2 or more, groundwater levels tend to rise. Increases or decreases in the drought indices would be associated with the increases or decreases in the rate of recharge of the Water Table aquifer. Because of the relatively large depth to the water table (at least 60 ft), prolonged wet or dry periods on the order of a year in duration are apparently required to affect the recharge to the water table at these depths.

As has been previously noted and as is evident from RAI Figure 2.4.12-2.4, water levels measured in the June 2005 to November 2006 period have remained relatively constant. Water levels measured during this period for wells 802A, 805A, 808, LT-7A, LT-12, and LT-13 fell roughly in the middle of their historical range. The annual precipitation, the PDSI, and the PDHI for 2004 to 2006 period have been relatively stable and near normal values. Due to the absence of any upward or downward trends in these indices, it is therefore expected that groundwater elevations in the Water Table aquifer would be relatively steady over this period.

RAI Table 2.4.12-2.1 Hydraulic Conductivity Values for the Water Table Aquifer (Barnwell sands and Utley Limestone) for the ESP Site

Observation Well No.	**Test Interval** [1] (ft bgs)	**Material** [2]	**Hydraulic Conductivity** [3] (ft/day)
OW-1003	72 - 91	"Reddish brown silty SAND (SM) with "Light tan silty SAND" with "Tan and grey clayey COQUINA"	0.12
OW-1005	143 - 169	"Pale yellow, silty SAND, calcareous (SM), fine-coarse-grained with shell pieces"	0.32
OW-1006	113 - 136	"Very light tan silty SAND (SM)" with "light gray COQUINA, unconsolidated" (OW-1006A) "Tan sandy and shelly CLAY (CH), saturated" with "Light tan, fine-coarse grained SAND with shell (SW)" (OW-1006)	1.40
OW-1007	99 - 120	"Tan fine-grained silty SAND (SM), saturated" with "Very light tan silty SAND (SM) becoming shelly" with "light olive grey CLAY (CH)"	2.65
OW-1009	81 - 98	"Very light tan silty SAND (SM)" with "Tan limestone shell hash, very light tan silty SAND (SM)" WITH "Brown silty CLAY".	1.10
OW-1010	70 - 92	"Tan poorly graded SAND with silt (SP-SM)" with "Brownish yellow clayey silty SAND (SC-SM), soft" with "White SHELL HASH"	0.18
OW-1012	71 - 94	"Brown SAND, fine-to-medium-grained with pale yellow silt (SM)" with "Pale olive silt (ML)" with "Pale yellow SILT, micaceous (ML)".	0.39
OW-1013	81 - 104	"Tan fine-to-medium-grained SAND (SP-SM) with tan or clay tubes or bioturbation" with "Light olive tan calcareous silty fine-grained SAND (SP-SM)" with "light olive tan calcareous CLAY (CL), wet but not saturated"	0.38
OW-1015	90 - 120	"Grayish white, fine-to-medium-grained SAND (SP) saturated" with Very light tan poorly graded SAND with silt (SP-SM)" with "Tan shelly (coarse) fine to medium grained clayey SAND (SC)".	0.44

Notes:
Borings OW-1001A, OW-1002A, OW-1003A, and OW-1005A were abandoned due to the use of 3.25-in hollow stem auger, which would not adequately accommodate well installation.
Boring OW-1006A was abandoned due to the of shortage hollow stem auger flights.
Boring log OW-1003 contained in SSAR Appendix 2.4A (Appendix E report) should read OW-1003A.The drilling method for boring OW-1006 is assumed to be 4.25" HSA (not described in SSAR Appendix 2.4A (Appendix E report)).

RAI Table 2.4.12-2.2 Hydraulic Conductivity Values for the Water Table Aquifer (Barnwell sands and Utley Limestone) for VEGP Units 1 and 2 Site

Well No.	**Test Interval** (ft bgs)	**Hydraulic Conductivity** [1] (ft/yr)	(ft/day)
Utley Limestone			
		Pumping Test Results	
1A	56 - 78	14,100	39
1B	68 - 78	125,400	343
1C	56 - 80	20,000	55
1D	56 - 80	44,100	121
2A	62 - 85	3,250	8.9
		Falling Head Test Results	
W-1	65 - 80	5,800	16
1A	63 - 78	600	1.6
W-2	69 - 85	980	2.7
2A	70 - 85	96	0.26
2B	69 - 84	360	1.0
2C	65 - 85	140	0.38
2D	70 - 85	2,100	5.7
		Constant Head Test Results	
1A	56 - 78	160	0.44
2A	56 - 85	3,200	8.8
2B	56 - 84	1,790	4.9
2D	56 - 85	1,190	3.3
Barnwell sands			
		Constant Head Test Results	
183	50 – 60	200	0.5
184	53 - 63	267	0.7
		Laboratory Test Results	
107A	13.8 – 14.4	302	0.8
	34 – 36	9.8	0.03
	62.5 - 63	27.4	0.08

Note:
[1] Values from UFSAR Tables 2.4.12-12 and -13

RAI Table 2.4.12-2.3 Historical Groundwater Levels for the Water Table Aquifer.

	Observation Well and Water Level Elevation (ft msl)												
	142	179	802A	803A	804	805A	806B	808	809	LT-1A/1B	LT-7/7A	LT-12	LT-13
23-Oct-71		154.3											
2-Nov-71		156.8											
10-Nov-71		160.3											
17-Nov-71		160.8											
23-Nov-71		161.1											
1-Dec-71		162.1											
7-Dec-71		162.4											
14-Dec-71		164.3											
23-Dec-71		164.6											
29-Dec-71		165.8											
5-Jan-72		166.1											
12-Jan-72		167.3											
19-Jan-72		168.1											
26-Jan-72		168.5											
3-Feb-72		168.6											
9-Feb-72		168.9											
23-Feb-72		169.8											
2-Mar-72		170.1											
9-Mar-72		170.3											
16-Mar-72		167.9											
21-Mar-72		170.2											
18-Apr-72		171.9											
1-May-73		174.1											
30-May-73		173.6											
27-Jul-73		172.3											
13-Oct-73		170.8											
3-Nov-73		170.4											
9-Dec-73		170.1											
7-Jan-74		168.9											

RAI Table 2.4.12-2.3 Historical Groundwater Levels for the Water Table Aquifer.

	Observation Well and Water Level Elevation (ft msl)												
	142	179	802A	803A	804	805A	806B	808	809	LT-1A/1B	LT-7/7A	LT-12	LT-13
10-Feb-74		166.6											
23-Mar-74		168.1											
17-Apr-74		167.4											
15-Aug-74		165.3											
11-Sep-74		165.1											
7-Jul-79		160.2		155.5	161.2	152.4							
26-Nov-79		161.8		155.1		153.0							
2-Jan-1980				155.1	161.2	152.9				137.2	141.6		
11-Jan-1980				155.1						136.8	141.7		
24-Jan-1980		161.0		154.9	161.0	138.2				136.8	141.6		
1-Feb-1980				154.9		138.5				136.5	141.1		
15-Feb-1980				155.0						136.6	141.2		
25-Mar-1980		157.9		154.7	161.0					136.2	142.1		
27-Jun-1980		162.0			161.4	137.5				137.0	140.6		
2-Sep-1980										136.4	139.0		
27-Sep-1980		161.7		154.7	161.1	153.3							
1-Dec-1980										135.6	140.2		
29-Dec-1980		161.1		154.4	160.9								
2-Mar-1981										135.8			
28-Mar-1981		159.3		154.0	160.3								
2-Apr-1981											139.7		
1-Jun-1981										135.4			
29-Jun-1981		158.0		153.6									
2-Jul-1981											139.5		
24-Dec-1981											140.2		
7-Feb-1982											139.6		
23-Mar-1982		158.8		152.6	159.1	150.8							
15-Jun-1982		158.8		152.4	159.0	151.0				135.6			
9-Jul-1982											140.7		

RAI Table 2.4.12-2.3 Historical Groundwater Levels for the Water Table Aquifer.

	Observation Well and Water Level Elevation (ft msl)												
	142	**179**	**802A**	**803A**	**804**	**805A**	**806B**	**808**	**809**	**LT-1A/1B**	**LT-7/7A**	**LT-12**	**LT-13**
15-Sep-1982		159.5		152.7	158.7	151.9							
20-Sep-82										137.0			
11-Dec-82	146.1	160.1		152.6	159.0	153.7							
18-Dec-82										135.1			
8-Mar-83	146.3				158.8	153.6							
9-Mar-83		159.6		152.6									
15-Mar-83										140.9			
22-Jun-83	152.3	159.7		155.1	159.0	156.1	152.7			151.4	149.9		
15-Sep-83	153.3	159.7		156.5			154.5						
21-Sep-83					159.7	156.8							
3-Oct-83											154.2		
15-Oct-83										153.8			
12-Dec-83	154.4	160.4		157.7	160.0	157.9	155.4						
14-Dec-83										156.4	155.9		
12-Mar-84	155.1					158.5	156.2						
13-Mar-84		159.9		158.2	160.1								
22-Mar-84										156.1	156.6		
11-Jun-84				158.9	160.5	159.9							
12-Jun-84		155.8					157.1			157.4	157.4		
13-Sep-84				159.8									
16-Sep-84					161.0								
18-Sep-84	156.5	150.9				160.6	157.4				157.7		
13-Dec-84	155.9	151.1		159.9	160.2	160.1	157.1			157.0			
31-Dec-84											158.0		
4-Feb-85	155.7	148.9		159.6	160.9	159.9	157.0			157.1			
30-Jun-85	155.5	150.2		159.6	161.0	159.5	156.9			152.0	152.0		
7-Jul-85	155.3	148.5		159.5	160.8	159.3	156.6	159.2	155.5	157.0		158.5	157.6
16-Jul-85	155.3	150.0		159.4	160.8	159.3	156.7	159.2	152.7	155.2	158.0	160.2	157.5
23-Jul-85	155.2	150.3		159.5	160.8	159.3	156.7	159.3	152.8	155.2	158.1	160.0	157.6

RAI Table 2.4.12-2.3 Historical Groundwater Levels for the Water Table Aquifer.

	Observation Well and Water Level Elevation (ft msl)												
	142	179	802A	803A	804	805A	806B	808	809	LT-1A/1B	LT-7/7A	LT-12	LT-13
31-Jul-85	155.3	150.6		159.5	160.9	159.3	156.8	159.8	152.8	155.2	158.0	160.0	157.5
7-Aug-85	155.4	148.6		159.4	160.9	159.3	157.0	160.0	152.8	155.3	158.1	160.2	157.7
14-Aug-85	155.3	148.6		159.4	160.8	159.2	156.2	160.3	152.7	155.3	158.0	160.2	157.7
21-Aug-85	155.4	148.6		159.4	160.8	159.3	157.1	160.4	152.8	157.2	158.1	160.4	157.8
28-Aug-85	155.6	148.8		159.5	160.9	159.4	157.2	160.5	152.5	157.3	158.2	160.5	157.7
4-Sep-85	155.5	148.8	159.0	159.6	161.0	159.6	157.2	160.4	152.8	157.4	158.3	160.8	157.8
11-Sep-85	155.5	148.8	159.0	159.5	161.0	159.6	157.2	160.6	152.9	157.4	158.4	161.1	158.1
18-Sep-85	155.4	148.8	159.0	159.5	160.8	159.5	157.2	160.5	152.8	157.3	158.4	161.1	158.0
25-Sep-85	155.6	148.8	159.0	159.3	160.9	159.6	157.3	160.4	152.9	157.5	158.5	161.4	158.1
6-Oct-85	155.6	148.8	159.1	159.6	160.9	159.7	157.3	160.3	152.9	157.5	158.5	161.5	158.1
9-Oct-85	155.5	148.8	159.0	159.6	160.9	159.6	157.2	160.2	152.9	157.3	158.3	161.3	158.0
16-Oct-85	155.5	148.8	159.2	159.7	160.8	159.6	157.4	160.3	152.9	157.6	158.7	161.5	158.2
23-Oct-85	155.5	148.8	159.1	159.7	160.7	159.7	157.3	160.2	152.9	157.5	158.8	161.5	158.3
30-Oct-85	155.7	148.8	159.2	159.8	161.1	159.9	157.5	160.2	153.0	157.7	159.0	162.0	158.5
6-Nov-85	155.5	148.7		159.5	160.8	159.7	157.2	160.1	152.9	157.4	158.5	161.6	158.4
13-Nov-85	155.5	148.8		159.5	161.0	159.8	157.2	160.1	152.9	157.3	158.5	161.5	158.0
20-Nov-85	155.6	148.9	159.2	159.8	161.0	159.7	157.3	160.2	153.1	157.4	158.5	161.5	158.1
27-Nov-85	155.6	148.8	159.1	159.6	160.6	159.8	157.4	160.1	153.0	157.6	158.7	161.6	158.1
4-Dec-85	155.7	148.8	159.1	159.7	160.8	159.6	157.4	160.1	153.0	157.5	158.5	161.3	158.4
11-Dec-85	155.8	148.8	159.2	159.9	161.1	159.9	157.6	160.3	153.0	157.8	158.8	161.6	158.3
18-Dec-85	155.8	148.8	159.2	159.7	160.9	159.9	157.6	160.4	153.0	157.7	158.9	161.5	158.3
28-Dec-85	155.9	148.8	159.3	159.8		159.9	157.7	160.5	153.0	157.8	158.6	161.6	158.6
2-Jan-86	156.0	148.9	159.4	159.8	161.0	159.8	157.7	160.5	153.1	157.8	158.6	161.6	158.4
10-Jan-86	156.1	148.9	159.6	160.0	161.4	159.7	157.9	160.5	153.3	158.2	158.8	161.8	158.3
15-Jan-86	155.7	148.7	159.4	159.8	160.7	159.8	157.7	160.6	152.9	157.9	158.8	161.9	158.3
22-Jan-86	156.0	148.8	159.4	159.8	161.0	160.0	157.2	160.5	153.1	157.8			158.7
29-Jan-86	156.0	148.8	159.5	160.0	161.2	160.2	157.7	160.5	153.1	157.9	159.2	161.8	158.8
5-Feb-86	156.0	148.7	159.5	159.9	161.1	160.1	157.6	160.6	153.0	157.9	159.2	162.0	158.6
12-Feb-86	155.9	148.8	159.4	159.9	160.9	160.0	157.6	160.5	153.0	157.7	158.8	161.5	158.8

RAI Table 2.4.12-2.3 Historical Groundwater Levels for the Water Table Aquifer.

	Observation Well and Water Level Elevation (ft msl)												
	142	179	802A	803A	804	805A	806B	808	809	LT-1A/1B	LT-7/7A	LT-12	LT-13
19-Feb-86	156.0	148.8	159.6	160.0	161.2	160.2	157.7	160.4	153.1	157.9	159.1	162.0	158.8
26-Feb-86	156.0	148.9	159.8	160.3	161.2	160.5	157.9	160.3	153.1	158.2	159.6	162.4	158.7
5-Mar-86	155.8	148.7	159.4	159.9	161.0	160.1	157.5	160.3	153.0	157.7	158.9	161.7	158.7
15-Mar-86	156.1	148.8	159.7	160.2	161.5	160.1	157.8	160.3	153.3	157.7	159.0	161.8	158.6
19-Mar-86	155.8	148.8	159.4	160.0	161.1	160.1	157.5	160.2	153.1	157.6	158.9	161.5	158.4
26-Mar-86	155.8	148.8	159.4	160.1	161.4	160.3	157.5	160.1	153.0	157.7	158.9	161.6	158.5
2-Apr-86	155.9	148.7	159.6	160.3	161.4	160.4	157.6	160.1	153.2	157.8	159.0	161.7	158.5
9-Apr-86	155.9	148.8	159.6	160.1	161.3	160.2	157.6	160.2	153.1	157.9	159.2	161.9	158.7
16-Apr-86	155.7	148.7	159.8	160.3	161.1	160.3	157.4	160.1	153.1	157.5	158.7	161.4	158.1
23-Apr-86	155.9	148.8	159.5	160.2	161.4	160.0	157.5	160.2	153.2	157.7	158.9	161.5	158.7
30-Apr-86	155.8	148.8	159.4	160.1	161.4	160.2	157.4	160.1	153.1	157.7	158.8	161.5	158.5
7-May-86	155.7	148.7	159.4	160.1	161.2	160.2	157.5	160.0	153.0	157.4	158.3	161.2	158.3
14-May-86	155.7	148.8	159.3	160.1	161.3	160.1	157.3	160.0	153.1	157.6	158.8	161.3	158.9
21-May-86	155.8	148.8	159.4	160.1	161.3	160.2	157.4	159.9	153.1	157.6	158.8	161.5	158.4
28-May-86	155.7	148.8	159.4	160.1	161.4	160.2	157.3	159.9	153.1	157.5	158.8	161.5	158.4
4-Jun-86	155.7	148.7	159.3	160.0	161.2	160.0	157.2	159.9	153.1	157.3	158.4	161.0	158.3
11-Jun-86	155.7	148.8	159.4	159.9	161.3	160.0	157.2	159.8	153.0	157.4	158.6	161.4	158.2
18-Jun-86	155.9	148.8	159.3	160.0	161.1	160.0	157.3	159.8	153.1	157.5	158.7	161.1	158.2
25-Jun-86	155.8	148.8	159.4	160.0	160.9	159.6	157.3	159.7	153.1	157.5	158.6	161.2	158.2
2-Jul-86	155.8	148.8	159.3	160.0	161.4	160.0	157.3	159.7	153.1	157.5	158.6	161.1	158.2
9-Jul-86	155.7	148.7	159.2	160.0	161.4	160.0	157.2	159.7	153.0	157.4	158.5	161.0	158.1
16-Jul-86	155.7	148.7	159.2	159.9	160.9	159.9	157.2	159.7	153.0	157.3	158.4	160.9	158.2
23-Jul-86	155.6	148.7	159.0	159.9	161.2	159.9	157.1	159.6	153.0	157.2	158.3	160.7	158.2
30-Jul-86	155.7	148.7	159.0	159.9	161.2	159.9	157.2	159.6	153.0	157.2	158.3	160.9	158.2
6-Aug-86	155.7	148.8	159.3	160.0	161.3	160.0	157.2	159.6	153.1	157.3	158.3	160.8	157.9
13-Aug-86	155.6	148.8	159.0	159.9	161.2	159.9	157.1	159.5	153.0	157.3	158.4	160.8	158.0
20-Aug-86	155.6	148.8	159.1	159.9	161.1	159.9	157.1	159.5	153.0	157.2	158.2	160.6	158.1
27-Aug-86	155.6	148.8	159.1	159.9	161.2	159.8	157.0	159.4	153.0	157.2	158.3	160.7	157.9
3-Sep-86	155.6	148.8	159.1	159.9	161.2	159.9	157.1	159.6	153.0	157.3	158.3	160.7	158.0

RAI Table 2.4.12-2.3 Historical Groundwater Levels for the Water Table Aquifer.

	Observation Well and Water Level Elevation (ft msl)												
	142	**179**	**802A**	**803A**	**804**	**805A**	**806B**	**808**	**809**	**LT-1A/1B**	**LT-7/7A**	**LT-12**	**LT-13**
10-Sep-86	155.6	148.7	159.1	159.9	161.2	159.8	157.1	159.6	152.9	157.3	158.3	160.7	157.9
17-Sep-86	155.5	148.7	159.0	159.9	161.0	159.8	157.0	159.7	152.9	157.4	158.5	160.5	157.8
24-Sep-86	155.5	148.7	159.0	159.8	161.0	159.8	157.0	159.9	152.9	157.6	158.2	160.5	158.0
1-Oct-86	155.7	148.8	158.9	159.9	161.0	159.9	157.0	159.9	153.0	157.6	158.3	160.7	157.8
11-Oct-86	155.6	148.8	159.0	160.0	161.1	159.9	157.0	159.8	152.9	157.1	158.1	160.5	157.9
15-Oct-86	155.5	148.8	159.1	159.9	161.1	159.9	157.1	159.9	152.9	157.0	158.2	160.5	158.0
22-Oct-86	155.6	148.8	159.1	159.9	161.2	159.9	157.1	159.8	153.0	157.0	158.2	160.5	157.7
29-Oct-86	155.5	148.8	159.0	159.8	160.9	159.8	157.1	159.9	152.9	156.9	158.2	160.6	157.9
5-Nov-86	155.6	148.8	159.1	159.6	161.2	159.9	157.2	159.8	153.0	157.2	158.2	160.7	158.0
12-Nov-86	155.6	148.8	159.1	159.6	161.1	159.8	157.2	159.7	152.96	157.2	158.3	160.6	157.9
19-Nov-86	155.5	148.8	159.2	159.7	160.9	160.0	157.3	159.8	152.8	157.5	158.6	160.9	158.0
26-Nov-86	155.6	148.8	159.2	159.6	160.9	159.9	157.2	159.6	152.9	157.3	158.3	160.7	158.2
3-Dec-86	155.6	148.8	159.0	159.7	160.9	160.0	157.2	159.6	152.8	157.1	158.0	160.5	157.9
31-Dec-86	155.9	148.1	159.0	159.8	160.9	159.8	157.5	159.4	153.0	157.6	158.6	160.8	158.1
10-Jan-87	156.0	148.9	159.1	160.1	160.9	160.1	157.8	159.3	153.1	158.0	158.9	161.2	158.1
14-Jan-87	156.0	148.8	159.2	160.1	160.8	160.0	157.6	159.1	153.1	158.1	159.1	161.3	158.3
21-Jan-87	155.9	148.7	159.3	160.1	160.8	159.9	157.5	159.2	152.8	159.7	159.1	161.4	158.4
28-Jan-87	156.2	148.8	159.4	160.1	161.2	159.9	157.9	159.5	153.0	158.1	158.9	161.1	158.3
Jan-88	156.7	148.8	160.5	161.8	161.9	161.4	158.2	159.7	153.4	158.2	159.0	160.9	158.6
Feb-88	156.7	148.9	160.7	163.0	162.1	161.6	158.4	159.7	153.3	158.3	159.2	161.1	159.0
Mar-88	156.6	148.8	160.4	161.8	162.1	161.5	158.2	159.3	153.3	158.3	159.2	161.1	158.7
Apr-88	156.7	148.8	160.4	161.6	162.2	161.4	158.1	159.3	153.3	158.3	159.3	161.2	158.9
May-88	156.3	148.7	159.9	161.3	161.7	161.0	157.8	159.0	153.5	157.9	158.8	160.6	158.3
Jun-88	156.2	148.8	159.9	161.1	161.7	161.2	157.8	159.8	153.5	157.9	158.8	160.5	158.3
16-Dec-94			158.8			160.0	156.0	159.4		156.8	155.8	158.3	156.6
13-Jun-95						161.0	156.6						
29-Jun-95			159.6					160.4		157.3	156.3	158.9	157.2
22-Sep-95										157.7	156.7	159.2	157.6
20-Dec-95			160.1							157.8	157.0	159.8	157.8

RAI Table 2.4.12-2.3 Historical Groundwater Levels for the Water Table Aquifer.

	Observation Well and Water Level Elevation (ft msl)												
	142	179	802A	803A	804	805A	806B	808	809	LT-1A/1B	LT-7/7A	LT-12	LT-13
21-Dec-95						161.6	157.0	160.2					
21-Mar-96										157.6	156.7	159.7	157.6
13-Jun-96			160.1					159.7		157.4	156.5	159.9	157.5
15-Sep-96										156.8	156.4	159.0	156.6
11-Dec-96						160.8	156.5	159.4					
30-Dec-96			159.5							157.3	156.4	159.1	157.3
13-Mar-97										157.1	157.7	159.7	157.7
19-Jun-97			159.0			160.7	156.5	159.2		156.8	156.0	158.6	156.8
31-Dec-97			158.9			160.7	156.6	159.0		156.7	155.8	158.4	156.7
24-Mar-98										157.6	156.5	159.2	157.6
23-Jun-98			158.8			160.8	156.7	159.2		157.1	156.1	159.0	157.1
28-Sep-98										157.3	156.5	159.1	157.4
23-Mar-99										158.8	157.8	160.0	158.8
8-Jun-99										158.5		160.6	
15-Jun-99							157.6						158.6
17-Jun-99			160.8			162.5		159.0			157.7		
17-Dec-99			159.7			160.9	156.9	158.6		156.9	156.1	159.6	157.6
22-Mar-00										158.5	157.3	159.0	158.1
2-Jun-00						159.7	156.0				157.0	158.3	156.5
5-Jun-00			158.6					158.3		156.8			
8-Sep-00										155.5	156.4	157.7	156.0
7-Dec-00			157.8			158.8	155.3	158.4		155.5	156.4	157.8	155.9
5-Mar-01										155.9	154.4	157.0	155.2
8-Jun-01			157.4			158.5	155.1			155.1	156.0	157.2	155.6
12-Jun-01								155.0					
14-Mar-02										155.3	156.1	157.7	155.7
5-Jun-02			157.0							154.7	155.5	156.9	155.3
7-Jun-02						157.7	154.6	158.0					
18-Sep-02										154.2	155.2	156.4	154.7

RAI Table 2.4.12-2.3 Historical Groundwater Levels for the Water Table Aquifer.

	Observation Well and Water Level Elevation (ft msl)												
	142	**179**	**802A**	**803A**	**804**	**805A**	**806B**	**808**	**809**	**LT-1A/1B**	**LT-7/7A**	**LT-12**	**LT-13**
5-Dec-02			156.1			156.9	154.0	157.6		154.2	153.4	156.2	154.7
10-Mar-03										153.9	154.7	155.8	154.3
18-Jun-03			156.9			159.0	154.8	160.0		154.7	154.0	156.3	155.1
4-Sep-03										155.6	154.5	157.3	155.9
9-Dec-03			158.7			160.0	156.2	160.6		156.2	155.0	158.0	156.7
3-Mar-04										156.3	155.2	158.4	156.9
3-Sep-04										156.1	157.0	158.4	156.7
17-Dec-04			158.5			159.5	155.9	158.6		156.0	155.2	158.2	156.6

Notes.

1) Well data for 23 Oct 1971 to 4 Feb 1985 contained in the Ground Water Supplement for VEGP Units 1 and 2 (Georgia Power 1985).

2) Well data for 30 Jun 1986 to 29 Dec 1985 contained in Observation Well Readings for VEGP Units 1 and 2, July – December 1985 (Georgia Power, 1986).

3) Well data for 2 Jan 1986 to 25 Jun 1986 contained in Observation Well Readings for VEGP Units 1 and 2, January - June 1986 (Georgia Power, 1986).

4) Well data for 2 Jul 1986 to 3 Dec 1986 contained in Observation Well Readings, for VEGP Units 1 and 2, July - December 1986, (Georgia Power, 1987).

5) Well data for 31 Dec 1986 to 28 Jan 1987 contained in Piezometer Weekly Readings Report for VEGP Units 1 and 2 (Georgia Power, 1987).

6) Well data for Jan 1988 to June 1988 contained in the Ground Water Supplement, July 1987 – June 1988 (Bechtel Civil Inc. 1988)

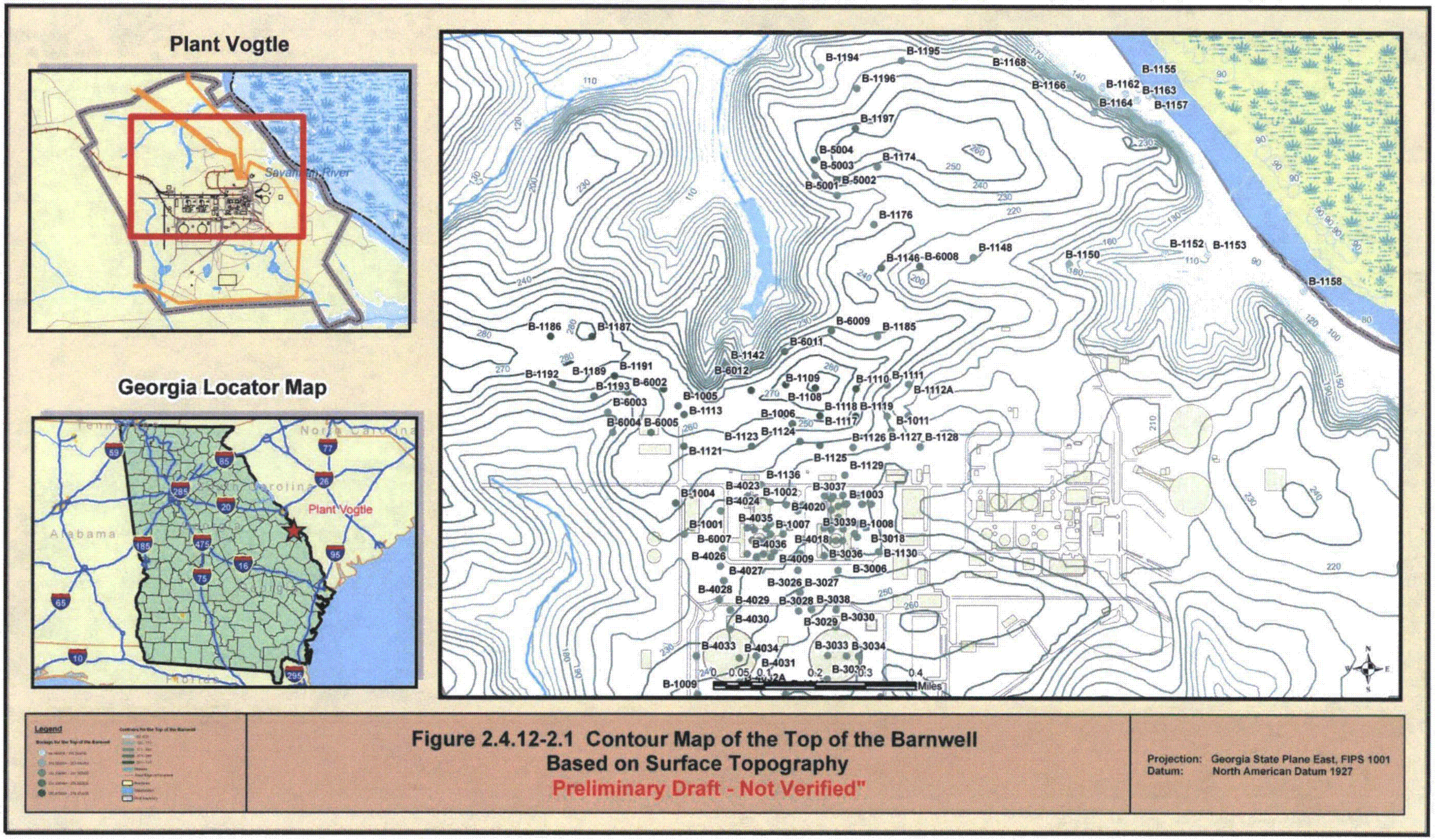
7) Well data for 16 Dec 1994 to 17 Dec 2004 contained in Bechtel Request for Information, RFI Number 25144-000-GRI-GEX-00028, SNC ALWR ESP Project (Bechtel Power Corporation, 2006)

RAI Table 2.4.12-2.4 Monthly Groundwater Level Elevations in the Water Table Aquifer

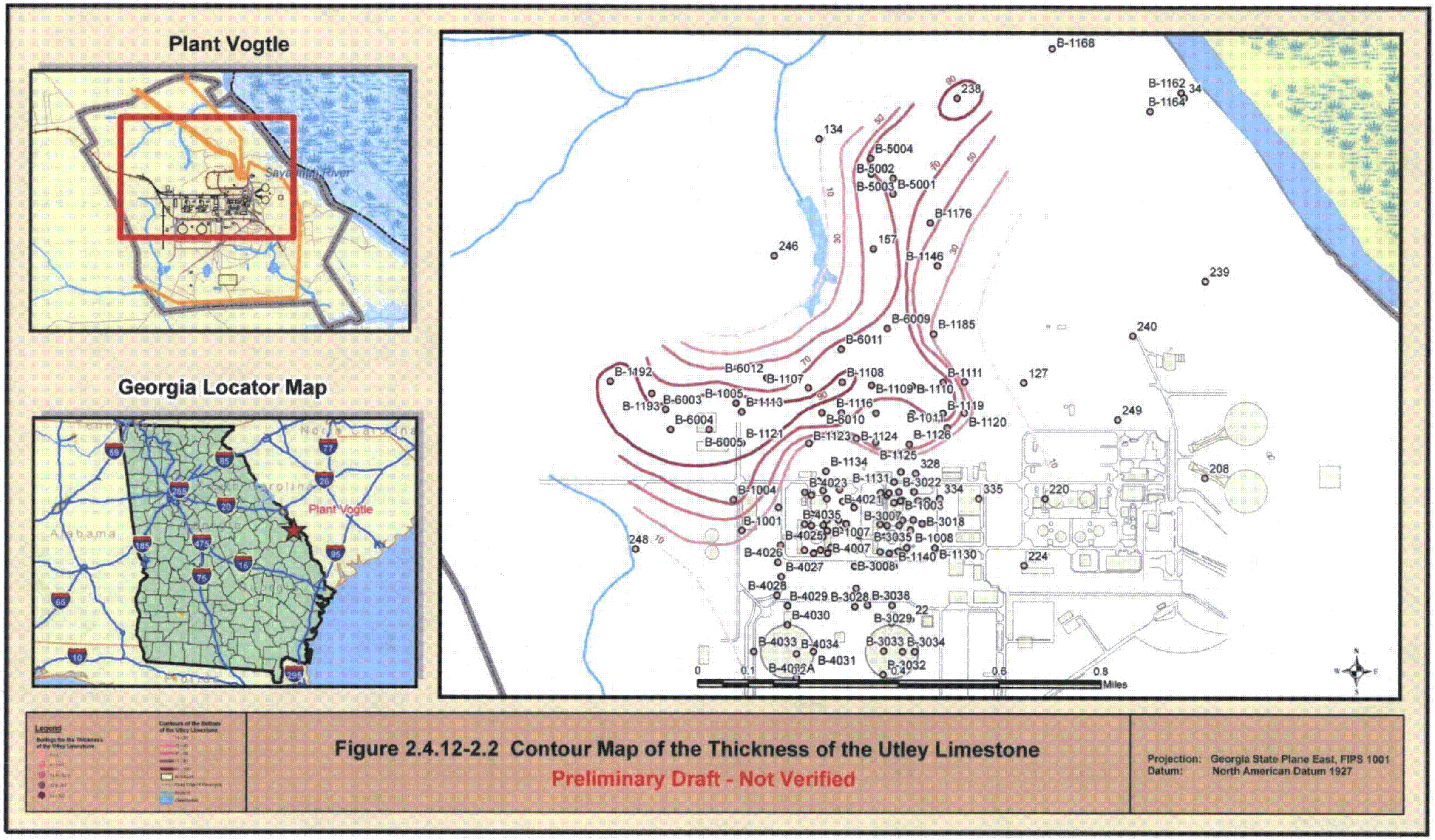
Well No.	Groundwater Level Elevation (ft msl)																	
	Jun-05	Jul-05	Aug-05	Sep-05	Oct-05	Nov-05	Dec-05	Jan-06	Feb-06	Mar-06	Apr-06	May-06	Jun-06	Jul-06	Aug-06	Sep-06	Oct-06	Nov-06
142	154.37	154.38	154.49	154.64	154.75	154.69	154.60	154.71	154.78	154.71	154.63	154.55	154.48	154.41	154.36	0.00	0.00	154.16
179	147.42	148.40	148.42	148.72	148.69	148.75	148.52	148.61	148.64	148.72	148.66	148.76	148.78	148.56	148.75	0.00	0.00	148.79
802A	157.88	157.86	158.07	158.23	158.29	158.34	158.28	158.28	158.39	158.23	158.17	158.09	157.99	157.91	157.89	0.00	0.00	157.56
803A	159.98	159.91	160.15	160.32	160.39	160.48	160.39	160.37	160.48	160.45	160.30	160.20	160.12	159.96	159.88	0.00	0.00	159.64
804	163.73	163.62	163.92	164.10	164.21	164.23	164.05	164.08	164.23	164.30	164.11	163.99	163.88	163.69	163.69	0.00	0.00	162.84
805A	158.53	158.57	158.84	158.98	159.09	159.09	159.05	158.94	158.92	158.98	158.82	158.82	158.63	158.53	158.45	0.00	0.00	158.19
806B	155.62	155.65	155.78	155.90	155.96	155.98	155.88	155.97	155.98	156.03	155.85	155.78	155.73	155.68	155.62	0.00	0.00	155.42
808	158.88	159.14	159.42	159.55	159.49	159.37	159.15	159.04	159.19	159.15	158.99	158.53	158.80	158.72	158.65	0.00	0.00	158.40
809	152.78	152.70	152.75	152.89	152.98	152.97	152.98	153.10	153.22	153.18	153.05	153.02	153.00	152.88	152.86	0.00	0.00	152.71
LT-1B	154.92	154.82	155.01	155.16	155.18	155.22	155.06	155.18	155.52	155.28	155.18	155.15	154.95	154.95	154.95	0.00	0.00	154.78
LT-7A	154.39	154.15	154.33	154.46	154.48	154.46	154.31	154.57	154.83	154.59	154.57	154.50	154.41	154.30	154.34	0.00	0.00	154.25
LT-12	158.21	157.90	158.07	158.22	158.31	158.28	158.21	158.53	158.66	158.48	158.54	158.48	158.23	158.19	158.18	0.00	0.00	158.11
LT-13	156.10	155.92	156.13	156.30	156.32	156.37	156.23	156.36	156.66	156.35	156.32	156.32	156.23	156.08	156.14	0.00	0.00	155.93
OW-1003	155.94	155.89	156.06	156.29	156.24	156.36	156.26	156.34	156.37	156.43	156.32	157.24	156.16	156.03	155.98	0.00	0.00	155.90
OW-1005	132.95	132.73	132.88	133.01	132.67	132.65	132.53	132.74	133.04	133.12	133.14	133.20	133.12	132.94	132.84	0.00	0.00	132.50
OW-1006	147.66	147.48	147.57	147.60	147.49	147.20	147.18	147.41	147.40	147.37	147.35	147.12	147.05	146.88	146.80	0.00	0.00	146.47
OW-1007	151.82	151.72	151.78	151.63	151.45	151.15	151.05	151.41	151.49	151.45	151.22	151.11	150.99	150.76	150.53	0.00	0.00	150.08
OW-1009	162.38	162.40	162.71	162.90	163.01	163.03	162.87	162.93	163.01	163.01	162.89	162.79	162.65	162.50	162.44	0.00	0.00	162.17
OW-1010	163.06	163.26	163.59	163.77	163.81	163.78	163.62	163.60	163.63	163.57	163.44	163.29	163.09	162.91	162.84	0.00	0.00	162.51
OW-1012	161.83	161.93	162.07	162.06	161.98	161.80	161.71	161.82	161.86	161.80	161.68	161.53	161.37	161.22	161.00	0.00	0.00	160.49
OW-1013	164.95	165.00	165.29	165.47	165.48	165.42	165.21	165.29	165.46	165.31	165.23	165.11	164.96	164.79	164.68	0.00	0.00	164.25
OW-1015	159.63	159.58	159.78	159.90	159.96	159.96	159.82	159.81	159.79	159.89	159.75	159.66	159.58	159.45	159.35	0.00	0.00	159.06

RAI Table 2.4.12-2.5 Minimum and Maximum Water Levels Recorded at Observation Wells 802A, 805A, 808, LT7A, LT12, and LT13

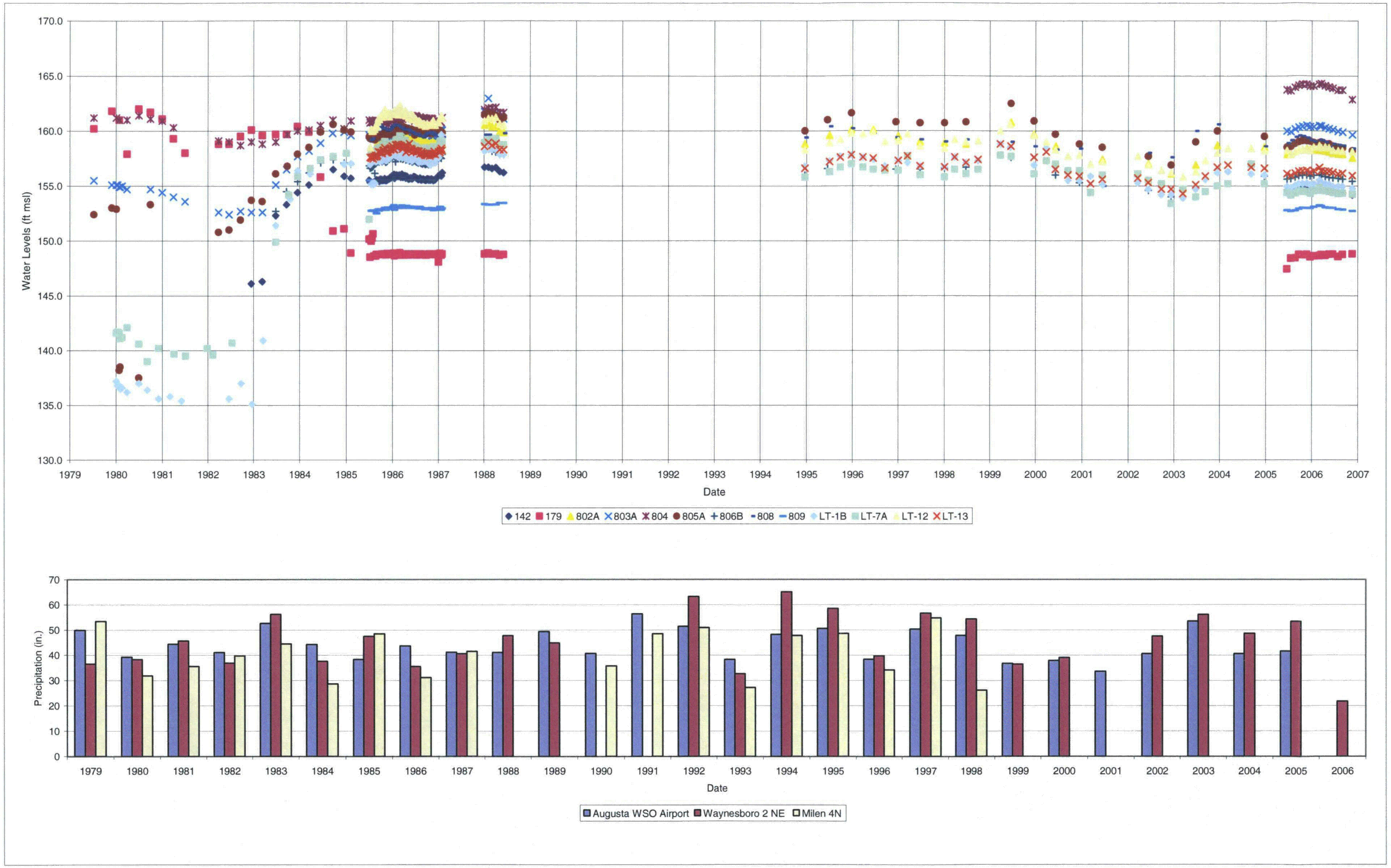
Observation Well	Minimum Water Level Elevation (ft msl)	Date	Maximum Water Level Elevation (ft msl)	Date
802A	156.1	5-Dec-02	160.8	13-Nov-85
805A	156.9	5-Dec-02	162.5	17-Jun-99
808	155.0	12-Jun-01	160.6	9-Dec-03
LT7A	152.0	30-Jun-85	159.6	25-Feb-86
LT12	155.8	10-Mar-03	162.4	26-Feb-86
LT13	154.3	10-Mar-03	159.0	1-Feb-88



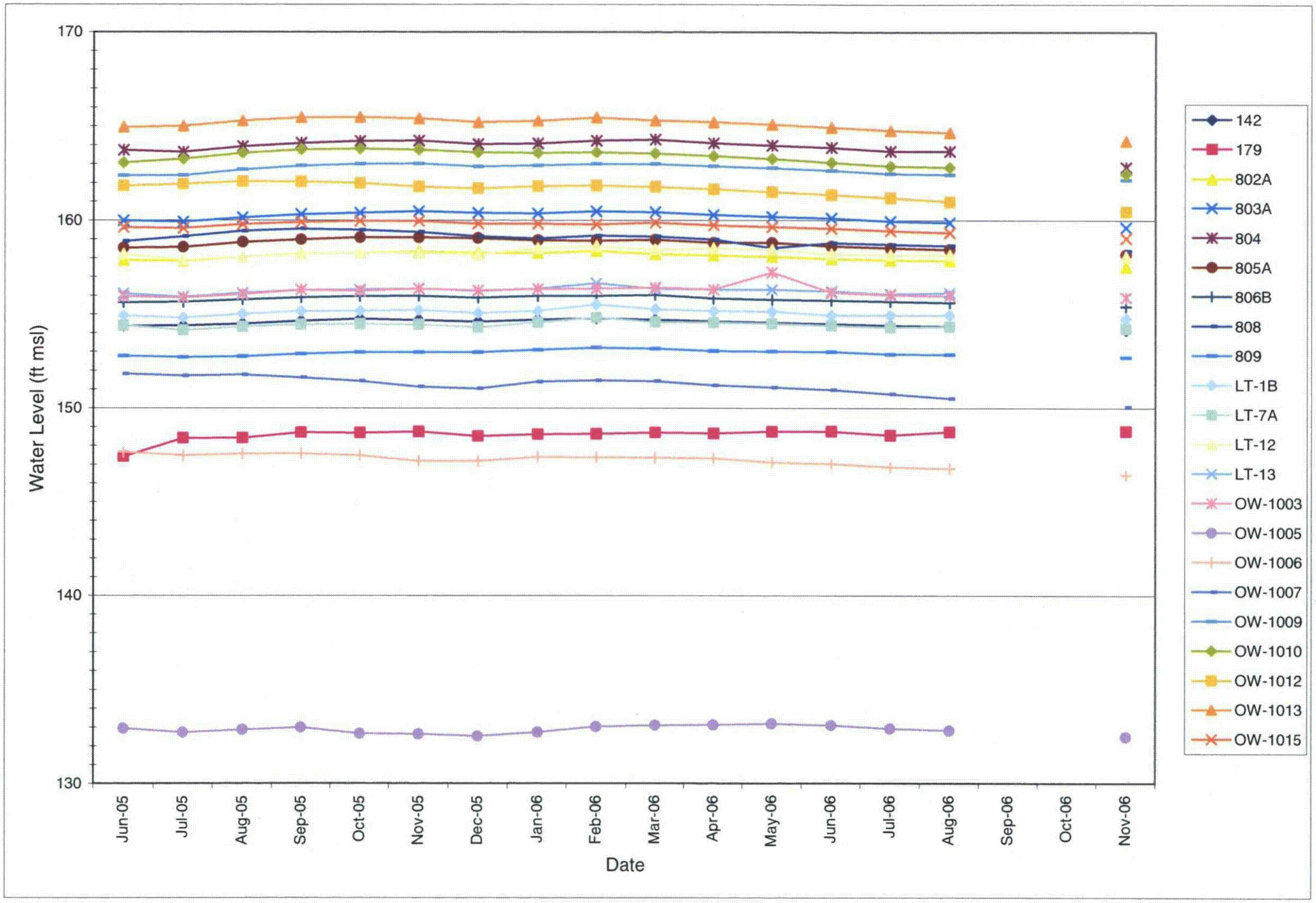
RAI Figure 2.4.12-2.1



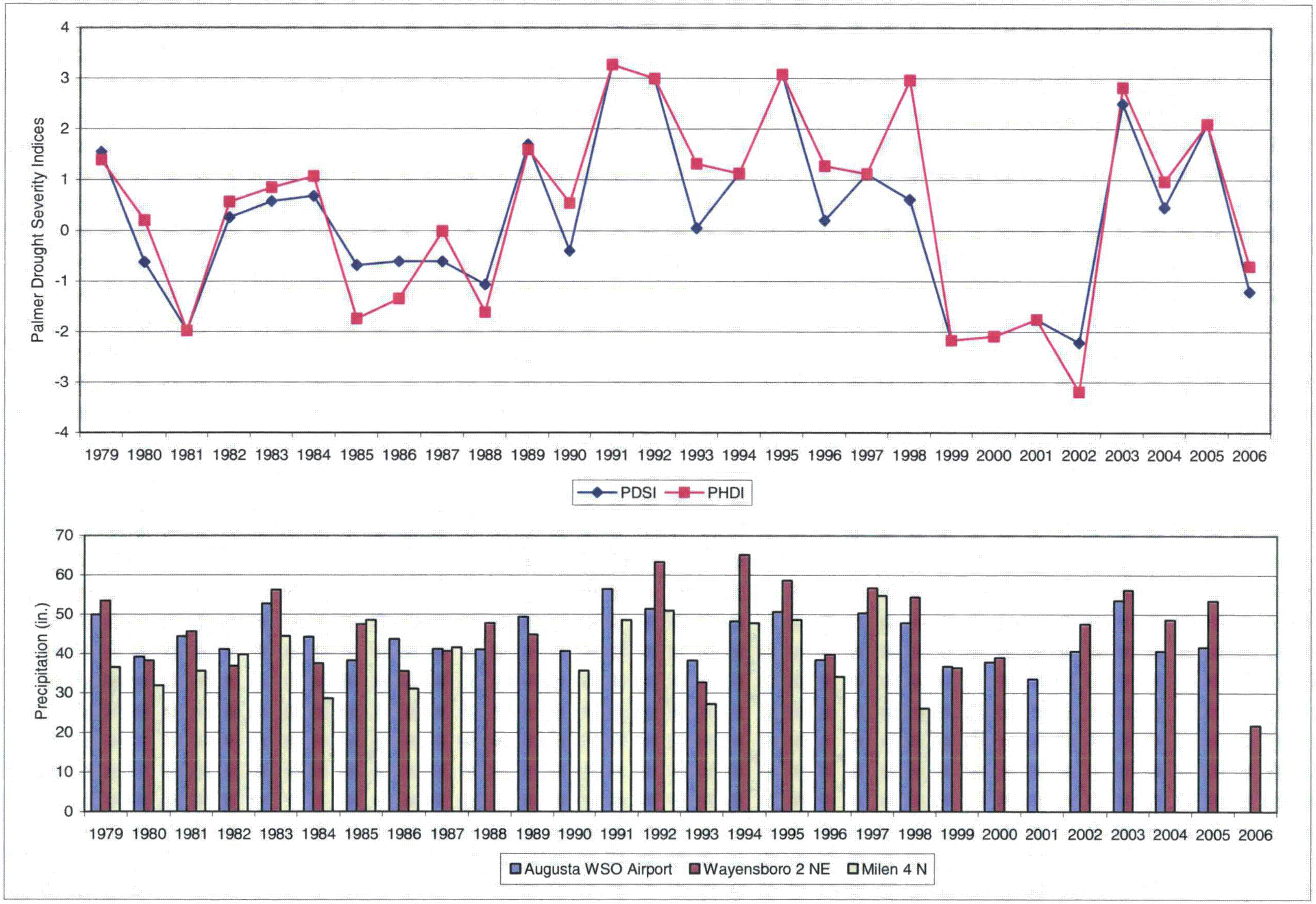
RAI Figure 2.4.12-2.2



RAI Figure 2.4.12-2.3 Water Table Aquifer Hydrographs and Total Annual Precipitation for the Period 1979 - 2006



RAI Figure 2.4.12-2.4 Water Table Aquifer Hydrographs for the Period 2005 - 2006



RAI Figure 2.4.12-2.5 Average Annual PDSI and PHDI for Georgia and Total Annual Precipitation for the Period 1979 – 2006

2.4.12-3a. Include in the SSAR all available information and data (including the historical groundwater levels at the Water Table aquifer near the ESP site, such as those at the 179, 809, 803A, and 804 observation wells), and update the contour maps depicting (i) the thickness of the Utley Limestone layer, and (ii) the top and (iii) the bottom elevations of the Blue Bluff Marl of the Lisbon Formation in the ESP site area. Please depict as much of the area as possible in the vicinity of the ESP site in the contour map(s), (i.e., include the area from the southern drainages to Telfair Pond, the northern drainage to Mallard Pond, and the eastern drainage to Savannah River).

Response.

RAI Table 2.4.12-3.1, following this response, summarizes all available historical groundwater level elevations for the Water Table aquifer for the period from 1971 to 2004 for the following observation wells: 142, 179, 802A, 803A, 804, 805A, 806B, 808, 809, LT-1B, LT-7A, LT-12 and LT-13. Historical groundwater level elevations from 1995 to 2004 for the Water Table aquifer are not available for the following observation wells: 142, 179, 803A, 804 and 809. RAI Figure 2.4.12-3.1, shows the hydrographs for each well.

RAI Table 2.4.12-3.1 and accompanying text will be included in the next revision of the ESP application. SSAR Figures 2.4.12-4 and -5 and ER Figures 2.3.1-13 and -14 will be combined and replaced with RAI Figure 2.4.12-3.1 in the next revision of the ESP application.

Contours showing the thickness of the Utley limestone beneath the VEGP site are shown on following RAI Figure 2.4.12-3.2. Contours of the top and bottom of the Blue Bluff Marl beneath the VEGP site are shown on following RAI Figures 2.4.12-3.3 and 2.4.12-3.4. RAI Figures 2.4.12-3.2, 2.4.12-3.3 and 2.4.12-3.4 were developed primarily from boring information obtained from the subsurface investigation program for the ESP application and preliminary boring information obtained from the current subsurface investigation for the COL application. The boring logs prepared for the ESP application are contained in SSAR Appendix 2.5A. The boring logs for the COL application are currently in preliminary form.

2.4.12-3 (b) In conjunction with the above information, data, and plots, include a discussion of (i) the continuity of the Utley Limestone, (ii) the composition and integrity of the Utley Limestone relative to Huddlestun and Summerour report (1996), and (iii) the presence or absence of Karst characteristics. Please incorporate the discussion into the subsurface conceptual model.

Response.

Contours showing the thickness of the Utley limestone are shown on following RAI Figure 2.4.12-3.2. The following RAI Table 2.4.12-3.2, shows the borings that encountered the Utley limestone and the borings that did not.

RAI Table 2.4.12-3.2 shows the Utley limestone is discontinuous across the VEGP site. Boring logs indicate that it was not encountered in about 19% of the borings. RAI Figure 2.4.12-3.2 shows that it underlies predominantly the northern part of the site between the Units 3 and 4 power block area and Mallard Pond. North of the power block area and east of Mallard Pond it ranges in thickness from about 50 to 100 feet. It becomes thinner to the south and east to a thickness typically less than 30 feet, where present.

The Utley limestone is described in SSAR Section 2.5.1.2.3.2 as a calcareous sand and biomoldic limestone with some silty and clayey sands and varying amounts of carbonate material and silicified zones. This description is generally consistent with the description presented in Huddlestun and

Summerour (1996) where it is described as a moldic, fossiliferous, variably glauconitic, variably sandy limestone with some beds consisting of calcareous sandstone or calcareous sand and minor amounts of clay minerals, some calcarenite beds, scattered shell fragments and other calcitic fossil debris and rare foraminifera.

Huddlestun and Summerour (1996) indicate that the Utley limestone may be absent locally in Burke County due to limestone dissolution. When drilling, the most revealing evidence for the occurrence of solution cavities or karst features is a sudden or rapid drop in the drill rod. At no time during drilling of the borings for the ESP or COL applications was any abnormal drill rod drop observed. However, a number of the borings reported loss of drilling fluid during drilling, indicative of the presence of voids, fractures or highly permeable zones in the Utley limestone. The borings that lost water during drilling are identified in RAI Table 2.4.12-3.2 and shown on RAI Figure 2.4.12-3.5. RAI Figure 2.4.12-3.5 was developed from boring information obtained from the subsurface investigation program for the ESP application and preliminary boring information obtained from the current subsurface investigation for the COL application. RAI Table 2.4.12-3.2 shows that approximately 53% of the borings lost water to the Utley limestone during drilling. RAI Figure 2.4.12-3.5 shows that these borings are located primarily in the power block area, immediately north of the power block area and northeast of Mallards Pond.

References.

Huddlestun, P.F. and Summerour, J.H., 1996, The Lithostratigraphic Framework of the Uppermost Cretaceous and Lower Tertiary of Eastern Burke County, Georgia, Georgia Department of Natural Resources Environmental Protection Division , Georgia Geologic Survey , Bulletin 127.

RAI Table 2.4.12-3.1 Historical Groundwater Levels for the Water Table Aquifer.

	Observation Well and Water Level Elevation (ft msl)												
Date	142	179	802A	803A	804	805A	806B	808	809	LT-1A/1B	LT-7/7A	LT-12	LT-13
23-Oct-71		154.3											
2-Nov-71		156.8											
10-Nov-71		160.3											
17-Nov-71		160.8											
23-Nov-71		161.1											
1-Dec-71		162.1											
7-Dec-71		162.4											
14-Dec-71		164.3											
23-Dec-71		164.6											
29-Dec-71		165.8											
5-Jan-72		166.1											
12-Jan-72		167.3											
19-Jan-72		168.1											
26-Jan-72		168.5											
3-Feb-72		168.6											
9-Feb-72		168.9											
23-Feb-72		169.8											
2-Mar-72		170.1											
9-Mar-72		170.3											
16-Mar-72		167.9											
21-Mar-72		170.2											
18-Apr-72		171.9											
1-May-73		174.1											
30-May-73		173.6											
27-Jul-73		172.3											

RAI Table 2.4.12-3.1 Historical Groundwater Levels for the Water Table Aquifer.

	Observation Well and Water Level Elevation (ft msl)												
Date	**142**	**179**	**802A**	**803A**	**804**	**805A**	**806B**	**808**	**809**	**LT-1A/1B**	**LT-7/7A**	**LT-12**	**LT-13**
13-Oct-73		170.8											
3-Nov-73		170.4											
9-Dec-73		170.1											
7-Jan-74		168.9											
10-Feb-74		166.6											
23-Mar-74		168.1											
17-Apr-74		167.4											
15-Aug-74		165.3											
11-Sep-74		165.1											
7-Jul-79		160.2		155.5	161.2	152.4							
26-Nov-79		161.8		155.1		153.0							
2-Jan-1980				155.1	161.2	152.9				137.2	141.6		
11-Jan-1980				155.1						136.8	141.7		
24-Jan-1980		161.0		154.9	161.0	138.2				136.8	141.6		
1-Feb-1980				154.9		138.5				136.5	141.1		
15-Feb-1980				155.0						136.6	141.2		
25-Mar-1980		157.9		154.7	161.0					136.2	142.1		
27-Jun-1980		162.0			161.4	137.5				137.0	140.6		
2-Sep-1980										136.4	139.0		
27-Sep-1980		161.7		154.7	161.1	153.3							
1-Dec-1980										135.6	140.2		
29-Dec-1980		161.1		154.4	160.9								
2-Mar-1981										135.8			
28-Mar-1981		159.3		154.0	160.3								
2-Apr-1981											139.7		

RAI Table 2.4.12-3.1 Historical Groundwater Levels for the Water Table Aquifer.

	Observation Well and Water Level Elevation (ft msl)												
Date	142	179	802A	803A	804	805A	806B	808	809	LT-1A/1B	LT-7/7A	LT-12	LT-13
1-Jun-1981										135.4			
29-Jun-1981		158.0		153.6									
2-Jul-1981											139.5		
24-Dec-1981											140.2		
7-Feb-1982											139.6		
23-Mar-1982		158.8		152.6	159.1	150.8							
15-Jun-1982		158.8		152.4	159.0	151.0				135.6			
9-Jul-1982											140.7		
15-Sep-1982		159.5		152.7	158.7	151.9							
20-Sep-82										137.0			
11-Dec-82	146.1	160.1		152.6	159.0	153.7							
18-Dec-82										135.1			
8-Mar-83	146.3				158.8	153.6							
9-Mar-83		159.6		152.6									
15-Mar-83										140.9			
22-Jun-83	152.3	159.7		155.1	159.0	156.1	152.7			151.4	149.9		
15-Sep-83	153.3	159.7		156.5			154.5						
21-Sep-83					159.7	156.8							
3-Oct-83											154.2		
15-Oct-83										153.8			
12-Dec-83	154.4	160.4		157.7	160.0	157.9	155.4						
14-Dec-83										156.4	155.9		
12-Mar-84	155.1					158.5	156.2						
13-Mar-84		159.9		158.2	160.1								
22-Mar-84										156.1	156.6		

RAI Table 2.4.12-3.1 Historical Groundwater Levels for the Water Table Aquifer.

	Observation Well and Water Level Elevation (ft msl)												
Date	**142**	**179**	**802A**	**803A**	**804**	**805A**	**806B**	**808**	**809**	**LT-1A/1B**	**LT-7/7A**	**LT-12**	**LT-13**
11-Jun-84				158.9	160.5	159.9							
12-Jun-84		155.8					157.1			157.4	157.4		
13-Sep-84				159.8									
16-Sep-84					161.0								
18-Sep-84	156.5	150.9				160.6	157.4				157.7		
13-Dec-84	155.9	151.1		159.9	160.2	160.1	157.1			157.0			
31-Dec-84											158.0		
4-Feb-85	155.7	148.9		159.6	160.9	159.9	157.0			157.1			
30-Jun-85	155.5	150.2		159.6	161.0	159.5	156.9			152.0	152.0		
7-Jul-85	155.3	148.5		159.5	160.8	159.3	156.6	159.2	155.5	157.0		158.5	157.6
16-Jul-85	155.3	150.0		159.4	160.8	159.3	156.7	159.2	152.7	155.2	158.0	160.2	157.5
23-Jul-85	155.2	150.3		159.5	160.8	159.3	156.7	159.3	152.8	155.2	158.1	160.0	157.6
31-Jul-85	155.3	150.6		159.5	160.9	159.3	156.8	159.8	152.8	155.2	158.0	160.0	157.5
7-Aug-85	155.4	148.6		159.4	160.9	159.3	157.0	160.0	152.8	155.3	158.1	160.2	157.7
14-Aug-85	155.3	148.6		159.4	160.8	159.2	156.2	160.3	152.7	155.3	158.0	160.2	157.7
21-Aug-85	155.4	148.6		159.4	160.8	159.3	157.1	160.4	152.8	157.2	158.1	160.4	157.8
28-Aug-85	155.6	148.8		159.5	160.9	159.4	157.2	160.5	152.5	157.3	158.2	160.5	157.7
4-Sep-85	155.5	148.8	159.0	159.6	161.0	159.6	157.2	160.4	152.8	157.4	158.3	160.8	157.8
11-Sep-85	155.5	148.8	159.0	159.5	161.0	159.6	157.2	160.6	152.9	157.4	158.4	161.1	158.1
18-Sep-85	155.4	148.8	159.0	159.5	160.8	159.5	157.2	160.5	152.8	157.3	158.4	161.1	158.0
25-Sep-85	155.6	148.8	159.0	159.3	160.9	159.6	157.3	160.4	152.9	157.5	158.5	161.4	158.1
6-Oct-85	155.6	148.8	159.1	159.6	160.9	159.7	157.3	160.3	152.9	157.5	158.5	161.5	158.1
9-Oct-85	155.5	148.8	159.0	159.6	160.9	159.6	157.2	160.2	152.9	157.3	158.3	161.3	158.0
16-Oct-85	155.5	148.8	159.2	159.7	160.8	159.6	157.4	160.3	152.9	157.6	158.7	161.5	158.2
23-Oct-85	155.5	148.8	159.1	159.7	160.7	159.7	157.3	160.2	152.9	157.5	158.8	161.5	158.3

RAI Table 2.4.12-3.1 Historical Groundwater Levels for the Water Table Aquifer.

	Observation Well and Water Level Elevation (ft msl)												
Date	**142**	**179**	**802A**	**803A**	**804**	**805A**	**806B**	**808**	**809**	**LT-1A/1B**	**LT-7/7A**	**LT-12**	**LT-13**
30-Oct-85	155.7	148.8	159.2	159.8	161.1	159.9	157.5	160.2	153.0	157.7	159.0	162.0	158.5
6-Nov-85	155.5	148.7		159.5	160.8	159.7	157.2	160.1	152.9	157.4	158.5	161.6	158.4
13-Nov-85	155.5	148.8		159.5	161.0	159.8	157.2	160.1	152.9	157.3	158.5	161.5	158.0
20-Nov-85	155.6	148.9	159.2	159.8	161.0	159.7	157.3	160.2	153.1	157.4	158.5	161.5	158.1
27-Nov-85	155.6	148.8	159.1	159.6	160.6	159.8	157.4	160.1	153.0	157.6	158.7	161.6	158.1
4-Dec-85	155.7	148.8	159.1	159.7	160.8	159.6	157.4	160.1	153.0	157.5	158.5	161.3	158.4
11-Dec-85	155.8	148.8	159.2	159.9	161.1	159.9	157.6	160.3	153.0	157.8	158.8	161.6	158.3
18-Dec-85	155.8	148.8	159.2	159.7	160.9	159.9	157.6	160.4	153.0	157.7	158.9	161.5	158.3
29-Dec-85	155.9	148.8	159.3	159.8		159.9	157.7	160.5	153.0	157.8	158.6	161.6	158.6
2-Jan-86	156.0	148.9	159.4	159.8	161.0	159.8	157.7	160.5	153.1	157.8	158.6	161.6	158.4
10-Jan-86	156.1	148.9	159.6	160.0	161.4	159.7	157.9	160.5	153.3	158.2	158.8	161.8	158.3
15-Jan-86	155.7	148.7	159.4	159.8	160.7	159.8	157.7	160.6	152.9	157.9	158.8	161.9	158.3
22-Jan-86	156.0	148.8	159.4	159.8	161.0	160.0	157.2	160.5	153.1	157.8			158.7
29-Jan-86	156.0	148.8	159.5	160.0	161.2	160.2	157.7	160.5	153.1	157.9	159.2	161.8	158.8
5-Feb-86	156.0	148.7	159.5	159.9	161.1	160.1	157.6	160.6	153.0	157.9	159.2	162.0	158.6
12-Feb-86	155.9	148.8	159.4	159.9	160.9	160.0	157.6	160.5	153.0	157.7	158.8	161.5	158.8
19-Feb-86	156.0	148.8	159.6	160.0	161.2	160.2	157.7	160.4	153.1	157.9	159.1	162.0	158.8
26-Feb-86	156.0	148.9	159.8	160.3	161.2	160.5	157.9	160.3	153.1	158.2	159.6	162.4	158.7
5-Mar-86	155.8	148.7	159.4	159.9	161.0	160.1	157.5	160.3	153.0	157.7	158.9	161.7	158.7
15-Mar-86	156.1	148.8	159.7	160.2	161.5	160.1	157.8	160.3	153.3	157.7	159.0	161.8	158.6
19-Mar-86	155.8	148.8	159.4	160.0	161.1	160.1	157.5	160.2	153.1	157.6	158.9	161.5	158.4
26-Mar-86	155.8	148.8	159.4	160.1	161.4	160.3	157.5	160.1	153.0	157.7	158.9	161.6	158.5
2-Apr-86	155.9	148.7	159.6	160.3	161.4	160.4	157.6	160.1	153.2	157.8	159.0	161.7	158.5
9-Apr-86	155.9	148.8	159.6	160.1	161.3	160.2	157.6	160.2	153.1	157.9	159.2	161.9	158.7
16-Apr-86	155.7	148.7	159.8	160.3	161.1	160.3	157.4	160.1	153.1	157.5	158.7	161.4	158.1

RAI Table 2.4.12-3.1 Historical Groundwater Levels for the Water Table Aquifer.

	Observation Well and Water Level Elevation (ft msl)												
Date	142	179	802A	803A	804	805A	806B	808	809	LT-1A/1B	LT-7/7A	LT-12	LT-13
23-Apr-86	155.9	148.8	159.5	160.2	161.4	160.0	157.5	160.2	153.2	157.7	158.9	161.5	158.7
30-Apr-86	155.8	148.8	159.4	160.1	161.4	160.2	157.4	160.1	153.1	157.7	158.8	161.5	158.5
7-May-86	155.7	148.7	159.4	160.1	161.2	160.2	157.5	160.0	153.0	157.4	158.3	161.2	158.3
14-May-86	155.7	148.8	159.3	160.1	161.3	160.1	157.3	160.0	153.1	157.6	158.8	161.3	158.9
21-May-86	155.8	148.8	159.4	160.1	161.3	160.2	157.4	159.9	153.1	157.6	158.8	161.5	158.4
28-May-86	155.7	148.8	159.4	160.1	161.4	160.2	157.3	159.9	153.1	157.5	158.8	161.5	158.4
4-Jun-86	155.7	148.7	159.3	160.0	161.2	160.0	157.2	159.9	153.1	157.3	158.4	161.0	158.3
11-Jun-86	155.7	148.8	159.4	159.9	161.3	160.0	157.2	159.8	153.0	157.4	158.6	161.4	158.2
18-Jun-86	155.9	148.8	159.3	160.0	161.1	160.0	157.3	159.8	153.1	157.5	158.7	161.1	158.2
25-Jun-86	155.8	148.8	159.4	160.0	160.9	159.6	157.3	159.7	153.1	157.5	158.6	161.2	158.2
2-Jul-86	155.8	148.8	159.3	160.0	161.4	160.0	157.3	159.7	153.1	157.5	158.6	161.1	158.2
9-Jul-86	155.7	148.7	159.2	160.0	161.4	160.0	157.2	159.7	153.0	157.4	158.5	161.0	158.1
16-Jul-86	155.7	148.7	159.2	159.9	160.9	159.9	157.2	159.7	153.0	157.3	158.4	160.9	158.2
23-Jul-86	155.6	148.7	159.0	159.9	161.2	159.9	157.1	159.6	153.0	157.2	158.3	160.7	158.2
30-Jul-86	155.7	148.7	159.0	159.9	161.2	159.9	157.2	159.6	153.0	157.2	158.3	160.9	158.2
6-Aug-86	155.7	148.8	159.3	160.0	161.3	160.0	157.2	159.6	153.1	157.3	158.3	160.8	157.9
13-Aug-86	155.6	148.8	159.0	159.9	161.2	159.9	157.1	159.5	153.0	157.3	158.4	160.8	158.0
20-Aug-86	155.6	148.8	159.1	159.9	161.1	159.9	157.1	159.5	153.0	157.2	158.2	160.6	158.1
27-Aug-86	155.6	148.8	159.1	159.9	161.2	159.8	157.0	159.4	153.0	157.2	158.3	160.7	157.9
3-Sep-86	155.6	148.8	159.1	159.9	161.2	159.9	157.1	159.6	153.0	157.3	158.3	160.7	158.0
10-Sep-86	155.6	148.7	159.1	159.9	161.2	159.8	157.1	159.6	152.9	157.3	158.3	160.7	157.9
17-Sep-86	155.5	148.7	159.0	159.9	161.0	159.8	157.0	159.7	152.9	157.4	158.5	160.5	157.8
24-Sep-86	155.5	148.7	159.0	159.8	161.0	159.8	157.0	159.9	152.9	157.6	158.2	160.5	158.0
1-Oct-86	155.7	148.8	158.9	159.9	161.0	159.9	157.0	159.9	153.0	157.6	158.3	160.7	157.8
11-Oct-86	155.6	148.8	159.0	160.0	161.1	159.9	157.0	159.8	152.9	157.1	158.1	160.5	157.9

RAI Table 2.4.12-3.1 Historical Groundwater Levels for the Water Table Aquifer.

	Observation Well and Water Level Elevation (ft msl)												
Date	**142**	**179**	**802A**	**803A**	**804**	**805A**	**806B**	**808**	**809**	**LT-1A/1B**	**LT-7/7A**	**LT-12**	**LT-13**
15-Oct-86	155.5	148.8	159.1	159.9	161.1	159.9	157.1	159.9	152.9	157.0	158.2	160.5	158.0
22-Oct-86	155.6	148.8	159.1	159.9	161.2	159.9	157.1	159.8	153.0	157.0	158.2	160.5	157.7
29-Oct-86	155.5	148.8	159.0	159.8	160.9	159.8	157.1	159.9	152.9	156.9	158.2	160.6	157.9
5-Nov-86	155.6	148.8	159.1	159.6	161.2	159.9	157.2	159.8	153.0	157.2	158.2	160.7	158.0
12-Nov-86	155.6	148.8	159.1	159.6	161.1	159.8	157.2	159.7	152.96	157.2	158.3	160.6	157.9
19-Nov-86	155.5	148.8	159.2	159.7	160.9	160.0	157.3	159.8	152.8	157.5	158.6	160.9	158.0
26-Nov-86	155.6	148.8	159.2	159.6	160.9	159.9	157.2	159.6	152.9	157.3	158.3	160.7	158.2
3-Dec-86	155.6	148.8	159.0	159.7	160.9	160.0	157.2	159.6	152.8	157.1	158.0	160.5	157.9
31-Dec-86	155.9	148.1	159.0	159.8	160.9	159.8	157.5	159.4	153.0	157.6	158.6	160.8	158.1
10-Jan-87	156.0	148.9	159.1	160.1	160.9	160.1	157.8	159.3	153.1	158.0	158.9	161.2	158.1
14-Jan-87	156.0	148.8	159.2	160.1	160.8	160.0	157.6	159.1	153.1	158.1	159.1	161.3	158.3
21-Jan-87	155.9	148.7	159.3	160.1	160.8	159.9	157.5	159.2	152.8	159.7	159.1	161.4	158.4
28-Jan-87	156.2	148.8	159.4	160.1	161.2	159.9	157.9	159.5	153.0	158.1	158.9	161.1	158.3
Jan-88	156.7	148.8	160.5	161.8	161.9	161.4	158.2	159.7	153.4	158.2	159.0	160.9	158.6
Feb-88	156.7	148.9	160.7	163.0	162.1	161.6	158.4	159.7	153.3	158.3	159.2	161.1	159.0
Mar-88	156.6	148.8	160.4	161.8	162.1	161.5	158.2	159.3	153.3	158.3	159.2	161.1	158.7
Apr-88	156.7	148.8	160.4	161.6	162.2	161.4	158.1	159.3	153.3	158.3	159.3	161.2	158.9
May-88	156.3	148.7	159.9	161.3	161.7	161.0	157.8	159.0	153.5	157.9	158.8	160.6	158.3
Jun-88	156.2	148.8	159.9	161.1	161.7	161.2	157.8	159.8	153.5	157.9	158.8	160.5	158.3
16-Dec-94			158.8			160.0	156.0	159.4		156.8	155.8	158.3	156.6
13-Jun-95						161.0	156.6						
29-Jun-95			159.6					160.4		157.3	156.3	158.9	157.2
22-Sep-95										157.7	156.7	159.2	157.6
20-Dec-95			160.1							157.8	157.0	159.8	157.8
21-Dec-95						161.6	157.0	160.2					

RAI Table 2.4.12-3.1 Historical Groundwater Levels for the Water Table Aquifer.

	Observation Well and Water Level Elevation (ft msl)												
Date	**142**	**179**	**802A**	**803A**	**804**	**805A**	**806B**	**808**	**809**	**LT-1A/1B**	**LT-7/7A**	**LT-12**	**LT-13**
21-Mar-96										157.6	156.7	159.7	157.6
13-Jun-96			160.1					159.7		157.4	156.5	159.9	157.5
15-Sep-96										156.8	156.4	159.0	156.6
11-Dec-96						160.8	156.5	159.4					
30-Dec-96			159.5							157.3	156.4	159.1	157.3
13-Mar-97										157.1	157.7	159.7	157.7
19-Jun-97			159.0			160.7	156.5	159.2		156.8	156.0	158.6	156.8
31-Dec-97			158.9			160.7	156.6	159.0		156.7	155.8	158.4	156.7
24-Mar-98										157.6	156.5	159.2	157.6
23-Jun-98			158.8			160.8	156.7	159.2		157.1	156.1	159.0	157.1
28-Sep-98										157.3	156.5	159.1	157.4
23-Mar-99										158.8	157.8	160.0	158.8
8-Jun-99										158.5		160.6	
15-Jun-99							157.6						158.6
17-Jun-99			160.8			162.5		159.0			157.7		
17-Dec-99			159.7			160.9	156.9	158.6		156.9	156.1	159.6	157.6
22-Mar-00										158.5	157.3	159.0	158.1
2-Jun-00						159.7	156.0				157.0	158.3	156.5
5-Jun-00			158.6					158.3		156.8			
8-Sep-00										155.5	156.4	157.7	156.0
7-Dec-00			157.8			158.8	155.3	158.4		155.5	156.4	157.8	155.9
5-Mar-01										155.9	154.4	157.0	155.2
8-Jun-01			157.4			158.5	155.1			155.1	156.0	157.2	155.6
12-Jun-01								155.0					
14-Mar-02										155.3	156.1	157.7	155.7

RAI Table 2.4.12-3.1 Historical Groundwater Levels for the Water Table Aquifer.

	Observation Well and Water Level Elevation (ft msl)												
Date	**142**	**179**	**802A**	**803A**	**804**	**805A**	**806B**	**808**	**809**	**LT-1A/1B**	**LT-7/7A**	**LT-12**	**LT-13**
5-Jun-02			157.0							154.7	155.5	156.9	155.3
7-Jun-02						157.7	154.6	158.0					
18-Sep-02										154.2	155.2	156.4	154.7
5-Dec-02			156.1			156.9	154.0	157.6		154.2	153.4	156.2	154.7
10-Mar-03										153.9	154.7	155.8	154.3
18-Jun-03			156.9			159.0	154.8	160.0		154.7	154.0	156.3	155.1
4-Sep-03										155.6	154.5	157.3	155.9
9-Dec-03			158.7			160.0	156.2	160.6		156.2	155.0	158.0	156.7
3-Mar-04										156.3	155.2	158.4	156.9
3-Sep-04										156.1	157.0	158.4	156.7
17-Dec-04			158.5			159.5	155.9	158.6		156.0	155.2	158.2	156.6

Notes.

1) Well data for 23 Oct 1971 to 4 Feb 1985 contained in the Ground Water Supplement for VEGP Units 1 and 2 (Georgia Power, 1985).

2) Well data for 30 Jun 1986 to 29 Dec 1985 contained in Observation Well Readings for VEGP Units 1 and 2, July – December 1985 (Georgia Power, 1986).

3) Well data for 2 Jan 1986 to 25 Jun 1986 contained in Observation Well Readings for VEGP Units 1 and 2, January - June 1986 (Georgia Power, 1986).

4) Well data for 2 Jul 1986 to 3 Dec 1986 contained in Observation Well Readings, for VEGP Units 1 and 2, July - December 1986 (Georgia Power, 1987).

5) Well data for 31 Dec 1986 to 28 Jan 1987 contained in Piezometer Weekly Readings Report for VEGP Units 1 and 2 (Georgia Power, 1987).

6) Well data for Jan 1988 to June 1988 contained in the Ground Water Supplement, July 1987 – June 1988 (Bechtel Civil Inc. 1988).

7) Well data for 16 Dec 1994 to 17 Dec 2004 contained in Bechtel RFI 25144-000-GRI-GEX-00028, SNC ALWR ESP Project (Bechtel Power Corporation, 2006).

(Preliminary COL Data – Not Final)

RAI Table 2.4.12-3.2 Presence of Utley Limestone in ESP and COL Site Borings

Boring No.	Coordinates (NAD 27)		Utley Lmst	Water Loss
	Northing	Easting		Reported
COL Boring Data (Preliminary)				
B-1105	1144168.36	620002.76	Present	
B-1107	1144153.75	620916.11	Present	
B-1108	1144214.07	621273.00	Present	
B-1109	1144180.46	621580.64	Present	
B-1110	1144170.91	622011.31	Present	
B-1111	1144212.59	622333.79	Present	Water loss
B-1112A	1144219.36	622561.49	Present	Water loss
B-1113	1143901.44	620217.17	Present	
B-1116	1143894.12	621264.65	Present	Water loss
B-1117	1143890.75	621628.43	Present	Water loss
B-1118	1143885.92	622007.97	Present	
B-1119	1143888.30	622333.77	Present	Water loss
B-1120	1143893.05	622558.49	Present	Water loss
B-1121	1143575.57	620216.27	Present	
B-1123	1143575.43	620921.98	Present	Water loss
B-1124	1143627.62	621421.59	Present	Water loss
B-1125	1143586.80	621628.20	Present	Water loss
B-1126	1143567.68	621980.43	Present	Water loss
B-1127	1143573.26	622332.29	Absent	
B-1128A [2]	1143572.55	622682.81		Water loss
B-1129	1143278.22	621893.74	Present	
B-1130	1142482.76	622250.00	Present	Water loss
B-1131	1143172.99	621823.06	Present	
B-1132	1142614.19	621450.08	Present	
B-1133	1142968.94	621451.15	Absent	
B-1134	1143282.88	621104.27	Present	
B-1136	1143178.11	621023.00	Present	
B-1138 [1]				Water loss
B-1139	1142289.86	621026.81	Present	
B-1140	1142290.16	621823.56	Present	Water loss
B-1142	1144416.58	620649.58	ND	
B-1146	1145428.36	622272.08	Present	Water loss
B-1148	1145537.78	623236.50	Absent	
B-1150	1145467.29	624235.30	Absent	
B-1152	1145581.68	625227.34	Absent	
B-1153	1145568.97	625673.46	Absent	
B-1154	1145664.20	626216.06	Absent	
B-1155	1147390.34	624936.42	Absent	
B-1156	1147302.50	624571.69	Absent	
B-1157	1147209.56	625062.18	Absent	
B-1158	1145194.92	626669.12	Absent	
B-1159	1147285.78	624954.51	Absent	

RAI Table 2.4.12-3.2 Presence of Utley Limestone in ESP and COL Site Borings

Boring No.	Coordinates (NAD 27)		Utley Lmst	Water Loss
	Northing	Easting		Reported
COL Boring Data (Preliminary)				
B-1161	1147363.37	624862.14	Absent	
B-1162	1147234.91	624815.03	Present	
B-1163	1147170.58	624938.82	Absent	
B-1164	1147039.33	624487.08	Present	
B-1166	1147452.97	623961.56	Absent	Water loss
B-1168	1147688.45	623467.78	Absent	
B-1170 [1]				
B-1172 [1]				
B-1174	1146476.06	622228.06	Present	
B-1176A	1145876.27	622195.21	Present	
B-1185	1144716.64	622232.17	Present	Water loss
B-1186	1144711.88	618818.88	ND	
B-1187	1144710.19	619259.61	ND	
B-1189	1144459.72	618997.50	ND	
B-1190 [1]				
B-1191	1144301.60	619490.75	ND	
B-1192	1144217.44	618840.90	Present	
B-1193	1144091.49	619277.79	Present	
B-1194	1147505.20	621631.61	NE	
B-1195	1147574.32	622481.27	NE	
B-1196	1147286.61	622013.91	NE	
B-1197	1146872.88	622002.10	NE	
B-3001	1142599.50	621799.64	Present	Water loss
B-3002A	1142599.97	621872.49	Present	Water loss
B-3003	1142599.85	621727.30	Present	Water loss
B-3004	1142447.42	621867.12	Present	Water loss
B-3005	1142717.58	621749.10	Present	
B-3006	1142425.58	621924.99	Present	
B-3007	1142718.50	621876.74	Present	Water loss
B-3008	1142425.35	621773.01	Present	Water loss
B-3009	1142484.48	621956.58	Present	
B-3010 [2]	1142634.86	622024.97		
B-3011	1142776.68	622024.86	Present	Water loss
B-3012	1142772.53	621911.91	Absent	Water loss
B-3013	1142842.89	621825.35	Present	Water loss
B-3014 [2]	1142799.43	621748.55		Water loss
B-3015	1142956.89	621823.95	Present	Water loss
B-3016	1142978.42	621913.43	Absent	Water loss
B-3017	1143034.35	621749.86	Present	Water loss
B-3018	1142738.11	622115.75	Present	Water loss
B-3019	1142977.36	622167.48	Present	Water loss
B-3020	1142977.94	622074.78	Present	Water loss
B-3021	1143070.22	622033.23	Present	Water loss

RAI Table 2.4.12-3.2 Presence of Utley Limestone in ESP and COL Site Borings

Boring No.	Coordinates (NAD 27)		Utley Lmst	Water Loss
	Northing	Easting		Reported
COL Boring Data (Preliminary)				
B-3022	1143069.84	621873.43	Present	Water loss
B-3023	1143061.11	621679.90	Present	Water loss
B-3024	1142905.82	621399.65	Absent	
B-3025	1142460.42	621425.34	Present	
B-3026	1142290.23	621403.73	Present	
B-3027	1142058.69	621423.26	Present	
B-3028	1141867.30	621408.76	Present	Water loss
B-3029	1141881.50	621803.88	Present	
B-3030	1141699.94	621799.67	Present	Water loss
B-3031	1141398.73	622042.01	Present	
B-3032	1141158.18	621709.53	Present	
B-3033	1141405.26	621715.21	Present	
B-3034	1141399.76	621914.68	Present	Water loss
B-3035	1142729.18	621675.37	Present	
B-3036	1142441.55	621675.96	Present	Water loss
B-3037	1143057.42	621768.87	Present	
B-3038	1141883.04	621543.13	Present	
B-3039 [2]	1142917.72	621753.54		Water loss
B-4001	1142599.97	620999.87	Present	
B-4002	1142600.25	621072.18	Present	Water loss
B-4003	1142599.93	620927.13	Present	
B-4004	1142459.68	621046.56	Present	
B-4005	1142714.97	620948.74	Present	
B-4006	1142719.63	621076.36	Present	Water loss
B-4007	1142426.19	621125.28	Present	
B-4008	1142424.22	620973.78	Present	Water loss
B-4009	1142486.09	621156.86	Present	
B-4010	1142667.58	621249.04	Present	Water loss
B-4011	1142773.07	621236.36	Present	Water loss
B-4013	1142842.72	621020.31	Absent	Water loss
B-4014	1142831.99	620950.23	Present	Water loss
B-4015	1142773.04	621115.24	Absent	Water loss
B-4016	1142996.39	621112.90	Absent	
B-4017	1143034.80	620949.92	Present	Water loss
B-4018	1142735.45	621315.51	Present	
B-4019	1142975.89	621371.41	Present	Water loss
B-4020A	1142969.39	621280.02	Present	
B-4021	1143092.61	621247.38	Present	Water loss
B-4022	1143081.30	621073.52	Present	Water loss
B-4023	1143062.36	620879.81	Present	Water loss
B-4024	1142904.78	620601.81	Present	Water loss
B-4025	1142510.01	620625.03	Present	Water loss
B-4026	1142330.16	620597.72	Present	
B-4027	1142180.05	620633.45	Present	Water loss

RAI Table 2.4.12-3.2 Presence of Utley Limestone in ESP and COL Site Borings

Boring No.	Coordinates (NAD 27)		Utley Lmst	Water Loss
	Northing	Easting		Reported
COL Boring Data (Preliminary)				
B-4028	1141984.20	620587.77	Present	
B-4029	1141874.85	620699.95	Present	
B-4030	1141676.68	620698.48	Present	Water loss
B-4031	1141399.83	620975.03	Present	
B-4032A	1141123.72	620794.66	Absent	
B-4033	1141398.11	620348.78	Present	
B-4034	1141375.68	620795.35	Present	Water loss
B-4035	1142729.08	620876.27	Present	Water loss
B-4036	1142457.21	620876.25	Present	Water loss
B-5001	1146177.05	621807.73	Present	Water loss
B-5002	1146339.76	621808.33	Present	Water loss
B-5003	1146386.61	621574.70	Present	Water loss
B-5004	1146547.79	621568.38	Present	Water loss
B-6002	1144134.10	619626.88	Present	
B-6003	1143925.02	619422.80	Present	
B-6004	1143718.15	619473.34	Present	Water loss
B-6005	1143717.98	619873.77	Present	
B-6006 [2]	1143069.79	620301.79		
B-6007	1142730.73	620301.79	Present	
B-6008 [2]	1145443.82	622676.36		Water loss
B-6009	1144773.69	621748.18	Present	Water loss
B-6010	1143893.34	621059.21	Present	
B-6011	1144557.94	621261.73	Present	Water loss
B-6012	1144256.66	620480.54	Present	Water loss
B-6013 [1]				
B-6014 [1]				
B-6015 [1]				

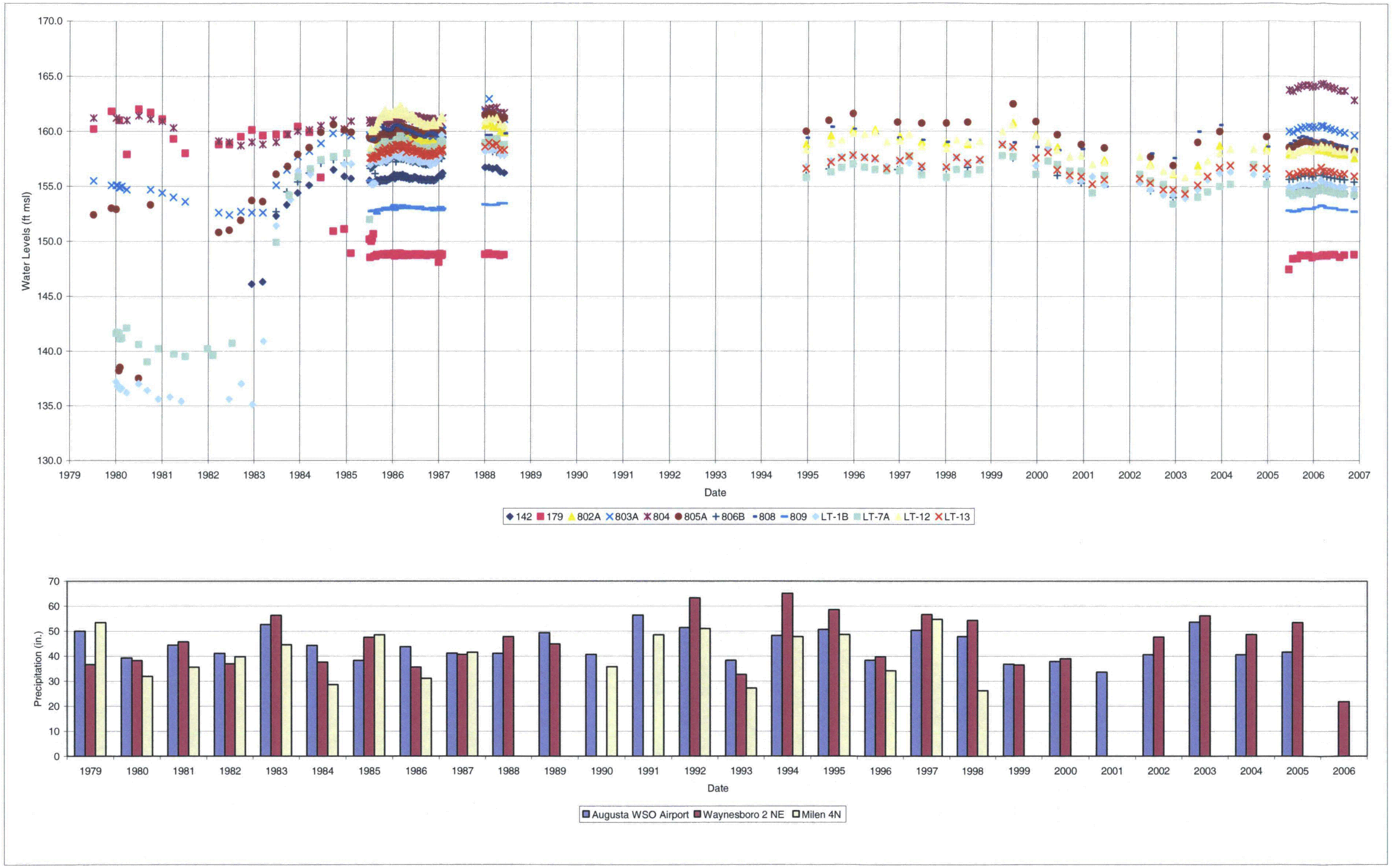
(Preliminary COL Data – Not Final)

RAI Table 2.4.12-3.2 Presence of Utley Limestone in ESP and COL Site Borings

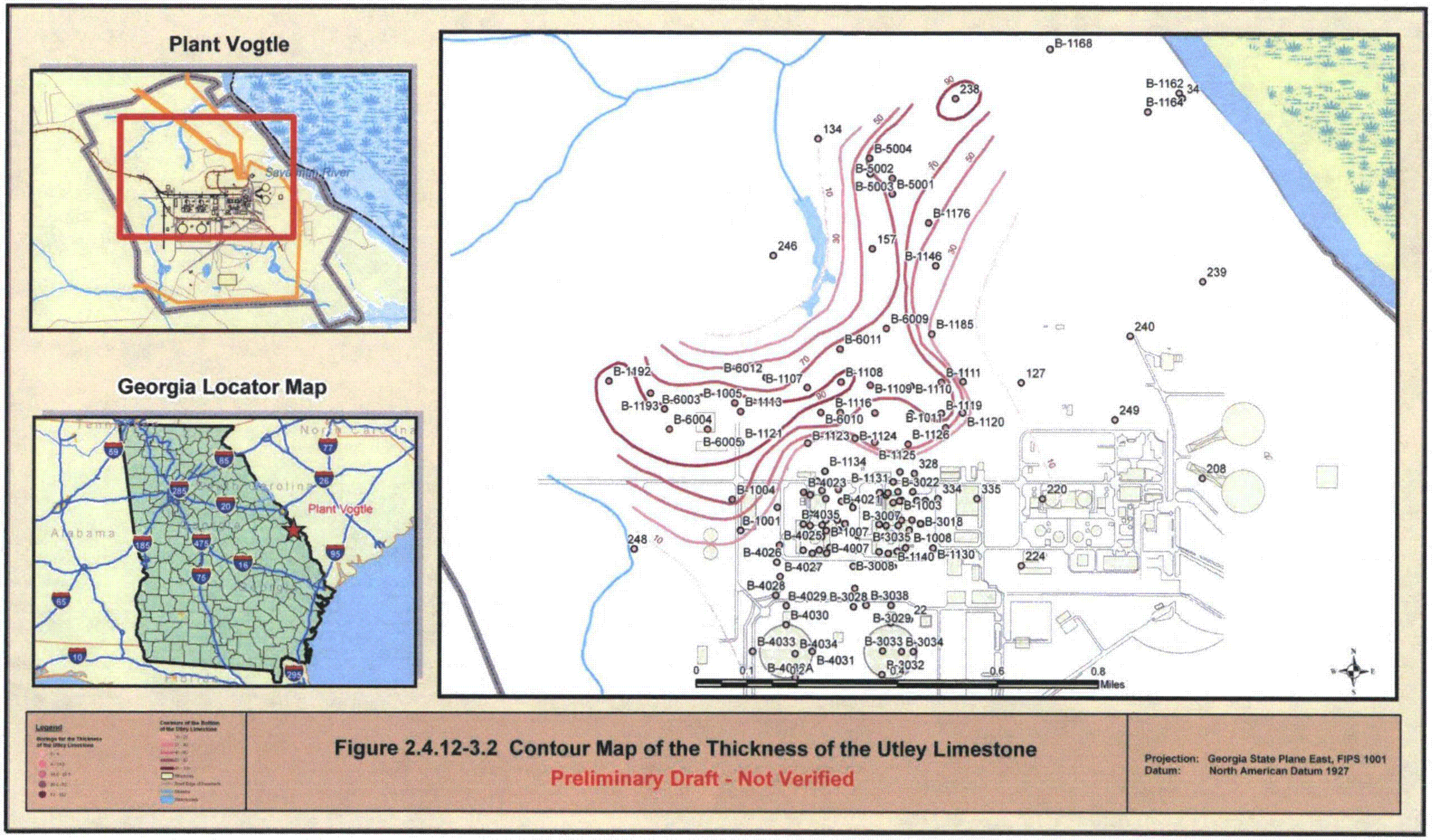
Boring No.	Coordinates (NAD 27)		Utley Lmst	Water Loss
	Northing	Easting		Reported
ESP Boring Data				
B-1001	1142661.92	620220.42	Present	Water loss
B-1002	1142998.52	620985.47	Absent	Water loss
B-1003	1142974.36	621889.85	Present	Water loss
B-1004	1142985.41	620131.44	Present	Water loss
B-1005	1143991.57	620155.35	Present	
B-1006	1143810.26	621342.9	Absent	Water loss
B-1007	1142662.29	621120.13	Present	Water loss
B-1008	1142670.93	621996.15	Present	Water loss
B-1009	1141000.54	620361.26	Absent	
B-1010	1141000.12	621279.68	Absent	
B-1011	1143741.13	622378.01	Present	
B-1013	1140976.08	622272.5	Absent	

Absent (%)	19%	
Water Loss (%)		53%

Note:
[1] No survey data available at the present time.
[2] No boring logs available at present time.
ND = Not determined, indicating that boring terminated in the Utley Lmst.
NE = Not encountered, indicating that the boring terminated in the Barnwell sands.



RAI Figure 2.4.12-3.1 Water Table Aquifer Hydrographs and Total Annual Precipitation for the Period 1979 - 2006



RAI Figure 2.4.12-3.2

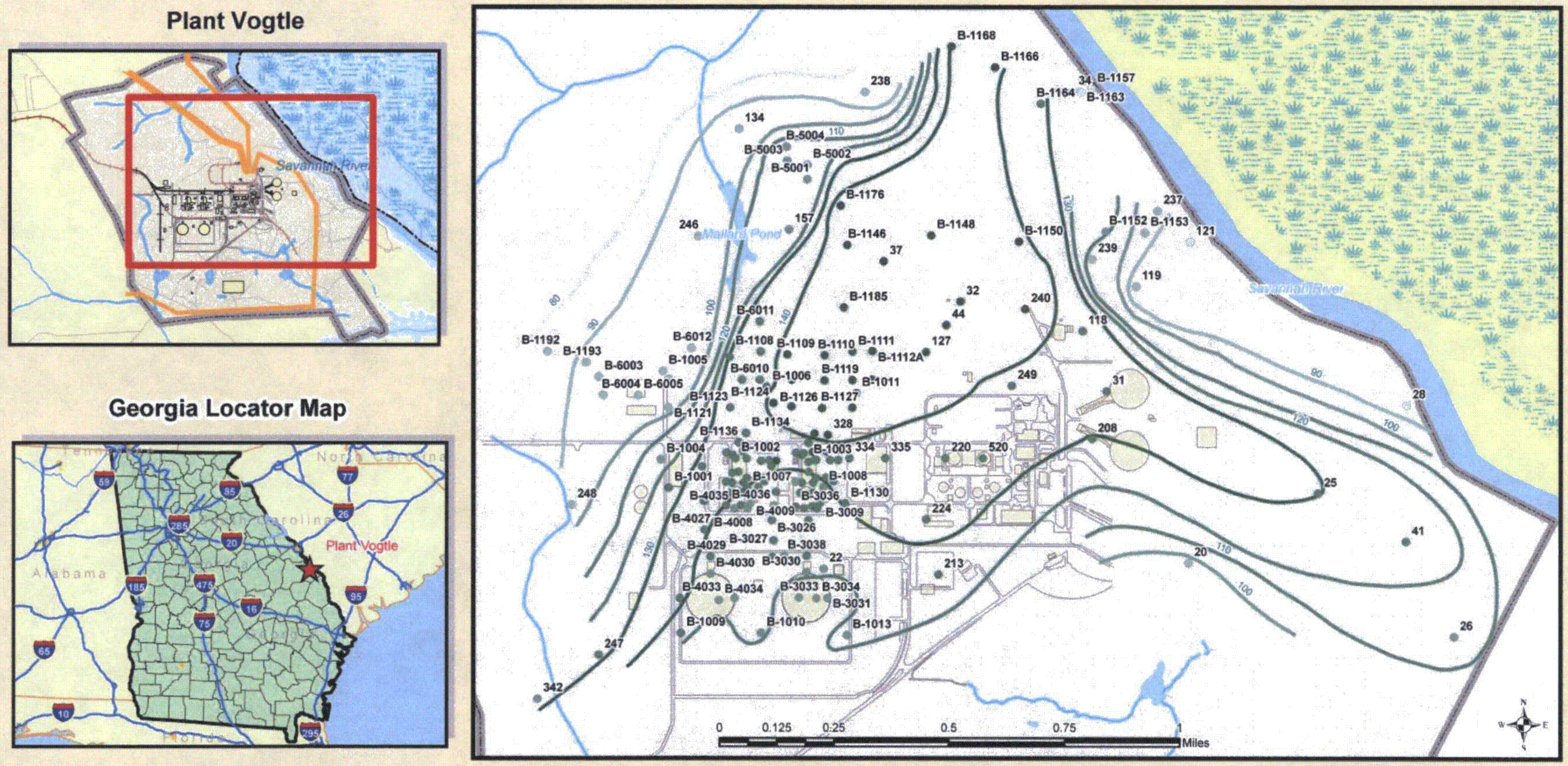
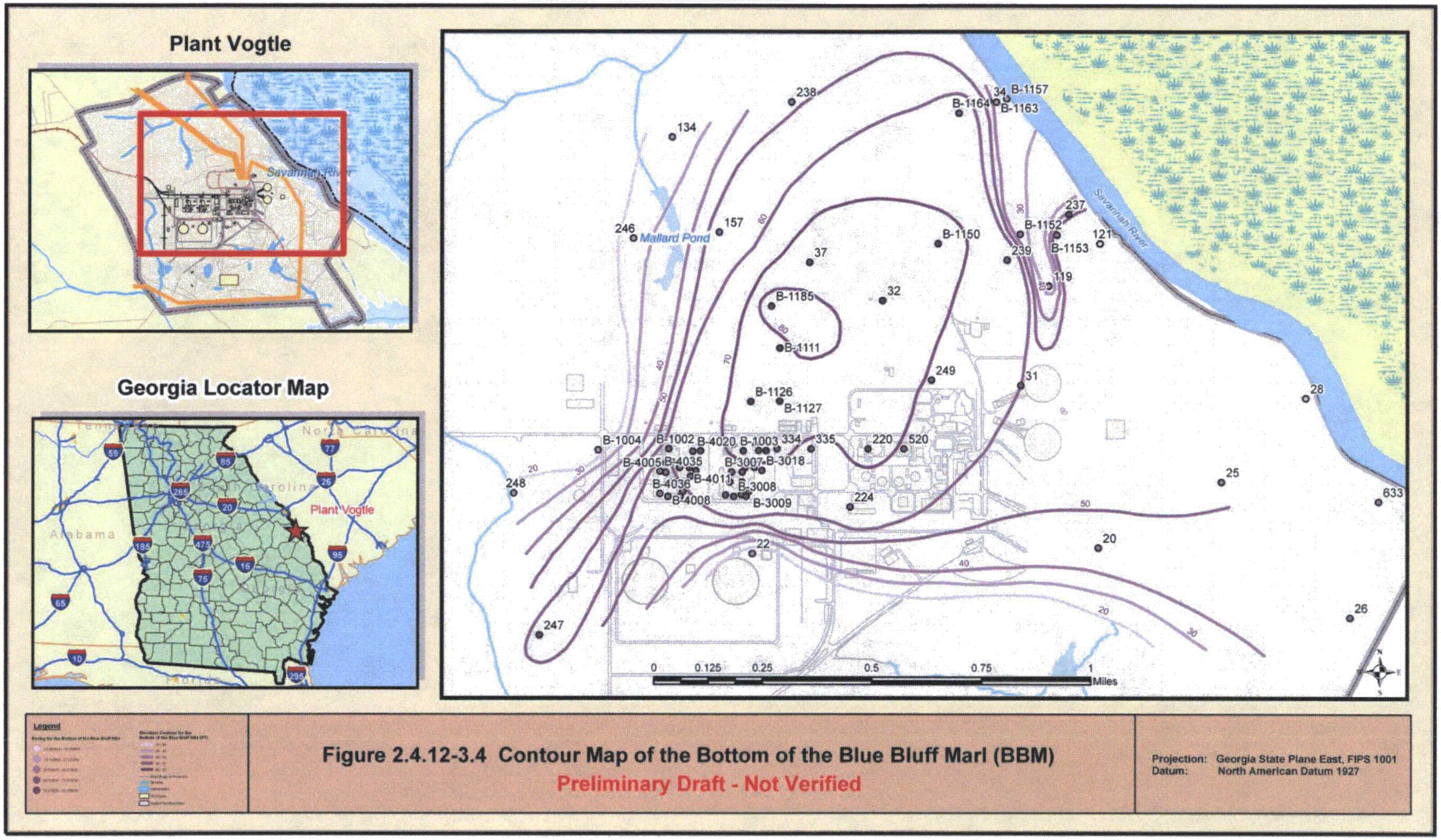


Figure 2.4.12-3.3 Contour Map of the Top of the Blue Bluff Marl (BBM)

Preliminary Draft - Not Verified

Projection: Georgia State Plane East, FIPS 1001
Datum: North American Datum 1927

RAI Figure 2.4.12-3.3



RAI Figure 2.4.12-3.4

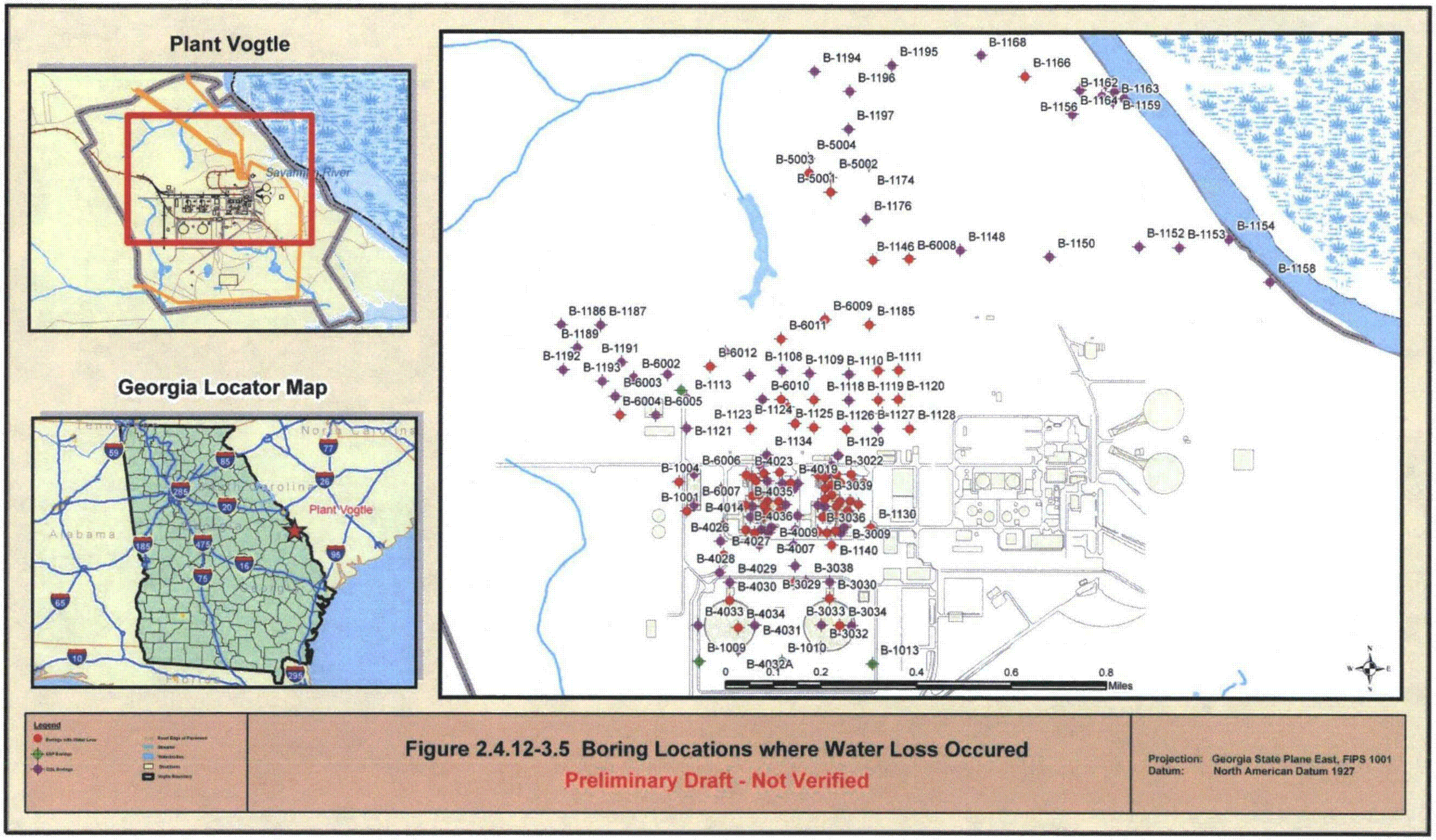


Figure 2.4.12-3.5 Boring Locations where Water Loss Occured

Preliminary Draft - Not Verified

RAI Figure 2.4.12-3.5

2.4.13-1 Provide and incorporate into the SSAR a discussion of the process used to establish that the conceptual model for the transport pathways and travel times presented in the SSAR represents the most conservative of the various other feasible alternative estimates by considering other pathways from the south of a proposed plant where a radwaste holding tank might be located. For example, a potential pathway might consist of travel of contaminants towards the west and then to the north towards the Mallard Pond.

Provide and incorporate into the SSAR a discussion of the process used to conservatively bound the hydraulic properties (gradient, hydraulic conductivity, porosity, etc.) used in safety related calculations. Provide a summary data set utilizing data from the ER and SSAR that presents the bounding hydraulic properties of soil/sediment overlying the Blue Bluff Marl of the Lisbon Formation.

Provide and incorporate into the SSAR new calculations of the accidental release from the effluent hold-up tank that utilize the above described and supported conceptual model and data.

Response:

SSAR Section 2.4.13 has been revised to describe the process used to establish that the conceptual model for transport pathways and travel times represents the most conservative of other feasible alternative estimates. The SSAR has been updated to discuss and identify the conservatively bounding hydraulic properties that are used in the radionuclide transport analysis. Using the bounding transport pathway, hydraulic properties, and resulting travel times, the radionuclide transport analysis of an accidental release from an effluent holdup tank has been revised and SSAR Section 2.4.13 has been updated. The revised SSAR section is included in Enclosure 2.

With respect to a potential pathway for contaminant travel towards the west and then to the north towards Mallard Pond, the updated piezometric contour maps that will be provided in the next revision of the ESP application indicate that the groundwater flow direction in the power block area is to the north-northwest towards Mallard Pond. Furthermore, the hydraulic conductivity of the backfill (3.3 ft/day) is expected to be greater than that of the sediments comprising the Barnwell Group, which was determined to range from 0.12 to 2.7 ft/day for the ESP site. The more conductive backfill along with the north-northwest hydraulic gradient suggest that any radionuclides released to the backfill would be preferentially transported towards the north. The available hydrogeologic data does not support a pathway to the west.

2.4.13-2 Discuss the process used to evaluate the potential for and the impact of chelation and complexation agents (e.g. organic acids) to mix with radiological liquid effluents either within the facility or along the transport pathway in the environment outside the facility. In this discussion, make a clear statement regarding whether or not it is possible for any chelation agents to be mixed with radiological liquid effluents within the ESP facility.

Response:

In the past, Vogtle has used chelating agents to enhance the treatment of wastewaters containing small amounts of radiological material. This routine practice was stopped a number of years ago, primarily because disposal facilities placed strict limits for certain chelating agents on wastes being disposed in the low level radiological waste landfills. The site does not prohibit the use of chelants, but rather requires a comprehensive evaluation prior to use. Vogtle has a Chemical Control procedure that requires evaluation of any chemicals used on or in plant systems and approval before use. For example, a chelating agent

(EDTA) was recently used in the Vogtle steam generator chemical cleaning project. This project required a detailed evaluation of all chemical use including waste disposal.

Vogtle is strictly controls the use of chemicals, including chelating agents, to ensure the use or disposal of wastes resulting from use does not adversely impact plant systems or the environment. Any future use of chelating agents at Vogtle will be tightly controlled. It is not anticipated that chelating agents would be used in applications where they could come in contact with radiological materials, due to the problems that could result from the presence of chelating agents in waste requiring disposal. There is no provision in place at Vogtle for use of chelating agents to mitigate a spill containing radiologically contaminated liquids, and the possibility of inadvertent mixing of spilled radiological material with chelating agents is extremely remote.

In summary, it would be extremely unlikely that a release of radiologically contaminated liquids could come in contact with chelating agents in a manner that would negatively alter the rate of transport for the spill and increase the time of travel to the nearest receptor.

2.4.13-3 The SSAR should include a description of the process followed, and the bases used, to estimate the groundwater outflow to Mallard Pond from the accidental release, and the estimate of the minimum discharge from Mallard Pond.

As discussed during the March 9, 2007, conference call, please revise the SSAR to correct the typographical error reporting a value of 0.7 gpm (2.65 lpm) as the groundwater outflow to the pond from the accidental release. Please include a redacted version of the calculation package showing all parameters, measurements, and assumptions used in the calculation of the 0.07 gpm (0.26 lpm) rate. Also please include a redacted version of the calculation package showing all parameters, measurements, and assumptions leading to the minimum discharge flow rate estimate of 250 gpm (946 lpm) for Mallard Pond. In both of these cases, the redaction should simply remove final calculated values.

Response:

SSAR Section 2.4.13 has been revised to describe the process and basis for estimating the groundwater discharge to Mallard Pond and the estimated flow rate in the stream discharging from Mallard Pond. The groundwater discharge to Mallard Pond has been updated to reflect the more conservative hydraulic properties that have been adopted for the radionuclide transport analysis. All parameters, measurements, and assumptions used to estimate the groundwater discharge to Mallard Pond have been documented in the revised SSAR Section 2.4.13. The revised text removed the 0.7 gpm value from the discussion. The revised SSAR section is included in Enclosure 2. The calculation package describing the flow measurements in the stream discharging from Mallard Pond is included as Enclosure 3.

Southern Nuclear Operating Company

AR-07-0639

Enclosure 2

Proposed Revision to SSAR Section 2.4.13

NOTE: This enclosure consists of a 20-page proposed ESP application section.

2.4.13 Accidental Releases of Liquid Effluents in Ground and Surface Waters

2.4.13.1 Groundwater

This section provides a conservative analysis of a postulated, accidental liquid release of effluents to the groundwater at the VEGP site. The accident scenario is described. The conceptual model used to evaluate radionuclide transport is presented, along with potential pathways of contamination to water users. The radionuclide transport analysis is described, and the results are summarized. The radionuclide concentrations to which a water user might be exposed are compared against the regulatory limits.

Results are considered acceptable if the concentrations are less than the maximum permissible concentrations (MPCs) included in 10 CFR Part 20, Appendix B, Table 2, Column 2. Because the identity and concentration of each radionuclide in the mixture are known, the ratio present in the mixture and the concentration otherwise established in 10 CFR Part 20 Appendix B for the specific radionuclide not in a mixture must also be determined. The sum of such ratios for all of the radionuclides in the mixture may not exceed "1" (i.e., "unity"). These criteria apply to the nearest potable water supply in an unrestricted area.

2.4.13.1.1 Accident Scenario

The accident scenario has been selected based on information developed by Westinghouse to assist AP1000 COL applicants in evaluating the accidental liquid release of effluents **(Westinghouse 2006)**. The accident scenario assumes an instantaneous release from one of the two effluent holdup tanks located in the lowest level of the AP1000 auxiliary building.

There are two effluent holdup tanks, each with a capacity of 28,000 gal., for each AP1000 unit. These tanks have both the highest potential radionuclide concentrations and the largest volume. Therefore, they have been selected by Westinghouse as the limiting tanks for evaluating an accidental release of liquid effluents that could lead to the most adverse contamination of groundwater or surface water, via the groundwater pathway.

Westinghouse estimated the radionuclide concentrations of the effluent holdup tanks to be 101 percent of the reactor coolant. Westinghouse determined the radionuclide concentrations in reactor coolant itself to be as follows:

- For tritium (H-3), a coolant concentration of 1.0 µCi/g should be used.
- Corrosion products (Cr-51, Mn-54, Mn-56, Fe-55, Fe-59, Co-58 and Co-60) should be taken directly from the AP1000 DCD, Table 11.1-2, *Design Basis Reactor Coolant Activity.*
- Other radionuclides should be based on the AP1000 DCD, Table 11.1-2 multiplied by 0.12/0.25 to adjust the failed fuel rate from the design basis to a conservatively bounding value for this analysis.

Based on these recommendations, the expected radionuclide concentrations in the effluent holdup tanks have been calculated, and the results are summarized in Table 2.4.13-1.

2.4.13.1.2 Conceptual Model

Figure 2.4.13-1 illustrates the conceptual model used to evaluate an accidental liquid release of effluent to groundwater, or to surface water via the groundwater pathway. The key elements and assumptions embodied in the conceptual model are described and discussed below.

As indicated in Section 2.4.13.1.1, the effluent holdup tanks are assumed to be the source of the release, with each tank having a volume of 28,000 gal. and the radionuclide concentrations as summarized in Table 2.4.13-1. These tanks are located at the lowest level of the auxiliary building, which has a floor elevation of approximately 186.5 ft msl and is approximately 25 to 35 ft above the water table, based on water table contour plots presented on Figures 2.4.12-7 through 2.4.12-11. One of these tanks is postulated to rupture, and 80 percent of the liquid volume (22,400 gal.) is assumed to be released in accordance with Section 15.7.3 of NUREG-0800. Flow from a tank rupture would initially flood the tank room, and begin to flow to the auxiliary building radiologically controlled area sump via floor drains as described in Section 3.4.1.2.2.2 of the AP1000 DCD. It is assumed that sump pumps are inoperable. According to the AP1000 DCD, this would result in the 22,400 gal. release flooding the balance of level 1 of the auxiliary building via the interconnecting floor drains. Once level 1 is flooded, it is assumed that a pathway is created that would allow the entire 22,400 gal. to enter the groundwater (unconfined aquifer) instantaneously. This assumption is very conservative because it requires failure of the floor drain system, plus it ignores the barriers presented by the 6-ft-thick basemat and the sealed, 3-ft-thick exterior walls of the AP1000 auxiliary building. Furthermore, there is a minimum of 20 ft of unsaturated zone beneath the basemat. Attenuation of radionuclide concentrations would occur during unsaturated zone transport as a consequence of adsorption, dispersion, and radioactive decay, which is not considered in this conservative analysis.

With the postulated instantaneous release of the contents of an effluent holdup tank to groundwater, radionuclides would enter the unconfined aquifer and migrate with the groundwater in the direction of decreasing hydraulic head. Hydraulic head contour maps for the unconfined aquifer presented in Figures 2.4.12-7 through 2.4.12-9 indicate that the groundwater pathway from a point of release in either of the AP1000 auxiliary buildings would be northward to Mallard Pond, a groundwater discharge area, as discussed in Section 2.4.12.1.3. Because the underlying Blue Bluff Marl has a very low vertical permeability, as is described in Section 2.4.12, groundwater flow in the unconfined aquifer is predominantly horizontal. The flow path is assumed to be a straight line between either auxiliary building and the south side of Mallard Pond, a distance of approximately 2,450 ft based on Figure 1-4. During saturated zone transport, radionuclide concentrations of the liquid released to the water table would be reduced by the processes of adsorption, hydrodynamic dispersion, and radioactive decay. There are no existing water-supply wells between the postulated release points and Mallard Pond that withdraw water from the unconfined aquifer. Based on the data in Table 2.4.12-10, all water-supply wells for the existing VEGP plant withdraw their water from the deeper, confined Tertiary and Cretaceous aquifers.

Mallard Pond serves as a groundwater discharge area for the unconfined aquifer. The radionuclides associated with a liquid release would enter the surface water system via Mallard Pond. Radionuclide concentrations would be diluted in the pond and in the stream running from the pond to the Savannah River. Groundwater flow into Mallard Pond is continuous, and the pond level is controlled by a spillway. Measurements of stream flow discharge from Mallard Pond and at points downstream indicate that flow increases progressively in magnitude before discharging to the Savannah River (Bechtel 1985). Upon discharge to the Savannah River, the stream flow would mix with the Savannah River flow, resulting in significantly further dilution prior to withdrawal by the nearest surface water user. As noted in Section 2.4.1, the nearest downstream industrial surface water users include the Fort James Operating Company and the Georgia Power Company. Both companies operate river intakes that withdraw water from the Savannah River near River Mile 45, which is about 106 miles downstream of the VEGP site. The City of Savannah Municipal and Industrial Plant, and the Beaufort-Jasper County Water and Sewer Authority are the nearest downstream municipal water users. The City of Savannah obtains water from Abercorn Creek where it enters the Savannah River near River Mile 29, which is about 122 miles downstream from the VEGP site. Beaufort-Jasper County withdraws water from the Savannah River via an 18-mile canal.

2.4.13.1.3 Radionuclide Transport Analysis

A radionuclide transport analysis has been conducted to estimate the radionuclide concentrations that might expose existing and future water users based on an instantaneous release of the radioactive liquid of an AP1000 effluent holdup tank. Analysis of liquid effluent release commenced with the simplest of models, using demonstratively conservative assumptions and coefficients. Radionuclide concentrations resulting from the preliminary analysis were then compared against the MPCs identified in 10 CFR Part 20, Appendix B, Table 2, Column 2, to determine acceptability. Further analysis, using progressively more realistic and less conservative assumptions and modeling techniques, was conducted when the preliminary results were not acceptable.

Radionuclide transport along a groundwater pathline is governed by the advection-dispersion-reaction equation **(Javandel et al. 1984)**, which is given as

$$R\frac{\partial C}{\partial t} = D\frac{\partial^2 C}{\partial x^2} - v\frac{\partial C}{\partial x} - \lambda RC \qquad (2.4.13\text{-}1)$$

where: C = radionuclide concentration; R = retardation factor; D = coefficient of longitudinal hydrodynamic dispersion; v = average linear velocity; and λ = radioactive decay constant. The retardation factor is defined from the relationship

$$R = 1 + \frac{\rho_b K_d}{n_e} \qquad (2.4.13\text{-}2)$$

where: ρ_b = bulk density; K_d = distribution coefficient; and n_e = effective porosity. The average linear velocity is determined using Darcy's law, which is

$$v = -\frac{K}{n_e}\frac{dh}{dx} \qquad (2.4.13\text{-}3)$$

where: K = hydraulic conductivity; and dh/dx = hydraulic gradient. The radioactive decay constant can be written as

$$\lambda = \frac{\ln 2}{t_{1/2}} \qquad (2.4.13\text{-}4)$$

where $t_{1/2}$ = radionuclide half-life. Conservatively neglecting hydrodynamic dispersion, Equation 2.4.13-1 can be integrated to yield

$$C = C_0 \exp(-\lambda t) \qquad (2.4.13\text{-}5)$$

where: C = radionuclide concentration; C_0 = initial radionuclide concentration; $t = LR/v$ = radionuclide travel time; and L = groundwater pathline length.

To estimate the radionuclide concentrations in groundwater discharging to Mallard Pond, Equation 2.4.13-5 was applied along the groundwater pathline that would originate at either of the liquid effluent release points beneath the AP1000 auxiliary buildings and terminate at Mallard Pond. The analysis was performed sequentially as described below.

2.4.13.1.3.1 Transport Considering Radioactive Decay Only

An initial screening analysis was performed considering radioactive decay only. This analysis assumed that all radionuclides migrate at the same rate as groundwater and considered no adsorption and retardation, which would otherwise result in a longer travel time and more radioactive decay. The concentrations of the radionuclides appearing in Table 2.4.13-1 were decayed for a period equal to the groundwater travel time from the point of release to Mallard Pond, using Equation 2.4.13-5 with R = 1. Radionuclides having concentrations less than 1 percent of their respective MPCs were eliminated from consideration because their concentrations would be well below their regulatory limits. Any radionuclides having a concentration greater than or equal to 1 percent of their MPC were retained for further evaluation.

Evaluating transport considering radioactive decay only requires an estimate of the groundwater travel time. The groundwater travel time has been estimated by considering the locations of the effluent holdup tanks, the hydrogeologic properties of the backfill, and conservative estimates of the hydraulic gradient and hydraulic conductivity of the water table aquifer. The total saturated zone travel time is the sum of three components: (1) travel time in the backfill; (2) travel time in the water table aquifer in the area between the backfill and the point at which the hydraulic gradient steepens near OW-1005; and (3) travel time between OW-1005 and Mallard Pond. The travel time in each is a function of the travel distance, hydraulic conductivity, effective porosity, and hydraulic gradient. The basis for estimating the travel time in each of these three segments is described below.

1. The travel distance in the backfill was determined to be about 460 ft, which represents the shortest distance between the portion of level 1 of the auxiliary building potentially flooded by a tank rupture and the northern extent of the power block excavation. This distance considers the 71 ft between column lines 7.3 and 11 of the auxiliary building (AP1000 Doc. No. APP-1010-P2-001), the 310 ft length of the turbine building (AP1000 Doc. No. APP-0030-X4-001), and the 80 ft between the turbine building and the northern extent of the power block excavation. A hydraulic conductivity of 1,220 ft/yr (3.3 ft/day) was conservatively assigned to the backfill, which is the maximum in situ value reported for the VEGP site and was obtained from Table 2.4.12-15 of the UFSAR **(SNC 2003)**. The effective porosity of the backfill was taken to be 0.34 as established in Section 2.4.13.1.1 of the UFSAR **(SNC 2003)**. Because the backfill for Units 3 and 4 will be obtained from the borrow areas used for Units 1 and 2 and compacted to the same criteria, the hydraulic conductivity and porosity values observed for Units 1 and 2 should be representative of Units 3 and 4. The hydraulic gradient in the backfill was conservatively estimated to be 0.014 ft/ft using the maximum water level observed at OW-1009 (El. 163.03 ft msl), the minimum water level observed at OW-1005 (El. 132.53 ft msl), and the distance between the two observation wells (2,209 ft). Based on the aforementioned, conservatively-established parameters, the groundwater travel time in the backfill was calculated to be 9.16 years.

2. The travel distance between the northern extent of the power block excavation and OW-1005 was determined to be 990 ft based on the location of OW-1005. Geotechnical borings included in Appendix 2.5A along with water table contour maps included in Section 2.4.12 indicate that groundwater flow from the power block area to the north and towards Mallard Pond will occur in the Utley limestone, as the data suggest that the limestone is continuous along this pathway. Test results given in Table 2.4.12-3 indicate that the in situ hydraulic conductivity of the Utley limestone ranges from 0.12 to 2.7 ft/day (boring logs for wells OW-1003, OW-1005, OW-1006, OW-1007, OW-1009, OW-1010, OW-1013, and OW-1015 indicate completion in the Utley limestone). UFSAR **(SNC**

2003) hydraulic testing results, adjacent to VEGP Units 1 and 2, indicate the possibility of localized, highly permeable zones in the Utley limestone. To address the possibility that similar zones are present north of Units 3 and 4, the maximum value reported in the UFSAR, 125,400 ft/year (343 ft/day), is used in this analysis. The effective porosity of the water table aquifer has been estimated to be 0.32 based on site-specific measurements, as noted in Section 2.4.12.1.4. Effective porosities of limestone formations are typically lower. A lower value of 0.10 has been adopted from the literature **(Heath 1998)** to provide a conservative estimate of the average linear velocity. The hydraulic gradient over this segment is assumed to be the same as that in the backfill (0.014 ft/ft). Using the parameters described above, a groundwater travel time of 0.06 years is estimated for this segment.

3. The travel distance between OW-1005 and Mallard is about 1,000 ft based on site topographic surveys. As with the prior segment, groundwater flow occurs in the Utley limestone and the same values for hydraulic conductivity (125,400 ft/yr) and effective porosity (0.10) are adopted. The hydraulic gradient is estimated to be 0.023 ft/ft using the maximum water level observed at OW-1005 (133.20 ft msl), the water surface elevation in Mallard Pond (110 ft msl), and the distance between the two (1,000 ft). A groundwater travel time of 0.03 years is estimated for this segment based the above parameters.

Summing the above travel times, the total travel time for this analysis is 9.25 years. Using Equation 2.4.13-5, the initial concentrations given in Table 2.4.13-1 were decayed for a period of 9.25 years. Table 2.4.13-2 summarizes the results considering only radioactive decay and identifies those radionuclides that would exceed their MPC by more than 1 percent. These include H-3, Mn-54, Fe-55, Co-60, Sr-90, I-129, Cs-134, and Cs-137.

2.4.13.1.3.2 Transport Considering Radioactive Decay and Adsorption

Radionuclides retained from the screening analysis (H-3, Mn-54, Fe-55, Co-60, Sr-90, I-129, Cs-134, and Cs-137) were further evaluated considering adsorption and retardation in addition to radioactive decay. Distribution coefficients values for Co-60, Sr-90, Cs-134, and Cs-137 were determined based on laboratory analyses of soil samples obtained from the VEGP site **(Kaplan and Millings 2006; MACTEC 2006)**, and are shown in Table 2.4.13-3. Sixteen soil samples were taken from shallow test pits located in potential borrow source areas for backfill that will be required for the new AP1000 units. Laboratory testing of these backfill samples yielded distribution coefficients that range from 1.4 to 15.3 mL/g for Co, 6.0 to 51.7 mL/g for Sr, and 3.5 to 56.2 mL/g for Cs. Three additional soil samples were obtained from a vibratory boring located near B-1003. The samples acquired from the vibratory boring represent the Utley limestone based on the boring log for B-1003. Testing of the Utley limestone samples resulted in distribution coefficients that range from 3.9 to 21.3 mL/g for Co, 14.4 to 17.4 mL/g for Sr, and 22.7 to 33.2 mL/g for Cs.

Distribution coefficients for Co, Sr, and Cs in the backfill were conservatively assigned the minimum value determined from the sixteen samples (1.4 mL/g for Co, 6.0 mL/g for Sr, and 3.5 mL/g for Cs). Distribution coefficients for Co, Sr, and Cs in the Utley limestone were conservatively assigned the minimum value observed for the three vibratory boring samples (3.9 mL/g for Co, 14.4 mL/g for Sr, and 22.7 mL/g for Cs). Distribution coefficients for H-3 and I-129, which have no or little tendency for adsorption, were taken to be zero for both the backfill and Utley limestone. Distribution coefficients for Mn-54 and Fe-55 were conservatively assumed to be zero in both the backfill and Utley limestone.

Retardation factors were calculated using Equation 2.4.13-2 with the distribution coefficients as stated above, effective porosities of 0.34 for the backfill and 0.10 for the Utley limestone, and a bulk density of 1.60 g/cm^3. Total radionuclide travel times were calculated by summing the radionuclide travel times in the backfill and the Utley limestone. Radionuclide concentrations were then determined at the point of discharge to Mallard Pond using Equation 2.4.13-5 and the appropriate initial concentration, decay rate, and total travel time. Results are summarized in Table 2.4.13-4 and indicate that H-3, Mn-54, Fe-55, Sr-90, I-129, and Cs-137 would exceed their respective MPC by more than 1 percent.

2.4.13.1.3.3 Transport Considering Radioactive Decay, Adsorption, and Dilution

The H-3, Mn-54, Fe-55, Sr-90, I-129, and Cs-137 discharging to surface water (Mallard Pond) would mix with other, uncontaminated groundwater discharging to surface water. A dilution factor was estimated to account for the mixing and dilution of contaminated groundwater with uncontaminated groundwater. The dilution factor is the ratio of the rate at which the postulated release would discharge to surface water (Mallard Pond) as contaminated groundwater to the total rate of groundwater discharge to surface water, which would include both uncontaminated and contaminated groundwater. The magnitude of the dilution factor was estimated as described below.

The rate at which a release from an effluent holdup tank discharges to surface water (Mallard Pond) is determined by the transport characteristics of the water table aquifer. A release from an effluent holdup tank would undergo unsaturated zone transport beneath the auxiliary building, followed by saturated zone transport first through the backfill and then through the Utley limestone, and would finally discharge to Mallard Pond. The discharge rate itself is a function of the Darcy velocity, and the assumed volume and dimensions of the resulting contaminant slug. The Darcy velocity was calculated to be 0.047 ft/day, using a hydraulic conductivity of 3.3 ft/day and a hydraulic gradient of 0.014 ft/ft. These values represent the hydrogeologic characteristics of the backfill as described previously. The volume of the liquid release has been assumed to be 22,400 gal. (2,995 ft^3), which represents 80 percent of the 28,000 gal. capacity of one effluent holdup tank (NUREG-0800, Section 15.7.3 recommends that 80 percent of the liquid volume be considered in this analysis). Considering the effective porosity of the backfill (0.34), the release would occupy about 8,810 ft^3 of the saturated backfill.

The shape of the resulting contaminant slug is assumed to be square in plan view and extend vertically throughout the entire saturated thickness of the backfill. Using 20 ft as a representative saturated thickness (water table to top of Blue Bluff Marl), the slug would have an area of about 440 ft^2 in plan view and a width of about 21 ft. The cross-sectional area of the contaminant slug normal to the groundwater flow direction would therefore be 20 ft by 21 ft or about 420 ft^2. The discharge rate of the contaminant slug is then the product of the Darcy velocity and the cross-sectional area, 20 ft^3/day or 0.10 gpm. The rate of total groundwater discharge to surface water has been estimated as 1,125 gpm at a point just downstream of the confluence of the stream discharging from Mallard Pond and its west branch. This value is the result of stream flow measurements that were taken in the months of June and July to support the licensing of VEGP Units 1 and 2 **(Bechtel 1985)**. Because the stream discharging from Mallard Pond and its west branch are both perennial streams, the stream flow measurements would represent the groundwater discharge. The resulting dilution factor is calculated as the ratio of 0.10 gpm to 1,125 gpm, or 9.1E-05.

This dilution factor is applied to the H-3, Mn-54, Fe-55, Sr-90, I-129, and Cs-137 concentrations reported in Table 2.4.13-4 to account for dilution in addition to radioactive decay and adsorption. Table 2.4.13-5 summarizes the resulting concentrations, which would represent the concentrations in the surface water at a point just downstream of the confluence of the stream discharging from Mallard Pond and its west branch. It is seen that the concentrations of each of these radionuclides are below their respective MPCs.

2.4.13.1.4 Compliance with 10 CFR Part 20

The radionuclide transport analysis presented in Section 2.4.13.1.3 demonstrates that each of the radionuclides that could be accidentally released to groundwater would be individually below its MPC. However, 10 CFR Part 20, Appendix B, Table 2, imposes additional requirements when the identity and concentration of each radionuclide in a mixture are known. In this case, the ratio present in the mixture and the concentration otherwise established in 10 CFR Part 20 Appendix B for the specific radionuclide not in a mixture must be determined. The sum of such ratios for all of the radionuclides in the mixture may not exceed “1” (i.e., “unity”) as indicated by Note 4 in Appendix B, 10 CFR Part 20.

This sum of fractions approach was applied to the radionuclide concentrations conservatively estimated in Section 2.4.13.1.3. Results are summarized in Table 2.4.13-6. The ratios for the mixture sum to 0.32, which demonstrates that an accidental liquid release of effluents in groundwater would not exceed 10 CFR Part 20 limits in the Mallard Pond stream prior to reaching the VEGP site property (EAB).

Compliance with 10 CFR Part 20 is further assured considering that the point at which compliance has been demonstrated is within the restricted area and not a potable water source. The stream discharging from Mallard Pond is a gaining stream that discharges to, and mixes with, the Savannah River. The entire reach of this stream, about 1.0 mi. in length, is within the restricted area and not a potable water supply. The nearest potable water supply in an unrestricted area to which the 10 CFR Part 20 requirements would apply is the Savannah River. Mixing of the tributary stream flow with the Savannah River flow would dilute radionuclide concentrations further. The magnitude of this additional dilution can be estimated from the ratio of the tributary stream flow rate (1,125 gpm) to the Savannah River flow rate. Using the 100-year drought flow, given as 3,298 ft^3/sec (1,480,000 gpm) in Section 2.4.11, to conservatively represent the Savannah River flow rate, a dilution factor of 7.6E-04 is calculated. Accounting for this additional dilution would further reduce radionuclide concentrations by a factor of about 1,000. Consequently, the ratios for the mixture would sum to a value much less than unity and well below the compliance limit.

2.4.13.2 Surface Water

No outdoor tanks contain radioactivity in the Westinghouse AP1000 design **(Westinghouse 2006)**. In particular, the AP1000 design does not require boron changes for load follow and does not recycle boric acid or reactor coolant water, so the boric acid tank is not radioactive. Because no outdoor tanks contain radioactivity, no accident scenario could result in the release of liquid effluent directly to the surface water.

Table 2.4.13-1 Radionuclide Concentrations in the AP1000 Effluent Holdup Tanks

Radionuclide	Design Basis Reactor Coolant Activity[1] (μCi/g)	Reactor Coolant Concentrations[2] (μCi/cm^3)	Effluent Holdup Tank Concentrations[3] (μCi/cm^3)
H-3	-	1.00E+00	1.01E+00
Cr-51	1.30E-03	1.30E-03	1.31E-03
Mn-54	6.70E-04	6.70E-04	6.77E-04
Mn-56	1.70E-01	1.70E-01	1.72E-01
Fe-55	5.00E-04	5.00E-04	5.05E-04
Fe-59	1.30E-04	1.30E-04	1.31E-04
Co-58	1.90E-03	1.90E-03	1.92E-03
Co-60	2.20E-04	2.20E-04	2.22E-04
Br-83	3.20E-02	1.54E-02	1.55E-02
Br-84	1.70E-02	8.16E-03	8.24E-03
Br-85	2.00E-03	9.60E-04	9.70E-04
Rb-88	1.50E+00	7.20E-01	7.27E-01
Rb-89	6.90E-02	3.31E-02	3.35E-02
Sr-89	1.10E-03	5.28E-04	5.33E-04
Sr-90	4.90E-05	2.35E-05	2.38E-05
Sr-91	1.70E-03	8.16E-04	8.24E-04
Sr-92	4.10E-04	1.97E-04	1.99E-04
Y-90	1.30E-05	6.24E-06	6.30E-06
Y-91m	9.20E-04	4.42E-04	4.46E-04
Y-91	1.40E-04	6.72E-05	6.79E-05
Y-92	3.40E-04	1.63E-04	1.65E-04
Y-93	1.10E-04	5.28E-05	5.33E-05
Nb-95	1.60E-04	7.68E-05	7.76E-05
Zr-95	1.60E-04	7.68E-05	7.76E-05
Mo-99	2.10E-01	1.01E-01	1.02E-01
Tc-99m	2.00E-01	9.60E-02	9.70E-02
Ru-103	1.40E-04	6.72E-05	6.79E-05
Rh-103m	1.40E-04	6.72E-05	6.79E-05
Rh-106	4.50E-05	2.16E-05	2.18E-05
Ag-110m	4.00E-04	1.92E-04	1.94E-04
Te-127m	7.60E-04	3.65E-04	3.68E-04
Te-129m	2.60E-03	1.25E-03	1.26E-03
Te-129	3.80E-03	1.82E-03	1.84E-03
Te-131m	6.70E-03	3.22E-03	3.25E-03
Te-131	4.30E-03	2.06E-03	2.08E-03
Te-132	7.90E-02	3.79E-02	3.83E-02
Te-134	1.10E-02	5.28E-03	5.33E-03
I-129	1.50E-08	7.20E-09	7.27E-09
I-130	1.10E-02	5.28E-03	5.33E-03

Radionuclide	Design Basis Reactor Coolant Activity[1] (μCi/g)	Reactor Coolant Concentrations[2] (μCi/cm³)	Effluent Holdup Tank Concentrations[3] (μCi/cm³)
I-131	7.10E-01	3.41E-01	3.44E-01
I-132	9.40E-01	4.51E-01	4.56E-01
I-133	1.30E+00	6.24E-01	6.30E-01
I-134	2.20E-01	1.06E-01	1.07E-01
I-135	7.80E-01	3.74E-01	3.78E-01
Cs-134	6.90E-01	3.31E-01	3.35E-01
Cs-136	1.00E+00	4.80E-01	4.85E-01
Cs-137	5.00E-01	2.40E-01	2.42E-01
Cs-138	3.70E-01	1.78E-01	1.79E-01
Ba-137m	4.70E-01	2.26E-01	2.28E-01
Ba-140	1.00E-03	4.80E-04	4.85E-04
La-140	3.10E-04	1.49E-04	1.50E-04
Ce-141	1.60E-04	7.68E-05	7.76E-05
Ce-143	1.40E-04	6.72E-05	6.79E-05
Pr-143	1.50E-04	7.20E-05	7.27E-05
Ce-144	1.20E-04	5.76E-05	5.82E-05
Pr-144	1.20E-04	5.76E-05	5.82E-05

[1] Values from AP1000 DCD Table 11.1-2.

[2] For tritium (H-3) a coolant concentration of 1.0 μCi/g is used; corrosion products (Cr-51, Mn-54, Mn-56, Fe-55, Fe-59, Co-58 and Co-60) are taken directly from the AP1000 DCD, Table 11.1-2; and other radionuclides are based on the AP1000 DCD, Table 11.1-2 multiplied by 0.12/0.25. The density of all liquids is assumed to be 1 g/cm^3.

[3] Values are 101% of the reactor coolant concentrations.

Table 2.4.13-2 Results of Transport Analysis Considering Radioactive Decay Only

Radionuclide	Effluent Holdup Tank Concentration[1] (μCi/cm³)	Half-life[2] (days)	Decay Rate[3] (days⁻¹)	MPC[4] (μCi/cm³)	Groundwater Concentration[5] (μCi/cm³)	Groundwater Concentration/ MPC
H-3	1.01E+00	4.51E+03	1.54E-04	1.00E-03	6.01E-01	6.01E+02
Cr-51	1.31E-03	2.77E+01	2.50E-02	5.00E-04	2.57E-40	5.14E-37
Mn-54	6.77E-04	3.13E+02	2.21E-03	3.00E-05	3.82E-07	1.27E-02
Mn-56	1.72E-01	1.07E-01	6.48E+00	7.00E-05	0.00E+00	0.00E+00
Fe-55	5.05E-04	9.86E+02	7.03E-04	1.00E-04	4.70E-05	4.70E-01
Fe-59	1.31E-04	4.45E+01	1.56E-02	1.00E-05	1.85E-27	1.85E-22
Co-58	1.92E-03	7.08E+01	9.79E-03	2.00E-05	8.35E-18	4.18E-13
Co-60	2.22E-04	1.93E+03	3.59E-04	3.00E-06	6.60E-05	2.20E+01
Br-83	1.55E-02	9.96E-02	6.96E+00	9.00E-04	0.00E+00	0.00E+00
Br-84	8.24E-03	2.21E-02	3.14E+01	4.00E-04	0.00E+00	0.00E+00
Br-85	9.70E-04	2.01E-03	3.44E+02	1.00E+00	0.00E+00	0.00E+00
Rb-88	7.27E-01	1.24E-02	5.59E+01	4.00E-04	0.00E+00	0.00E+00
Rb-89	3.35E-02	1.06E-02	6.54E+01	9.00E-04	0.00E+00	0.00E+00
Sr-89	5.33E-04	5.05E+01	1.37E-02	8.00E-06	3.91E-24	4.89E-19
Sr-90	2.38E-05	1.06E+04	6.54E-05	5.00E-07	1.91E-05	3.82E+01
Sr-91	8.24E-04	3.96E-01	1.75E+00	2.00E-05	0.00E+00	0.00E+00
Sr-92	1.99E-04	1.13E-01	6.16E+00	4.00E-05	0.00E+00	0.00E+00
Y-90	6.30E-06	2.67E+00	2.60E-01	7.00E-06	0.00E+00	0.00E+00
Y-91m	4.46E-04	3.45E-02	2.01E+01	2.00E-03	0.00E+00	0.00E+00
Y-91	6.79E-05	5.85E+01	1.18E-02	8.00E-06	2.82E-22	3.53E-17
Y-92	1.65E-04	1.48E-01	4.68E+00	4.00E-05	0.00E+00	0.00E+00
Y-93	5.33E-05	4.21E-01	1.65E+00	2.00E-05	0.00E+00	0.00E+00
Nb-95	7.76E-05	3.52E+01	1.97E-02	3.00E-05	1.01E-33	3.36E-29
Zr-95	7.76E-05	6.40E+01	1.08E-02	2.00E-05	1.01E-20	5.03E-16
Mo-99	1.02E-01	2.75E+00	2.52E-01	2.00E-05	0.00E+00	0.00E+00
Tc-99m	9.70E-02	2.51E-01	2.76E+00	1.00E-03	0.00E+00	0.00E+00
Ru-103	6.79E-05	3.93E+01	1.76E-02	3.00E-05	9.11E-31	3.04E-26
Rh-103m	6.79E-05	3.90E-02	1.78E+01	6.00E-03	0.00E+00	0.00E+00
Rh-106	2.18E-05	4.63E-04	1.50E+03	NA[6]	0.00E+00	
Ag-110m	1.94E-04	2.50E+02	2.77E-03	6.00E-06	1.66E-08	2.77E-03
Te-127m	3.68E-04	1.09E+02	6.36E-03	9.00E-06	1.73E-13	1.92E-08
Te-129m	1.26E-03	3.36E+01	2.06E-02	7.00E-06	6.90E-34	9.85E-29
Te-129	1.84E-03	4.83E-02	1.44E+01	4.00E-04	0.00E+00	0.00E+00
Te-131m	3.25E-03	1.25E+00	5.55E-01	8.00E-06	0.00E+00	0.00E+00
Te-131	2.08E-03	1.74E-02	3.98E+01	8.00E-05	0.00E+00	0.00E+00
Te-132	3.83E-02	3.26E+00	2.13E-01	9.00E-06	0.00E+00	0.00E+00
Te-134	5.33E-03	2.90E-02	2.39E+01	3.00E-04	0.00E+00	0.00E+00
I-129	7.27E-09	5.73E+09	1.21E-10	2.00E-07	7.27E-09	3.63E-02
I-130	5.33E-03	5.15E-01	1.35E+00	2.00E-05	0.00E+00	0.00E+00
I-131	3.44E-01	8.04E+00	8.62E-02	1.00E-06	1.17E-127	1.17E-121
I-132	4.56E-01	9.58E-02	7.24E+00	1.00E-04	0.00E+00	0.00E+00
I-133	6.30E-01	8.67E-01	7.99E-01	7.00E-06	0.00E+00	0.00E+00
I-134	1.07E-01	3.65E-02	1.90E+01	4.00E-04	0.00E+00	0.00E+00
I-135	3.78E-01	2.75E-01	2.52E+00	3.00E-05	0.00E+00	0.00E+00
Cs-134	3.35E-01	7.53E+02	9.21E-04	9.00E-07	1.50E-02	1.66E+04

Radionuclide	Effluent Holdup Tank Concentration[1] (μCi/cm³)	Half-life[2] (days)	Decay Rate[3] (days⁻¹)	MPC[4] (μCi/cm³)	Groundwater Concentration[5] (μCi/cm³)	Groundwater Concentration/ MPC
Cs-136	4.85E-01	1.31E+01	5.29E-02	6.00E-06	1.17E-78	1.95E-73
Cs-137	2.42E-01	1.10E+04	6.30E-05	1.00E-06	1.96E-01	1.96E+05
Cs-138	1.79E-01	2.24E-02	3.09E+01	4.00E-04	0.00E+00	0.00E+00
Ba-137m	2.28E-01	1.81E-03	3.84E+02	NA[6]	0.00E+00	
Ba-140	4.85E-04	1.27E+01	5.46E-02	8.00E-06	4.20E-84	5.25E-79
La-140	1.50E-04	1.68E+00	4.13E-01	9.00E-06	0.00E+00	0.00E+00
Ce-141	7.76E-05	3.25E+01	2.13E-02	3.00E-05	4.02E-36	1.34E-31
Ce-143	6.79E-05	1.38E+00	5.02E-01	2.00E-05	0.00E+00	0.00E+00
Pr-143	7.27E-05	1.36E+01	5.10E-02	2.00E-05	1.25E-79	6.26E-75
Ce-144	5.82E-05	2.84E+02	2.44E-03	3.00E-06	1.53E-08	5.10E-03
Pr-144	5.82E-05	1.20E-02	5.78E+01	6.00E-04	0.00E+00	0.00E+00

[1] Values from Table 2.4.13-1.

[2] Values from NUREG/CR-5512, Table E.1 **(Kennedy and Strenge 1992)**, and U. S. Department of Health Radiological Health Handbook **(USDOH 1970)** for Sr-92, Rh-106, and Ba-137m.

[3] Values calculated from Equation 2.4.13-4.

[4] Maximum Permissible Concentrations (MPCs) from 10 CFR Part 20, Appendix B, Table 2, Column 2

[5] Values calculated from Equation 2.4.13-5 for a travel time of 9.25 years.

[6] Maximum Permissible Concentration (MPC) is not available.

Table 2.4.13-3 Results of K_d Analysis

Soil Sample	K_d Value (mL/g)		
	Co	Sr	Cs
Samples From Potential Borrow Sources Areas			
A-10(a)	8.1	13.2	56.2
C-7	3.9	9.0	14.8
D-10	1.7	7.8	9.9
E-7	10.1	25.7	19.9
E-12	15.3	51.7	10.7
G-9	7.9	9.8	> 25.5
J-11	13.5	9.2	> 47.4
K-10	15.2	10.0	19.3
L-7	1.7	11.4	18.8
M-5	7.3	9.3	16.8
N-3	5.8	10.7	7.8
P-8	6.5	7.0	5.3
Q-7	3.2	9.3	14.6
H-6	1.4	6.0	3.5
S-9	3.0	8.6	19.3
R-8	2.1	10.5	13.5
Samples From Barnwell Formation (Utley Limestone)			
B-1003V-55-65	10.9	17.4	> 30.1
B-1003V-65-75	3.9	15.0	22.7
B-1003V-75-82	21.3	14.4	33.2

Source: Kaplan and Millings 2006

Table 2.4.13-4 Results of Transport Analysis Considering Radioactive Decay and Adsorption

Radionuclide	Backfill			Utley Limestone			Total Travel Time[3] (years)	Groundwater Concentration ($\mu Ci/cm^3$)	Groundwater Concentration / MPC
	Distribution Coefficient (cm^3/g)	Retardation Factor[1]	Travel Time[2] (years)	Distribution Coefficient (cm^3/g)	Retardation Factor[1]	Travel Time[2] (years)			
H-3	0.0	1.0	9.16	0.0	1.0	0.09	9.25	6.01E-01	6.01E+02
Mn-54	0.0	1.0	9.16	0.0	1.0	0.09	9.25	3.82E-07	1.27E-02
Fe-55	0.0	1.0	9.16	0.0	1.0	0.09	9.25	4.70E-05	4.70E-01
Co-60	1.4	7.6	69.48	3.9	63.4	5.75	75.24	1.15E-08	3.83E-03
Sr-90	6.0	29.2	267.70	14.4	231.4	21.00	288.71	2.41E-08	4.82E-02
I-129	0.0	1.0	9.16	0.0	1.0	0.09	9.25	7.27E-09	3.63E-02
Cs-134	3.5	17.5	159.98	22.7	364.2	33.06	193.03	2.18E-29	2.42E-23
Cs-137	3.5	17.5	159.98	22.7	364.2	33.06	193.03	2.85E-03	2.85E+03

[1] Values calculated from Equation 2.4.13-2 using a bulk density of 1.60 g/cm^3 and effective porosities of 0.34 and 0.10 for the backfill and Utley limestone, respectively.

[2] Travel time calculated as the product of the retardation factor and groundwater travel time (9.16 years for backfill and 0.09 years for Utley limestone).

[3] Total travel time calculated as the sum of backfill and Utley limestone travel times.

[4] Groundwater concentration calculated from Equation 2.4.13-5 using total travel time.

Table 2.4.13-5 Results of Transport Analysis Considering Radioactive Decay, Adsorption, and Dilution

Radionuclide	Groundwater Concentration[1] (μCi/cm^3)	Surface Water Concentration[2] (μCi/cm^3)	Surface Water Concentration / MPC
H-3	6.01E-01	5.45E-05	5.45E-02
Mn-54	3.82E-07	3.46E-11	1.15E-06
Fe-55	4.70E-05	4.26E-09	4.26E-05
Sr-90	2.41E-08	2.18E-12	4.37E-06
I-129	7.27E-09	6.59E-13	3.29E-06
Cs-137	2.85E-03	2.58E-07	2.58E-01

[1] Values from Table 2.4.13-4.

[2] Surface water concentrations calculated as the product of the groundwater concentration and the dilution factor (9.1E-05).

Table 2.4.13-6 Compliance with 10 CFR Part 20

Radionuclide	Concentration / MPC			
	Decay[1]	Decay and Adsorption[2]	Decay, Adsorption, and Dilution[3]	Minimum
H-3	6.01E+02	6.01E+02	5.45E-02	5.45E-02
Cr-51	5.14E-37			5.14E-37
Mn-54	1.27E-02	1.27E-02	1.15E-06	1.15E-06
Mn-56	0.00E+00			0.00E+00
Fe-55	4.70E-01	4.70E-01	4.26E-05	4.26E-05
Fe-59	1.85E-22			1.85E-22
Co-58	4.18E-13			4.18E-13
Co-60	2.20E+01	3.83E-03		3.83E-03
Br-83	0.00E+00			0.00E+00
Br-84	0.00E+00			0.00E+00
Br-85	0.00E+00			0.00E+00
Rb-88	0.00E+00			0.00E+00
Rb-89	0.00E+00			0.00E+00
Sr-89	4.89E-19			4.89E-19
Sr-90	3.82E+01	4.82E-02	4.37E-06	4.37E-06
Sr-91	0.00E+00			0.00E+00
Sr-92	0.00E+00			0.00E+00
Y-90	0.00E+00			0.00E+00
Y-91m	0.00E+00			0.00E+00
Y-91	3.53E-17			3.53E-17
Y-92	0.00E+00			0.00E+00
Y-93	0.00E+00			0.00E+00
Nb-95	3.36E-29			3.36E-29
Zr-95	5.03E-16			5.03E-16
Mo-99	0.00E+00			0.00E+00
Tc-99m	0.00E+00			0.00E+00
Ru-103	3.04E-26			3.04E-26
Rh-103m	0.00E+00			0.00E+00
Rh-106[4]	0.00E+00			0.00E+00
Ag-110m	2.77E-03			2.77E-03
Te-127m	1.92E-08			1.92E-08
Te-129m	9.85E-29			9.85E-29
Te-129	0.00E+00			0.00E+00
Te-131m	0.00E+00			0.00E+00
Te-131	0.00E+00			0.00E+00
Te-132	0.00E+00			0.00E+00
Te-134	0.00E+00			0.00E+00
I-129	3.63E-02	3.63E-02	3.29E-06	3.29E-06
I-130	0.00E+00			0.00E+00
I-131	1.17E-121			1.17E-121
I-132	0.00E+00			0.00E+00
I-133	0.00E+00			0.00E+00
I-134	0.00E+00			0.00E+00

Radionuclide	Concentration / MPC			
	Decay[1]	Decay and Adsorption[2]	Decay, Adsorption, and Dilution[3]	Minimum
I-135	0.00E+00			0.00E+00
Cs-134	1.66E+04	2.42E-23		2.42E-23
Cs-136	1.95E-73			1.95E-73
Cs-137	1.96E+05	2.85E+03	2.58E-01	2.58E-01
Cs-138	0.00E+00			0.00E+00
Ba-137m[4]	0.00E+00			0.00E+00
Ba-140	5.25E-79			5.25E-79
La-140	0.00E+00			0.00E+00
Ce-141	1.34E-31			1.34E-31
Ce-143	0.00E+00			0.00E+00
Pr-143	6.26E-75			6.26E-75
Ce-144	5.10E-03			5.10E-03
Pr-144	0.00E+00			0.00E+00
			Sum of Ratios =	0.32

[1] Table 2.4.13-2.

[2] Table 2.4.13-4.

[3] Table 2.4.13-5.

[4] No MPCs are published for Rh-106 and Ba-137m. However, the half-lives for these radionuclides are short (less than one day) and they decay to near zero values. Their ratios have been taken as zero.

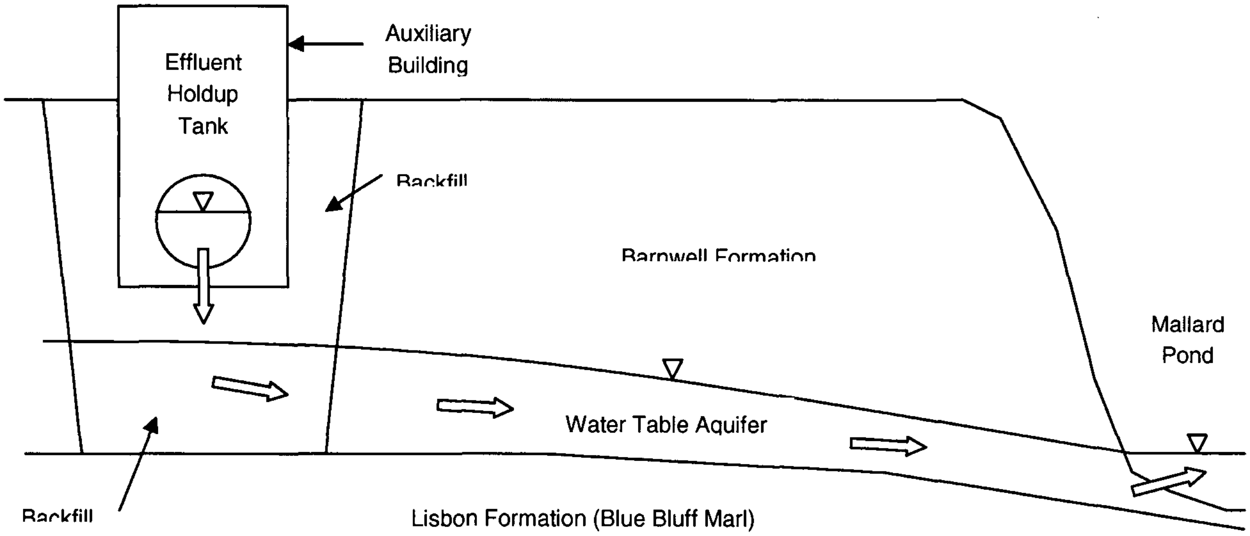


Figure 2.4.13-1 Conceptual Model for Evaluating Radionuclide Transport in Groundwater

Section 2.4.13 References

(Bechtel 1985) Bechtel Corporation, Flow Rate in Mathes Pond Stream & West Branch Stream, Calculation Number G-008, Vogtle Nuclear Power Plant, Job No. 9510-091, 1985.

(Heath 1998) Heath, R. C., Basic Ground-Water Hydrology, U. S. Geological Survey Water-Supply Paper 2220, 1998.

(Javandel et al. 1984) Javandel, I., Doughty, C. and Tsang, C-F, *Groundwater Transport: Handbook of Mathematical Models, Water Resources Monograph 10*, American Geophysical Union, 1984.

(Kaplan and Millings 2006) Kaplan, D. I., and Millings, M. R., Distribution Coefficients for the Vogtle Early Site Permit, WSRC-TR-2006-00246, Savannah River National Laboratory, Washington Savannah River Company, Aiken, South Carolina, July 2006.

(Kennedy and Strenge 1992) Kennedy, W. E., and Strenge, D. L., NUREG/CR-5512, *Residual Radioactive Contamination From Decommissioning*, Volume 1, Pacific Northwest Laboratory, October 1992.

(MACTEC 2006) Report of Soil and Groundwater Sampling and Laboratory Testing, Southern Advanced Light Water Reactor, Early Site Permit, Vogtle Electric Generating Plant, Burke County, Georgia, MACTEC Project No. 6141-06-0090, MACTEC Engineering and Consulting, Inc., June 2006.

(SNC 2003) Southern Nuclear Operating Company, Updated Final Safety Analysis Report, Vogtle Electric Generating Plant, Revision 11, May 2003.

(USDOH 1970) U. S. Department of Health, Education, and Welfare, Radiological Health Handbook, January 1970.

(Westinghouse 2006) AP1000 Calculation Note Number APP-WLS-M3C-021, Liquid Tank Rupture Evaluation (COL Item 15.7.6), Revision 0, Westinghouse Electric Company, LLC, 2006.

Southern Nuclear Operating Company

AR-07-0639

Enclosure 3

Requested Reference Documents

NOTE: The following documents are contained on Enclosure 3:

1. Bechtel Power Corporation and Georgia Power Company, 1980, Final Report on Dewatering and Repair of Erosion in Category I Backfill in Power Block Area, Vogtle 5.16

2. Bechtel Power Corporation, 1972, Aquifer Tests for Construction Dewatering, Vogtle 8.7.1

3. VEGP Bechtel Calculation G-008, September 27, 1985

AR-07-0639
Enclosure 3
RAI Response

1. Bechtel Power Corporation and Georgia Power Company, 1980, Final Report on Dewatering and Repair of Erosion in Category I Backfill in Power Block Area, Vogtle 5.16

NOTE: This document is 137-pages.

VOGTLE 9510

Dewatering and Repair of Erosion in Category I Backfill in Power Block Area

FINAL REPORT

August 15, 1980

VOGTLE 9510
5.16 Final Report on Dewatering & Repair of Erosion in Category I Backfill in Power Block Area

PLU

GGR

Vogtle Project

August 15, 1980

attachment filed X1BG03-C3

Mr. Jim Bailey
Southern Company Services
P. O. Box 2625
Birmingham, Alabama 35202

Subject: Plant Vogtle - Units 1 & 2
Backfill Erosion Report

File No. X2AP01 X1BG03-C3

Correspondence No. ACPM-G-3

Dear Jim,

Please find attached "Final Report on Dewatering and Repair of Erosion in Category I Backfill in Power Block Area" for your use during submittals to the NRC as required.

Sincerely,

H. H. Gregory, III

H. H. Gregory, III
Assistant Construction
Project Manager

HHG/mfk

Attachment

xc: D. E. Dutton w/a
B. L. Lex w/a
J. E. Mahlmeister w/o
K. M. Gillespie w/o
W. M. Johnston, Jr. w/o

PLANT VOGTLE UNITS 1 AND 2

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

Prepared By

BECHTEL POWER CORPORATION

and

GEORGIA POWER COMPANY

Date: August 15, 1980

SUBMITTED AND APPROVED

TABLE OF CONTENTS

TABLE OF CONTENTS, continued

APPENDIX

LIST OF FIGURES

LIST OF FIGURES, continued

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
- [ ] PREPARE REPLY FOR MY SIGNATURE
- [ ] NOTE AND RETURN TO ME
- [ ] TAKE APPROPRIATE ACTION
- [ ] RETURN WITH MORE DETAILS
- [ ] PER YOUR REQUEST
- [ ] NOTE AND SEE ME ABOUT THIS
- [ ] SIGNATURE
- [ ] PLEASE ANSWER
- [ ] FOR YOUR INFORMATION
- [ ] FOR YOUR APPROVAL
- [ ] 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. <u>Electrical Tunnel Unit 2</u>

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. <u>Containment Unit 2 - Partial Tendon Gallery</u>

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. Auxiliary Building and NSCW Towers

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 of 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 corrected (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. Dewatering Systems

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'±). 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'±, 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.

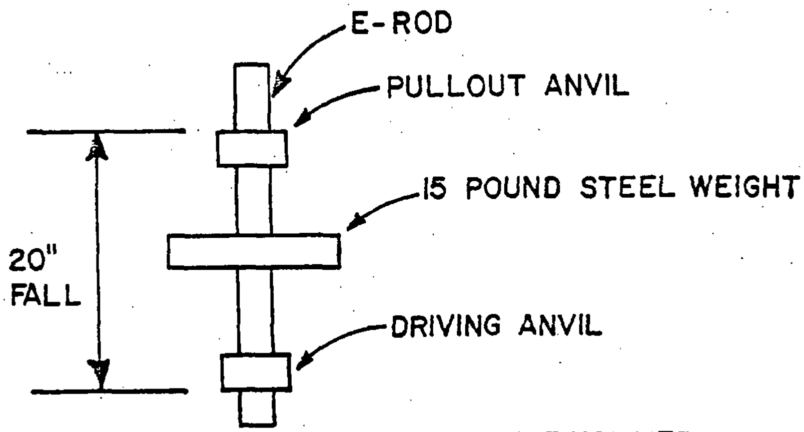
A sketch showing the Shelby tube sampler used in sample extraction is included in the Appendix. A 3-inch diameter, 30-inch long Shelby tube was attached to a 2-foot length of pipe by means of a heavy adaptor. The driving head was then screwed into the pipe. A flat plate was welded on top of the driving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

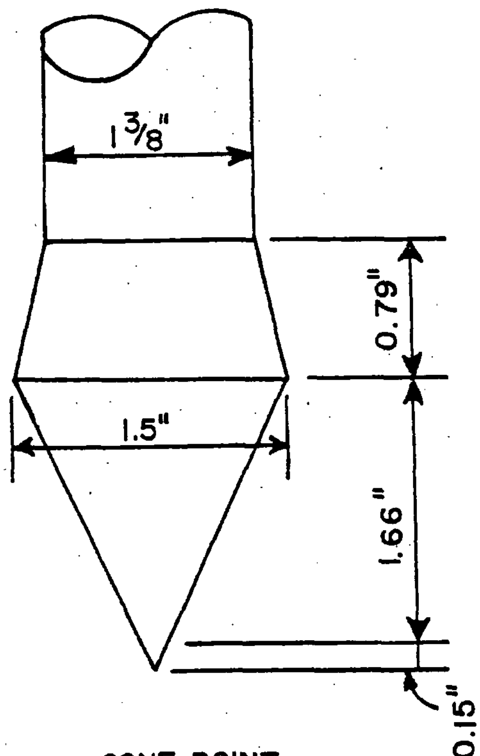
The volume of the sample inside the tube was determined by first measuring the distances inside the tube from the top of the sample to the top of the tube and the bottom of the sample from the bottom of the tube. These distances were subtracted from the total length of the tube sampler and then multiplied 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.

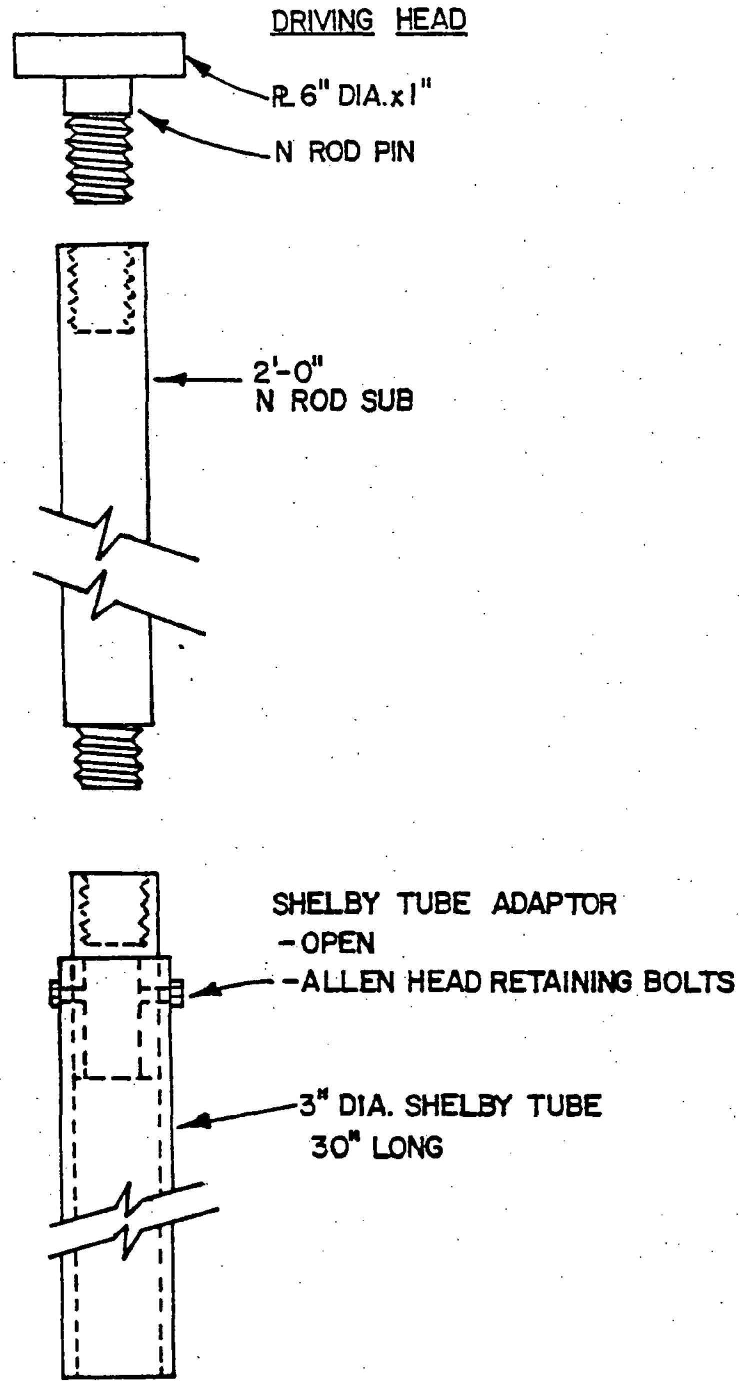


SLIDING DRIVE HAMMER

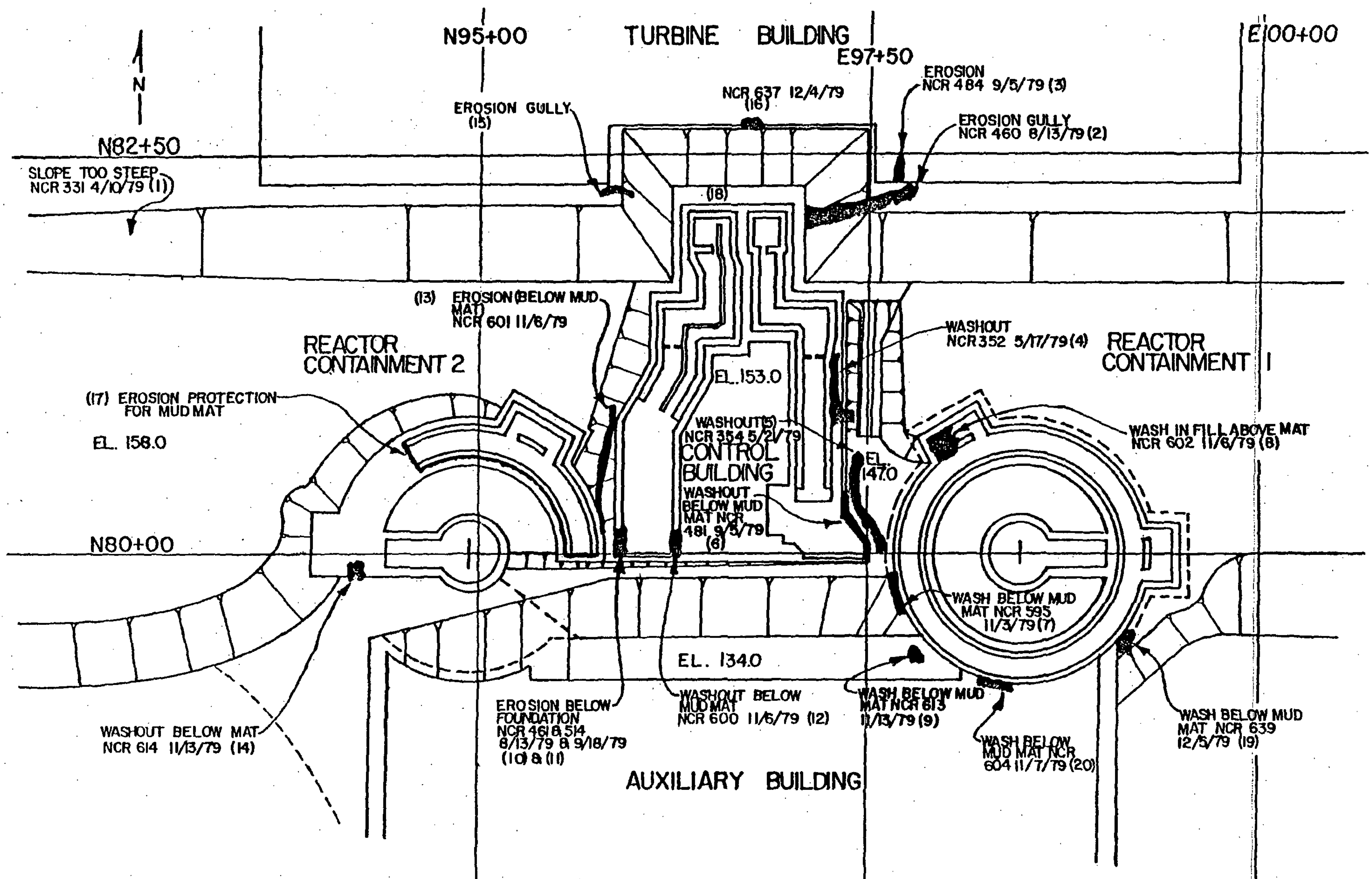


CONE POINT

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



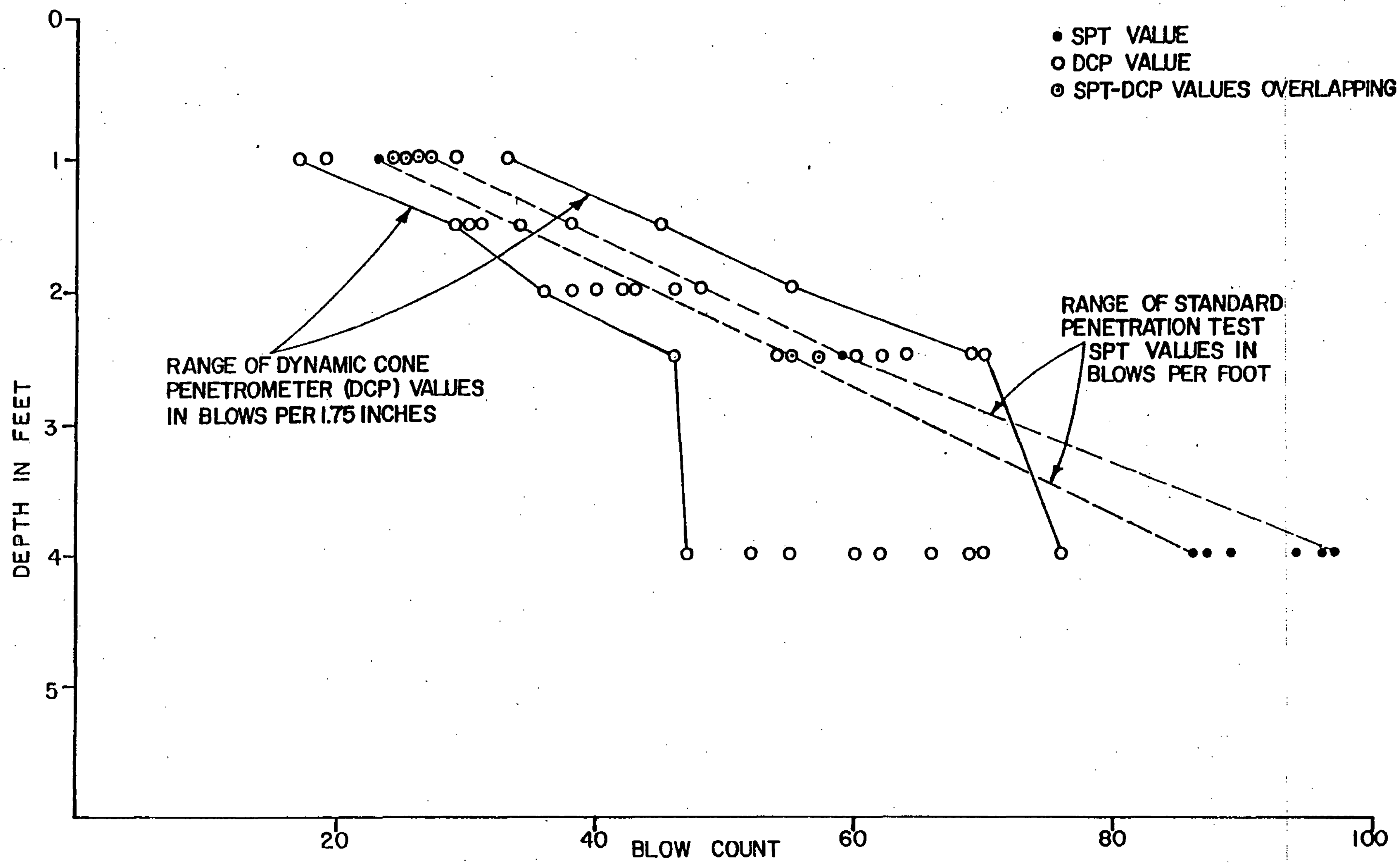
GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
SHELBY TUBE SAMPLER		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	A - 2	



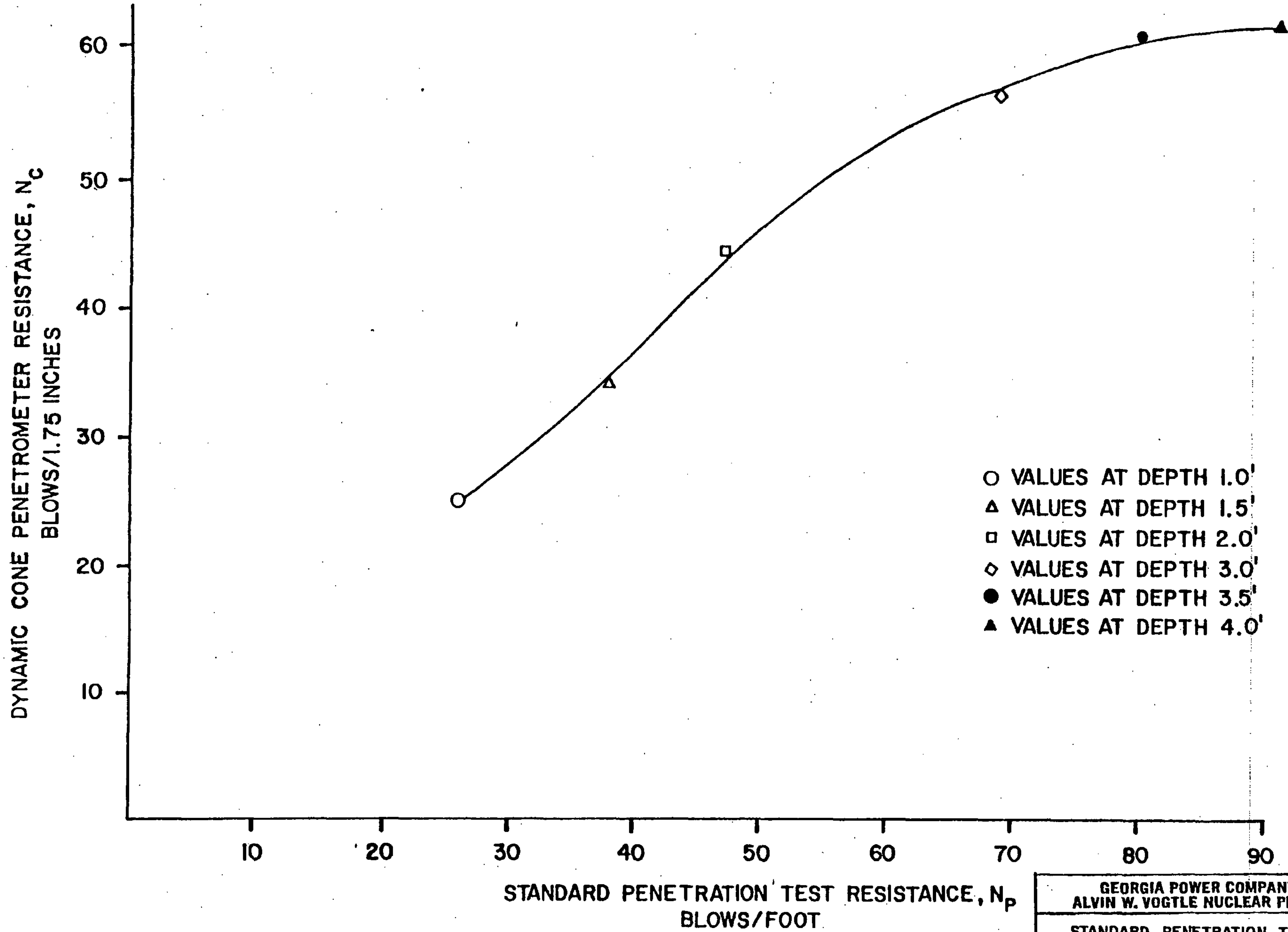
GEORGIA POWER COMPANY
ALVIN W. VOGTLE NUCLEAR PLANT

PLAN OF POWER BLOCK SHOWING LOCATIONS OF ERODED AREAS

SCALE: 1" = 80'	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 1	



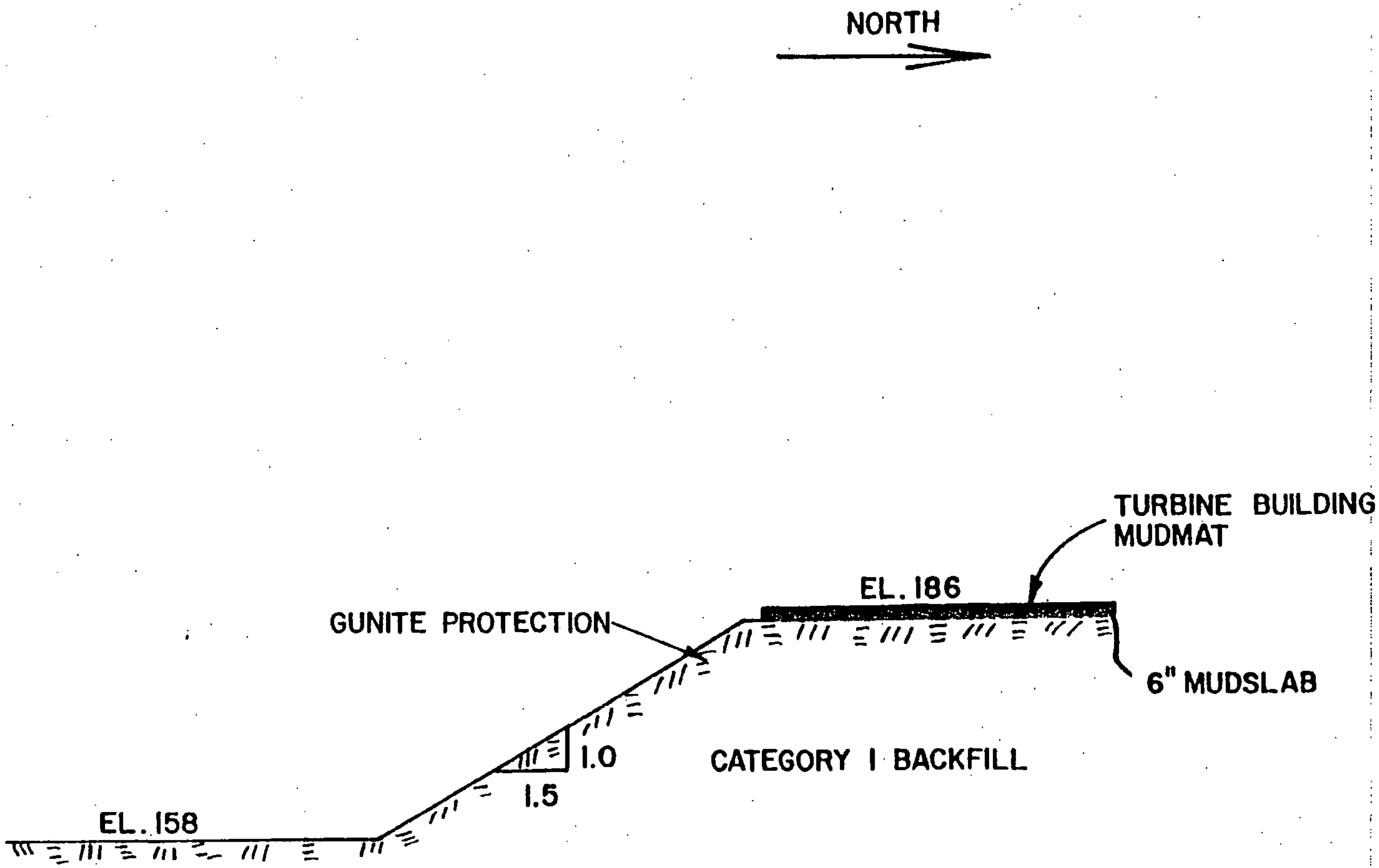
GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
STANDARD PENETRATION TEST AND DYNAMIC CONE PENETROMETER TEST BLOWCOUNTS VS. DEPTH		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 2	



GEORGIA POWER COMPANY
ALVIN W. VOGTLE NUCLEAR PLANT

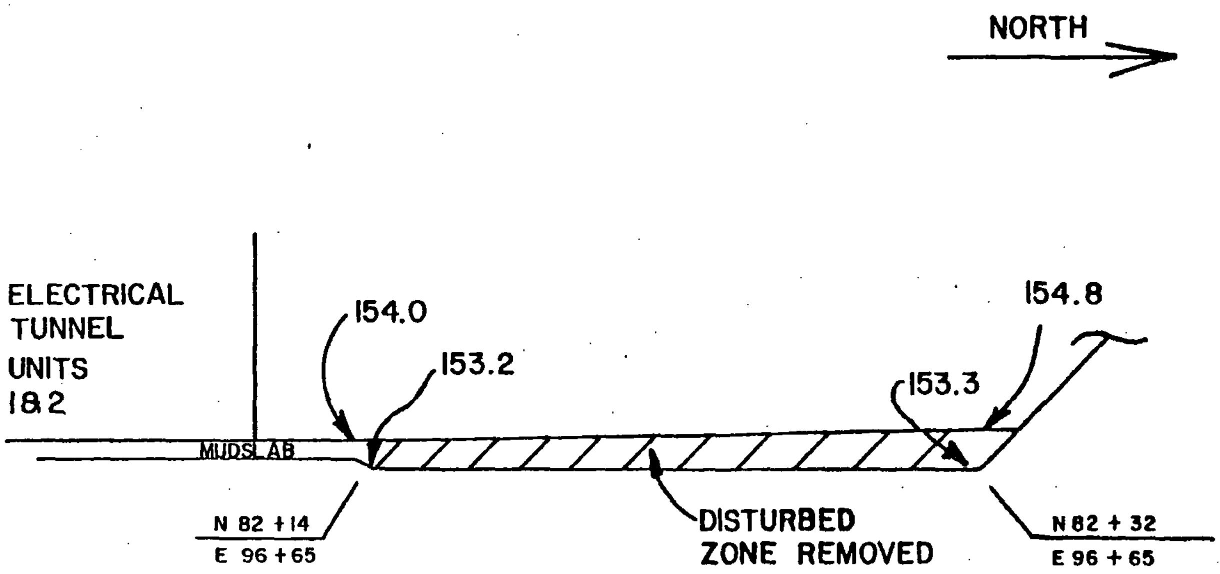
STANDARD PENETRATION TEST AND DYNAMIC CONE PENETROMETER TEST CALIBRATION CURVE

SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 3	



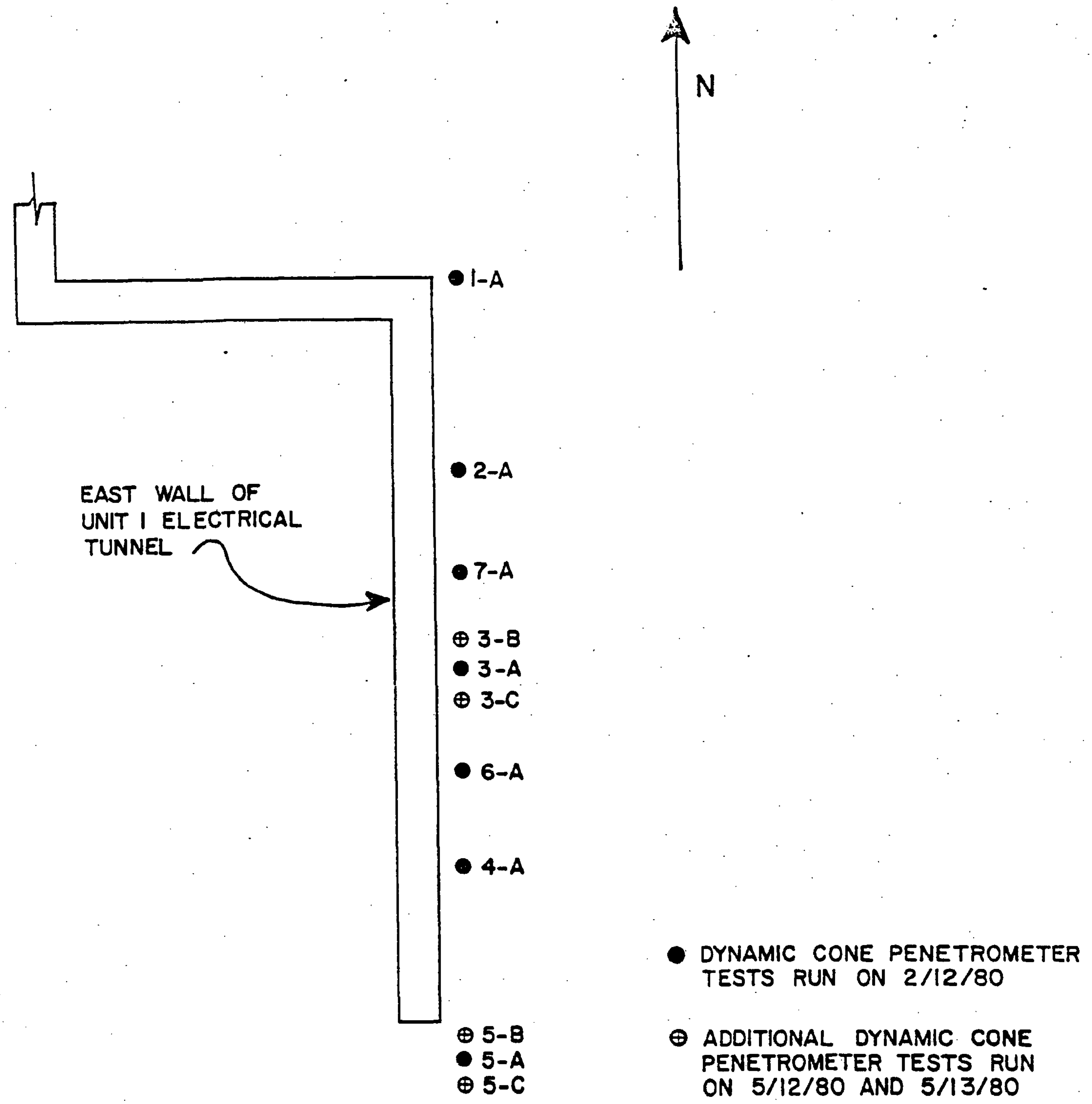
NOT TO SCALE

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	



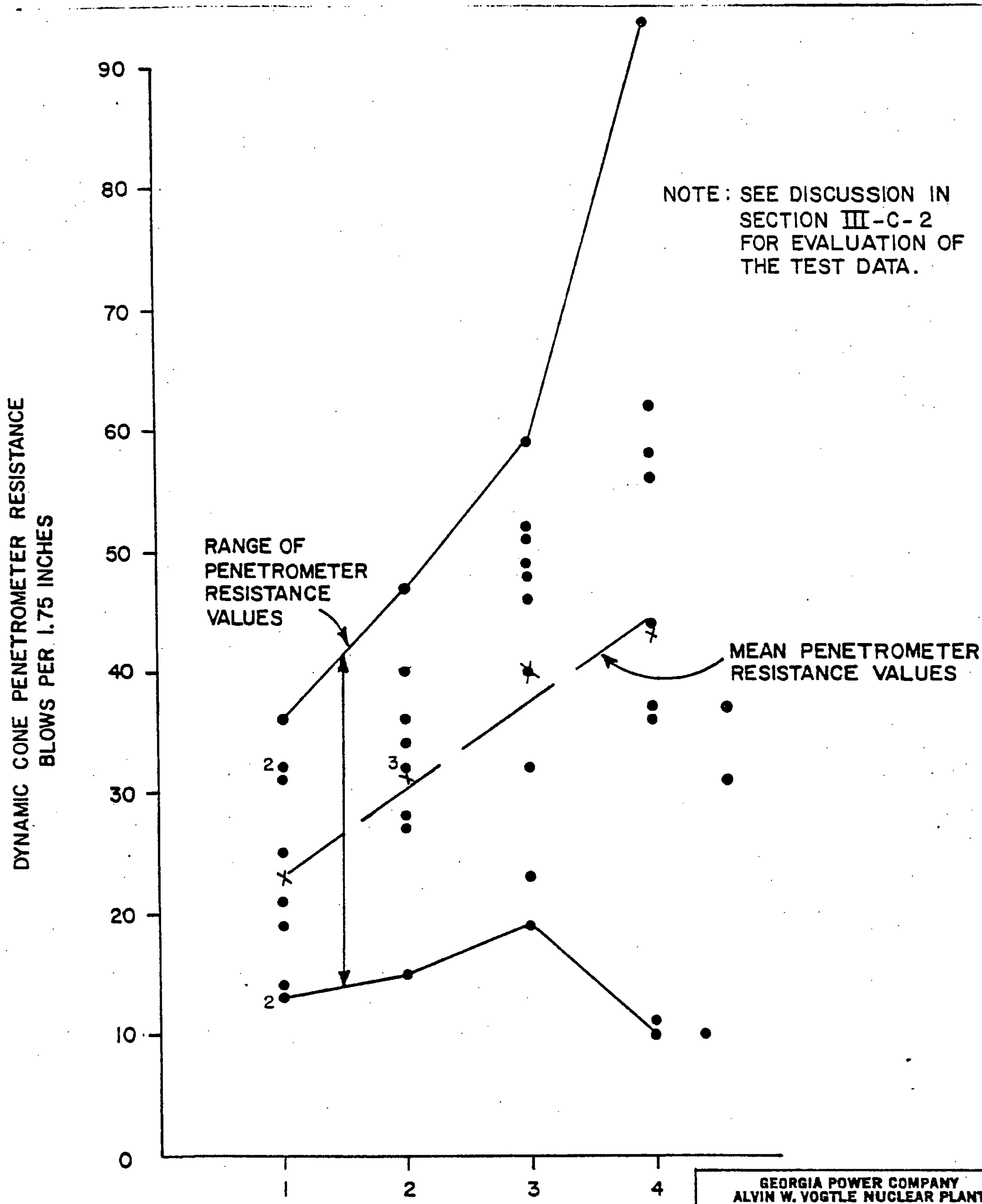
NOT TO SCALE

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	



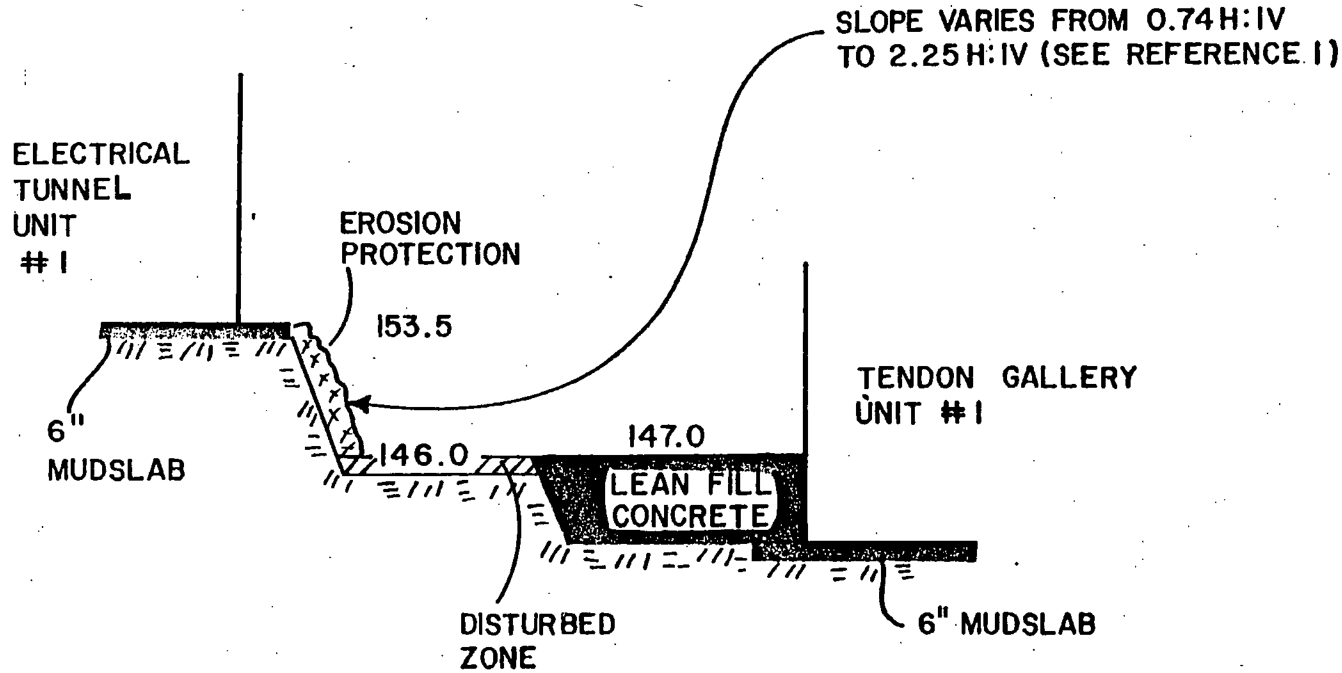
PLAN SHOWING LOCATIONS OF DYNAMIC CONE PENETROMETER TESTS ADJACENT TO ELECTRICAL TUNNEL EAST WALL.

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

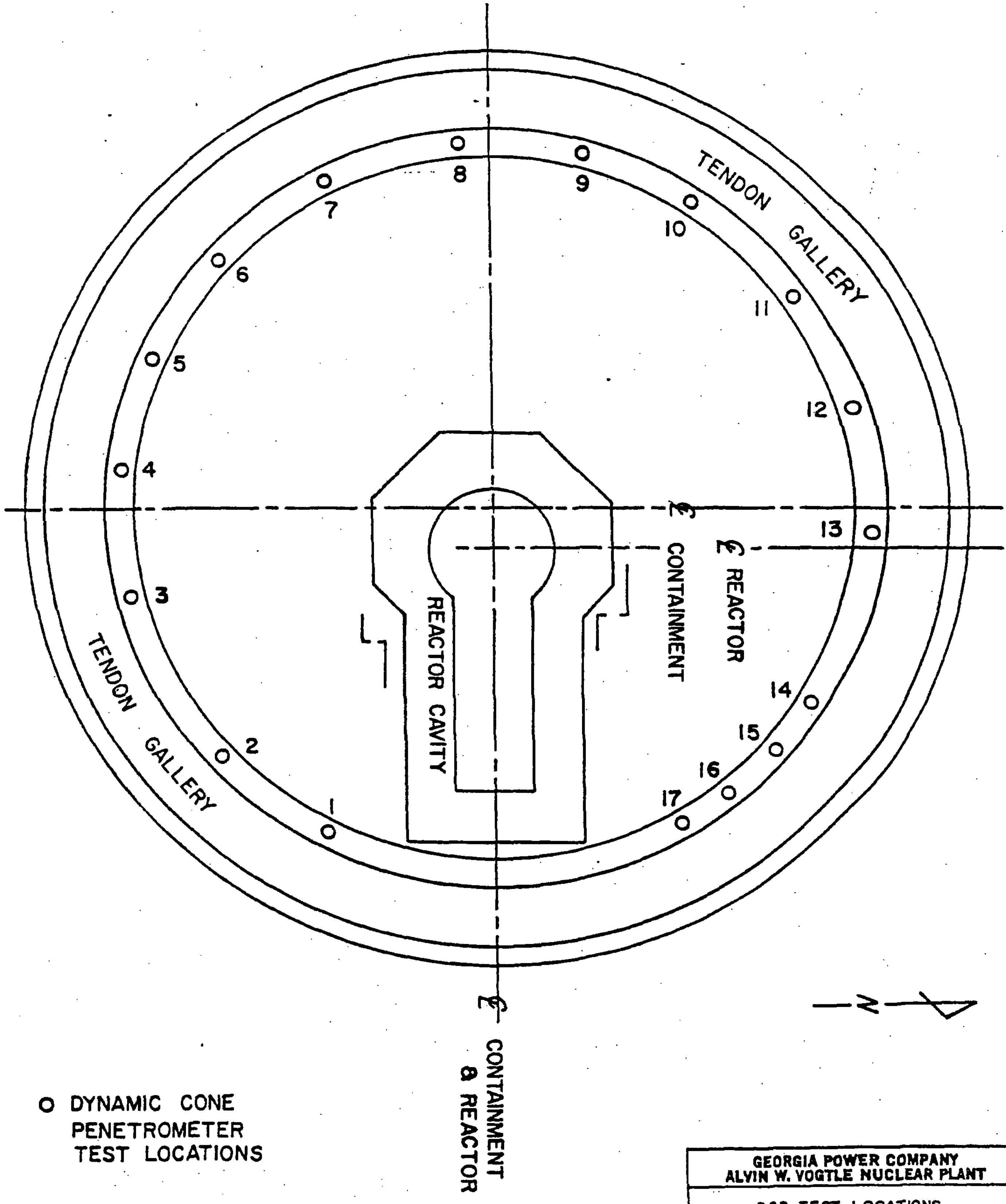
GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
ELECTRICAL TUNNEL EAST WALL DCP RESISTANCE VERSUS DEPTH		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 7	



NOT TO SCALE

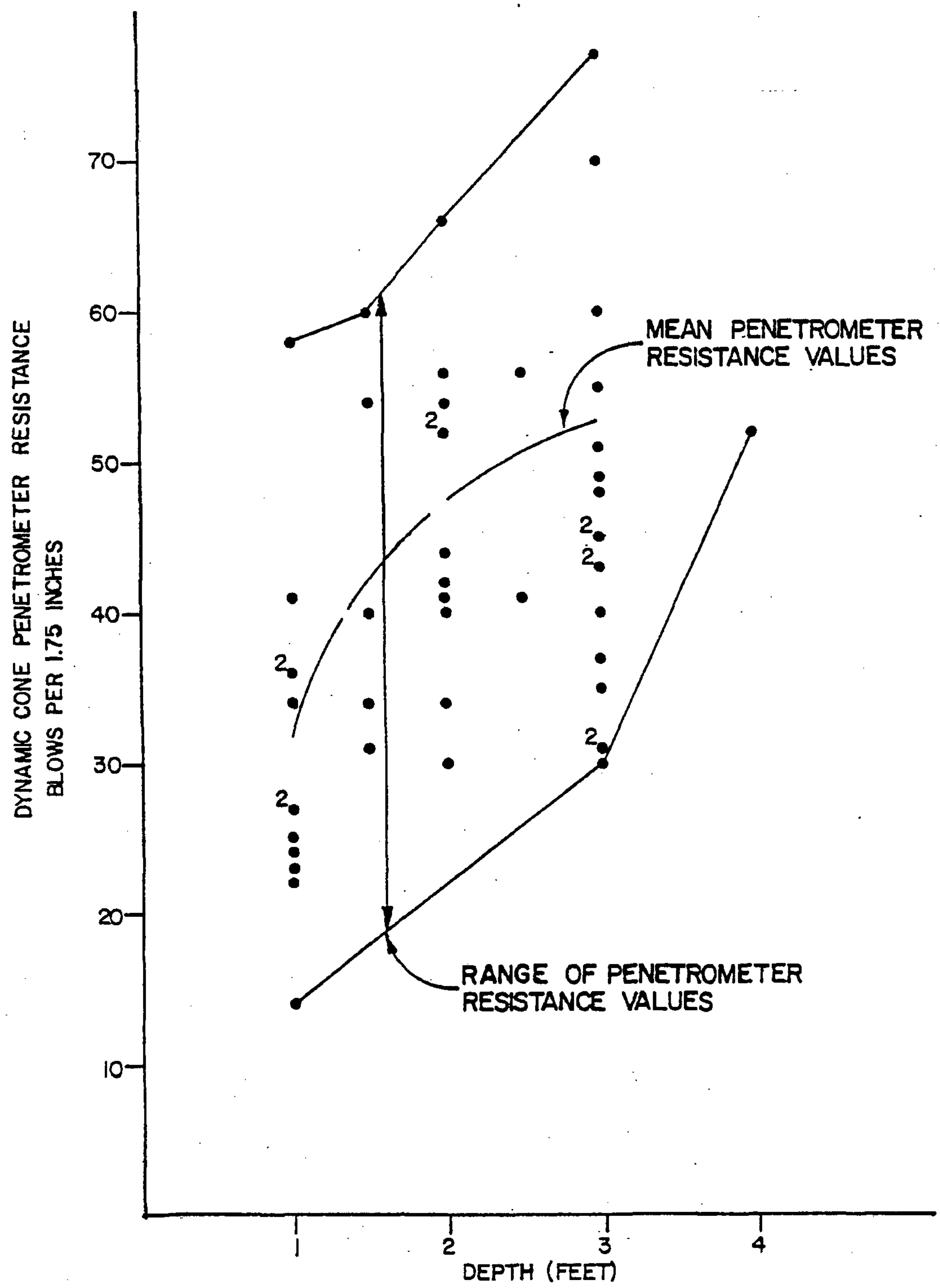
TYPICAL SECTION SHOWING EXTENT OF DISTURBED ZONE REMOVED IN AREA BETWEEN UNIT I ELECTRICAL TUNNEL AND CONTAINMENT.

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	

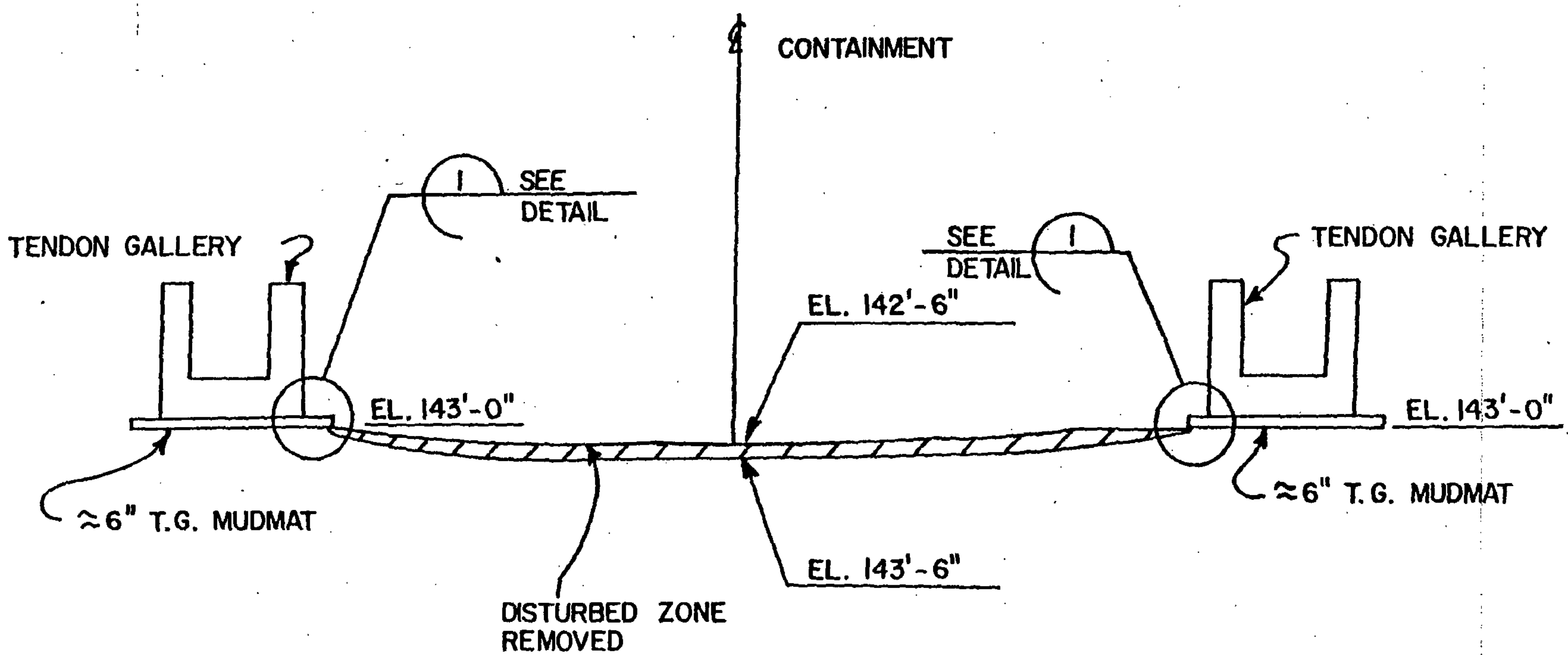


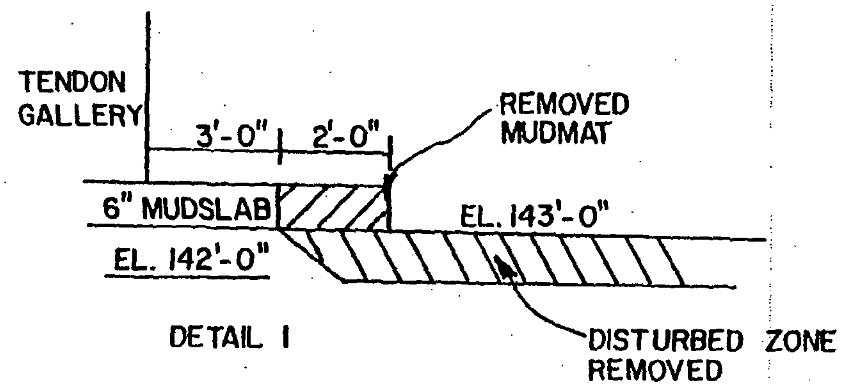
O DYNAMIC CONE PENETROMETER TEST LOCATIONS

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	

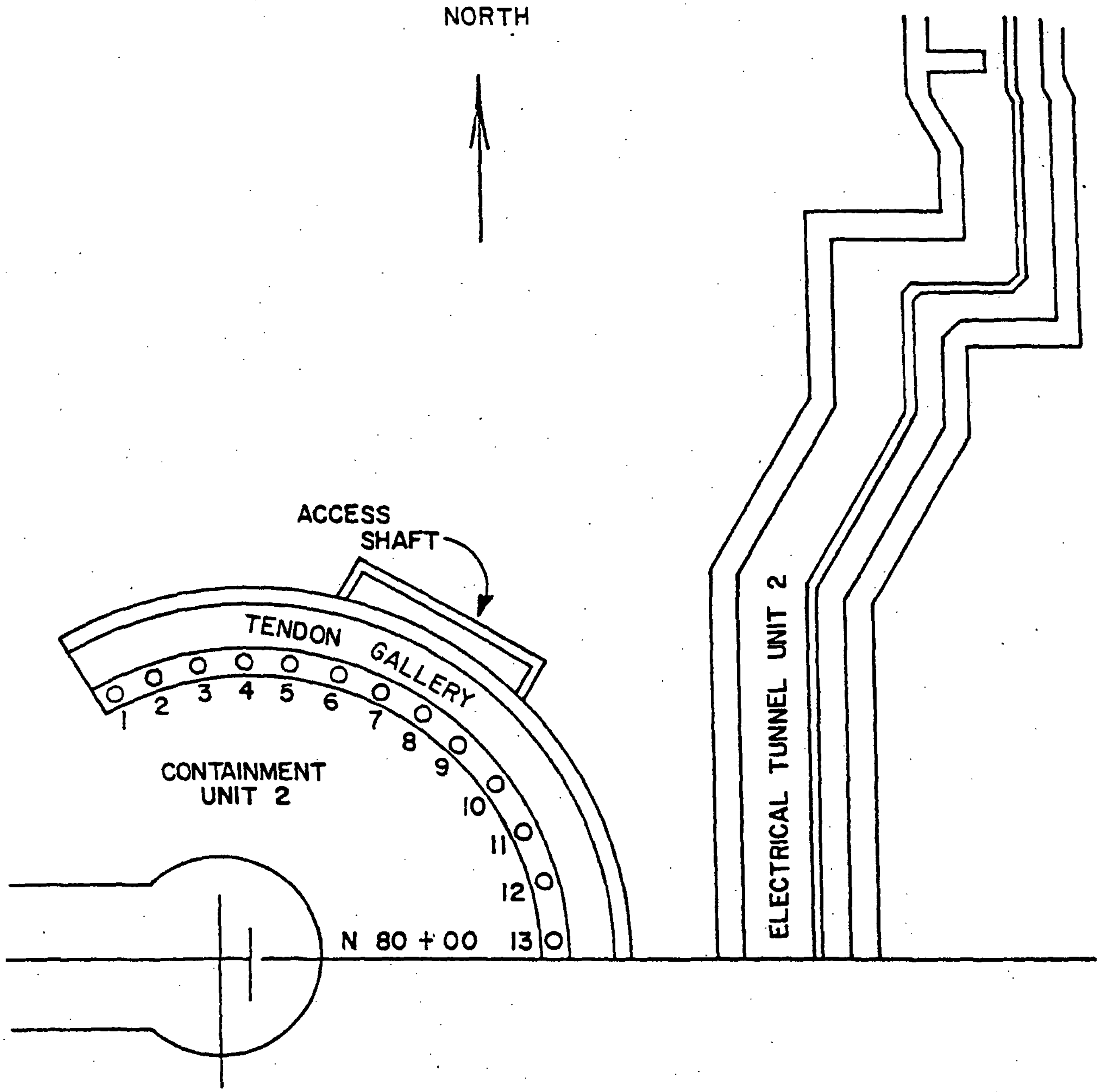




DETAIL I

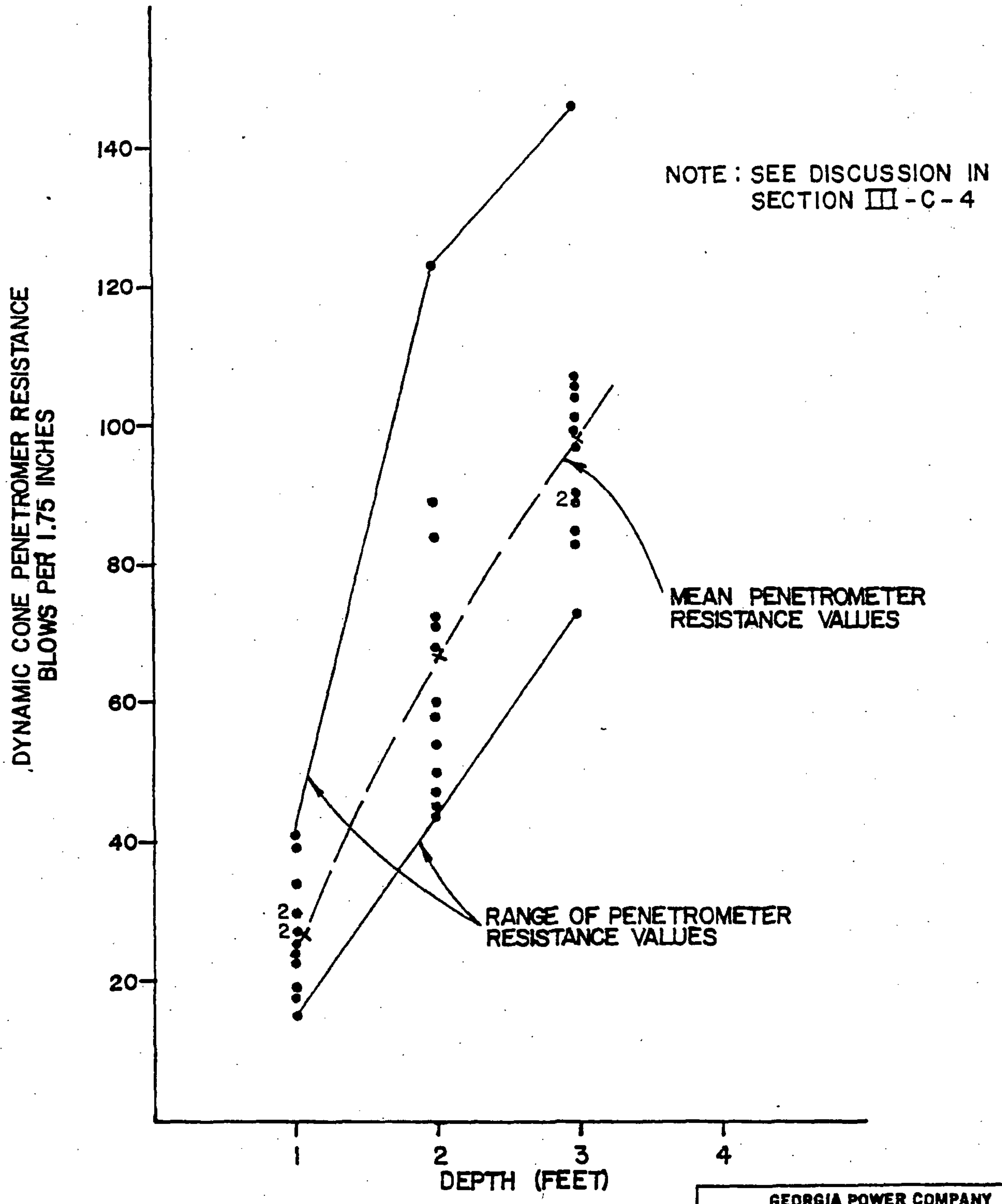
NOT TO SCALE

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

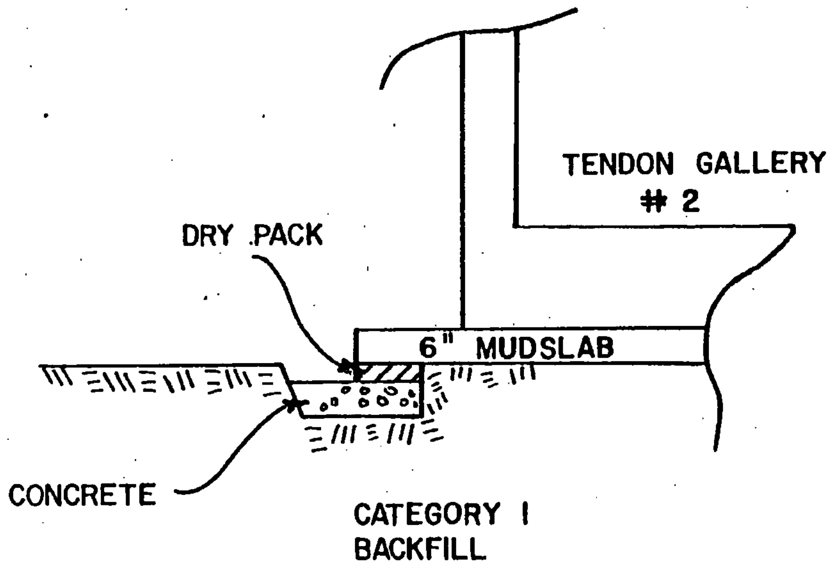


O DYNAMIC CONE PENETROMETER TEST LOCATIONS

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	

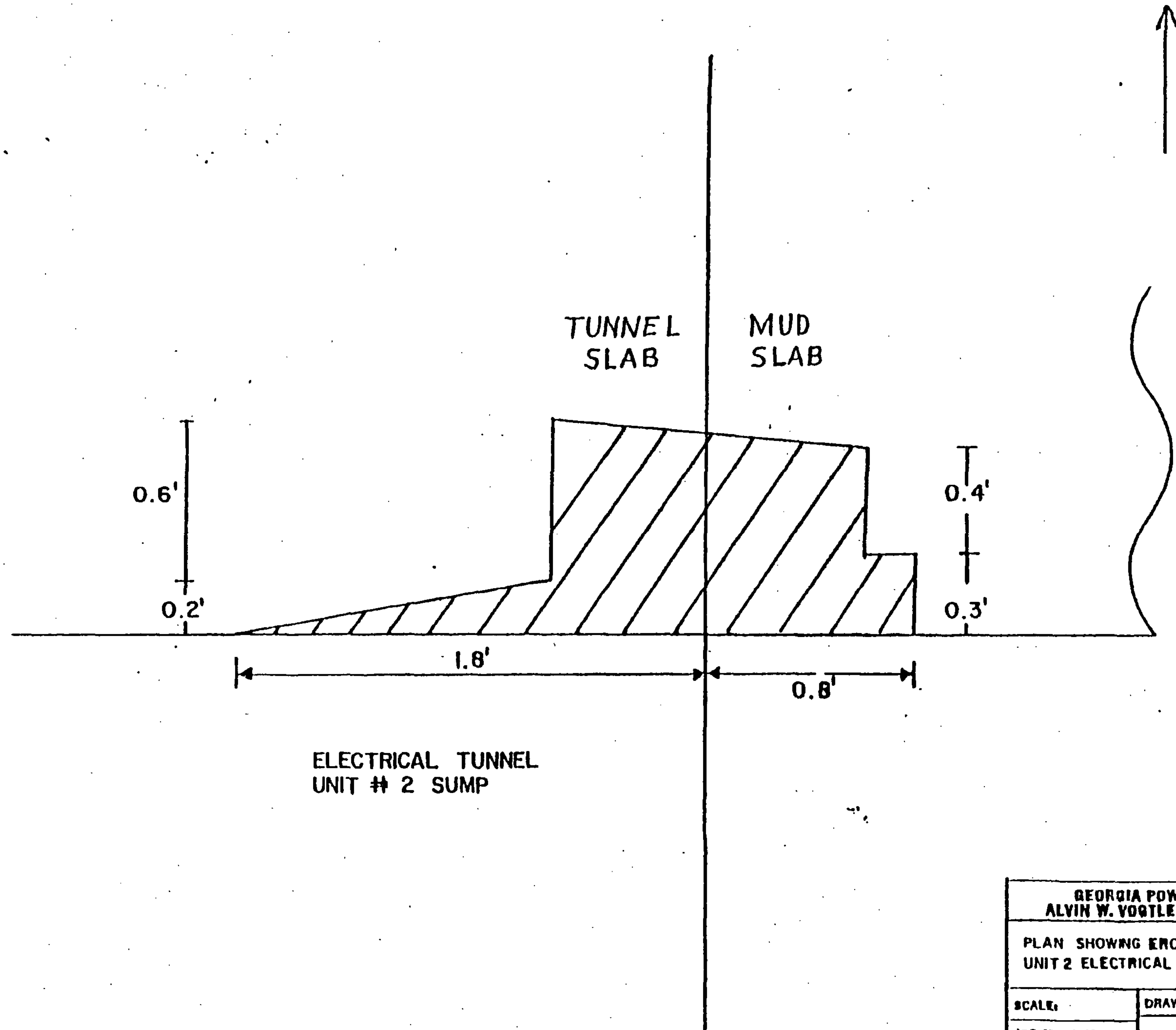


GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
DYNAMIC CONE PENETROMETER RESISTANCE VERSUS DEPTH FOR UNIT 2 TENDON GALLERY		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 13	

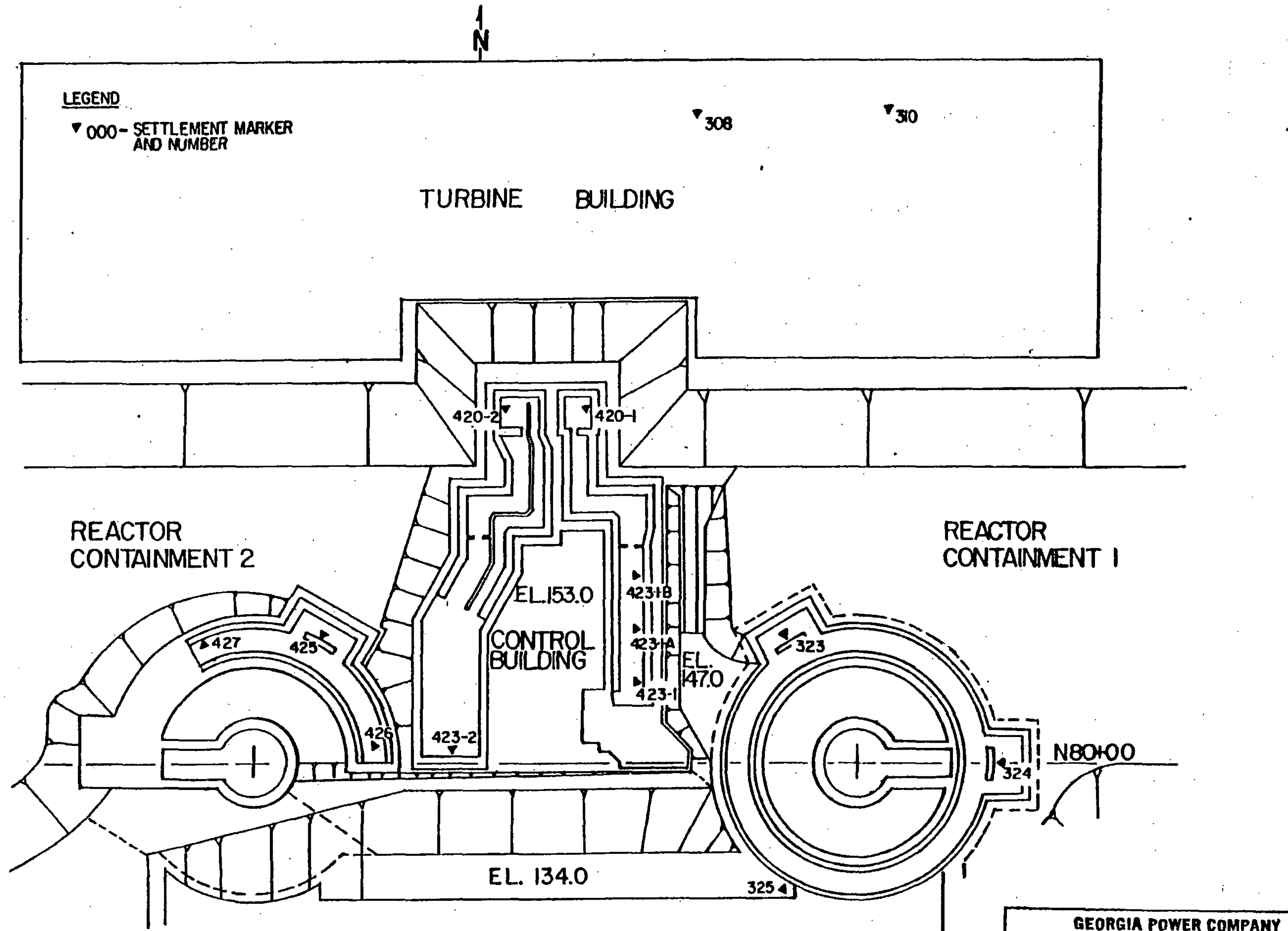


NOT TO SCALE

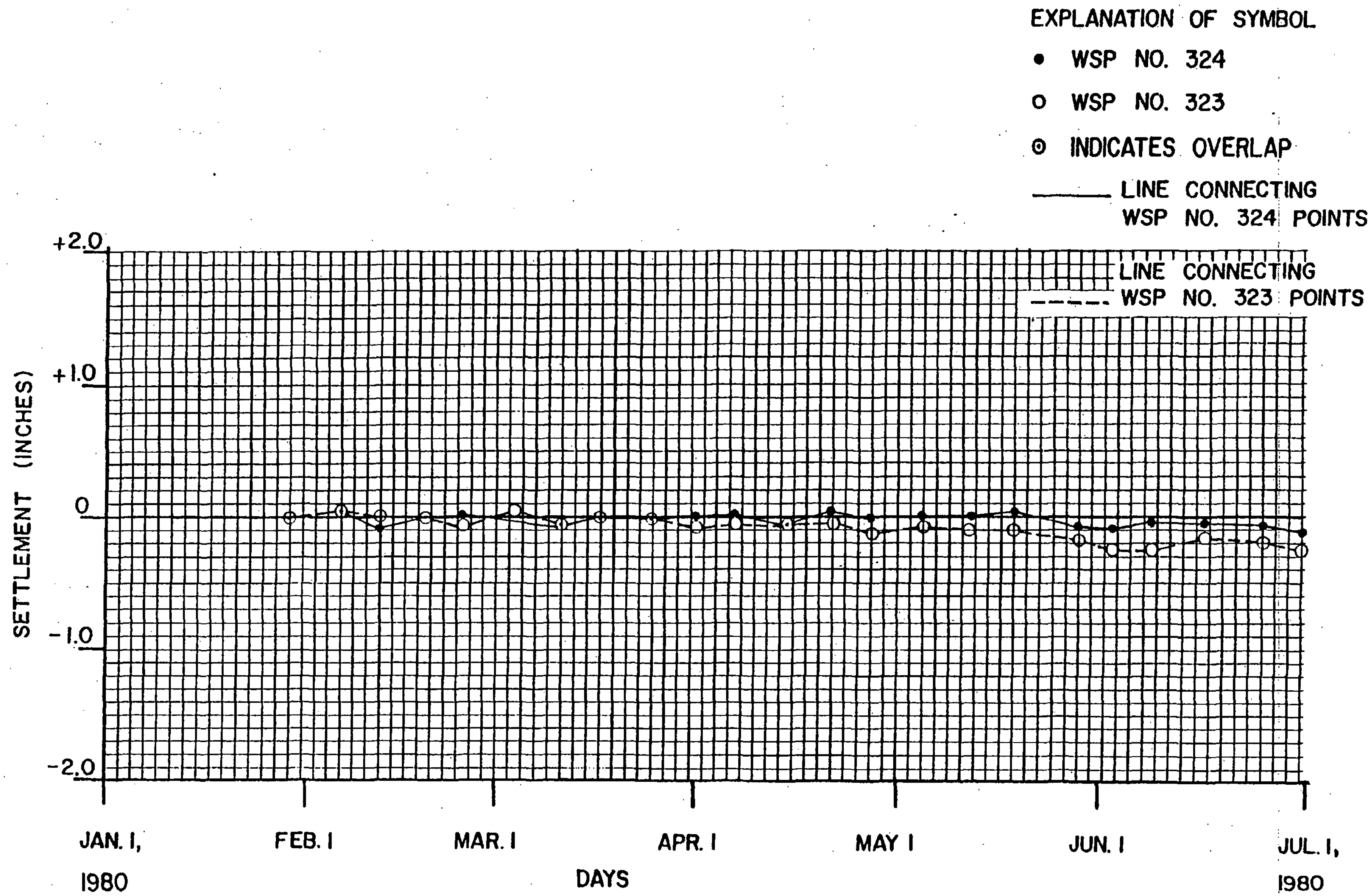
GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
SKETCH SHOWING PROCEDURE USED TO REPAIR UNIT 2 TENDON GALLERY MUDSLAB		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 14	



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	



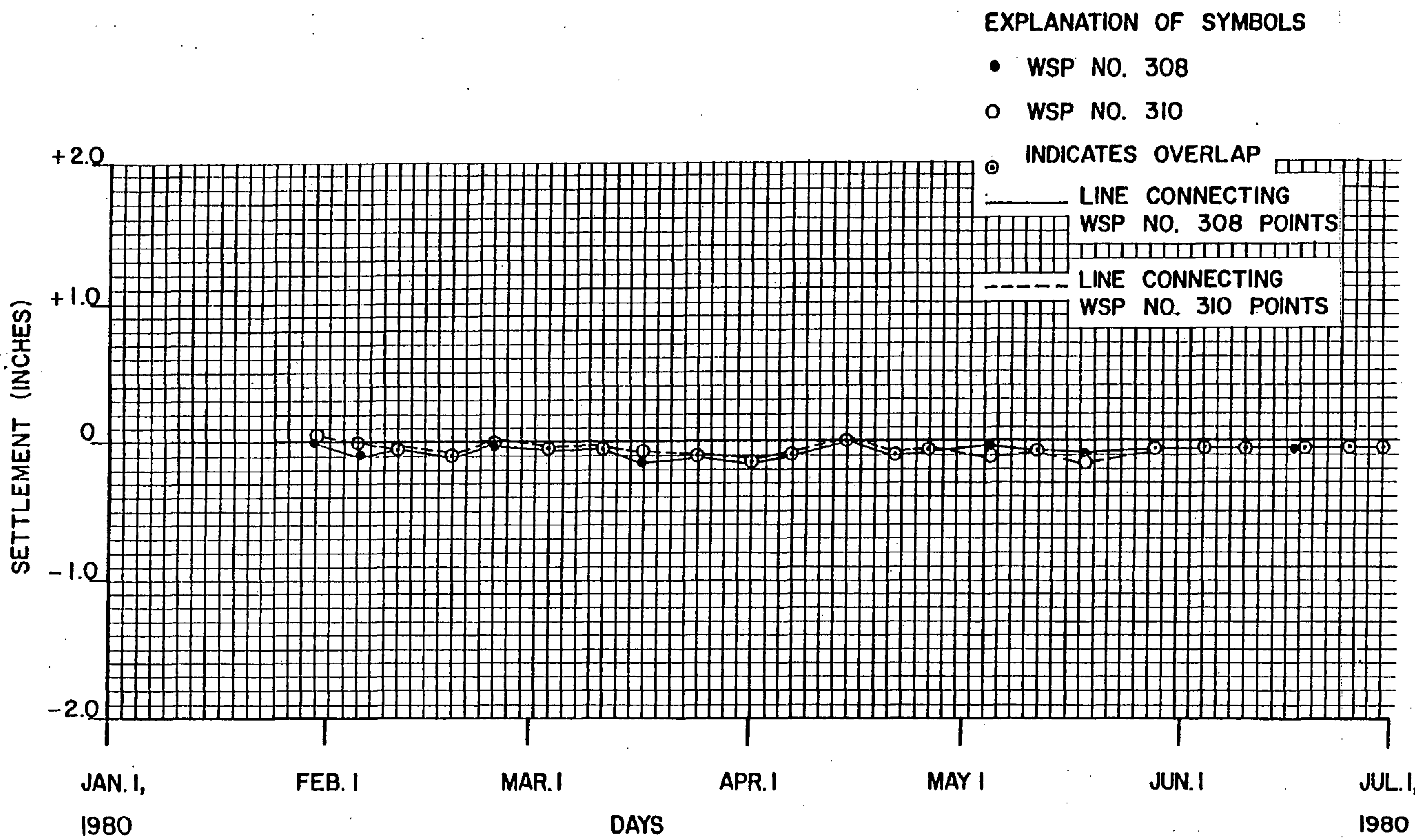
GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
PLAN SHOWING LOCATIONS OF SETTLEMENT MARKER POINTS		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 16	



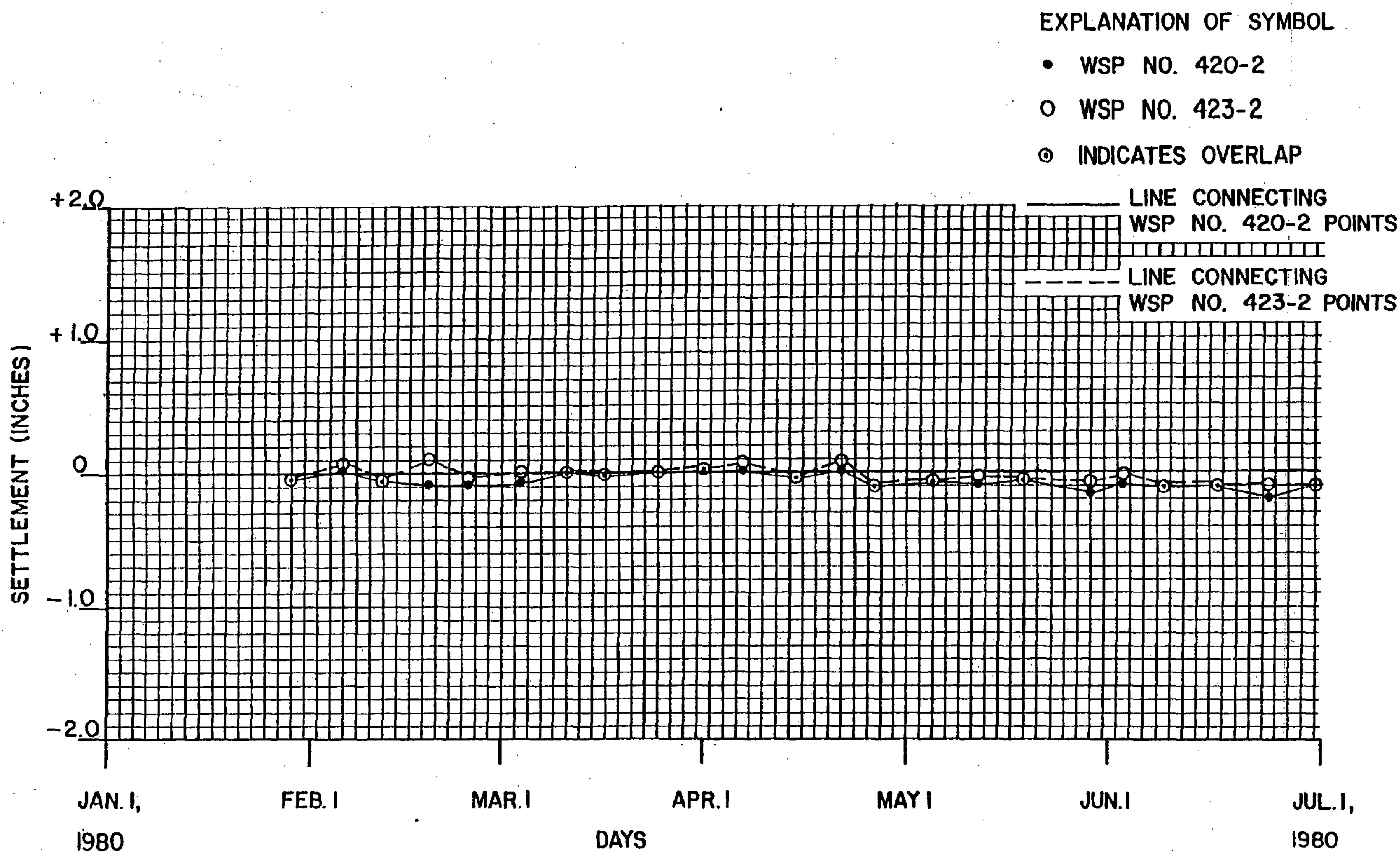
GEORGIA POWER COMPANY
ALVIN W. VOGTLE NUCLEAR PLANT

SETTLEMENT VERSUS TIME FOR CONTAINMENT UNIT I

SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 16-1	



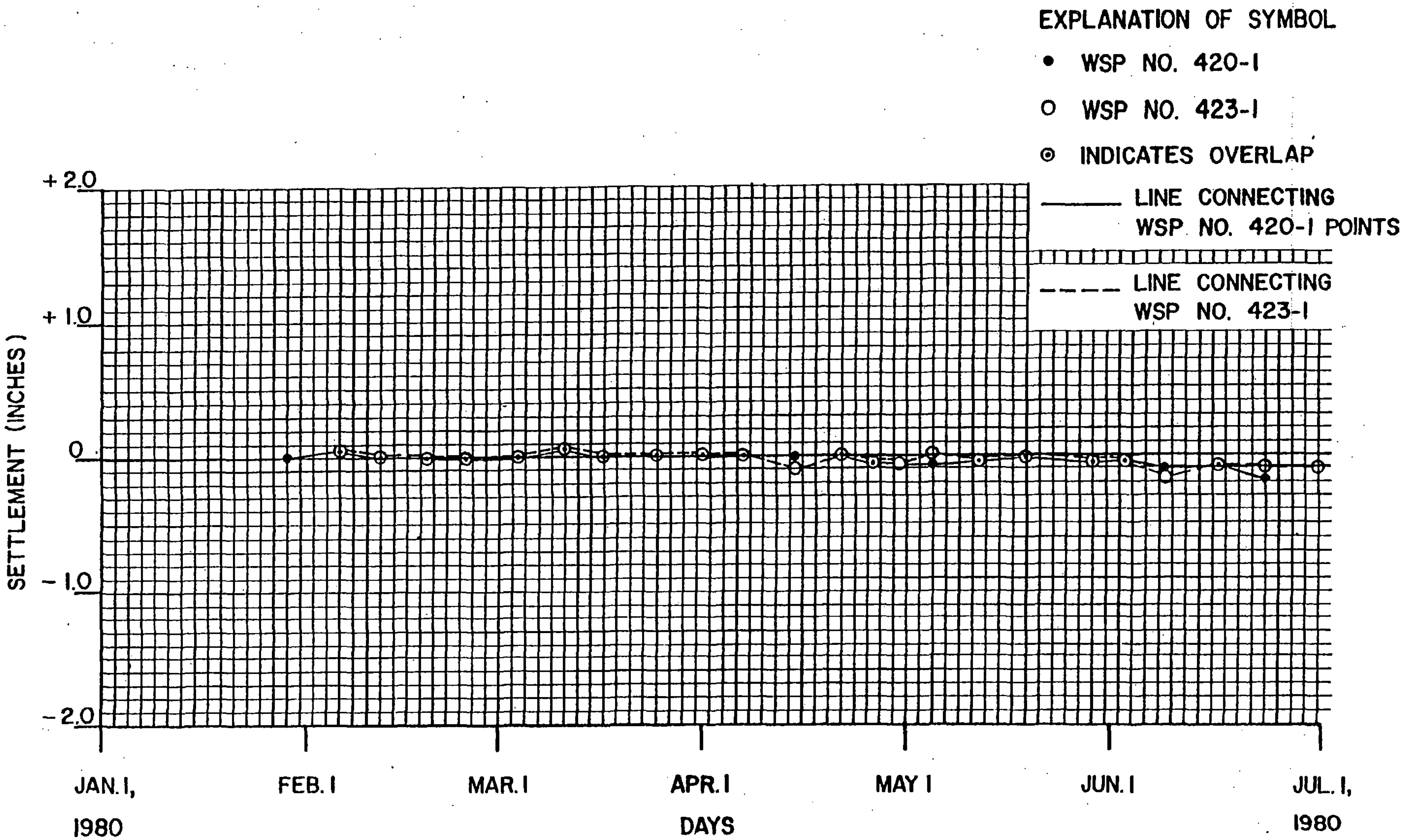
GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
SETTLEMENT VERSUS TIME FOR TURBINE BUILDING UNITS 1 & 2		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 16-2	



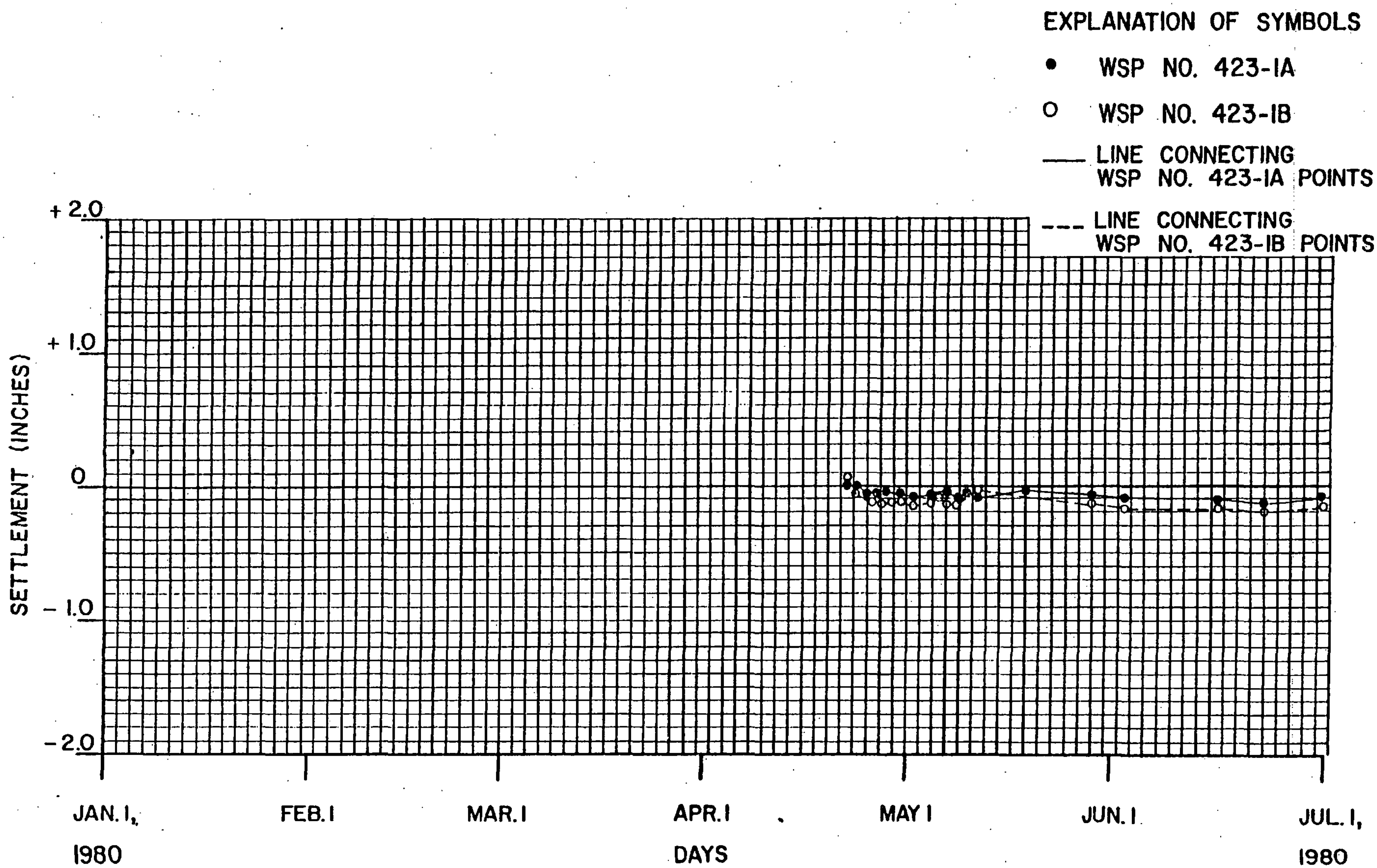
GEORGIA POWER COMPANY
ALVIN W. VOGTLE NUCLEAR PLANT

SETTLEMENT VERSUS TIME FOR ELECTRICAL TUNNEL UNIT 2

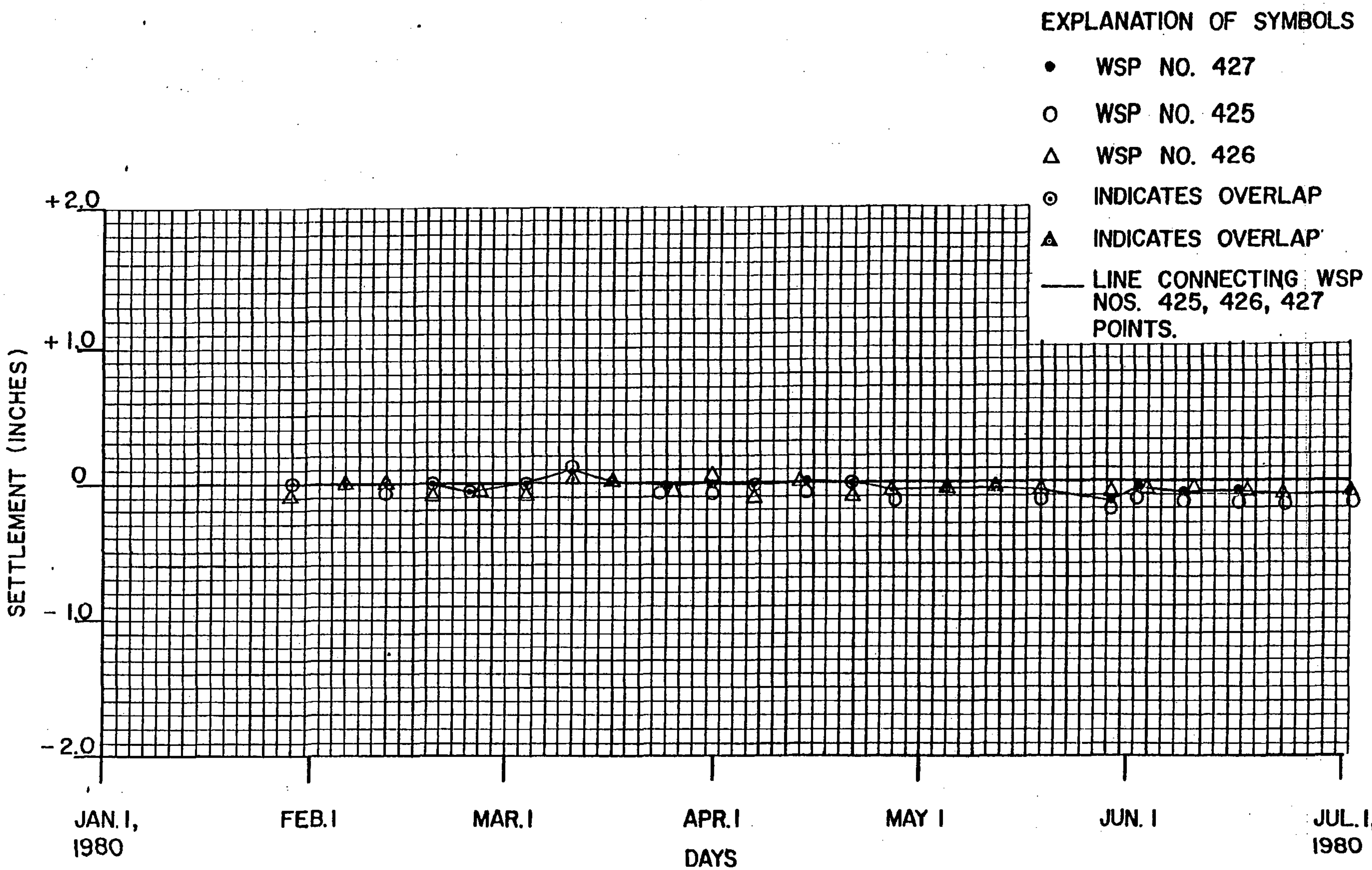
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 16-3	



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



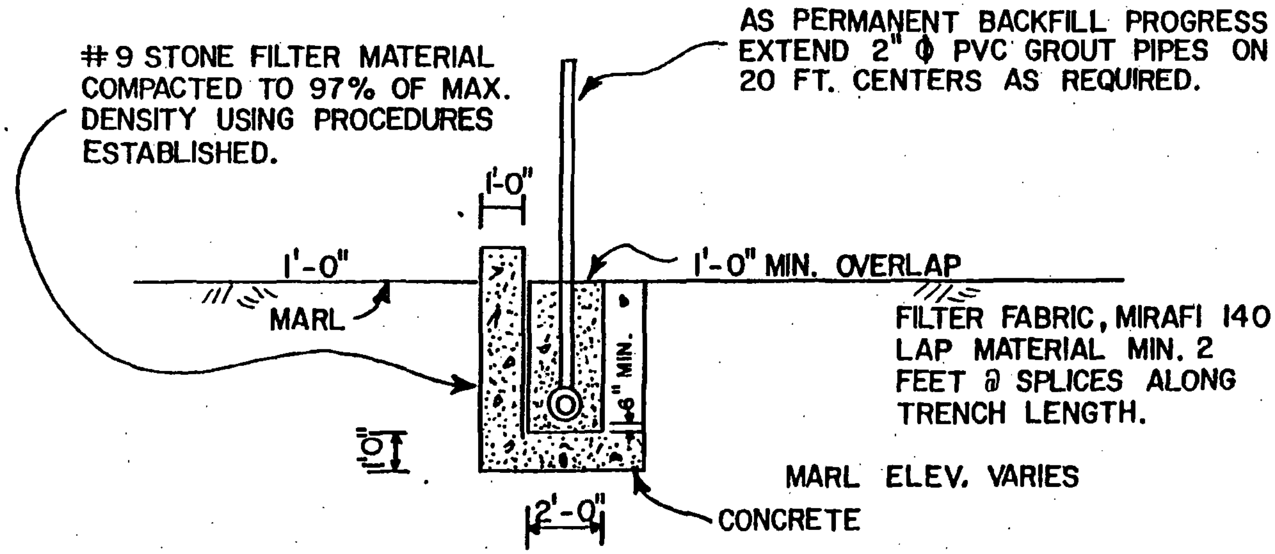
GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
SETTLEMENT VERSUS TIME FOR ELECTRICAL TUNNEL UNIT I		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 16-4B	



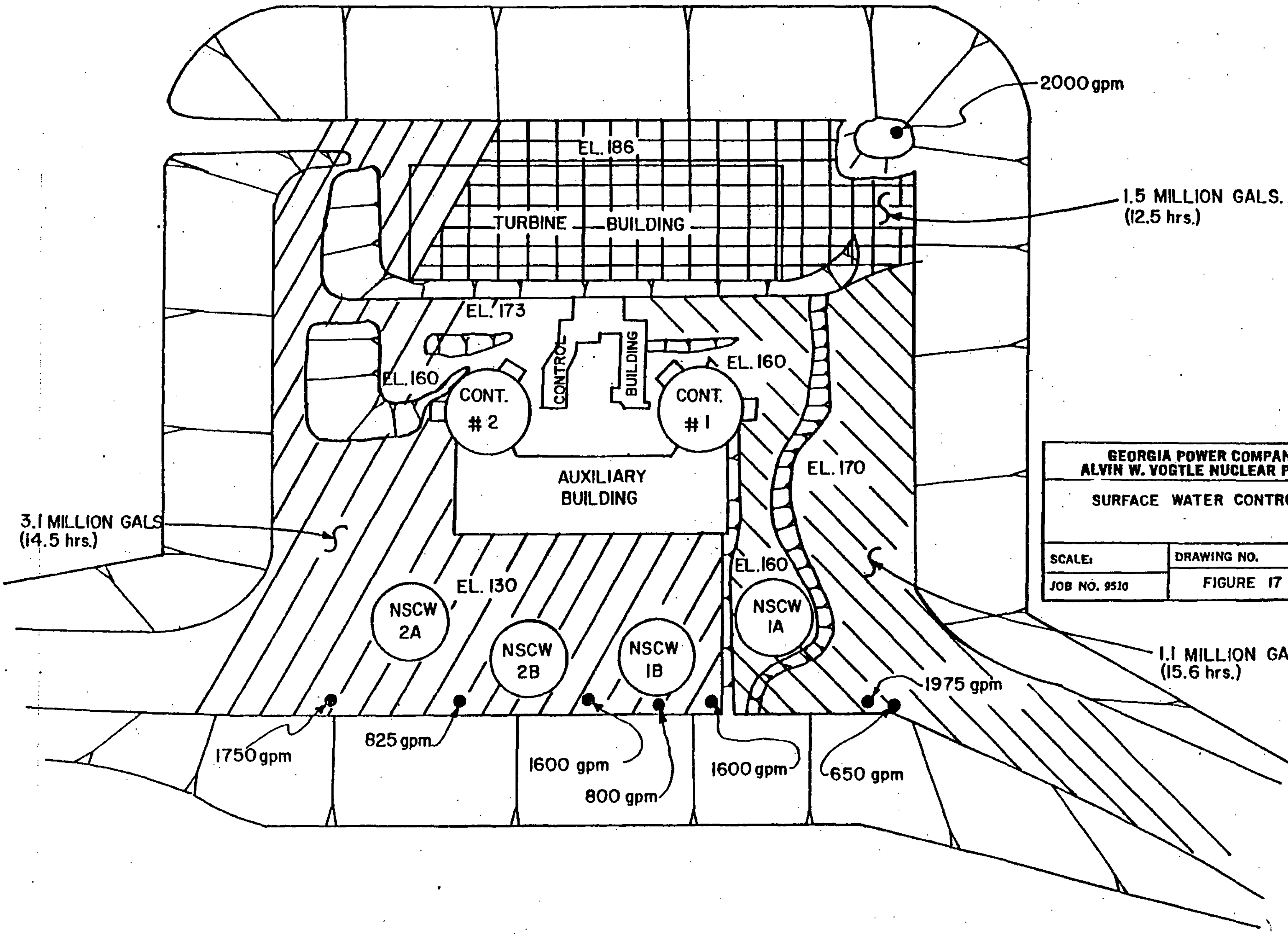
GEORGIA POWER COMPANY
ALVIN W. VOGTLE NUCLEAR PLANT

SETTLEMENT VERSUS TIME FOR CONTAINMENT UNIT 2

SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 16-5	



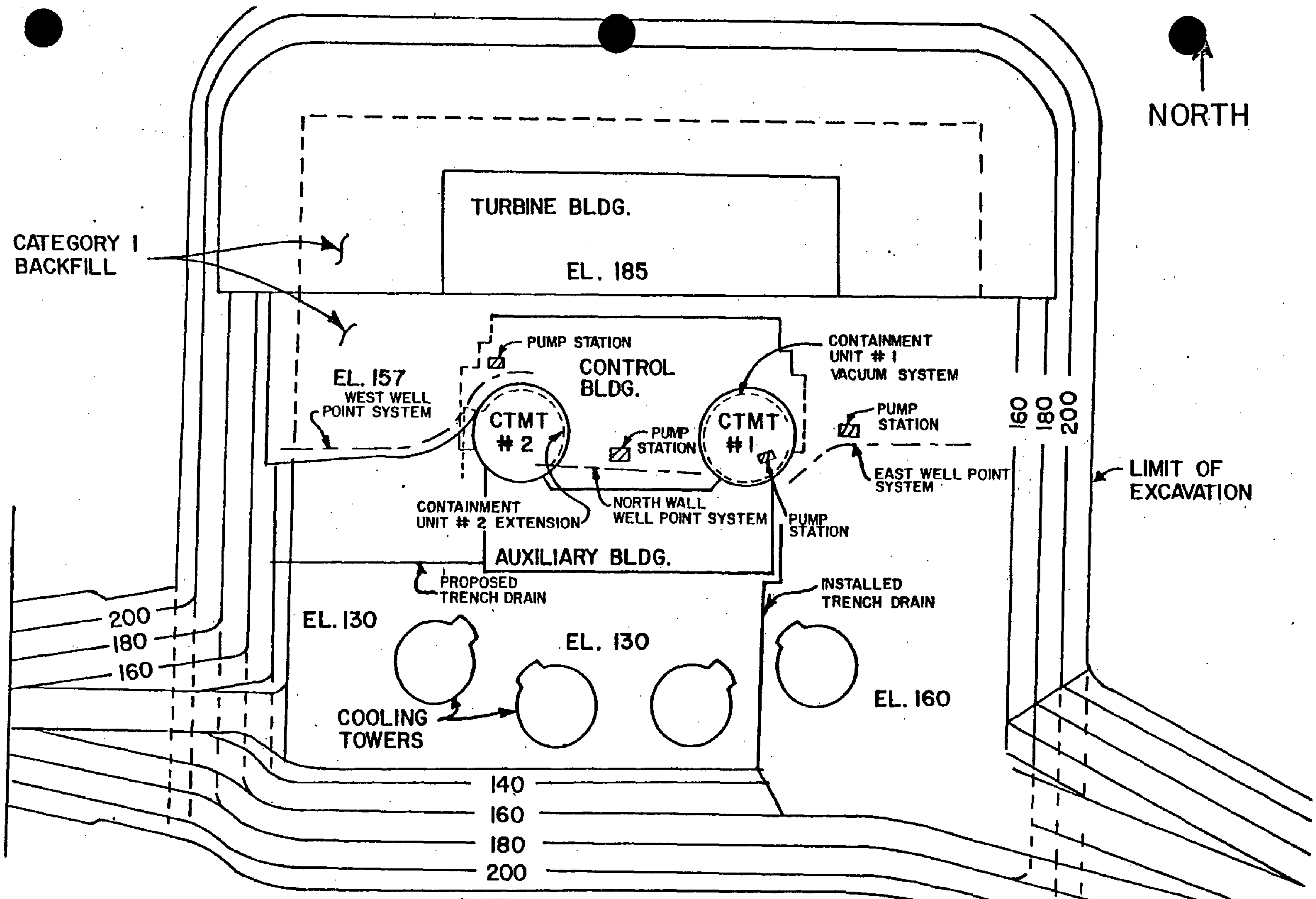
GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
TRENCH DRAIN TYPICAL SECTION		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 18	



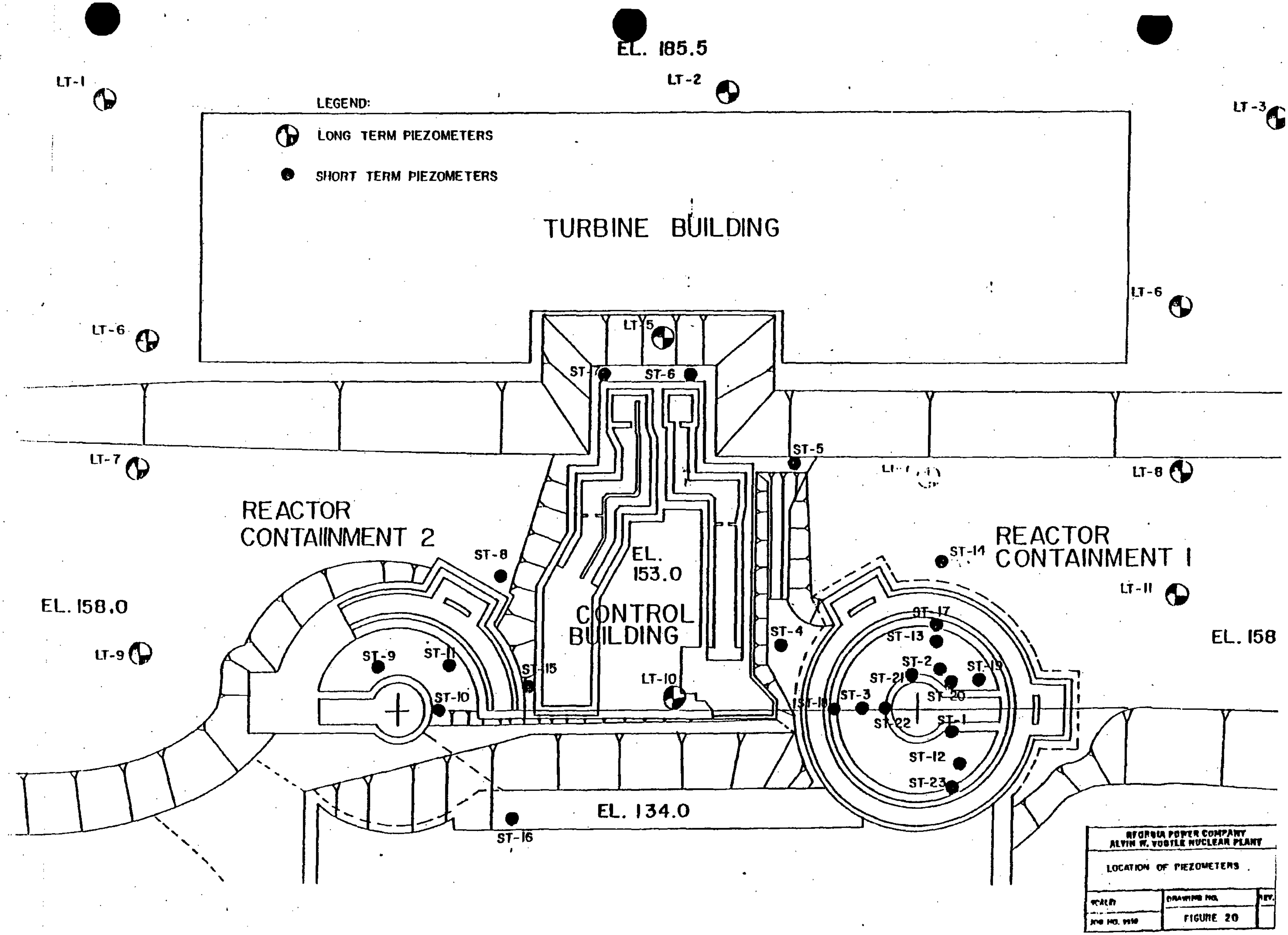
GEORGIA POWER COMPANY
ALVIN W. VOGTLE NUCLEAR PLANT

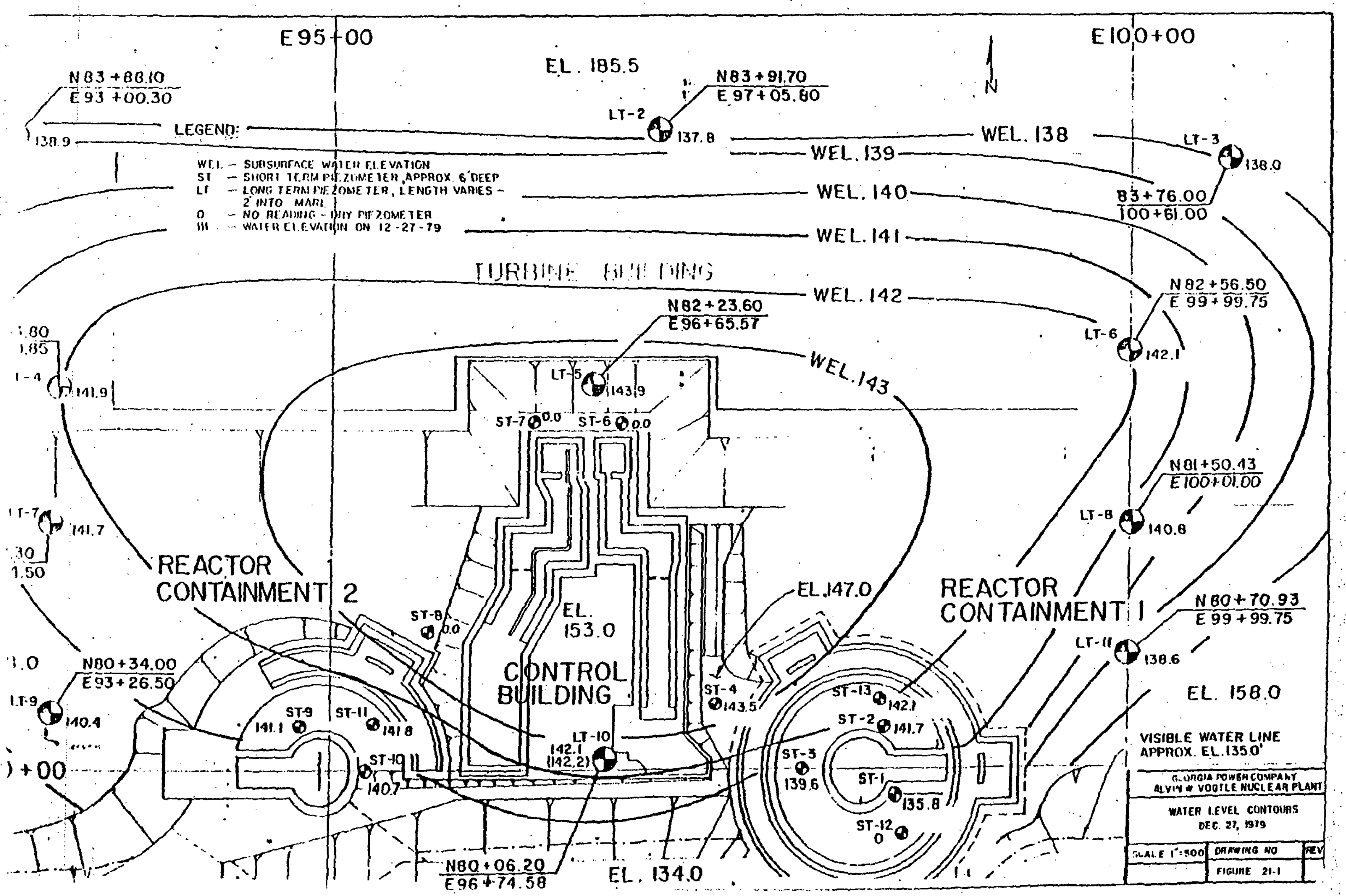
SURFACE WATER CONTROL

SCALE:	DRAWING NO.	REV
JOB NO. 9510	FIGURE 17	

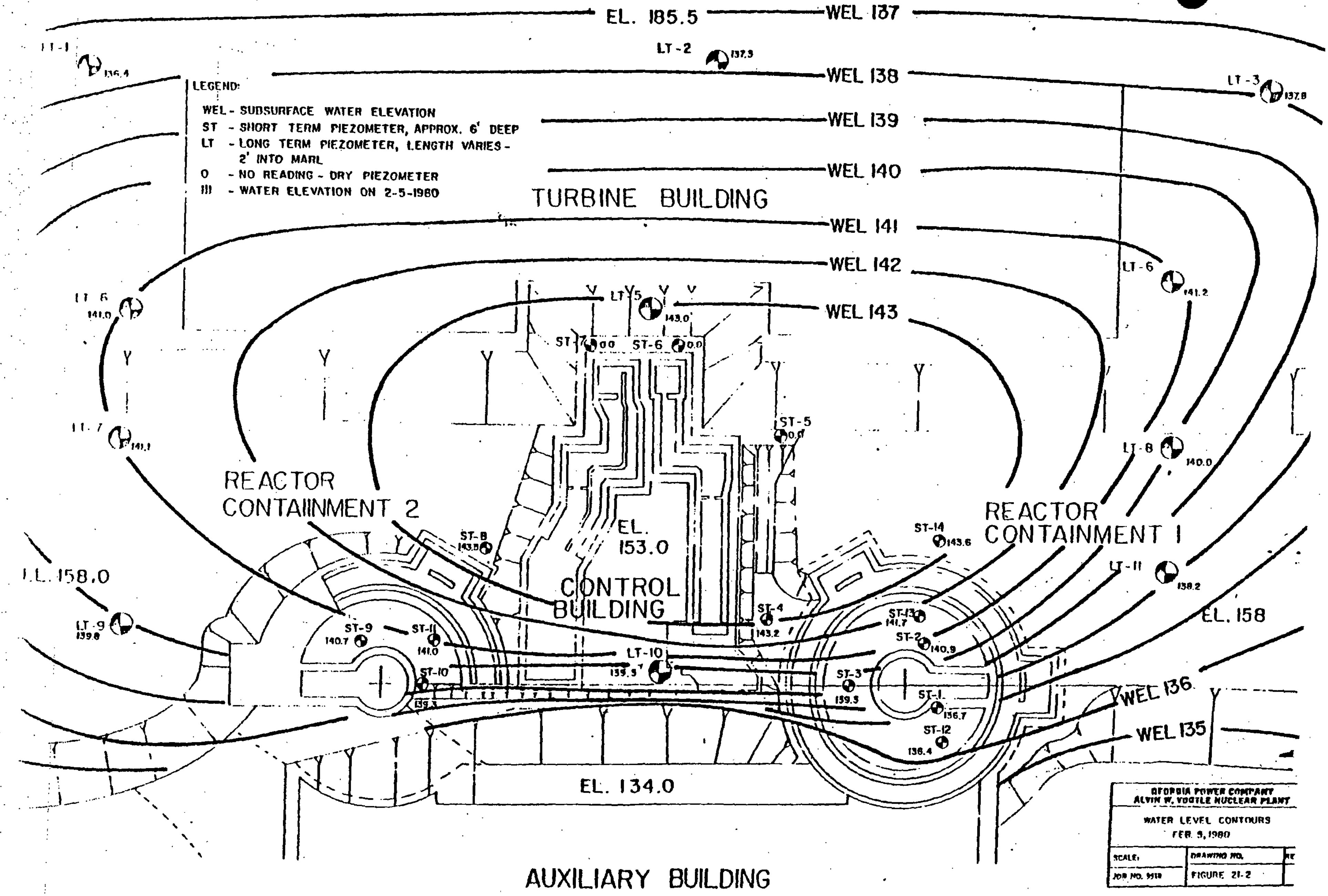


GEORGIA POWER COMPANY ALVIN W. VOGTLE NUCLEAR PLANT		
LOCATION OF DEWATERING SYSTEMS		
SCALE:	DRAWING NO.	REV.
JOB NO. 9510	FIGURE 19	

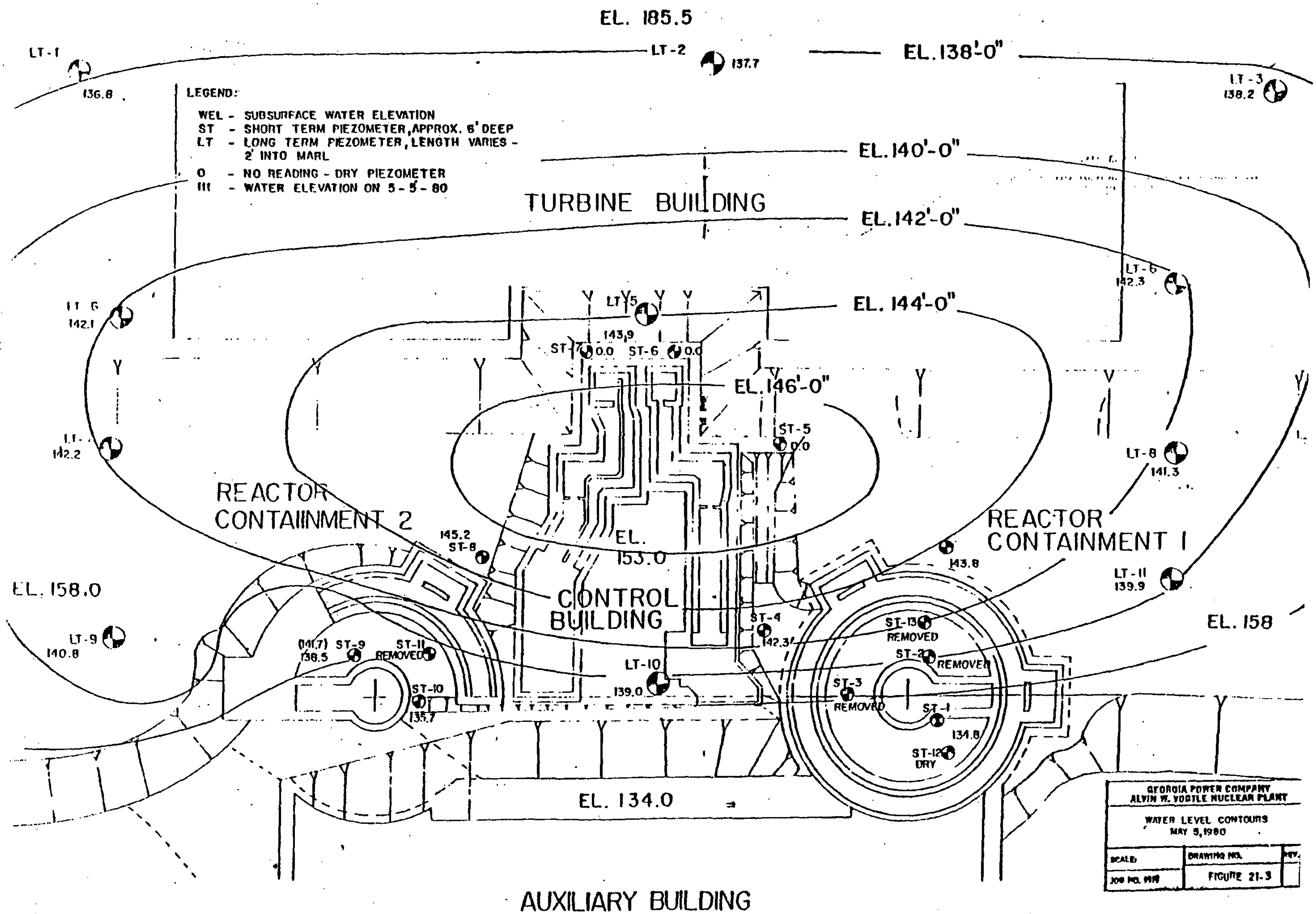

EL. 185.5
LT-1
LT-2
LT-3
LEGEND:
LONG TERM PIEZOMETERS
SHORT TERM PIEZOMETERS
TURBINE BUILDING
LT-6
LT-5
LT-6
ST-7
ST-6
ST-5
LT-7
LT-8
REACTOR CONTAINMENT 2
ST-8
EL. 153.0
CONTROL BUILDING
REACTOR CONTAINMENT 1
ST-14
LT-11
EL. 158.0
EL. 158
ST-17
ST-13
ST-4
LT-9
ST-9
ST-11
ST-15
ST-2
ST-19
ST-21
LT-10
ST-10
ST-18
ST-3
ST-20
ST-22
ST-1
ST-12
ST-23
EL. 134.0
ST-16
GEORGIA POWER COMPANY
ALVIN W. VOGTLE NUCLEAR PLANT
LOCATION OF PIEZOMETERS
SCALE:
DRAWING NO.
REV.
FIGURE 20



E95+00
E100+00
N
EL. 185.5
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
E97+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+23.60
E96+65.57
N82+56.50
E99+99.75
LT-6
142.1
141.9
LT-5
143.9
ST-7
0.0
ST-6
0.0
N81+50.43
E100+01.00
LT-8
140.8
LT-7
141.7
REACTOR CONTAINMENT 2
REACTOR CONTAINMENT 1
EL. 147.0
ST-8
0.0
EL. 153.0
CONTROL BUILDING
N80+70.93
E99+99.75
LT-11
138.6
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
EL. 158.0
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
REV
FIGURE 21-1



EL. 185.5
WEL 137
WEL 138
WEL 139
WEL 140
WEL 141
WEL 142
WEL 143
WEL 136
WEL 135
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 2-5-1980
TURBINE BUILDING
LT-2 137.9
LT-3 137.8
LT-6 141.2
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-8 143.8
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-10 139.5
ST-4 143.2
ST-13 141.7
ST-2 140.9
ST-3 139.5
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:
DRAWING NO.
FIGURE 21-2



GEORGIA POWER COMPANY
ALVIN W. VOGTLE NUCLEAR PLANT

WATER LEVEL CONTOURS
MAY 5, 1980

SCALE:	DRAWING NO.	REV.
JOB NO. 1919	FIGURE 21-3	

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		Field Test			Laboratory Test		Percent Compaction	Remarks
		N	E	γ_w (pcf)	W (%)	γ_d (pcf)	γ_d (max) (pcf)	OMC (%)		
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...

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...

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	

TABLE 3

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

...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
	3.0	26	49	sample attempted
10	1.5	22	54	
	2.0	--	--	Shelby tube
	3.0	28	45	sample attempted
11	1.5	19	40	
	2.0	--	--	Shelby tube
	2.5	14	41	sample attempted
	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
	3.0	17	37	sample attempted
	4.0	15	52	
13	1.5	18	34	
	2.0	--	--	Shelby tube
	3.0	29	77	sample attempted
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°F though actual temperature range is about 35-100° 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: (Does not include existing groundwater)

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

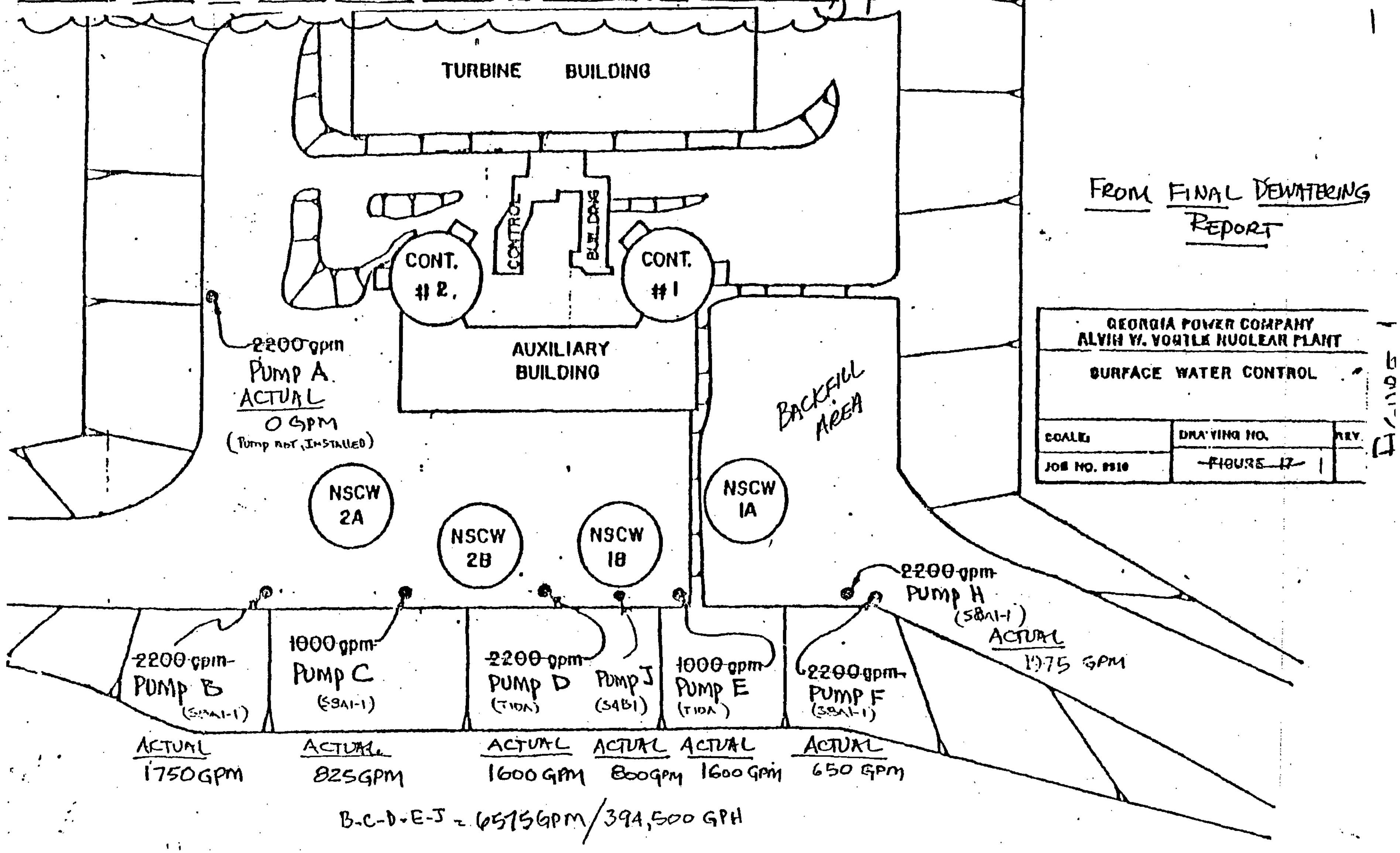
3. Hydraulic Institute Standards, Thirteenth Edition, Hydraulic Institute

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

VIII. ATTACHMENTS

umping capacity

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



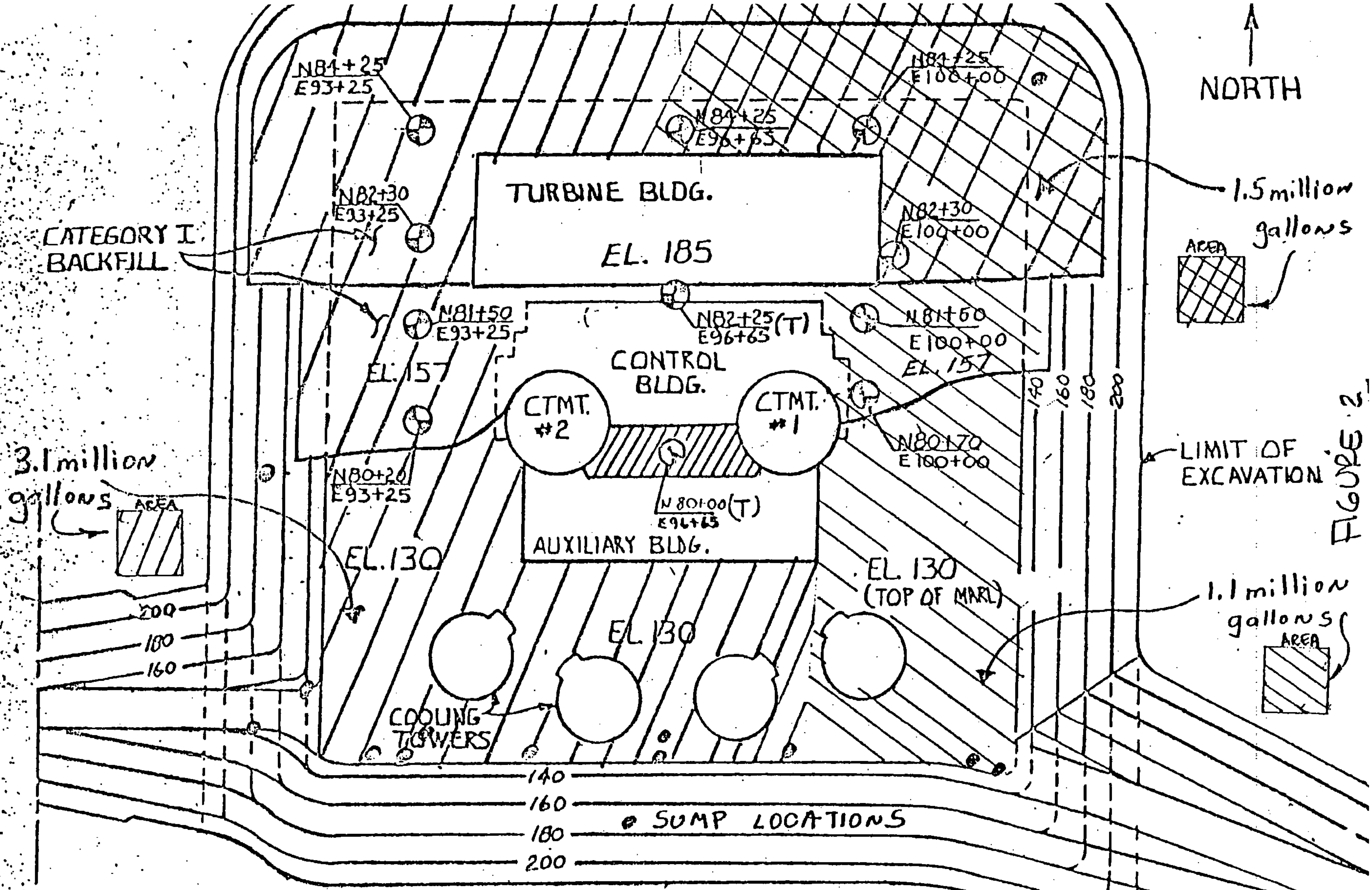


FIGURE 2

NOTES: 1. ⊕ 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:

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

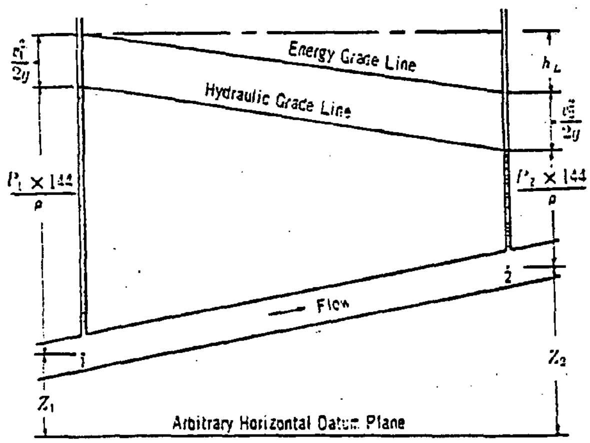


Figure 1-4
Energy Balance for Two Points in a Fluid

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

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

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

1. $h_l = \frac{0.00259\, k Q^2}{d^4}$

2. $R_e = 123.9\,\frac{d v \rho}{\mu}$

3. $V = \frac{0.408\, Q}{d^2}$

4. $d_1^2 V_1 = d_2^2 V_2$

5. $bhp = \frac{Q H \rho}{247000\, e\rho}$

FIGURE 4

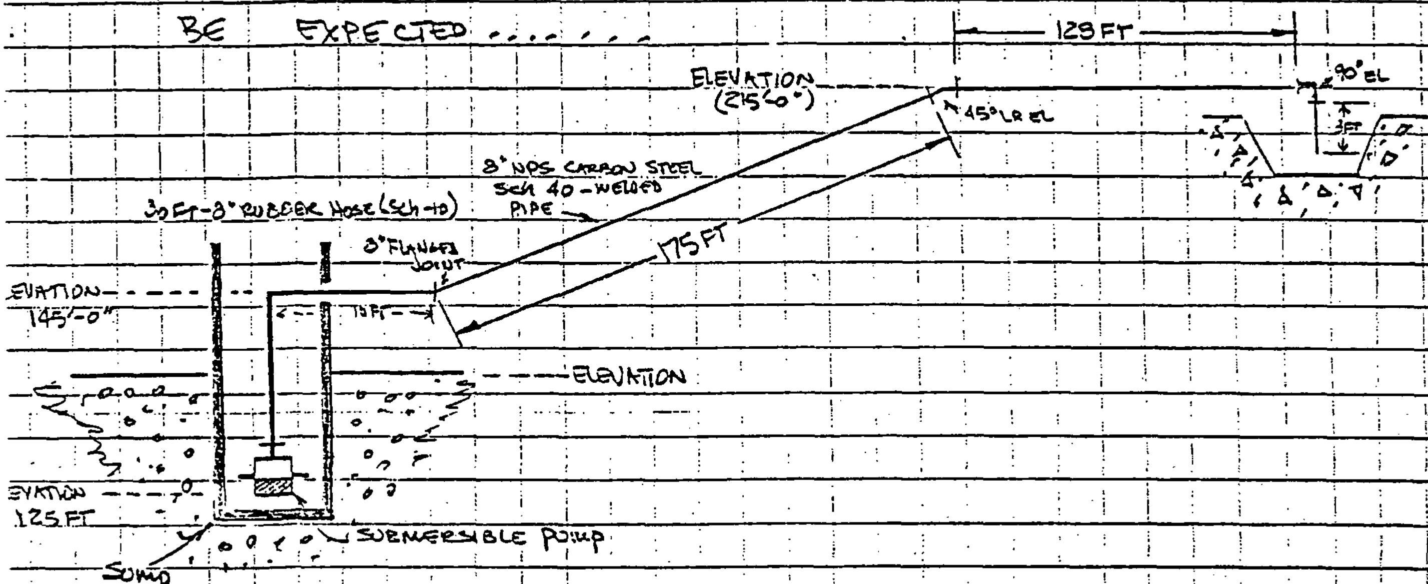
1756—12M—1-81-05 GEORGIA POWER COMPANY Sheet ____ of ____

Subject DEWATERING PUMP PERFORMANCE Project VOGTLE

POWER BLOCK WEST SLOPE – PUMP A

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

$d = \underset{C.S.P}{7.981}$ & $\underset{HOSE}{8.329}$

PUMP NOT INSTALLED

APPENDIX A-1

GEORGIA POWER COMPANY Sheet ___ of ___

Subject ______ Project ______

Computed by ______ Date ______ Checked by ______ Date ______

SOLUTION:

I. FORMULAE

SEE FIGURE A

APPENDIX A-2

II COMPUTE SYSTEM PIPING HEAD LOSS —

$$h_l = \frac{0.00259\,K\,Q^2}{d^4}$$

K = 25 f_T for LONG RADIUS 45° ELBOW
K = 30 f_T for 90° ELBOW
K = 1 for PIPE EXIT
K = $f\frac{L}{D}$ for STRAIGHT PIPE
PIPE ENTRANCE NOT CONSIDERED

K FOR PIPE

$$K = f_1\frac{L}{D} + f_2\frac{L}{D} \text{ (FOR HOSE \& C.S. PIPE)}$$

f_1 = 0.015 FOR CARBON STEEL; f_2 = 0.02 FOR HOSE (ESTIMATED)

$$\therefore K = \frac{(0.015)(306)(12)}{7.981} + \frac{(0.02)(30)(12)}{8.329}$$

K = 6.9 + 0.86 = 7.76 FOR PIPING

K = (2)(25)(0.015) = .75 FOR 2-45° LR ELS (CONSIDERED HOSE/PIPE JUNCT A LR EL)
K = (2)(30)(0.015) = .9 FOR 2-90° ELS

K_{TOTAL} = 1 + 0.75 + 0.9 + 7.76 = 10.41

APPENDIX A-3

GEORGIA POWER COMPANY

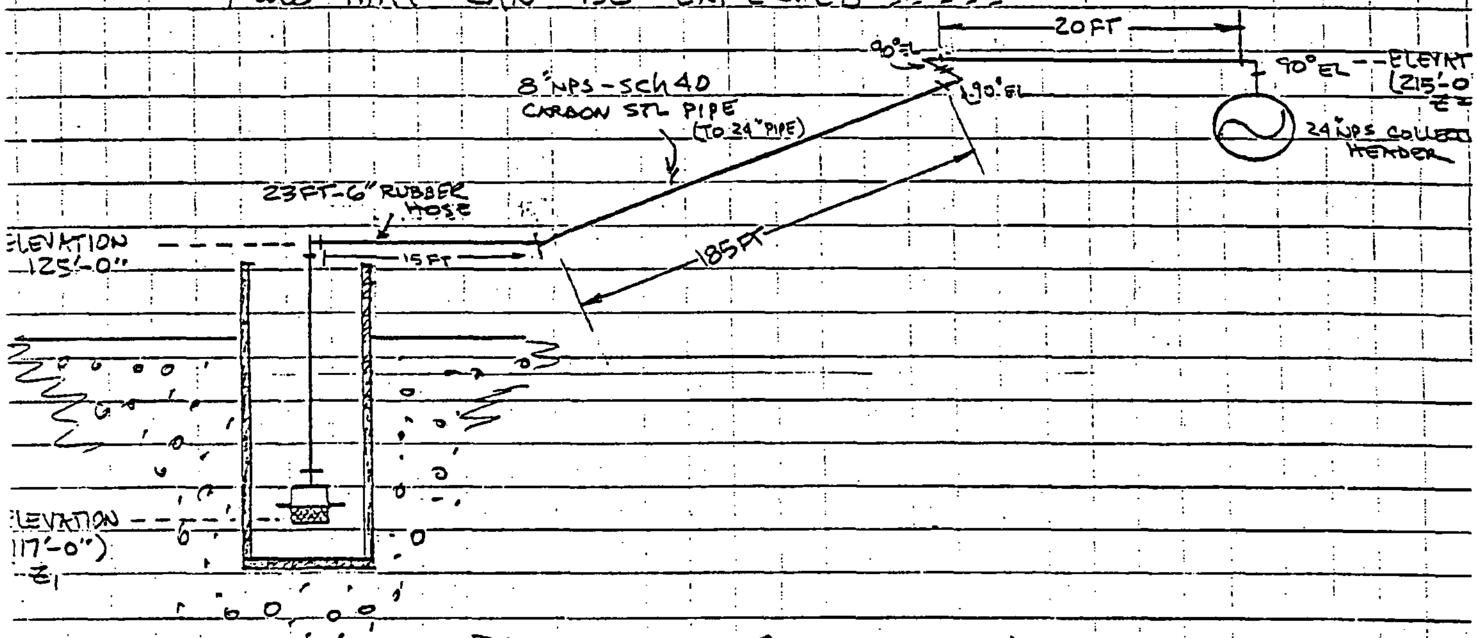
Sheet ______ of ______

Subject: DEWATERING PUMP PERFORMANCE — POWER BLOCK SOUTH SLOPE - PUMP B

Project: VOGTLE

Computed by: D. DAY   Date: 8-8-80   Checked by: ______   Date: ______

I. 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 S8A1-1

RATED 2600 GPM @ 55 FT HD

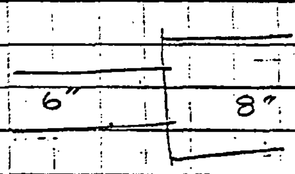
$d_1 = 6.065$ $d_2 = 7.981$

$f_{T_1} = 0.015$ $f_{T_2} = 0.014$

II COMPUTE SYSTEM PIPING HEAD LOSS

$$h_l = \frac{0.00259\, K Q^2}{d^4}$$

$K = 30 f_T$ for 90° ELBOWS
$K_2 = 0.53$ for SUDDEN ENLARGEMENT
$K = 1$ for PIPE ENTRANCE
$K = 1$ for PIPE EXIT
$K = f \frac{L}{D}$ for PIPING



$\beta = \frac{d_1}{d_2} = 0.76$

$K_2 = \frac{(1-\beta^2)^2}{\beta^4} = 0.53$

K for PIPE

$$K = f_{T_1} \frac{L}{D_1} + f_{T_2} \frac{L}{D}$$

$$K = \frac{(0.015)(23)(12)}{6.065} + \frac{(0.014)(205)(12)}{7.981}$$

$K = 0.68 + 4.32 = 5.0$

$K = (4)(30)(0.015) = 1.8$ for 4-90° ELBOWS

$K_{TOTAL} = 1.8 + 0.53 + 1.0 + 1.0 + 5.0 = 9.33$

Subject ____ Project ____

Computed by ____ Date ____ Checked by ____ Date ____

III COMPUTE h_l OVER RANGE OF INTEREST –

$$h_l = \frac{0.00259\,K\,Q^2}{d^4} \qquad K_{TOTAL} = 9.33$$

$$h_l = \frac{0.00259\,(8.65)(Q^2)}{(7.981)^4} + \frac{(0.00259)(0.68)(Q^2)}{(6.065)}$$

$$h_l = (5.5\times10^{-6})(Q^2) + (1.3\times10^{-6})(Q^2)$$

$h_l = 1.38 + 0.33 = 1.71$ FT for 500 GPM
$h_l = 5.50 + 1.3 = 6.8$ FT for 1000 GPM
$h_l = 22.0 + 5.2 = 27.2$ FT for 2000 GPM
$h_l = 37.18 + 8.79 = 45.97$ FT for 2600 GPM
$h_l = 49.5 + 11.7 = 61.2$ FT for 3000 GPM

IV $H_{REQ'D} = Z_1 - Z_2 + h_{l_{1-2}} + \frac{V_2^2}{2g}$

$$\boxed{\frac{V_2^2}{2g} = \frac{\left(\frac{0.408\,Q}{d^2}\right)^2 \text{FT}^2/\text{SEC}^2}{64.4 \ \text{FT/SEC}^2}}$$

a. $H_{REQ'D_{3000}} = 98 + 61.2 + 5.73 = 164.93$ FT for 3000 GPM

b. $H_{REQ'D_{2600}} = 98 + 45.97 + 4.31 = 148.28$ FT for 2600 GPM

c. $H_{REQ'D_{2000}} = 98 + 27.2 + 2.55 = 127.3$ FT for 2000 GPM

d. $H_{REQ'D_{1000}} = 98 + 6.8 + 0.64 = 105.4$ FT for 1000 GPM

e. $H_{REQ'D_{500}} = 98 + 1.71 + 0.16 = 99.9$ FT for 500 GPM

2765—12M—1-31-66

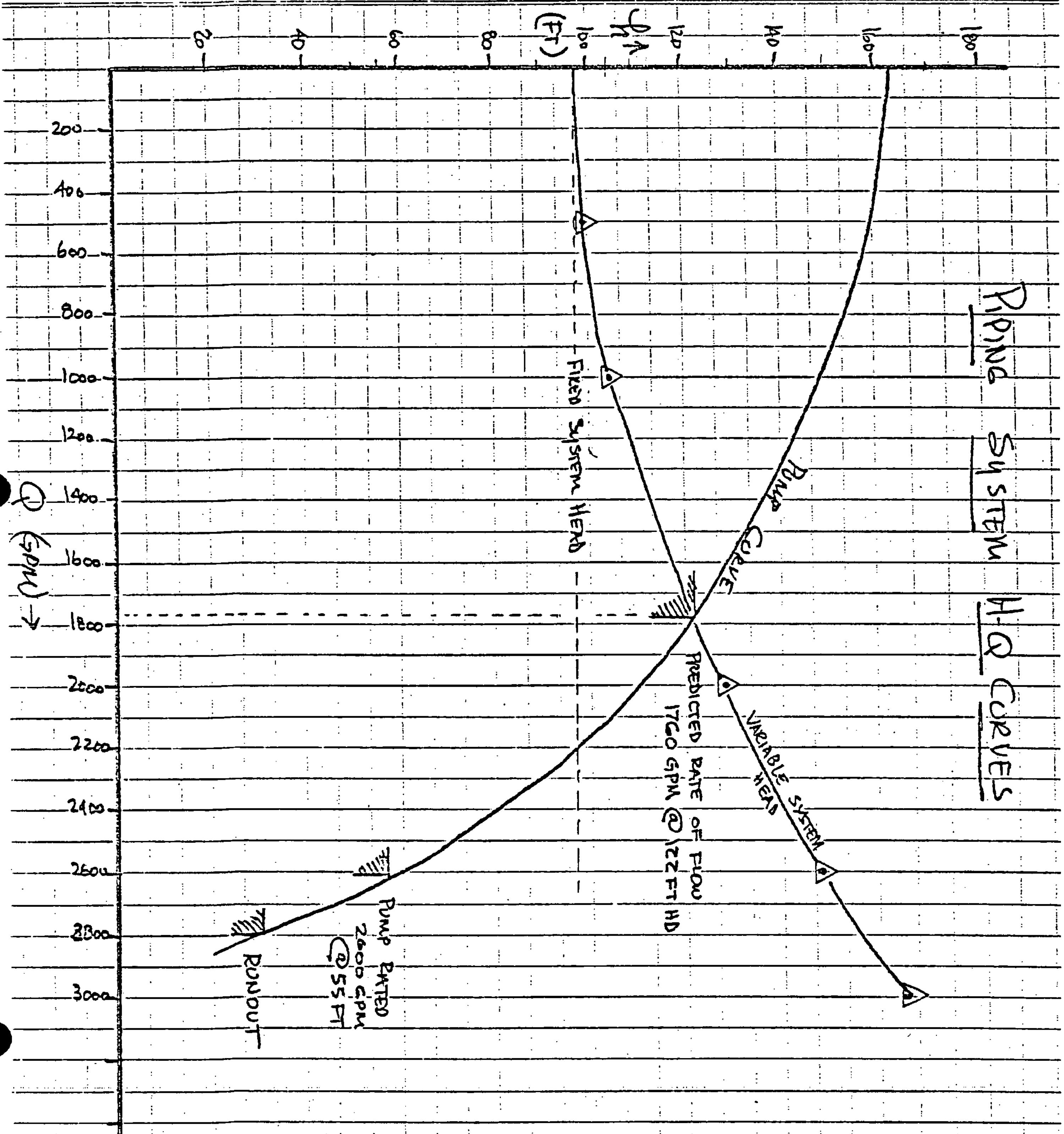
GEORGIA POWER COMPANY

Sheet 4 of 5

Subject DEWATERING PUMP PERFORMANCE — POWER BLOCK SOUTH SLOPE - PUMP B

Project VOGTLE

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



APPENDIX B-4

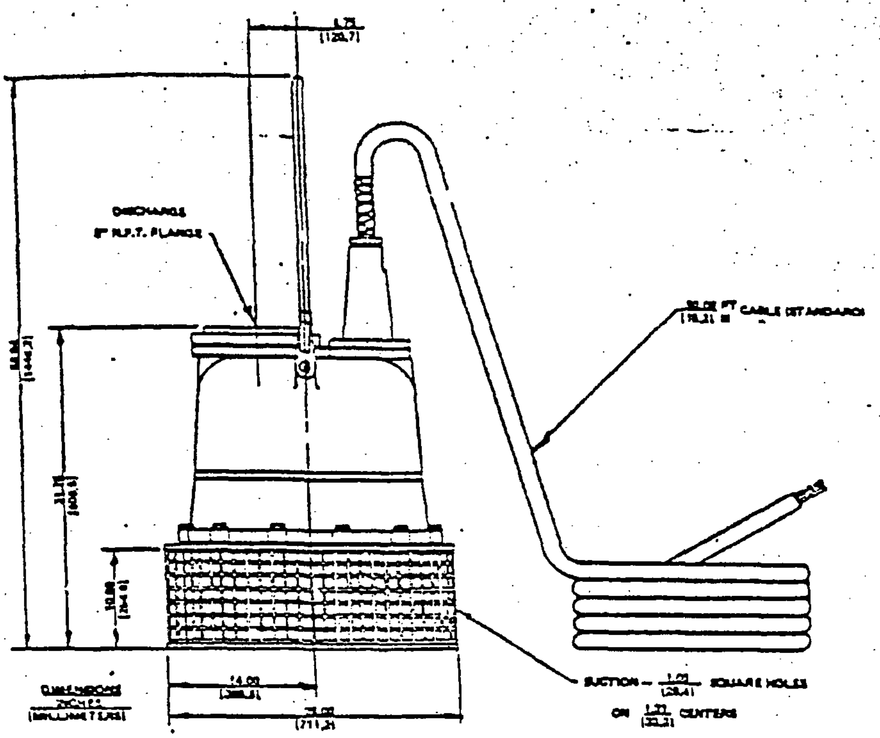
SHEET 5 of 5

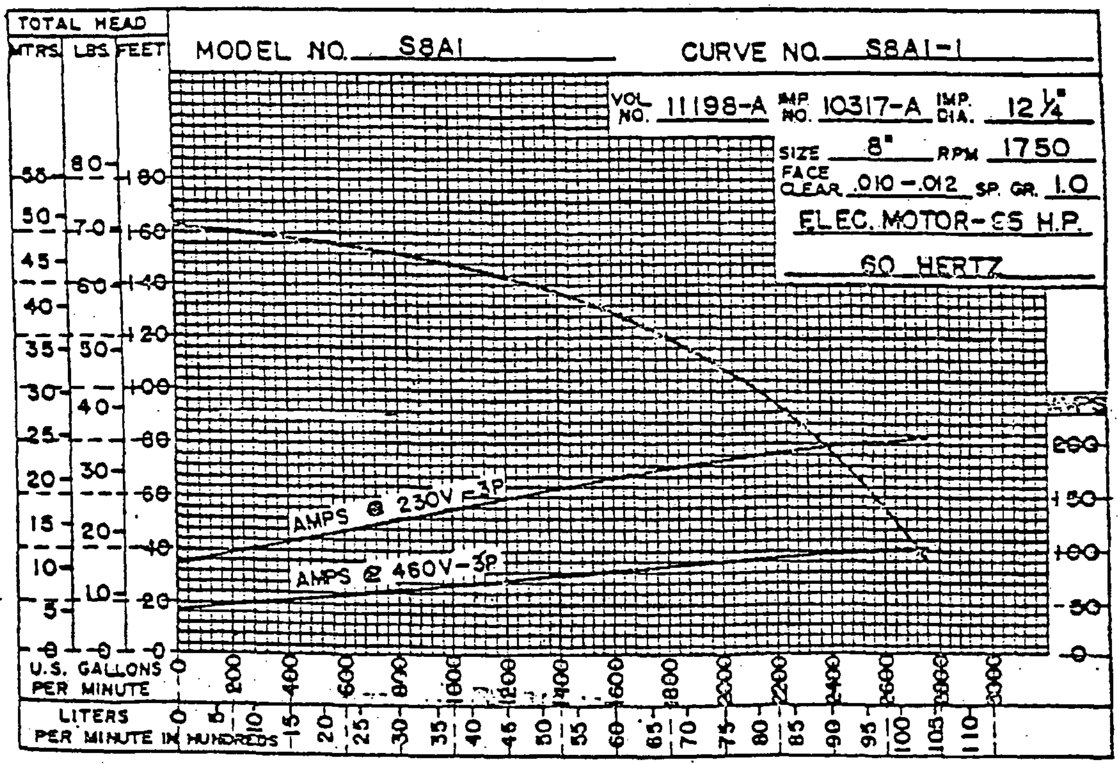
SPECIFICATION DATA

OVERALL DIMENSIONS and WEIGHTS APPROXIMATE

SECTION 130, PAGE 700

PUMP AND CABLE: Net Weight 935 lbs.
Shipping Weight 1070 lbs.
MOTOR CONTROL: Shipping Weight 150 lbs.
EXPORT SIZE: Pump: 36.0 Cu. Ft.





T-10-10-75-REV.

GORMAN-RUPP

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GORMAN-RUPP OF CANADA LIMITED • ST. THOMAS, ONTARIO, CANADA

APPENDIX B-5

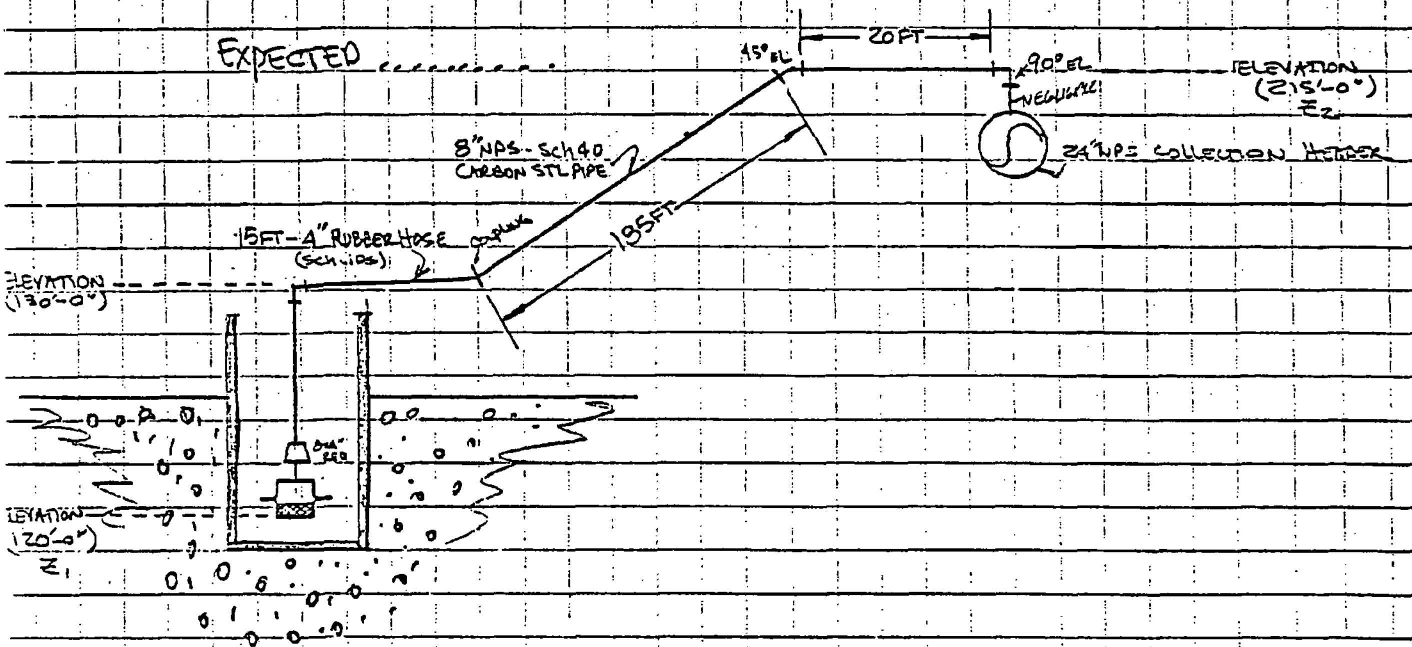
2706—12M—1-31-66 GEORGIA POWER COMPANY Sheet ____ of ____

Subject DEWATERING PUMP PERFORMANCE Project VOGTLE

POWER BLOCK SOUTH SLOPE - PUMP C

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 S8A1-1

RATED 2600 GPM @ 55 FT HD

$d_1 = 7.981_{c.s.}$ $d_2 = 4.260$

APPENDIX C-1

2766—12M—1-31-66

GEORGIA POWER COMPANY

Sheet 2 of

Subject

Project

Computed by Date Checked by Date

SOLUTION:

I. FORMULAE

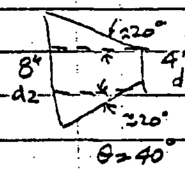
SEE FIGURE 4

APPENDIX C-2

II. Compute System Piping Head Loss —

$$h_f = \frac{0.00259\, K Q^2}{d^4}$$

K = 16 fT for 45° elbows
K = 30 fT for 90° elbows.
K = 8.76 for 8x4 reducer
$K = f\frac{L}{D}$ for piping
Pipe entrance & exit not included



$$K = \frac{0.8 \sin\frac{\theta}{2}(1-\beta^2)}{\beta^4}$$

$$\beta = \frac{d_1}{d_2} = \frac{4}{8} = .5$$

K for pipe —

$K = f_1\frac{L}{D} + f_2\frac{L}{D}$ (for hose & C.S. pipe)

$f_1 = 0.015$ for C.S. pipe ; $f_2 = 0.02$ for rubber hose (estimated)

$$K = \frac{(0.015)(205)(12)}{7.981} + \frac{(0.02)(15)(12)}{4.260}$$

$K = 4.62 + 0.85 = 5.47$

$f_T = 0.015$ (estimated all components)

$K = (2)(16)(0.015) = 0.48$ for 2-45° els
$K = (2)(30)(0.015) = 0.9$ for 2-90° els

$K_{TOTAL} = 0.48 + 0.90 + 8.76 + 5.47 = 15.61$

III COMPUTE hl OVER RANGE OF INTEREST —

$$hl = \frac{0.00259\,KQ^2}{d^4} \qquad K_{TOTAL} = 15.61 \qquad K_{7.981} = 0.48 + 0.45 + 4.62 = 5.55$$

$$K_{4.260} = 0.45 + 8.76 + 0.85 = 10.06$$

$$hl = \frac{(0.00259)(5.55)(Q^2)}{(7.981)^4} + \frac{(0.00259)(10.06)(Q^2)}{(4.260)^4}$$

$$hl = (3.54\times10^{-6})(Q^2) + 7.9\times10^{-5}(Q^2)$$

$hl = 0.89 + 19.75 = 20.64$ for 500 GPM
$hl = 3.54 + 79.0 = 82.54$ for 1000 GPM
$hl = 14.16 + 316.0 = 330.16$ for 2000 GPM
$hl = 23.93 + 534.0 = 557.93$ for 2600 GPM
$hl = 31.86 + 711.0 = 742.86$ for 3000 GPM

IV $H_{REQ'D} = Z_1 - Z_2 + hl_{1-2} + \frac{V_2^2}{2g}$

$$\boxed{\frac{V_2^2}{2g} = \frac{\left(\frac{0.408Q}{d^2}\right)^2 \ FT^2/SEC^2}{64.4\ FT/SEC^2}}$$

a. $H_{REQ'D_{3000}} = 95 + 742.86$ x for 3000 GPM

b. $H_{REQ'D_{2600}} = 95 + 557.93$ x for 2600 GPM

c. $H_{REQ'D_{2000}} = 95 + 330.16$ x for 2000 GPM

d. $H_{REQ'D_{1000}} = 95 + 82.54$ x for 1000 GPM

e. $H_{REQ'D_{500}} = 95 + 20.64$ for 500 GPM

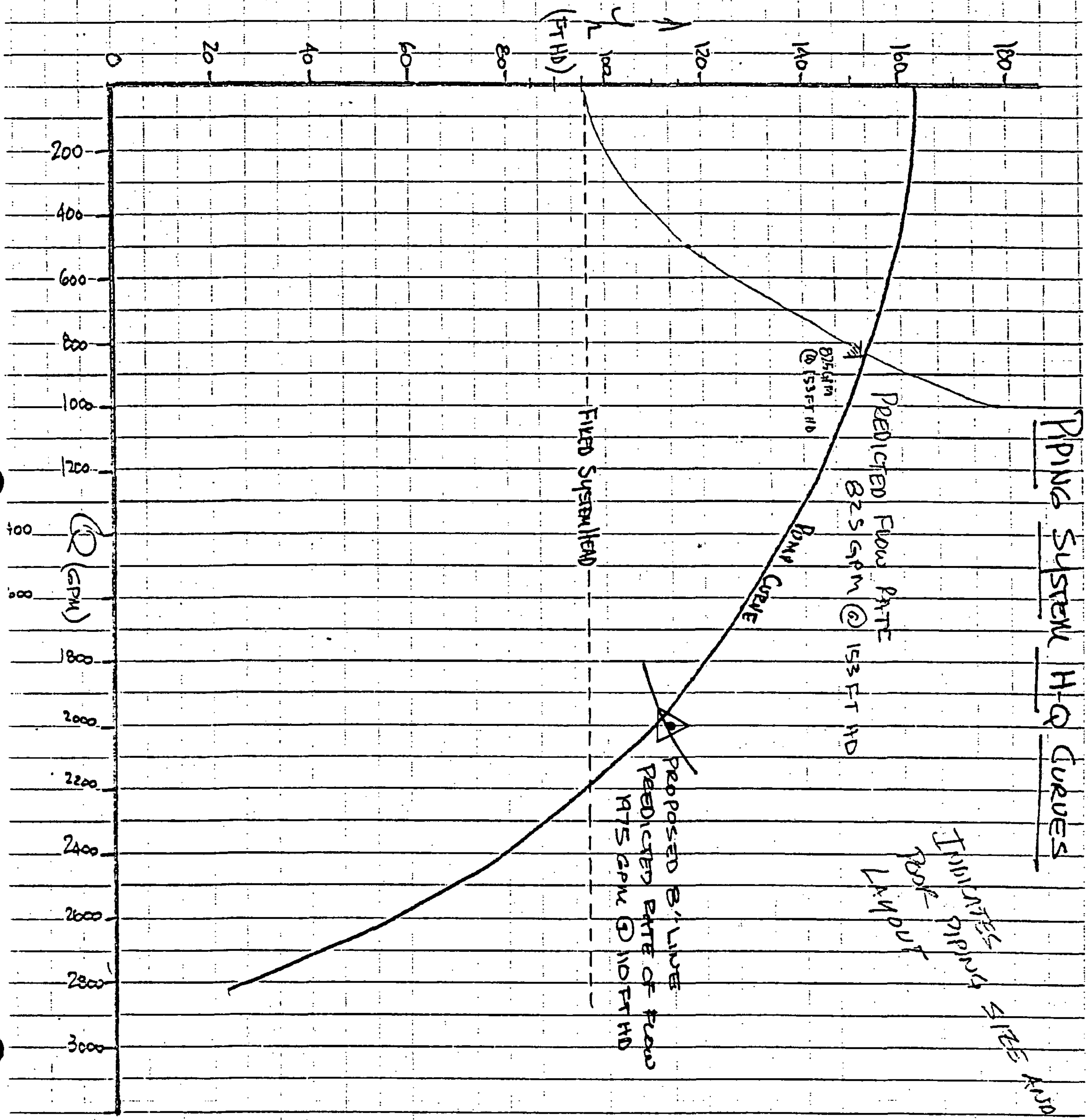
GEORGIA POWER COMPANY

Sheet ______ of ______

Subject: DEWATERING PUMP PERFORMANCE — POWER BLOCK SOUTH SLOPE – PUMP C

Project: VOGTLE

Computed by: D. DAY Date: 8-13-80 Checked by: ______ Date: ______



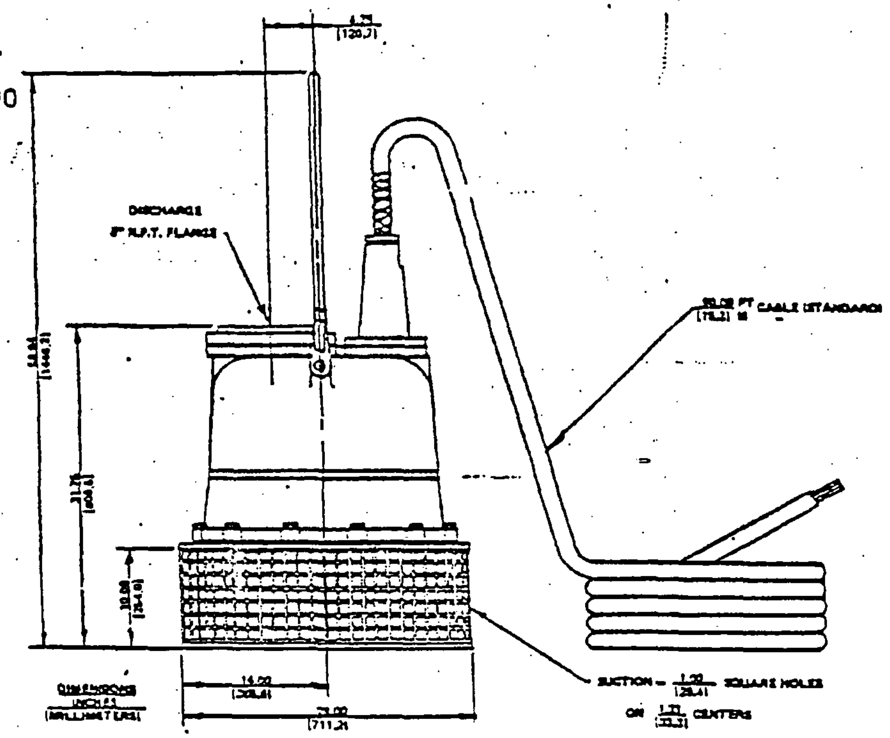
APPENDIX C-5

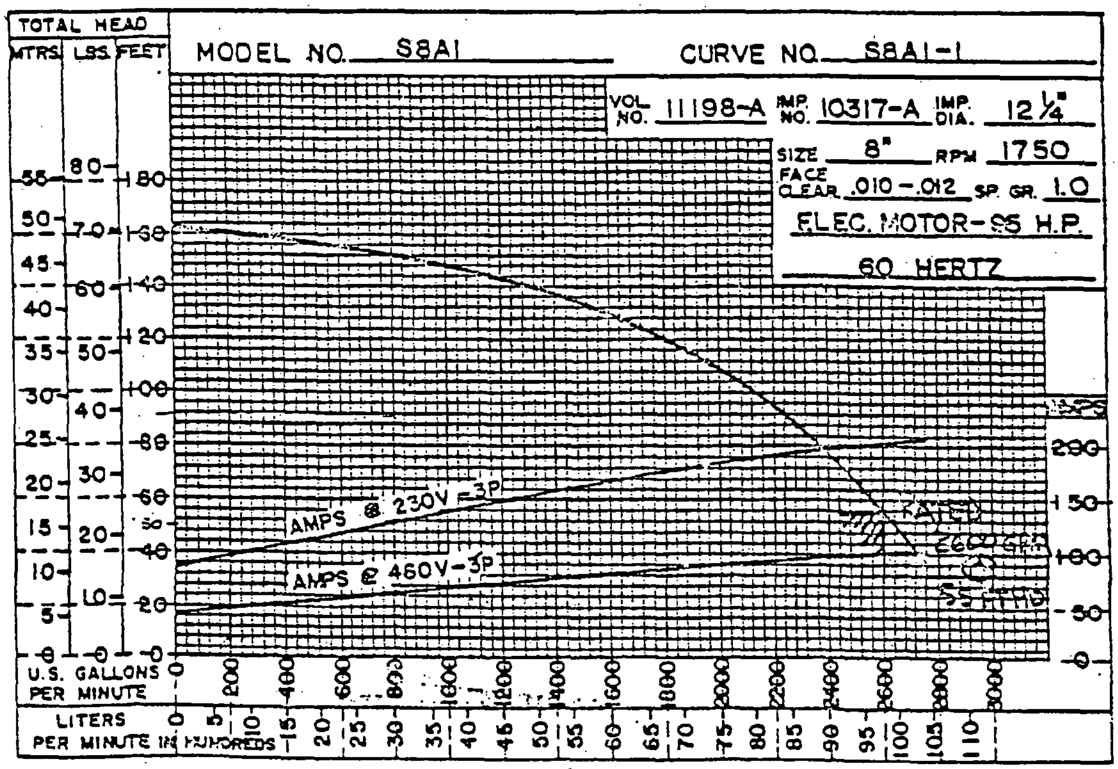
SPECIFICATION DATA

OVERALL DIMENSIONS and WEIGHTS APPROXIMATE

SECTION 130, PAGE 700

PUMP AND CABLE: Net Weight 935 lbs.
Shipping Weight 1070 lbs.
MOTOR CONTROL: Shipping Weight 150 lbs.
EXPORT SIZE: Pump: 36.0 Cu. Ft.





T-10-10-75-REV.

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APPENDIX C-6

2766—12M—1-81-65

GEORGIA POWER COMPANY

Sheet 1 of 1

Subject DEWATERING PUMP PERFORMANCE Project VOGTLE

POWER BLOCK SOUTH SLOPE - PUMP D

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

PIPING SYSTEM LOSSES FOR THIS PUMP ARE ALMOST IDENTICAL TO THOSE LOSSES ENCOUNTERED IN THE PUMP E CONFIGURATION. FOR THAT REASON, A SEPARATE ANALYSIS FOR THIS PUMP WILL NOT BE PERFORMED. BASED ON SIMILARITY APPROACH, VALUES OBTAINED IN PUMP E CALCULATION SHALL APPLY FOR PUMP D WITH AN ESTIMATED ERROR NOT TO EXCEED -1% TO +5% OF PREDICTED FLOW RATE.

(SEE APPENDIX E)

APPENDIX D

GEORGIA POWER COMPANY

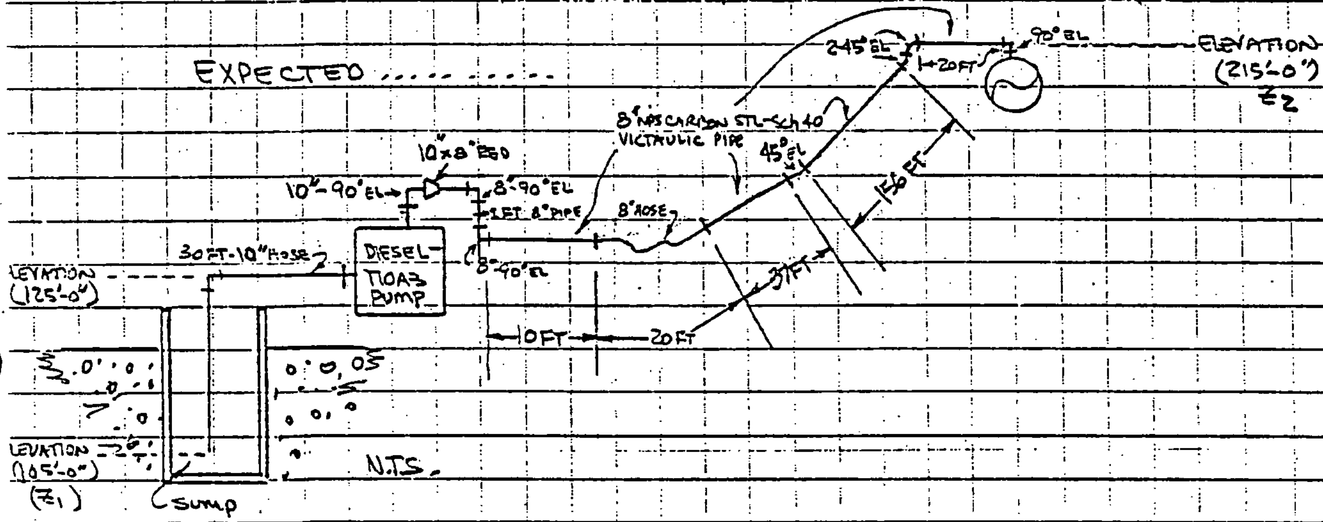
Sheet 1 of 7

Subject: DEWATERING PUMP PERFORMANCE — Project: VOGTLE

POWER BLOCK SOUTHEAST CORNER - PUMP E

Computed by: D. DAY — Date: 8-6-80 — 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 T10A3-B-4031-C, SIZE 10"x10"

RATED 2750 GPM @ 90 FT HD

$d_1 = 10.420"$ (10" HOSE)

$d_2 = 7.981$ (8" PIPE)

$d_3 = 8.329$ (8" HOSE)

$f_T = 0.015$

APPENDIX E-1

2766—12M—1-81-64 GEORGIA POWER COMPANY Sheet 2 of 7

Subject ______ Project ______

Computed by ______ Date ______ Checked by ______ Date ______

SOLUTION :

I FORMULAS

SEE FIGURE 4

APPENDIX E-2

Subject ______ Project ______

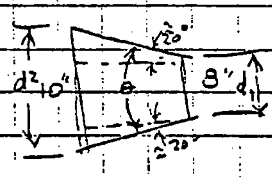
Computed by ______ Date ______ Checked by ______ Date ______

II Compute system piping head loss –

$$h_l = \frac{0.00259 K Q^2}{d^4}$$

CRANE A-26

K = 30 f_T for 90° elbows
K = 16 f_T for 45° elbows
K = 2 for 10"×8" reducer
K = $f \frac{L}{D}$ for straight pipe
Pipe entrance & exit not included



$$K = \frac{0.8 \sin\frac{\theta}{2}(1-\beta^2)}{\beta^4}$$

$$\beta = \frac{d_1}{d_2}$$

K for pipe

$$K = f_1 \frac{L}{D} + f_2 \frac{L}{D} \text{ (for hose \& CS pipe)}$$

$f_1 = 0.015$ for C.S.; $f_2 = 0.02$ for rubber hose (estimated)

$$K = \underbrace{\frac{(0.015)(224)(12)}{7.981}}_{\text{(C.S. pipe)}} + \underbrace{\frac{(0.02)(20)(12)}{8.329}}_{\text{(8" hose)}} + \underbrace{\frac{(0.02)(30)(12)}{10.420}}_{\text{(10" hose)}}$$

K = 6.32 for piping

f_T = 0.015 (estimated all components)

K = (5)(0.015)(30) = 2.25 for 5–90° elbows
K = (3)(0.015)(16) = .72 for 3–45° elbows

$$K_{TOTAL} = 2.25 + 0.72 + 6.32 + 2 = 11.29$$

Subject ______ Project ______

Computed by ______ Date ______ Checked by ______ Date ______

III Compute h_l over range of interest

$$h_l = \frac{0.00259 K Q^2}{d^4}$$ K = 5.05 for 8" CS pipe; K = .58 for 8" hose; K = 0.69 for 10" hose

$$h_l = \frac{(0.00259)(5.05)(Q)^2}{(7.981)^4} + \frac{(0.00259)(0.58)(Q)^2}{(8.329)^4} + \frac{(0.00259)(0.69)(Q)^2}{(10.420)^4}$$

$$h_l = (3.2\times10^{-6})(Q^2) + (3.1\times10^{-7})(Q^2) + (1.5\times10^{-7})(Q^2)$$

$h_l = 0.80 + 0.08 + 0.04 = 0.92$ for 500 GPM
$h_l = 3.2 + 0.31 + 0.15 = 3.66$ for 1000 GPM
$h_l = 12.8 + 1.24 + 0.6 = 14.64$ for 2000 GPM
$h_l = 21.63 + 2.10 + 1.01 = 24.74$ for 2600 GPM
$h_l = 28.80 + 2.79 + 1.35 = 32.94$ for 3000 GPM

IV $H_{REQ'D} = Z_1 - Z_2 + h_{l_{1-2}} + \frac{V_2^2}{2g}$ $\left[\frac{V^2}{2g} = \frac{\left(\frac{0.408 Q}{d^2}\right)^2 FT^2/SEC^2}{64.4\ FT/SEC^2}\right]$

a. $H_{REQ'D_{3000}} = 110 + 32.94 + 5.73 = 148.67$ FT for 3000 GPM

b. $H_{REQ'D_{2600}} = 110 + 24.74 + 4.31 = 139.05$ FT for 2600 GPM

c. $H_{REQ'D_{2000}} = 110 + 14.64 + 2.55 = 127.19$ FT for 2000 GPM

d. $H_{REQ'D_{1000}} = 110 + 3.66 + 0.64 = 114.3$ FT for 1000 GPM

e. $H_{REQ'D_{500}} = 110 + 0.92 + 0.16 = 111.08$ FT for 500 GPM

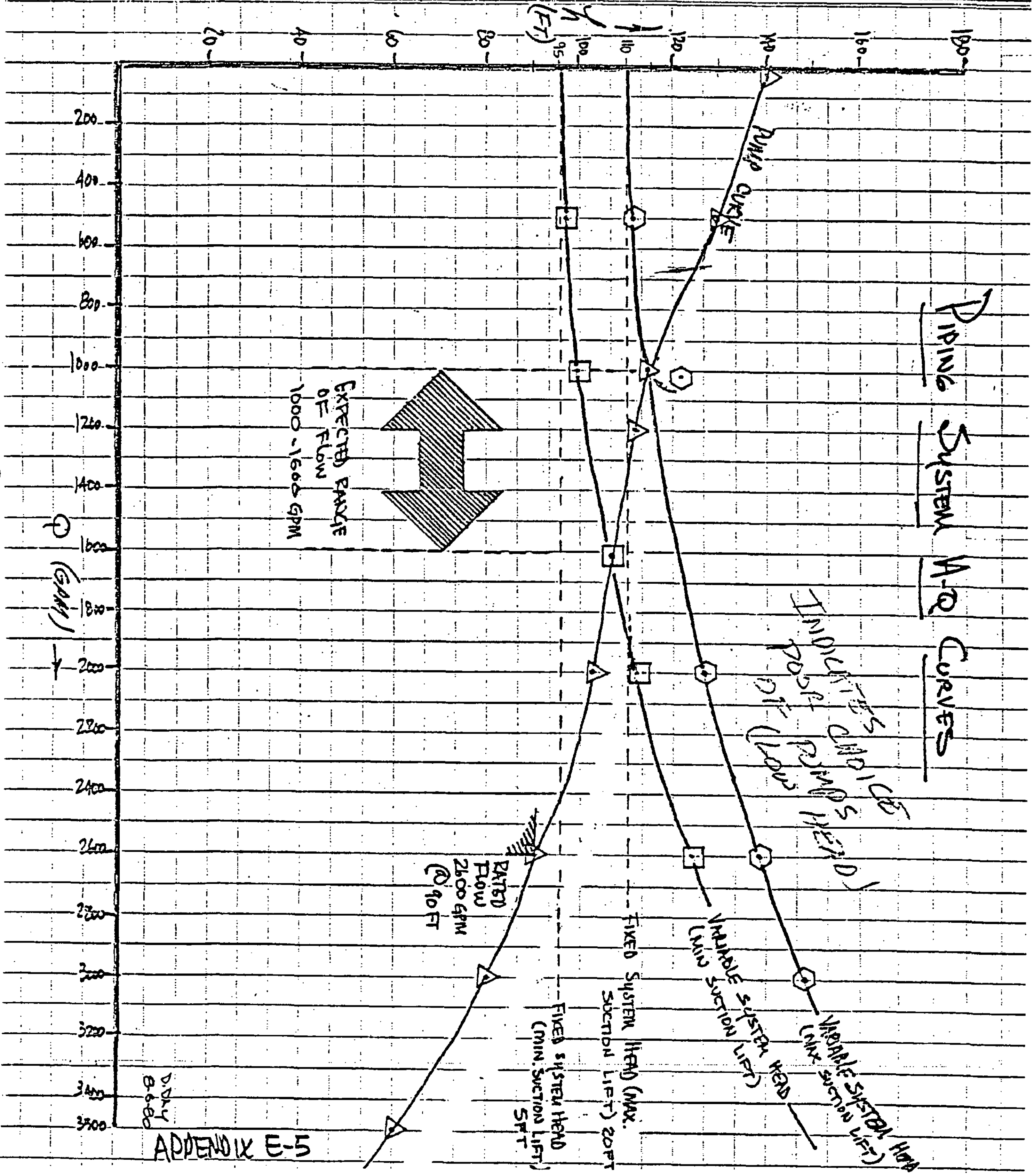
2746—12M—1-21-08

GEORGIA POWER COMPANY

Subject DEWATERING PUMP PERFORMANCE Project VOGTLE

POWER BLOCK SOUTHEAST CORNER - PUMP E

Computed by D. DAY Date 8-6-80 Checked by ____ Date ____



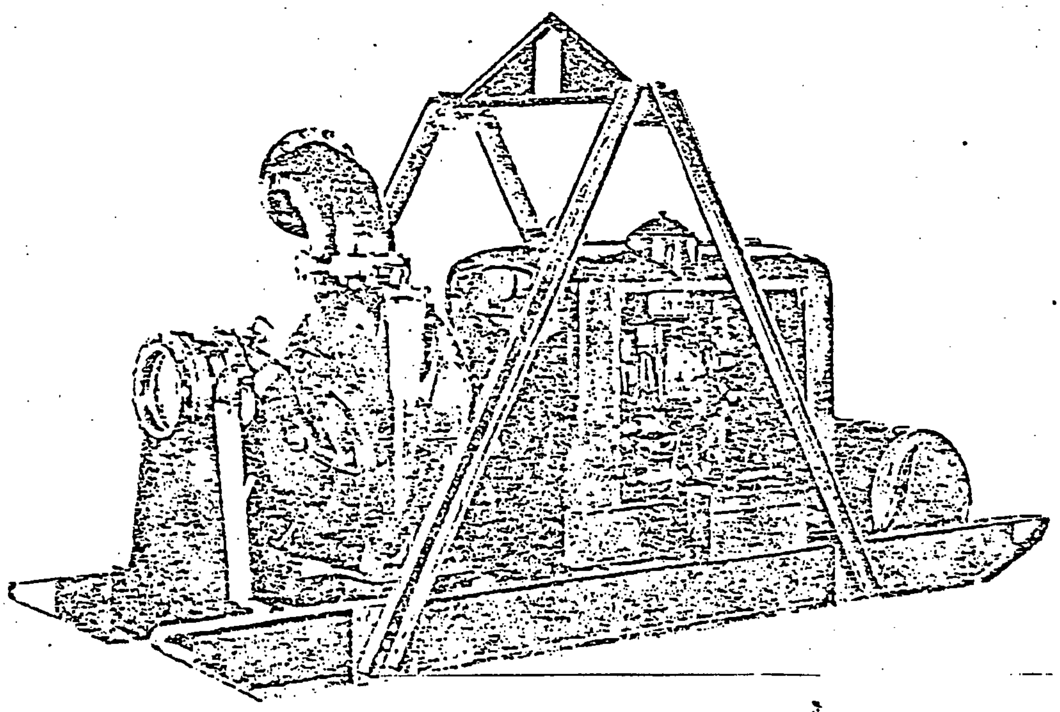
APPENDIX E-5

Self Priming Centrifugal

DIESEL ENGINE DRIVEN

MODEL T10A3-B-4031-C

Size 10" x 10"



PUMP SPECIFICATIONS

Size: 10" Suction - 10" Discharge
Suction: Standard fitted with NPT Flange
Discharge: Standard fitted with NPT Flange
All Flanges: Gray Iron No. 30 — Class 125 Rated
Casing: Gray Iron No. 30 — Hydrostatic Test Pressure 150 lbs. Maximum Operating Pressure 80 lbs.
Impeller; Open Type, 2-Vane: Ductile Iron No. 60-40-18. Handles 3" spherical solids
Clean Out Cover: Removable — Gray Iron No. 30
Wear Ring: Ductile Iron No. 120-90-02
Wear Plate: Steel
Shaft Sleeve: Type 304 Stainless Steel
Impeller Shaft: Type 17-4 PH Stainless Steel
Bearing Housing: Gray Iron No. 30
Ball Bearings: Sealed, permanently lubricated
Seal: Oil lubricated, mechanical, with tungsten titanium carbide seal rings
O-Rings: Neoprene
Gaskets: Buna/Cork

ENGINE SPECIFICATIONS

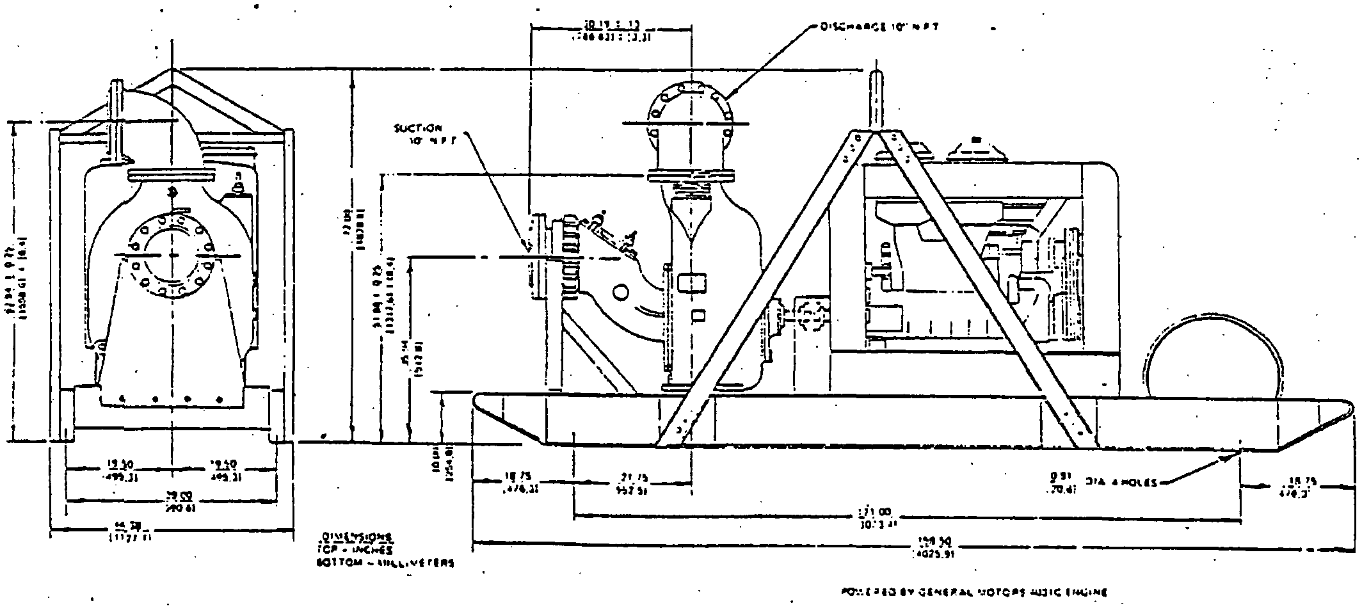
Model: GMC 4031C
Type: 4 cylinder, 2 cycle Diesel engine
Displacement: 284 cu. in.
Governor: Hydraulic
Lubrication: Forced Feed
Fuel Metering: Injectors
Air Cleaner: Oil Bath
Oil Reservoir: 17 Qt.
Starter: 12 volt electric
Fuel Tank: 55 Gal.
Engine Run Time: 9.55 Hours
GMC Published Performance
Max. Dyn. BHP 101 @ 1800 RPM
Max. Cont. BHP 85 @ 1600 RPM

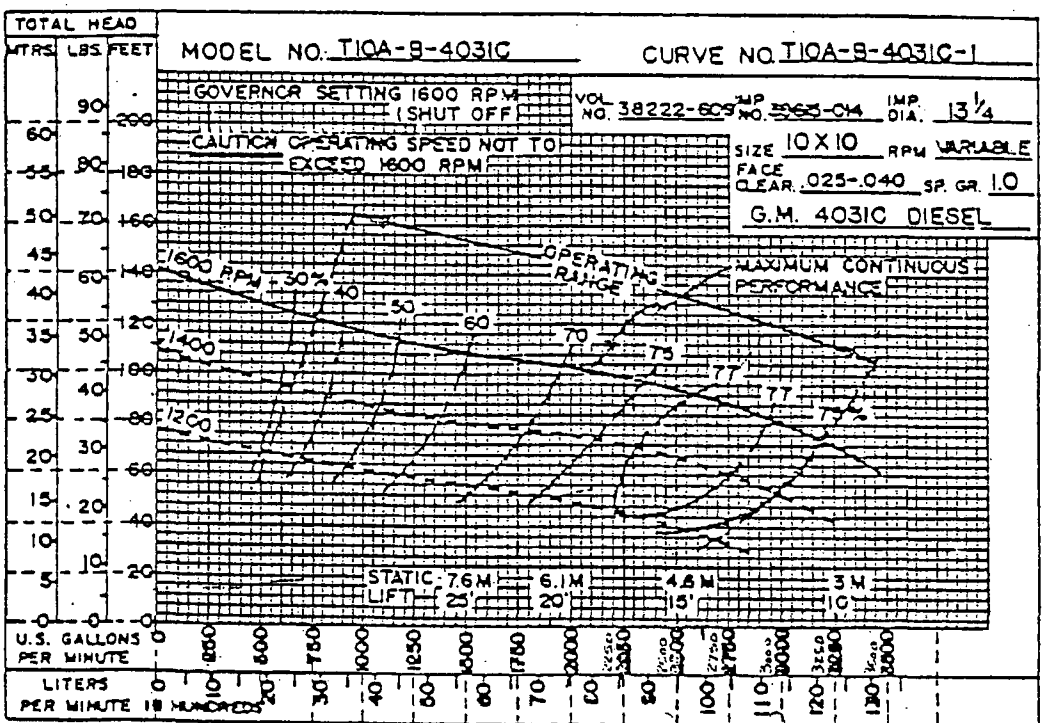
SPECIFICATION DATA

OVERALL DIMENSIONS and WEIGHTS APPROXIMATE

NET UNIT WEIGHT – 4900 lbs.

SHEET 7 OF 7





C-3-18-74

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APPENDIX E-7

2706—12M—1-31-66

GEORGIA POWER COMPANY

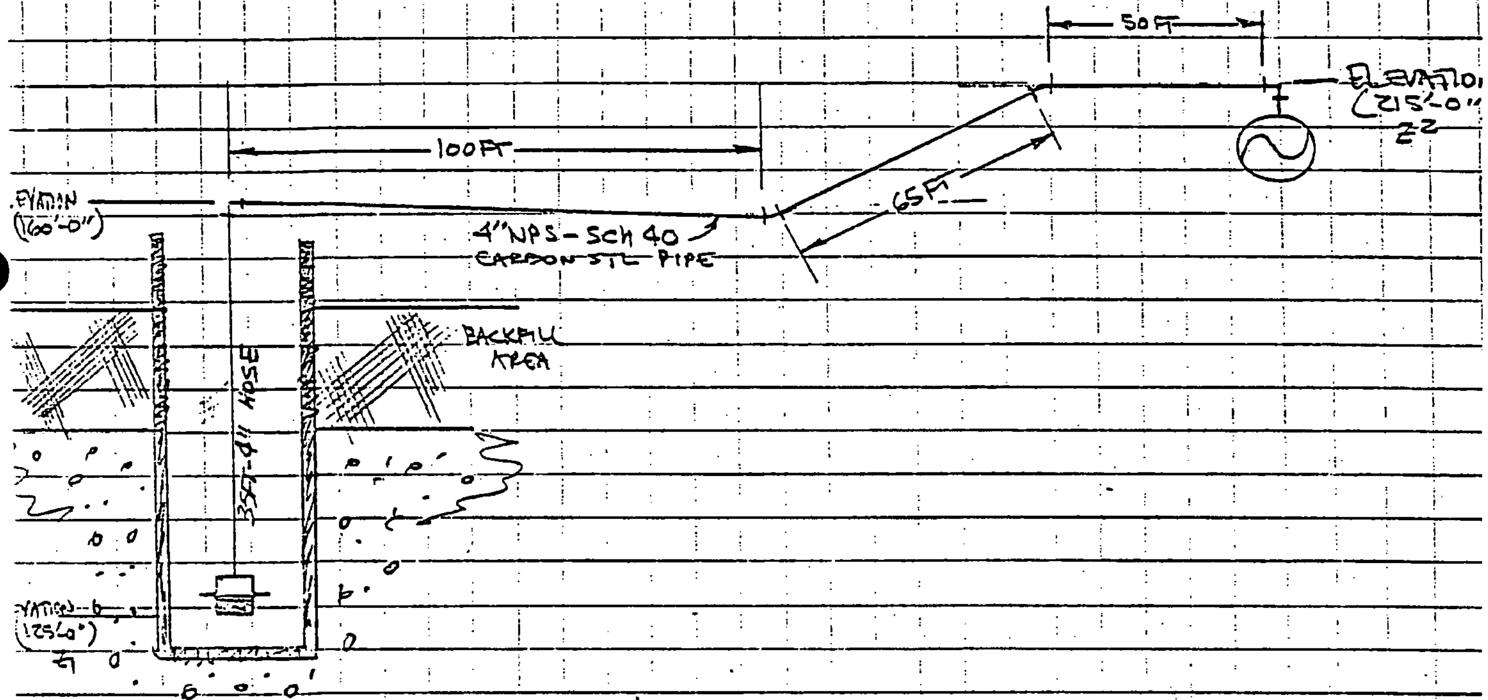
Sheet_____ of _____

Subject: DEWATERING PUMP PERFORMANCE — Project: VOGTLE

POWER BLOCK SOUTH SLOPE - BACKFILL AREA - PUMP F

Computed by: D. DAY — Date: 8-12-80 — Checked by: _____ — Date: _____

I. 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 THAN CAN BE EXPECTED........



PUMP SPECS: (SEE ATTACHED DATA SHEET)
GORMAN-RUPP MODEL S8A1-1
RATED 2600 GPM @ 55 FT HD

APPENDIX F-1

Subject ____ Project ____

Computed by ____ Date ____ Checked by ____ Date ____

II COMPUTE SYSTEM PIPING HEAD LOSS —

$$hf = \frac{0.00259\, KQ^2}{d4}$$

K = 30 FT for 90° ELs
K = 16 FT for 45° ELs
K = 1 for PIPE ENTRANCE
K = 1 for PIPE EXIT
$K = f\frac{L}{D}$ for PIPING

K for PIPE

$$K = f\frac{L}{D} = \frac{(0.017)(250)(12)}{4.026}$$

K = 12.67

K = (2)(.017)(30) = 1.02 for 2-90° ELs
K = (2)(.017)(16) = 0.54 for 2-45° ELs

K_{TOTAL} = 1.02 + 0.54 + 1.0 + 1.0 + 12.67

K_{TOTAL} = 16.23

PROPOSED 8" LINE

$$K = f\frac{L}{D}$$

$$K = \frac{(0.014)(250)(12)}{7.981}$$

K = 5.26

K_{TOTAL} = 8.82

2766—1252—1-31-66 GEORGIA POWER COMPANY Sheet____of____

Subject________________ Project________________

Computed by________ Date________ Checked by________ Date________

III COMPUTE hl OVER RANGE OF INTEREST –

$$hl = \frac{0.00259\,KQ^2}{d^4} \qquad K_{TOTAL} = 16.23$$

$$hl = \frac{(0.00259)(16.23)\,Q^2}{(4.026)^4}$$

$$hl = 1.6\times10^{-4}\,Q^2$$

$hl_{500} = 40$ FT $\qquad hl_{800} = 102.4$

$hl_{1000} = 160$ FT

$hl_{2000} = 640$ FT

$hl_{2600} = 1081.6$ FT

$hl_{3000} = 1440$ FT

INDICATES EXTRAORDINARY SYSTEM HEAD LOSSES (INCORRECT PIPE SIZE SELECTION)

RECOMMENDED 8" LINE

$K_{TOTAL} = 8.82$

$hl_{500} = 1.41$ FT

$hl_{1000} = 5.63$ FT

$hl_{2000} = 22.52$ FT

$hl_{2600} = 38.06$ FT

$hl_{3000} = 50.67$ FT

IV $HEAD_{REQ'D} = Z_1 - Z_2 + hl_{1-2} + \frac{V_2^2}{2g}$

$$\boxed{\frac{V_2^2}{2g} = \frac{\left(\frac{0.408\,Q}{d^2}\right)^2 \text{ FT}^2/\text{SEC}^2}{64.4 \text{ FT/SEC}^2}}$$

$H_{REQ'D\ 3000} = 90 + 1440 +$

$H_{REQ'D\ 2600} = 90 + 1081.6 +$

$H_{REQ'D\ 2000} = 90 + 640 +$

$H_{REQ'D\ 1000} = 90 + 160 +$

$H_{REQ'D\ 500} = 90 + 40 + 2.46 = 132.5$

RECOMMENDED 8" LINE

$H_{REQ'D\ 3000} = 90 + 50.67 + 5.73 = 1$

$H_{REQ'D\ 2600} = 90 + 38.06 + 4.31 = 13$

$H_{REQ'D\ 2000} = 90 + 22.52 + 2.55 = 11$

$H_{REQ'D\ 1000} = 90 + 5.63 + 0.64 = 9$

$H_{REQ'D\ 500} = 90 + 1.41 + 0.16 = 91$

APPENDIX F-3

2766—12M—1-31-66

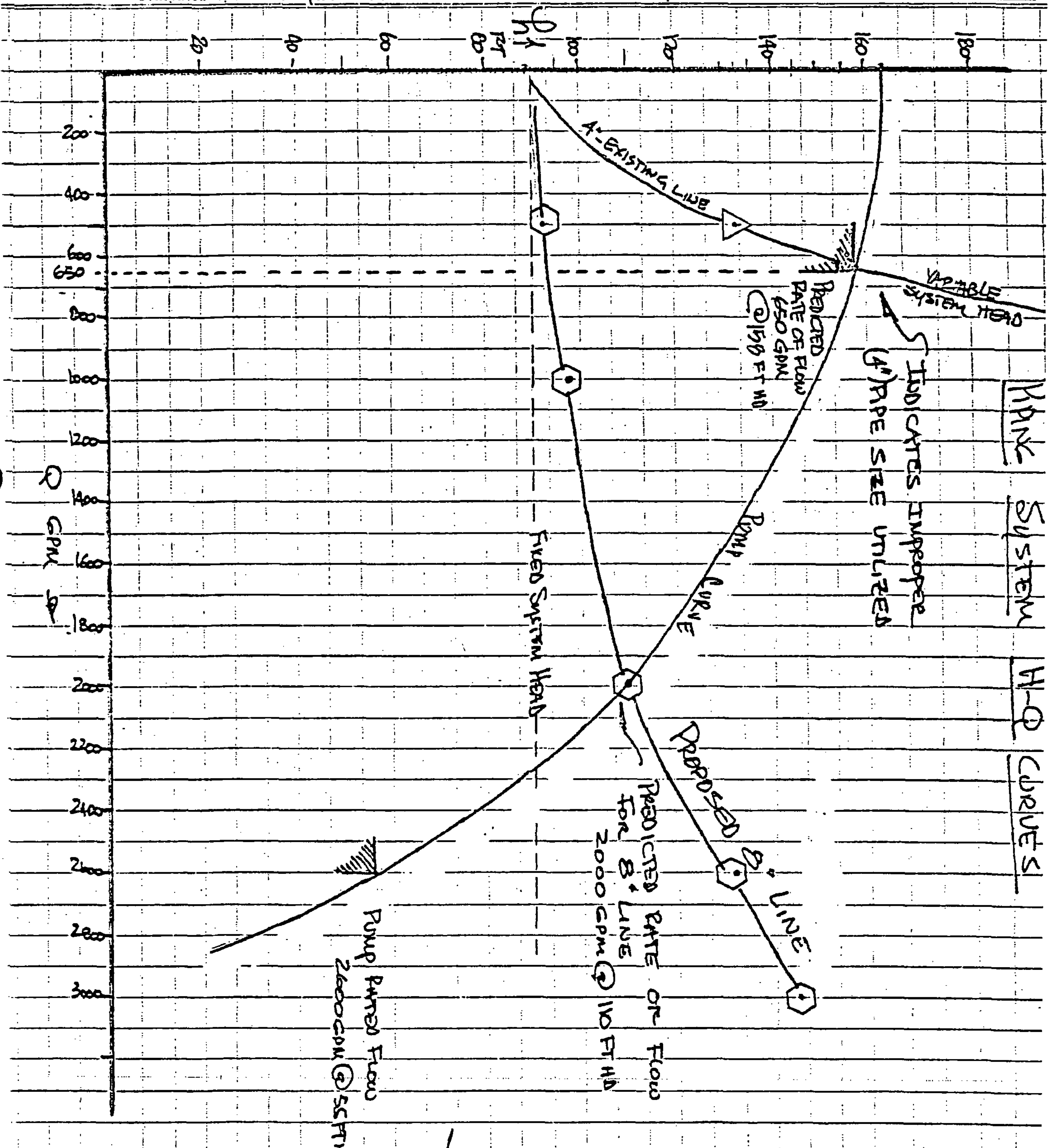
GEORGIA POWER COMPANY

Sheet ______ of ______

Subject: DEWATERING PUMP PERFORMANCE — POWER BLOCK SOUTH SLOPE - PUMP F

Project: VOGTLE

Computed by: D. DAY Date: 8-13-80 Checked by: ______ Date: ______



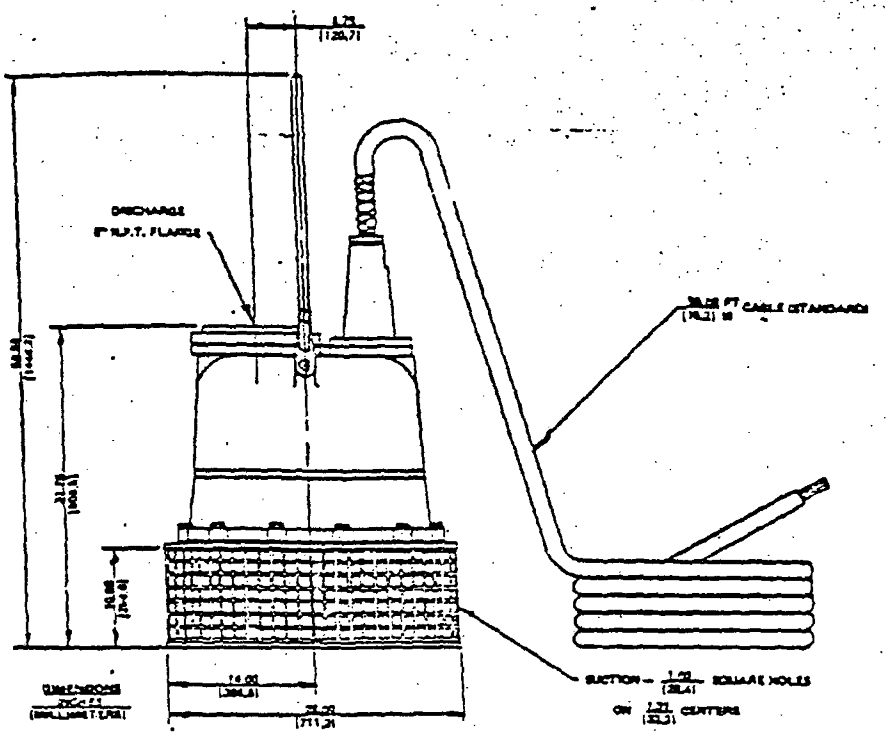
APPENDIX F4

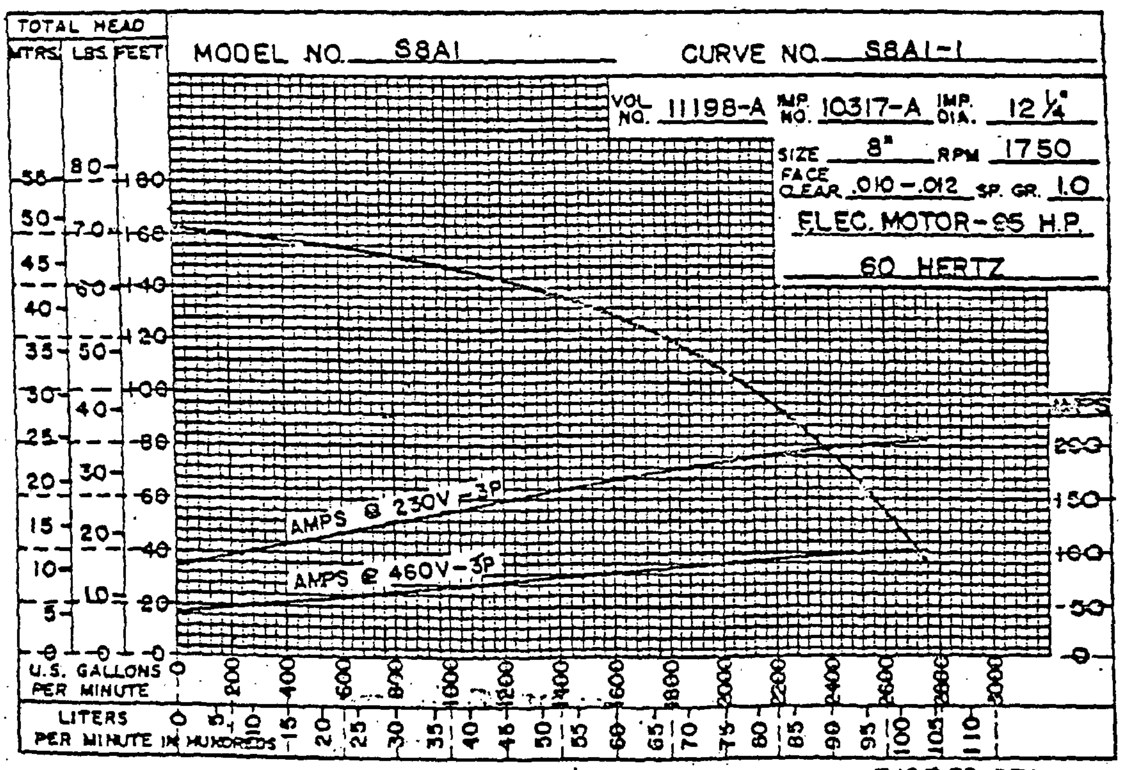
SPECIFICATION DATA

OVERALL DIMENSIONS and WEIGHTS APPROXIMATE

SECTION 130, PAGE 700

PUMP AND CABLE: Net Weight 835 lbs.
Shipping Weight 1070 lbs.
MOTOR CONTROL: Shipping Weight 150 lbs.
EXPORT SIZE: Pump; 36.0 Cu. Ft.





T-10-10-75-REV.

THE GORMAN-RUPP COMPANY • MANSFIELD, OHIO

GORMAN-RUPP OF CANADA LIMITED • ST. THOMAS, ONTARIO, CANADA

APPENDIX F-5

2766—12M—1-31-66

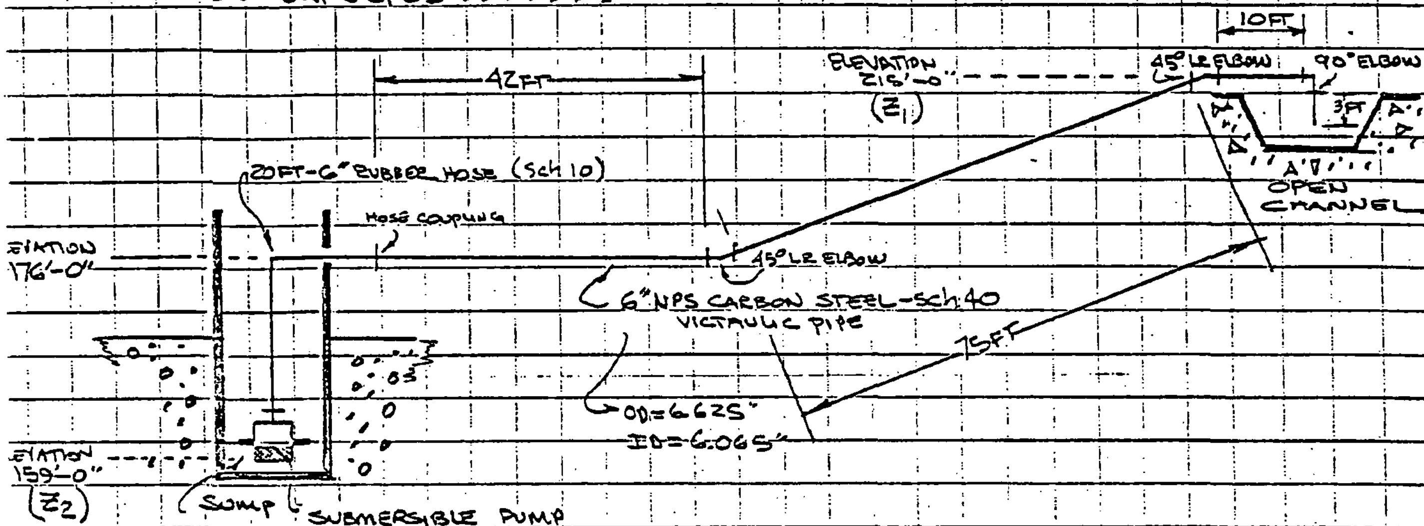
GEORGIA POWER COMPANY

Subject DEWATERING PUMP PERFORMANCE Project VOGTLE

POWER BLOCK NORTHEAST CORNER - PUMP G

Computed by D. DAY Date 8-6-80 Checked by ______ Date ______

I. 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 S8A1-1

RATED - 2600 GPM @ 55 FT HD

$d = 6.065_{CSP}$ & 6.357_{HOSE}

$f_T = 0.015$

APPENDIX G-1

2766—12M—1-31-66

GEORGIA POWER COMPANY

Subject ______ Project ______

Computed by ______ Date ______ Checked by ______ Date ______

SOLUTION:

I. FORMULAE

SEE FIGURE 4

II. COMPUTE SYSTEM PIPING HEAD LOSS –

$$hl = \frac{0.00259\,K\,Q^2}{d^4}$$

$K = 25 f_T$ LONG RADIUS 45° ELBOWS

$K = 30 f_T$ 90° ELBOWS

$K = 1.0$ PIPE EXIT

$K = f\frac{L}{D}$ STRAIGHT PIPE

PUMP FLANGE REDUCER, HOSE COUPLING AND VICTAULIC COUPLINGS NOT CONSIDERED

APPENDIX G-2

II CONT'D

K FOR PIPE —

$K = f_1 \frac{L}{D} + f_2 \frac{L}{D}$ (FOR HOSE & CS PIPE)

$f_1 = 0.015$ FOR CARBON STL; $f_2 = 0.02$ FOR RUBBER HOSE (ESTIMATED)

$\therefore K = \frac{(0.015)(120)(12)}{6.065} + \frac{(0.02)(20)(12)}{6.357}$ (USED Sch 10S FOR HOSE ID)

K = 4.32 FOR PIPING

$f_T = 0.015$ (ESTIMATED ALL COMPONENTS)

$K = (2)(0.015)(25) = 0.75$ FOR 2-45° LR ELS
$K = (2)(0.015)(30) = 0.9$ FOR 2-90° ELS

$K_{TOTAL} = 0.75 + 0.9 + 4.32 = 5.97$

III. COMPUTE hl OVER RANGE OF INTEREST

$hl = \frac{0.00259 KQ^2}{d4}$

$hl = \frac{(0.00259)(5.97)(Q)^2}{(6.065)^4} = (1.143 \times 10^{-5})(Q)^2$

$hl = (1.143 \times 10^{-5})(100)^2 = .11$ FT for 100 GPM
$hl = (1.143 \times 10^{-5})(500)^2 = 2.86$ FT for 500 GPM
$hl = (1.143 \times 10^{-5})(1000)^2 = 11.43$ FT for 1000 GPM
$hl = (1.143 \times 10^{-5})(2000)^2 = 45.72$ FT for 2000 GPM
$hl = (1.143 \times 10^{-5})(2600)^2 = 77.27$ FT for 2600 GPM

Subject ____ Project ____

Computed by ____ Date ____ Checked by ____ Date ____

$$\frac{V^2}{2g} = \frac{\left(\frac{0.408Q}{d^2}\right)^2 FT^2/SEC^2}{64.4\ FT/SEC^2}$$

IV $H_{REQ'D} = (Z_1 - Z_2) + h_{l_{1-2}} + \frac{V^2}{2g}$

a. $H_{REQ'D_{2600}} = 56 + 77.27 + 12.91 = 146.18$ FT for 2600 GPM

b. $H_{REQ'D_{2000}} = 56 + 45.72 + 7.64 = 109.36$ FT for 2000 GPM

c. $H_{REQ'D_{1000}} = 56 + 11.43 + 1.91 = 69.34$ FT for 1000 GPM

d. $H_{REQ'D_{500}} = 56 + 2.86 + 0.48 = 59.34$ FT for 500 GPM

2766—12M—1-31-66

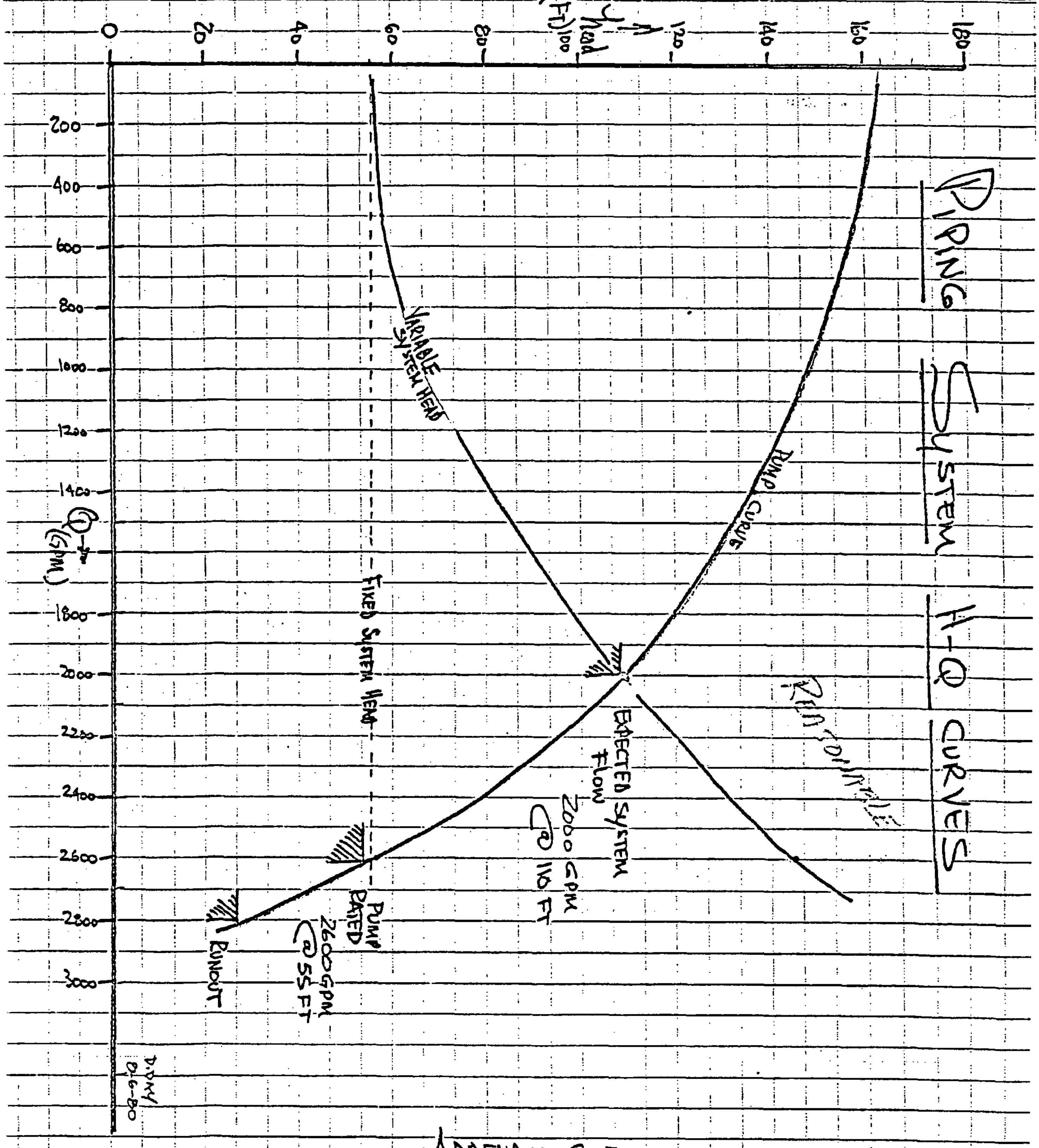
GEORGIA POWER COMPANY

Sheet 5 of 6

Subject DEWATERING PUMP PERFORMANCE POWER BLOCK NORTHEAST CORNER – PUMP G

Project VOGTLE

Computed by D. DAY Date 8-6-80 Checked by _____ Date _____



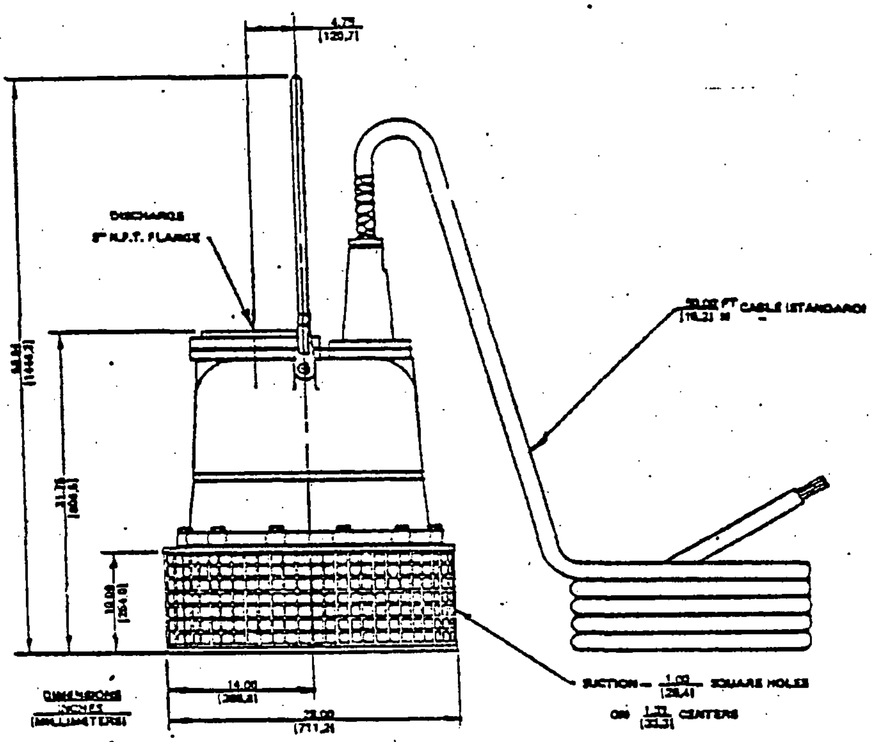
APPENDIX G-5

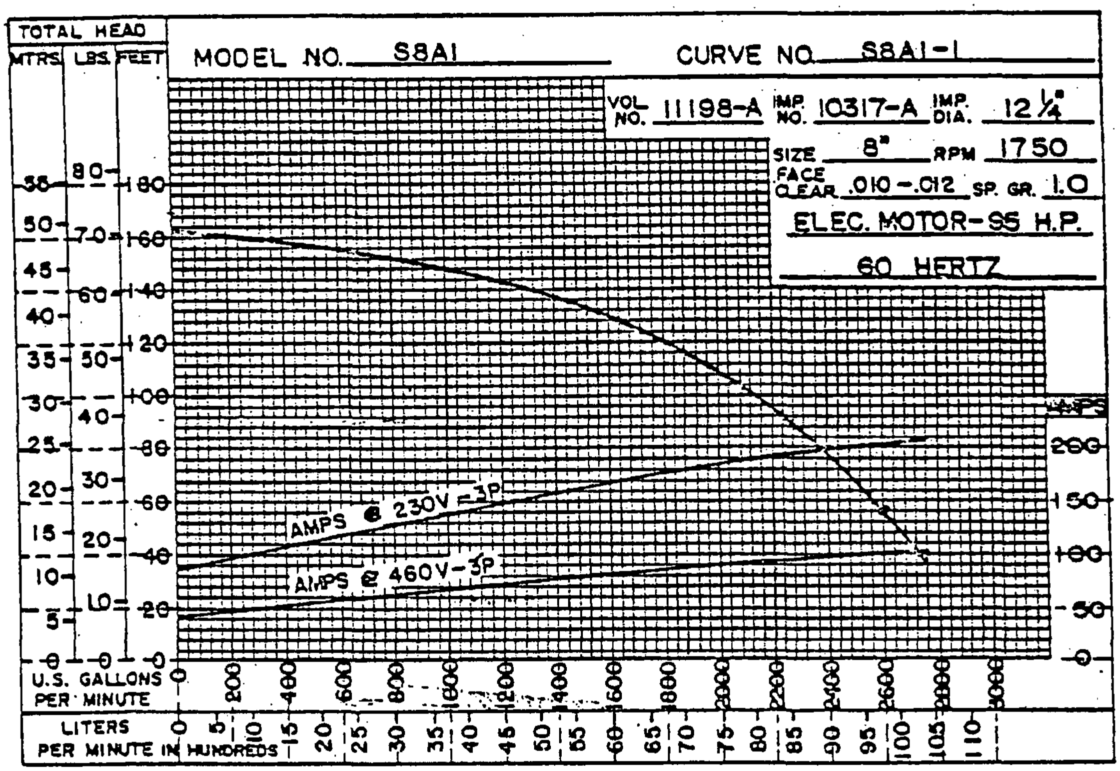
SPECIFICATION DATA

OVERALL DIMENSIONS and WEIGHTS APPROXIMATE

SECTION 130, PAGE 700

PUMP AND CABLE: Net Weight 935 lbs.
Shipping Weight 1070 lbs.
MOTOR CONTROL: Shipping Weight 150 lbs.
EXPORT SIZE: Pump: 36.0 Cu. Ft.





T-10-10-75-REV.

2766—12M—1-81-65

GEORGIA POWER COMPANY

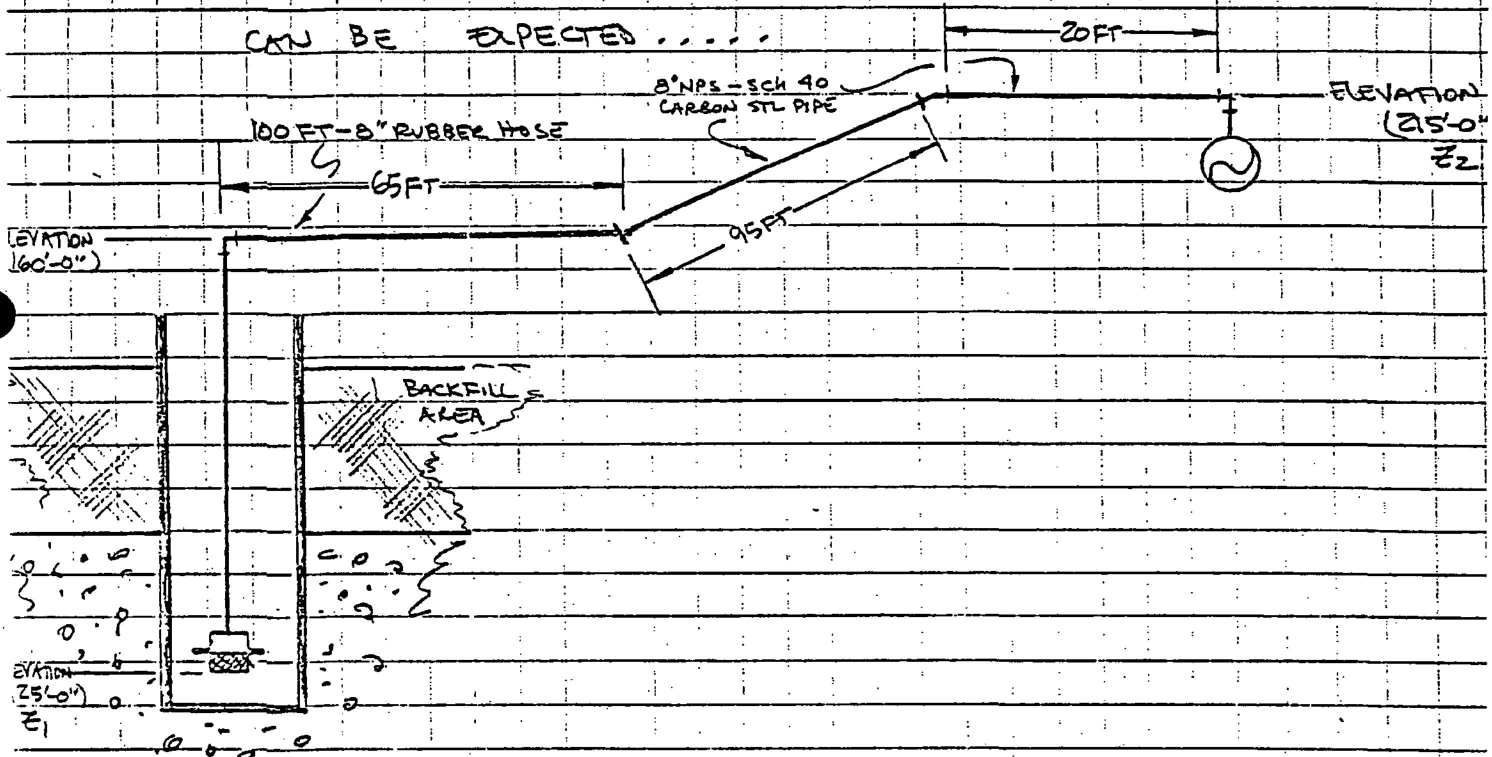
Sheet ____ of ____

Subject: DEWATERING PUMP PERFORMANCE — Project: VOGTLE

POWER BLOCK SOUTH SLOPE – BACKFILL AREA – PUMP H

Computed by: D. DAY — Date: 8-8-86 — 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 . . . . .



PUMP SPECS: (SEE ATTACHED DATA SHEET)

GORMAN-RUPP MODEL S8A1-1

RATED 2600 GPM @ 55 FT HD

ADDENDUM H-1

3766—12M—1-61-66 GEORGIA POWER COMPANY Sheet ____ of ____

Subject ____ Project ____

Computed by ____ Date ____ Checked by ____ Date ____

II COMPUTE SYSTEM PIPING HEAD LOSS –

$$h_l = \frac{0.00259\,KQ^2}{d^4}$$

K = 30 FT for 90° ELBOW
K = 16 FT for 45° ELBOW
K = 1 for PIPE ENTRANCE
K = 1 for PIPE EXIT
$K = f\frac{L}{D}$ for PIPING

K for PIPE

$$K = f\frac{L}{D} = \frac{(0.015)(215)(12)}{7.981}$$

K = 4.85

K = (2)(30)(0.015) = 0.90 for 2-90° ELS
K = (2)(16)(0.015) = 0.48 for 2-45° ELS

$K_{TOTAL} = 0.90 + 0.48 + 1.0 + 1.0 + 4.85 = 8.23$

APPENDIX H-2

III COMPUTE h_l OVER RANGE OF INTEREST —

$$h_l = \frac{0.00259 K Q^2}{d^4} \qquad K_{TOTAL} = 8.23$$

$$h_l = \frac{(0.00259)(8.23)(Q^2)}{(7.981)^4}$$

$$h_l = (5.25 \times 10^{-6})(Q^2)$$

$h_{l_{500}} = 1.31$ FT for 500 GPM

$h_{l_{1000}} = 5.25$ FT for 1000 GPM

$h_{l_{2000}} = 21.02$ FT for 2000 GPM

$h_{l_{2600}} = 35.52$ FT for 2600 GPM

$h_{l_{3000}} = 47.28$ FT for 3000 GPM

IV $H_{REQ'D} = Z_1 - Z_2 + h_{l_{1-2}}' + \frac{V_2^2}{2g}$ $\qquad \left[\frac{V_2^2}{2g} = \frac{\left(\frac{0.408Q}{d^2}\right)^2 \text{ FT}^2/\text{SEC}^2}{64.4 \text{ FT/SEC}^2} \right.$

$H_{REQ'D\ 3000} = 47.28 + 90 + 5.73 = 143.01$ FT for 3000 GPM

$H_{REQ'D\ 2600} = 35.52 + 90 + 4.31 = 129.83$ FT for 2600 GPM

$H_{REQ'D\ 2000} = 21.02 + 90 + 2.55 = 113.57$ FT for 2000 GPM

$H_{REQ'D\ 1000} = 5.25 + 90 + 0.64 = 95.89$ FT for 1000 GPM

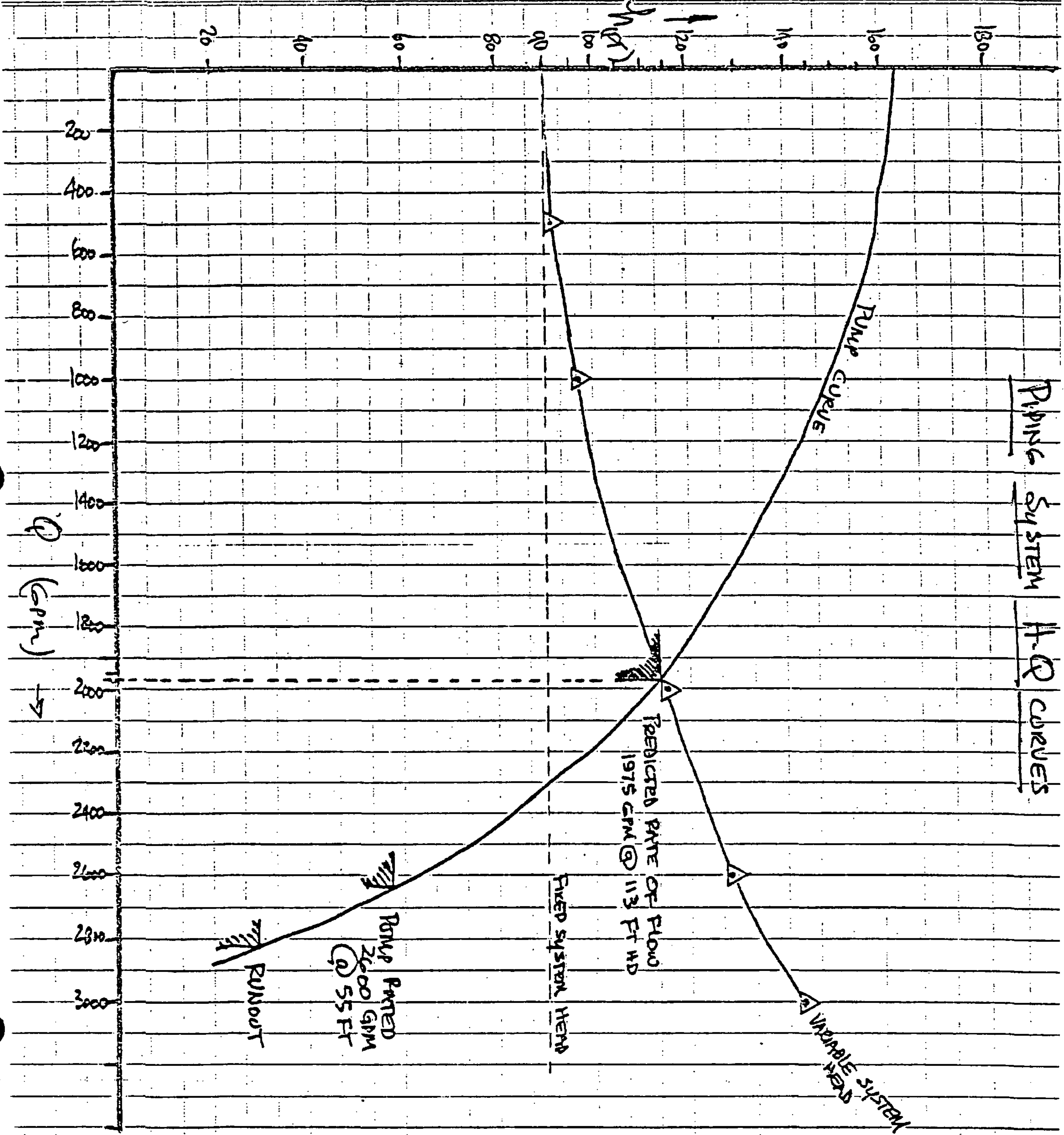
$H_{REQ'D\ 500} = 1.31 + 90 + 0.16 = 91.47$ FT for 500 GPM

27662-12M—1-31-66 GEORGIA POWER COMPANY Sheet ____ of ____

Subject: DEWATERING PUMP PERFORMANCE Project: VOGTLE

POWER BLOCK SOUTH SLOPE - BACKFILL AREA - PUMP H

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



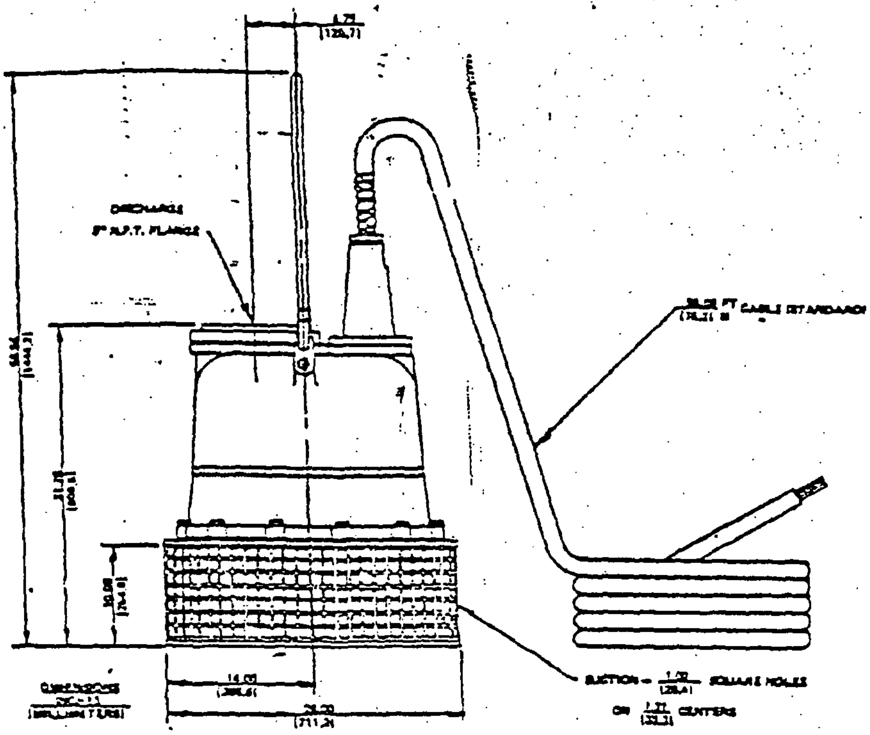
APPENDIX H-4

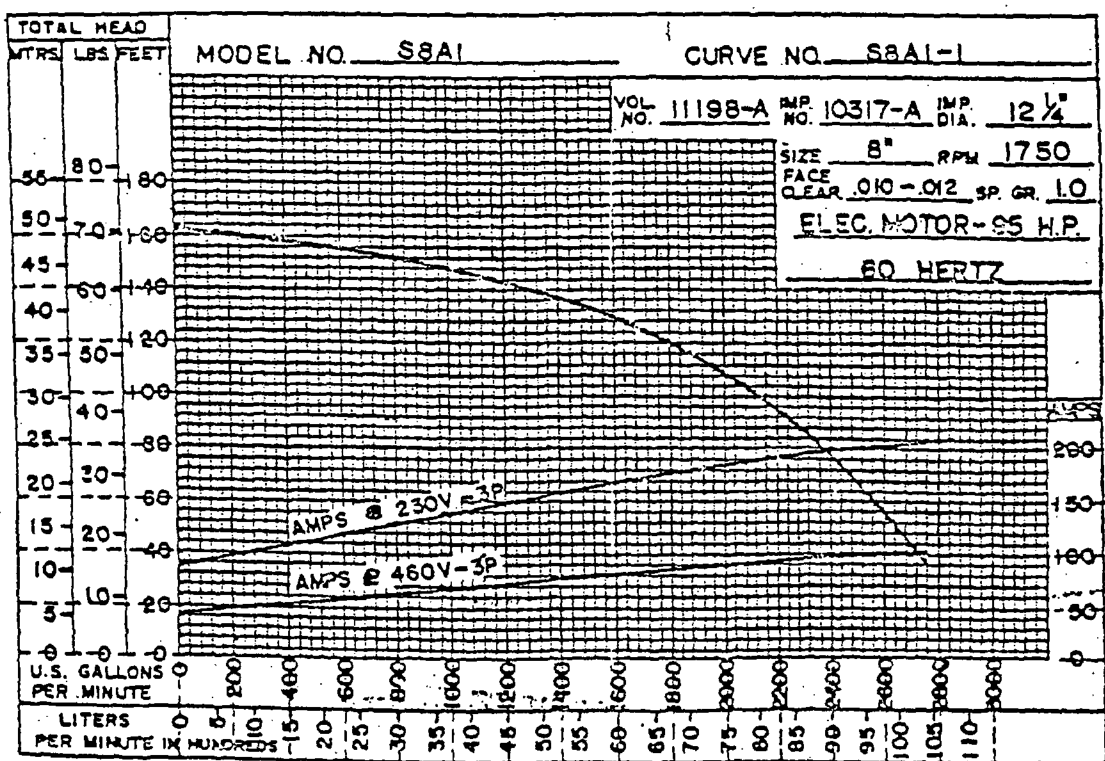
SPECIFICATION DATA

OVERALL DIMENSIONS and WEIGHTS APPROXIMATE

SECTION 130, PAGE 700

PUMP AND CABLE: Net Weight 935 lbs.
Shipping Weight 1070 lbs.
MOTOR CONTROL: Shipping Weight 150 lbs.
EXPORT SIZE: Pump: 36.0 Cu. Ft.





T-10-10-75-REV.

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GORMAN-RUPP OF CANADA LIMITED • ST. THOMAS ONTARIO CANADA

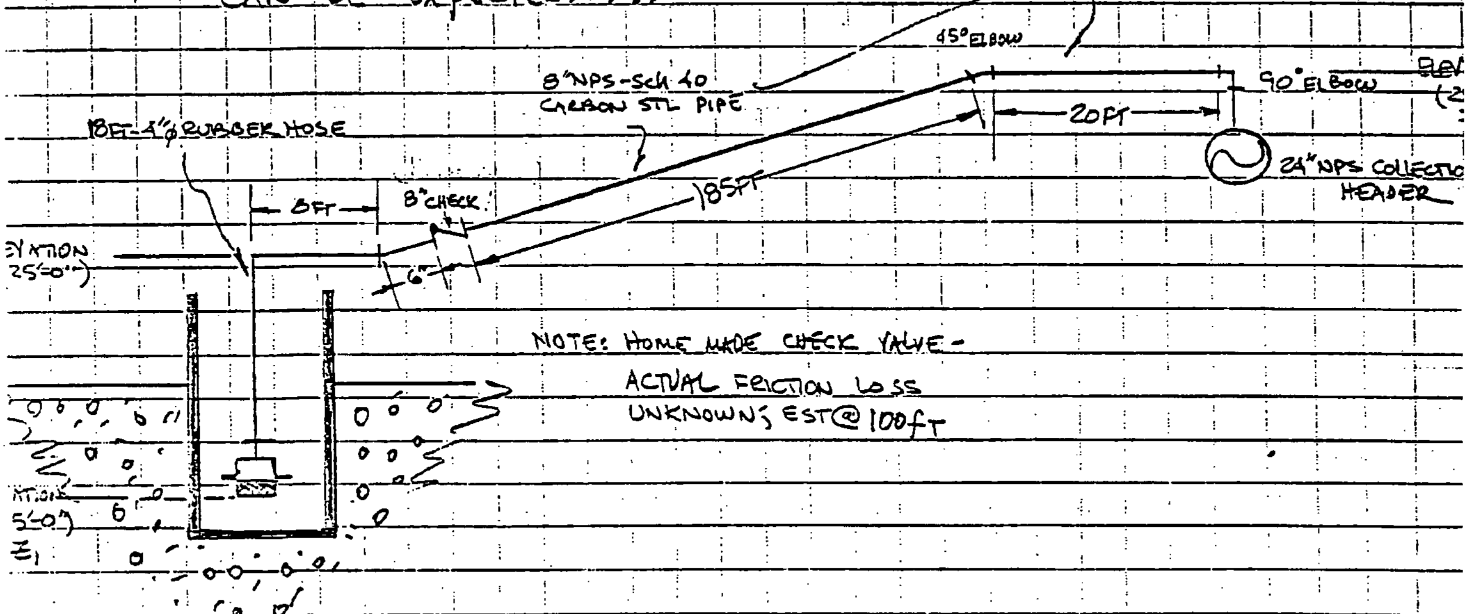
Appendix II-E

Subject: DEWATERING PUMP PERFORMANCE — POWER BLOCK SOUTH SLOPE - PUMP J

Project: VOGTLE

Computed by: D. DAY Date: 8-8-80 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.....



PUMP SPECS: (SEE DATA SHEET ATTACHED)

GORMAN-RUPP MODEL S4B1

RATED 900 GPM @ 90 FT HD

$d_1 = 4.026$ $d_2 = 7.981$

$f_{T_1} = 0.017$ $f_{T_2} = 0.014$

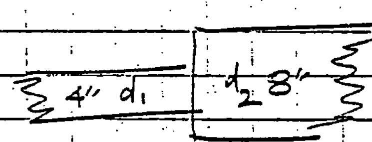
APPENDIX J-1

II COMPUTE SYSTEM HEAD LOSS –

$$h_l = \frac{0.00259\, K Q^2}{d^4}$$

K = $30 f_T$ for 90° ELBOW
K = 1 for PIPE EXIT
K = 1 for PIPE ENTRANCE
K = $100 f_T$ for home made CHECK VALVE
K_2 = 9.0 for SUDDEN ENLARGEMENT
K = $f \frac{L}{D}$ for piping
K = $16 f_T$ for 45° ELBOW

SUDDEN ENLARGEMENT



$\beta = \frac{d_1}{d_2} = 0.$

CRANE A

$$K_2 = \frac{(1-\beta^2)^2}{\beta^4} = 9$$

K for pipe –

$$K = f_{T1}\frac{L}{D} + f_{T2}\frac{L}{D}$$

$$K = \frac{(0.017)(18)(12)}{4.026} + \frac{(0.014)(206)(12)}{7.981}$$

K = 0.91 + 4.34 = 5.25

K = (2)(30)(.014) = 0.84 for 2-90° ELS
K = (1)(16)(.014) = 0.22 for 1-45° EL
K = (1)(100)(.014) = 1.4 for 1-CHECK VALVE

K_{TOTAL} = 0.84 + 1.0 + 1.0 + 1.4 + 9.0 + 5.25 + 0.22 = 18.71

Subject ______ Project ______

Computed by ______ Date ______ Checked by ______ Date ______

III COMPUTE h_l OVER RANGE OF INTEREST —

$$h_l = \frac{0.00259\,KQ^2}{d^4} \qquad K_{TOTAL} = 18.71$$

$$h_l = \frac{0.00259(0.91)(Q^2)}{(4.026)^4} + \frac{0.00259(4.34)(Q^2)}{(7.981)^4}$$

$$h_l = (8.97\times10^{-6})(Q^2) + (2.77\times10^{-6})(Q^2)$$

$h_l = 0.36 + 0.11 = 0.47$ FT for 200 GPM
$h_l = 1.44 + 0.44 = 1.88$ FT for 400 GPM
$h_l = 3.23 + 1.00 = 4.23$ FT for 600 GPM
$h_l = 5.74 + 1.77 = 7.51$ FT for 800 GPM
$h_l = 8.97 + 2.77 = 11.74$ FT for 1000 GPM

IV $H_{REQ'D} = Z_1 - Z_2 + h_{l_{1-2}} + \frac{V_2^2}{2g}$

$$\boxed{\frac{V_2^2}{2g} = \frac{\left(\frac{0.408\,Q}{d^2}\right)^2 \text{ FT}^2/\text{SEC}^2}{64.4 \text{ FT/SEC}^2}}$$

a. $H_{REQD_{1000}} = 100 + 11.74 + 0.64 = 112.38$ FT

b. $H_{REQD_{800}} = 100 + 7.51 + 0.41 = 107.92$ FT

c. $H_{REQD_{600}} = 100 + 4.23 + 0.23 = 104.46$ FT

d. $H_{REQD_{400}} = 100 + 1.88 + 0.10 = 101.98$ FT

e. $H_{REQD_{200}} = 100 + 0.47 + 0.02 = 100.49$ FT

$R_{e_{8''}} = 123.9\,\frac{dv\rho}{\mu} = 2.8\times10^5$

friction factor by Crane A-25 = 0.017 for 8"

APPENDIX J-3

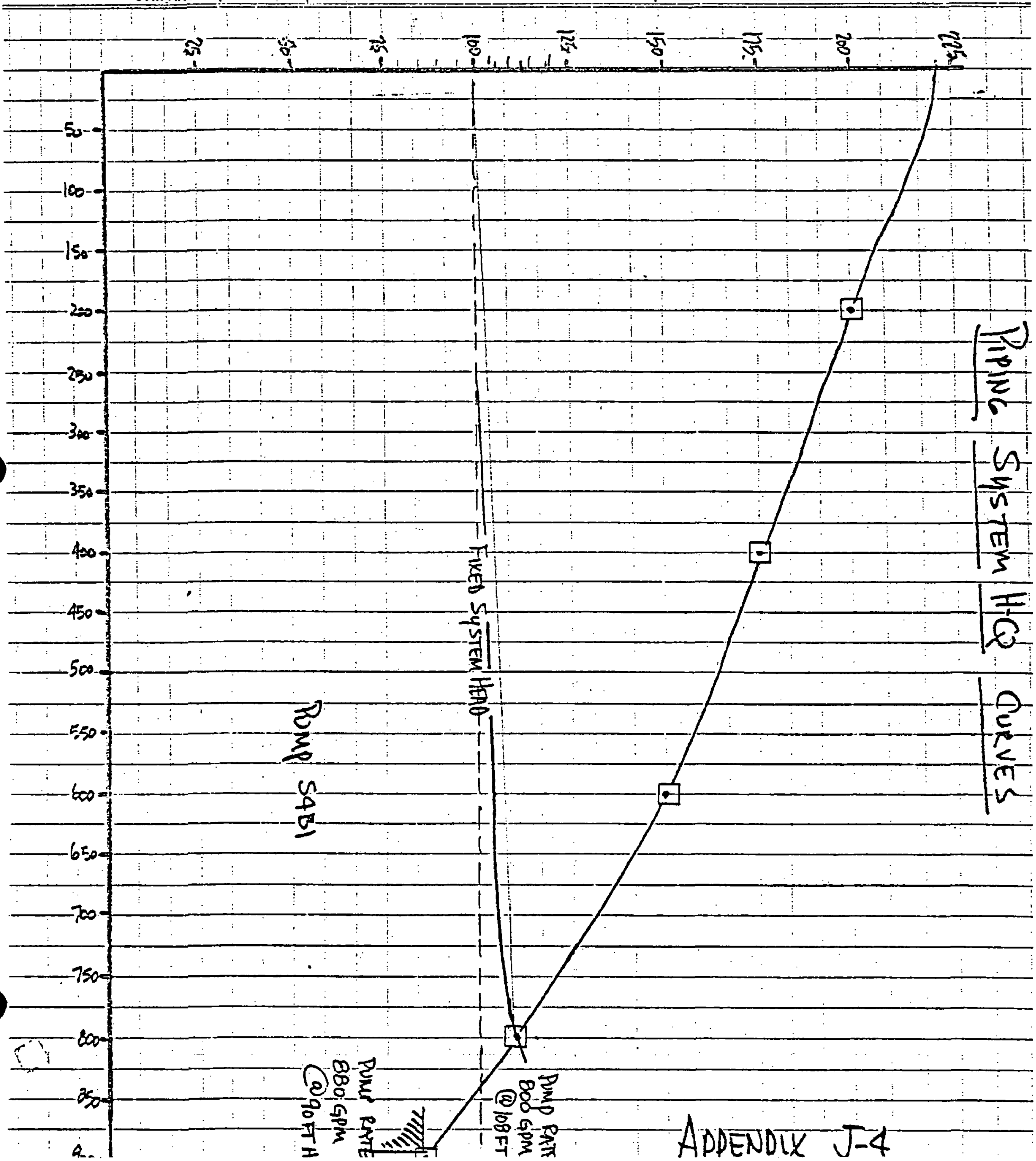
GEORGIA POWER COMPANY

Sheet____ of ____

Subject: DEWATERING PUMP PERFORMANCE
POWER BLOCK SOUTH SLOPE – PUMP J

Project: VOGTLE

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



APPENDIX J-4

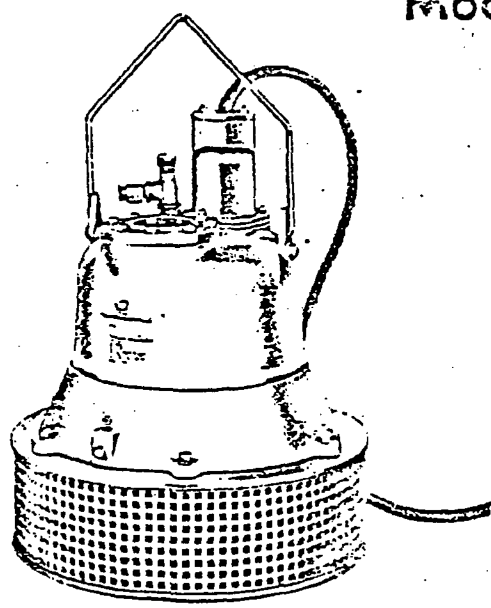
SPECIFICATION DATA

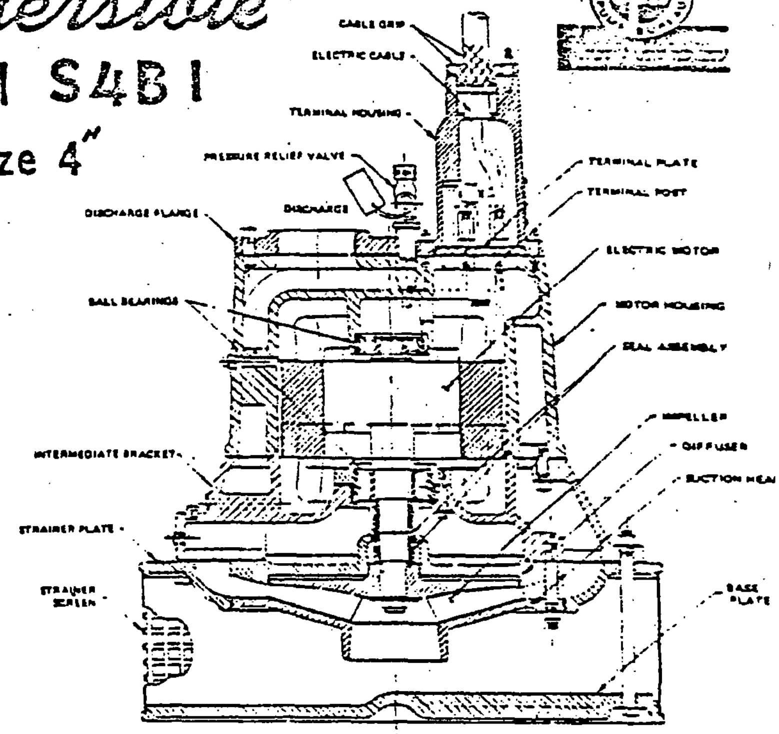
ELECTRIC MOTOR DRIVEN

Submersible

Model S4B1

Size 4"



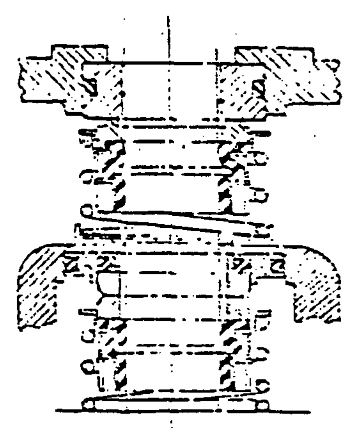


S4B1

PART	MATERIAL
Motor Housing	Aluminum
Intermediate Bracket	Aluminum
Impeller	Ductile Iron
Suction Head	Ductile Iron
Diffuser	Ductile Iron
Hardware (Internal)	Stainless Steel
Bearings, Ball	1 Single, 1 Double Row
Control Box	Standard
Hoisting Bail	Standard
Strainer 358 sq.in.area	5/8 inch sq. openings
Staging Adapter	Optional
Liquid level switch	Optional
Liquid level control	Optional

WARNING
DO NOT USE IN EXPLOSIVE ATMOSPHERE OR FOR PUMPING VOLATILE LIQUIDS

SEAL – Two. Operate in Oil.
UPPER – Carbon and Ni-Resist Wearing Faces
LOWER – Tungsten Carbide Wearing Faces.



MOTOR: Oil Filled, 50 H.P., 60 Hertz, 1750 RPM.
Max. KW Input: 43 KW

Available: Three Phase, 460 and 575 Volts.

CABLE: 6 Wire, 3 Conductor, 3 ground, No. 6 AWG, Type GGC. Available in lengths required. 50 Ft. provided standard. For different lengths, specify at time of order.

STANDARD CONTROL BOX: Type 3R Rainproof Starter, 3 pole with 3 coil overload relay and 3 pole magnetic trip circuit breaker; three position selector switch; automatic off-manual.

Max. Temp. of liquid pumped, 120°F

Recommended generator for across the line starting, 60 KW.

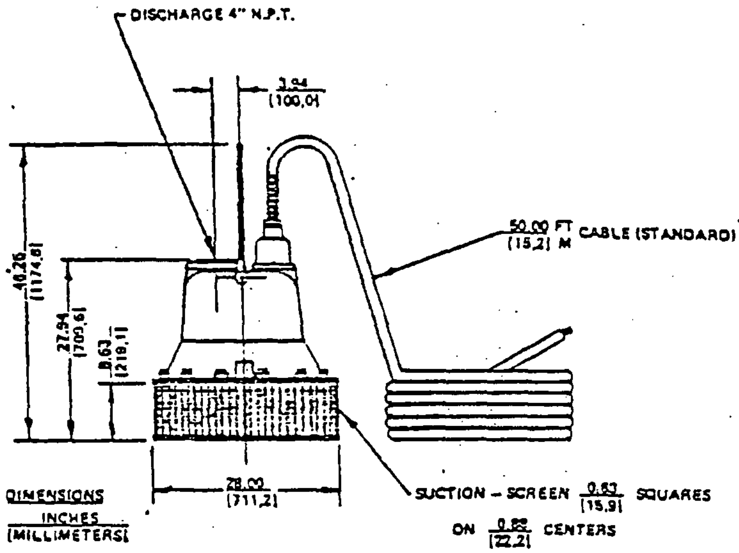
Appendix J-5

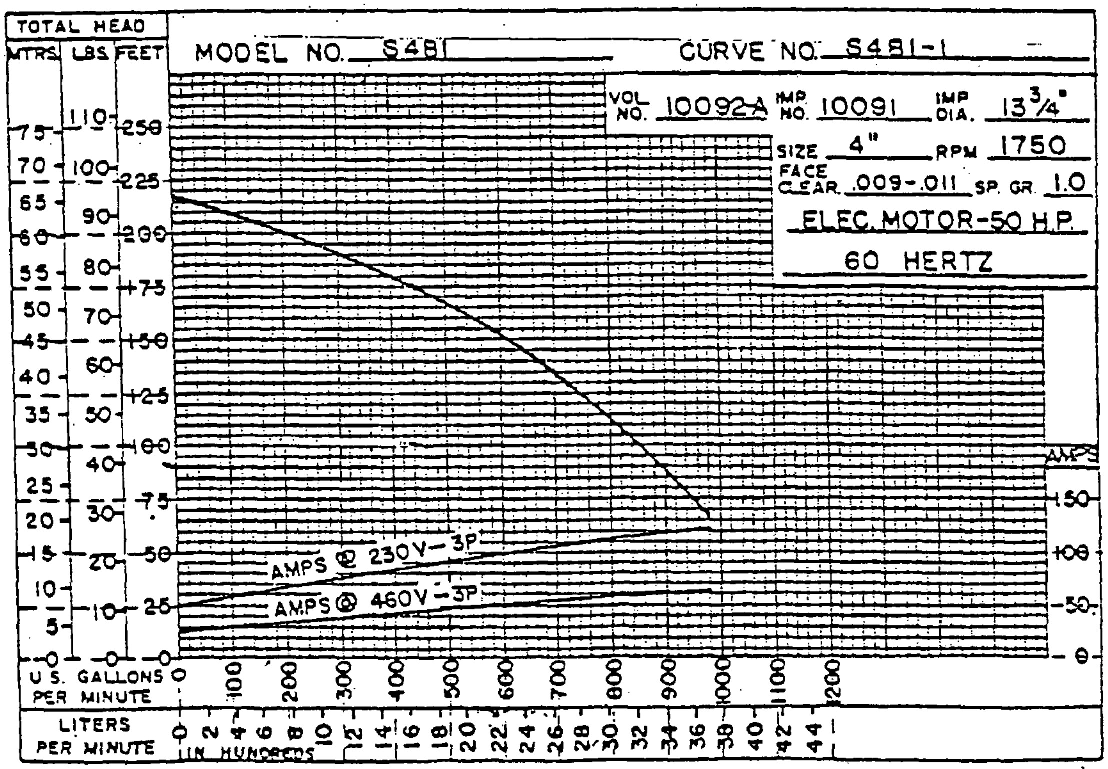
SPECIFICATION DATA

OVERALL DIMENSIONS and WEIGHTS APPROXIMATE

SECTION 130, PAGE 400

PUMP AND CABLE:	Net Weight	
	460 & 575V	— 400 lbs.
	Shipping Weight	
	460 & 575V	— 530 lbs.
MOTOR CONTROL:	Shipping Weight	
	460 & 575V	— 140 lbs.
EXPORT SIZE:	Pump:	31 cu. ft.
	Control:	9.5 cu. ft.





880 GPM @ 90 FT HD

T-10-9-75-REV.

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APPENDIX J-6

GEORGIA POWER COMPANY

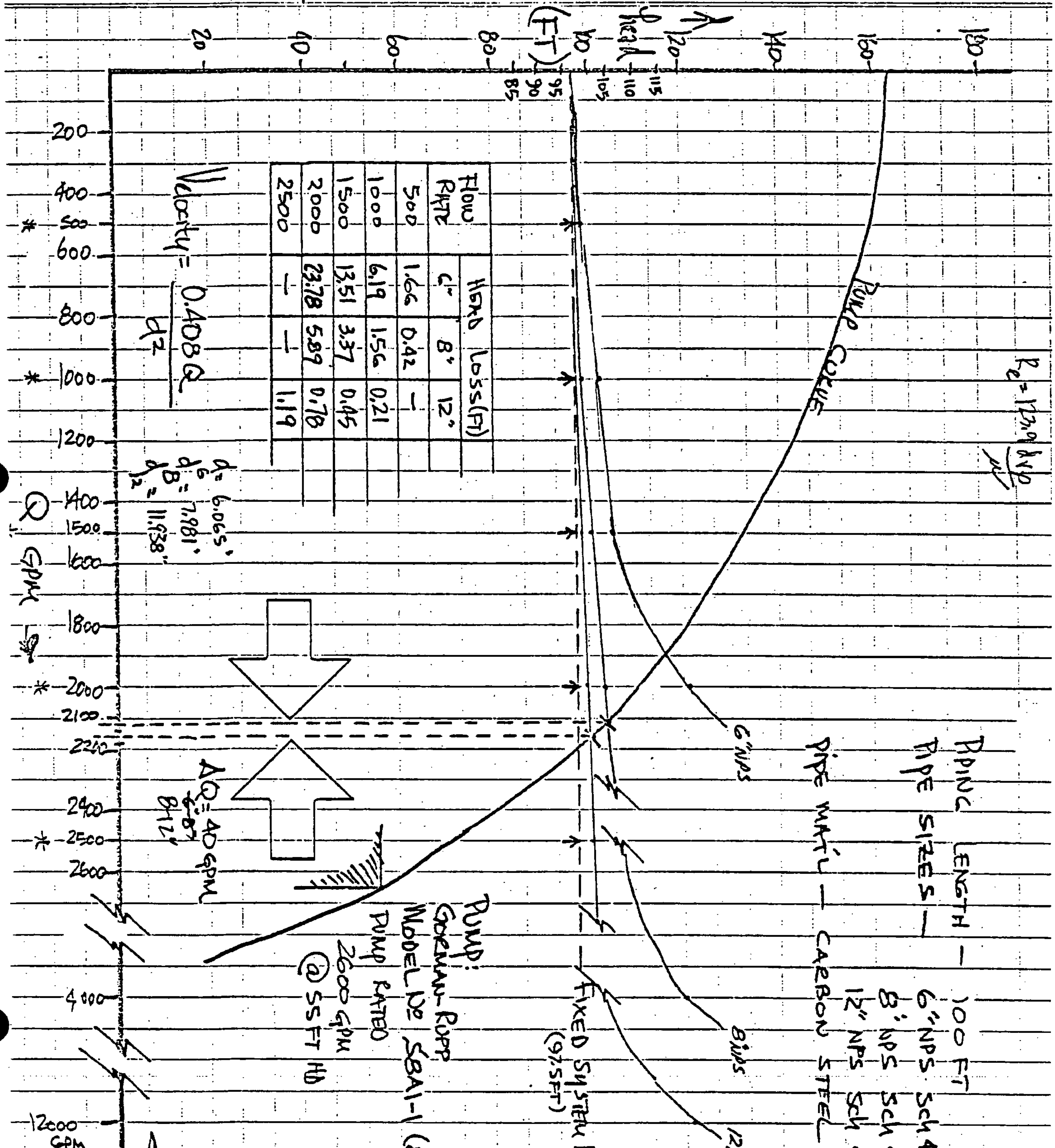
Sheet 1 of 2

Subject: DEWATERING PUMP PERFORMANCE

Project: VOGTLE

COMPARATIVE PIPE LOSSES FOR 6"-8"-12" NPS

Computed by: D. DAY Date: 8-10-80 Checked by: ______ Date: ______



FLOW RATE	HEAD LOSS (FT) 6"	8"	12"
500	1.66	0.42	—
1000	6.19	1.56	0.21
1500	13.51	3.37	0.45
2000	23.78	5.89	0.78
2500	—	—	1.19

APPENDIX K-1

GEORGIA POWER COMPANY

Subject ______ Project ______

Computed by ______ Date ______ Checked by ______ Date ______

CONCLUSIONS:

I. FOR THE 8" PUMP, THE SIX-INCH LINE BEGINS TO EXHIBIT HIGH LOSSES AT THE UPPER FLOW RATES BEYOND 2000 GPM . . . . (UNDERSIZED)

II. THE 8" LINE IS OPTIMUM FOR THE PUMP FLOW RATES INVOLVED . . . .

III THE 12" LINE IS MUCH LARGER THAN REQUIRED FOR THE FLOW RATES INVOLVED (OVERSIZED). CHANGING LINE SIZES FROM 8" TO 12" FOR THIS APPLICATION ONLY RESULTS IN A CHANGE IN FLOW OF 40 GPM IN THE 2000 GPM REGION AND WOULD BE IMPRACTICAL AND ECONOMICALLY UNJUSTIFIED . . .

IV FOR GENERAL WATER FLOW SERVICE, REASONABLE VELOCITIES FOR THE FLOW OF WATER THROUGH PIPE IS FROM 4-10 FEET PER SECOND . . . . MAINTAINING REASONABLE VELOCITIES MINIMIZES SYSTEM HEAD LOSSES ATTRIBUTABLE TO PIPING.

[illegible]

2766—12M—1-31-66 GEORGIA POWER COMPANY Sheet ____ of ____

Subject DEWATERING SYSTEM Project VOGTLE

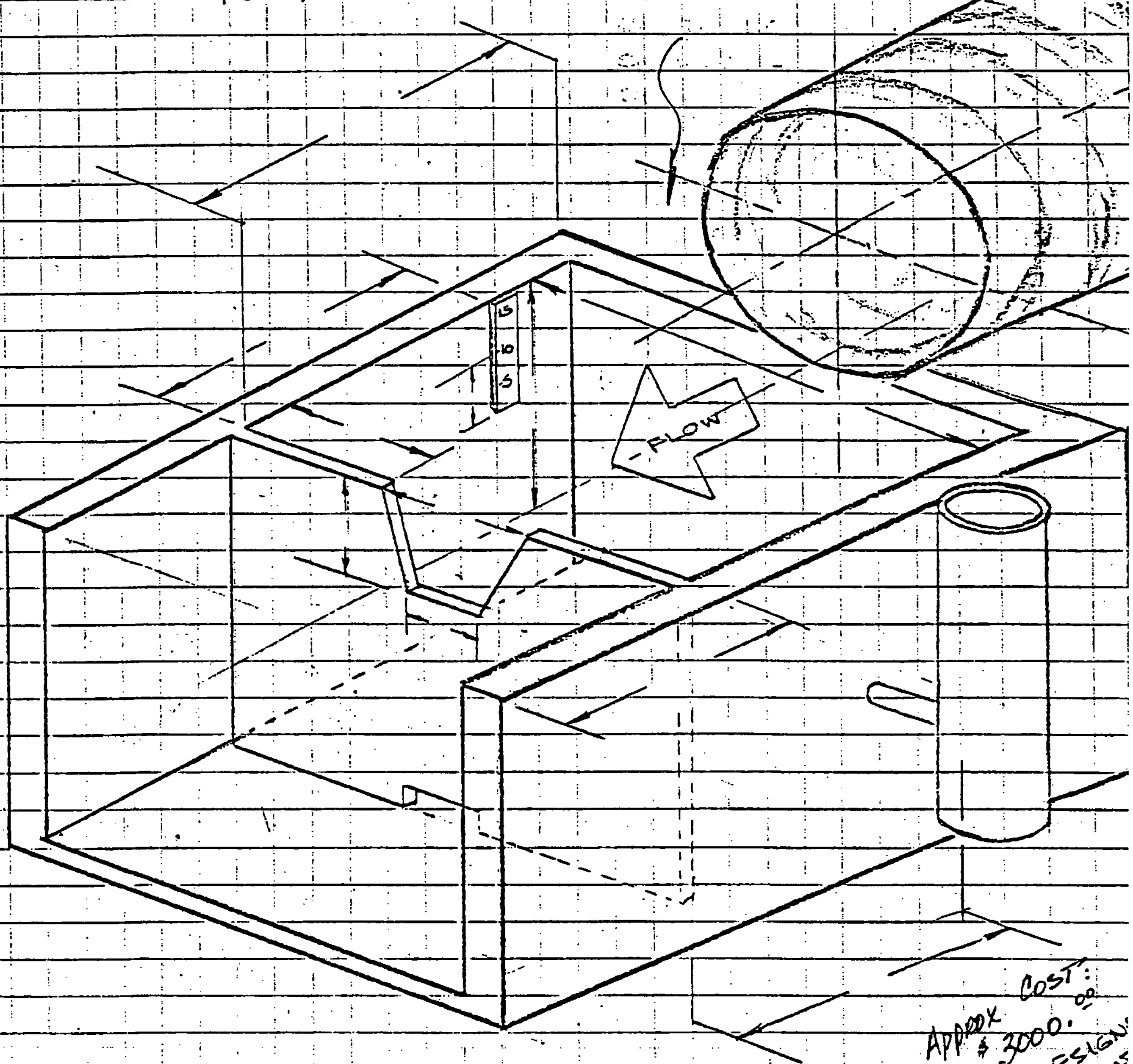
WEIR CONSTRUCTION DETAILS

Computed by D. DAY Date 8-14-80 Checked by ____ Date ____

FLOW RANGE =

500 – 15,000 GPM

MAY REQUIRE STILLING WELL TO REDUCE TURBULENCE



APPROX COST: $ 3000.00 SITE DESIGN AND FABRICAT

5-FT CIPPOLETTI WEIR

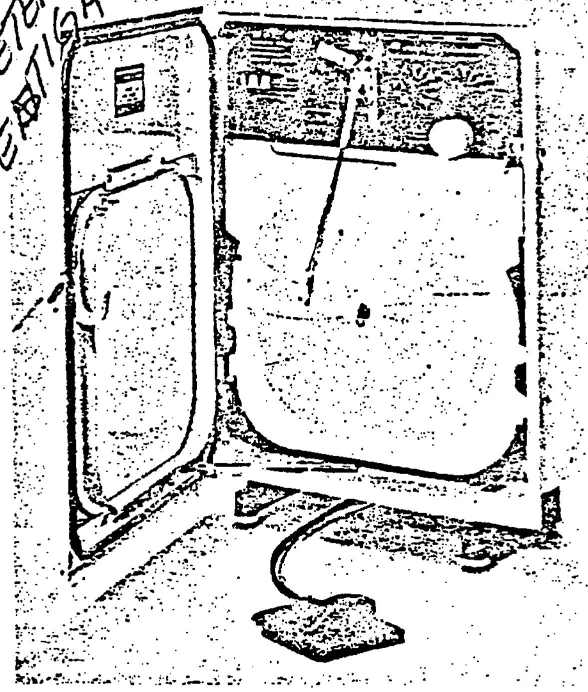
FIGURE 5-1

VMI FLOWMETER Model 250

The Velocity-Modified FLOWMETER (VMFM), Model 250, is used extensively in:

- Industrial and municipal waste water systems
- Process liquid and slurry systems
- EPA Permit Application studies
- Storm water management systems

ALTERNATE FLOWMETER BEING INVESTIGATED!



The Velocity-Modified FLOWMETER, Model 250, uniquely measures volumetric fluid flow in open-channel and pipe systems of any configuration.

Readily installed without system modifications, and often without interrupting operations, the Model 250 continuously senses fluid velocity and level. Recorded volumetric flow is the automatically computed product of flow velocity times the instantaneous cross-sectional area of fluid in the conduit. This extraordinary instrument provides accurate, trouble-free service even in sludge, floating debris, and dilute acids or caustics.

Designed for permanent installation, the Model 250 is both compact and durable. Its control console, circuitry, and recorder are packaged in a dust-proof, cast-aluminum enclosure. The enclosure, in turn, is cabled to encapsulated dual sensors mounted on a conduit adaptor appropriate for the system geometry. The capsule contains an electromagnetic velocity sensor and a compact, bubble-type level-transducer. It is this powerful combination that creates the capability to measure forward and reverse flows accurately in open channels and filled or partially filled pipes of any shape.

The advanced features of the Model 250 FLOWMETER overcome obstacles that are major problems for other measurement techniques: The effects of manhole surcharging are automatically eliminated. Estimates of conduit roughness and slope of a fluid surface, needed to generate error-prone Manning-equation approximations, are not required by the Model 250. The characteristic flow reversals in tidal or storage systems are accommodated up to 5% of system capacity. Although volumetric flow is computed and recorded, both the velocity and level of the fluid may be read out at the flick of a switch.

The total life-cycle cost of the Model 250 FLOWMETER is highly favorable. Because its use requires no system modifications, installation costs are minimal; and being simple to operate and maintain, little added training of personnel is demanded. The accurate data that it delivers can be relied upon when pivotal operating decisions must be made, or capital plans and budgets formulated.

The economics and long, trouble-free service life of the Velocity-Modified FLOWMETER, Model 250, ensure that you will find it to be an entirely satisfactory purchase. We invite your inquiry.

SPECIFICATIONS

MEASUREMENT

Volumetric flow in millions of gallons per day (MGD) in filled or partially filled pipes from 8-in. to 60-in. diameters

Velocity Measurement

Method: Electromagnetic (Faraday effect)
Range: -0.5 to +10 ft/sec (-0.5 to +20 ft/sec optional); zero stability ±0.05 ft/sec
Accuracy: ±2%

Level Measurement

Method: Air bubbler with electronic pressure transducer
Range: 0.5 in. to 60 in. of water depth
Accuracy: ±2%

Flow Calculation

Method: Conversion of water level and pipe size to flow area; conversion of point velocity reading to average velocity; multiplication of flow area by average velocity
Range: 11 selectable ranges up to 0-200 MGD with 5% reverse-flow capability
Accuracy: ±2%
Repeatability: ±0.5%

STANDARD OUTPUTS

Recorder

Flow Range: See above
Flow Velocity: -0.5 to ±10 ft/sec full scale
Flow Level: 0 to 100% of full conduit size
4-20 mA: Optional
Time Pulse: Optional
Flow Proportional Contact Closure: ≈180-ms pulse every x000 gallons (x is customer-selected)

Flow Totalizer

Display: 6 digits, non-resettable
Resolution: 1000 gallons per count

MATERIALS

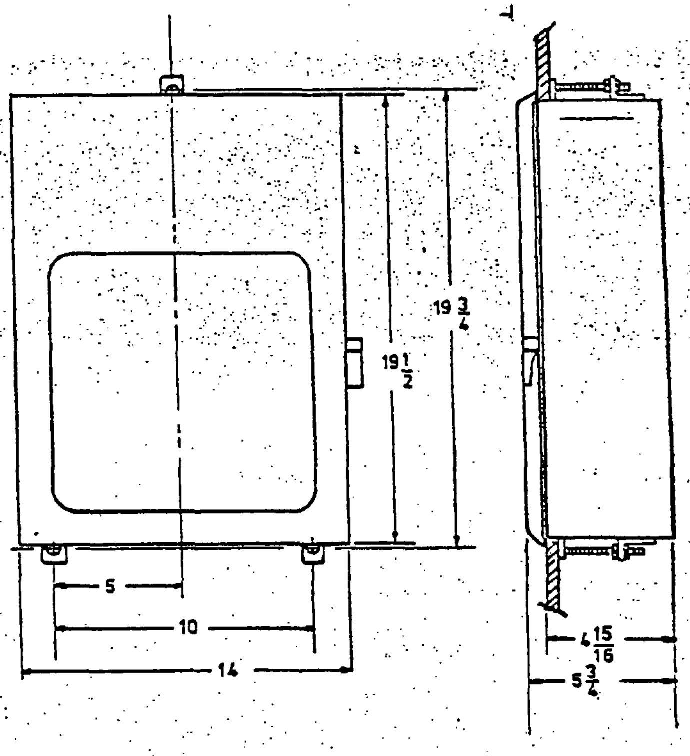
Sensor: Polyurethane exposed to flow
Sensor Cable: Twinax polyurethane rubber outer jacket exposed to flow
Air Tubes: Tygon
Sensor Mounting Band: Type 304 stainless steel exposed to flow
Recorder Housing: Cast aluminum dust-proof enclosure

ENVIRONMENTAL CONDITIONS

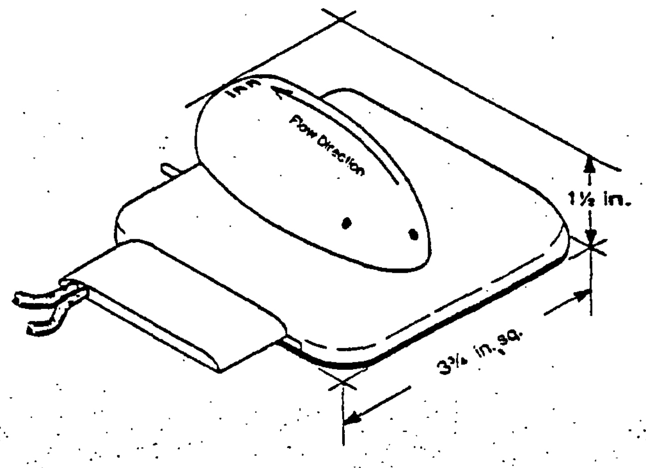
Electronics: Temperature limits 30°F to 112°F (-1°C to +40°C); outdoor installations require weatherproof enclosure
Sensor: Flow temperature range 32°F to 160°F (0°C to 65°C)

POWER REQUIREMENTS

120 VAC, 60 Hz (220 VAC, 50 Hz optional)
25 W rms



Enclosure Dimensions



Sensor Dimensions

Cascade Control Company
INSTRUMENTATION AND CONTROL EQUIPMENT
P.O. BOX 825
ROSWELL, GA. 30077
PHONE (404) 475-8504

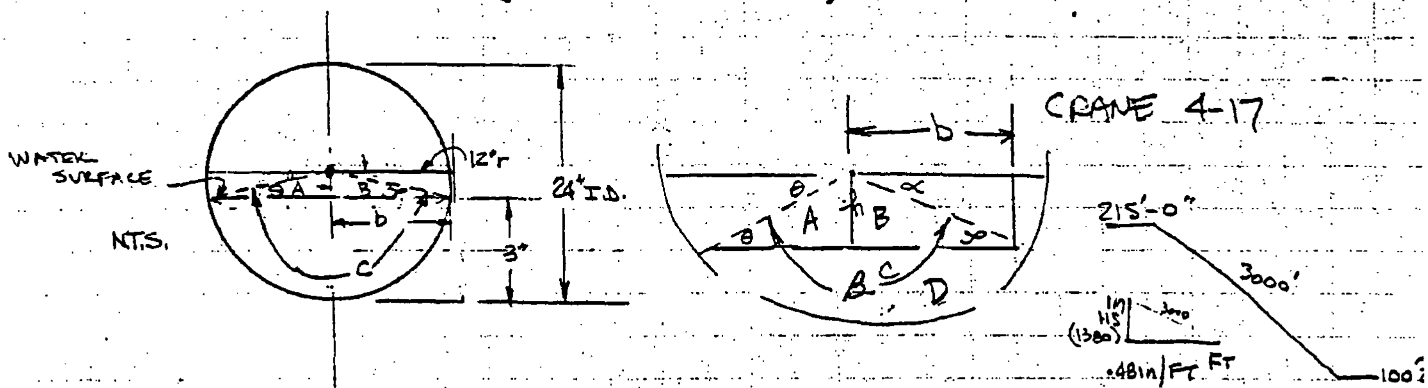
Jan. '80

GEORGIA POWER COMPANY

Subject ESTIMATION OF DEWATERING LINE EFFLUENT Project VOGTLE

D. DAY 8-14-80

GIVEN: A GALVANIZED CARBON STEEL CORRUGATED PIPE HAS A STEADY UNIFORM FLOW OF WATER @ 60°F WHICH FILLS THE PIPE 3" FROM THE BOTTOM. THE PIPE HAS AN INSIDE DIAMETER OF 24-inches and a slope of 0.5 inch per foot. NOTE THE SKETCH THAT FOLLOWS: (OBSERVED 8-14-80, 3 P.M.)



Find: THE FLOW RATE IN GALLONS PER MINUTE;

Solution:

PIPE IS 0.125 OR 1/8 FULL

1. $Q = 19.65 \sqrt{\frac{h_l D}{L}}$

Since pipe is flowing partially full an equivalent diameter based upon hydraulic radius is substituted for D in Equation 1 — $D = 4R_H$

2. $Q = 19.65 d^2 \sqrt{\frac{h_l 4 R_H}{fL}} = 39.3 d^2 \sqrt{\frac{h_l R_H}{fL}}$

3. $R_H = \frac{\text{CROSS SECTIONAL FLOW AREA (SQ FT)}}{\text{WETTED PERIMETER (FT)}}$

4. $R_e = \dfrac{473 q \rho}{R_H \mu} = 1.054 \dfrac{Q \rho}{R_H \mu}$

5. DEPTH OF FLOWING WATER EQUALS = 3"

6. $AREA_{24"/2} = \pi R^2/2 = \dfrac{(3.14)(12)^2}{2} = 226$ sq in

7. $\sin \theta = \dfrac{9}{12} = 0.75$

$\theta = 48.6°$

$\alpha = 48.6°$

$\beta = 180 - 2(48.6) = 82.8°$

8. AREA D = AREA C − AREA A − AREA B

9. AREA C = $\dfrac{\pi d^2}{4}\left[\dfrac{180-(2)(48.6)}{360}\right]$

$= \dfrac{\pi d^2}{4}[0.23]$

$= 104$ in^2

$\tan \alpha = \dfrac{o}{a}$

$\tan 48.6° = \dfrac{9}{b}$

$b = \dfrac{9}{\tan 48.6} = \dfrac{9}{1.13} = 7.93"$

10. AREA A = AREA B = $\frac{1}{2}bh = \frac{1}{2}(7.93)(9) = 35.71$ in^2

11. AREA D = 104 in^2 − 2(35.71) in^2 = 32.59 in^2

12. CROSS SECTIONAL FLOW AREA = $(32.59\ \text{in}^2)(6.944 \times 10^{-3}) = 0.225$ SQ FT

13. $d^2 = \dfrac{4a}{\pi} = \dfrac{4(32.59)}{\pi} = 41.52$

14. $h_l = \Delta h = \dfrac{0.48}{12} = 0.04$ FT/FT

15. WETTED PERIMETER = $\pi d \left(\dfrac{180-82.8}{360}\right) = \dfrac{20.36 \text{ in}}{12} = 1.7$ FT

16. $R_H = \dfrac{0.225}{1.7} = 0.132$

GEORGIA POWER COMPANY

Sheet ____ of ____

Subject ____ Project ____

Computed by ____ Date ____ Checked by ____ Date ____

17. EQUIV. diam $d = 48 R_H$

$d = 48\ (0.132)$

$d = 6.34$

18. RELATIVE ROUGHNESS $\frac{\epsilon}{D} = .0005$ (EST CONCRETE)

19. $f = 0.017$

20. $Q = (39.3)(41.52)\sqrt{\frac{(0.04)(0.132)}{(0.017)(1)}}$

$Q = 909.37$ GPM

Estimated Flow Attributable To Groundwater 8-14-80, 3PM...

AR-07-0639
Enclosure 3
RAI Response

2. Bechtel Power Corporation, 1972, Aquifer Tests for Construction Dewatering, Vogtle 8.7.1

NOTE: This document is 21-pages.

VOGTLE 9510

8.7.1 Aquifer Tests for
Construction Dewatering

8-7.a

May 3,
C. R.
SHEET

Vogtle Nuclear Plant Excavation
Proposal for dewatering test well program

Test wells (2) - 80 feet deep, 10" Ø hole, 4" dia. casing, 10 feet of well screen, gravel pack.

Obs. Pts. (4) - two 80 feet deep; and two 65 feet deep, 4" dia. with 2" casing, 10 feet of well screen.

1. Drilling, setting casing and gravel pack of test wells;
   est. 160 feet - cost per linear foot
2. Drilling, setting casing and gravel pack of obs. pts.
   est. 290 feet - cost per linear foot
3. Cleaning and development of test wells
   est. 40 hrs. (20 each) - cost per hour
4. Test Pumping of wells
   est. 144 hrs. (72 each) - cost per hour
5. Move in, set up, and clean up
   linear sum cost

TOTAL COST ESTIMATE

Pay Item	Unit of Measure	Cost per Unit	Estimated Total Units	Total Cost
1	linear foot	$25.00	160	$4000.00
2	linear foot	$ 6.00	290	1740.00
3	hour	$30.00	40	1200.00
4	hour	$30.00	144	4320.00
5	lump sum	$3,000.00	1	3000.00
TOTAL COST				$14,260.00

FORM BC-S 116
20M ISPCO

BECHTEL

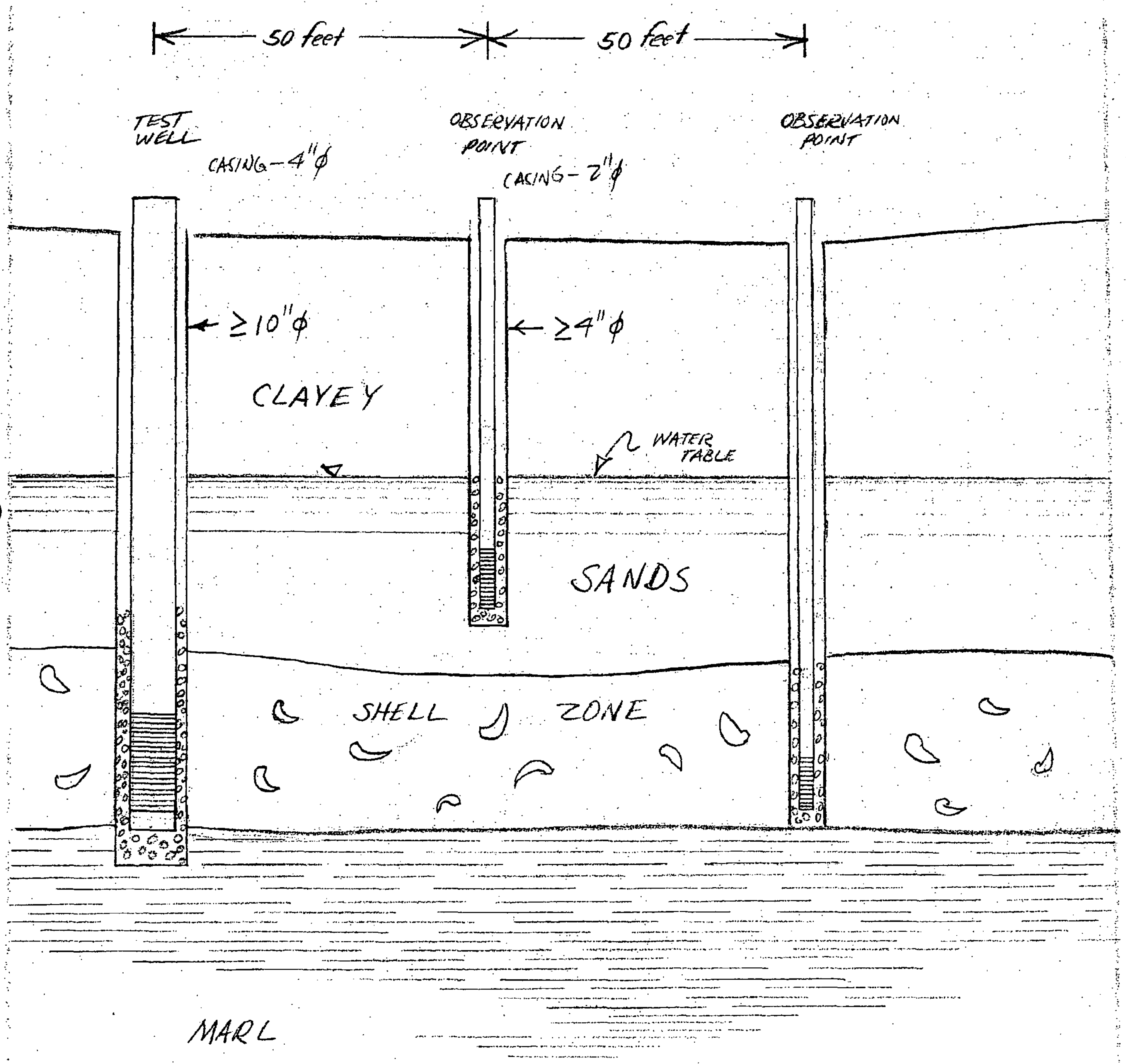
CALCULATION SHEET

BECHTEL CORPORATION
4620 SEVILLE AVE.
VERNON, CALIFORNIA

SIGNATURE Clifford Farrell DATE May 3, 1972 CHECKED ______ DATE ______

PROJECT Vogtle Nuclear Plant Excavation JOB NO. 9510-001

SUBJECT Test Wells for Dewatering SHEET 2 OF 2 SHEETS



SKETCH — NOT TO SCALE

FORM BC-S 116
20M ISPCO

CALCULATION SHEET

BECHTEL CORPORATION
4620 SEVILLE AVE.
VERNON, CALIFORNIA

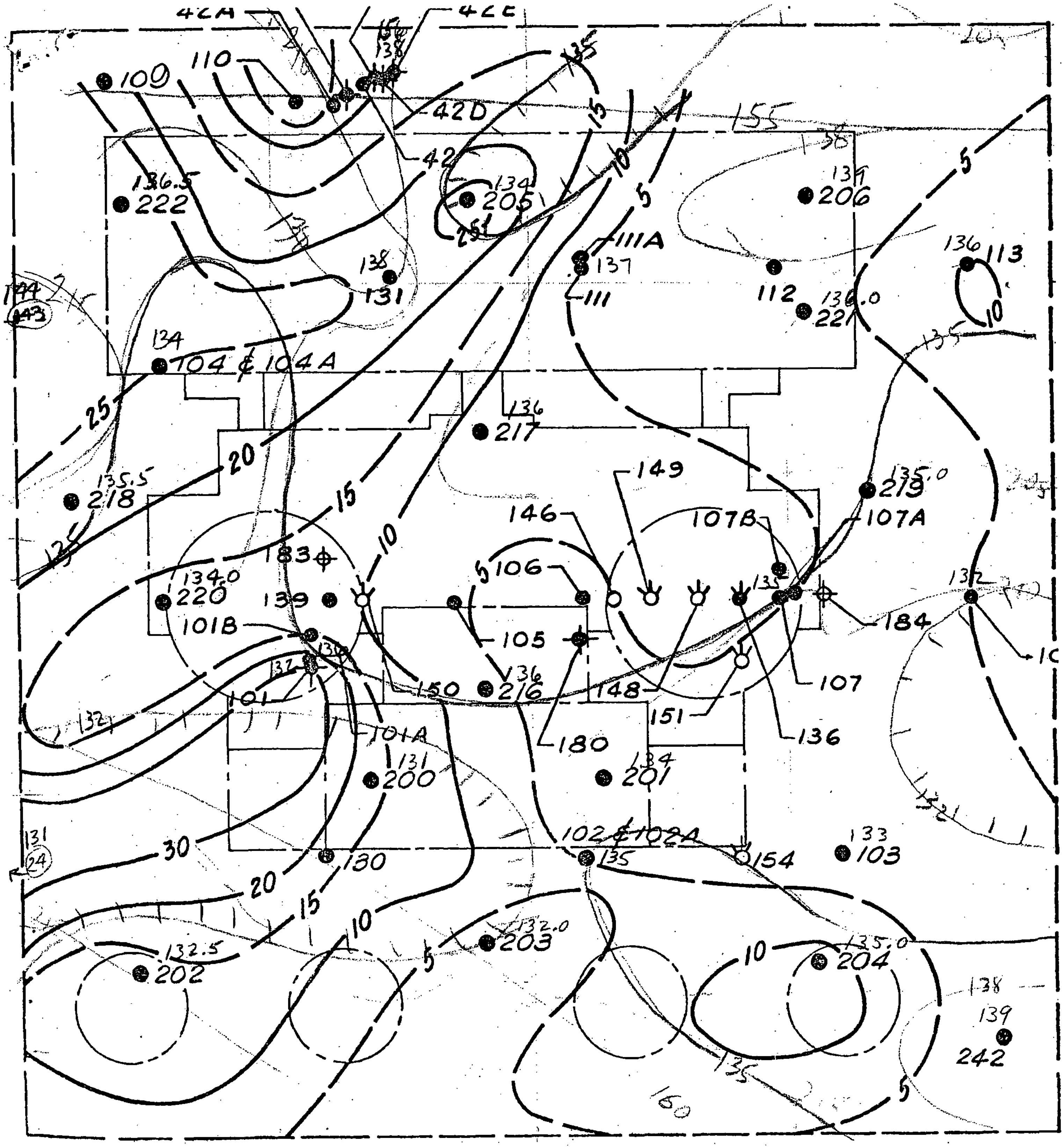
SIGNATURE ______ DATE ______ CHECKED ______ DATE ______

PROJECT ______ JOB NO. ______

SUBJECT ______ SHEET ______ OF ______ SHEETS

DH #	E	N
(GEOL)		
114	623,526	1,143,503
116	623,928	1,143,503
113	624,026	1,143,256
OBSERV. PTS (SHALLOW)		
42D	623,571	1,143,403
245	623,917	1,143,501
140	622,702	1,142,846
143	622,738	1,143,283
OBSERV. PTS. (DEEP)		
101	623,517	1,142,945
42A	623,535	1,143,380
24	623,093	1,142,850

TEST WELLS FOR DEWATERING



DETAIL A

SCALE 1" = 100'

135 - T.O.C.

156 - W.L.

F. 7.

Bechtel Corporation

Inter-office Memorandum

To Files

Date May 10, 1972

Subject Investigation for Dewatering of Plant Excavation, Vogtle Nuclear Plant Job No. 9510-001

From C. R. Farrell

Of Geology

Copies to W. Holland
A. Luft
C. McClure
W. Ferris

At E & I Division

On Friday, April 28, R. Bush, consulant to the project, attended a meeting in our offices to discuss dewatering problems we might expect in the excavation for the plant site, and at the water intake structure near the river. I briefly attended the meeting to provide clarification of our intrepretation of ground water conditions at the site.

Site Excavation Dewatering

Mr. Bush is concerned that wells might not be an effective means for dewatering the area. He is basing this concern on the information collected to date; pump-in tests within the shallow sands and the description of materials in the shell zone overlying the marl (bearing unit). Although the experience of drilling and knowledge of the materials suggests that the shell zone is relatively high in permeability, it is not certain that it would act as an effective underdrain for dewatering the overlying sands. Should the proposed plan for well points in the shell zone not adequately drain the sands, serious delay in construction scheduling, as much as 2 or 3 months, could occur. I agreed with Bush that our knowledge of the permeabilities was not firm enough to preclude this possibility. It was decided that a testing program be conducted.

Test wells selected at two sites, representing the most favorable conditions and the least favorable conditions, as evidenced from our exploration of the site for Units 1 and 2, will provide data to evaluate a well system. After selecting the sites, and preparing a tentative construction plan, I contacted Layne-Atlantic of Savannah, Georgia, concering their availability to do the work. After verifying their willingness, I contacted R. Bush by telephone, Thursday, May 3, to review the details of test well construction.

There was apparently some misunderstanding as to Bush's primary objective for the test wells; I had thought it was to determine the permeability of the shell zone. Although this will be desireable, Bush is first concerned about the maximum yield of wells. Construction wise, this does not make a large difference (primarily it will call for 15 to 20 feet of perforations opposite the upper sands also, in order to intercept all inflows of water available to the well.

With these additional factors in mind, the test wells and observation points to be constructed will consist of the following:

<u>Test Wells</u> (2)

Depth: 80 feet (± 5 feet)
Diameter of bore: 12-inch
Casing diameter: 6-inch
Well screen: length; 15 feet
diameter; 4-inch
slot opening; 1/8-inch

<u>Observation Points</u>

Quantity: 3 points for each well
Depth: 80 feet (± 5 feet)
Diameter of bore: 4-inch
Casing diameter: 2-inch
Screen: length; 15 feet
diameter; 2-inch
slot opening; 1/8-inch

After placing the screen and casing in the bore, the annular space in the wells and the observation points will be filled with clean, fine-gravel up to height of 15 feet above the screened intervals. During placement of the gravel, clean water will be pumped through the casing to clean the hole of drilling fluid. The observation points will then be "pumped" by air injection to confirm hydraulic continuity with the aquifer zone.

The wells will be developed by pumping, possibly preceded by air injection. It is anticipated that 8 to 12 hours of development will be sufficient before commencing a testing of the well. The pumping tests will be conducted at a constant discharge rate for a continuous period of 72 hours (3 days).

I have asked Terry Scafidi of Layne-Atlantic to submit an estimate of cost for the work as a lump sum to be added to the present contract. He will submit an estimate by the end of this week. They would be able to conduct the work following completion of the test well construction and testing.

Intake Structure

Invert elevation of the intake structure adjacent to the Savannah River will be at elevation 54 feet, or approximately 10 feet below the base of the marl. Piezometric levels measured at various depths below the marl in the vicinity of the plant site indicate the level below the marl is at elevation 110 feet.

However, where the confining marl is breached, as in the river channel, the upward flow reduces the point hydraulic head, and it is believed that piezometric levels adjacent to the river will not be as high as 110 feet. This will be significant both for dewatering at the intake structure and in considering possible uplift pressures. It is therefore, recommended that an observation point be placed at the intake structure, to a depth corresponding to elevation 45. The point should be isolated by grouting the annular space above elevation 65. This could possibly be done by a Law Engineering drilling rig presently at the site conducting soils exploration for Units 3 and 4. Following completion of that work, a piezometer could be easily constructed by them, as they are familiar with the site and have placed similar ones in the vicinity. It is my understanding that data for dewatering conditions are not needed for the PSAR, so that construction of the piezometers can be planned on the availability of a drilling rig. If it is not convenient for Law Engineering to do it, we can arrange for placement of the point by Layne-Atlantic.

CR Farrell

C. R. Farrell

P. 7.

R. Y. Bush
Consulting Engineer
543 N. Stanford Avenue
Fullerton, California 92631
Telephone (714) 879-7812

DEWATERING STUDY

ALVIN W. VOGTLE NUCLEAR PLANT
GEORGIA POWER COMPANY

Purpose:

The purpose of this report is to present the results of our study of the dewatering problem anticipated in connection with the construction of the subject project.

Description of Study

This investigation consisted of a review of preliminary construction drawings; studies of geological information which included borings logs, a draft of a ground water report by Mr. C. Farrell of Bechtel, and various maps of geological conditions at the site; studies of rainfall intensity as related to possible flood damage in the excavation area; analyses of pump test data obtained by your personnel; and the preliminary design of a combination dewatering and storm water pumping system. As of the date of this report, the writer has not had an opportunity to personally visit the project site.

Groundwater Conditions

The report draft on groundwater conditions by Mr. Farrell provided valuable information. Significant items contained in this report are:

How deep?

1. "The impervious marl, or bearing unit, acts as an aquiclude (impervious barrier) to groundwater."
2. The only source of recharge to the unconfined groundwater above the marl is rainfall, and
3. A highly pervious shell zone of limited thickness (10'+-) exists directly above the marl.

January 12, 1973

The report describes the outflow from Mathes Pond as an estimated 300 gpm which is considered to be the amount corresponding to a final equilibrium condition during dewatering. It is pointed out that initial pumping for dewatering would be considerably greater than this amount. An excellent check on the 300 gpm was obtained by a planimeter measurement of the tributary area to the site, which appears to be about 367 acres. For this area, a rate of 300 gpm would correspond to 50 inches per year with 30% infiltration, both reasonable values.

?

Data obtained from two pump tests were analyzed. Descriptions of the test wells follow:

1. Well #1-total depth 94'; white sand with shells 72'-80'; marl below 80'; coordinates N1, 142,660 and E623,570.
2. Well #2-total depth 87'; white sand with shells 52'-61'; shell, hard, limestone 61'-85'; marl below 85'; coordinates N1, 143,225 and E623,075. In addition to the pumped wells, 8 observation wells, 4 per test well, were installed to permit the measurement of water levels during pumping.

Well #1 was pumped for approximately four days at rates of generally in the range of 30 to 38 gpm. Well #2 was pumped for about 27 hours at rates of 10 to 15 gpm. Pumping on well #2 was discontinued due to the lack of response of the water levels in the observation wells. Additional "pump in" tests were performed on well #2 observation wells. Due to the relatively small rate of pumping from well #2 and the correspondingly small amount of lowering of water, a quantitative evaluation of permeability was not possible in this case. This test did indicate that transmissibility at this location is very small.

Data obtained from test well #1 was analysed on the basis of nonequilibrium methods, using data obtained during both drawdown and rebound periods. Attached plots indicate fair agreement between the various observation wells with the exception of 1-c.

The erratic behavior was due to interruptions in the rate of pumping and to a lesser extent due to variations in barometric pressure during a storm period.

Based on our analysis, the transmissibility of the unconfined aquifer is estimated to be in the range of 0.7 to 1.8 ft^2/min. (7,560 to 19,440 gal/day ft.). Corresponding permeability values, based on an aquifer thickness of 10' would be 0.07 to 0.18 ft./min. Average permeabilities in the area are probably less than this due to the fact that well #1 was probably located in a relatively high permeability area. Considering the variable nature of the shell zone, a wide range of local permeability should be anticipated.

Permeable material is considered 300-1000 gals/day/ft.^{2}

Dewatering and Pumping

Although the apparent permeability of the shell zone is relatively high, because of its limited thickness, the transmissibility of the aquifer is quite low. Due to this condition, which results in low individual well capacity, the application of predraining methods employing deepwells or wellpoints is not considered practical or economically feasible.

The volume of water to be removed during the initial dewatering period until the final "equilibrium condition" is reached, is estimated at about 140,000,000 gal.? An average rate of 1,000 gpm would therefore require about 100 days which should coincide reasonably well your anticipated excavation rate. An initial dewatering plant having a minimum capacity of approximately 1,500 gpm is recommended. The rate of pumping would gradually decrease with time until finally the sustained condition, estimated at 300 gpm, is reached.

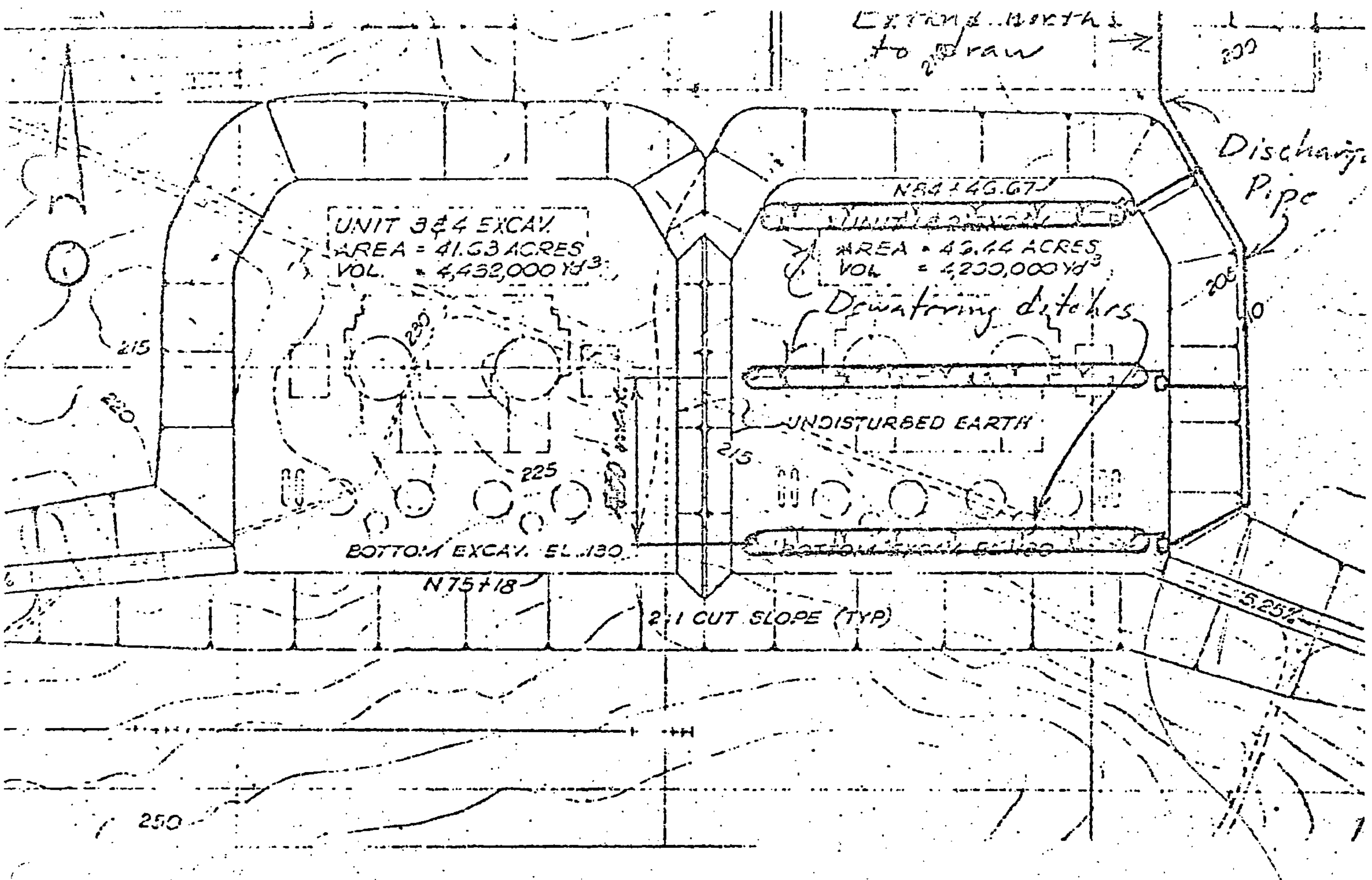
A system of ditches and sumps is recommended to perform this dewatering. The basic scheme is illustrated on figures 3,4,5,6. It should be emphasized that the sketches are of necessity quite rough and should be considered as schematic only. It is recognized that various construction considerations unknown to the writer could necessitate the extensive revision of the layouts as proposed.

The basic dewatering scheme proposed consists of the following:

1. Preliminary excavation is made to an elevation slightly above the initial water table.
2. Ditches are excavated across the excavation area to allow the wet materials to drain by gravity flow through the ditches to sumps from which the water is pumped. It should be noted that the spacing of the ditches is indicated as a 400' maximum. This is to insure that dewatering of the materials between ditches occurs in a reasonably short period of time.
3. Excavation continues to the surface of the marl, the bearing material, at which time the rate of pumping should have diminished to a relatively small rate, approaching the sustained rate.
4. At this time ditches are excavated in the marl to provide drainage during periods of high intensity rainfall. This item is discussed in greater detail subsequently.
5. Prior to backfill, a perimeter porous drain pipe is installed to allow dewatering during the backfill period. This drain leads to vertical pump wells from which the water can be pumped during the backfilling operation. This pumping on the perimeter drain would continue until backfill has reached a sufficiently high elevation, and the weight of the concrete placed is sufficiently heavy so that no further control of hydrostatic uplift is required.

Stormwater Pumping

The major pumping requirement will be to remove stormwater from the excavation during periods of high intensity rainfall which must be anticipated in this area. The combined effects of this high intensity rainfall with the extremely large area of the excavation results in extremely high rates of pumping required during storms to keep the excavation free of water. Figure 7 illustrates a simple plot of rainfall intensity versus gal/min for the area



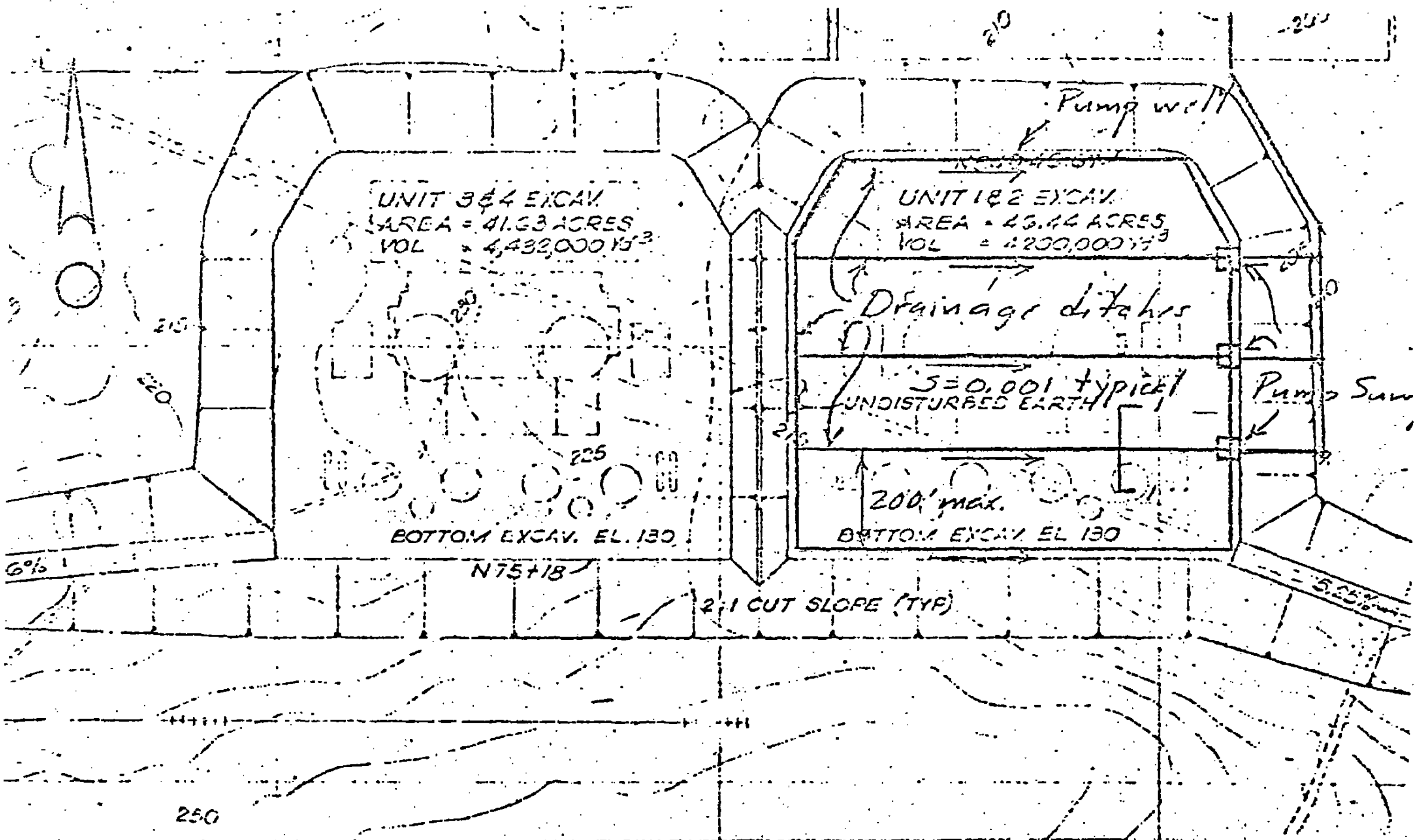
Stage I

1. Install discharge pipe. Size of line to be based on maximum design rate of pumping to handle storm water.

2. Excavate with scraper equipment down to about 3' ± above water table.

3. Excavate drainage-dewatering ditches with Gradall or backhoe. Maximum spacing between ditches to be 400'. Provide initial pumping capacity of 1500 gpm minimum. Final pumping rate for dewatering estimated at 300 gpm.

Figure 3

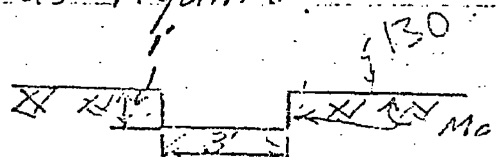


Stage II

1. After completion of excavation to subgrade, cut storm water drainage ditches in marl as indicated. Install storm water pumps.

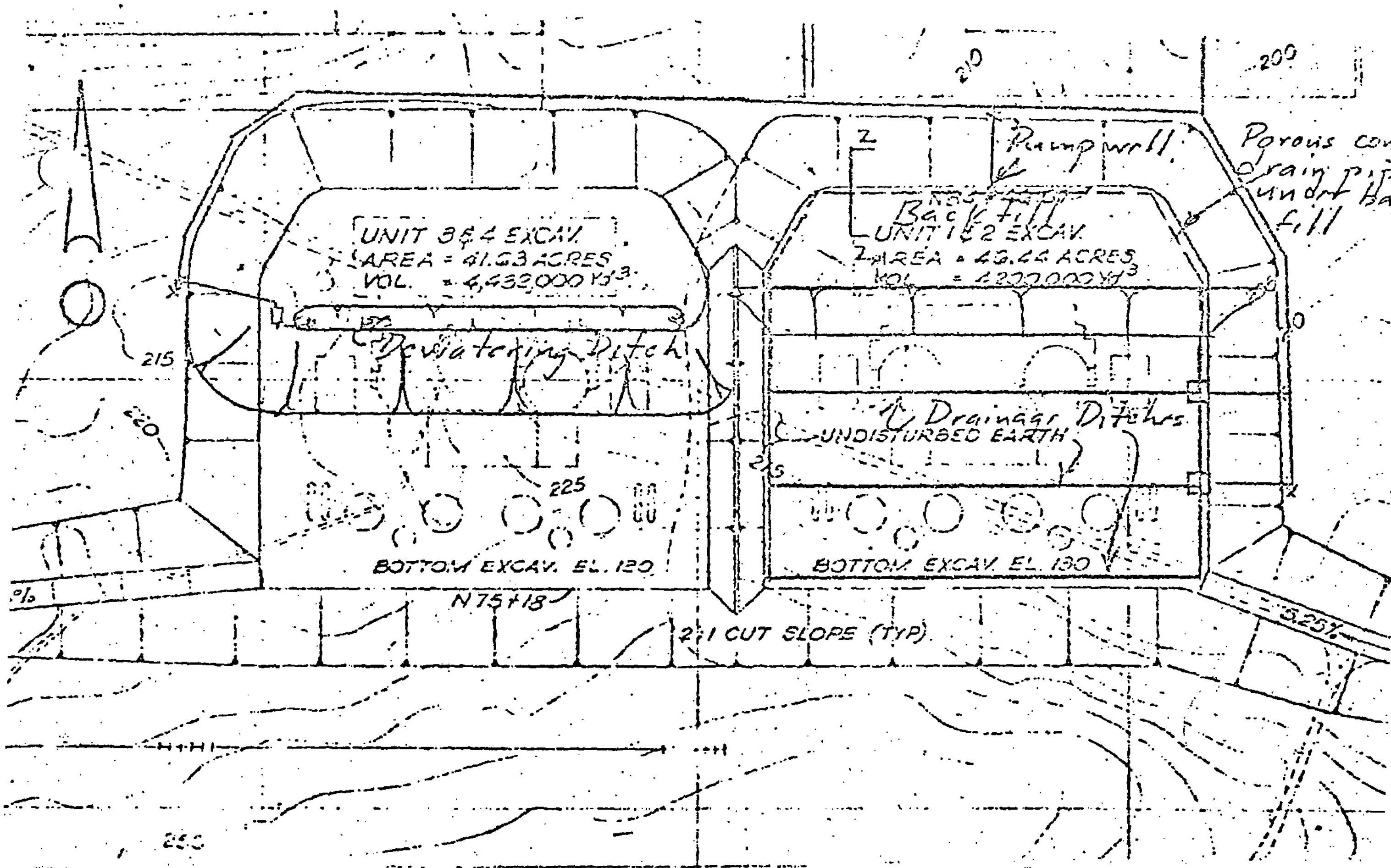
2. Prior to starting backfill, install porous concrete drain pipe around perimeter in a properly graded filter envelope to allow dewatering during period of backfilling up to water table. Install pump well in backfill.

3. Provide appropriate ditches in backfilled areas to allow drainage of storm water as required.



Section 1-1 Typ.
N.T.S.

Figure 4

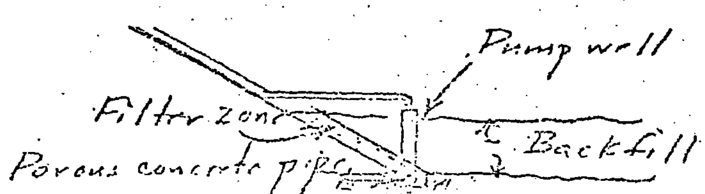


Stage III (Phase III Excavation, Drwng SK-C-5(

1. Dewatering in Units 1 & 2 Turbine Bldg. area by perimeter drain during backfill.

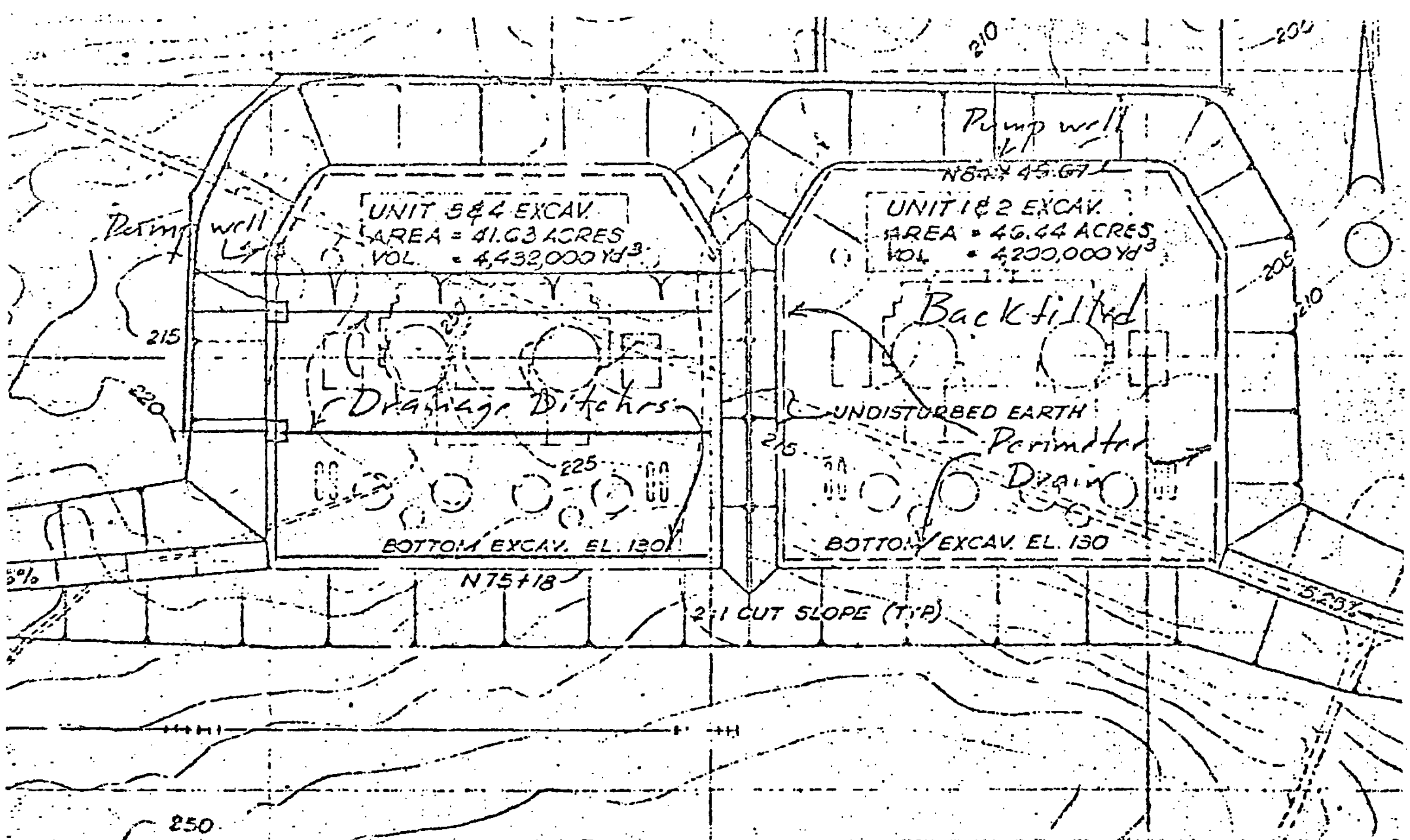
2. Install additional discharge piping to Units 3 & 4 area. Dewatering by ditching similar to Stage I.

3. Provide storm water pumping equipment in areas as required.



Section 2-2 Typical
N.T.S.

Figure 5



Stage IV (Phase IV Excavation, Drwng SK-C-51)

1. Continue pumping on perimeter drain in Unit 1&2 area as required to (a) permit backfilling in "the dry," (b) control hydrostatic uplift under partially completed structures, and (c) control seepage from east for completion of construction of Units 3 and 4

2. Install additional discharge piping to south portion of Units 3 and 4 area. Dewater by ditching similar to Stage I.

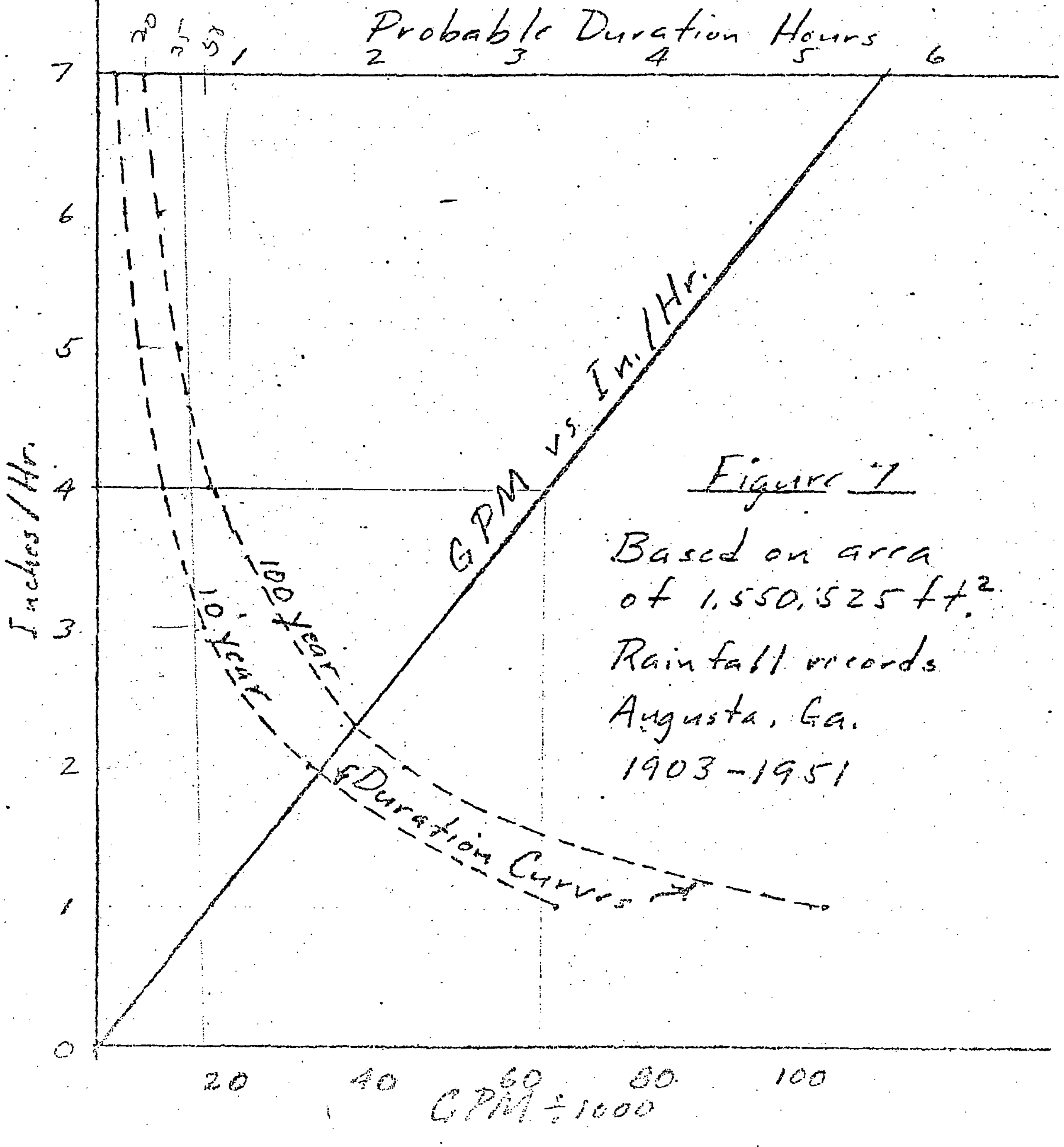
3. Excavate ditches in marl for handling storm water similar to Stage II.

4. Install perimeter drain around north, west, and south sides of Unit 3 and 4 area. Complete construction.

Figure 6

PROJECT Alvin W. Vogtle Nuclear Plant PROJECT NO. 230
BY RWB DATE 1/11/75 CHECKED BY ______ DATE ______ SHEET NO. ___ OF ___

Stormwater Pumping Study



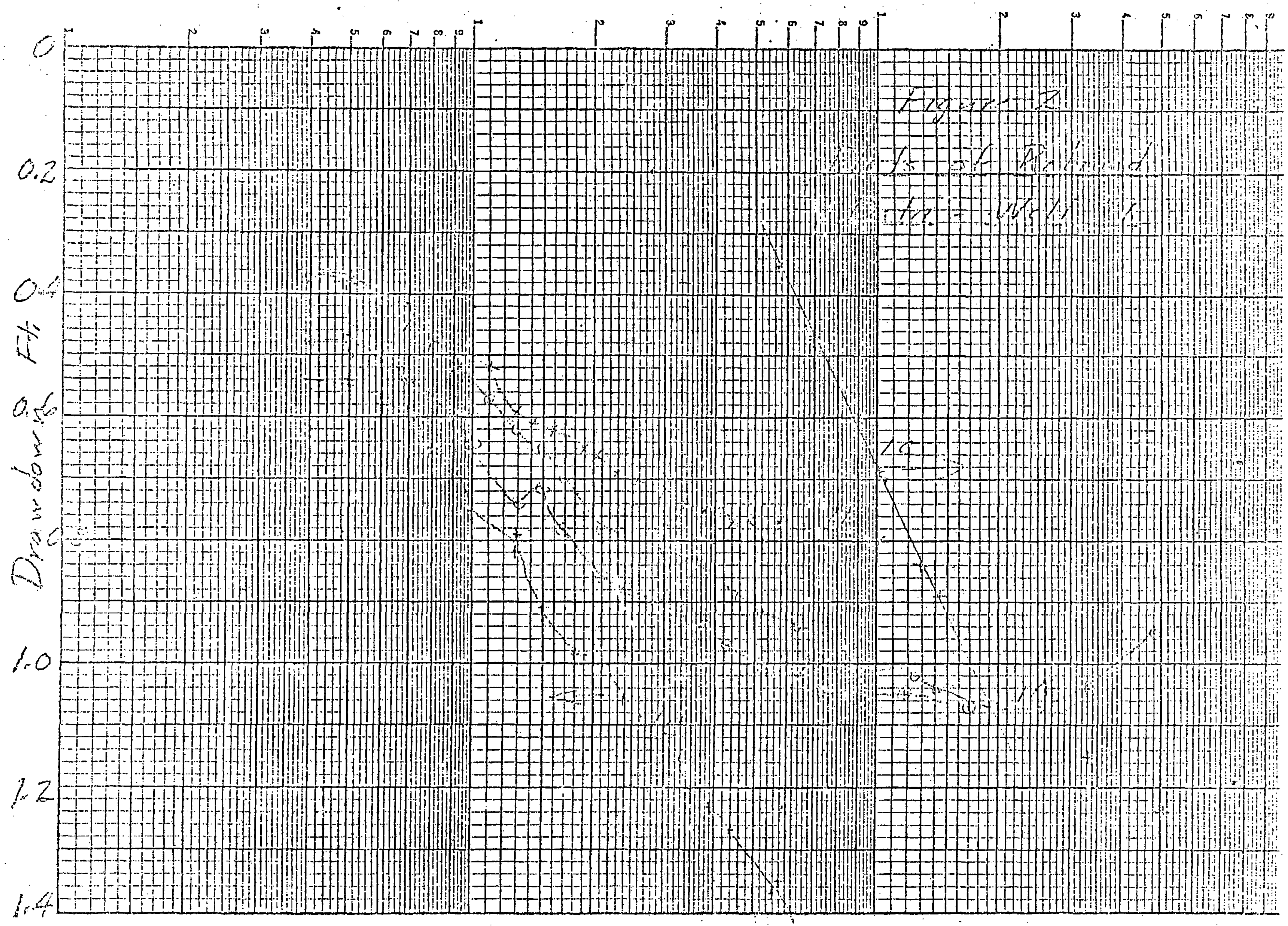
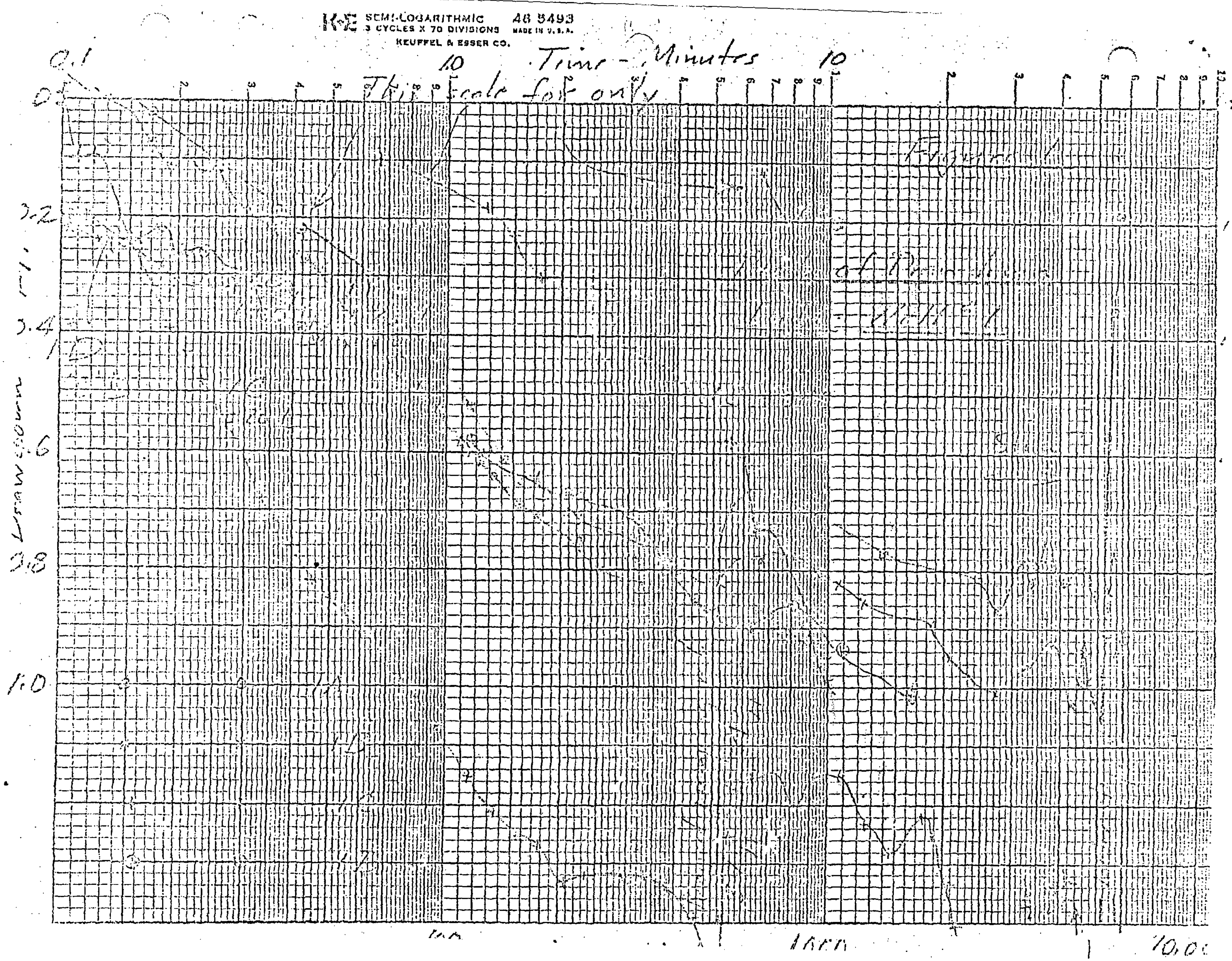


Figure 2
Drawdown Ft.
0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1 2 3 4 5 6 7 8 9 1 2 3 4 5 6 7 8 9 1 2 3 4 5 6 7 8 9



K+E SEMI-LOGARITHMIC 46 5493
3 CYCLES X 70 DIVISIONS MADE IN U.S.A.
KEUFFEL & ESSER CO.
Time - Minutes
This scale for only
0.1
10
10
0.2
0.4
0.6
0.8
1.0
1000

represented by Units 1 and 2. It is assumed that the top of the excavation slope is provided with proper drainage ditches and that therefor only rainfall falling on the actual excavation area would be pumped from it. Consideration should be given to the use of appropriate stabilizing materials to the slope to minimize erosion.

During normal conditions, only a portion of the pumping equipment would be required to operate for dewatering; that is, handling the groundwater entering the excavation. The design of the discharge piping for the combined system would obviously be based on the pumping rate during the storm period. The actual size of the system must be based on a careful consideration of the financial consequences of a heavy rainstorm due to damage caused to concrete and other operations, weighed against the probability of extreme storms occurring say of the 50 to 100 year variety. We will not attempt to evaluate this complex problem since we are not sufficiently acquainted with the various cost and construction considerations involved on this project. It would appear that a pumping plant to provide reasonable protection against storm damage should have a capacity in the range of from 5,000 to 10,000 gpm.

5000 + 1000

Conclusions and Recommendations

1. Although the permeability of the shell zone immediately above the marl appears to be quite high, due to its limited thickness, the transmissibility in this area is quite low, in the range of 0.7 to 1.8 $ft.^2$ / min.
2. Due to the limited thickness of the pervious zone directly above the marl, along with other considerations such as the difficulty and high expense of drilling, the application of predraining methods employing wellpoints or deepwells is considered impractical and economically not feasible.
3. A method of ditches and sumps should be used to perform the dewatering of the excavation.
4. The size of the pumping plant provided should be based on a consideration of handling stormwater since this pumping rate will greatly exceed the anticipated rate of dewatering.
5. ~~A perimeter drain should be installed to allow dewatering and~~

5. A perimeter drain should be installed to allow dewatering and hydrostatic uplift control during backfill operations.

R. Y. Bush

January 12 1973

3. VEGP Bechtel Calculation G-008, September 27, 1985

NOTE: This document is 18-pages.

CALCULATION COVER SHEET

XZCF-S-121

PROJECT Vogtle Nuclear Power Plant JOB NO. 9510-091 DISCIPLINE Geology

SUBJECT Flow Rate in Mathes Pond Stream & West Branch Stream FILE NO.

CALC.NO. G-008

ORIGINATOR J. C. [signature] DATE 9/27/85

CHECKER Barbara Turner DATE 9/27/85 NO. OF SHEETS 18 incl. Cover Sheet

Q Class 01C

RECORD OF ISSUES

NO.	DESCRIPTION	BY	DATE	CHKD	DATE	APPRD	DATE	DATE FILMED
	THIS CALC IS APPLICABLE TO: UNIT 1 [ ] 2 [ ] A [✓] RE [signature] DATE 7/26/88 EGS [signature] DATE 7/26/88							
0	~~Initial Issue for~~	JCI	11/20/85	BLT	11/20/85	[signature]	11/20/85	
	Project Use	JCI	9/27/85	BLT	9/27/85	—	—	

PRELIMINARY CALC. ☐ COMMITED PRELIMINARY DESIGN CALC. ☐

SUPERSEDED CALC. ☐ FINAL CALC. ☒

STATEMENT OF PROBLEMS:

Determine the stream discharge rate in the Mathes Pond Drainage Basin. The Mathes Pond Drainage Basin consists primary of the Mathes Pond and West Branch streams.

SOURCES OF DESIGN CRITERIA:

IOM from J.C. Isham to C.R. Farrell dated 7/25/85 "Assessment of Discharge from Mathes Pond and West Branch streams". Field observations by J.C. Isham in June & July of 1985.

SOURCES OF FORMULA & REFERENCES:

U.S. Dept. of Interior, Bureau of Reclamation, Water Measurement Manual, 1967.

U.S. Dept. of Interior, Bureau of Reclamation, Design of Small Dams, 1973.

Missouri Water Well Drillers Assoc., Water Well Handbook, K.E. Anderson, 1963.

0510 (11-74)

BECHTEL

CALCULATION SHEET

DATE 9/27/85

DESIGN BY J. C. Isham DATE 9/25/85 CHECKED BY B. L. Turner SHEET NO. 1

PROJECT Vogtle Nuclear Power Plant JOB NO. 9510-091

SUBJECT Flow Rate in Mathes Pond Stream & West Branch Stream CALCULATION NO. G-008 FILE NO. X2CF-5-121

The purpose of this calculation is to determine the flow rate in the Mathes Pond Drainage Basin (Mathes Pond & West Branch Streams). The flow rates determined in this calculation will be used as input data for the Hypothetical Spill Analysis (Section 2.4.13 of FSAR).

The source of data for this calculation are observations & field measurements made in June and July, 1985 by J.C. Isham (see IOM from J.C. Isham to C.R. Farrell dated July 25, 1985).

0510 (11-74)

BECHTEL

CALCULATION SHEET

DATE 9/27/85

DESIGN BY J.C. Isham DATE 9/25/85 CHECKED BY B.L. Turner SHEET NO. 2

PROJECT Vogtle N.P.P. JOB NO. 9510-091

SUBJECT Flow Rate in Mathes Pond & West Branch streams CALCULATION NO. G-008 FILE NO. XZCF-S-121

I. Flow Rate in West Branch Stream

A. 500 feet upstream from confluence of West Branch & Mathes Pond streams (site #1)

stream dimensions = 10' wide & .083' deep
velocity (v) = 2 ft/sec (surface float velocity)

crossectional Area (A) = 10' x .083' = 0.83 sq ft

Flow Rate (Q) = A x v

= 2 ft/sec x .83 sq ft

Q = 1.67 ft³/sec

Because the stream is extremely shallow (1" depth) with numerous rocks breaking the surface causing eddy currents it is assumed that the actual (Q_a) may be 1/3 of the calcuated value.

Q_a = .56 ft³/sec or 249.3 gpm

say 250 gpm

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CALCULATION SHEET

DATE 9/27/85

DESIGN BY J.C. Isham DATE 9/25/85 CHECKED BY B.L. Turner SHEET NO. 3

PROJECT Vogtle N.P.P. JOB NO. 9510-091

SUBJECT Flow Rate Mathes Pond & West Branch Streams CALCULATION NO. G-008 FILE NO. XZCF-S-121

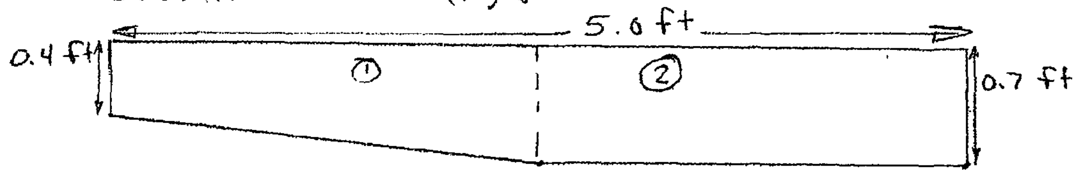
I. B. West Branch Stream immediately upstream of the confluence with Mathes Pond Stream (site #4 on attached map), page 16.

Stream Dimensions: 5 ft width

0.7 ft depth - center
0.7 ft depth - right bank
0.4 ft depth - left bank

Velocity (v) = 0.75 ft/sec (surface float velocity)

Crossectional Area (A):



Area ① = $\frac{.4+.7}{2} \times 2.5$ = 1.38 ft²

Area ② = .7 × 2.5 = 1.75 ft²
+

Total Area (A) 3.13 ft²

The Water Measurement Manual (WMM) p 158 has tabulated a series of correction coefficients which relates the surface velocity to average stream velocity. These coefficients are based on the roughness of the channel & the depth of the channel.

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CALCULATION SHEET

DATE 9/27/85

DESIGN BY J.C. Isham DATE 9/25/85 CHECKED BY B.R. Turner SHEET NO. 4

PROJECT Vogtle N.P.P. JOB NO. 9510-091

SUBJECT Flow Rate Mathes Pond & West Branch Streams CALCULATION NO. G-008 FILE NO. X2CF-5-121

The correction coefficient given for a smooth stream with an average depth of 1.0 ft is 0.66. Taking into consideration that the measured stream is shallower and rougher a correction coefficient of 0.5 may be appropriate.

Corrected Velocity (V_c) = .75 ft/sec x .5 = .375 ft/sec ✓

Flow Rate (Q) = $A \times V_c$

= 3.13 ft^2 x .375 ft/sec

Q = 1.17 ft^3/sec ✓ or 526.8 gpm ✓

Say 525 gpm ✓

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CALCULATION SHEET

DESIGN BY J.C. Isham	DATE 9/25/85	CHECKED BY B.L. Turner	DATE 9/27/85
			SHEET NO. 5
PROJECT Vogtle N.P.P.			JOB NO. 9510-091
SUBJECT Flow Rate Mathes Pond & West Branch streams	CALCULATION NO. G-008		FILE NO. XZCF-5-121

II Flow Rate in Mathes Pond Stream

A. Flow from Mathes Pond Drain (site #2 on attached map, page 16)

Size of drain = 1.5 ft diameter

Height of water above drain = 0.125 ft

1) Calculate the discharge from the pond based on the use of the drop inlet (morning glory drain) spillway equation: U.S. Dept of Interior Bureau of Reclamation, Design of Small Dams, 1974 pages 415-417.

$$Q = C_o\ (2\pi R_s)\ H_o^{3/2} \checkmark$$

where: Q = flow rate (ft^3/sec)

C_o = coefficient, based on the relationship of size of the drain, height of water above the drain, & heigh of drain above the base of the pond. From Figure 283 in Design of Small Dams C_o is estimated to be 3.9 ft/sec. ✓

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CALCULATION SHEET

0510 (11-74)

DATE 9/27/85

DESIGN BY J. C. Isham DATE 9/25/85 CHECKED BY B. L. Turner SHEET NO. 6

PROJECT Vogtle N.P.P. JOB NO. 9510-091

SUBJECT Flow Rate Mather Pond & West Branch Streams CALCULATION NO. G-008 FILE NO. X2CF-5-121

R_s = radius of drain (ft)

H_o = head of water above drain (ft)

∴

$$Q = 3.9 \times 2\pi \times .75\ (.125)^{3/2}$$

Q = .81 ft^3/sec or 363.5 gpm

say

360 gpm

2) Calculate the discharge from the pond based on the use of the pipe "drop inlet" equivalent to rectangular weir equation. Water Well Handbook, K. Anderson, 1966, page 149

$$Q = 3.33\,(L - 0.2\,H)\,H^{3/2}$$

Where: Q = flow rate (ft^3/sec).

L = length of weir (circumference of pipe) (ft).

H = head of water above weir (pipe) (ft).

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CALCULATION SHEET

0510 (11-74)

DESIGN BY J.C. Ishan	DATE 9/26/85	CHECKED BY R.L. Turner	DATE 9/27/85
			SHEET NO. 7
PROJECT Vogtle N.P.P.			JOB NO. 9510.091
SUBJECT Flow Rate Mathes Pond & West Branch Stream		CALCULATION NO. G-008	FILE NO. X2CF-S-121

$L = \pi\ 1.5 = 4.71\ ft$

$Q = 3.33(4.71 - [0.2 \times .125])(.125)^{3/2}$

$= 0.689\ ft^3/sec$ or 309.2 gpm

say 310 gpm

Average of the two method of analysis

II. A.1	morning glory inlet	360 gpm
II. A.2	rectangular weir-pipe inlet	310 gpm

Average flow rate 335 gpm

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CALCULATION SHEET

DATE 9/27/85

DESIGN BY J.C. Isham DATE 9/26/85 CHECKED BY B. L. Turner SHEET NO. 8

PROJECT Vogtle N.P.P. JOB NO. 9510-091

SUBJECT Flow Rate in Mathes Pond & West Branch Streams CALCULATION NO. G-008 FILE NO. X2CF-5-121

II. B. Flow rate in Mathes Pond Stream 100 ft downstream of Mathes Pond Dam (Site #3 on attached map).

Stream Dimensions: 4.0 ft. width
0.33 ft. depth

Velocity (v) : 0.6 ft/sec (surface float velocity)

Crossectional Area (A) = 4.0 x .33 = 1.32 ft²

The correction coefficient (conversion from surface float velocity to average stream velocity) given in the WMM for a smooth stream with an average depth of 1.0 foot is 0.66. Taking into consideration that the stream bed is fairly smooth but the stream depth is .33 ft a correction coefficient of 0.62 may be appropriate. (see page 17 for extrapolation of velocity correction data.)

CALCULATION SHEET

0510 (11-74)

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DATE 1/27/85

DESIGN BY J.C. Isham DATE 9/26/85 CHECKED BY B.L. Turner SHEET NO. 9

PROJECT Vogtle N.P.P. JOB NO. 9510-091

SUBJECT Flow Rate in Mathes Pond & West Branch Streams CALCULATION NO. G-008 FILE NO. X2CF-S-121

Corrected velocity (V_c) $= .6\ ft/sec \times .62$

$= .372\ ft/sec$ ✓

$Q = V_c \times A$

$= .372 \times 1.32$

$= 0.491\ ft^3/sec$ or $220.4\ gpm$ ✓

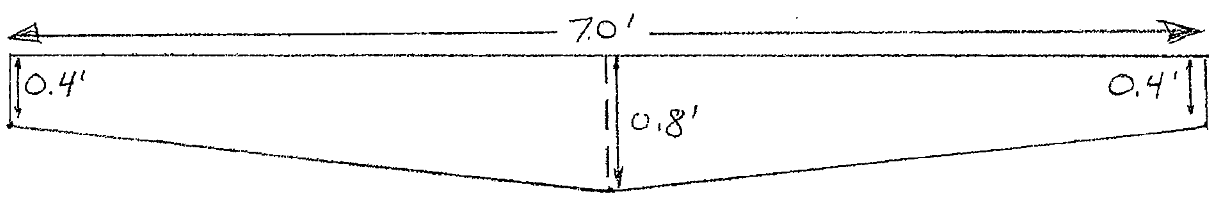
say 220 gpm

II.C. Flow rate in Mathes Pond Stream 300 ft downstream of Mathes Pond Dam (Site #5)

Stream Dimensions: 7.0 ft width
0.8 ft center depth
0.4 ft right & left bank depth

velocity (v) = 0.5 ft/sec (surface float velocity)

crossectional Area (A):



$$A = 2\left(\left[\frac{.4+.8}{2}\right] \times 3.5\right) = 4.2\ ft^2$$ ✓

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CALCULATION SHEET

DATE 9/27/85

DESIGN BY J.C. Ishan DATE 9/26/85 CHECKED BY B.L. Turner SHEET NO. 10

PROJECT Vogtle N.P.P. JOB NO. 9510-091

SUBJECT Flow Rate in Mathes Pond & West Branch Streams CALCULATION NO. G-008 FILE NO. X2CF-5-121

The stream bed has some obstructions, but it is fairly smooth. The average depth of the stream is .6 ft. Based on these considerations a velocity correction coefficient of .64 may be appropriate. (see page 17 for depth vs correction coefficient plot) ✓

Corrected velocity (vc) $= 0.5 \times .64$

$= 0.32$ ft/sec ✓

$Q = A \times vc$

$= 4.2 \times .32$

$= 1.344$ ✓ ft^3/sec or 603.2 gpm ✓

say 600 gpm

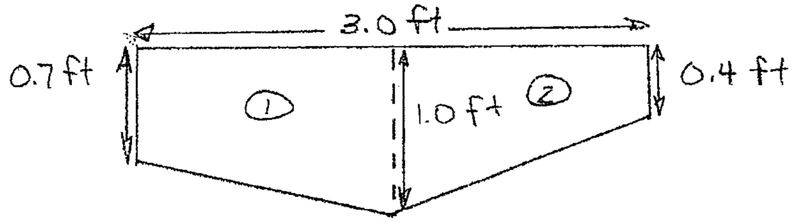
III. D. Flow Rate in Mathes Pond Stream immediately upstream of the confluence with West Branch Stream (site #4 on attached map, page 16)

stream dimensions: 3.0 ft width

1.0 ft depth - center
0.4 ft depth - right bank
0.7 ft depth - left bank

velocity (v) = 1.0 ft/sec (surface float velocity)

Crossectional Area (A) =



Area ① = $\frac{.7+1.0}{2} \times 1.5$ = 1.275 ft² ✓

Area ② = $\frac{.4+1.0}{2} \times 1.5$ = 1.050 ft² ✓

+

Total Area (A) 2.33 ft²

Average depth = $\frac{\left(\frac{.7+1.0}{2}\right) + \left(\frac{.4+1.0}{2}\right)}{2}$ = .775 ft ✓

0510 (11-74)

BECHTEL

CALCULATION SHEET

DATE 9/27/85

DESIGN BY J.C. Isham DATE 9/26/85 CHECKED BY R. L. Turner SHEET NO. 12

PROJECT Vogtle N.P.P. JOB NO. 9510.091

SUBJECT Flow Rate in Mathes Pond & West Branch Streams CALCULATION NO. G-008 FILE NO. X2CF-5-121

The velocity correction coefficient (page 158 WMM) given for a smooth stream with an average depth of 1.0 foot is 0.66. Taking into account that the measured stream is shallower and rougher (numerous marsh plants) a correction coefficient of 0.5 may be appropriate.

Corrected velocity (v_c) $= 1.0 \times .5$

$= .5$ ft/sec ✓

$Q = A \times v_c$

$= 2.33 \times .5$

$= 1.17$ ft³/sec or 525 gpm ✓

say 525 gpm ✓

0510 (11-74)

BECHTEL

CALCULATION SHEET

DATE 9/27/85

DESIGN BY J.C. Ishan DATE 9/26/85 CHECKED BY B.L. Turner SHEET NO. 13

PROJECT Vogtle N.P.P. JOB NO. 9510-091

SUBJECT Flow Rate in Mathes Pond & West Branch Streams CALCULATION NO. G-008 FILE NO. X2CF-S-121

Summary Sheet

I. West Branch Stream

Location	Calc. Page	Flow Rate (GPM)
500 ft upstream from confluence with Mathes Pond Stream (site #1)	2	250 ✓
Immediately upstream of confluence with Mathes Pond Stream (site #4)	4	525 ✓

II. Mathes Pond Stream

Location	Calc. Page	Flow Rate (GPM)
Mathes Pond Drain (site #2)	8	335 ✓
100 ft downstream from drain (site #3)	9	220 ✓
300 ft downstream from drain (site #5)	10	600 ✓
Immediately upstream of confluence with Mathes Pond Stream (site #4)	12	525 ✓

0510 (11-74)

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CALCULATION SHEET

DATE 7/27/85

DESIGN BY J. C. Isham DATE 9/26/85 CHECKED BY B. L. Turner SHEET NO. 14

PROJECT Vogtle N.P.P. JOB NO. 9510-091

SUBJECT Flow Rate in Mathes Pond & West Branch Streams CALCULATION NO. G-008 FILE NO. X2CF-5-121

The flow rate appears to increase in the downstream direction for both of the streams. This trend is defendable because these are effluent streams. Seeps and springs have been observed in the stream channels.

The apparent decreases in flow in Mathes Pond Stream between sites 2 & 3 and 5 & 4 may be a reflection of the accuracy of the flow measurements.

In general the accuracy of stream flow measurements obtained by the surface float method is limited by a number of factors.

0510 (11-74)

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CALCULATION SHEET

DATE 9/27/85

DESIGN BY J.C. Isham DATE 9/27/85 CHECKED BY B.L. Turner SHEET NO. 15

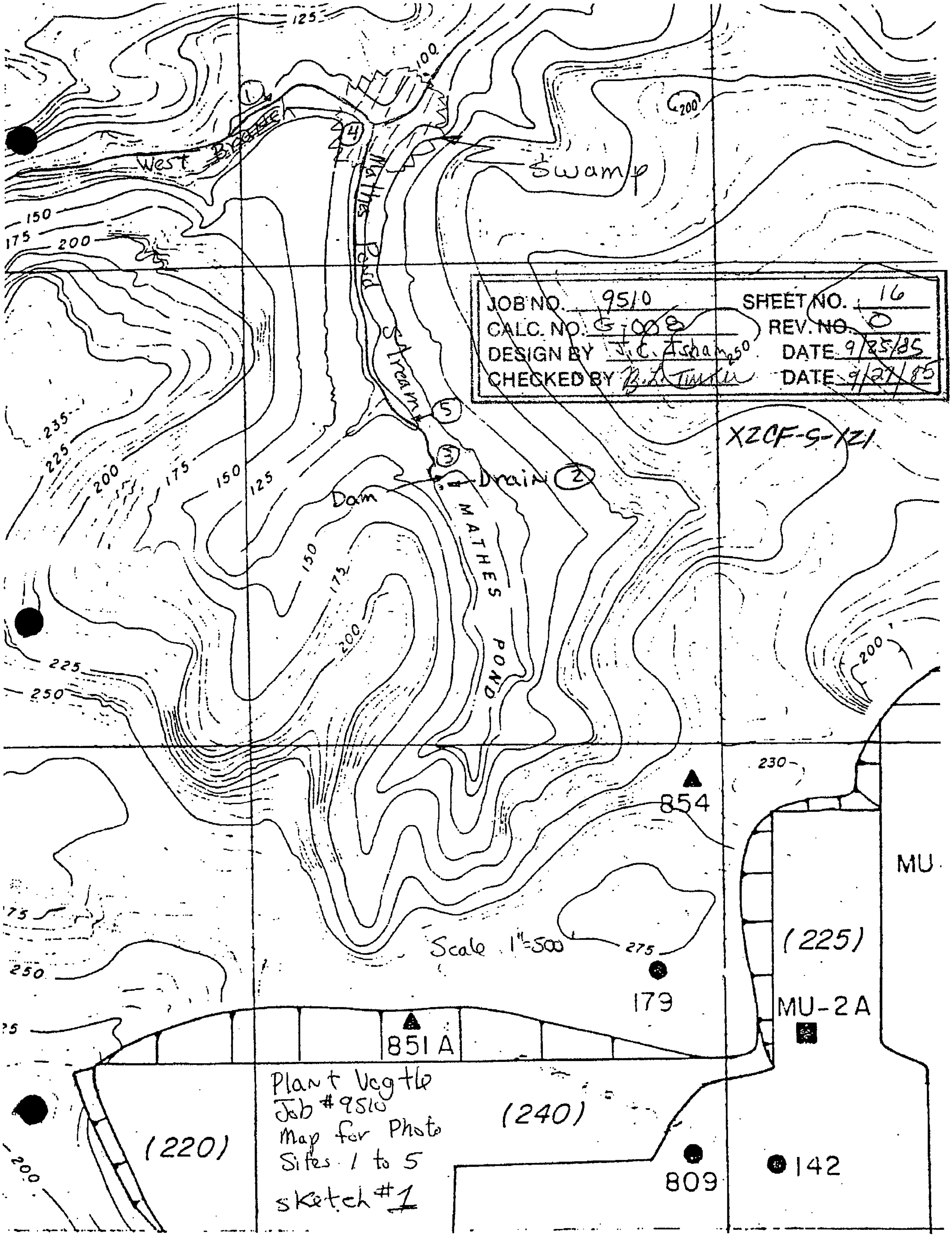
PROJECT Vogtle N.P.P. JOB NO. 9510-091

SUBJECT Flow Rate in Mathes Pond & West Branch Stream CALCULATION NO. G-008 FILE NO. XZCF-5-121

These factors include the lack of precision in the correction coefficients, stream roughness estimates, and experimental errors in measuring time and distances. It is estimated, as a result of these factors (especially the shallow and rough nature of the stream channel), that the accuracy of the flow measurements are approximately 25%.

III Conclusion :

The total flow from the Mathes Pond Drainage Basin (combined flow from Mathes Pond & West Branch streams) could range from 800 to 1200 gpm



JOB NO. 9510
SHEET NO. 16
CALC. NO. G-008
REV. NO. 0
DESIGN BY J.C. Isham
DATE 9/25/85
CHECKED BY B.L. Turner
DATE 9/27/85
XZCF-S-121
West Branch
Swamp
Mathes Pond
Stream
Dam
Drain
MATHES POND
854
MU
(225)
MU-2A
179
851A
Scale 1"=500
(220)
(240)
809
142
Plant Vogtle
Job #9510
Map for Photo
Sites 1 to 5
sketch #1

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KEUFFEL & ESSER CO. MADE IN U.S.A.

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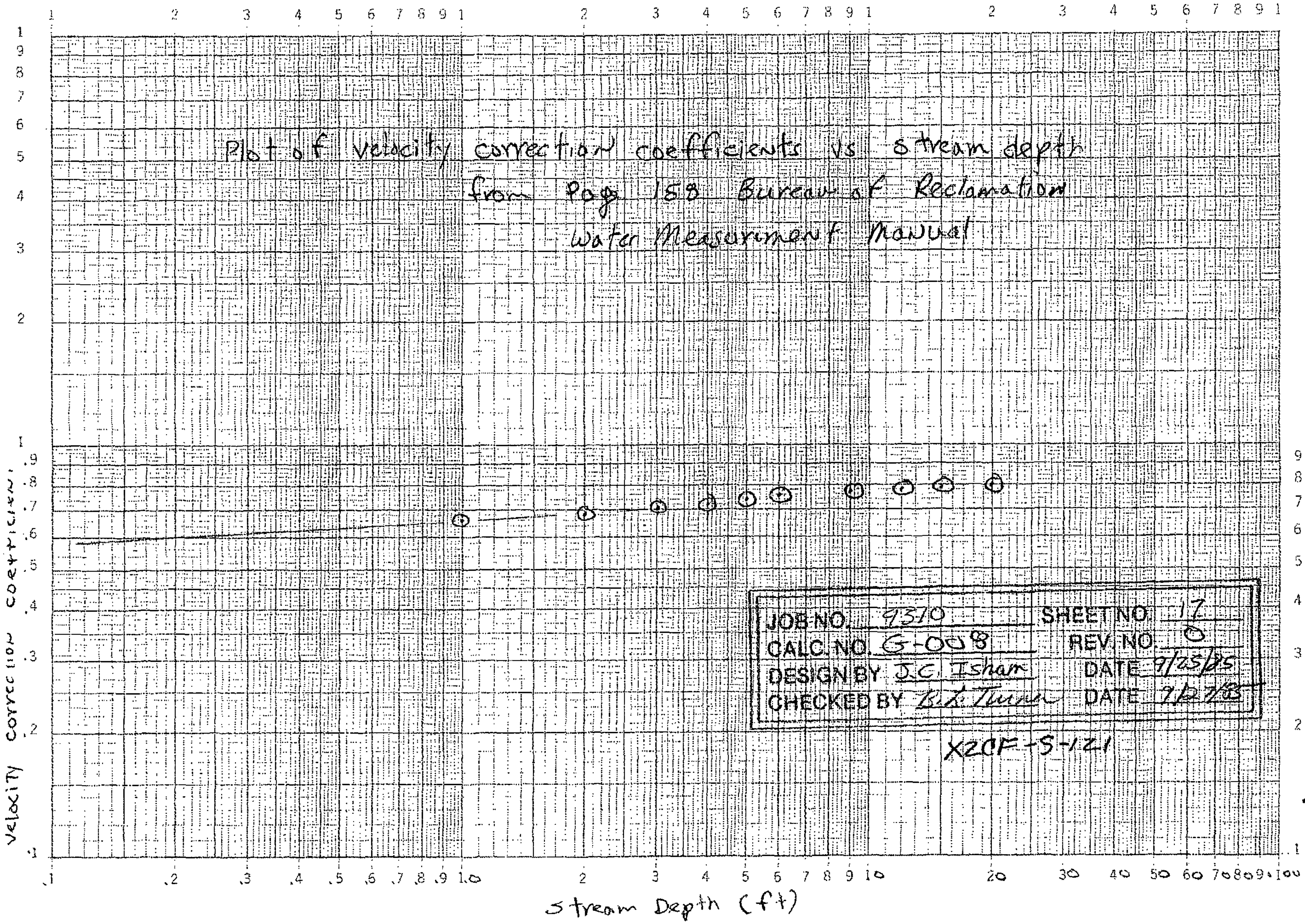

Page 11