2. FOIA 84-463

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON D.C. 20555

JAN 24 1984

Docket Nos. 50-524/525

MEMORANDUM FOR: Elinor Adensam, Chief
Licensing Branch #4, DL

FROM: Ronald L. Ballard, Chief
Environmental & Hydrologic Engineering Branch, DE

SUBJECT: HYDROLOGIC ENGINEERING SAFETY QUESTIONS FOR VOGTLE
OL REVIEW

Plant Name: Vogtle Electric Generating Plant
Licensing Stage: OL
Docket No. 50-524/525

Attached are Hydrologic Engineering Safety Questions for transmittal to the applicant prior to the February 6 site visit. We consider the attached as Draft Questions, since some may be resolved or revised and others may be generated as a result of the site visit.

We are in the process of initiating a contract that will include the review of the mechanical draft cooling tower (UHS) performance at Vogtle. Our contractor may request additional information from the applicant in order to complete his review. However, since that contract has not yet been finalized we cannot now provide an estimated date for those questions.

This review was performed by Gary B. Staley of the Hydrologic Engineering Section, phone X28003.

Ronald L. Ballard
Ronald L. Ballard, Chief
Environmental & Hydrologic
Engineering Branch
Division of Engineering

Attachment: As stated

cc: W. Johnston
O. Parr
W. Gammill
M. Miller
M. Fliegel
J. Kane
G. Staley

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Vogtle Electric Generating Plant
Hydrologic Engineering Safety Questions
Docket Nos. 50-424/425

240.1 Your design basis ground water level of elevation 165.0 ft msl is not substantiated by some of the observation well readings in the water table aquifer. Well numbers 124 and 142 have readings in excess of elevation 200 ft msl for several quarters. Figure 2.4.12-7, Sheet 2, shows a groundwater elevation of about 145 ft msl near well number 129, whereas Table 2.4.12-7, Sheet 2, shows an elevation of 176.0 ft msl for first quarter 1980 for well number 129.

It appears from your discussion in Section 2.4.12 that your design bases value (165.0 ft msl) may represent more of an average value rather than an upper limit. The design basis groundwater should not be exceeded during the life of the plant. Provide additional justification to support your selected design basis ground water level of 165.0 ft msl. Your justification should include reasons for apparently disregarding some observed higher recorded values in the vicinity of the main plant area. Alternately, you may provide a revised (higher) design basis ground water level that can be supported by the records and will reflect a value that is not likely to be exceeded during the life of the plant. Your response should also include consideration of historic rainfall records in comparison to what has occurred during your groundwater monitoring period.

See Response - Amend. 5 4/84

240.2 You have not provided sufficient information for the staff to review your provisions for site drainage. Provide the following information:

1. Full size (unreduced) drawings for Figure 2.4.1-2, sheets 1 and 2.

2. On the drawings mark the contributing drainage area and subbasins.

3. The drainage area, time of concentration, runoff coefficient and peak discharge (for the PMP) for each subbasin.

4. Elevations at each change in grade for all peripheral roads and railroads. Also provide sufficient spot elevations on all flat or gently sloping areas (main plant area, parking lots, switchyard etc.) such that the staff will be able to determine slopes or elevation limits.

5. Arrows on drawings to indicate assumed flow paths for overland and ditch flow.

6. Ditch cross sections and invert elevations at extremities and at each change in grade or size.

7. Locate all culverts (used for PMP discharge) on the drawings and provide the type and shape of pipe, inlet and outlet invert elevations and shape or type of inlet.

8. The design basis water surface elevation for safety-related structures in the main power block area as a result of local PMP on the site area. You should also provide the maximum water surface elevation (due to local PMP) for each subbasin that contributes flow in the vicinity of the power block.

See Response-Amend 5 4/84

240.3 Your estimates for probable maximum precipitation (PMP) in Table 2.4.2-2 are 20% to 30% less conservative than the values estimated by the staff using Hydrometeorological Report (HMR) 51 and 52. Since our Standard Review Plan 2.4.2 allows for at most a 5% difference, this discrepancy must be resolved. Since both staff and applicant values have been interpolated from HMR 51 and 52, there is apparently some judgemental error in interpretation. For the purpose of resolving this difference, we have listed below the values the staff determined and the appropriate HMR Figure number that was used:

1 hour 1 sq. mi PMP	19.1 inches	Fig 24, HMR 52
5 to 60 minute ratio	0.323	Fig 36, HMR 52
15 to 60 minute ratio	0.506	Fig 37, HMR 52
30 to 60 minute ratio	0.736	Fig 38, HMR 52

Provide your revised values and additional discussion to substantiate those values if different from the staff's.

See Response - Amend 5 4/84

240.4 Provide a tabulation of existing groundwater users and a map showing the location and other pertinent information as described i Section 2.4.13.2 of NUREG-75/094.

See Response - Amend 5 4/84

240.5 FSAR Section 2.4.13 is incomplete. Describe the nearest downgradient groundwater and/or surface water users and show that a postulated release from the most critical radwaste storage tank (which you must identify or cross reference) will result in concentrations at the nearest downgradient user that are less than those identified in 10 CFR Part 20, Appendix B, Table II, Column 2.

3. FOIA 84-463

1/24/84
1 of 6
J. Kune

Review Comments - VOGTLE FSAR

VEGP - Vogtle Electric Generating Plant - Pressurized water reactor, 3411 MWt (ea
Location - Southwest side of Savannah River in Burke Co., Ga. - 26 miles SE of Augusta
Across the Savannah River from DOE Savannah River Plant

Scheduled FUEL LOADING DATES
Unit 1 September 1986
Unit 2 March 1988

Table 1.8-1 DEVIATIONS FROM SRP

Par. 2.5.4.15 Only deterministic approach (not probabilistic) used in LIQUEFACTION ANALYSIS

Par. 3.7.B.1.5 SRP indicates vibratory motion should be applied @ FOUNDATION level. For VEGP the seismic design input (for deeply embedded structures) was applied @ GRADE LEVEL

Par. 3.7.B.2.15 SRP indicates for soil-structure interaction studies - use both half-space & finite element methods of analysis
For VEGP, finite element method used for deeply embedded structures
half-space method for shallow founded structures

Seismic analysis of VEGP for deeply embedded structures does not have a free-field response spectra at foundation level that is enveloped by the design response spectra - required by SRP

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Par. 3.8.2.8, 3.8.3.8, 3.8.4.8 ACI-318 was used rather than ACI-349

DEVIATION FROM PSAR to FSAR - Table 1.3.2-1 (Sheet 1 of 6) indicates change in compaction criteria from 97%

Review Comments - VOGTLE FSAR (cont.)

Par. 9.2.5.6 Ultimate heat sink should be capable of providing cooling water for 30 days (See Section 9.2)

SITE CONDITIONS (Pg. 1.2.1-2)

Savannah River - Normal water elev. 80, low water elev. 70

Pg. 2.4.1-1) Plant Site - Original elevations ranged from 225 to 250 (excluding intake struct.
Final Plant Grade - El. 220 ft (MSL) (Actually 219-5' Pg. 2.5.4-

Pg. 1.2.1-2) Evidence of solutioning due to leaching of calcareous sands by local near-surface SUBSIDENCE. Solutioning stated to be in materials ABOVE structure's bearing layer.

TYPICAL GEOLOGIC CONDITIONS

Ground level

80' to 110' Red-yellow SANDS & CLAYS

lower levels of sands are calcareous & leachable

70' Hard, impervious CLAY (the CLAY MARL)

Dense SANDS to Very Dense Sands

750' thick

SITE is approx. 104 miles from epicenter of 1886 Charleston earthquake

* * PMF Elevation of 165 ft (allowing for 27' of wave run-up, Pg. 2.4.2-1.
allows for upstream dam failures & surges
without run-up - PMF elevation = 138 ft

Pgs. 1.2.2-1 thru 1.2.2-3 Briefly IDENTIFIES CAT. 1 Structures
See Fig. 1.2.2-1 for location of structures

Review Comments - VOGTLE FSAR (cont.)

Volume 2. -Section 1.9 GPC indicates their position with respect to NRC Regulatory Guid

Reg. Guide	GPC Position (Page No.)
1.70 Std Format	Conformed - See Section 1.1.6
1.132 "Site Investigations etc Mar. 1979	Conformed (Pg. 1.9-111)
1.138 "Lab Investigation, etc. Apr. 1978	[GPC indicates investigations were completed prior to issuance of guide - Pg. 1.9-115]
1.127 "Inspection of Water Controlled Structures __" Mar. 1979	[GPC indicates that VEGP does not have water control structures that are Category 1]
1.29 "Seismic Design Classif.	Conformed (Pg. 1.9-23)
1.60 "Design Response Spectra	Conformed (Pg. 1.9-55) See Sect. 3.7.1
1.61 "Damping Values ---"	See Section 3.7.B.1 & 3.7.N.1 for discussion on conformance (Pg. 1.9-55)

Volume 3 - Entirely on Meterology

Volume 4

Pg. 2.4.2-1 There are NO SAFETY-RELATED structures that COULD BE AFFECTED by FLOODS & FLOOD WAVES

Pg. 2.4.2-1 The makeup water RIVER INTAKE STRUCTURE, and INTAKE CANAL is not seismic Cat. 1. This
Pg 2.4.8.1 is because it is a secondary backup to the makeup water system - the nuclear service cooling water which obtains water from wells & has 30 day supply w/ wells pumping

1/24/84
4-10
J. Kane

Review Comments - VOGTLE FSAR (cont.)

Volume 4 (cont.)

Pg. 2.4.4-1 13 Upstream Dams - 3 Major Dams - Hartwell (El. 675±), Clark Hill (El. 335±, concrete dam), Richard B. Russell (El. 475±)

ASSUMED WORST POSSIBLE SCENARIO OF UPSTREAM DAM FAILURES:

- Jocassee Dam fails under SPF conditions & earthquake. Assumed breach would permit 30' high wave to develop below dam
- At Keowee Dam (15 miles downstream of Jocassee) the arriving wave is 16' high & Keowee is assumed to fail allowing for SPF conditions to exist before failure
- At Hartwell Dam (51 miles downstream of Keowee) the arriving wave is 15' high & Hartwell Dam is assumed breached
- At Richard B Russell Dam the arriving wave is 20' and overtops the Russell Dam
- At Clark Hill Dam a surging 15 ft wave overtops Clark Hill Dam
- At VEGP the resulting river would reach el. 141 ft & with surge wave and wind caused runup would reach el. ~~152~~ 168

NSCWS - Nuclear Service Cooling Water System

Pg. 2.4.11-3 Two NSCWS cooling tower basins for EACH UNIT

During Normal plant operation - need 406 gpm of makeup water because of evaporative losses, drift & cooling tower blowdown

During Accident Condition - need 1270 gpm decreasing to 210 gal/min after 30 days

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Pg. 2.4.12-1 Ground Water

LOWER AQUIFER - CRETACEOUS AQUIFER (also called TUSCALOOSA AQUIFER)

In power block site - Elev. ranges - check borings in site area

1/25/84
5 of 10
J. Kane

Review Comments - VOGTLE FSAR (cont.)
Volume 5 - Section 2.5 Geology, Seismology & Geotechnical Engineering

2.5.1-1) EXTENT OF INVESTIGATIONS (BORINGS)
PSAR STAGE - 474 holes over 69,000 ft of drilling
Post-PSAR STAGE - 111 holes

2.5.1-1) FIELD INVESTIGATIONS INCLUDED:
- Geologic Mapping
- Drilling borings - Electric logging
- Geophysical survey (shallow & deep refraction seismic lines, cross-hole velocities)
- Groundwater studies
- Water pressure tests & Menard pressure meter tests to determine IN SITU properties of plant ~~...~~ structure bearing stratum - the MARL
- Solubility tests (carbonate analysis)

2.5.1-1) LABORATORY INVESTIGATIONS INCLUDED:
- Normal classification (moisture, unit weight, A.L.
- Shear tests (triaxial & direct & dynamic
- Consolidation
- Relative Density
- Resonant column
- Compaction (moisture-density relationship)

J. Kane

Review Comments - VOGTLE FSAR (cont.)

Volume 5 - Section 2.5 (cont.)

2.5.1-1c) For SSE, intensity of VII - VIII, Peak horiz. ground acceleration = 0.20g
OBE, peak horiz. ground acceleration = 0.12g

.1-1d) The Category 1 structures (Auxiliary building, instrumentation cavity of reactor containments & nuclear service cooling water towers) founded directly on the MARL (Grn-gry sandy CLAY - soft rock) which averages about 90' depth from ground surface

.1-1d) All other powerblock structures (? Categ 1) are founded on compacted backfill which extends in depth to the MARL stratum

.5.1-17 A deep boring, TW-1 reached deeply into the lower dense to v. dense SAND but did not reach bedrock

2.5.1-18 In Savannah River width - the bearing layer marl has been eroded and replaced with recent alluvium - See Fig. 2.5.1-14

Figure 2.5.1-23 present EXCAVATION PLAN to Marl Bearing Stratum and provides locations of explorations into Marl layer (El. 130 ft±)

Description of MARL bearing layer - See Pgs vol.5 2.5.1-19 to 2.5.1-22

Geologic Units A thru J (Between El. 132 ft± to El. 108.6± (Mapped in Auxil. Bldg Area.)

Pg 2.5.1-19 Unit A - Bottom El @ avg el 120 Dk gry silty to clayey MARL w/ lt. gry sandy laminations, subject to dessication ? Scattered shell fragments

Unit B - 1' to 4' thick, Gry sandy MARL, less subject to dessication - poorly cemented

Unit C - lt. gry LIMESTONE - about 1' thick (avg.)

Unit D - med. gry sandy to silty MARL, highly fossiliferous - about 8' thick

Unit E - about 1' thick, lt. gry indurated LIMESTONE, fossiliferous (El. range 121' to 116')

Unit F - Med gry sandy to silty MARL 1' to 4' thick, fossiliferous w/ clay pods

Unit G - Lt. to Dk gry laminated MARL - sandy laminations

7.1-10
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REVIEW COMMENTS - VOGTLE FSAR (cont.)

2.5.1-21 vol 5 Unit H - Dense gray MARL, 1' to 6' thick of shell fragments

Unit I - Lt gry LIMESTONE, <1' thick

Unit J - bottom of Auxil Bldg. fdn excavation - med gry MARL

2.5.1-38) Discussion on HEAVING of MARL stratum following power block excavation - See Fig 2.5.1-38 for location

From 1974 to 1977 - measured rebound of 1" to 1.6"

2.5.4-2 Upper Sand Stratum (To Approx El 140) has POTENTIAL for LIQUEFACTION

UTLEY LIMESTONE (El. 140 to El. 135±) was solutioned & had CAVITIES

2.5.4-3 Engineering Properties of Soils - See Tables 2.5.4-1 & 2.5.4-2

UPPER SAND STRATUM (also contains silty & clayey SAND lenses)

Sands vary in density from very loose to dense

Clay lenses from ~~very~~ soft to medium

SHEAR STRENGTH (c & ϕ)

		Table 2.5.4-2 ADOPTED IN DESIGN
U-U	$c = 2100$ psf, $\phi = 6°$ to $c = 440$ psf, $\phi = 32°$	$c = 2300$ psf, $\phi = 6°$
C-U	$c = 1650$ psf, $\phi = 17°$ to $c = 4000$ psf, $\phi = 25°$	$c = 1000$ psf, $\phi = 18°$
C-D	$\phi = 33°$ to $\phi = 34.5°$	$\phi = 34°$

pg 2.5.4-3 DYNAMIC SHEAR MODULUS (G)

for Upper Sands Adopted $G = 2300$ KSF (@ 10^{-4}% strain)

Basis for adopted $G = \frac{\gamma}{g} \cdot V_s^2$ where $V_s \approx 800$ ft/sec? - Variation considered

$\gamma = 115$ pcf

VOGTLE - SETTLEMENT STUDIES

Procedure Used To Estimate Settlement in CLAY MARL

INITIAL ASSUMPTIONS

1. Both "immediate" and "primary" consolidation settlements were estimated

IMMEDIATE SETTLEMENTS

Assumptions - Low permeability - hence constant volume (no pore water loss) and therefore $\nu = 0.5$

- Used elastic theory and adopted Young's modulus based on undrained shear tes
  note difference in selection of strength from below
  Adopted $E_s = 400 \cdot C = 400 \times 10{,}000\ lb/ft^2 = 4000000\ lb/ft^2 = 4000\ k/ft^2$

See PSAR, App C Pg. 2C-30,31

- Used SEPOL computer program to estimate settlements. Broke layer into approx. 2' deep zones, computed strain = (? from $\frac{\text{(load) pressure change}}{E}$) $(1-\nu)$; then computed settlements from $\Delta H = \text{strain} \times H$ where H = thickness of layer

PRIMARY SETTLEMENTS (or CONSOLIDATION SETTLEMENT)

Assumptions - With sufficient time, pore water seepage would occur & volume would decrease and primary consolidation would occur

- Used work of Skempton to correlate plasticity index of clay marl to ratio of $C/\bar{p}$ where C = undrained shear strength and $\bar{p}$ = effective vertical pressure @ sample depth

- Plasticity index ranged from 2 to 70 - Adopted average PI = 25 Unrealistic to make selection

- From Skempton's curve of PI vs $C/\bar{p}$, determined $C/\bar{p}$ ratio = 0.2
- From wide scatter of undrained shear strengths vs elevation (PSAR Fig 2C-6, App. C) selected C = 16,000 psf **(Unrealistic to make selection)**

$$\therefore \frac{C}{\bar{p}} = 0.2 = \frac{16{,}000\ psf}{\bar{p}} ; \quad \bar{p} = \frac{16{,}000\ psf}{0.2} = 80{,}000\ psf$$

- Since $\bar{p}$ = 80,000 psf is considerably greater than existing effective vertical pressure - concluded clay marl is over consolidated and primary settlements would be small

Procedure (cont.) Used to Estimate Primary (or CONSOLIDATION) Settlements

1. Clay Marl stratum divided into 3 twenty foot thick depth zones
2. Because of power block excavation to El. 130 it was necessary to estimate the amount of rebound after having computed the reduction in effective vertical pressure (Details not given – need to have correlated with dewatering effects which were preceding excavation to El. 130) Was lab consolidation test results used to estimate amount of rebound?
3. Used lab consolidation test results and computed structure & backfill loads to estimate amount of recompression & consolidation settlement

$$\rho_c = \frac{C_r \cdot h}{1+e_o} \log_{10} \frac{\bar{\sigma}_v + \Delta\sigma_z}{\bar{\sigma}_v}$$

where ρ_c = consolidation settlement
C_r = compression index
h = layer thickness
e_o = initial void ratio
$\bar{\sigma}_v$ = in situ effect. vertical stress at mid depth of layer
$\Delta\sigma_z$ = effective additional vertical stress @ layer mid-depth due to loads @ top of clay marl surface

4. $\frac{C_r}{1+e_o}$ = 0.0046 (average value) was used in settlement computations
5. Total (immediate and primary (or consolidation) settlements and net settlements (beyond the amount of rebound) that were estimated @ PSAR stage can be seen on Fig 2C-15, App. C of PSAR

REVIEW COMMENTS - VOGTLE FSAR (cont.)

pg. 2.5.4-3) CLAY MARL STRATUM (BLUE BLUFF MARL)

Hard to very hard brittle, slightly cemented calcareous CLAY (resembling calcareous siltstone or claystone

Top 15' of clay marl, V_c = 5000 fps, remaining 55' depth, V_c = 7000 fps

SHEAR STRENGTH	ADOPTED in Design
U-U C = 260 psf to C = 500,000 psf 1.8 psi to 3472 psi	C = 10,000 psf Table 2.5.4 - 2

(31 borings were drilled into the CLAY MARL in 1977 after foundation excavation to El. 132± (top of marl) had been completed - See Jan 1978, Vol. II, Part 3 report (yellow-black covering) entitled "Report on Backfill Material Investigation" by Law Engineering. Of the 31 borings, 9 borings showed core recovery less than 70% and, of these 9 borings, 8 borings showed depth intervals of no recovery. (Poor recovery @ El. 127 & El. 105 depth intervals)

SETTLEMENT (COMPRESSIBILITY) CHARACTERISTICS (See added sheets 8a & 8b)

pg 2.5.4-6 EXPLORATION PROGRAMS (also see App. 2B of Vol. 5 for boring inventory)
Initially started explorations in 1971

pg 2.5.4-7 Extent of Power Block Excavation - Approx 1400' by 1000' (Larger Area to El. 13.
Slopes 2H to 1V
Total volume = 5,000,000 yd^3

Deeper, Smaller Area (Auxil. Bldg) to El. 108 measured 120 ft by 440 ft
Slopes vertical in clay marl

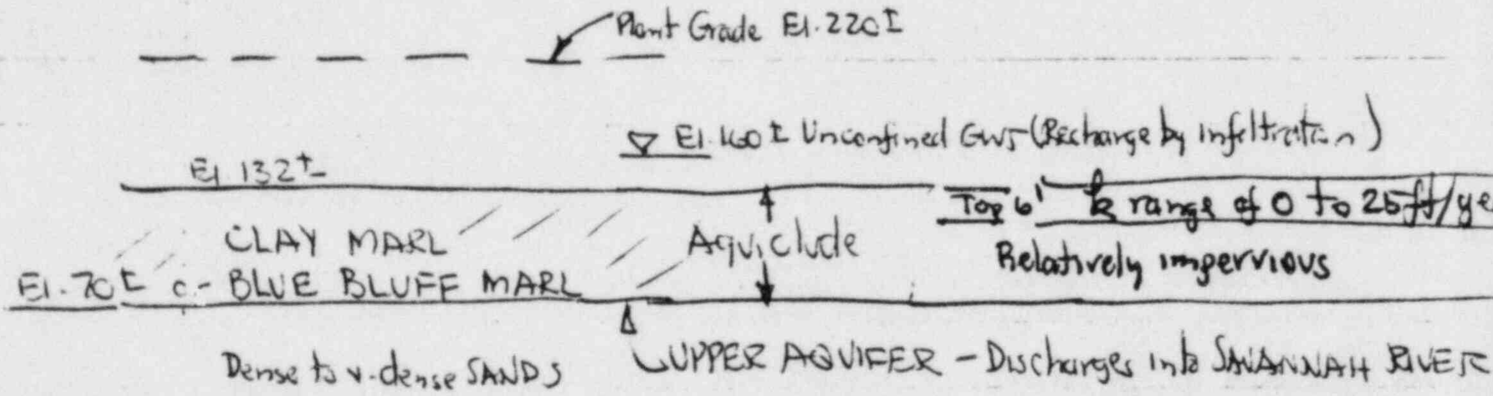
REVIEW COMMENTS - VOGTLE FSAR (cont.)

Foundation Excavation started in May 1974 & went to El. 145 by Sept. 1974
Remaining Excavation (To El. 130 & 108) not completed until Feb 1977 to Oct 1977

Limited blasting of Utley Limestone was required

2.5.4-9) CLAY MARL was excavated by RIPPING

2.5.4-20) Groundwater Conditions - See Sheet 9a & 9b for pressure diagram



* pg. 2.5.4-20) Power block structures designed to GROUNDWATER El. 160

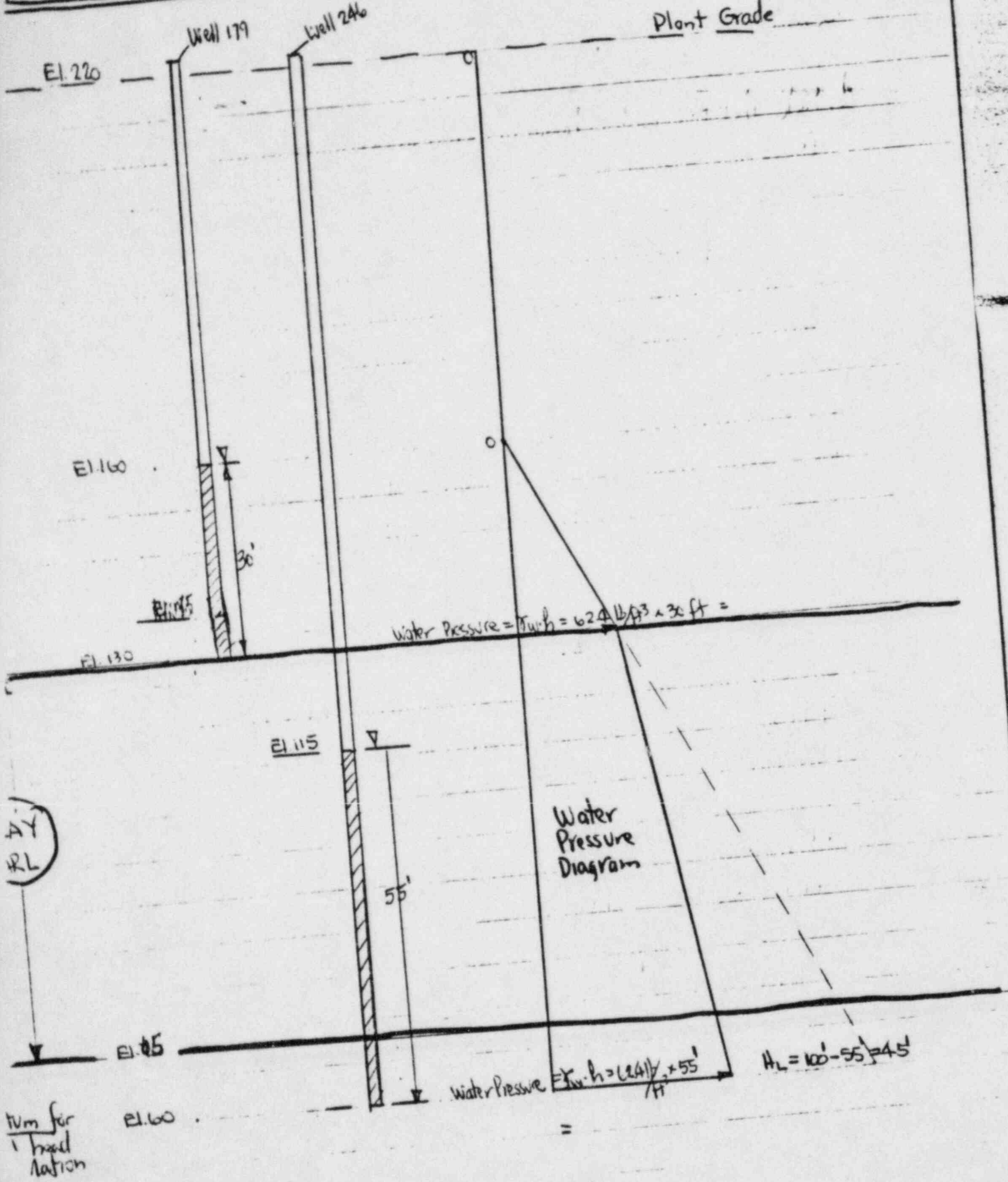
SETTLEMENT STUDIES

pg 2.5.4-26)

Settlement of Structures on Cat. 1 Backfill (sand & silty sands)
Estimated only "immediate settlements" (not consolidation settlements) for structures on Cat. 1 Backfill
Soil Parameters used for Cat. 1 backfill
$\gamma_m = 120 pcf$, $\gamma_{sat} = 130 pcf$, Poisson's ratio = 0.4, $E_s = 1500 ksf$

REVIEW COMMENT - VOGTLE FSAR (cont.)

J. Kane



Water Pressure Head Loss Through CLAY MARL = 45'

(over)

Review Comments - VOGTLE FSAR (cont.)

Comments

1. Note that piezometric heads measured in observation wells (or piezometers) are WATER PRESSURE HEADS $= \frac{P}{\gamma_w}$

Measuring the piezometric level permits the water pressure to be calculated at the tip elev. of the well (or piezometer) from equation water pressure $= \gamma_w \cdot h$
where h = height water rises in well (or piezometer) above the tip elevation

2. Note distinction between total head and pressure head
where total head = Elevation head + Pressure head + Velocity Head

$$= H + \underbrace{\frac{P}{\gamma}}_{\text{what is determined in well or piezometer}} + \text{Negligible and neglected for water seeping thru soil}$$

3. If total head were to be calculated for wells 179 and 246

$T.H._{246}$ = Elev. head + Pressure head

$T.H._{246} = 0 + 55' = 55'$

$T.H._{179}$ = Elev. head + Pressure head

$T.H._{179} = (El\ 130 - El.\ 60) + 30'$

$T.H._{179} = 70' + 30'$

$T.H._{179} = 100'$

4. The head loss thru the CLAY MARL $= T.H._{179} - T.H._{246}$

$= 100' - 55'$

$H_L = 45'$

REVIEW COMMENTS - VOGTLE FSAR (cont.)

pg. 2.5.4-27) Settlement of Structures on CLAY MARL stratum

Estimated "immediate settlements"

Used soil parameters

- E_s range from 4000 ksf to 10,000 ksf, γ_{sat} = 115 pcf, Poisson's ratio = 0.5

Estimated "consolidation settlements" $\rho_c = \frac{C_c \cdot h}{1+e_0} \log\left(\frac{\sigma_v' + \Delta\sigma_v}{\sigma_v'}\right)$

Used soil parameters

- Allowed for rebounding (after heaving) using rebound index, C_r, from lab consolidation tests on clay marl. This resulted in an average value of $\frac{C}{1+e_0} = 0.0046$

Caisson Design - See pg. 2.5.4-29

REVIEW COMMENTS - VOGTLE FSAR (cont.)

pg. 2.5.4-27) Settlement of Structures on CLAY MARL stratum

Estimated "immediate settlements"

Used soil parameters

- E_s range from 4000 ksf to 10,000 ksf, γ_{sat} = 115pcf, Poisson's ratio = 0.5

Estimated "consolidation settlements" $\rho_c = \frac{C_c \cdot h}{1+e_o} \log \left(\frac{\sigma_v' + \Delta\sigma_v}{\sigma_v'} \right)$

Used soil parameters

- Allowed for rebounding (after heaving) using rebound index, Cr, from lab consolidation tests on clay marl. This resulted in an average value of $\frac{C}{1+e_o} = 0.0046$

Caisson Design - See pg. 2.5.4-27