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September 14, 1984

United States Nuclear Regulatory Commission
Washington, DC 20555

ATTENTION: Mr. George W. Knighton, Chief
Licensing Branch 3
Office of Nuclear Reactor Regulation

SUBJECT: Beaver Valley Power Station - Unit No. 2
Docket No. 50-412
Response to Draft SER Open Item 178

Gentlemen:

The response to the NRC Geotechnical Engineering Section's Draft SER Open Item No. 178 is provided in Attachment 1. The associated revisions to FSAR Section 2.5.4 are provided in Attachment 2.

DUQUESNE LIGHT COMPANY

By E. J. Woolever
E. J. Woolever
Vice President

JDO/wjs
Attachments

cc: Ms. M. Ley, Project Manager (w/a)
Mr. E. A. Licitra, Project Manager (w/a)
Mr. G. Walton, NRC Resident Inspector (w/a)

SUBSCRIBED AND SWORN TO BEFORE ME THIS 14th DAY OF September, 1984.
Anita Elaine Reiter
Notary Public

ANITA ELAINE REITER, NOTARY PUBLIC
ROBINSON TOWNSHIP, ALLEGHENY COUNTY
MY COMMISSION EXPIRES OCTOBER 20, 1986

B001
1/1

COMMONWEALTH OF PENNSYLVANIA)
) SS:
COUNTY OF ALLEGHENY)

On this 14th day of September, 1984, before me, a Notary Public in and for said Commonwealth and County, personally appeared E. J. Woolever, who being duly sworn, deposed and said that (1) he is Vice President of Duquesne Light, (2) he is duly authorized to execute and file the foregoing Submittal on behalf of said Company, and (3) the statements set forth in the Submittal are true and correct to the best of his knowledge.

Anita Elaine Reiter
Notary Public

ANITA ELAINE REITER, NOTARY PUBLIC
ROBINSON TOWNSHIP, ALLEGHENY COUNTY
MY COMMISSION EXPIRES OCTOBER 20, 1986

ATTACHMENT 1

Draft SER Open Item No. 178 (Section 2.5.4.3.4) - Densification of Soils (Liquefaction analysis of soils in the vicinity of the intake structure and sliding stability analysis for the intake structure):

The possibility, and the consequences, of liquefaction of the granular materials in the vicinity of the intake structure were thoroughly evaluated by the applicant (and reviewed by the NRC staff) at the construction permit stage as seen from the PSAR for BVPS-2, Amendment 13, dated February 28, 1974. Since liquefaction of these soils was considered likely, the applicant densified two areas west and east of the intake structure, each measuring 90' x 75', using the Terra Probe method. Areas immediately north of the intake structure and beneath the structure were not densified.

The effectiveness of the Terra Probe densification was evaluated by performing liquefaction analyses of the soils in the vicinity of the intake structure using the data obtained by verification borings drilled in the densified areas. For analyzing the liquefaction potential of the soils beneath and north of the intake structure, borings drilled in the vicinity prior to densification (includig the only preconstruction boring drilled beneath the intake structure) were used. The evaluation using the SSE indicated that the soils within the densified zones should not liquefy. The soils directly beneath the intake structure had a minimum factor of safety against liquefaction of 1.3 with the ground water level at el 665 ft (corresponding to normal river water level), and 1.1 with the ground water level at el. 690 ft. The applicant has, thus, shown that the soils east and west of the intake structure, and beneath the structure, have some margin of safety against liquefaction for the combination of SSE and 25-year flood.

The applicant has also performed, but not yet docketed, a sliding stability analysis for the intake structure. In addition to this analysis, the applicant must also reevaluate and docket the liquefaction potential analysis of the soils beneath, and east, and west of the intake for the combination of OBE and a ground water level corresponding to the standard project flood (el. 705 ft) as recommended by SRP 2.4.4.

Response:

FSAR Section 2.5.4.8.1 describes the liquefaction analysis of the soil at the main intake structure. As discussed in this section, raising the water level will not affect the results of the liquefaction analysis. The results presented are for the most conservative case of the SSE load and a fully saturated soil profile. Directly beneath the main intake structure, the SSE + 25-year flood condition is more conservative than the OBE + standard project flood condition. This is due to the fact that applied shear stress is a function of acceleration and total vertical overburden stress. Raising the water level in the intake bays to standard project flood elevation 705 ft. and reducing the acceleration to the OBE level produces lower applied shear stresses than for the SSE + 25-year flood condition, and thus a higher factor of safety against liquefaction. See revised FSAR Figure 2.5.4-36 for a summary of the liquefaction analysis beneath the main intake structure. Also, see revised FSAR Section 2.5.4.8.1 for a discussion of sliding stability. These FSAR revisions are provided in Attachment 2 and will be incorporated into a future FSAR amendment.

Only one pre-construction boring, boring 4, was drilled beneath the intake structure, and none were drilled in the channel area immediately north of the structure. Since neither area was densified, for the liquefaction analyses it was assumed that borings drilled prior to densification were representative of conditions beneath and north of the structure.

The results of the analyses are presented on Figures 2.5.4-33 through 2.5.4-36. The densified area south of the riverward sheetpile walls has satisfactory factors of safety against liquefaction with all values at or above 1.6 (Figure 2.5.4-33). Offshore, the soils within the densified zone are not susceptible to liquefaction as shown by the preponderance of samples having acceptable factors of safety. Two samples at a depth less than 5 feet in two different borings have unsatisfactory factors of safety (Figure 2.5.4-35), but this is neither significant nor unusual due to low confining stress at shallow depths. Results of the analysis of the soils directly beneath the structure are shown on Figure 2.5.4-36. For the river at el 665 feet, the minimum computed factor of safety against liquefaction is 1.3. For the case with water level at el 690 feet, two samples had a factor of safety of 1.1, and the remainder had higher factors of safety. Therefore, the soils east and west of the intake, and beneath the intake, have an adequate factor of safety against liquefaction. ← Insert "A" (pg. 2.5.4-19a)

The soils directly in front of the intake structure were not densified. This area has been dredged to approximately el 645 ft. As shown on Figure 2.5.4-34, ten samples between el 645 ft and el 634 ft had factors of safety less than 1.1. Most of these samples occur within the top 5 to 10 feet of the soil profile; however, one sample at approximately el 623 ft was unsatisfactory. Similarly, from samples above el 645 ft along the channel slopes prior to densification, approximately the upper 10 feet of soil is loose and may also liquefy. Therefore, when performing the dynamic slope stability analysis of the intake channel, the upper 10 feet along the slopes outside of the densified zone and below the dredge line were assumed to be liquefied at the end of the seismic event.

Insert B → (pg. 2.5.4-19a)

Two cross-sections of the intake channel slope at the locations shown on Figure 2.5.4-32 were analyzed for dynamic slope stability using the computer program LEASE II (SWEC 1980). One section was taken adjacent to the intake structure through the densified zone while the other section was taken approximately 100 feet from the intake structure beyond the densified zone.

The upper 10 feet of loose soil along the undensified slope and below the dredge line is susceptible to liquefaction. The pore pressure buildup in the loose zone during the seismic event is accounted for by reducing the friction angle from 25° for the drained case to 17° for the undrained case. This is conservative and assumes the pore pressure parameter equals 1, which is appropriate for loose soils (Lambe & Whitman 1969). A static, post-earthquake slope stability

Insert "A"

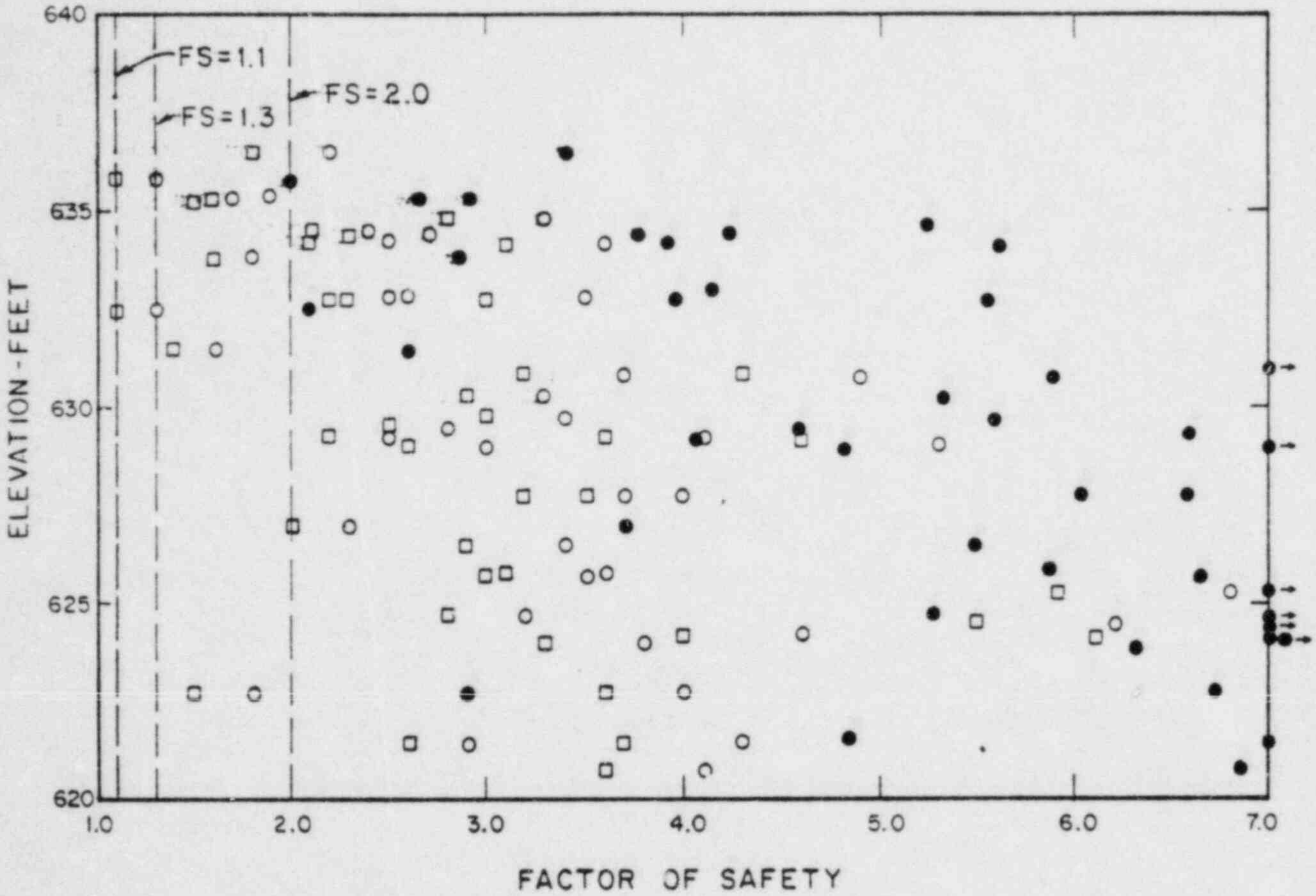
Figure 2.5.4-36 presents the results of the liquefaction analysis of the soils beneath the main intake structure for the following loading combinations:

1. SSE + normal water at elevation 665 ft.
2. SSE + 25-year flood at elevation 690 ft.
3. OBE + standard project flood at elevation 705 ft.

The minimum factors of safety against potential liquefaction were computed as 1.3, 1.1, and 2.0, respectively, for these loading combinations.

Insert "B"

Figure 2.5.4-65 presents the loading diagram used to calculate the factor of safety against sliding of the main intake structure. The water level within the intake structure is the same as the river level. During plant operation a maximum of one bay can be dewatered which would reduce the frictional resisting force along the base of the structure. During a seismic event, undrained shear behavior will govern sliding stability of the intake structure. Changes in vertical stresses at the soil-structure interface will cause a corresponding change in pore pressure. Therefore, the effective contact pressure will remain constant and equal to the effective building weight (total building weight minus static buoyant force). Consequently, only the horizontal component of interial force is considered in the sliding stability analysis. Under the conservative conditions of the SSE, standard project flood, and one intake bay empty, the factor of safety against sliding is 1.3 which is satisfactory. The dynamic sliding stability analysis of the intake structure was conservatively performed without taking into account passive resistance of the soil.



LEGEND

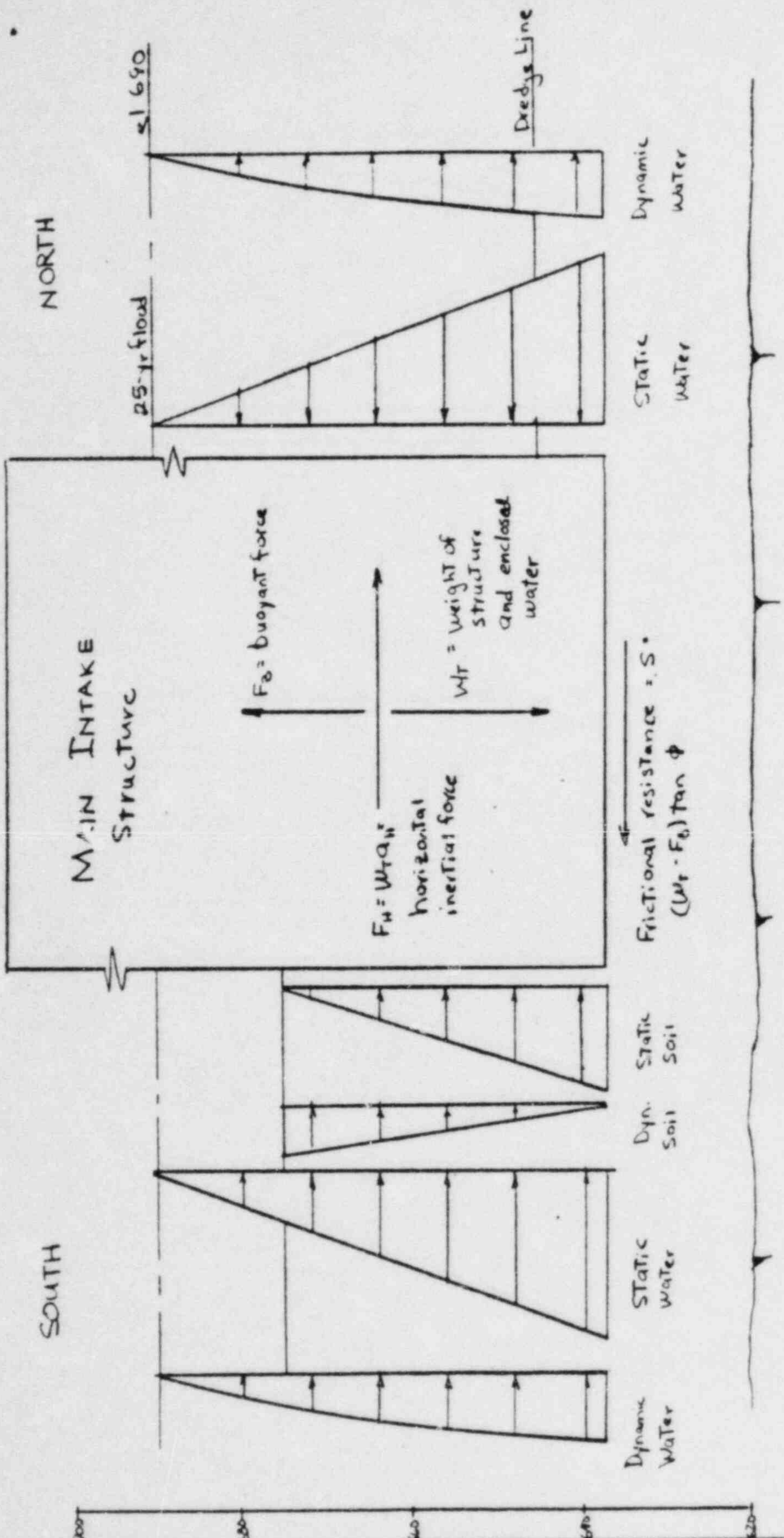
- ○ : NORMAL WATER LEVEL EL. 665' + SSE
- □ : 25 YEAR FLOOD EL. 690' + SSE
- ● : STANDARD PROJECT FLOOD EL. 705' + OBE

NOTE

ANALYSIS IS BASED ON SOIL DATA FROM BORINGS PRIOR TO DENSIFICATION

537T-548T, 4

FIGURE 2.5.4-36
LIQUEFACTION ANALYSIS OF SOILS BENEATH MAIN INTAKE STRUCTURE
BEAVER VALLEY POWER STATION - UNIT 2
FINAL SAFETY ANALYSIS REPORT



$$F.S._{SLIDING} = \frac{S}{F_H + \Sigma \text{ lateral soil forces} + \Sigma \text{ lateral water forces}}$$

Ref Fig. 2.5.4-42 for method of calculation of lateral forces

	TITLE Fig. 2.5.4-65	SCALE N.T.S
CHECKED JWM	Main Intake Structure	DATE 8-30-84
CORRECT	Dynamic Sliding Stability	SKETCH NUMBER
APPROVED		