TENNESSEE VALLEY AUTHORITY

CHATTANOOGA, TENNESSEE 37401

400 Chestnut Street Tower II

February 1, 1982

Director of Nuclear Reactor Regulation
Attention: Ms. E. Adensam, Chief
Licensing Branch No. 4
Division of Licensing
U.S. Nuclear Regulatory Commission
Washington, DC 20555

Dear Ms. Adensam:

In the Matter of the Application of) Docket Nos. 50-390
Tennessee Valley Authority) 50-391

Enclosed for NRC review is information concerning sheet pile walls at Watts Bar Nuclear Plant. This information should resolve open item 7 of the draft Safety Evaluation Report.

If you have any questions concerning this matter, please get in touch with D. P. Ormsby at FTS 858-2682.

Very truly yours,

TENNESSEE VALLEY AUTHORITY

L M Mills

L. M. Mills, Manager
Nuclear Regulation and Safety

Sworn to and subscribed before me
this 1st day of Feb. 1982
Paulette H. White
Notary Public
My Commission Expires 9-5-84

Enclosure

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ENCLOSURE

WATTS BAR NUCLEAR PLANT UNITS 1 AND 2
SHEET PILE WALLS

Question

A description of the method of static and dynamic analysis of the tieback sheetpile walls, and the results of the analysis should be promptly provided by the applicant for staff review. The applicant should include in the FSAR a list of soil-related parameters, including anchorage used in the design, the design values of these parameters, and the bases for obtaining the design values used in the analysis.

Response

Tieback Sheet Pile Walls

(i) Description of the Method of Static and Dynamic Analysis

As stipulated in Section 3.8.4.3.2 of the WBN FSAR, all components of the tieback sheet pile walls were designed for the three loading cases and allowable stresses of table 3.8.4-10 of the FSAR. These components include the sheet piles, wales, anchor rods, and deadmen as shown on figures 3.8.4-54 and 3.8.4-55 of the FSAR.

The static loads of the required load cases were calculated following the method presented in Chapter 11 of Foundation of Structures by C. W. Dunham, McGraw-Hill, 1962. The applicable portion of this text is presented in figures 1 and 2. A value of 210 psf per foot of depth was used for the unit passive pressure (Pn) and 32 psf per foot of depth was used for the unit active pressure (Pa) in the calculations.

The seismic loads used in each of the required load cases were calculated by the following approach:

1. The average acceleration of the soil acting against the sheet pile wall was determined for the 1/2 SSE and SSE conditions (0.26g and 0.53g, respectively).

2. Using the average acceleration of the retained soil mass, the dynamic earth pressures were calculated in accordance with TVA Report No. 8-194, "Dynamic Effect of Earthquake on Engineering Structures," Appendix E. This method gives the dynamic earth pressure as a percentage of the active earth pressure. The resulting load diagram of triangular shape is applied with its apex at the base of the wall and maximum ordinate at the fill surface. In this instance, the dynamic earth pressure for the 1/2 SSE condition was 72 percent of static and for the SSE condition was 200 percent of static.

The deadmen were sized by an equivalent fluid pressure method. With this approach, a net soil pressure equal to the passive soil resistance minus the active soil pressure is determined at the center of the deadman. The deadman is then sized so that its bearing area multiplied by the net soil pressure is greater than the imposed factored load by a factor of safety of 1.25. The governing condition was Case III listed in WBN FSAR Table 3.8.4-10. The equivalent passive fluid weight used in these calculations was 300 pcf, and the equivalent active fluid weight was 32 pcf.

The sliding stability of the entire compacted earthfill contained by the sheet pile walls was checked under seismic conditions.

The method of analysis was:

1. Assume each soil particle in the soil mass under consideration is subjected to its maximum horizontal acceleration.

2. These accelerations times their respective masses when summed result in the shear force that must be resisted to prevent failure.

3. The shear strength of the soil is to resist this force.

The maximum acceleration at the surface of the soil deposit is determined by multiplying the known maximum acceleration at the base of the deposit by an amplification factor. Amplification factors were determined for a uniform soil mass assuming an infinitely long soil deposit. The amplification curves for 10- and 20-percent damping are shown in Figure 3. Soil damping of 10 percent was used in the analysis. The variation of maximum acceleration through the depth of the deposit was assumed to be linear. The analysis was performed using the soil properties of $C=1200$ psf, $\phi=15^{\circ}$, $\gamma=120$ pcf, and a shear wave velocity of 1200 fps.

(ii) Results of Static and Dynamic Analysis

The tieback sheet pile walls and anchorage system adhere to the allowable stresses of Table 3.8.4-10 of the WBN FSAR when subjected to the load cases listed in the table.

The deadmen maintain a factor of safety against failure of greater than 1.25 under a safe shutdown earthquake.

The earthfill contained by the sheet pile walls is stable against sliding failure during a safe shutdown earthquake.

(iii) Bases of Obtaining Design Values for Soil-Related Parameters

Sheet Pile Wall and Deadman Analysis

The active pressure of 32 psf is based on $\phi=32^{\circ}$ for the backfill soil and a wall friction value of 16° for the sheet piles. These values are listed in section 3.8.1.3 of the WBN FSAR and referenced by section 3.8.4.3.1.

The passive pressures of 210 psf per foot of depth used in the sheet pile design and 300 psf per foot of depth used in the deadman design were specified in section 6.2 of "TVA Design Criteria for Intake Pumping Station and Concrete Structues," WB-DC-20-19.

6.2 General Design Requirements

Each wall shall be connected by ties to a common concrete "dead man" placed midway between the walls in the earthfill. The ties shall be steel rods or cables and be embedded in the "dead man" at one end and to the sheet piling on the other. The wale shall be on the inside of the wall and each sheet pile shall be bolted to the wale to transfer the reaction of the pile to the wale. This gives better protection against missile damage.

The "dead man" shall be designed in accordance with the alternate design method of the 1971 edition of the ACI 318 Code for the following conditions:

1. Both walls exerting forces simultaneously
2. One wall only attached to "dead man"

Drains shall be provided along the walls to eliminate any hydrostatic pressure due to fluctuations of reservoir level.

Passive resistance of earth shall be assumed as follows:

Sheet pile wall - 210 psf per foot of depth
Dead man - 300 psf per foot of depth

The values have been confirmed by soil borings that indicate the presence of well compacted clay material that will develop at least the design passive pressures.

Sliding Stability Analysis

The soil values used of C=1200 psf and $\phi=15^{\circ}$ are based on unconsolidated-undrained tests performed on the remolded intake channel soils shown in figure 2.5-251 of the WBN FSAR. These were assumed to be representative of the compacted fill placed between the sheet pile walls. The shear wave velocity of 1200 fps and unit weight of 120 pcf were judged to be representative of the compacted fill based on past experience with similar soils.

When the embedment of DB is considerable, it can support the shear at the bottom of the piling, and perhaps fix that end against rotation. In this case, the bending moment at Q and in the portion QB may be estimated in the following manner, referring to Fig. 11-13(b):

1. Assume the active-pressure diagram to be the rectangle $QUEB$.
2. Assume the passive-pressure diagram to be the triangle DVB, where angle α represents p_n p.s.f. per ft.
3. Assume that the passive pressure will be developed as rapidly as possible to support the piling. Excessive length of DB over that required will be assumed to fix the piling in position but not to add materially to the reaction or the bending moments.
4. Find the distance x such that the minimum passive resistance $DV'B'$ will support one-half of the pressure caused by the diagram $QUE'B'$. Thus

$$\frac{QUE'B'}{2} = DV'B'$$

$$E'B' = p_a h \qquad \text{and} \qquad B'V' = p_n x$$

$$\frac{1}{2}\, p_a h(L_5 + x) = p_n \frac{x^2}{2}$$

$$p_n x^2 - p_a h x - p_a h L_5 = 0 \qquad (11\text{-}7)$$

Solve for x.

5. The resultant passive pressure P_1 will act at $x/3$ above B'. Assume this line of action to be the support point for the piling, thus determining the span $L = QW$.
6. Assume the span QW to be fixed at both ends, and compute the fixed-end moment at Q as $\frac{1}{12} p_a h L^2 - (p_n DW/60)a^3(5 - 3a)L^2$, where the latter term is explained by reference to Fig. 11-13(c).
7. Assume the reaction at Q from span QW to be $\frac{1}{2} p_a h L - p_n(DW)^3/6L$.

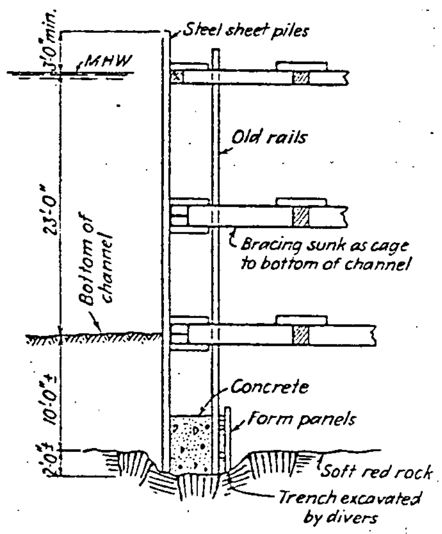

FIG. 11-15. Cofferdam with concrete seal used for Ferry St. Bridge piers over Quinnipiac River by C. W. Blakeslee and Sons, New Haven, Conn.

Figure 1: Procedure to estimate bending moment in sheetpile wall.

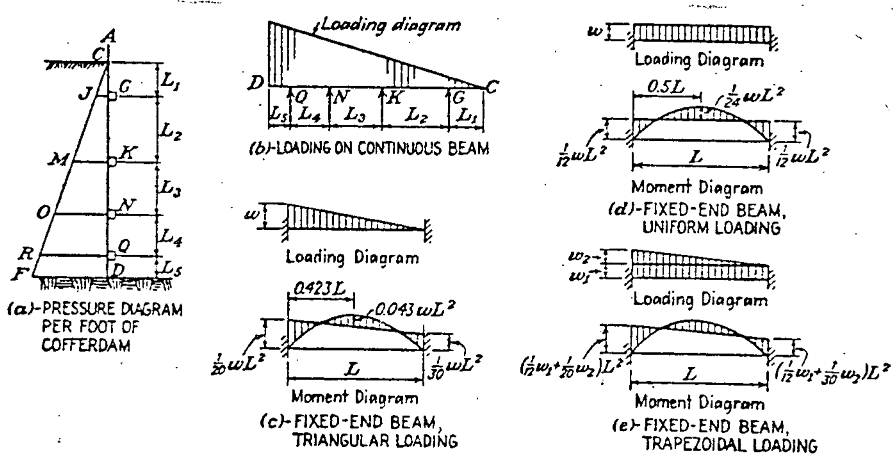

FIG. 11-12. Data for analysis of multibraced cofferdam, cantilevered at top and bottom.

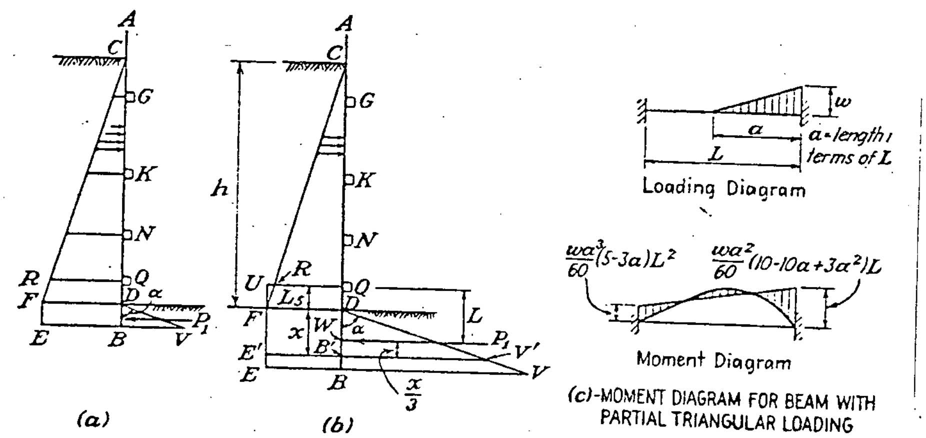

FIG. 11-13. Assumed pressure diagrams at bottom of embedded piling of multibraced cofferdam.

Figure 2: Figures used in procedure to estimate bending moment in sheetpile wall (Reference Figure 1)

AMPLIFICATION IN A UNIFORM SOIL LAYER
FOUR ARTIFICIAL EARTHQUAKES FOR WATTS BAR

Solution for $\frac{\partial^2 u}{\partial t^2} + \frac{c}{\rho}\frac{\partial u}{\partial t} - V_s^2 \frac{\partial^2 u}{\partial z^2} = -\frac{\partial^2 u_g}{\partial t^2}$