

TENNESSEE VALLEY AUTHORITY

CHATTANOOGA, TENNESSEE 37401  
400 Chestnut Street Tower II

April 27, 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

Enclosure 1 provides information concerning safety factors for sheetpile walls at Watts Bar Nuclear Plant. This information was requested informally by the NRC on March 9, 1982. Also provided, as Enclosure 2, is additional information concerning the use of sheetpile walls. This is a revised version of information provided by my letter to you dated February 1, 1982.

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, Manager  
Nuclear Licensing

Sworn to and subscribed before me  
this 27<sup>th</sup> day of April 1982

Bryant M. Lowery  
Notary Public

My Commission Expires 4/8/86

Enclosures

cc: U.S. Nuclear Regulatory Commission  
Region II  
Attn: Mr. James P. O'Reilly, Regional Administrator  
101 Marietta Street, Suite 3100  
Atlanta, Georgia 30303

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ENCLOSURE 1

WATTS BAR NUCLEAR PLANT UNITS 1 AND 2  
SHEETPILE WALL SAFETY FACTORS

Question:

What is the factor of safety of the tieback sheetpile walls?

Response:

As shown on WBN FSAR Figures 3.8.4-54 and 3.8.4-55 the tieback sheetpile walls are composed of six basic components. These components are:

1. Sheetpile
2. Wale
3. Anchor Plate at Sheetpile
4. Tie Rod Connecting Sheetpile and Deadman
5. Anchorage at Deadman
6. Deadman

The minimum factor of safety (F.S.) and ratio of allowable stress or load to applied stress or load are listed in Table 1 for Load Case III Safe Shutdown Earthquake (SSE). For steel components, the factor of safety is defined as the yield stress divided by the applied stress. For concrete, the factor of safety is defined as the ultimate load divided by the applied load.

Factors of safety are presented only for Load Case III (SSE). Load Case I (normal) and Case II (Operating Basis Earthquake (OBE) or one-half SSE) were also considered in the design as required by FSAR Table 3.8.4-10. In some instances these conditions controlled the design of different components at different elevations. However, since the required factors of safety for these loading conditions are larger, they were not included in Table 1.

The factors of safety required by WBN FSAR Table 3.8.4-10 for all load cases are listed in Table 2.

Table 1

Factors of Safety (F.S.)  
and Allowable to Applied Stress (or Load) Ratios  
for the Tieback Sheetpile Wall Components  
Under Load Case III (SSE) Condition

<u>Component</u>	<u>Required F.S. per WBN FSAR Table 3.8.4-10</u>	<u>Provided F.S.*</u>	<u>Allowable/Applied</u>
Sheetpile	1.38	1.43	28,000 ksi/27,000 ksi = 1.04
Wale	1.0	1.07	32,000 ksi/29,950 ksi = 1.07
Anchor Plate at Wale	1.0	1.09	28,800 ksi/26,520 ksi = 1.09
Tie Rod	1.0	1.15	28,800 ksi/25,000 ksi = 1.15
Anchorage at Deadman (Concrete Controls)	2.40	2.51	278 K/266K = 1.05
Deadman/ Stability	1.0	1.12	1419K/1262K = 1.12
Deadman/ Reinforced Concrete (Steel Controls)	1.11	1.6	54,000 ksi/37,420 ksi = 1.44

\*For steel, F.S. = Yield Stress/Applied Stress

For concrete, F.S. = Ultimate Load/Applied Load

For soil, F.S. = Ultimate Load/Applied Load

Factors of Safety (F.S.) Required by  
WBN FSAR Table 3.8.4-10

Load Case*	Factors of Safety for Reinforced Concrete Components		Factors of Safety for Steel Components	
	Concrete Ultimate/Allowable	Reinf. Steel Yield/Allowable	ASTM A36 (Shapes, Plates and Rods) Yield/Allowable	ASTM A328 (Sheetpiles) Yield/Allowable
I (Normal Condition)	2.22**	2.5	1.67 (Tension Rods) 1.52 (Compact Shapes) 1.33 (Plates)	2.14
II (OBE Condition)	2.22**	2.0	Same as Case I	2.14
III (SSE Condition)	1.33**	1.11	1.0	1.38

\*See WBN FSAR Table 3.8.4-10 for Load Case Description

\*\*Bending

Question:

What is the factor of safety against sliding for the earthfill contained by the sheet pile walls during a safe shutdown earthquake?

Response:

The factor of safety against sliding for the earthfill contained by the sheetpile walls during a safe shutdown earthquake is 1.3.

ENCLOSURE 2

WATTS BAR NUCLEAR PLANT UNITS 1 AND 2  
SHEETPILE 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 sheetpile walls were designed for the three loading cases and allowable stresses of table 3.8.4-10 of the FSAR. These components include the sheetpiles, 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 sheetpile 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 sheetpile 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 sheetpile 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 1.10 under a safe shutdown earthquake.

The earthfill contained by the sheetpile 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  $16^\circ$  for the sheetpiles. 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 or 355 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 Structures," 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 sheetpiling on the other. The wale shall be on the inside of the wall and each sheetpile 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 and 355 psf per foot of depth for Load Case III

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  $\alpha$  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

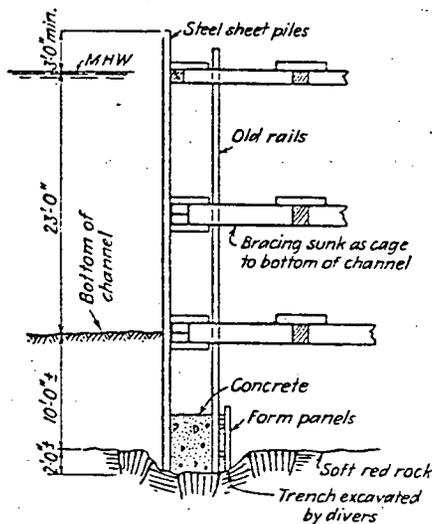


FIG. 11-15. Cofferdam with concrete seal used for Ferry St. Bridge piers over Quinnipiac River by C. W. Blakeslee and Sons, New Haven, Conn.

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

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

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

$$p_n x^2 - p_a h x - p_a h L_s = 0 \quad (11-7)$$

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  $\frac{1}{2} p_a h L - p_n (DW)^2/6L$ .

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

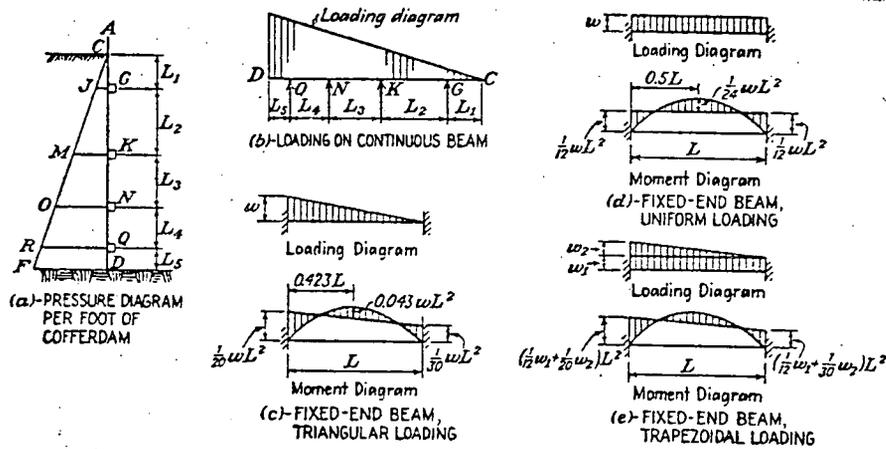


FIG. 11-12. Data for analysis of multibraced cofferdam, cantilevered at top and bottom.

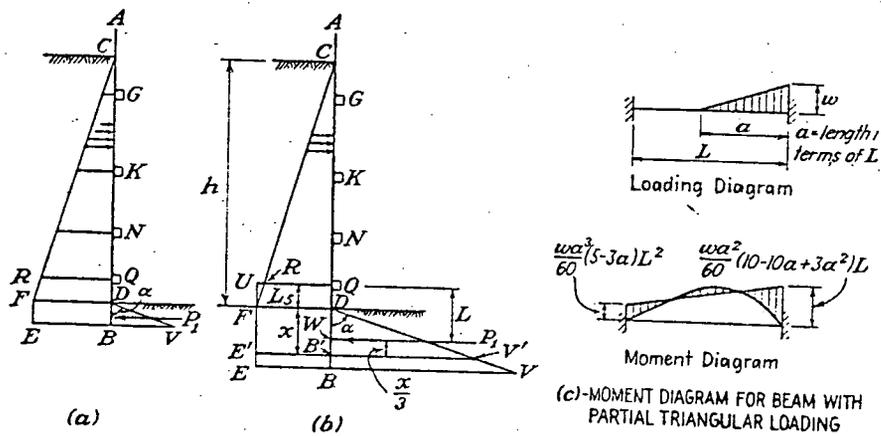


FIG. 11-13. Assumed pressure diagrams at bottom of embedded piling of multibraced cofferdam.

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  $\frac{\partial^2 u}{\partial t^2} + \frac{c}{\rho} \frac{\partial u}{\partial t} - v_s^2 \frac{\partial^2 u}{\partial x^2} = - \frac{\partial^2 y}{\partial t^2}$

