TENNESSEE VALLEY AUTHORITY

CHATTANOOGA, TENNESSEE 37401
400 Chestnut Street Tower II

July 21, 1983

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

Dear Ms. Adensam:

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

Please refer to L. M. Mills' letters to you dated November 30, 1982 and March 25, 1983, which transmitted information requested by NRC as a result of a geotechnical audit conducted September 22-24, 1982.

Your letter dated June 1, 1983 requested TVA to provide responses to specific questions concerning the analysis of sheetpile walls and seismic analysis of buried pipes. Enclosures 1 and 2 to this letter provide our responses to these questions.

If you have any questions concerning this matter, please get in touch with D. B. Ellis at FTS 858-2681.

Very truly yours,

TENNESSEE VALLEY AUTHORITY

D S Kammer

D. S. Kammer
Nuclear Engineer

Sworn to and subscribed before me
this 21st day of July 1983

Bryant M. Lowery
Notary Public
My Commission Expires 4/8/86

Enclosures (2)
cc: U.S. Nuclear Regulatory Commission (Enclosures)
Region II
Attn: Mr. James P. O'Reilly, Regional Administrator
101 Marietta Street, NW, Suite 2900
Atlanta, Georgia 30303

B001
1/1

ENCLOSURE 1
ANALYSIS OF SHEETPILE WALLS

Question:

1. Identify the load cases and load combinations for which passive pressure with submerged conditions were used in the analysis.

2. Provide and justify the soil properties and other analysis parameters used in the additional investigation.

3. What are the factors of safety resulting from the analysis?

Response:

The load cases and load combinations using passive pressure with submerged conditions are listed in Table 1. The table also includes the soil properties and other parameters used in the analyses. The factor of safety for each case at the controlling section are shown on the right side of the table. Figure 1 shows a diagram of the controlling section, using submerged earthfill for the passive pressure.

The soil properties used in the analysis were based on a TVA design standard that was in effect when the wall was first analyzed. The design standard had been used for many years at both hydro and thermal projects with satisfactory results. The design standard was based on Coulomb's equation for calculating earth pressures. The angle of internal friction ($\emptyset = 32^{\circ}$) used in the standard was based on experience of what was appropriate for the soils in the Tennessee Valley. The effect of seismic forces were accounted for by using the TVA "Shaking Table" experiments.

TABLE 1

SHEETPILE WALL ANALYSIS
USING
PASSIVE PRESSURE UNDER SUBMERGED CONDITIONS

LOAD CASE	LOAD COMBINATION	SOIL WEIGHT γ_m (pcf)	SOIL WEIGHT γ_{sub} (pcf)	EARTH PRESSURE COEFFICIENTS K_a	EARTH PRESSURE COEFFICIENTS K_p	ANGLE OF INTERNAL FRICTION ϕ	SAFETY FACTOR
I	P + S	120	65	0.307	3.25	32^{o}	1.68
IA	P + S + W	120	65	0.307	3.25	32^{o}	1.00
III	P + S + E'	120	65	0.307	3.25	32^{o}	1.23

P - Earth Pressure

S - Surcharge (200 psf)

W - Hydro Static Load - water table at elev 700 for active pressure
- water table at grade (elev 684 for controlling case) for passive pressure

E' - Safe shutdown earthquake (0.18g at top of rock and 0.68g at elev 710)
Water table at grade (elev 684 for controlling case) for passive pressure

NOTE: Bedrock is nominally found between elevations 660 to 665

Watts Bar Nuclear Plant
Sheet Pile Retaining Wall
Controlling Section for Design

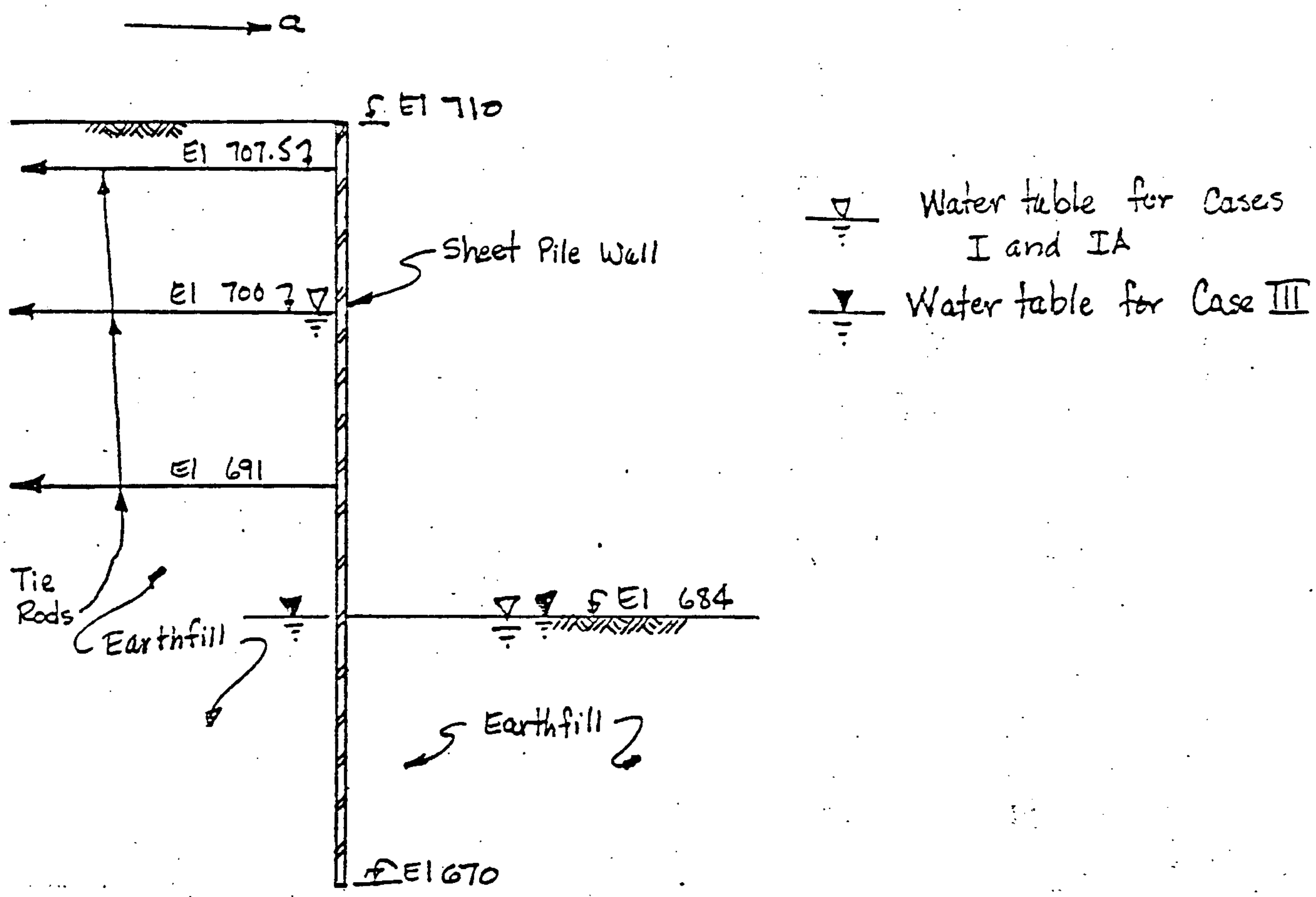


Figure 1

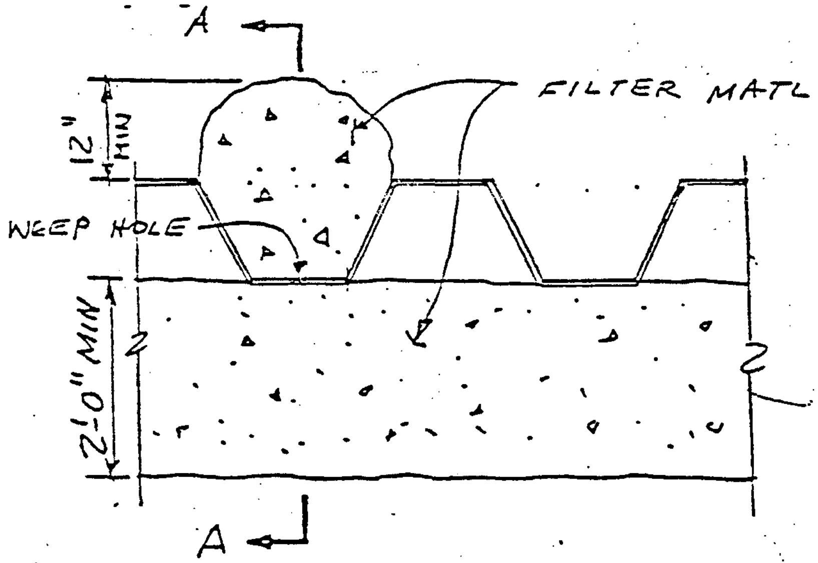
Question:

4. Provide a discussion with sketches illustrating the technique being used to correct the locations of the weep holes. How will you ensure the continued function of the weep holes in the future?

Response:

As shown on the attached sketch of a plan and section of the correction needed to allow the weep holes to drain, the earthfill covering the weep holes on the outside of the wall was removed to a depth (9 inches minimum) below the weep holes and replaced with a free draining granular material.

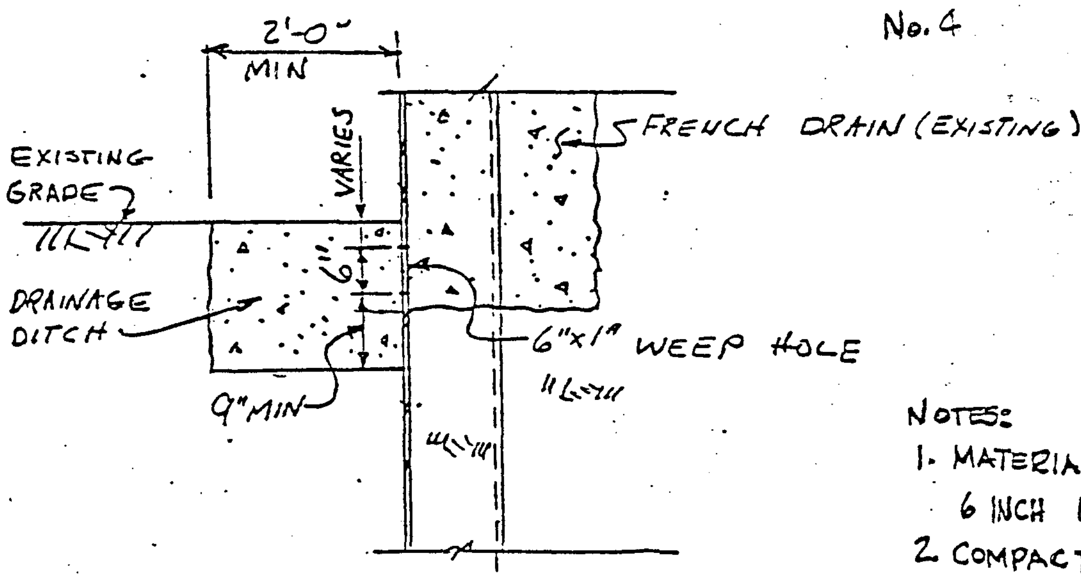
An adequate quantity of the drain material was placed along the sheetpile wall to ensure free drainage. In addition, the analysis tabulated for question 1 above shows that the wall is stable for the case where the backfill inside the wall is saturated to elevation 700.



PLAN

FILTER MATERIAL GRADATION

SIEVE SIZE	PERCENT PASSING BY WEIGHT
1½ INCHES	100
3/4 INCH	30-75
3/8 INCH	5-15
No. 4	0-5



A - A

NOTES:
1. MATERIAL PLACED IN 6 INCH LIFTS
2. COMPACTED WITH SMALL ROLLERS OR POWER TAMPERS MAKING 5 OR 6 PASSES.

SEISMIC ANALYSIS OF BURIED PIPES

Question 1:

What is the basis for the values of soil properties used in the calculations?

Response :

The soil properties used in the calculations for the buried piping were based on field and labortory testing. The soil properties used in the analysis are shown in FSAR Table 2.5-12 and 2.5-17. FSAR Table 2.5-12 contains the results of an evaluation of the static soil test results along the pipeline and the conduit alignment and an onsite borrow area. These results were developed from the soil test data presented in FSAR Tables 2.5-10 and -11 for the IE conduit alignment, 2.5-24 for the ERCW pipeline alignment, and 2.5-25 for the onsite borrow area. The test data are shown in graphical form on Figures 2.5-206 through -208 for the IE conduit alignment, 2.5-241 through -243 for the ERCW pipeline alignment, and 2.5-244 through -246 for the onsite borrow area. Tables 2.5-17 contains the results of geophysical testing along the pipeline and conduit alignment. The results from the geophysical testing were averaged to obtain a design shear wave velocity for use in the piping analysis. The average shear wave velocity was 1266 fps, but to allow for variation in the soil properties, the shear wave velocity was varied by $\pm$ 30 percent. For the resuls of our analysis, the upper variation (1266 + 30 percent) of the shear wave velocity controlled the analysis of the piping.

Soil properties were used in the following equations in the analysis:

Mean earth pressure on pipe -

$$\overline{\sigma}_m = \sigma_v \left(\frac{1 + 2 K_o}{3} \right)$$

$\overline{\sigma}_m$ = Mean earth pressure on pipe

σ_v = Major principal stress, generally overburden pressure

K_o = At rest lateral earth pressure coefficient ($K_o = 0.8$)

Pipe friction -

$$f = \pi D [c + \overline{\sigma}_m \tan \phi] f_r$$

f = Friction force along pipe axis per unit length

D = Pipe diameter

f_r = Coefficient of interface friction

C = Cohesion of the soil

$\overline{\sigma}_m$ = Mean earth pressure on pipe

ϕ = Angle of internal friction of the soil

Modulus of subgrade reaction -

$$K_s = \frac{0.65}{D}\left[\frac{E_s D^4}{E_p I_p}\right]^{1/12}\left[\frac{E_s}{1-\mu^2}\right] \quad \text{(Vesic's equation)}$$

$$E_s = 2(1+m)G_s$$

$$G_s = \rho V_s^2$$

$$K = K_s D$$

$$\lambda = \sqrt[4]{\frac{K}{4E_p I_p}} \quad \text{(Hetenyi)}$$

D = Pipe diameter

K_s = Modulus of subgrade reaction

K = Soil spring constant per unit length

E_p = Young's modulus for the pipe

E_s = Young's modulus for the soil

I_p = Moment of inertia for the pipe

μ = Poisson's ratio

G_s = Shear modulus of the soil

V_s = Shear wave velocity

ρ = Soil mass density

λ = Characteristic of the system

Question 2:

The submittal states that the soil would have localized failure before it could resist the magnitude of the calculated pressure. Define and justify the approach used to solve this concern and identify the specific locations and mechanisms of the failure zones.

Response:

TVA's opinion that a localized soil failure rather than a pipe failure would occur is based on the judgment that the soil cannot deliver the load theoretically being imposed on the pipe without deformation of the soil occurring. Our approach in making this judgment was to make a simplified evaluation of the dynamic bearing capacity of the soil for each pipe diameter being considered. The simplified evaluation was made using Terzaghi's equation assuming a strip footing and ignoring the effect of the circular shape of the pipe. The value of the dynamic bearing capacity was then compared with the pressure being exerted by the soil on the pipe based on the piping analysis. The ratio of pressure being exerted by the soil on the pipe to the dynamic soil bearing capacity varied from a ratio of approximately 6:1 for the 24-inch-diameter pipe to 25:1 for the 8-inch-diameter pipe. These ratios were high enough that in our judgement the soil will development some type of failure zone allowing the soil to deflect or deform rather than overstressing the pipe. The locations for this type of failure zone would be at the junction of the smaller diameter pipe to the larger diameter pipe.

Question 3:

Provide the input seismic datausedin the analysis and the corresponding soil strains.

Response:

The seismic input used in the calculations was based on 0.18g acceleration at the top of rock. This seismic input as amplified through the soil using the factor given in section 2.2.5 of the design criteria WB-DC-40-31.5 and was used to calculate the bending, shear, and displacement that would be experienced by the piping system. This design criteria was provided in response to audit item No. 5 submitted by letter to E. Adensam dated November 30, 1982. The seismic input of 0.22g was also used to determine the axial force in the pipe, but it was not the controlling factor inthe analysis. The axial force in the pipe is limited by the friction force that could be transmitted to the pipe by the soil. This friction force is used in conjunction with the fundamental period of the soil profile and the shear wave velocity of the soil to determine the maximum axial force experienced by the pipe. The dynamic shear wave velocity for the soil was taken as 1266 fps. This was the average value of the results of the geophysical soil measures performed along the ERCW pipeline as reported in Table 2.5-17 of the FSAR. The shear wave velocities were varied by ± 30 percent in the piping analysis.

The following equations show the seismic input used in the analysis of buried piping:

1. Equations used to calcualte bending moment, shear, and displacement in the pipe.

$$A = \frac{(T\ )^2}{(2\ )} a$$

A = Amplitude of the displacement of the pipe

a = Acceleration of the soil

T = Period of the soil deposit

Equation used to calculate stress caused by axial movement. Two methods for calculation soil strain -

1. By maximum slippage length

$$\varepsilon_m = \frac{F_{max}}{AE} = \frac{f\, l_m}{AE}$$

ε_m = Maximum axial strain

F_{max} = Maximum axial force = $f l_m$

l_m = Maximum slippage length = $\frac{L}{4}$ = $V_s T$

L = Wave length

H = Soil deposit height

f = Friction force along pipe axis per unit length

2. By particle velocity method

- may be too conservative because it assumes no soil slippage between soil and pipe.

$$\varepsilon_m = \frac{V_p}{C_p} \quad \text{or} \quad \varepsilon_m = \frac{V_p}{2V_s}$$

Where V_p = 48 in/sec (a_m)*; a_m in terms of g.

a_m = Maximum ground acceleration

V_p = Maximum particle velocity of the soil

C_p = Maximum compression wave velocity

V_s = Maximum shear wave velocity

Based on site conditions (pipe diameter, depth of cover, profile depth, etc.) at various locations along the pipe, the pipe maximum pipe strain varies from 1.98 x 10^{-4} in/in to 5.30 x 10^{-4} in/in. The soil strain values are based on an input seismic acceleration of 0.22g and shear wave velocity varied from 886 ft per seconds to 1646 ft per second (1266 ft per second $\pm$ 30 percent) varied from 5.37 x 10^{-4} in/in to 10.00 x 10^{-4} in/in.

$$D.A.F. = \frac{\gamma_B V_{SB}}{\gamma_T V_{ST}}$$

DAF = Dynamic amplification factor for the soil layer

γ_B = Density of the base rock

γ_T = Average density of the soil layer

V_{SB} = Shear move velocity in the base rock

V_{ST} = Shear wave velocity in the soil layer

$$M = \pi \frac{EIA(DAF)}{(L/2)^2}$$

$$L = V_s \ T$$

M - Maximum bending moment

E = Young's modulus for the pipe

I = Moment of inertia for the pipe

L = Wave length

V_s = Shear wave velocity of soil

Question 4:

What are the intensification factors used in the analysis?

Response:

The intensification factors used in the analysis were based on the ASME code, section III.

The following intensification factors were used in the analysis of the elbows and tee sections:

Elbow Pipe Diameter	i (intensification factor)
36"	5.64
30"	4.98
16"	3.22
14"	2.93
8"	2.44

Tee Junctions Pipe Diameter	i (intensification factor)
30"-24" tee junction	15.3
30"-14" tee junction	11.5
30"- 8" tee junction	7.6

The above intensification factors for tee-intersections are conservative. Intensification factors based on the ASME code are higher than factors recommended for tee-intersections by manufacturers literature. A factor of 4.71 is recommended for tee-intersections with a 30-inch-diameter pipe. This lower factor was recommended in Bulletin 789 entitled "Bonney Forge, Weldolet, Stress Intensification Factor."

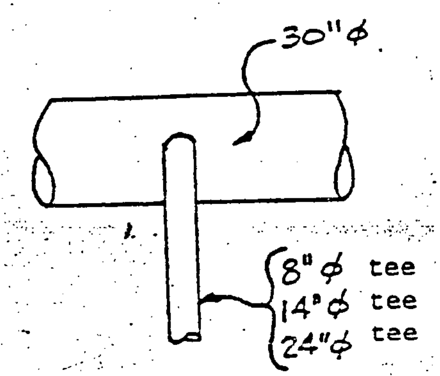
Question 5:

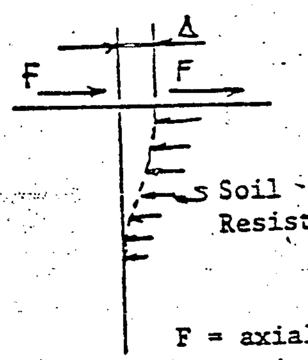
Provide the details of an analysis which demonstrates that failure of the small diameter pipes will not occur under seismic loadings. Include assumptions, failure criteria, calculation procedures and results.

Response:

A failure analysis was not made for the tee-junctions which were indicated in the analysis to be overstressed. The analysis that was made was done to calculate the stresses that would occur in the pipe and to compare those results with the allowables as determined by the ASME Code, Section III for steel pipe. When it was determined that the aforementioned tee-intersections were overstressed, the input parameters, assumptions and other analysis as well as the results were reviewed. Based on that review, as listed in response to audit action item No. 4 by letter to E. Adensam dated November 30, 1982, it was judged that the pipe would not fail. This engineering judgement was based primarily on the very small movements that were calculated to occur, but the other listed factors contributed to the judgement that the pipe would not fail.

The method used in the analysis of the tee-junctions was developed by Shah and Chu (1974)[1]. The worst case for the analysis was found when axial soil strain caused by the earthquake results in a differential movement in the small diameter pipe that is connected to a larger diameter pipe by means of a tee-junction.





F = axial force due to seismic event

Δ = movement caused by seismic event

[1] Shah, H. H. and Chu, S.L., "Seismic Analysis of Underground Structural Elements," ASCE Journal of the Power Division, Vol 100, No. PO1, July, '74, pp. 53-62.

The analyses were performed using the Shah and Chu (1974) procedure in conjunction with the applicable soil and pipe properties, and axial strain on the pipe due to the seismic event. The moments from the above analyses were used in the appropriate ASME equations and the resultant stresses were compared with the allowable stresses. The attached table provides the pertinent input data to the analysis and the results for the various tee-intersections.

SOIL PROPERTIES	30" Main Line 8" Branch	30" Main Line 14" Branch	30" Main Line 24" Branch
Unit Weight (γ) pcf	120	120	120
Cohesion (c) psf	2100	2100	2100
Angle of Friction (ϕ)	6^o	6^o	6^o
At Rest Pressure Coef (K_o)	0.8	0.8	0.8

SOIL PROFILE			
Shear Wave Velocity (V_s) fps	1650	1650	1650
Poisson's Ratio (μ)	0.4	0.4	0.4
Depth of Profile (H) ft	41.7	33.0	39.5
Depth of Pipe (h) ft	9.9	8.7	6.5

PIPE PROPERTIES	8"	14"	24"	30"
Outside Diameter (D) in.	8.625	14.0	24.0	30.0
Wall Thickness (t) in.	0.322	0.375	0.375	0.375
Young's Modulus (E_p) psi	$28x10^6$	$28x10^6$	$28x10^6$	$28x10^6$
Moment of Inertia (I_p) in^4	72.5	372.8	1943	3829
Pipe to Soil Friction Ratio (f_r)	0.5	0.5	0.5	0.5
Cross-Sectional Area (A) in^2	8.40	16.05	27.83	34.90
Section Modulus (Z) in^3	16.81	53.2	161.9	255.3

ANALYSIS RESULTS	30" Main Line 8" Branch	30" Main Line 14" Branch	30" Main Line 24" Branch
Young's Modulus of Soil (E_s) psi	98,000	196,000	196,000
Maximum Strain (E_m) in/in	.0003606	.000285	.000238
Shear (S) lb.	86,700	108,800	190,000
Moment (M) 16-in	$7.95x10^5$	$1.27x10^6$	$2,85x10^6$
Deflection (D) in.	0.070	0.035	0.040
ASME Intensification Factor (i)	7.6	11.5	15.3
Bending Stress Calculated (σ_b) psi	270,000	205,000	202,000
Stress-Allowable (σ_A) psi	42,000	42,000	42,000