TENNESSEE VALLEY AUTHORITY
CHATTANOOGA, TENNESSEE 37401
5N 157B Lookout Place

AUG 18 1986

Director of Nuclear Reactor Regulation
Attention: Mr. B. J. Youngblood, Project Director
PWR Project Directorate No. 4
Division of Pressurized Water Reactor (PWR)
Licensing A
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Dear Mr. Youngblood:

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

Submitted herewith are 60 copies of Amendment 59 to the Watts Bar Nuclear Plant Final Safety Analysis Report (FSAR).

Please note that the material in this amendment related to rod withdrawal reanalysis (FSAR Chapter 15) represents significant changes to material which was previously reviewed and discussed in the Watts Bar Safety Evaluation Report (SER) and may require further review. The rod withdrawal reanalysis was necessary to allow plant operation in mode 3 with the operation of two reactor coolant pumps. The conclusions reached in the SER are not expected to be affected with these changes.

An instruction sheet for incorporating this amendment into the FSAR is included with each copy.

If there are any questions, please get in touch with James Young at (615) 365-8837.

Very truly yours,

TENNESSEE VALLEY AUTHORITY

R Gridley

R. Gridley, Director
Nuclear Safety and Licensing

Sworn to and subscribed before me
this 18th day of August 1986

Susan C Parker
Notary Public
My Commission Expires 2/7/90

Enclosure (60)
cc: See page 2

B001
3/60

Director of Nuclear Reactor Regulation

AUG 18 1986

cc: U.S. Nuclear Regulatory Commission (Enclosure)
Region II
Attention: Dr. J. Nelson Grace, Regional Administrator
101 Marietta Street, NW, Suite 2900
Atlanta, Georgia 30323

2.5.3.8 Results of Faulting Investigations

Details of regional, site, and foundation faulting, dating techniques used, and results, are provided in Sections 2.5.3.2 and 2.5.3.4.

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2.5.4 Stability of Subsurface Materials

2.5.4.1 Geologic Features

The Conasauga formation of the Middle Cambrian age is the principal foundation rock found at the site. This formation is discussed in Sections 2.5.1.1.6, 2.5.1.2.7, and 2.5.1.2.9.

2.5.4.2 Properties of Subsurface Materials

2.5.4.2.1 In Situ Soils

2.5.4.2.1.1 General Description

The unconsolidated deposits overlying bedrock are composed primarily of alluvial deposits on the elevated flood plain near the lake shore and terrace materials, deposited by the Tennessee River when flowing at a higher level, over the bench that covers most of the site area. The alluvium is composed of fine-grained, finely sorted silts and clays, with micaceous sand and some quartz gravel. The thickness of the unit varies, but drilling showed an average thickness of approximately 25 feet. Near the base of the terrace bench the alluvial deposits thin out to a feather edge. Included in the alluvial material are some fairly well defined beds of tough, blue-gray clay, containing carbonized fragments of wood. These are interpreted as old slough fillings. 50

The terrace deposits are much older than the recent flood plain deposits and their edge is marked by a distinct topographic bench some 30 feet high which lies from 200 to 1000 feet northwest of the edge of Chickamauga Lake. Recent drillings show the thickness of the terrace deposits to vary from a minimum of 31 feet to a maximum of 46 feet. The average thickness is 40 feet.

Approximately the upper half of the unit is composed of sandy, silty clay and the lower half is much coarser, consisting of pebbles, cobbles, and small boulders of quartz or quartzitic sandstone embedded in a sandy clay matrix.

In contrast to the conditions at the Sequoyah site, very little residual material derived from weathering of the underlying shale is present under the terrace deposits at the Watts Bar site. In a few holes a foot or two of residual clay was encountered, but in most instances the terrace deposits are immediately underlain by a few feet of soft but unweathered shale.

2.5.4.2.1.2 Investigations

Soil investigations were conducted at the site for the major features. Figure 2.5-185 and 2.5-185a shows the locations of borings for the soil investigations for the various features at the site. 50 32 50

The field exploration for in situ soils consisted of split-spoon borings, using standard penetration test procedures for all features, and borings for undisturbed sampling for most features, auger borings to determine the top of rock, and test pits to obtain undisturbed samples. Although most borings were made using dry procedures, some borings, specifically for the liquefaction study, were made using drilling mud with a fishtail bit to advance the boring. 50

The split-spoon borings were made at the plant site using the methods specified in ASTM D 1586. The purpose of these borings was to obtain disturbed soil samples for laboratory testing and to determine the standard penetration resistance, N, of the in situ soils. Disturbed samples from the split-spoon borings were sealed in glass jars after removal from the soil samples and taken to TVA's Materials Testing Laboratory for tests.

Undisturbed soil sample borings were made to obtain undisturbed samples for laboratory testing. The soil samples were obtained using various types of samplers. The ends of each tube were sealed immediately after removal from the boring to preserve the natural moisture content of the sample. 50

The types of borings made for any feature were based on the design requirements for each feature. In addition to the borings made for SPT samples and undisturbed samples, a number of locations were tested using a cone penetrometer. The results of this testing are described in reference 167.

Field Investigation Techniques Along the ERCW Pipeline and 1E Conduit Alignments 50

As a result of the NRC's interest in the techniques used in the field for the investigations of the soils along the ERCW pipeline and 1E conduit alignments, the following specific information is furnished.

The initial field investigation was completed between July 24 and August 19, 1979 with two Mobile model 3-50 drills. The standard penetration test (SPT) borings were advanced by dry methods using 3-3/8 inch inside diameter (id) hollow stem augers. Standard 2 inch split-barrel samplers complying with specification ASTM D

1586 and equipped with light duty spring retainers were used for sampling. The string of tools was exclusively AW drill rods. Tables 2.5-28 and -29 provide information on the weight of the drill rod for each split-spoon sample. Safety-type 140-lb drive hammers were used. One wrap of rope was used on the cathead. Blow counts were recorded for each 0.5' interval driven and sample recovery recorded. Drilling and sampling were in accordance with ASTM D 1586 procedures. Sample descriptions were recorded on both the drilling log and sample tags. Samples were immediately sealed in glass pint jars and temporarily stored in an onsite building to avoid extreme temperatures.

The undisturbed sampling borings were also advanced by dry methods, but using 6 inch id hollow stem augers. Samples were taken with 5 inch diameter thin-walled tubes attached to a piston-type sampler conforming to specifications in ASTM D 1587. Samples were sealed on both ends with at least 1 inch of beeswax-paraffin sealing wax. Depths of sample recovery were recorded on drill logs and sample tags. Samples were transported on rubber-padded racks for temproary storage to an onsite building to avoid extreme temperatures. A covered vehicle with rubber-padded racks was used to transport the samples from temporary storage to TVA's Singleton Materials Engineering Laboratory. Certified soils technicians performed all handling, moving, and transportation of specimens. 50

A subsequent field exploration was completed between May 30 and July 3, 1979. Equipment used was a CME-55 drill and a Mobile B-50 drill. The methods and sampling equipment used on the SPT borings exactly match those described above for the report of March 17, 1976. Tables 2.5-28 and -30 provide information about the drilling equipment used for each boring.

Rotary drilling methods were used between sampling elevations in the undisturbed sample borings. Bentonite drilling fluid was used. The 5-1/2 inch wide drag bit was equipped with baffles which deflected the drilling fluid upward. Samples were obtained with 5 inch diameter thin-walled tubes attached to a piston sampler.

Samples were sealed on both ends with a beeswax-paraffin mixture and temporarily stored onsite to protect them from extreme temperatures. They were transported to the laboratory on rubber-padded racks in a vehicle driven by a soils technician.

No engineering testing was required on these samples. However, following standard practice, the tube samples were extracted and unit weights and general classification tests conducted and recorded.

Additional SPT borings were completed between November 4 and 24, 1981. All borings were drilled with a Mobile B-61 drill. Procedures followed the recommendations in Table 2.5-31. Tables 2.5-32 and -33 provide information about the drill rig and equipment used for each boring.

On all Watts Bar Nuclear Plant ERCW assignments, one drill operator was assigned to, and stayed with, a specific drill. Exceptions would normally occur only in case of illness or other personal emergencies. Such situations are not documented.

Ropes used in drilling standard penetration test borings are normally replaced when noticeably worn on the initiative of either the driller or inspector. There are no specific guidelines or documentation. During the 1975 and 1979 investigations, it is judged that the ropes were used and somewhat limp. During the 1981 investigations, the ropes were new and stiff in accordance with specific instructions. 50

During all investigations, a 140-lb Mobile safety-type drive hammer, model 006981, was used.

Test pits were excavated by a Gradall excavator equipped with a 3-yd^3 smooth bucket. Side walls were excavated to about a 1 to 1 slope. Dewatering was facilitated by installing a section of perforated 18 inch diameter pipe surrounded by a $\pm$ 3/4 inch crushed stone filter. Undisturbed samples were obtained by benching into the side wall and hand trimming 1-ft^3 blocks with handtools. The trimmed top and sides were covered with three alternating layers of cheesecloth and paraffin. The sample was then cut at the bottom which was covered in a similar manner. Samples were placed in a wooden box surrounded with damp sawdust padding. A soil technician immediately transported the blocks on styrofoam pads to the laboratory.

The following laboratory tests were made on all split-spoon samples.

1. Moisture content

2. Atterberg limits (ASTM D 423 and D 424)

3. Grain size tests (ASTM D 422)

4. Classification (ASTM D 2487)

For features where undisturbed borings were located on basis of information obtained from the split-spoon borings, the results obtained from these tests were used in the assessment of the existing soil characteristics. In order to assure continuity between the split-spoon borings and the companion undisturbed

penetration data given in Figure 2.5-190. The results of the laboratory testing are summarized in Table 2.5-6.

In order to assure a safe foundation for the building, the fine grained soils above the in situ gravel were removed and replaced with granular fill as illustrated in Figure 2.5-226. The criteria for the granular fill is discussed in Section 2.5.4.5.2. 27

Intake Pumping Station and Channel (Category I Feature) 27

The intake channel is a man-made feature extending approximately 800 feet from the edge of the reservoir through the flood plain to the intake pumping station. The bottom of the channel is elevation 660, and is 50 feet wide. Channel earth side slopes are one vertical on four horizontal. The nominal ground surface is elevation 695. Groundwater is near elevation 685. The location of the channel with respect to the plant layout is shown in Figure 2.1-5. The channel is illustrated in Figure 2.4-99. 27

The layout of holes is as shown on Figure 2.5-185 and 2.5-185a. Three lines of borings at about 200 feet spacing (borings 30-34, 35-39, and 41-45) were laid out parallel to the intake channel with additional borings near the river bank (40 and 46) and in the slough on the flood plain (47 and 48). Boring 29 was drilled during early site investigation. The layout included 19 (30-48) standard-penetration split-spoon borings with sufficient undisturbed borings (adjacent to split-spoon borings) to sample all types of soils in the profile. 50

Initial alternate split-spoon borings were taken to the top of rock to obtain general rock elevations and to confirm the presence of firm gravel in the lower part of the soil profile as was indicated in the earlier site exploration. Successive split-spoon borings were made into the firm gravel between these initial borings in order to confirm the general uniformity of soils above the gravel. Borings 33, 36, 38, and 44 were not made since the uniformity of the profile was disclosed by the other borings. Graphic logs of all borings are shown on Figures 2.5-191 through 2.5-195.

The graphic logs reveal firm silty and sandy gravel under the entire intake channel area, below about elevation 665 across the flood plain, and below elevation 675 at the intake structure. Above the gravel are lean clays, silts of low plasticity, and silty sands.

Index tests for soil classification, moisture, mechanical analysis, and Atterberg limits on the split-spoon samples were used to select locations for undisturbed borings for sampling all types of soils in the profile. Since the split-spoon borings confirmed gravel in the lower part of the soil profile, the undisturbed samples were taken only in soils above the gravel. Five

continuous undisturbed borings were made beside split-spoon borings at these locations. Sampling and testing of the basal gravel is described below.

Laboratory tests on undisturbed samples are recorded in Tables 2.5-7 through 2.5-9. Included in these tables is standard-penetration data in the split-spoon boring adjacent to each undisturbed boring. On both the logs and the tables, samples are identified by capital letters and lowercase letters.

The identification system of using uppercase and lowercase letters can be described as follows. Capital letters identify the representative undisturbed samples that are subjected to strength tests. Lowercase letters identify other undisturbed samples and disturbed samples considered to be the same soil as a corresponding capital letter sample. Letter designations on graphic logs and data tabulations permit easier reading of the soil profile and provide a record of the adequacy of the selective sampling and testing. The letter designations are completely arbitrary and apply only to the one project, or even to a single project feature.

The process for selecting test samples is described as follows. Split-spoon standard penetration borings are made to explore the area. The disturbed samples are examined for index properties. This data, with penetration records, is used to determine soil types distribution in the profile in order to select specific locations at which to obtain inclusive representative undisturbed samples for strength testing. Letters are assigned to the indicated separate soil types. The undisturbed samples are taken and tested also for index properties, and density and void ratio. Letter designations are then finalized, with some changes in previous designations, and with possible variations in some 'same' samples' properties, because of judgment designation based on all properties. Representative undisturbed samples are given capital letter identifications and are tested for strength. 27

The silty sand is deposited on top of the firm gravel from approximate elevation 665 to 680, and the lean clay (or silt) from approximate elevation 680 to 695. Strength properties of these soils in-situ were obtained from the test results shown on Tables 2.5-7 through 2.5-9. The results of the shear tests are plotted in graphical form (Figures 2.5-247 through 2.5-250) and a value of C and O was selected for design. 27

The soils exploration disclosed a possible weak layer of lean clay soil at approximate elevation 690 to 685 in borings US-30 and US-36, which are on opposite sides of the channel near the reservoir. The test results indicate the minimum strength properties of this material as 0 = 3 and c = 500 psf.

Cohesive soil samples were tested for sensitivity. Of the many samples, the four with sensitivity greater than 2 were remolded to in-situ density and moisture content and had unconsolidated-undrained (Q) shear tests run. The results are shown in Tables 2.5-8 and 2.5-9. The tests do not indicate serious strength loss. 27

The liquefaction potential of the site soil deposits are discussed in Section 2.5.4.8.

The basal gravel is located on top of rock at approximate elevation 650, and extends to approximate elevation 665. A trench was made in the flood plain for access to the basal gravel for undisturbed sampling. Gravel sizes up to 6 inches and water conditions in wet weather prevented useful undisturbed sampling. Successive essentially saturated grab samples were taken to a depth of 4 feet with a reasonably tight clamshell bucket. Fines contents in the samples so obtained were about 3 percent, compared with about 5 to 10 percent in previous boring sampling. Samples were scalped to maximum 2-inch size, scooped into a 12-inch-cube direct shear box, consolidated in submerged condition under equivalent overburden pressure of 3000 psf, and sheared under submerged conditions. Figures 2.5-203, 2.5-204, and 2.5-205 show gradation and shear test results. The gravel strength used in design is 0 = 42 and c = 0. The shear test results show an 0-load intercept of 0.4 to 0.6 tsf, representing interlock of particles in the shear box. Since the magnitude of this effect in the gravel mass cannot be assured, it is ignored in the basic stability analyses.

Due to unexpected soil conditions encountered during the excavation of the intake channel, an additional investigation was made and this information is provided in Section 2.5.5.2.2. 50

<u>Class 1E Electrical Conduits Alignment (Category I Feature)</u>

The Class 1E conduits furnish electrical power and control for the pumps, valves, screens, control boards, etc., at the intake pumping station. The soils investigation for the conduit alignment was to establish the dynamic soil properites along the alignment and to provide soil strength information for any slopes that would have to be qualified if the conduits were constructed in the slopes. 50

The layout for the soils investigation is shown on Figure 2.5-273. 50
The graphic logs for borings 49 through 63 are shown on Figures 2.5-196 and 2.5-197, and for borings 171 through 177 are shown on Figures 2.5-174 through 2.5-280. The graphic logs indicate that the over-burden varies from 24 to 60 feet thick. Weathered shale is encountered at depths of 10 to 32 feet. The water table was established between El. 690 and El. 710. Figure 2.5-281 shows a profile along the IE conduit bank from the ERCW pump station to the main plant with borings spaced along the alignment. 50

The overburden consists primarily of lean clay (CL) and silt (ML and MH) with small quantities of silty and gravelly sand (SM and G-SM), and silty gravel (GM ang GP-GM). Below the top of the weathered shale the laminated shaly materials classify as sand-sized soil. The granular portion of the soils above the shale is made up of silicious and micaceous sand and subangular to rounded sandstone and cherty gravel. 32

The standard penetration test results shown on the graphic logs indicates soils of a medium to stiff consistency with blow counts usually between 10 and 30 blows per foot. In a few instances, usually near the water table, the penetration results indicate a loose or soft consistency. The liquefaction potential of the samples with a loose consistency was evaluated and is discussed in Section 2.5.4.8.

The results of the laboratory testing are summarized in Tables 2.5-10 and 2.5-11. The strength values in design are represented in Table 2.5-12. The values used for design (Table 2.5-12) are low averages for all of the strength data shown in Tables 2.5-10 and 2.5-11. The results for each type shear test are plotted in graphical form (Figures 2.5-206 through 2.5-208), and a value below the average for c and o is selected to be a conservative value to use in the design. There were no sensitive soils encountered in the investigation. The dynamic soil properties are discussed in Section 2.5.4.4.2.

ERCW Piping Alignment (Category I Feature)

The Essential Raw Cooling Water piping furnishes water for cooling the reactor during emergency condition. Additional piping along the same alignment furnishes water for extinguishing fires (High Pressure Fire Protection (HPFP) piping). The soils investigation for the piping alignment was to establish the dynamic soil properties along the alignment and to provide soil strength information for any slopes that would have to be qualified if the piping were constructed in the slopes. The results of the investigation of the dynamic soil properties are provided in Section 2.5.4.4. The location of the borings for the soils investigation is shown on Figure 2.5-185. The graphic logs for all soil borings are shown on Figure 2.5-198 through 2.5-202 and 2.5-282 through 2.5-333. The graphic logs indicate an overburden that varies from 10 to 66 feet, and averages 37 feet. Weathered shale was encountered at the surface at one boring location and at depths up to 37 feet over the remaining portion of the site. Bedrock ranges from El. 668 to El. 699 with an average elevation of 685.6. 24 50 50

Alluvial soils consist of lean to fat clay, CL and CH, lean to highly plastic silt, ML and MH, along with smaller amounts of silty and clayey sand, SM and SC, and silty and clayey gravel, GM

Except for some soft areas at the bedrock surface, both the shale and the limestone appear fresh and unweathered, and there is no evidence of solution in the limestone horizons.

In order to make an engineering analysis of the rock, and to provide a tenable means of logging the complex geology in the core, the following classification system was developed:

Rock Type	Description
0	Core Loss--Zones of core not recovered either because of grinding between harder overlying and underlying strata or because they were too soft or fragmented to be recovered by conventional hard rock drilling methods. No cavities were observed with the borehole TV apparatus. For engineering purposes, all zones of core loss are assumed to be Type 1 rock.
1	Soft Shale--Material which, when removed from the core barrel, can be easily scratched with a fingernail, or is in such small, although sound and unweathered pieces, that it resembles an agglomerate rather than solid rock. The physical character of the soft shale is generally a function of intense folding and crinkling, and is not generally attributed to weathering. In most instances the individual shale particles are bounded by slickensided surfaces.
2	Hard Shale--Shale which, when removed from the core barrel, cannot be easily scratched with a fingernail. It is recovered in relatively large discrete pieces which may break down upon exposure to discs of shale--so-called "poker chips". Slickensides are present but are not as plentiful as in the soft shale, and are generally concentrated along bedding surfaces.
3	Limestone--Pieces of core that are either entirely composed of limestone or are composed predominantly of limestone with a few thin shale stringers.

Photographs showing the general nature of the three rock types are presented in Figure 2.5-209. Watts Bar Nuclear Plant shale specimens tested in the University of Illinois Rock Mechanics Laboratory [142] demonstrate the following additional physical properties:

2.5.4.2.2.7 Behavior of Watts Bar Lock

2.5.4.2.2.7.1 Moduli Calculated from Lock Settlement

Watts Bar Dam, constructed in 1939, is situated less than 2 miles upstream from the present nuclear plant site. The left end of the dam and the lock are founded on the same material which will form the foundation for the nuclear plant. During construction, a number of settlement points were placed on various blocks within the lock, and settlements were monitored for the period between April 1940 ad June 1942. Most of the lock was placed within an original water course, and less than 5 feet of alluvium and 5 feet of rock were excavated to foundation grade. A simplified plan of the lock foundation is shown in Figure 2.5-218, and a typical settlement curve is shown in Figure 2.5-219.

With known settlements and foundation load distributions, the deformation moduli shown in Figure 2.5-218 have been calculated using Newmark's chart for vertical displacements 146 which assumes a perfectly flexible foundation. This assumption is most valid for point E, for which a large part of the settlement can be attributed to the adjacent fill. The effective modules at point E should be compared with that of a footing 100 to 200 feet in diameter at the nuclear plant. The modulus of deformation at point E, 99,000 psi, is equal to the highest modulus hole under a 200-foot-diameter footing at the nuclear plant, indicating that the assumptions made concerning the modulus of deformation for deep rock at the nuclear plant are slightly conservative.

For points A through D on the upper guard well, moduli range from 23,500 psi to 89,000 psi, with the two higher moduli on the lock side of each block. Since the blocks are high with respect to their base size, and since they are triangular in shape with the heaviest load on the lock side of the foundation, additional calculations have been made assuming them to be rigid with rotation about their centers of gravity. The resulting moduli, 43,000 and 49,000 psi, should be compared with moduli calculated for footings at the nuclear plant approximately 25 feet in diameter and founded 5 feet into shale. From Table 2.5-14, the average E for a 10-foot footing founded 10 feet into rock is 33,000 psi. Since the modulus for a larger footing would be greater, the calculated moduli at the lock and at the nuclear plant appear to agree perfectly.

The calculations based on lock settlement verify the validity of assumptions and techniques used to estimate deformation moduli at the nuclear plant, ans settlement of the lock serves as a large-scale foundation test for the nuclear plant foundation.

2.5.4.2.2.7.2 Settlement of Lock and Nuclear Plant as a Function of Time

Figure 2.5-219 shows settlement of block R-10 (point F in Figure 2.5-218) with respect to three important events: (1) completion of construction of the block, (2) flooding of the cofferdam, and (3) the start of reservoir filling. By August 1940, when the block was completed, approximately 65 percent of the total settlement had taken place. Eight months later, settlements were nearly complete. As the cofferdam was flooded, reducing the effective stress on the block foundation, settlement ceased, and one-third of the previous settlement was recovered as heave during the next six months. Between April 1941 and January 1942, the cyclic nature of the settlement curve is most likely response to the lake level behind Chickamauga Dam.

The average foundation stress at block R-10 was approximately 6.5 ksf before the cofferdam was flooded, a stress similar to those expected at the nuclear plant. However, at the lock the effective weight of removed overburden was only 0.8 ksf as opposed to 4.1 ksf at the nuclear plant. Because of this difference, it is expected that an even greater percentage of total settlement will be realized during construction of the nuclear plant.

Settlement should cease as the water table is allowed to retain its original elevation around the nuclear plant, and as shown at the lock, heave can be expected at this time as effective stresses on the foundation are reduced.

Since two periods of differential movement are expected during construction, and during raising of the water table around the plant installation of utility lines passing between buildings was delayed as long as possible during the plant construction to allow the water table to almost return to the original ground water level around the plant. In order to be conservative, utilities passing between buildings were designed to allow for a differential settlement across building lines of at least 1 inch.

2.5.4.2.2.8 Excavation Experience in the Rutledge Shale at Watts Bar Lock

Exploratory holes were drilled in the area of the Watts Bar Dam navigation lock by TVA and Corps of Engineers personnel. The following remarks [150] relate the experience during drilling and subsequent excavation.

> . . . Most of these holes penetrated dark gray sandy fissile shale with thin layers of interbedded dense gray sandstone. The core recovery was poor, leaving only the hard sandy shale and thin layer of sandstone after the soft fissile shale was washed away by the drilling operation. The cores indicated a

rather level rock formation with little weathering at the surface.

After the foundation excavation was started the true nature of the rock surface was revealed. The shale was soft and weathered for a foot or two below the surface, and below this limit the shale was consistently uniform in character although the dip of the beds varied between wide limits. The shale was somewhat harder than was at first expected and disintegrated very little on exposure to the weather. It proved practical to dig the shale with a power shovel without blasting.

At the powerhouse where blasting was required, the perimeter of the area was line drilled prior to blasting to minimize damage to adjacent rock.

Core drilling experiences at the lock have been duplicated at the nuclear plant site except that core loss decreased with depth and the 'sandstone' at the lock site has been reinterpreted to be a glauconitic limestone. It is expected that excavation conditions at the nuclear plant will duplicate those at the lock.

During excavation, it was found that weathered shale could be identified as being brown, rusty, and 'rotten,' while fresh shale was dark gray to black. The rock under the lock is described as having hundreds of small, sharp, overtured folds plunging to the northeast. It was found that the small-scale folds 'strengthened the shale against sliding, but they always made hand scaling difficult for it was impossible to follow any one bed.'

Information from the 'Final Geologic Report of the Watts Bar Project' by P. P. Fox [141] states:

To obtain a satisfactory surface on the shales under the lock, a saw-toothed surface was cut into the shale, after an attempt to scaleblocks R-10, L-2, and R-2 as a flat surface. This method proved to be much faster and better than any other tried. On the flat surfaces innumerable small, partially detached, loose, and fragile particles of shale existed in spite of all care in scaling, but by the notched method the hard beds could be exposed on the top of the benches and the softer shale left undisturbed in the nearly vertical faces.

TVA Technical Report No. 9 describes the excavation operation as follows:

. . . Power shovels excavated to within 6 inches of the neat line and grade for the lock walls. The final excavation was performed by hand picks and pneumatic tools approximately 24 hours before the placing of concrete. The final scaling left the bedrock stepped or with a saw-tooth relief, the more horizontal areas cut to a plant along the more durable

sandstone strata. Experience proved that the rock did not disintegrate as rapidly as expected and that final scaling of only 3 inches from walls and floors, where no truck traffic was expected, would be sufficient . . .

Several specimens of shale from the clear plant site were tested with a newly developed slaking durability test in the University of Illinois Rock Mechanics Laboratory. [142] The test is based on the percent of an oven-dry specimen retained in a 2-mm mesh drum after 10 minutes of rotation. This value is compared with the plasticity index of the shale. The test is sensitive to the shale's ability to withstand stress relief and cyclic wetting and drying. Since the shales at Watts Bar are nonplastic, cyclic wetting and drying is not of chief concern because it does not cause appreciable swelling and shrinkage. However, the percent retained in the durability test was 30 percent for type 1 shale and 80 percent for type 2 shale. From past experience, durability becomes a matter of concern when the percent retained falls below 95 percent. The low durability of the Watts Bar shale is attributed to its slickensided and contorted nature, which makes it susceptible to deterioration upon stress relief.

The results of the durability test are borned out in observations made by Fox [141] during construction of Watts Bar lock. Upon examination of the lock foundation, Mr. Fox expressed the opinion that wetting and drying was not the chief source of deterioration in the shale, but that stress relief causing parting along preexisting planes of weakness, such as joints and slickensided surfaces, was most responsible for deterioration. Mr. Fox thought that final scaling should be followed in less than two hours by placement of the first lift of concrete. As experience was gained, it was found that stress relief was not a problem if the first pour came within 24 hours of scaling.

2.5.4.2.2.9 <u>Recommendations for Design</u>

2.5.4.2.2.9.1 <u>Settlement Analysis</u>

The following recommendations and design considerations are based on the findings of the settlement analysis. If foundation stresses differ appreciably from those assumed in Section 2.5.4.2.2.6.2, settlement values should be adjusted linearly.

1. Design Turbine Building, turbine mats, Reactor Buildings, and Auxiliary Building to behave independently, and provide means to accommodate 1-inch differential settlement where critical utility lines cross from one to another. Install these lines after the water table has been allowed to rise to its natural position around the plant.

2. Maximum probable differential settlement between the edge and center of the reactor mat, and between the end and center of

the turbine mat, assuming them to be perfectly flexible would be 0.3 inches. Design mats to be rigid using De Simone's [138] criteria including the effective stiffness of the superstructure.

3. Maximum probable differential settlement between reactors is 0.20 inches.

4. Maximum probable differential settlement between adjacent individual column footings in the Turbine Building, assuming 10 ksf D.L. + L.L, is 0.20 inches.

5. Settlement of entire plant with respect to the surrounding area will be 1 to 2 inches. At least part of this will be recovered as the water table rises around the structure after construction.

6. Techniques outlined in this report were used to calculate effective subgrade moduli for any size footing founded at any depth into rock. These moduli were used in the design of rigid mats.

7. Bench marks have been established on the foundations of each building to observe settlements during construction.

2.5.4.2.2.9.2 Recommendations for Rock Excavation

In response to the experiences related from the lock, the following recommendations are made concerning preparation of the nuclear plant foundation.

1. The foundation excavation should be brought to 2 to 6 inches above final grade with power equipment. Final scaling was done with hand-operating equipment used with extreme care. In local areas of harder limestone, power equipment may prove inadequate for excavation. Where this situation occured detailed excavation procedures were developed and approved by the Project Manager, Chief Civil Engineer, and Chief Geologist.

2. The shale was excavated in a saw-tooth fashion with the less steeply inclined areas being located on more resistant layers where possible. Flat horizontal surfaces should be avoided.

3. All weathered material at the surface of the bedrock was removed before establishing final grade. Criteria for the definition of weathered rock was established as the foundation is exposed. In general, soft, rust-colored shale was removed.

4. Once final grade is reached, a layer of poured concrete was placed, if possible, during the same working day that final grade is reached, but in all cases within 48 hours of scaling for horizontal surfaces and within 14 days of scaling for vertical surface.

   Where precipitation occured before placing the surface coating, additional scaling was done as necessary. The thickness of concrete was such that the highest points of rock are covered with at least 2 inches of concrete. Thicker coats were used as required where heavy equipment is in continuous operation.

2.5.4.2.2.10 Evaluation of Settlement

2.5.4.2.2.10.1 Initial Settlement Monitoring

The initial program to monitor settlement began in October 1973 to record structural movements during construction. The location of the settlement stations are provided in Figures 3.8.4-66 and -67. Figure 2.5-58 shows details of a typical settlement monument. In general, the monuments were read monthly until 1978, when the frequency was reduced to quarterly for selected monuments and terminated for the rest. The recorded settlement data are given in Tables 2.5-67 through -70. The differential settlement readings between rock-supported structures are provided in Table 2.5-71. During the course of construction several settlement stations became inaccessible (and are so labeled) either because they were physically buried or were impossible or extremely difficult to reach. All accessible settlement stations were last read during December 1981 and January 1982 when three additional surveys were run. During each of these surveys all accessible stations were read except for locations SS-4, -5, -6, -7, -13, -14, -15, and -16. These eight were read once in January 1982 and again in a special survey during February 1982. All eight are in highly congested areas and were previously labeled as inaccessible. These stations were only accessible using specially modified level tripods and survey rods. Six of these eight stations are in the annulus.

During construction, several monuments were relocated for various reasons. Monuments 1 and 2 had associated relocated monuments 1A, 1B, and 2A. These and their corresponding old monuments were all read for a sufficiently long period that they constitute essentailly separate data. The diesel generator building monuments were relocated from the inside to the outside of the building in April 1980. In excess of 7 years' data are available for the interior monuments. Other monuments were relocated several months after original installation and were subsequently read for periods of more than 7 years. These are not identified

by a letter (1A, 2B, etc), but the relocation dates are given in Tables 2.5-69 and -70 and the monuments are identified as reset. Their settlements prior to reset are small and are not carried forward; in effect truncating several months of earlier records. This is done for two reasons. First, the total duration of record (over 7 years) is large compared to the several months truncated. Second, the magnitudes of the 'measured settlements' at the time of reset are small and generally less than the apparent random fluctuations of the dta. Absolute magnitude of error allowed in typical surveys of these monuments is on the order of from 0.01 to 0.06 feet depending on the monument and length of run. However, the actual error of closure for the surveys was generally less than 0.01 feet.

2.5.4.2.2.10.2 Evaluation of the Program

All rock supported category I structures were designed to meet the settlement requirements stated in section 2.5.4.2.2.9.2 for total settlement of 1 to 2 inches and differential settlements of 1 inch. The settlement of rock supported structures was not deemed to control the design of the building. Table 2.5-72 provides the design settlement (total and differential), the maximum recorded settlement, and the current settlement measurement for each rock supported structure. The maximum recorded settlement shown in Table 2.5-72 is for a single monthly reading with both preceding and following readings indicating smaller settlement. The current settlement reading is the average of the three readings from December 1981 to January 1982 unless otherwise noted. 50

The time versus settlement plots of unit 1 and 2 Reactor Building of Figures 2.5-586 and 2.5-587 reflect the latest reliable data available. Readings were discontinued June 1978 because settlement stations became inaccessible. The survey data of January and February 1982 were not used for the plots of the units 1 and 2 reactor building because the surveyors experienced considerable difficulty in reaching the settlement stations in the annulus. However, the current data is presented in Table 2.5-20. For the unit 1 reactor building, the maximum and minimum settlement stations were inaccessible after March 1978, and October 1977 respectively. The unit 2 reactor building settlement was discontinued in June 1978 because the settlement stations became inaccessible.

Updated time versus settlement plots are provided in Figures 2.5-588 and 2.5-589 for the Auxiliary Control Building, the Diesel Generator Building, and the Intake Pumping Station.

The measured settlements have not approached the design criteria of 1 inch of differential settlement between buildings or 1 to 2 inches of total settlement with respect to the surrounding area. In general the maximum settlement of rock-supported structures

had occurred by 1977, and thereafter the settlements have been stable. The maximum settlement of 0.056 feet (0.67 inches) was recorded April 1980. The maximum differential settlement of 0.038 feet between the reactor building unit 1 and the auxiliary building was recorded August 3, 1977. The measured differential settlement of 0.060 feet, August 1977, between settlement stations (SS) 18 and 23 was judged to be a measurement error for three reasons. First, the differential settlements one month before and after were recorded to be 0.018 feet and 0.024 feet respectively. Second, the latest reading between SS18 and SS23 was recorded to be 0.023 feet of differential settlement. Third, the maximum settlement recorded a year before and after the error was 0.036 feet between SS18 and SS23.

We have fulfilled our commitment of monitoring rock-supported structures since the structure loading is essentially complete on all rock-supported buildings, and all the total and differential settlements are well within the design criteria allowables. Settlement readings will no longer be reported for rock-supported structures.

The settlement monitoring of the soil-supported category I structures include the diesel generator building and the waste packaging area. Table 2.5-72 provides the design settlement (total and differential), the maximum recorded settlement, and the current settlement measurement for these structures. The diesel generator building is founded on compacted 1032 crushed stone (an engineered granular fill) underlain by basal gravel and bedrock. This foundation was deemed to have negligible settlement or differential settlement. The diesel generator building as laid out and designed is not controlled by settlement. The connections were designed to allow for a differential settlement of 3 inches for the ERCW piping, 3/4 inch for OZ expansion/deflection fitting (on electrical conduits), and 1 inch between the cable trays and rigid conduits in conduit banks (electrical conduits). 50

The design of the waste packaging area was similar to that of the diesel generator building. The settlement of the building was taken into account when designing the connections between the buildings. The waste packaging area does not have connecting ERCW pipes, other Category I pipes, or Category I electrical conduits. The waste packaging area was designed for 1 inch of differential settlement between the auxiliary building and itself.

Based on our evaluation, the total and differential settlements are not significant; there are no trends being exhibited; there has been no adverse structural performance; and there are no anticipated problems from the settlement of Category I structures.

2.5.4.2.2.10.3 Differential Settlement Not Incorporated in Design Criteria

The design 1-inch differential settlement between adjacent rock-supported structures was not incorporated into the design of piping and electrical components passing between adjacent rock-supported structures. The affected items pass between the unit 1 reactor building and auxiliary building and between the unit 2 reactor building and the auxiliary building. This design deficiency is covered in NCR WBNCEB8108.

The effect of the failure to include the 1-inch differential settlement between adjacent rock-supported structures would be limited to HVAC duct, cable trays, Category 1 piping, instrument lines, and conduit (plus their related supports) which pass between adjacent buildings. Through evaluation TVA has determined that all such HVAC duct, cable trays, and their supports can withstand a 1-inch settlement as is. TVA has also determined by analysis of settlement data on all category 1 structures in the main plant area that the differential settlement of adjacent structures would not be 1 inch, but rather the maximum differential would be less than 1/2 inch. (This 1/2-inch figure is based on settlement which occurred in 1976 and early 1977 which is before the great majority of utility lines were installed.) The analysis also demonstrates that after 1982 additional settlement will be less than 1/4 inch. 50

By the engineering judgment of TVA design personnel, the conservatism inherent in the design of the plant is sufficient to accept the effects of this settlement on Category I piping, conduit, and instrumentation lines without causing line failure or adversely affecting safe operation of the plant.

To confirm this analysis, a system for monitoring future differential settlement was developed.

2.5.4.2.2.10.4 Monitoring Program for Differential Movement

Instrumentation for monitoring future differential settlement has been designed and installed. Details of the relative movement detectors (RMDs) are shown in Figure 2.5-590. Their locations are shown on Figure 3.8.4-66.

2.5.4.3 Exploration

The relationship between Category I foundations and the in situ soil or fill materials are described in the following sections: 50

Plan -		
In Situ Soil Investigations	Section 2.5.4.2.1	
Borrow Investigations	Section 2.5.4.5	50
Excavation and Backfill	Section 2.5.4.5	

The corresponding information with regard to rock is found in Section 2.5.1.2.6.

2.5.4.4 Geophysical Surveys

2.5.4.4.1 Rock Characteristics

The rock characteristics have been discussed in Sections 2.5.1.2.7 and 2.5.4.2.2.1 with regard to dynamic moduli.

2.5.4.4.2 Soil Characteristics

In situ soil dynamic studies were made at the Watts Bar site to obtain data for computation of elastic moduli for earthquake design criteria. Tests consisted of the following:

1. Down-hole seismic surveys for 4 stations in the intake channel, for 1 station for the diesel generator building, and for 25 stations for the Class IE conduit and ERCW piping alignments.

2. Seismic refraction surveys along two lines in the Diesel Generator Building and along four lines in the intake channel.

2.5.4.4.2.1 Equipment

1. Intake Channel and Diesel Generator Building

   The equipment used to record the time arrivals for compressional and shear wave velocities was a Bison signal enhancement seismograph, Model 1570B, a Bison strip chart recorder, a Hall-Sears MP-4 pressure type geophone, and a 8-hz Mark Products geophone.

2. Class IE Conduits and ERCW Piping Alignments

   The equipment used for recording seismic waveforms through the soil consisted of a Bison Instruments signal enhancement seismograph, model 1575, strip chart seismic recorder and blaster, and a Hall-Sears MP-4 hydrophone.

2. Class II accounted for 54 percent and classified sandy, lean clay, CL, with an optimum moisture content of 17.9 percent and a maximum density of 101.1 pcf. Soils represented by this class had an average moisture content of 23.3, or 54 percent above optimum.

3. Class III amounted to 35 percent and classified sandy, plastic silt, MH, with an optimum moisture content of 21.8 percent and a maximum density of 101.1 pcf. Soils represented by this class had an average moisture content of 24.5, or 2.7 percent above optimum.

In summary, this investigation in the main plant area has established that borrow soils of a satisfactory type, but of relatively high moisture content, are present in cut areas at the site. Materials of acceptable moisture content were present in the transformer yard, the 500-kV switchyard, and cooling tower areas. Materials in the reactor building, plant building, and intake channel areas required drying before using these soils as fill.

The borrow soil types identified in the intake channel were remolded and tested for shear strength. The results are shown in Table 2.5-21. The results are shown in graphical form (Figure 2.5-251). As indicated by the graphical plot (Figure 2.5-251) the value selected for design is conservative. 27

Onsite Borrow Areas

The onsite borrow areas which are shown on Figure 2.5-221 and 2.5-221a were completed after the completion of the borrow investigation in the main plant area described above. The results of the borrow investigation are described in relationship to the results obtained from the borrow investigation in the main plant area. 50

Area 1, which covers approximately 13 acres, lies about 2500 feet east of the main plant area. The soil profile, as established by the eight auger borings drilled in this area, consists of 8 to 13 feet of reddish-brown silt, ML and MH, underlain by 7 to 16 feet of brown lean clay, CL. (see Figure 2.5-223). The majority of these fine-grained essentially impervious soils have natural moisture contents near the plastic limit. This area will supply about 280,000 cubic yards of borrow material.

Area 2 is located about 3700 feet west-northwest of the main plant site and covers about 8 acres. The predominant soil type encountered in this area is a lean clay, CL. (see Figure 2.5-223). Some silt of medium plasticity, MH, and a small amount of fat clay, CH, are also present. As in area 1, the upper soils are reddish brown to a depth of 4 to 7 feet with the underlying

soils colored brown. This area will supply about 170,000 cubic yards of borrow.

In area 3, two borings were drilled and reddish brown to brown lean clay, CL, was encountered. (See Figure 2.5-223). Secondary soils are lean silt, ML, and clayey gravel, GC with the gravel consisting of subrounded quartizite. This area is located about 3600 feet west-southwest of the power plant and extends over 3 acres. About 50,000 cubic yards of fill will be available from area 3.

In area 4, located southwest of the plant and covering approximately 6 acres, seven borings were drilled and alluvial sandy lean clay, CL. was encountered, (See Figure 2.5-224. Secondary soils are silty sand, SM, and sandy silt, ML, which are slightly micaceous. This area, located southwest of the power plant, covers about 100,000 cubic yards of borrow.

Area 7 covers approximately 12 acres and is borings located southwest of the main plant area. Eleven borings were drilled and the predominant soil encountered was a lean clay (CL). Secondary soils encountered were a lean clayey-silt (CL-ML) and a lean silt (ML). Minor quantities of a fat clayey-silt (CH-MH) and a fat silt (MH) were also encountered. Approximately 145,000 cubic yards of material is available from the area.

Subsoils in the areas are similar in texture and plasticity to the borrow soils determined in the soil investigation previously reported. Soil properties are listed in Table 2.5-19. The natural moisture contents are from 2 to 8 percent above optimum. Close moisture control during placement will be required to assure adequate compaction. In summarizing, these additional borrow sources at Watts Bar Nuclear Plant will yield approximately 0.7 million cubic yards of impervious fill with satisfactory characteristics. 44

Borrow area number 4 was selected as the source for any soil necessary for the construction qualified fills. The area was selected based on; (1) the quantity of borrow material available, and (2) the information provided on the graphic logs. Table 2.5-25 presents laboratory test data on the borrow classes available in borrow area 4. The strength values used for design are shown in Table 2.5-12. The results for each type shear test are plotted in graphical form (Figures 2.5-244 through 2.5-246), and a conservative value below the average for c and 0 is selected for use in the design. The values used for design (Table 2.5-12) are low averages for the strength data shown in Table 2.5-25. 24

Due to the need to construct the underground barrier trenches to resolve the issue of potentially liquefiable soils along portions of the ERCW piping and 1E conduit alignments, several additional onsite borrow areas were investigated for use as safety-related 50

fill. The additional areas are shown on Figures 2.5-220, -221, and -221a. These areas are identified as Trench A, Trench B, Areas 9, 10, 11, 12, 13, and 2c, and the future 161-kV switchyard. The central laboratory investigated each of these areas and developed moisture-density compaction curves (ASTM D 698) for each area. The testing identified several soil classes for each area. The laboratory strength testing consisted of consolidated-undrained (R) shear tests on each soil class. Samples were molded to 95 percent of maximum dry density (ASTM D 698) and 3-percent below optimum moisture content. All samples were subsequently saturated prior to shearing. Due to the desire for a higher design cohesion, borrow classes with a cohesion intercept (c) less than 0.2 tons/ft² were retested at a higher density. These samples were remolded to 100 percent of maximum dry density (ASTM D 698) and 3-percent below optimum moisture content. All samples were saturated prior to shearing. The test results for each borrow area are shown on Tables 2.5-45 through -53. The results of this testing were evaluated to provide soil properties to use in the design and analysis of the underground barrier trenches.

The backfill used for Trench A came from borrow areas Trench A, 9, 10, 2c, and the future 161-kV switchyard. Thus, materials from these areas were evaluated for the Trench A design soil properties. Since two different degrees of compaction were used in Trench A, separate evaluations were made. The first evaluation, shown on Figure 2.5-520, was for Earthfill A which was placed at 95 percent of maximum dry density, and the second evaluation, shown on Figure 2.5-521, was for Earthfill A1 which was placed at 100 percent of maximum dry density. In the second evaluation, the data for sands was deleted from the evaluation, since only fine-grained soils were used for Earthfill A1. 50

The backfill used for Trench B came from borrow areas Trench B, 12, 2c, 13, and the future 161-kV switchyard. Thus, materials from those areas were evaluated for the Trench B design soil properties. Since two different degrees of compaction were also used in Trench B, separate evaluations were made. The first evaluation, shown on Figure 2.5-522, was for Earthfill A which was placed at 95 percent of maximum dry density, and the second evaluation, shown on Figure 2.5-523, was for Earthfill A1 which was placed at 100 percent of maximum dry density. In the second evaluation, the data for sands was deleted from the evaluation, since only fine-grained soils were used for Earthfill A1. Figure 2.5-583 provides a summary of the above borrow evaluations.

2.5.4.5.1.3 <u>Field Work</u>

Prior to construction, the central laboratory prepares a family of compaction curves for all soil classes at the site (see Figure 2.5-235, 2.5-271, 2.5-524 through 2.5-533). The soil classes are further divided into subclasses for use by the inspectors of

backfill placing and the project laboratory for construction control and day-to-day testing of fill compaction. These tests are performed by the project laboratory and are for dry density, moisture content, and degree of compaction. A minimum of at least one set of tests for each 2000 cubic yards of fill placed is performed throughout the course of the work. Additional sampling and testing are done as required by the inspectors or engineers in charge. 50

The quality of the backfill is documented by measuring the in-place density. The inplace compaction is expressed as a percent of the maximum density at optimum moisture content for the backfill material being placed. A backfill log book is maintained containing all pertinent information concerning daily backfill operation. 44 40

In addition, a penetrometer is used, correlated with penetration charts prepared by the central laboratory (see Figure 2.5-234, 2.5-272, and 2.5-534 through 2.5-543) to maintain a continual check on the compaction of the backfill. At Watts Bar Nuclear Plant, Class A backfill is placed around all Category I structures. This material, which is selected earth placed in not more than 6-inch layers, has a minimum required compaction of 95 percent of the maximum standard density at optimum moisture content. Class A1 backfill used in portions of the underground barrier trenches has the same requirements except it has a minimum required compaction of 100 percent of maximum dry density of optimum moisture content. 44 50 5

The limits of excavation and the backfill placed around the Category I structures are shown in Figures 2.5-225 and 2.5-226.

Class B backfill is placed around non-Category I structures. This material, which is selected earth placed in not more than 9-inch layers, has a minimum required compaction of 90 percent of the maximum standard density at optimum moisture content.

A third class of fill is also used, Class C, using unclassified fills to be placed in approximately 12-inch layers and compacted with hauling equipment. This fill class is used in areas not requiring Class A or B fills, or highway and railroad fills, such as spoil areas.

The fill used to form the channel slopes in the intake channel is composed of material originally excavated from the intake channel. The material is compacted to 95 percent of maximum density at optimum moisture content.

Earthfill borrow areas are worked in a manner which ensures a suitable material for compaction. They are excavated in layers so that widely varying soil classes are not mixed during placement and compaction. Any conditioning which the soil requires is

normally accomplished in the borrow areas prior to hauling it to the earthfill site. This conditioning includes control of moisture content and removal of deleterious materials. All borrow areas are maintained such that adequate drainage of ground water and surface runoff is provided. Drainage will be accomplished by sloping excavations, crowning, channels, dikes, sumps, and pumping, as necessary. 50

Compaction of large areas of earthfill is accomplished using crawler-drawn or self-propelled sheep-foot rollers. Soils in areas of limited access are compacted with small power tampers or rollers. Compaction and all other earthwork is suspended during periods of inclement weather. 50

In areas where earthfills with differing compaction requirements adjoin, the compacted fill with the higher degree of compaction is placed prior to the placement of fill of lower density requirements. 50

2.5.4.5.1.4 Construction Control

All earthfills are placed in accordance with the provisions of TVA's General Construction Specification No. G-9 for Rolled Earthfill for Dams and Power Plants.\\The following information summarizes the construction control which is described in that document. This program is also applicable for all engineered granular fills. 50

All fill operations are accomplished in the presence of a trained inspector. The inspector has the authority to suspend fill operations whenever weather or material conditions are judged unsuitable. His responsibilities include material quality, selection, excavation, hauling, placement, and compaction control. During placement, periodic construction control tests are made to ensure that a suitable fill is obtained. This testing determines soil classification, moisture content, inplace density, relative density (granular fill only), and degree of compaction (earthfill only). The frequency of testing is as specified in General Construction Specification G-9. The inspector may require additional testing to conclusively identify material or check compaction. A project laboratory has been established at the plant site to perform the necessary testing. Project drawings and a series of construction control procedures relay unique construction requirements to the construction personnel. 50

2.5.4.5.2 Granular Fill

2.5.4.5.2.1 General

Granular fill materials are used at the site for several purposes; such as structural fill, backfill, to establish a working surface, and for road foundations. The material is 50

obtained from offsite commercial sources. The location and use of any type of material is determined by the engineer for any safety-related feature.

Section 1032 Material -

A granular fill material, consisting of crushed stone or sand and gravel, placed around and below safety-related features in lieu of earthfill in certain locations. The granular fill material is suitable for compaction to a dense, stable mass and consists of sound, durable particles which are graded within the following limits:

Passing	Percent by Weight Minimum	Maximum
1-1/4 inch Sieve	100	
1 inch Sieve	95	100
3/4 inch Sieve	70	100
3/8 inch Sieve	50	85
No. 4 Sieve	33	65
No. 10 Sieve	20	45
No. 40 Sieve	8	25
No. 200 Sieve	0	10

The material is free of soft friable particles, salt, alkali, organic matter; or an adherent coating and reasonably free of thin, flat, or elongated pieces. 50

Laboratory shear strength tests were performed on the granular material to establish design properties. The testing consisted of triaxial (Q&R) and direct (S) shear tests. The tests were made on samples compacted to 70 percent and 80 percent of maximum relative density (ASTM D 2049). The samples' composition were varied to provide three separate gradations for testing.

The three gradations tested are as follows:

Sieve Size	Percent (by Weight) Passing Maximum Fines	Average Fines	Minimum Fines
1-1/4 inch	100	100	100
1 inch	100	100	95
3/4 inch	100	88	70
3/8 inch	85	67	51
No. 4	65	49	33
No. 10	45	32	20
No. 40	25	17	8
No. 200	10	5	0

Minimum and maximum densities were determined in accordance with ASTM D 2049.

The triaxial shear tests (Q&R) were made in a 4-inch diameter testing machine on particles passing the 3/4-inch sieve. The direct shear tests (S) were made using a 12-inch square shear box on particles passing the 1-1/4-inch sieve. The results of the shear testing are shown on Table 2.5-54, and the values to use for design are shown on Table 2.5-55. Figures 2.5-544 through -547 are graphical plots of the test results with the adopted design values for each type of shear test.

The apparent shear strength values for the R test are not presented because the test results were determined to be inconsistent. On tests at 80-percent relative density, two of the three sets of the R tests showed significant negative pore water pressures during the tests. It is unrealistic for a saturated fill of this granular material to develop negative pore pressures. During earthquakes, pool drawdowns, or conditions of steady seepage, a crushed stone fill would more likely develop positive pore pressures rather than negative pore pressures. Thus as indicated on Table 2.5-55, pore pressures will be incremented during analysis to check the effect of pore pressure buildup.

The test results indicate that the coarse particle-size distribution (minimum fine distribution) produces a slightly higher friction angle along with a marked increase in cohesion intercept. Part of the 'cohesion' appears to be the result of interlocking of the angular particles. Overall, the shear strength increases as particle size increases. 50

Consolidation tests were not made on the granular material, since consolidation would be negligible at the densities the fill is placed and because any connections between adjacent structures would not be made until after any minor consolidation had occurred.

In areas where this granular material is placed adjacent to an earthfill, the granular fill is placed and compacted prior to the placement of the earthfill. Granular fill is placed and compacted to a relative density as specified on drawings or in construction specifications and as determined by ASTM D 2049. The moisture content of the material is adjusted as necessary to obtain the required relative density. The construction control program for granular fill is discussed in Section 2.5.4.5.1.4.

As a result of inquiries by NRC about the granular material used to support the Diesel Generator Building, the following tables and figures are provided:

1. Table 2.5-56 showing the compaction results;

2. Figure 2.5-548 showing a statistical summary of the compaction test results; and

3. Table 2.5-57 showing sieve analysis results on the material stockpile during the period which the granular fill material was placed for the Diesel Generator Building.

Section 1075 Material -

A free-draining granular fill material, consisting of crushed stone or sand and gravel, frequently used to establish a working surface on top of soil or weathered rock, or to develop a good interface between earthfill and weathered rock, or to act as a surface cover for an area such as a switchyard.

The granular fill material is graded within the following limits:

Sieve Size	Percent (by Weight) Passing: Bottom Layer	Alternate Bottom Layer	Top 2' Layer
1-1/2 inch	100	100	-
1 inch	90-100	-	-
3/4 inch	40-75	30-75	100
1/2 inch	15-35	-	90-100
3/8 inch	0-15	5-15	40-75
No. 4	0-5	0-5	5-25
No. 8	-	-	0-10
No. 16	-	-	0-5

50

The material is free of soft friable particles, salt, alkali, organic matter, or an adherent coating and reasonably free of thin, flat, or elongated pieces.

In areas where the material is used, it is placed and compacted using a procedural specification given on drawings or in construction specifications.

2.5.4.6 Groundwater Conditions

The normal ground water level for the main plant is at elevation 726, which is 2 feet below plant grade. This level was determined when making soil borings in the main plant area. Each structure in the main plant area is designed for the normal ground water level at elevation 710 for service load conditions, although most structural design for hydrostatic forces are controlled by the PMF.

In order to control groundwater seepage into any structure, each construction joint up to grade has a seal embedded across the joint to prevent the passage of water. Dewatering during construction is controlled by routing any seepage into the

and conduit alignments are discussed in Section 2.5.4.2.1.2. The purpose of the additional borings was to decrease the spacing of borings to approximately 100 feet along the piping and conduit alignments and to investigate apparent soft layers of fine sands. The majority of the additional borings were made with dry drilling procedures using a hollow stem auger to advance the borings. The borings added to investigate apparent soft layers were made with wet drilling procedures using drilling mud and fishtail bit to advance the boring.

Profiles were prepared to show the relationship of the safety-related piping and conduits to the soil deposits. A profile of the ERCW pipes is provided in Figures 2.5-549 through -553. The 1E electrical conduit profiles are provided in Figures 2.5-554 through -556. The profiles provide the following information:

a. Pertinent boring logs along the routes showing blow counts and the classification of the in situ soil.

b. The elevatin of original grade, final grade, to of weathered shale, and the top of rock. (Note: Fill material was used to backfill around the pipes and to achieve final grade.)

c. The electrical conduit and ERCW pipelines and their elevation to scale. 50

d. The 24-hour water table and the design groundwater.

Figure 2.5-273 is a plan of the piping and conduit which shows the locations where the sections are drawn.

Test pits were excavated at two locations along the pieline and are discussed in Section 2.5.4.2.1.2. The testpits were made to obtain undisturbed block samples to determine in situ densities and for cyclic triaxial testing.

A series of standpipe piezometers were intalled along the pipeline to monitor the groundwater to establish a design groundwater level. The result of this monitoring is discussed in Section 2.5.4.6.

Several evaluations of potential soil liquefaction have been made. The criteria have varied from (1) evaluations based upon empirical rules derived from records of historical events, to (2) evaluations using laboratory cyclic shear testing, to (3) evaluations using empirical rules based on standard penetration testing, grain size analysis, and a correlation procedure developed to Seed and Idriss (Reference 1).

Liquefaction Evaluation - Based on Empirical Rules

The first approach to the evaluation of the liquefaction potential of soils was done using grain size distribution curves in conjunction to the earlier outlined empirical rules. Only those silty sands above the top of weathered shale were evaluated since all materials below that point are merely rock fragments which have been given a soil classification.

Figures 2.5-557 through -563 are grain-size distribution curves for SM material in borings SS-50, SS-60, and SS-63. These curves show that the silty sands are well-graded materials rather than the uniformly graded materials one associates with liquefiable soils. In addition, a comparison of these gradation curves with those for materials which are known to liquefy, including the potentially liquefiable sand encountered in the intake channel, shows that the gradation characteristics are not typical of those for liquefiable soils.

The electrical conduits follow the route defined by borings 49, 50, 51, 52, 53, 57, and 58. The graphic logs of the borings for the Class 1E electrical conduits show that the SM and G-SM materials are not present in extensive layers. These materials are present in isolated pockets. For instance, the SM and G-SM material in SS-50 extending from elevations 689.0 to 699.0 is not encountered in either SS-49, SS-51, or SS-59. Rather, thin layers of silty sand no more than 1' to 1-1/2' thick are found at elevations which do not correlate with boring 50. Approximately 5' of SM and G-SM material is found above the top of weathered shale in boring SS-53. However, the undisturbed boring SS-53 which was made only 5' away encountered no SM or G-SM material above top of weathered shale. One therefore concludes that the silty sand in SS-53 is not an extensive layer. The layers of silty sand in borings SS-60 and SS-63 are not extensions of the same layers since the silty sand of SS-53 is an isolated pocket. Similarly, the remainder of the borings show no evidence to suggest extensive layering of silty sands. 50

Borings 59 and 51 clearly establish that the silty sands encountered in the other borings for the electrical conduits are not extensions of the potentially liquefiable sands encountered in the intake channel since only a thin layer of silty sand is encountered in 51 and none in 59. The investigation of the intake channel revealed that the continuous layers of silty sand tapered out at the intake pumping station. This investigaion substantiates that the silty sands shown in borings along the 1E conduits are not continuous layers. Finally, a comparison of the gradation characteristics of the potentially liquefiable sands of the intake channel (a typical gradation curve is presented in Figure 2.5-566) with those of the silty sands along the conduit and pipeline routes, shown in Figures 2.5-557 through -565, shows that the sands are not similar.

to the location of SM matrial encountered in the borings. The resulting cyclic stress ratio is 0.32.

The profile selected and analyzed is based on boring SS-50-1 (Figure 2.5-339) which contained the most SM material. The surface elevation is 716.9. Around elevations 685.0 and 690.0 the blow count increases to +50 and is identified as 'top of weathered shale.' This is assumed as 'top of rock' for the liquefaction evaluation. Thus the depth of the profile is 30'. The water table is about 15' to 20' below the ground surface in borings SS-50, SS-50-1, SS-65, and SS-65-1. thus the water table is assumed to be 15' to 20' below the ground surface. The profile analyzed is fairly typical of those along the ERCW route. This generalized soil profile is shown graphically in Figure 2.5-569.

The soil unit weight (moist) was assumed to be 120 lb/ft^3. The shear wave velocity of the soil is taken as 1000 ft/s. This value is in agreement with data obtained from the intake channel and elsewhere on the site. The strain dependent shear modulus and damping ratio properties of these soils are assumed to conform with the relationships developed by Seed for sand. The coefficient of earth pressure at rest (K_0) is conservatively taken as 0.5. All soil properties are assumed to be constant with depth.

The rock has a unit weight of 165 lb/ft^3 and a shear wave velocity of 5900 ft/s.

The postulated site SSE is based on an intensity MM VII-VIII or VIII event. Such an event would be approximately a magnitude 5.5 to perhaps a 6.0. The seismic input at the site is defined as a 0.18 g earthquake at top of rock. The liquefaction evaluation was performed using an artificial accelerogram which conforms to Regulatory Guide 1.60 requirements. Peak accelerations of 0.18 g, 0.225 g, and 0.25 g were considered. 50

The accelerogram was also high band pass filtered to eliminate frequencies greater than 5 Hz for three cases and 25 Hz for two cases. In all, five different analyses were performed and are listed below.

Case	Maximum Acceleration	Applied at Top of	Upper Frequency Cutoff
1	0.25 g	Ground	5 Hz
2	0.18	Ground	5
3	0.225	Ground	5
4	0.25	Ground	25
5	0.18	Rock	25

The most appropriate seismic loading is case 1 where the 0.25 g accelerogram is applied at top of ground with a 5-Hz upper frequency cutoff. The results from case 1 essentially envelop all other cases except for case 5 where the input is at top of rock.

The dynamic response analysis was performed using the computer program SHAKE. Irregular shear stress time histories were not calculated. The equivalent uniform cyclic stress was taken as 65 percent of the maximum cyclic shear stress within each layer of the profile as calculated by SHAKE.

The results of the analyses are given in Table 2.5-59. The maximum and equivalent uniform stresses within each layer and the peak accelerations at the top of each layer are summarized in Table 2.5-59 for all five earthquake input conditions.

For material located about 17.5' below the surface (approximately the elevation of the samples which were tested cyclically), the maximum shear stress is:

$$\tau \text{ max} - 500 \text{ lb/ft}^2$$

The average shear stress is:

50

$$\tau_{avg} - 0.65\ \tau_{max} = 325 \text{ lb/ft}^2$$

The vertical pressure at 17.5' is:

$$\sigma_v = \delta_h = (120 \text{ pcf})(17.5') = 2100 \text{ lb/ft}^2$$

Assuming $K_o = 0.5$, the horizontal stress is:

$$\sigma_h = 0.5\ \sigma_v = 1050 \text{ lb/ft}^2 \text{ use } 1000 \text{ lb/ft}^2$$

The cyclic stress ratio is:

$$\frac{\sigma_d}{2\sigma_3} = \frac{\tau}{\sigma_3} = \frac{325 \text{ lb/ft}^2}{1000 \text{ lb/ft}^2} = 0.32$$

Figure 2.5-353 shows the most susceptible sample will survive six load cycles with this stress ratio. Only five uniform load cycles should occur from a 0.18 to 0.25 g event. This event is an intensity VIII earthquake and is characterized as an m_{blg} 5.8. Extrapolating Seed and Idriss data:

drilling mud and a fishtail bit. The results of this additional drilling are given in Section 2.5.4.2.1.1. The correlation procedure developed by Seed and Idriss (1981) differentiates between clean sands and dirty sands. The fine sands at the site are relatively dirty or silty with fines content (minus No. 200 sieve) ranging up to 50 percent. The cyclic performance of the silty sands is better than for clean sands when both materials have the same blow count or other similiar characteristics.

This increased performance under cyclic loading is accounted for in the Seed and Idriss (1981) procedure. The additional cyclic tests were made on undisturbed block samples obtained from two test pits excavated along the ERCW piping alignment.

Several reports have been submitted to the NRC on this evaluation. The initial report, reference 166, was issued February 8, 1982.

The liquefaction evaluation was based on a seismic input of 0.18 g at the top of ground. The report also provided the results of a study to project the seasonal high groundwater levels for use in the liquefaction evaluation. These seasonal high groundwater levels were used in this and all subsequent liquefaction evaluations as the design groundwater. Additional water level readings from the groundwater monitoring program are given in Table 2.5-58. This evaluation concluded that although a few samples (3) had factors of safety against potential liquefaction less than one, the areas of concern were isolated and unlikely to cover a large lateral area and the potential settlement was insignificant. The second report, reference 167, was an update of the initial report, reference 166. The second report, reference 167, basically expressed the same conclusions as the initial report, reference 166. However, this report, reference 167, contained cyclic triaxial test results showing that the samples that had factors of safety less than one, using the Seed and Idriss (1981) procedures, would not liquefy. 50

Due to NRC concerns about the seismic input to the liquefaction evaluation, an additional study was made. The seismic input to the evaluation was 0.22 g at the top of ground. This revised seismic input was based on the results of a site-specific study and as discussed in Section 2.5.4.2. The third report, reference 168, containing the results of this evaluation, was issued in November 1982. The report indicated that additional samples were susceptible to liquefaction, but the samples were localized and there were no indications that the liquefiable zones were continuous. In addition, the cyclic triaxial tests showed that these additional samples would not liquefy. The potential settlement due to the postulated liquefaction was also considered minimal.

As a result of several meetings with the NRC and the NRC's review references 166, 167, and 168, the seismic input and the procedure for evaluating liquefaction were changed. The seismic input was changed from 0.22 g to 0.40 g at top of ground as discussed in Section 2.5.2.4. The procedure for evaluating liquefaction was changed from the Seed and Idriss (1981) procedure to the Seed and Idriss (1971) procedure. Both procedures are simplified methods for evaluating the liquefaction potential of sands, but the Seed and Idriss (1981) procedure provides a modification that accounts for presence of fines in the sand samples. In order to resolve the issue of potentially liquefiable soils at the site, TVA used the Seed and Idriss (1971) procedure.

A report has not been issued on the liquefaction evaluation based on a seismic input of 0.40 g at top-of-ground and the Seed and Idriss (1971) procedure. However, the results are presented as follows:

1. Tables 2.5-62 through -64 tabulates the samples that would potentially liquefy, i.e., (FS 1.0).

2. Figure 2.5-273 shows the layout of the ERCW piping and 1E conduits and the location of the sections that show the piping and conduit profiles.

3. Figures 2.5-571 through -575 show profiles of the ERCW piping and the borings along the alignment. The borings have been marked to indicate the design groundwater, top of weathered shale, and the samples that will potentially liquefy. 50

4. Figures 2.5-576 through -579 show profiles of the 1E conduit banks and the borings along the alignment. The borings have been marked to indicate the design groundwater, top of weathered shale, and the samples that will potentially liquefy.

The result of this evaluation is that the zones of potentially liquefiable materials are apparently continuous in some areas along the pipeline and conduit alignments and that some method of remedial treatment is needed. The method of remedial treatment to prevent the lateral flow of liquefied soils, the method of analysis, and the results are described in Sections 2.5.5.1.2 and 2.5.5.2.3.

As discussed in Section 2.5.4.6, the groundwater level was revised to reflect an estimated 25-year groundwater. The influence of this slightly higher groundwater on the liquefaction analysis and potential settlement due to liquefaction was discussed with the NRC staff. The staff indicated they concur with our judgement that the higher groundwater will have

negligible effects on the results of the liquefaction and settlement evaluations, therefore no additonal evaluations for the piping or conduits are needed.

The potential settlement of the soils along the ERCW pipeline and 1E conduit alignments, due to an earthquake sufficient to cause liquefaction, were evaluated for each report, references 166, 167, and 168. All studies revealed that the potential settlement was insignificant or minimal and the performance of the piping or conduits would not be affected. When the peak ground acceleration was increased to 0.40 g (see Section 2.5.2.4) and the method of evaluating for potential liquefaction was changed 50

to the Seed and Idriss (1971) procedure, the extent of the soils that would potentially liquefy increased, thereby significantly increasing the amount of potential settlement. The theoretical settlement at each boring location along the ERCW pipeline and 1E conduit alignments was calculated twice. The initial settlement evaluation was based on a paper by Lee and Albaise (1974) (Reference 165). The second evaluation was based on a criteria provided by the NRC staff. The method and results of each evaluation are described below.

The evaluation based on Lee and Albaise's paper assumed the test data for a Monterey sand was applicable and the in situ relative density of the fine sands was 50%. Using test data for a Monterey sand is conservative, since the D_{50} for the fine sands at the Watts Bar site is in the range of 0.07 mm to 0.15 mm, and the test data shown in Figure 6 of the Lee and Albaise paper indicates that a finer sand will experience a lower volumetric strain. The use of an in situ relative density of 50% is also conservative, since the relative densities of the undisturbed block samples from the test pits ranged from 61% to 69% for two of the samples and above 70% for the other sample. The test data shown in Figure 7 of the Lee and Albaise paper indicates that a soil with a lower relative density will experience a higher volumetric strain. Based on Figure 7 of the Lee and Albaise paper, a Monterey sand sample with an initial relative density of 50% that subsequently liquefies will experience approximately 1.5% volumetric strain. For the initial settlement evaluation sand (SM or SP) samples that were theoretically susceptible to liquefaction were considered to experience 1.5% volumetric strain, and silt (ML) samples were considered to experience 0.75% volumetric strain. Figures 2.5-571 through -578 show the potential settlement calculated using the 1.5% strain (1.5%E) criteria at each boring along the pipeline and conduit alignments. 50

The criteria specified by the NRC staff is shown in Table 2.5-65 has a maximum volumetric strain of 6%. The criteria specifies a volumetric strain even for samples that will not liquefy. The results of the evaluation for potential settlement at each boring along the pipeline and conduits using the 6% strain (6%E) criteria are also shown on Figures 2.5-571 through -578. As can be noted, the potential settlement using the 6% criteria is significantly higher than the results using the 1.5% strain criteria. However, in order to resolve the issue of the potential settlement due to soil liquefaction, the results of the settlement evaluation based on the NRC staff's criteria (6%) was used for evaluating the need for remedial treatment for the pipeline and conduits. The evaluation of the piping for the potential settlement along the ERCW piping alignment is described in Section 3.7.3.12. The evaluation of the conduits for the potential settlement along the 1E conduit alignment is discussed in Section 3.7.2.1.2.

ation and testing are presented in Section 2.5.4.2.1.3. Characteristics of the slopes and the underlying soil deposit are also presented in Section 2.5.4.2.1.3.

2.5.5.1.2 Underground Barrier for Protection Against Potential Soil Liquefaction

The underground barrier is a manmade feature extending along the ERCW pipeline and 1E conduit alignments in the area north of the intake pump station and south of the cooling towers and 500-kV switchyard. The purpose of the underground barrier is to prevent the lateral flow of soils should an earthquake occur that could liquefy some of the soils below the ERCW piping and 1E conduits. The underground barrier is located between the safety-related piping and conduits and the area towards which the material would attempt to flow should the soils liquefy. The liquefaction evaluation is presented in Section 2.5.4.8.

The underground barrier will be constructed by excavating two trenches. The location of the underground barrier trenches are shown on Figures 2.5-580 and -581. The locations were based on the extent of the potentially liquefiable soils along the piping and conduit alignments as shown on Figures 2.5-571 through -578. Figure 2.5-582 shows the layout of the underground barrier trenches in relation to the borings which indicate potentially liquefiable material.

The trenches will be backfilled with soils excavated from the trenches, if acceptable, and soil from approved onsite borrow areas. The method of construction and construction control will be in accordance with the requirements and notes on Figures 2.5-580 and -581. The results of the soils investigation and testing of the borrow materials is described in Section 2.5.4.5.1. The design and analysis of the underground barrier is described in Section 2.5.5.2.3. 50

As can be seen on the layout (Figure 2.5-582) and on the profiles, some borings with potentially liquefiable material will not be included in the area encompassed by the underground barriers and no remedial treatment is being planned. Each of these areas is discussed in detail as follows:

1. At boring SS-143 (Figure 2.5-571, sheet 2 of 4) and its associated borings (SS-143A, B, and C), the soil is localized; the liquefiable material is a thin layer which would produce small settlements. In three of the borings, it is unrealistic to expect the material to liquefy. The G-SP-SM (elevation 693.0) in boring SS-143 is part of the basal gravel that exists at the site (the 'G' indicates the sample has greater than 12% gravel); the CL-ML (elevation 697.0) in boring SS-143C should not liquefy due to the high percentage of fines; and the SM (elevation 696.0) in boring SS-143B with

a blow count of 21. The results of an extensive test program on the basal gravel is discussed in Section 2.5.4.2.1.3 (In Situ Basal Gravel).

2. At borings SS-146 and SS-147 (Figure 2.5-571, sheet 2 of 4) both samples shown to be susceptible to liquefaction are in the basal gravel. Also, the blow counts (13 and 18) of the samples (13 and 18) indicate a fairly firm material.

3. At boring SS-153 (Figure 2.5-571, sheet 3 of 4) the sample (G-SW-SM at elevation 707.0) represents a thin isolated pocket and the sample is in the basal gravel.

4. In the main plant area (Figures 2.5-571, sheet 4 of 4, 2.5-572 through - 575, and 2.5-577 and -578), there are no problems related to soil flow during liquefaction since there are no slopes in the area. Potential settlement in this area is discussed in Section 2.5.4.8.

5. In the southern part of the switchyard, soils encountered in borings SS-53, SS-54, SS-55, SS-62, and SS-61, show some liquefaction potential. However, liquefaction does not appear to be realistic. In boring SS-53 (Figure 2.5-579) the two samples, an ML (elevation 711.0) and an SM (elevation 707.0) with apparent liquefaction potential have high blow counts (20 and 18), and one, the ML, has a high plasticity index (PI=18.4). In boring SS-54 (Figure 2.5-579) the two samples, an ML (elevation 703.0) and an SM (elevation 701.0) that apparently would liquefy have high blow counts (19 and 21) and have medium to high plasticity indices (PI = 10.4 and 16.8). At boring SS-55 (Figure 2.5-579) the two ML samples (elevations 714.0 and 709.0) have blow counts that are good to high (14 and 19) and the plasticity indices are high (PI = 18.4 and 14.3). At boring SS-62 (Figure 2.5-579) the blow count of the potentially liquefiable material (elevation 687.0) is good (14) and the plasticity index is high for an SM (PI = 13.8) and, in addition, the layer is very thin and is probably weathered shale rather than alluvium. At boring SS-61 (Figure 2.5-579) the material is localized, located at the surface where it will not affect any soils overlying it; and it is a long distance from the conduit bank. 50

2.5.5.2 Design Criteria and Analysis

2.5.5.2.1 Design Criteria and Analyses for the Essential Raw Cooling Water Intake Channel Slopes

The static design cases and the conditions and factors of safety associated with each are shown below:

Case	Factor of Safety
1. Normal operating condition with reservoir elevation 675, groundwater elevation 685.	1.5
2. Sudden drawdown due to loss of downstream dam: groundwater elevation 685; reservoir drawdown elevation 685 to 666.	1.1
3. Construction condition: groundwater elevation 685, channel dry.	1.25

The earthquake design cases are the same as Case 1 and 2 above combined with a Safe Shutdown Earthquake. The minimum factor of safety must be equal to or greater than 1.0. 24

Static Analysis

Slip circle analysis using the Modified Swedish method were performed for the static design Case 2. The critical circle, which has a factor of safety of 2.5, is shown in Figure 2.5-238. The combination of events comprising design Cases 1 and 3 are less than those for Case 2. Since the factor of safety for Case 2 is 2.5, then the factor of safety for Cases 1 and 3 will be greater than that required for these cases.

The soils exploration in Section 2.5.4.2.1.3 disclosed a possible weak layer of lean clay soil at approximate elevation 680 to 685 in borings US-30 and US-36, which are on opposite sides of the channel near the reservoir. The test results indicate the minimum strength properties of this material as 0 = 3° and c = 500 psf. Wedge analyses were performed for design Case 2 assuming a failure plane at elevation 680, using these strength properties under the wedge. The minimum wedge, which has a factor of safety of 3.7, is shown in Figure 2.5-238. By inspection again, design Cases 1 and 3 are satisfied.

Earthquake Analysis

The soils exploration results presented in Section 2.5.4 revealed some silty sands that were possibly subject to liquefaction under earthquake excitation. Section 2.5.4.8 deals with the evaluation of the liquefaction potential following the performance of cyclic triaxial shear tests on these silty sands. The cyclic testing program showed that this material would liquefy when subjected to earthquake motion. It was therefore decided to excavate this material and compact it back into place as described in Section 2.5.4.5. Section 2.5.4.2.1.3 presents the results of normal shear tests on the remolded channel area soils. The more important cohesion value is conservatively taken as 1200 psf; 24

friction angle is assigned an average value of 15 degrees. The two values are the same as the undisturbed values for the in situ clay above elevation 680.

A dynamic 2-dimensional finite element analysis for earthquake design Case 2 was performed on the intake slopes using in situ test results in Section 2.5.4.2.1.3. By inspection, design Case 2 controls for seismic analysis and is the only case considered. The finite element analysis considered the soil to behave as an elastic medium with a constant damping of 10 percent of critical. The soil deposit was modeled from the centerline of the intake channel to a distance of 350 feet beyond the crest of the intake channel side slope and from the top of the soil deposit to a fixed boundary at bedrock. Earthquake motion is input into the model at this fixed boundary at bedrock. The four artificial time histories discussed in Section 3.7 were each in turn used as input directly into the base of the model. Acceleration profiles for the intake channel were then prepared for the accelerations produced by each of the four records. An average acceleration was then calculated from the four profiles for use as seismic coefficients and taking into account the location of the various failure planes to be investigated by a pseudostatic approach (see Figure 2.5-237). Accordingly, for a failure plane at elevations 650.0 (at or directly above bedrock) to 665.0, a seismic coefficient of 0.30 g was calculated for the SSE. Similarly, a seismic coefficient of 0.40 g was calculated for the SSE for a failure plane at elevation 680.0 24

Pseudostatic wedge analyses were performed for earthquake design Case 2 using seismic coefficients obtained from the finite element analysis, to determine the lateral extent of excavation required to obtain a minimum factor of safety equal to or greater than 1.0. The soil properties used in the pseudostatic wedge analyses are the same as in the static analyses except the liquefiable sand is assumed to have no strength (Figure 2.5-237).

Section A-A, Figure 2.5-237 shows the wedge failure at elevation 650 which has a factor of safety equal to 1.12. The factor of safety increases for wedges considered further behind the crest of the slope.

Section B-B, Figure 2.5-237 shows the wedge failure at elevation 665. The factor of safety is 1.04 if the failure plane is in the firm gravel and is 1.57 if the failure plane occurs in the replacement material. The factor of safety increases for wedges considered further behind the crest of the slope. As a result of these analyses, excavation will be made down to firm gravel and laterally back to the point directly below the crest of the slope, and then to the surface with a slope not steeper than one vertical on 1.5 horizontal as shown in Figure 2.5-239.

exploration program determined that the lowest bedrock elevation occurred near the mouth of the channel downstream side at elevation 650. In addition, a program of cyclic triaxial testing of the sandy material and static testing of the clay material, under R conditions (saturated, consolidated, undrained) in both cases, was instituted. Sandy material from two representative locations were tested at TVA's Singleton Materials Laboratory. The results of that testing are presented in Table 2.5-26. The results of static R testing on the clay material are shown in Table 2.5-27. The results of the exploration and testing program were evaluated to determine the need for additional analysis. These results indicated a probable liquefaction of the sand layer during a seismic event. In addition, the strength properties of the clay layer were too low to stabilize overlying slopes. Additional analyses have been made to determine new limits of excavation to top of rock for the downstream side of the channel extending from the reservoir to approximately halfway to the pumping station.

Additional Analyses

As outlined above, additional stability analyses were made for those portions of the downstream side of the channel with bedrock elevations ranging from 656 (approximately halfway to the reservoir) to 650 at the reservoir end of the channel. The analyses assumed that the excavated material would be compacted and placed as fill in the same manner as that used in other areas of the intake channel. The strength properties of the remolded material are = 15° and c = 1200 psf, the same values used in the original analysis, as determined by tests on the remolded soil. 28
The liquefiable material adjoining the remolded slopes is assumed to have no strength. The most critical design case has been established above to be that for sudden drawdown plus an SSE, for which the minimum factor of safety is 1.0. Therefore, the results presented below are for that case only.

From bedrock elevation 660 to elevation 656 the limits of excavation will be as shown in Figure 2.5-256. The factor of safety for a wedge failure along a plane at 656 is 1.12. The slope is therefore stable against failure by sliding.

Figure 2.5-257 shows the limits of excavation for a section with a bedrock elevation of 650. The factor of safety for a wedge failure along a plane at elevation 650 is 1.0. This factor of safety is considered adequate, since it was computed with the use of extremely conservative assumptions. As shown on Figure 2.5-257, the factor of safety was computed assuming that the entire zone of sandy material extending from elevation 680 to 650 liquefies completely during a seismic event. This is a very conservative assumption. Furthermore, the assumption has been made that no shear strength exists along the failure plane where it passes through the sandy zone; again, this is a very

conservative assumption. Even a small amount of shear strength in the liquefiable zone along the failure plane would make the safety factor greater than 1.0.

The final configuration of the rockfill side slopes at the reservoir end of the intake channel, as discussed in Section 2.5.5.2.1, are also affected by the unexpected soild conditions encountered. On the upstream side of the mouth of the intake channel the firm gravel layer will be left in place and rockfill placed on top of it from elevation 665 to 695. On the downstream side the rockfill will be placed on bedrock down to elevation 650.

Figure 2.5-258 shows a typical cross section of the rockfill slopes on the upstream side of the channel. The factor of safety against sliding along a plane at elevation 665 is 1.5.

The downstream side of the channel with rockfill placed on a bedrock elevation of 650 is shown in Figure 2.5-259. The factor of safety for a wedge failure at 650 is 1.30, and the slope is therefore stable.

2.5.5.2.3 <u>Design Criteria and Analysis for the Underground Barrier for the ERCW Pipeline and 1E Conduit Alignment</u>

The location of the underground barrier is shown on Figures 2.5-580 and -581. The underground barrier was analyzed for the following cases: 50

Case	Required Factor of Safety
1. Safe Shutdown Earthquake, but prior to liquefaction	1.0
2. Safe Shutdown Earthquake after liquefaction, but prior to dissipation of pore water pressure	1.0

Section 2.5.4.6 describes the study made to determine the design groundwater for the piping and conduit alignments. The results of that groundwater study were used in the analyses of the underground barrier. As discussed in Section 2.5.4.6, the groundwater level was revised to reflect a 25-year groundwater. The influence of this slightly higher groundwater on the analysis of the underground barrier was discussed with the NRC staff. The staff indicated they concur with our judgement that the higher groundwater will have a negligible effect on the results of the stability analysis, thus not requiring any additional evaluation of the stability of the underground barrier.

Figure 2.5-583 shows a loading diagram of how the underground

barrier was analyzed. Seven sections of the barrier were analyzed. Figure 2.5-582 shows the locations of the seven sections. The most critical sections were Section 1 for Trench A and Sections 6 and 7 for Trench B. Case 2 is the controlling case in the analysis for each section, since passive earth pressure is included in Case 1, but assumed to be zero for Case 2. Figure 2.5-583 shows the results of the analysis. Case 1 was dropped from the analysis, when it became obvious that Case 2 controlled the design and analysis of the barrier. Due to the urgency to complete the construction of the barriers prior to fuel load, the trench excavation was started prior to completion of the laboratory testing of the backfill soils. The barrier width was based on assumed design soil properties. The results of the evaluation of the initial laboratory shear strength tests showed that the design cohesion was approximately half the needed cohesion to stabilize the barrier. To eliminate the need to widen the barrier, additional laboratory shear strength tests were made on backfill soils remolded to a higher level (100% Standard Compaction ASTM D 698) of compaction. The results of this testing showed that the cohesion was increased sufficiently to allow the barrier to be stable. The test results are presented in Section 2.5.4.5.1.

Since it was not necessary for the entire barrier to be constructed at the higher compaction level (100%), additional analyses were made to determine what elevation the lower compaction level (95%) could be used. The results of this analysis are given on Figure 2.5-583. Figure 2.5-584 shows the final grading for the area of the underground barrier. 50

2.5.5.3 Logs of Borings

Refer to Section 2.5.4.3 for the location of all in situ soil borings. Refer to Section 2.5.1.2.6 for the location of all rock borings.

2.5.5.4 Compaction Specifications

The compaction specification for earth and rock fills are discussed in Sections 2.5.4.5.1.3 and 2.5.4.5.2.2 respectively.

2.5.6 Embankments

There are no embankments at the site which are used for plant flood protection or for impouding cooling water required for the operation of the nuclear power plant.

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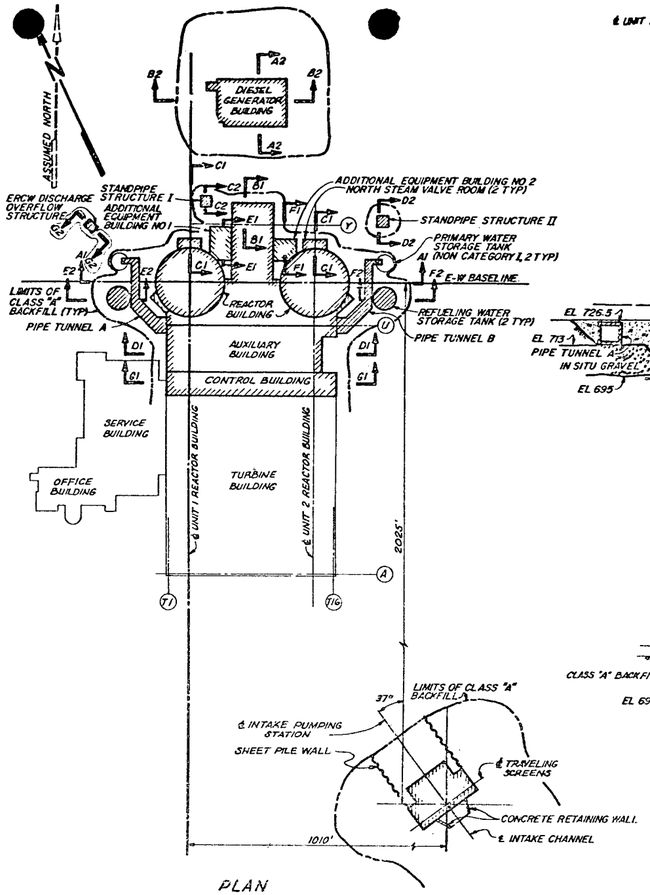
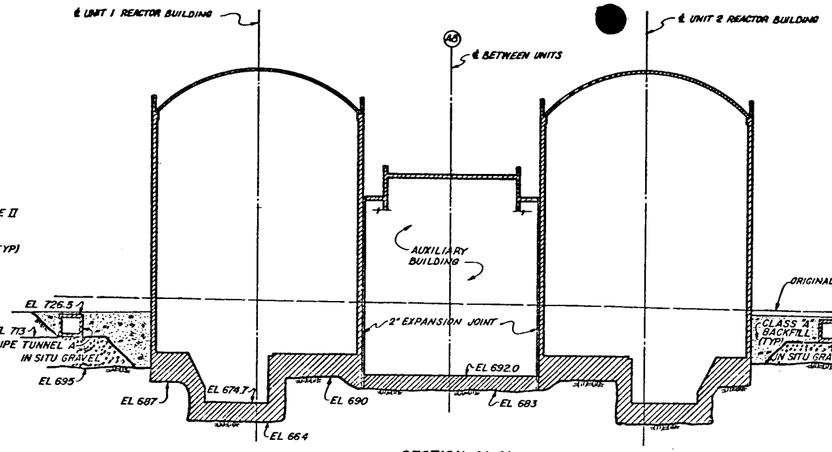
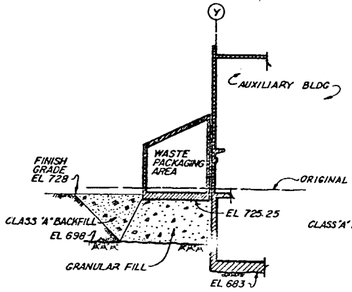


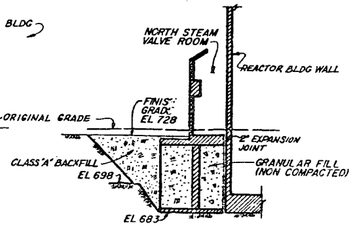
PLAN
SHOWING LOCATION OF CATEGORY I STRUCTURES



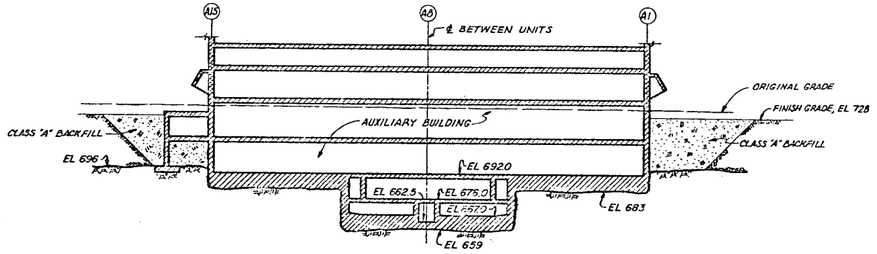
SECTION A1-A1



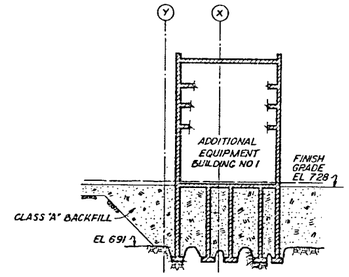
SECTION B1-B1



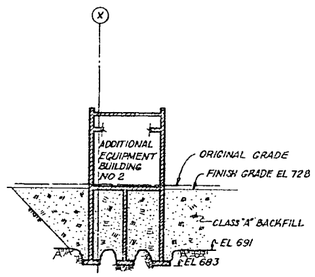
SECTION C1-C1



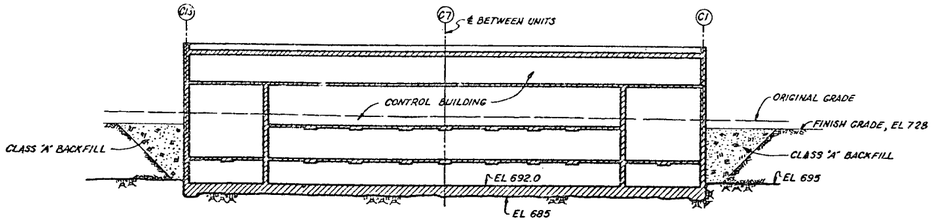
SECTION D1-D1



SECTION E1-E1



SECTION F1-F1



SECTION G1-G1

PRC APERTURE CARD

Also Available On Aperture Card

LEGEND:

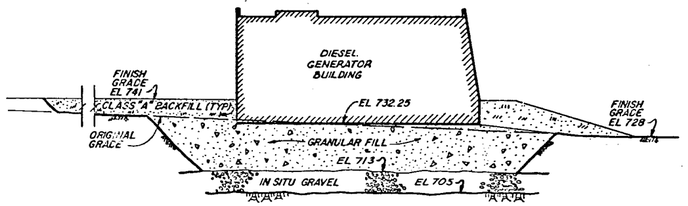
- CATEGORY I STRUCTURES
- SOUND ROCK
- EXISTING EARTH
- CLASS "A" BACKFILL
- GRANULAR FILL
- IN SITU GRAVEL

Revised by Amendment 48

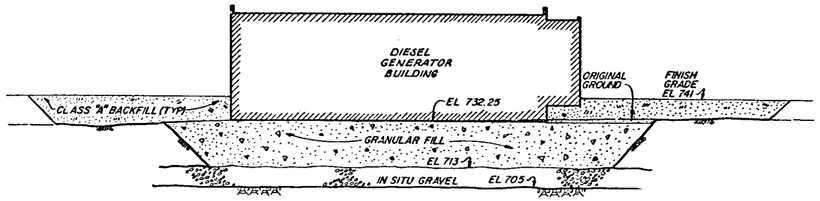
WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

MAIN PLANT
EXCAVATION AND BACKFILL
CATEGORY I STRUCTURES SHEET 1
TVA DWG NO. 10W335 R2
FIGURE 2.5-225

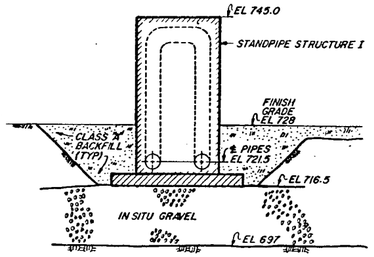
8307010284-07



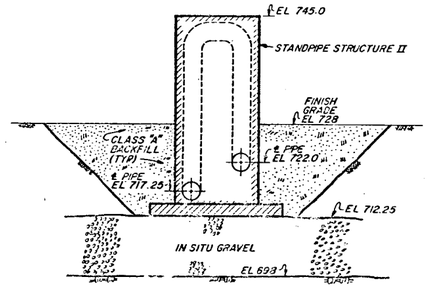
SECTION A2-A2



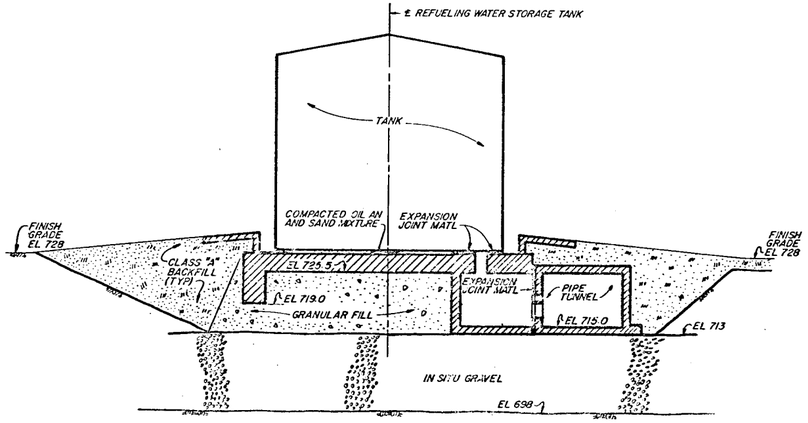
SECTION B2-B2



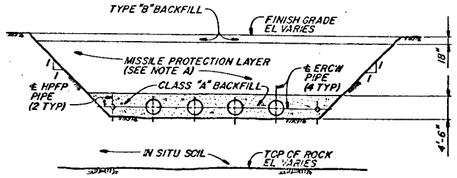
SECTION C2-C2



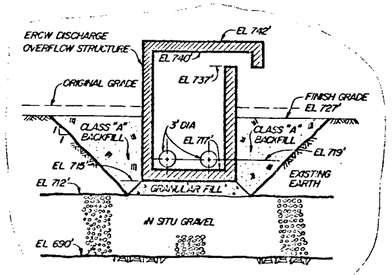
SECTION D2-D2



SECTION E2-E2
SECTION F2-F2 (OPP HAND)



TYPICAL SECTION
ERCW AND HPFP PIPING



SECTION G2-G2

NOTE A:
MISSILE PROTECTION LAYER MAY CONSIST OF: MINIMUM OF 10 FT OF TYPE "B" BACKFILL, 7 FT OF CRUSHED STONE OR 18" THICK CONCRETE SLAB.

Also Available On Aperture Card

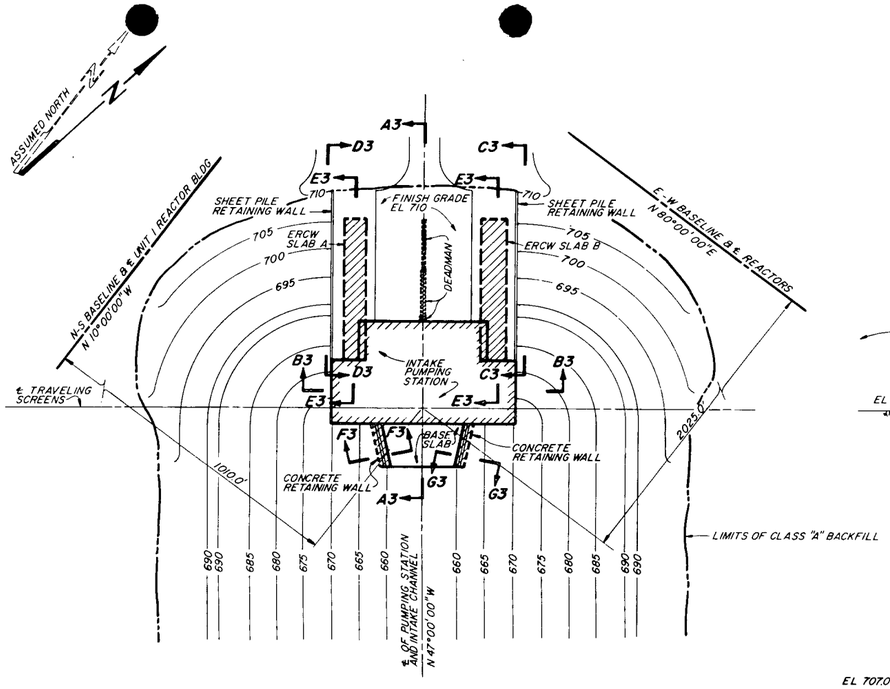
PRC APERTURE CARD

Revised by Amendment 48

WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
MAIN PLANT EXCAVATION AND BACKFILL CATEGORY I STRUCTURES SHEET 2 TVA DWG NO. 10W336 R2 FIGURE 2.5-226

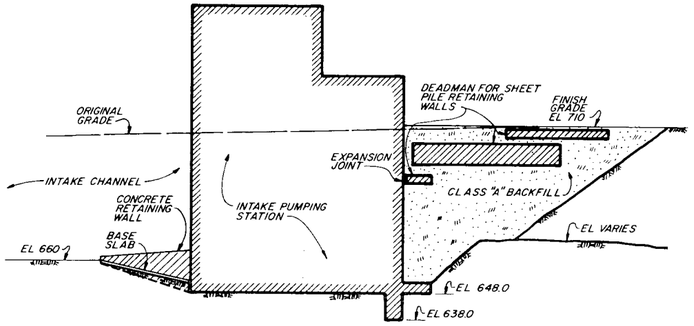
FSAR

8307010284-D8

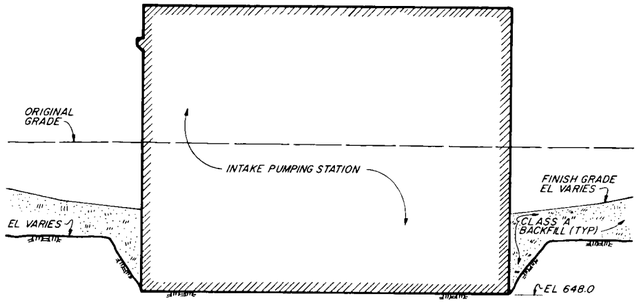


PLAN

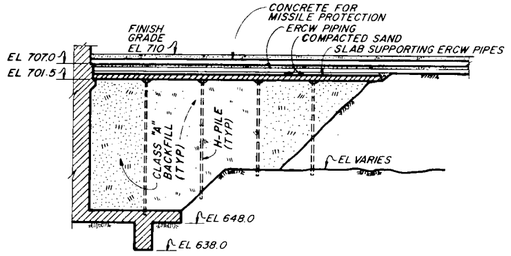
1"=40' 0 40 80



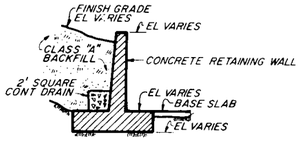
SECTION A3-A3



SECTION B3-B3



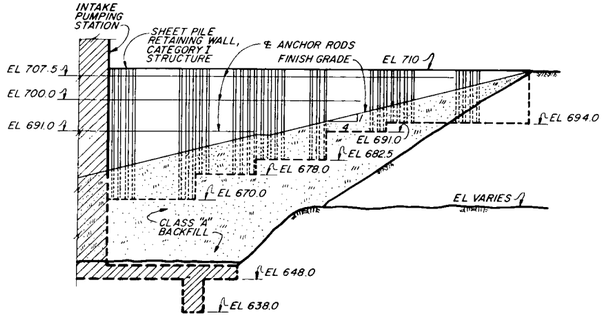
SECTION E3-E3



SECTIONS F3-F3 & G3-G3

NTS

NOTE:
FOR LEGEND SEE 10W335



SECTION C3-C3
SECTION D3-D3 (OPP HAND)

NTS

Revised by Amendment 45

1"=20' 0 20 40
EXCEPT AS NOTED

COMPANION DRAWINGS:
10W335 & 10W336

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

EXCAVATION AND BACKFILL
CATEGORY I STRUCTURES
SHEET 3
TVA DWG NO. 10W337 R0
FIGURE 2.5-226a

EXCAVATION NOTES:

1. A STOCKPILE AREA FOR FINE GRAINED CLAYS AND SILTS, AND A SEPARATE STOCKPILE AREA FOR THE SANDS AND SILTY SANDS ARE TO BE ESTABLISHED. SEPARATE STOCKPILES FOR EACH TRENCH MAY BE ESTABLISHED AT THE OPTION OF THE FIELD. AS EACH TRENCH IS EXCAVATED THE FINE GRAINED CLAYS AND SILTS ARE TO BE VISUALLY SEPARATED FROM THE SANDS AND SILTY SANDS AND DISTRIBUTED TO THE APPROPRIATE STOCKPILE. THE STOCKPILE AREAS ARE TO BE ESTABLISHED IN A MANNER THAT WILL ALLOW DRAINAGE OF THE STOCKPILED MATERIAL IN ORDER THAT IT CAN BE RECLAIMED FOR BACKFILL. THE SURFACES OF THE STOCKPILE AREAS ARE TO BE GRADED TO PREVENT PONDING AND TO MINIMIZE INFILTRATION OF RAINFALL AND RUNOFF.
2. MATERIAL ENCOUNTERED IN THE EXCAVATION THAT WAS PREVIOUSLY SPOILED DURING PLANT CONSTRUCTION SHALL BE SPOILED IN A NEW LOCATION.
3. BASEL GRAVEL MAY BE ENCOUNTERED BELOW THE SANDS AND SILTY SANDS IN MANY AREAS OF THE TRENCH EXCAVATION. THE BASEL GRAVEL SHALL BE SPOILED.
4. EACH TRENCH IS TO BE EXCAVATED TO THE WEATHERED SHALE (SAPROLITE). THE EXCAVATION IS TO BE CARRIED INTO THE WEATHERED SHALE TO A DEPTH WHERE THE SAPROLITE MATERIAL EXHIBITS ROCK-LIKE CHARACTERISTICS SUCH AS BEDDING STRUCTURE AND JOINTS. THE DEPTH OF EXCAVATION INTO THE WEATHERED SHALE SHALL BE DETERMINED BY A QUALIFIED SOILS INSPECTOR. EXCAVATION TO SOUND, UNWEATHERED ROCK IS NOT REQUIRED.
5. BEST MANAGEMENT PRACTICE FOR RUNOFF SHOULD BE USED FOR STOCKPILES AND SPOILPILE AREAS.
6. EACH TRENCH SHALL BE DEWATERED AND MAINTAINED IN A MANNER THAT WILL ALLOW THE EXCAVATION AND PLACEMENT OF EARTHFILL TO BE DONE IN AN ENVIRONMENT SUFFICIENTLY DRY TO COMPLY WITH THE MOISTURE CONTENT REQUIREMENTS OF EARTHFILL NOTE 3. DEWATERING IS ALSO NECESSARY TO MAINTAIN THE STABILITY OF EXCAVATION SLOPES AND ADJACENT STRUCTURES AND FEATURES. THE DEWATERING TECHNIQUES USED BY CONST MUST BE EFFECTIVE AND RELIABLE AND MEET THE APPROVAL OF EN DES. PARTICULAR CARE SHALL BE TAKEN TO PREVENT THE MOVEMENT, MIGRATION, FLOW, SLUMPING, OR LOSS OF THE SANDS AND SILTY SANDS IN THE TRENCH EXCAVATIONS. PROGRESSIVELY GRADED REVERSE FILTERS OF SANDS, GRAVELS, AND/OR CRUSHED STONE, AS APPROPRIATE, SHALL BE IMMEDIATELY PLACED OVER ANY SANDY MATERIALS THAT EXHIBIT TENDENCIES FOR MOVEMENT, MIGRATION, FLOW, SLUMPING, OR LOSS.
7. PRIOR TO PLACEMENT OF ANY BACKFILL, THE SURFACE OF THE WEATHERED SHALE SHALL BE REASONABLY WELL CLEANED OF ANY SOIL OR LOOSE DEBRIS AND/OR ANY ROCK OVER 4": THAT MAY REMAIN AFTER THE EXCAVATION PROCESS. AIR OR WATER SHALL NOT BE USED IN THE CLEANUP OF THE WEATHERED SHALE SURFACE.
8. THE PROCESS OF EXCAVATING INTO THE WEATHERED SHALE TO THE SPECIFIED DEPTH, CLEANING THE SURFACE, AND PLACEMENT OF THE GRANULAR MATERIAL AS SPECIFIED IN EARTHFILL NOTE 1 SHALL BE KEPT AS SHORT AS REASONABLE TO PREVENT DETERIORATION OF THE WEATHERED SHALE SURFACE.

EARTHFILL NOTES:

AFTER THE TRENCH HAS BEEN EXCAVATED TO THE SPECIFIED DEPTH (EXCAV NOTE 4) THE FOLLOWING STEPS SHALL BE TAKEN TO BACKFILL EACH TRENCH:

1. PLACE AND COMPACT A MINIMUM OF 12 INCHES OF GRANULAR MATERIAL MEETING THE REQUIREMENTS OF SECTION 1075 (BOTTOM LAYER) OF GENERAL CONSTRUCTION SPECIFICATION T-1. THE FOLLOWING GRADATION IS ALSO ACCEPTABLE.

SQUARE SIEVE SIZE	PERCENT PASSING BY WEIGHT
1-1/2 INCHES	100
3/4 INCH	30-75
3/8 INCH	5-15
NO. 4	0-5

THE GRANULAR MATERIAL SHALL BE PLACED IN MAXIMUM 10 INCH LOOSE LIFTS AND COMPACTED WITH A MINIMUM OF 6 COMPLETE PASSES BY A DYNAPAC CA25 VIBRATORY ROLLER, OR AN EN DES APPROVED EQUAL.

2. EARTHFILL TO FILL THE TRENCHES SHALL BE OBTAINED FROM STOCKPILES AND BORROW AREAS APPROVED BY EN DES. THE PURPOSE OF THE BACKFILLING SEQUENCE PROVIDED BELOW IS TO PLACE THE SANDS AND SILTY SANDS AT A HIGHER ELEVATION AND AT A HIGHER DENSITY THAN THEY NATURALLY EXIST. THE MATERIAL FOR BACKFILLING THE TRENCHES SHALL BE OBTAINED FROM THE FOLLOWING SOURCES IN THE ORDER SHOWN.
   (a) MATERIAL FROM THE STOCKPILE OF FINE-GRAINED CLAYS AND SILTS ESTABLISHED DURING THE TRENCH EXCAVATION. THIS MATERIAL SHOULD BE DISTRIBUTED UNIFORMALLY AND COMPACTED ALONG THE LENGTH OF THE TRENCH.
   (b) MATERIAL FROM THE STOCKPILE OF SANDS AND SILTY SANDS, ESTABLISHED DURING THE TRENCH EXCAVATION. THIS MATERIAL SHOULD BE DISTRIBUTED UNIFORMALLY AND COMPACTED ALONG THE LENGTH OF THE TRENCH.
   (c) MATERIAL FROM APPROVED BORROW AREAS MAY BE USED TO SUPPLEMENT ANY ADDITIONAL MATERIAL NEEDED FOR FILLING THE TRENCHES.
   (d) MATERIAL FOR BACKFILLING TRENCH A SHALL BE OBTAINED FROM TRENCH A STOCKPILE, BORROW AREAS 9,10, AND 2C, AND MATERIAL FROM REGRADING FUTURE 161 KV SWITCHYARD.
   (e) MATERIAL FOR BACKFILLING TRENCH B SHALL BE OBTAINED FROM TRENCH B STOCKPILE, BORROW AREAS 12(13) AND 2C, AND MATERIAL FROM REGRADING FUTURE 161 KV SWITCHYARD.

   A MINIMUM OF 10 FEET OF FINE GRAINED MATERIAL FROM CATEGORIES (a) AND (c) ABOVE SHALL BE PLACED BEFORE MATERIAL FROM CATEGORY (b) CAN BE PLACED.
3. EARTHFILL SHALL BE UNIFORMLY COMPACTED IN LAYERS WHICH WHEN COMPACTED DO NOT EXCEED A THICKNESS OF 6 INCHES. COMPACTION SHALL BE ACCOMPLISHED WITH A TAMPING (SHEEPSFOOT) ROLLER (REV FACTOR 3-50, OR AN EN DES APPROVED EQUAL). TWO EARTHFILL TYPES ARE DEFINED AS FOLLOWS:
   (a) TYPE A: EARTHFILL COMPACTED TO AT LEAST 95% OF MAXIMUM DRY DENSITY AS DETERMINED BY ASTM D698 (STANDARD PROCTOR).
   (b) TYPE A1: EARTHFILL COMPACTED TO AT LEAST 100% OF THE MAXIMUM DRY DENSITY AS DETERMINED BY ASTM D698.

   MOISTURE CONTENT OF ALL EARTHFILL SHALL BE WITHIN ± 3% OF OPTIMUM MOISTURE CONTENT.
4. EARTHFILL PLACEMENT INSTRUCTIONS:
   (a) GENERAL: THE DATUM FOR FILL PLACEMENT SHALL BE A PLANE SURFACE CONNECTING THE TOP OF SHALE ON OPPOSING TRENCH SIDEWALLS AS DETERMINED BY THE SOILS INSPECTOR. IN CASES WHERE IRREGULARITIES IN THE TRENCH BOTTOM RESULT IN LESS THAN THE REQUIRED DEPTH OF TYPE A1 EARTHFILL NOTIFY EN DES GEOLOGY AND GEOTECHNICAL ENGINEERING GROUP OF THE CIVIL ENGINEERING SUPPORT BRANCH (CEB). SUCH CASES WILL BE EVALUATED ON AN INDIVIDUAL BASIS AND VERBAL INSTRUCTIONS PROVIDED TO THE FIELD.
   (b) TRENCH A:
      (1) TYPE A1 EARTHFILL SHALL BE PLACED FROM THE GRANULAR 1075 MATERIAL TO A LINE 10 FEET ABOVE THE DATUM.
      (2) TYPE A EARTHFILL SHALL BE USED FROM THE TOP OF THE TYPE A1 MATERIAL TO FINAL GRADE.
   (c) TRENCH B: EARTHFILL PLACEMENT INSTRUCTIONS FOR TRENCH B ARE A FUNCTION OF THE DEPTH FROM FINAL GRADE TO THE DATUM.
      (1) FOR 20 FEET OR LESS: TYPE A1 EARTHFILL SHALL BE PLACED FROM THE GRANULAR 1075 MATERIAL TO A LINE 5 FEET ABOVE THE DATUM. THE REMAINDER OF THE TRENCH SHALL BE TYPE A EARTHFILL.
      (2) FOR A DEPTH GREATER THAN 20 FEET BUT LESS THAN 25 FEET: TYPE A1 EARTHFILL SHALL BE PLACED FROM THE GRANULAR 1075 MATERIAL TO A LINE 10 FEET ABOVE THE DATUM. THE REMAINDER OF THE TRENCH SHALL BE TYPE A EARTHFILL.
      (3) FOR A DEPTH GREATER THAN 25 FEET BUT LESS THAN 45 FEET: GRANULAR MATERIAL MEETING THE REQUIREMENTS OF NOTE 1 SHALL BE PLACED TO WITHIN 25 FEET OF FINAL GRADE. TYPE A1 EARTHFILL SHALL BE PLACED FROM A DEPTH OF 25 FEET TO 15 FEET. THE REMAINDER OF THE TRENCH SHALL BE TYPE A EARTHFILL.
      (4) FOR A DEPTH GREATER THAN 45 FEET: CONTACT THE GEOLOGY AND GEOTECHNICAL ENGINEERING GROUP IN CEB FOR SPECIAL INSTRUCTIONS.
5. IN-PLACE DRY DENSITY TESTS USING THE SAND CONE (ASTM D1556) OR RUBBER BALLOON (ASTM D2167) TEST METHODS SHALL BE MADE AT A RATE OF 1 TEST FOR EACH 2000 CUBIC YARDS OF EARTHFILL PLACED (IN PLACE VOLUME). BLOCK SAMPLES SHALL BE OBTAINED AS OUTLINED IN SECTION 11.3 OF GENERAL CONSTRUCTION SPECIFICATION G-9, EXCEPT THAT THE MINIMUM REQUENCY OF SAMPLING SHALL CONFORM TO EACH OF THE FOLLOWING:
   (a) ONE SAMPLE SHALL BE TAKEN FOR EACH 50,000 CUBIC YARDS OF FILL PLACED THROUGHOUT THE COURSE OF THE WORK.
   (b) ONE SAMPLE SHALL BE TAKEN FOR EACH 20 DAYS OF FILL PLACING THROUGHOUT THE COURSE OF THE WORK.
   (c) A MINIMUM OF THREE SAMPLES SHALL BE TAKEN IN EACH TRENCH. A MINIMUM OF ONE OF THESE THREE SAMPLES IN EACH TRENCH SHALL BE TAKEN IN THE SAND OR SILTY SAND (SEE BACKFILL NOTE 2b) IF MORE THAN 10,000 CUBIC YARDS ARE PLACED. A MINIMUM OF ONE OF THESE THREE SAMPLES IN EACH TRENCH SHALL BE TAKEN FROM THE FILL COMPACTED TO 100% OF MAXIMUM DRY DENSITY.
6. EXCEPTIONS AND SUBSTITUTIONS TO THE ABOVE MATERIAL OR PLACEMENT SEQUENCE ARE:
   (a) GRANULAR MATERIAL MEETING THE REQUIREMENTS OF SECTION 1032 OF GENERAL CONSTRUCTION SPECIFICATION T-1 MAY BE USED IN LIEU OF ANY OF THE ABOVE EARTHFILL MATERIALS. THE GRANULAR MATERIAL SHALL BE PLACED IN A MAXIMUM LOOSE LIFT THICKNESS OF 10 INCHES AND UNIFORMALLY COMPACTED WITH A VIBRATORY ROLLER TO AN AVERAGE RELATIVE DENSITY OF 85% OR GREATER FOR ALL TESTS, WITH A MINIMUM OF 80% RELATIVE DENSITY FOR INDIVIDUAL TESTS AS DETERMINED BY ASTM D2049 PROCEDURES.

      THE MOISTURE CONTENT SHALL BE ADJUSTED AS NECESSARY TO ASSURE ADEQUATE COMPACTION. IN-PLACE DENSITY TESTS USING THE SAND CONE (ASTM D1556) OR RUBBER BALLOON (ASTM D2167) OR NUCLEAR MOISTURE-DENSITY GAUGE (ASTM D2922 AND D3017) TEST METHODS SHALL BE MADE AT A RATE OF 1 PER EVERY 500 CUBIC YARDS OF GRANULAR MATERIAL PLACED WITH A MINIMUM OF ONE TEST EACH DAY THE MATERIAL IS PLACED. COMPLETE DOCUMENTATION OF QUANTITY AND LOCATIONS WHERE THE MATERIAL WAS USED SHALL BE RECORDED AND SUBMITTED TO EN DES FOR REVIEW WITH THE MONTHLY FILL QUALITY CONTROL REPORTS REQUIRED BY G-9.
   (b) EARTHFILL FROM BORROW AREAS APPROVED FOR USE IN THE TRENCHES BY EN DES MAY BE SUBSTITUTED FOR ANY OF THE MATERIALS EXCAVATED FROM THE TRENCHES AND STOCKPILED FOR USE AS BACKFILL.

Added by Amendment 50

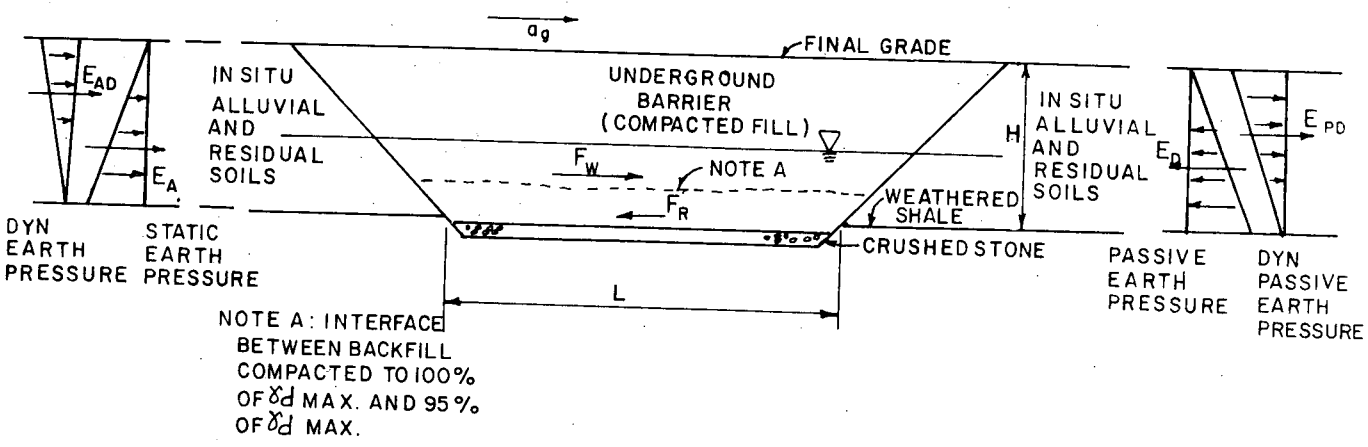
FIRST ISSUE FOR ECN 3960

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

YARD
UNDERGROUND BARRIERS FOR
POTENTIAL SOIL LIQUEFACTION
TVA DWG NO. 10N213-2 R2
FIGURE 2.5-581

FIGURE 2.5-583
WATTS BAR NUCLEAR PLANT
REMEDIAL TREATMENT FOR POTENTIAL SOIL LIQUEFACTION
STABILITY ANALYSIS SUMMARY

LOAD DIAGRAM



ANALYSIS CASES

CASE	DESCRIPTION
I	DURING EARTHQUAKE BUT PRIOR TO LIQUEFACTION (PASSIVE PRESSURE ASSUMED TO ACT)
II	DURING EARTHOUAKE BUT AFTER LIQUEFACTION (NO PASSIVE PRESSURE ASSUMED)

F_R - SLIDING RESISTANCE DUE TO THE SHEAR STRENGTH OF THE COMPACTED FILL ($FR = \Sigma N_{EFF} \tan \phi + CL$)

E_A - EARTH PRESSURE * $= \frac{\gamma H^2 Ka}{2}$

E_P - PASSIVE EARTH PRESSURE * $= \frac{\gamma H^2 K_P}{2}$

FACTOR OF SAFETY

$$FS = \frac{F_R + (E_P - E_{PD})}{E_A + E_{AD} + F_W} \geq 1.0$$

$$FS = \frac{F_R}{E_A + E_{AD} + F_W} \geq 1.0$$

F_W - HORIZONTAL SEISMIC FORCE CAUSED BY THE ACCELERATION OF THE UNDERGROUND BARRIER.

E_{AD}- DYNAMIC EARTH PRESSURE*

E_{PD}- DYNAMIC PASSIVE EARTH PRESSURE*

* INCLUDES WATER PRESSURE

MATERIAL PROPERTIES

	UNIT WEIGHTS (PCF)			R TEST (NAT'L MOISTURE)		R TEST (SATURATED)	
	γ_M	γ_{SAT}	γ_{SUB}	ϕ	C(TSF)	ϕ	C(TSF)
IN SITU MATERIALS							
ALLUVIAL CLAYS AND SILTS	120	123	61	28°	0.4	14°	0.2
ALLUVIAL SANDS							
PRIOR TO LIQUEFACTION	119	124	62	28°	0.4	14°	0.2
AFTER LIQUEFACTION	-	120	58	-	-	0°	0
BASEL GRAVEL	120	130	68	-	-	30°	0
COMPACTED FILL (BORROW MATERIALS)							
@ 95% $\gamma_{D_{MAX}}$							
TRENCH A	117	126	64	-	-	15°	0.1
TRENCH B	117	126	64	-	-	15°	0.1
@ 100% $\gamma_{D_{MAX}}$							
TRENCH A	123	130	68	-	-	14°	0.25
TRENCH B	123	130	68	-	-	14°	0.35
				Q TEST		R & S TEST	
CRUSHED STONE				ϕ	C(TSF)	ϕ	C(TSF)
1032 SECTION MATERIAL	135	143	81	39°	1.0	40°	0.5
1075 SECTION MATERIAL	135	143	81	40°	0	40°	0

UNDERGROUND BARRIER ANALYSIS SUMMARY

	CASE I[1]	CASE II[4]	CASE II[5]
TRENCH A			
SECTION 1	1.7	1.1	1.8
SECTION 2	(SEE NOTE 2)	1.2	1.5
SECTION 3	1.4	1.4	1.7
SECTION 4	1.6	1.2	2.0
TRENCH B			
SECTION 5	1.8	1.1	1.3
SECTION 6	2.6	1.5	1.04
SECTION 7	NM	1.04[6]	1.04

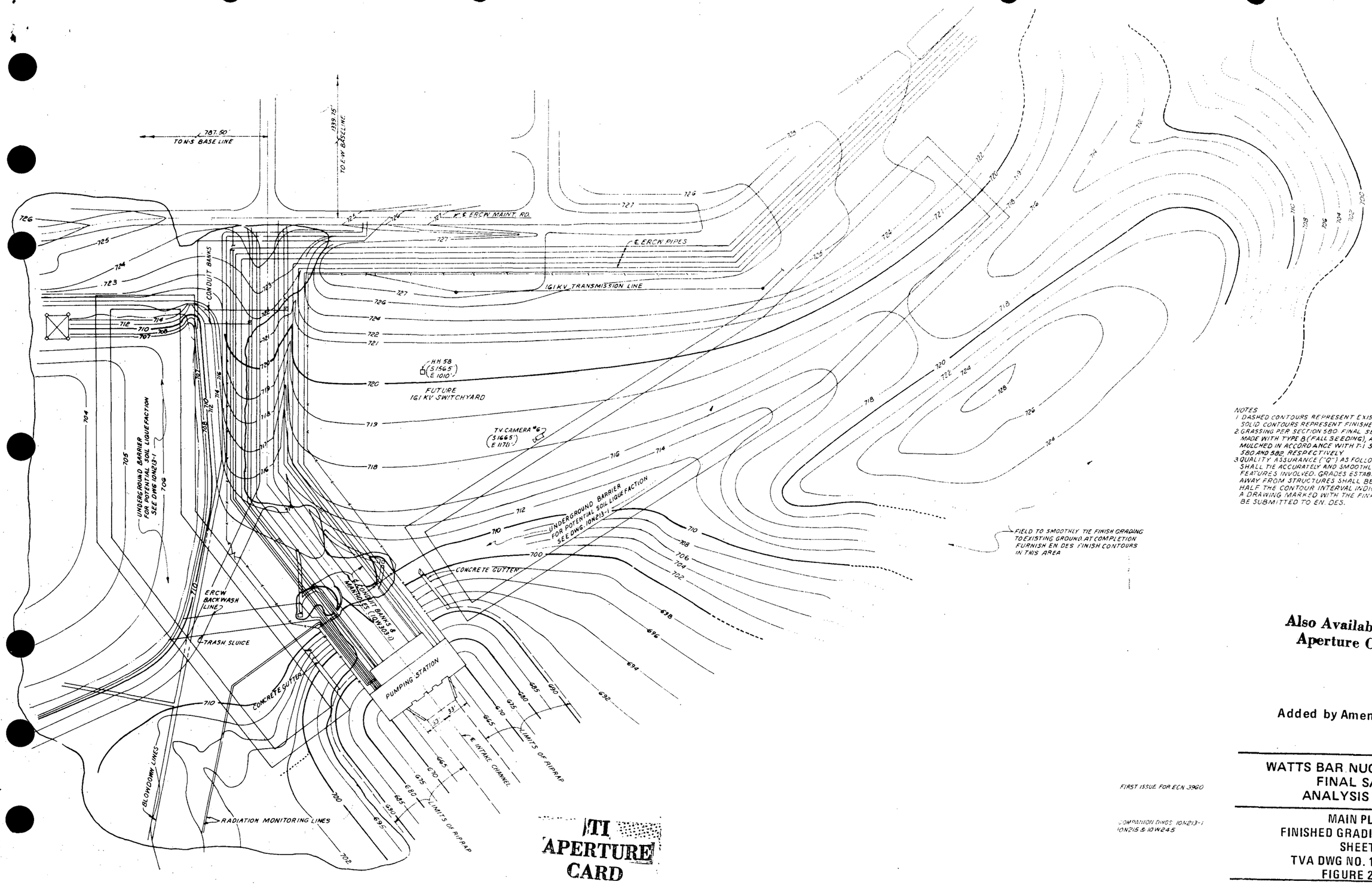
NOTES:

1. CASE I ANALYSIS WAS DONE USING ASSUMED SOIL PROPERTIES FOR THE BACKFILL (ϕ = 14° C = 0.2 TSF) AND EXISTING GRADES. RESULTS OF THE ANALYSIS OF CASE II USING THE SAME BACKFILL PROPERTIES IDENTIFIED CASE II AS THE CONTROLLING CASE. CASE II WAS THEN REDONE USING ACTUAL TEST RESULTS.
2. THE REGRADING OF THE FUTURE SWITCHYARD WAS NECESSARY TO STABILIZE SECTION 2.
3. NM - NOTE MADE
4. SLIDING PLANE AT TOP OF CRUSHED STONE.
5. SLIDING PLANE AT INTERFACE OF 95%/100% $\gamma_{D_{MAX}}$ COMPACTED FILL.
6. SECTION 7 WAS ALSO CHECKED AT THE CRUSHED STONE/WEATHERED SHALE INTERFACE DUE TO THE THICK SECTION OF 1075 MATERIAL PLACED. THE FACTOR OF SAFETY WAS 1.02.

Added by Amendment 50

Also Available On Aperture Card

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT
REMEDIAL TREATMENT FOR POTENTIAL SOFT
LIQUEFACTION STABILITY ANALYSIS
SUMMARY
LOAD DIAGRAM
FIGURE 2.5-583



NOTES
1. DASHED CONTOURS REPRESENT EXISTING GROUND, SOLID CONTOURS REPRESENT FINISHED GRADE.
2. GRASSING PER SECTION 580. FINAL SEEDING SHALL BE MADE WITH TYPE B (FALL SEEDING), AND FERTILIZED AND MULCHED IN ACCORDANCE WITH T.I SPECIFICATIONS 580 AND 582 RESPECTIVELY
3. QUALITY ASSURANCE ("Q") AS FOLLOWS: FINISHED GRADES SHALL TIE ACCURATELY AND SMOOTHLY TO ALL STRUCTURAL FEATURES INVOLVED. GRADES ESTABLISHED FROM CONTOURS AWAY FROM STRUCTURES SHALL BE ACCURATE WITHIN HALF THE CONTOUR INTERVAL INDICATED FOR THE AREA. A DRAWING MARKED WITH THE FINAL CONTOURS SHALL BE SUBMITTED TO EN. DES.

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FIRST ISSUE FOR ECN 3960

COMPANION DWGS. 10N213-1
10N215 & 10W245

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

MAIN PLANT
FINISHED GRADING & PAVING
SHEET 8
TVA DWG NO. 10W245-1 R1
FIGURE 2.5-584

8405080183-80

9.4.3.2.2 Building Cooling System (Chilled Water)

The purpose of the building cooling system is to supplement the general ventilation system and to maintain temperatures at less than the design maximum in the general spaces of the Auxiliary Building. The cooling system consists of two 100-percent-capacity packaged water chillers rated at 400 tons nominal capacity each, two 100-percent-capacity primary loop circulating pumps designed for 800 gpm at 70 ft. head each, two 100-percent capacity secondary loop circulating pumps designed for 800 gpm at 100 feet head each, six fan-coil type air handling units, and associated piping, ductwork, and controls. 52

Primary and secondary chilled water circulating loops are designed for mixing supply and return water to obtain a variable coil inlet temperature normally 47°F to 72°F to minimize unnecessary heat removal. A primary loop pump provides circulation of water through the water chiller. The secondary loop pump circulates chilled water to air intake heating/cooling coils and also to the six air handling units located in various areas where ventilation air alone is not sufficient to maintain the 104°F maximum space temperature.

The twelve heating/cooling coils located in the building air intake at El 737.0 are designed to cool a total of 200,000 cfm of outside air from 97°F to 85°F when supplied with 72°F chilled water, or to heat the outside supply air from 0°F to 60°F when supplied with 240°F hot water.

The locations and capacities of the chilled water air handling units are as follows:

Unit	El.	Air (cfm)	Water (gpm	Capacity (Btuh)
1A	737.0	12,750	46	237,000
1B	713.0	21,900	81	415,000
1C	692.0	5,800	16	77,000
2A	737.0	12,750	46	237,000
2B	713.0	16,800	62	322,000
2C	692.0	5,850	16	77,000

52

The chilled water system is designed for manual startup with automatic mixing of primary and secondary loop flows by means of thermostatically controlled two-way control valves. Flow to heating/cooling coils and to air handling units is individually controlled at each terminal unit by three-way modulating control valves. The seasonal changeover from heating to cooling or from cooling to heating is done by the manual operation of system changeover valves located in the mechanical equipment rooms on El. 737.0.

9.4.3.2.3 Safety Feature Equipment Coolers

The safety feature equipment coolers are described in Section 9.4.5.3.

9.4.3.2.4 Shutdown Board Room Air-Conditioning System

Shutdown board rooms are located on El 757 of the Auxiliary Building with a firewall separating Units 1 and 2 equipment. The electrical boards for either unit can provide the service necessary for the safe shutdown of both plant units following an accident in either unit.

Each of the four fan-coil units is designed to cool 34,000 scfm from 76.5°F DB and 63.5°F WB to 53.5°F DB and 52.4°F WB when supplied with 225 gpm of chilled water at 42°F. The total coil capacity per unit is 878,800 Btuh (73.2 tons) minimum. Each unit contains a single centrifugal fan driven by a nominal 60-hp motor to operate against 7.0-inch water gauge external static pressure. The water chiller has the capacity to cool a minimum of 450 gpm of water from 52°F to 42°F when supplied with 560 gpm maximum of essential raw cooling water 85°F, and is rated at 2,250,000 Btuh (187.5 tons). 52

Environmental control for the Auxiliary Control Room is maintained by the shutdown board room air-conditioning system. The four shutdown board room air-conditioning units are arranged so that any one of the units can provide the necessary cooling required by the Auxiliary Control Room. Four unit heaters provide heating as required to maintain the design ambient conditions. Each shutdown board room air-conditioning system is connected to an emergency power source as well as a source of cooling water that will be available under all conditions. The Shutdown Board Room Air-Conditioning System is designed to meet Safety Class 2b and Seismic Category I requirements.

In the improbable event that both shutdown board room air-conditioning units serving one subarea are inoperable, an arrangement is available for cooling the shutdown board and Auxiliary Control Room from the standby system of the control building air-conditioning system. Diverting dampers will be manually adjusted to obtain this emergency air supply.

The centrifugal pressurizing fans are each designed to supply 1000 cfm against 2.5-inch water gauge static pressure, and each is driven by a nominal 3/4-hp motor. They maintain the shutdown board rooms at a slight positive pressure with respect to the surrounding areas. 52

Each of the two air-conditioning units and each of the two pressurizing air supply fans serving one set of shutdown board rooms are powered by different power trains.

TABLE OF CONTENTS

TABLE OF CONTENTS (CONT.)

LIST OF FIGURES

LIST OF FIGURES (Continued)

15.1.9.6 TURTLE

TURTLE is a two-group, two-dimensional neutron diffusion code featuring a direct treatment of the nonlinear effects of xenon, enthalpy, and Doppler. Fuel depletion is allowed.

TURTLE was written for the study of azimuthal xenon oscillations, but the code is useful for general analysis. The input is simple, fuel management is handled directly, and a boron criticality search is allowed.

TURTLE is further described in Reference 17 .

15.1.9.7 TWINKLE

The TWINKLE program is a multi-dimensional spatial neutron kinetics code, which was patterned after steady state codes presently used for reactor core design. The code uses an implicit finite-difference method to solve the two-group transient neutron diffusion equations in one, two and three dimensions. The code uses six delayed neutron groups and contains a detailed multi-region fuel-clad-coolant heat transfer model for calculating pointwise Doppler and moderator feedback effects. The code handles up to 2000 spatial points, and performs its own steady state initialization. Aside from basic cross-section data and thermal-hydraulic parameters, the code accepts as input basic driving functions such as inlet temperature, pressure, flow, boron concentration, control rod motion, and others. Various edits are provided, e.g. channelwise power, axial offset, enthalpy, volumetric surge, pointwise power, and fuel temperatures.

The TWINKLE Code is used to predict the kinetic behavior of a reactor for transients which cause a major perturbation in the spatial neutron flux distribution.

TWINKLE is further described in Reference 18 .

15.1.9.8 WIT

WIT is a one-region neutron kinetics program with a single axial lump description of thermal kinetics making it useful in the analysis of transients in a heterogeneous reactor core consisting of fuel rods, fuel rod clad, and water moderator and coolant. The code is basically a core model and therefore generally useful for fast reactivity transients which terminate before there is significant feedback from the remainder of the plant, i.e. transients shorter than the loop transit time. 53

WIT is used in safety analysis of reactivity accidents from a subcritical condition.

WIT is further described in Reference 19 .

15.1.9.9 PHOENIX

The PHOENIX code calculates the individual loop flows, core flow and pump speeds as a function of time subsequent to failure of any number of the reactor coolant pumps. The analysis is based on a momentum balance around each reactor coolant loop and across the reactor core. This momentum balance is combined with the continuity equation, a pump momentum balance and the pump characteristics. Any number of reactor coolant loops are accommodated up to a maximum of 6.

PHOENIX is further described in Reference [20].

15.1.9.10 THINC

The THINC Code is described in Section 4.4.3.1.

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20. Bordelon, F. M., 'Calculation of Flow Coastdown After Loss of Reactor Coolant Pump (PHOENIX Code),' WCAP-7973, September, 1972.

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44

TABLE 15.1-2

SUMMARY OF INITIAL CONDITIONS AND COMPUTER CODES USED

FAULTS	COMPUTER CODES UTILIZED	REACTIVITY COEFFICIENTS ASSUMED: MODERATOR TEMPERATURE (Δk/°F)	REACTIVITY COEFFICIENTS ASSUMED: MODERATOR DENSITY (Δk/gm/cc	REACTIVITY COEFFICIENTS ASSUMED: DOPPLER	INITIAL NSSS THERMAL POWER OUTPUT ASSUMED* (MWt)
CONDITION II					
Uncontrolled RCC Assembly Bank Withdrawal from Subcritical Condition	WIT, FACTRAN	$+1 \times 10^{-5}$	---	See Figure 15.1-5	0
Uncontrolled RCC Assembly Bank Withdrawal at Power	LOFTRAN	---	Figure 15.1-7 and 0.43	lower and upper (1)	3425
RCC Assembly Misalignment	THINC, TURTLE LOFTRAN FRACTRAN	---	Figure 15.1-7	upper (1)	3425
Uncontrolled Boron Dilution	NA	NA	NA	NA	0 and 3425
Partial Loss of Forced Reactor Coolant Flow	PHOENIX, LOFTRAN THINC, FACTRAN	---	Figure 15.1-7	lower (1)	2397 and 3425
Startup of an Inactive Reactor Coolant Loop	MARVEL, THINC	---	0.43	lower (1)	2397
Loss of External Electrical Load and/or Turbine Trip	LOFTRAN	---	Figure 15.1-7 and 0.43	upper (1)	3425
Loss of Normal Feedwater	BLKOUT	---	NA	NA	3579
Loss of Off-Site Power to the Station Auxiliaries (Station Blackout)	BLKOUT	---	NA	NA	3425

(Sheet 1 of 4)

Revised by Amendment 53

The neutron flux response to a continuous reactivity insertion is characterized by a very fast rise terminated by the reactivity feedback effect of the negative Doppler coefficient. This self limitation of the power excursion is of primary importance since it limits the power to a tolerable level during the delay time for protective action. Should a continuous RCCA withdrawal accident occur, the transient will be terminated by the following automatic features of the Reactor Protection System:

1. Source Range High Neutron Flux Reactor Trip - actuated when either of two independent source range channels indicates a neutron flux level above a preselected manually adjustable setpoint. This trip function may be manually bypassed only after an intermediate range flux channel indicates a flux level above a specified level. It is automatically reinstated when both intermediate range channels indicate a flux level below a specified level.

2. Intermediate Range High Neutron Flux Reactor Trip - actuated when either of two independent intermediate range channels indicates a neutron flux level above a preselected manually adjustable setpoint. This trip function may be manually bypassed only after two of the four power range channels are reading above approximately 10 percent of full power and is automatically reinstated when three of the four channels indicate a power level below this value. | 53

3. Power Range High Neutron Flux Reactor Trip (Low Setting) - acutated when two out of the four power range channels indicate a power level above approximately 25 percent of full power. This trip function maybe manually bypassed when two of the four power range channels indicate a power level above approximately 10 percent of full power and is automatically reinstated only after three of the four channels indicate a power level below this value.

4. Power Range High Neutron Flux Reactor Trip (High Setting) - actuated when two out of the four power range channels indicate a power level above a preset setpoint. This trip function is always active.

In addition, control rod stops on high intermediate range flux level (one of two) and high power range flux level (one out of four) serve to discontinue rod withdrawal and prevent actuation of the intermediate range flux level trip and the power range flux level trip, respectively. | 53

15.2.1.2 Analysis of Effects and Consequences

Method of Analysis

This transient is analyzed by two digital computer codes. The WIT-6 3 Code is used to calculate the reactivity transient and

hence the nuclear power transient. This code includes the simulation of six delayed neutron groups and the core thermal and hydraulic feedback equations. The FACTRAN [4] Code is then used to calculate the thermal heat flux transient based on the nuclear power transient calculated by the WIT-6 Code. FACTRAN also calculates the fuel and clad temperatures.

In order to give conservative results for a startup accident, the following assumptions are made concerning the initial reactor conditions:

1. Since the magnitude of the power peak reached during the initial part of the transient for any given rate of reactivity insertion is strongly dependent on the Doppler coefficient, conservative values (low absolute magnitude) as a function of temperature are used. See Section 15.1.6 and Table 15.1-2.

2. Contribution of the moderator reactivity coefficient is negligible during the intial part of the transient because the heat transfer time between the fuel and the moderator is much longer than the neutron flux response time. However, after the initial neutron flux peak, the succeeding rate of power increase is affected by the moderator reactivity coefficient. A conservative value of +1 pcm/°F, which is appropriate for beginning of core life at hot zero power, is used in the analysis to yield the maximum peak heat flux. This value is conservative since the moderator coefficient in a rodded core is expected to be negative. See Section 15.1.6 and Table 15.1-2.

3. The reactor is assumed to be at hot zero power. This assumption is more conservative than that of a lower initial system temperature. The higher initial system temperature yields a larger fuel-water heat transfer coefficient, larger specific heats, and a less negative (smaller absolute magnitude) Doppler coefficient all of which tend to reduce the Doppler feedback effect thereby increasing the neutron flux peak. The initial effective multiplication factor is assumed to be 1.0 since this results in the worst nuclear power transient.

4. Reactor trip is assumed to be initiated by power range high neutron flux (low setting). The most adverse combination of instrument and setpoint errors, as well as delays for trip signal actuation and rod cluster control assembly release, is taken into account. A 10 percent increase is assumed for the power range flux trip setpoint raising it from the nominal value of 25 percent to 35 percent. Previous results, however, show that rise in the neutron flux is so rapid that the effect of errors in the

trip setpoint on the actual time at which the rods are released is negligible. In addition, the reactor trip insertion characteristic is based on the assumption that the highest worth RCCA is stuck in its fully withdrawn position. See Section 15.1.5 for RCCA insertion characteristics. 53

5. The maximum positive reactivity insertion rate assumed is greater than that for the simultaneous withdrawal of the combination of the two sequential control banks having the greatest combined worth at maximum speed (45 inches/minute). Control rod drive mechanism design is discussed in Section 4.2.3. 53

6. The initial power level was assumed to be below the power level expected for any shutdown condition. The combination of highest reactivity insertion rate and lowest initial power produces the highest peak heat flux.

Results

The calculated sequence of events for this accident is shown on Table 15.2-1.

Figures 15.2-1 through 15.2-3 show the transient behavior for the indicated reactivity insertion rate with the accident terminated by reactor trip at 35 percent nominal power. This insertion rate is greater than that for the two highest worth sequential control banks, both assumed to be in their highest incremental worth region. It is also greater (by more than a factor of 10) than the maximum insertion rate of the part length RCCA's.

Figure 15.2-1 shows the nuclear power transient. The nuclear power overshoots the full power nominal value but this occurs for only a very short time period. Hence, the energy release and the fuel temperature increases are relatively small. The heat flux response, of interest for DNB considerations, is shown on Figure 15.2-2. The beneficial effect of the inherent thermal lag in the fuel is evidenced by a peak heat flux less than the full power nominal value. There is a large margin to DNB during the transient since the rod surface heat flux remains below the design value, and there is a high degree of subcooling at all times in the core. Figure 15.2-3 shows the response of the average fuel, cladding, and coolant temperature. The average fuel temperature increases to a value lower than the nominal full power value.

15.2.1.3 Conclusions

In the event of a RCCA withdrawal accident from the subcritical condition, the core and the Reactor Coolant System are not adversely affected, since the combination of thermal power and

the coolant temperature result in a DNBR greater than the limiting value of 1.30. Thus, there will be no cladding damage and no release of fission products to the Reactor Coolant System as a result of DNB. This conclusion regarding the DNBR is based on the fact that the analysis shows that nominal full power values for the coolant average temperature, heat flux, and fuel pellet average temperature are not exceeded.

15.2.2 UNCONTROLLED ROD CLUSTER CONTROL ASSEMBLY BANK WITHDRAWAL AT POWER

15.2.2.1 Identification of Causes and Accident Description

Uncontrolled rod cluster control assembly (RCCA) bank withdrawal at power results in an increase in the core heat flux. Since the heat extraction from the steam generator lags behind the core power generation until the steam generator pressure reaches the relief or safety valve setpoint, there is a net increase in the reactor coolant temperature. Unless terminated by manual or automatic action, the power mismatch and resultant coolant temperature rise would eventually result in DNB. Therefore, in order to avert damage to the fuel clad the Reactor Protection System is designed to terminate any such transient before the DNBR falls below 1.30.

The automatic features of the Reactor Protection System which prevent core damage following the postulated accident include the following:

1. Power range neutron flux instrumentation actuates a reactor trip if two out of four channels exceed an overpower setpoint.

2. Reactor trip is actuated if any two out of four ΔT channels exceed an overtemperature ΔT setpoint. This setpoint is automatically varied with axial power imbalance, coolant temperature and pressure to protect against DNB.

3. Reactor trip is actuated if any two out of four ΔT channels exceed an overpower ΔT setpoint. This setpoint is automatically varied with axial power imbalance to ensure that the allowable heat generation rate (kw/ft) is not exceeded.

4. A high pressurizer pressure reactor trip actuated from any two out of four pressure channels which is set at a fixed point. This set pressure is less than the set pressure for the pressurizer safety valves.

5. A high pressurizer water level reactor trip actuated from any two out of three level channels which is set at a fixed point.

Following reactor trip, the plant will approach a stabilized condition at hot standby; normal plant operating procedures may then be followed. The operating procedures would call for operator action to control RCS boron concentration and pressurizer level using the CVCS, and to maintain generator level through control of the main or auxiliary feedwater system. Any action required of the operator to maintain the plant in a stabilized condition will be in a time frame in excess of ten minutes following reactor trip. 30

15.2.14.3 Conclusions

Results of the analysis show that spurious safety injection with or without immediate reactor trip presents no hazard to the integrity of the Reactor Coolant System.

DNBR is never less than the initial value.

If the reactor does not trip immediately, the low pressurizer pressure reactor trip will be actuated. This trips the turbine and prevents excess cooldown thereby expediting recovery from the incident. 53

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TABLE 15.2-1

TIME SEQUENCE OF EVENTS FOR CONDITION II EVENTS

Accident	Events	Time (sec.)
Uncontrolled RCCA Withdrawal from a Subcritical Condition		
	Initiation of uncontrolled rod withdrawal 90 pcm/sec reactivity insertion rate from 10^{-13} of nominal power	0
	Power range high neutron flux low setpoint reached	9.2
	Peak nuclear power occurs	9.3
	Peak clad temperature occurs	10.1
	Peak heat flux occurs	10.1
	Peak average fuel temperature occurs	11.2
	Rods begin to fall into core	9.7

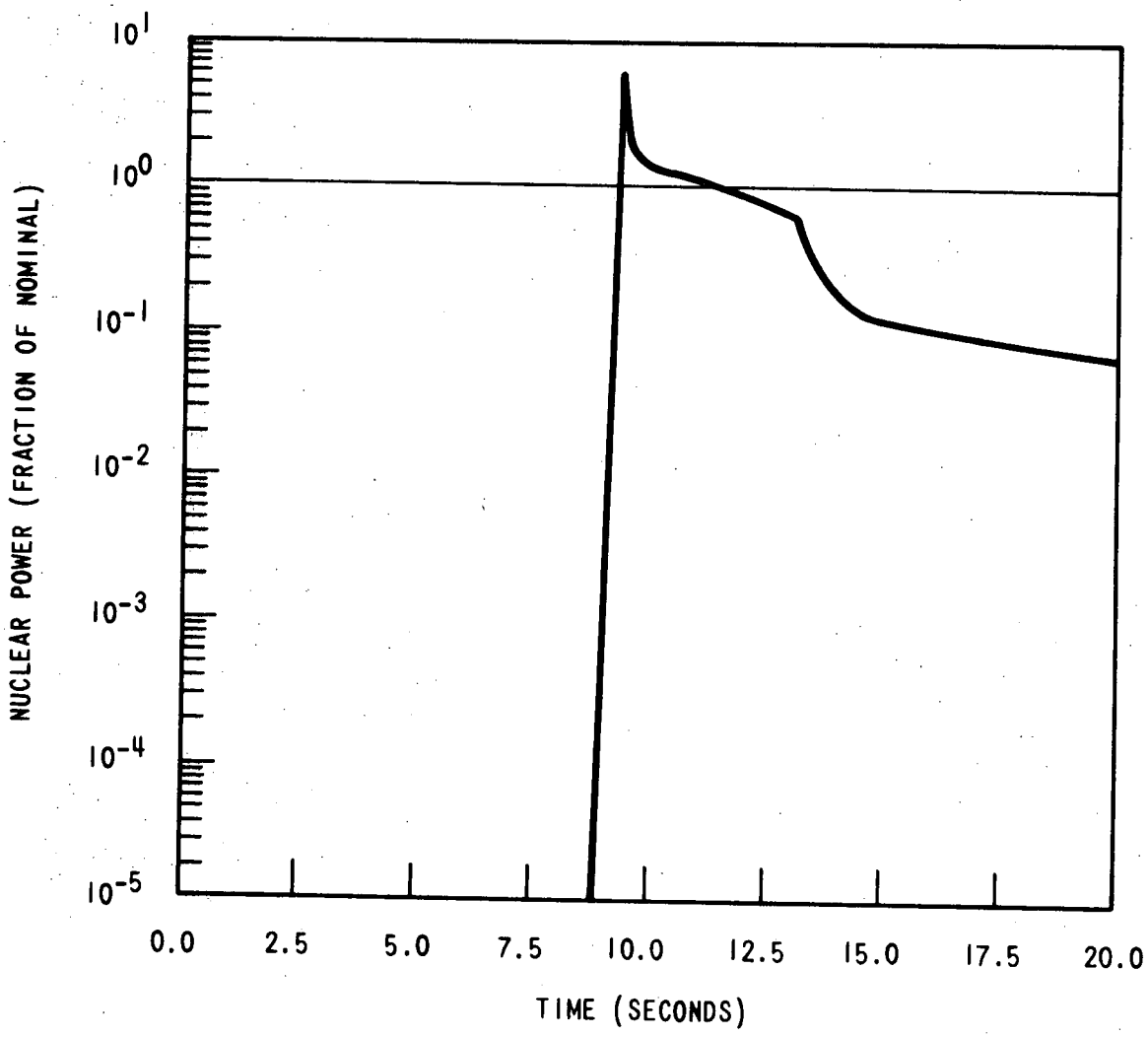

Figure 15.2-1 Uncontrolled Rod Withdrawal from a Subcritical Condition
Nuclear Power Versus Time

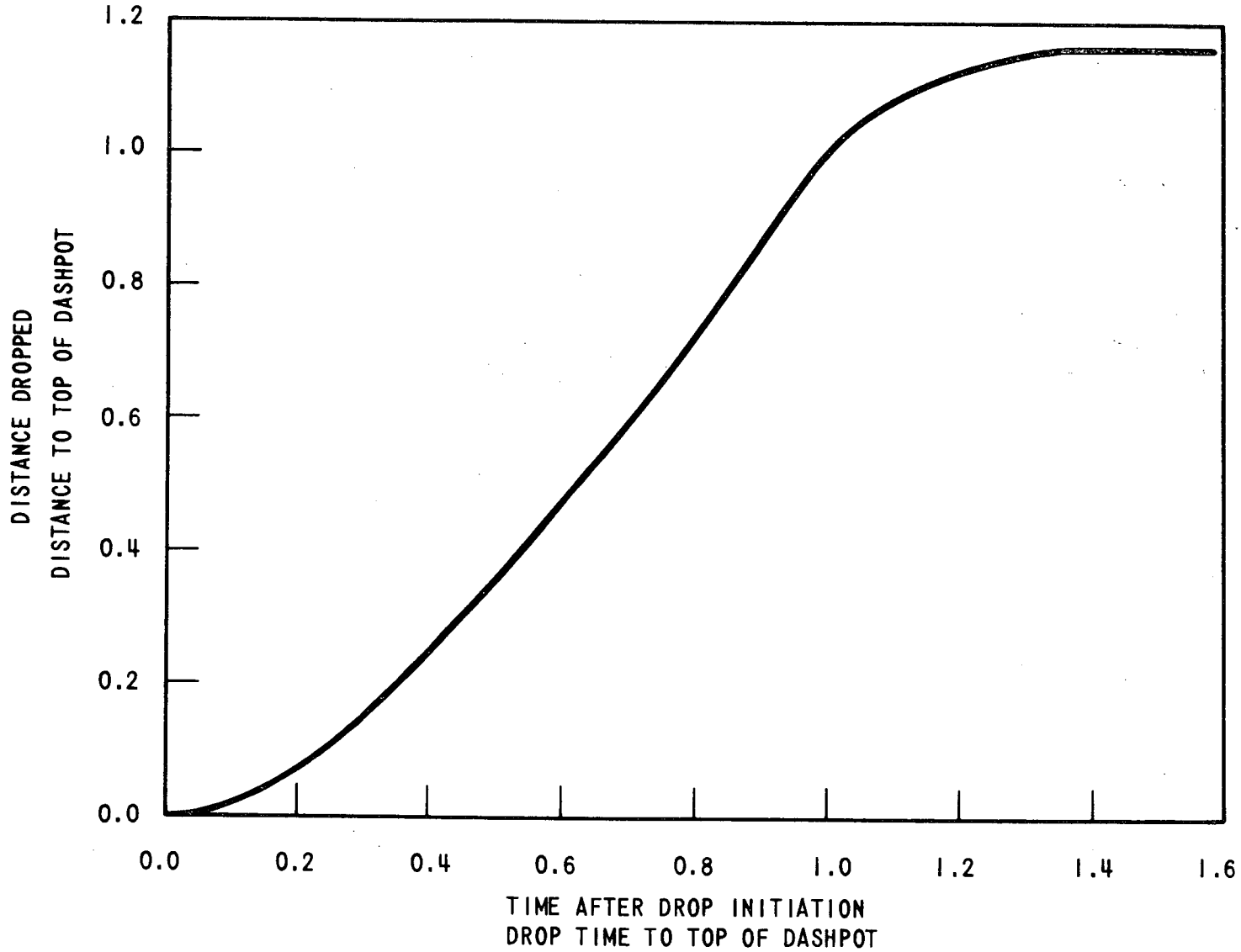

Figure 15.1-2

Revised by Amendment 31

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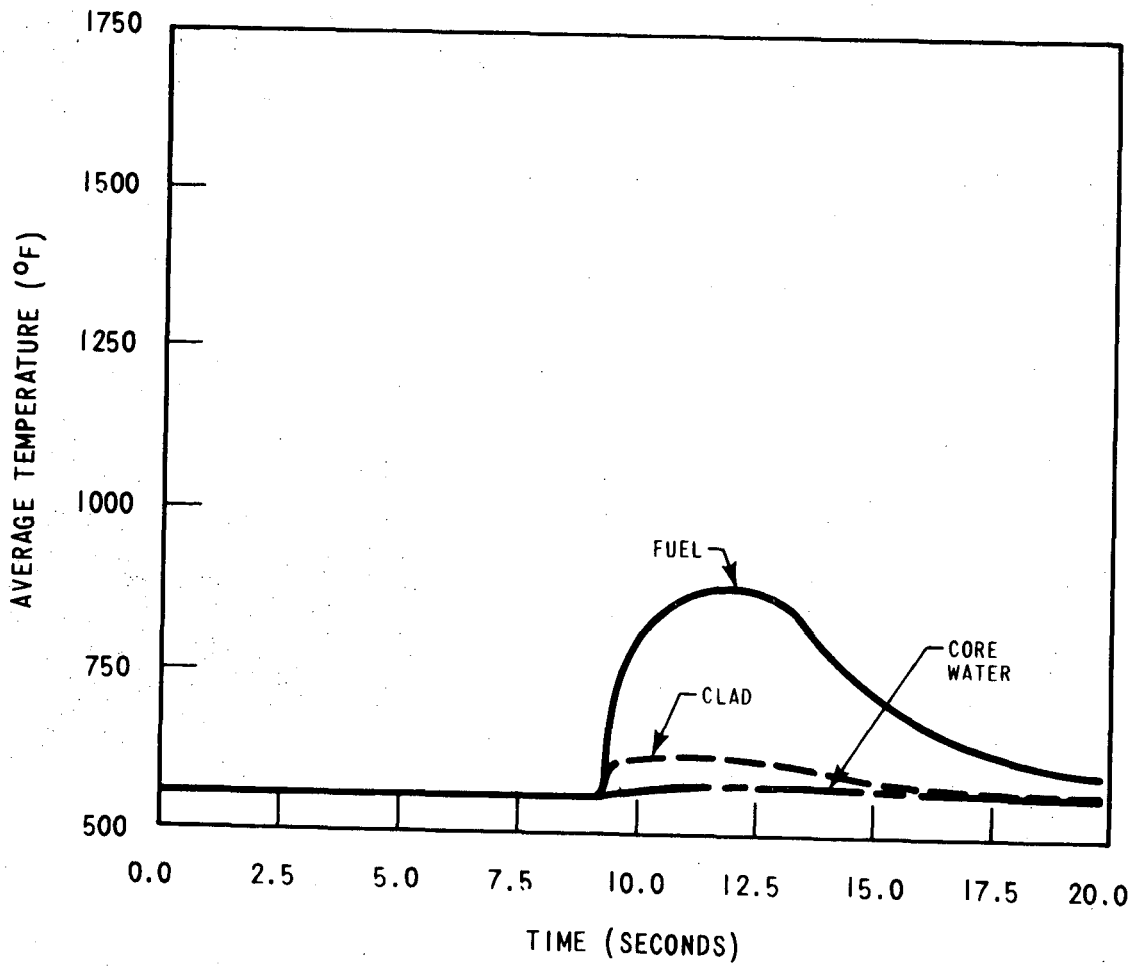

Figure 15.2-3 Uncontrolled Rod Withdrawal from a Subcritical Condition, Temperature Versus Time, Reactivity Insertion Rate 90×10^{-5} $\Delta K/Sec$