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TENNESSEE VALLEY AUTHORITY

CHATTANOOGA, TENNESSEE 37401

November 10, 1972

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Mr. John F. O'Leary, Director
Directorate of Licensing
United States Atomic Energy Commission
Washington, DC 20545

Dear Mr. O'Leary:

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

Submitted herewith is Amendment 18 to the TVA application for units 1 and 2 of the Watts Bar Nuclear Plant. Amendment 18 consists of the results of postulated potential dam failure flooding which was noted in the staff safety evaluation and miscellaneous PSAR editorial changes.

Instructions for incorporating the new and revised material into the preliminary safety analysis report are included with the amendment.

Very truly yours,

J. E. Gilleland

J. E. Gilleland
Assistant to the Manager of Power

Enclosure

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Subscribed and sworn to before me this 10th day of Nov 1972

Carolyn B. Roberts

Notary Public

My Commission Expires 5-7-74

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INSTRUCTIONS FOR AMENDMENT EIGHTEEN PAGE CHANGES

The following instructional information and check list is furnished to help you insert Amendment Number Eighteen into the Watts Bar Nuclear Plant PSAR.

Since in most cases the original PSAR contains information printed on both sides of a sheet of loose leaf paper, a new sheet is furnished to replace sheets containing superseded material. As a result, the front or back of a sheet may contain information that is merely reprinted rather than changed.

Only pages which contain amended (i.e., added, deleted, or revised) information are identified with the cipher "WBNP-18" at the top of the page. Further, where amended information is new or revised, a vertical bar has been inscribed adjacent to the information in the outside margin of the page.

Discard the old sheets and insert the new sheets, as listed below. Keep these instruction sheets in the front of Volume I to serve as a record of changes.

WATTS BAR NUCLEAR PLANT

Amendment No. 18

Revised 11-29-72 LB

INSTRUCTION SHEET

Remove (Front/Back)	Insert (Front/Back)
List of Amendments i/ List of Amendments ii	List of Amendments i/ List of Amendments ii
2.7A-24/2.7A-25	2.7A-24/2.7A-25
-	Insert Appendix 2.7B
7.2-29/7.2-30	7.2-29/7.2-30
Table 14.5-11(b)/14.5-11(c)	Table 14.5-11(b)/14.5-11(c)
E.2-7/E.2-8	E.2-7/E.2-8
E.2-13/E.2-14	E.2-13/E.2-14
Table E.2-1	Table E.2-1
Q2.11.3-1	Q2.11.3-1
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	Q2.11.3-16
	Q2.11.3-17
	Q2.11.3-18
	Q2.11.3-19
	Q2.11.3-20
	Q2.11.3-21

Remove (Front/Back)	Insert (Front/Back)
	Table Q2.11.3-1
	Fig. Q2.11.3-1
	Fig. Q2.11.3-2
	Fig. Q2.11.3-3
	Fig. Q2.11.3-4
	Fig. Q2.11.3-5
	Fig. Q2.11.3-6
	Fig. Q2.11.3-7
	Fig. Q2.11.3-8
	Fig. Q2.11.3-9
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	Fig. Q2.11.3-20
	Fig. Q2.11.3-21
	Fig. Q2.11.3-22
	Fig. Q2.11.3-23
	Fig. Q2.11.3-24
	Fig. Q2.11.3-25
	Fig. Q2.11.3-26

50-390/391

RESULTS OF POSTULATED POTENTIAL DAM FLOODING...SUBMITTED AS PART OF AMDT# 18

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Both the Sequoyah and Watts Bar Nuclear Plants are designed using the same earthquake criteria except as noted below. ~~The operating basis earthquake and design basis earthquake ground accelerations used at Watts Bar are identical to those used at Sequoyah.~~ ~~However,~~ There is a difference in the analytical techniques used to show that the structures meet the criteria. The seismic design of Watts Bar structures will be based on dynamic analyses with the structure response being computed by the "response spectrum" method. The seismic design for Sequoyah is based on dynamic analyses with the structure response computed by the "time-history" method.

Insert

The containment penetrations are designed alike for Watts Bar and Sequoyah. The penetrations differ in minor details and pressure ratings, but they basically provide the same degree of protection from mechanical, thermal, and pressure induced loads.

The same containment isolation criteria were applied identically to Sequoyah and Watts Bar for establishing the particular valving arrangements for pipes that penetrate the containment. There are no changes to systems that perform similar functions in the two plants.

Containment testing and reliability for both plants is ensured by following similar procedures and quality assurance programs. Continuing programs will be performed to ensure that deterioration below acceptable standards does not occur and that a high standard of performance is maintained throughout the operating life of both plants.

E.2.6 Engineered Safety Features

The engineered safety features of the Watts Bar and Sequoyah Nuclear Plant will be essentially identical. The Emergency Core Cooling, containment isolation, leakage detection, and containment spray systems of both plants are designed to essentially the same criteria and except for minor capacity differences in the containment spray system necessitated by the

Insert

Both the Sequoyah and Watts Bar Nuclear Plants are designed for the same ground accelerations for the operating basis earthquake and the design basis earthquake. The response spectra are different, with the Watts Bar spectra having peak amplification factors greater than the Sequoyah spectra.

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E.2.13 Initial Tests and Operation

The preoperational and startup tests for both the Watts Bar and Sequoyah Nuclear Plants will be prepared and performed by essentially the same organizations within TVA and Westinghouse. Since the designs of these plants parallel each other so closely, these tests will cover essentially the same systems and require meeting similar acceptance criteria. In all cases the tests will follow the AEC's Guide for the Planning of Preoperational Testing Programs and Guide for the Planning of Initial Startup Programs.

E.2.14 Safety Analysis

For both the Watts Bar and Sequoyah Nuclear Plants, all postulated accidents that would consequently release fission products to the environment were analyzed at an extrapolated core power level of 3582 MWt. Similar accidents were analyzed for both plants. In the Watts Bar safety analysis, advantage was taken of more recent advances in analytical techniques. The consequences of all the accidents were found to be within the 10 CFR 100 reference values. The strong similarity between corresponding systems for the two plants, along with the larger exclusion distance for the Watts Bar plant ([redacted] 3900 feet versus 1920 feet) results in lower offsite doses from postulated accidents at Watts Bar than at Sequoyah.

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TABLE E.2-1

Comparison of Design Parameters for the Watts Bar and Sequoyah Nuclear Plants Containment Vessels

	Sequoyah	Watts Bar
Containment Vessel Internal Design Pressure	10.8 psig	13.5 psig
Initial LOCA Peak Pressure	8.4 psig*	8.4 psig
Long Term LOCA Peak Pressure	12 psig	15 psig
Sub-Compartment Design Pressures		
Reactor Vessel Annulus	100 psig	100 psig
Pipe Sleeve	900 psig	900 psig
Compartment Above Reactor	30 psig	30 psig
Steam Generator Enclosure	17 psig	19 psig
Pressurizer Enclosure	15 psig	15 psig
Compartments Outside Crane Wall In Lower Compartment	12 psig	15 psig
Upper Compartment	12 psig	15 psig
Divider Barrier Pressure Differential	12 psig	12 psig
Upper Crane Wall Differential Pressure		
Near Open End	9 psig	9.2 psig
Remainder	8 psig	8.3 psig
Ice Condenser Compartment	12 psig	15 psig
Upper Compartment Design Temperature	170°F	190°F
Lower Compartment Design Temperature	244°F	250°F
Ice Condenser Design Temperature	240°F	250°F
Operating Conditions		
Pressure	0.3 psig	0.3 psig
Upper Compartment Temperature	110°F	110°F
Lower Compartment Temperature	120°F	120°F
Ice Condenser Temperature	15°F	15°F
18 Mass Release During LOCA	546,440 lbs	516,000 ~~[illegible]~~ lbs
Energy Release During LOCA	348 x 10^6 Btu	351.7 ~~[illegible]~~ x 10^6 Btu 2
Deign Leak Rate	0.25 ~~[illegible]~~ %/day	0.25 ~~[illegible]~~ %/day
Containment Vessel External Design Pressure	0.5 psig	0.5 psig

*SQNP PSAR gave 9.0 psig but later calculations give the value listed.

TABLE 14.5-4

CALCULATED MAXIMUM DIFFERENTIAL PRESSURES ACROSS UPPER CRANE WALL

Peak ΔP (psi)

	Element	7-8-9	10-11-12	13-14-15	16-17-18	19-20-21	22-23-24
17	DECL	7.0	5.8	4.5	4.7	5.9	6.8
	DEHL	8.4	6.8	5.4	5.4	6.9	8.2

TABLE 14.5-5

INITIAL PEAK PRESSURE IN CONTAINMENT

Guillotine Break in Compartment 1

	BREAK	COMMENTS	(PSIG) P_{MAX}
17	DECL	FSAR Data Old TMD	9.8
	DECL	C_D = 1.0 100% Entrainment	12.21
	DEHL	C_D = 1.0 100% Entrainment	13.12
	DEHL	C_D = 1.0 50% Entrainment	11.24

In addition to the above, the following will be tested periodically at shutdown:

1. Reactor Coolant Pump Breakers (open to trip)

2. Manual Trip

The reactor coolant pump breakers cannot be tripped at power without causing a plant upset by loss of power to a coolant pump. However, the reactor coolant pump breaker open trip logic can be tested at power. Manual trip cannot be tested at power without causing a reactor trip since operation of either manual trip switch actuates both Train A and Train B. These two trips, however, only provide
17 backup protection to other trip functions. Note, however, that manual trip could also be initiated from outside the control room by such means as manually tripping the turbine which would then initiate reactor trip, or manually tripping one of the reactor trip breakers.

The pump bus undervoltage, pump bus underfrequency, turbine trip--reactor trip, and safety injection trip cannot be tested at power without possibly causing a plant upset or damage to equipment; however, the reactor protection logic trains for the above reactor trips are ~~designed~~ shall ~~to be tested~~ completely at power. Annunciation is provided in the control room to indicate when a train is in test (which results in the tested train being bypassed), and when a reactor trip breaker is bypassed. Details of the logic system testing are given in WCAP 7672.

The design of the protection system as defined by IEEE 279-1971 will
13 comply with our understanding of the intent of Safety Guide 22, Periodic Testing of Protection System Actuation Functions.

Logic Channel Testability

The general design features and testability of the logic system are described in Reference (1).

INSTANTANEOUS DAM FAILURE COMPUTATIONS - WATTS BAR

	1963 Flood	Summer Levels
Initial conditions		
Bottom elevation	665.0	665.0
Headwater elevation	743.0	741.0
Tailwater elevation	697.0	684.5
Headwater depth, feet	78.0	76.0
Tailwater depth, feet	32.0	19.5
H_B,* feet		
Theory	20.6	23.4
Model	22.0	25.5
V_W,* feet per second		
Theory	46.7	46.1
Model	49.3	49.3
U_B,* feet per second		
Theory	15.2	21.9
Model	23.3	23.1

*H_B is the wave height, V_W is the wave velocity, and U_B is the velocity of the water after the wave front has passed.

For practical use the TVA model accommodates initial flows at time of failure and also failures that are gradual and partial. The theoretical relationships do not. Figure 30 shows in detail the short segment of the maximum possible flood which is influenced by the Watts Bar embankment failure. Both flow and resulting average stages at Watts Bar Dam tailwater and at Watts Bar Nuclear Plant site are shown for a 7-hour period as solved by the refined unsteady flow model.

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In this case of the maximum possible flood, the potential for a bore or wave front created by the gradual erosion followed by the relatively rapid, last stage of failure of the earth embankment was investigated. A bore so generated could strike a ridge on the left bank and a part of it be reflected across the stream toward the nuclear plantsite. At a time just prior to the last stage of failure, the conditions tabulated below would exist.

CONDITIONS PRIOR TO RAPID, LAST STAGE OF FAILURE
OF WATTS BAR EARTH EMBANKMENT
DURING MAXIMUM POSSIBLE FLOOD

Elevation				Depth, Feet				Tailwater
Channel Bottom	Over-bank	Head-Water	Tail-water	Head-water	Tail-water	Over-bank	Q, CFS	Velocity, FPS
665.0	700±	760.9	728.5	95.9	63.5	28.5	1,030,000	8.6

The approximate flow field streamlines just prior to the last stage of failure are shown by solid lines on figure 31. At the end of complete failure of the earth embankment the discharge will have increased rapidly to 1,310,000 cfs. Any bore thus generated is conservatively estimated to spread laterally at about 10 degrees. The approximate streamline pattern for a bore is also shown on figure 31 by arrowed lines. The average bore height at the dam would be at most about 2.5 feet. This height would be reduced as the wave travels downstream and expands. Its height would be about 1 foot when it strikes Blalock Ridge on the left bank. The ridge is heavily wooded and the trees would absorb much of the wave energy. To be conservative, however, a perfect reflection of double the incident amplitude was allowed. Thus a

The local height over the 750-foot length of the failed embankment would be about 12 feet.

QUESTION 2.11.3

The ability of Watts Bar Dam to withstand severe earthquakes without causing a loss of any nuclear power plant safety functions may be demonstrated in either of two ways; either by showing in detail that the dam can withstand severe earthquake induced stresses, or by proving its arbitrarily assumed failure would not cause a loss of safety-related functions. The PSAR indicated the assumption was made to postulate the arbitrary failure of the dam, but a detailed analysis of the effects on safety-related structures and equipment was not presented. Substantiate that sufficient protection will be provided to safety-related structures and equipment to prevent a loss of function due to the static and dynamic effects of the postulated failure.

ANSWER

As a result of the requirement that the Sequoyah Nuclear Plant Final Safety Analysis Report must conform to the AEC proposed "Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants," issued February 1972, with elaboration and clarification by the AEC staff, TVA has completed a detailed flood and seismic analysis for all dams upstream from Sequoyah Nuclear Plant to determine the potential for flooding as a result of seismic failures. This same program also determines the potential for flooding at the Watts Bar Nuclear Plant.

TVA has considered what is thought to be the worst conditions of flooding for which the nuclear plant can be subjected to, both as a result of postulated failure of single dams or combinations of dams.

It should be clearly understood that these studies have been made solely to ensure the safety of the Watts Bar Nuclear Plant against failure by floods caused from excessive rainfall or by the assumed failure of dams due to seismic forces. TVA is of the strong opinion that the chances of the assumed events occurring approach zero probability. But to assure that safe shutdown of the Watts Bar Nuclear Plant is not impaired by flood waters TVA has in these studies added conservative assumptions to conservative assumptions to be able to show that the plant can be safely controlled even in the event that all these unlikely events occur in just the proper sequence.

By furnishing this information TVA does not infer or concede that its dams are inadequate to withstand great floods and/or earthquakes that may be reasonably expected to occur in the TVA region under consideration. TVA has a program of inspection and maintenance carried out on a regular schedule, to keep its dams safe. Instrumentation of the dams to help keep check on their behavior was installed in many of the dams during original construction. Other instrumentation has been added since and is still being added as the need may appear or as new techniques become available. In short, TVA has confidence that its dams are safe against catastrophic destruction by any natural forces that could be expected to occur.

The analysis consisted of the following:

1. Determination of the flood level at the nuclear plant site resulting from one-half the maximum possible flood, as defined in Appendix 2.7A of the PSAR, with the associated flood levels at upstream reservoirs, coincident with an increase in the flood level at the nuclear plant site resulting from the postulated seismic failure of an upstream dam or dams (simultaneously) caused by an Operating Basis Earthquake as defined in Section 2.9.2 of the PSAR.

2. Determination of the flood level at the nuclear plant site resulting from a 25-year flood, with the associated flood levels at upstream reservoirs, coincident with an increase in the flood level at the nuclear plant site resulting from the postulated seismic failure of an upstream dam or dams (simultaneously) caused by a Design Basis Earthquake as defined in Section 2.9.2 of the PSAR.

It is to be noted that the Operating Basis Earthquake identified in Condition I is defined in the PSAR as having a peak horizontal acceleration value of 0.09g at the rock foundation. ~~This is a more severe earthquake than that required by the AEC proposed "Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants" for combining with this flood condition. The AEC guide requires a maximum earthquake based on historic seismicity.~~ remove In Section 2.9.2 of the PSAR, TVA has identified that the maximum intensity which has been felt at the site in the recorded history of the area is probably MM V and certainly no more than MM VI. Horizontal ground acceleration values are 0.015g and 0.03g respectively for these earthquakes. It is to be noted that this is ground surface acceleration and is greater than the accompanying acceleration of the rock surface on which the dams are founded. This comparison simply reiterates the extreme conservatism contained in the analysis.

A summary of the results of the analyses for these two conditions is given in Table Q2.11.3-1. Watts Bar Nuclear Plant and upstream dams are located as shown on Figure Q2.11.3-1.

The summary shows that the effect on downstream dams from flood waves resulting from postulated seismic failure of upstream dams has been investigated, and it has been determined that no downstream dam with reservoirs of significant volume would fail. However, because of a close margin of safety against overtopping in one postulated seismic failure situation, Fort Loudoun embankment was failed arbitrarily to determine the consequences. This, in turn, led to a close margin of safety against failure from overtopping for the Watts Bar embankment, and it also was failed arbitrarily to determine the consequences.

The effect of postulated bridge failure of the spillway gates at Watts Bar and Fort Loudoun Dams is included in the analyses.

A general discussion of the analyses follows.

Concrete Structures

The standard method of computing stability is used. The maximum base compressive stress, average base shear stress, the factor of safety against overturning, and the shear strength required for a shear-friction factor of safety of 1 are determined. To find the shear strength required to provide a safety factor of 1, a coefficient of friction of 0.65 is assigned at the elevation of the base under consideration.

The analyses for earthquake are based on the static analysis method as given in Engineering for Dams by Hinds, Creager, and Justin with increased hydrodynamic pressures determined by the method developed by Jorge I. Bustamante and Armando Flores as contained in their paper entitled "Water Pressure on Dams Subject to Earthquakes," Journal of the Engineering Mechanics Division, ASCE Proceedings, October 1966. These analyses include applying masonry inertia forces and increased water pressure to the structure resulting from the acceleration of the structure horizontally in the upstream direction and simultaneously in an upward direction. The masonry inertia forces are determined by a dynamic analysis of the structure which takes into account amplification of the accelerations above the foundation rock.

No reduction of hydrostatic or hydrodynamic forces due to the decrease of the unit weight of water from the downward acceleration of the reservoir bottom is included in this analysis.

Waves created at the free surface of the reservoir by an earthquake are considered of no importance. In the paper "Hydrodynamic Pressures on Dams During Earthquake," by Anil K. Chopra, Journal of the Engineering Mechanics Division, ASCE Proceedings, December 1967, it is stated that Bustamante and others concluded that surface waves may be neglected without introducing significant error. O. C. Zienkiewicz in Water Power, September 1964, stated that surface waves are normally only possible if catastrophically large displacements of the earth's surface occur, and cannot be generated in any magnitude by tremors alone. It is our judgment that before waves of any significant height have time to develop the earthquake will be over. The duration of earthquake used in this analysis is in the range of 20 to 30 seconds.

The effect of vertically traveling waves will tend to be dampened by the silt accumulation on the reservoir bottom.

Effects of silt on structures is not considered. There is only a small amount of silt now present and the accumulation rate is very slow. Reservoir sedimentation has been measured by TVA for many years, and it has been determined that the annual rate of accumulation of silt in the reservoirs under consideration is very low, varying from a low of 0.026 percent to a maximum of 0.142 percent ("Sedimentation in TVA Reservoirs, TVA Report No. 0-6693", Division of Water Control Planning, February 1968).

Embankment

The standard slip circle analysis is used. The effect of the earthquake is taken into account by applying the appropriate static inertia force to the dam mass within the assumed slip circle.

In the analysis, the embankment design constants used, including the shear strength of the materials in the dam and the foundation, are the same as those used in the original stability analysis.

Although detailed dynamic soil properties are not available, a value for seismic amplification through the soil has been assumed based on previous studies pertaining to TVA's nuclear plants. These studies have indicated maximum amplification values slightly in excess of two for a rather wide range of shear wave velocity to soil height ratios. For these analyses, a straight line variation is used with an acceleration at the top of the embankment being two times the top of rock acceleration.

Hydrologic Routing

Flow as a result of flood and postulated dam failure is routed by steady flow methods in the bulk of the analysis, with unsteady methods being used only downstream from rapid earth embankment failures. Further use of unsteady flow methods is not considered necessary because the margin between plant grade and flood levels (7 feet or more as shown in Table Q2.11.3-1) is more than adequate to account for possible differences in results from the two methods.

Where Watts Bar Dam embankment is overtopped, unsteady flow methods are used for routing through the Watts Bar reservoir.

The customary methods of flood routing, steady or unsteady, (identified below as single unit routing) make no allowance for the possibility that flow in the deeper, smoother channel sections of a stream valley may outrun the flow in the more shallow, less smooth sections of the overflow plain.

In a method of approximate test flows are divided between channel and overbank and a comparable division of valley storage is also made. The channel flow is then routed through its part of the total valley storage, and the overbank flow is routed through its part. The resulting two outflows are then added together for comparison to the single unit routing.

Available backwater curves make it possible to separate total flow into channel and overbank flow in a satisfactory way. A curve plot of these respective flows allows a sensible approximation of the flow division to be made for each routing interval. The flow separation is based on cross section geometry. The valley storage separation is also accomplished in a satisfactory way by typical cross section geometry.

There is a second choice of flow separation at the upstream end of routing, the point immediately below the dam for which failure is postulated, identified below as failure geometry. It consists of a best estimate of where water under the postulated mode of failure will be directed as a result of failure. Both separations are computed through their separate valley storage.

The variation in routing techniques was tested on two reaches totaling 22 miles of the Holston River below Cherokee Dam. Following is a summary of results of the three routing techniques:

	Crest Flow (Cubic Feet Per Second)		
		Separated Routing	
Mile	Single Unit Routing	Failure Geometry	Cross Section Geometry
52.3	1,951,000	1,951,000	1,951,000
40	1,200,000	1,010,000	1,090,000
30	980,000	630,000*	750,000*

*Highest of a broad, double crest.

As can be seen by the above tabulation both separation techniques produced lower routed results than the single unit technique. The cause is clearly explainable. Acceleration influences of the deeper, smoother part of the cross section and drag influence of the more shallow, rougher part of the cross section cause a time separation of the two crests with a resulting lower total crest than with routing accomplished in a single unit.

Despite the lower crests resulting from flow-storage routing separation, TVA believes that the single unit technique is the more correct solution and has used this technique throughout the analysis.

Details of Analysis for Condition 1 (1/2 Maximum Possible Flood + OBE)

Watts Bar Dam

Stability analyses for the powerhouse and spillway sections result in the conclusion that these structures are judged not to fail. The analyses show low stresses with all the spillway base in compression and about 75 percent of the powerhouse base in compression. Results are given in Figure Q2.11.3-2. Dynamic analysis of the concrete structures resulted in the determination that the base acceleration is not amplified at levels above the base.

Slip circle analysis of the earth embankment section results in a factor of safety of 1.52, and the embankment is judged not to fail. Results are given in Figure Q2.11.3-3.

For the condition of peak discharge at the dam for one-half maximum possible flood, the spillway gates are in the wide open position (see Figure Q2.11.3-4) with the bottom of the gates above the water. The forces in the gate members resulting from an OBE, including amplification of acceleration appropriate for the period of the gate, are a small percentage of the forces for which the gates are designed, i.e., forces resulting from the gates being in the closed position with water to the top of the gates.

Analysis of the bridge structure for forces resulting from an OBE, including amplification of acceleration appropriate for the period of the structure, results in the determination that the bridge will not fail in the upstream or downstream direction. In the direction of the bridge span, across the reservoir on top of the dam, the bridge will fail as a result of shearing the anchor bolts connecting the bridge girders to the towers at each end of the spillway. The downstream bridge girders will strike the spillway gates with the upstream girders striking the concrete spillway piers an instant later. The impact of the girders striking the gates will fail the bolts which anchor the gate trunnions to the pier anchorages allowing the gates to fall on the spillway crest and be washed into the channel below the dam. The flow over the spillway crest will be the same as that prior to bridge and gate failure, i.e., peak discharge for one-half maximum possible flood with gates in the wide open position.

The above condition results in a water level at the nuclear plant site of elevation 711.5, well below the 728 plant grade.

TVA considers the most severe condition imaginable is one in which the OBE occurs at the onset of the main portion of the one-half maximum possible flood flow into Watts Bar Reservoir. For practical purposes, spillway gates would be in the closed position at the time of the OBE with consequent bridge failure as described above. The gate hoisting machinery would be inoperable as a result of being struck by the bridge with the result that the peak discharge would occur with the gates closed and the bridge deck and girders lying on top of the spillway piers. Analysis of the concrete portions of the dam for the headwater for this condition show that they will not fail.

For the condition described above with the most probable embankment breaching from overflow, the outflow of Watts Bar Dam would increase rapidly from about 200,000 cfs prior to the breach to about 660,000 cfs when breaching is complete. Breach time would be about five hours.

The 660,000 cfs breach flow is the crest. By unsteady flow routing the tailwater level at Watts Bar Dam would rise to elevation 717.5. The flood level would, of course, be somewhat lower at the plant site and safely below plant grade, elevation 728.

For flow conditions between the 25-year flood and one-half maximum possible flood, when the bottom of the gates are in the water, failure of the bridge during an OBE, with consequent striking of the gates by the downstream bridge girders will result in failure of the gate lifting chains. The gates will rotate to the closed position. This condition is less severe than that described above for gates remaining closed during one-half maximum possible flood, consequently the resulting flood levels were not determined.

Fort Loudoun Dam

Stability analyses for the powerhouse and spillway sections result in the conclusion that these structures are judged not to fail. The analyses show low base stresses, with near two-thirds of the base in compression. Results are given in Figure Q2.11.3-5.

Slip circle analysis of the earth embankment results in a factor of safety of 1.26, and the embankment is judged not to fail. Results are given in Figure Q2.11.3-6.

The spillway gates and bridge are of the same design as those at Watts Bar Dam. Conditions of failure during an OBE are the same. However, coincident failure at Fort Loudoun and Watts Bar does not occur.

For the critical case of Fort Loudoun bridge failure at the onset of the main portion of one-half maximum possible flood flow into Fort Loudoun Reservoir, it was found that the inflows are much less than the condition resulting from simultaneous failure of Cherokee and Douglas as described on pages Q2.11.3-16 and -17.

No further analysis is made, for it is concluded that in the event of bridge failure at Fort Loudoun, there is adequate time (more than 36 hours) to place Watts Bar Nuclear Plant in a shutdown mode to accept flooding.

Tellico Dam

No part of the dam is judged to fail. Results of the stability analyses for a typical nonoverflow block and a typical spillway block are shown in Figure Q2.11.3-7. The result of the stability analysis of the earth embankment is shown in Figure Q2.11.3-8 and indicates a factor of safety of 1.28.

Norris Dam

Results of the stability analyses for a typical spillway block and a typical nonoverflow section of maximum height are shown on Figure Q2.11.3-9. Since only a small percentage of the spillway base is in compression, this structure is judged to fail. The high nonoverflow section with a small percentage of the base in compression and with high compressive and shearing stresses is judged to fail.

Figure Q2.11.3-10 shows the condition of the dam after failure. Based on stability analyses the nonoverflow blocks remaining in place are judged to withstand the OBE. Blocks 33 through 44 are judged to fail by overturning.

The location of the debris is not based on any calculated procedure of failure, because it is believed that this is not possible. However, it is TVA's judgment that the failure mode shown is one logical assumption, and although there may be many other equally logical assumptions, the amount of channel obstruction would probably be about the same. The sensitivity to lodgment of debris was not calculated for Norris because such an analysis for Fontana as discussed on pages Q2.11.3-14 and -15 was considered representative.

The hydrologic routing for this failure ignores the likely failure of Melton Hill Dam because of the small capacity of its reservoir. The headwater at Watts Bar Dam does not exceed the top of the concrete or earth portion, and hence no additional failure from the Norris flood wave occurs. The resulting water level at the nuclear plant site is elevation 719, well below 728 plant grade.

Cherokee Dam

Results of the stability analysis for a typical spillway block are shown in Figure Q2.11.3-11. Based on this analysis, the spillway is judged stable at the foundation base elevation 900. Analyses made for other elevations above elevation 900, but not shown in Figure Q2.11.3-11, indicate the resultant of forces falls outside the base at elevation 1010. The spillway is assumed to fail at that elevation.

The nonoverflow dam is embedded in fill to elevation 981.5 and is considered stable below that elevation. However, stability analysis indicates failure will occur above the fill line.

The powerhouse intake is very massive and backed up by the powerhouse. Therefore, it is judged able to withstand the OBE without failure.

Results of the analysis for the highest portion of the south embankment is shown on Figure Q2.11.3-12. The analysis was made using the same shear strengths of material as were used in the original analysis and shows a factor of safety of 0.85. Therefore, the south embankment is assumed to fail during the OBE. Since the north embankment and saddle dams 1, 2, and 3 are generally about one-half or less as high as the south embankment, they are judged to be stable for the OBE.

Figure Q2.11.3-13 shows the assumed condition of the dam after failure. All debris from the failure of the concrete portion is assumed to be located in the channel below the failure elevations AND WAS NOT considered to be an obstruction to flow.

No hydrologic results are given for the single failure of Cherokee Dam since simultaneous failure of Cherokee and Douglas is discussed on pages Q2.11.3-16 and -17.

Douglas Dam

Results of the stability analysis for a typical spillway block are shown in Figure Q2.11.3-14. The upper part of the Douglas spillway is approximately 12 feet higher than Cherokee, but the amplification of the rock surface acceleration is the same. Therefore, based on the Cherokee analysis, it is assumed that the Douglas spillway will fail at elevation 937 which corresponds to the assumed failure elevation of the Cherokee spillway.

The Douglas nonoverflow dam is similar to that at Cherokee and is embedded in fill to elevation 927.5. It is considered stable below that elevation. However, based on the Cherokee analysis, it is assumed to fail above the fill line. The abutment nonoverflow blocks 1 through 5 and 29 through 35, being short blocks, are considered able to resist the OBE without failure.

The powerhouse intake is very massive and backed up downstream by the powerhouse. Therefore, it is considered able to withstand the OBE without failure.

Results of the analysis of the saddle dam shown on Figure Q2.11.3-15 indicate a factor of safety of 1. Therefore, the saddle dam is considered to be stable for the OBE.

Figure Q2.11.3-16 shows the portions of the dam judged to fail and the portions judged to remain. All debris from the failed portions is assumed to be deposited in the channel below the failure elevations, and was not considered to be an obstruction to flow.

No hydrologic results are given for the single failure of Douglas Dam since it is determined that the potential exists for simultaneous failure of Cherokee and Douglas as discussed on pages Q2.11.3-16 and -17.

Fontana Dam

No stability analysis was made for this dam to include the effects of the OBE. Fontana is a high dam constructed with three longitudinal contraction joints in the higher blocks. Although the joints are keyed and grouted, it is possible that the grouting was not fully effective. Consequently, there is some question as to how this structure will respond to the motion of a severe earthquake. Therefore, it is assumed that Fontana Dam will not resist the OBE without failure.

Figure Q2.11.3-17 shows the part of the dam judged to remain in its original position after failure and the assumed location of the debris of the failed portion. The location of the debris after failure is one logical assumption based on an assumed failure of the dam at the longitudinal contraction joints. There may be other logical assumptions, but the amount of channel obstruction would probably be about the same.

The higher blocks 9 through 27 containing either two or three longitudinal joints are assumed to fail. Right abutment blocks 1 through 8 and left abutment blocks 28 and beyond were judged to be stable for the following reasons:

a. Their heights are less than one-half the maximum height of the dam.

b. None of these blocks have more than one longitudinal contraction joint, and some have no longitudinal joints.

c. The back slope of Fontana Dam is 1 on 0.76 which the original stability analysis shows is flatter than that required for stability for the normal static loadings.

The hydrologic routing for this failure includes the assumption that the 3
Alcoa dams below Fontana fail completely. Volumes of their reservoirs are too small to influence results. The headwaters at Fort Loudoun, Tellico, and Watts Bar do not exceed the tops of the earth embankments and the concrete portions are stable for this headwater condition. Hence no additional failure from the Fontana flood wave occurs. The resulting water level at the nuclear plant site is elevation 718, well below 728 plant grade. On the chance that Tellico Dam is never completed, the Fontana failure was repeated for that situation. The water level at the nuclear plant then would reach elevation 720.

To determine the sensitivity of the water level at the nuclear plant site to the degree and mode of the Fontana failure, a routing was made in which the top three blocks shown in Section A-A, Figure Q2.11.3-17, are removed. This amounts to lowering the obstruction 83 feet below the level formerly postulated over a distance of 500 feet.

As a result, the initial flood wave would increase from a postulated peak flow of 5,000,000 cfs to 7,500,000 cfs. When routed the 61 miles to the mouth of the Little Tennessee River, however, the increase amounts to only about 50,000 cfs. Although continued routing through the Tellico-Fort Loudoun Reservoirs complex and through Watts Bar Reservoir would further decrease the difference, the entire 50,000 cfs can be applied directly to the former Watts Bar outflow hydrograph with the resulting increase of only 2 feet in elevation. There remains an 8 foot margin below plant grade.

TVA concludes that this test has been applied in a conservative manner and that the results show a relatively low degree of sensitivity and that a comfortable margin of safety remains.

Simultaneous Dam Failures (One-Half Maximum Possible Flood + OBE)

Based on attenuation studies of the OBE, only simultaneous failures of Cherokee and Douglas Dams need be considered since the dams are only 15 miles apart. For an OBE located midway between these two dams, it is assumed that both dams could fail simultaneously and the portions remaining judged to be as given in Figure Q2.11.3-12 and -16 for single dam failures.

The hydrologic routing for this postulated simultaneous failure results in peak headwater at Fort Loudoun Dam of elevation 829.5, one-half foot below the top of the earth embankment, and resulting in headwater elevation of 748.8 at Watts Bar Dam with a water surface of elevation 720 at the nuclear plant site, 8 feet below plant grade. The routing shows that the embankment at Fort Loudoun is not overtopped, having 6 inches of freeboard. The embankment at Watts Bar is not overtopped, having 8.2 feet of freeboard.

Because of the narrow margin of freeboard at Fort Loudoun, conservatism is provided by postulating the arbitrary failure of the earth embankment at approximately the peak headwater time. This results in a headwater elevation of 749.8 at Watts Bar Dam, one foot higher than the elevation for the routing with Fort Loudoun embankment intact. This level is 7.8 feet below the earth embankment at Watts Bar. However, this level is only tenths of a foot below the headwater elevation at which flow over the spillway changes from free flow to orifice flow as a result of the lower portion of the spillway gates being submerged (see Figure 17, Appendix 2.7A to the PSAR). This change in flow conditions will sharply increase the headwater elevation.

Although the routing does not indicate this point is reached, additional conservatism is introduced, because of the narrow margin, by postulating that the flow conditions do change to orifice flow. The Watts Bar headwater will increase to elevation 758.5, 1.5 feet above the top of the earth embankment. A failure analysis was made for this condition, with the result that, for this small amount of overtopping, the embankment would not fail even if the flow continued for more than four days.

Regardless of this finding, a third element of conservatism is introduced by arbitrarily failing the Watts Bar earth embankment. This results in a water level of elevation 732 at the nuclear plant site, 4 feet above grade.

As a result of the above described analysis, TVA concludes that it is extremely unlikely for water to exceed plant grade at Watts Bar Nuclear Plant in one-half the maximum possible flood coupled with simultaneous seismic (OBE) failure of Cherokee and Douglas Dams. For grade to be exceeded all of three narrow factors of safety would have to be violated. There is, however, more than 36 hours from the time of seismic failure to the time at which the analysis determines plant grade could be exceeded. It has previously been determined that the plant can be placed in a shutdown mode to accept flooding within 36 hours after failure.

The hydrograph for Watts Bar Dam is given in Figure Q2.3.11-18 for this flood condition.

Details of Analysis for Condition 2 (25-Year Flood + DBE)

Watts Bar Dam

Based on stability analysis for the powerhouse and spillway sections and slip circle analysis of earth embankment, it is judged that postulated failure does not occur. Figure Q2.11.3-19 provides results for maximum probable headwater elevation 745 and minimum tailwater elevation 675 with water forces as determined by Westergaard. Figure Q2.11.3-20 provides results for the earth embankment. A reanalysis of the spillway section using headwater elevation 745 and tailwater elevation 696 for a 25-year flood and hydrodynamic forces in accordance with Bustamante and Flores results in the determination that about 13 feet of the spillway base is in compression with the stress on the rock at the toe being low, and the factor of safety against overturning is 1.03. However, even if the dam is arbitrarily removed instantaneously, then the level at the nuclear plant site is 723, 5 feet below plant grade.

Fort Loudoun Dam

Results of the stability analysis are shown on Figure Q2.11.3-21. Since the resultant of forces falls outside the base, a portion of the spillway is judged to fail. Based on previous modes of failure for Cherokee and Douglas, the spillway is judged to fail above elevation 750.

The results of the slip circle analysis for the highest portion of the embankment is shown on Figure Q2.11.3-22. Since the factor of safety is less than one, the embankment is assumed to fail.

No analysis was made for the powerhouse under DBE. However, an analysis was made for the OBE with no water in the units, a condition believed to be extremely remote to occur during the OBE. Since the stresses were low and a large percentage of the base was in compression, it is considered that the addition of water in the units would be a stabilizing factor, and the powerhouse is judged not to fail.

Figure Q2.11.3-23 shows the condition of the dam after assumed failure. All debris from the failure of the concrete portions is assumed to be located in the channel below the failure elevations.

No hydrologic routing for the single failure of Fort Loudoun is made since simultaneous failure with Tellico and Fontana, as well as with Tellico, Norris, and Douglas is discussed on pages Q2.11.3-20 and -21.

Tellico Dam

No analysis was made for Tellico for the DBE. Due to the similarity to Fort Loudoun, the spillway and entire embankment are judged to fail in a manner similar to Fort Loudoun. Figure Q2.11.3-24 shows the condition of the dam after failure with all debris assumed located in the channel below the failure elevation.

No routing for the single failure of Tellico is made for the reasons given above for Fort Loudoun.

Single Failure of Other Upstream Dams

It is obvious that the occurrence of a DBE will produce the same postulated failure as identified for the occurrence of an OBE. It is obvious that for a single dam failure, as well as the postulated simultaneous failures of Cherokee and Douglas, that the flood levels at the nuclear plant site will be less than that determined for the OBE + one-half maximum possible flood.

Simultaneous Dam Failures (25-Year Flood + DBE)

TVA believes that multiple dam failures are an extremely unlikely event. TVA's search of the literature reveals no record of failure of concrete dams from earthquake. The postulation of a DBE of 0.18 g acceleration is a very conservative upper limit in itself (as stated in Section 2.9.2 TVA has determined this as 0.14 g). In addition, the DBE must be located in a very precise region to have the potential for multiple dam failures. In order to fail three dams, Norris, Cherokee, and Douglas, the epicenter of a DBE must be confined to a relatively small area, the shape of a football, about 10 miles wide and 20 miles long. In order to fail four dams, Norris, Douglas, Fort Loudoun, and Tellico, the epicenter of a DBE must be confined to a triangular area with sides of approximately one mile in length. However, as an extreme upper limit the above two combinations of dams are postulated to fail as well as the combination of Fort Loudoun, Tellico, and Fontana.

Fort Loudoun, Tellico, and Fontana Dams

A DBE centered between the Fort Loudoun-Tellico complex and Fontana could be postulated to fail these three dams. Using the failure modes given in Figure Q2.11.3-17, -23, and -24, hydrologic routing results in a level at the nuclear plant site of less than elevation 718, 10 feet below plant grade. In this routing the Alcoa dams downstream from Fontana are assumed to not exist since their storage is insignificant with respect to the total routed. The concrete and earth dam at Watts Bar is not overtopped and no failure from flood wave occurs.

Tellico is presently under construction and, if for some reason, the dam is not completed prior to the postulated event, the simultaneous failure of Fort Loudoun and Fontana would result in a level at the nuclear plant site less than that given above.

Norris, Cherokee, and Douglas Dams

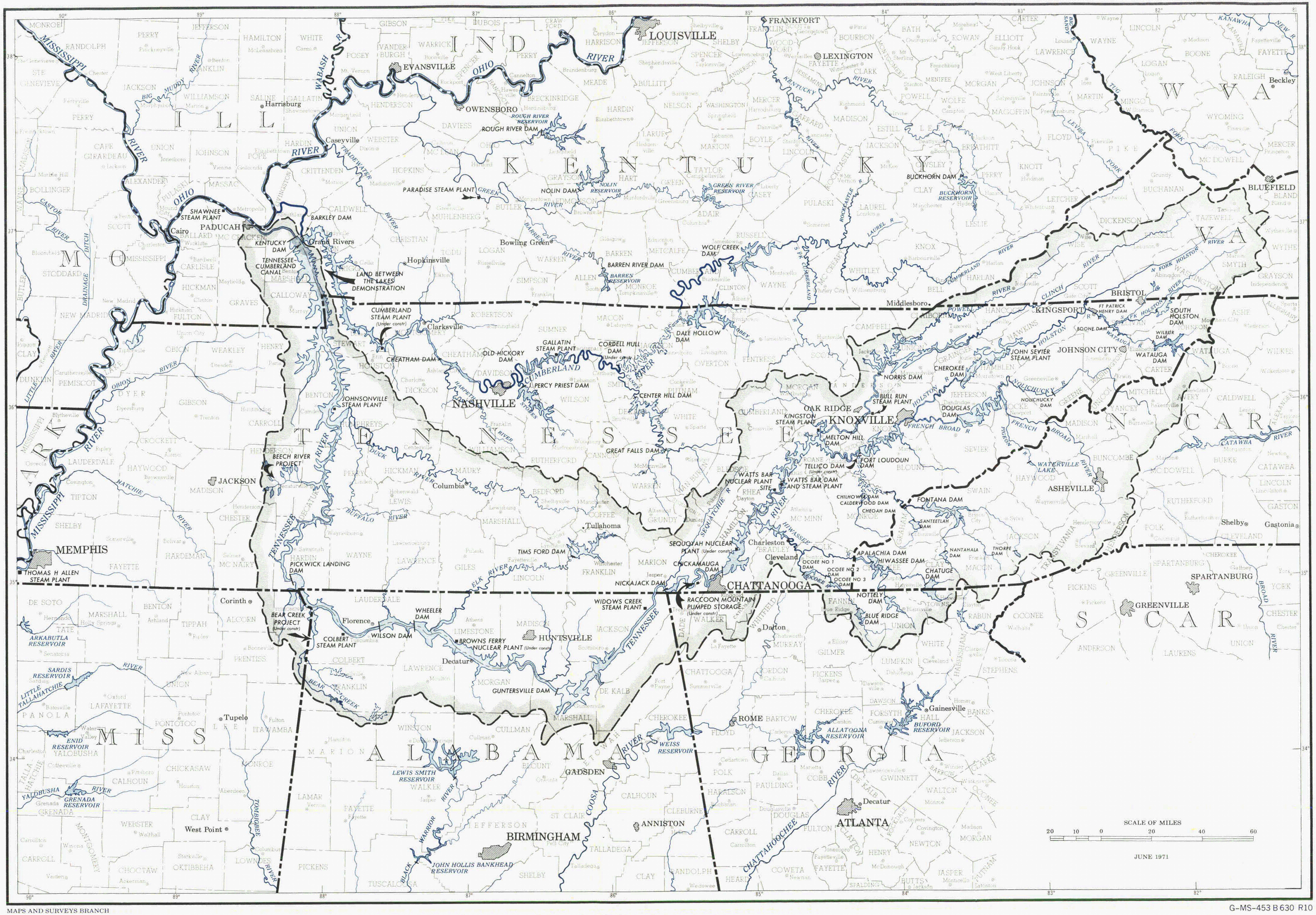
Figure Q2.11.3-25 shows the location of a DBE, and its attenuation, which produces 0.15 g at Norris, 0.09 g at Cherokee and Douglas, 0.08 g at Fort

Loudoun and Tellico, 0.05 g at Fontana, and 0.03 g at Watts Bar. Fort Loudoun, Tellioc, and Watts Bar have previously been judged to not fail for the OBE (0.09 g). As stated before it is believed that multiple dam failure is extremely remote, and it seems reasonable to exclude Fontana on the basis of being the most distant in the cluster of dams under consideration. Norris, Cherokee, and Douglas are postulated to fail simultaneously, and the portions judged to remain are as given in Figures Q2.11.3-10, -13, and -16 for single dam failure.

Hydrologic routing for this postulated failure combination results in the determination that the headwaters at Fort Loudoun and Watts Bar do not exceed the top of the concrete and earth portions of the dams. Therefore, no additional failure from the flood wave occurs. The resulting water level at the nuclear plant site is less than elevation 721, 7 feet below plant grade.

Norris, Douglas, Fort Loudoun, and Tellico Dams

Figure Q2.11.3-26 shows the location of a DBE, and its attenuation, which produces 0.12 g at Norris, 0.08 g at Douglas, 0.12 g at Fort Loudoun and Tellico, 0.07 g at Cherokee, 0.06 g at Fontana, and 0.04 g at Watts Bar. Cherokee is judged not to fail at 0.07 g; Watts Bar has previously been judged not to fail at 0.08 g; and for the same reasons as given above, it seems reasonable to exclude Fontana in this failure combination. Norris, Douglas, Fort Loudoun, and Tellico are postulated to fail simultaneously and the portions judged to remain are as given in Figures Q2.11.3-10, -16, -23, and -24 for single dam failure.



MAPS AND SURVEYS BRANCH

G-MS-453 B 630 R10

Figure Q2.11.3-1

(Added by Amendment 1[illegible])

TABLE Q2.11.3-1

WATTS BAR NUCLEAR PLANT

SUMMARY OF FLOOD RESULTING FROM POSTULATED SEISMIC FAILURE OF UPSTREAM DAMS

	Fort Loudoun Headwater	Watts Bar Headwater	Watts Bar Dam Q, CFS	Watts Bar Nuclear Plant Flow	Watts Bar Nuclear Plant Elevation
OBE Failures With One-Half Maximum Possible Flood					
Norris	-	748	650,000	650,000	719
Cherokee-Douglas					
Fort Loudoun-Watts Bar embankments intact	829.5	748.8	680,000	680,000	720
Arbitrary Fort Loudoun-Watts Bar embankment failure	-	749.8	1,200,000	1,150,000	732
Fontana					
With Tellico constructed	823.8	747.6	630,000	630,000	718
With Tellico not constructed		748.5	664,000	664,000	720-
With certain debris removed[1]			680,000	680,000	720-
Watts Bar					
Gate opening prevented by bridge failure	815	762	660,000		717.5[2]

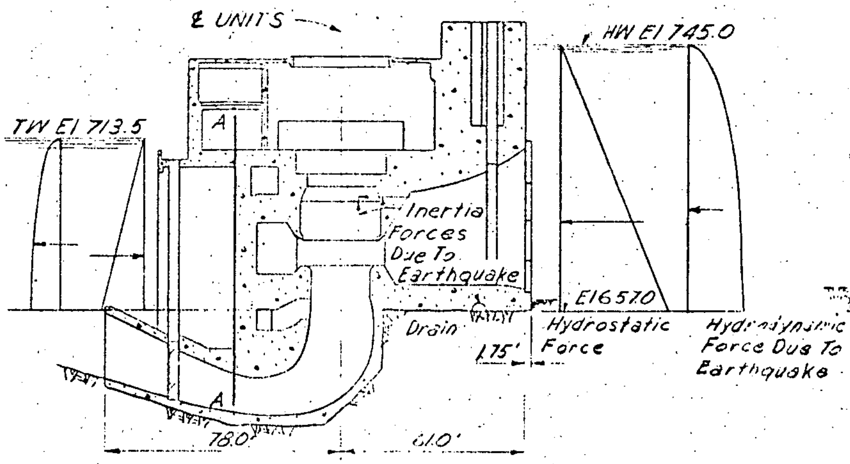
Fort Loudoun gates

DBE Failures With 25-Year Flood					
Fort Loudoun, Tellico, Fontana	-	747	618,000	618,000	718-
Norris, Cherokee, Douglas	822	748.5	673,000	673,000	721-
Norris, Douglas, Fort Loudoun, Tellico	-	745.3	570,000	570,000	716-

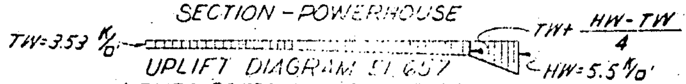
Note 1. Provides sensitivity of water level at nuclear plant site to amount and position of debris at failed dam.

Note 2. Elevation is given for Watts Bar Dam. Elevation at nuclear plant will be less.

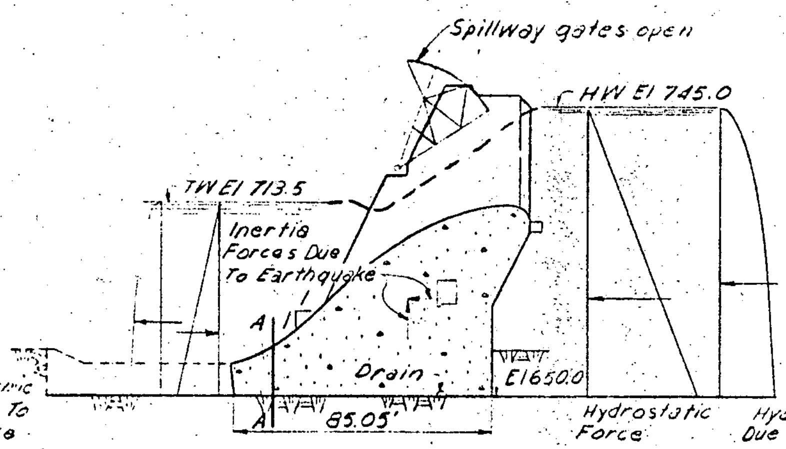
EXHIBIT 2



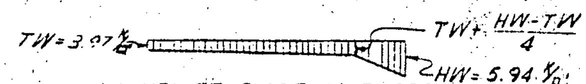
SECTION - POWERHOUSE



UPLIFT DIAGRAM El 657
UPLIFT PRESSURE ASSUMED TO ACT ON 100% OF BASE AREA

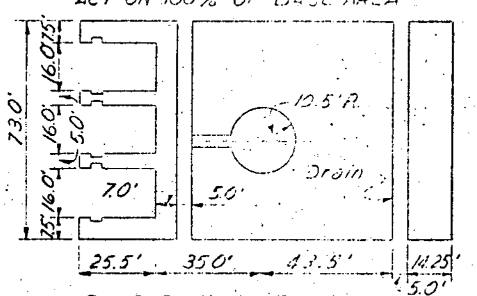


TYPICAL BLOCK - SPILLWAY



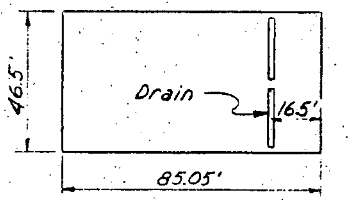
UPLIFT DIAGRAM El 650
UPLIFT PRESSURE ASSUMED TO ACT ON 100% OF BASE AREA

NOTES:
1. OBE earthquake inertia forces assumed as 0.09 g horizontally and 0.06 g vertically.
2. Spillway gates were assumed open for this analysis.

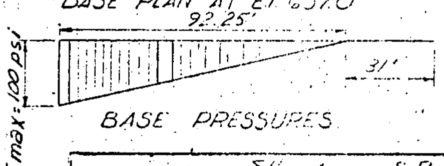


BASE PLAN AT El 657.0

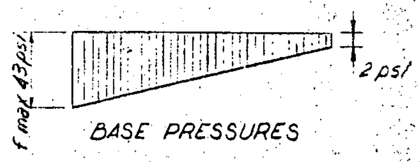
Note A:
The powerhouse and spillway structures are well keyed into the rock foundation. The rock formations are severely folded with the dip generally in a dnstr direction varying from 10° to 40°. Any failure would require cross bed shear of the rock. Rock of this type has cross bed shear strength much in excess of that reqd for a Factor of Safety of 1.



BASE PLAN AT El 650.0



BASE PRESSURES



BASE PRESSURES

* Shear, s, that is reqd for Q=1 is calculated from shear-friction formula, $Q = \frac{0.65\,\Sigma V + SA}{\Sigma H}$. A is assumed to be entire base area.

ΣV	ΣH	$\frac{\Sigma H}{\Sigma V}$	Avg. Shear, S	S Reqd For Q=1	f max	$FS = \frac{\Sigma M_R}{\Sigma M_O}$	Vertical Shear on Plane A-A
28,064K	18,548K	0.66	12.6 psi (entire base)	Note A 0.33 psi* (0.47 psi)**	100 psi	1.38	27 psi

ΣV	ΣH	$\frac{\Sigma H}{\Sigma V}$	Avg. Shear, S	S Reqd For Q=1	f max	$FS = \frac{\Sigma M_R}{\Sigma M_O}$	Vertical Shear on Plane A-A
12,704 K	10,799 K	0.85	19 psi	4.5 psi*	43 psi	1.25	28 psi

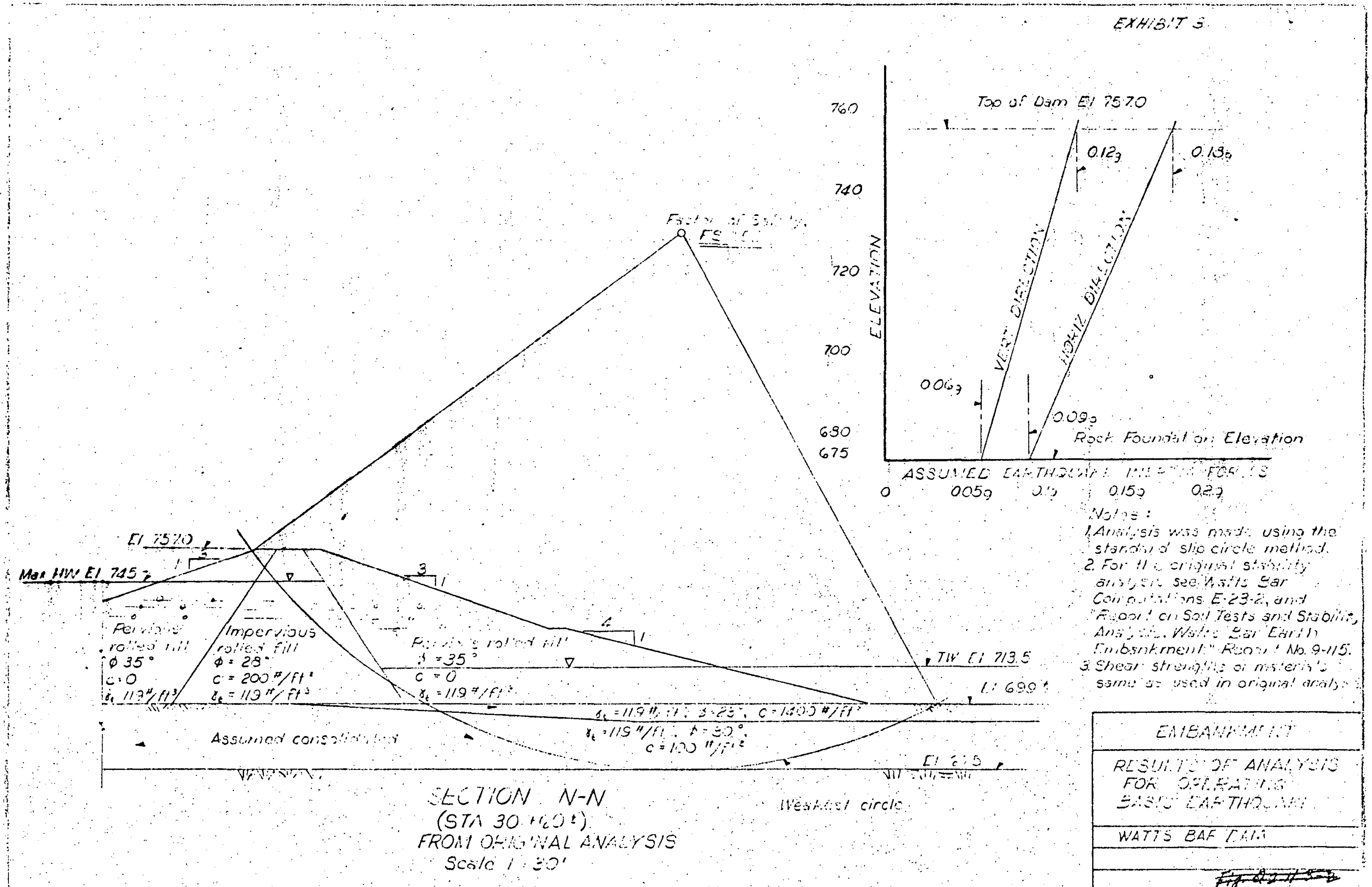
** Shear, S, reqd for Q=1 considering portion of base in compression (no tension), instead of entire base area.

Scale 1"=40'

POWERHOUSE & SPILLWAY
RESULTS OF ANALYSIS FOR OPERATING BASIS EARTHQUAKE

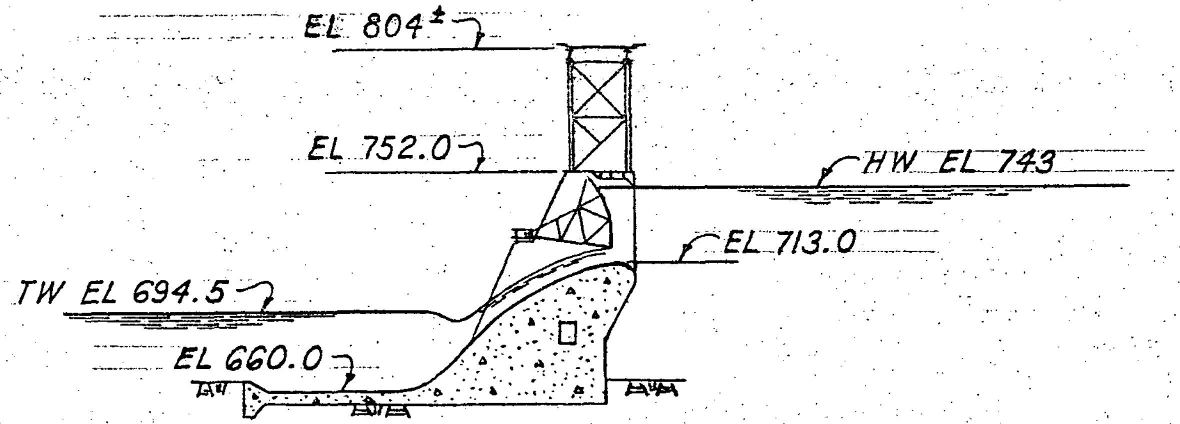
WATTS BAR
TENNESSEE VALLEY AUTHORITY
DIVISION OF ENGINEERING DESIGN
SUBMITTED | RECOMMENDED | APPROVED
KNOXVILLE

FIG. Q2.11.3-2

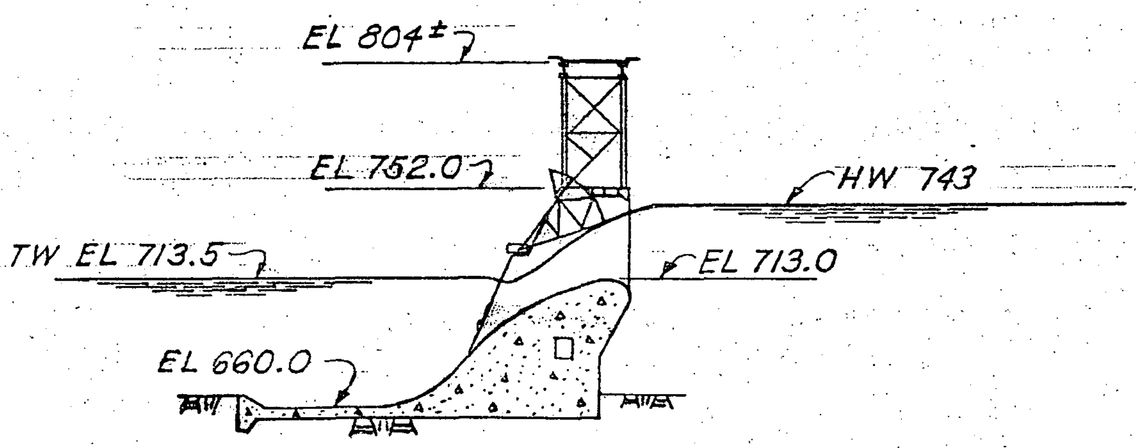


SECTION N-N
(STA 30+60±)
FROM ORIGINAL ANALYSIS
Scale 1" = 30'

FIG. Q2.11.3-3



25 YEAR FLOOD
SCALE 1" = 80'

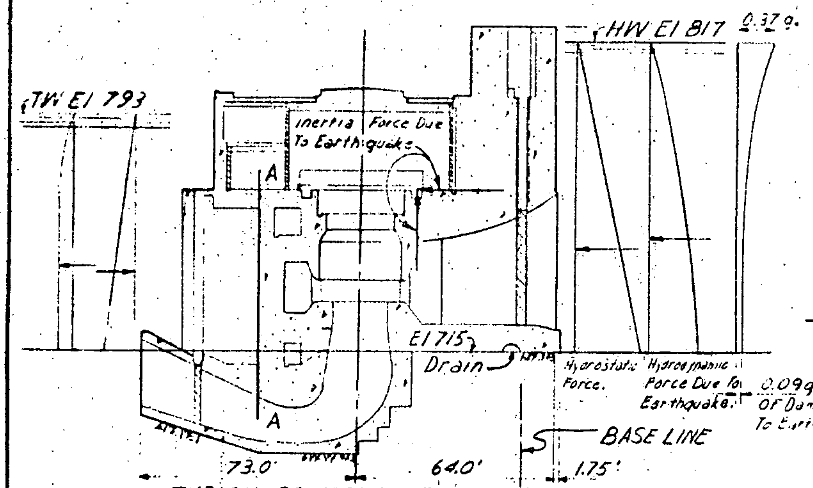


$\frac{1}{2}$ MAX POSSIBLE FLOOD
SCALE 1" = 80'

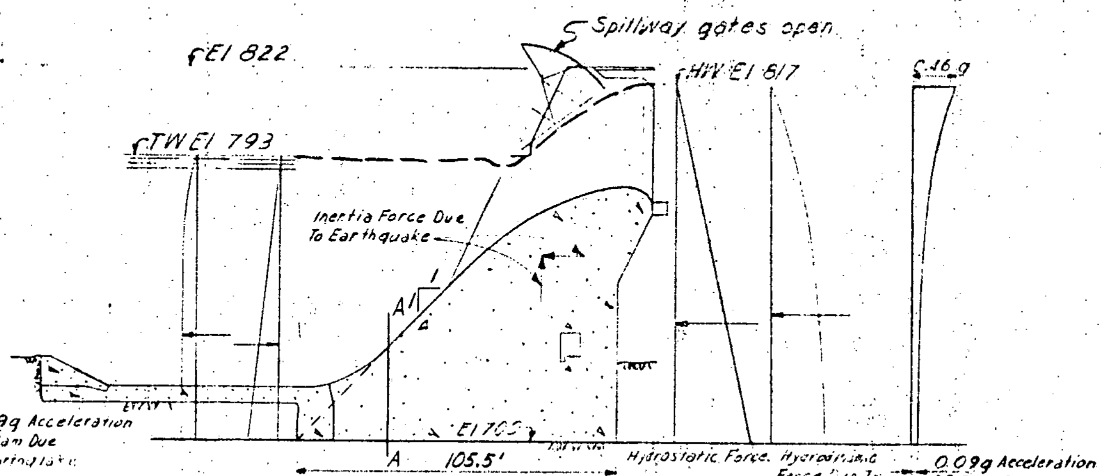
SPILLWAY GATE POSITIONS
FOR
25 YEAR FLOOD
$\frac{1}{2}$ MAX POSSIBLE FLOOD

FIGURE Q2.11.3-4
ADDED BY AMENDMENT 17

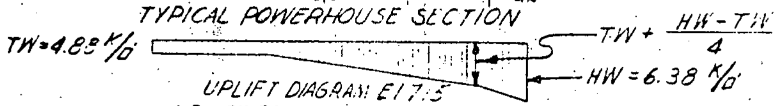
EXHIBIT 9



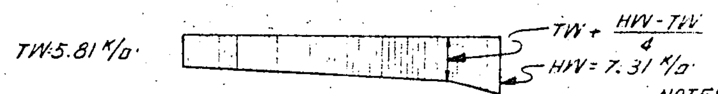
TYPICAL POWERHOUSE SECTION



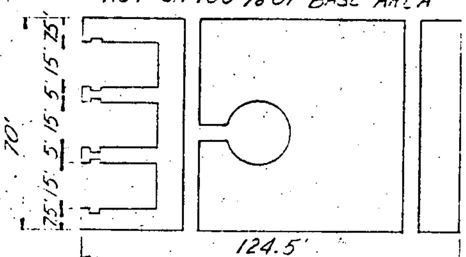
TYPICAL SPILLWAY SECTION



UPLIFT DIAGRAM El 715
UPLIFT PRESSURE ASSUMED TO ACT ON 100 % OF BASE AREA

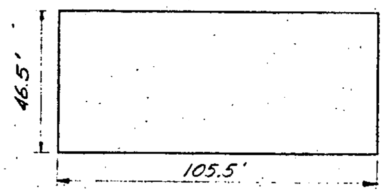


UPLIFT DIAGRAM El 700
UPLIFT PRESSURE ASSUMED TO ACT ON 100 % OF BASE AREA

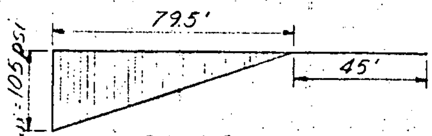


BASE PLAN El 715

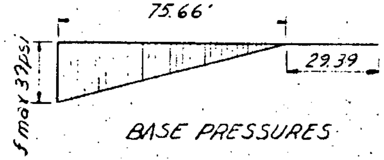
* Shear, s that is reqd for Q=1 is calculated from shear-friction formula, $Q = \frac{0.65\,\Sigma V + SA}{\Sigma H}$, A is assumed to be entire area.
** Shear stress, s, reqd for Q=1 considering portion of base in compression instead of entire base area.



BASE PLAN El 700



BASE PRESSURES



BASE PRESSURES

NOTES:

1. Vertical acceleration of powerhouse and spillway at base assumed to be 0.06g. By dynamic analysis, amplification of acceleration above the base was determined to be 0.22g at top for the powerhouse section and 0.[illegible]g at the top for the spillway.
2. Horizontal acceleration of powerhouse and spillway at base assumed to be 0.09g. By dynamic analysis, amplification of acceleration above the base was determined to be 0.37g at the top for the powerhouse section and 0.46g at the top for the spillway.
3. Spillway gates assumed open for this analysis.

Scale 1" = 40'

ΣV	ΣH	$\frac{\Sigma H}{\Sigma V}$	Avg shear	S reqd for Q=1	f max	$FS = \frac{\Sigma M_R}{\Sigma M_O}$	Vertical shear on plane A-A
22,762K	22,541K	0.99	24.2psi (entire base)	8.3psi * (14.6psi) **	105psi	1.18	26 psi

ΣV	ΣH	$\frac{\Sigma H}{\Sigma V}$	Avg shear	S reqd for Q=1	f max	$FS = \frac{\Sigma M_R}{\Sigma M_O}$	Vertical shear on plane A-A
19,488K	14,382K	0.75	20psi (entire base)	2.38psi * (3.30psi) **	39psi	1.26	17psi

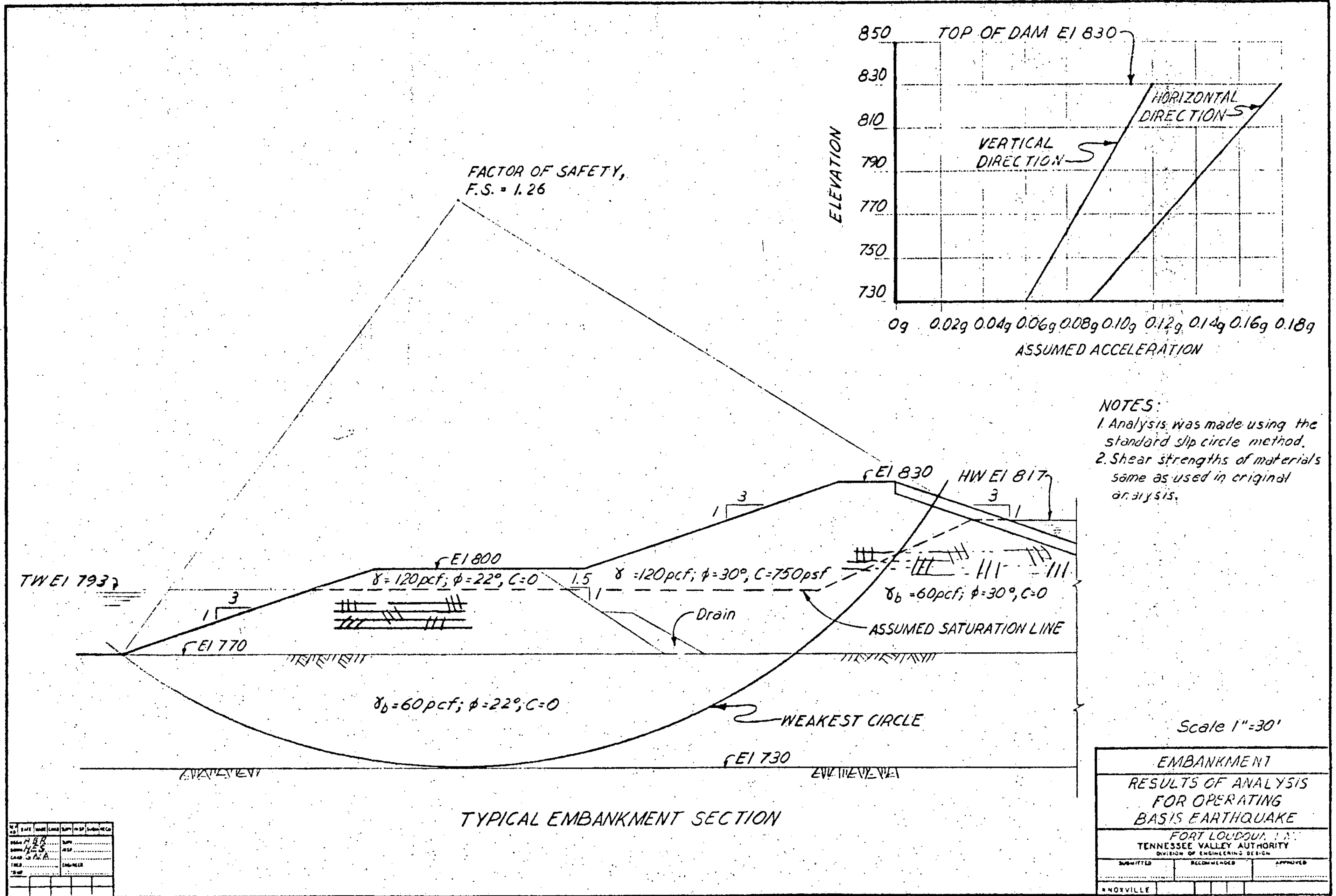
POWERHOUSE & SPILLWAY
RESULTS OF ANALYSIS FOR OPERATING BASIS EARTHQUAKE

FORT LOUDOUN
TENNESSEE VALLEY AUTHORITY
DIVISION OF ENGINEERING DESIGN

KNOXVILLE

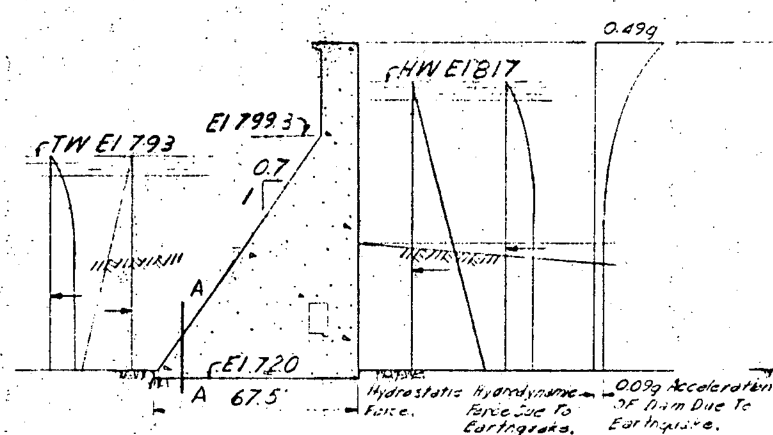
Fig. Q2.11.3-5

EXHIBIT 10

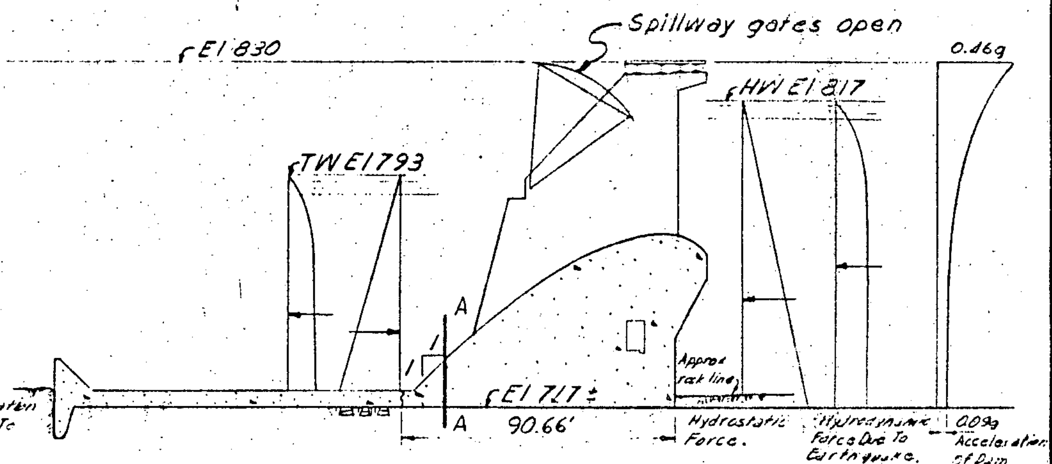


~~Figure Q2.11.3-6~~

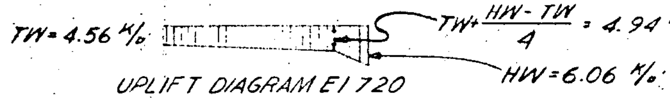
Fig. Q2.11.3-6



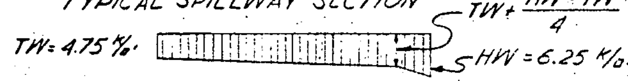
TYPICAL NONOVERFLOW SECTION



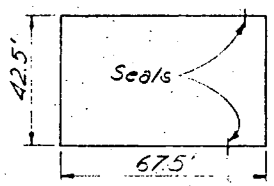
TYPICAL SPILLWAY SECTION



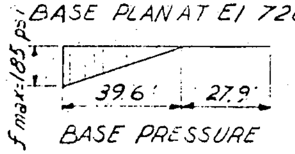
UPLIFT DIAGRAM El 720
UPLIFT PRESSURE ASSUMED TO ACT ON 100 % OF BASE AREA



UPLIFT DIAGRAM El 717
UPLIFT PRESSURE ASSUMED TO ACT ON 100 % OF BASE AREA



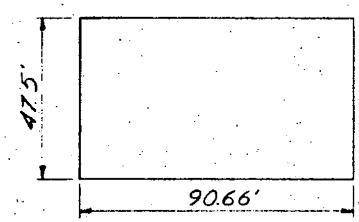
BASE PLAN AT El 720



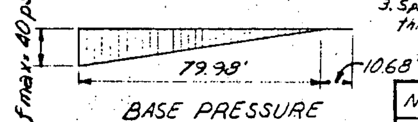
BASE PRESSURE

* Shears, that is reqd for Q=1 is calculated from shear-friction formula, $Q = \frac{0.65\Sigma V + SA}{\Sigma H}$, A is assumed to be entire area.

** Shear stress, s, reqd for Q=1 considering portion of base in compression instead of entire base area.



BASE PLAN AT El 717



BASE PRESSURE

NOTES:
1. Vertical acceleration of nonoverflow and spillway at base assumed to be 0.06g. By dynamic analysis, amplification of acceleration above the base was determined to be 0.12g at top for the nonoverflow section and 0.17g at the top for the spillway.
2. Horizontal acceleration of nonoverflow and spillway at base assumed to be 0.09g. By dynamic analysis, amplification of acceleration above the base was determined to be 0.49g at the top for the nonoverflow section and 0.46g at the top for the spillway.
3. Spillway gates assumed open for this analysis.

Scale 1" = 40'

ΣV	ΣH	$\frac{\Sigma H}{\Sigma V}$	Avg Shear, S	S Reqd For Q=1	f max	$F.S. = \frac{\Sigma M_R}{\Sigma M_O}$	Vertical Shear on Plane A-A
13,602K	9715K	0.72	24psi	2.17psi* (3.96 psi)**	185psi	1.14	20psi

ΣV	ΣH	$\frac{\Sigma H}{\Sigma V}$	Avg Shear, S	S Reqd For Q=1	f max	$F.S. = \frac{\Sigma M_R}{\Sigma M_O}$	Vertical Shear on Plane A-A
11,094K	9,891K	0.89	16psi	4.3psi* (4.9psi)**	40psi	1.16	40psi

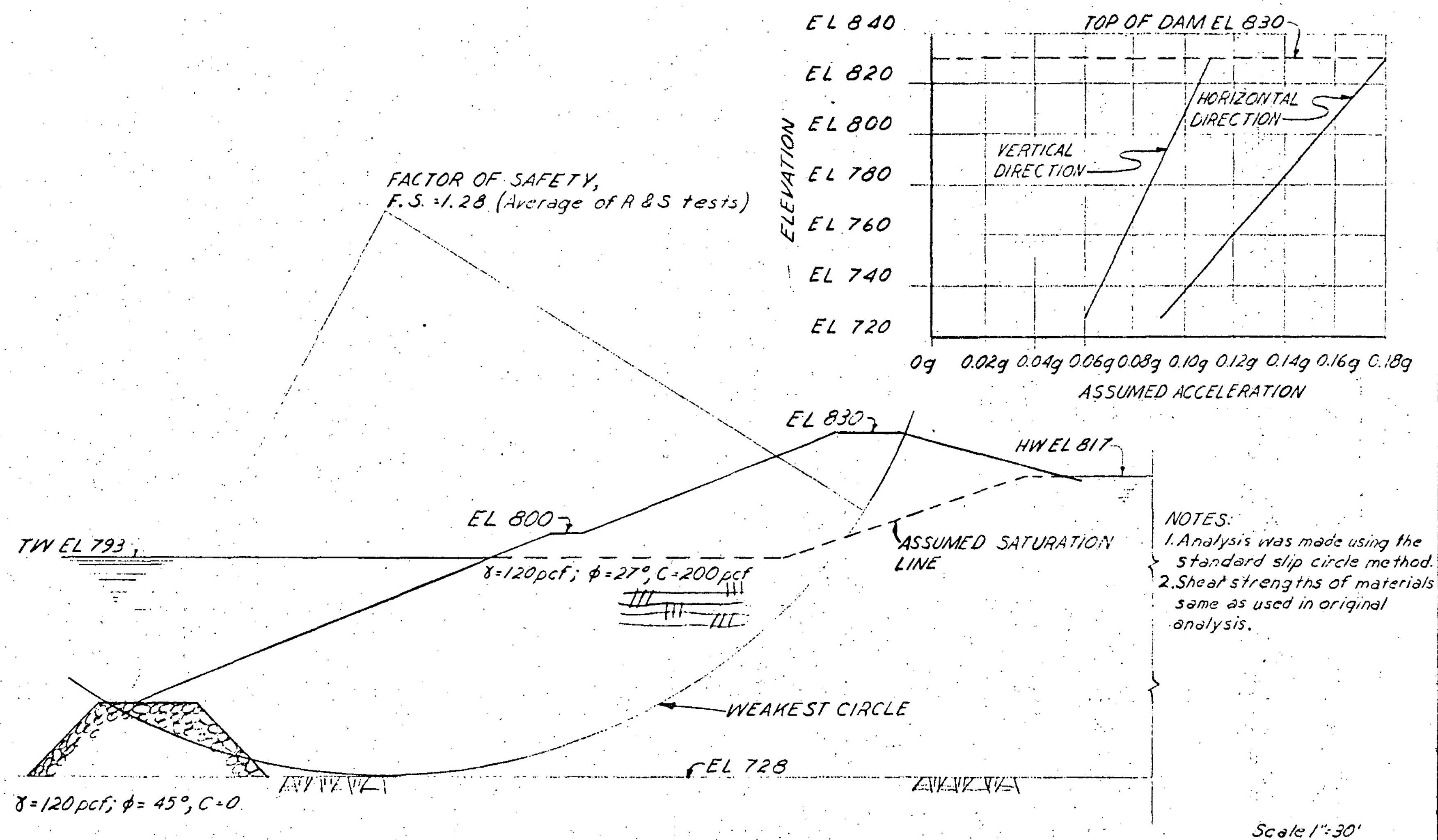
NONOVERFLOW & SPILLWAY

RESULTS OF ANALYSIS FOR OPERATING BASIS EARTHQUAKE

TELLICO DAM
TENNESSEE VALLEY AUTHORITY
DIVISION OF ENGINEERING DESIGN
SUBMITTED | RECOMMENDED | APPROVED
KNOXVILLE

FIG. Q2.11.3-7

EXHIBIT 12



TYPICAL EMBANKMENT SECTION

EMBANKMENT

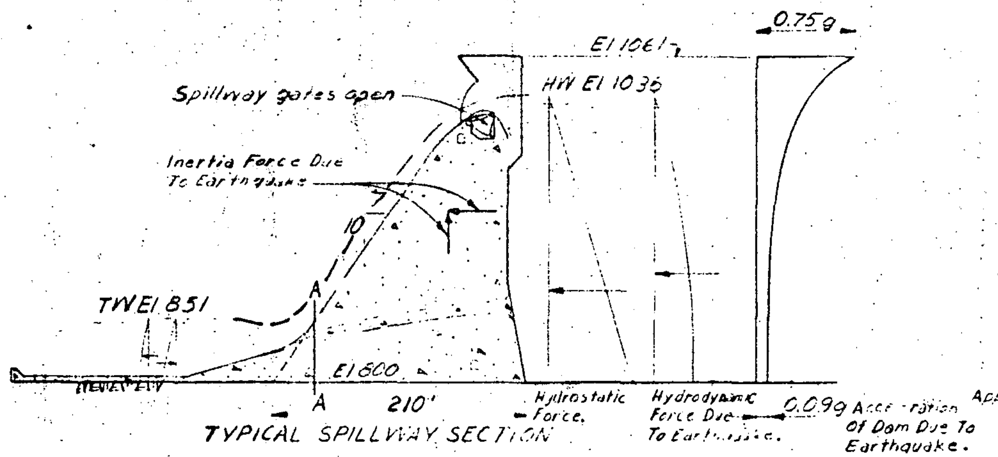
RESULTS OF ANALYSIS FOR OPERATING BASIS EARTHQUAKE

TELLICO DAM
TENNESSEE VALLEY AUTHORITY
DIVISION OF ENGINEERING DESIGN

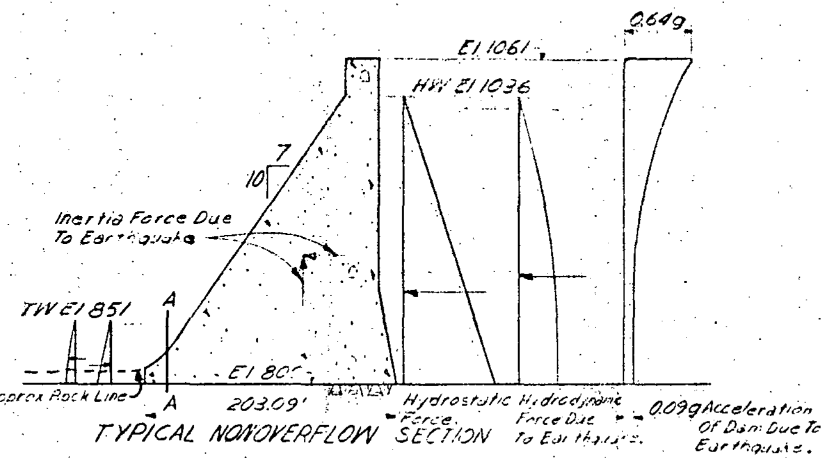
KNOXVILLE

FIG. Q2.11.3-8

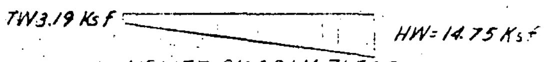
EXHIBIT 1



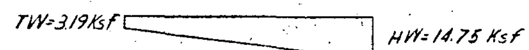
TYPICAL SPILLWAY SECTION



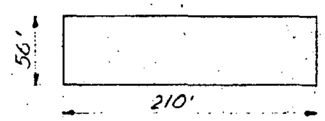
TYPICAL NONOVERFLOW SECTION



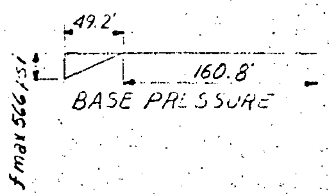
UPLIFT DIAGRAM El 800
UPLIFT PRESSURE ASSUMED
TO ACT ON 100 % OF BASE AREA



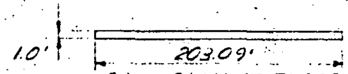
UPLIFT DIAGRAM El 800
UPLIFT PRESSURE ASSUMED
TO ACT ON 100 % OF BASE AREA



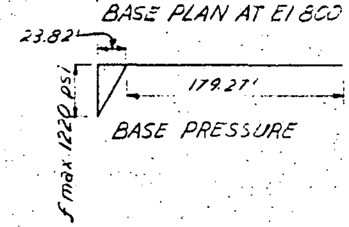
BASE PLAN AT El 800



BASE PRESSURE



BASE PLAN AT El 800



BASE PRESSURE

* Shear, s, that is reqd for Q=1 is calculated from shear-friction formula, $Q = \frac{0.65 \Sigma V + SA}{\Sigma H}$, A is assumed to be entire area.

** Shears, reqd for Q=1 considering portion of base in compression (no tension), instead of entire base area.

NOTES:

1. Vertical acceleration of nonoverflow and spillway at base assumed to be 0.06g. By dynamic analysis, amplification of acceleration above the base was determined to be 0.14g at the top for the nonoverflow section and 0.14g at the top for the spillway.
2. Horizontal acceleration of nonoverflow and spillway at base assumed to be 0.09g. By dynamic analysis, amplification of acceleration above the base was determined to be 0.64g at the top for the nonoverflow section and 0.75g at the top for the spillway.
3. Spillway gates assumed open for this analysis.

Scale 1"=100'

ΣV	ΣH	$\frac{\Sigma H}{\Sigma V}$	Avg Shear, s	S Reqd For Q=1	f max	FS: $\frac{\Sigma M_R}{\Sigma M_O}$	Vertical Shear on plane A-A
112,616 K	143,587K	1.28	85 psi (entire base)	42 psi* (177psi)**	566psi	1.25	247 psi

ΣV	ΣH	$\frac{\Sigma H}{\Sigma V}$	Avg Shear, s	S Reqd For Q=1	f max	FS: $\frac{\Sigma M_R}{\Sigma M_O}$	Vertical Shear on plane A-A
2101 K	2786K	1.33	95 psi (entire base)	49 psi* (415psi)**	1220 psi	1.03	535 psi

SPILLWAY & NONOVERFLOW

RESULTS OF ANALYSIS FOR OPERATING BASE EARTHQUAKE

NORRIS DAM
TENNESSEE VALLEY AUTHORITY
DIVISION OF ENGINEERING DESIGN

SUBMITTED | RECOMMENDED | APPROVED

KNOXVILLE

~~Figure 8 Exhibit~~

Fig. Q2.11.3-9

EXHIBIT 13

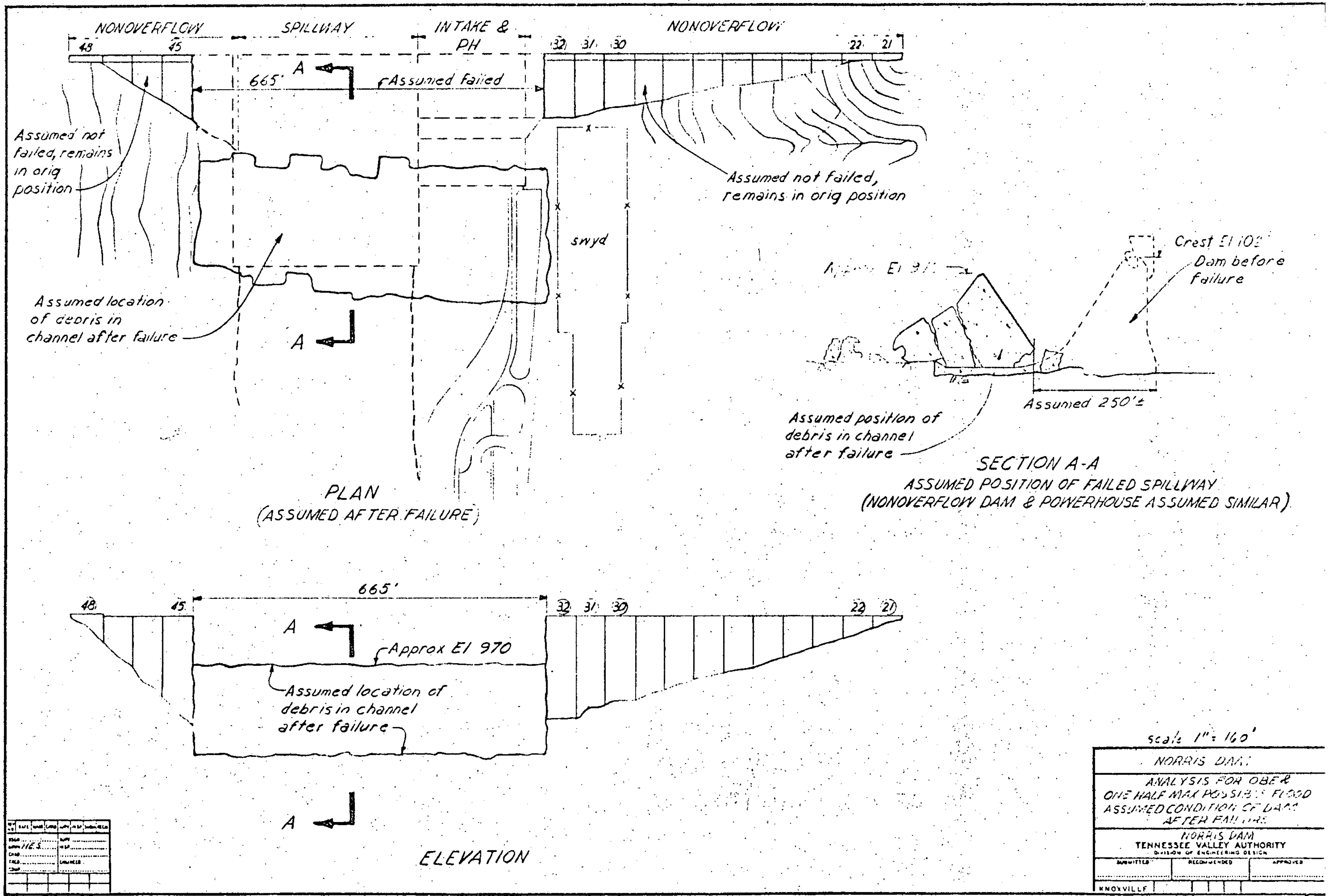
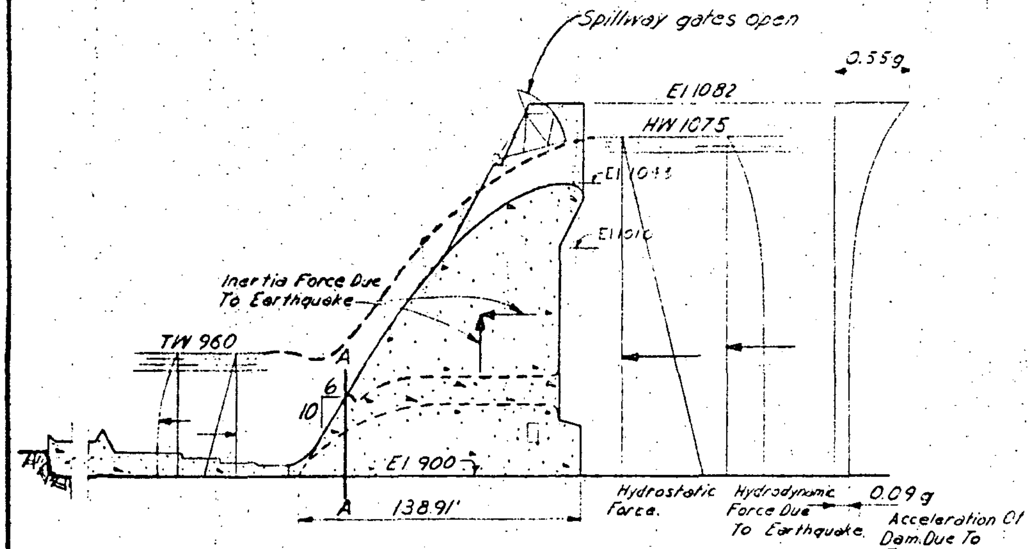
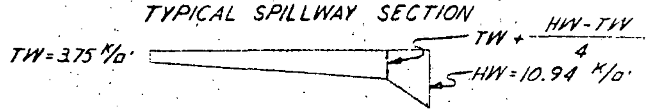


FIG. Q2.11.3-1

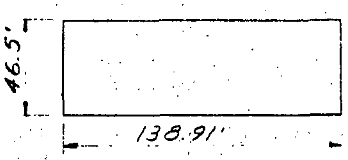
EXHIBIT 5



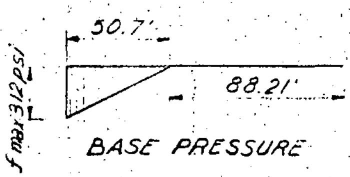
TYPICAL SPILLWAY SECTION



UPLIFT DIAGRAM El 900
UPLIFT PRESSURE ASSUMED TO ACT ON 100 % OF BASE AREA



BASE PLAN AT El 900



BASE PRESSURE

* Shear, s, that is reqd for Q=1 is calculated from shear-friction formula, $Q = \frac{0.65\,\Sigma V + SA}{\Sigma H}$, A is assumed to be entire area.

** Shear stress, s, reqd for Q=1 considering portion of base in compression instead of entire base area.

ΣV	ΣH	$\frac{\Sigma H}{\Sigma V}$	Avg Shear, S	S Reqd For Q=1	f max	$FS = \frac{\Sigma M_R}{\Sigma M_O}$	Vertical Shear on Plane A-A
53,007K	57,276K	1.08	61 psi (entire base)	25 psi * (67 psi) **	312 psi	1.13	173 psi

NOTES:
1. Vertical acceleration of the spillway at the base assumed to be 0.06 g. By dynamic analysis, amplification of acceleration above the base was determined to be 0.11 g at the top.
2. Horizontal acceleration of the spillway at the base assumed to be 0.09 g. By dynamic analysis, amplification of acceleration above the base was determined to be 0.55 g at the top.
3. Spillway gates assumed open for this analysis.

Scale 1" = 60'

SPILLWAY & NONOVERFLOW

RESULTS OF ANALYSIS FOR OPERATING BASIS EARTHQUAKE

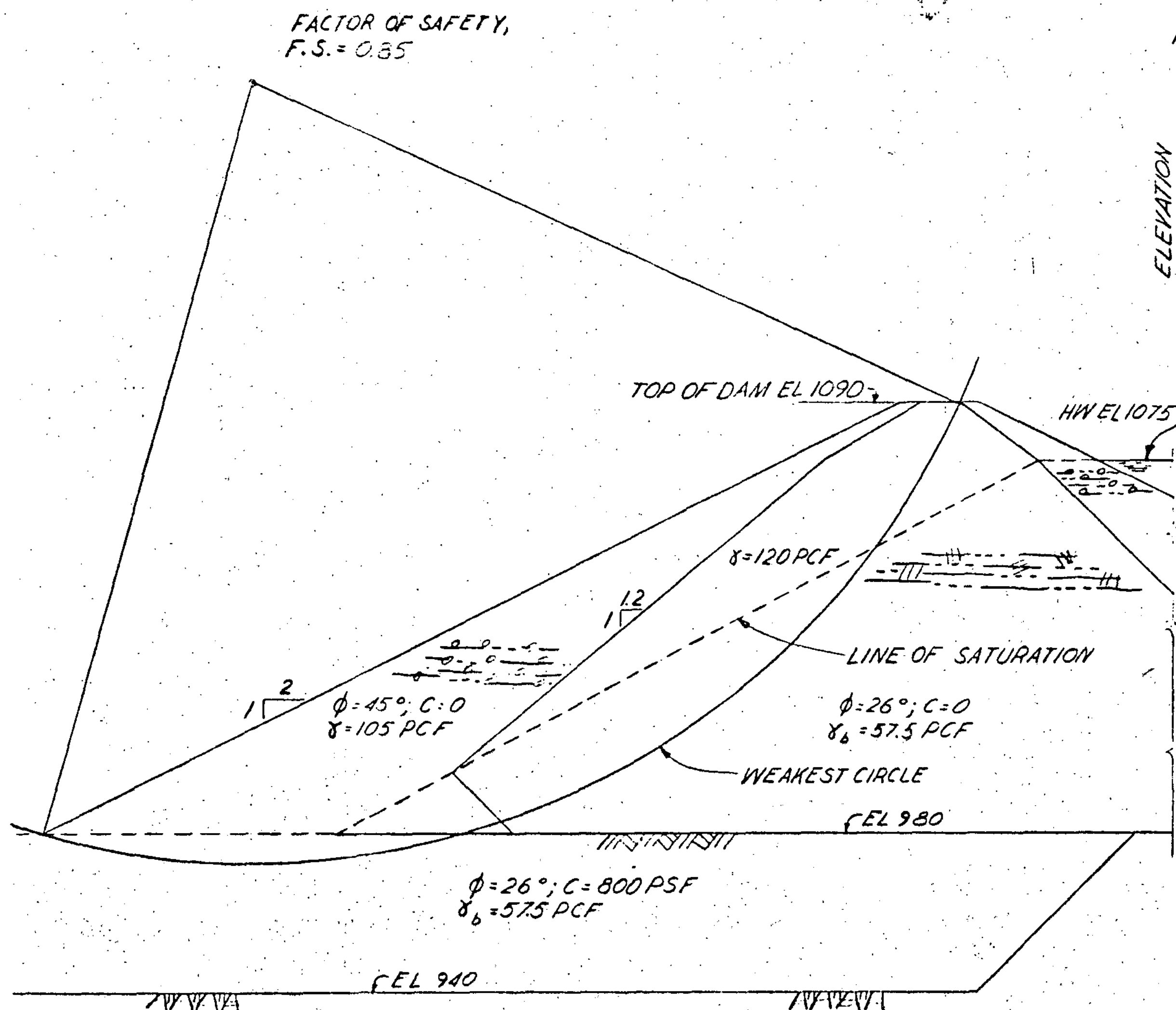
CHEROKEE DAM
TENNESSEE VALLEY AUTHORITY
DIVISION OF ENGINEERING DESIGN

SUBMITTED | RECOMMENDED | APPROVED

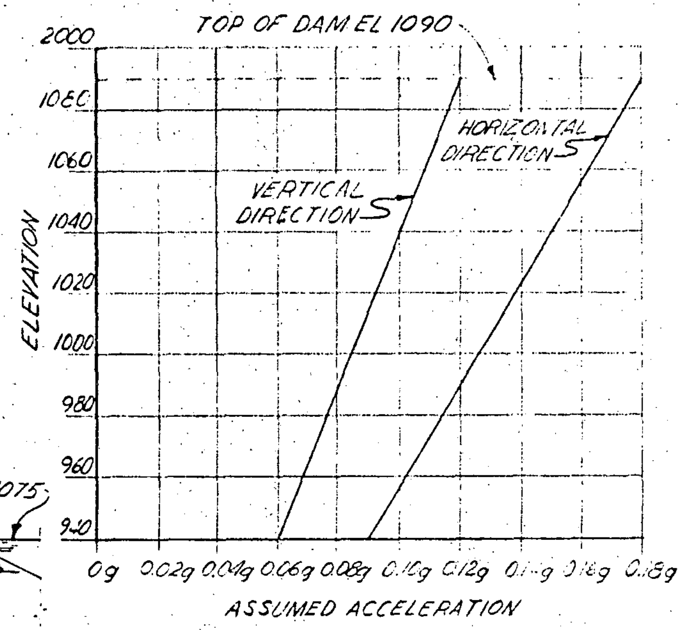
KNOXVILLE

FIG. Q2.11.3-1

EXHIBIT 6



TYPICAL EMBANKMENT SECTION



NOTES:
1. Analysis was made using the standard slip circle method.
2. Shear strengths of materials same as used in original analysis.

Scale 1"=30'

EMBANKMENT
RESULTS OF ANALYSIS FOR OPERATING BASIS EARTHQUAKE
CHEROKEE DAM
TENNESSEE VALLEY AUTHORITY
DIVISION OF ENGINEERING DESIGN
SUBMITTED | RECOMMENDED | APPROVED
KNOXVILLE

FIG. Q2.11.3-1

EXHIBIT 15

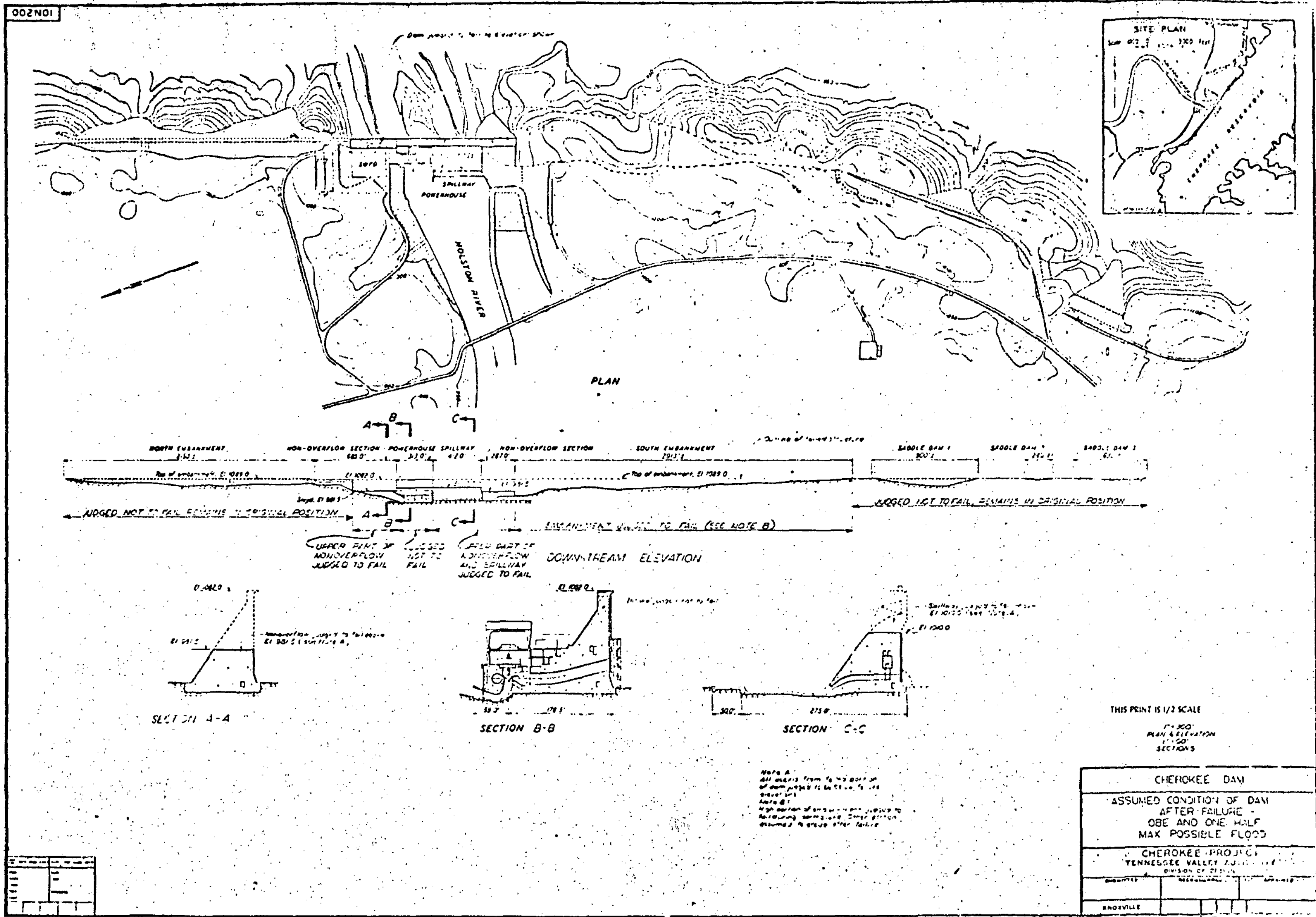
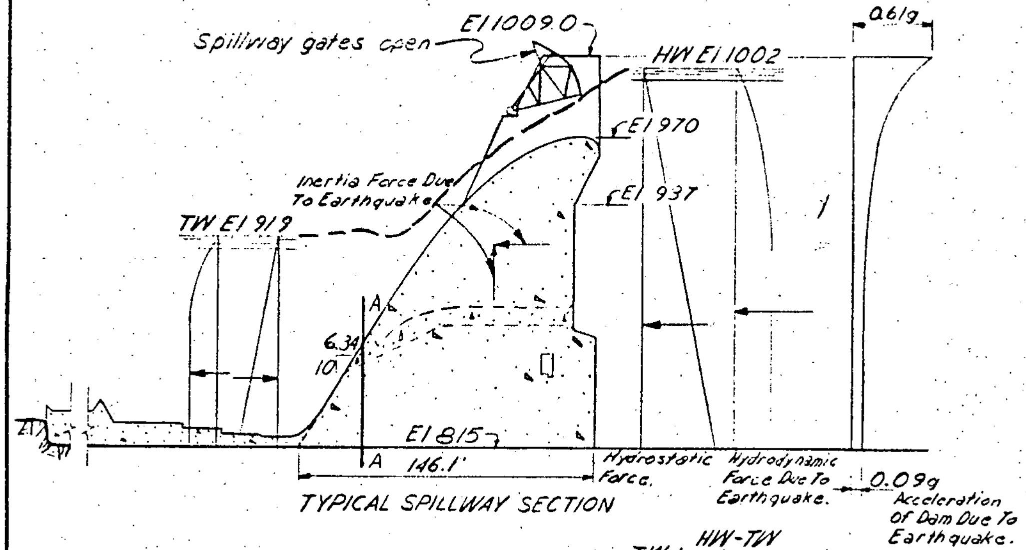
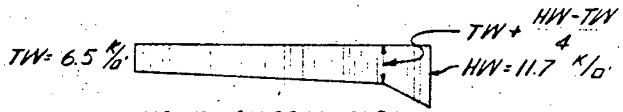


Fig. Q2.11.3-13

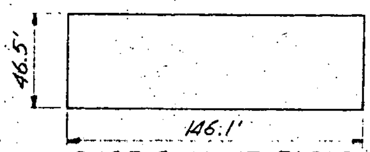
EXHIBIT 7



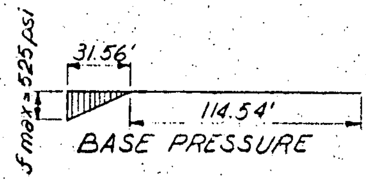
TYPICAL SPILLWAY SECTION



UPLIFT DIAGRAM El 815
UPLIFT PRESSURES ASSUMED
TO ACT ON 100% OF BASE AREA



BASE PLAN AT El 815



BASE PRESSURE

* Shear, s, that is reqd for Q=1 is calculated from shear-friction formula, $Q = \frac{0.65\Sigma V + SA}{\Sigma H}$, A is assumed to be entire area.

** Shear stress, s, reqd for Q=1 considering portion of base in compression instead of entire base area.

NOTES:
1. Vertical acceleration of the spillway at the base assumed to be 0.06 g. By dynamic analysis, amplification of acceleration above the base was determined to be 0.13 g at the top.
2. Horizontal acceleration of the spillway at the base assumed to be 0.09 g. By dynamic analysis, amplification of acceleration above the base was determined to be 0.61 g at the top.
3. Spillway gates assumed open for this analysis.

ΣV	ΣH	$\frac{\Sigma H}{\Sigma V}$	Avg Shear, s	s Reqd For Q=1	f max	$FS = \frac{\Sigma M_R}{\Sigma M_O}$	Vertical Shear on Plane AA
55,483K	60,245K	1.09	61.6 psi (entire base)	25 psi* (170 psi)**	525 psi	1.06	156 psi

Scale 1" = 60'

SPILLWAY & NONOVERFLOW

RESULTS OF ANALYSIS FOR OPERATING BASIS EARTHQUAKE

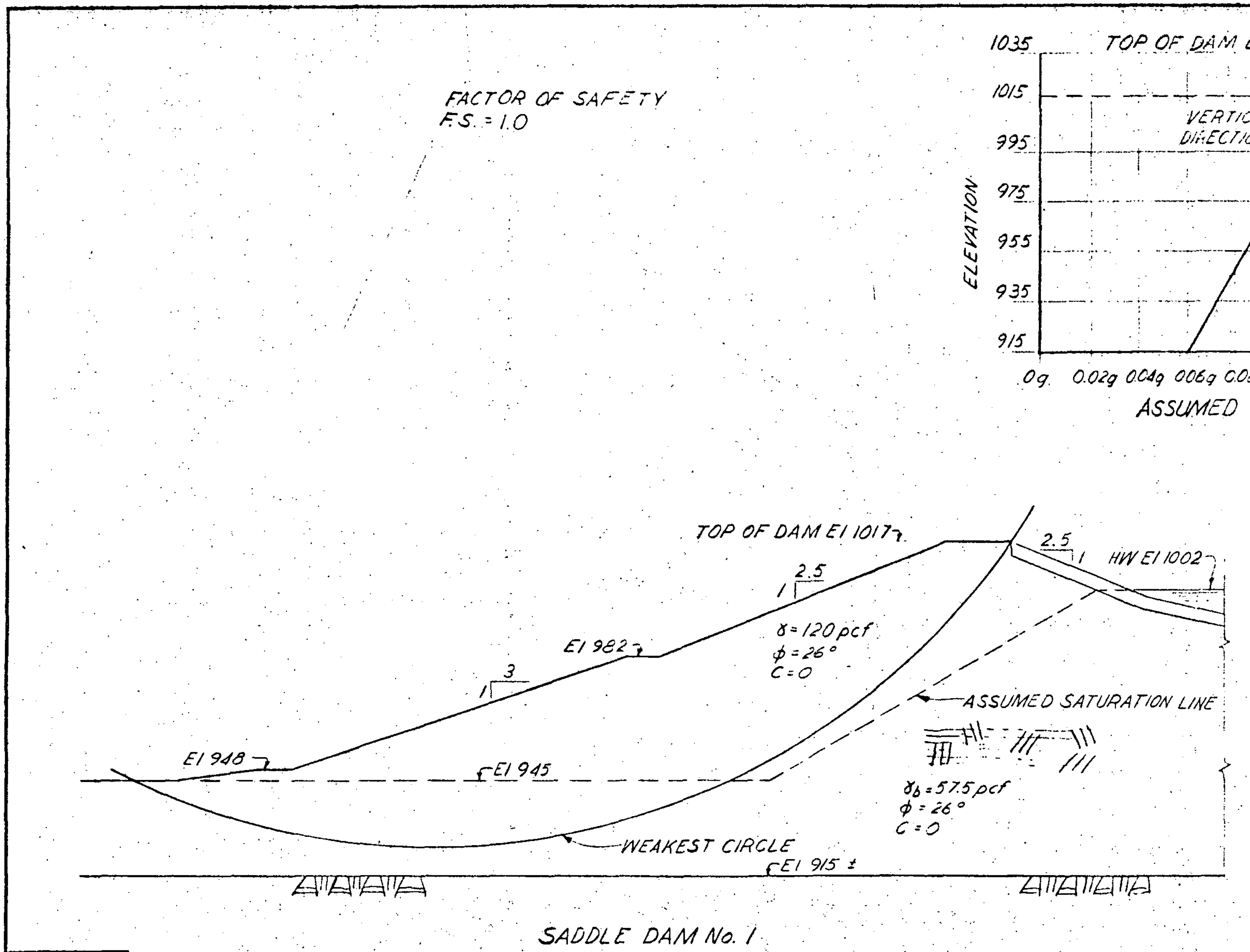
DOUGLAS DAM
TENNESSEE VALLEY AUTHORITY
DIVISION OF ENGINEERING DESIGN

SUBMITTED | RECOMMENDED | APPROVED

KNOXVILLE

FIG. Q2.11.3-1

EXHIBIT 8



NOTES:
1. Analysis was made using the standard slip circle method.
2. Shear strengths of materials same as used in original analysis.

Scale 1" = 30'

SADDLE DAM No. 1
RESULTS OF ANALYSIS FOR OPERATING BASIS EARTHQUAKE
DOUGLAS DAM
TENNESSEE VALLEY AUTHORITY
DIVISION OF ENGINEERING DESIGN
SUBMITTED RECOMMENDED APPROVED
KNOXVILLE

FIG. Q2.11.3-1

EXHIBIT 16

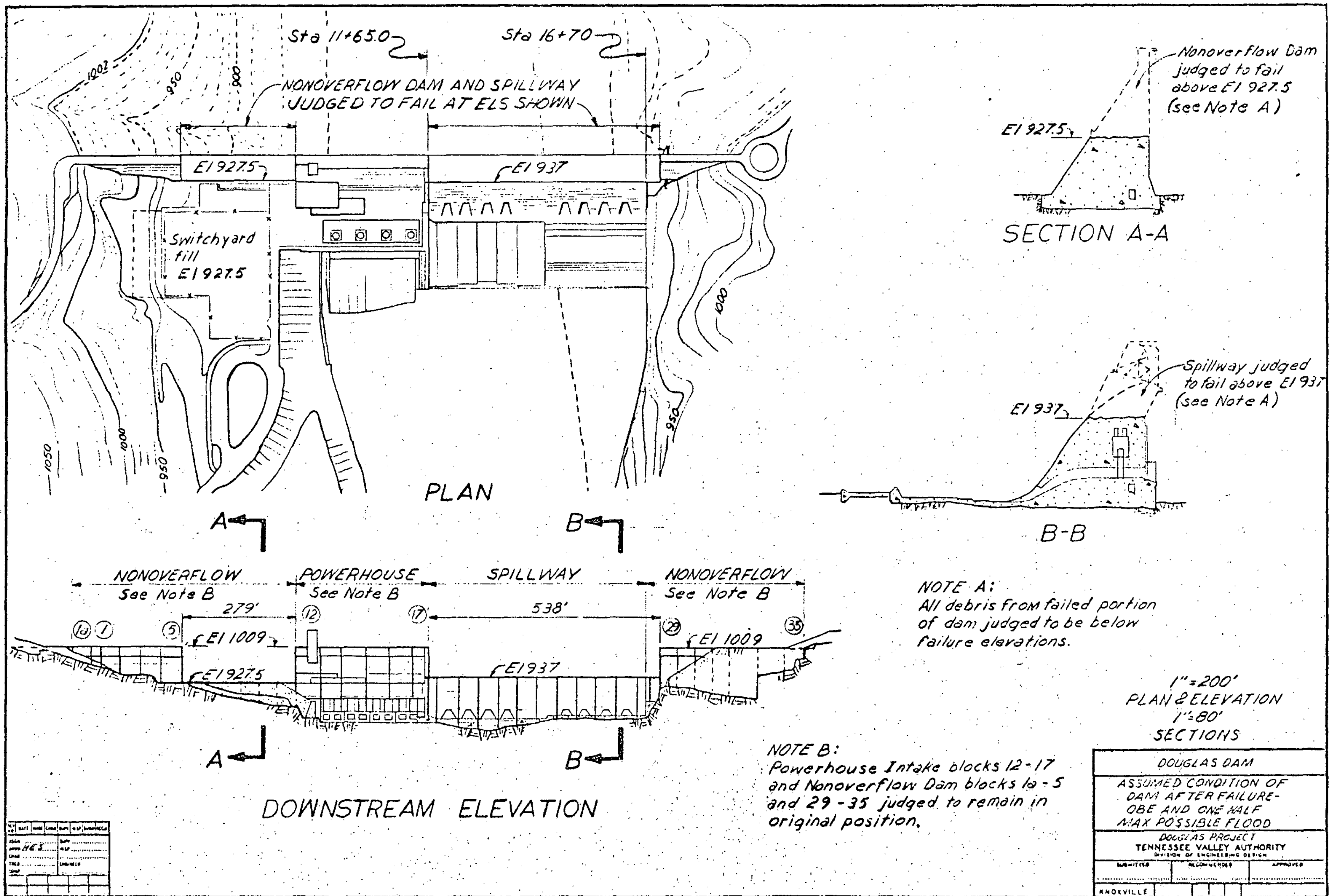
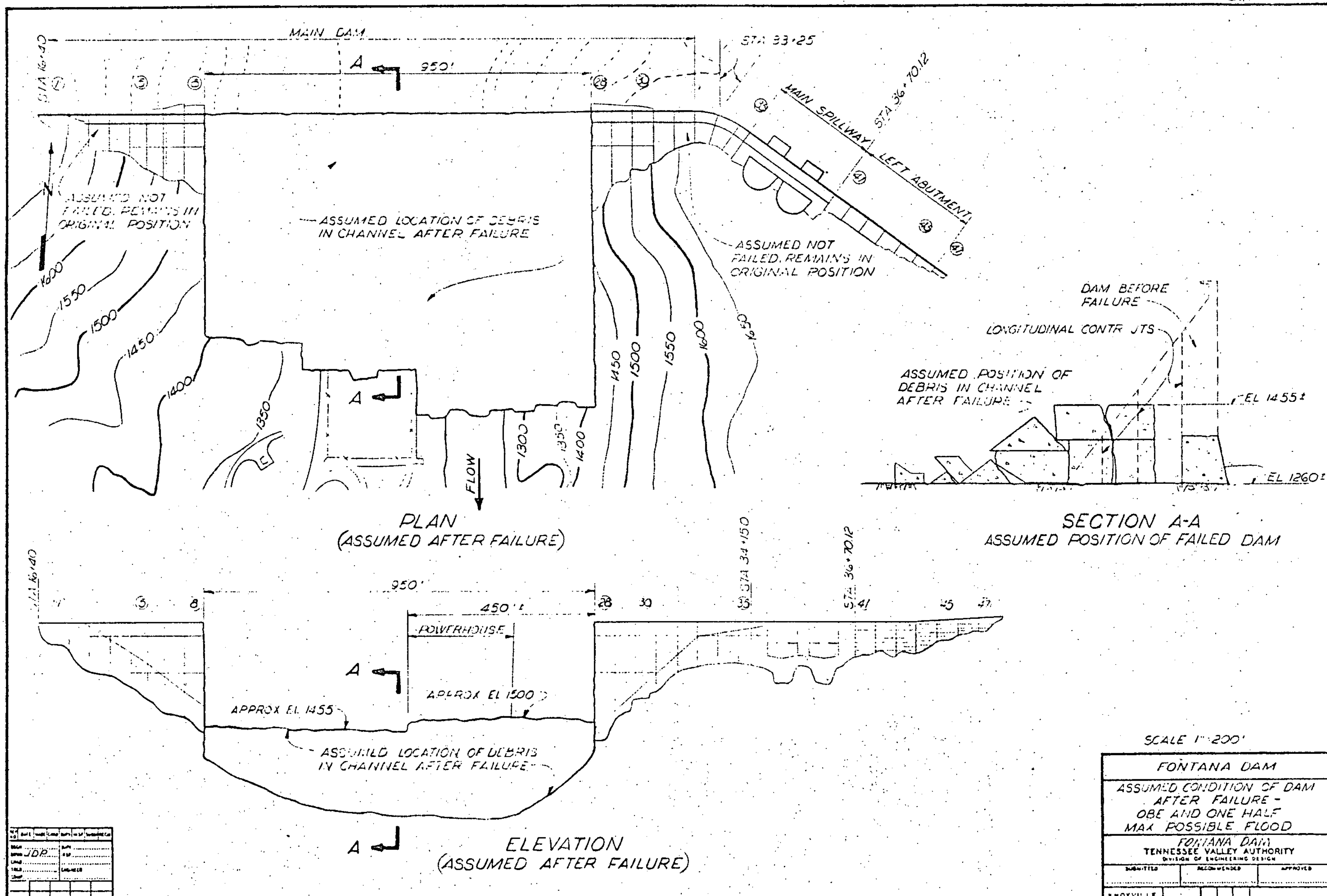


FIG. Q2.11.3-16

EXHIBIT 17



SCALE 1" = 200'

FONTANA DAM

ASSUMED CONDITION OF DAM AFTER FAILURE - OBE AND ONE HALF MAX POSSIBLE FLOOD

FONTANA DAM
TENNESSEE VALLEY AUTHORITY
DIVISION OF ENGINEERING DESIGN

KNOXVILLE

Fig. Q2.11.3-17

NO. 3164 12 BY 10 DIVISIONS PER INCH. 120 BY 70 DIVISIONS. GRAPH PAPER PRINTED IN U. S. A.

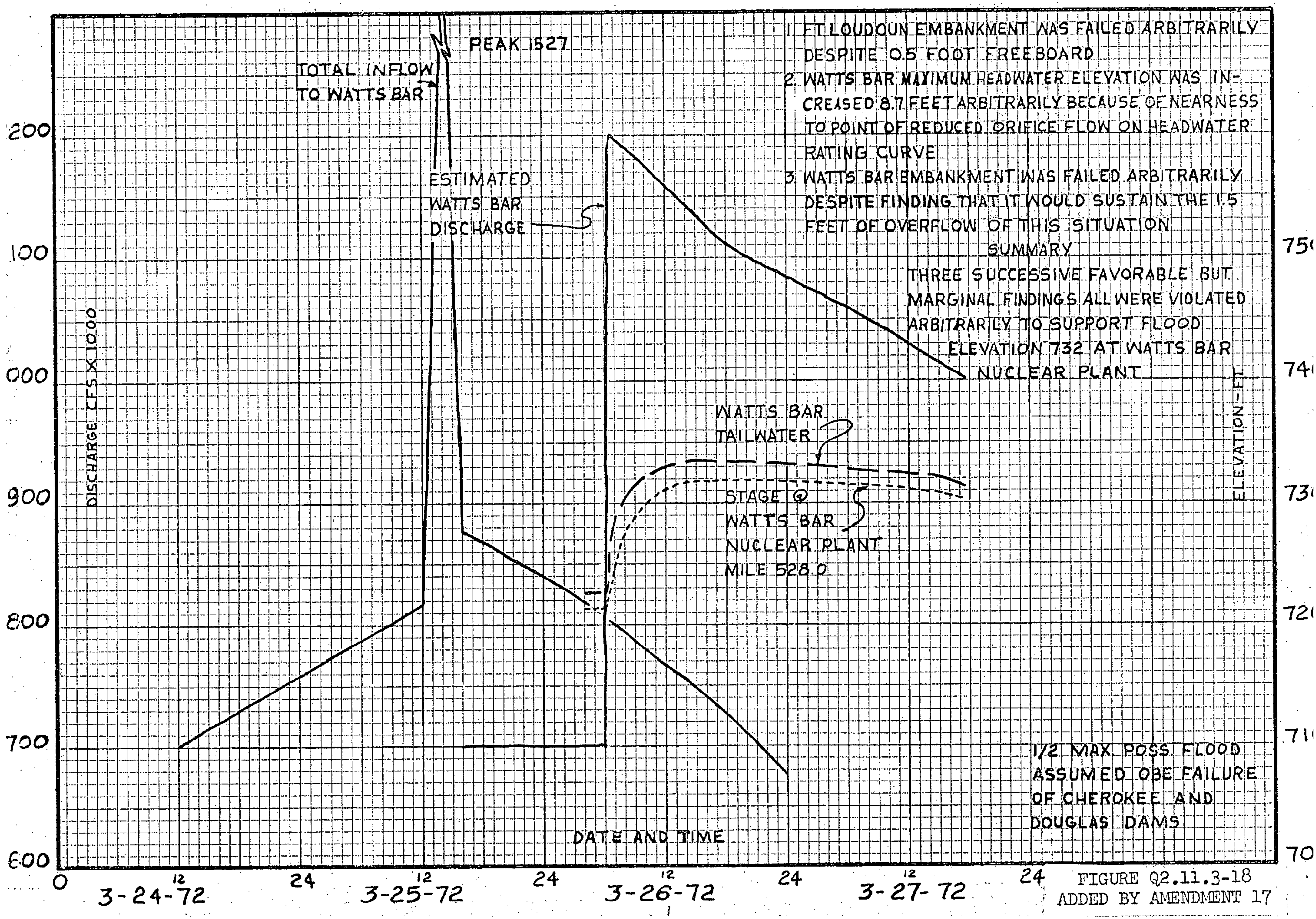
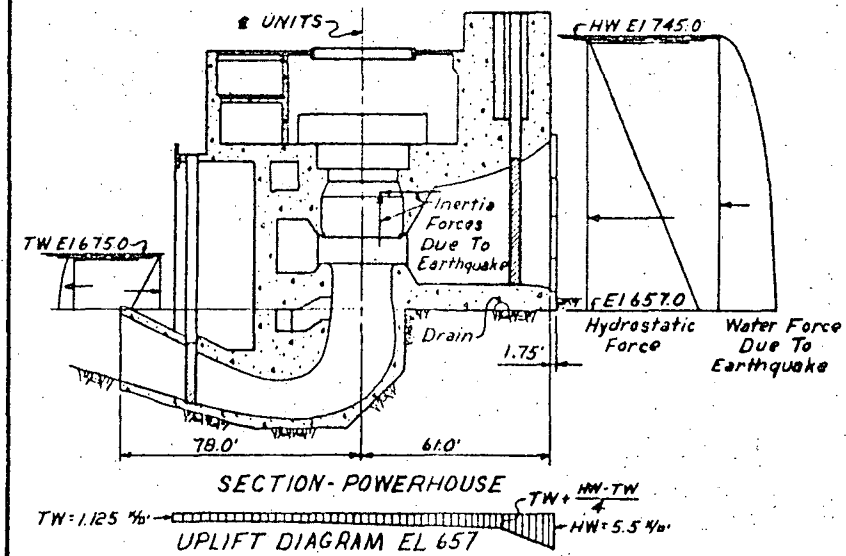


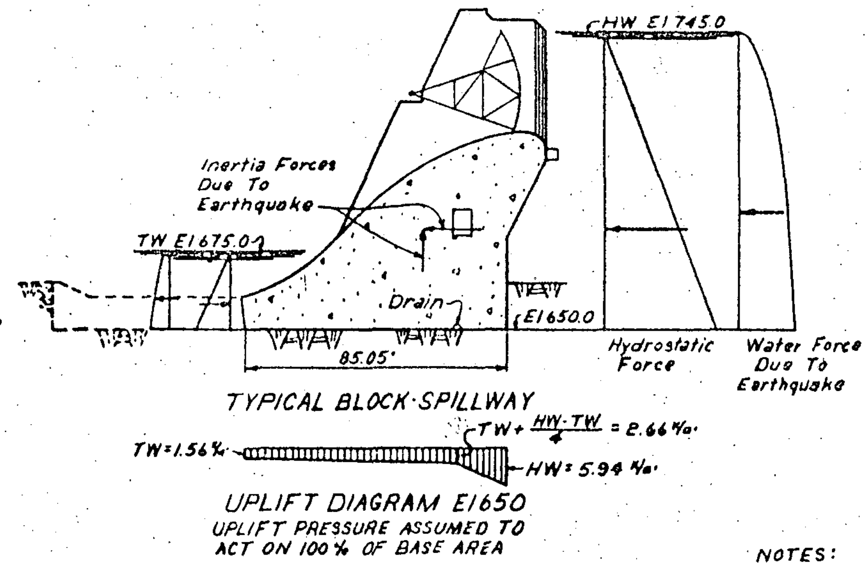
FIGURE Q2.11.3-18
ADDED BY AMENDMENT 17



SECTION- POWERHOUSE

UPLIFT DIAGRAM EL 657

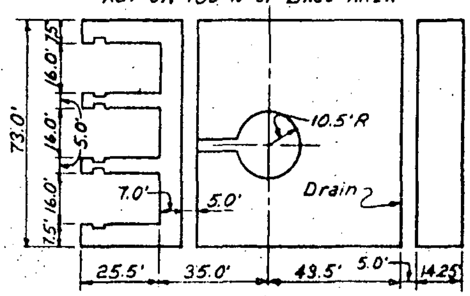
UPLIFT PRESSURE ASSUMED TO ACT ON 100% OF BASE AREA



TYPICAL BLOCK-SPILLWAY

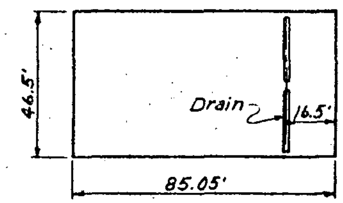
UPLIFT DIAGRAM El 650

UPLIFT PRESSURE ASSUMED TO ACT ON 100% OF BASE AREA

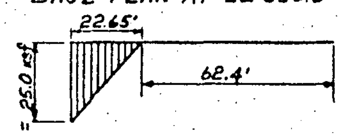


BASE PLAN AT EL 657.0

Note A:
The powerhouse and spillway structures are well keyed into the rock foundation. The rock formations are severely folded with the dip generally in a dnstr direction varing from 10° to 40°. Any failure would require cross bed shear of the rock. Rock of this type has cross bed shear strength much in excess of that reqd for a Factor of Safety of 1.

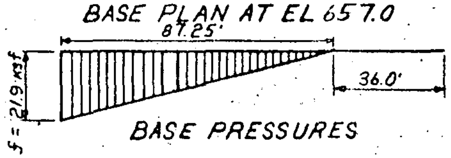


BASE PLAN AT EL 650.0



BASE PRESSURES

NOTES:
1. DBE earthquake inertia forces assumed as 0.18g horizontally and 0.12g vertically.
2. Results given are based on the static analysis as given in ENGINEERING FOR DAMS by Hinds, Creager, & Justin, pp 279-286.
3. For original stability analysis of structures see drawings 41N608 & 51N202 and Watts Bar computation books E-41-2 and E-51-5.



BASE PRESSURES

* Shear, S, that is reqd for Q=1 is calculated from shear-friction formula, $Q = \frac{0.65 V_R + SA}{H_R}$.
A is assumed to be entire base area.

Scale 1" = 40'

V_R	H_R	$\frac{H_R}{V_R}$	Avg Shear, S	S Reqd For Q=1*	f_{max}	$FS = \frac{\Sigma M_R}{\Sigma M_{OT}}$
38,129K	30,535K	0.8	4.6 ksf (32 psi)	0.9 ksf (6 psi) Note A	21.9 ksf (152 psi)	1.54

V_R	H_R	$\frac{H_R}{V_R}$	Avg Shear, S	S Reqd For Q=1*	f_{max}	$FS = \frac{\Sigma M_R}{\Sigma M_{OT}}$
13,144K	19,763K	1.5	5.0 ksf (34.8 psi)	2.9 ksf (20 psi) Note A	25.0 ksf (174 psi)	1.2

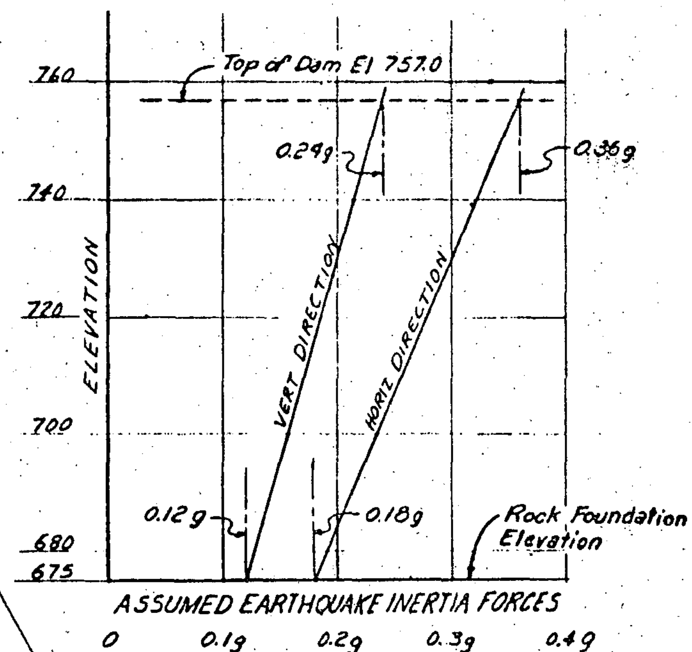
POWERHOUSE & SPILLWAY

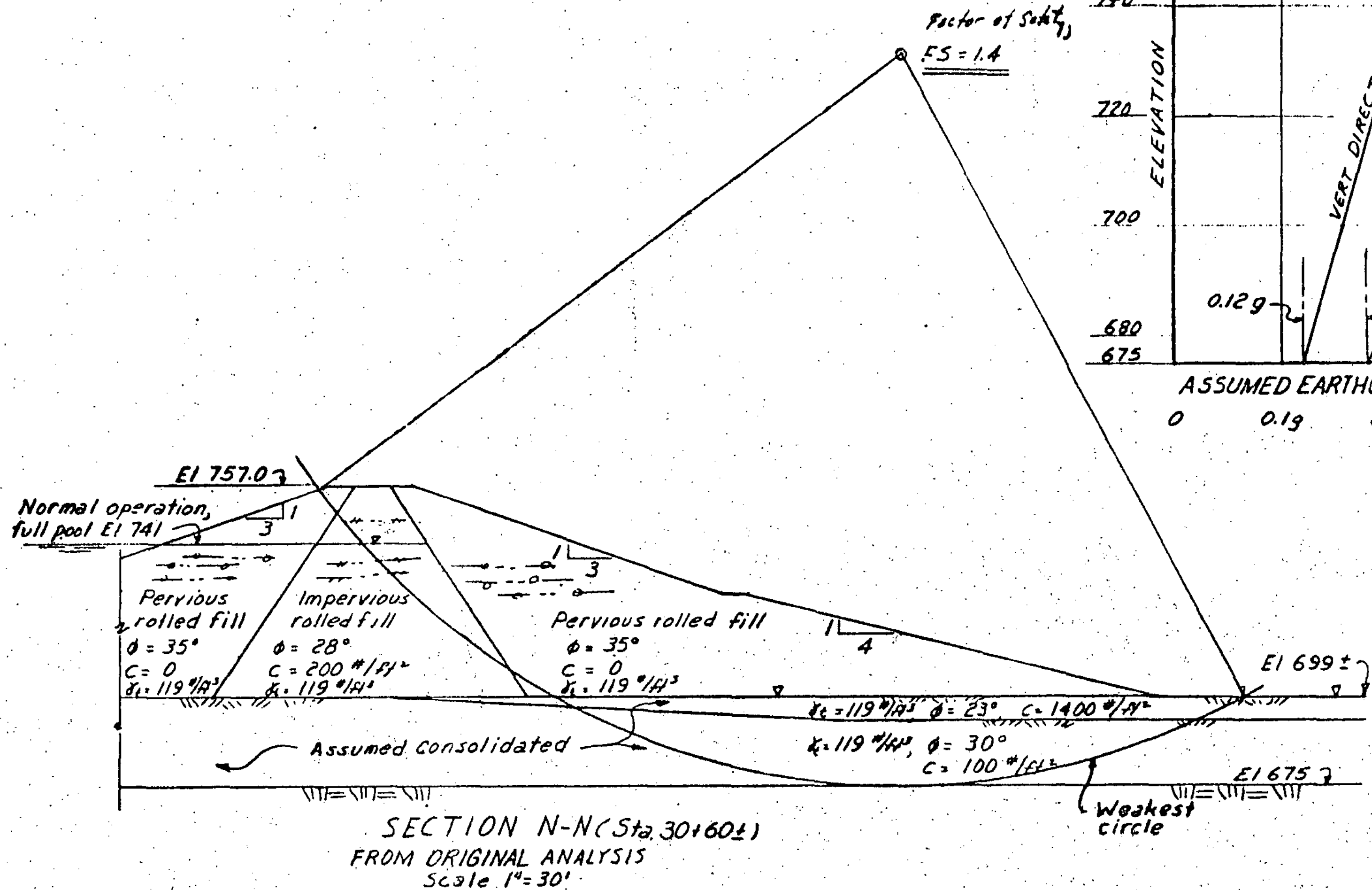
RESULTS OF ANALYSIS FOR DESIGN BASIS EARTHQUAKE

WATTS BAR PROJECT
TENNESSEE VALLEY AUTHORITY

FIGURE 02.11.3-19

"ADDED BY AMENDMENT 5"





SECTION N-N (Sta. 30+60±)
FROM ORIGINAL ANALYSIS
Scale 1" = 30'

Notes:
1. Analysis was made using the standard slip circle method.
2. For the original stability analysis see Watts Bar Computations E-23-2, and "Report on Soil Tests and Stability Analysis, Watts Bar Earth Embankment" Report No 9-115.
3. Shear Strengths of materials same as used in orig analysis.

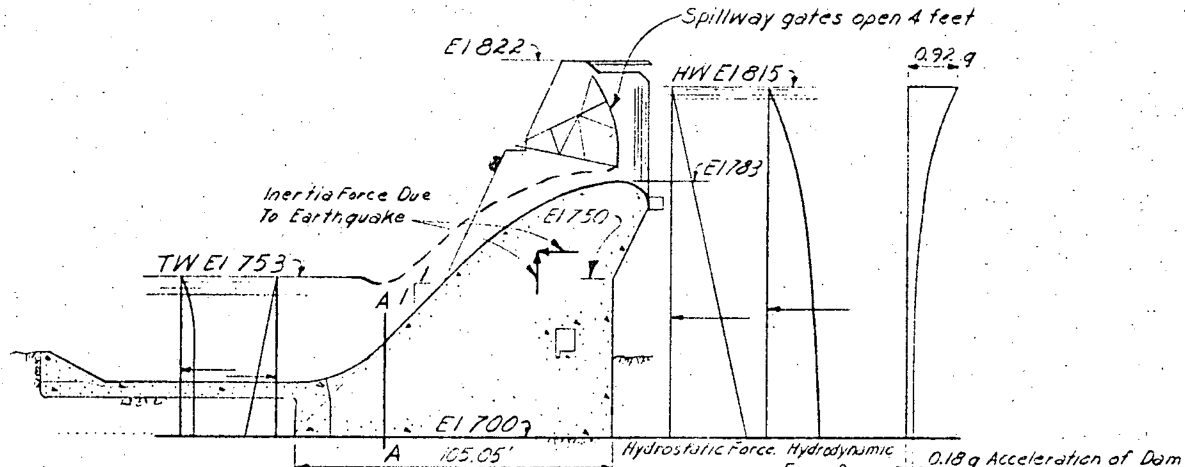
EMBANKMENT

RESULTS OF ANALYSIS FOR DESIGN BASIS EARTHQUAKE

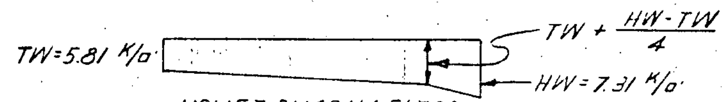
WATTS BAR PROJ

FIGURE Q2.11.3-~~2~~ 20

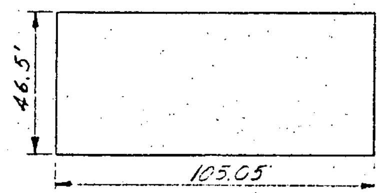
"ADDED BY AMENDMENT 5"



TYPICAL SPILLWAY SECTION



UPLIFT DIAGRAM El 700
UPLIFT PRESSURE ASSUMED TO ACT ON 100% OF BASE AREA



BASE PLAN El 700

NOTES:
1. Vertical acceleration of spillway at base assumed to be 0.12g. By dynamic analysis, amplification of acceleration above the base was determined to be 0.24 g at top.
2. Horizontal acceleration of spillway at base assumed to be 0.18g. By dynamic analysis, amplification of acceleration above the base was determined to be 0.92g at top.
3. Spillway gates assumed open 4 feet for this analysis.

* Shear, s, that is reqd for Q=1 is calculated from shear-friction formula, $\frac{0.65\,\Sigma V + SA}{\Sigma H}$, A is assumed to be entire area.

** For base at El 700 resultant falls outside base under DBE.

BASE PRESSURE**

ΣV	ΣH	$\frac{\Sigma H}{\Sigma V}$	Avg shear	S reqd for Q=1	f max	FS $\frac{\Sigma M_R}{\Sigma M_O}$
18,254 K	29,534 K	1.62	42 psi entire base	25 psi*	* *	0.9

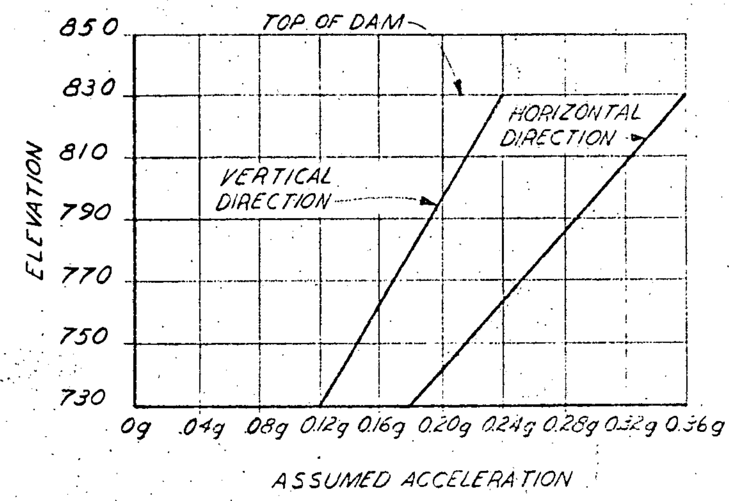
Scale 1" = 40'

SPILLWAY
RESULTS OF ANALYSIS FOR DESIGN BASIS EARTHQUAKE
FORT LOUDOUN DAM
TENNESSEE VALLEY AUTHORITY
DIVISION OF ENGINEERING DESIGN
SUBMITTED RECOMMENDED APPROVED
KNOXVILLE

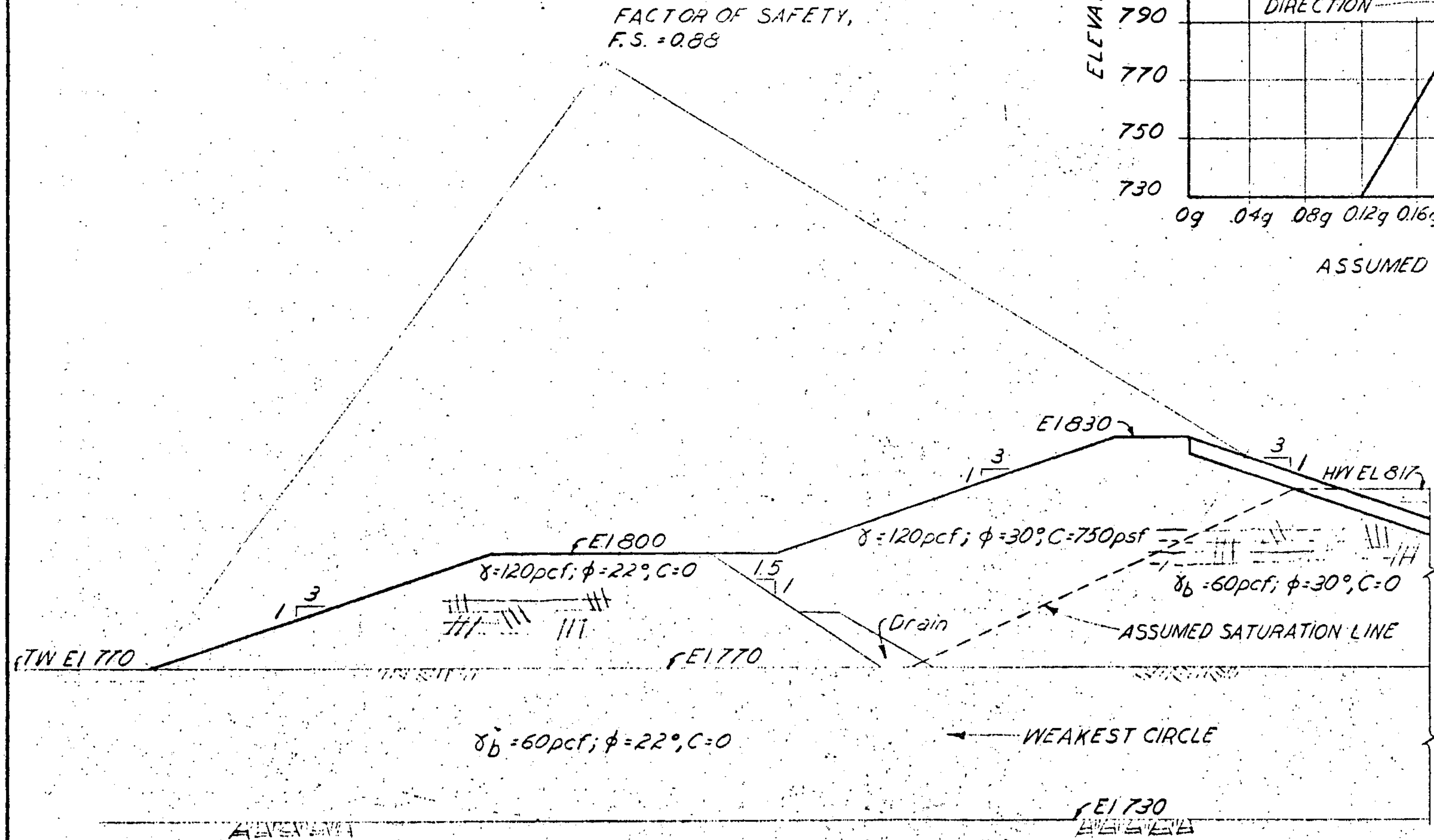
HES

Fig. Q2.11.3-2

EXHIBIT 21



NOTES:
1. Analysis was made using the standard slip circle method.
2. Shear strengths of materials same as used in original analysis.



Scale 1" = 30'

TYPICAL EMBANKMENT SECTION

EMBANKMENT
RESULTS OF ANALYSIS FOR DESIGN BASIS EARTHQUAKE
FORT LOUDOUN DAM
TENNESSEE VALLEY AUTHORITY
DIVISION OF ENGINEERING DESIGN
SUBMITTED RECOMMENDED APPROVED
KNOXVILLE

Fig. Q2.11.3-22

EXHIBIT 22

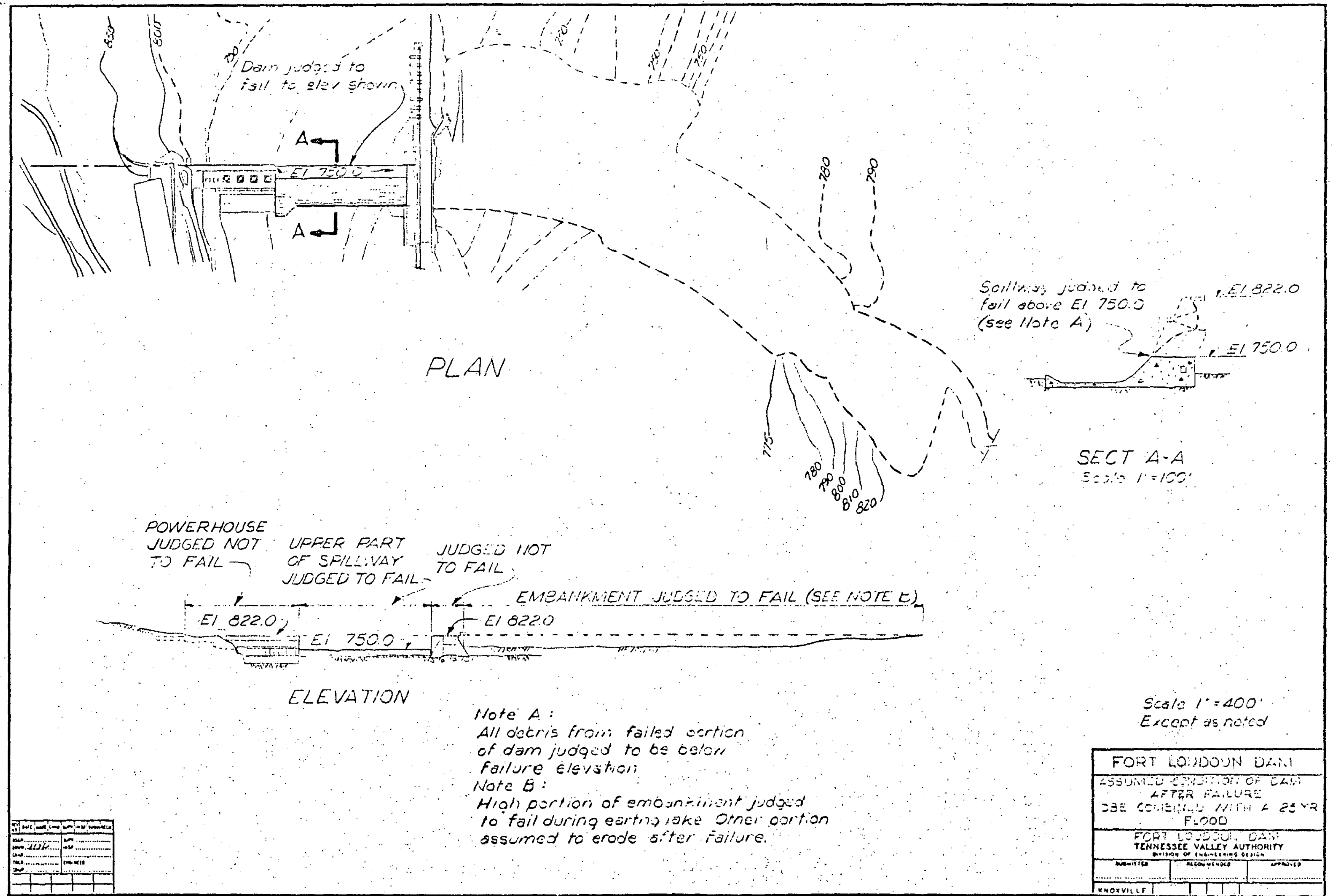


Fig. 2.11.3-23

EXHIBIT 23

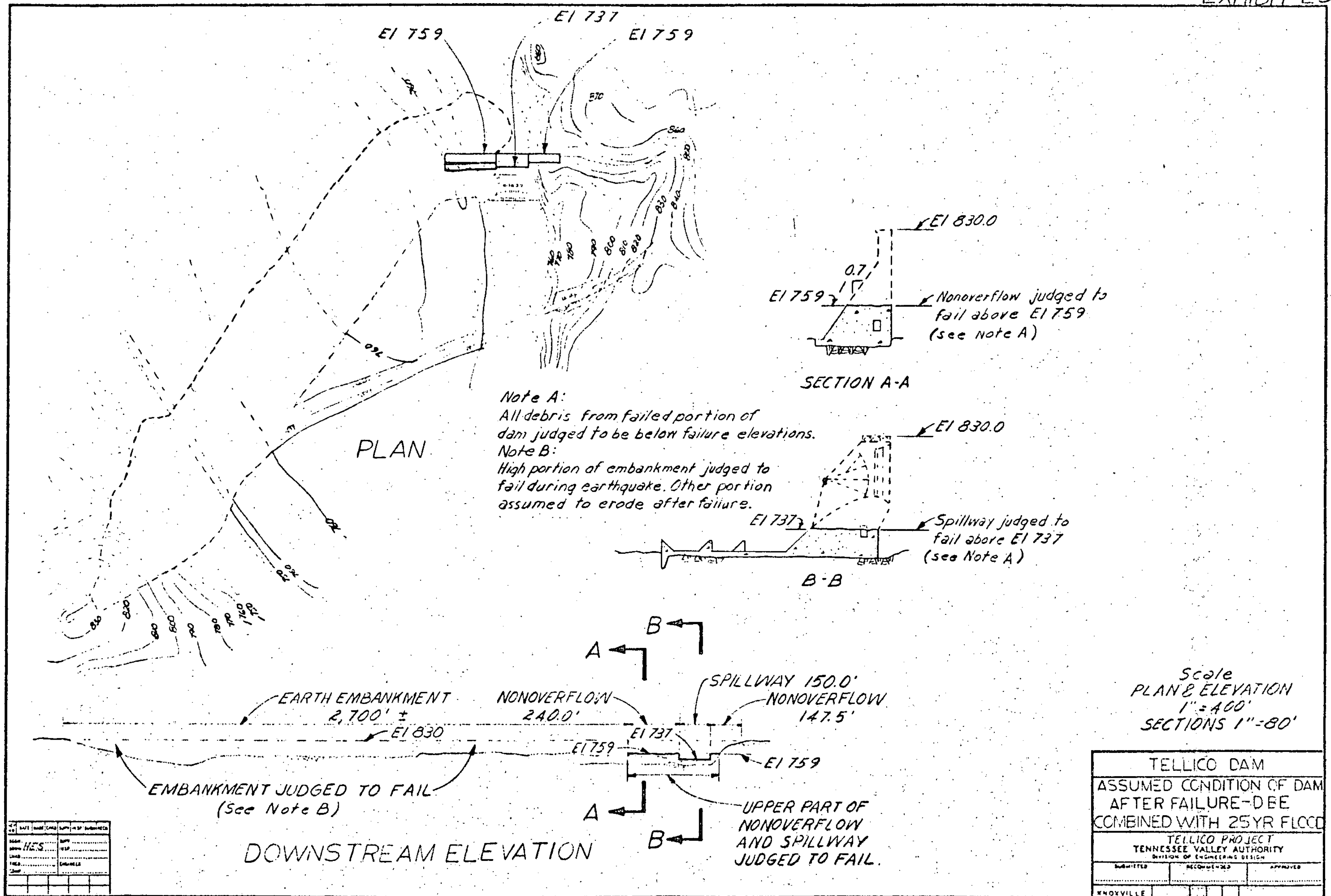
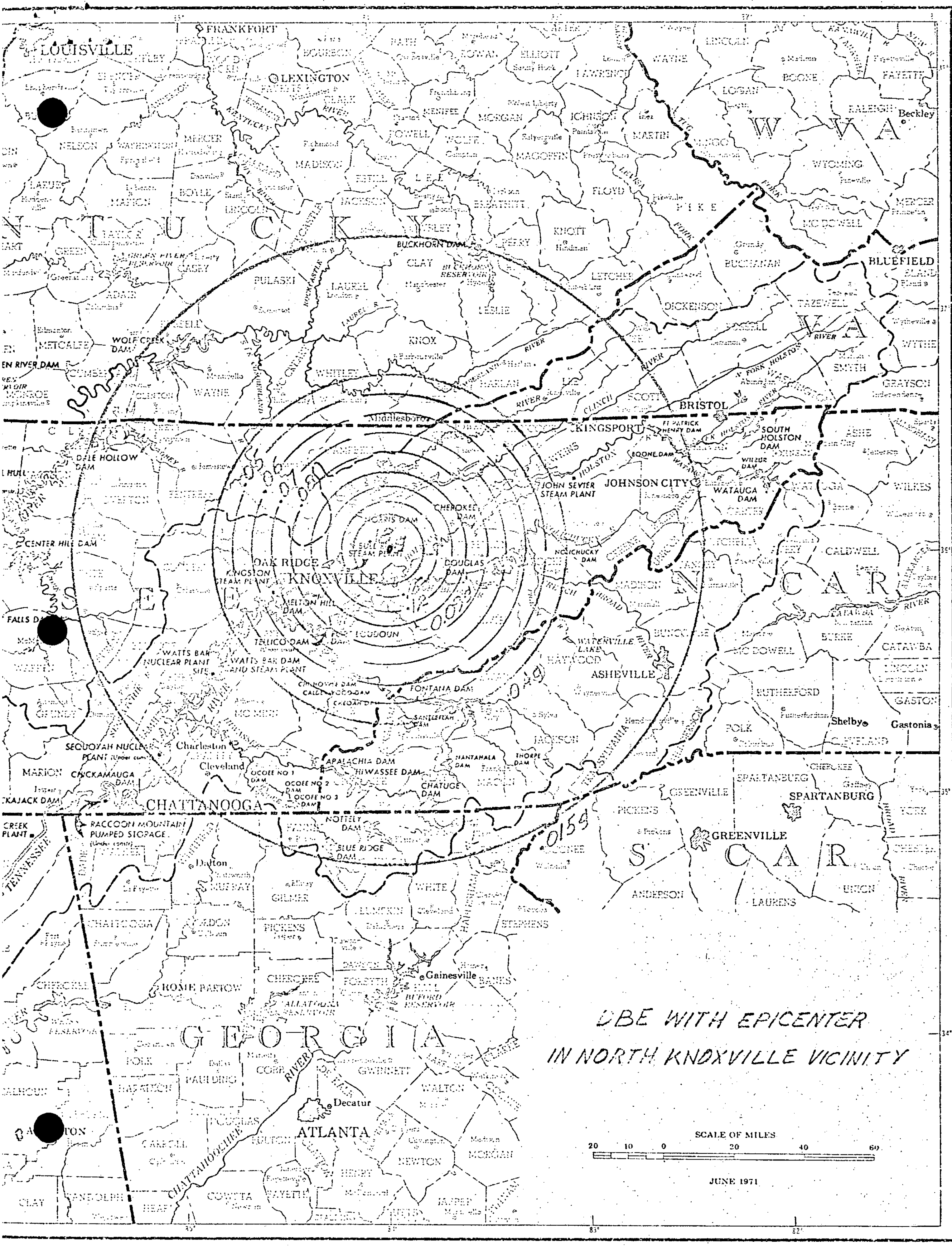


Fig. Q2.11.3-24



DBE WITH EPICENTER IN NORTH KNOXVILLE VICINITY

SCALE OF MILES

20 10 0 20 40 60

JUNE 1971

G-MS-453 B630 R10

Fig. A2.11.3-


DBE WITH EPICENTER
IN WEST KNOXVILLE VICINITY
SCALE OF MILES
20 10 0 20 40 60
JUNE 1971
KNOXVILLE
OAK RIDGE
CHATTANOOGA
ASHEVILLE
JOHNSON CITY
KINGSPORT
BRISTOL
LEXINGTON
FRANKFORT
LOUISVILLE
ATLANTA
Decatur
Gainesville
Dalton
Cleveland
Charleston
Middlesboro
SPARTANBURG
GREENVILLE
Shelby
Gastonia
BLUEFIELD
Beckley
NORRIS DAM
CHEROKEE DAM
DOUGLAS DAM
FORT LOUDOUN DAM
MELTON HILL DAM
KINGSTON STEAM PLANT
WATTS BAR NUCLEAR PLANT SITE
WATTS BAR DAM AND STEAM PLANT
SEQUOYAH NUCLEAR PLANT
CHICKAMAUGA DAM
RACCOON MOUNTAIN PUMPED STORAGE
FONTANA DAM
APALACHIA DAM
HIWASSEE DAM
CHATUGE DAM
NANTAHALA DAM
THORPE DAM
OCOEE NO 1 DAM
OCOEE NO 2 DAM
OCOEE NO 3 DAM
NOTTELY DAM
BLUE RIDGE DAM
NOLICHUCKY DAM
JOHN SEVIER STEAM PLANT
BOONE DAM
FT PATRICK HENRY DAM
SOUTH HOLSTON DAM
WATAUGA DAM
WILBUR DAM
WATERVILLE LAKE
BUCKHORN DAM
WOLF CREEK DAM
DALE HOLLOW DAM
CENTER HILL DAM
ALLATOONA RESERVOIR
BUFORD RESERVOIR
WEISS RESERVOIR
0.099
.079
.03
.0159
GEORGIA
W VA

Fig. 2.11.3-26

Appendix 2.7B

WAVE ANALYSIS FOLLOWING EMBANKMENT FAILURE

In the case of the maximum possible flood, the possibility of a steep-fronted wall of water caused by the final sudden failure of the Watts Bar Dam earth embankment moving downstream and striking the plant was investigated. Two aspects of the problem were examined:

1. The magnitude of a wave striking a ridge on the left bank and being reflected toward the plant on the right bank.
2. The magnitude of a wave caused by a rapid rise of the water surface in the middle of the river adjacent to the plant. This wave would move from the middle of the river toward the plantsite.

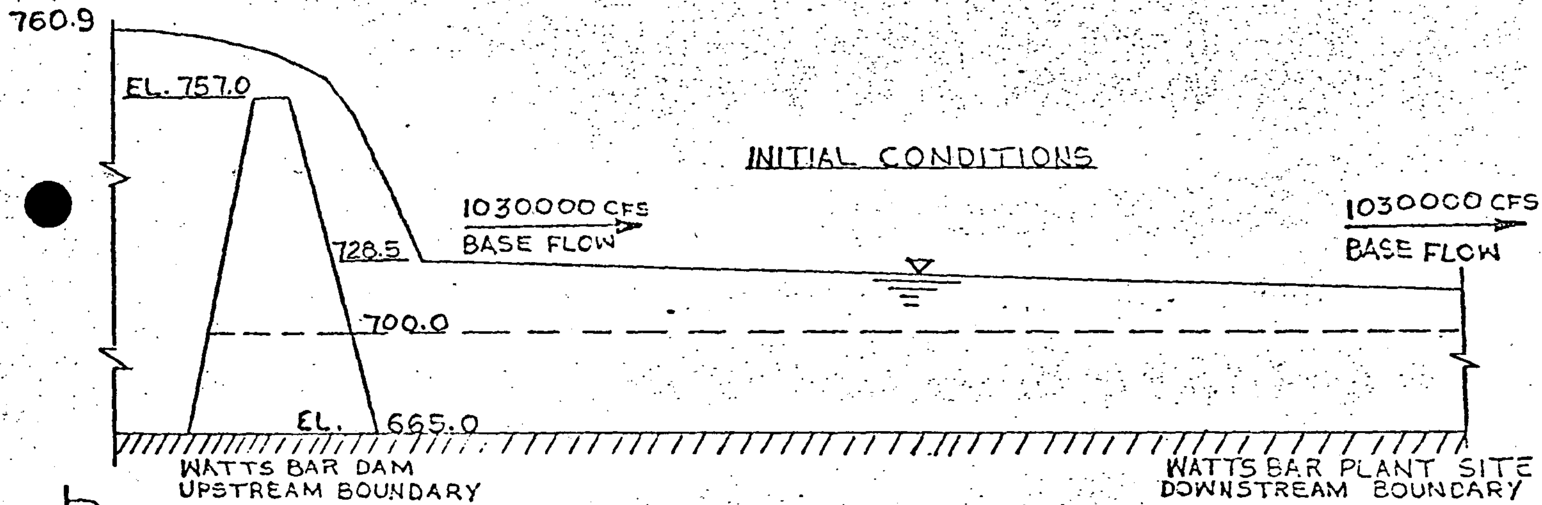
At the end of complete failure of the earth embankment the discharge would increase rapidly from 1,030,000 cfs to 1,310,000 cfs, a sudden increase of 280,000 cfs. The portion of the earth embankment that fails is about 750 feet long and is located on the left overbank where the bottom elevation is about 700 feet, the same elevation to which the dam is finally eroded. A mathematical model channel 750 feet wide, Figure 1, produced a rapid rise of about 12.0 feet in the tailwater with the incremental discharge increase due to the failure prescribed. This is shown on Figure 2. Because the total width of the section is about 3,000 feet, the average tailwater rise over the entire section would be (750/3000) (12.0) = 3.0 feet. This agrees well with the average 2.5-foot rise given in Appendix 2.7A.

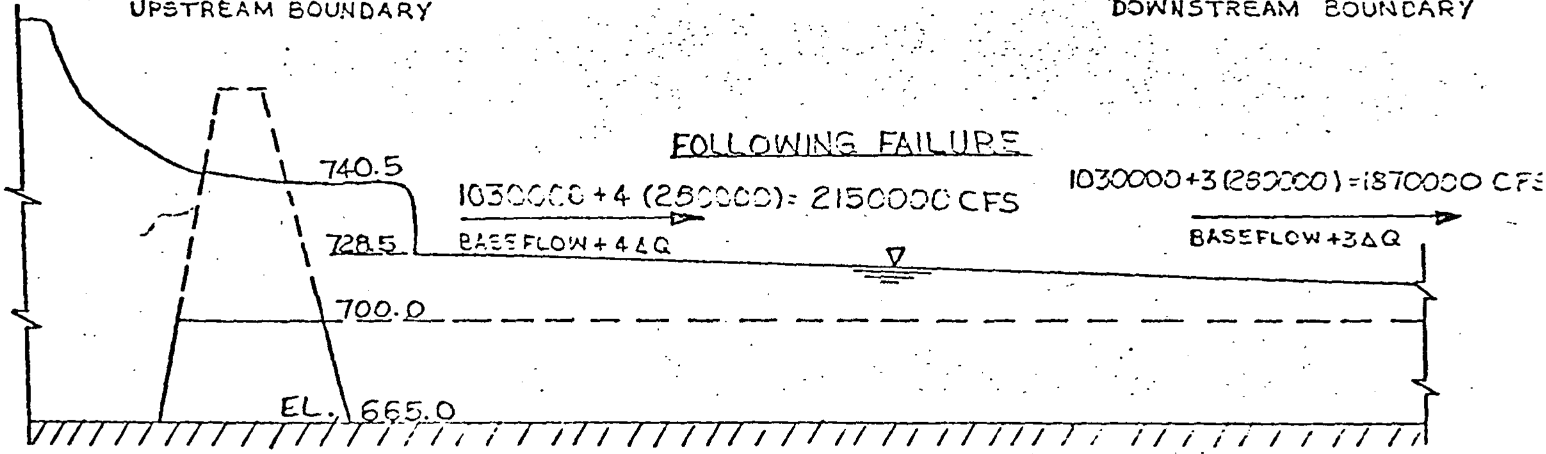
Ignoring any channel bottom elevation differences which would cause the wave to move faster or slower according to the local depth, the wave would expand at about 45° laterally from each side of the 750-foot-wide breach. At the plant location 2 miles downstream from the breach the wave would be expanded over the entire channel width of about 6,500 feet. Thus, the average rise in the river adjacent to the plant would be (750/6500) (12.0) = 1.4 feet. This is about the same rise shown on figure 30 of Appendix 2.7A. It may, therefore, be concluded that the analysis reported in Appendix 2.7A in which the average 2.5-foot tailwater rise was used is valid.

As a further check on the analysis, the following procedure was used with the detailed model (ΔX = 2125 feet) as shown on Figure 3.

At the instant before embankment failure the flow from Watts Bar Dam would be 1,030,000 cfs. Following the failure of the 750-foot earth embankment the flow would increase to 1,310,000 cfs--a ΔQ of 280,000 cfs. From the 750-foot-wide model described above, it was found that a ΔQ of 280,000 cfs caused a 12.0-foot rise in tailwater over the breach width.

An ultraconservative result would be the stage rise adjacent to the plant caused by a hypothetical 12.0-foot rise in tailwater over the entire 3,000-foot width. This procedure requires no assumption about the angle of lateral spreading of the failure wave because the 12.0-foot rise is already artificially spread over the 3,000-foot tailwater width. Because the breach width of 750 feet is one-fourth of the total width of 3,000 feet, a $\Delta Q = 4(280,000) = 1,120,000$ cfs was prescribed at the 3,000-foot-wide tailwater section of the model to produce a 12.0-foot rise. Conditions before and after the earth embankment failure for this study are summarized and illustrated in the following diagram.





The method outlined above accounts for the ΔQ caused by only the 750-foot embankment failure. The stage rise at the downstream boundary (plantsite) would be about 1.6 feet using this procedure, as shown on Figure 4. This figure is conservative since a 12.0-foot rise was used over the entire tailwater width, whereas in reality the 12.0-foot rise occurs only over the 750-foot embankment width.

The second aspect of the problem involves the possibility of the rapid rise in water level in the river adjacent to the plant producing a transverse wave that would be directed toward the plant. This wave would be roughly comparable to the transverse flow which can occur as a flood wave expands from the main channel into an overbank area.

In the Watts Bar case, at the time of embankment failure the overbank water depth is already 20 to 25 feet (the average flood-plain bottom is at elevation 700 feet$\pm$). Therefore, nothing comparable to the transverse flow described above can occur. However, this possibility was studied using a model extending from the center of the river channel to the adjacent plant wall, as shown on Figure 5. The upstream boundary condition for this model was the rapid stage rise which would occur in the river opposite the plant resulting from the unsteady flow routing described in Appendix 2.7A, figure 30.

The downstream boundary condition was zero flow to get the maximum rise at the plant. The output from the model showed a stage rise equal to that which occurred at the upstream boundary as shown on Figure 6. This analysis is valid only through the first reflection at the downstream boundary (about 6 minutes, 3 minutes in each direction). However, the first reflection gives the maximum rise.

Flood Control Branch
November 1972

750' WIDE MODEL

$\Delta X = 2125'$

5 X-SECTIONS

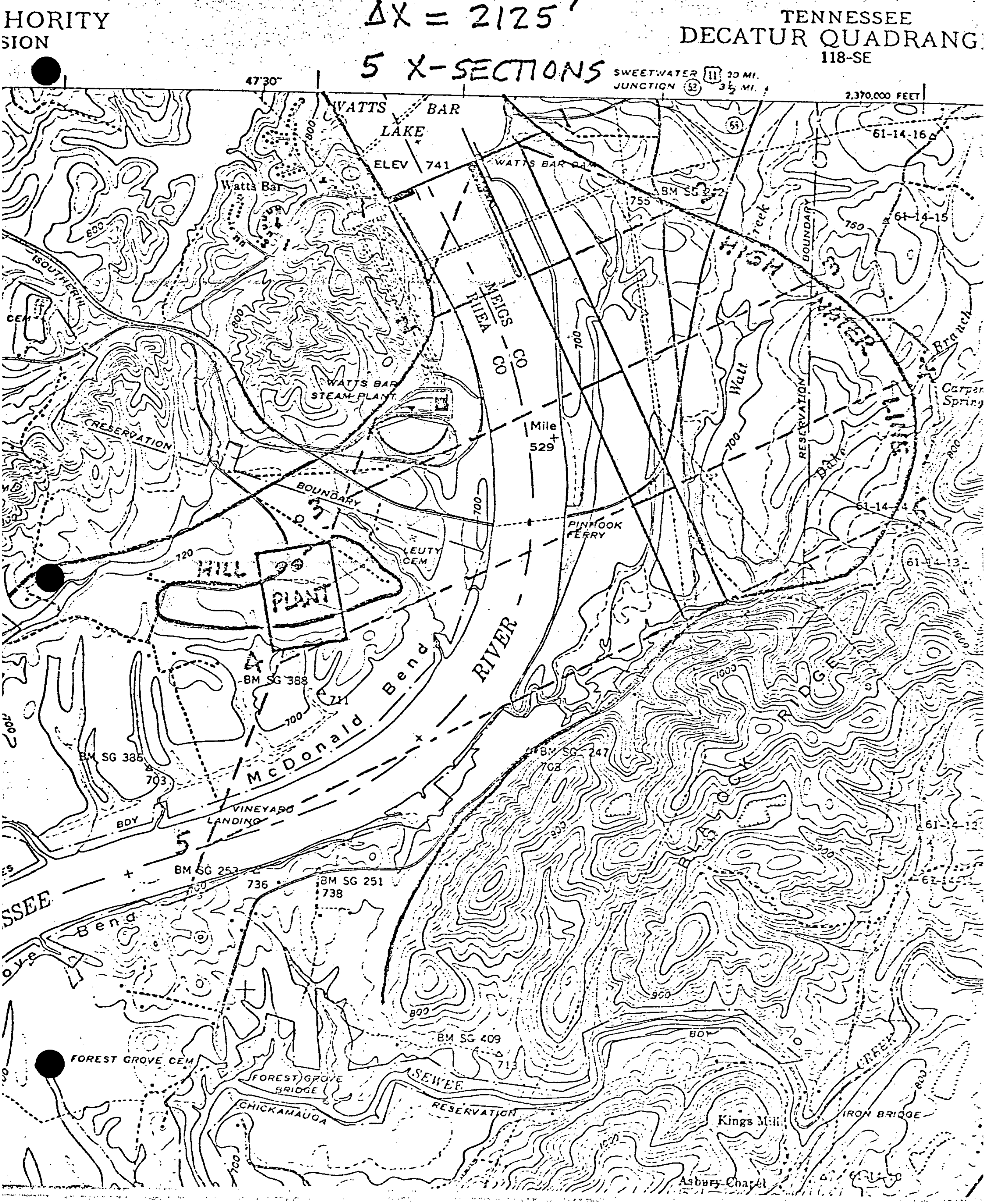
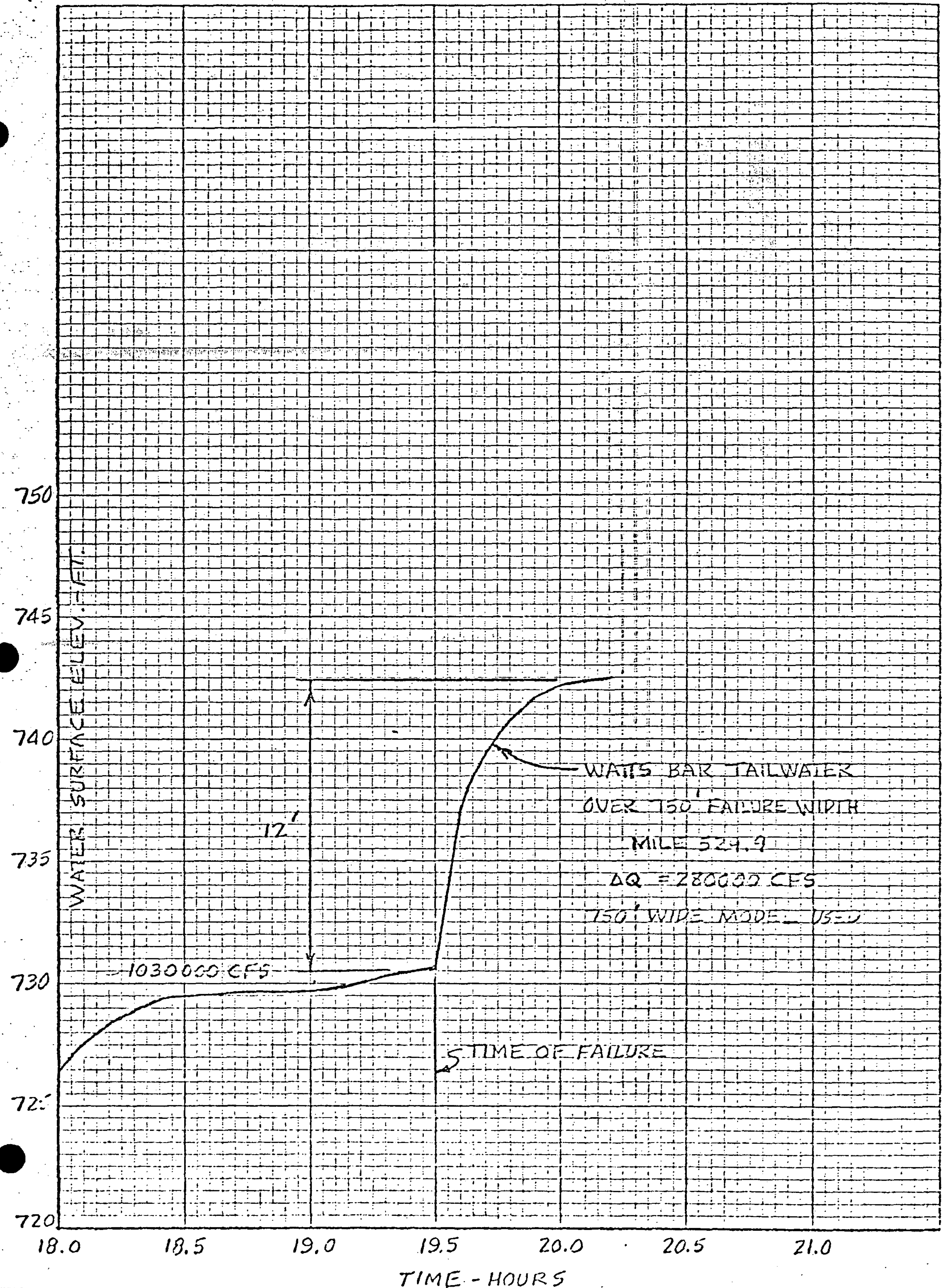


PLATE 2



WATER SURFACE ELEV. - FT.
750
745
740
735
730
725
720
TIME - HOURS
18.0
18.5
19.0
19.5
20.0
20.5
21.0
12'
1030000 CFS
TIME OF FAILURE
WATTS BAR TAILWATER
OVER 750' FAILURE WIDTH
MILE 529.9
ΔQ = 280000 CFS
750' WIDE MODE USED

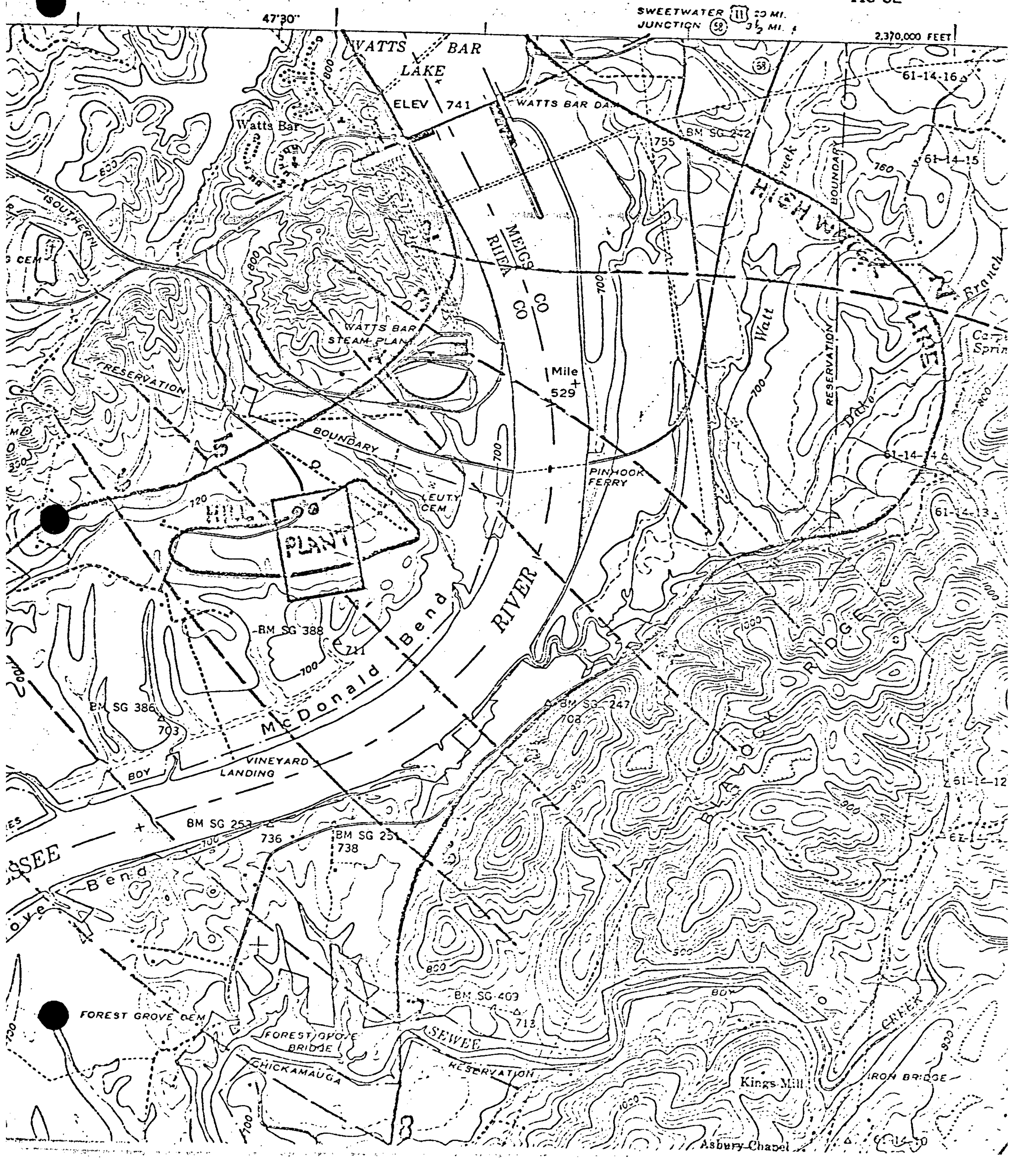
DETAILED MODEL

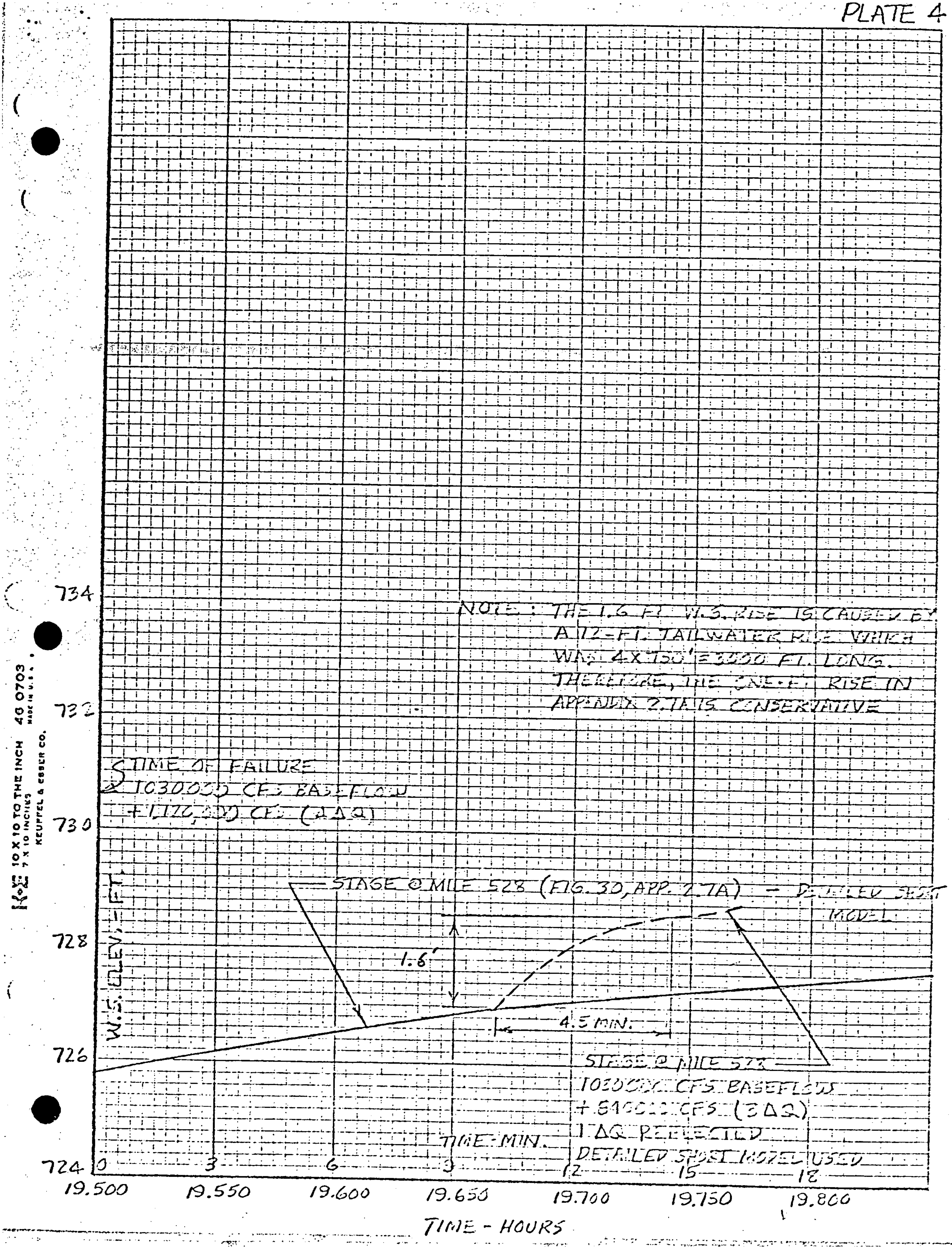
$\Delta X = 2508'$

17 X-SECTIONS

THORITY
SION

TENNESSEE
DECATUR QUADRANG
118-SE




734
732
730
728
726
724
W.S. ELEV. - FT.
NOTE: THE 1.6 FT W.S. RISE IS CAUSED BY
A 12-FT. TAILWATER RISE WHICH
WAS 4 X 750' = 3000 FT. LONG.
THEREFORE, THE ONE-FT RISE IN
APPENDIX 2.7A IS CONSERVATIVE
TIME OF FAILURE
1030,000 CFS BASEFLOW
+ 1,120,000 CFS (2ΔQ)
STAGE @ MILE 528 (FIG. 30, APP. 2.7A) - DETAILED SPORT MODEL
1.6'
4.5 MIN.
STAGE @ MILE 528
1030000 CFS BASEFLOW
+ 840000 CFS (3ΔQ)
1 ΔQ REFLECTED
DETAILED SPORT MODEL USED
TIME-MIN.
0
3
6
9
12
15
18
19.500
19.550
19.600
19.650
19.700
19.750
19.800
TIME - HOURS
K&E 10 X 10 TO THE INCH 7 X 10 INCHES KEUFFEL & ESSER CO. 46 0703 MADE IN U.S.A.

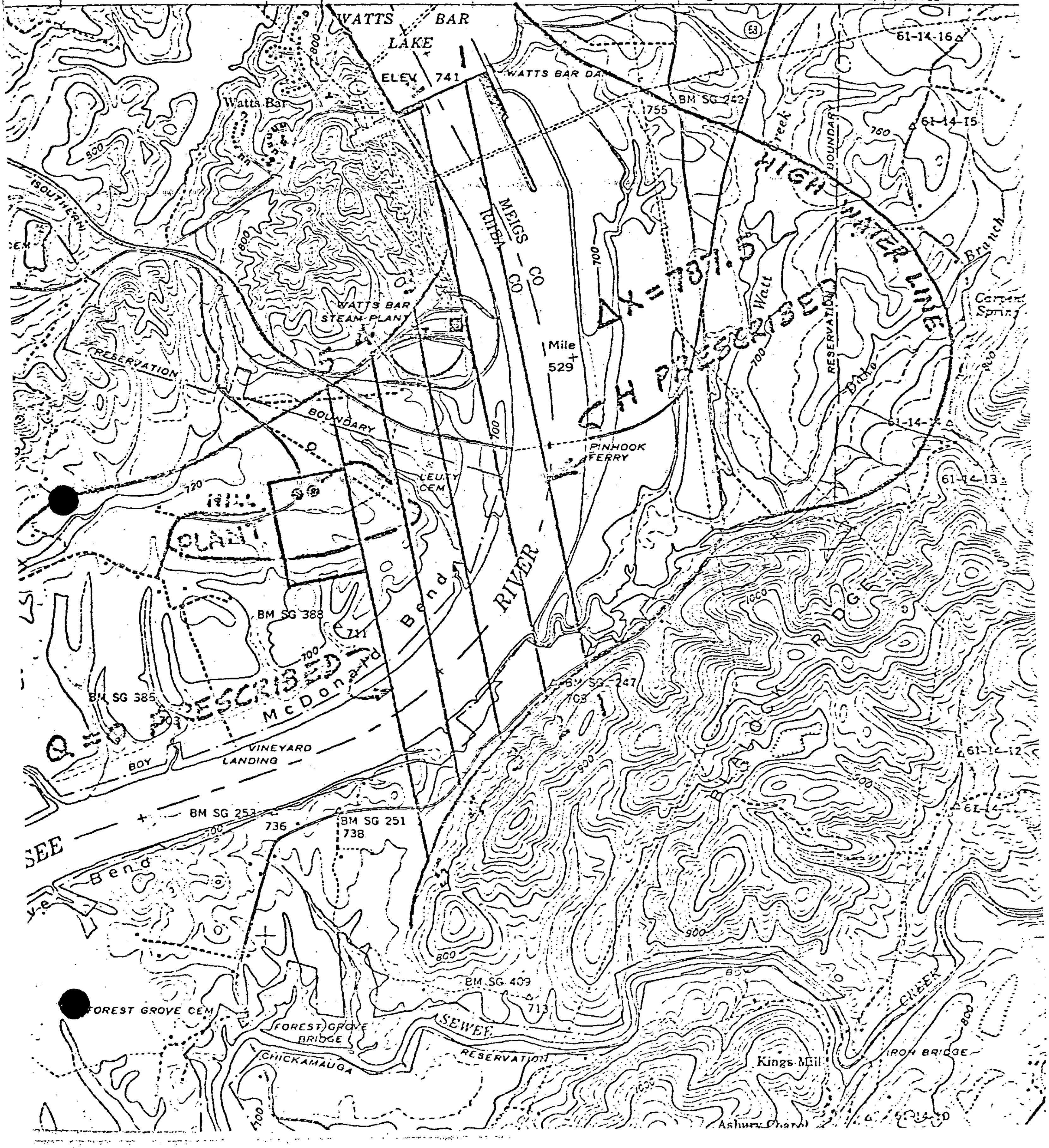
MODEL ⊥ TO PLANT

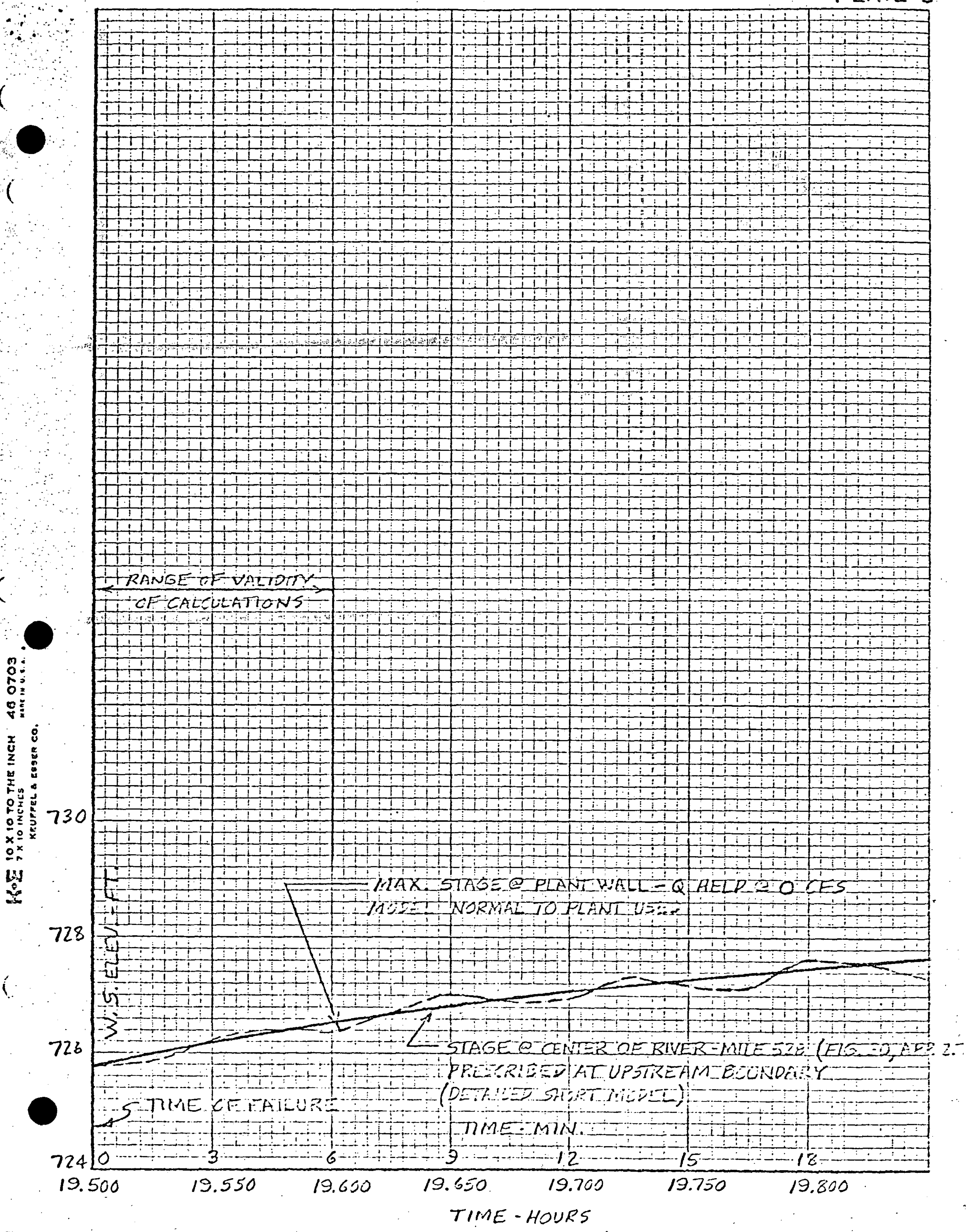
ΔX=787.5'

5 X-SECTIONS

HORITY
ION

TENNESSEE
DECATUR QUADRANGI
118-SE




RANGE OF VALIDITY OF CALCULATIONS
MAX. STAGE @ PLANT WALL - Q HELD @ 0 CFS
NORMAL TO PLANT
STAGE @ CENTER OF RIVER-MILE 528
PRESCRIBED AT UPSTREAM BOUNDARY
(DETAILED SHORT MODEL)
TIME OF FAILURE
TIME - MIN.
0
3
6
9
12
15
18
W. S. ELEV - FT.
724
726
728
730
19.500
19.550
19.600
19.650
19.700
19.750
19.800
TIME - HOURS
K&E 10 X 10 TO THE INCH 7 X 10 INCHES 46 0703
KEUFFEL & ESSER CO.