TEXAS UTILITIES SERVICES INC.
2001 BRYAN TOWER · DALLAS, TEXAS 75201

TXX-2941 January 31, 1979

Mr. R. Naventi
Licensing Project Manager
Light Water Reactors Branch No. 4
Division of Project Management
Office of Nuclear Reactor Regulation
U. S. Nuclear Regulatory Commission
Washington, D.C. 20555

COMANCHE PEAK STEAM ELECTRIC STATION
NRC ROUND ONE HYROLOGY QUESTIONS
DOCKET NOS. 50-445 & 50-446
FILE NO. 10010

Dear Mr. Naventi:

Enclosed are our initial responses to your round one questions on hydrology (Q371.4-Q371.13). As agreed, these are being transmitted to you by letter to expedite response time. These responses will be retransmitted to the Commission in FSAR Amendment 4. Also enclosed in response to the above questions are three copies of drawing numbers FN-SCR-5, 9, 11, 14, 17, 26 and 37.

If you have any questions about this matter, please contact this office.

Sincerely,

Richard Werner

Richard Werner

RAW:tls
Enclosure
cc: H. C. Schmidt

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Q371.4 You have not demonstrated that the service spillway will not fail during the occurrence of a Probable Maximum Flood. According, provide the following additional information regarding the spillway and appurtenant structures.

(1) Provide the height of the spillway chute ides downstream of the crest in the chute. Document the freeboard provided and the basis for its selection. Provide a drawing of the chute showing the height of the sides for the entire length together with a profile of water surface elevations for the Probable Maximum Flood. Provide the "n" values and velocity distribution coefficients that were used and the bases for their selection.

(2) Provide more detailed and larger scale drawings in plan and profile of the approach channel, spillway and appurtenant structures.

(3) Provide a detailed plan view of the transition area between the stilling basin and the spillway discharge channel.

(4) Discuss the gradation limits of the 24 inches and the 48 inch riprap to be provided on the sides of the discharge channel. Provide the median rock size to be used.

(5) Provide the equations used to define the upstream and downstream quadrants of the ogee crest. Also, provide the radius of curvature of the transition between the downstream quadrant and the spillway and the coordinates of the points of tangency.

(6) Define the location and length of the hydraulic jump in the stilling basin and assure that the side walls are of sufficient height to contain this jump.

(7) Provide a tailwater rating curve and a water surface profile in the spillway discharge channel. Discuss the computational technique used to derive this profile.

R371.4 (1) Drawing number FN-SCR-14, transmitted to the commission by letter dated January 31, 1979, shows the water surface profile through the spillway during the probable maximum flood. The chute walls are 9 feet high and provide a minimum freeboard of 4.1 feet. The required freeboard was computed as 3.9 feet (reference: U.S. Bureau of Reclamation, Design of Small Dams, Second Edition, page 393). To determine the wall heights a velocity distribution coefficient of 1.0 and a Manning's n of 0.018 were used for conservatism.

(2) Drawing numbers FN-SCR-11, FN-SCR-14, FN-SCR-26, and FN-SCR-37, transmitted as above, give additional details of the approach channel, spillway and service outlet.

(3) Drawing number FN-SCR-9, transmitted as above shows a detailed plan view of the transition area between the stilling basin and spillway discharge channel.

(4) The riprap was graded within the following limits:

| Percent Lighter By Weight | Limits of Stone Weight (Lbs.) 24-inch Riprap | 48-inch Riprap |
|---|---|---|
| 100 | 607 - 243 | 4859 - 1944 |
| 50 | 180 - 121 | 1440 - 972 |
| 15 | 90 - 38 | 720 - 304 |

The median rock size was 150 pounds for the 24-inch riprap and 1,200 pounds for the 48-inch riprap.

(5) Drawing number FN-SCR-17, transmitted as above, gives details of the spillway crest and transition curve.

(6) The location and length of the hydraulic jump during the probable maximum flood are shown on drawing number FN-SCR-14. This location and length are for a Manning's n of 0.008 and a velocity distribution coefficient of 1.0. The stilling basin walls are 35.5 feet high, and the jump height is 34.2 feet.

(7) The tailwater rating curve for the stilling basin design and the water surface profile in the spillway discharge channel during the probable maximum flood are shown in Figure 371.4. The tailwater rating curve for the spillway was obtained from a backwater analysis of the flow in the spillway discharge channel. The Corps of Engineers' HEC II computer program was used for the analysis. A Manning's n of 0.030 was used for the channel. The water surface elevation at the downstream end of the discharge channel was obtained from a backwater analysis of the existing conditions on Squaw Creek. Studies showed

that the tailwater rating curve was not sensitive to water levels at the downstream end of the discharge channel.

Q371.5 You have not demonstrated that the auxiliary spillway is designed to safely discharge the Probable Maximum Flood without failure. Accordingly, provide the following additional information.

(1) Detailed drawings in plan and profile.

(2) Discuss velocities caused by the Probable Maximum Flood discharge over the spillway and demonstrate that these velocities are low enough to preclude failure of the unlined spillway.

(3) Describe the composition of the spillway crest.

(4) Provide the basis for design of any erosion control structures.

(5) Demonstrate that a Probable Maximum Flood discharge through the spillway will not endanger the Squaw Creek Dam embankment.

(6) Provide a tailwater rating curve.

R371.5 (1) Drawing number FN-SCR-5, transmitted to the commission by letter dated January 31, 1979, is a detailed drawing of the emergency spillway.

(2&3) The maximum velocity along the emergency spillway will be 10 feet per second during the probable maximum flood. Velocities at the downstream edge of the spillway will be higher. These velocities will cause some erosion damage. The erosion will not be severe, as the spillway cut is into limestone. In

places, surface materials (up to 2 feet thick) overlying the limestone consist of softer materials, graded for drainage. The crest of the spillway has a concrete wall at Elevation 783.0, anchored into the limestone. The frequency of operation of the emergency spillway is in excess of 100 years. The erosion damage will not endanger the dam or reduce its storage capacity.

(4) The only erosion control structure is the concrete wall on the spillway crest. Its purpose is to maintain a uniform elevation for the entire length of the crest.

(5) The emergency spillway discharges into a tributary of Squaw Creek whose confluence with Squaw Creek is 7,000 feet downstream of the dam. A water surface profile in Squaw Creek showed that the water level at the toe of the dam will reach elevation 656 during the probable maximum flood. The tributary through which emergency spillway discharges will flow is separated from the dam by a large hill composed of a thin overburden overlying limestone. The Squaw Creek Dam embankment will not be endangered by flows through the emergency spillway.

(6) A tailwater rating curve was not needed for design of the emergency spillway, since the flow will pass through critical depth at the downstream edge of the spillway.

Q371.6 Provide the rip-rap gradation limits for the Safe Shutdown Impoundment Spillway. Provide the velocities through the spillway.

R371.6 There is no riprap in the SSI Dam spillway channel. The channel is excavated into limestone. The velocity through the spillway will reach 10.3 feet per second.

Q371.7 There are many discrepancies between various tables, figures and the text. Some of these are listed below. These and others should be corrected.

(1) Page 2.4-31 shows the storage of Squaw Creek Reservoir (SCR) at elevation 775 feet to be 151,953 acre-feet while Table 2.4-17 shows this as 150,953 acre-feet.

(2) Page 2.4-31 also shows the area of SCR at elevation 770 feet to be 3043 acres while Table 2.4-17 shows this as 3084 acres.

(3) Page 2.4-14 shows the area of SCR at elevation 775 feet to be 3,228 acres while Table 2.4-17 shows this as 3272 acres.

(4) Page 2.4-31 shows the storage of SCR at levation 770 as 135,062 acre-feet while page 2.4-49 shows this as 135,360 acre-feet.

(5) Page 2.4-1 shows the elevation of the operating deck of the service water intake structure as 796 feet while page 2.4-15 states this 795 feet.

(6) Page 2.4-19 shows the maximum water surface in the Safe Shutdown Impoundment (SSI) to be 790.7 feet while Table 2.4-15 shows 791.8.

(7) Page 2.4-29 and Figure 2.4-14 show the effective fetch of the SCR as 1.28 miles while page 2.4-18 and page 2.4-32 show 1.56 miles.

(8) Page 2.4-37 shows the effective fetch for the SSI as 0.42 mile while figure 2.4-15 shows 0.36 miles.

(9) Riprap thickness on page 2.4-37 should be 24 inches instead of 24 feet as shown.

(10) Note on bottom of Table 2.4-24 makes reference to figure 2.4.13.2.1.2-1. Shouldn't this be figure 2.4-33?

(11) In section 2.5.4.6 you state, "As discussed in Section 2.5.4.5, groundwater was not encountered in the primary unweathered Glen Rose Limstone." Section 2.5.4.5 does not contain this description. Please correct this reference.

(12) Figure 2.5.5-77 shows the piezometric level of boring P-9 at a minimum elevation of 750 feet, but page 2.5-133 states that the piezometric level in boring P-10 is at elevation 670 feet. Furthermore, the logs of these two borings show the base of the Glen Rose formation at elevation 610. This means that the static water levels are 60 feet and 140 feet above the base of the Glen Rose formation for borings P-9 and P-10, respectively. Explain then your statement in the previous question that groundwaer was not encountered in the Glen Rose Limestone.

(13) On page 2.5-133 you state that the static water level in the Twin Mountains formation was observed in boring P-10 at elevation 670 feet. As mentioned above, elevation 670 is in the Glen Rose formation.

(14) On page 2.5-133 you state that groundwater observations for piezometers installed at the site are provided on fig. 2.5.5-5. This should be fig. 2.5.5-77.

R371.7 The following changes have been made to the FSAR as indicated:

(1) The Squaw Creek Reservoir storage at elevation 775, shown on page 2.4-31, should be 150,953 acre-feet.

(2) The Squaw Creek Reservoir area at elevation 770, shown on page 2.4-31, should be 3,084 acres.

(3) The Squaw Creek Reservoir area at elevation 775, shown on page 2.4-14, should be 3,272 acres.

(4) The Squaw Creek Reservoir storage at elevation 770, shown on page 2.4-49, should be 135,062 acre-feet.

(5) The elevation of the Service Water Intake Structure operating deck, shown on page 2.4-19, should be 796 feet.

(6) The SSI maximum water surface elevation shown on page 2.4-19, should be 790.5. Table 2.4-15 has been revised and now shows the same maximum elevation.

(7) The effective fetch of SCR is 1.28 miles. Pages 2.4-18 and 2.4-32 have been corrected.

(8) The effective fetch for the SSI is 0.36 miles. Page 2.4-37 has been corrected.

(9) Page 2.4-37 has been corrected to show a riprap thickness of 24 inches.

(10) Table 2.4-24 has been changed to reference Figure 2.4-33.

(11) The reference by Seciton 2.5.4.6 to Section 2.5.4.5 has been corrected.

(12&13) The groundwater in the Twin Mountains formation is under artesian pressure. The piezometric level in the Twin Mountains as measured by Boring P-10 is at 670 feet. However, due to the imprevious Glen Rose overlying the Twin Mountains, a boring must penetrate into the Twin Mountains, to allow the water to rise to its piezometric level.

The piezometer in Boring P-9 was blocked, as shown on Figure 2.5.5-77, and no reading was possible. This piezometer was reinstalled, but only to Elevation 749.8, which is in the Glen Rose formation. The readings show that after a couple of months, the piezometer ws dry.

(14) The reference to Figure 2.5.5-5 on page 2.5-133 was to location of borings. In the same paragraph it does say that the observations are summarized in Figure 2.5.5-77.

Q371.8 In developing hydrographs for flood analyses, you divided the SCR catchment into three areas, the upper and lower areas and the area within the reservoir. It appears that only the first two were considered in developing a Probale Maximum Flood. Provide additional information showing that the area within the reservoir was considered, or revise your computations by assuming that all of the Probable Maximum Precipitation which falls on the reservoir contributes to the total Probable Maximum Flood.

R371.8 The flood routings for Squaw Creek Reservoir and the Safe Shutdown Impoundment did include allowance for rainfall on the surface of the reservoirs. Tables 2.4-11 and 2.4-15 have been expanded with additional columns to show the rainfall volumes included.

Q371.9 The available storage in the SSI will be reduced by sediment depletion from 367 acre-feet to about 300 acre-feet during the life of the plant. Discuss sedimentation effects on the service water intake structures. Provide assurance that the intake will not be clogged. Discuss your monitoring and maintenance programs that will be implemented to detect and remove sediment.

R371.9 The SSI is a quiescent body of water. It receives silt form a small drainage area and, therefore, sedimentation buildup is expected to be slow and gradual. There is no formal program to directly monitor sedimentation accumulation at the service water intake structure. However, if sedimentation begins to significantly accumulate at the service water intake structure, it would be indirectly detected during the Technical Specification required monthly service water pump surveillance tests performed in accordance with ASME, Section XI. Since significant sedimentation accumulation occurs over a long period of time, the monthly interval established by the Technical Specifications for the service water pump surviellance test is sufficient for detecting the gradual pump performance deterioration due to sedimentation. When significant service water pump performance deterioration is determined from the results of surveillance test trends and it is determined to be the result of sedimentation accumulation, then the sedimentation will be removed to restore pump performance.

Q371.10 Provide the basis for your conclusion that water from the Service Water Discharge Structure enters the SSI at a point remote enough from the Service Water Intake Structure and at a velcity high enough to ensure adequate mixing, dispersion and evaporative cooling of the effluent.

R371.10 The service water system effluent flows through an open channel type discharge canal prior to entering the SSI. Cooling occurs throughout the approximately 1300 ft. length of the discharge canal before the service water effluent mixes with the SSI bulk fluid. The discharge canal outlet enters the SSI in a direction away from the service water intake structure. Mixing and dispersion of the effluent occurs when the SSI fluid reverses direction in order to pass to vicinity of the intake structure. The minimum shore line distance between the discharge canal and intake structure is 1500 ft. Further cooling of the mixed fluid occurs en route. Because the intake is submerged and the discharge is on the surface, there is vertical as well as horizontal separation. See Figure 9.2-2.

Q371.11 Provide the basis for your statement that an effective porosity of 0.28 is conservative. (Section 2.4.13.3.3)

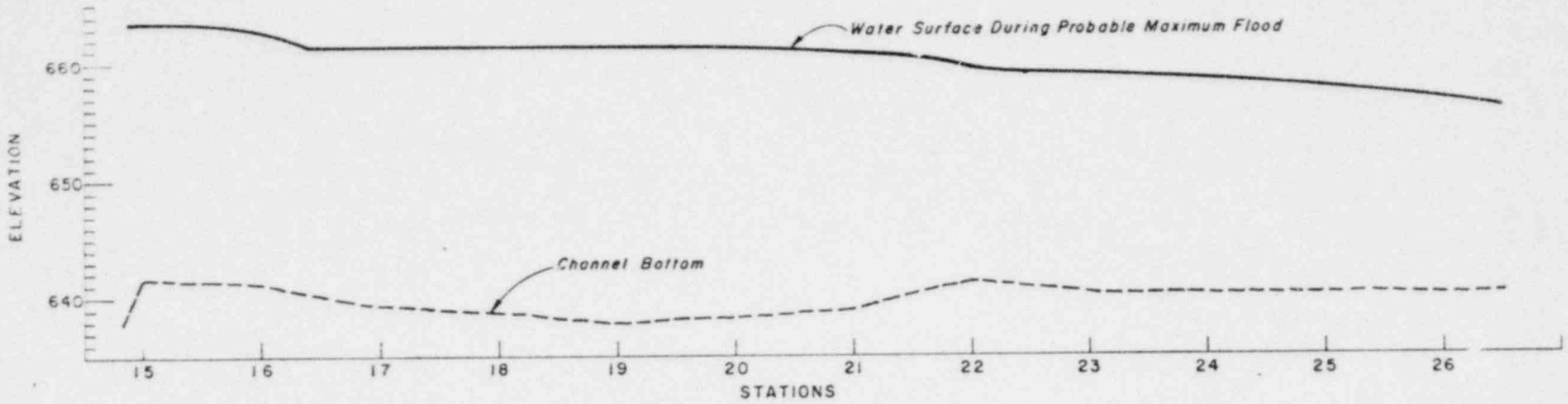
R371.11 Based upon a wet density of 135 pounds per cubic foot for the Twin Mountains formation, the porosity could vary between 0.28 and 0.36 for specific gravities of 2.65 and 2.8 respectively. However, the lower the porosity, the faster the seepage velocity and the greater the rate of dispersion. Therefore, since the lower bound of the porosity was used in the dispersion computations, the results obtained are conservative.

Q371.12 Explain your statement in section 3.8.5.1.5 that, "ground water is not expected to reach higher that 775 feet because of the impermeable nature of the rock," when in figure 2.5.5-77 you show piezometric water levels as high as 830 feet and the packer test results shown in table 2.5.6-1 indicate that the Glen Rose formation is not uniformly of low permeability but rather contains more permeable lenses.

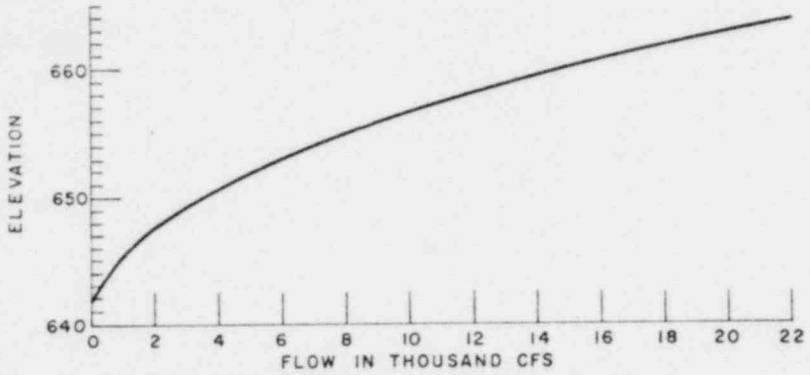
R372.12 All peizometric levels recorded on Figure 2.5.5.77 are measures of pearched water in the upper zone of the Glen Rose Formation measured in the immediate area of each piezometer. These water levels resulted from surface run-off and are not a true measure of any permanent groundwater in the formation. These piezometers were installed during preliminary design work at the site and before the plant site was excavated to plant grade (elevation 810). In Table 2.5.6-1 only zones at a depth range of 194 feet to 214 feet (elevation 649.04 to 629.04 feet) recorded any water loss during the Packer Test. By reviewing the Log of Boring for Boring P-10 it can be seen that zone of sandstone and sand lenses are the cause of the water losses recorded in the Packer Test. The remaining Glen Rose Formation is uniformly of low permeability in its in situ state. However, excavation and the subsequent backfilling with pervious material of the duct banks, piping etc. throughout the plant site has changed the rock formation from an impervious material to a factured and previous material. The horizontal and vertical extent of the fractures and backfilling varies over the site. A field monitoring program will be established, as soon as construction activities permit, to determine if groundwater conditions have changed at the site.

Q371.13 You have not demonstrated that your subsurface groundwater design level, which is normal maximum water level in Squaw Creek Reservoir, elevation 775 feet, is conservative. We note that the water level in borehole P-4, which is located between the two reactor units, fluctuated between elevation 780 and 830 during the period when water level observations were made. (Figure 2.5.5-77) You should therefore, substantiate and show by pertinent analyses that your design groundwater levels will never be exceeded. Alternately, you should use a more conservative groundwater design level.

R371.13 See response to question 371.12.




PROFILE OF SPILLWAY DISCHARGE CHANNEL




TAILWATER RATING CURVE AT STATION 14+87

COMANCHE PEAK S E S
NUCLEAR PLANT
UNITS 1 and 2

FLOW IN SPILLWAY
DISCHARGE CHANNEL

FIGURE 371.4

These values are indicative of the magnitude of precipitation losses expected on the SCR catchment due to its similarity to the Paluxy watershed in regard to general topography, geology, soil types and land usage. The values require evaluation, however, in light of some differences between the two watersheds, including: 1) the SCR catchment is generally not as steep as the Paluxy watershed; 2) the SCR watershed is much smaller; and 3) much of the relatively level, low-lying portion of the SCR catchment will be inundated by the reservoir.

Although the first difference described above tends to increase the precipitation losses on the SCR catchment in relation to that of the Paluxy River, the other two differences indicate a decrease in losses. Smaller drainage areas, such as the SCR catchment, generally have lower losses than larger areas, and submergence of much of the creek alluvium removes a section of the catchment which should have the most infiltration capacity. Thus, it is prudent to adopt lower estimates of losses on the SCR catchment. This conclusion is further supported by the relatively short duration of the available historical storms studied, in comparison to the 48-hour storm used in computing the PMF. In view of these factors, an initial loss estimate of 0.5 inches and an infiltration rate of about 0.1 inch per hour is considered appropriate for the catchment.

### 2.4.3.3 Runoff Model

#### 2.4.3.3.1 Description of Squaw Creek Reservoir Catchment

Figure 2.4-1 illustrates the 64-square mile SCR catchment and the reservoir limits corresponding to a selected flood-stage at Elevation 780 feet.

The maximum normal operating reservoir level will be Elevation 775 feet. The higher level has been utilized as the pre-flood condition.

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Q371.7 During the PMF, pool level will rise from Elevation 775 feet (area 3,272 acres) to Elevation 789.7 feet (area 3,863 acres).

2.4.3.3.2 Synthetic Hydrographs

Synthetic methods for developing a runoff model have been undertaken. Three methods of synthetic hydrograph development were considered:

1. Use of Snyder's Unit hydrograph relations presented by the U.S. Army Corps of Engineers (USACE) [14].

2. Use of dimensionless hydrographs presented by the Soil Conservation Service (SCS) [14].

3. Use of triangular hydrograph techniques presented by the U.S. Bureau of Reclamation [12].

The empirical relations developed by Snyder have also been shown to be reliable through widespread usage. To employ this method, two coefficients which depend upon drainage basin characteristics are computed from hydrologic records for a representative portion of the drainage area under study, or for nearby catchment of similar characteristics [15]. Snyder's method was adopted for development of a runoff model for Squaw Creek.

2.4.3.3.3 Hydrograph Development

For purposes of analyses, the SCR catchment has been divided into three areas, as shown on Figure 2.4-1 and described below:

1. The Upper Squaw Creek catchment, which consists of about 38 square miles of land located above the reservoir area.

Except for a few existing small farm ponds, there are no present or planned structures upstream of SCR; therefore, the effect of such structures was not considered in developing the PMF hydrograph.

The PMF was routed through the SCR assuming the reservoir level at the beginning of the PMF was at elevation 775, the maximum operating level. All discharge was assumed to occur over the uncontrolled spillways of Squaw Creek Dam.

All streams in the SCR basin empty directly into SCR; therefore no channel routing coefficients were required. The applicability of the stream course response model to handle the PMF is discussed in Section 2.4.3.3.4. The ability of the SCR dam to withstand the PMF and coincident wave action is discussed in Section 2.4.3.6.

### 2.4.3.5 Water Level Determinations

The mass curve, the capacity-area-depth curves, and the spillway rating curves (Figure 2.4-9) are used in routing the PMF through the reservoir to evaluate water level. The resulting peak reservoir level is Elevation 789.7.

In routing, the reservoir water surface has been assumed to be nearly horizontal, and the volume of water in the reservoir has been assumed to be directly related to the reservoir elevation. These are reasonable assumptions in view of the shape and depth of the SCR. These assumptions allow the principle of continuity expressed as a storage equation (It - s = t, where I and are the average rates of inflow and outflow for the time t, and s is the change in water volume during time t) to be applied directly to the routing problem [16].

### 2.4.3.6 Coincident Wind Wave Activity

The magnitude of the wind tide and wave runup are dependent upon the wind velocity, fetch and reservoir depth. The wind direction must coincide with the fetch direction. An overload wind velocity of 40 miles per hour has been approved by the USACE for use in determining freeboard requirements in the Fort Worth District. This 40 mph wind velocity is the highest that may reasonably be assumed to occur coincidentally with the probable maximum flood [17].

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The effective fetch length for wave generation was determined for the center of Squaw Creek Dam (fetch of 1.28 miles) and for the exposed side of the CPSES plant location (fetch of 1.25 miles). It was also determined for the Safe Shutdown Impoundment Dam and the protected side of the CPSES plant location, but freeboard requirements were found to be less than two feet for these locations, so discussion is not included here. Computation of effective fetch considered radial lines at angles up to 42 degrees from the central or primary fetch line, as recommended by the USACE [10].

The average depth of the reservoir at PMF, Elevation 789.7 feet, is approximately 55 feet, and the longest theoretical deep-water wave length (for waves reaching the center of Squaw Creek Dam) is 46 feet, so the ratio of water depth to wave length is well over one-half, and the reservoir can be considered to have "deep water" [10].

Computation [10] utilizing data for wind velocity, fetch length, and reservoir depth yield the results shown in Table 2.4-14. The Table illustrates the maximum runup and setup of smooth and riprapped banks on the Squaw Creek Dam and at the exposed side of the CPSES plant area. As can be seen from the Table, wave runup and wind tide at the dam and plant are about 4 and 5.0 feet and elevations reached are 793.7 feet and 794.7 feet, respectively. Due to the much shorter fetch available around the area, water level elevation reached at the SSI is about

791.3 feet. All plant facilities are above the maximum wave runup and setup elevation of 794.7 feet. The Service Water Intake Structure is the only safety-related structure subject to wave action. The elevation at the operating deck is approximately 796, above the maximum expected wave runup. Section 2.4.10 discusses the effect of wave runup and wind tide on all pertinent safety-related facilities. 4 Q371.7

### 2.4.3.7 Flood Evaluations for Safe Shutdown Impoundment

A 40-ft wideopen channel with a crest at elevation 769.5 feet above mean sea level (0.5 feet below minimum operating pool) and a channel slope of .003 has been cut through the peninsula that consitutes the south abutment of the Safe Shutdown Impoundment (SSI) Dam. Water can pass freely between the SSI and SCR, and the water surface in both water bodies normally will be at the same elevation. Figure 2.4-11 is a graph of the discharge characteristics of the SSI spillway. Table 2.4-15 outlines predicted performance of the SSI during occurrence of simultaneous Probable Maximum Floods on the over-all SCR watershed and the SSI watershed. The maximum level reached in the SSI during the PMF is computed to be 790.5 feet, leaving a freeboard in the SSI of 5.5 feet. Further details on the Safe Shutdown Impoundment Dam are given in Section 2.4.8.2.2. Table 2.4-16 gives the unit hydrograph parameters for the SSI watershed, and Figure 2.4-12 shows the unit hydrograph for the SSI watershed. 4 Q371.7

## 2.4.4 POTENTIAL DAM FAILURES (SEISMICALLY INDUCED)

There are no impoundments other than small farm ponds on the SCR catchment; therefore, a postulated dam failure upstream on Squaw Creek is not appropriate for the CPSES facilities. The farm ponds on the catchment have a combined volume which is less than one percent of the reservoir volume and are insignificant.

Failure of Squaw Creek Dam itself presents no danger of flooding the CPSES, as the Station is above the PMF water level. The possibility of damage to Squaw Creek Dam by backwater due to flooding on the Brazos River in the event of a postulated domino-type failure of Morris Sheppard Dam and DeCordova Bend Dam is ruled out in Section 2.4.4.3.

### 2.4.4.1 Reservoir Description

Present and possible future reservoirs which might be considered to have an influence on the site from a safety or water-supply standpoint are described in Tables 2.4-1 through 2.4-3, and their locations are shown in Figures 2.4-5 and 2.4-6.

### 2.4.4.2 Dam Failure Permutations

Considering CPSES safety, the most severe dam failure permutation conceivable is the failure of Morris Sheppard Dam and the subsequent domino-type failure of DeCordova Bend Dam. The 25-year floods of both the Brazos and Paluxy rivers can be added into the effects of the combined dam breaks without significantly intensifying the results. Failure of the Lake Whitney Dam, downstream, would not have an adverse effect on CPSES.

The detailed analysis of the most severe dam failure permutation is presented in Sections 2.4.4.3 and the effect of landslides into the reservoir is discussed in Section 2.4.2.2.

There will be no commercial water traffic on SCR, and no possible blockage of any water course in the site region could affect the plant.

### 2.4.4.3 Unsteady Flow Analysis of Potential Dam Failures

The possibility of damage to Squaw Creek Dam due to failure of Morris Sheppard Dam and DeCordova Bend Dam was examined by initiating a number

## 2.4.8 COOLING WATER CANALS AND RESERVOIRS

### 2.4.8.1 Canals

No canals are involved.

### 2.4.8.2 Reservoirs

#### 2.4.8.2.1 Squaw Creek Reservoir (SCR)

SCR is a cooling lake for CPSES. The location and configuration of the reservoir are shown in Figures 2.4-5 and 2.4-6. Table 2.4-17 gives the area and capacity characteristics of the SCR site, based on planimeter measurements from U.S. Geological Survey quadrangle maps entitled Hill City, Texas, and Nemo, Texas, scale 1:24,000. The volumes and areas indicated are those of the entire reservoir, including the reserve storage within the Safe Shutdown Impoundment (SSI) described in Section 2.4.8.2.2 below. The performance capability of SCR as operational cooling pond was evaluated through mathematical modeling as documented in Reference [40].

Under normal conditions, the reservoir will remain in the five-foot range between a normal minimum operating level at elevation 770.0 and the crest of the service spillway at elevation 775.0. At the lower drawdown limit, elevation 770.0, the over-all surface area will be 3,084 acres, and the content will be 135,062 acre-feet. When filled to the top of conservation storage, at elevation 775.0, the area will be 3,272 acres and the capacity 150,953 acre-feet. 4 Q371.7

1. Squaw Creek Dam

The layout of Squaw Creek Dam is shown in Figure 2.4-16. A typical cross-section of the embankment is shown in Figure 2.4-17. The top of the dam is at elevation 796.0. The central section is constructed of

select, impervious material, wetted and rolled, with a cutoff trench extending down to impervious foundation material. The outer zones of the embankment are of less select material. A filter system separates the impervious central zone from the less select outer zone on the downstream side and extends outward to the downstream toe to provide drainage and protection for the core.

The reservoir side of the dam is protected by rip-rap and gravel blanket from the top of the embankment to elevation 760.0, which is ten feet below the minimum operating level. The top width of the embankment is 20 feet, exclusive of the gravel blanket and rip-rap. Design for the rip-rap was based on an average over-water wind of 95 mph (Probable maximum wind - 200 year frequency) over the effective fetch distance. The method of computing the effective fetch distance set forth in Department of the Army Office of the Chief of Engineers ETL 1110-2-8, 1 August 1968, was adopted. Mimimum layer thicknesses were determined using the requirements set forth in EM 1110-2-2300 (April 1959) and EM 1110-2-1601 (July 1970). The specific gravity of the rock was assumed to be 2.3. The result are as follows:

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a. Effective Fetch - 1.28 miles
b. Significant Wave Hgt. - 5.0 ft.
c. Layer Thickness - 33 Inches
d. Average Rock Size - 22 Inches

2. Spillways

The service spillway is an uncontrolled structure (i.e., without gates), 100 ft wide, with a standard ogee crest at elevation 775.0. Additional discharge capacity for protection from extreme floods is provided by a broad-crest emergency spillway, 2,200 ft wide, excavated through the rock of the north abutment at elevation 783.0. A 12-inch diameter makeup water pipeline crosses the emergency spillway along its crest. This line is placed in a trench cut in limestone and is covered

1110-2-2300 (April 1959) and EM 1110-2-1601 (July 1970). The specific gravity of the rock was assumed to be 2.3. The results are as follows:

| | | | |
|---|---|---|---|
| a. | Effective Fetch - | 0.36 mi | 4 Q371.7 |
| b. | Significant Wave Hgt. - | 2.6 ft | |
| c. | Layer Thickness - | 24 in | 4 Q371.7 |
| d. | Average Rock Size - | 15 in | |

Estimated sediment production from the Panther Branch watershed above the SSI during the 30-yr projected service life of CPSES was derived based on analytical procedures for small watersheds as described in Reference [38]. The anticipated reduction in storage capacity of the SSI during that period due to accumulation of sediment was found to be 69 ac-ft, of which 64 ac-ft would be below elevation 770.0 and the remaining 5 ac-ft above elevation 770.0. Comparative plots of area and capacity characteristics before and after sedimentation are shown in Figure 2.4-22. Table 2.4-19 outlines the predicted area and capacity values at the end of the 30-year period.

A detailed layout of the SSI design features is given in Figure 2.4-23.

Seismic Design Criteria for the SSI are discussed in Section 3.7.

The ability of the SSI to meet criteria of Regulatory Guide 1.27 is discussed in Section 9.2.5.

## 2.4.9 CHANNEL DIVERSIONS

The SCR catchment has developed streams with distinct valleys and has sustained numerous farm ponds. Therefore, diversion of water from the catchment appears impossible. The reservoir is formed in the Glen Rose formation, a predominately limestone sequence. Information developed regarding this formation indicated it is relatively impermeable and free of sinkholes and solutioning. Thus, significant loss of water is improbable.

Lake Granbury, which is on the Brazos River, will be a major source of makeup cooling water. The loss of Lake Granbury makeup water due to the diversion of the Brazos River is highly improbable. Above Lake Granbury, the Brazos River channel is cut into bedrock which precludes any reasonable possibility of the river changing its channel significantly within the life of the CPSES and thus affecting the supply of water.

Extraction of groundwater, oil, and gas from the region is relatively nominal. It is concluded that subsidence sometimes associated with these extractions will not occur in the vicinity of the CPSES. The potential of subsidence at the site is discussed in detail in Section 2.5.1.2.6.

## 2.4.12 DISPERSION, DILUTION, AND TRAVEL TIME OF ACCIDENTAL RELEASES OF LIQUID EFFLUENTS IN SURFACE WATERS

### 2.4.12.1 Introduction

This section provides a conservative analysis of a postulated accidental release of radioactive material in surface waters adjacent to the site. The postulated release was assumed to occur due to an accidental rupture of the waste holdup tank which is located in the Auxiliary Building near the Containment Building.

The volume of the tank is 30,000 gallons and at the time of rupture it was assumed that the tank was 80 percent full. The assumed quantities of radionuclides in the tank at the time of rupture are given in Table 2.4-20.

It was conservatively assumed that all the liquid radwaste (24,000 gallons, or $7.36 \times 10^{-2}$ acre-feet) is spilled into Squaw Creek Reservoir. Minimum dilution in Squaw Creek Reservoir would occur at minimum pool elevation 770.00 feet (msl), corresponding to a storage volume of 135,062 acre-feet. Assuming complete mixing, the minimum dilution factor is $135{,}360/(7.36 \times 10^{-2})$ or $1.84 \times 10^{6}$. 4 Q371.7

The instantaneous concentrations in Squaw Creek Reservoir are calculated by dividing the concentrations in the tank by the dilution factor. Due to the decay characteristics of the radionuclides, the concentrations will decrease with time. The equation used to define the concentration of any radionuclide for certain periods of time is:

$$C_t = 2^{\left(-\frac{t}{t^{\frac{1}{2}}}\right)} \times C_o \qquad \text{(Reference 42)}$$

where,

$C_o$ = Concentration in time zero
$t$ = Time interval considered
$t_{\frac{1}{2}}$ = Half life of radionuclide
$C_t$ = Concentration at time t

The concentration of each radionuclide in Squaw Creek Reservoir at the end of the first day and at the end of the first month is shown in Table 2.4-21.

Under normal operating conditions, there will be no controlled release of water to Squaw Creek from the Squaw Creek Reservoir. Instead, an average of 52,600 acre-feet of water per year will be pumped to Squaw Creek Reservoir from Lake Granbury and 26,400 acre feet of water per year will be pumped back to Lake Granbury. The "Partially Mixed Model," described in the NRC Reg. Guide 1.113 (May, 1976), was used to calcute radionuclide concentrations in Lake Granbury due to this pumpage.

The blowdown of Lake Granbury is conservatively assumed to be the 50-year low flow of the Brazos River at Granbury, which is 35 cfs (Figure 2.4-24). The spillway crest elevation of Lake Granbury is 658.0 feet and the corresponding reservoir capacity is 15,440 acre-feet. Water released from Lake Granbury will travel down the Brazos River and enter Whitney Reservoir approximately 75 miles downstream.

Given a low flow condition of 35 cfs and by assuming a roughness coefficient of 0.035, it is estimated that travel of the radionuclides will be 3 days, 20 hours. In the portion of the Brazos River between Lake Granbury and Whitney Reservoir, there are no significant tributaries which might affect the radionuclide concentrations in the

TABLE 2.4-11
SQUAW CREEK RESERVOIR PROBABLE MAXIMUM FLOOD

| Hours | Probable Maximum Storm Rainfall (Inches) | Losses (Inches) | Rainfall Excess (Inches) | Probable Maximum Flood Hydrograph (cfs) | Probable Maximum Flood Hydrograph (ac-ft) | Rainfall on Reservoir (ac-ft) | Cumulative Inflow (ac-ft) |
|---|---|---|---|---|---|---|---|
| 3 | .5 | .5 | .0 | 0 | 0 | 158 | 158 |
| 6 | .5 | .3 | .2 | 951 | 118 | 158 | 434 |
| 9 | .6 | .3 | .3 | 2,358 | 410 | 18[illegible] | 1,033 |
| 12 | .7 | .3 | .4 | 3,774 | 760 | 22[illegible] | 2,014 |
| 15 | .8 | .3 | .5 | 5,140 | 1,105 | 253 | 3,372 |
| 18 | .8 | .3 | .5 | 6,002 | 1,382 | 253 | 5,007 |
| 21 | .9 | .3 | .6 | 6,864 | 1,595 | 284 | 6,886 |
| 24 | .9 | .3 | .6 | 7,473 | 1,778 | 284 | 8,948 |
| 27 | 2.3 | .3 | 2.0 | 14,424 | 2,585 | 726 | 12,259 |
| 30 | 5.5 | .3 | 5.2 | 34,370 | 5,665 | 1,737 | 19,661 |
| 33 | 20.0 | .3 | 19.7 | 121,907 | 14,958 | 6,317 | 40,936 |
| 36 | 3.5 | .3 | 3.2 | 142,576 | 33,962 | 1,105 | 76,003 |
| 39 | .6 | .3 | .3 | 66,914 | 25,258 | 189 | 101,450 |
| 42 | .5 | .3 | .2 | 21,545 | 10,402 | 158 | 112,010 |
| 45 | .5 | .3 | .2 | 7,796 | 3,281 | 158 | 115,449 |
| 48 | .5 | .3 | .2 | 4,079 | 1,389 | 158 | 116,996 |
| 51 | | | | 2,118 | 768 | | 117,764 |
| 54 | | | | 907 | 375 | | 118,139 |
| 57 | | | | 329 | 153 | | 118,292 |
| 60 | | | | 115 | 55 | | 118,347 |
| 63 | | | | 40 | 19 | | 118,366 |
| 66 | | | | 15 | 7 | | 118,373 |
| 69 | | | | 4 | 2 | | 118,375 |
| 72 | | | | 2 | 1 | | 118,376 |
| Totals: | 39.1 | 5.0 | 34.1 | | | | |

TABLE 2.4-15

SAFE SHUTDOWN IMPOUNDMENT PROBABLE MAXIMUM FLOOD

| Time in Hours | Incremental Rainfall (Inches) | Incremental Rainfall Excess (Inches) | PMF Hydrograph (cfs) | PMF Hydrograph (ac-ft) | Rainfall on SSI Surface (ac-ft) | Cumulative Inflow (ac-ft) | SSI Surface Elevation | SSI Spillway Flow (cfs) |
|---|---|---|---|---|---|---|---|---|
| 3 | .5 | .0 | 0 | 0 | 1 | 1 | 775.0 | 0 |
| 6 | .5 | .2 | 111 | 14 | 2 | 17 | 775.3 | 60 |
| 9 | .6 | .3 | 200 | 30 | 2 | 58 | 775.5 | 130 |
| 12 | .7 | .4 | 285 | 60 | 3 | 121 | 775.7 | 200 |
| 15 | .8 | .5 | 354 | 79 | 3 | 203 | 776.0 | 270 |
| 18 | .8 | .5 | 368 | 89 | 3 | 295 | 776.5 | 320 |
| 21 | .9 | .6 | 431 | 99 | 3 | 397 | 777.0 | 350 |
| 24 | .9 | .6 | 442 | 108 | 4 | 509 | 777.5 | 370 |
| 25 | .6 | .5 | 504 | 39 | 4 | 552 | 777.8 | 380 |
| 26 | .8 | .7 | 851 | 55 | 4 | 611 | 778.1 | 380 |
| 27 | 1.2 | 1.1 | 1,311 | 88 | 4 | 703 | 778.6 | 830 |
| 28 | 2.2 | 2.1 | 2,103 | 140 | 10 | 853 | 779.3 | 1,900 |
| 29 | 2.6 | 2.5 | 3,586 | 231 | 11 | 1,095 | 780.3 | 3,000 |
| 30 | 2.9 | 2.8 | 4,683 | 340 | 11 | 1,446 | 781.5 | 4,200 |
| 30-1/2 | 2.4 | 2.35 | 5,151 | 203 | 10 | 1,659 | 782.1 | 4,600 |
| 31 | 2.6 | 2.55 | 5,883 | 227 | 11 | 1,897 | 782.9 | 4,900 |
| 31-1/2 | 5.9 | 5.85 | 7,451 | 275 | 25 | 2,197 | 784.2 | 6,200 |
| 32 | 9.3 | 9.25 | 10,079 | 361 | 40 | 2,598 | 785.5 | 8,400 |
| 32-1/2 | 1.6 | 1.55 | 15,828 | 526 | 7 | 3,131 | 787.5 | 11,600 |
| 33 | 1.5 | 1.45 | 20,849 | 799 | 6 | 3,936 | 790.1 | 17,600 |
| 33-1/2 | 1.0 | .95 | 17,3[illegible] | 798 | 4 | 4,738 | 790.5 | 18,400 |
| 34 | .9 | .85 | 13,321 | 633 | 4 | 5,375 | 789.8 | 15,400 |
| 34-1/2 | .7 | .65 | 10,124 | 483 | 3 | 5,861 | 789.2 | 10,200 |
| 35 | .4 | .35 | 7,783 | 370 | 2 | 6,233 | 789.5 | 7,200 |
| 35-1/2 | .3 | .25 | 6,107 | 285 | 1 | 6,519 | 789.7 | 5,600 |
| 36 | .2 | .15 | 4,787 | 225 | 1 | 6,745 | 789.8 | 4,600 |
| 37 | .2 | .1 | 2,832 | 314 | 1 | 7,060 | 789.7 | 3,500 |
| 38 | .2 | .1 | 1,538 | 178 | 1 | 7,239 | 789.2 | 2,000 |
| 39 | .2 | .1 | 838 | 97 | 1 | 7,337 | 788.5 | 1,500 |
| 42 | .5 | .2 | 178 | 101 | 2 | 7,440 | 786.4 | 600 |
| 45 | .5 | .2 | 145 | 40 | 2 | 7,482 | 784.9 | 300 |
| 48 | .5 | .2 | 146 | 36 | 1 | 7,519 | 783.9 | 100 |
| 51 | .0 | .0 | 28 | 22 | | 7,541 | 783.3 | 100 |
| 54 | .0 | .0 | 3 | 4 | | 7,545 | 782.9 | 100 |
| | 44.9 | 39.9 | | | | | | |

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TABLE 2.4-24

PUBLIC SUPPLY, INDUSTRIAL AND IRRIGATION WELLS, 0-20 MILES

| Well Number[a] | Piezometric Elevation (ft) and Date | | Yield (gpm) | Drawdown[b] (feet) | Use[c] | |
|---|---|---|---|---|---|---|
| 1 | 701 | 3/15/68 | 100 | 90 | PS | City of Walnut Springs |
| 2 | - | -- | 135 | 90 | PS | City of Walnut Springs |
| 3 | 628 | 8/24/66 | - | 110 | Irr | James Smith |
| 4 | 890 | 10/14/65 | 69 | 0 | Irr | Lee Manning |
| 5 | 974 | 3/27/69 | 150 | 0 | Irr | J. W. Waldie |
| 6 | 992 | 10/14/65 | 150 | 0 | Irr | J. W. Waldie |
| 7 | - | -- | 550 | 0 | Irr | Triangle Ranch |
| 8 | - | -- | - | 0 | Irr | Triangle Ranch |
| 9 | 845 | 10/26/65 | 120 | 0 | Irr | Stanley Allen |
| 10 | 834 | 3/27/69 | 46 | 0 | Irr | Stanley Allen |
| 11 | 868 | 3/26/69 | - | 30 | Irr | E. L. Huffman |
| 12 | 855 | 10/15/65 | - | 30 | Irr | E. L. Huffman |
| 13 | 615 | 9/13/60 | - | 40 | Irr | Roy Kenedy |
| 14 | 620 | 1960 | 614 | 100 | PS | City of Glen Rose |
| 15 | 627 | 9/21/60 | 250 | 100 | PS | City of Glen Rose |
| 16 | 620 | 6/19/30 | 50 | 100 | PS | Young Women's Christian As. |
| 17 | 611 | 9/14/60 | - | 90 | Irr | Squaw Creek Cemetery As. |
| 18 | 608 | 7/20/66 | 222 | 190 | Ind | Texas Lime Company |
| 19 | 575 | 9/15/60 | 100 | 140 | Ind | Texas Cedar Oil Company |
| 20 | 48[illegible] | 6/06/68 | - | 210 | PS | U.S. Army Corps of Engrs. |
| 21 | - | -- | - | 0 | PS | City of Tolar |
| 22 | - | -- | - | 0 | PS | City of Tolar |
| 23 | - | -- | 65 | 0 | PS | City of Granbury (9 wells) |
| 24 | - | -- | - | 25 | Irr | L. L. Williams |
| 25 | - | -- | - | 100 | PS | Camp El Jesom |

[a] Well locations are shown on Figure 2.4.33.

[b] Estimated drawdown, based on original static, piezometric level, before 1900.

[c] Use: Ind, Industrial; Irr, Irrigation; PS, Public Supply.

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The compaction requirements on the bedding material, placed in both the Class I Electrical Duct Banks and Service Water Pipe Trench, is a minimum density of not less than 80% of the relative density as determined by ASTM Test Designation D2049, latest revision effective prior to September 4, 1975. In-place density shall be determined in accordance with ASTM D1556 (Sand Cone), ASTM D2167 (Balloon), of ASTM D2922 (Nuclear). Figure 2.5.4-39 shows the gradation requirements of the bedding material.

### 2.5.4.6 Groundwater Conditions

A detailed description of groundwater is presented in Section 2.4.13.
4 No groundwater was encountered during excavation for the plant
Q371.7 foundations. Groundwater observations from piezometers installed at the site are provided on Figure 2.5.5-77.

### 2.5.4.7 Response of Soil and Rock to Dynamic Loading

During design, adopted values for the cyclic shear strength for Class I
2 backfill and bedding material were based on the published data for granular soils. A discussion of the published data is presented in Section 2.5.6.4.3.4. Since the $D_{50}$ of the bedding material was approximately the same as for Filter "A" for the SSI Dam, the material was assumed to have essentially the same cyclic characteristics. Class I backfill material has a $D_{50}$ of approximately 15mm and since this material is coarser than the published data utilized, the cyclic strength characteristics of this material should be greater. For design, the cyclic strength criteria was assumed based on the data
4 presented on Figure 2.5.6-48. The specific criteria as spelled out in Gibbs & Hill Specification 2323-SS-8, Section 10 was that the material with the gradation limits as shown on Figure 2.5.4-38 shall not allow
2 development of shear strain larger than 5% under specific corresponding stress conditions shown on Table No. 2.5.4-11.