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October 12, 1985
ST-HL-AE-1418
File No.: G9.17

Mr. George W. Knighton, Chief
Licensing Branch No. 3
Division of Licensing
U. S. Nuclear Regulatory Commission
Washington, DC 20555

South Texas Project
Units 1 and 2
Docket Nos. STN 50-498, STN 50-499
Responses to DSER/FSAR Items -
FSAR Sections 2.4, 3.4 and 3.5

Dear Mr. Knighton:

The attachments enclosed provide STP's response to Draft Safety Evaluation Report (DSER) or Final Safety Analysis Report (FSAR) items.

The item numbers listed below correspond to those assigned on STP's internal list of items for completion which includes open and confirmatory DSER items, STP FSAR open items and open NRC questions. This list was given to your Mr. N. Prasad Kadambi on October 8, 1985 by our Mr. M. E. Powell.

The attachments include mark-ups of FSAR pages which will be incorporated in a future FSAR amendment unless otherwise noted below.

The items which are attached to this letter are:

Attachment	Item No.*	Subject
1	F 3.4-1	FSAR Section 2.4 (See Note 1) FSAR Section 3.4 (See Note 2) FSAR Section 3.5

Note 1 - Section 2.4 is being clarified based upon resent re-analysis.

Note 2 - Section 3.4 is being clarified based upon resent flood re-analyses. It is also being clarified to reflect the project position regarding the Main Cooling Reservoir breach (see ST-HL-AE-1240, ST-HL-AE-1093, and ST-HL-AE-1103).

* Legend

D - DSER Open Item C - DSER Confirmatory Item

F - FSAR Open Item Q - FSAR Question Response Item

L1/DSER/ad

B001
1/1

If you should have any questions concerning this matter, please contact Mr. Powell at (713) 993-1328.

Very truly yours,

M. R. Wisenburg
Manager, Nuclear Licensing

SMH/bl

Attachments: See above

L1/DSER/ad

cc:

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9.3.3.2 System Description. The Equipment and Floor Drain System is segregated into radioactive and nonradioactive classes of liquid wastes and is described in the following sections. A description of processing and discharge facilities is given in Section 11.2. Protection from the effects of internal flooding ~~will be~~ are discussed in Section 3.4. 2 Q10.10

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Reactor coolant grade equipment drains are collected within the RCB in the reactor coolant drain tank, which is part of the LWPS. See Section 11.2.2 for further discussion.

9.3.3.2.1 Drainage Provisions: The majority of radioactive and nonradioactive drains except for those servicing filters and demineralizers, are routed to open equipment drains or combination equipment and floor drains. 40

Floor drains are placed throughout the plant to collect leakage and aid in floor cleaning. These floor drains are connected to collection sumps or tanks having pumps to convey the collected volume to the appropriate collection tanks for processing, or they are connected directly to the collection tank.

Certain gravity drain lines receive sump pump flow from the lower elevation radioactive sumps. These gravity drain lines carry the sump pump flow, along with leakage collected, directly to the LWPS collecting tanks.

The radioactive drain system gravity drain lines are designed for the largest of the following flows: 40 Q10.10

1. Equipment Leakage

2. Expected sump pump discharge flows

3. Controlled drainage of vessels or equipment

4. Decontamination washdowns from simultaneous use of two 1-inch hoses at 35 gal/min flow each.

Piping is arranged to prevent crud pockets where accumulation of solids may occur. Drain lines are adequately sloped to ensure complete drainage of piping. Slotted-cover plates are an integral part of all floor drains to prevent solids from entering drain piping and causing subsequent clogging.

Drains from the solid waste processing area are equipped with screens and strainers to prevent solid particles and bead resins from entering the LWPS collection tank. 40

Safety-related tanks are located in compartments having elevated openings. Sumps and pumps are provided in these compartments, except for the boric acid tank compartment, to transfer the liquids to the appropriate tanks for processing. The boric acid tank compartment is provided with a normally covered floor drain. 40

Nonsafety-related tanks, except the floor drain tank (FDT), are located in cubicles containing floor drains and elevated thresholds.

2.4.1 Hydrologic Description

2.4.1.1 Site and Facilities. The STP site is situated near the west bank of the Colorado River in Matagorda County, Texas The East-West centerline of the Reactor Building is opposite river mile 1[illegible].4. The site is 12 miles south-southwest of Bay City, Texas, and 8 miles north-northwest of Matagorda, Texas, near FM 521 (see Figure 2.4.1-1). 43

The MCR is formed by a 12.4-mile-long earthfill embankment constructed above the natural ground surface, totally enclosing 7,000 acres of surface area at a normal maximum operating level of El. 49 ft. Reservoir makeup water is withdrawn from the Colorado River adjacent to the site by pumping intermittent flows and providing reservoir storage to account for periods during which makeup is unavailable. The ultimate heat sink is a cooling pond designated as the Essential Cooling Pond (ECP). It is external to and derives its makeup from MCR. A well is also provided to augment the makeup water to the ECP in the event of failure of the ECP makeup from the MCR. 43 43

Ground slopes are minimal, and both deciduous and coniferous trees are sparsely scattered throughout the site. Surface elevations range from about El.30 ft at the north end of the site to between El. 15 ft and 20 ft at the south end, as illustrated in Figure 2.4.1-2. Plant grade is at El. 28 ft.

The westerly divide of the lower Colorado River Basin passes through approximately the center of the site in a northwest-to-southeast direction. A drainage area of approximately 4.5 square miles north and west of the reservoir drains to a relocated channel for Little Robbins Slough which is parallel to, and west of, the westerly embankment of the reservoir (see Figure 2.4.1-3).

The critical safety-related flood levels result from either an assumed instantaneous breach of the MCR embankment opposite the plant site or the failure of upstream dams on the Colorado River as discussed in Section 2.4.4. Calculations show a maximum flood wave runup on the structures to El. 50.8 ft. Consequently, the sill elevations of doors of seismic Category I structures are placed above this elevation or are otherwise protected. Structures are designed to withstand both hydrostatic and hydrodynamic effects that would result from the postulated embankment failure described above. Specific elevations of structures and plant flood protection measures are discussed in Section 3.4. 43 43

2.4.1.2 Hydrosphere.

2.4.1.2.1 Surface Water:

2.4.1.2.1.1 Colorado River -

2.4.1.2.1.1.1 General Description of the Basin - The Colorado River Basin, shown on Figure 2.4-2, contains approximately 41,800 square miles and is oriented generally along a northwest-to-southeast direction. The 600-mile length of the basin extends for the southeastern portion of New Mexico to Matagorda Bay in southeast Texas at the Gulf of Mexico. The width of the basin increases from 85 miles in the upper portion to about 170 miles in the area of Stacey, Texas, then narrows to about 30 miles near Austin, Texas. From Austin the basin gradually continues to narrow to about 4 miles wide near

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from 73 in. in the upper portion to 53 in. near the Gulf. Average annual runoff ranges from 50 acre-ft/mile^2 in the upper portion to 350 acre-ft/mile^2 in the lower reaches. Floods occur in the basin on an average of every 4 to 5 years. This frequency of floods is compatible with the fact that a severe tropical storm may be expected to cross the Texas coast on an average of about once every 3 years. These tropical storms are a principal cause of floods in
the hydrologic region discussed herein. Flood-producing precipitation is also 43
caused by orographic storms which occur when moisture-laden warm air masses from the Gulf are forced upward by the escarpment and are cooled by the higher air masses.

All principal tributaries of the Colorado River are upstream of Lake Travis. Normal and flood flows in these streams are regulated by reservoirs. The evolution of these tributaries probably can be attributed to the rugged and varying topography from which they drain. Their channels are well incised and their flood plains are moderately steep. Downstream from Austin only a few minor tributaries are to be found. Below Columbus, definable tributaries are almost nonexistent.

2.4.1.2.1.1.2 The Colorado River Near the Site - To evaluate the response of the Colorado River Basin in the lower reaches near the site, it is necessary to have an understanding of the upstream reaches. Generally, the
main channel of the Colorado River has the capacity to contain flows ranging 43
from a 6-year to a 21-year return interval from Austin to the Gulf of Mexico. Thus, in any given year there is a 5- to 16-percent chance that river flows will encroach upon the floodplains. The magnitude of these flows ranges from about 320 ft^3/sec/mile^2 near Austin to about 28 ft^3/sec/mile^2 near Bay City.

For large peak flows the attenuation of peak discharges downstream from 43
Columbus becomes more pronounced than for smaller flows. This phenomenon is explained by a comparison of the floodplain and river valley complex of the areas above and below Columbus. Above Columbus, the floodplain varies from about 2.5 to 5.5 miles wide. ~~Floodplain slopes normal to the channel vary from about 2.5 to 5.5 miles wide.~~ Floodplain slopes normal to the channel vary from about 5 ft/mi to 12 ft/mi, and the floodplain is the floor of a well-defined valley. This kind of river valley complex provides little opportunity for storage of storm runoff, especially in floods of greater magnitudes.

Below Columbus, the floodplain width varies from 4 to 8 miles within the basin, and slopes normal to the channel average between 0.5 ft/mi and 1.5 ft/mi. In this area, no discernible valley exists, and floods have occurred which have extended beyond the basin divides with interbasin spillage. Thus,
this part of the basin provides great storage capacity and significant peak 43
reduction.

Pertinent stream-flow gage data for the four active gages and three abandoned gages below Mansfield Dam are shown in Table 2.4.1-25 and the locations of these gages are shown on Figure 2.4.1-4. Physical data pertinent to the river characteristics are shown in Table 2.4.1-26.

2.4.1.2.1.2 Little Robbins Slough - The principal drainage feature in the STP site area other than the Colorado River is Little Robbins Slough shown
on Figure 2.4.1-3. This drainage course flows south to a coastal marsh area 43

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Among the hurricanes which have crossed the Texas coast was Hurricane Carla
which caused a surge of 12.3 ft near Port O'Connor in September 1961,
resulting in flooding of the coastal area of Tres Palacios Bay and Matagorda
Bay. Table 2.4.5-2 lists major historical hurricanes which have caused significant surge flooding the general coastal area of the STP site. The effects
of hurricane surge are discussed in Section 2.4.5.

2.4.2.2 Flood Design Considerations. The regulatory positions of RG 43
1.59, have been used as the basis for flood design considerations. Of the
several possible causes of flooding given in RG 1.59, only tsunami flooding
(see Section 2.4.6), ice flooding (see Section 2.4.7), and flooding due to
landslides (see Section 2.4.4) are not considered critical in establishing the
flood design bases for the STP site. Floods that could result from all other
causes, or combinations thereof, are analyzed for the Colorado River, local
site drainage and the MCR. The critical flood levels at STP determined from
these analyses, result from a postulated breach of a portion of the north 43
embankment of the MCR which determine controlling levels for the power block
and ~~south, east, and west faces of~~ the ECWIS ~~and from the failure of the dams
on the Colorado River which determined the controlling levels for the north
face of the ECWIS~~ as discussed in Section 2.4.4. Safety-related structures
and components are designed to withstand the ~~effects of flooding~~ flood levels from these 43
postulated events. Section 3.4 presents a discussion of the flood protection
for safety-related structures and components. Table 2.4-1 presents a summary
of the flooding conditions studied, with the exception of the PMP on the
Cooling Reservoir which is presented in detail in Section 2.4.8. 43

2.4.2.3 Effects of Local Intense Precipitation. There are two local
drainage areas adjacent to the plant structures. Considering a PMP of a point
rainfall magnitude, a PMF on either of these two adjacent areas would result
in water levels in the plant area that would be above plant grade.

The larger of these two areas lies west and northwest of the plant structures
and contains 4.5 square miles of land surface. This area drains into relocated Little Robbins Slough. The PMF from this area is estimated conservatively to have a peak discharge of 8,000 ft^3/sec. It would cause a water 43
level of about 32 ft at the site.

The other drainage area adjacent to the plant structures lies northeast of the
plant and contains about 0.6 square miles. It drains easterly and southeasterly away from the plant structures through natural streams and into some
plant area ditches. The critical point of flow in the discharge of a PMF peak
from this area would be at the concrete culvert under the plant access road,
just southeast of the ECP. This culvert is designed for a 50-year flood. A
local PMP would cause overtopping of the plant access road. Since the eleva- 43
tion of the access road in that area is 30.75 ft for a length of at least 700
linear ft, a water level of 32 ft over a broad-crested weir 700 ft long would
pass a discharge of 2450 ft^3/sec assuming a conservatively low value of the
weir coefficient $C = 2.5$. For the 0.6-square mile drainage area, this would 43
represent a peak discharge of about 4080 ft^3/sec/sq mi, or over two times the
peak discharge per square mile calculated for the 4.5-square mile drainage
area on the west side of the plant. Therefore, by inspection, the water level
of 32 ft caused by PMF on Little Robbins Slough would be higher than the PMF
on the stream draining the 0.6 sq. mile area. 43

A PMF on either adjacent drainage area would result in short-term overloading of the plant drainage system but would not enter plant area buildings.

The normal drainage system for the drainage of roofs of plant structures has
been designed for a maximum intensity of 8 in./hr. However, the design of
plant roofs for a PMP is based on the assumption that these drains would be 43
plugged. Under these conditions, water would accumulate on the roof until it
overtopped the low curb around the perimeter of the roof and flowed over the
side. The height of this curb is designed so that the maximum depth realized 43
during the maximum hourly intensity of a PMP will not produce a load which
exceeds the roof design loads.

Probable maximum precipitation for local intense precipitation was taken from
Reference 2.4.3-2. Point rainfall (10 square miles or less) was determined
from all-season envelope and was found to be 32.5 in. for a 6-hours duration
at the STP site. After losses were estimated and deducted, the total excess
rainfall was 31.76 in., reflecting a highly conservative runoff coefficient of
97.7 percent. The rainfall excess was distributed in accordance with the
USACE procedures for determining the standard project flood (Reference 43
2.4.2-1).

A Snyder unit hydrograph was used to develop runoff from the 4.5-square mile
drainage area adjacent to the west side of the plant. Parameters were esti-
mated by analyzing records of rainfall-runoff characteristics of a gaged
watershed near the site which has similar hydrometeorological characteristics.
The data investigated included seven storms of record which produced approxi- 43
mately 1 in. of runoff. Each of the events analyzed was reduced to a
unit-graph and the seven unit-graphs were averaged. Snyder's parameters were
estimated from the average unit-graph, and adjustments were made to these
parameters when they were applied to the area under study. The PMP excess
rainfall was applied to the resulting 1-hour unit hydrograph (Figure 2.4.3-13) 43
and a peak discharge of 6,400 ft^3/sec was calculated. To account for non-
linearity between normal and intense rainfall, the calculated peak was
increased by 25 percent, resulting in a peak discharge of 8,000 ft^3/sec from 43
the 4.5-square mile area.

The design of site drainage facilities in the main plant area is based on a
50-year storm using the rational method. ~~and assuming runoff coefficients of~~ 43
~~0.85 for paved areas and 0.35 for grassed areas.~~ Should any blockage of site
drainage facilities occur during storm conditions, the net result would be to
delay runoff and increase times of concentration. This would have the result
of reducing flood peaks; however, since total inundation of plant grade is
calculated to be greater for the flood conditions discussed Section 2.4.4, it
is concluded that local PMP flooding is not critical to flood design. 43

2.4.3 Probable Maximum Flood on Streams and Rivers

PMF determinations are made for six conditions which represent the most criti- 43
cal hydrometeorological events, or combinations thereof, which may be expected
to affect the STP site. The probable maximum precipitation, as applied to the 43
MCR, is discussed in Section 2.4.8.

The Fort Worth District Office of the USACE, in conjunction with a hydrologic
study related to the proposed Columbus Bend Reservoir, has determined that a 43

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3. Model No. 3

Model No. 3 simulated the flood-wave impact on the south embankment of the ECP. Between the MCR embankment and the south ECP embankment, the ground elevation varies between 25 ft and 29 ft approximately. The crest elevations of the ECP embankment and the interior dike are, respectively, 34 ft and 38 ft.

For a sufficiently large breach, all breach flow would approach one-dimensional. In this model, all of the breached flow was assumed to go over the interior dike, which, being 4 ft higher than the surrounding dike, would cause the water surface to attain its maximum elevation. (For breach location see Figure 2.4.4-14.)

The assumption that all the breached flow goes over the interior dike is conservative, since in reality part of the breached flow would be free to go around the ECP embankment. 43

The reach between the MCR embankment and the ECP was divided into several sections. The distances between consecutive sections were variable. During computations smaller distances were used near the MCR embankment to accommodate the very steep initial wave fronts. The time step used in the simulation was 1.44 secs.

The initial MCR water surface elevation was assumed at El. 50.5 ft which is the maximum elevation realized during a Standard Project Precipitation (see Section 2.4.8.2.1). The MCR water surface elevation was held constant at El. 50.5 ft at the upstream boundary.

The boundary was placed sufficiently far upstream such that it does not significantly influence the maximum water surface elevation realized during passage of breached flow over the interior dike. The downstream boundary was selected at the interior dike. A rating curve calculated using discharge characteristics of embankment shaped weir was used as downstream boundary condition.

2.4.4.3 Water Levels at the Plant Site.

2.4.4.3.1 Water Level Resulting From Failures of Upstream Dams: Analysis of the two major dam failure sequences and the failure of the MCR embankment as described above show that the critical flood levels on the power block ~~including the east, west,~~ and ~~south faces of~~ the ECWIS are determined by the MCR embankment failure. ~~while the controlling elevation on the north face of the ECWIS is caused by the upstream dams failures.~~ The MCR embankment failure is also the controlling event for buoyancy consideration. 43

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The failure of Mansfield Dam, without Columbus Bend Reservoir in the system, would produce a maximum stillwater level of El. 32 ft MSL in the plant area. With the proposed Columbus Bend Dam failure a plant site maximum stillwater elevation of 31.7 ft MSL would be realized.

43

The maximum water surface elevation realized at the plant structures is the sum of the stillwater elevation plus any wind setup and wave runup. The wind-wave phenomena produced by a 2-year 50-mph wind speed were investigated

assuming several different directions. It is concluded that the wind wave generated along fetch line A, as shown on Figure 2.4.3-30, will produce the maximum water elevation as well as the maximum wave force on structures. The average depth used for setup calculation along fetch line A for a stillwater elevation of 32 ft MSL is about 15 ft. The wind setup at plant structures would be 1.6 ft. Relatively shallow water depths near the power block limit the maximum wave height that could occur. A maximum depth-limited wave height of 4.3 ft resulting in a runup of 9.8 ft above the stillwater elevation 2
against smooth impervious unprotected vertical walls was determined using methodologies 43 Q
described in Reference 2.4.4-14. ~~Thus, a maximum water surface elevation of~~ 3
~~43.4 ft, including setup and runup, would occur for the case of the Mansfield~~ 1
~~Dam failure.~~

2.4.4.3.2 Water Levels Resulting From Cooling Reservoir Embankment
Breach: The maximum water level realized at the south face of power block 43
structures which results from the instantaneous flood-surge runup caused by the postulated embankment breach is El. 50.2 ft MSL and occurs 38 seconds after the instantaneous and total removal of a 2,030-ft section of the embankment. This water level is determined by use of model no. 1 as described in
Section 2.4.4.2.2.2.2.1. Forces resulting from the flood-wave surge as dis- 43
cussed in Section 3.4.

The quasi-steady-state condition resulting from the postulated 1,890 ft embankment breach produces the maximum stillwater elevations on all sides of
the power block structures. The maximum water depths and buoyancy elevations 43
resulting from the quasi-steady-state condition are shown in Table 2.4.4-3.
The results pertaining to power block structures are determined by use of 43
model no. 2, as discussed in Section 2.4.4.2.2.2.2. The data obtained from the Danish Hydraulic Institute computer output for the embankment breach analyses are computed for grid line intersections. Since the building outlines in the model are at half-grid spaces, the computed data must be extrapolated to determine the water surface at the face of power block structures. At the north and south faces, the water levels for the two grid points immediately to the north or south of the model outline are determined. Then the linear extraploation is performed to dertermine the water surface at the model outline. At the east and west faces, the water levels fro the two grid points immediately to east or west of the model outline are determined. The water surface at the model outline is determined by linear extrapolation. These elevations so determined are considered to be the water surface elevations at the actual face of power block structures.

2.4.4.3.2.1 Confirmation of Model Results by One-Dimensional Analysis - 43
To obtain a design confirmation of the DHI two-dimensional modeling, an analysis was made to establish an "order of magnitude" result, utilizing a one-dimensional model which is acknowledged to be less refined than the two-dimensional calculations. Theoretical treatment of one-dimensional unsteady flows is given in References 2.4.4-11, 2.4.4-12 and 2.4.4-13. The
modified NWS Dam-Break program was used to predict the evolution of the surge 43
against the walls of the power block structures assuming that these walls represented a dead end. This is a very conservative assumption, since in reality not only would the breached waters go around the power block structures, they would also flow between them. It is, therefore, reasonable to say that the maximum water surface elevation of 52.7 ft against the walls of the

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75,000 ft^3/sec. Recent backwater studies indicate that the bankfill capacity in this reach has increased to about 100,000 ft^3/sec, probably due in part to the dredging of a 14-ft deep channel with a 100-ft width for a distance of 15.5 miles above the Gulf Intracoastal Waterway.

A considerable number of channel improvements were completed by the USACE south of Bay City in connection with the navigation project authorized by Congress under Section 7 of the Rivers and Harbors Act of August 8, 1917. Dredging carried out between river mile 22 (turning basin) and the Gulf Intracoastal Waterway stabilized the course of the river. The dredged material was deposited along both banks of the river and the spoil areas were enclosed by embankments. A considerable portion of the abandoned river channel north of the STP site and in the vicinity of Selkirk Island (discussed previously in Section 2.4.9.1) was filled in. Hence, shifting of the Colorado River channel beyond its present dredged alignment is prevented.

2.4.9.4 Channel Diversion. Due to flood regulation by upstream reservoirs and the responsibility for channel stabilization and improvement delegated by Congress to USACE, channel diversion is not considered to be a significant factor to the safety of the STP site.

2.4.10 Flooding Protection Requirements

Safety-related plant features are designed to withstand combinations of flood conditions and wave runup as discussed in Section 2.4.1.1. Maximum water level elevations at the plant site under these conditions are given in Table 2.4-1 and 2.4.4-3. Protection of safety-related structures and components, including the effects of floods and waves, is discussed in Section 3.4.

The roof decks of the seismic Category I structures are drained by sloping the concrete to the roof drains or scuppers. The roof drainage piping is embedded in the concrete roof slabs and piped to grade along the outside walls. The roof piping is sized for a rainfall intensity of 8 in./hr. In the event that the roof drainage system becomes overloaded or plugged, the excess water will 43
flow over the 6-in.-high curb and down the face of the building. Loads caused by water sheet flow over the roof, or snow in combination with other meteorological or seismic events, will not approach the 50 lb/ft^2 construction load for which the roofs are designed.

Plant-area drainage facilities are designed to pass a 50 year rainfall event 43
without interference with traffic mobility, structures or process areas. More intense precipitation may cause flooding of certain roadways and yard areas. To determine the flooding caused by a local PMP, the plant area was analyzed as an impoundment with the peripheral roads and rail spurs as the impounding dike. Considering the discharge due to over-the-road weir flow alone, ~~as would be the case with an incapacitated underground weir flow alone~~, as would be the case with an incapacitated underground drainage system, flood levels thus caused would not exceed El. 30 ft. Since the breach of the dams on the Colorado River as described in Section 2.4.4 will produce more critical flood levels at the plant, further detailed analysis of local drainage was not 43
undertaken.

The MCR for STP is an earthfill dam as described in Section 2.5.6. The only inflow to the MCR other than direct precipitation is the makeup water pumped 43

The Liquid Waste Processing System (LWPS) releases radionuclides into the Circulating Water System where the concentrations are diluted to below the limits defined in 10CFR20, Appendix B, Table 2. Further dilution occurs in the MCR. Consequently, concentrations of radionuclides in seepage resulting in relief well discharge would be below the MPC limits of 10CFR20.

2.4.13.3.2.3 Seepage From the Main Cooling Reservoir - The shallow aquifer at the site consists of two units, an upper shallow aquifer and a lower shallow aquifer. These are shown in section on Figures 2.4.13-3A | 44
through 2.4.13-3C, with supplemental information concerning groundwater gradients in both the upper and lower shallow aquifers and the results of observations made in each aquifer on Figures 2.4.13-17 through 2.4.13-19. As described in Section 2.5.6 and the figures attached thereto, a system of some 669 relief wells has been installed in the embankment around the reservoir to | 44
relieve excess hydrostatic pressure. The seepage analysis, discussed herein, takes flow from the relief wells into account. Different piezometric levels | 44
have been measured in each portion of the shallow aquifer. This is shown on Figure 2.4.13-18. Therefore, for purposes of this analysis, it is assumed that seepage from the MCR will enter only the upper portion of the shallow | 44
aquifer. Seepage discharging from the reservoir will be composed of two about 700
parts: (1) seepage that is collected and discharged through ~~the 669~~ relief | 44
wells located around the reservoir and (2) seepage through the upper shallow aquifer that bypasses the relief wells and continues downgradient.

| 44

Flow is broken into two components, the portion expected to be discharged through relief wells and the portion expected to bypass the relief wells and exit downgradient from the site in the upper shallow aquifer. As indicated in Section 2.5.6, relief well seepage will be collected in toe and drainage | 44
ditches around the periphery of the reservoir embankment and discharged at various locations offsite.

~~As indicated,~~ total seepage from the MCR is estimated to be 3,530 gal/min, or | 44
approximately 5,700 af/yr. Of that, approximately 68 percent, or 3,850 af/yr, would be discharged through the relief wells.

2.4.13.4 Monitoring or Safeguard Requirements. Any significant changes in shallow- and deep-zone piezometric levels and associated basic groundwater flow pattern will be detected by periodic monitoring. To accomplish this, | 44
shallow and deep piezometers have been installed at appropriate locations around the site. The piezometer monitoring program is discussed in Section 2.5.4.13 and Appendix 2.5.C. Piezometers are of sufficient diameter, i.e., 2 in. inside diameter, to allow water sampling. In the unlikely event of an | 44
operational accident, piezometers will be sampled frequently for three or more years and the waters tested for indications of contamination. Subsequent monitoring, or modification of this schedule, would depend on initial results.

Should hazardous levels of contamination ever reach the shallow groundwater aquifers at the project perimeter, additional sampling stations would be installed downstream to track the migration and dilution of hazardous components. Furthermore, additional deep-zone monitoring would also be instituted as a precautionary measure, both in onsite project supply wells and in wells and deep piezometers installed downstream, as deemed desirable under prevailing circumstances. In the extremely unlikely event of contaminated

The following events have been analyzed in Section 2.4 to determine high water level. 49

1. Probable maximum flood

2. Probable maximum hurricane surge

3. Failure of upstream dams on the Colorado River

4. Breach of the Main Cooling Reservoir (MCR) embankment 49

The occurrence of high water levels at the plant will not be a sudden event and there will be adequate warning time of potential flooding. For the case of upstream dam failures, the shortest warning time is estimated at 65 hours. 49
Also, warning of impending floods from the Colorado River as well as the approach of hurricanes will allow sufficient time to safely protect the plant. An evaluation has been completed (see Section 2.5.6) that concludes the MCR embankment will not fail under any creditable design basis event. 49

As described in Section 2.4.11, no emergency protective measures are required to safely shut down the plant in the event of extreme low water levels in the MCR as plant shutdown will be initiated when water level in the MCR falls to El. 25.5 ft. Due to the size of the MCR such a drop in water level necessarily would be a gradual event and, for the reasons discussed in Section 2.4.11.6, is extremely remote. 49

Section 9.2.5 describes the normal and emergency operation of the ECP. No emergency ~~protection~~ measures are required to safely shut down the plant providing an operating level above El. ~~24.5~~ 25.5 ft is maintained. ~~The plant will be shut down when the water level in the ECP reaches 24.5 ft~~. If the ECP level falls below 25.5 ft and cannot be restored to operable range within the time specified in plant technical specifications, shutdown will be initiated.

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Section 9.5 describes the normal and emergency operation of the ECP. No emergency protection measures are required to safely shutdown the plant provoding an operating level above El. 25.5 ft is maintained. The plant will be shutdown when the water level in the the ECP reaches 25.5 ft.

That the failure of the spillway gates to operate in time of flood will not cause failure of the reservoir embankment is explained principally by the fact that there is no watershed contributing to reservoir water other than the rainfall on the reservoir surface itself. In the case of the design flood, the SPS, 25.28 in. of rainfall would be placed on the reservoir. This amount, of top of EL. 49.0 ft, the normal maximum operating water surface, gives a water surface elevation of 51.1 ft. With the gates operating, the routing of the SPS through the reservoir results in a maximum water surface elevation of 50.5 ft. Thus, with the spillway gates nonoperative, the water surface is only 0.60 ft higher than with the gates operating. The lowest elevation of the perimeter embankment is 65.75 ft. Thus, with gates operating and with passage of the SPS, there is a freeboard of 15.25 ft. With gates nonoperative, this is reducted to 14.65 ft, again a difference of 0.60 ft. As a result of the conservatism used in the calculation of wind-wave setup and runmup, the reduction in freeboard of 0.60 ft is not considered significant. 6

If the SPS is followed by the probable maximum sotrm, an interval of 3 dry days is allowed between storms. Water flowing over the top of the gates at El. 49.5 ft will reduce the above-mentioned water surface El. of 51.1 ft, because of low head, to only approximately 51.0 ft in this 3-day interval. At this time the probable maximum storm, amounting to 45.82 in. of rainfall in 48 hours, is routed through the reservoir, withe gates remaining nonoperative and flow being over the top of the gates. A reservoir water surface elevation of approximatley 54.4 ft results. With this water surface, the wave runup and wind setup is computer for a wind speed of 66 mph at the lowest point of resevoir embankment, in the vicinity of station 660 + 00, where the elevation of embankment is 65.75. Wave runup was computed to be 4.25 ft and wind setup to be 1.86 ft. These two values added to the maximum water surface El. 54.4 ft results in an elevation of 60.5 ft, 5.2 ft under top of embankment EL. 65.75 ft.

TABLE 2.4-1

SUMMARY OF FLOOD ANALYSES RESULTS

	Description of Event and Attendant Conditions	Discharge Resulting From Event (1,000 ft^3/sec)	Antecedent Discharge (1,000 ft^3/sec)	Base Flow (1,000 ft^3/sec)	Total Discharge Used in Analysis (1,000 ft^3/sec)	Maximum Stillwater Elevation (ft MSL)	Wind Velocity Applied (mph)	Maximum Flood Elevation at Plant (ft MSL)
1.	SDF from proposed Columbus Bend Dam routed to STP site in coincidence with peak of SPF from 755-square mile area above site and 50,000 ft^3/sec base flow	648	260	50	958	28.8	---	---
2.	Natural PMF on uncontrolled area between Mansfield Dam and Bay City, 50,000 ft^3/sec base flow and SPF on same area 3 days antecedent.	863	0	50	913	28.7	---	---
3.	SDF from Mansfield Dam routed to STP site with SPF 3,520 square miles above Bay City 3 days after PMP on area above Lake Travis and with 50,000 ft^3/sec base flow.	648	0	50	698	28.0	---	---
4.	PMF on area between propsed Columbus Bend Dam and STP site incoincidence with SPF release from proposed Columbus Bend Dam and 50, 000 ft^3/sec base flow.	520	324	50	894	28.5	---	---
5.	Extreme PMF selected from envelope curve. Based on assumption of no flood control for entire 28,770 square miles above Bay City.	1,750	--	---	1,750	30.8	---	---
6.	PMF on 4.5-square mile drainage area local to Cooling Reservoir, contributing to relocated Little Robbins Slough.	8	---	---	8	32.0	---	---
7.	Instantaneous, complete failure of Mansfield and of Buchanan Dam in coincidence with a river flow equal to SPF at Bay City.	---	432	50	1,900	32.0	50*	43.4(2)

*The only case for which conincident wind-wave activity was considered.
(See Sections 2.4.3.6 and 2.4.4.3.1)

1) Design basis flood for all structures except the north face of the ECWIS.
2) Design basis flood for the north face of the ECWIS.

43
2 Q37 1
44
43

TABLE 2.4-1 (Continued)

SUMMARY OF FLOOD ANALYSES RESULTS

Description of Event and Attendant Conditions	Discharge Resulting From Event (1,000 ft^3/sec)	Antecedent Discharge (1,000 ft^3/sec)	Base Flow (1,000 ft^3/sec)	Total Discharge Used in Analysis (1,000 ft^3/sec)	Maximum Stillwater Elevation (ft MSL)	Wind Velocity Applied (mph)	Maximum Flood Elevation at Plant (ft MSL)	
8. Intantaneous, complete failure of proposed Columbus Bend Dam and of Mansfield Dam in coincidence with a river flow equal t SPF at Bay City.	---	432	50	1,800	31.7	---	---	43
9. Intantaneous removal of 2,000-ft-long (nominal) section of the north embankment of the Cooling Reservoir, assuming an initial water surface elevation of 50.5 ft.	---	---	---	---	---	---	50.8(1)	44 43
10. The peak of a Bay City 100-year flood in coincidence with the maximum PMH surge at the river's mouth.	---	193	---	193	26.7	---	---	

*The only case for which conincident wind-wave activity was considered. (See Sections 2.4.3.6 and 2.4.4.3.1)

1) Design basis flood for all structures except the north face of the ECWIS.
2) Design basis flood for the north face of the ECWIS. 43

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TABLE 2.4.4-3

MAXIMUM RESULTS OF POSTULATED FAILURES

		Reactor Containment Building				Mechanical-Electrical Auxiliaries Building				Fuel-Handling Building				Diesel-Generator Building				ECW Intake Structure				Auxiliary Feedwater ~~Condensate~~ Storage Tank				
		N	S	E	W	N	S	E	W	N	S	E	W	N	S	E	W	N	S	E	W	N	S	E	W	
Cooling Reservoir Embankment Breach	Ground Elevation (ft MSL)				28.0	28.0	28.0	28.0	28.0		28.0	28.0	28.0	28.0	28.0	28.0	28.0	25.0	25.0	10.0	34.0	28.0	28.0	28.0	28.0	
	Max. Water Elevation (ft)				49.0	44.5	50.6	48.1	50.6		50.8	50.6	50.3	44.5	44.5	44.5	44.5	40.8	40.8	40.8	40.8	50.0	50.0	50.0	50.0	43
	Bouyancy Elevation (ft MSL)				49.0	46.5	50.6	48.1	50.6		50.8	50.6	50.3	44.5	44.5	44.5	44.5	40.8	40.8	40.8	40.8	50.0	50.0	50.0	50.0	43
Colorado River Dam Failure	Ground Elevation (ft MSL)				28.0	28.0	28.0	28.0	28.0		28.0	28.0	28.0	28.0	28.0	28.0	28.0	25.0	25.0	10.0	34.0	28.0	28.0	28.0	28.0	
	Max. Water Elevation (ft)				43.4	43.4	43.4	43.4	43.4		43.4	43.4	43.4	43.4	43.4	43.4	43.4	~~43.4~~ 39.5'	39.3	39.3	39.3	43.4	43.4	43.4	43.4	43
	Bouyancy Elevation (ft MSL)				33.6	33.6	33.6	33.6	33.6		33.6	33.6	33.6	33.6	33.6	33.6	33.6	33.6	33.6	33.6	33.6	33.6	33.6	33.6	33.6	43 Q371.2 Q371.1

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3.4 WATER LEVEL (FLOOD) DESIGN

The methods of analysis used to determine the design basis flood condition are discussed in Section 2.4 These methods are consistent with the requirements of Regulatory Guide 1.59. ~~Revision 0.~~

The ~~flood~~ floods protection measures ~~and structural design methods~~ used to accommodate static and dynamic loadings ~~ings~~s on Category I structures ~~due to flooding are discussed herein. The measures~~ generally fall under the category of "incorporated barriers" as specified in regulatory position C.1.c of Regulatory Guide 1.102. ~~Revision 1.~~

3.4.1 Flood Protection

External Flood Protection Measures for Seismic Category I Structures.

3.4.1.1 Design Flood Conditions and Elevations. A wide range of conditions has been investigated to establish flood elevations for which the plant and its relative structures are designed. The resulting design conditions that produce maximum water level at the plant and its related structures are as follows:

1. Cooling Reservoir Embankment Breach

The flooding due to a postulated MCR ~~reservoir~~ embankment ~~branch~~ breach produces the maximum water level ~~along the south, east and west faces of the plant~~ around the power block structures as well as ~~provided~~ the controlling water elevations for buoyancy calculations. This is also the controlling phenomena in determining the maximum water level at the ~~south, east and west faces of the~~ Essential Cooling Water ~~(ECW)~~ Intake Structure (ECWIS). ← Insert I

The maximum water level on a vertical face at the south end of the plant structures is El. 50.8 ft mean sea level (MSL), which is 22.8 ft above plant grade. This maximum elevation occurs during a quasi-steady-state condition after a breach of the MCR ~~Cooling Reservoir~~ embankment and is based on an instantaneous~~, nonmechanistic~~ removal of 2,000 ft of the embankment opposite the power block structures ~~plant site~~. This maximum elevation occurs on the south face of the Fuel-Handling Building (FHB) of Unit 1. The selection of postulated embankment breach widths and the assumptions made in determining the maximum flood elevations are described in Section 2.4.4.

Figure 3.4-1 shows a general section through the plant. Figure 3.4-2 shows the maximum steady-state water surface profile, and the corresponding relationship of sill elevations for entrances to seismic Category I building areas.

Total inundation of the MCR Essential Cooling Pond (ECP) occurs only under the condition of ~~reservoir~~ embankment breach and does not affect the safe shutdown capability of the plant. The maximum water level calculated to occur at the ECWIS ~~ECW Intake Structure~~ is El. 40.8 ft~~; however, this structure is provided with flood protection up to El. 49.0 ft~~. ← Insert II

2. Colorado River Dam Breach

The instantaneous, complete failure of Mansfield Dam in coincidence with a river flow equal to a standard project flood (SPF) at Bay City, as described

INSERT I

Studies and analyses on the MCR embankment have demonstrated that an adequate margin of safety can be maintained for all credible failure mechanisms (see Section 2.5.6). Accordingly, mechanistic effects (such as scour and erosion) associated with a postulated failure of the MCR embankment need not be evaluated.

INSERT II

All safety-related structures, systems and components listed in Table ~~3.5-1~~ 3.2-1 under the heading "System, Component, or Structure" are protected against the effects of external flooding by: Q010.2

1) Being designed to withstand the maximum flood level and associated effects and remain functional (such as Seismic Category I Structures and the Category I Auxiliary Feedwater Storage Tank) or

2) Being housed within Seismic Category I Structures which are designed as in 1 above.

Flood protection of safety-related structures, systems, and components is provided for postulated flood levels and conditions described in Section 2.4

Seismic Category I structures are designed to withstand the maximum flood levels ~~and associated effects~~ by:

1. Having external walls and slabs of structures designed to resist the hydrostatic and hydrodynamic forces associated with surge-wave runup and steady-state water level.

2. Ensuring the overall stability of the total structure against overturning and sliding due to the hydrostatic and hydrodynamic forces associated with surge-wave runup and steady state water level, and

3. Ensuring that the total structure will not float due to buoyancy forces.

in Section 2.4.4, is, in general, the controlling phenomenon in determining the maximum water level along the north face of the plant structures and along the north embankment of the ECP. The maximum stillwater level caused by the Colorado River dam breach at the plant site area is El. 34.1 ft MSL. The maximum nonhurricane-wind setup and wind-wave runup on the vertical face at the north end of the plant structures is 8.1 ft above El. 34.1 ft MSL. Therefore, the maximum water level realized on the north face of plant structures is El. 42.2 ft MSL. The corresponding setup and runup for the north ECP embankment and the north face of the ECW Intake Structure results in a maximum water level of 46.9 ft MSL. As demonstrated in this section, the plant and all safety-related facilities are designed to withstand or are protected from the effects of these flood conditions and still remain operational to permit a safe shutdown of the plant. 2 Q371. 1

Insert III

Table 3.4-1 shows the water depths which were used in the force and buoyancy calculations. These depths were developed based on the values shown for depths presented in Table 2.4.4-3.

3.4.1.2 Design Provisions for Flood Protection. All safety-related systems and components, except the auxiliary feedwater storage tank (AFST), are protected against the effects of external flooding by being housed within seismic Category I structures which are designed to withstand the maximum flood levels and remain functional. The AFST is a seismic Category I structure and is designed to withstand these maximum flood levels and associated effects. 31 31

Seismic Category I structures are designed to withstand the maximum flood levels and associated effects by:

1. Having external walls and slabs of structures designed to resist the hydrostatic and hydrodynamic forces associated with surge-wave runup and steady-state water level.

2. Ensuring the overall stability of the total structure against overturning and sliding due to the hydrostatic and hydrodynamic forces associated with surge-wave runup and steady-state water level.

3. Ensuring that the total structure will not float due to buoyancy forces.

An investigation of seismic Category I structures has been made for the flood levels and associated effects as previously described. The design for gross effects upon the structure, ~~as mentioned in subparagraphs 2 and 3 above,~~ incorporates safety factors greater than 1.1. All exterior seismic Category I building openings are located above the maximum steady-state flood level or are equipped with watertight doors when located below this profile, except as stated below. 46 Q240. 02N

Exceptions to the above-stated design access basis for exterior building openings in seismic Category I structures are: (1) the opening for the truck bay in the radwaste loading area of the Mechanical-Electrical Auxiliaries Building (MEAB); (2) the opening for the rail car in the spent fuel cask loading area of the FHB, and (3) Tendon Gallery Access Shaft (TGAS) cover. These areas are not protected from flooding because they do not have any safety-related systems and components. In addition, the first two areas are separated from the remainder of the building by walls which do not contain openings below the 46 Q240. 02N 46

INSERT III

Figure 3.4-1 shows a general section through the plant. Figure 3.4-2 shows the seismic Category I buildings, the maximum steady-state water surface profile, and the corresponding relationship of sill elevations for entrances to seismic Category I buildings.

The TGAS is provided with a external watertight cover to prevent flood water from entering the TG.

water surface elevation

maximum ~~surge-wave runup height~~ corresponding to their location. ~~The access opening from the Tendon Gallery into the MEAB will be provided with a watertight door.~~ 46 Q240. 02N

Insert IV

The exterior wall openings in the ECW Intake Structure are located below the maximum water levels of El. 40.8 and 46.9 ft MSL and are equipped with six waterproof doors. Each door permits access to a single compartment of the structure, and the doors are controlled so that only one door may be opened at one time. The dividing walls between compartments preclude propagation of flooding from one compartment to another. 2 Q371. 1

45.0 tight water surface elevation

The three maintenance knockout panels in the exterior walls of the Diesel-Generator Building, which are located below the maximum ~~wind-wave runup height~~ of ~~El. 42.2~~ ft MSL, are water~~proofed~~ and designed for the hydrostatic forces. Each knockout panel allows access to only one of the three separate compartments within the structure, and only one panel may be removed at one time. The dividing walls betweeen the compartments preclude propagation of flooding from one compartment to another. 2 Q371. 1

tight room

The maintenance knockout panel in the exterior wall of the ~~MEAB, which is~~ located below the maximum steady-state water level shown on Figure 3.4-2. ~~is~~ water~~proofed and designed for the chamber~~ housing the component cooling water heat exchangers ~~with~~in the MEAB. ~~This chamber prevents floodwaters from entering the remainder of the structure. A waterproof door, which allows access from the chamber to the remainder of the structure, may be opened only when the knockout panel is in place.~~ are

These panels are

Insert V

All exterior seismic Category I building wall and slab surfaces below grade are waterproofed. This conservatively protects the substructure of seismic Category I building from groundwater, which ~~usually fluctuates betweeen 2 ft and 10 ft below grade~~. No waterproofing is provided on exterior wall or slab surfaces above grade to protect against the effects of surge-wave run-up because of its short duration. All construction joints in exterior walls and slabs (except for localized areas of blockouts) are provided with waterstops to El. ~~60~~ 50 ft and can withstand hydrostatic effects.

Insert VI

provided with check valves not

~~No~~ drains are ~~connected~~ such that the external flooding ~~c~~would result in internal flooding through the inadvertent introduction of water through these drains into seismic Category I structures.

Insert VII

Leakage from groundwater into the FHB is prevented by the use of waterproofing on exterior wall and slab surfaces located below grade. Should groundwater inleakage occur, it is handled by the pumps in the FHB sump, the three-train compartment sumps, and the ~~cask loading~~ transfer cart area sump.

The safety-related portions of the MEAB are protected from flooding due to liquids originating within the building, and from groundwater by the following methods:

1. Major tanks containing liquids in the MEAB are housed in watertight compartments which will retain the contents of the tank. This includes the following tanks.

is expected to stabilize between El. 17 ft and El. 26 ft (1 to 10 ft. below grade) after decommissioning of the construction dewatering system.

INSERT IV

The safety related equipment in the ECWIS is protected from the effects of the design basis flood. The personnel access doors on the west wall are provided with watertight doors; all other doors and openings are above the flood level. The dividing walls and doors between the ECWIS compartments minimize the potential for the propagation of flooding from one compartment to another.

INSERT V

Since nonmechanistic effects from the MCR breach need not be evaluated, there is adequate time to replace the knockout panels for the remaining flood events of concern.

INSERT VI

All seismic joints between Category I structures contain dual 9 inch water stops capable of withstanding potential seismic and hydrostatic effects. Cracks in concrete are minimized by imposing strict QA and QC procedures on the quality of concrete and construction techniques.

INSERT VII

The ducts banks are sealed so as to prevent backflow into safety related areas. The cable in the duct banks is designed/specified for submerged installations.

Boric Acid Storage — Refueling Water Storage
Recycle Holdup — Reactor Makeup Water Storage
Spent Resin Storage — Volume Control
Floor Drain

2. A sufficient number of sumps and sump pumps are located at El. 10 ft to collect and remove leakage originating primarily from floor drains located in various areas and elevations of the MEAB. Their number and size are such that they accommodate postulated failure of equipment and piping not located in watertight compartments without flooding (i.e., rendering inoperable) any safety-related equipment in the MEAB.

~~3.~~ 2. Leakage of groundwater into the MEAB is prevented by the use of waterproofing on exterior wall and slab surfaces located below grade. Should groundwater leakage occur, it will be collected in the sumps and pumped to the Liquid Waste Processing System.

Insert VIII

(See Section 9.3.3 for a description of the Floor Drain System~~, which is equipped to protect safety-related equipment from the effects of leakage of systems within the building.)~~

3.4.2 Analysis Procedures

Insert IX

3.4.2.1 Phenomena Considered in Design Load Calculations. The phenomena considered in design load calculations are, as described in Section 3.4.1, the Cooling Reservoir embankment breach and the Colorado River dam breach. These phenomena were determined to be the events which produce the maximum flood elevations and attendant maximum wave runup on structures (see Section 2.4).

In the case of the Cooling Reservoir embankment breach, an initial reservoir water surface elevation is assumed as El. 50.5 ft, which corresponds to the level of an SPF. Since no fetch distance can be associated with the Cooling Reservoir embankment failure, no separate wind-wave and associated phenomena are considered in the force calculation.

In the case of the Colorado River dam breach, a 60-mph wind is assumed to generate the corresponding waves. The explanation of fetch and wave calculations for this phenomenon is discussed in Subsection 2.4.4.3.

3.4.2.2 Flood-Force Application. The design flood conditions and elevations have been determined from an analysis of the phenomena discussed in ~~Sub~~section 3.4.1.~~1~~.

In order to establish the controlling load~~ing~~ conditions resulting from the embankment breach, both instantaneous surge wave runup as well as the longer term, quasi-steady state conditions ~~are~~ were analyzed. The wave runup condition conservatively assumes that the maximum total force perpendicular to the south face of the plant structures includes a dynamic component in addition to the associated hydrostatic forces. The quasi-steady state condition assumes that only the hydrostatic component contributes to the development of the total force for this case. The latter condition resulted in higher water surface elevations and greater hydraulic loads on power block structures. 7 Q130.7

INSERT VIII

3.4.1.2 Internal Flood Protection Measures.

Safety related systems, structures, and components are protected against internal flooding caused by postulated piping through-wall cracks and breaks, tank or vessel rupture, and by inadvertent actuation of the fire protection (water) system. Protection is provided for safety related systems and components that are required to attain and maintain a safe shutdown condition and prevent unacceptable radiological dose consequences. Watertight doors, curbs, wall penetration seals and drainage systems (see Section 9.3.3) are provided to mitigate the effects of internal flooding on essential systems and components. An appendix to Chapter 3 will be provided to include basic assumptions, results, and specific protection measures as part of the Integrated Hazards Analyses (pipe break, flooding, etc.).

INSERT IX

For external flooding, the design basis events considered in design load calculations are as described in Section 3.4.1.

The vertical bouyant loading condition is the force equal to the weight of water displaced by a structure. The discussion of lateral and vertical loadings is presented in the following subsections. Table 3.4-1 shows a summary of different lateral loadings at various locations around plant and 7 ECP structures, caused by their respective controlling flood conditions. Q130.7 Procedures used to determine flood loadings are identified in ~~Sub~~Ssections 3.4.2.2.1 and 3.4.2.2.2.

3.4.2.2.1 Lateral Loading: (hydrostatic)

3.4.2.2.1.1 Lateral Loading on the ~~South Side of Plant~~ Power Block Structures - The analysis of the lateral force on the ~~south side of the plant~~ power block structures considered both the instantaneous wave runup and the quasi-steady state conditions. This analysis determined that the maximum total lateral force on ~~the south side of~~ the ~~plant~~ power block structures occurs when the maximum water level is reached during the quasi-steady state condition. Table 3.4-1 shows the ~~controlling~~ lateral forces surge exerted on ~~the south side of the~~ different power block ~~plant~~ structures. These lateral forces are treated as triangular loadings on a vertical surface, varying at a rate of 62.4 $lb/ft^2/ft$ of structure depth. The procedures used to determine the dynamic and hydrostatic loadings for the above analysis conditions are discussed below:

1. Dynamic Force

The dynamic force on the south side of the power block ~~plant~~ structures is determined by application of linear momentum principles. The flow from the ~~Cooling Reservoir~~ MCR is assumed to be normal to the south side of the power block ~~plant~~ structures. Therefore, the dynamic force exerted on the structures can be expressed by the following momentum equation (see Reference 3.4-~~2~~1):

$$F = \rho Q V_o$$

where:

F = dynamic force normal to plant structure 7
ρ = density of flow
Q = flow rate Q130.7
V_o = ~~average~~ velocity of flow

The maximum value of $\rho Q V_o$ during surge formation is calculated. This is the contribution of momentum flux to the dynamic force. The ~~other~~ contribution of the unsteadiness of momentum field is insignificant.

2. Hydrostatic Force

The lateral hydrostatic force is determined by the following equation (see Reference 3.4-~~2~~1):

$$F_{Hyd} = 1/2\ \gamma_w h^2$$

where:

F_{Hyd} = hydrostatic force, lb/ft of width

h = water depth, ft

γ_w = unit weight of water, lb/ft^3

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3.4.2.2.1.2 Lateral Loading on the ~~South Side of~~ ECW IS ~~Intake Structure~~ and the South ECP Embankment - The determination of the maximum lateral force on the ~~south side of the~~ ECW IS ~~Intake Structure~~ considered both instantaneous and quasi-steady state conditions. The maximum total force on ~~the south side of~~ the ECW IS ~~Intake Structure~~ is a result of the ~~Cooling~~ MCR ~~Reservoir~~ embankment breach ~~as analyzed by model No. 2, as~~ discussed in ~~Sub~~section 2.4.4.2.2. This force is the result of a water elevation of 41.0 ft mean sea level during the quasi-steady state condition. ~~and is calculated to be 870 kip/ft of structure width. Table 3.4-1 shows the data and results of this force calculation.~~

Since the ~~height of the~~ south ECP embankment crest elevation is 34.0 ft MSL, it would be overtopped by the flood wave resulting from the MCR embankment breach. ~~9.0 ft and overtopping occurs, several conditions were analyzed. The conditions included the~~ initial surge wave runup, the period between the initial surge ~~and when the~~ ECP is filled, and the period after the ECP ~~is filled~~. The maximum total force on the most southern ~~point of~~ the ECP embankment is analyzed by model No. 3 as ~~discussed~~ in Subsection 2.4.4.2.2. This maximum force ~~is a result of an upstream depth of 21.4 ft and a downstream depth of 7.2 ft.~~ The ~~maximum lateral force acting on the~~ south ECP embankment is designed to withstand the ~~...~~ lateral force based on the maximum water elevation resulting from MCR embankment breach. ~~7.9 kip/ft length of the ECP embankment. For purposes of design, the small magnitude~~ dynamic component was neglected and ~~this force~~ considered to be totally hydrostatic. ~~Procedures~~ for determining both dynamic and hydrostatic ~~forces considered in the above conditions are discussed below.~~ 7

Q130.7

I. Dynamic Force

The dynamic force ~~in a flow field is determined by multiplying the mass of~~ flow by the acceleration of flow. The acceleration includes both local acceleration and translative acceleration. The local acceleration is the change of velocity with respect to time, and the translative acceleration is the change of velocity with respect to location. Therefore, the dynamic force can be calculated using the following two-dimensional equations expressed in a rectangular coordinate system:

$$F_x = \rho Q_x \left(\frac{\delta u}{\delta t} + u\frac{\delta u}{\delta x} + v\frac{\delta u}{\delta y} \right)$$

$$F_y = \rho Q_x \left(\frac{\delta v}{\delta t} + u\frac{\delta v}{\delta x} + v\frac{\delta v}{\delta y} \right)$$

where F_x, F_y = dynamic force components in the x and y direction, respectively.

ρ = density of flow

Q_x, Q_y = flow rate in the x and y direction, respectively.

A finite difference scheme was used to approximate the above equations. The numerical values of u and v at different time levels, t, and different locations, x and y, were obtained from the two-dimensional embankment breach analysis described in Subsection 2.4.4.2.2. The flow rate in the x and y direction was based on the flux densities for the corresponding time, t, and location x and y.

2. Hydrostatic Force

The hydrostatic force is determined as shown in Subsection 3.4.2.2.1.1. 7 Q130.7

3.4.2.2.1.3 Lateral Loading on the North Face of Plant and ECW Intake Structure and on the North ECP Embankment - The lateral loading on the north side of the plant and ECW Intake Structure and on the north ECP embankment is controlled by the maximum nonhurricane, wind-wave force associated with the maximum flood caused by upstream dam breach in coincidence with a riverflood flow equivalent to an SPF at Bay City (see Section 2.4.4). The maximum stillwater level, not including wind setup at the plant site, is at El. 34.1 ft MSL.

As explained in Subsection 2.4.4.3, the controlling wave force on the plant structures results from a broken wave and the controlling wave force at the ECW Intake Structure and the north ECP embankment results from a nonbreaking wave. The methods used for calculating the force on north faces of structures due to broken waves, or nonbreaking waves, are presented in Reference 3.4-1. 7 Q130.7

The method of adjustment in the force on the north ECP embankment is presented in Reference 3.4-1. 7 Q130.7

With the maximum water-surface elevation for broken waves, the maximum wind-wave force acting on plant structures was calculated to be 7.6 kip/ft. The force on the ECW Intake Structure was caused by nonbreaking waves with the resulting force of 9.0 kip/ft of structure width. The maximum force on the north ECP embankment results when the maximum water depth is 21.9 ft. Since the ECP embankment is only 9 ft high, an adjustment in the force diagram was made to reflect the force over that height. The method of adjustment in the force on the north ECP embankment is presented in Reference 3.4-1. The maximum force on the north ECP embankment was calculated to be 5.9 kip/ft of embankment length. The forces and their respective water depths for the plant structures and the ECW structures are given in Table 3.4-1. 2 7 Q130.7

A typical force diagram is shown on Figure 3.4-3. The lateral force distribution, shown in Figure 3.4-3, was multiplied by a dynamic load factor to account for the response of a structure. The surge-wave action produces a forcing function on a structure which is closely approximated by an isosceles-triangular load pulse. The natural period of the maximum surge-wave pulse, t, is greatly in excess of the natural period of any seismic Category I structure, T. As the ratio, t/T, becomes larger, the dynamic load factor of 1.0 is applicable for this type of loading and the pressure distribution shown on Figure 3.4-3 does not need modification. 7 Q130.7

3.4.2.2.1.4 Lateral Loading on East and West Face of Plant Structures - Lateral forces acting along the east or west face of plant structures are entirely hydrostatic, whether resulting from the Cooling Reservoir embankment breach or from the Colorado River dam breaches. Such

~~forces are calculated using the formula given in Subsection 3.4.2.2.1.1. The maximum depths and the resulting forces are given in Table 3.4-1.~~

3.4.2.2.2 Vertical Loading: ~~The vertical hydrostatic loading acting on a horizontal plane is the weight of water above that plane. The vertical buoyant load that affects the stability of a structure is the buoyant force which is equal to the weight of water displaced by the structure.~~ Table 3.4-1 shows the elevations of maximum water surface used for buoyancy calculations. ~~based on average depths.~~ The maximum buoyant force is calculated by assuming that the granular backfill around the structures is completely saturated so that the buoyant force will occur as soon as water arrives at the plant area. ~~The maximum buoyancy effects for the ECW intake structure are based on a maximum water-surface El. of 41.0 ft MSL based on the steady-state condition.~~

III

The roofs of Seismic Category I structures are designed to withstand the weight of the accumulated PMP, assuming completely clogged drains (see Section 2.4.2.3).

REFERENCES

Section 3.4:

~~3.4-1 U.S. Army Coastal Engineering Research Center, "Shore Protection Manual," Third Edition, 1977.~~

3.4-~~2~~1 Streeter, Victor L., "Fluid Mechanics," 3rd Edition, McGraw-Hill Book Company, Inc., 1962. 7 Q130.

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TABLE 3.4-1

FLOOD LOADS FOR CATEGORY I STRUCTURES
(FOR NOTATION SEE FIG. 3.4-3)

		REACTOR CONTAINMENT BUILDING				MECH & ELECT AUXILIARIES BUILDING				FUEL HANDLING BUILDING				DIESEL GENERATOR BUILDING				ESSENTIAL COOLING WATER ~~STRUCTURE~~ INTAKE STRUCTURE				AUXILIARY FEEDWATER ~~CONDENSATE~~ STORAGE TANK				ALL CAT I STRS MAX WATER LEVEL DIFFERENTIAL N-S, E-W FOR OVERTURNING EFFECT (HYDROSTATIC ONLY)	
DIRECTIONS		N	S	E	W	N	S	E	W	N	S	E	W	N	S	E	W	N	S	E	W	N	S	E	W	N-S	E-W
~~COOLING RESERVOIR EMBANKMENT BREACH~~	El A				28.0	28.0	28.0	28.0	28.0		28.0	28.0	28.0	28.0	28.0	28.0	28.0	25.0	25.0	10.0	34.0	28.0	28.0	28.0	28.0		
	H_1				21.0	16.5	23.0	21.0	23.0		23.0	23.0	23.0	11.0	17.0	17.0	11.0	16.0	16.0	31.0	7.0	22.0	22.0	22.0	22.0		17
	P_T				0	0	0	0	0		0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	21.6 FT	14.7 FT
	P_S				1.31	1.03	1.44	1.31	1.44		1.44	1.44	1.44	0.69	1.06	1.06	0.69	1.00	1.00	1.93	0.44	1.37	1.37	1.37	1.37		
	F_T				13.8	8.5	16.5	13.8	16.5		16.5	16.5	16.5	~~3.8~~ 9.0	9.0	9.0	3.8	8.0	8.0	30.0	1.5	15.1	15.1	15.1	15.1		
	H_2				7.0	5.5	7.7	7.0	7.7		7.7	7.7	7.7	~~3.7~~ 5.7	5.7	5.7	3.7	5.3	5.3	10.3	2.3	7.3	7.3	7.3	7.3		
	Bouy EL				49.0	44.5	51.0	49.0	51.0		51.0	51.0	51.0	~~39.0~~ 45.0	45.0	45.0	39.0	41.0	41.0	41.0	41.0	50.0	50.0	50.0	50.0		45
COLORADO RIVER DAM BREACH (3)	EL A				28.0	28.0	28.0	28.0	28.0		28.0	28.0	28.0	28.0	28.0	28.0	28.0	25.0	25.0	10.0	34.0	28.0	28.0	28.0	28.0		
	H_1				10.9	14.2	10.9	10.9	10.9		10.9	10.9	10.9	14.2	10.9	10.9	10.9	21.9	15.4	30.4	6.4	10.9	10.9	10.9	10.9		
	P_T				0	0.26	0	0	0		0	0	0	0.26	0	0	0	0.27	0	0	0	0	0	0	0		
	P_S				0.68	0.88	0.68	0.68	0.68		0.68	0.68	0.68	0.88	0.68	0.68	0.68	0.76	0.96	1.09	0.40	0.68	0.68	0.68	0.68	3.3 FT	0 FT
	F_T				3.7	7.6	3.7	3.7	3.7		3.7	3.7	3.7	7.6	3.7	3.7	3.7	9.0	7.4	28.7	1.3	3.7	3.7	3.7	3.7		
	H_2				3.6	6.9	3.6	3.6	3.6		3.6	3.6	3.6	6.9	3.6	3.6	3.6	6.8	6.1	10.1	2.1	3.6	3.6	3.6	3.6		
	Bouy EL				37.1	37.1	37.1	37.1	37.1		37.1	37.1	37.1	37.1	37.1	37.1	37.1	37.1	37.1	37.1	37.1	37.1	37.1	37.1	37.1		

Q371.1

~~1) NORTH AND SOUTH SIDES ARE REFERENCED ONLY TO INTAKE STRUCTURE~~ 2
~~2) FOR NOTATIONS SEE FIGURE 3.4-3~~ 2
~~3) THIS CONDITION IS COMBINED WITH 55 MPH WIND~~ 2

2

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Question 010.10

To adequately evaluate Section 9.3.3, "Equipment and Floor Drainage Systems," provide additional information and detail explaining what is provided in each safety related compartment or area to assure the plant can be safely shut down after a postulated pipe break or crack in any system passing through or terminating in the compartment or area. Describe the protection provided, i.e., equipment or isolable compartment structure or area. In the case where equipment is provided for protection of the safety related components or system, describe what protective equipment is provided, where it is installed, and what function(s) does it perform to assure protection from flooding, of the safety related equipment in the compartment or area. Indicate what operator action, if any, and within what time interval it is required to prevent flooding of safety related equipment. Also, provide the results of an analysis that demonstrates compartment and/or area drains serving safety related components or systems have been sized for maximum flow conditions.

3.4.1.2

Response

~~See revised Section 9.3.3.2 and new Table 9.3-13.~~

See revised Section 3.4 and Table 3.4-2 for a discussion of the protection provided from internal flooding. See also revised section 9.3.3.2.1 for drain line flow sizing.

Question 010.2

Expand Section 3.4.1 to include the following:

1. Identify the safety-related systems and components that should be protected against floods, and show their relation to design flood levels and conditions.

2. Describe the structures that house safety-related equipment, including an identification of exterior or access openings and penetrations that are below the design flood levels.

3. Discuss the means of providing flood protection (e.g., pumping systems, stopplugs, water tight doors, and drainage systems) for safety related equipment that may be vulnerable to floods because of its location and the protection provided to cope with potential inleakage from such occurrences as cracks in structure walls, leaking water stops, and effects of wind wave action.

Response: See revised FSAR Section 3.4.1.

1. All safety-related systems and components listed in Table 3.5-1 under the heading "System, Component, or Structure" are protected against the effects of external flooding by:

   1) Being designed to withstand the maximum flood level and remain functional (such as Seismic Category I Structures and Condensate Storage Tank) or

   2) Being housed within Seismic Category I Structures which are designed as in 1 above.

   Table 3.4.1 shows the flood levels under various loading conditions on all the Category I structures.

2. Refer to Section 3.8 for the description of all Category I Structures that house safety-related equipment.

   Refer to Figure 3.4-2 which shows all the major openings and penetrations and the maximum flow profile during flooding. The figure also indicates the type of doors provided for these openings to make it waterproof during flooding.

3. The means of providing flood protection for safety-related equipment that may be vulnerable to flooding has been discussed in the response to NRC acceptance review question 010.10.

Response (Continued)

The potential inleakage from such occurrences as cracks in structural walls, leaking water stops, and effects of wind wave action is highly improbable because of the following preventive measures:

1) All construction joints in exterior walls and slabs are provided with water stops to el. 60 ft. and can withstand hydrostatic effects. All Seismic Joints between Category I Structures contain dual 9" water stops capable of withstanding potential seismic and hydrostatic effects.

2) Cracks in concrete will be minimized by imposing strict QA and QC procedures on the quality of concrete and construction techniques.

3) The effects of wind wave action has been taken into consideration when designing walls and construction joints.

Question 010.15

Your response to our Request 010.2 is not complete. Identify major penetrations on exterior walls of safety-related buildings that are below the design flood levels. Explain the methods of flood protection provided for these penetrations.

Response : See FSAR Section 3.4.1 and Figure 3.4-2.

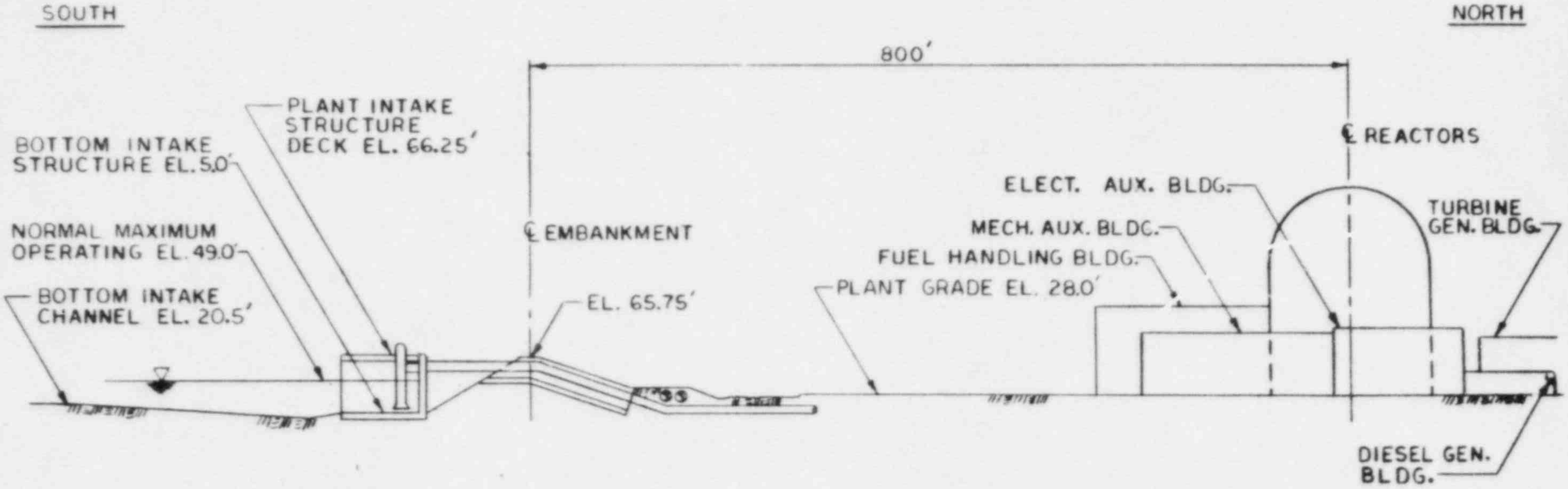
The design provisions for flood protection are provided in Section 3.4.1.2. Revised Figure 3.4-2 illustrates the locations of openings in Category I structures, located below El. 55.0 ft MSL. The actual design maximum flood elevations for the different buildings are defined in Table 3.4-1. The table below identifies the openings of exterior exposed walls below the actual design maximum flood elevation and the type of protection provided.

Table Q010.15

Openings Below Maximum Flood Elevation

Building	Elev.	Description	Protection
Fuel Handling	50'-5"	Cask Contamination Area Door	Watertight Door
M&E Auxiliary	35'-6"	Entrance Door	Watertight Door
M&E Auxiliary	26'-0"	3 Knock-Out Panels	Bolted Steel Panel w/Neoprene Gasket
M&E Auxiliary	41'7"	Entrance Door	Watertight Door
M&E Auxiliary	29'-0"	Entrance Door	Watertight Door
Diesel Generator	25'0"	3 Knock-Out Panels	Concrete Block, w/Steel Plates
Containment	39'9"	Auxiliary Lock	Double Interlocking Watertight Doors
ECW Intake	34'0"	6 Entrance Doors	Watertight Doors

Note: The watertight doors have neoprene seals attached to the door and compressed against a plate attached to the jamb.



SOUTH TEXAS PROJECT
UNITS 1 & 2

TYPICAL SECTION
THROUGH PLANT

Figure 3.4-1

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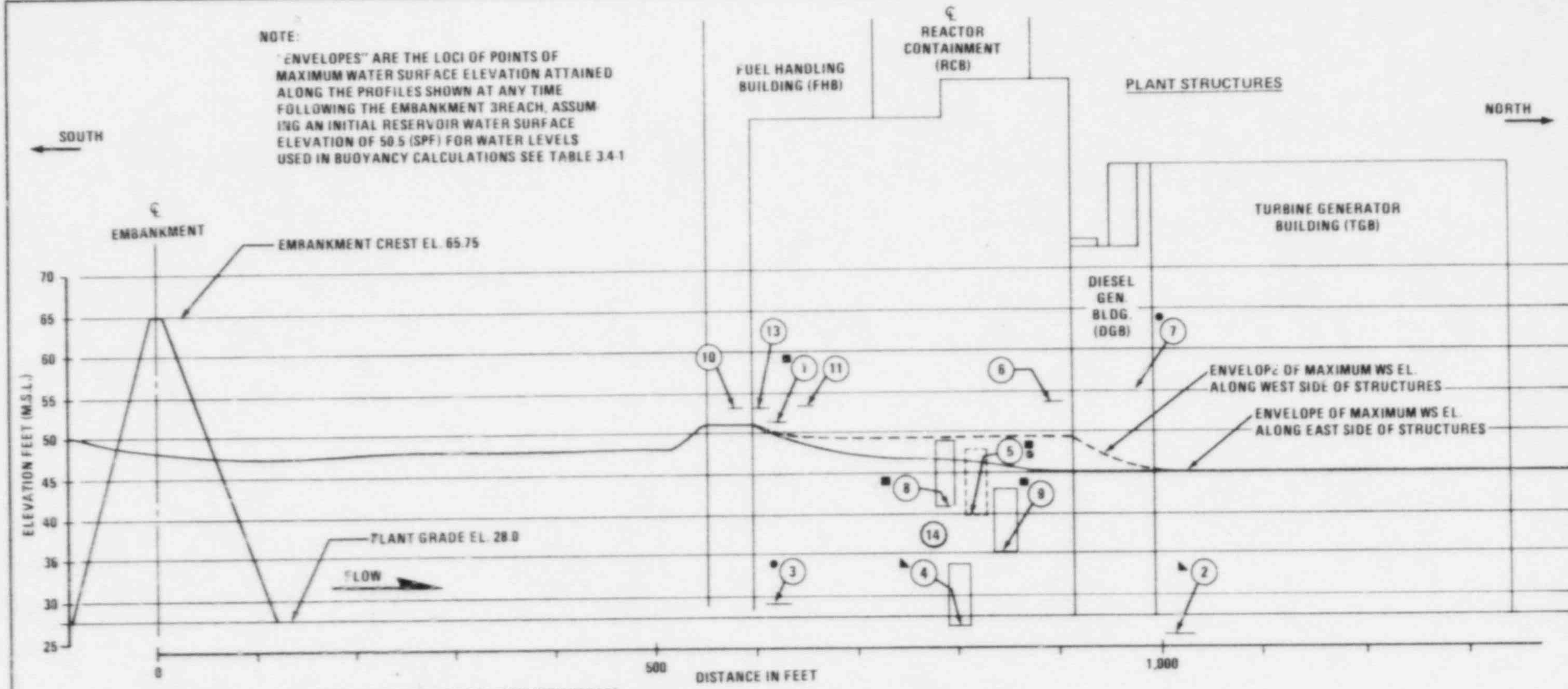


OPENINGS IN SAFETY RELATED STRUCTURES

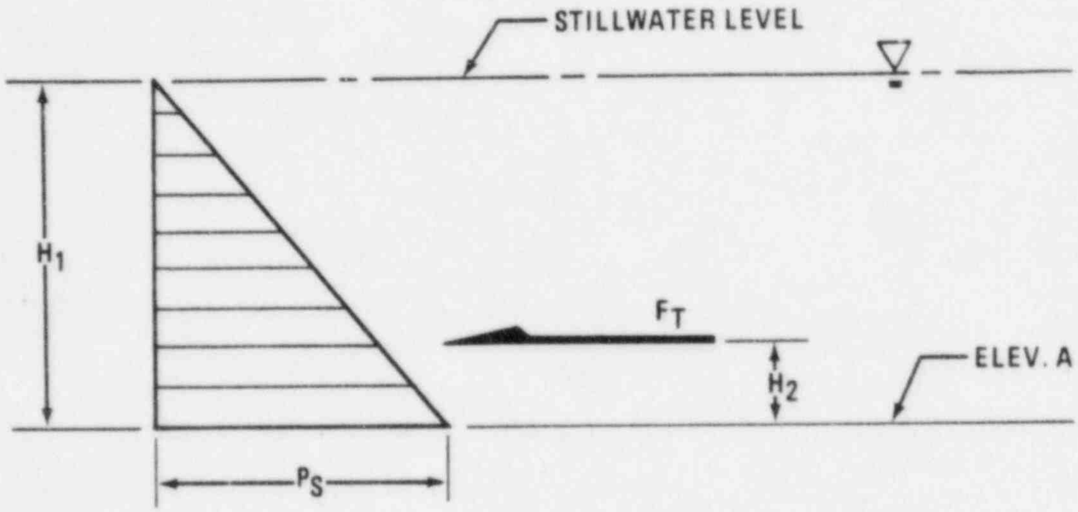
OPENING NO.	DESCRIPTION	ELEV.	OPENING NO.	DESCRIPTION	ELEV.
■ 1	CASK DECONTAMINATION AREA DOOR	50.5'	■ 8	ENTRANCE TO MEAB	41.6'
◣ 2	DGB KNOCKOUT PANELS	25.0'	■ 9	ENTRANCE TO MEAB	35.5'
■ 3	DOOR TO STORAGE AREA	29.0'	10	FUEL CASK DELIVERY DOCK (TOP OF FLOOD CONTROL BARRIER)	53.0'
◣ 4	EQUIPMENT REMOVAL KNOCKOUT PANELS FOR MEAB	26.0'	11	OPENING TO HVAC AREA IN FHB	53.25'
■ • 5	℄ RCB AUXILIARY ACCESS DOOR (6 – 2" DIA.)	39.75'	■ 12	(NOT SHOWN) DOORS TO ESSENTIAL COOLING WATER INTAKE STRUCTURE	34.0'
6	ENTRANCE TO MEAB	53.75'	13	EXIT FROM SOLID WASTE TO TRUCK BAY MEAB	53.0'
• 7	ENTRANCE TO DGB	55.0'	14	TGAS (W OF RCB)	38.0'

LEGEND:
- ◣ WATERTIGHT KNOCKOUT PANELS, CLOSED EXCEPT DURING COLD SHUTDOWN.
- ■ WATERTIGHT DOOR
- • DOORS ON WEST SIDE OF STRUCTURES

SOUTH TEXAS PROJECT UNITS 1 AND 2

MAXIMUM FLOW PROFILES THROUGH THE PLANT

Figure 3.4-2



P_S = LATERAL HYDROSTATIC PRESSURE IN KSF

H_1 = HEIGHT OF MAXIMUM WATER LEVEL ATTAINED AT FACE OF STRUCTURE

H_2 = POINT OF APPLICATION OF RESULTANT FORCE, F_T

F_T = RESULTANT FORCE IN K/FT

SOUTH TEXAS PROJECT UNITS 1 & 2

TYPICAL FORCE DIAGRAM

Figure 3.4-3

Question 130.7

Identify in this section of the FSAR the procedures used for transforming the dynamic flood effects to loads, and how these procedures compare with those delineated in the U.S. Army Coastal Engineering Research Center Technical Report No. 4, if applicable.

Response

~~Subsection 3.4.2.2 has been revised and expanded to provide a more thorough discussion of the procedures used to transform dynamic flood effects into loads. It should be noted that, with the single exception of the loadings on the north face of the plant and ECW Intake Structures and on the north ECP Embankment, the maximum design loads are governed by a quasi-steady state condition in which the contributing forces are totally hydrostatic or in which the dynamic components are negligible. For the case with dynamic components contributing to the maximum design load, the procedures given in the Shore Protection Manual, U.S. Army Corps of Engineers, Coast Engineering Research Center, 3rd Edition, 1977, were employed to transform the dynamic flood effects into loads. These procedures supercede that organization's previous procedures given in Technical Report No. 4. The procedures of the Shore Protection Manual for a solitary wave were also used to confirm the results obtained from the embankment breach analysis. The procedure for a solitary wave was a hydrostatic form of analysis.~~

As discussed in detail in ~~FSAR~~ Section 3.4.2.2, the maximum design loads are governed by a quasi-steady state condition. Thus, the referenced technical report is not applicable.

 Amendment 7, 7/16/79

STET

steam generator (SG) main feed pumps. These pumps and their drive turbines are protected from overspeed by ~~redundant~~ overspeed trips, and neither is considered to be a source of missiles.

b. HVAC and chiller fans were reviewed. Chillers have very low rpm fans which are not a credible source of missiles. Nearly all of the HVAC fans are separated from safety-related equipment and cable trays to the extent that postulated missiles do not pose a safety hazard. The supply subsystem fan is the only fan which might be a source of missiles and is located in the Mechanical-Electrical Auxiliaries Building (MEAB) at El. 60.0 ft. The blades of this fan are made of aluminum and are postulated to impact the housing at 26.7 ft/sec. The housing is 1/4-in.-thick steel and would contain such a missile.

c. The diesel generators (DGs) are designed to withstand overspeeds of 125 percent; redundant mechanical and electrical overspeed trips operate at 110 percent overspeed. The only portion of the diesels considered to be a credible source for postulated missiles is the turbocharger, which is not speed controlled and operates at high rpm. The turbocharger rotors weigh 270 pounds and are mounted on the diesels. In the event of failure, only one DG unit would be affected since each is separated from adjacent units by 2-ft-thick reinforced concrete walls which would contain any turbocharger missile.

d. Motor generator (MG) set flywheels were reviewed to determine missile generation potential. The fabrication specifications of the MG set flywheels control the material to meet American Society for Testing and Materials (ASTM) A533-70a, Grade B, Class I, with inspections in accordance with MIL-I-45208A and flame-cutting and machining operations governed to prevent flaws in the material. | 44
Nondestructive testing for nil-ductility (ASTM-E-208), Charpy V-notch (ASTM A593-69), ultrasonic (ASTM A578-71b and A577-70a), and magnetic particles (ASME Section III, NB2545) has been performed on each flywheel material lot. | 44
In addition to these requirements, stress calculations have been performed consistent with guidelines of American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel (B&PV) Code, Section III, Appendix A to show the combined primary stresses due to centrifugal forces and to show that the shaft interference fit does not exceed one-third of the yield strength at normal operating speeds (1,800 rpm) and does not exceed two-thirds of the yield strength at 25 percent overspeed. However, no overspeed is expected for the following reason: The flywheel weighs approximately 1,300 lbs and is 35.26 in. in diameter by 4.76 in. wide. The flywheel mounted on the generator shaft, which is directly coupled to the motor shaft, is driven by a 200-hp, 1,800-rpm synchronous motor. The torque developed by the motor is insufficient for overspeed. Therefore, there are no credible missiles from the MG sets.

3.5.1.1.3 <u>Gravitational Missiles</u>: Virtually the only significant gravitational missiles would be from overhead cranes. As discussed in Section 9.1.4, overhead cranes either have interlocks or are single-failure-proof or | 36

a postulated

Water-borne missiles would have a maximum velocity of 20 ft/sec, corresponding to the conservatively calculated maximum water speed resulting from ~~the~~ failure of the Cooling Reservoir embankment. Such missiles could consist of waterborne debris such as automobiles, utility poles, wooden planks, etc. The effects from such missiles are considerably less severe than the effects of the postulated tornado missiles.

Missiles resulting from hurricane winds could be postulated to be similar to the types of missiles generated by tornadoes; however, due to the lower hurricane wind speeds, the effects would be less severe than the effects of tornado-generated missiles. Tornado-generated missiles are used as design basis missiles for STP. A maximum tornado wind speed of 360 mph consistent with a Region I design basis tornado of RG 1.76 (April 1974) is used to calculate the missile velocities. The design parameters for tornado missiles are summarized in Table 3.5-9.

Structures, systems, and components whose failure could prevent safe shutdown of the reactor or result in significant uncontrolled release of radioactivity are protected from such failure due to design tornado and wind and missile loading by the following methods: 36

1. Structure or component is designed to withstand tornado wind loading or tornado missile. 36

2. Component is housed within a structure which is designed to withstand the tornado wind loading and tornado missile loading.

The only exception to the above is the Isolation Valve Cubicle (IVC) where the probability of a tornado missile strike on the roof is demonstrated by analysis to be much less than 1×10^{-7}. 36

Although RG 1.117 (June 1976) is not applicable to STP, the design of STP is such that the structures, systems, and components specified in the appendix to the guide are protected against tornadoes and tornado missiles. As a result, STP is in compliance with RG 1.117. Information on barriers used to protect the principal systems is given in Table 3.5-10. 36

3.5.1.5 <u>Missiles Generated by Events Near the Site</u>. As discussed in Section 2.2.3, missiles originating from events near the site, such as from explosions, do not impact safety-related structures or components and do not constitute design basis events.

3.5.1.6 <u>Aircraft Hazards</u>. As discussed in Section 2.2.2, there are no airports within 10 miles with greater than 500 d^2 operations per year or farther than 10 miles with greater than 1,000 d^2 operations per year (d is the distance to the airport); therefore, aircraft activities from nearby airports do not constitute a hazard to STP.

The only nearby military aviation activity was flight route OB-19, which was used by the U.S. Air Force and Navy for low-level navigation-bombing training flights for jet aircraft, but the route was cancelled as of January 30, 1975. In fact, the route was not used for 3 years prior to that. Thus, there is no hazard to STP from military aviation activity.

TABLE 3.5-1 (Continued)

SAFETY CLASS SYSTEMS AND COMPONENTS AND SEISMIC CATEGORY I STRUCTURES TO BE PROTECTED

System, Component, or Structure	Location	External Missile Protection	Internal Missile Protection	FSAR Reference
Essential Cooling Water System				Section 9.2.1.2
Essential cooling water pumps	ECWIS	B	D	
Strainers	ECWIS	B	D	
Screen Wash System	ECWIS	B	D	
Piping	MAB, DGB, ECWIS	B	D	
Valves	MAB, DGB, ECWIS	B	D	
Auxiliary Feedwater System				Section 10.4.9
Pumps	IVC	B	D	
Pump turbine	IVC	B	D	
AFW piping from AFST to AFW pumps	IVC	B	D	
AFW piping & valves from AFW pumps to SGs	RCB/IVC	B	D	
AFW pump test/recirc. liner inside IVC	IVC	B	D	
Feedwater System				Section 10.4.7
Those portions of the FW System extending from and including the secondary side of the SGs up to and including the first restraint outside the valve cubicle and connected piping up to and including the first valve that is either normally closed or capable of automatic closure during all modes of normal reactor operation.	RCB/IVC	B	D	
Feedwater control valves	IVC	B	D	

TABLE 3.5-10

BARRIERS FOR TORNADO MISSILES
(Sheet 2 of 2)

Protected Systems and Components	Missile Barrier	Concrete Thickness (in) Walls	Roof	Concrete Strength (PSI)	Curing Time (Days)
Main steam line isolation valves and auxiliary feedwater pumps	Containment Structure Wall	48	-	5500	90
	Isolation Valve Cubicle Wall	24(a)	(b)	4000	28
Auxiliary feedwater storage tank	Concrete Tank Walls and Roof	30(c)	30(c)	4000	28
Auxiliary feedwater ~~transfer pumps,~~ lines and valves	Valve Pit	24	24	4000	28
Essential cooling water system piping	Underground	NA	NA		
Class 1E outside electrical raceway system	Underground	NA	NA		

(a) Minimum thickness.
(b) Roof is metal deck. Risk analysis for tornado missile strike yields probability of $<10^{-7}$.
(c) Including 1/4" stainless steel plate liner.

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