2.4S.2 Floods

The following site-specific supplement addresses COL License Information Item 2.14.

This section identifies historical flooding at the STP 3 & 4 site. It identifies and summarizes the individual types and combinations of flood-producing phenomena considered in establishing the flood design basis for safety-related plant features. The potential effects of local intense precipitation are also discussed in this section.

2.4S.2.1 Flood History

The major natural events that may cause flooding near the STP 3 & 4 site are flooding from the Colorado River, hurricane-induced storm surges from the Gulf of Mexico, and local site flooding from Little Robbins Slough.

Little Robbins Slough is an ephemeral stream near the STP 3 & 4 site with its headwaters located approximately 2 mi northwest of the site. It has a total drainage area of approximately 4 sq. miles at the STP 3 & 4 site. Little Robbins Slough enters the STP site by crossing an irrigation canal of the Texas Gulf Coast Irrigation District through a siphon structure, as indicated on the United States Geological Survey (USGS) topographic map (Reference 2.4S.2-1). It passes under Highway FM 521 through pipe culverts. There are no stream gauge data or flood records available for Little Robbins Slough. The flood potential for Little Robbins Slough is discussed in connection with flood elevations from a local intense precipitation (also defined as the local probable maximum precipitation or local PMP) storm event in Subsection 2.4S.2.3.

The major historical hurricanes that affected the STP site are discussed in Subsection 2.4S.5. Table 2.4S.5-1 provides a chronological list of all major hurricanes that affected the Texas coast between 1900 and 2005. One of the most notable hurricanes that devastated the Texas Coast near the site is Hurricane Carla. This Category 4 hurricane (hurricane category in Saffir-Simpson Hurricane Scale) had landfall near Port O'Connor, Texas on September 11, 1961. It produced a peak surge elevation of approximately 20 ft MSL near the head of the Lavaca Bay (Reference 2.4S.2-2). North of the Gulf Inter Coastal Waterway and near the STP site, the peak surge elevation from Hurricane Carla was estimated to be about 16.5 ft MSL (Reference 2.4S.2-2).

The USGS maintains a network of stream gauging stations in the Colorado River basin. Three gauge stations, at Bay City (USGS gauge number 08162500), Wharton (08162000) and Columbus (08161000), are located closest to the site, as shown in Table 2.4S.1-3. As discussed in Subsection 2.4S.1, larger floodplain width downstream from the City of Columbus results in a large attenuation of flood peaks downstream from the Columbus gauge. Consequently, stream gauging data from Bay City and Wharton are more representative of the streamflow conditions near the STP 3 & 4 site. The USGS gauge station at Bay City is located approximately 16 mi upstream from the site. The gauge at Wharton is located further upstream, approximately 50 mi from the site. Because the watershed boundary downstream of Wharton is very narrow, the contributing river basin drainage areas for the two stations are very similar in size (Table 2.4S.1-3). The locations of the gauge stations are shown in Figure 2.4S.1-8.

Annual peak streamflow data at Bay City are available for the water years 1940 and from 1948 to 2006 (Reference 2.4S.2-3). A water year starts on October 1 of the preceding year and ends on September 30 of the current year. For example, the water year 1940 ranged from October 1, 1939 to September 30, 1940. At Wharton the period of records ranges between water years 1919 and 2006 (Reference 2.4S.2-4). Annual peak streamflow data for the period of record for the two stations are presented in Tables 2.4S.2-1 and 2.4S.2-2, respectively. The variations of annual peak stream flow are also shown in Figure 2.4S.2-1 and Figure 2.4S.2-2. The two gauge stations are located below the series of dams on the Colorado River, which are discussed in Subsection 2.4S.1 and shown in Figure 2.4S.1-5. Of the dams and reservoirs in the lower Colorado River basin, Mansfield Dam along with Lake Travis is the most downstream dam that provides the maximum floodwater storage and management. The Mansfield Dam was constructed in 1942. For the water years ranging from 1942 to 2006, the highest observed peak streamflow in the Lower Colorado River at Bay City was 84,100 cfs on June 26, 1960. At Wharton the observed peak streamflow for the same period was 74,800 cfs recorded on October 23, 1960. The historic peak at Wharton was recorded on June 20, 1935, with a peak streamflow of 159,000 cfs (Reference 2.4S.2-4).

The ten highest recorded flood elevations at Bay City are shown in Table 2.4S.2-3 (References 2.4S.2-5 and 2.4S.2-6). Although reported by the USGS, the flood levels before 1948 were recorded by others. The historical highest flood levels at Bay City were due to flooding in the uncontrolled basin of the Colorado River (elevation 56.1 ft MSL in 1913, 55.4 ft MSL in 1922, 55.0 ft MSL in 1929 etc., as shown in Table 2.4S.2-3). After the construction of the dams, the highest flood levels observed in the Lower Colorado River at Bay City were 46.4 ft MSL and 38.67 ft MSL in the 1960 and 1995 water years, respectively.

Flood elevations near the STP 3 & 4 site are available from published studies. Halff Associates Inc. (Reference 2.4S.2-7) studied the flood hydrology of the Lower Colorado River as part of the Colorado River Flood Damage Evaluation Project – Phase I for the U.S. Army Corps of Engineers and the Lower Colorado River Authority. For the October 1998 flood event, which represents one of the highest recorded flow rates on the Lower Colorado River, with a peak flow of 81,800 cfs at the Bay City gauge, Halff Associates Inc. computed a water surface elevation of approximately 21.0 ft MSL upstream of Highway FM 521 bridge crossing. The bridge crossing is located east of the STP site.

The USGS recently established a water surface elevation gauge station on the Colorado River Bypass Channel near Matagorda (08162506), approximately 10 mi south of the STP 3 & 4 site. The gauge station has data records from October 1999 to May 2007. A maximum recorded water surface elevation of 7.05 ft MSL was observed at this station (on the East Colorado River) on November 27, 2004 (Reference 2.4S.2-8). The streamflow magnitude for this day was approximately 72,900 cfs at the Bay City gauge on the Colorado River (Reference 2.4S.2-3).

There are no records of ice sheet formation or ice jam events on the Colorado River or Little Robbins Slough, as discussed in Subsection 2.4S.7. Also, there are no records of any landslide (submarine or subaerial) or distant tsunami source-induced flooding events at the STP 3 & 4 site. Historical tsunami events are discussed in Subsection 2.4S.6.

2.4S.2.2 Flood Design Considerations

The design basis flooding (DBF) elevation for the STP 3 & 4 site is determined by considering a number of different flooding scenarios. The potential flooding scenarios applicable and investigated for the site include the following: probable maximum flood (PMF) on streams and rivers, potential dam failures, probable maximum surge and seiche flooding, probable maximum tsunami, flooding due to ice effects, and potential flooding caused by channel diversions. In applicable cases, the flooding scenarios were investigated in conjunction with other flooding and meteorological events, such as wind generated waves and tidal levels, as recommended in the guidelines presented in ANSI/ANS 2.8-1992 (Reference 2.4S.2-9). Detailed discussions on each of these flooding events and how they were estimated are found in Subsections 2.4S.3 through 2.4S.7, and Subsection 2.4S.9.

The estimation of the PMF water level on the Colorado River is discussed in Subsection 2.4S.3. Three different combinations of parameters including PMP storm events and antecedent water levels, contributing catchment areas, upstream reservoir releases, and base flow conditions are considered in estimating the PMF streamflow magnitude. The maximum PMF water level for the Colorado River at the STP 3 & 4 site has been determined to be at elevation 26.3 ft MSL. Because the site grade for the STP 3 & 4 power block and the ultimate heat sink (UHS) areas are located above elevation 32 ft MSL the PMF elevation would not cause a flooding risk to any of the safety-related systems, structures and components (SSCs).

The impacts of postulated dam failures on the STP 3 & 4 safety-related SSCs are discussed in Subsection 2.4S.4. Two aspects of flooding are considered. First, flood elevation at the site is investigated as a result of cascading failure of dams in the Colorado River basin and its tributaries upstream of the site. The resulting water level at the site, including coincidental wind set-up and wave run-up is 34.4 ft MSL. Second, the flood elevation at the site is investigated due to the failure of the Main Cooling Reservoir (MCR) embankment. A maximum flood elevation of 38.8 ft MSL was determined at the STP 3 & 4 site as a result of the MCR embankment breach. Based on this, the DBF is conservatively established as 40.0 ft MSL. The MCR embankment breach flood level is above the site grade and the ground floor elevation of the safety-related SSCs in the power block area. Therefore, all power block safety-related structures will require appropriate flood protection measures below elevation 40.0 ft MSL, such as water tight doors and components that will prevent any flooding of the safety-related SSCs. The UHS and reactor service water (RSW) pump house is contiguous with the UHS basin. The UHS basin and RSW pump house are water tight below elevation 50 ft MSL. Flooding of these structures due to DBF is therefore precluded. Flood protection requirements are discussed in Subsection 2.4S.10.

Probable maximum surge and seiche flooding as a result of the probable maximum hurricane (PMH) in the Gulf of Mexico is presented in Subsection 2.4S.5. The probable maximum storm surge (PMSS) water level at the STP 3 & 4 site is estimated to be elevation 31.1 ft MSL. This elevation is lower than the flood elevation at the site due to the postulated breach of the MCR embankment and cascading failures of dams in the Colorado River. The PMSS flood elevation is also lower than all entrance elevations of safety-related SSCs at the STP 3 & 4 site.

Subsection 2.4S.6 describes the estimation of the probable maximum tsunami (PMT) water level. The maximum water level associated with a PMT at the STP 3 & 4 site is 11.5 ft MSL. Therefore, the PMT would not be a flood risk to the STP 3 & 4 site. As discussed in Subsections 2.4S.7 and 2.4S.9, it is unlikely that ice effect and channel diversions, respectively, would pose any flood risk to the STP 3 & 4 site.

The maximum water level due to a local PMP storm event is estimated and discussed in Subsection 2.4S.2.3. The maximum water level in the power block area due to a local PMP storm event is estimated to be at elevation 36.6 ft MSL. The maximum flood elevation due to the local PMP storm event is also above the ground floor elevations for the safety-related SSCs in the power block area. However, this elevation is lower than the flood elevation due to the postulated breach of the MCR embankment or cascading failures of dams in the Colorado River. Flood protection measures adopted for the DBF would adequately provide the required protection to the safety-related SSCs against flooding due to a local PMP storm event.

2.4S.2.3 Effects of Local Intense Precipitation

The effects of local intense precipitation (or local PMP) in the vicinity of the STP 3 & 4 site are discussed in this subsection. The site drainage system used in the analysis of flooding due to a local PMP storm is conceptualized such that it would facilitate drainage away from the plant safety-related SSCs during non-PMP flood events.

2.4S.2.3.1 Probable Maximum Precipitation Depths

The design basis for the local intense precipitation is the all season one sq. mile or point PMP as obtained from the U.S. National Weather Service (NWS) Hydrometeorological Reports No. 51 and 52 (HMR-51 and HMR-52) (References 2.4S.2-10 and 2.4S.2-11). Table 2.4S.2-4 presents the one sq. mile PMP depths for various durations at the STP 3 & 4 site with the 5-minute and 1-hour one sq. mile PMP depths estimated to be 6.4 in. and 19.8 in., respectively. The 5-min and 1-hour local PMP depths on one sq. mile exceed the corresponding maximum rainfall rates of 6.2 in. (15.7 cm) and 19.4 in. (49.3 cm), respectively, specified in the reference ABWR DCD Tier 1 Table 5.0, and Tier 2 Table 2.0-1 (see STP DEP T1 5.0-1). Justification for the departure specific to the STP 3 & 4 site is further discussed in Table 2.0-2. The estimated rainfall depths presented in HMR-51 are for precipitation duration ranging from 6 hrs to 72 hrs, and for drainage areas from 10 sq. miles to 20,000 sq. miles. Based on these rainfall depths, HMR-52 provides a procedure for estimating short duration point (or one sq. mile) PMP depths for up to 1 hr rainfall duration. PMP rainfall depths for duration between 1 hr and 6 hrs are obtained by logarithmic fit of rainfall depths available for different storm durations. Figure 2.4S.2-3 is a plot of the PMP depths against corresponding storm durations, as obtained from HMR-51 and HMR-52. The PMP depths for 2 hrs and 3 hrs durations are also shown on the figure.

2.4S.2.3.2 Local Drainage Components and Subbasins

The STP 3 & 4 site is located northwest of the STP 1 & 2 site adjacent to the plant access road from the north. The proposed site layout and drainage system are shown in Figure 2.4S.2-4. The site grade in the immediate vicinity of the STP 3 & 4 power block area ranges from an elevation of about 30 ft MSL to 36.6 ft MSL, with generally higher elevation on the northern side. The center of the power block area has the highest grade elevation of about 36.6 ft MSL, which slopes down towards the corners with a 0.4% gradient. The corners of the power block area are at an elevation of approximately 32 ft MSL.

The STP 3 & 4 power block and UHSs areas are contained within a security perimeter. Drainage from the area within the security perimeter is collected in two drainage channels labeled the East and West Channels, running north-south that drain to an east-west running Main Drainage Channel (MDC) north of the power block area and outside the security perimeter (Figure 2.4S.2-4). The security perimeter includes personnel security fences at grade elevation and a concrete vehicular barrier outside the security fences. Drainage across the security barrier is permitted through narrow, grated openings in the concrete vehicular barrier and underground culverts in the security fences at the drainage ditches. For purposes of the local PMP analysis the openings in the vehicular barrier and the culverts are conservatively assumed to be blocked. Consequently, during a PMP storm event drainage from within the area enclosed by the security perimeter would likely take place by overtopping of the concrete barrier.

The East Channel is located within the security perimeter along the north access road (Figure 2.4S.2-4). Drainage from the STP 3 power block area and UHS is collected at catch basins and diverted to the East Channel by connecting drainage pipes. Similarly, for the STP 4 power block area, and UHS drainage collected at catch basins is diverted by pipes to the West Channel (Figure 2.4S.2-4). During a PMP storm event all catch basins and pipe flows are assumed inoperative; consequently, surface runoff from the power block area would be collected in the drainage channels as overflows. Runoff from building roofs is conservatively assumed to contribute to surface runoff without any delay. Roof runoff is also collected by the East and West Channels.

Outside the security perimeter, drainage from the STP 1 & 2 switchyard, located east of the north access road, is directed towards the main drainage channel. West of the security perimeter, drainage from the area within the west access road (Figure 2.4S.2-4) and the Main Drainage Channel are collected by a small channel that flows north towards the Main Drainage Channel.

Apart from the drainage from the area within the security perimeter, the Main Drainage Channel also collects runoff from a portion of the area bounded by Highway FM 521. The Main Drainage Channel first runs westward parallel to the security barrier north of the STP 3 & 4 power block area, turns southwestward near the northwest corner of the security barrier, runs south parallel to the west MCR embankment, and finally drains to

Little Robbins Slough south of the MCR. Prior to crossing the West Access Road the Main Drainage Channel also joins with Little Robbins Slough at a second location by a link channel (Figure 2.4S.2-4). Approximately 500 ft south of the link channel junction, both the Main Drainage Channel and Little Robbins Slough cross under the West Access Road through pipe culverts.

Considerable area north of Highway FM 521 contributes to Little Robbins Slough, as shown in Figure 2.4S.2-5. Runoff from this area crosses Highway FM 521 also through pipe culverts. In the event of a PMP storm it is likely that Highway FM 521 would be overtopped and runoff from this area would contribute to the flow in Little Robbins Slough.

For the analysis of local PMP flooding, the STP 3 & 4 site drainage area has been divided into seven subbasins based on USGS topographic maps, site aerial survey, and the location of roads and barriers. These subbasins, North1, North2, North3, PBN1, PBW1, PBW and PBE, are shown in Figure 2.4S.2-5. Contributing drainage areas for each of the subbasins are presented in Table 2.4S.2-5. Runoff from these subbasins provides the peak discharges entering the East and West Channels, the Main Drainage Channel, and Little Robbins Slough (Figure 2.4S.2-4).

2.4S.2.3.3 Peak Discharges

The U.S. Army Corps of Engineers computer program HEC-HMS (Reference 2.4S.2-12) was used to develop the hydrologic model and determine peak discharges in the drainage basins. The land surface cover for the STP 3 & 4 subbasins consists primarily of developed impervious areas, fallow land, irrigated land, and small grass surfaces. Within the catchment areas south of Highway FM 521, Laewest series (LaA) soil is present, which consists of very deep, moderately well drained, very slowly permeable, clayey soils on uplands. These soils are formed in calcareous, clayey marine sediments (Reference 2.4S.2-13). North of FM 521, in addition to LaA soil series, loamy Dacosta (DaA) and Edna (EdA) soil series are also present in the irrigated areas. These soils are also very slowly permeable and moderately drained. All these soil series fall in hydrologic soil group ‘D’. To estimate peak discharges from a PMP storm event, however, all surfaces are conservatively assumed to be impervious to reflect a saturated ground condition prior to the start of the PMP storm event.

The times of concentration (tc) for the subbasins are estimated using the methodologies suggested by the U.S. Natural Resources Conservation Service (NRCS), as given in TR-55 Manual (Reference 2.4S.2-14). To account for non-linearity effects during extreme flood conditions, the computed tc were reduced by 25% in accordance with guidance from the U.S. Army Corps of Engineers Engineering Manual EM-1110-2-1417 (Reference 2.4S.2-15). The lag times are estimated as 60% of tc (Reference 2.4S.2-16).

Runoff from the subbasins North1 and North2 contributes to Little Robbins Slough after crossing Highway FM 521 through pipe culverts. It is assumed that during a local PMP storm event runoff would accumulate upstream of FM 521 in addition to the flow through the culverts to Little Robbins Slough. When the upstream storage elevation exceeds the crest elevation of FM 521, overtopping of FM 521 would contribute additional flow to Little Robbins Slough. Because the culverts are located upstream of the site, allowing these culverts to flow into the basin increases the local PMP discharges to the site. Thus, the culverts were assumed to be open, rather than blocked, as a conservative modeling approach for the STP 3 & 4 site.

The subbasin areas, local PMP intensities, and lag times were input to the HEC-HMS computer model. A runoff curve number of 98, representing impervious surfaces (Reference 2.4S.2-14), is used in the model for the entire drainage area. The NRCS dimensionless unit hydrograph option in HEC-HMS was utilized for the developments of the peak discharges from the various subbasins. A storm duration of 6 hrs was specified along with 5 min duration for incremental rainfall intensities.

A schematic of the HEC-HMS model is given in Figure 2.4S.2-6, and resulting peak discharges along with the time of peak for different subbasins are presented in Table 2.4S.2-6. Although the subbasin PBN1 drains to the Main Drainage Channel over its length, in the HEC-HMS model the drainage is directed towards the drainage element OutFlow as shown in Figure 2.4S.2-6. Distribution of flow from PBN1 to the Main Drainage Channel is accounted for in the hydraulic model developed for the system to compute the PMP flood elevation. The hydraulic model is discussed in Sub-section 2.4S.2.3.4. Table 2.4S.2-6 shows that because of longer lag times, upstream storage and overflow of Highway FM 521, the combined peak discharge from the subbasins North1 and North2 occur at hour 6:25 at the upstream boundary of Little Robbins Slough (LRS, see Figure 2.4S.2-6 for drainage elements). As a result, the flood peak in Little Robbins Slough at the confluence with the Main Drainage Channel occurs much later than the combined flood peak discharge in the Main Drainage Channel (MDC4 in Figure 2.4S.2-6) and the subbasin PBN1. Therefore, the combined peak discharge from the Main Drainage Channel and PBN1 is the largest contributor to the peak discharge from the entire local PMP basin (OutFlow in Figure 2.4S.2-6), even though the combined peak discharge from Little Robbins Slough (LRS) and North3 is of comparable magnitude to the peak discharge from the entire basin. This also indicates that flooding in Little Robbins Slough alone would not produce a flood magnitude at the STP 3 & 4 site that is higher than the flood magnitude produced by the local PMP flood event from the site.

2.4S.2.3.4 Hydraulic Model Setup

The computer program HEC-RAS, also developed by the U.S. Army Corps of Engineers (Reference 2.4S.2-17), was used to estimate the peak local PMP water levels in the power block area. Cross sections for the drainage channels included in the model were developed at locations shown in Figure 2.4S.2-7. The Main Drainage Channel has a bottom width of 30 ft and a longitudinal gradient of 0.061 percent. The bottom elevation at cross section 5280 (Figure 2.4S.2-7) was 24 ft MSL. In the HEC-RAS model this cross section was also used unchanged at the upstream cross section 5380. Both the East and West Channels have a bottom width of 10 ft with longitudinal gradients of 0.298 percent and 0.756 percent, respectively. The bottom elevation of the upstream cross sections (cross section 1690 in Figure 2.4S.2-7) of both the channels was 29 ft MSL. The Main Drainage Channel and the East and West

Channels all have a horizontal (H) to vertical (V) side slope of 3:1 (H:V). Little Robbins Slough is a natural stream for which the cross sections were obtained from aerial survey data. Figure 2.4S.2-7 shows very wide floodplain areas associated with several channels. The floodplain areas likely provide storage volume and contribute to channel conveyance when floodwater rises above the floodplain. These were modeled with a high surface roughness coefficient, as discussed later in this sub-section. The cross section data were input into the HEC-RAS model. The two downstream cross sections on Little Robbins Slough intersect the adjacent cross sections on the Main Drainage Channel. This was allowed in the model to avoid a sudden change in cross sectional properties from the immediate upstream cross sections, in addition to defining a portion of the upstream cross sections as ineffective flow area in the HEC-RAS model. The area of overlap was approximately 1 percent of the area of subbasin North3. Because the water surface elevation at the junction of Little Robbins Slough with the Main Drainage Channel is governed by the water surface elevation in the Main Drainage Channel, the overlap in cross section would have insignificant impact on the model results.

The inflow discharges were also input into the HEC-RAS model. The discharges were estimated from the HEC-HMS discharge hydrographs. Table 2.4S.2-6 indicates that subbasin peak discharges do not occur at the same time. The peak discharge at the downstream outflow location (OutFlow) occurs within 25 min of the peak discharges for subbasins PBE, PBW, PBN1, and PBW1. The peak discharge in Little Robbins slough (LRS) occurs much later than those for the subbasins. To determine the maximum water surface elevation within the power block area, it is therefore conservatively assumed that the peak discharges for subbasins PBE, PBW, PBN1, and PBW1 occur simultaneously with the outflow peak. For subbasin North3 and branch LRS the flow discharges corresponding to the time of peak outflow discharge (hour 3:35) were used to develop the inflow discharge for the HEC-RAS model. Because the peak discharges in the subbasins near the power block area are assumed to occur simultaneously, the total discharge at the outflow location in the HEC-RAS model is greater than the corresponding total peak discharge obtained from the HEC-HMS model.

The peak discharge obtained for a subbasin in HEC-HMS was first distributed to the most upstream cross section of a stream reach in HEC-RAS in proportion to the area contributing to that cross section and the total area of the subbasin. The remaining portion of the peak discharge is then distributed equally among the remaining cross sections within the receiving channel reach. For example, subbasin PBE has a total area of approximately 0.089 sq. miles that contributes a peak flow of 1443.3 cfs (Table 2.4S.2-6) to the East Channel. The most upstream cross section of the East Channel receives flow from approximately 0.039 sq. miles area. Therefore, a peak flow of (0.039/0.089 × 1443.3 =) 632.5 cfs was assigned at the upstream cross section. The remaining flow was divided equally between the remaining cross sections and added to the upstream peak flow. The same approach was also adopted for the West Channel. Peak flow from the subbasin PBN1 was distributed similarly over the Main Drainage Channel. For the Main Drainage Channel, two segments of the channel reach were considered in distributing the peak flow discharge based on uniformity in cross section extent and areas associated with the uniform cross section channel reach. Additionally, peak flow from subbasin PBW1 was added to the Main Drainage

Channel as a point inflow at a location where a small stream drains the runoff from this subbasin. The outflow from USLRS (Figure 2.4S.2-6) was used as inflow at the upstream cross section on Little Robbins slough, whereas runoff from the subbasin North3 was distributed equally over the cross sections of Little Robbins Slough. Downstream of a junction, the combined flow from the channels upstream of the junction was specified in addition to any fraction of peak discharge at the cross section.

The total discharge to the channels is assumed to remain within the model basin and drains through the downstream boundary on the Main Drainage Channel at the West Access Road crossing. Consequently overflow from the model area to adjacent drainage areas to the west, east, and southeast is not allowed for a local PMP storm event. Because additional subbasin discharge from the area outside Highway FM 521 contributes to the total flow in the model, the resulting water surface elevation within the model domain would be conservative.

The vehicular security barrier surrounding the plant area (Figure 2.4S.2-4) was modeled as a vertical wall on three sides (east, west, and south) of the power block area. Where the security barrier crosses the East and West Channels north of the STP 3 & 4 power block area, in-line weirs were used to model the extent and flow over the barrier. As water level inside the security perimeter exceeds the top elevation of the barrier, flow would begin to spill over the barrier depending on the water level in the Main Drainage Channel at the channel junction. The antecedent water level within the security perimeter to pass the peak flow from the East and West Channels was assumed to be the same as the height of the security barrier. The combined flow from the Main Drainage Channel and Little Robbins Slough discharges to the area south of the West Access Road through pipe culverts. Because the pipe culverts were assumed to be blocked, outflow would take place only by overtopping of the road, which was also modeled as an in-line weir structure in HEC-RAS. The width and breadth of the in-line structure were obtained from the aerial survey data for the West Access Road in this area. A broad-crested weir coefficient of 2.6 was assumed for all the inline structures.

Flow interactions at the junctions were analyzed with the energy junction equation option in HEC-RAS. Depending on the surface cover of the drainage channels and floodplains, various Manning’s *n* values were used in the model as recommended in Reference 2.4S.2-18. In the power block area, the East and West Channels have a gravel bottom with sides of dry rubble or riprap. An *n* value of 0.036 is used for these channels. The main drainage channel consists of straight reach sodded earthen banks for which a Manning’s *n* value of 0.033 is used. Little Robbins Slough is an excavated earthen bank winding and sluggish stream with short grasses. An *n* value of 0.033 is also recommended for such streams. For the floodplains, a Manning’s *n* value of 0.07 is conservatively assumed for those of the East and West Channels that represents a surface with heavy weeds. For all other floodplains a Manning’s *n* of 0.16 was used considering that the floodplains consist of medium to heavy brush on the surface.

The HEC-RAS model simulation was performed for a steady state sub-critical flow condition for which only the downstream boundary condition is required on the Main Drainage Channel. A sensitivity analysis of model results indicated that the flow over the weir structure at the West Access Road crossing would be controlled by an upstream flow condition when the downstream boundary water level is below approximately elevation 34 ft MSL. Considering that a large portion of the flow is intercepted upstream of the West Access Road, it is very unlikely that the water surface elevation downstream of the West Access Road would be above 34 ft MSL. Consequently a downstream boundary with constant water level at 34 ft MSL is selected for the simulation of local PMP flood elevations.

2.4S.2.3.5 Flood Elevations

The HEC-RAS computer model simulation was used to estimate the maximum water surface elevation within the STP 3 & 4 power block area. Model simulation results showed that the maximum water surface elevation within the power block area was elevation 36.6 ft MSL. This elevation is conservatively assumed to affect the entire power block area of STP 3 & 4. This flooding elevation is higher than the power block grade elevation and the ground floor slab elevation of the safety-related SSCs. However, the local PMP water surface elevation is less than the flood elevation estimated from the postulated breach of the MCR embankment, which was estimated to be at elevation 38.8 ft MSL, as discussed in Subsection 2.4S.4. Flood protection measures for the safety-related SSCs against flooding due to the MCR embankment breach are sufficient to provide protection against flood elevation due to the local PMP storm event.

Runoff from roofs of structures in the power block area would contribute to flows in the East and West Channels. Because the power block area would be inundated during a local PMP storm event, flooding of the safety-related SSCs due to sheet flow from roof and surface runoff is not relevant.

The RSW pump house is contiguous with the UHS basin. The lowest elevation associated with the entrances or openings of these safety-related structures is the RSW pump house slab elevation that is at 50 ft MSL, below which the UHS is water tight. Flooding of these structures due to a local PMP storm event is therefore precluded.

2.4S.2.4 References

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2.4S.2-2 “Verification Study of a Bathystrophic Storm Surge Model,” G.P.-Carayannis, Technical Memorandum No. 50, Coastal Engineering Research Center, U.S. Army Corps of Engineers, May 1975.

2.4S.2-3 “Peak Streamflow for Texas, USGS 08162500 Colorado River near Bay City, TX,” United States Geological Survey, National Water Information System. Available at http://nwis.waterdata.usgs.gov/tx/nwis/peak?site_no=08162500&agency_cd=USGS&format=html, accessed April 27, 2007.

2.4S.2-4 “Peak Streamflow for Texas, USGS 08162000 Colorado River at Wharton, TX,” United States Geological Survey, National Water Information System. Available at http://nwis.waterdata.usgs.gov/tx/nwis/peak?site_no=08162000&agency_cd=USGS&format=html, accessed April 27, 2007.

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2.4S.2-6 “Water Data Report TX-2005, 08162500 Colorado River near Bay City, TX,” The United States Geological Survey, Texas, 2006.

2.4S.2-7 “Colorado River Flood Damage Evaluation Project, Final Report, Phase I, Volume I and Volume II,” prepared for the Lower Colorado River Authority and Fort Worth District Corps of Engineers, Halff Associates, Inc., July 2002.

2.4S.2-8 “USGS 08162506 Colorado River Bypass Channel Near Matagorda, TX,” United States Geological Survey, National Water Information System. Available at http://waterdata.usgs.gov/tx/nwis/nwisman/?site_no=08162506&agency_cd=USGS, accessed June 3, 2007.

2.4S.2-9 “Determining Design Basis Flooding at Power Reactor Sites,” ANSI/ANS-2.8-1992, Historical Technical Reference, American Nuclear Society, July 1992.

2.4S.2-10 “U.S. Department of Commerce National Oceanic and Atmospheric Administration (NOAA), Probable Maximum Precipitation Estimates, United States East of the 105th Meridian,” Hydrometeorological Report No. 51, NOAA, June 1978.

2.4S.2-11 "U.S. Department of Commerce National Oceanic and Atmospheric Administration, Application of Probable Maximum Precipitation Estimates - United States East of the 105th Meridian," Hydrometeorological Report No. 52, NOAA, August 1982.

2.4S.2-12 "HEC-HMS, Hydrologic Modeling System, Version 3.0.1," U.S. Army Corps of Engineers, Hydrologic Engineering Center, April 2006.

2.4S.2-13 "Soil Survey of Matagorda County, Texas," U.S. Department of Agriculture, Natural Resources Conservation Service (In Cooperation with Texas Agricultural Experiment Station), 2001.

2.4S.2-14 "Urban Hydrology for Small Watersheds, Technical Release 55," U.S. Department of Agriculture, Soil Conservation Service (now known as Natural Resources Conservation Service), June 1986.

2.4S.2-15 "EM 1110-2-1417 Flood-Runoff Analysis," U.S. Army Corps of Engineers, August 1994.

2.4S.2-16 "HEC-HMS, Hydrologic Modeling System, Technical Reference Manual," U.S. Army Corps of Engineers, Hydrologic Engineering Center, March 2000.

2.4S.2-17 "HEC-RAS, River Analysis System, Version 3.1.3", U.S. Army Corps of Engineers, Hydrologic Engineering Center, May 2005.

2.4S.2-18 "Open-Channel Hydraulics," Chow, Ven Te, 1959.

Table 2.4S.2-1 Peak Streamflow in the Lower Colorado River at Bay City (USGS Gauge No. 08162500)

Water Year	Date	Gauge Height [1] (ft)	Streamflow [2] (cfs)
1940	July 4, 1940	46.6	83,300
1948	May 28, 1948	No data	6,390
1949	April 28, 1949	34	36,000
1950	June 4, 1950	30.51	24,800
1951	June 7, 1951	25.75	12,000
1952	May 29, 1952	29.07	20,100
1953	May 1, 1953	30	23,300
1954	December 5, 1953	24.83	10,000
1955	May 21, 1955	25.74	11,900
1956	October 10, 1955	20.95	4,460
1957	May 1, 1957	41.83	53,000
1958	October 17, 1957	42.77	59,200
1959	April 13, 1959	34.48	34,200
1960	June 26, 1960	46.4	84,100
1961	September 15, 1961	44.09	66,400
1962	November 15, 1961	29.8	21,000
1963	February 22, 1963	22.69	8,580
1964	September 19, 1964	21.96	7,800
1965	May 20, 1965	31.05	27,000
1966	December 6, 1965	25.64	15,200
1967	September 23, 1967	27.5	19,000
1968	June 26, 1968	37.49	49,500
1969	February 23, 1969	27.92	24,200
1970	May 19, 1970	26.34	21,900
1971	October 25, 1970	24.76	19,400
1972	May 13, 1972	27.34	24,600
1973	June 15, 1973	38.7	60,800
1974	September 16, 1974	32.14	38,400
1975	May 28, 1975	34.45	48,900
1976	April 22, 1976	23.47	19,900
1977	April 24, 1977	34.2	50,300
1978	September 15, 1978	22.96	19,700
1979	June 9, 1979	29.9	40,400
1980	May 19, 1980	19.2	14,300
1981	June 18, 1981	30.95	42,100

Table 2.4S.2-1 Peak Streamflow in the Lower Colorado River at Bay City (USGS Gauge No. 08162500) (Continued)

1982	November 3, 1981	32.48	46,400
1983	May 23, 1983	22.9	22,600
1984	October 17, 1983	16.02	10,700
1985	October 25, 1984	24.6	24,500
1986	November 28, 1985	24.23	23,600
1987	June 17, 1987	34.32	50,500
1988	March 19, 1988	18.24	12,200
1989	May 16, 1989	14.6	7,740
1990	February 22, 1990	13.74	6,720
1991	April 16, 1991	24.04	23,200
1992	December 27, 1991	38.9	69,600
1993	June 22, 1993	30.28	38,500
1994	May 17, 1994	18.69	12,000
1995	October 20, 1994	38.67	71,100
1996	June 26, 1996	16.76	12,200
1997	March 19, 1997	29.29	37,900
1998	October 15, 1997	26.09	33,000
1999	October 24, 1998	40.95	81,800
2000	June 12, 2000	12.98	7,380
2001	September 1, 2001	22.36	22,800
2002	July 16, 2002	27.05	33,000
2003	November 8, 2002	32.56	48,300
2004	June 26, 2004	24.38	25,300
2005	November 27, 2004	41.73	73,800
2006	July 26, 2006	12.53	6,930

Source: Reference 2.4S.2-3

[1] Gauge height is measured from the National Geodetic Vertical Datum of 1929 (NGVD 29).

[2] Peak streamflow data for 1940 water year is indicated as a historical peak in Reference 2.4S.2-3. Peak streamflow data for all other water years may have been affected by regulation or diversion.

Table 2.4S.2-2 Peak Streamflow in the Lower Colorado River at Wharton (USGS Gauge No. 08162000)

Water Year	Date	Gauge Height [1] (ft)	Streamflow [2] (cfs)
1919	June 18, 1919	No data	37,600
1920	October 15, 1919	No data	39,600
1921	September 14, 1921	No data	35,900
1923	May 3, 1923	No data	29,300
1924	November 5, 1923	No data	32,800
1925	May 15, 1925	No data	22,100
1935	June 20, 1935	38.2	159,000
1938	July 30, 1938	37.4	145,000
1939	July 18, 1939	11.48	12,600
1940	July 3, 1940	35.99	100,000
1941	November 26, 1940	35.3	92,000
1942	April 9, 1942	22.35	38,900
1943	October 21, 1942	9.3	8,330
1944	March 17, 1944	14.96	19,700
1945	April 2, 1945	19.8	36,400
1946	March 14, 1946	19.5	35,600
1947	November 6, 1946	16.98	27,300
1948	May 27, 1948	8.4	7,800
1949	April 27, 1949	20.9	37,900
1950	June 4, 1950	17.55	28,600
1951	June 6, 1951	11.15	13,200
1952	May 29, 1952	12.95	17,400
1953	May 1, 1953	17.9	29,900
1954	December 5, 1953	11.2	13,300
1955	May 20, 1955	10.4	10,100
1956	October 7, 1955	5.7	4,610
1957	April 30, 1957	28.9	54,200
1958	October 17, 1957	30	58,500
1959	April 13, 1959	20.6	33,300
1960	June 27, 1960	27.5	53,000
1961	September 15, 1961	30.9	59,600

Table 2.4S.2-2 Peak Streamflow in the Lower Colorado River at Wharton (USGS Gauge No. 08162000) (Continued)

1962	November 14, 1961	15.7	21,600
1963	February 21, 1963	8.6	9,680
1964	September 19, 1964	7.3	7,990
1965	May 19, 1965	20.8	33,300
1966	May 3, 1966	12.72	16,300
1967	September 23, 1967	12.43	15,700
1968	June 26, 1968	28.06	55,400
1969	February 22, 1969	17.45	25,500
1970	May 18, 1970	17	24,600
1971	October 24, 1970	15.72	22,400
1972	May 13, 1972	16.96	24,500
1973	June 15, 1973	32.25	59,400
1974	September 15, 1974	25.9	40,800
1975	May 27, 1975	28.5	50,800
1976	April 21, 1976	18.14	21,000
1977	April 23, 1977	32.35	53,000
1978	September 14, 1978	17.7	20,300
1979	June 8, 1979	28.55	43,400
1980	May 16, 1980	13.05	13,500
1981	June 17, 1981	28.5	43,300
1982	November 3, 1981	30.55	49,300
1983	May 23, 1983	28.85	23,700
1984	May 20, 1984	16.19	5,140
1985	February 25, 1985	23.5	14,900
1986	November 27, 1985	29.94	25,400
1987	June 17, 1987	41.48	51,600
1988	March 19, 1988	22.42	12,900
1989	May 20, 1989	18.44	7,410
1990	May 6, 1990	15.77	4,470
1991	January 12, 1991	31.63	28,800
1992	December 27, 1991	45.31	61,900
1993	June 21, 1993	33.06	31,800

Table 2.4S.2-2 Peak Streamflow in the Lower Colorado River at Wharton (USGS Gauge No. 08162000) (Continued)

1994	May 16, 1994	22.62	13,800
1995	October 20, 1994	40.69	49,600
1996	September 23, 1996	20.47	11,900
1997	March 18, 1997	31.29	30,600
1998	October 15, 1997	28.85	25,200
1999	October 23, 1998	48.72	74,800
2000	June 12, 2000	18.71	8,650
2001	March 16, 2001	23.06	14,500
2002	November 19, 2001	31.05	30,100
2003	November 8, 2002	39.16	46,000
2004	June 13, 2004	29.26	25,600
2005	November 26, 2004	48.32	73,200
2006	May 10, 2006	14.49	3,820

Source: Reference 2.4S.2-4

[1] Gauge height is measured from the NGVD 29.

[2] Peak streamflow data for 1935 water year is indicated as a historical peak in Reference 2.4S.2-4. Peak streamflow data from 1938 to 2006 water years may have been affected by regulation or diversion.

Table 2.4S.2-3 Recorded Highest Flood Elevations at Bay City

Rank	Gauge Height (ft NGVD 29 or MSL)	Date
1	56.10	12/10/1913
2	55.40	5/8/1922
3	55.00	6/1/1929
4	54.60	6/22/1935
5	53.40	8/2/1938
6	52.20	10/5/1936
7	47.60	11/27/1940
8	46.60	7/4/1940
9	46.40	6/26/1960
10	38.67	10/20/1994

Source: References 2.4S.2-3, 2.4S.2-5, and 2.4S.2-6

Table 2.4S.2-4 Short Duration PMP Depths at the STP 3 & 4 Site

PMP Duration & Area	6-hr, 10 mi^2 Ratio	1-hr, Point Location Ratio	Source	PMP Depth (in)
72 hr, 10 mi^2	-	-	HMR 51 - Fig. 22	55.7
48 hr, 10 mi^2	-	-	HMR 51 - Fig. 21	51.8
24 hr, 10 mi^2	-	-	HMR 51 - Fig. 20	47.1
12 hr, 10 mi^2	-	-	HMR 51 - Fig. 19	37.8
6 hr, 10 mi^2	-	-	HMR 51 - Fig. 18	32.0
3 hr	-	-	Fitted from Figure 2.4S.2-3	29.7
2 hr	-	-	Fitted from Figure 2.4S.2-3	26.6
1 hr, point location [1]	0.62	-	HMR 52 - Fig. 23	19.8
30 min, point	-	0.73	HMR 52 - Fig. 38	14.5
15 min, point	-	0.50	HMR 52 - Fig. 37	9.9
5 min, point [2]	-	0.32	HMR 52 - Fig. 36	6.4

Source: Reference 2.4S.2-10 and 2.4S.2-11

[1] The local PMP rainfall magnitude for 1 hr point location (1 mi^2) exceeded the maximum rainfall rate (1 hr, 1 mi^2) specified in the ABWR DCD (Tier 1 Table 5.0, and Tier 2 Table 2.0-1), which is further discussed in Table 2.0-2.

[2] The local PMP rainfall magnitude for 5 min point location (1 mi^2) exceeded the maximum short term rainfall rate (5 min, 1 mi^2) specified in the ABWR DCD (Tier 1 Table 5.0, and Tier 2 Table 2.0-), which is further discussed in Table 2.0-2.

Table 2.4S.2-5 Subbasin Drainage Areas

Sub-Basin	Drainage Area (mi^2)
North 1	1.466
North 2	0.298
North 3	0.177
PBN1	0.319
PBW1	0.049
PBW	0.135
PBE	0.089
Total	**2.533**

Table 2.4S.2-6 PMP Peak Discharges in STP 3 & 4 Subbasins and Drainage Elements

Hydrologic Element	Drainage Area (mi^2)	Peak Discharge (cfs)	Time of Peak	Runoff Volume (in)
LRS	1.764	7686.9	26Jul2007, 06:35	32.04
MDC2	0.089	1428.7	26Jul2007, 03:30	31.68
MDC3	0.224	3588.4	26Jul2007, 03:35	31.68
MDC4	0.273	3937.5	26Jul2007, 03:35	31.68
North 1	1.466	7971.5	26Jul2007, 05:30	31.68
North 2	0.298	1773.1	26Jul2007, 05:15	31.68
North 3	0.177	1457.3	26Jul2007, 04:25	31.68
OutFlow	2.533	9852.0	26Jul2007, 03:35	31.93
PBE	0.089	1443.3	26Jul2007, 03:25	31.68
PBN1	0.319	4243.8	26Jul2007, 03:35	31.68
PBW	0.135	2304.4	26Jul2007, 03:25	31.68
PBW1	0.049	1367.7	26Jul2007, 03:10	31.68
US LRS	1.764	7690.3	26Jul2007, 06:25	31.96
US MDC2	0.089	1443.3	26Jul2007, 03:25	31.68
US MDC3	0.224	3635.2	26Jul2007, 03:25	31.68
US MDC4	0.273	3976.3	26Jul2007, 03:30	31.68

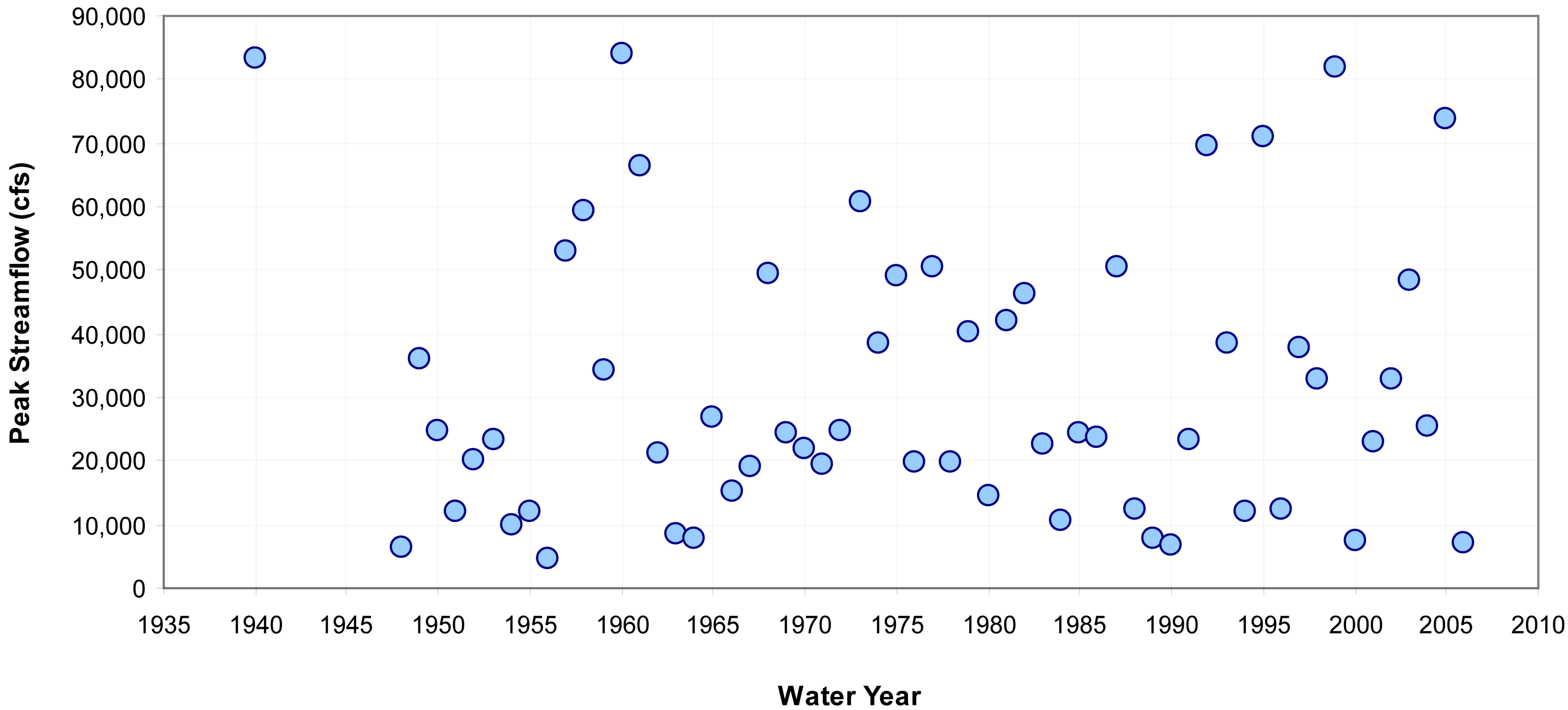

Figure 2.4S.2-1 Peak Streamflow on the Lower Colorado River near Bay City, Texas (USGS gauge number 08162500)

Source: Reference 2.4S.2-2

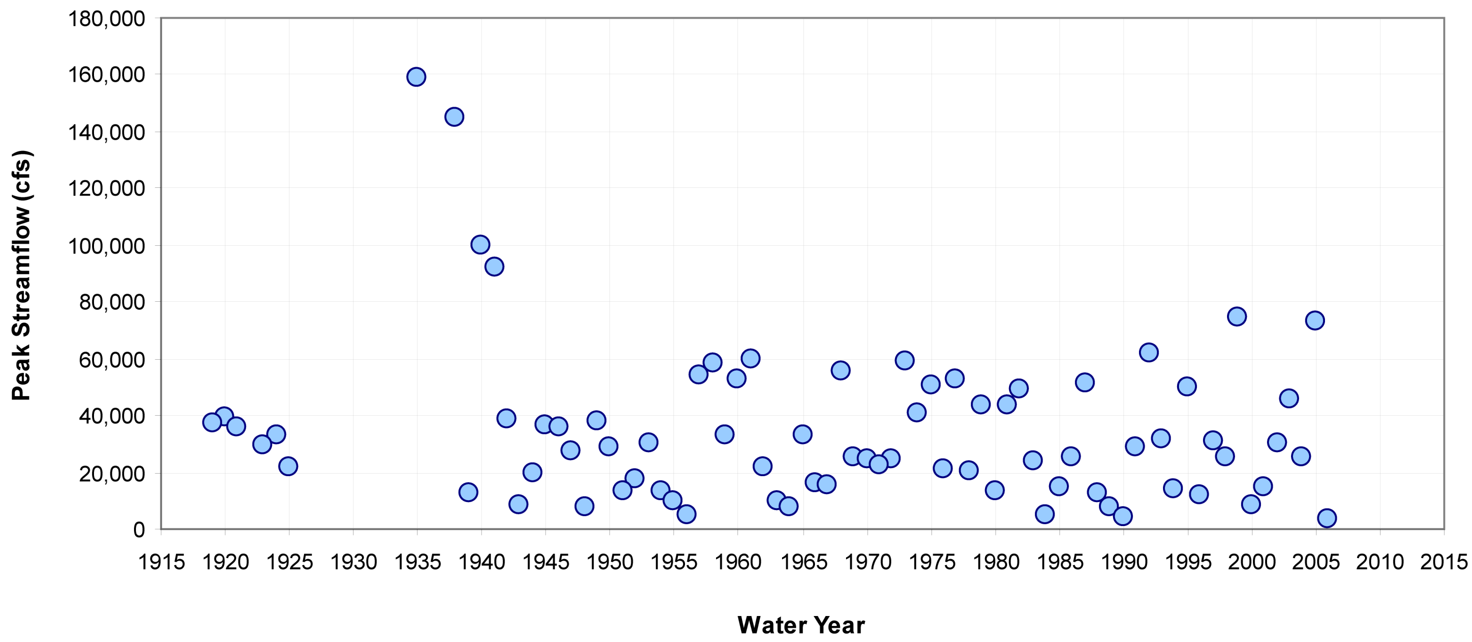

Figure 2.4S.2-2 Peak Streamflow on the Lower Colorado River at Wharton, Texas (USGS gauge number 08162000)

Source: Reference 2.4S.2-3

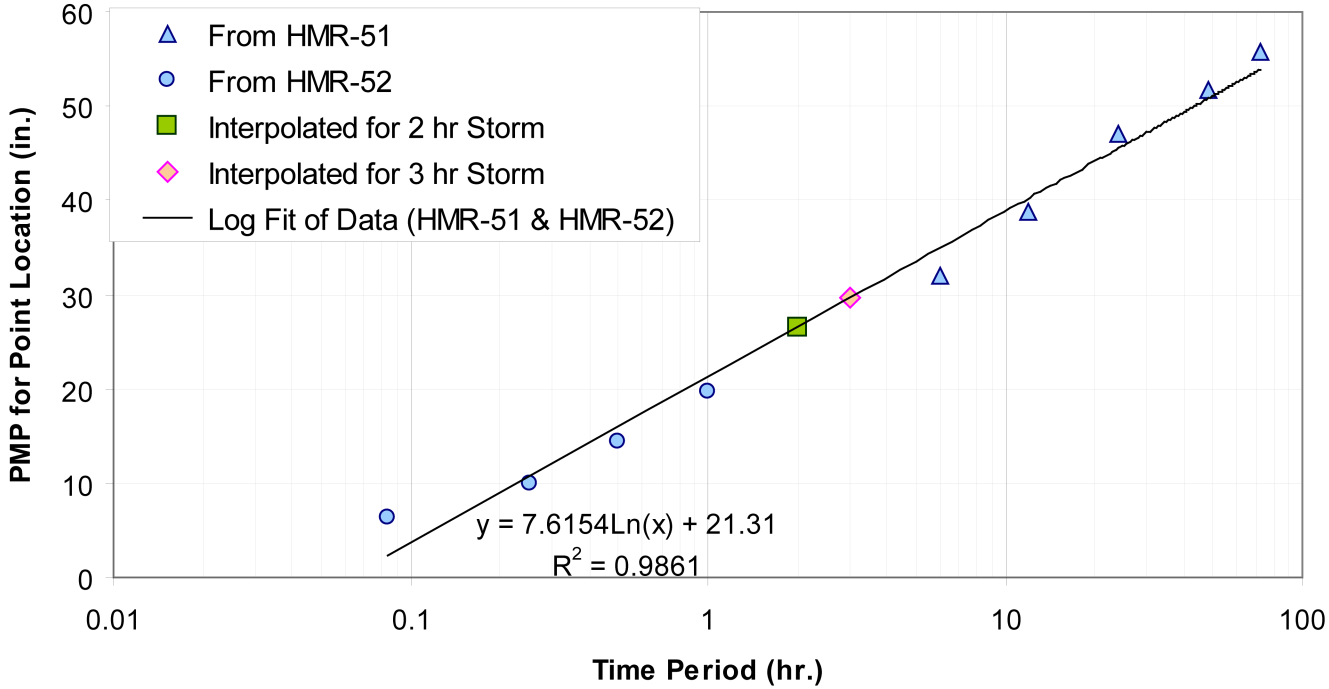

Figure 2.4S.2-3 Determination of 2- and 3-hr PMP Depths at the STP 3 & 4 Site

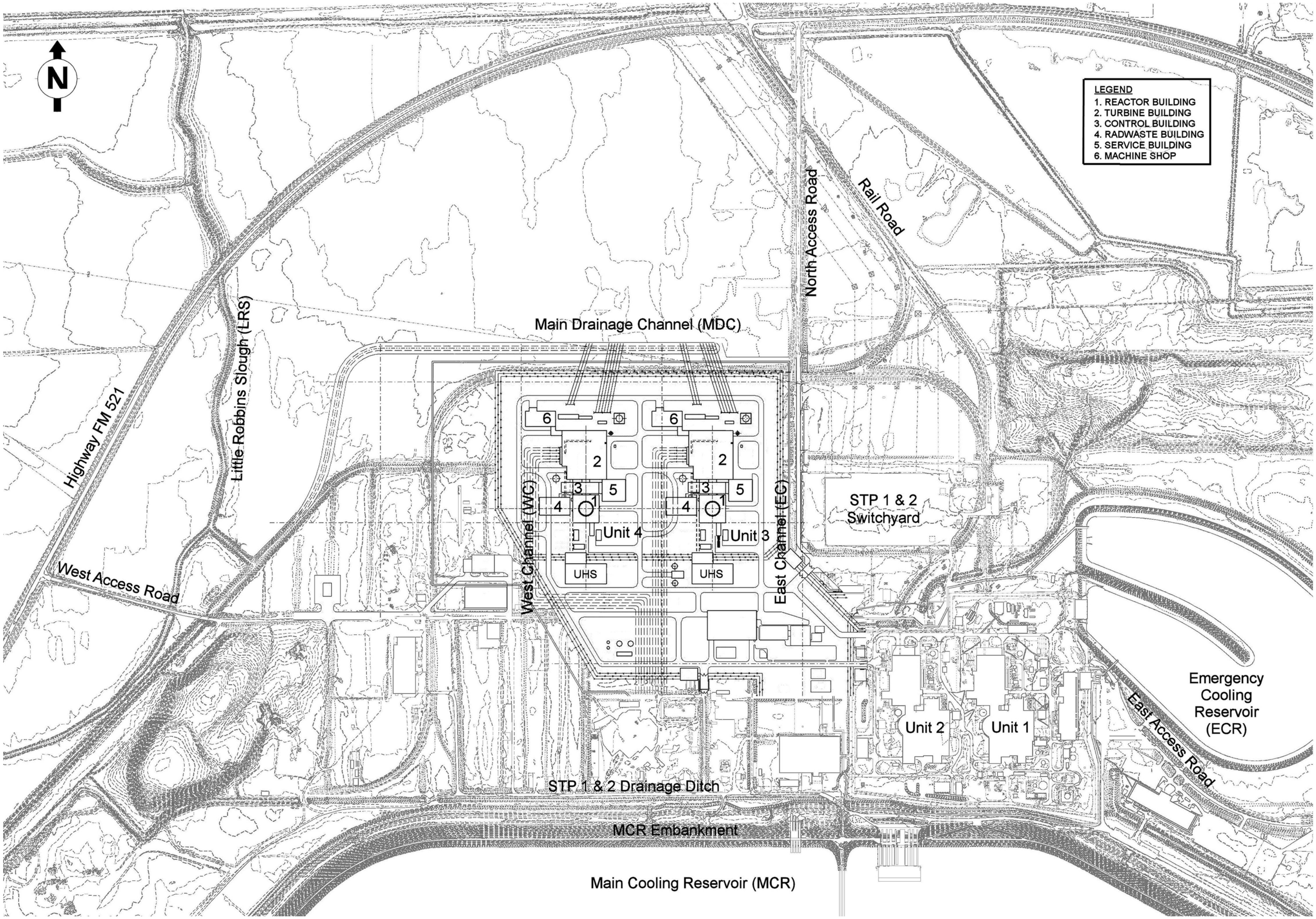

Figure 2.4S.2-4 Site Layout and Major Drainage Routes

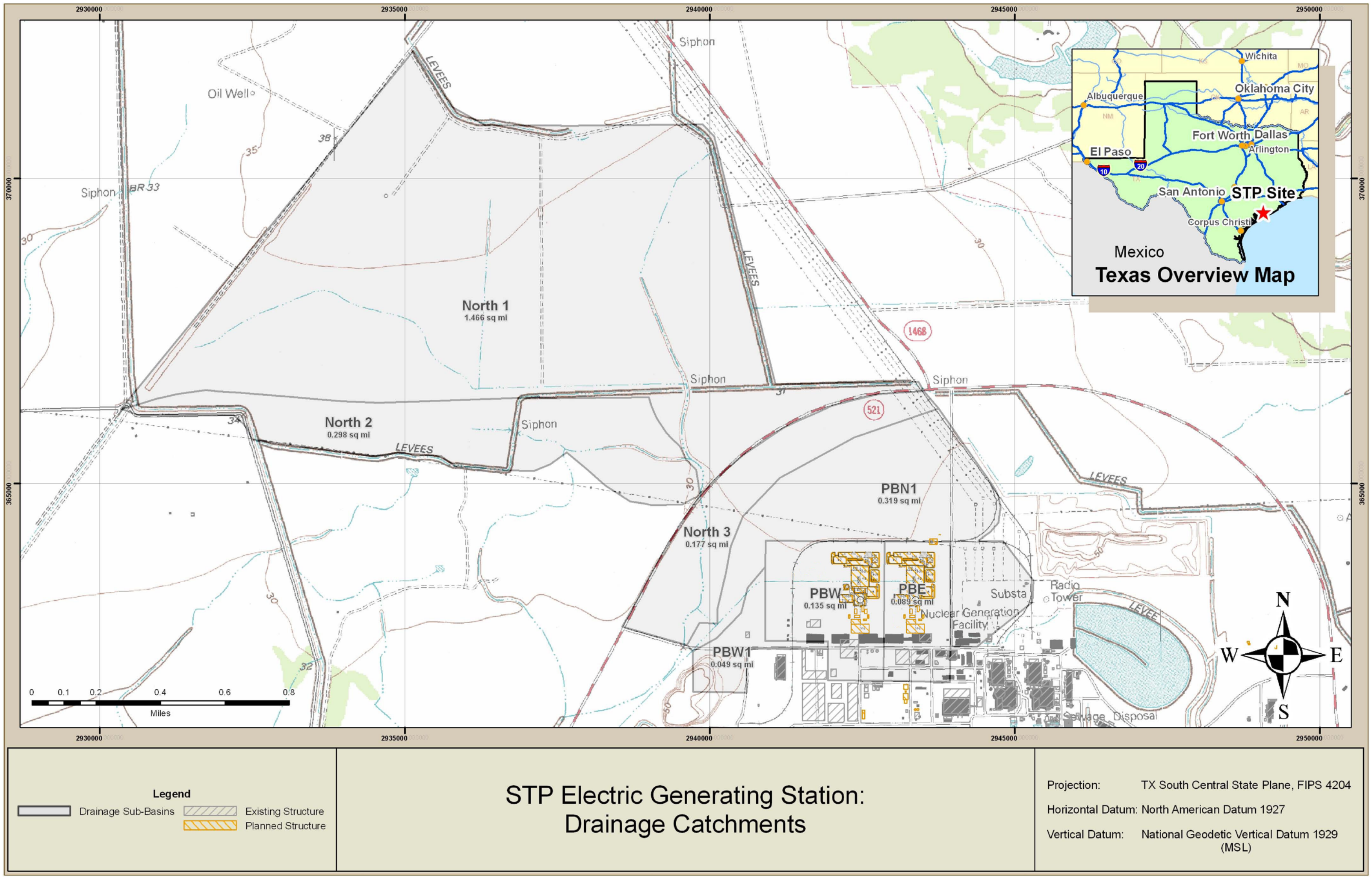

Figure 2.4S.2-5 Components of the Drainage System and Subbasin Areas. The vertical datum used is NGVD 1929

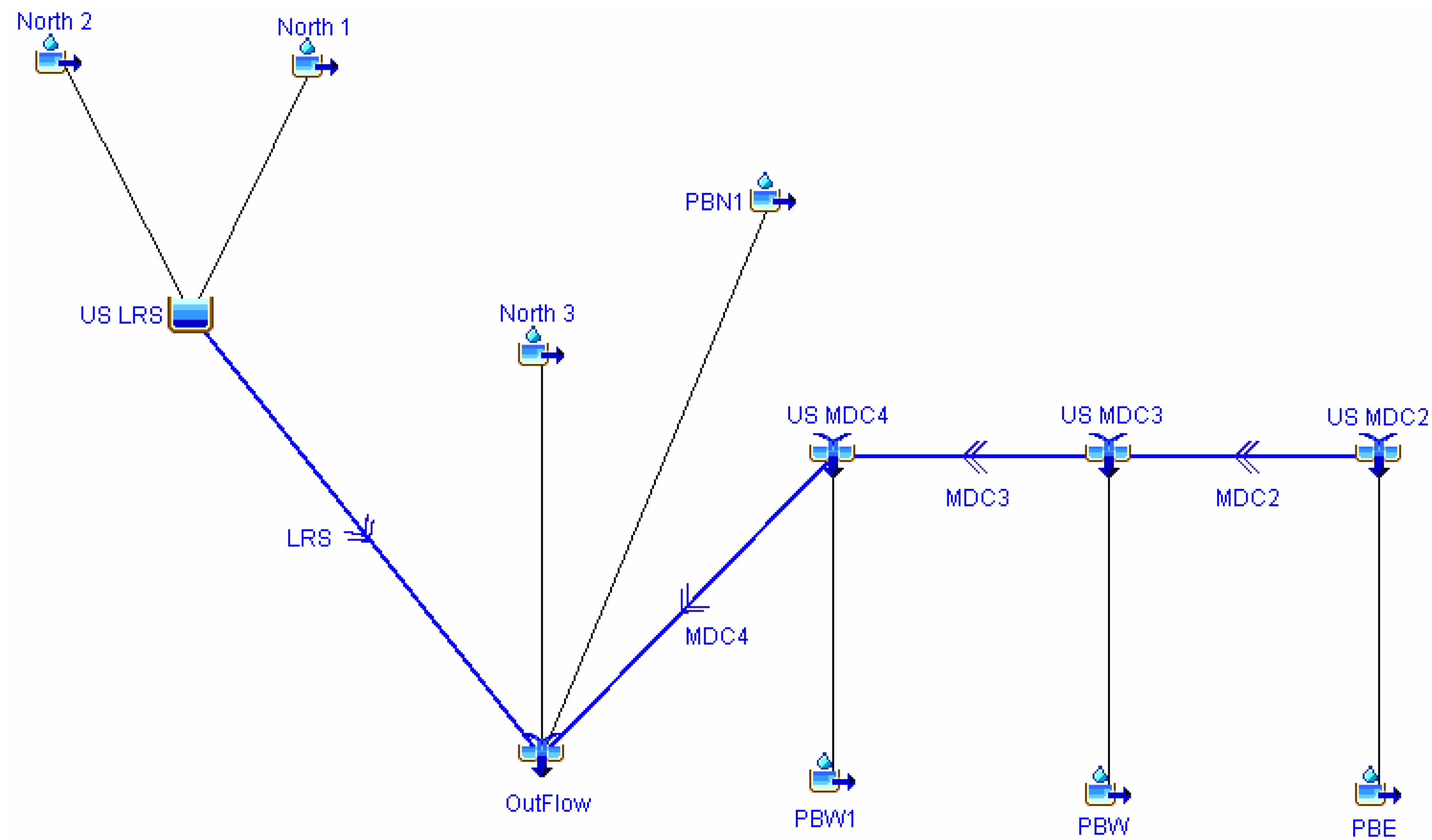

Figure 2.4S.2-6 HEC-HMS Model Hydrologic Diagram of the STP 3 & 4 Drainage System

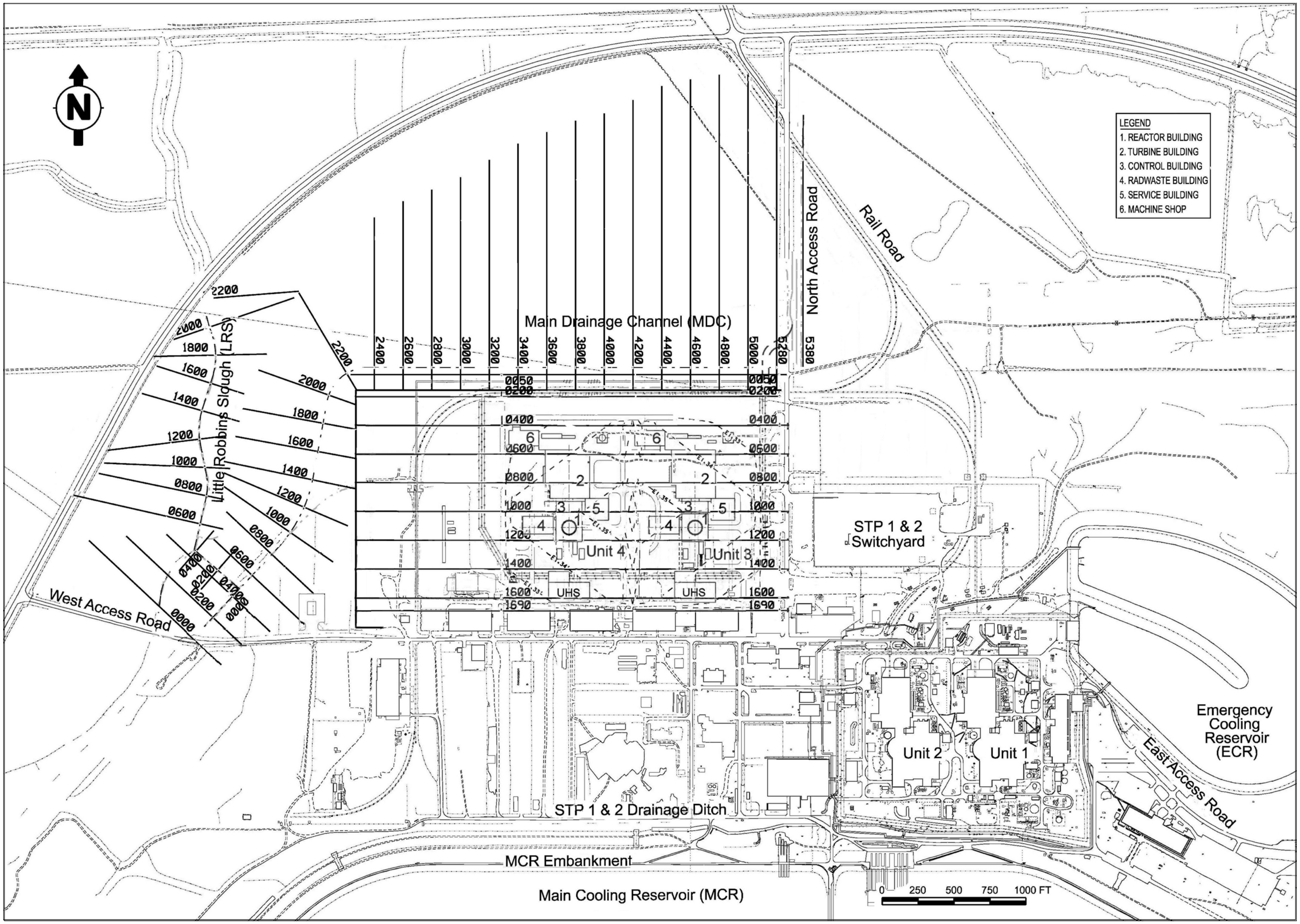

Figure 2.4S.2-7 Extents and Locations of Channel Crosssections