# COAL COMBUSTION RESIDUALS
# SURFACE IMPOUNDMENT
# INFLOW DESIGN FLOOD CONTROL SYSTEM PLAN
# (REV. 1)

## COLETO CREEK POWER STATION
**FANNIN, TEXAS**

**JANUARY 24, 2018**
**(ORIGINAL VERSION: OCTOBER 13, 2016)**

*Prepared for:*

**COLETO CREEK POWER, LP**
Coleto Creek Power Station
Fannin, Texas

*Prepared by:*

**BULLOCK, BENNETT & ASSOCIATES, LLC**
Engineering and Geoscience
Registrations:  Engineering F-8542, Geoscience 50127

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BBA Project No. 17266

**Certification Statement 40 *CFR* § 257.82 — Inflow Design Control System Plan for a CCR Surface Impoundment**

**CCR Unit: Coleto Creek Power, LP; Coleto Creek Power Station; Coleto Creek Primary Ash Pond**

I, Daniel Bullock, being a Registered Professional Engineer in good standing in the State of Texas, do hereby certify, to the best of my knowledge, information, and belief that the information contained in this plan has been prepared in accordance with the accepted practice of engineering. I certify, for the above referenced CCR Unit, that the information contained in the Inflow Design Control System Plan, dated January 24, 2018, meets the requirements of 40 *CFR* § 257.82.

Daniel B. Bullock

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1/24/2018

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## 1.0 SITE SUMMARY

Coleto Creek Power, LP operates the Coleto Creek Power Station located at 45 FM 2987 near the city of Fannin in Goliad County, Texas (Figure 1). One boiler is operated at the facility to generate electricity for distribution to the area power grid. The boiler uses coal as the primary fuel and fuel oil as a backup fuel. There are two streams of coal combustion residuals (CCR) generated at this plant. Bottom ash is collected from the boiler, combined with water, and transferred in slurry form for disposal in the facility's surface impoundment (Primary Ash Pond). Fly ash is collected from the boiler exhaust and transported pneumatically to two storage silos. From there, the fly ash is loaded into enclosed dry haul hoppers for off-site beneficial use. Fly ash not meeting required beneficial reuse specifications is combined with water and pumped to the Coleto Creek Primary Ash Pond for disposal. Bottom ash in the Primary Ash Pond is routinely recovered for beneficial reuse via excavation, screening, and placement in covered dump trucks for transport off site.

The CCR slurry is pumped directly to the 190-acre Primary Ash Pond where the majority of solids settle out of the carrier water leaving only deminimis amounts. The treated water can then flow into a 10-acre Secondary Pond. The facility's Texas Pollutant Discharge Elimination System (TPDES) Permit No. WQ0002159000 allows for the discharge of up to 0.64 million gallons per day (gpd) of water from the Secondary Pond to the adjacent Coleto Creek Reservoir. Because the Primary Ash Pond and Secondary Pond are hydraulically connected (a levee failure of the Secondary Pond and the associated rapid dewatering could impact the stability of the Primary Ash Pond), both ponds are considered in this assessment even though the Secondary Pond is not regulated under the CCR Rule.

Pursuant to Rule 40 *CFR* §257.82(a), "the owner or operator of an existing or new CCR surface impoundment…must design, construct, operate, and maintain an inflow design flood control system." 40 *CFR* §257.82(c) requires the owner or operator of the existing CCR surface impoundments to "…prepare initial and periodic inflow design flood control system plans for the CCR unit." This *Inflow Design Flood Control System Plan*[1] has been prepared to meet the requirements of the rule. This plan should be amended at any time that CCR management

[1]This *Inflow Design Flood Control System Plan* replaces the initial *Inflow Design Flood Control System Plan* dated October 13, 2016.

operations substantially change. In addition, this plan will be updated every five years in accordance with §257.82(c)(4). A copy of this Plan will be maintained in the facility's operating record and publicly accessible internet site.

## 2.0 HYDRAULIC ANALYSIS

According to §257.82(a)(1) and (2), the inflow design flood control system must adequately manage flow into the CCR unit during and following the peak discharge of the inflow design flood as defined by the rule. In addition, the inflow design flood control system must adequately manage flow from the CCR unit to collect and control the peak discharge resulting from the inflow design flood. As noted in the *Coleto Creek Power Station Structural Integrity Report* (BBA, December 2017), the Primary Ash Pond is classified as having a Low Hazard Potential. The inflow design flood, therefore, is defined in §257.82(a)(3)(iii) as the 24-hour, 100-year flood.

The Coleto Creek Primary Ash Pond and Secondary Pond are currently operated as a relatively closed system. The ponds are completely surrounded by dikes that range from approximately four (4) to 39 ft above grade for the Primary Ash Pond and up to 56 ft for the Secondary Pond (Sargent & Lundy Engineers, 1978). The only sources of storm water accumulation, therefore, are the rain that falls within the surface impoundment boundary and incidental runoff from the dike crest. No other facility storm water is reportedly pumped into the ponds. Water from the ponds can be siphoned from the Secondary Pond at a maximum rate of approximately 0.64 million gpd and discharged to the adjacent “hot side” of the Coleto Creek Reservoir. Water levels in the pond are currently maintained below approximate elevation 136 ft NAVD88.

Bullock, Bennett and Associates, LLC (BBA) contracted Naismith Marine Services (Naismith) of Corpus Christi, Texas to complete a land and bathymetric site survey in July 2016 for the purpose of evaluating current conditions at the ponds and to obtain approximate as-built dike cross sections in areas of interest. Naismith surveyed the local plant vertical control datum and determined plant control to be equivalent to North American Vertical Datum (NAVD88) of 1988 although plant control refers to mean sea level (MSL). Figure 2 provides the results of the July 2016 survey. Based on the 2016 survey the crest height generally appears to be constructed to elevation 140 ft NAVD88, however, areas were identified to be as low as approximate elevation 139.7 ft. This lower elevation is used to evaluate available capacity in the ponds.

The staff gauge elevation was also measured during the 2016 site topography and bathymetry survey. The survey found that the staff gauge mark of 140.0 corresponds to an elevation of approximately 140.4 ft NAVD88.

Because no significant inflow of outside storm water occurs and no conventional spillway is present, the surface impoundment must be operated so that it can contain the entirety of the design storm as well as the inflow of water/CCR from normal plant operations that occurs during the same period. The Primary Ash Pond is currently partially full of CCR, and water storage capacity remains primarily in the north portion of the pond, between approximate elevations 106 ft and 139.7 ft NAVD88 (the lowest dike crest elevation recorded in the recent survey). The available remaining liquid capacity of the Primary Ash Pond and Secondary Pond based on 2016 survey data is presented below for elevations between 135.0 ft and 139.7 ft NAVD88.

**TABLE 1**
**ESTIMATED AVAILABLE LIQUID CAPACITY**
**(for elevations between 135 ft and 139.7 ft)**

| Elevation (ft NAVD88) | Primary Ash Pond Available Capacity (acre-ft) | Secondary Pond Available Capacity (acre-ft) | Cumulative Available Capacity (acre-ft) |
|---|---|---|---|
| 135-136 | 87.61 | 8.26 | 95.87 |
| 136-137 | 181.48 | 16.70 | 198.18 |
| 137-138 | 284.90 | 25.32 | 310.22 |
| 138-139 | 402.71 | 28.32 | 431.03 |
| 139-139.7 | 538.19 | 44.20 | 582.39 |

*Available capacity estimates were calculated using the July 2016 topographic and bathymetric survey data, and AutoCAD volume surface models.

Maximum precipitation values for a 100-year, 24-hour storm were evaluated from various data sources. The most applicable and appropriate value was obtained from *San Antonio River Basin Regional Modeling Standards for Hydrology and Hydraulic Modeling,* Table 7: USGS Adjusted IDF Rainfall Values for Goliad County (SARA, September 2013). Maximum precipitation values presented in this publication were based on the USGS publication “Atlas of Depth-Duration Frequency of Precipitation Annual Maxima for Texas” (Asquith, June 2004). The total rainfall for the 100-year design storm is listed as 11.40 inches in a 24-hour period.

Multiplying the entire surface area of the ponds (200 acres) plus one-half of the total dike crest area (approximately 5 acres) by the total rainfall of the design storm gives a total accumulated storm water volume of approximately 195 acre-ft. Adding the flow of CCR plus sluice water from normal plant operations (2.6 million gpd, or approximately 8 acre-ft/day), the total liquid volume that must be safely contained is approximately 203 acre-ft. For the purposes of this evaluation it was assumed that no water was discharged to Coleto Creek Reservoir. Based on the wind and wave run-up estimates (Section 3.0), 1.7 ft of freeboard should be available above the elevation of containment of the design storm rainfall event. Therefore, since the low point of the perimeter dike is approximately elevation 139.7 ft, the rain event should be contained within or below elevation 138.0 ft (maximum surcharge pool elevation). And, based on Table 1 data, the design storm event can be contained within the interval between elevations 136.1 ft and 138.0 ft, therefore, the maximum storage pool elevation should be maintained at or below elevation 136.1 ft. The maximum storage pool elevation of 136.1 ft NAVD88 equates to a staff gauge reading of 135.7 ft.

Given these elevations are related back to a low point identified on the dike system, it’s possible that isolated low points could be brought to surrounding dike design grades and thus potentially increase the acceptable maximum storage elevations slightly if needed in the future.

# 3.0 WIND AND WAVE RUN-UP ANALYSIS

Wind and wave run-up effects were estimated using guidance contained in the document *Freeboard Criteria and Guidelines for Computing Freeboard Allowances for Storage Dams* (USBR, 1981). Equation 3 of USBR was used to calculate wave run-up as follows:

$$R_s = \frac{H_s}{0.4 + (H_s/L)^{0.5} \cot \Theta}$$

where:
$R_s$ = wave run-up
$H_s$ = significant wave height, 1.8 ft
L = deep water wave length, 27.08 ft
$\Theta$ = angle of upstream face of the dam with the horizon, 18 deg

$H_s$ was calculated using Figure 9 in the USBR guidelines. Figure 9 determines significant wave height from the effective fetch (Fe) and the design wave velocity. Effective fetch is estimated to be ½ of wave fetch (F). F was determined to be the longest over water tangent normal to the dam and was measured at 3,818 ft (.72 mi) which leaves Fe at 1,909 ft (0.36 miles). Design wind velocity was determined from Figure 3 of the USBR guidelines, Fastest Mile of Record-Summer. This measurement was used because it yielded the highest velocity and therefore the most conservative measurement. Wind velocity was determined to be 63 mph. After applying the wind velocity ratio (wind over water) from Table 2 of 1.08 for a Fe of 0.5 miles (rounded up), the design wind velocity was determined to be 68 mph.

L was calculated using the Equation 2, $L = 5.12T^2$, with T being wave period. T was found with Figure 10 of USBR to be 2.3 seconds. When applied to the equation, L is determined to be 27.08 ft. $\Theta$ is 18 degrees as the dam has a side slope of approximately 3 horizontal to 1 vertical.

When all variables are applied to equation 3 of the USBR guidelines, the wave run-up is calculated to be 1.5 ft.

The wind setup in feet is calculated using Equation 4 of the USBR guidelines as follows:

$$S = \frac{U^2F}{1400D}$$

where:
U = design wind velocity over water in miles per hour, 68 mph
F = wind fetch in miles, 0.72 miles
D = average water depth along the central radial in feet, conservatively estimated to equal 10 ft

The wind setup is calculated to equal 0.2 ft.

The required freeboard is the wave run-up plus the wind setup. The total required freeboard, therefore, is 1.7 ft.

## 4.0 CHANNEL FLOW EVALUATION

As noted previously, CCR has built up in the Primary Ash Pond such that approximately half of the impoundment is "dry". There are two channels that are maintained in the deposited CCR through which the sluiced ash is conveyed (Figure 3A). The East Channel conveys bottom ash sluice water and the West Channel conveys fly ash sluice water. Because these channels are adjacent to the Primary Ash Pond dikes and are also constrained on the interior bank by CCR that is mounded above the maximum dike height of 140 ft NAVD88, it is necessary to evaluate whether they can manage the peak flow from the 100-year, 24-hour design storm without overtopping the dikes.

The US Army Corps of Engineers (USACE) Hydraulic Engineering Center's (HEC) River Analysis System software (HEC-RAS ver. 5.0.1) was used to evaluate water flow through each of the channels. Naismith collected elevation survey data for three cross sections across each of the two channels at the locations shown on Figure 3A. This information was then input into the HEC-RAS model. The channels are excavated from the CCR material and have some grass and weeds present along the base and on the sides. The channels were thus modeled as earthen channels, with a maximum Manning coefficient of 0.033.

In order to estimate the peak water flow through each channel, two methods were used. First, the maximum rainfall contribution was manually estimated by reviewing the 100-yr rainfall amounts for the duration periods ranging from 5 minutes up to 24 hours as reported in Table 7 of the *San Antonio River Basin Regional Modeling Standards for Hydrology and Hydraulic Modeling* document (SARA, September 2013) and then calculating the total contribution of rainfall to each segment of the channel area defined by the corresponding cross section. It was then conservatively assumed that the entire quantity of water from the period with the greatest flow rate in cubic feet per second would immediately enter the channel with no lag time. It was also assumed that the channel was operating normally with either fly ash (West Channel) or bottom ash (East Channel) sluice water present. Modeling input parameters are summarized in Appendix A.

As an alternative, the USACE HEC Hydraulic Modeling System software (HEC-HMS ver. 4.2) was used to estimate the peak flow rate. This program accepts rainfall estimates for periods greater than 1 hour and estimates the rate for shorter durations. This program also takes into account lag times and other site specific parameters that can affect flow over larger land areas. The peak flow rates estimated using HEC-HMS were approximately half those estimated using the manual method. Because the manual method results in higher flow rates, and therefore is more conservative, they were used as the basis for input into the HEC-RAS flow model.

The results of the HEC-RAS modeling for each channel cross section are presented in Figure 3B. As shown in this figure, the estimated water levels in each of the channels at the selected cross sections do not exceed the elevation of the perimeter dike system. The channels as currently configured, therefore, appear to manage the anticipated peak flow from a 100-year design storm without overtopping. The channels should continue to be maintained to allow free flow of sluice water and storm water into the “wet” side of the Primary Ash Pond.

## 5.0 SUMMARY

The Coleto Creek Primary Ash Pond is considered an existing CCR surface impoundment that is regulated under 40 *CFR* Part 257 Subpart D – Standards for the Disposal of Coal Combustion Residuals in Landfills and Surface Impoundments. §257.82(c) requires that existing CCR surface impoundment prepare a written *Inflow Design Control System Plan* to ensure that the surface impoundment is operated such that inflows to and from the impoundment from a design storm are adequately controlled. Because the Primary Ash Pond has a Low Hazard classification, the design storm is the 100-year, 24-hour rain event.

Based on the estimated rainfall accumulation associated with the design storm event, and wind and wave run-up estimates, the maximum storage pool elevation should be set to 136.1 ft NAVD88 (staff gauge elevation of 135.7 ft), which would provide containment for the design storm and allow 1.7 ft of additional freeboard for wave action. Furthermore, the East and West channels should be maintained to allow the cumulative flow of ash sluice water and peak rainwater flow from the design 100-year storm into the "wet" side of the Primary Ash Pond.

The results of this analysis show that overtopping during the design storm would not occur due to the accumulation of storm water. Therefore, the Primary Ash Pond currently has sufficient hydraulic capacity to manage the design storm at the highest operating liquid level.

## 6.0 REFERENCES

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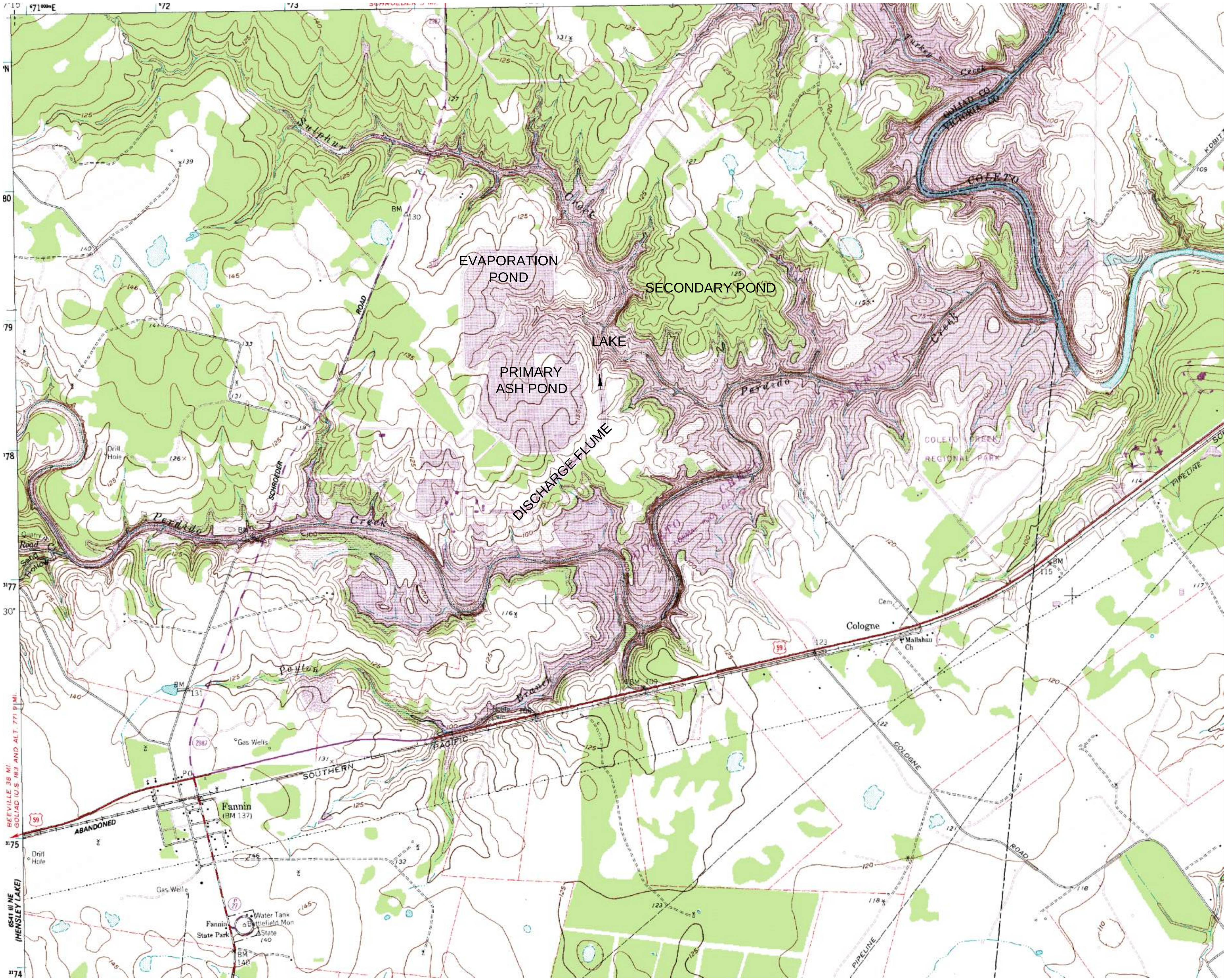
USBR. (1981). *Freeboard Criteria and Guidelines for Computing Feeboard Allowances for Storage Dams, ACER Technical Memorandum No. 2.* Assistant Commissioner-Engineering and Research, U.S. Department of the Interior, Bureau of Reclamation.

**FIGURES**




N

APPROXIMATE SCALE: 1" = 2500'

0 1250 2500 5000

NOTE: CONTOUR DATA SHOWN ON U.S.G.S. MAP IN AREAS OF ASH PONDS ARE REPRESENTATIVE OF CONDITIONS PRIOR TO ASH POND CONSTRUCTION.

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12-28-2017

SOURCE: U.S.G.S. FANNIN TEXAS, DATE: 2016, SCALE 1:24000, 10' CONTOURS. NAD1983, NAVD1988.

**Coleto Creek Power, LP**

Figure 1

U.S.G.S. AREA MAP

| PROJECT: 17266 | BY: RR | DATE: DEC 2017 | CHECKED: DBB |
|---|---|---|---|

Bullock, Bennett & Associates, LLC
Engineering and Geoscience
Texas Registrations: Engineering F-8542, Geoscience 50127

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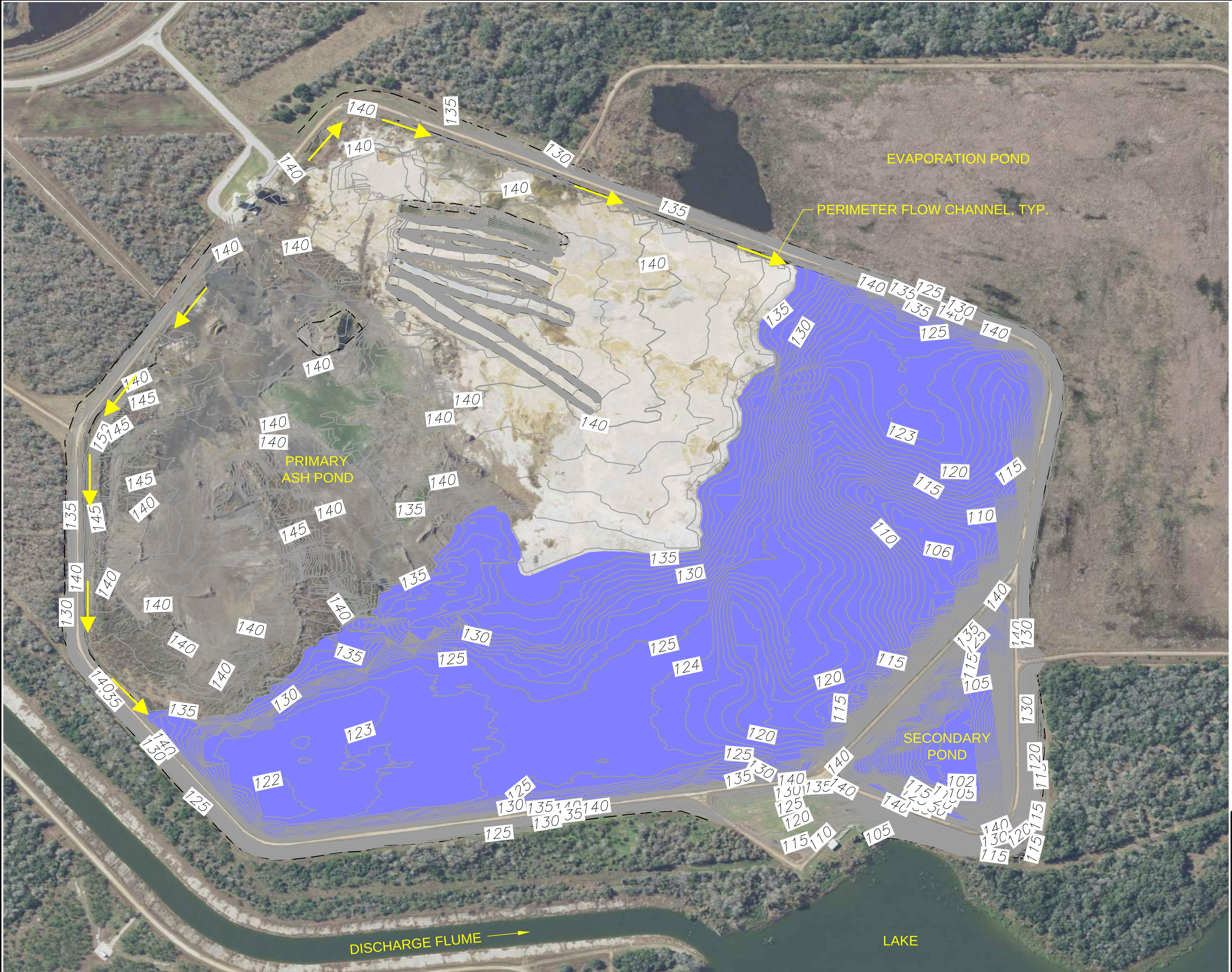


N

SCALE IN FEET

0 200 400 800

SOURCES:

ON-GROUND TOPOGRAPHIC AND BATHYMETRIC SURVEY PROVIDED BY NAISMITH MARINE SERVICES ON JULY 2016. HORIZONTAL DATUM: NAD83, TEXAS CENTRAL SOUTH ZONE, US FEET. VERTICAL DATUM: NAVD88.

AERIAL PHOTO PROVIDED BY IMAGEPATCH.COM EARTHSTAR GEOGRAPHICS, DATE: MAY-OCT 2011.

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**Coleto Creek Power, LP**

Figure 2

SITE TOPOGRAPHY MAP

| PROJECT: 15214-3 | BY: RR | DATE: DEC 2017 | CHECKED: DBB |
|---|---|---|---|

Bullock, Bennett & Associates, LLC
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PARTIAL PLAN

APPROXIMATE SCALE: 1" = 300'

0 150 300 600

SOURCES:

ON-GROUND TOPOGRAPHIC AND BATHYMETRIC SURVEY PROVIDED BY NAISMITH MARINE SERVICES ON JULY 2016. HORIZONTAL DATUM: NAD83, TEXAS CENTRAL SOUTH ZONE, US FEET. VERTICAL DATUM: NAVD88.

AERIAL PHOTO PROVIDED BY IMAGEPATCH.COM EARTHSTAR GEOGRAPHICS, DATE: MAY-OCT 2011.

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**Coleto Creek Power, LP**

Figure 3A

FLOW CHANNEL
CROSS SECTIONS

| PROJECT: 17266 | BY: RR | DATE: DEC 2017 | CHECKED: DBB |
|---|---|---|---|

Bullock, Bennett & Associates, LLC
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Texas Registrations: Engineering F-8542, Geoscience 50127

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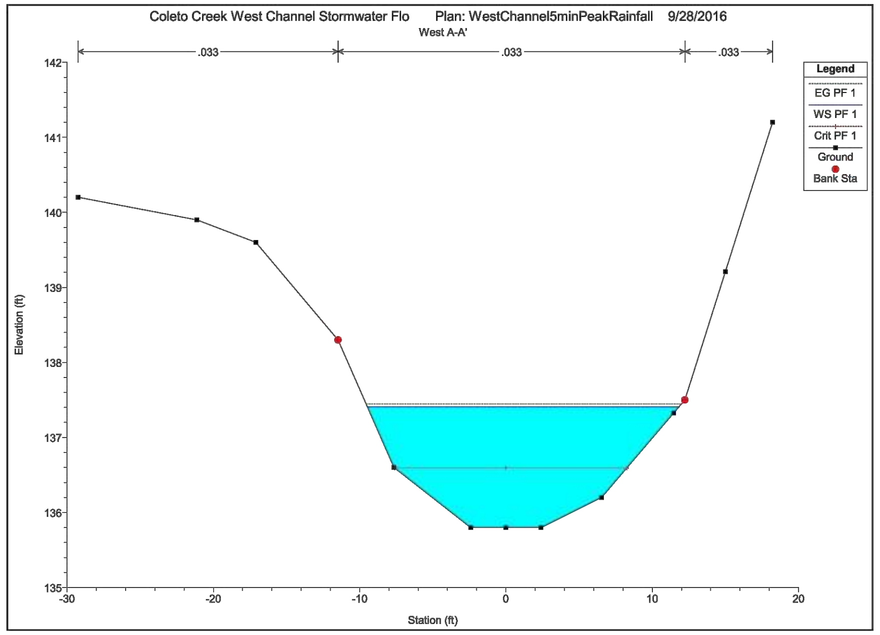
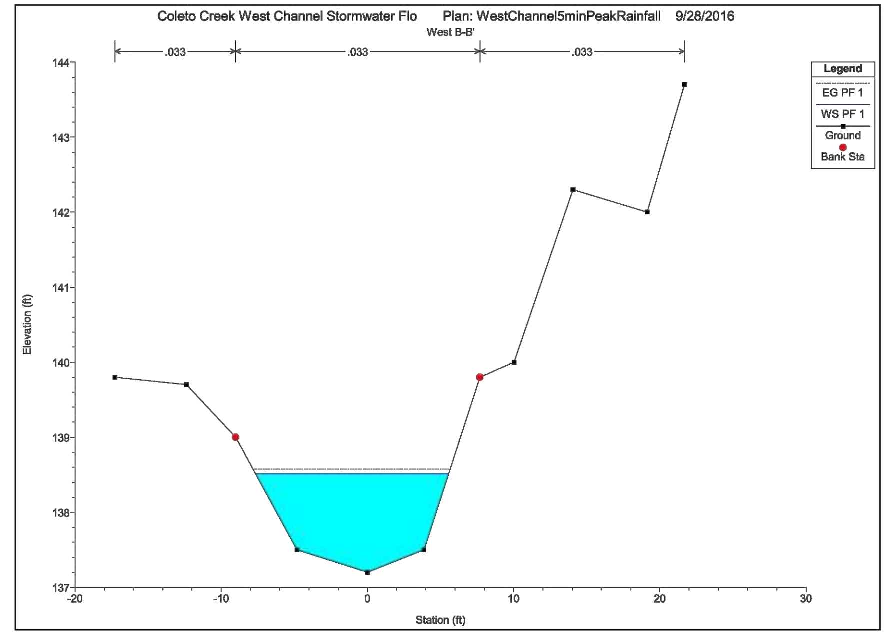



SECTION A-A'

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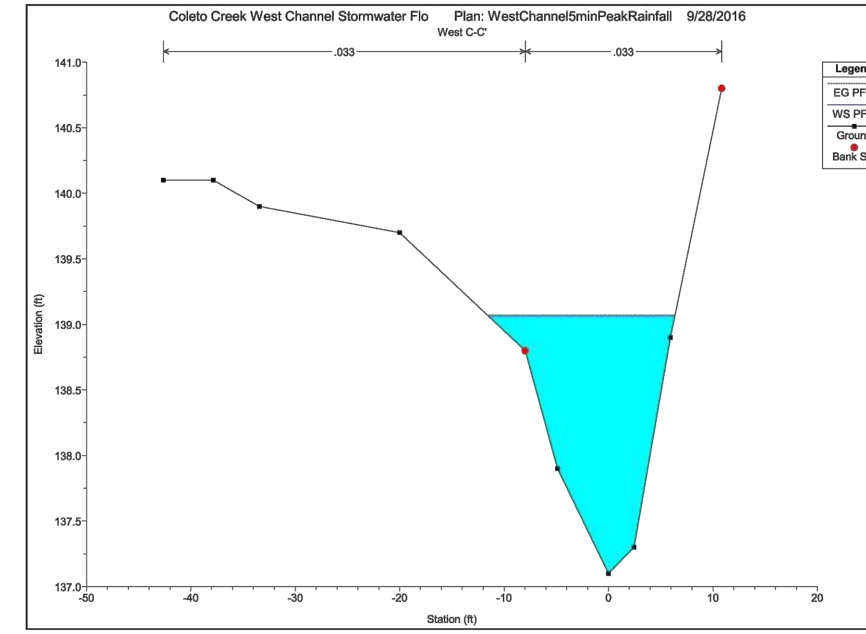
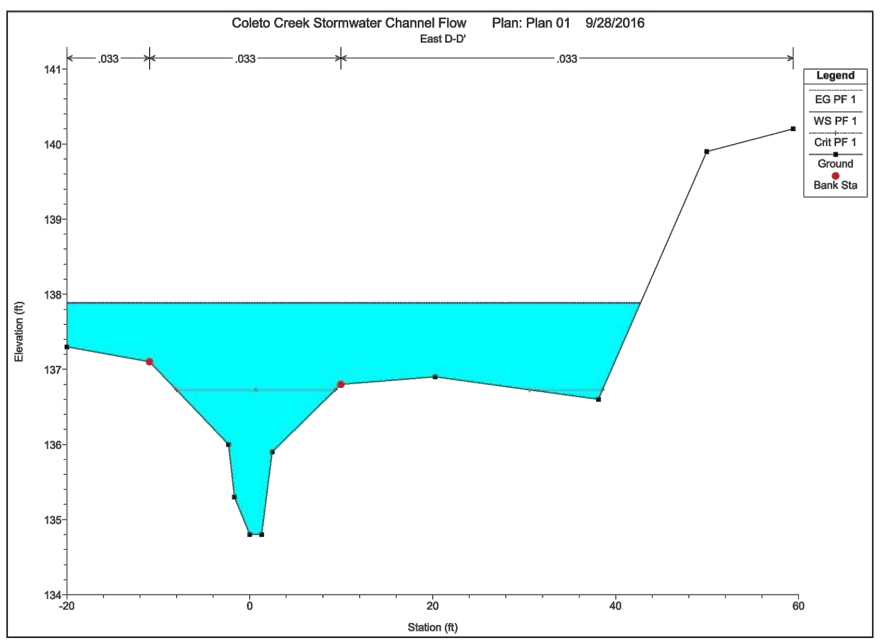



SECTION C-C'

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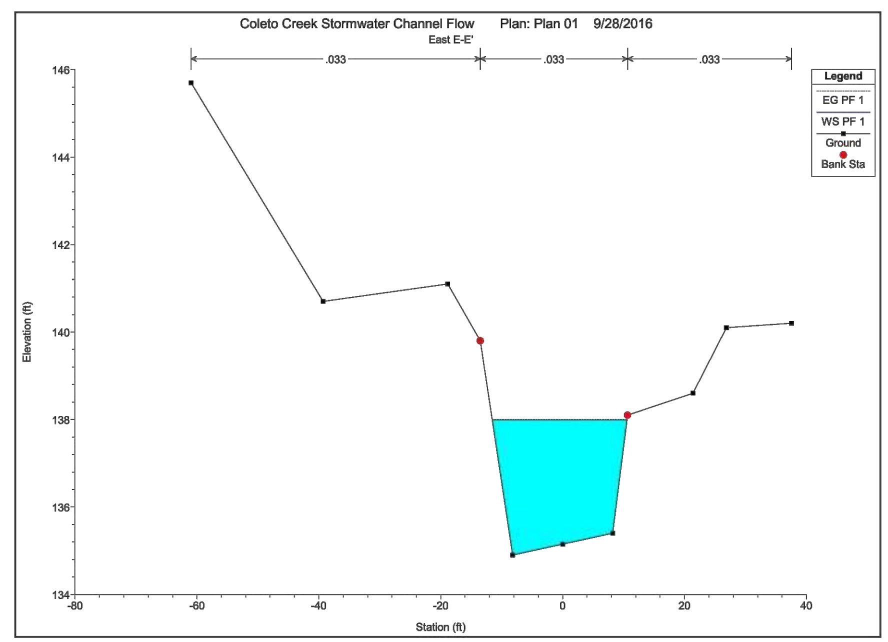
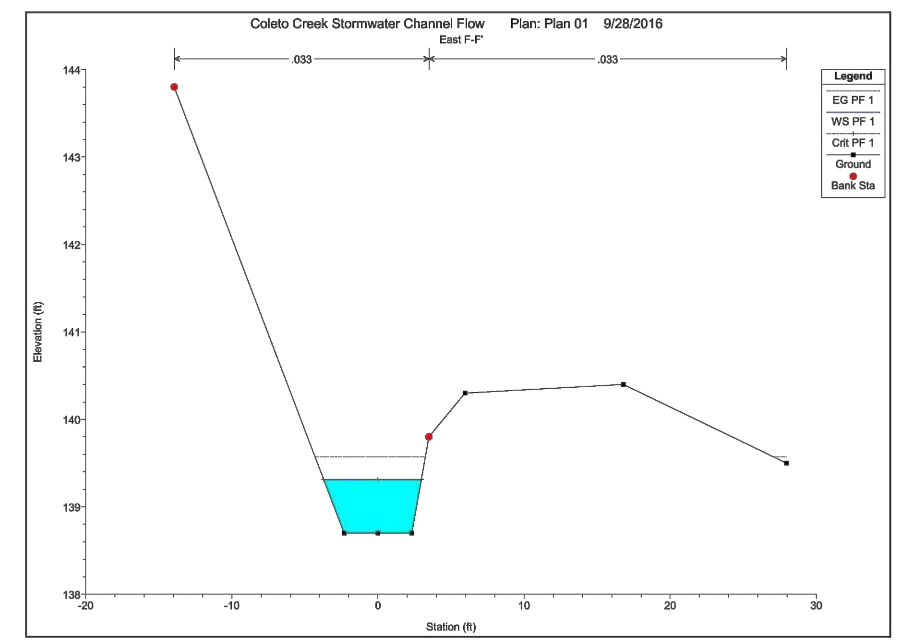



SECTION E-E'




SECTION F-F'

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**Coleto Creek Power, LP**

Figure 3B

FLOW CHANNEL
CROSS SECTIONS

| PROJECT: 17266 | BY: RR | DATE: DEC 2017 | CHECKED: DBB |
|---|---|---|---|

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# APPENDIX A

# CHANNEL FLOW EVALUATION

# PEAK STORM WATER FLOWS THROUGH EAST CHANNEL
## Coleto Creek Primary Ash Pond

| | | |
|---|---|---|
| Total Sluiced Ash Flow | 2.6 mgd | (includes bottom ash and fly ash, and approximately 80% water/20% ash mixture) |
| | 30.1 gal/sec | |
| | 4.0 cfs | |
| Percent Bottom Ash | 80% | (bottom ash sluice represents approximately 80% of total ash sluice flow rates) |
| Flow to East Channel | 3.2 cfs | |

Area Contributing Rainfall to Channel

| Channel Segment | Width (ft) | Length (ft) | Area (sf) |
|---|---|---|---|
| Upstream (F-F') | 35 | 905 | 31675 |
| Mid-channel (E-E') | 100 | 765 | 76500 |
| Downstream (D-D') | 100 | 460 | 46000 |

Note: Width includes from center of dike crest to center of ash berm crest.

Rainfall Values (SARA, 2013 - Table 7, Goliad County, 100-year Storm)

| Rainfall Duration | | Rainfall (in) | Rainfall Rate (f/s) | Flow Rate (cfs) Upstream | Flow Rate (cfs) Midstream | Flow Rate (cfs) Downstream |
|---|---|---|---|---|---|---|
| 5 | min | 1.25 | 3.47E-04 | 11.0 | 26.6 | 16.0 |
| 15 | min | 2.5 | 2.31E-04 | 7.3 | 17.7 | 10.6 |
| 30 | min | 3.38 | 1.56E-04 | 5.0 | 12.0 | 7.2 |
| 1 | hr | 4.4 | 1.02E-04 | 3.2 | 7.8 | 4.7 |
| 2 | hr | 5.5 | 6.37E-05 | 2.0 | 4.9 | 2.9 |
| 3 | hr | 6.55 | 5.05E-05 | 1.6 | 3.9 | 2.3 |
| 6 | hr | 7.8 | 3.01E-05 | 1.0 | 2.3 | 1.4 |
| 12 | hr | 9.3 | 1.79E-05 | 0.6 | 1.4 | 0.8 |
| 24 | hr | 11.4 | 1.10E-05 | 0.3 | 0.8 | 0.5 |

where:
Rainfall Rate = Rainfall / Duration
Flow Rate Contribution to Segment = Rainfall Rate x Channel Segment Area

Highest Flow Rate (Sluice Flow Rate + Rainfall Flow Rate + Previous Segment Flow Rate)

| | |
|---|---|
| Upstream | 14.2 cfs |
| Mid-channel | 40.8 cfs |
| Downstream | 56.8 cfs |

Downstream Boundary Condition (Slope calculated between segments D-D' and E-E')

| | | |
|---|---|---|
| Elevation Change | 0.1 ft | |
| Distance | 621 ft | |
| Downstream Slope | 2.E-04 ft/ft | (note: actual slope downstream of modeled section is steeper) |

Manning Coefficient for Earthen Channel, Winding/Sluggish, Grass/Some Weeds

Maximum 0.033 (Provided in HEC-RAS program)

# PEAK STORM WATER FLOWS THROUGH WEST CHANNEL
## Coleto Creek Primary Ash Pond

| | | |
|---|---|---|
| Total Ash Flow | 2.6 mgd | (includes bottom ash and fly ash, and approximately 80% water/20% ash mixture) |
| | 30.1 gal/sec | |
| | 4.0 cfs | |
| Percent Bottom Ash | 20% | (fly ash sluice represents approximately 20% of total ash sluice flow rates) |
| Flow to East Channel | 0.8 cfs | |

| | Area Contributing Rainfall to Channel | | |
|---|---|---|---|
| Channel Segment | Width (ft) | Length (ft) | Area (sf) |
| Upstream (C-C') | 50 | 705 | 35250 |
| Mid-channel (B-B') | 40 | 840 | 33600 |
| Downstream (A-A') | 50 | 660 | 33000 |

Note: Width includes from center of dike crest to center of ash berm crest.

Rainfall Values (SARA, 2013 - Table 7, Goliad County, 100-year Storm)

| | | Rainfall | Flow Rate (cfs) | | |
|---|---|---|---|---|---|
| Rainfall Duration | Rainfall (in) | Rate (f/s) | Upstream | Midstream | Downstream |
| 5 min | 1.25 | 3.47E-04 | 12.2 | 11.7 | 11.5 |
| 15 min | 2.5 | 2.31E-04 | 8.2 | 7.8 | 7.6 |
| 30 min | 3.38 | 1.56E-04 | 5.5 | 5.3 | 5.2 |
| 1 hr | 4.4 | 1.02E-04 | 3.6 | 3.4 | 3.4 |
| 2 hr | 5.5 | 6.37E-05 | 2.2 | 2.1 | 2.1 |
| 3 hr | 6.55 | 5.05E-05 | 1.8 | 1.7 | 1.7 |
| 6 hr | 7.8 | 3.01E-05 | 1.1 | 1.0 | 1.0 |
| 12 hr | 9.3 | 1.79E-05 | 0.6 | 0.6 | 0.6 |
| 24 hr | 11.4 | 1.10E-05 | 0.4 | 0.4 | 0.4 |

where:
Rainfall Rate = Rainfall / Duration
Flow Rate Contribution to Segment = Rainfall Rate x Channel Segment Area

Highest Flow Rate (Sluice Flow Rate + Rainfall Flow Rate + Previous Segment Flow Rate)

| | |
|---|---|
| Upstream | 13.0 cfs |
| Mid-channel | 24.7 cfs |
| Downstream | 36.2 cfs |

Downstream Boundary Condition (Slope calculated between segments A-A' and B-B')

| | | |
|---|---|---|
| Elevation Change | 1.4 ft | |
| Distance | 1015 ft | |
| Downstream Slope | 0.001 ft/ft | (note: actual slope downstream of modeled section is steeper) |

Manning Coefficient for Earthen Channel, Winding/Sluggish, Grass/Some Weeds

Maximum 0.033 (Provided in HEC-RAS program)