AECOM

# J.K. SPRUCE POWER PLANT -

# PLANT DRAINS POND -

Initial Inflow Design Flood Control System Plan

July 2023
AECOM Project 60566130

Delivering a better world

Prepared for:

CPS Energy Calaveras Power Station
12940 U.S. Highway 181 South
San Antonio, Texas 78223

Prepared by:

AECOM
12640 Briarwick Drive, Suite 250
Austin, TX 78729
aecom.com

**J.K. Spruce Power Plant –**

**Plant Drains Pond – Initial Inflow Design Flood Control System Plan**

## TABLE OF CONTENTS

# J.K. SPRUCE POWER PLANT
# INITIAL INFLOW DESIGN FLOOD CONTROL SYSTEM PLAN
# TAC Title 30, Part 1, §352, Subchapter G, §352.821 and 40 CFR § 257.82
# PLANT DRAINS POND (PDP)

## Hydrologic and Hydraulic Capacity Criteria

***30 TAC §352.821 Hydrologic and Hydraulic Capacity Requirements for Coal Combustion Residuals Surface Impoundments.*** *The commission adopts by reference 40 Code of Federal Regulations §257.82 (Hydrologic and hydraulic capacity requirements for CCR surface impoundments) as amended through the April 17, 2015, issue of the Federal Register (80 FR 21301).*

### *40 CFR § 257.82 Hydrologic and hydraulic capacity requirements for CCR surface impoundments*

*(a) The owner or operator of an existing or new CCR surface impoundment or any lateral expansion of a CCR surface impoundment must design, construct, operate, and maintain an inflow design flood control system as specified in paragraphs (a)(1) and (2) of this section.*

*(1) 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 specified in paragraph (a)(3) of this section.*

*(2) 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 specified in paragraph (a)(3) of this section.*

*(3) The inflow design flood is:*

*(i) For a high hazard potential CCR surface impoundment, as determined under § 257.73(a)(2) or § 257.74(a)(2), the probable maximum flood;*

*(ii) For a significant hazard potential CCR surface impoundment, as determined under § 257.73(a)(2) or § 257.74(a)(2), the 1,000-year flood;*

*(iii) For a low hazard potential CCR surface impoundment, as determined under § 257.73(a)(2) or § 257.74(a)(2), the 100-year flood; or*

*(iv) For an incised CCR surface impoundment, the 25-year flood.*

*(b) Discharge from the CCR unit must be handled in accordance with the surface water requirements under §257.3-3.*

*(c) Inflow design flood control system plan –*

*(1) Content of the Plan. The owner or operator must prepare initial and periodic inflow design flood control system plans for the CCR unit according to the timeframes specified in paragraphs (c)(3) and (4) of this section. These plans must document how the inflow design flood control system has been designed and constructed to meet the requirements of this section. Each plan must be supported by appropriate engineering calculations. The owner or operator of the CCR unit has completed the inflow design flood control system plan when the plan has been placed in the facility's operating record as required by §257.105(g)(4).*

*(2) Amendment of the Plan. The owner or operator of the CCR unit may amend the written inflow design flood control system plan at any time provided the revised plan is placed in the facility's operating record as required by §257.105(g)(4). The owner or operator must amend the written inflow design flood control system plan whenever there is a change in conditions that would substantially affect the written plan in effect.*

*(3) Timeframes for preparing the initial plan -*

*(i) Existing CCR surface impoundments. The owner or operator must prepare the initial inflow design flood control system plan no later than October 17, 2016.*

*(ii) New CCR surface impoundments and any lateral expansion of a CCR surface impoundment. The owner of operator must prepare the initial inflow design flood control system plan no later than the date of initial receipt of CCR in the CCR unit.*

*(4) Frequency for revising the plan. The owner or operator must prepare periodic inflow design flood control system plans required by paragraph (c)(1)*

## Hydrologic and Hydraulic Capacity Documentation

This Initial Inflow Design Flood Control System Plan (Plan) document has been prepared for the new Plant Drains Pond (PDP) at the J.K. Spruce Power Plant. CPS Energy is the operator of the J.K. Spruce Power Plant.

This Plan has been prepared in accordance with the requirements prescribed in §257.82 of the Federal Register, Volume 80, Number 74, dated April 17, 2015 (U. S. Government, 2015) for hydrologic and hydraulic capacity requirements for existing and new Coal Combustion Residual (CCR) surface impoundments. Section §257.82 is reproduced in the column to the left for reference purposes. This document serves as the initial plan described in §257.82 (c).

The PDP is a new CCR surface impoundment facility with two cells (identified as "west" and "east") with dikes on all sides. The site of the PDP is located on the east side of the existing landfill haul road and slopes from northwest to south east towards the cooling water canal. The maximum height of embankment, on the east embankment of the east cell, is approximately 15 feet. The inside crest elevation is 514.8 feet above mean sea level (amsl).

The PDP only receives inflows from plant discharges and from direct precipitation. The maximum normal operating water surface elevation is set at 512.8 feet amsl, providing 2.0 feet of freeboard as available storage volume for the direct precipitation resulting from Inflow Design Flood (IDF). The elevation-area-capacity relationship information is provided in Table 1.

Inflows to the PDP are pumped from the Plant. Outflows from the PDP are pumped to the PDP Clarifiers and then flow by gravity to a distribution/sample box where they can be discharged through new internal Outfall 714 into the Station Discharge Canal #2 (Outfall 007) or returned to the PDP.

In a separate certification by CPS dated June 26, 2023, a qualified professional engineer described the basis for an Initial Hazard Potential Classification for the PDP in accordance with the requirements of 40 CFR § 257 and designated the PDP as a "Significant Hazard" Potential CCR Surface Impoundment. Therefore, in accordance with § 257.82(a)(3)(ii), the inflow design flood is the 1,000-year flood.

Figure 1, "Plant Drains Pond, Stormwater Diversion Schematic", depicts the drainage diversion site improvements made upgradient of the PDP site in order to route the 1,000-year flood around the PDP. Under preexisting conditions, there were two watersheds tributary to the PDP site, labelled as "North Culvert Watershed" and "South Culvert Watershed", separated by the existing "North Inlet Channel" that directed flow from the North Culvert Watershed to the existing "North Culvert" under the haul road. The South Culvert Watershed drained south to the existing "South Culvert" under the haul road.

The engineering calculations supporting the design of site improvements to route the 1,000-year flood around the new PDP are presented in Attachment 1, "Plant Drains Pond – Drainage Calculations". The improvements are summarized as follows:

North Culvert Watershed:

1. Increase capacity of the existing North Inlet Channel by widening the base and constructing a downstream berm.
2. Extend the North Inlet Channel by 200 feet to a new inlet for a new three-barrel, 24-inch CMP culvert under the haul road.
3. Construct a broad weir to spread channel flows that exceed the 10-year flood across the haul road and into a natural swale that directs flows away from the PDP.
4. Remove existing North Culverts.

*of this section every five years. The date of completing the initial plan is the basis for establishing the deadline to complete the first periodic plan. The owner or operator may complete any required plan prior to the required deadline provided the owner or operator places the completed plan into the facility's operating record within a reasonable amount of time. In all cases, the deadline for completing a subsequent plan is based on the date of completing the previous plan. For purposes of this paragraph (c)(4), the owner or operator has completed an inflow design flood control system plan when the plan has been placed in the facility's operating record as required by §257.105(g)(4).*

*(5) The owner or operator must obtain a certification from a qualified engineer stating that the initial and periodic inflow design flood control system plans meet the requirements of this section.*

*(d) The owner or operator of the CCR unit must comply with the record keeping requirements specified in §257.105(g), the notification requirements specified in §257.106(g), and the internet requirements specified in §257.107(g).*

South Culvert Watershed:

1. Construct a V-ditch parallel to, and on the west side of, the haul road to direct flows south to the South Culvert and to prevent localized ponding.
2. Leave as-is the existing South Culverts, which have sufficient capacity to pass the peak runoff from the 1,000-year flood.
3. Construct a new two-barrel, 24-inch CMP culvert under the toe of the Clarifier complex on the south embankment of the PDP.
4. Grade the area between the two sets of culverts to direct surface flows towards the downgradient set.

Haul Road:

1. Construct a one-foot-high Stormwater Diversion Berm, located between the haul road and the west embankment crest of the PDP, to prevent incidental run-on into the PDP and to collect and channel runoff from the haul road to the south and around the PDP.

Required Plan Contents

1. *"§ 257.82(a)(1) 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 specified in paragraph (a)(3) of this section."*

   Runoff from upstream tributary basins is diverted around the surface impoundment by the enlarged North Inlet Channel, the new North Culverts, the Stormwater Diversion Berm along the west edge of the PDP, and the South Culvert system. The remaining basin area tributary to the PDP consists of the pond itself and half the crest width of the surrounding embankment.

2. *"§ 257.82(a)(2) 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 specified in paragraph (a)(3) of this section."*

   The PDP is sized to contain the direct precipitation resulting from the 1,000-year, 24-hour precipitation event, estimated at 19.3 inches based on "NOAA Atlas 14 Point Precipitation Frequency Estimates: TX." The tributary areas, including half the crest width of the perimeter berms, are 1.95 and 1.96 acres for the west and east cells, respectively. The PDP freeboard depth is sufficient to manage the direct precipitation resulting from the inflow design flood without discharge. In addition, the central divider berm of the PDP incorporates two shallow spillways (invert elevation of 514.3 feet amsl) to allow overflow of water from the more-full cell to the less-full cell during a major storm event.

3. *"§ 257.82(a)(3) The inflow design flood is: . . . (ii) For a significant hazard potential CCR surface impoundment, as determined under § 257.73(a)(2) or § 257.74(a)(2), the 1,000-year flood."*

   As identified in accordance with § 257.74(a)(2), the PDP has been categorized as a significant hazard potential CCR surface impoundment; therefore, the inflow design flood is the 1,000-year flood.

4. *"§ 257.82(b) Discharge from the CCR unit must be handled in accordance with the surface water requirements under §257.3-3."*

   The PDP provides hydraulic retention to buffer the flow to the PDP Clarifiers and allow the larger solids to settle. Wastewater discharged to the PDP drains by gravity to the PDP Sump. Wastewater entering the sump is sent to the PDP Clarifiers for fine solids removal and final polishing. Clarified effluent flows by gravity to a distribution/sample box where it can be discharged through new internal Outfall 714 and into the Station Discharge Canal #2 (Outfall 007) or returned to the PDP. Sampling and discharge at outfalls will be in accordance with the guidelines outlined within the TPDES permit.

5. *"§ 257.82(c)(1) Content of the Plan. The owner or operator must prepare initial and periodic inflow design flood control system plans for the CCR unit according to the timeframes specified in paragraphs (c)(3) and (4) of this section. These plans must document how the inflow design flood control system has been designed and constructed*

*to meet the requirements of this section. Each plan must be supported by appropriate engineering calculations. The owner or operator of the CCR unit has completed the inflow design flood control system plan when the plan has been placed in the facility's operating record as required by §257.105(g)(4)."*

This Plan describes how the inflow design flood control system has been designed and constructed to meet the requirement to manage the designated inflow design flood. The engineering calculations supporting the design of site improvements to route the 1,000-year flood around the new PDP are presented in Attachment 1, "Plant Drains Pond – Drainage Calculations". This *Initial Inflow Design Flood Control Plan* serves as the initial plan prescribed herein.

*6.* *"§ 257.82(c)(2) Amendment of the Plan. The owner or operator of the CCR unit may amend the written inflow design flood control system plan at any time . . . whenever there is a change in conditions that would substantially affect the written plan in effect."*

CPS Energy acknowledges this requirement.

*7.* *"§ 257.82(c)(3) Timeframes for preparing the initial plan - . . . (ii) New CCR surface impoundments and any lateral expansion of a CCR surface impoundment. The owner of operator must prepare the initial inflow design flood control system plan no later than the date of initial receipt of CCR in the CCR unit."*

The PDP is a new CCR impoundment at the J.K. Spruce Power Plant. The Initial Inflow Design Flood Control System Plan is included herein.

CPS Energy acknowledges this requirement.

*8.* *"§ 257.82(c)(4) Frequency for revising the plan. The owner or operator must prepare periodic inflow design flood control system plans . . . every five years. . . the owner or operator has completed an inflow design flood control system plan when the plan has been placed in the facility's operating record as required by §257.105(g)(4)."*

CPS Energy acknowledges this requirement.

*9.* *"§ 257.82(c)(5) The owner or operator must obtain a certification from a qualified engineer stating that the initial and periodic inflow design flood control system plans meet the requirements of this section."*

This document is certified and sealed by a qualified professional engineer.

*10.* *"§ 257.82(d) The owner or operator of the CCR unit must comply with the record keeping requirements specified in §257.105(g), the notification requirements specified in §257.106(g), and the internet requirements specified in §257.107(g)."*

CPS Energy acknowledges this requirement.

**Certification Statement 40 CFR § 257.82(c)(5) – Initial Inflow Design Flood Control System Plan for a New CCR Surface Impoundment**

**CCR Unit: CPS Energy; J.K. Spruce Power Plant; Plant Drains Pond**

I, Alexander W. Gourlay, 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 certification 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 initial inflow design flood control system plan dated July 14, 2023 meets the requirements of 40 CFR § 257.82.

Alexander W. Gourlay, P.E.
*Printed Name*

July 14, 2023
*Date*

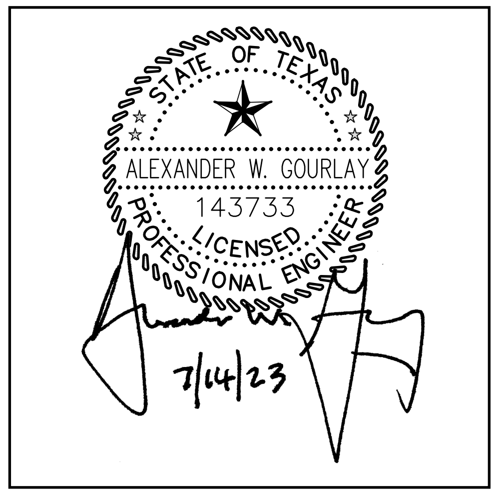
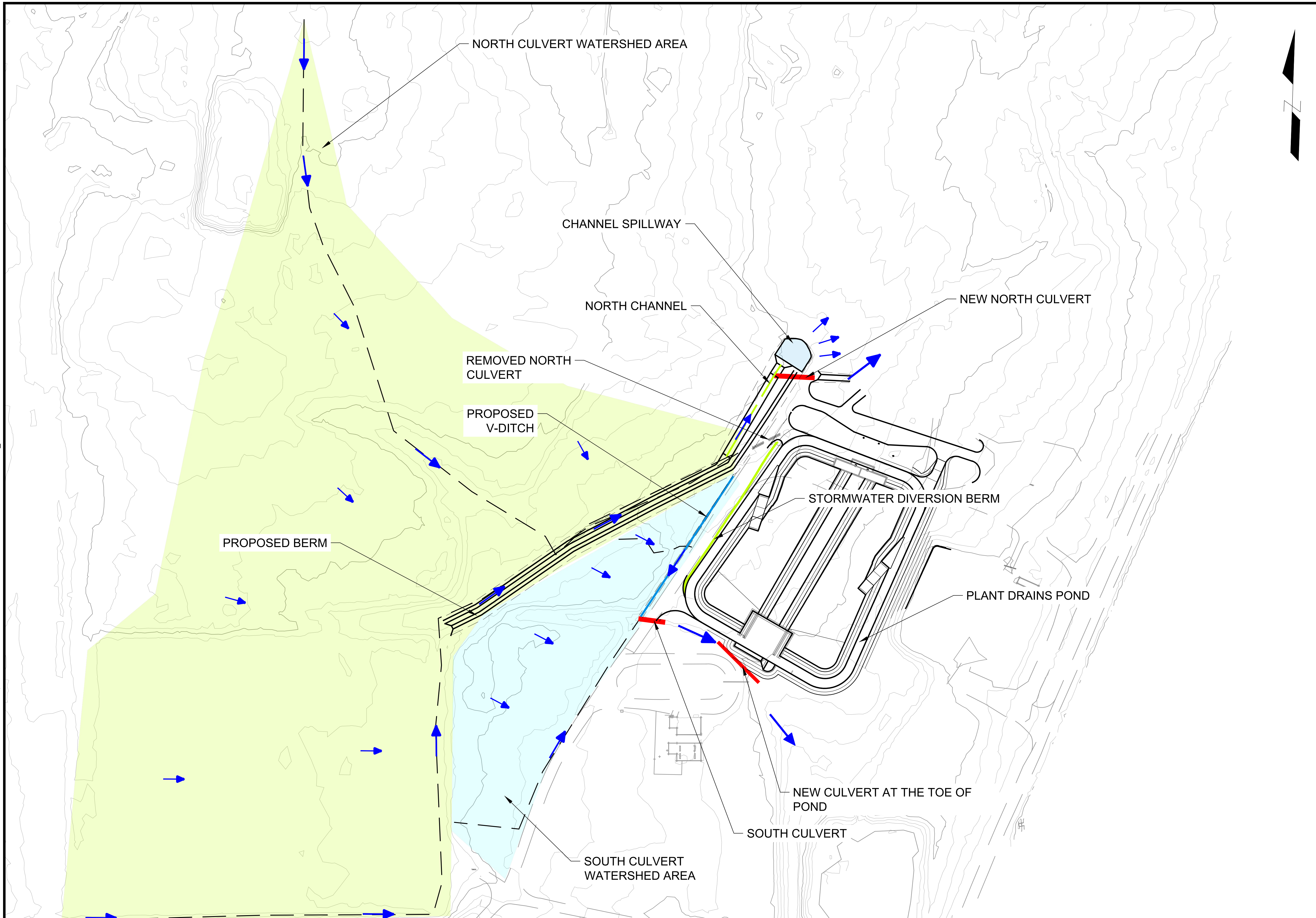


**FIGURE 1**

**PLANT DRAINS POND –**

**STORMWATER DIVERSION SCHEMATIC**




Issue Status: SUBMITTAL

AECOM

Figure: 1

**PLANT DRAINS POND**
**STORMWATER DIVERSION SCHEMATIC**
J K SPRUCE POWER PLANT
AECOM PROJECT NO. 60566130

Date: 2022-06-24

**TABLE 1**

**PLANT DRAINS POND –**

**ELEVATION-AREA-CAPACITY RELATIONSHIP**

**TABLE 1 -**
**ELEVATION-AREA-CAPACITY RELATIONSHIP**
**J.K. SPRUCE PLANT DRAINS POND**

CPS Spruce Drains Pond Project
Pond Capacities for IFC 5/4/22

AECOM Project No. 60566130
Prepared by Alan Proctor 05/09/22
Checked by Sandy Gourlay 05/10/22

| POND: | ELEV. | AREA (SF) | AREA (AC) | ELEVATION DIFFERENCE (FT.) | INTERVAL VOLUME (AC-FT.) | CUMULATIVE VOLUME (AC-FT.) | |
|---|---|---|---|---|---|---|---|
| WEST CELL | 507.11 | 90 | 0.00 | - | - | 0.00 | Bottom of pond at sump |
| | 508.00 | 36161 | 0.83 | 0.9 | 0.37 | 0.37 | |
| | 509.00 | 47107 | 1.08 | 1.0 | 0.96 | 1.33 | |
| | 510.00 | 50932 | 1.17 | 1.0 | 1.13 | 2.45 | |
| | 511.00 | 54850 | 1.26 | 1.0 | 1.21 | 3.67 | |
| | 512.00 | 58941 | 1.35 | 1.0 | 1.31 | 4.97 | |
| | 512.80 | 61367 | 1.41 | 0.8 | 1.10 | 6.08 | Max normal operating level |
| | 513.00 | 62952 | 1.45 | 0.2 | 0.29 | 6.36 | |
| | 514.00 | 66981 | 1.54 | 1.0 | 1.49 | 7.85 | |
| | 514.30 | 68518 | 1.57 | 0.3 | 0.47 | 8.32 | Spillway invert |
| | 514.80 | 72968 | 1.68 | 0.5 | 0.81 | 9.13 | Crest Inside Elevation |
| | 515.00 | 84760 | 1.95 | 0.2 | 0.36 | 9.49 | Crest road crown |
| | | | | | | | |
| EAST CELL | 507.11 | 90 | 0.00 | - | - | 0.00 | Bottom of pond at sump |
| | 508.00 | 36087 | 0.83 | 0.9 | 0.37 | 0.37 | |
| | 509.00 | 47096 | 1.08 | 1.0 | 0.95 | 1.32 | |
| | 510.00 | 51024 | 1.17 | 1.0 | 1.13 | 2.45 | |
| | 511.00 | 55040 | 1.26 | 1.0 | 1.22 | 3.67 | |
| | 512.00 | 59226 | 1.36 | 1.0 | 1.31 | 4.98 | |
| | 512.80 | 61802 | 1.42 | 0.8 | 1.11 | 6.09 | Max normal operating level |
| | 513.00 | 63327 | 1.45 | 0.2 | 0.29 | 6.38 | |
| | 514.00 | 67441 | 1.55 | 1.0 | 1.50 | 7.88 | |
| | 514.30 | 68972 | 1.58 | 0.3 | 0.47 | 8.35 | Spillway invert |
| | 514.80 | 72968 | 1.68 | 0.5 | 0.81 | 9.16 | Crest Inside Elevation |
| | 515.00 | 85429 | 1.96 | 0.2 | 0.36 | 9.53 | Crest road crown |

**ATTACHMENT A**

**AECOM, 2022. *Plant Drains Pond – Drainage Calculations*.**

**Prepared for: CPS Energy**

**AECOM Job No. 60566130, June 2022.**

| Project<br>Spruce Plant Drains Project | Job No.<br>60566130 |
|---|---|
| **Client**<br>CPS Energy | **Department/Discipline**<br>Civil |
| **Software Name**<br>FlowMaster, HY8 | |

| Calculation Rev. No. | Originator Self Check (name and signature) | Reviewer/Checker (name and signature) | Independent Peer Reviewer (if used/required) (name & signature) | Approver (name & signature) |
|---|---|---|---|---|
| 1 | Alireza Samieadel<br>Alireza Samieadel | Todd Ringsmuth | | Alexander Gourlay |
| | | | | |

*Add rows as required*

**Calculation Objective:**
Determine the dimensions for channels and culverts utilized to divert stormwater flows from the 1,000-year storm event around the Plant Drains Pond. The diversion system includes the following: North Channel, North Culverts, South Culverts, South Area Channel, and Pond Culverts. Peak flows will be estimate for each component of the diversion system and used to size the channels and culverts.

**Calculation Methodology:**
NOAA Precipitation data is utilized to develop precipitation depths and intensities. The Rational Method and TxDOT Hydraulics Manual are used to estimate peak flows. FlowMaster and HY8 computer programs were used to estimate channel and culvert capacities.

**References / Inputs/ Field Data:**
See calculations

**Assumptions:** (Include comments on need to revise calculations after more data is collected/confirmed and/or after assumptions have been verified.)
See calculations

**Conclusions including confirmations to be obtained:**
See calculations

**This calculation is complete and ready for Discipline Review:**

| Alireza Samieadel | Alireza Samieadel | 06/16/2022 |
|---|---|---|
| **Originator Name** | **Signature** | **Date** |

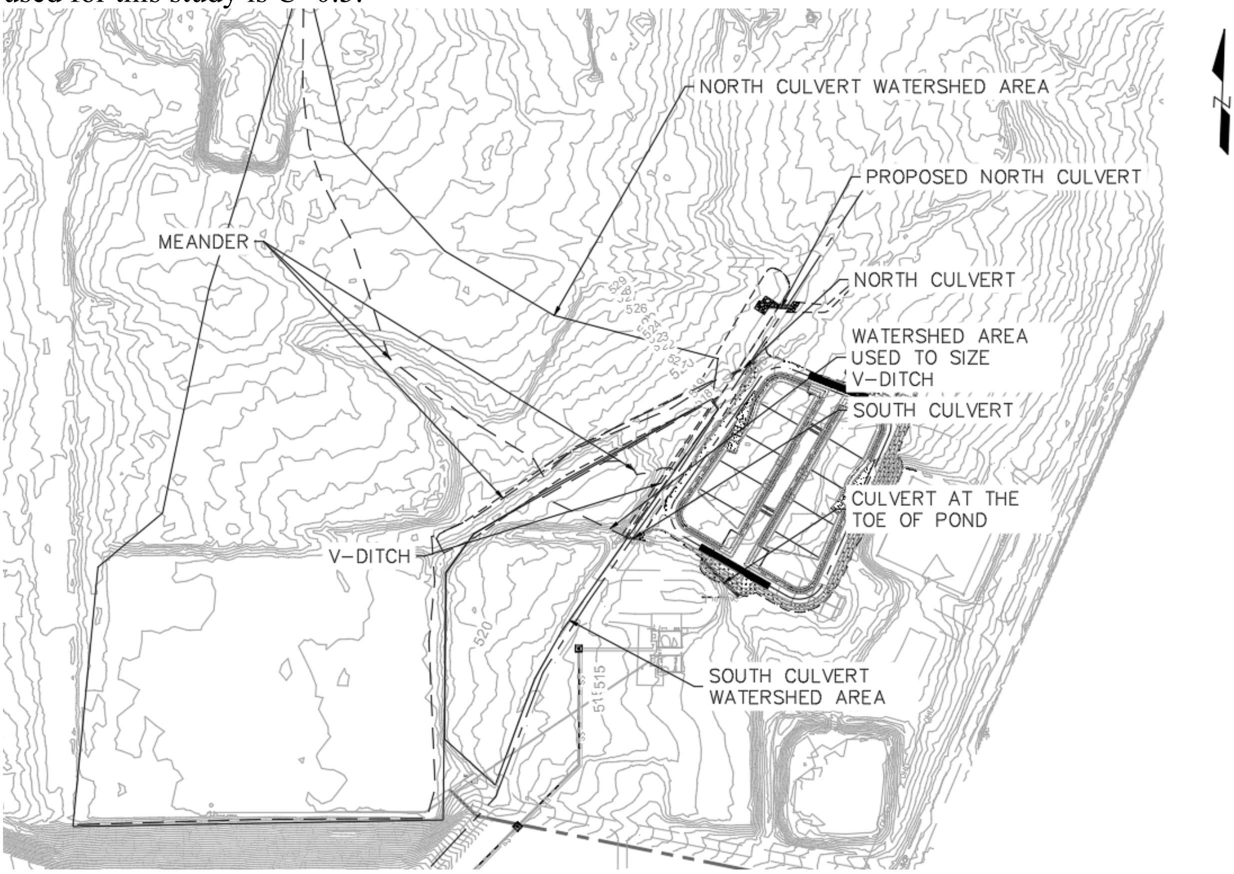
## Background and Goal

The purpose of this calculation is to develop sizes for channels and culvert utilized to divert stormwater flows from the 1,000-year storm event around the proposed Plant Drains Pond. Stormwater currently drains from west to east and is managed by the existing North Channel, North Culvert, and South Culverts. With the construction of the proposed Plant Drains Pond, several improvements are required to divert the design stormwater flows around the pond. Those improvements will consist of the following:

- Modifying the North Channel to increase the channel capacity.
- Installing new North Culverts to convey flow in the North Channel under the existing road. The existing North Culverts will be blocked or removed.
- Constructing a new channel (v-ditch) along the roadway in the South Culvert area. The new channel is required to convey stormwater to the existing South Culverts.
- Constructing new culverts and channelization at the south edge of the Plant Drains Pond to convey stormwater from the existing South Culverts to the east.

## Site characteristics

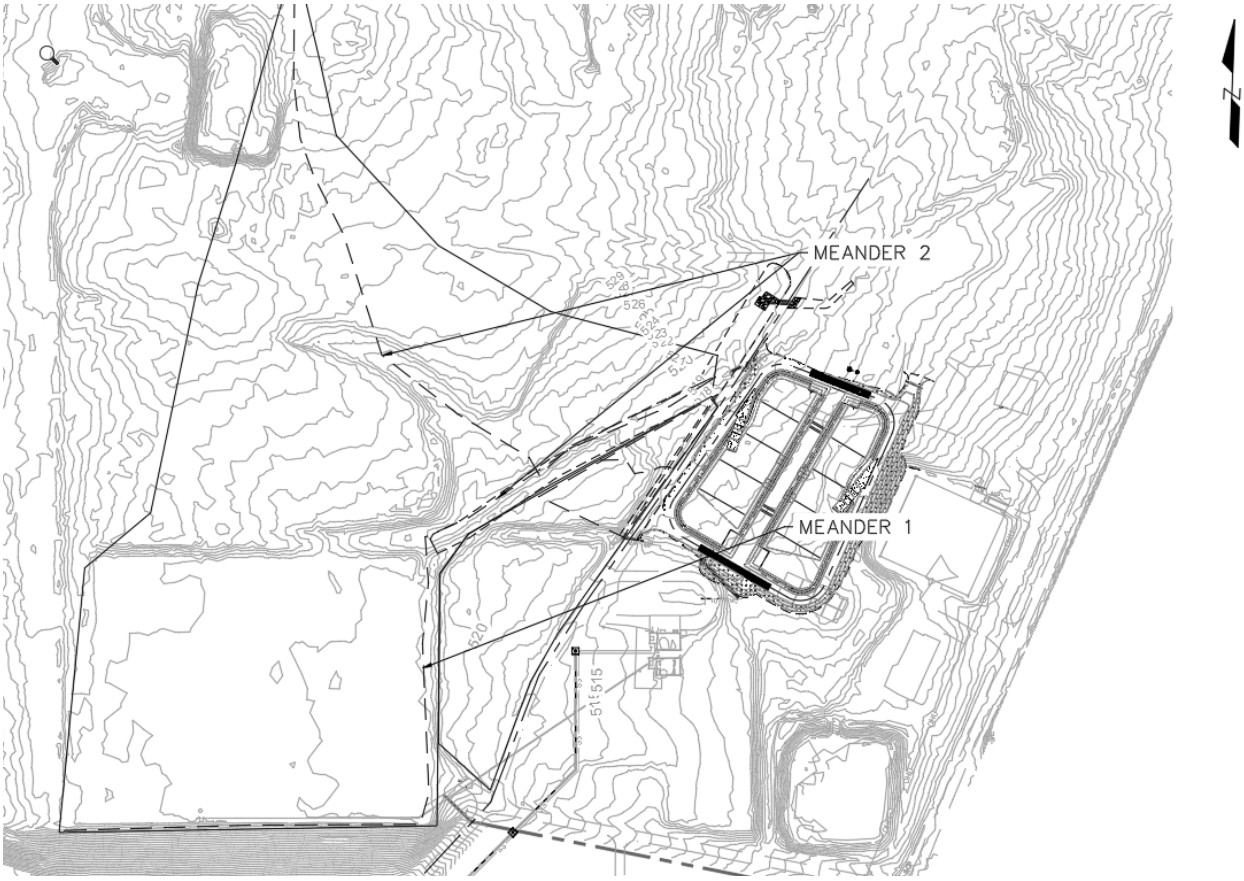
The area west of the pond consists of the North Culvert and South Culvert Watersheds with areas of 30.2 acres and 4.5 acres, respectively. The area tributary to the v-ditch within the South Culvert Watershed is 1.15 acres. The runoff coefficient is a weighted average of sandy soil (substation) with average slope (C=0.1-0.15) and cultivated land (C=0.2-0.5). The weighted average runoff coefficient used for this study is C=0.3.




NORTH CULVERT WATERSHED AREA
PROPOSED NORTH CULVERT
MEANDER
NORTH CULVERT
WATERSHED AREA USED TO SIZE V-DITCH
SOUTH CULVERT
CULVERT AT THE TOE OF POND
V-DITCH
520
SOUTH CULVERT WATERSHED AREA

# 1. North Culvert Watershed Peak Flow Estimation

## 1.1 Time of concentration




Using Kirpich Method, time of concentration can be calculated using the watershed information. Two meanders were evaluated, as shown below.

| Meander 1 | |
|---|---|
| Elevation Drop | From 524.85’ to 517.66 |
| Length | 2230 ft |
| Slope | 0.0032 ft/ft |

| Meander 2 | |
|---|---|
| Elevation Drop | From 542’ to 518’ |
| Length | 1856.6 ft |
| Slope | 0.013 ft/ft |

The Kirpich Method formula is as follows:

$$t_{ch} = KL^{0.770} S^{-0.385}$$

*Equation 4-15.*

***Where*:**

$t_{ch}$ = the time of concentration, in minutes

$K$ = a units conversion coefficient, in which K = 0.0078 for traditional units and K = 0.0195 for SI units

$L$ = the channel flow length, in feet or meters as dictated by K

$S$ = the dimensionless main-channel slope

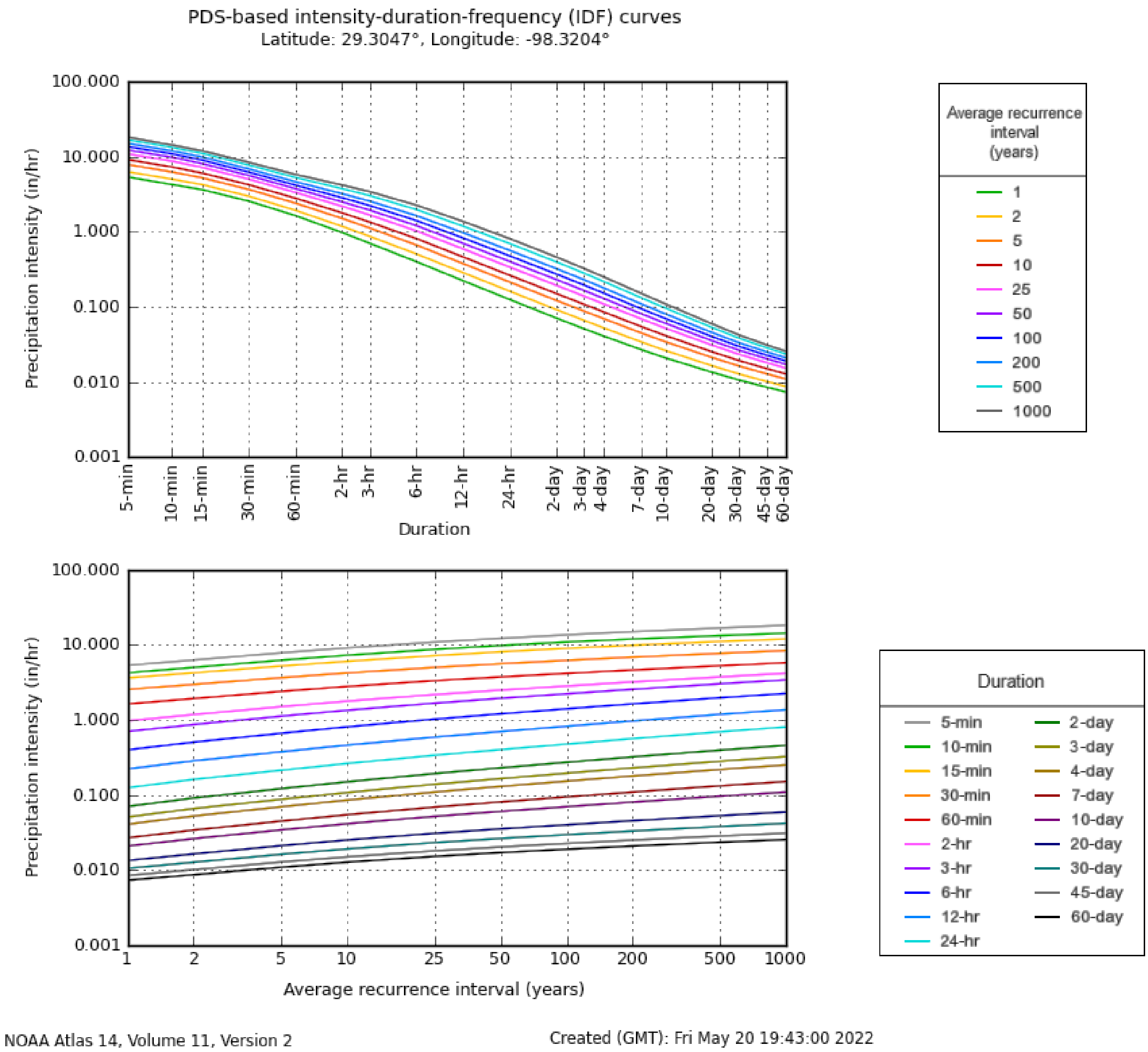
From Hydraulic Design Manual published by TxDOT 2019
http://onlinemanuals.txdot.gov/txdotmanuals/hyd/hyd.pdf

**Results are as follow:**

| Meander | Tc (min) |
|---|---|
| 1 | 26.97 |
| 2 | 14.08 |

### 1.2 Rainfall Intensity

Using data obtained from NOAA's national weather service (https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=tx) for the coordinates of the, the following figures are IDF Curves 9 (intensity-duration-frequency).




For the calculated time of concentration, the precipitation intensities were estimated for the 10-year and 1,000-year storm events. The data for the 10-year storm event will be used to evaluate alternate culvert capacities and sizing at the North Culvert.

### 1,000-year Precipitation Intensity

| Meander | Tc (min) | Precipitation intensity (in/hr) |
|---|---|---|
| 1 | 26.97 | 9.6 |
| 2 | 14.08 | 12.5 |

### 10-year Precipitation Intensity

| Meander | Tc (min) | Precipitation intensity (in/hr) |
|---|---|---|
| 1 | 26.97 | 3.8 |
| 2 | 14.08 | 5.3 |

### 1.3 Peak Flow Estimation

The Rational Method formula estimates the peak flowrate at a specific location in a watershed as a function of the drainage area, runoff coefficient, and mean rainfall intensity for a duration equal to the time of concentration. The Rational Method formula is:

$$Q = \frac{CIA}{Z}$$

Equation 4-20.

**Where**:

$Q$ = maximum rate of runoff (cfs or m$^3$/sec.)

$C$ = runoff coefficient

$I$ = average rainfall intensity (in./hr. or mm/hr.)

$A$ = drainage area (ac or ha)

$Z$ = conversion factor, 1 for English, 360 for metric

From Hydraulic Design Manual published by TxDOT 2019
http://onlinemanuals.txdot.gov/txdotmanuals/hyd/hyd.pdf

In this report, units used are A(ac), i(in/hr) and Q(cfs).

The peak flowrates were estimated as follows:

**1,000-year Peak Flow**

| Meander | Runoff Coefficient | Tc (min) | Precipitation intensity (in/hr) | Area (ac) | Peak Flow (cfs) |
|---|---|---|---|---|---|
| 1 | 0.3 | 26.97 | 9.6 | 30.2 | 86 |
| 2 | 0.3 | 14.08 | 12.5 | 30.2 | 113 |

**10-year Peak Flow**

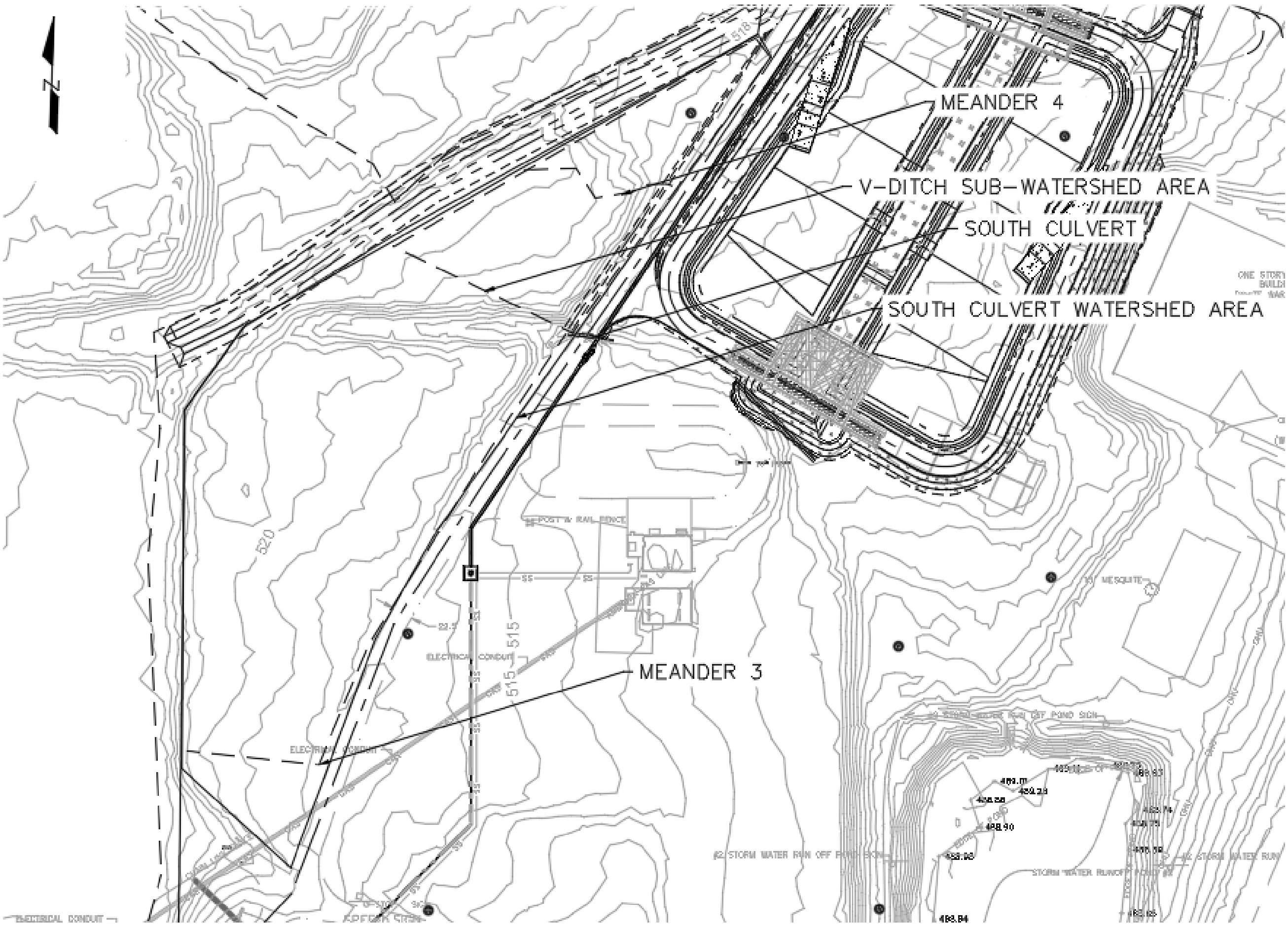
| Meander | Runoff Coefficient | Tc (min) | Precipitation intensity (in/hr) | Area (ac) | Peak Flow (cfs) |
|---|---|---|---|---|---|
| 1 | 0.3 | 26.97 | 3.8 | 30.2 | 34 |
| 2 | 0.3 | 14.08 | 5.3 | 30.2 | 48 |

For each storm event, the larger estimated peak flow will be used for design: 113 cfs for the 1,000-year storm event and 48 cfs for the 10-year storm event.

## 2. South Culvert Watershed Peak Flow Estimation

This calculation estimates peak flows for the South Culvert Watershed and the portion of that watershed that drains to the proposed v-ditch located on the west side of the road. The v-ditch will capture and convey stormwater from and area north of the South Culverts to prevent overtopping of the road.

## 2.1 Time of concentration




Using Kirpich Method, time of concentration can be calculated using the watershed information.

| Meander 3 (South Culvert Watershed) | |
|---|---|
| Elevation Drop | From 517.7 to 514.5 |
| Length | 651 |
| Slope | 0.005 ft/ft |
| Time of Concentration based on Kirpich Method | 8.8 min >> Use 10 min |

| Meander 4 (V-Ditch Sub-Watershed) | |
|---|---|
| Elevation Drop | From 522 to 515.6 |
| Length | 180 |
| Slope | 0.004 ft/ft |
| Time of Concentration based on Kirpich Method | 3.6 min >> Use 10 min |

The time of concentration is limited to no less than 10 minutes for this calculation because shorter times give unrealistic intensities. Many intensity-duration-frequency curves are constructed from curve-smoothing equations and not based on actual data collected at intervals shorter than 15 to 30 minutes. Making the curves shorter involves extrapolation, which is not reliable. Rainfall takes time to generate runoff within a defined basin. [https://wsdot.wa.gov/publications/manuals/fulltext/M23-03/Chapter2.pdf]

## 2.2 Rainfall Intensity

Rainfall intensity was estimated using the data and methodology presented in section 1.2. For the calculated time of concentration, a precipitation intensity of 11.6 in/hr was estimated for the 1,000-year storm event for a time of concentration of 10 minutes.

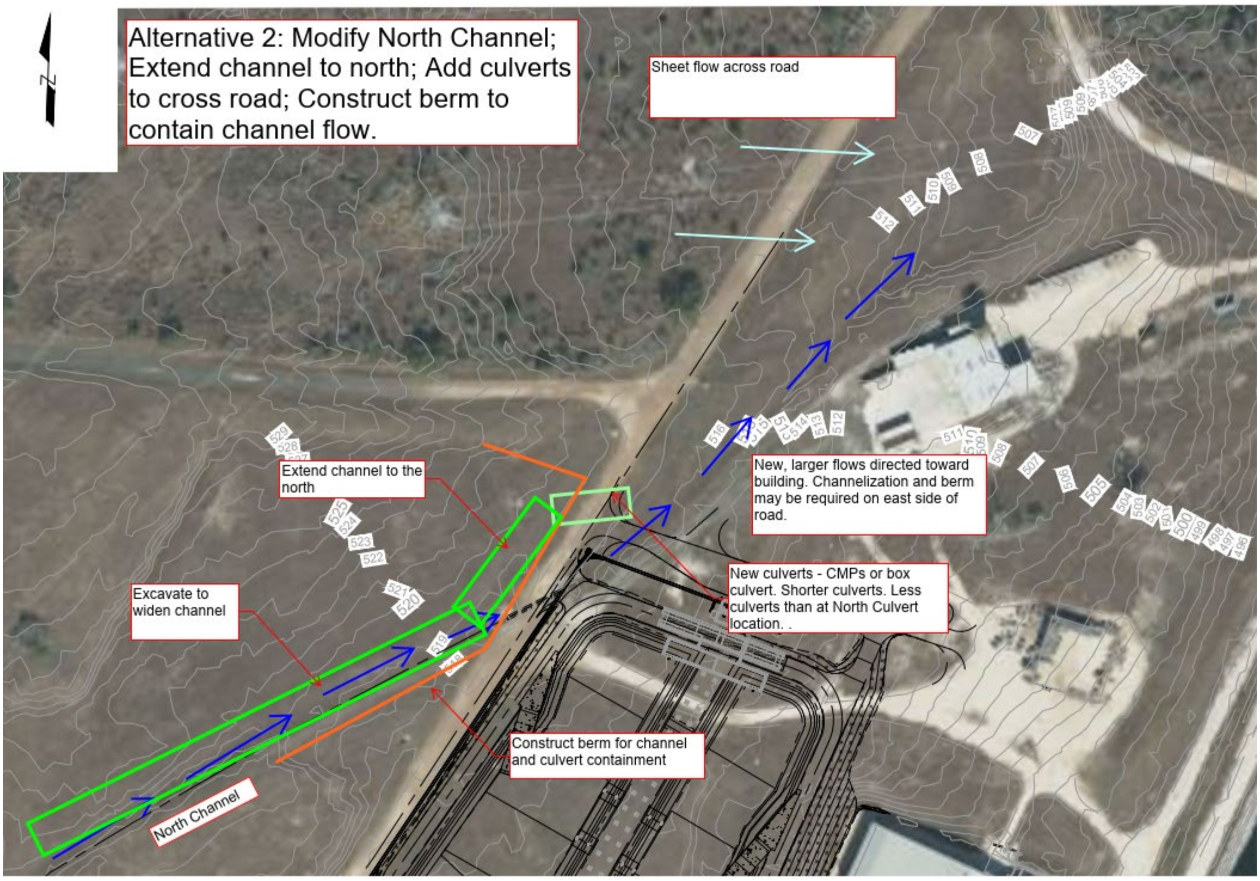
## 2.3 Peak Flow Estimation

The peak flow was estimated using the Rational Method formula, as discussed in Section 1.3. The estimated peak flows for the referenced watersheds are as follows:

- South Culvert Watershed peak flow is 15.7 cfs (Q=CIA=0.3x11.6x4.5=15.7 cfs)
- V-Ditch Watershed peak flow is 4.0 cfs (Q=CIA=0.3x11.6x1.15= 4.0 cfs)

# 3. North Culvert Watershed Channel and Culvert Sizing

The stormwater from the North Culvert Watershed is captured by the existing North Channel and conveyed to the existing North Culverts. With the construction of the Plant Drains Pond, the existing North Culverts with be removed or blocked and the culverts will be relocated north of the pond. The North Channel will be improved and extended to the new culvert location. The North Channel is designed to contain and convey the 1000-year peak flow to the new culverts. At the culvert location (end of channel on the west side of the road), the 10-year peak flow will pass through the culverts with no overtopping of the channel. For the 1,000-year storm event, flow above the 10-year peak flow will pass through a “spillway” to the north and cross over the road to the existing channel east of the road.



### 3.1 North Culvert Hydraulics Analysis

The design will consist of three 24-inch diameter corrugated metal pipe (CMP) culverts crossing from the North Channel to the east beneath the road. The inlet invert elevation will be 515.25 ft; outlet invert elevation will be 515.0 ft, and pipe length is 90 ft. The HY8 computer program was utilized to evaluate the hydraulics. The estimated headwater elevation for the 10-year peak flow is 518.99 ft, as shown on the following table.

| Total Discharge (cfs) | Culvert Discharge (cfs) | Headwater Elevation (ft) | Inlet Control Depth (ft) | Outlet Control Depth (ft) | Flow Type | Normal Depth (ft) | Critical Depth (ft) | Outlet Depth (ft) | Tailwater Depth (ft) | Outlet Velocity (ft/s) | Tailwater Velocity (ft/s) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.00 | 515.25 | 0.000 | 0.000 | 0-NF | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 5.00 | 5.00 | 516.02 | 0.655 | 0.770 | 2-M2c | 0.689 | 0.444 | 0.444 | 0.272 | 3.208 | 1.678 |
| 10.00 | 10.00 | 516.36 | 0.948 | 1.109 | 2-M2c | 1.017 | 0.636 | 0.636 | 0.408 | 3.881 | 2.146 |
| 15.00 | 15.00 | 516.64 | 1.188 | 1.388 | 2-M2c | 1.322 | 0.783 | 0.783 | 0.516 | 4.385 | 2.465 |
| 20.00 | 20.00 | 516.89 | 1.409 | 1.644 | 2-M2c | 1.717 | 0.915 | 0.915 | 0.608 | 4.757 | 2.713 |
| 25.00 | 25.00 | 517.14 | 1.620 | 1.893 | 2-M2c | 2.000 | 1.028 | 1.028 | 0.690 | 5.124 | 2.919 |
| 30.00 | 30.00 | 517.41 | 1.831 | 2.155 | 7-M2c | 2.000 | 1.126 | 1.126 | 0.764 | 5.487 | 3.096 |
| 35.00 | 35.00 | 517.75 | 2.049 | 2.497 | 7-M2c | 2.000 | 1.222 | 1.222 | 0.833 | 5.803 | 3.251 |
| 40.00 | 40.00 | 518.19 | 2.280 | 2.935 | 7-M2c | 2.000 | 1.311 | 1.311 | 0.898 | 6.107 | 3.390 |
| 45.00 | 45.00 | 518.68 | 2.532 | 3.427 | 7-M2c | 2.000 | 1.392 | 1.392 | 0.958 | 6.428 | 3.516 |
| 48.00 | 48.00 | 518.99 | 2.694 | 3.743 | 7-M2c | 2.000 | 1.437 | 1.437 | 0.993 | 6.621 | 3.587 |

Therefore, to contain the 10-year peak flow within the channel, the “spillway” elevation will be set at elevation 519.0 ft. The North Channel will be constructed with an invert elevation matching the culvert inlet invert elevation (515.25 ft) at the culvert location followed by a constant slope of 0.01 ft/ft towards the north and spillway point. The new channel segment will be excavated to have a 20 ft bottom width, 3.5:1 side slope on both sides, and constructed with a constant slope to match the existing channel bottom elevation. Downstream of the culverts, a channel will be excavated to convey flow from the new culverts into the existing channel east of the road.

The right bank of the channel will also be formed by an earthen berm. The crest elevation of that berm will be based on elevation of flow through the “spillway” using the 1,000-year peak flow. Based on the channel dimensions, the “spillway” will have a crest length of 46 ft. The “spillway” is analyzed as a broad-crested weir using FlowMaster. Assuming the culverts convey the 48 cfs (the 10-year peak flow) during the 1,000-year storm event, the resulting “spillway” flow of 65 cfs (113 cfs minus 48 cfs) results in a headwater elevation of 519.6 ft, as shown in the following figure.

| Project Description | |
|---|---|
| Solve For | Headwater Elevation |

| Input Data | |
|---|---|
| Discharge | 65.00 cfs |
| Crest Elevation | 519.00 ft |
| Tailwater Elevation | 518.00 ft |
| Crest Surface Type | Gravel |
| Crest Breadth | 30.00 ft |
| Crest Length | 46.0 ft |

| Results | |
|---|---|
| Headwater Elevation | 519.64 ft |
| Headwater Height Above Crest | 0.64 ft |
| Tailwater Height Above Crest | -1.00 ft |
| Weir Coefficient | 2.74 ft^(1/2)/s |
| Submergence Factor | 1.000 |
| Adjusted Weir Coefficient | 2.74 ft^(1/2)/s |
| Flow Area | 29.6 ft² |
| Velocity | 2.20 ft/s |
| Wetted Perimeter | 47.3 ft |
| Top Width | 46.00 ft |

To prevent overtopping of the channel during the 1,000-year storm event, the berm will be set at elevation 521.25 ft, which provides 1.65 feet of freeboard at the "spillway" location.

### 3.2 North Channel Hydraulics Analysis

The North Channel needs to have the capacity to contain and convey a 1,000-year peak flow. The existing channel slope was estimated to be 0.007 ft/ft using LiDAR topographic data. The existing channel bottom width varies. For design analysis, a bottom width of 8 ft was used. The design side slopes will be 3.5:1. The Manning's Roughness Coefficient for the grass-lined channel is estimated to be 0.035. Utilizing FlowMaster, the flow depth was estimated to be 22.5 inches (1.9 ft), as shown on the figure below.

| Project Description | |
|---|---|
| Friction Method | Manning Formula |
| Solve For | Normal Depth |

| Input Data | |
|---|---|
| Roughness Coefficient | 0.035 |
| Channel Slope | 0.007 ft/ft |
| Left Side Slope | 3.500 H:V |
| Right Side Slope | 3.500 H:V |
| Bottom Width | 8.00 ft |
| Discharge | 113.30 cfs |

| Results | |
|---|---|
| Normal Depth | 22.5 in |
| Flow Area | 27.3 ft² |
| Wetted Perimeter | 21.7 ft |
| Hydraulic Radius | 15.1 in |
| Top Width | 21.13 ft |
| Critical Depth | 17.7 in |
| Critical Slope | 0.018 ft/ft |
| Velocity | 4.15 ft/s |
| Velocity Head | 0.27 ft |
| Specific Energy | 2.14 ft |
| Froude Number | 0.643 |
| Flow Type | Subcritical |

The channel should be designed to a depth of 3.0 ft, which provides 1.1 ft of freeboard. At existing channel locations, the design depth can be achieved by increasing the existing berm height. At new channel locations, the design depth can be achieved with a combination of excavation and berm construction. The berm must be constructed with a minimum elevation of 521.25 ft to contain the "spillway" overflow depth, which may result in channel depths greater than 3 ft in the northern portions of the North Channel.

## 4. South Culvert Watershed Channel and Culvert Sizing

The stormwater from the South Culvert Watershed is directed to the existing South Culverts. With the construction of the Plant Drains Pond, the area downstream of the South Culverts will be modified and require channelization of flows at the toe of the pond embankment and installation of culverts to convey stormwater east of the pond embankment. In addition, a v-ditch will be constructed west of the road to convey stormwater to the existing south culverts.

### 4.1 V-Ditch Channel Hydraulics Analysis:

The v-ditch channel on the west side of the road needs to have the capacity to contain and convey a 1,000-year peak flow. The existing channel slope was estimated to be 0.008 ft/ft using LiDAR topographic data. The design analysis evaluated a v-ditch (zero bottom width) and bottom widths of 2 and 4 ft. The design side slopes will be 3.5:1. The Manning's Roughness Coefficient for the grass-lined channel is estimated to be 0.035. Utilizing FlowMaster, the following flow depths were estimated:

| Bottom Width (ft) | Normal Depth (inches) |
|---|---|
| 0 | 9.3 |
| 2 | 6.6 |
| 4 | 5 |

The design will utilize a v-ditch (zero bottom width) and a depth of 2 ft, which provides over 1 ft of freeboard. The FlowMaster data file for the v-ditch hydraulic analysis is shown below.

| Project Description | |
|---|---|
| Friction Method | Manning Formula |
| Solve For | Normal Depth |

| Input Data | |
|---|---|
| Roughness Coefficient | 0.035 |
| Channel Slope | 0.008 ft/ft |
| Left Side Slope | 3.500 H:V |
| Right Side Slope | 3.500 H:V |
| Bottom Width | 0.00 ft |
| Discharge | 4.00 cfs |

| Results | |
|---|---|
| Normal Depth | 9.3 in |
| Flow Area | 2.1 ft² |
| Wetted Perimeter | 5.6 ft |
| Hydraulic Radius | 4.5 in |
| Top Width | 5.42 ft |
| Critical Depth | 7.3 in |
| Critical Slope | 0.028 ft/ft |
| Velocity | 1.90 ft/s |
| Velocity Head | 0.06 ft |
| Specific Energy | 0.83 ft |
| Froude Number | 0.539 |
| Flow Type | Subcritical |

| GVF Input Data | |
|---|---|
| Downstream Depth | 0.0 in |
| Length | 0.0 ft |
| Number Of Steps | 0 |

| GVF Output Data | |
|---|---|
| Upstream Depth | 0.0 in |
| Profile Description | N/A |
| Profile Headloss | 0.00 ft |
| Downstream Velocity | 0.00 ft/s |
| Upstream Velocity | 0.00 ft/s |
| Normal Depth | 9.3 in |
| Critical Depth | 7.3 in |
| Channel Slope | 0.008 ft/ft |
| Critical Slope | 0.028 ft/ft |

## 4.2 South Culvert Hydraulic Analysis

The existing south culverts consist of one 24-inch and one 30-inch diameter CMP culverts. The 1,000-year peak flow is estimated to be 15.7 cfs (see Section 2.3). The details of the culverts are provided below.

**Culvert Data Summary - Culvert 1**

Barrel Shape: Circular

Barrel Diameter: 2.50 ft

Barrel Material: Corrugated Steel

Embedment: 0.00 in

Barrel Manning's n: 0.0150

Culvert Type: Straight

Inlet Configuration: Thin Edge Projecting

Inlet Depression: NONE

**Culvert Data Summary - Culvert 2**

Barrel Shape: Circular

Barrel Diameter: 2.00 ft

Barrel Material: Corrugated Steel

Embedment: 0.00 in

Barrel Manning's n: 0.0150

Culvert Type: Straight

Inlet Configuration: Thin Edge Projecting

Inlet Depression: NONE

**Site Data**

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 511.10 ft

Outlet Station: 50.00 ft

Outlet Elevation: 510.39 ft

Number of Barrels: 1

The intent of this analysis is to verify that no changes are required for the South Culverts. The estimated headwater elevation for the 1,000-year peak flow is 512.6 ft, as shown on the following table. This headwater elevation is below the top of the culverts. The existing culverts have sufficient capacity to convey the 1,000-year peak flow without overtopping the road.

### Summary of Culvert Flows at Crossing: South Culvert

| Headwater Elevation (ft) | Total Discharge (cfs) | Culvert 2 Discharge (cfs) | Culvert 1 Discharge (cfs) | Roadway Discharge (cfs) | Iterations |
|---|---|---|---|---|---|
| 511.10 | 0.00 | 0.00 | 0.00 | 0.00 | 0 |
| 511.52 | 1.60 | 0.75 | 0.85 | 0.00 | 5 |
| 511.71 | 3.20 | 1.50 | 1.69 | 0.00 | 6 |
| 511.86 | 4.80 | 2.24 | 2.55 | 0.00 | 5 |
| 511.98 | 6.40 | 2.98 | 3.41 | 0.00 | 4 |
| 512.10 | 8.00 | 3.73 | 4.25 | 0.00 | 4 |
| 512.20 | 9.60 | 4.45 | 5.14 | 0.00 | 4 |
| 512.30 | 11.20 | 5.16 | 6.03 | 0.00 | 3 |
| 512.39 | 12.80 | 5.87 | 6.93 | 0.00 | 3 |
| 512.48 | 14.40 | 6.57 | 7.83 | 0.00 | 3 |
| **512.56** | **15.70** | **7.14** | **8.56** | **0.00** | **3** |
| 514.54 | 48.64 | 20.01 | 28.63 | 0.00 | Overtopping |

### 4.3 South Culvert Downstream Channel Hydraulic Analysis

The channel between South Culvert and the new culvert at the toe of pond embankment was modeled using FlowMaster. The channel right bank will be formed by natural ground and left bank will be formed by the new pond embankment slope (looking downstream). The details of the channel configuration, input data, and model results are shown below.

| Input Data | |
|---|---|
| Channel Slope | 0.003 ft/ft |
| Discharge | 15.70 cfs |

**Section Definitions**

| Station (ft) | Elevation (ft) |
|---|---|
| 0+00 | 513.41 |
| 0+34 | 513.66 |
| 0+50 | 513.41 |
| 0+99 | 512.00 |
| 0+99 | 511.98 |
| 1+00 | 512.00 |
| 1+08 | 512.80 |
| 1+10 | 512.89 |
| 1+15 | 514.30 |
| 1+25 | 514.50 |

**Roughness Segment Definitions**

| Start Station | Ending Station | Roughness Coefficient |
|---|---|---|
| (0+00, 513.41) | (1+25, 514.50) | 0.035 |

| Options | |
|---|---|
| Current Roughness Weighted Method | Pavlovskii's Method |
| Open Channel Weighting Method | Pavlovskii's Method |
| Closed Channel Weighting Method | Pavlovskii's Method |

| Results | |
|---|---|
| Normal Depth | 9.1 in |
| Elevation Range | 512.0 to 514.5 ft |
| Flow Area | 12.9 ft² |
| Wetted Perimeter | 33.9 ft |
| Hydraulic Radius | 4.6 in |
| Top Width | 33.88 ft |
| Normal Depth | 9.1 in |
| Critical Depth | 6.0 in |
| Critical Slope | 0.028 ft/ft |
| Velocity | 1.22 ft/s |
| Velocity Head | 0.02 ft |

CPS Channel Design.fm8
5/15/2022

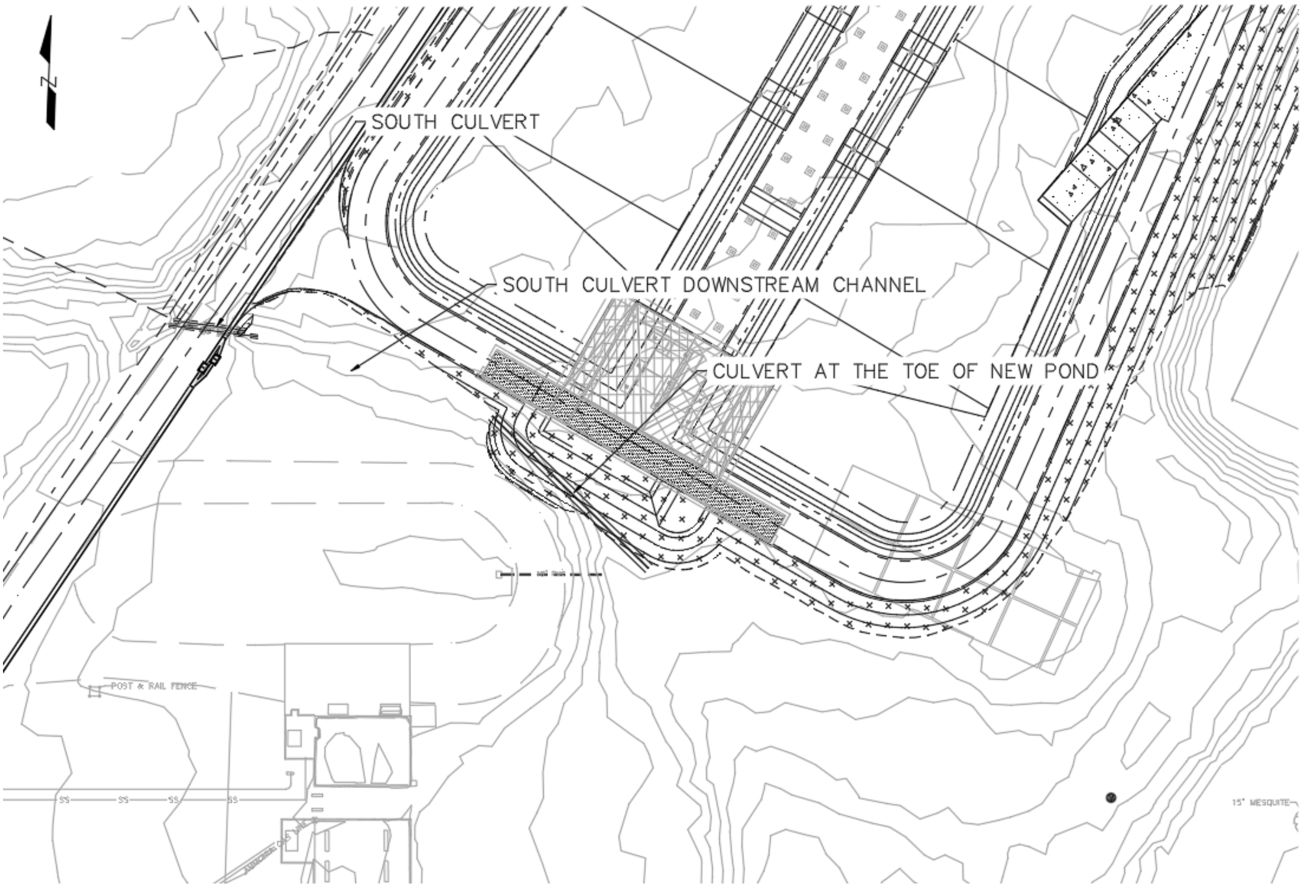
Bentley Systems, Inc. Haestad Methods Solution Center
27 Siemon Company Drive Suite 200 W
Watertown, CT 06795 USA +1-203-755-1666

FlowMaster
[10.02.00.01]
Page 1 of 2

The results show an estimated water depth of 9.1 inches. The constructed pond embankment crest will be at elevation of 514.5 ft. The existing roadway to the south is at elevation 513.0 ft. The highest channel invert elevation is 510.4 ft (the same elevation as the outlet of the South Culverts). Therefore, the maximum water surface elevation in the channel during the 1,000-year storm event will be 511.15 ft, which results in approximately 3.2 ft of freeboard to the pond embankment crest.

### 4.4 Pond Culvert Hydraulic Analysis

The culverts to be installed at the toe of the pond embankment will consist of 2 24-inch CMPs to drain the stormwater from the South Culverts to the east of the pond (see figure below). The 1,000-year peak flow is estimated to be 15.7 cfs (see Section 2.3). The details of the culverts are provided below.




**Site Data**

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 510.00 ft

Outlet Station: 120.00 ft

Outlet Elevation: 509.00 ft

Number of Barrels: 2

**Culvert Data Summary**

Barrel Shape: Circular

Barrel Diameter: 2.00 ft

Barrel Material: Corrugated Steel

Embedment: 0.00 in

Barrel Manning's n: 0.0240

Culvert Type: Straight

Inlet Configuration: Thin Edge Projecting

Inlet Depression: NONE

The analysis shows that the headwater elevation for the new culverts is 511.67 ft, as shown on the following table. The headwater elevation is below the top of the culverts and below the pond embankment crest (elevation 514.5 ft), and below the adjacent roadway (elevation 513 ft).

| Total Discharge (cfs) | Culvert Discharge (cfs) | Headwater Elevation (ft) | Inlet Control Depth (ft) | Outlet Control Depth (ft) | Flow Type | Normal Depth (ft) | Critical Depth (ft) | Outlet Depth (ft) | Tailwater Depth (ft) | Outlet Velocity (ft/s) | Tailwater Velocity (ft/s) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.00 | 510.00 | 0.000 | 0.000 | 0-NF | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 1.60 | 1.60 | 510.49 | 0.442 | 0.488 | 2-M2c | 0.361 | 0.302 | 0.302 | 0.223 | 2.683 | 0.665 |
| 3.20 | 3.20 | 510.70 | 0.638 | 0.700 | 2-M2c | 0.506 | 0.435 | 0.435 | 0.336 | 3.174 | 0.853 |
| 4.80 | 4.80 | 510.87 | 0.794 | 0.867 | 2-M2c | 0.624 | 0.534 | 0.534 | 0.425 | 3.558 | 0.984 |
| 6.40 | 6.40 | 511.01 | 0.923 | 1.011 | 2-M2c | 0.731 | 0.622 | 0.622 | 0.502 | 3.838 | 1.085 |
| 8.00 | 8.00 | 511.14 | 1.043 | 1.141 | 2-M2c | 0.823 | 0.697 | 0.697 | 0.570 | 4.103 | 1.169 |
| 9.60 | 9.60 | 511.26 | 1.155 | 1.262 | 2-M2c | 0.914 | 0.765 | 0.765 | 0.633 | 4.340 | 1.242 |
| 11.20 | 11.20 | 511.38 | 1.264 | 1.375 | 2-M2c | 0.998 | 0.833 | 0.833 | 0.690 | 4.520 | 1.306 |
| 12.80 | 12.80 | 511.48 | 1.369 | 1.484 | 2-M2c | 1.083 | 0.896 | 0.896 | 0.745 | 4.698 | 1.364 |
| 14.40 | 14.40 | 511.59 | 1.471 | 1.589 | 2-M2c | 1.166 | 0.953 | 0.953 | 0.795 | 4.875 | 1.416 |
| 15.70 | 15.70 | 511.67 | 1.553 | 1.673 | 2-M2c | 1.234 | 0.997 | 0.997 | 0.835 | 5.017 | 1.455 |

*********************************************************************************

Straight Culvert

Inlet Elevation (invert): 510.00 ft,    Outlet Elevation (invert): 509.00 ft

Culvert Length: 120.00 ft,    Culvert Slope: 0.0083

*********************************************************************************

aecom.com

aecom.com