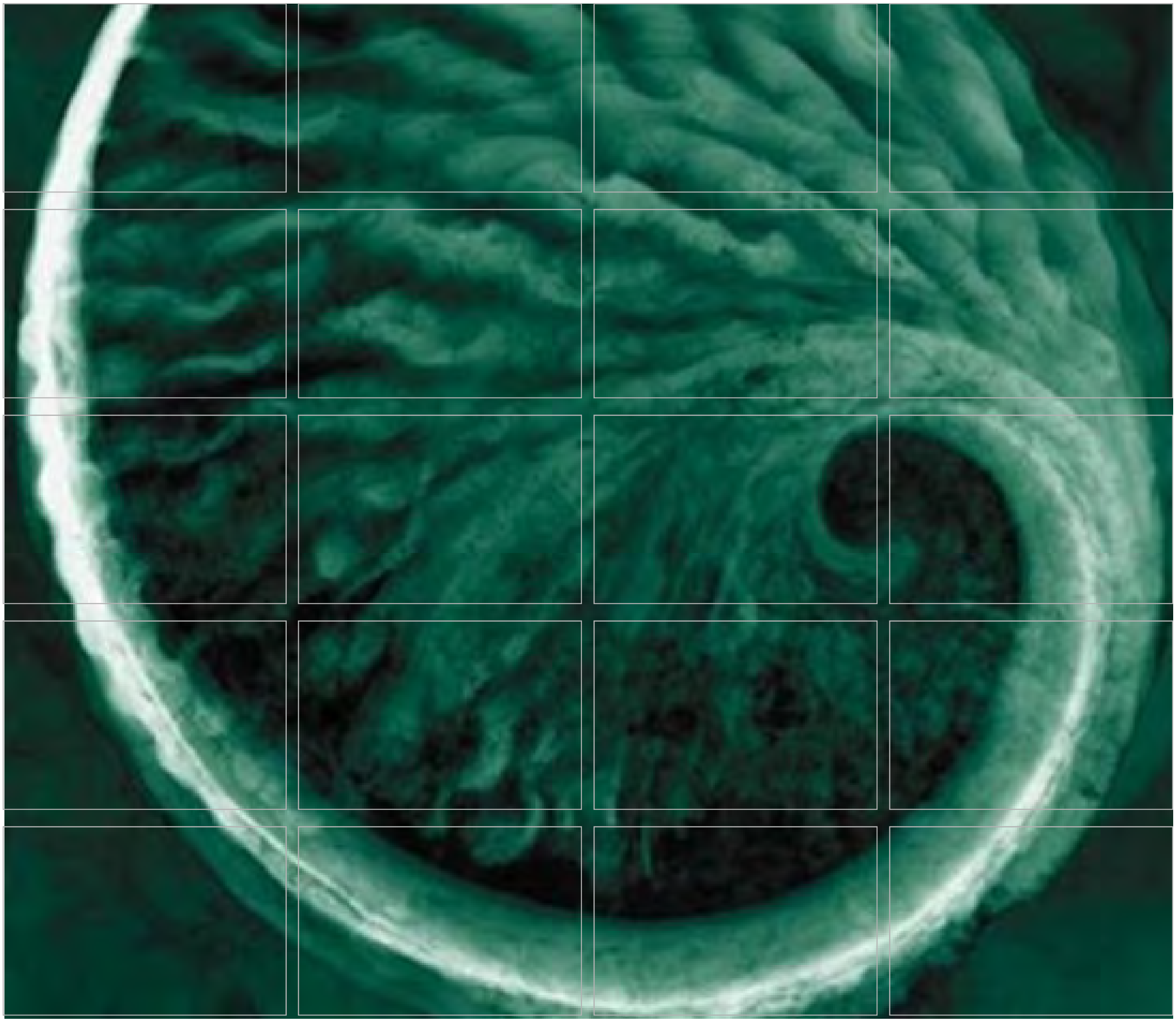

Inflow Design Flood Control System Plan

San Miguel Electric Cooperative, Inc.

Atascosa County, Texas

October 18, 2016

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San Miguel Electric Cooperative, Inc.

Inflow Design Flood Control System Plan

Christine, Texas

October 18, 2016

Project No. 0303548
Christine, Texas

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APPENDICES

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1.0 PURPOSE AND OBJECTIVES

San Miguel Electric Cooperative, Inc. (San Miguel) is an electric utility that owns and operates a 440-MW mine-mouth, lignite-fired electric power generating plant (the plant) and associated mining facilities in Atascosa County, Texas. The plant generates (CCR) that are subject to regulation under Title 40, Code of Federal Regulations, Part 257 (40 CFR §257)(the CCR Rule). San Miguel operates two CCR surface impoundments at the plant:

(1) the Ash Water Transport Ponds (Ash Ponds), and

(2) the Equalization Pond (EP).

The CCR Rule requires that each CCR surface impoundment control stormwater runoff from a design storm event. The return frequency of the design storm event depends on the hazard classification of the CCR surface impoundment.

The CCR Rule requires that San Miguel prepare an initial Inflow Design Flood (IDF) Control System Plan for each CCR surface impoundment no later than October 17, 2016 in accordance with 40 §CFR 257.82(c)(3)(i). The CCR Rule requires that San Miguel review and update the IDF Control System Plan for each surface impoundment at five year intervals following completion of the initial IDF Control System Plan in accordance with 40 §CFR 257.82(c)(4). San Miguel must also amend the plan in the future "whenever there is a change in conditions that would substantially affect the written plan…" in accordance with 40 §CFR 257.82(c)(2).

The CCR Rule requires each owner and operator of a CCR Surface Impoundment to "…construct, operate, and maintain an inflow design flood control system…". The inflow design flood control system must adequately manage flow into and out of the CCR unit resulting from the inflow design flood required for the corresponding CCR surface impoundment hazard potential classification designated in 40 §CFR Part 257.82(a)(3) and as determined in accordance with 40 §CFR Part 257.73(a)(2).

This document includes the initial IDF Control System Plan for the Ash Ponds and for the EP at the San Miguel plant.

2.0 *ASH PONDS INFLOW DESIGN FLOOD CONTROL SYSTEM PLAN*

This section is the initial IDF Control System Plan for the Ash Ponds, including:

- unit description;
- process flow rates;
- hazard potential classification;
- design storm precipitation;
- stormwater runoff coefficient analysis;
- stage-storage analysis;
- design storm routing analysis; and
- IDF control analysis.

2.1 *ASH PONDS UNIT DESCRIPTION*

The Ash Ponds are two connected CCR surface impoundments constructed by San Miguel in 1977 as part of the original plant construction. The Ash Ponds are located generally south of the plant and west of the EP; see Figure 1.

The adjoining Ash Ponds are separated by earthen dikes and hydraulic gates. As shown on San Miguel drawings (T & G, 1977a), each of the two Ash Ponds is approximately 2450 feet long and 240 feet wide at the dike crest interior top of bank. The Ash Ponds have a common dike crest elevation and are shown to be approximately 20 feet deep from the dike crest to the pond bottom. Based on those dimensions, the total area inside the two Ash Ponds is approximately 27.0 acres. The total area drained to the Ash Ponds, including the interior and the dike crest areas, is approximately 32.0 acres.

The Ash Ponds receive and store bottom ash transport water overflow from hydrobins used to dewater bottom ash, economizer ash, and pyrites CCR produced by the plant. Roughly the same flow rate and volume of water is pumped from the Ash Ponds to the plant ash transport water system as the plant ash transport water system returns to the Ash Ponds.

The Ash Ponds also have the capability to receive pumped stormwater from the plant floor drainage (approximately 7 acres). Some areas of the plant floor are pumped to the flue gas desulfurization (FGD) waste treatment system thickeners but may instead be pumped to the Ash Ponds. For the purposes of this IDF Control System Plan, it is assumed that all stormwater drainage from the plant floor is pumped to the Ash Ponds during the design storm duration.

San Miguel records show that the Ash Ponds received CCR prior to, on, and subsequent to October 14, 2015. Consequently, in accordance with 40 §CFR 257.53, the Ash Ponds are classified as “existing” CCR surface impoundments.

San Miguel records indicate that portions of the Ash Ponds were constructed above the elevation of the adjoining exterior ground surface. Consequently, in accordance with 40 §CFR 257.53, the Ash Ponds are not classified as an "incised" CCR surface impoundment.

2.2 *ASH PONDS PROCESS FLOW RATES*

According to the San Miguel facility water balance (San Miguel Water Balance, 1982) and discussions with San Miguel personnel, the Ash Ponds can receive process flows from the following sources:

- Bottom ash, pyrites, and economizer ash transport water;
- Lignite Yard Retention Pond (also known as the Coal Pile Runoff Pond);
- Water Well Storage Pond (also known as the Raw Water Pond);
- Cooling tower blowdown water;
- Plant floor drainage;
- Direct precipitation;
- Stormwater runoff;
- Boiler feedwater treatment wastewater; and
- Equalization Pond.

Under the plant's Texas Pollutant Discharge Elimination System (TPDES) Permit No. WQ0002601000 for the facility, wastewater from the two Ash Ponds is not authorized for discharge; rather, the plant reuses ash transport water from the ponds to the extent practical and relies on evaporation to manage the water level in the Ash Ponds. For the purposes of this IDF Control System Plan, it is assumed the only net contributions from the process flows to the Ash Ponds during a storm event are plant floor drainage, direct precipitation, and stormwater runoff.

The Ash Ponds were not constructed with an emergency spillway to control overflow from the Ash Ponds. Therefore, in order to prevent uncontrolled flow over the Ash Pond perimeter dikes, the volume of water contained in the Ash Ponds above the Normal Dry Weather (NDW) freeboard needs to be equal or less than the maximum volume of plant floor drainage, direct precipitation, and stormwater runoff received, less a top surface freeboard of at least six inches to account for uncertainty in dike crest elevation and wave action when filled by water accumulated during the IDF storm event.

2.3 ASH PONDS HAZARD POTENTIAL CLASSIFICATION

CCR surface impoundment hazard potential is classified in accordance with 40 §CFR 257.53. Hazard potential classification of the Ash Ponds is based on San Miguel assessment of the potential for loss of life, economic loss, environmental damage, disruption of lifeline facilities, or impact other concerns in accordance with 40 §CFR 257.73(a)(2) and 257.73(f). For this Inflow Design Flood Control System Plan, the Ash Ponds were evaluated as having "Low" hazard potential in accordance with the hazard potential classification data in "Hydrologic and Hydraulics Report for Coal Combustion Waste Impoundments" (HDR, 2015) prepared in response to a recommendation in a letter from the Environmental Protection Agency (EPA) to San Miguel asserting that the hydrologic and hydraulic capacity of the impoundments should be evaluated (EPA, 2014). The EPA recommendation was made in response to conclusions in "Assessment of Dam Safety of Coal Combustion Surface Impoundments, Final Report", an assessment of the Ash Ponds and EP prepared by CDM Smith under contract with the EPA (CDM Smith, 2014).

2.4 ASH PONDS INFLOW DESIGN FLOOD PRECIPITATION

Design storm precipitation data for a range of 100-year storm durations were obtained from Technical Paper No. 40 and Technical Paper No. 49 published by the U.S. Weather Bureau. Precipitation for the 3-day IDF design storm was interpolated from the data obtained from Technical Paper No. 40 and Technical Paper No. 49 by generating a logarithmic line of best fit as shown on Figure 2. Precipitation data used for this analysis are shown in Table 1.

As discharge from the Ash Ponds is not authorized under the plant's TPDES wastewater discharge permit, and as the Ash Ponds were not constructed with an emergency spillway to control overflow from the Ash Ponds, the critical design storm duration to evaluate the hydraulic capacity of Ash Ponds is the IDF design storm that produces the maximum total precipitation. The applicable critical IDF design storm for the Ash Ponds is the 3-day 100-year storm. Consistent with Texas Commission on Environmental Quality (TCEQ) guidance in "Hydrologic and Hydraulic Guidelines for Dams in Texas, Dam Safety Program" (TCEQ, 2007), IDF design storm durations longer than three days do not need to be evaluated.

The Federal Emergency Management Administration (FEMA) Flood Insurance Rate Maps (FIRM) show part of the Ash Ponds footprint within the 100-year flood plain boundary of the nearby Caballos Creek (FEMA, 2010). However, the FIRM flood plain delineation appears to be inaccurate in the area adjacent to the Ash Ponds based on flood plain elevation, FIRM flood boundary, ground surface elevation shown on the corresponding USGS topographic map, and the ground contours of the constructed Ash Ponds berms.

The observed 100-year flood plain contours in the FIRM map adjacent to the Ash Ponds are approximately 297 feet North American Vertical Datum, 1988 (NAVD

88). San Miguel drawings of the Ash Ponds show the elevation of the Ash Ponds dike crests to be 315 feet, unknown datum (T&G, 1977a). The Ash Ponds drawing elevation datum is otherwise consistent with USGS topographic contours that are NAVD 88 elevation datum. Therefore, the Ash Ponds dike crests are approximately 18 feet above the 100-yr flood plain.

Consequently, the Ash Ponds dike crests are sufficiently above the 100-year floodplain to prevent inflow of flood waters from the adjacent Caballos Creek; however, a hydraulic study would be required to confirm that conclusion. A Letter of Map Revision (LOMR) application to FEMA with results of the hydraulic study would be required to update the FIRM flood plain boundary and elevation. A hydraulic study and LOMR were not included in this scope of work.

2.5 ASH PONDS STORMWATER RUNOFF COEFFICIENT ANALYSIS

The volume of stormwater drained to the Ash Ponds during the design storm was calculated using the rational formula:

$$Q = C \times Pt \times A$$

Where:

Q = flow rate, cubic feet per second
C = stormwater runoff coefficient
Pt = precipitation for the corresponding design storm duration
A = drainage area

While the total drainage area is constant for any water stage in the Ash Ponds, the part of the total area that is subject to direct precipitation (*i.e.*, the pond water surface) and the area of exposed dike crest and side slopes vary with pond stage (*i.e.,* the height of the water in the ponds relative to the dike crests). As a result, the value of the area-weighted value of the stormwater runoff coefficient (the composite value of C, or Cc) varies with Ash Ponds water stage.

San Miguel data, as well as the results of calculations of the composite stormwater runoff coefficient are shown in Table 2. The values of the composite stormwater runoff coefficient as a function of NDW freeboard are shown on Figure 3. Using these data, a second-order polynomial formula was developed (as a best fit approximation of that relationship) for use in the Ash Ponds design stormwater routing described below.

2.6 ASH PONDS STAGE-STORAGE ANALYSIS

The Ash Ponds were not constructed with an emergency spillway to control overflow from the Ash Ponds at a stage above the minimum 6-inch freeboard below the perimeter dike crest. Therefore, in order to prevent uncontrolled flow over the Ash Pond perimeter dikes, the volume of water contained in the Ash

Ponds above the NDW freeboard needs to be equal or less than the maximum volume of process water and stormwater received and less a top surface freeboard of at least six inches when filled by water accumulated during the design storm.

San Miguel data used to model the variation of the capacity above a range of NDW water surface and below the 6-inch minimum freeboard are shown in Table 3. The Ash Ponds dike crest was assumed to be a single and constant overflow elevation. A reserve 6-inch minimum freeboard was assumed to account for uncertainty in dike crest elevation and wave action.

A plot of the stage-storage relationship is shown in Figure 4. Using these data, a second-order polynomial formula, also shown on Figure 4, was developed (as a best fit approximation of that relationship) for use in the design stormwater routing described below.

2.7 ASH PONDS DESIGN STORM ROUTING ANALYSIS

The process and stormwater runoff flow volumes of each inflow stream were calculated for a range of design storm durations assuming a NDW freeboard level at the beginning of the storm. Those results were used to calculate the total volume of the stormwater to be contained in the Ash Ponds for the corresponding duration.

The volume of water contained above the NDW freeboard and the minimum 6-inch freeboard was calculated using the relationship obtained in the stage-storage relationship. The trial Ash Ponds NDW freeboard level was adjusted until the net volume of process water and stormwater equaled the volume of water contained between the NDW freeboard and the 6-inch minimum freeboard for the corresponding design storm duration.

A summary of the Ash Ponds 100-year design storm routing data and calculations are shown in Table 4. The maximum Ash Ponds NDW freeboard required below the dike crest to contain the net process and 100-year design storm streams with no CCR solids in the storage space that would otherwise displace accumulated water is 2.0 feet[1]. A summary of values used in the Ash Pond design storm routing analysis is shown in Table A-1, Appendix A.

2.8 ASH PONDS INFLOW DESIGN FLOOD CONTROL ANALYSIS

Based on San Miguel data and published precipitation data, the Ash Ponds can contain inflow from process streams and stormwater runoff from the 100-year IDF design storm with a 6-inch freeboard below the dike crest if the normal dry

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[1] If solids exist above the recommended freeboard they must be removed so as to not displace stormwater during a storm event and thus reduce the capacity of the impoundments.

weather water surface and the top surface of CCR solids in the Ash Ponds are maintained at or below a level 2.0 feet below the Ash Ponds dike crest.

3.0 EQUALIZATION POND INFLOW DESIGN FLOOD CONTROL SYSTEM PLAN

This section is the initial IDF Control System Plan for the EP, including:

- unit description;
- process flow rates;
- hazard potential classification;
- design storm precipitation;
- stormwater runoff coefficient analysis;
- stage-storage analysis;
- design storm routing analysis; and
- IDF control analysis.

3.1 EQUALIZATION POND UNIT DESCRIPTION

The EP is a CCR surface impoundment constructed by San Miguel in 1977 as part of the original plant construction. The EP is located generally southeast of the plant and east of the Ash Ponds; see Figure 1.

The northern part of the EP western dike adjoins the Water Well Storage Pond. The southern part of the EP western dike adjoins a plant electrical substation. The EP northern, eastern, and southern dikes adjoin undeveloped off-site property and Caballos Creek (a wet weather creek).

As shown on San Miguel drawings (T&G, 1977a), the EP is approximately 20 feet deep from the crest of the dike to the bottom and total area inside the EP is approximately 23.7 acres. The total area drained to the EP, including the interior and the dike crest areas, is approximately 28.5 acres.

San Miguel records show that the EP received CCR prior to, on, and subsequent to October 14, 2015. Consequently, in accordance with 40 §CFR 257.53, the EP is classified as an "existing" CCR surface impoundment.

Elevations of parts of the EP dike crests are above the elevation of the adjoining exterior ground surface. Consequently, in accordance with 40 §CFR 257.53, the EP is not classified as an "incised" CCR surface impoundment.

3.2 EQUALIZATION POND PROCESS FLOW RATES

According to the San Miguel water balance (San Miguel Water Balance, 1982) and discussions with San Miguel personnel, the EP can receive process flows from the following sources:

- Flue Gas Desulfurization Scrubber Waste Treatment System waste water;

- Sanitary treated waste water; and
- Ash Ponds water.

Under TPDES Permit No. WQ0002601000 for the facility, wastewater from the EP is not authorized for discharge; rather, the plant relies on evaporation to manage the water level in the EP. For the purposes of this IDF Control System Plan, it is assumed the only net contributions from the above process flows to the EP during a storm event are direct precipitation and stormwater runoff.

The EP was not constructed with an emergency spillway to control overflow from the EP. Therefore, in order to prevent uncontrolled flow over the EP perimeter dikes, the volume of water contained in the EP above the NDW freeboard needs to be equal or less than the maximum volume of direct precipitation, and stormwater runoff received, less a top surface freeboard of at least six inches to account for uncertainty in dike crest elevation and wave action when filled by water accumulated during the IDF design storm.

3.3 EQUALIZATION POND HAZARD POTENTIAL CLASSIFICATION

CCR surface impoundment hazard potential is classified in 40 §CFR 257.53. Hazard potential classification of the EP is based on San Miguel assessment of the potential for loss of life, economic loss, environmental damage, disruption of lifeline facilities, or impact other concerns in accordance with 40 §CFR 257.73(a)(2) and 257.73(f). For this Inflow Design Flood Control System Plan, the EP was evaluated as having “Low” hazard potential in accordance with the hazard potential classification data in “Hydrologic and Hydraulics Report for Coal Combustion Waste Impoundments” (HDR, 2015) prepared in response to a recommendation in a letter from the EPA to San Miguel asserting that the hydrologic and hydraulic capacity of the impoundments should be evaluated (EPA, 2014). The EPA recommendation was made in response to conclusions in “Assessment of Dam Safety of Coal Combustion Surface Impoundments, Final Report”, an assessment of the Ash Ponds and EP prepared by CDM Smith under contract with the EPA (CDM Smith, 2014).

3.4 EQUALIZATION POND DESIGN STORM PRECIPITATION

Design storm precipitation data for a range of 100-year storm durations were obtained from Technical Paper No. 40 and Technical Paper No. 49 published by the U.S. Weather Bureau. Precipitation for the 3-day IDF design storm was interpolated from the data obtained from Technical Paper No. 40 and Technical Paper No. 49 by generating a logarithmic line of best fit as shown on Figure 2. Precipitation data used for this analysis are shown in Table 1.

As discharge from the EP is not authorized under the plant’s TPDES wastewater discharge permit, and as the EP was not constructed with an emergency spillway to control overflow from the EP, the critical design storm duration to evaluate the hydraulic capacity of EP is the IDF design storm that produces the maximum total precipitation. The applicable critical IDF design storm for the EP is the 3-day 100-year storm. Consistent with TCEQ guidance in "Hydrologic and Hydraulic Guidelines for Dams in Texas, Dam Safety Program" (TCEQ, 2007), IDF design storm durations longer than three days do not need to be evaluated.

The FEMA FIRM show part of the EP footprint within the 100-year flood plain boundary of the nearby Caballos Creek (FEMA, 2010). However, the FIRM flood plain delineation appears to be inaccurate in the area adjacent to the EP based on the flood plain elevation, the FIRM flood plain boundary, the ground surface elevation shown on the corresponding USGS topographic contour map, and the ground contours of the constructed EP berms.

The observed 100-year flood plain contours in the FIRM map adjacent to the EP appear to be approximately 288 feet North American Vertical Datum, 1988 (NAVD 88). San Miguel drawings of the EP show the elevation of the dike crests to be 295 feet, unknown datum (T&G, 1977a). The EP drawing elevation datum is otherwise consistent with USGS topographic contours that are NAVD 88 elevation datum. Therefore, the EP dike crests are approximately 7 feet above the 100-yr flood plain.

Consequently, the EP dike crests are sufficiently above the 100-year floodplain to prevent inflow of flood waters from the adjacent Caballos Creek; however, a hydraulic study would be required to confirm that conclusion. An LOMR application to FEMA with results of the hydraulic study would be required to update the FIRM flood plain boundary and elevation. A hydraulic study and LOMR were not included in this scope of work.

3.5 EQUALIZATION POND STORMWATER RUNOFF COEFFICIENT ANALYSIS

The volume of stormwater drained to the EP during the design storm was calculated using the rational formula:

$$Q = C \times Pt \times A$$

Where:

Q = flow rate, cubic feet per second
C = stormwater runoff coefficient
Pt = precipitation for the corresponding design storm duration
A = drainage area

While the total drainage area is constant for any water stage in the EP, the part of the total area that is subject to direct precipitation (*i.e.*, the pond water surface) and the area of exposed dike crest and side slopes vary with pond stage (*i.e.*, the height of the water in the ponds relative to the dike crests). As a result, the value of the area-weighted value of the stormwater runoff coefficient (the composite value of C, or Cc) varies with the EP water stage.

San Miguel data used and the results of calculations of the value of the composite stormwater runoff coefficient are shown in Table 5. The values of the composite stormwater runoff coefficient as a function of NDW freeboard are shown on Figure 5. Using these data, a second-order polynomial formula was developed (as a best fit approximation of that relationship) for use in the EP design stormwater routing described below.

3.6 EQUALIZATION POND STAGE-STORAGE ANALYSIS

The EP was not constructed with an emergency spillway to control overflow of the EP at a stage above the minimum 6-inch freeboard below the dike crest. Therefore, in order to prevent uncontrolled flow over the EP dike crest, the volume of water contained in the EP above the NDW freeboard needs to be equal or less than the maximum volume of process water and stormwater received less a top surface freeboard of at least six inches when filled by water accumulated during the design storm.

San Miguel data used to model the variation of the capacity above a range of NDW water surface and below the 6-inch minimum freeboard are shown in Table 6. The EP dike crest was assumed to be a single and constant overflow elevation. A reserve 6-inch minimum freeboard was assumed to account for uncertainty in dike crest elevation and wave action.

A plot of the stage-storage relationship is shown in Figure 6. Using these data, a second-order polynomial formula, also shown on Figure 6, was developed (as a best fit approximation of that relationship) for use in the EP design stormwater routing described below.

3.7 EQUALIZATION POND DESIGN STORM ROUTING ANALYSIS

The volume of EP stormwater inflow was calculated for a range of design storm durations assuming a trial NDW freeboard level at the beginning of the storm. Those results were used to calculate the total volume of the stormwater to be contained in the EP for the corresponding duration.

The volume of water contained above the NDW freeboard and the minimum 6-inch freeboard was calculated using the relationship obtained in the stage-storage relationship. The trial EP NDW freeboard level was adjusted until the net volume of process water and stormwater equaled the volume of water contained between the NDW freeboard and the 6-inch minimum freeboard for the corresponding design storm duration.

A summary of the EP 100-year design storm routing data and calculations are shown in Table 7. The maximum EP NDW freeboard required below the dike crest to contain the net process and 100-year design storm streams with no CCR solids in the storage space that would otherwise displace accumulated water is

1.7 feet[2]. A summary of values used in the EP design storm routing analysis is shown in Table A-2, Appendix A.

3.8 EQUALIZATION POND INFLOW DESIGN FLOOD CONTROL ANALYSIS

Based on San Miguel data and published precipitation data, the EP can contain the net inflow from process streams and stormwater runoff from the 100-year IDF design storm with a 6-inch freeboard below the dike crest if the normal dry weather water surface and the top surface of CCR solids in the EP are maintained at or below a level 1.7 feet below the EP dike crest.

[2] If the EP contains solids above the 1.7 foot recommended freeboard level they must be removed so as to not displace stormwater during a storm event and thus reduce the capacity of the impoundment.

4.0 *PROFESSIONAL ENGINEER'S CERTIFICATION*

40 CFR Part 257.82 requires that this IDF Control System Plan meet the requirement of the CCR Rule. In addition, a professional engineer must certify that this initial plan and any amendments to the IDF Control System Plan meet requirements of the CCR Rule. Certification for this initial IDF Control System Plan is provided below.

I hereby certify that I have reviewed the hydrologic and hydraulic capacity and inflow design flood control systems for the Ash Ponds and Equalization Pond CCR surface impoundments at the San Miguel Electric Cooperative, Inc. plant located in Atascosa County, Texas, and attest that the Inflow Design Flood Control System Plan for each of those CCR surface impoundments has been prepared in accordance with 40 §CFR Part 257.82.

Seal:

STATE OF TEXAS
EDWARD DOYON MAIN
65502
REGISTERED
PROFESSIONAL ENGINEER

E. Doyon Main, P.E.
Printed Name of Licensed Professional Engineer

Signature of Licensed Professional Engineer

10/18/2016
Date

5.0 REFERENCES

Information used for this Inflow Design Flood Control System Plan for the Ash Ponds and EP are from San Miguel and public sources listed below. San Miguel source documents are included in Appendix B.

5.1 SAN MIGUEL DOCUMENTS

The following San Miguel documents were used as sources of information used for this Inflow Design Flood Control System Plan for the Ash Ponds and EP.

T&G, 1977a — Sludge Disposal Basin, 69 kV Substation & Temp. Parking Area, San Miguel Plant Unit No. 1, Drawing No. C-12, Rev. 0, Tippet & Gee, Inc., April 1, 1977, revised April 5, 1977.

T&G, 1977b — Site Plan Section No. 8, San Miguel Plant Unit No. 1, Drawing No. 1-C-37, Rev. 0, Tippet & Gee, Inc., April 1, 1977, revised August 18, 1977.

T&G, 1980a — Site Plan and Vicinity Map, San Miguel Plant Unit No. 1, Drawing No, 1-C-1C Rev 3, Tippet & Gee, Inc., April 1, 1977, revised April 14, 1980.

T&G, 1980b — Site Plan Section No. 6, San Miguel Plant Unit No. 1, Drawing No. 1-C-35, Rev 16, Tippet & Gee, Inc., April 1, 1977, revised August 6, 1980.

T&G, 1980c — Site Plan Section No. 11, San Miguel Plant Unit No. 1, Drawing No. 1-C-40, Rev. 6, Tippet & Gee, Inc., April 1, 1977, revised June 13, 1980.

T&G, 1980d — Site Plan Section No. 11, San Miguel Plant Unit No. 1, Drawing No. 1-C-41, Rev. 4, Tippet & Gee, Inc., April 1, 1977, revised April 14, 1980.

T&G, 1980e — Site Plan Section No. 13, San Miguel Plant Unit No. 1, Drawing No. 1-C-42, Rev. 2, Tippet & Gee, Inc., April 1, 1977, revised April 14, 1980.

T&G, 1981 — Site Plan Section No. 4, San Miguel Plant Unit No. 1, Drawing No. 1-C-33, Rev. 7, Tippet & Gee, Inc., April 1, 1977, revised May 13, 1981.

San Miguel, 1982 — Facility Water Balance, San Miguel Electric Cooperative, Inc., ca. 1982.

EPA, 2014 — Request for Action Plan regarding San Miguel Cooperative Inc.'s San Miguel Electric Plant, United States Environmental Protection Agency, May 2, 2014.

HDR, 2015 — Hydrologic and Hydraulics Report for Coal Combustion Waste Impoundments, San Miguel Electric Cooperative, Inc., HDR Engineering, Inc., March, 2015.

5.2 PUBLIC SOURCE DOCUMENTS

The following public source documents were used as sources of information used for this Inflow Design Flood Control System Plan for the Ash Ponds and EP.

USWB, 1961 — Technical Paper No. 40, Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years, U.S. Department of Commerce, Weather Bureau, 1961, Rev. January 1963.

USWB, 1964 — Technical Paper No. 49, Two- to Ten-Day Precipitation for Return Periods of 2 to 100 Years in the Contiguous United States, U.S. Department of Commerce, Weather Bureau, 1964.

TCEQ, 2007 — Hydrologic and Hydraulic Guidelines for Dams in Texas, Dam Safety Program, Texas Commission on Environmental Quality, Guidance No. GI 364, January 2007.

FEMA, 2010 — Flood Insurance Rate Map, Atascosa County, Texas and Incorporated Areas, Federal Emergency Management Agency, Number 480014, Panel 0675C, November 4, 2010

CDM Smith, 2014 — Assessment of Dam Safety of Coal Combustion Surface Impoundments, Final Report, CDM Smith, Project No. 93083.1801.044.SIT.SANMG, March 2014, Rev. April 2014.

Tables

October 2016
Project No. 0303548

Environmental Resources Management
206 East 9th Street, Suite 1700
Austin, Texas 78701
(512) 459-4700

TABLE 1

100-Year Design Precipitation

Inflow Design Flow Control Plan for CCR Surface Impoundments
San Miguel Electric Cooperative, Inc., Atascosa County, Texas

Storm Duration		Total Precipitation	Source[1,2]
hours	days	inches	
12	0.5	8.7	Chart 42, TP-40, p. 49
24	1	10.2	Chart 49, TP-40, p. 56
	2	11.7	Figure 17, TP-49, p. 11
	3	12.8	Interpolated, see Figure 2
	4	13.5	Figure 23, TP-49, p. 17
	7	15.0	Figure 29, TP-49, p. 22

NOTES:

1. "TP-40" data are from the indicated part of "Technical Paper No. 40", U.S. Weather Bureau, 1961, Rev. January 1963.
2. "TP-49" data are from the indicated part of "Technical Paper No. 49", U.S. Weather Bureau, 1964.

TABLE 2

Ash Ponds Stage-Composite Runoff Coefficient

Inflow Design Flow Control Plan for CCR Surface Impoundments
San Miguel Electric Cooperative, Inc., Atascosa, County, Texas

Depth Below Dike Crest vf	Total Drainage Area (At)[1] acres	Pond Length At Depth Below Dike Crest (Lw)[2,3] lf	Pond Width At Depth Below Dike Crest (Ww)[2,3] lf	Water Surface Area At Depth Below Dike Crest (Aw)[4] acres	Open Water Storm Water Runoff Coeff. (Cw)[5] in/in	Slope Area At W.S. Depth Below Dike Crest (As)[6] acres	Slope Area Storm Water Runoff Coeff. (Cs)[5] in/in	Composite Storm Water Runoff Coeff. (Cc)[7] in/in
0	32.0	2,450	240	27.0	1.00	5.0	0.70	0.95
0.5		2,448	238	26.7		5.3		0.95
1		2,445	235	26.4		5.6		0.95
2		2,440	230	25.8		6.2		0.94
3		2,435	225	25.2		6.9		0.94
4		2,430	220	24.5		7.5		0.93

ABBREVIATIONS AND ACRONYMS:

cf	cubic feet
Coeff.	coefficient
Dwg.	drawing
in/in	inches per inch
lf	linear feet
No.	number
sf	square feet
vf	vertical feet
W.S.	water surface

NOTES:
1. Data and calculations of the total drainage area are shown on Table A-1.
2. The length and width at the dike crest and at the depth below dike crest of each Ash Pond is based on dimensions and 2.5H:1V side slopes as shown on Dwg. Nos. 1-C-1-C, Rev.3, Tippet & Gee, Inc., Rev. 4/14/1980; 1-C-33, Rev. 7, Tippet & Gee, Inc., Rev. 5/13/1981; 1-C-37, Rev. 0, Tippet & Gee, Inc., Rev. 8/8/1977; and 1-C-40-C, Rev.6, Tippet & Gee, Inc., 6/13/1980.
4. Aw = Lw*Ww/(43,560 sf/acre).
5. Runoff coefficient sources are shown on Table A-1.
6. As = At-Aw.
7. Cc = (Cw*Aw+Cs*As)/(Aw+As).

TABLE 3

Ash Ponds Stage-Storage Data

Inflow Design Flow Control Plan for CCR Surface Impoundments
San Miguel Electric Cooperative, Inc., Atascosa County, Texas
ERM Project No. 0303548

Depth Below Dike Crest (h)	Each Pond Length At Dike Crest (a)[1]	Each Pond Width At Dike Crest (b)[1]	Each Pond Length At Depth Below Dike Crest (c)[2]	Each Pond Width At Depth Below Dike Crest (d)[2]	Total Volume Below Dike Crest, Both Ash Ponds[3]	Total Volume Below 0.5-ft. Freeboard, Both Ash Ponds[4]
vf	lf	lf	lf	lf	MMgal	MMgal
0	2,450	240	2,450	240	0.0	0.0
0.5			2,448	238	4.4	0.0
1			2,445	235	8.7	4.3
2			2,440	230	17.2	12.8
3			2,435	225	25.5	21.1
4			2,430	220	33.6	29.2

ABBREVIATIONS AND ACRONYMS:

cf	cubic feet
lf	linear feet
MMgal	million gallons
vf	vertical feet

NOTES:

1. The length and width at the dike crest of each Ash Pond is from Dwg. No. 1-C-1-C, Rev.3 Tippet & Gee, Inc., 4/14/1980 and 1-C-40-C, Rev.6, Tippet & Gee, Inc., 6/13/1980.
2. The length and width of each Ash Pond at the depth below dike crest is based on the 2.5H:1V side slope calculated from dimensions and elevations shown on Dwg. No. 1-C-40-C, Rev. 6, Tippet & Gee, Inc., 6/13/1980.
3. The total volume of both Ash Ponds was calculated as the product of (h/6)*(a*b+(a+c)*(b+d)+c*d) multiplied by 2 (the number of identical Ash Ponds), and converted to units of million gallons by multiplying by 7.48 gallons per cubic feet and dividing by 1,000,000.
4. The total volume below 0.5-foot freeboard was calculated as the total volume below the dike crest at that depth minus the total volume in the one-foot freeboard to account for uncertainty in dike crest elevation and wave action.

TABLE 4

Ash Ponds Design Flood Inflow Rates, 100-Year Storm

Inflow Design Flow Control Plan for CCR Surface Impoundments
San Miguel Electric Cooperative, Inc., Atascosa County, Texas

		Storm Water Runoff[1]		12-Hour Storm Event			1-Day Storm Event			2-Day Storm Event			3-Day Storm Event		
		Runoff Coefficient	**Drainage Area**	**Ash Pond NDW Freeboard[3]**	**Total Precip.[4]**	**Total Volume[5]**	**Ash Pond NDW Freeboard[3]**	**Total Precip.[4]**	**Total Volume[5]**	**Ash Pond NDW Freeboard[3]**	**Total Precip.[4]**	**Total Volume[5]**	**Ash Pond NDW Freeboard[3]**	**Total Precip.[4]**	**Total Volume[5]**
Stream	**Description**	**in/in**	**acres**	**vf**	**in.**	**MMgal**	**vf**	**in.**	**MMgal**	**vf**	**in.**	**MMgal**	**vf**	**in.**	**MMgal**
1	Ash Pond Direct Precipitation	1.00	27.0	1.5	8.7	7.1	1.7	10.2	8.4	1.9	11.7	9.6	2.0	12.8	10.5
2	Ash Pond Perimeter Storm Water Runoff	0.70	5.0												
3	Plant Floor Drainage[2]	0.95	7	-----		1.6	-----		1.8	-----		2.1	-----		2.3
	Total Inflow Volume					8.7			10.2			11.7			12.8
	Total Volume Contained Below 0.5-Foot Minimum Freeboard[6]					8.7			10.2			11.7			12.8

ABBREVIATIONS AND ACRONYMS:

ID	identification	NDW	normal dry weather
in.	inches	No.	Number
in/in	inches per inch	Precip.	precipitation
kgpd	thousand gallons per day	vf	vertical feet
MMgal	million gallons	yr	year

NOTES:

1. Ash Pond runoff coefficients and exposed areas were used to calculate composite runoff coefficient; see Table 2 and Figure 3.
2. The storm water runoff coefficient and drainage area is of the Plant Floor based on surface cover type and dimensions in Dwg. No. 1-C-35, Rev 16, Tippet & Gee, Inc., rev. 8/6/1980
3. The Ash Pond NDW freeboard was selected to produce a volume above the NDW equal the Total Inflow for the corresponding design storm duration. The Ash Pond NDW freeboard includes 0.5-foot freeboard at the maximum storage volume to contain wave action.
4. Total precipitation for the design storm duration indicated; see Table 1 and Figure 2.
5. Total Volume is the sum of the direct precipitation, stormwater runoff, and pumped plant floor drainage. The stormwater runoff volume was calculated using the composite runoff coefficient calculated using the indicated Ash Pond NDW Freeboard and the Stage-Composite Runoff Coefficient relationship developed using data shown on Table 2 and shown on Figure 3.
6. The total volume contained above the Ash Pond NDW freeboard level and below the 0.5-foot freeboard level at the indicated total volume based on data in Table 3 and as shown on Figure 4.

TABLE 5

Equalization Pond Stage-Composite Runoff Coefficient

Inflow Design Flow Control Plan for CCR Surface Impoundments
San Miguel Electric Cooperative, Inc., Atascosa County, Texas
ERM Project No. 0303548

Depth Below Dike Crest	Total Drainage Area (At)[1]	W.S. Width At Depth Below Dike Crest (Ww)[2]	W.S. Length At Depth Below Dike Crest (Lw)[2]	Water Surface Area At Depth Below Dike Crest, Calc. (Awc)[3]	Pond Water Surface Area Measured:Calc (Awm/Awc)	Water Surface Area At Depth Below Dike Crest, Measured (Awm)[4]	Open Water Storm Water Runoff Coeff. (Cw)[5]	Slope Area At Depth Below Dike Crest (As)[6]	Slope Area Storm Water Runoff Coeff. (Cw)[5]	Composite Storm Water Runoff Coeff. (Cc)[7]
vf	sf	lf	lf	acres	%	acres	in/in	acres	in/in	in/in
0	28.5	710	1,570	25.6	92%	23.7	1.00	4.8	0.70	0.95
0.5		707	1,567	25.4	92%	23.5		5.0		0.95
1		704	1,564	25.3	92%	23.2		5.3		0.94
2		698	1,558	25.0	91%	22.8		5.7		0.94
3		692	1,552	24.7	91%	22.4		6.1		0.94
4		686	1,546	24.3	91%	22.0		6.5		0.93

ABBREVIATIONS AND ACRONYMS:

calc	calculated
cf	cubic feet
Coeff.	coefficient
in/in	inches per inch
lf	linear feet
sf	square feet
vf	vertical feet
W.S.	water surface

NOTES:
1. Total area approximated based on drawing Nos. 1-C-41, Rev. 4 and 1-C-42, Rev. 2, Tippet & Gee, Inc. 4/14/1980
2. The Equalization Pond is an irregular shape but was approximated as a truncated rectangular prism for the stage-storage analysis. Length and width of the EP was approximated from Dwg. Nos. 1-C-41, Rev. 4 and 1-C-42, Rev. 2, Tippet & Gee, Inc. 4/14/1980.
3. Awc = Lw*Ww/(43,560 sf/acre).
4. The measured area of the Equalization Pond was used in composite runoff coefficient calculations.
5. Runoff coefficient sources are shown on Table A-2.
6. As = At-Awm.
7. Cc = (Cw*Aw+Cs*As)/(Aw+As).

TABLE 6

Equalization Pond Stage-Storage Data

Inflow Design Flow Control Plan for CCR Surface Impoundments
San Miguel Electric Cooperative, Inc., Atascosa County, Texas

Depth Below Dike Crest (d)	EP Area Calculated at Depth Below Dike Crest (Awc)[1]	W.S. Width At Depth Below Dike Crest (Ww)[1]	W.S. Length At Depth Below Dike Crest (Lw)[1]	Pond Water Surface Area Measured:Calc (Awm/Awc)[2]	Water Surface Area At Depth Below Dike Crest, Measured (Awm)[2]	EP Total Volume Below Dike Crest[3]	Total Volume Below 0.5-ft. Freeboard[4]
vf	acres	lf	lf	%	acres	MMgal	MMgal
0	25.6	710	1,570	92%	23.7	0.0	0.0
0.5		707	1,567	92%	23.6	3.8	0.0
1		704	1,564	92%	23.5	7.7	3.8
2		698	1,558	91%	23.4	15.3	11.5
3		692	1,552	91%	23.3	22.9	19.1
4		686	1,546	91%	23.2	30.5	26.7

ABBREVIATIONS AND ACRONYMS:

cf	cubic feet
EP	Equalization Pond
lf	linear feet
MMgal	million gallons
vf	vertical feet

NOTES:

1. The Equalization Pond is an irregular shape but was approximated as a truncated rectangular prism for the stage-storage analysis. Length and width of the EP was approximated from Dwg. Nos. 1-C-41, Rev. 4 and 1-C-42, Rev. 2, Tippet & Gee, Inc. 4/14/1980. The EP pond water surface area was calculated by the product of the approximate length and width AWC = Lw * Ww.
2. The area used in volume calculations was corrected for the area measured vs. approximated by the dimensions Lw & Ww.
3. The total volume of the EP at the indicated Depth Below Dike Crest was calculated as the product of (h/3)*(Awm0+Awmd+(Awm0*Awmd)^0.5)*43560 converted to units of million gallons by multiplying by 7.48 gallons per cubic feet and dividing by 1,000,000.
4. The total volume below 0.5-foot freeboard was calculated as the total volume below the dike crest at that depth minus the total volume in the one-foot freeboard to account for uncertainty in dike crest elevation and wave action.

TABLE 7
Equalization Pond Design Flood Inflow Rates, 100-Year Storm

Inflow Design Flow Control Plan for CCR Surface Impoundments
San Miguel Electric Cooperative, Inc., Atascosa County, Texas

		Storm Water Runoff[1]		12-Hour Storm Event			1-Day Storm Event			2-Day Storm Event			3-Day Storm Event		
		Runoff Coefficient	**Drainage Area**	**SSP NDW Freeboard[2]**	**Total Precip.[3]**	**Total Volume[4]**	**SSP NDW Freeboard[2]**	**Total Precip.[3]**	**Total Volume[4]**	**SSP NDW Freeboard[2]**	**Total Precip.[3]**	**Total Volume[4]**	**SSP NDW Freeboard[2]**	**Total Precip.[3]**	**Total Volume[4]**
Stream	**Description**	**in/in**	**acres**	**vf**	**in.**	**MMgal**	**vf**	**in.**	**MMgal**	**vf**	**in.**	**MMgal**	**vf**	**in.**	**MMgal**
1	EP Direct Precipitation	1.00	23.7	1.3	8.7	6.4	1.5	10.2	7.4	1.6	11.7	8.5	1.7	12.8	9.4
2	EP Perimeter Storm Water Runoff	0.70	4.8												
Total Inflow Volume						6.4			7.4			8.5			9.4
Total Volume Contained Below 0.5-Foot Minimum Freeboard[5]						6.4			7.4			8.5			9.4

ABBREVIATIONS AND ACRONYMS:

EP	Equalization Pond	NDW	normal dry weather
ID	identification	No.	Number
in.	inches	Precip.	precipitation
in/in	inches per inch	vf	vertical feet
kgpd	thousand gallons per day	yr	year
MMgal	million gallons		

NOTES:
1. EP runoff coefficients and exposed areas were used to calculate composite runoff coefficient; see Table 5 and Figure 5.
2. The EP NDW Freeboard was selected to produce a volume above the NDW equal the Total Inflow for the corresponding design storm duration. The EP NDW Freeboard includes 0.5-foot freeboard at the maximum storage volume to contain wave run-up.
3. Total precipitation for the design storm duration indicated; see Table 1.
4. Total Volume is the sum of the Process Flow and the stormwater runoff calculated using the composite runoff coefficient calculated using the indicated EP NDW Freeboard and the Stage-Composite Runoff Coefficient relationship developed using data shown on Table 5 and shown on Figure 4.
5. The total volume contained above the EP NDW Freeboard Level and below the 1-foot freeboard level at the indicated total volume based on data in Table 6 and as shown on Figure 6.

Figures

October 2016
Project No. 0303548

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Austin, Texas 78701
(512) 459-4700

Environmental Resources Management

DESIGN: PD/CJ	DRAWN: EFC	CHKD.: .
DATE: 8/22/2016	SCALE: AS SHOWN	REVISION: 0
W.O.NO.: K:\GIS\SMECI\Christine_TX\MXD\Fig1_SitePlan.mxd		

FIGURE 1
SITE PLAN
CCR Inflow Design Flood Control System Plan
San Miguel Electric Cooperative, Inc. Facility
Atascosa County, Texas

ERM

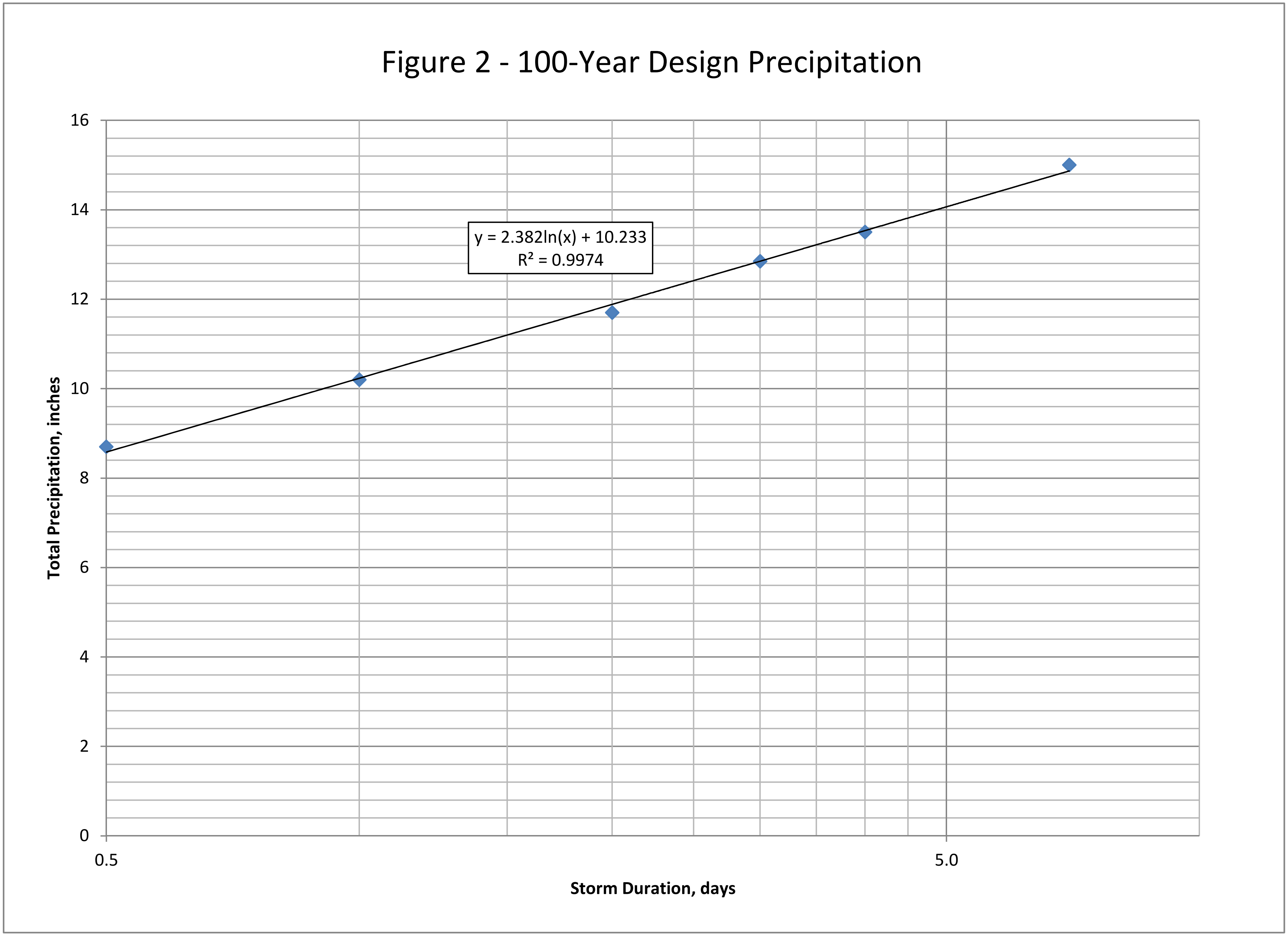
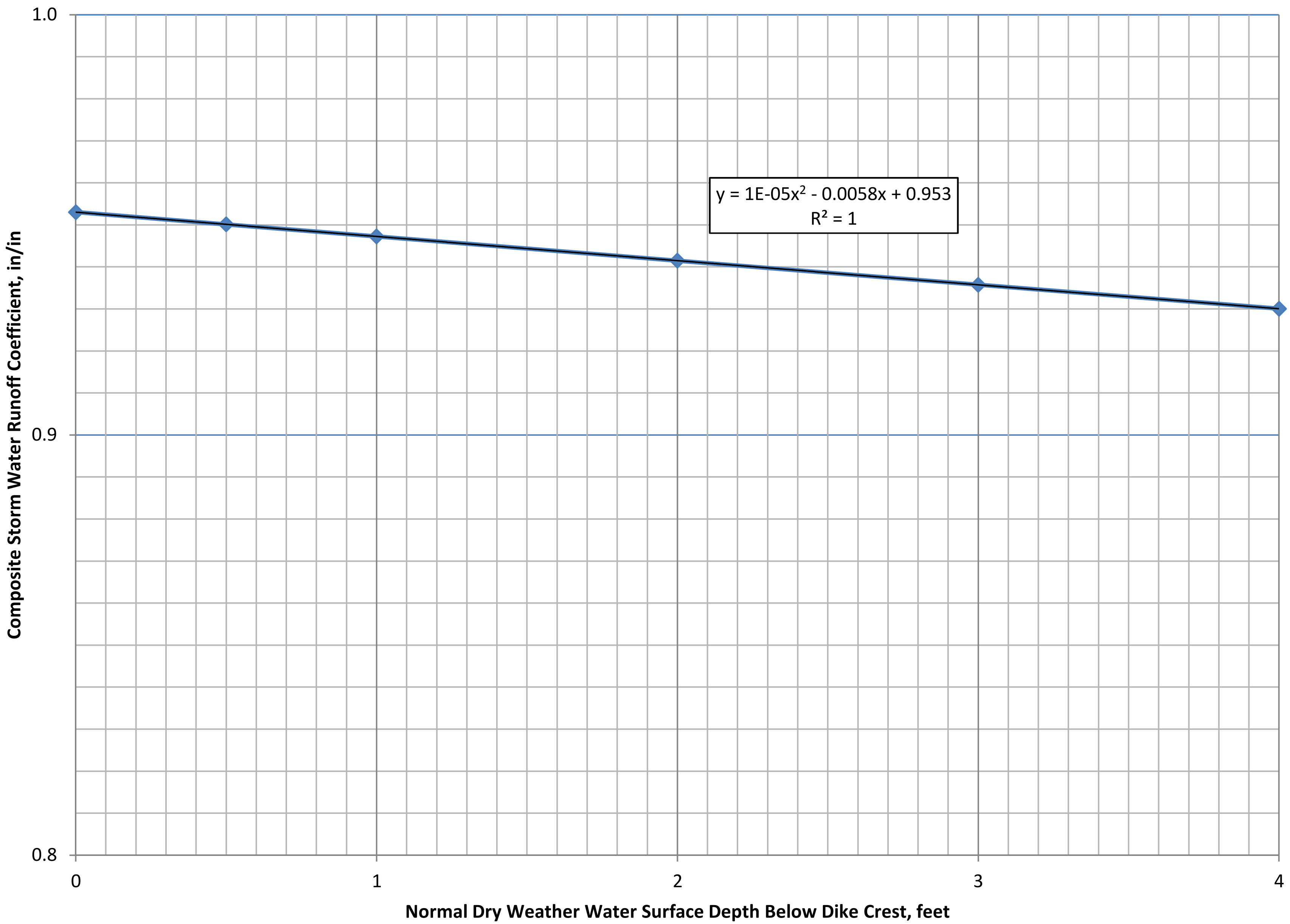


Figure 2 - 100-Year Design Precipitation
y = 2.382ln(x) + 10.233
R² = 0.9974
16
14
12
10
8
6
4
2
0
Total Precipitation, inches
0.5
5.0
Storm Duration, days

Figure 3 - Ash Ponds Stage-Composite Runoff Coefficient Curve



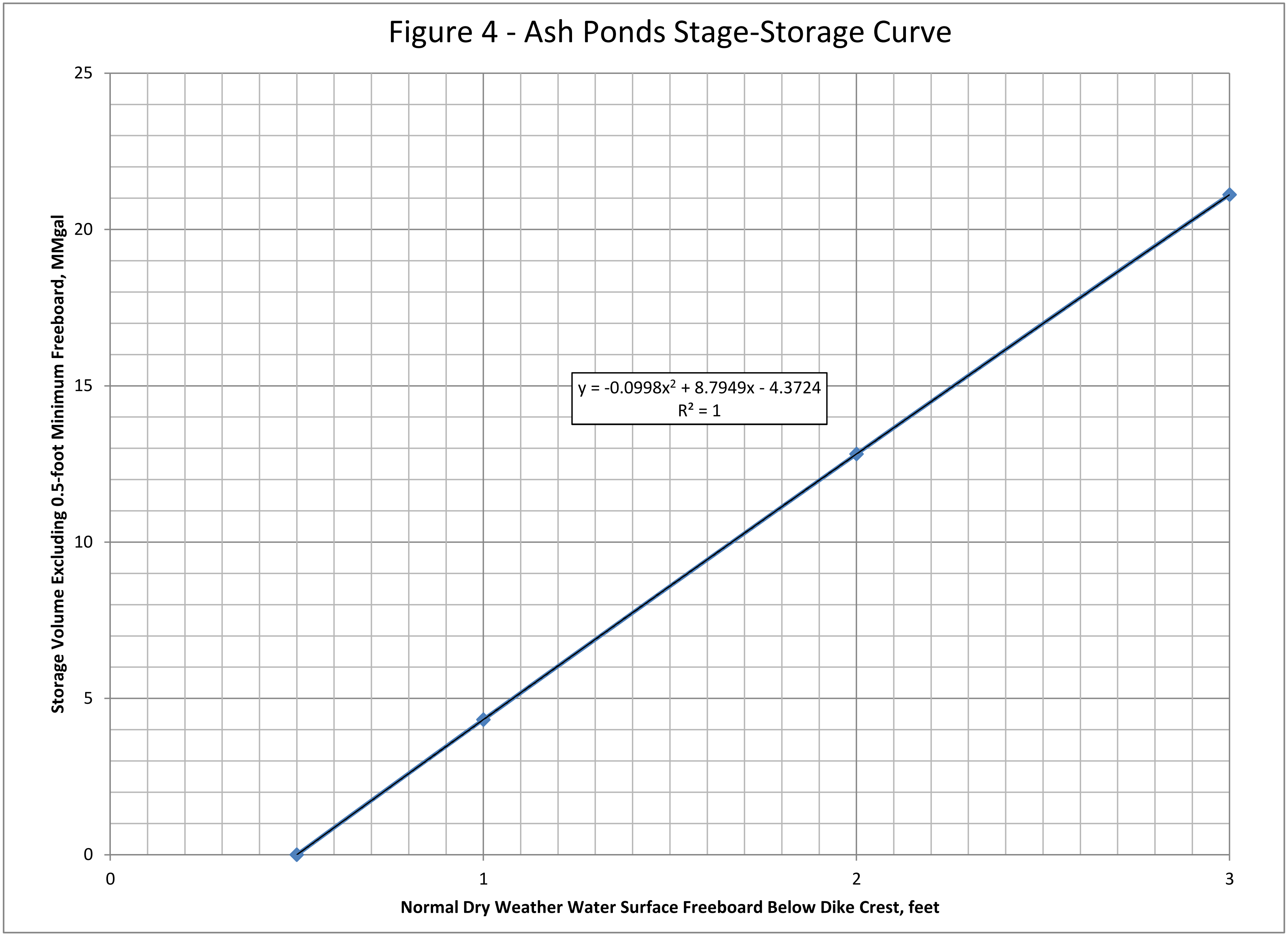


Figure 4 - Ash Ponds Stage-Storage Curve
25
20
15
10
5
0
0
1
2
3
y = -0.0998x² + 8.7949x - 4.3724
R² = 1
Storage Volume Excluding 0.5-foot Minimum Freeboard, MMgal
Normal Dry Weather Water Surface Freeboard Below Dike Crest, feet

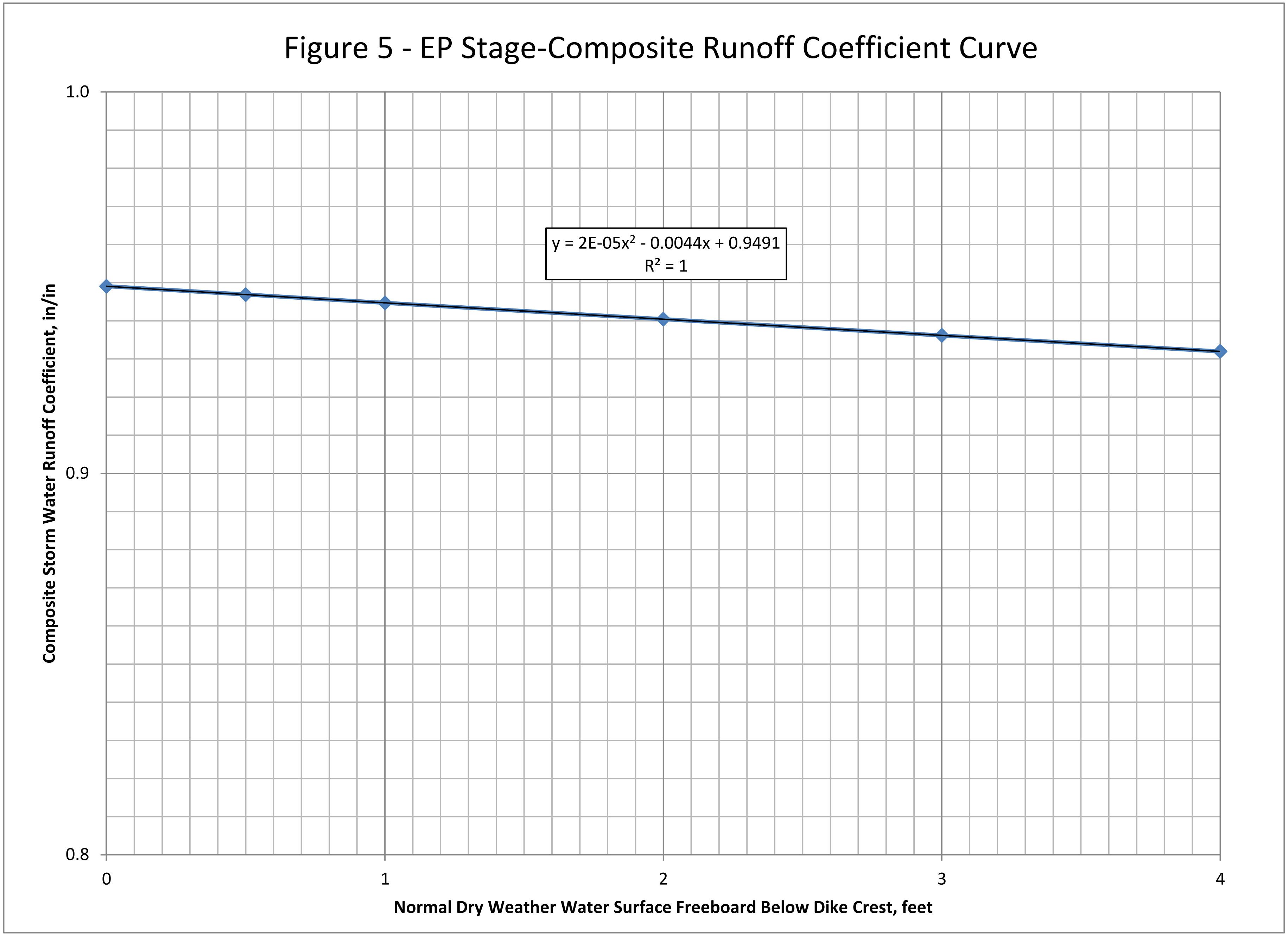


Figure 5 - EP Stage-Composite Runoff Coefficient Curve
1.0
0.9
0.8
0
1
2
3
4
y = 2E-05x² - 0.0044x + 0.9491
R² = 1
Composite Storm Water Runoff Coefficient, in/in
Normal Dry Weather Water Surface Freeboard Below Dike Crest, feet

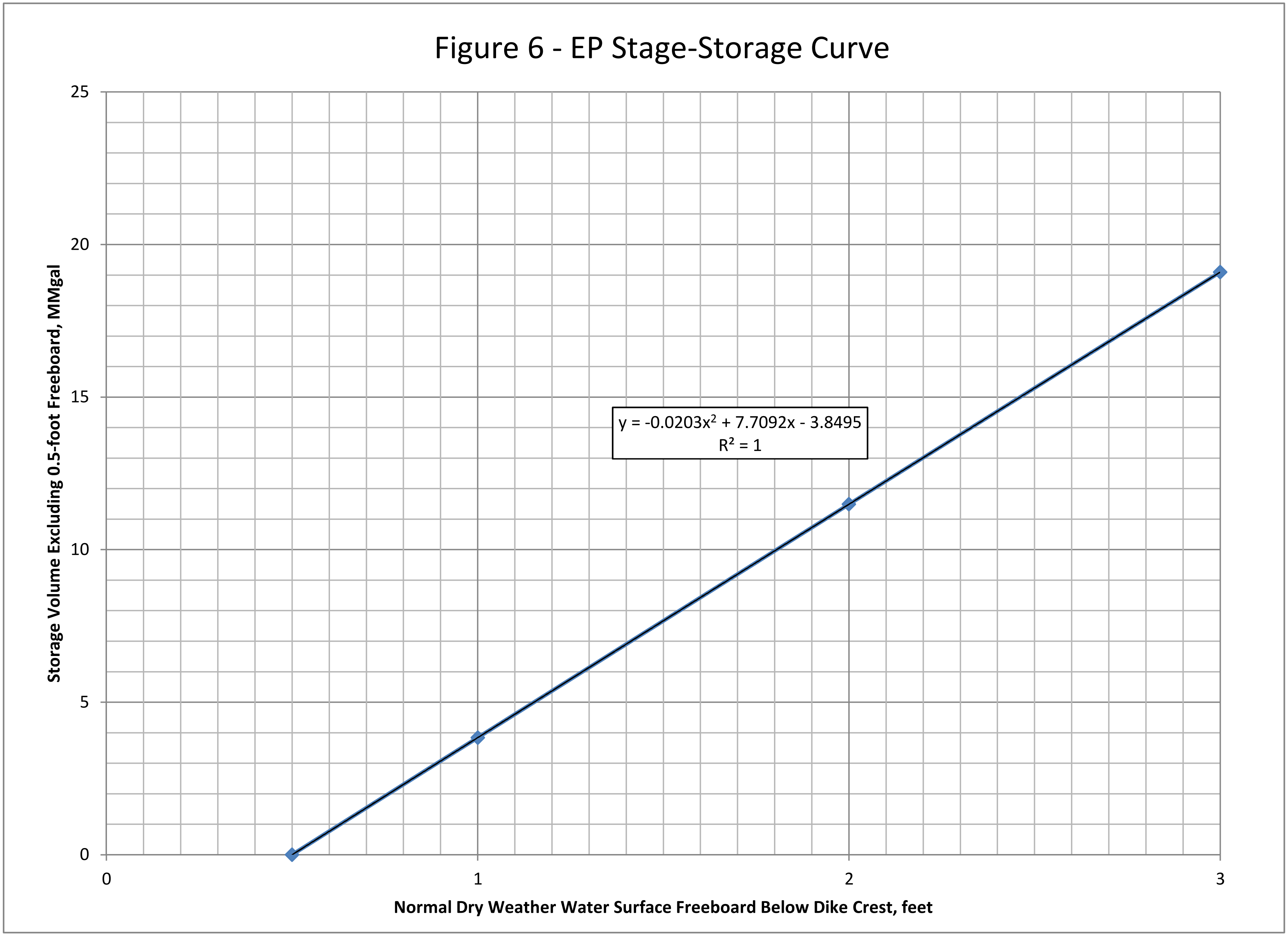


Figure 6 - EP Stage-Storage Curve
y = -0.0203x² + 7.7092x - 3.8495
R² = 1
Storage Volume Excluding 0.5-foot Freeboard, MMgal
Normal Dry Weather Water Surface Freeboard Below Dike Crest, feet
0
5
10
15
20
25
0
1
2
3

Design Flood Inflow Capacity Calculations

Appendix A

October 2016

Project No. 0303548

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TABLE A-1

Ash Ponds Design Flood Inflow Capacity Calculations

Inflow Design Flow Control Plan for CCR Surface Impoundments
San Miguel Electric Cooperative, Inc., Atascosa County, Texas

Calculation Inputs

Parameter	Estimated Quantity	Estimate Units	Source
Gallons per cubic foot	7.48	gal/cf	
Square feet per acre	43,560	ft/acre	
Concrete Runoff Coefficient	0.95	-	TxDOT Hydraulic Design Manual, July 2016, Table 4-10
Steep Grassed Slopes Runoff Coefficient	0.70	-	TxDOT Hydraulic Design Manual, July 2016, Table 4-10
Ponded Water Runoff Coefficient	1.00	-	

Ash Water Transport Pond Stage-Storage and Overflow Calculations

Parameter	Estimated Quantity	Estimate Units	Source
Side Slopes	2.5	H: 1V	
Int. TOB El., Et	315	ft - datum unknown	Dwg. No. 1-C-40-C, Rev.6, Tippet & Gee, Inc., 6/13/1980
Int. TOS El., Eb	295	ft - datum unknown	Dwg. No. 1-C-40-C, Rev.6, Tippet & Gee, Inc., 6/13/1980
Pond Total Depth (each of 2 Ash Ponds), D	20	vf	Calculation: D=Et-Eb
Pond Bottom Area (each of 2 Ash Ponds)			
Bottom length, Lb	2,350	lf	Dwg. No. 1-C-1-C, Rev.3, Tippet & Gee, Inc., 4/14/1980
Bottom width, Wb	140	lf	Dwg. No. 1-C-40-C, Rev.6, Tippet & Gee, Inc., 6/13/1980
Bottom area, Ab	329,000	sf (each AP)	Calculation: Ab=Lb*Wb
Pond Top Area (each of 2 Ash Ponds)			
Top length, Lt	2,450	lf	Dwg. No. 15-C-235, Rev.1, Tippet & Gee, Inc., 8/16/1977
Top width, Wt	240	lf	Dwg. No. 15-C-235, Rev.1, Tippet & Gee, Inc., 8/16/1977
Top area, At	588,000	sf (each AP)	Calculation: At=Lt*Wt
Top area	13.5	acres (each AP)	Convert units at 43,560 sf/acre
Pond Top Area (both ponds)	27.0	acres (total)	Calculation: At(both ponds)=At*2
Ash Pond Total Drainage Area			
Dike Crest Width Drained to Ash Pond, Wtda	280	lf	Assume all of the perimeter dike crest drains to the interior
Dike Crest Length Drained to Ash Pond, Ltda	2,490	lf	Assume 100% of the interior dike crests drain to the interior
Top Area, Atda	1,394,400	sf (both APs)	Atda = Ltda*Wtda
Ash Pond Total Drainage Area	32.0	acres	Convert units at 43,560 sf/acre

ABBREVIATIONS AND ACRONYMS:

cf	cubic feet	lf	linear feet
Dwg.	Drawing	No.	Number
El.	Elevation	Rev.	Revision
ft	feet	sf	square feet
ft W	feet width	TOB	top of bank
gal	gallons	TOC	toe of slope
H	horizontal	TxDOT	Texas Department of Transportation
Int.	interior	V	vertical
		vf	vertical feet

TABLE A-2

Equalization Pond Design Flood Inflow Capacity Calculations

Inflow Design Flow Control Plan for CCR Surface Impoundments
ERM Project No. 0303548

Calculation Inputs

Parameter	Estimated Quantity	Estimate Units	Source
Gallons per cubic foot	7.48	gal/cf	
Square feet per acre	43,560	ft/acre	
Concrete Runoff Coefficient	0.95	-	Table 4-10, Hydraulic Design Manual, TxDOT, July 2016
Steep Grassed Slopes Runoff Coefficient	0.70	in/in	Table 4-10, Hydraulic Design Manual, TxDOT, July 2016
Ponded Water Runoff Coefficient	1.00	in/in	

Equalization Pond Stage-Storage Calculations

Parameter	Estimated Quantity	Estimate Units	Source
Side Slope	3.0	H:1V	
Int. TOB El., Et	295	ft - datum unknown	
Int. TOS El., Eb	275	ft - datum unknown	Drawing Nos. 1-C-41, Rev. 4 and 1-C-42, Rev. 2, Tippet & Gee, Inc., 4/14/1980
Pond Total Depth, D	20	vf	Calculation: D=Et-Eb
EP Interior Area At Int. TOS			
Bottom width, Wb	590	ft	Width at Int. TOS, Drawing No. C-12, Tippet & Gee, Inc. 4/5/1977
Bottom length, Lb	1450	ft	Length at Int. TOS, Drawing No. C-12, Tippet & Gee, Inc. 4/5/1977
Bottom area, Ab (calculated)	19.64	acres	Bottom area, approximated Lb * Wb
Bottom area, Ab (measured)	16.25	acres	Total area measured based on drawing No. C-12, Rev. 0, Tippet & Gee, Inc., 4/5/1977
EP Interior Area At Int. TOB			
Top width, Wt	710	ft	Width at Int. TOB, Drawing No. C-12, Tippet & Gee, Inc. 4/5/1977
Top length, Lt	1,570	ft	Length at Int. TOB, Drawing No. C-12, Tippet & Gee, Inc. 4/5/1977
Top area, At (calculated)	25.6	acres	Top area, approximated Lt * Wt
Top area, At (measured)	23.7	acres	Total area measured based on drawing No. C-12, Rev. 0, Tippet & Gee, Inc., 4/5/1977
EP top area Int. TOS measured:calculated	92%	-	
EP bottom area Int. TOB measured:calculated	83%	-	
EP Total Drainage Area			
Top area	28.5	acres	Total area approximated based on drawing No. C-12, Rev. 0, Tippet & Gee, Inc., 4/5/1977

ABBREVIATIONS AND ACRONYMS:

cf	cubic feet	Int.	Interior
Dwg.	Drawing	in	inches
El.	Elevation	No.	Number
EP	Equalization Pond	sf	square feet
ft	feet	TOB	top of bank
ft W	feet, width	TOS	toe of slope
gal	gallons	TxDOT	Texas Department of Transportation
H	horizontal	V	vertical

Reference Documents

Appendix B

October 2016
Project No. 0303548

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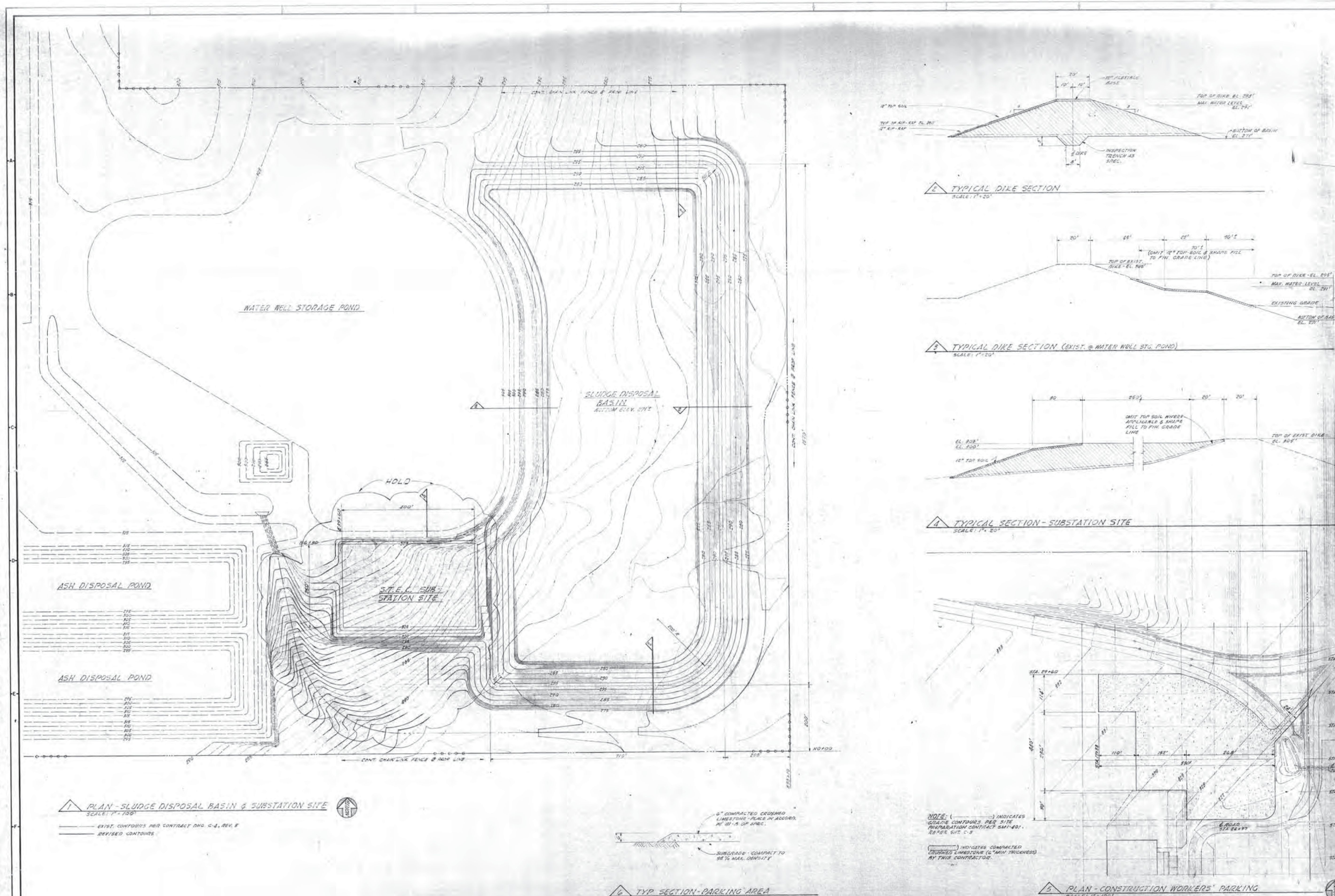
Appendix B-1

T&G, 1977a — Sludge Disposal Basin, 69 kV Substation & Temp. Parking Area, San Miguel Plant Unit No. 1, Drawing No. C-12, Rev. 0, Tippet & Gee, Inc., April 1, 1977, revised April 5, 1977.



NOTES

1. ALL FILL INDICATED 1 SHALL BE AS SPECIFIED FOR CONTROLLED COMPACTED FILL.

REV	DATE	BY	DESCRIPTION
0		PK	ADD THIS DRAWING TO CONTRACT DRAWINGS

SCALE AS NOTED
DRAWN P.R.K.
DATE 4-5-77
CHECKED C.A.D.
APPROVED

STATE OF TEXAS — William G. Hollowell — 15137 — REGISTERED PROFESSIONAL ENGINEER

STATE OF TEXAS — M. L. HUGHES — 25324 — REGISTERED PROFESSIONAL ENGINEER

TIPPETT & GEE, INC.
CONSULTING ENGINEERS
ABILENE TEXAS

SAN MIGUEL PLANT
UNIT NO. 1
B.E.P.C. S.T.E.C.

SLUDGE DISPOSAL BASIN, 69 KV SUBSTATION & TEMP. PARKING AREA

JOB NO.	REV.
SMI-401	0

DRAWING NUMBER
C-12

Appendix B-2

T&G, 1977b — Site Plan Section No. 8, San Miguel Plant Unit No. 1, Drawing No. 1-C-37, Rev. 0, Tippet & Gee, Inc., April 1, 1977, revised August 18, 1977.

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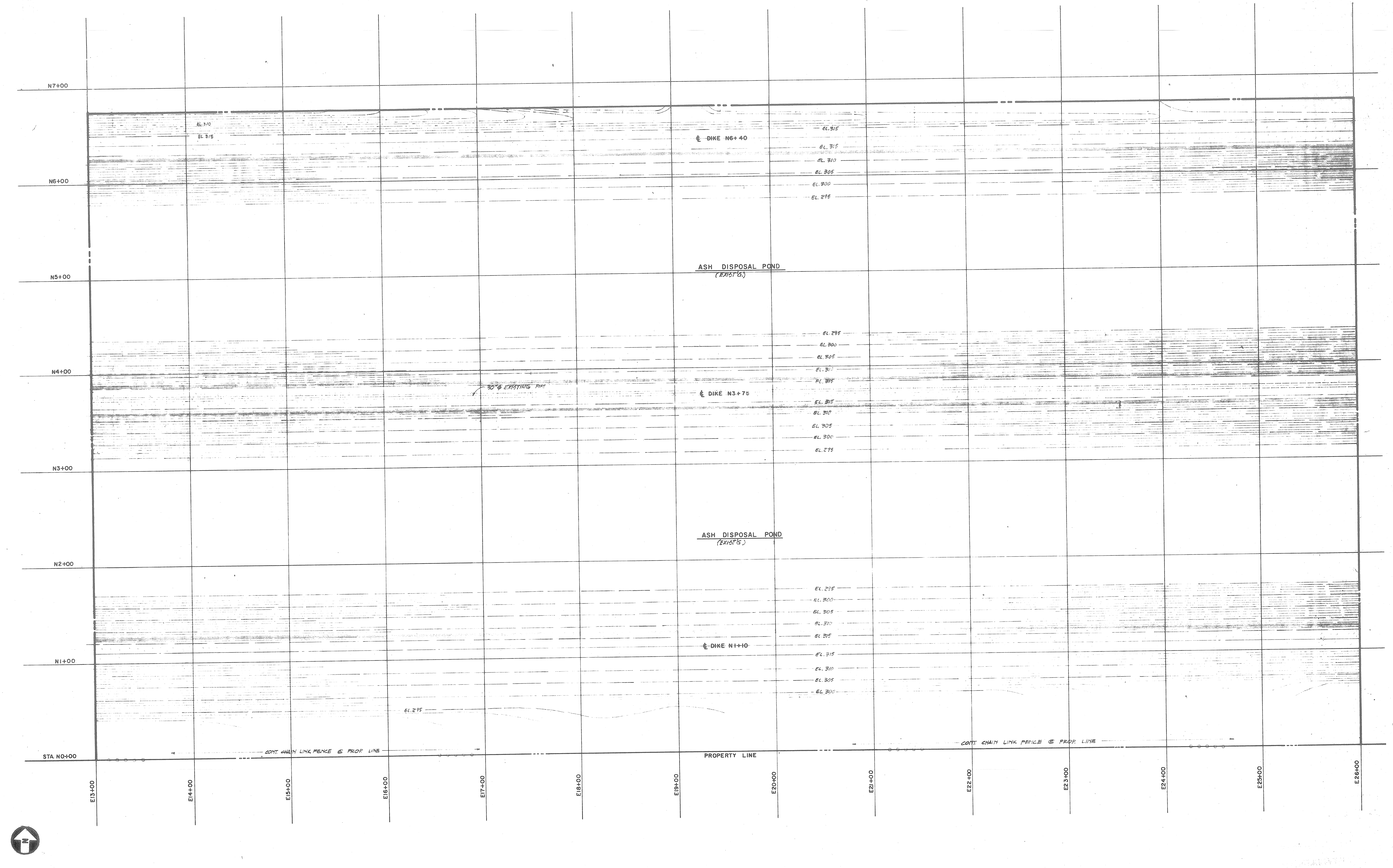

NOTES

REV	DATE	BY	DESCRIPTION
A	5-18-77		REVISED PER ADDENDUM NO. 1
0	8-18-77		FINAL BID SET

SCALE 1"=40'
DRAWN P.G.M.
DATE 4-1-77
CHECKED C.A.D.
APPROVED M.L.H., W.G.H.

STATE OF TEXAS M.L. HUGHES 25324 REGISTERED PROFESSIONAL ENGINEER

STATE OF TEXAS William G. Hollowell 15137 REGISTERED PROFESSIONAL ENGINEER

STATE OF TEXAS CHARLES A. DOBY 37593 REGISTERED PROFESSIONAL ENGINEER

TIPPETT & GEE, INC.
CONSULTING ENGINEERS
ABILENE TEXAS

SAN MIGUEL PLANT
UNIT NO. 1
B.E.P.C. S.T.E.C.

SITE PLAN
SECTION NO. 8

JOB NO. SMI-406
REV. 0
DRAWING NUMBER 1-C-37

Appendix B-3

T&G, 1980a	Site Plan and Vicinity Map, San Miguel Plant Unit No. 1, Drawing No, 1-C-1C Rev 3, Tippet & Gee, Inc., April 1, 1977, revised April 14, 1980.

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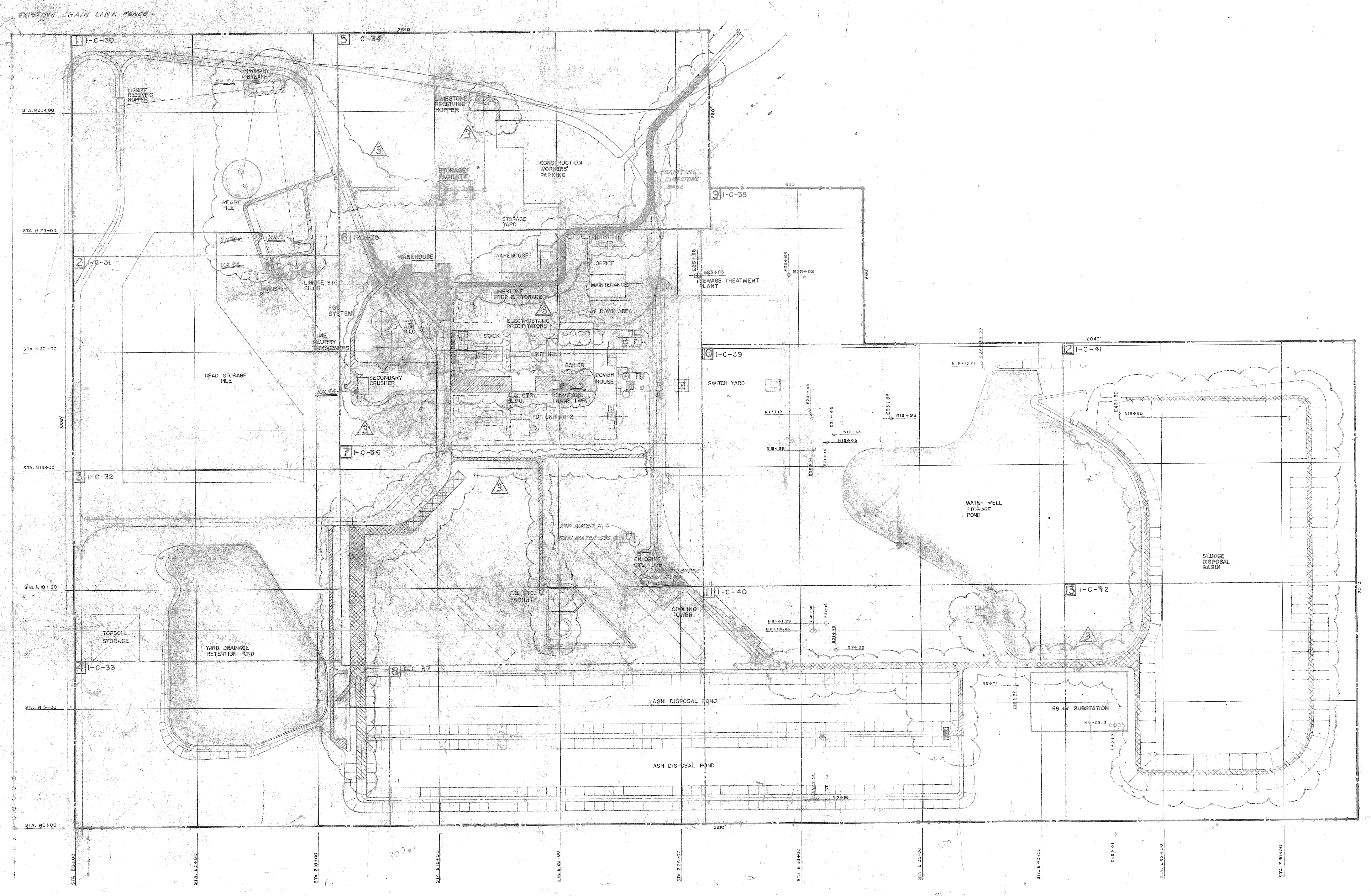
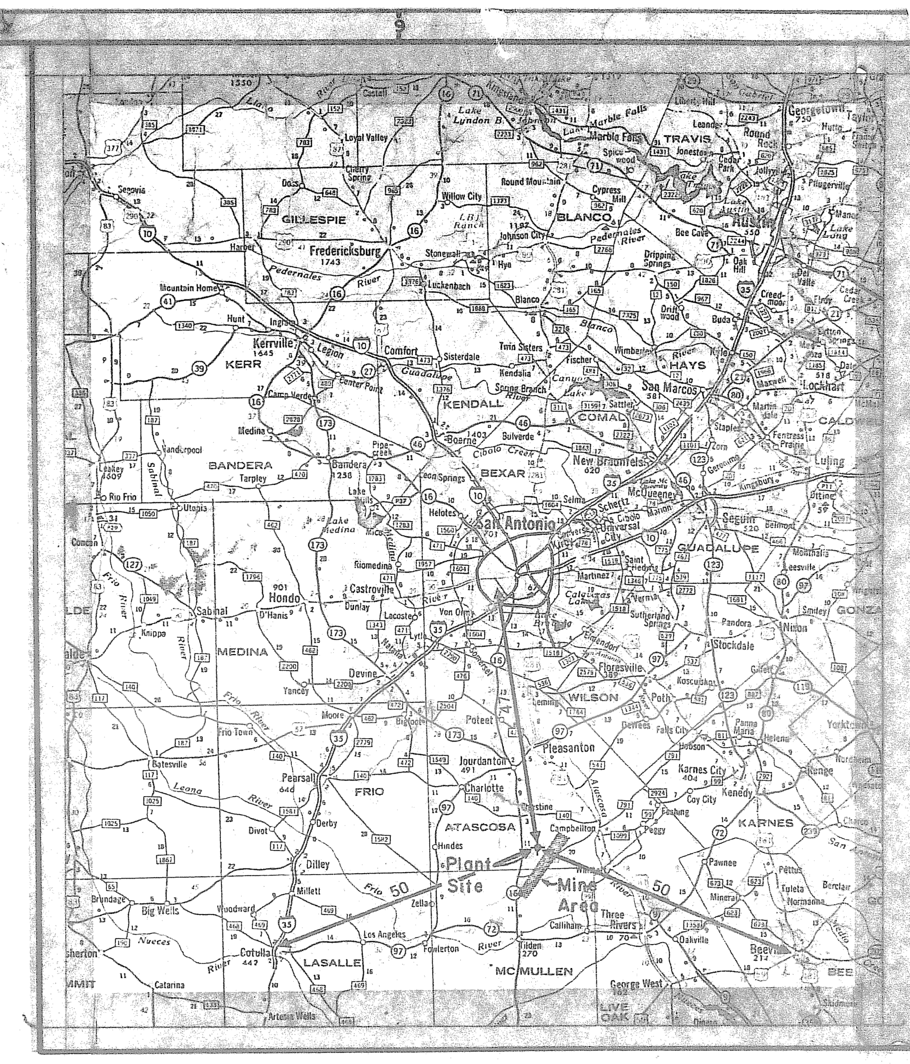


PLANT SITE PLAN

SCALE: 1" = 200'



VICINITY MAP

NO SCALE

LEGEND	
— — — —	EXISTING CONTOURS; EXISTING UTILITIES
———	FINISH CONTOURS; FINISH UTILITIES
	PAVING & BASE AS SPEC. THIS CONTRACT
	TYPE "A" FLEX. BASE ROADS BY THIS CONTRACTOR
	EXISTING TYPE "A" FLEX. BASE ROADS
	ASPHALT PAVING ON EXISTING BASE
	CONCRETE PAVING & BASE AS SPEC. THIS CONTRACT

3

NOTES

REV	DATE	BY	DESCRIPTION
A	5-18-77	WL	REVISED PER ADDENDUM NO. 1
B	7-1-77		REVISED PER ADDENDUM NO. 4
0	8-18-77		FINAL BID SET
1	9-22-77	R.L.H.	EXTENDED ROAD AT WAREHOUSE
2	4-6-79	LW	CORRECTED TO AGREE W/OTHER DWGS. ADDED LAMBERT GRID.
3	4-14-80	JB	ADD. TYPE "A" FLEX. BASE RDS., NOTED EXISTING FLEX. BASE RDS., REV. LEGEND & AREA SOUTH OF BOILER PLAN.

SCALE: AS NOTED
DRAWN: BJK
DATE: 4-1-77
CHECKED: C.A.D.
APPROVED: M.L.H. W.G.H.

STATE OF TEXAS — M. L. HUGHES — 25524 — REGISTERED PROFESSIONAL ENGINEER
STATE OF TEXAS — William G. Hollowell — 15137 — REGISTERED PROFESSIONAL ENGINEER
STATE OF TEXAS — CHARLES A. DOBY — 37593 — REGISTERED PROFESSIONAL ENGINEER

TIPPETT & GEE, INC.
CONSULTING ENGINEERS
ABILENE TEXAS

SAN MIGUEL PLANT
UNIT NO. 1
B.E.P.C. S.T.E.C.

PLANT SITE PLAN AND VICINITY MAP

JOB NO. SMI-406
REV. 3
DRAWING NUMBER I-C-I-C

Appendix B-4

T&G, 1980b	Site Plan Section No. 6, San Miguel Plant Unit No. 1, Drawing No. 1-C-35, Rev 16, Tippet & Gee, Inc., April 1, 1977, revised August 6, 1980.

Environmental Resources Management
206 East 9th Street, Suite 1700
Austin, Texas 78701
(512) 459-4700

NOTES

1. CURB & GUTTER & CONC. APRON NOT UNDER THIS CONTRACT
2. CURB & GUTTERS @ OFFICE & MAINT. BLDG. UNDER THIS CONTRACT. PAVING IN THIS AREA UNDER THIS CONTRACT.
3. DET. 1/1-C-15 TYP. ALONG ENTIRE WIDTH OF BOILER SLAB.

REV	DATE	BY	DESCRIPTION
12	3-24-80	JB	CLARIFIED FIN. CONTOURS - FGD WASTE BLDG.
13	4-14-80	LB	CONSOLIDATED REV. BOX, ADDED RD. AROUND THICKENER TANKS, INDICATED BASE MATERIAL ADJACENT TO BOILER SLAB; INDICATED SUBGRADE & BASE COURSES THIS CONTRACT.
14	4-28-80	JB	REV. TO AGREE W/DWG. 1-C-161, REV. 5
15	7-15-80	LB	ADDED FENCE
16	8-6-80	LB	ADDED ARMCO SLOTTED DRAIN - N. OF R.R. - LAY DOWN AREA
A	6-3-77	JLF	REVISED PER ADDENDUM NO. "3"
B	7-1-77	V.W.	REVISED PER ADDENDUM NO. 4
C	8-3-77	JF	REVISED PER ADDENDUM NO. "5"
0	8-18-77		FINAL BID SET
1	9-23-77	R.L.H.	ADDED TWO CONST. JTS. W/ NOTE & DWG. REFER., LAYOUT CURB NOTE, ADDED ELEVS., REVISED CONTOURS, REVISED TRANSF. PWR. STA. "N&O", REVISED TRANSF. PWR. STA. NOTES "A&B", "G&H", "RES", "T&U", ERASED DASHED LINE OF ROAD, ADDED LINE TO END OF MAINT. DOCK, ADDED TWO PREFAB. TIMBER CROSSINGS WITH NOTES & REFER. NOTES, ADDED STORM SEWER LINE NO. 4 WITH DIMS. & NOTES.
2	12-9-77	W.L.	DELETED RD. CONST. JTS. NOTED C. R.R. BY OTHERS. RELOCATED BENCH MARK #3 BY OTHERS

SCALE 1" = 40'
DRAWN R.G.M.
DATE 4-1-77
CHECKED C.A.D.
APPROVED M.L.H., W.G.H.

M. L. HUGHES 25324 — STATE OF TEXAS REGISTERED PROFESSIONAL ENGINEER
William G. Hollowell 15137 — STATE OF TEXAS REGISTERED PROFESSIONAL ENGINEER
CHARLES A. DOBY 37593 — STATE OF TEXAS REGISTERED PROFESSIONAL ENGINEER

TIPPETT & GEE, INC.
CONSULTING ENGINEERS
ABILENE TEXAS

SAN MIGUEL PLANT
UNIT NO. 1
B.E.P.C. S.T.E.C.

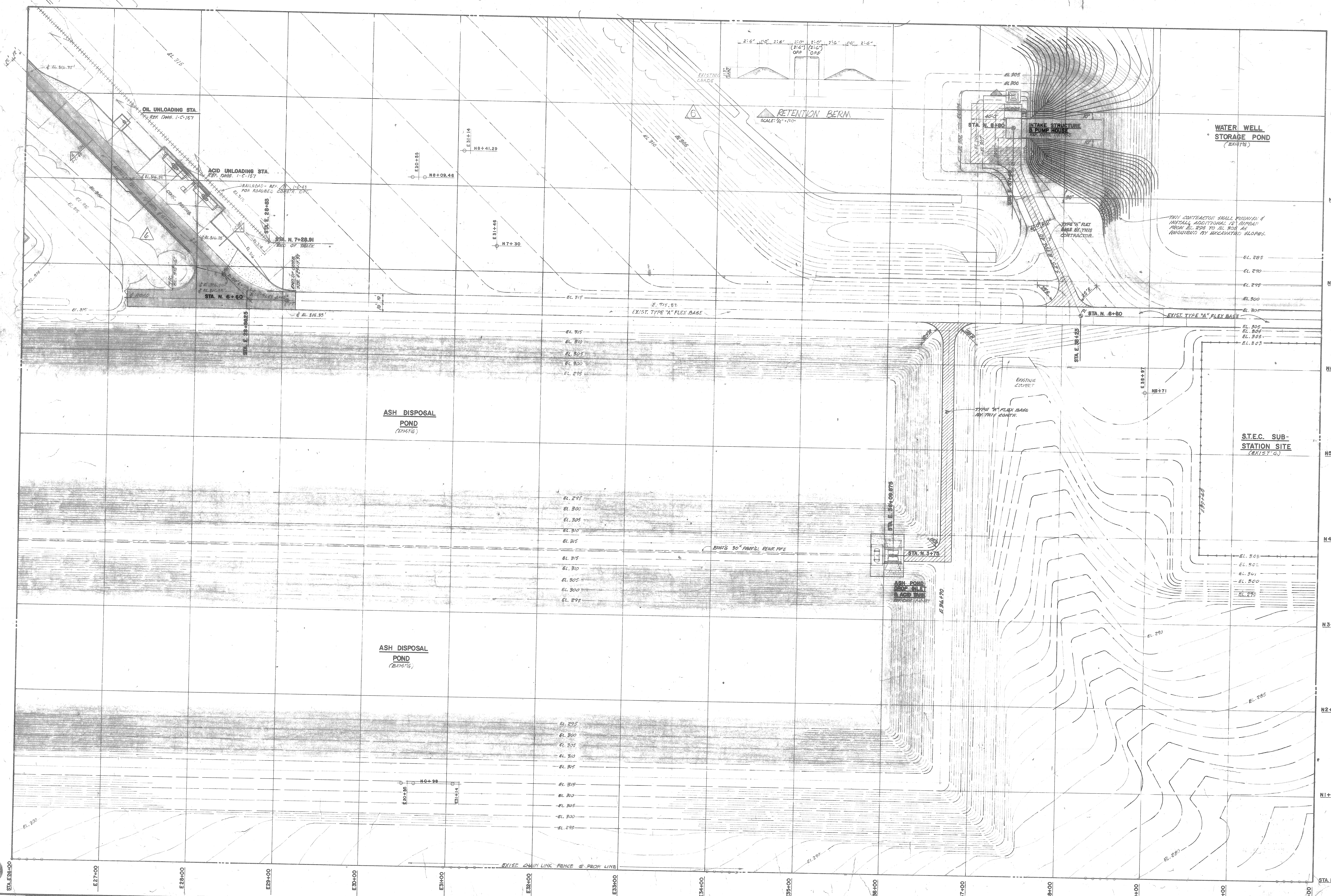
SITE PLAN
SECTION NO. 6

JOB NO. SMI-406
REV. 16
DRAWING NUMBER 1-C-35

Appendix B-5

T&G, 1980c — Site Plan Section No. 11, San Miguel Plant Unit No. 1, Drawing No. 1-C-40, Rev. 6, Tippet & Gee, Inc., April 1, 1977, revised June 13, 1980.

Environmental Resources Management
206 East 9th Street, Suite 1700
Austin, Texas 78701
(512) 459-4700



NOTES

CONTRACTOR SHOULD EXAMINE THE GENERAL, MECHANICAL AND ELECTRICAL CONSTRUCTION DRAWINGS, AND ALL PERTINENT EQUIPMENT MANUFACTURER'S DRAWINGS TO DETERMINE THE EXISTENCE AND LOCATION OF GROUNDING CABLES, PIPE PENETRATIONS, CONDUIT PENETRATIONS, BLOCKOUTS, DRAINS, ETC. AS ARE REQUIRED TO BE INSTALLED UNDER OTHER PORTIONS OF THIS CONTRACT.

REV	DATE	BY	DESCRIPTION
6	6-18-80	LKS	REVISED CONTOURS AT COOLING TOWER & DRAINAGE DITCH.

REV	DATE	BY	DESCRIPTION
A	5-18-77	WL	REVISED PER ADDENDUM NO. "1"
B	7-1-77	W.L.	REVISED PER ADDENDUM NO. "4"
0	8-18-77		FINAL BID SET
1	9-23-77	R.L.H.	REVISED CONTOURS, REVISED OR ADDED ELEVATIONS & ELEVATION POINTS, ADDED REFERENE NOTE
2	12-9-77	R.H.	CORRECTED CONTOURES TO AGREE WITH SMI-401 CONTRACT DELETED REF. NOTE
3	3-3-78	LW	CORRECTED CONTOURES @ E27+00 & N7+00 TO AGREE W/ BID SET.
4	7-31-78	LB	ADDED RETENTION BERM ON PLAN & IN SECTION
5	4-14-80	LB	REV. WIDTH & BASE MAT'L. ON RD. TO INTAKE STRUCT., INDICATED FENCE.

SCALE 1" = 40'
DRAWN P.G.H.
DATE 4-1-77
CHECKED C.A.D.
APPROVED M.L.H., W.G.H.

STATE OF TEXAS M. L. HUGHES 25324 REGISTERED PROFESSIONAL ENGINEER

STATE OF TEXAS William G. Hollowell 15137 REGISTERED PROFESSIONAL ENGINEER

STATE OF TEXAS CHARLES A. DOBY 37593 REGISTERED PROFESSIONAL ENGINEER

TIPPETT & GEE, INC.
CONSULTING ENGINEERS
ABILENE TEXAS

SAN MIGUEL PLANT
UNIT NO. 1
B.E.P.C. S.T.E.C.

SITE PLAN
SECTION NO. 11

JOB NO. SMI-406
REV. 6
DRAWING NUMBER I-C-40

Appendix B-6

T&G, 1980d — Site Plan Section No. 11, San Miguel Plant Unit No. 1, Drawing No. 1-C-41, Rev. 4, Tippet & Gee, Inc., April 1, 1977, revised April 14, 1980.

Environmental Resources Management
206 East 9th Street, Suite 1700
Austin, Texas 78701
(512) 459-4700

CONTRACTOR SHOULD EXAMINE THE GENERAL, MECHANICAL AND ELECTRICAL CONSTRUCTION DRAWINGS, AND ALL PERTINENT EQUIPMENT MANUFACTURER'S DRAWINGS TO DETERMINE THE EXISTENCE AND LOCATION OF GROUNDING CABLES PIPE PENETRATIONS, CONDUIT PENETRATIONS, BLOCKOUTS, DRAINS, ETC. AS ARE REQUIRED TO BE INSTALLED UNDER OTHER PORTIONS OF THIS CONTRACT.

NOTES

REV	DATE	BY	DESCRIPTION
A		WL	REV. PER ADDENDUM NO. "1"
0	8-18-77		FINAL BID SET
1	9-23-77	R.L.H.	ADDED TWO RIPRAPS WITH DIMENSIONS, NOTES, & REFERENCE SECTION MARKS, REFERENCE NOTES, ADDED 10" DRAINLINE, ADDED 20" WELL WATERLINE
2	12-9-77	R.H.	CHANGED RIPRAP TO CONC. SLAB
3	4-6-79	LW	DELETE (1) SPLASH BLOCK & (3) PIPE SUPPORTS.
4	4-14-80	JB	NOTED EXIST. TYPE "A" FLEX. BASE RD.

SCALE 1"=40'
DRAWN P.G.M.
DATE 4-1-77
CHECKED C.A.D.
APPROVED M.L.H., W.G.H.

M.L. HUGHES 25324
William G. Holloway 15137
CHARLES A. DOBY 37593

TIPPETT & GEE, INC.
CONSULTING ENGINEERS
ABILENE TEXAS

SAN MIGUEL PLANT
UNIT NO. 1
B.E.P.C. S.T.E.C.

SITE PLAN
SECTION NO. 12

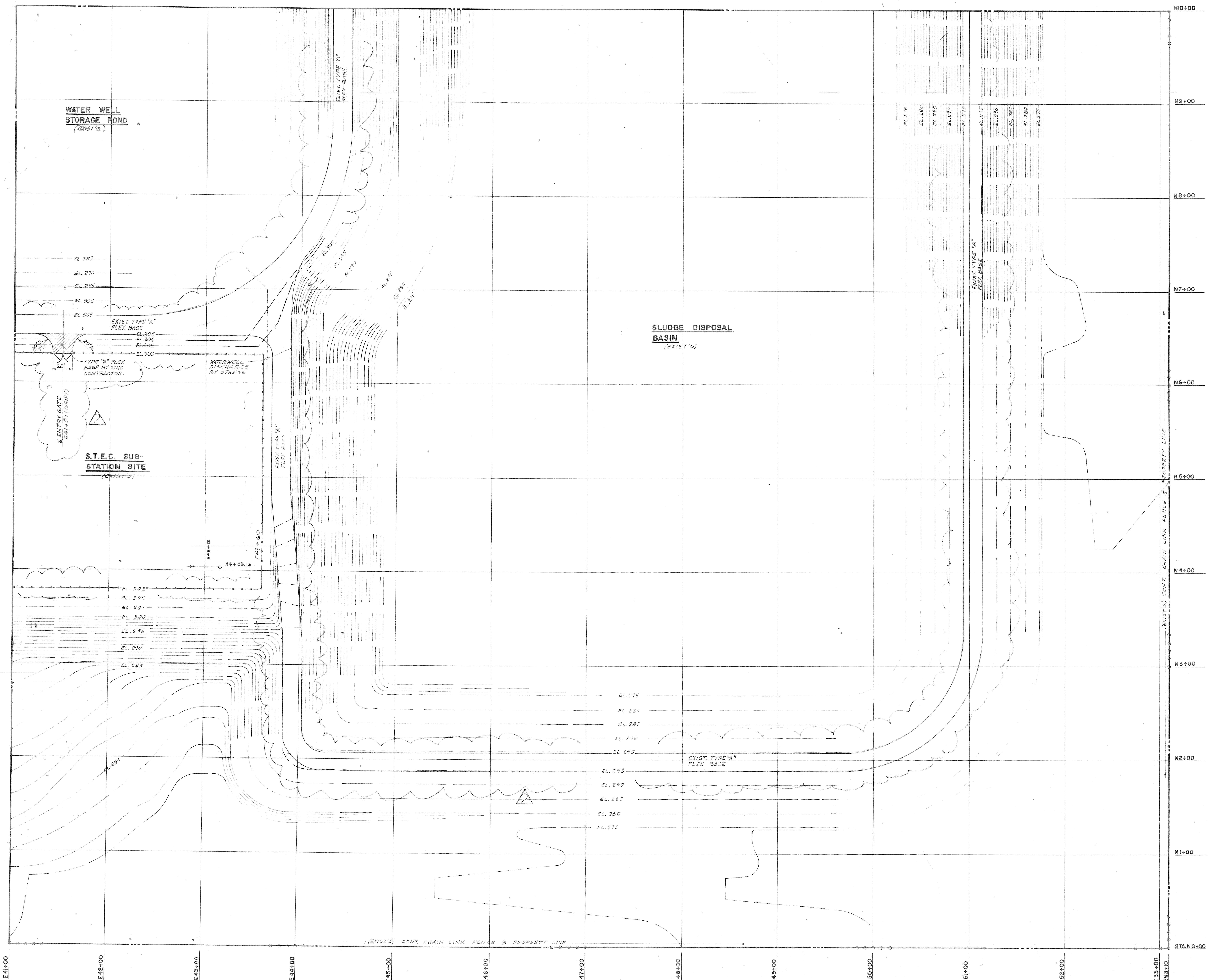
JOB NO.	REV.
SMI-406	4

DRAWING NUMBER
1-C-41

Appendix B-7

T&G, 1980e	Site Plan Section No. 13, San Miguel Plant Unit No. 1, Drawing No. 1-C-42, Rev. 2, Tippet & Gee, Inc., April 1, 1977, revised April 14, 1980.

Environmental Resources Management
206 East 9th Street, Suite 1700
Austin, Texas 78701
(512) 459-4700



NOTES

REV	DATE	BY	DESCRIPTION
A	5-18-77	WL	REVISED PER ADDENDUM NO. "1"
0	8-18-77		FINAL BID SET
1	4-6-79	LW	LOCATED WATERWELL DISCHARGE
2	4-14-80	LB	ADDED FENCE AROUND SUB-STATION SITE; ADDED ACCESS RD. @ SUB-STA. ENTRANCE; REV. EXIST. CONTOURS TO AGREE W/CONTRACT SMI-401; NOTED EXIST. TYPE "A" FLEX BASE RDS.

SCALE 1" = 40'
DRAWN R.G.M.
DATE 4-1-77
CHECKED C.A.D.
APPROVED M.L.H., W.G.H.

M. L. HUGHES 28524
William G. Hollowell 15137
CHARLES A. DOBY 37593

TIPPETT & GEE, INC.
CONSULTING ENGINEERS
ABILENE TEXAS

SAN MIGUEL PLANT
UNIT NO. 1
B.E.P.C. S.T.E.C.

SITE PLAN
SECTION NO. 13

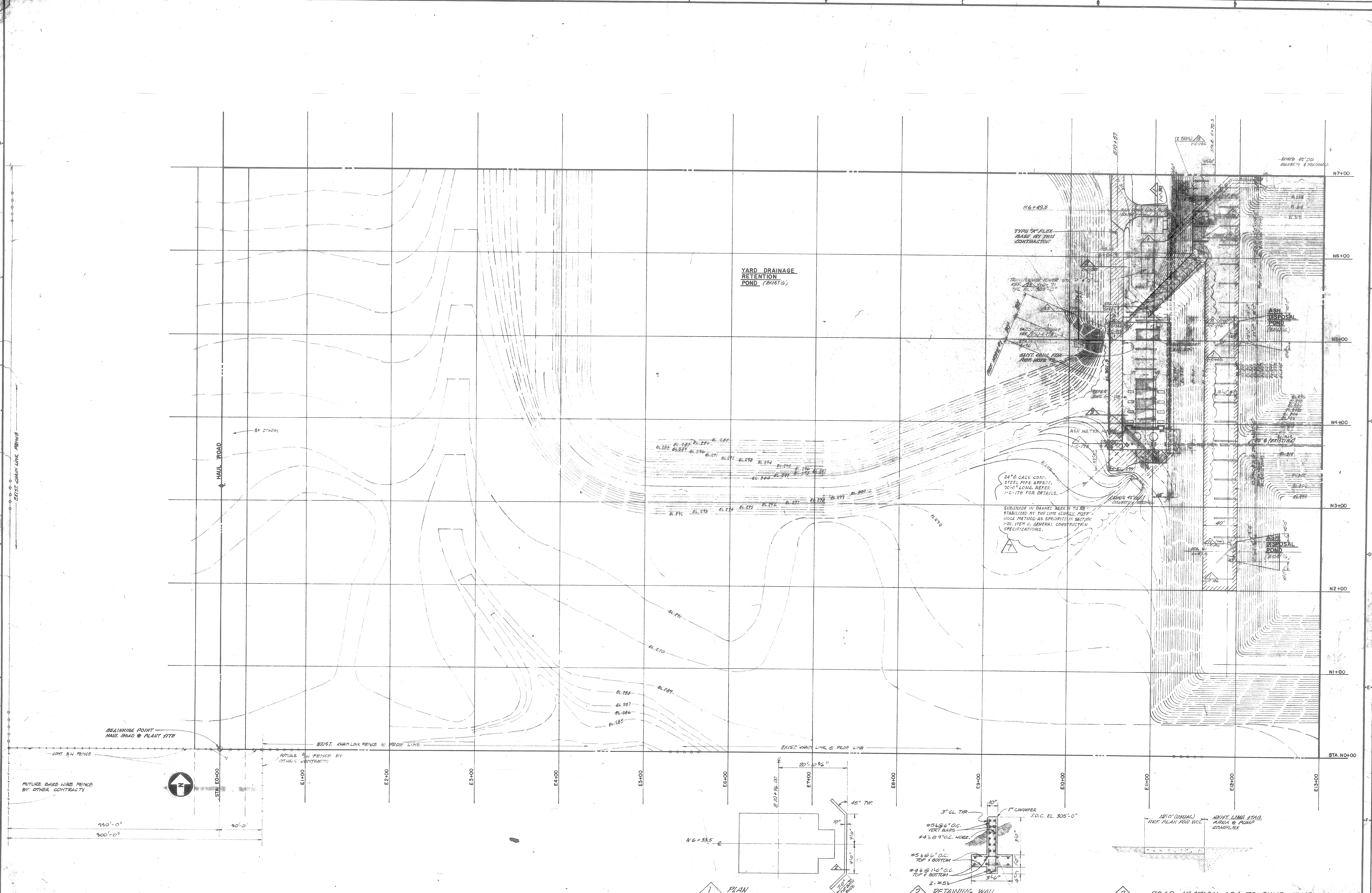
JOB NO.	REV.
SMI-406	2

DRAWING NUMBER
1-C-42

Appendix B-8

T&G, 1981	Site Plan Section No. 4, San Miguel Plant Unit No. 1, Drawing No. 1-C-33, Rev. 7, Tippet & Gee, Inc., April 1, 1977, revised May 13, 1981.

Environmental Resources Management
206 East 9th Street, Suite 1700
Austin, Texas 78701
(512) 459-4700



NOTES

1. THIS CONTRACTOR TO FURNISH & INSTALL 12" RIPRAP ALONG SLOPE AROUND CONC. PUMP SUPPORT STRUCT. AS SHOWN. RIPRAP SHALL EXTEND FULL HT. OF SLOPE EL. 285' TO EL. 300'.
2. EXISTING CONC. FOUNDATION TO BE REMOVED TO TOP OF PIER, EL. 299'-0"

REV.	DATE	BY	DESCRIPTION
6	4-14-80	JB	REV. CONTOURS @ YARD RETENTION PUMP; ADDED TYPE "A" FLEX BASE ROAD; INDICATED FLEX. BASE BY THIS CONTR. @ ASH POND DIKE; DEFINED & NOTED AREA TO RECEIVE RIPRAP; INDICATED & NOTED FDN. TO BE REMOVED; ADD. 3
7	5-13-81	LB	REV. CONTOURS & ADDED RETAINING WALL. ADDED FLEX BASE ROAD

REV	DATE	BY	DESCRIPTION
A	5-18-77		REVISED PER ADDENDUM NO. 1
B	6-3-77	JLF	REVISED PER ADDENDUM NO. 3
C	7-1-77	JLF	REVISED PER ADDENDUM NO. 4
0	8-18-77		FINAL BID SET
1	9-23-77	R.L.H.	REVISED REF. NOTE, ADDED REF. NOTE
2	6-9-78	SMM	REVISED PIPE FOUNDATIONS
3	10-20-78	LW	CORRECTED PULLBOX LOCATION.
4	6-1-79	LB	CHG'D. 3 TO 19; REVISED ELEV. OF TRANSFORMER POWER STA. REMOVED MOTOR CONTROL CENTER FDN.; REVISED STATION COORDINATE
5	1-23-80	LB	REV. PIPE SUPPORTS AS NOTED.

SCALE 1" = 40'
DRAWN R.C.M.
DATE 4-1-77
CHECKED C.A.D.
APPROVED M.L.H., W.G.H.

STATE OF TEXAS M.L. HUGHES 25324 REGISTERED PROFESSIONAL ENGINEER
STATE OF TEXAS William G. Hollowell 15137 REGISTERED PROFESSIONAL ENGINEER
STATE OF TEXAS CHARLES A. DOBY 37593 REGISTERED PROFESSIONAL ENGINEER

TIPPETT & GEE, INC.
CONSULTING ENGINEERS
ABILENE TEXAS

SAN MIGUEL PLANT
UNIT NO. 1
B.E.P.C. S.T.E.C.

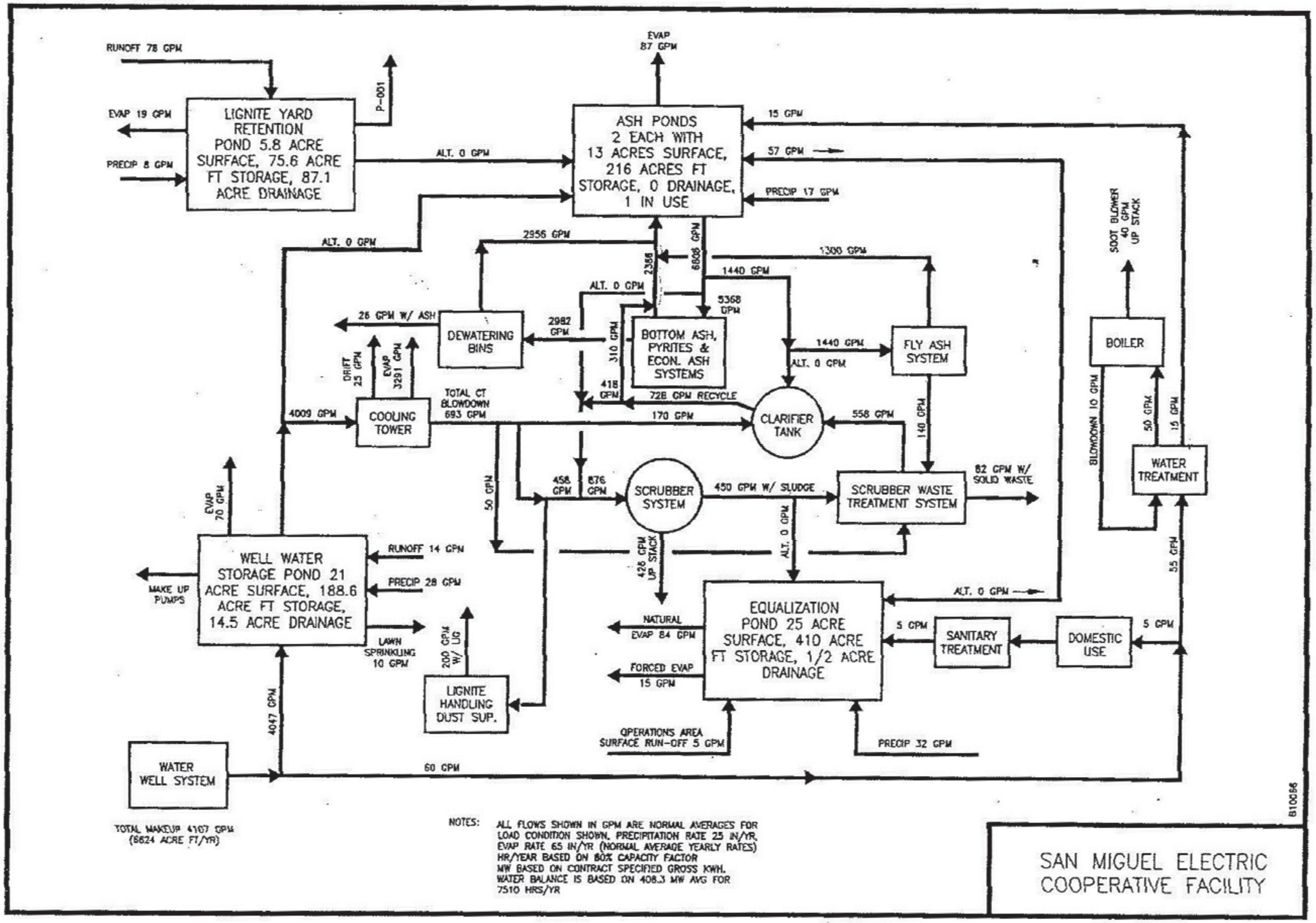
SITE PLAN
SECTION NO. 4

JOB NO. SMI-406
REV. 7
DRAWING NUMBER 1-C-33

Appendix B-9

San Miguel, 1982 Facility Water Balance, San Miguel Electric Cooperative, Inc., ca. 1982.

Environmental Resources Management
206 East 9th Street, Suite 1700
Austin, Texas 78701
(512) 459-4700


RUNOFF 78 GPM
EVAP 19 GPM
PRECIP 8 GPM
LIGNITE YARD RETENTION POND 5.8 ACRE SURFACE, 75.6 ACRE FT STORAGE, 87.1 ACRE DRAINAGE
P-001
ALT. 0 GPM
EVAP 87 GPM
ASH PONDS 2 EACH WITH 13 ACRES SURFACE, 216 ACRES FT STORAGE, 0 DRAINAGE, 1 IN USE
15 GPM
57 GPM
PRECIP 17 GPM
ALT. 0 GPM
2956 GPM
2356
6808 GPM
1300 GPM
1440 GPM
ALT. 0 GPM
5368 GPM
26 GPM W/ ASH
DEWATERING BINS
2982 GPM
310 GPM
BOTTOM ASH, PYRITES & ECON. ASH SYSTEMS
1440 GPM
FLY ASH SYSTEM
ALT. 0 GPM
SOOT BLOWER 40 GPM UP STACK
BOILER
DRIFT 25 GPM
EVAP 3291 GPM
418 GPM
728 GPM RECYCLE
4009 GPM
COOLING TOWER
TOTAL CT BLOWDOWN 693 GPM
170 GPM
CLARIFIER TANK
558 GPM
140 GPM
BLOWDOWN 10 GPM
50 GPM
15 GPM
WATER TREATMENT
EVAP 70 GPM
50 GPM
458 GPM
876 GPM
SCRUBBER SYSTEM
450 GPM W/ SLUDGE
SCRUBBER WASTE TREATMENT SYSTEM
82 GPM W/ SOLID WASTE
ALT. 0 GPM
WELL WATER STORAGE POND 21 ACRE SURFACE, 188.6 ACRE FT STORAGE, 14.5 ACRE DRAINAGE
RUNOFF 14 GPM
PRECIP 28 GPM
MAKE UP PUMPS
LAWN SPRINKLING 10 GPM
200 GPM W/ UG
426 GPM UP STACK
ALT. 0 GPM
55 GPM
NATURAL EVAP 84 GPM
FORCED EVAP 15 GPM
EQUALIZATION POND 25 ACRE SURFACE, 410 ACRE FT STORAGE, 1/2 ACRE DRAINAGE
5 GPM
SANITARY TREATMENT
DOMESTIC USE
5 GPM
LIGNITE HANDLING DUST SUP.
4047 GPM
OPERATIONS AREA SURFACE RUN-OFF 5 GPM
PRECIP 32 GPM
WATER WELL SYSTEM
60 GPM
B10066
TOTAL MAKEUP 4107 GPM (6624 ACRE FT/YR)
NOTES: ALL FLOWS SHOWN IN GPM ARE NORMAL AVERAGES FOR LOAD CONDITION SHOWN. PRECIPITATION RATE 25 IN/YR. EVAP RATE 65 IN/YR (NORMAL AVERAGE YEARLY RATES) HR/YEAR BASED ON 80% CAPACITY FACTOR MW BASED ON CONTRACT SPECIFIED GROSS KWH. WATER BALANCE IS BASED ON 408.3 MW AVG FOR 7510 HRS/YR
SAN MIGUEL ELECTRIC COOPERATIVE FACILITY

Appendix B-10

EPA, 2014	Request for Action Plan regarding San Miguel Cooperative Inc.'s San Miguel Electric Plant, United States Environmental Protection Agency, May 2, 2014.

Environmental Resources Management
206 East 9th Street, Suite 1700
Austin, Texas 78701
(512) 459-4700

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY
WASHINGTON, D.C. 20460

May 2, 2014

OFFICE OF
SOLID WASTE AND
EMERGENCY RESPONSE

VIA E-MAIL

Mr. Michael Kezar
General Manager
San Miguel Electric Cooperative
P.O. Box 280
Jourdanton, Texas 78026-0280

Re: Request for Action Plan regarding San Miguel Electric Cooperative Inc.'s San Miguel Electric Plant

Dear Mr. Kezar,

On August 30, 2012 the United States Environmental Protection Agency ("EPA") and its engineering contractors conducted a coal combustion residual (CCR) site assessment at the San Miguel Electric Cooperative Inc.'s San Miguel Electric Plant facility. The purpose of this visit was to assess the structural stability of the impoundments or other similar management units that contain "wet" handled CCRs. We thank you and your staff for your cooperation during the site visit. Subsequent to the site visit, EPA sent you a copy of the draft report evaluating the structural stability of the units at the San Miguel Electric Cooperative Inc.'s San Miguel Electric Plant facility and requested that you submit comments on the factual accuracy of the draft report to EPA. Your comments were considered in the preparation of the final report.

The final report for the San Miguel Electric Cooperative Inc.'s San Miguel Electric Plant facility is attached.

This report includes a specific condition rating for the CCR management units and recommendations and actions that our engineering contractors believe should be undertaken to ensure the stability of the CCR impoundments located at the San Miguel Electric Cooperative Inc.'s San Miguel Electric Plant facility. These recommendations are listed in Enclosure 1.

Since these recommendations relate to actions which could affect the structural stability of the CCR management units and, therefore, protection of human health and the environment, EPA believes their implementation should receive the highest priority. Therefore, we request that you inform us on how you intend to address each of the recommendations found in the final report. Your response should include specific plans and schedules for implementing each of the recommendations. If you will not implement a recommendation, please provide a rationale. Please provide a response to this request by **June 4, 2014**. Please send your response to:

Mr. Stephen Hoffman
U.S. Environmental Protection Agency (5304P)

1200 Pennsylvania Avenue, NW
Washington, DC 20460

If you are using overnight or hand delivery mail, please use the following address:

Mr. Stephen Hoffman
U.S. Environmental Protection Agency
Two Potomac Yard
2733 S. Crystal Drive
5^{th} Floor, N-5838
Arlington, VA 22202-2733

You may also provide a response by e-mail to hoffman.stephen@epa.gov, dufficy.craig@epa.gov, kelly.patrickm@epa.gov and englander.jana@epa.gov.

You may assert a business confidentiality claim covering all or part of the information requested, in the manner described by 40 C. F. R. Part 2, Subpart B. Information covered by such a claim will be disclosed by EPA only to the extent and only by means of the procedures set forth in 40 C.F.R. Part 2, Subpart B. If no such claim accompanies the information when EPA receives it, the information may be made available to the public by EPA without further notice to you. If you wish EPA to treat any of your response as "confidential" you must so advise EPA when you submit your response.

EPA will be closely monitoring your progress in implementing the recommendations from this report and could decide to take additional action if the circumstances warrant.

You should be aware that EPA will be posting the report for this facility on the Agency website shortly.

Given that the site visit related solely to structural stability of the management units, this report and its conclusions in no way relate to compliance with RCRA, CWA, or any other environmental law and are not intended to convey any position related to statutory or regulatory compliance.

Please be advised that providing false, fictitious, or fraudulent statements of representation may subject you to criminal penalties under 18 U.S.C. § 1001.

If you have any questions concerning this matter, please contact Mr. Hoffman in the Office of Resource Conservation and Recovery at (703) 308-8413. Thank you for your continued efforts to ensure protection of human health and the environment.

Sincerely,
/Barnes Johnson /, Director
Office of Resource Conservation and Recovery

Enclosures

Enclosure 1

San Miguel Electric Cooperative Inc.'s San Miguel Electric Plant Recommendations (from the final assessment report)

CONCLUSIONS

Conclusions Regarding Structural Soundness of the CCW Impoundments

Structural stability documentation appears to be adequate. A geotechnical report, prepared by Arias & Associates, Inc. (Arias), was provided, and it included slope stability analyses for all required load conditions, with the exception of rapid drawdown and liquefaction. Because the impoundments do not include spillways or overflow structures, and liquids are pumped over the embankments, rapid drawdown conditions were considered only likely in the event of a breach. The potential for liquefaction is considered unlikely due to the subsurface soil conditions and low seismic hazard level.

Slope stability analyses were provided for steady-state seepage, maximum surcharge pool, and seismic conditions, as well as the assessment for liquefaction potential. In general, slope stability safety factors for load conditions analyzed are satisfactory.

Conclusions Regarding the Hydrologic/Hydraulic Safety of CCW Impoundments

No hydrologic and hydraulic information was provided by San Miguel to indicate CCW impoundments hydrologic/hydraulic safety. A target pool elevation of at least 18 inches of freeboard at both the Ash Pond and Sludge Basin was the only hydraulic information provided by San Miguel. During the site visit, both ponds were below the target pool elevation. Because no hydrologic/ hydraulic documentation was provided, the hydrologic/hydraulic safety is judged to be inadequate.

Conclusions Regarding Adequacy of Supporting Technical Documentation

Supporting data and documentation for the Ash Pond and Sludge Basin includes required structural stability analyses for normal operating pool, steady state conditions; maximum surcharge pool condition; and normal operating pool under seismic loading conditions. An assessment of liquefaction potential was also provided, with the conclusion that liquefaction is considered to be very unlikely based on existing subsurface soil conditions and the stated 6% chance of a seismic event of a magnitude 5.0 or greater occurring over a 250-year period. Technical documentation of the embankment stability under a sudden drawdown loading condition was not provided because rapid drawdown conditions were considered only likely in the event of a breach. CDM Smith agrees with the rationale provided regarding embankment stability, liquefaction potential, and rapid drawdown conditions. Supporting documentation for structural stability is considered to be adequate.

Because no supporting data or documentation was provided for hydrologic/hydraulic safety of the impoundments, it is considered to be inadequate.

Conclusions Regarding Description of the CCW Impoundments

The record drawings and descriptions of the CCW impoundments provided by San Miguel representatives appear to be consistent with the visual observations by CDM Smith during site assessment.

Conclusions Regarding Field Observations

During visual observations and site assessments, CDM Smith observed an area of potential seepage near the toe of the Ash Pond's west embankment, erosion rills on the interior and exterior slopes of the Ash Pond embankments and several rodent burrows on the crest and exterior slope of the Ash Pond embankments. An area of erosion, approximately 5 feet wide, was also observed on the interior slope of the Ash Pond's east embankment. According to San Miguel representatives this erosion was a result of leakage from a water well pipe traversing the Ash Pond embankment. The water well pipe had been repaired at the time of the site assessment. Soils had eroded or settled from under the Sludge Basin's stormwater inlet structure. Other observations of the Sludge Basin embankments included erosion rills on west embankment interior slope and an area of erosion on the interior slope of the west embankment, near the submersible pump outlet structure.

Conclusions Regarding Adequacy of Maintenance and Methods of Operation
Current maintenance and operation procedures appear to be generally adequate.
There was documentation regarding seepage at the Ash Pond in the 1980s. The pond liner was reconstructed in 1987, but an area of potential seepage was observed during the CDM Smith site assessment in the vicinity of one of the areas that had documented seepage in the 1980s. There was no evidence of previous spills or release of impounded liquids outside the plant property.
Conclusions Regarding Adequacy of Surveillance and Monitoring Program
Surveillance and monitoring procedures include weekly checks of the impoundments by the Plant Environmental Engineer for leaks or deficiencies, and recording pool levels for both the Ash Pond and Sludge Basin. Additionally, level gages are checked six times daily by the operations department.
Instrumentation for the Ash Pond and Sludge Basin consists of local level gages, used by operations to record impoundment levels. In addition to the current surveillance and monitoring program, the area of potential seepage at the west embankment exterior slope of the Ash Pond should be monitored.
Because of the erosion into the Ash Pond's east embankment slope from a leaking pipe, the surveillance and monitoring program should be revised to include more-detailed inspections.
Conclusions Regarding Suitability for Continued Safe and Reliable Operation
Main embankments do not show evidence of unsafe conditions requiring immediate remedial efforts, although maintenance to correct deficiencies noted above is required.
As described by San Miguel representatives operating procedures for the Ash Pond and Sludge Basin include methods of controlling the water levels in the lagoons, but no formal documentation was provided to CDM Smith.

RECOMMENDATIONS
Recommendations Regarding the Hydrologic/Hydraulic Safety
It is recommended that a qualified professional engineer determine the required flood frequency and evaluate the hydrologic and hydraulic capacity of the CCW impoundments to withstand design storm events without overtopping.
Recommendations Regarding the Technical Documentation for Structural Stability
It is recommended that a qualified professional engineer reevaluate the impoundments for structural stability should conditions from those included in the Arias & Associates, Inc. structural stability analyses change.
Recommendations Regarding Field Observations
CDM Smith recommends corrective actions be taken for the specific conditions identified below:

- Erosion rills – Erosion rills were observed on the interior slopes of the Sludge Basin and the interior and exterior slopes of the Ash Pond. Structural fill should be placed and compacted in the rills and graded to adjacent existing contours. The area should be sodded or reseeded.
- Surface erosion - Structural fill should be placed and compacted, graded to adjacent existing contours, and sodded or reseeded. Alternatively, riprap or other armoring could be used. Riprap or other armoring is recommended for the west, north, and east interior slopes to reduce the potential for erosion.
- Rodent burrows - Rodent burrows were observed on the crest and exterior embankment of the Ash Pond. Although not seen on other embankments, vegetation cover may have hidden additional rodent burrows. CDM Smith recommends San Miguel accurately document areas disturbed by animal activity, remove the animals, and backfill the burrows with compacted structural fill to protect the integrity of the embankments.

- Potential seepage area - CDM Smith observed an area of potential seepage at the west embankment exterior slope of the Ash Pond. CDM Smith recommends San Miguel take the following actions:
    - Cut back and maintain vegetation in the area to facilitate monitoring the condition
    - Develop a regular surveillance program to monitor areas of seepage and potential seepage to measure the rate, volume, and turbidity of flow emerging from the embankment slope; and
    - Develop and execute a geotechnical exploration program that includes additional test borings and installation of piezometers and other instrumentation to analyze and regularly monitor embankment seepage and stability.

Recommendations Regarding Surveillance and Monitoring Program

Monitoring for potential seepage at the exterior embankment slopes is recommended for both the Ash Pond and Sludge Basin considering historical issues with seepage. Potential areas of seepage may be more readily assessed after clearing of trees and dense vegetation on embankment slopes. It is recommended that vegetation on the impoundment embankments be maintained with seasonal mowing, as necessary, for animal control and surveillance and monitoring of embankments.

Recommendations Regarding Continued Safe and Reliable Operation

Inspections should be made following periods of heavy and/or prolonged rainfall, and the occurrence of these events should be documented. Inspection procedures should be documented and inspection records should be retained at the facility for a minimum of three years.

Major repairs and slope restoration should be designed by a registered professional engineer experienced with earthen dam design.

None of the conditions observed require immediate attention or remediation, however, the above recommendations should be implemented to maintain continued safe and reliable operation of the CCW impoundments.

Appendix B-11

HDR, 2015	Hydrologic and Hydraulics Report for Coal Combustion Waste Impoundments, San Miguel Electric Cooperative, Inc., HDR Engineering, Inc., March, 2015.

Technical Memorandum

Date: April 28, 2015

Project: H&H Study and Geotechnical Exploration Program – Ash Ponds – San Miguel Electric

To: Lane William – San Miguel Electric

From: Rolland Boehm, PE; Dave Vogt, PE

STATE OF TEXAS
ROLLAND GENE BOEHM
101598
LICENSED PROFESSIONAL ENGINEER
4/28-15

Subject: **Embankment Seepage – Ash Pond "A"**

Information and Background

San Miguel Electric Cooperative (SMEC) is the owner and operator of an electric generating plant near Christine, Texas. The facility includes two ash ponds (A & B) and an equalization pond. The two 13-acre ash ponds are located south of the electric generating plant, where as the 25-acre equalization pond is located southeast. By EPA definition, all three of the ponds are considered impoundments of Coal Combustion Residuals (CCRs).

SMEC retained HDR in September of 1014 to perform an H&H study of the three ponds, as well as to develop a plan to assess seepage at the toe of the westerly embankment of Ash Pond A. The seepage was first noted in August of 2012, and formally documented in an EPA assessment report that was completed by CDM Smith and issued final to SMEC in May of 2014. Since that time, SMEC has cut back the vegetation in this area, as well performed regular surveillance monitoring for changed conditions (seepage rate, volume, turbidity).

Based on the findings presented in the assessment report, EPA made certain action requests of SMEC. The requests were outlined in a letter to SMEC, dated May 2, 2014. In response, SMEC commented to these requests, which included complying with a request to perform an H&H study on the ponds, as well as develop and execute a geotechnical exploration program in the seepage area. In general, the geotechnical program is to include test borings, installation of piezometers and other instrumentation to analyze and regularly monitor embankment seepage and stability.

HDR's has developed a geotechnical exploration plan on behalf of SMEC, as described herein. The plan materially satisfies the development portion of SMEC's commitment to the EPA. Its actual execution could be included as a component of much larger effort needed to satisfy current CCR regulations, which were recently promulgated under the Resource Conservation and Recovery Act (RCRA).

Record Drawings

SMEC provided HDR with several record drawings of the ash ponds, dated May 28, 1996. Based on these drawings, the westerly portion of the embankment associated with Ash Pond A has the following general configuration:

- Sideslopes = 2.5H:1V (interior and exterior)
- Crest Width = 40 feet
- Height Above Outside Toe ≈ 12 feet
- Height Above Interior Excavated Bottom Grade = 20 feet (per design)
- Composition: Fill and Undisturbed Native Soil (pond partially developed by excavation)

Soil Conditions

Arias & Associates completed a Geotechnical Engineering Study of the ash pond embankments in 2012. The study was performed as part of the site assessment that was completed by CDM Smith, and included by appendix in their final report. The study included a 39-foot deep soil boring on top of the west embankment in the area of interest. The boring encountered the following soil profile:

Depth (ft)	Layer Thickness (ft)	Soil Type	Remarks
0 to 1	1	silty Gravel (Base Course)	Access Road
1 to 5	4	stiff to very stiff Lean Clay	Fill
5 to 22	17	very stiff to hard Fat Clay	Native – gypsum seam noted a 10 feet*
22 to 28	6	hard Lean Clay	Native
28 to 39	11	very dense Silty Fine Sand	Native

*Seepage exit area appears to be located in native Fat Clay soils, possibly along gypsum seam.

Recent Site Visit

HDR personnel visited the site on October 7, 2014. Observations of the seepage area were made as part of this visit. The following is a list of these observations:

- Wet soils along the toe for approximately 50-60 lineal feet
- Wet soils appear to creep 3 to 4 feet up the slope in certain discrete areas
- Few cattails and wet soil type grasses present
- Scrub trees growing on the slope were recently removed, though roots still remain
- Water was ponded in small isolated low areas along approximately 15 lineal feet of the toe
- Soil within the small ponded areas generally very soft
- Ponded water was clear and not flowing
- No indication seepage water has recently flowed beyond the immediate toe area.
- No signs of slope distress or material piping

Recommended Geotechnical Exploration Program

The following Geotechnical Exploration Program is developed and recommended based on the available information, site observation, and experience.

1. Conduct two (2) exploratory soil borings on top of the embankment crest, immediately above the seepage area. One boring to be extended to a depth of 35 feet and the other boring to a depth of 12 feet.

2. Perform six (6) to eight (8) shallow borings along the toe of the slope to better define the limits of impact. Borings to extend approximately 5 feet in depth or until firm non-impacted soils are encountered.

3. Collect soil samples at 2-1/2 foot intervals for 35-foot deep boring and continuous in all other borings. Thin walled tube sampling to be used in the cohesive soils and split-spoon sampling in granular or non-cohesive soil.

4. Perform detailed logging during completion of the borings. Including visual classification of the encountered soils, density or consistency of the soils, presence of seams, fissures or

slickensides, the moisture condition of the soil, free flowing water, borehole caving, and other pertinent drilling and sampling observations.

5. Install groundwater piezometers in the two deeper boreholes (i.e. competed on the embankment). Piezometers to be constructed of 2-inch diameter Schedule 40 PVC riser pipe and slotted well screen. The deeper piezometer to be installed with a 20-foot section of slotted screen, whereas the shallower piezometer to include a 5-foot section of slotted screen. In both cases, the screen slots will be 0.010 inch in width. Additionally, piezometer installation to include a proper sand filter pack, bentonite seal plug, encasement grout, and above-grade protection. Each piezometer to be developed by purge and surge method to remove fines that may have collected within the piezometer or slotted screen during the installation period.

6. Record depth to water in each piezometer several days after installation (and development) and then routinely (e.g. weekly) until water levels appear to stabilize.

7. Place tracer dye in the shallow piezometer. Observe when, or if, tracer dye reaches seepage area.

8. Perform laboratory testing on select soil samples. Testing to include moisture content, Atterberg limits, grain size, unconfined compressive strength, and undrained triaxial compression (with pore water pressure measurement).

9. Evaluate the above information to determine the lateral extent of the seepage area, depth to groundwater, line seepage or preferential seepage paths thru the embankment, and shear strength of the soil (toe and embankment).

10. Perform stability analyses to determine the current factor of safety against a slope failure. Compare computed factor of safety with the criteria indicated in the new CCR regulations.

11. Develop corrective action alternatives, e.g. reconstruct a portion of the embankment, seepage interception or control, ballast material placement at the toe, etc.

12. Summarize all findings and recommendations in a Geotechnical Engineering Study Report, sealed by an engineer licensed in the State of Texas.

Interim Monitoring

In the meantime, it is recommended that personnel from SMEC continue to monitoring the seepage area on a routine basis, making note of seepage quantities, the size and depth of inundated areas, the turbidity of the seepage water emerging from the embankment slope, and any noticeable signs of slope distress. Keeping the vegetation in the immediate area cut back and well maintained will aid in making these observations.

Should conditions appear to worsen, then it is further recommended that an engineering firm be consulted immediately to determine if a temporary, though quickly implementable measure would be necessary to stabilize the area.

STATE OF TEXAS
RODRIGO VIZCAINO
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LICENSED
PROFESSIONAL ENGINEER

4/9/15

HYDROLOGIC AND HYDRAULICS REPORT for

Coal Combustion Waste Impoundments

Prepared for:

San Miguel Electric Cooperative, Inc.

Prepared by:

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1.0 PURPOSE & OBJECTIVE

The purpose of this report is to provide documentation of an analysis of the hydrology and hydraulics associated with three of the Coal Combustion Waste (CCW) Impoundments of the San Miguel Electric Plant.

This study is in response to an Environmental Protection Agency (EPA) site inspection on August 30, 2012. As shown in **Appendix A**, the EPA's letter dated May 4, 2014 included a recommendation that a qualified professional engineer determine the required flood frequency and evaluate the hydrologic and hydraulic capacity of the CCW impoundments to withstand design storm events without overtopping. The objective of the study is to estimate the maximum water surface elevation in each impoundment that would result from occurrence of the design storm. The capacity of the impoundments will be evaluated to see if they can withstand the design storm without overtopping.

1.1 Background and Scope of Study

There are two ash impoundments situated to the south of the electric plant and each has an approximate surface area of 13-acres. The excess ash from the electric plant is deposited in these impoundments and the water in the impoundments is used to cool the plant by a method of recirculation. The two ash impoundments are connected at the eastern end via a small weir structure and most of the ash is kept only in one of the impoundments. This ensures that there is always capacity to store water in one and availability to clear the excess ash in the other. The two ash impoundments and equalization basin were built in 1977, and copies of the available record drawings are in **Appendix A**. These are described below:

- Ash Impoundment 1: 13 acre surface, 216 acre-ft maximum storage capacity

- Ash Impoundment 2: 13 acre surface, 216 acre-ft maximum storage capacity

- Equalization Impoundment: 25 acre surface, 410 acre-ft maximum storage capacity

These are the total impoundment storage volumes and include the volume up to the normal pool and the flood storage volume from the normal pool to the top of berm. The following items summarize the general tasks and goals performed during the study under the scope of work.

1. Determine design storm rainfall and estimate the hydrologic parameters of each sub basin, including the time of concentration, unit hydrograph and loss rate parameters.

2. Develop the elevation – area – storage – discharge relationship for each of the 3 impoundments using the latest available topography and record drawings of the facility.

3. Prepare a HEC-HMS model of the 3 impoundments, and apply the design storm rainfall depths for a range of storm durations to determine the critical storm duration for the impoundments.

4. Compare the maximum water surface elevation to the top of embankment elevation for each of the 3 impoundments to calculate the amount of freeboard.

1.2 Assumptions

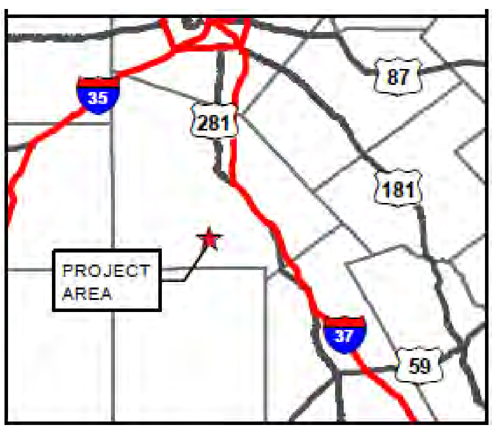
The report and scope of work do not include:

- An evaluation of compliance with the design criteria in Texas Commission of Environmental Quality (TCEQ)'s "Design and Construction Guidelines for Dams in Texas", RG-394 dated August 2009.
- A breach analysis, downstream inundation mapping or any additional effort to develop an Emergency Action Plan for the impoundments.
- A detailed geotechnical analysis of the embankments or bed.
- Any subsurface drainage analysis or evaluation of seepage.
- Any revisions to a FEMA Flood Insurance Rate Map.
- Consideration of a higher viscosity of liquid contained within the impoundments.
- Topographic surveying.
- A hydrologic and hydraulic analysis of the Lignite Yard Retention Impoundment or the Well Water Storage Impoundment.

1.3 Site Location, FEMA Map, and Data Collection

The San Miguel Electric Cooperative facility is located in unincorporated area of Atascosa County, which has a Census 2010 population of 44,911. It is located between IH-35 and IH-37 near Christine, Texas as shown in **Figure 1.**

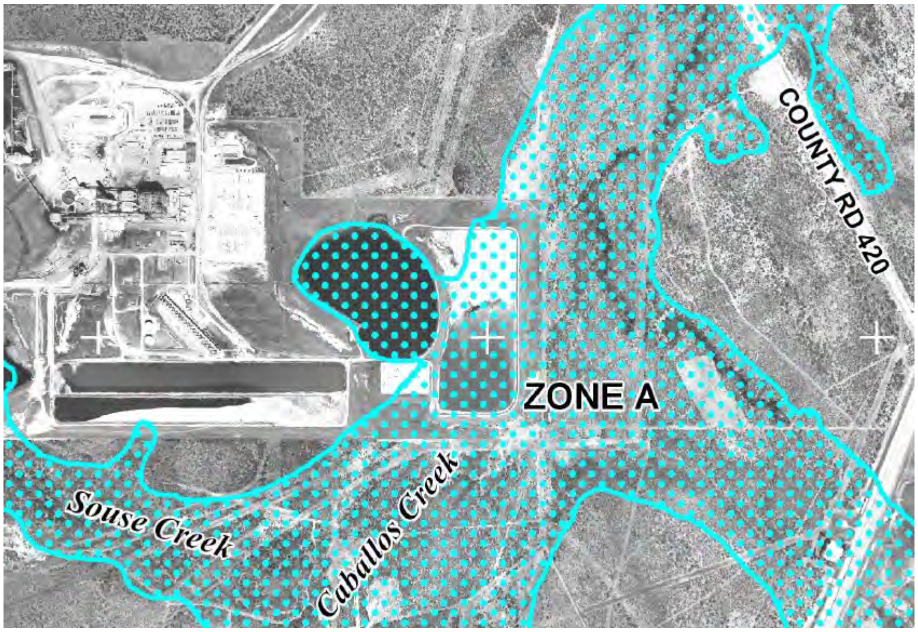
Figure 1: Location Map – San Miguel Electric Plant



Caballos Creek is located immediately south of the impoundments. The Creek's floodplain is currently mapped as a FEMA flood hazard area (Zone A). This means that there is no detailed hydraulic study for the stream completed to date and the floodplain is an approximation.

Caballos Creek generally flows northeast and is depicted along with the San Miguel Plant on FEMA Flood Insurance Rate Map (FIRM) No. 48013C0675C. The FIRM is located in **Appendix B** for reference. It is important to note the Zone A 100-year event (1% AEP) floodplain delineation shows the equalization impoundment (eastern most impoundment) as within the floodplain zone. This is believed to be an inaccurate delineation, but can only be corrected via a more detailed hydraulic study of Caballos Creek along with a Letter of Map Revision (LOMR) application to FEMA (these services are not part of this scope of work). See **Figure 2** (full FEMA FIRM Map provided in **Appendix B**).

Figure 2: FEMA Zone A Delineation



The **data collected** for this task are as follows:

- FEMA Map - FIRM data (located in **Appendix B**).
- Available impoundments as-built sheets and plant water balance diagram (both located in **Appendix B**).
- USGS topographic map with 10' contours (1961 USGS), and recent aerial images (2012 NAIP).
- Rainfall Data – USGS Atlas of Depth Duration Frequency Precipitation for Texas and TCEQ HMR 51 (both in **Appendix A**)
- Soil Data for Atascosa County based on a 2014 web soil survey (**Appendix A**)

The following data were provided by the San Miguel Electric Cooperative (available in **Appendix B**):

- Plant Site Plan and Vicinity Map
- Site Preparation Sections and Details.
- Water Balance Diagram
- Sludge Disposal Basin Plans

USGS 10’ contour data was used for the hydrologic analysis and aerial photography was used to identify drainage patterns as well as dimensions of each impoundment structure. The coordinate system used for this study is the North Central Texas State Plane 4202 NAD 83.

1.4 Site Reconnaissance Visit

A site visit was performed on October 9, 2014. The main objective of the visit was to understand the functionality of the impoundments as well as to note localized drainage patterns and floodplain proximity. A photo log of the visit has been developed and is located in **Appendix C**.

1.5 Applicable Regulations

The TCEQ Sunset Legislation, House Bill 2694 from the 82nd Texas Legislative Session amended §12.052, Subsection (a) of the Texas Water Code (TWC) and added Subsections (b-1), (e-1), (e-2) and (e-3). Under TWC §12.052 (e-1), beginning on September 1, 2013 owners of dams located on private property are exempt from meeting TCEQ dam safety requirements if the dam impounds less than 500 acre-feet at maximum capacity, has a hazard classification of low or significant, is located in a county with a population of less than 350,000 and is not located inside the corporate limits of a municipality. The San Miguel Electric Cooperative facility is located in Atascosa County unincorporated area, which has a Census 2010 population of 44,911, and there are no habitable structures downstream, which results in a low hazard classification.

Based on the facts above, under TWC §12.052 (e-1), the CCW impoundments are exempt from TCEQ dam safety regulations. As such, HDR will not evaluate compliance with the design criteria in TCEQ’s “Design and Construction Guidelines for Dams in Texas”, RG-473 dated August 2009. Dam owners still have to comply with maintenance and operation requirements, and there is no exemption expiration date.

The CCW impoundments are also subject to the MSHA regulations at 30 CFR § 77.216-2, for water, sediment, or slurry impoundments and impounding structures, which include minimum plan requirements, including changes or modifications. The MSHA regulations require a certification by a registered engineer that “the design of the impounding structure is in accordance with current, prudent engineering practices for the maximum volume of water, sediment, or slurry which can be impounded therein and for the passage of runoff from the designed storm which exceeds the capacity of the impoundment; or, in lieu of the certification, a report indicating what additional investigations, analyses, or improvement work are necessary before such a certification can be made, including what provisions have been made to carry out such work in addition to a schedule for completion of such work.” Providing this certification is beyond the scope of the current analysis.

The EPA is currently under a rule making progress to regulate for the first time, coal combustion residuals (CCRs) under the Resource Conservation and Recovery Act (RCRA) to address the risks from the disposal of CCRs generated from the combustion of coal at electric utilities and independent power producers, as described in Docket ID No. EPA-HQ-RCRA-2009-0640. The proposed rule requires a certification similar to the current MSHA regulations. Once this rule is promulgated, it will require a separate study and report that is signed by a Professional Engineer in Texas to ensure those specific requirements are evaluated.

The hydrologic and hydraulic analysis will follow the design criteria in TAC Chapter 299, and the design storm methodology in Chapters 4, 5 and 6 of TCEQ's Hydrologic and Hydraulic Guidelines for Dams in Texas, GI-364 dated January 2007 (**TCEQ GI-364**). A copy is available in **Appendix B**.

1.6 Size and Hazard Classification

The Texas Administrative Code (TAC) classifies dams to assure appropriate safety considerations. The three size classifications (small, intermediate and large), based on height of dam or impoundment capacity, and the three hazard classifications (low, significant and high), are combined to indicate a dam's downstream hazard potential. Thus, the classification assignment reflects the hazard potential associated with assumed failure of the dam. For example, dams located such that resulting failure could be catastrophic are classified so as to require a higher degree of design consideration than would be required for similar dams located in remote areas. Classification does not indicate the physical condition of a dam.

Subchapter B of Section 299 of the TAC (copy located in **Appendix B**) lists the following three size classifications, based on the height of the dam or maximum reservoir storage capacity as shown in **Table 1** below. The appropriate size is the largest category determined for either storage or height. With a total maximum storage volume of 842 acre-ft and a maximum height of 24 ft for all 3 impoundments, the San Miguel CCW Impoundments are clearly in the small category.

Table 1: Size Classifications

Category	Storage (Ac-Ft)	Height (Ft.)
Small	Less than 1000	Less than 40
Intermediate	Equal to or Greater than 1000 & less than 50,000	Equal to or Greater than 40 & less than 100
Large	Equal to or Greater than 50,000	Equal to or Greater than 100

Subchapter B of Section 299 of the TAC lists the following three hazard potential classifications, as shown in **Table 2** below. Hazard classification pertains to potential loss of human life and/or property damage within either existing or potential developments in the area downstream of the dam in event of failure or malfunction of the dam or appurtenant facilities. Hazard classification does not indicate any condition of the dam itself. Dams in the low hazard potential category are normally those in rural areas where failure may damage farm buildings, limited agricultural improvements and county roads. Significant hazard potential category dams are usually those in predominantly rural areas where failure would not be expected to cause loss of human life, but may cause damage to isolated homes, secondary highways, minor railroads, or cause interruption of service or use (including the design purpose of the facility) of relatively important public utilities. Dams in the high hazard potential category are usually those in or near urban areas where failure would be expected to cause loss of human life, extensive damage to agricultural, industrial or commercial facilities, important public utilities (including the design purpose of the facility), main highways or railroads. With no habitable structures located downstream and only limited agricultural lands and county roads downstream, the San Miguel CCW impoundments are clearly in the low hazard category.

Table 2: Hazard Potential Classifications

Category	Loss of Human Life	Economic Loss
Low	None expected (No permanent structures for human agricultural improvements)	Minimal (Undeveloped to occasional structures or habitation)
Significant	Possible, but not expected (A small number of inhabitable structures)	Appreciable (Notable agricultural, industrial or commercial development)
High	Expected (Urban development or large number of inhabitable structures)	Excessive (Extensive public, industrial, commercial or agricultural development)

1.7 Design Storm

TAC §299.1 provides the hydrologic criteria listed in **Table 3**, which are the minimum acceptable spillway design flood (SDF) for proposed dams, including those to be constructed in accordance with Texas Water Code, §11.142. Per **Table 3**, the small low hazard impoundments are required to pass 25% of the Probable Maximum Flood (25% PMF).

Table 3: Hydrologic Criteria for Dams

Hazard	**Size**	**Minimum Design Hydrograph**
Low	Small Intermediate Large	25% PMF 25% PMF to 50% PMF 100% PMF
Significant	Small Intermediate Large	25% PMF to 50% PMF 50% PMF to 100% PMF 100% PMF
High	Small Intermediate Large	100% PMF 100% PMF 100% PMF

1.8 Computer Programs

The following computer programs were used as part of this study to calculate and analyze the hydrology and hydraulics at the project site.

- The USACE's HEC-HMS version 3.5 (Aug. 2010) was used for hydrologic computations.
- ESRI's ArcGIS 10.1 (2012) was used for mapping and topographic analysis.

2.0 IMPOUNDMENT HYDROLOGIC ANALYSIS

The following sections provide a detailed description of existing hydrologic conditions and methodologies used to evaluate the maximum pool elevation for each impoundment. The three CCW impoundments are not connected to the “storm water” impoundments at the site. They do not have any spillways since they are not designed to release any excess water or ash to any offsite area.

2.1 Ash Impoundments Description and Functionality

There are two ash impoundments situated to the south of the electric plant and each has an approximate 13-acre surface area. The ash carryover from the Dewatering Basins and Hydroveyor System at the plant is deposited in these impoundments and the water in the impoundments is used to cool the plant bottom ash hopper by recirculation. The two ash impoundments are connected at the eastern end via a small weir structure and most of the ash is kept only in one of the impoundments. This ensures that there is always capacity to store water in one and availability to clear the excess ash in the other. During the site visit, the plant operators mentioned that an 18-inch minimum freeboard from the top of the impoundment embankment is always maintained in the ash impoundments and equalization basin. The **Water Balance Diagram (Appendix B**) helps to further explain the re-circulation process among the plant and its impoundments.

2.2 Equalization Impoundment Description and Function

The equalization impoundment has a 25-acre surface area and it provides relief to the ash impoundments in the event that these become saturated with too much excess ash or water. The equalization impoundment also recirculates water back to the ash impoundments as needed and also maintains the minimum 18-inch freeboard.

2.3 Rainfall and Probable Maximum Precipitation (PMP)

A design storm analysis was completed in order to evaluate the water surface elevations for the design storm flood (or 25% of the PMP) at the three impoundments.

A temporal distribution of the Texas Hydro-Meteorological Report (HMR 51) was developed and followed the values according to the schema provided in the TCEQ GI-364. There was no evidence of a site-specific PMP study performed. Since the contributing drainage area to each of the three impoundments is less than ten (10 square miles), there is no aerial reduction for the spatial distribution of the PMP (as indicated in TCEQ GI-364). The HMR 51 PMP 72-hour duration storm has a total precipitation depth of 52.72 inches. The design storm depths were calculated based on 1-hour intervals for the following range of durations: 1, 2, 3, 6, 12, 24, 48, and 72 hours. These design storm depths were then used in the analysis to determine the critical storm duration for the impoundments. The minimum design storm duration (per Table 4.1 of TCEQ GI-364) is 1 hour.

As a comparative analysis, a 500-year storm design event (0.2% AEP) was evaluated. However, this depth is only for information purposes and does not have a bearing in the determination of a Probable Maximum Flood as dictated by TCEQ GI-364. The 500-year 24-hour rainfall depth was obtained from the intensity-duration-frequency (IDF) rainfall data in the USGS Atlas for Texas (Atascosa County). The San Antonio River Authority (SARA) also published (2004) similar IDF rainfall values. These depths are very similar to those published in the USGS Atlas. Therefore, only the USGS Atlas values were used in the analysis.

The various rainfall values used (HMR 51, USGS, SARA) are shown in **Appendix A**.

2.4 Hydrology

The drainage area draining to each CCW impoundment was delineated using the San Miguel Plant Sludge Disposal Basin plans (C-12) located in Appendix B along with 10' USGS contours and aerial imagery in GIS. The area delineated for the inflow hydrograph to the Ash Impoundments was 31.9 acres. The ash impoundments outer rim area is 27 acres so there are 5 acres of dry land draining into the ash water impoundments. A drainage area of 28.5 acres was delineated for the equalization impoundment. The equalization impoundment has a dry area of 6.5 acres draining into the impoundment and an outer rim area (per normal pool elevation) of 22 acres. The **Drainage Area Map** can be viewed within **Appendix A.**

The SCS (NRCS) Unit Hydrograph Method was used to compute the inflow hydrograph contributing to each impoundment. The SCS Curve Number (CN) Method was used to calculate excess precipitation and runoff from each contributing sub-basin by accounting for initial abstraction, land use, soil cover, and impervious cover percentages. A weighted CN was calculated based on the computed soils and ground cover. The majority of the ground cover in the impoundments is water, which was modeled as 100% impervious. The small dry area draining to the ash water impoundment is treated as an independent sub-basin and had a CN of 88 with a land cover of poor grass and the majority of the soil values falling into a soil group D. For the equalization impoundment, the dry land cover was also considered an independent sub-basin and had CN of 89 (soil group D with a poor grass cover embankment). The **Curve Number calculations summary** and soil map is provided in **Appendix A**.

The NRCS unit hydrograph method was used to compute the inflow hydrograph to the impoundments. The NRCS Technical Release 55 Time of Concentration (Tc) Method was used to estimate the lag time (TLag) for the contributing drainage area of each impoundment. With this method, Tc is the summation of sheet flow, shallow concentrated flow, and channel flow travel times. TLag is calculated as 60% of the Tc. For the sub-basin represented by the water in each impoundment, the assumption was that rain falling directly on the water in the impoundments does not have a lag. The dry areas resulted in very small Tc's due to their size. The Tc calculation summary is available in **Appendix A.**

Table 4 provides a summary of the hydrologic parameters used to compute the flows at each impoundment.

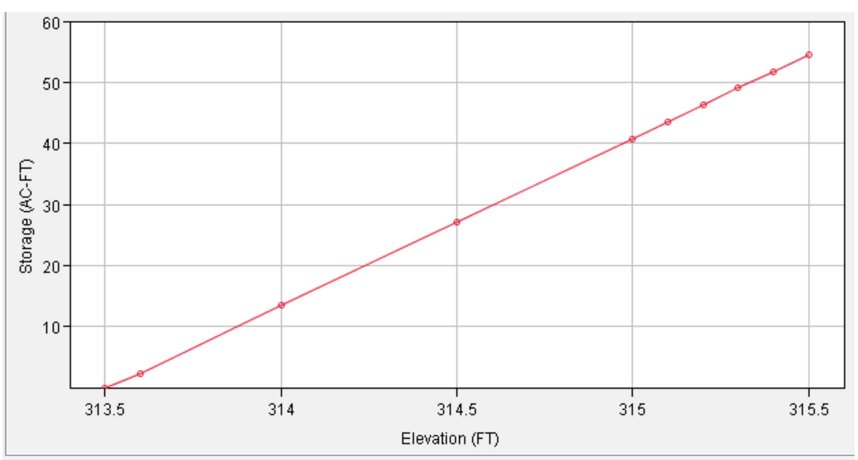
Table 4: Impoundment Hydrologic Parameters Summary

Basin	Area (acres)	Lag Time = 06.Tc (min)	Soil Groups	Weighted Observed Curve Number
Ash Impoundment – Sub-Basin 1	4.9	0.2	C/D	88
Ash Impoundment – Sub-Basin 2	27.0	None	None	100% Impervious
Equaliz. Impoundments – Sub-Basin 3	6.5	0.7	C/D	89
Equaliz. Impoundments – Sub-Basin 4	22.0	None	None	100% Impervious

2.5 Ash Impoundments Hydrologic Analysis and Modeling

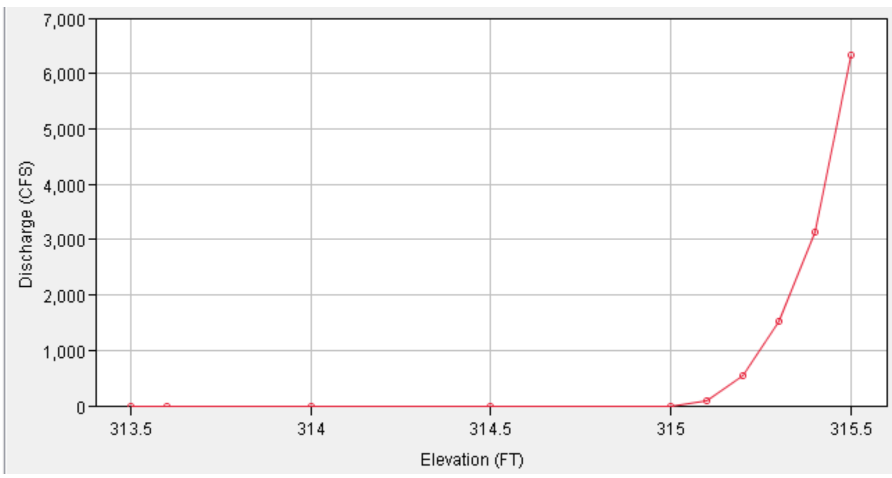
The two ash Impoundments (built in 1977) were modeled as one reservoir since they are hydraulically connected. The storage was assumed to be based on a starting water surface elevation at 18-inches of freeboard (above the normal pool) to the top of berm since this is the normal maintained elevation by the San Miguel Plant. The two ash impoundments have a combined flood storage capacity in the 18 inches of freeboard of 40.7 acre-ft . A stage-storage-discharge relationship was developed using available as-built reservoir plans in order to model the ash impoundments. **Figure 3** shows a graphical representation of the elevation versus storage relationship used in the hydrologic modeling. It is based on the impoundments' physical dimensions and has a maximum flood storage at elevation 315 (18 inches above the water pool elevation of 313.5). Note that 6 inches of additional depth above the berm were assumed for model computational purposes in the event that there is overtopping, so the curve extends to a total flood storage of 54.5 acre-feet at elevation 315.5.

Figure 3: Ash Impoundments – Stage vs. Storage Relationship



A discharge versus stage relationship was calculated using the two impoundments. The starting water surface elevation for the analysis was 313.5. The overflow berm areas were assumed to function as a broad crested weir with a crest elevation at 315.0 ft (40.7 acre-ft storage). The two ash impoundments do not have a spillway or any type of discharge structure. The assumption was made that overflow would occur along the entire perimeter of the berms since there is no available ground survey to confirm a low area of possible discharge. It is unlikely that this overflow would occur simultaneously over the entire perimeter. The modeling assumption was to start with 1,000 feet of weir length for the first 0.1' stage over the top of berm. The weir length was increased as the stage increases until elevation 315.5 at which the total impoundment combined perimeter length of 5,800 linear feet is active. **Figure 4** shows the stage- discharge relationship that was used to model the berm overflow. The curve shows a maximum discharge of 6,330 cfs at stage 315.5 ft.

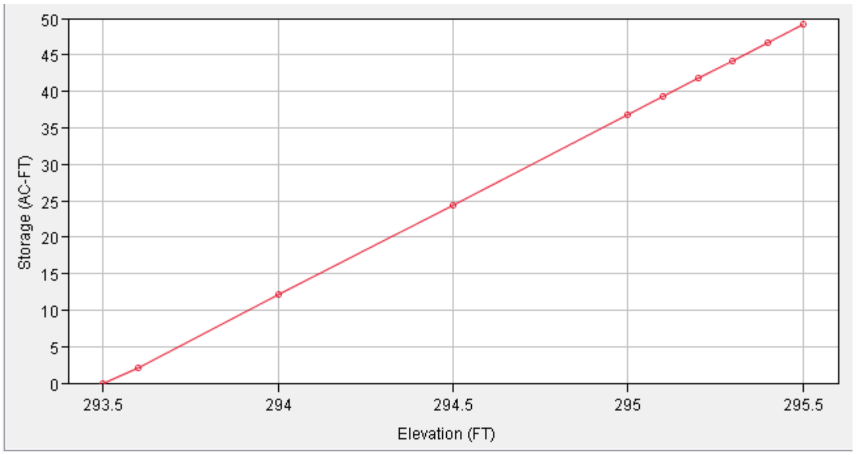
Figure 4: Ash Impoundments Stage vs. Discharge relationship



2.6 Equalization Impoundment Hydraulic Analysis and Modeling

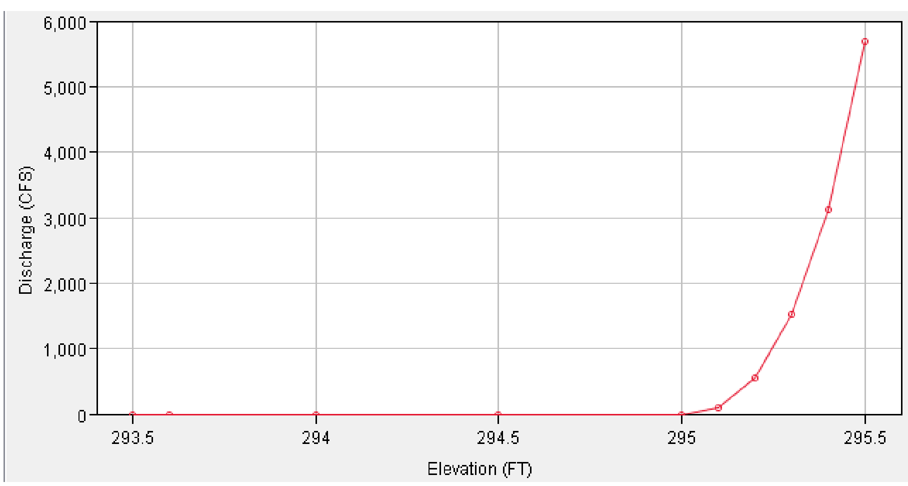
The equalization impoundment was also built in 1977. Its storage was also assumed to be based on the 18-inches of freeboard for the analysis. The storage capacity calculated at the top of berm is 36.8 acre-ft. **Figure 5** shows a graphical representation of the elevation vs. storage relationship used in the hydrologic modeling. It is based on the impoundment's physical dimensions and has a maximum storage at elevation 295 ft (18 inches above the water pool elevation of 293.5 ft). Six inches of additional depth above the berm were also assumed for model computational purposes.

Figure 5: Equalization Impoundment – Stage vs. Storage Relationship



A discharge vs. stage relationship was calculated and the overflow berm areas were assumed to function as a broad crested weir with crest elevation of 295 ft (36.8 ac-ft storage). The impoundment does not have a spillway or any sort of discharge structure. As with the ash impoundments, the same assumption was made that the overflow occurs along the entire perimeter of the impoundment. It is unlikely that this overflow would occur simultaneously over the entire perimeter so the same gradual increase in weir length occurs as with the ash impoundments. The active weir length was gradually increased from 1,000 feet at 295.1 to the total 5,200 linear feet of weir at elevation 295.5. The starting water surface elevation for the analysis was 293.5. **Figure 6** shows the stage-storage relationship that was used to model the outfall hydrograph conditions. A maximum discharge of 5,680 cfs is observed at stage 295.5.

Figure 6: Equalization Impoundment - Stage vs. Discharge Relationship



2.7 Modeling and Results

The parameters in **Table 4**, the stage-storage-discharge relationships of each impoundment, and the PMP rainfall data with various storm durations were all used to set up the various modeling scenarios (in HEC-HMS 4.0) to determine the design storm peak flood elevation for 25% of the Probable Maximum Flood (25% PMF). A one-minute time-step interval was used for computation purposes for all storm scenarios.

Table 5 provides the resulting design storm precipitation depths, storage, and maximum water surface elevation for the various storm durations modeled for the Ash Water combined impoundments:

Table 5: Ash Water Impoundments Modeling Results

Watershed Size:	31.87 acres						
Max Pond Storage:	40.73 ac-ft						
Top of Berm (overflow)	315.0						
Meteorologic Model & Storm Duration:	**PMP - HMR 51 - 72 hours**	**PMP - HMR 51 - 48 hours**	**PMP - HMR 51 - 24 hours**	**PMP - HMR 51 - 12 hours**	**PMP - HMR 51 - 6 hours**	**PMP - HMR 51 - 3 hours**	**PMP - HMR 51 -1 hour**
MODELING RESULTS							
PMP Depth (in):	52.7	49.7	44.4	37.4	30.6	24.9	19.3
Design Storm Precip. (1/4 PMP) Depth(in):	13.2	12.4	11.1	9.3	7.6	6.2	4.8
Design Storm (1/4 PMF) (cfs):	424	399	357	300	245	200	153
Design Storm Peak Storage (ac-ft):	45.5	45.3	45.1	44.7	44.4	36.4	12.1
Design Storm (1/4 PMF)Peak Flood Elevation:	315.2	315.2	315.2	315.1	315.1	314.8	314.0
Freeboard:	-0.2	-0.2	-0.2	-0.1	-0.1	0.2	1.0

The 100% PMP depth for a 72-hour duration storm is 52.7 inches of rain, with 19 inches occurring in the most intense 1 hour duration. Likewise, the 25% PMP depth for a 72-hour duration storm is 13.2 inches of rain, with 4.8 inches occurring in the most intense 1 hour duration. The design storm discharges (25% PMF) for storm durations greater than 6 hours exceed the storage capacity of the impoundments and overflow the berm. The observed overflow stage is not getting much higher than the berms because the berms' length is the entire perimeter of the impoundment (i.e., long weir length = low head). The 1- and 3-hour PMFs have lesser flow generated and show 1 and 0.2 feet of freeboard respectively.

The 500-year storm event (0.2% AEP) was also modeled for comparison purposes only and is not part of the PMF analysis. The results were 12.7 inches of rain accumulate to an inflow runoff volume of 33.9 acre-ft. This is less than the 40.7 acre-ft capacity and therefore there is no overflow (there is 0.2 feet of freeboard). Even though the 500-year event has a higher rainfall depth, the storm is more of a flash flood event in which the total rainfall volume is less than the 25% PMP rainfall volume. There is no overflow observed during the 500-year event with 0.2 feet of freeboard.

Table 6 provides the resulting design storm precipitation depths, storage, and maximum flood elevation for the various storm durations modeled for the Equalization impoundment:

Table 6: Equalization Impoundments Modeling Results

Watershed Size:	28.46 acres						
Max Pond Storage:	36.77 ac-ft						
Top of Berm (overflow)	295.0						
Meteorologic Model & Storm Duration:	**PMP - HMR 51 - 72 hours**	**PMP - HMR 51 - 48 hours**	**PMP - HMR 51 - 24 hours**	**PMP - HMR 51 - 12 hours**	**PMP - HMR 51 - 6 hours**	**PMP - HMR 51 - 3 hours**	**PMP - HMR 51 - 1 hour**
MODELING RESULTS							
PMP Depth (in):	52.7	49.7	44.4	37.4	30.6	24.9	19.3
Design Storm Precip. (1/4 PMP) Depth(in):	13.2	12.4	11.1	9.3	7.6	6.2	4.8
Design Storm Discharge (1/4 PMF) (cfs):	378	357	319	268	219	179	137
Design Storm Peak Storage (ac-ft):	40.8	40.7	40.5	40.2	39.9	32.3	10.6
Design Storm (1/4 PMF)Peak Flood Elevation:	295.2	295.2	295.1	295.1	295.1	294.8	293.9
Freeboard:	-0.2	-0.2	-0.1	-0.1	-0.1	0.2	1.1

The ash impoundment generates more runoff than the equalization impoundment due to its larger surface. The results indicate that the design storm discharges (25% PMF) for storm durations greater than 6 hours exceed the storage capacity of the impoundments by less than 1 acre-ft. This causes an overflow of the berm for these events. The 1- and 3-hour events have lesser flow generated and show 1.1 and 0.2 feet of freeboard respectively.

The 500-year storm event (0.2% AEP) was also modeled for comparison purposes only and the results were 12.7 inches of rain with storage of 30.1 acre-ft. There is no overflow observed with 0.3 feet of freeboard. Even though the 500-year event has a higher rainfall depth, the storm is more of a flash flood event in which the total rainfall volume occurs in les time than the PMP and has less than the 25% PMP rainfall volume.

2.8 Conclusions

As described in Section 1.6 above, the CCW impoundments at the San Miguel Electric Plant are classified as low hazard and small size per the TCEQ criteria for dams in Texas. The area downstream does not appear to have the potential for a significant loss of life, and there are no structures or agricultural land that would create a significant economic loss.

However, the majority of the design storm rainfall is overflowing each of the impoundments when the total rainfall depths exceed 7 inches. The 18-inches of freeboard provided in each impoundment is not adequate to withstand these design storm discharges (25% PMF). With this significant overflow, it is possible that the berms on all three impoundments could erode and breach since they are likely not designed to withstand this condition and there is no spillway for emergency overflows.

Possible preliminary recommendations to avoid either berm overflow or breach are listed below. These recommendations are preliminary and limited to the information available when performing this study. They all assume that the plant's recirculation system would not be compromised during a significant rainfall event and that the minimum freeboard amount for each recommendation would be maintained at all times. Further studies and inspections (such as geotechnical, hydraulic, and ground survey) would be required to further validate these concepts. In addition, the EPA is currently under a rule making progress for CCW impoundments. Once this rule is promulgated, a separate study and report that is signed by a Professional Engineer in Texas should be conducted to ensure that those specific requirements are evaluated for the San Miguel Electric Plant:

- Lower the normal pool elevation that is maintained for each impoundment to increase the freeboard to a depth where overtopping would be prevented. The recommendation would be to increase the freeboard depth from 18 inches to 22 or 24 inches based on the results presented in this study.

- Raise the existing berms surrounding each of the impoundments. This could be achieved by re-grading the landward (exterior) side of the berm to a 3H:1V slope. This would make the proposed berm less steep than the existing 2H:1V berm and would add more side-slope stability. It would cover more land area, but there appears to be sufficient land on the exterior side of the berms to perform this. The reconstructed berms would need to be raised to elevation 316.0 and 296.0 at the ash-water and equalization ponds respectively. The berm modification would impede any overflow for the calculated flows in this study.

- Armor the existing earthen berms at both impoundments to withstand a small amount of overtopping without failure. The armoring could be concrete slope protection or articulated concrete blocks. This would impede any possible breach of berms, but would still allow the overtopping to continue after the 6-hour rainfall event as calculated in this study. The outer toe of the exterior side of berms would need to be re-graded with earthen ditches to convey the runoff overspill into the existing storm water channels in the facility. These outfall channels currently convey runoff to Caballos Creek.

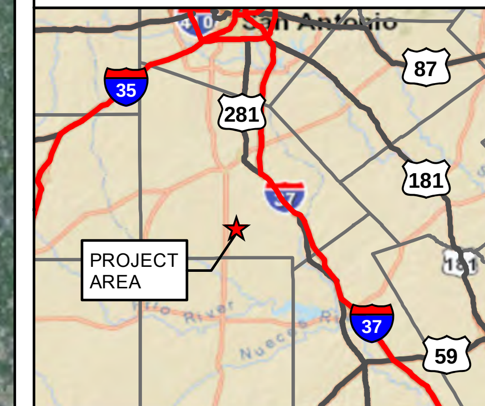
- Install an emergency overflow spillway below the top of berm elevation of each impoundment. The specification and details of the spillway would need to be evaluated as well as the downstream channel conveying the overflow to Caballos Creek. A filtration system would be required to allow only "environmentally acceptable" fluids into the receiving stream.

- Construct a new holding pond onsite to provide the additional storage volume. Provide new pumps at the CCW impoundments that would transfer flow from significant rainfall events to the new holding pond. The existing ponds would still have the 18" of freeboard, but the new pumps would prevent any overspill. The pumps would need to be sized for a discharge rate that is adequate to maintain the 25% PMF at the top elevation of the berms.

Appendix A

SAN MIGUEL H&H

PROJECT LOCATION



LEGEND

- DRAINAGE AREA
- CABALLOS CREEK
- 10-FOOT INTERVAL CONTOUR
- TOPO LINE
- TC FLOW PATH
- IMPOUNDMENT PERIMETER

SOURCE: USGS; SMECI

0 200 400
FEET

N

HDR

DEC 2014 | FIGURE 1

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HMR 51 PMP Calculations for San Miguel Mine/Power Plant 28.701N, -98.469W

Yellow highlights indicate point of inflection on TCEQ temporal distribution graphs (33% of storm duration over at 3 hours and greater)

HMR 51 PMP Values for a basins 10 sq. mi or less

Duration	Rainfall
1-hour	19.25
3-hour	24.90
6-hour	30.55
12-hour	37.35
24-hour	44.40
48-hour	49.70
72-hour	52.72

HMR_51-PMP

Temporal Distribution of PMP as per TCEQ GI-364 Guidelines

Minutes	1-hour PMP Rainfall Accumulation	3-hour PMP Rainfall Accumulation
5	1.60	1.04
10	3.21	2.07
15	4.81	3.11
20	6.42	4.15
25	8.02	5.19
30	9.62	6.22
35	11.23	7.26
40	12.83	8.30
45	14.44	9.34
50	16.04	10.37
55	17.64	11.41
60	19.25	12.45
65		12.97
70		13.49
75		14.01
80		14.52

Hours	6-hour PMP Rainfall Accumulation	12-hour PMP Rainfall Accumulation	24-hour PMP Rainfall Accumulation	48-hour PMP Rainfall Accumulation	72-hour PMP Rainfall Accumulation
1	9.17	6.54	4.44	2.64	1.87
2	18.33	13.07	8.88	5.28	3.73
3	21.39	19.61	13.32	7.92	5.62
4	24.44	26.15	17.76	10.56	7.51
5	27.50	27.55	22.20	13.20	9.40
6	30.55	28.95	26.64	15.84	11.29
7		30.35	31.08	18.48	13.18
8		31.75	35.52	21.12	15.07
9		33.15	36.08	23.79	16.96
10		34.55	36.63	26.47	18.85
11		35.95	37.19	29.14	20.74
12		37.35	37.74	31.81	22.63
13			38.30	34.48	24.51
14			38.85	37.15	26.40
15			39.41	39.82	28.29
16			39.96	42.25	30.18

Depth-Duration-Frequency Relationship

County: Atascosa
Source: TxDOT 2014 Hydraulic Manual: Chapter 4 Section 13
Precip Data: USGS Atlas of Depth-Duration Frequency Precipitation for Texas (2004)

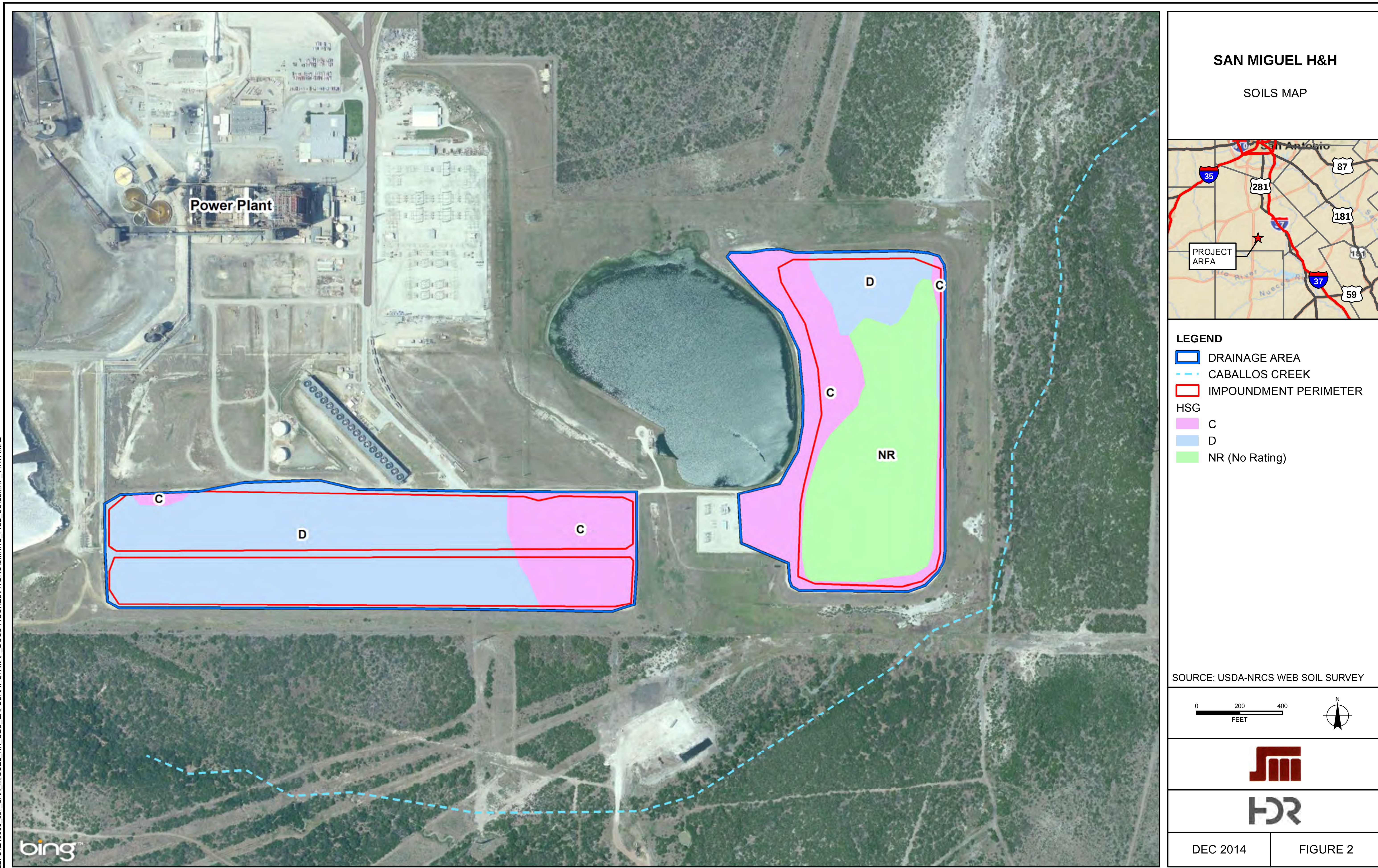
	2-yr	**5-yr**	**10-yr**	**25-yr**	**50-yr**	**100-yr**	**500-yr**
Prob	**50 percent**	**20 percent**	**10 percent**	**4 percent**	**2 percent**	**1 percent**	**0.2 percent**
Int Duration	1 hour	1 hour	1 hour	1 hour	1 hour	1 hour	1 hour
Storm dur	1 day	1 day	1 day	1 day	1 day	1 day	1 day
Int Position	50%	50%	50%	50%	50%	50%	50%
Storm Area							
Depths	inches	inches	inches	inches	inches	inches	inches
15 min	1.1	1.4	1.6	1.9	2.2	2.4	3
1 hr	1.8	2.4	2.8	3.3	3.8	4.4	5.8
2 hr	2	2.9	3.5	4.3	5	5.8	8
3 hr	2.3	3.2	4	4.9	5.8	6.5	9.5
6 hr	2.6	3.6	4.6	5.5	6.5	7.5	10
12 hr	3	4.4	5.1	6.5	7.5	9	12
1 day	3.4	4.7	5.8	7	8.6	9.8	13

County: Karnes (borders Atascosa on the east)
Source: San Antonio River Authority - Regional Hydrology and Hydraulic Standards
Precip Data: USGS Atlas of Depth-Duration Frequency Precipitation for Texas (2004)

TABLE 6: USGS ADJUSTED IDF RAINFALL VALUES FOR KARNES COUNTY

Total Rainfall for Frequency (Inches)

Duration	10-Year	25-Year	50-Year	100-Year	500-Year
5 minute	0.84	0.98	1.11	1.22	1.55
15 minute	1.62	1.98	2.29	2.50	3.20
30 minute	2.28	2.66	3.00	3.30	4.20
1 hour	2.80	3.38	3.85	4.35	5.80
2 hour	3.55	4.20	4.90	5.50	7.50
3 hour	3.90	4.80	5.65	6.50	8.80
6 hour	4.40	5.50	6.40	7.30	10.30
12 hour	5.05	6.40	7.50	8.60	12.00
24 hour	6.00	7.30	8.80	9.90	13.20
2-day	6.70	8.20	9.60	11.00	14.20
3-day	7.10	8.60	10.10	11.80	14.80
4-day	7.50	9.05	10.55	12.15	15.45
5-day	7.90	9.50	11.00	12.50	16.10
7-day	8.20	10.00	11.60	12.60	16.60



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Job No.	240666 - 037	Calc No.	00001

UNIT HYDROGRAPH METHOD SUMMARY

Project	San Miguel H&H and Geo Exploration	Computed	JF
System	Hydrologic Analysis - HEC-HMS	Date	10/28/2014
Component	Computations	Reviewed	RV
Task	Hydrologic Summary Table	Date	11/10/2014

Ash Water Transport Impoundments

Sub-Basin	Land Use Classification	Area (acre)	Percent Area	Soil Group	Condition II Curve Number	CN* Percent Area
1	Embankment - Poor condition (grass cover < 50%)	1.3	26%	C	86	22.2
	Embankment - Poor condition (grass cover < 50%)	3.6	74%	D	89	66.0
	Total:	**4.9**			**Weighted CN =**	**88**
2	Water	27.0	-	-	-	-
	Total:	**27.0**			**Imperviousness =**	**100%**

Equalization Impoundments

Sub-Basin	Land Use Classification	Area (acre)	Percent Area	Soil Group	Condition II Curve Number	CN* Percent Area
3	Embankment - Poor condition (grass cover < 50%)	4.3	66%	C	86	56.7
	Embankment - Poor condition (grass cover < 50%)	0.6	10%	D	89	8.6
	Pavement	1.6	24%	C	98	23.9
	Total:	**6.5**			**Weighted CN =**	**89**
4	Water	22.0	-	-	-	-
	Total:	**22.0**			**Imperviousness =**	**100%**

Note: Soils data was taken from USDA-NRCS Web Soil Survey; the data is dated Dec. 12, 2013
Land use data was developed by HDR using NAIP Aerial Imagery for Atascosa County (2012)
Curve Number values based on 2014 TxDOT Hydraulic Design Manual (Table 4-21)
No CN climatic adjustment required per Figure 4-21 of the Manual

LAG TIME CALCULATIONS SUMMARY

Sub Basin	Sheet Flow				Shallow Concentrated Flow				Flood wave	Tc	T_{lag} =0.6Tc	T_{lag}	Drainage Area
	Length	Slope	n	Tc	Length	Slope	Velocity	Tc	Tc	total			
	ft	ft/ft		min	ft	ft/ft	ft/s	min	(min)	min	Min	Hrs	acres
1	60	0.250	0.011	0.26						0.3	0.2	0.0	4.9
2	0	0.001	0.011	0.00					0.00	0.0	0.0	0.0	27.0
3	100	0.100	0.011	0.57	160	0.100	5.1	0.5		1.1	0.7	0.0	6.5
4	0	0.001	0.011	0.00					0.00	0.0	0.0	0.0	22.0

Ash Ponds - non-water
Ash Ponds - rain on water does not have a la
Equalization Pond - non-water
Equalization Pond -rain on water does not ha

n = Manning's roughness coefficient
Tc = time of concentration
T_{lag} = lag time

Overland flow is the initial flow over plane surfaces after raindrops impact the surface.

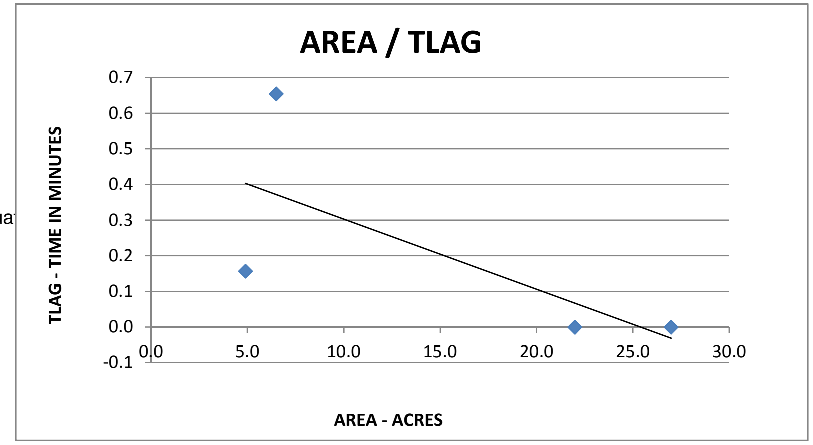
$$T_t = \frac{.007(nL)^{0.8}}{P_2^{0.5} s^{0.4}}$$ TR-55 equation 3-3

Where:
T_t = travel time (hr)
n = Manning's roughness coefficient
L = flow length (ft)
P_2 = 2-year, 24-h rainfall depth (in.) = 4.02" for Atascosa County (per TxDOT 24-Hour Rainfall Depth vs. Frequency for TX counties)
s = slope of hydraulic grade line (land slope, ft/ft)

Shallow concentrated flow occurs after overland flow and describes the flows which accumulate in similar paths but not yet form a channelized pattern.
Velocity is estimated using **Figure 3-1**, given in TR-55, a nomograph in which velocity is directly proportional to slope and is dependent on the paved or unpaved nature of the surface.
Travel time is then calculated using the following equation:

$$T_t = \frac{L}{3600V}$$

Where:
T_t = travel time (hr)
L = flow length (ft)
V = velocity (ft/s)



Storm Sewer and Channel Flow times of concentration are calculated using manning's equa

$$V = \frac{1.49 r^{2/3} s^{1/2}}{n}$$ TR-55 equation 3-4

Where:
V = average velocity (ft/s)
r = hydraulic radius (ft) and is equal to a/p_w
a = cross sectional flow area (ft^2)
p_w = wetter perimeter (ft)
s = slope of hydraulic grade line (pipe slope, ft/ft)
n = Manning's roughness coefficient
After velocity is calculated, travel time is calculated using the equation shown for shallow concentrated flow.

Job No.	240666 - 0	Calc No.	00001

STAGE-STORAGE-DISCHARGE CALCULATIONS

Project	San Miguel H&H and Geo Exploration	Computed	JF
System	Hydrologic Analysis - HEC-HMS	Date	11/10/2014
Component	Computations	Reviewed	RV
Task	Stage-Storage_Discharge Table	Date	11/25/2014

Ash Water Transport Impoundments

$g=32.17$ ft/s^2 $Q=CLH^{3/2}$

Stage (ft)	Elevation (ft)	Length (ft)	Width (ft)	Area (ac)	Storage (ac-ft)	Hydraulic Head (H) (ft)	Weir Length (L) (ft)	Discharge Coefficient C	Discharge (cfs)
0	313.5	2447.5	237.5	26.7	0.00				0
0.083	313.6	2447.9	237.9	26.7	2.23				0
0.5	314.0	2450.0	240.0	27.0	13.42				0
1	314.5	2452.5	242.5	27.3	27.00				0
1.5	315.0	2455.0	245	27.6	40.73	0.0	0.0	0.0	0
1.6	315.1	2455.0	245	27.6	43.49	0.1	1000.0	3.087	98
1.7	315.2	2455.0	245	27.6	46.25	0.2	2000.0	3.1	552
1.8	315.3	2455.0	245	27.6	49.01	0.3	3000.0	3.087	1,522
1.9	315.4	2455.0	245	27.6	51.77	0.4	4000.0	3.1	3,124
2	315.5	2455.0	245	27.6	54.54	0.5	5800.0	3.087	6,330

Elevation (ft)	Storage (ac-ft)
313.5	0.00
313.6	2.23
314	13.42
314.5	27.00
315	40.73
315.1	43.49
315.2	46.25
315.3	49.01
315.4	51.77
315.5	54.54

Elevation (ft)	Discharge (cfs)
313.5	0.00
313.6	0.00
314	0.00
314.5	0.00
315	0.00
315.1	97.62
315.2	552.22
315.3	1521.74
315.4	3123.82
315.5	6330.23

Storage (ac-ft)	Discharge (cfs)
0.00	0.00
2.23	0.00
13.42	0.00
27.00	0.00
40.73	0.00
43.49	97.62
46.25	552.22
49.01	1521.74
51.77	3123.82
54.54	6330.23

Job No.	240666 - 037	Calc No.	00001

STAGE-STORAGE-DISCHARGE CALCULATIONS

Project	San Miguel H&H and Geo Exploration	Computed	JF
System	Hydrologic Analysis - HEC-HMS	Date	11/10/2014
Component	Computations	Reviewed	RV
Task	Stage-Storage_Discharge Table	Date	**12/14/2014**

Equalization Impoundment

$g=32.17$ ft/s^2 $Q=CLH^{3/2}$

Stage (ft)	Elevation (ft)	Area (ac)	Storage (ac-ft)	Hydraulic Head (H) (ft)	Weir Length (L) (ft)	Discharge Coefficient C	Discharge (cfs)	
0	293.5	24.28	0.00				0	->
0.083	293.6	24.31	2.02				0	
0.5	294.0	24.43	12.18				0	
1	294.5	24.59	24.44				0	
1.5	295.0	24.75	36.77	0.0	0.0	0.0	0	->
1.6	295.1	24.83	39.25	0.1	1000.0	3.087	98	->
1.7	295.2	24.83	41.73	0.2	2000.0	3.087	552	
1.8	295.3	24.83	44.21	0.3	3000.0	3.087	1,522	
1.9	295.4	24.83	46.70	0.4	4000.0	3.087	3,124	
2	295.5	24.83	49.18	0.5	5203.0	3.087	5,679	->

Elevation (ft)	Storage (ac-ft)
293.5	0.00
293.6	2.02
294	12.18
294.5	24.44
295	36.77
295.1	39.25
295.2	41.73
295.3	44.21
295.4	46.70
295.5	49.18

Elevation (ft)	Discharge (cfs)
293.5	0.00
293.6	0.00
294	0.00
294.5	0.00
295	0.00
295.1	98
295.2	552
295.3	1522
295.4	3124
295.5	5679

Storage (ac-ft)	Discharge (cfs)
0.00	0.00
2.02	0.00
12.18	0.00
24.44	0.00
36.77	0.00
39.25	98
41.73	552
44.21	1522
46.70	3124
49.18	5679

Job No.	240666 - 037	Calc No.	00001

HEC-HMS RESULTS SUMMARY

Project	San Miguel H&H and Geo Exploration	Computed	JF
System	Hydrologic Analysis - HEC-HMS	Date	12/1/2014
Component	Computations	Reviewed	RV
Task	Hydrologic Summary Table	Date	12/5/2014

ASH WATER TRANSPORT IMPOUNDMENTS STUDY - RESULTS

Impoundment Details

Watershed Size: 31.87 acres
Max Pond Storage: 40.73 ac-ft
Top of Berm (overflow) 315.0

Meteorologic Model & Storm Duration:	PMP - HMR 51 - 72 hours	PMP - HMR 51 - 48 hours	PMP - HMR 51 - 24 hours	PMP - HMR 51 - 12 hours	PMP - HMR 51 - 6 hours	PMP - HMR 51 - 3 hours	PMP - HMR 51 -1 hour
MODELING RESULTS							
PMP Depth (in):	52.7	49.7	44.4	37.4	30.6	24.9	19.3
Design Storm Precip. (1/4 PMP) Depth(in):	13.2	12.4	11.1	9.3	7.6	6.2	4.8
Design Storm (1/4 PMF) (cfs):	424	399	357	300	245	200	153
Design Storm Peak Storage (ac-ft):	45.5	45.3	45.1	44.7	44.4	36.4	12.1
Design Storm (1/4 PMF)Peak Flood Elevation:	315.2	315.2	315.2	315.1	315.1	314.8	314.0
Freeboard:	-0.2	-0.2	-0.2	-0.1	-0.1	0.2	1.0

Run 1

Meteorologic Model: 500-year Frequency Storm Event
Control: 24 hours / 15 minute interval
Overflow Elevation: 315.0

Max Pond Storage at 315: 40.73 ac-ft

RESERVOIR ROUTING RESULTS

Peak Inflow: 198.2 CFS
Peak Discharge: 0 CFS
Inflow Volume: 12.75 IN
Discharge Volume: **0** IN
Date/Time Peak Inflow: **01-01-2000 / 12:30**
Date/Time Peak Discharge: NONE
Peak Storage: 33.9 AC-FT
Peak Elevation: 314.7 FT
No overflow

Job No.	240666 - 037	Calc No.	00001

HEC-HMS RESULTS SUMMARY

Project	San Miguel H&H and Geo Exploration	Computed	JF
System	Hydrologic Analysis - HEC-HMS	Date	12/1/2014
Component	Computations	Reviewed	RV
Task	Hydrologic Summary Table	Date	12/16/2014

EQUALIZATION IMPOUNDMENT STUDY - RESULTS

Impoundment Details

Watershed Size: 28.46 acres
Max Pond Storage: 36.77 ac-ft
Top of Berm (overflow) 295.0

Meteorologic Model & Storm Duration:	PMP - HMR 51 - 72 hours	PMP - HMR 51 - 48 hours	PMP - HMR 51 - 24 hours	PMP - HMR 51 - 12 hours	PMP - HMR 51 - 6 hours	PMP - HMR 51 - 3 hours	PMP - HMR 51 -1 hour
MODELING RESULTS							
PMP Depth (in):	52.7	49.7	44.4	37.4	30.6	24.9	19.3
Design Storm Precip. (1/4 PMP) Depth(in):	13.2	12.4	11.1	9.3	7.6	6.2	4.8
Design Storm Discharge (1/4 PMF) (cfs):	378	357	319	268	219	179	137
Design Storm Peak Storage (ac ft):	40.8	40.7	40.5	40.2	39.9	32.3	10.6
Design Storm (1/4 PMF)Peak Flood Elevation:	295.2	295.2	295.1	295.1	295.1	294.8	293.9
Freeboard:	-0.2	-0.2	-0.1	-0.1	-0.1	0.2	1.1

500-yr Run

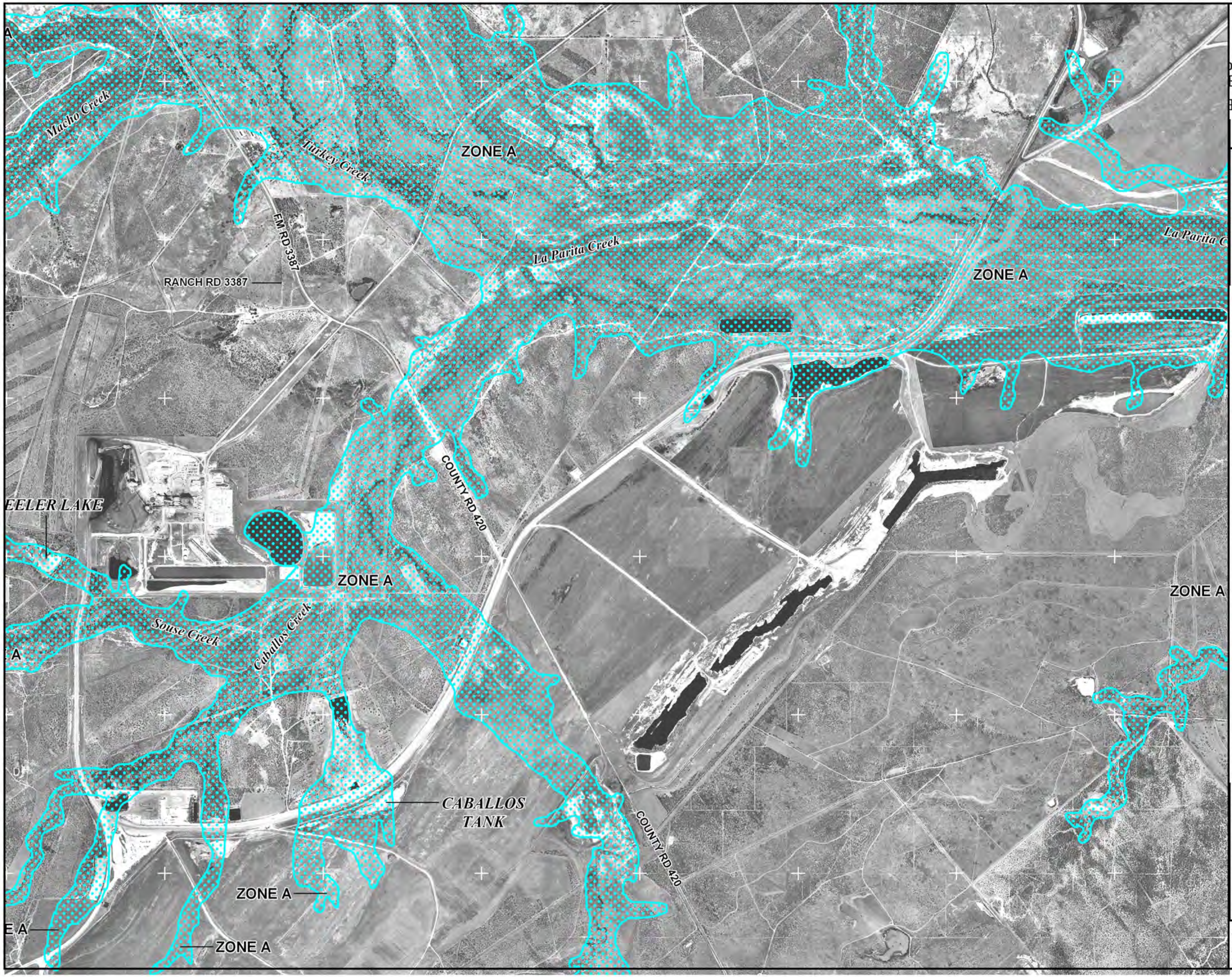
Meteorologic Model: 500-year Frequency Storm Event
Control: 24 hours / 15 minute interval

RESERVOIR ROUTING RESULTS

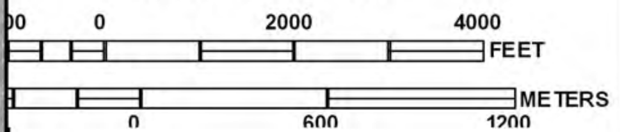
Peak Inflow:	244	CFS
Peak Discharge:	0	CFS
Inflow Volume:	12.7	IN
Discharge Volume:	**0**	IN
Date/Time Peak Inflow:	**01-01-2000 / 12:15**	
Date/Time Peak Discharge:	NONE	
Peak Storage:	30.1	AC-FT
Peak Elevation:	294.7	FT

No overflow

Appendix B



MAP SCALE 1" = 2000'



NATIONAL FLOOD INSURANCE PROGRAM

NFIP

PANEL 0675C

FIRM

FLOOD INSURANCE RATE MAP

ATASCOSA COUNTY, TEXAS

AND INCORPORATED AREAS

PANEL 675 OF 750

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
ATASCOSA COUNTY, UNINCORPORATED AREAS	480014	0675	C

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
48013C0675C

EFFECTIVE DATE
NOVEMBER 4, 2010

Federal Emergency Management Agency

This is an official copy of a portion of the above referenced flood map. It was extracted using F-MIT On-Line. This map does not reflect changes or amendments which may have been made subsequent to the date on the title block. For the latest product information about National Flood Insurance Program flood maps check the FEMA Flood Map Store at www.msc.fema.gov

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY
WASHINGTON, D.C. 20460

May 2, 2014

OFFICE OF
SOLID WASTE AND
EMERGENCY RESPONSE

VIA E-MAIL

Mr. Michael Kezar
General Manager
San Miguel Electric Cooperative
P.O. Box 280
Jourdanton, Texas 78026-0280

Re: Request for Action Plan regarding San Miguel Electric Cooperative Inc.'s San Miguel Electric Plant

Dear Mr. Kezar,

On August 30, 2012 the United States Environmental Protection Agency ("EPA") and its engineering contractors conducted a coal combustion residual (CCR) site assessment at the San Miguel Electric Cooperative Inc.'s San Miguel Electric Plant facility. The purpose of this visit was to assess the structural stability of the impoundments or other similar management units that contain "wet" handled CCRs. We thank you and your staff for your cooperation during the site visit. Subsequent to the site visit, EPA sent you a copy of the draft report evaluating the structural stability of the units at the San Miguel Electric Cooperative Inc.'s San Miguel Electric Plant facility and requested that you submit comments on the factual accuracy of the draft report to EPA. Your comments were considered in the preparation of the final report.

The final report for the San Miguel Electric Cooperative Inc.'s San Miguel Electric Plant facility is attached.

This report includes a specific condition rating for the CCR management units and recommendations and actions that our engineering contractors believe should be undertaken to ensure the stability of the CCR impoundments located at the San Miguel Electric Cooperative Inc.'s San Miguel Electric Plant facility. These recommendations are listed in Enclosure 1.

Since these recommendations relate to actions which could affect the structural stability of the CCR management units and, therefore, protection of human health and the environment, EPA believes their implementation should receive the highest priority. Therefore, we request that you inform us on how you intend to address each of the recommendations found in the final report. Your response should include specific plans and schedules for implementing each of the recommendations. If you will not implement a recommendation, please provide a rationale. Please provide a response to this request by **June 4, 2014**. Please send your response to:

Mr. Stephen Hoffman
U.S. Environmental Protection Agency (5304P)

1200 Pennsylvania Avenue, NW
Washington, DC 20460

If you are using overnight or hand delivery mail, please use the following address:

Mr. Stephen Hoffman
U.S. Environmental Protection Agency
Two Potomac Yard
2733 S. Crystal Drive
5th Floor, N-5838
Arlington, VA 22202-2733

You may also provide a response by e-mail to hoffman.stephen@epa.gov, dufficy.craig@epa.gov, kelly.patrickm@epa.gov and englander.jana@epa.gov.

You may assert a business confidentiality claim covering all or part of the information requested, in the manner described by 40 C. F. R. Part 2, Subpart B. Information covered by such a claim will be disclosed by EPA only to the extent and only by means of the procedures set forth in 40 C.F.R. Part 2, Subpart B. If no such claim accompanies the information when EPA receives it, the information may be made available to the public by EPA without further notice to you. If you wish EPA to treat any of your response as "confidential" you must so advise EPA when you submit your response.

EPA will be closely monitoring your progress in implementing the recommendations from this report and could decide to take additional action if the circumstances warrant.

You should be aware that EPA will be posting the report for this facility on the Agency website shortly.

Given that the site visit related solely to structural stability of the management units, this report and its conclusions in no way relate to compliance with RCRA, CWA, or any other environmental law and are not intended to convey any position related to statutory or regulatory compliance.

Please be advised that providing false, fictitious, or fraudulent statements of representation may subject you to criminal penalties under 18 U.S.C. § 1001.

If you have any questions concerning this matter, please contact Mr. Hoffman in the Office of Resource Conservation and Recovery at (703) 308-8413. Thank you for your continued efforts to ensure protection of human health and the environment.

Sincerely,
/Barnes Johnson /, Director
Office of Resource Conservation and Recovery

Enclosures

Enclosure 1

San Miguel Electric Cooperative Inc.'s San Miguel Electric Plant Recommendations (from the final assessment report)

CONCLUSIONS

Conclusions Regarding Structural Soundness of the CCW Impoundments

Structural stability documentation appears to be adequate. A geotechnical report, prepared by Arias & Associates, Inc. (Arias), was provided, and it included slope stability analyses for all required load conditions, with the exception of rapid drawdown and liquefaction. Because the impoundments do not include spillways or overflow structures, and liquids are pumped over the embankments, rapid drawdown conditions were considered only likely in the event of a breach. The potential for liquefaction is considered unlikely due to the subsurface soil conditions and low seismic hazard level.

Slope stability analyses were provided for steady-state seepage, maximum surcharge pool, and seismic conditions, as well as the assessment for liquefaction potential. In general, slope stability safety factors for load conditions analyzed are satisfactory.

Conclusions Regarding the Hydrologic/Hydraulic Safety of CCW Impoundments

No hydrologic and hydraulic information was provided by San Miguel to indicate CCW impoundments hydrologic/hydraulic safety. A target pool elevation of at least 18 inches of freeboard at both the Ash Pond and Sludge Basin was the only hydraulic information provided by San Miguel. During the site visit, both ponds were below the target pool elevation. Because no hydrologic/ hydraulic documentation was provided, the hydrologic/hydraulic safety is judged to be inadequate.

Conclusions Regarding Adequacy of Supporting Technical Documentation

Supporting data and documentation for the Ash Pond and Sludge Basin includes required structural stability analyses for normal operating pool, steady state conditions; maximum surcharge pool condition; and normal operating pool under seismic loading conditions. An assessment of liquefaction potential was also provided, with the conclusion that liquefaction is considered to be very unlikely based on existing subsurface soil conditions and the stated 6% chance of a seismic event of a magnitude 5.0 or greater occurring over a 250-year period. Technical documentation of the embankment stability under a sudden drawdown loading condition was not provided because rapid drawdown conditions were considered only likely in the event of a breach. CDM Smith agrees with the rationale provided regarding embankment stability, liquefaction potential, and rapid drawdown conditions. Supporting documentation for structural stability is considered to be adequate.

Because no supporting data or documentation was provided for hydrologic/hydraulic safety of the impoundments, it is considered to be inadequate.

Conclusions Regarding Description of the CCW Impoundments

The record drawings and descriptions of the CCW impoundments provided by San Miguel representatives appear to be consistent with the visual observations by CDM Smith during site assessment.

Conclusions Regarding Field Observations

During visual observations and site assessments, CDM Smith observed an area of potential seepage near the toe of the Ash Pond's west embankment, erosion rills on the interior and exterior slopes of the Ash Pond embankments and several rodent burrows on the crest and exterior slope of the Ash Pond embankments. An area of erosion, approximately 5 feet wide, was also observed on the interior slope of the Ash Pond's east embankment. According to San Miguel representatives this erosion was a result of leakage from a water well pipe traversing the Ash Pond embankment. The water well pipe had been repaired at the time of the site assessment. Soils had eroded or settled from under the Sludge Basin's stormwater inlet structure. Other observations of the Sludge Basin embankments included erosion rills on west embankment interior slope and an area of erosion on the interior slope of the west embankment, near the submersible pump outlet structure.

Conclusions Regarding Adequacy of Maintenance and Methods of Operation
Current maintenance and operation procedures appear to be generally adequate.
There was documentation regarding seepage at the Ash Pond in the 1980s. The pond liner was reconstructed in 1987, but an area of potential seepage was observed during the CDM Smith site assessment in the vicinity of one of the areas that had documented seepage in the 1980s. There was no evidence of previous spills or release of impounded liquids outside the plant property.
Conclusions Regarding Adequacy of Surveillance and Monitoring Program
Surveillance and monitoring procedures include weekly checks of the impoundments by the Plant Environmental Engineer for leaks or deficiencies, and recording pool levels for both the Ash Pond and Sludge Basin. Additionally, level gages are checked six times daily by the operations department.
Instrumentation for the Ash Pond and Sludge Basin consists of local level gages, used by operations to record impoundment levels. In addition to the current surveillance and monitoring program, the area of potential seepage at the west embankment exterior slope of the Ash Pond should be monitored.
Because of the erosion into the Ash Pond's east embankment slope from a leaking pipe, the surveillance and monitoring program should be revised to include more-detailed inspections.
Conclusions Regarding Suitability for Continued Safe and Reliable Operation
Main embankments do not show evidence of unsafe conditions requiring immediate remedial efforts, although maintenance to correct deficiencies noted above is required.
As described by San Miguel representatives operating procedures for the Ash Pond and Sludge Basin include methods of controlling the water levels in the lagoons, but no formal documentation was provided to CDM Smith.

RECOMMENDATIONS
Recommendations Regarding the Hydrologic/Hydraulic Safety
It is recommended that a qualified professional engineer determine the required flood frequency and evaluate the hydrologic and hydraulic capacity of the CCW impoundments to withstand design storm events without overtopping.
Recommendations Regarding the Technical Documentation for Structural Stability
It is recommended that a qualified professional engineer reevaluate the impoundments for structural stability should conditions from those included in the Arias & Associates, Inc. structural stability analyses change.
Recommendations Regarding Field Observations
CDM Smith recommends corrective actions be taken for the specific conditions identified below:

- Erosion rills – Erosion rills were observed on the interior slopes of the Sludge Basin and the interior and exterior slopes of the Ash Pond. Structural fill should be placed and compacted in the rills and graded to adjacent existing contours. The area should be sodded or reseeded.
- Surface erosion - Structural fill should be placed and compacted, graded to adjacent existing contours, and sodded or reseeded. Alternatively, riprap or other armoring could be used. Riprap or other armoring is recommended for the west, north, and east interior slopes to reduce the potential for erosion.
- Rodent burrows - Rodent burrows were observed on the crest and exterior embankment of the Ash Pond. Although not seen on other embankments, vegetation cover may have hidden additional rodent burrows. CDM Smith recommends San Miguel accurately document areas disturbed by animal activity, remove the animals, and backfill the burrows with compacted structural fill to protect the integrity of the embankments.

- Potential seepage area - CDM Smith observed an area of potential seepage at the west embankment exterior slope of the Ash Pond. CDM Smith recommends San Miguel take the following actions:
    - Cut back and maintain vegetation in the area to facilitate monitoring the condition
    - Develop a regular surveillance program to monitor areas of seepage and potential seepage to measure the rate, volume, and turbidity of flow emerging from the embankment slope; and
    - Develop and execute a geotechnical exploration program that includes additional test borings and installation of piezometers and other instrumentation to analyze and regularly monitor embankment seepage and stability.

Recommendations Regarding Surveillance and Monitoring Program

Monitoring for potential seepage at the exterior embankment slopes is recommended for both the Ash Pond and Sludge Basin considering historical issues with seepage. Potential areas of seepage may be more readily assessed after clearing of trees and dense vegetation on embankment slopes. It is recommended that vegetation on the impoundment embankments be maintained with seasonal mowing, as necessary, for animal control and surveillance and monitoring of embankments.

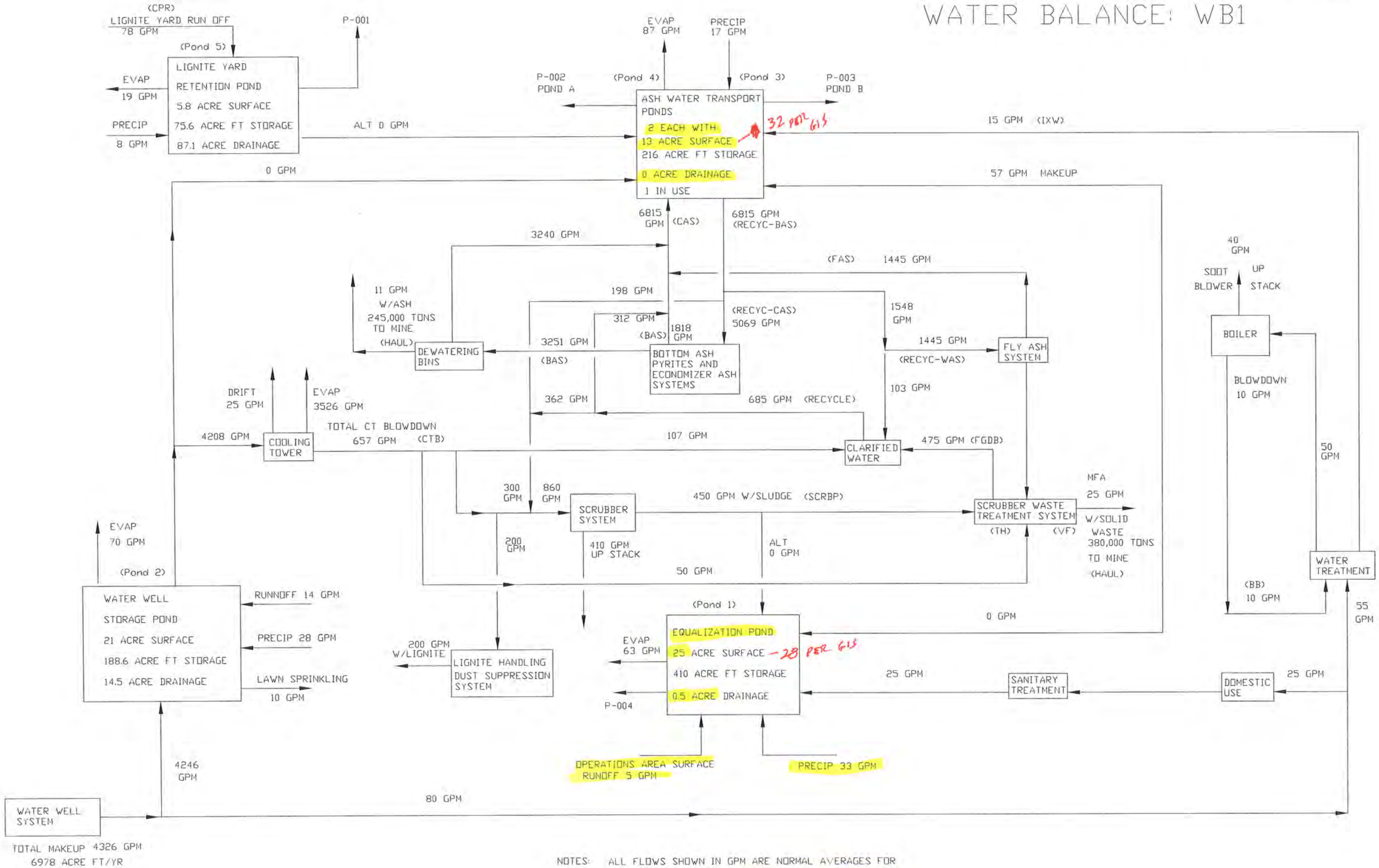
Recommendations Regarding Continued Safe and Reliable Operation

Inspections should be made following periods of heavy and/or prolonged rainfall, and the occurrence of these events should be documented. Inspection procedures should be documented and inspection records should be retained at the facility for a minimum of three years.

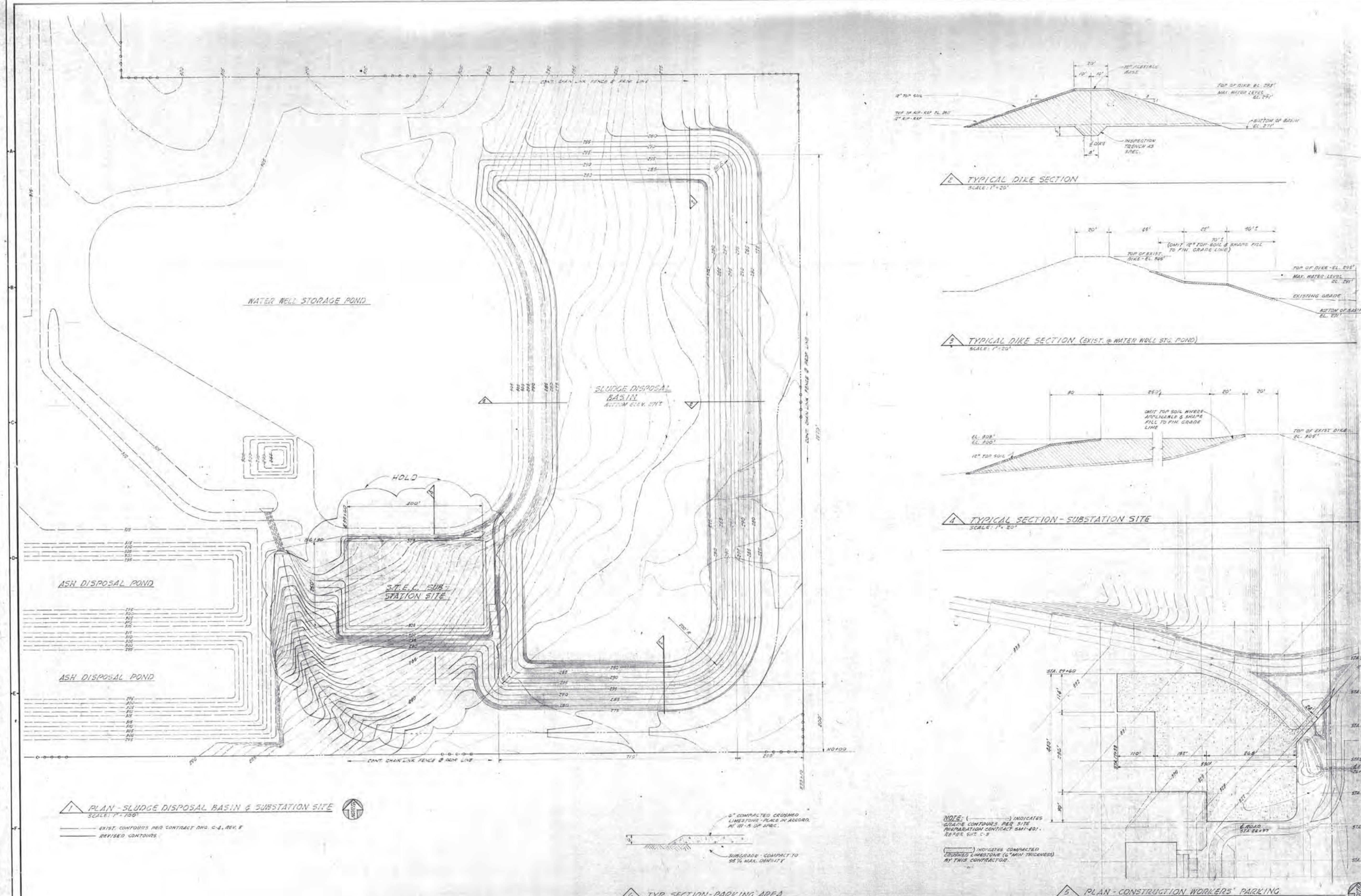
Major repairs and slope restoration should be designed by a registered professional engineer experienced with earthen dam design.

None of the conditions observed require immediate attention or remediation, however, the above recommendations should be implemented to maintain continued safe and reliable operation of the CCW impoundments.

PLANT ID: 508
PLANT NAME: SAN MIGUEL
WATER BALANCE: WB1



NOTES: ALL FLOWS SHOWN IN GPM ARE NORMAL AVERAGES FOR LOAD CONDITION SHOWN. PRECIPITATION RATE 25 IN/YR EVAPORATION RATE 65 IN/YR. (NORMAL AVERAGE YEARLY RATES) HR/YR BASED ON 80% CAPACITY FACTOR MW BASED ON CONSTANT SPECIFIED GROSS KWH WATER BALANCE IS BASED ON 434 MW AVG FOR 7440 HRS



NOTES
1. ALL FILL INDICATED 1 SHALL BE AS SPECIFIED FOR CONTROLLED COMPACTED FILL.

REV	DATE	BY	DESCRIPTION
0		PK	ADD THIS DRAWING TO CONTRACT DRAWINGS

SCALE AS NOTED
DRAWN P.R.K.
DATE 4-8-77
CHECKED E.A.D.

STATE OF TEXAS William G. Hollowell 15137 REGISTERED PROFESSIONAL ENGINEER
STATE OF TEXAS M.L. HUGHES 25324 REGISTERED PROFESSIONAL ENGINEER

TIPPETT & GEE, INC.
CONSULTING ENGINEERS
ABILENE TEXAS

SAN MIGUEL PLANT
UNIT NO. 1
B.E.P.C. S.T.E.C.

SLUDGE DISPOSAL BASIN,
69 KV SUBSTATION &
TEMP. PARKING AREA

JOB NO. SMI-401
REV. 0
DRAWING NUMBER C-12

Hydrologic and Hydraulic Guidelines for Dams in Texas

Dam Safety Program
Texas Commission on Environmental Quality

GI-364
January 2007

Contents

Tables

Figure

CHAPTER 1

Introduction to Guidelines

1.0 Introduction

These guidelines present instructions, standards, and accepted procedures for the hydrologic and hydraulic analysis of existing and proposed dams in Texas. They also clarify the expectations of the TCEQ with respect to submitted analyses, and simplify review by standardizing processes and elements so they will be acceptable to the reviewer. Though the guidelines are relatively specific, the engineer may always submit alternate procedures that either are more conservative or are sufficiently explained and justified.

Dams and spillways designed to comply with TCEQ rules, using hydrologic and hydraulic procedures of the Natural Resources Conservation Service, are acceptable, provided that they are shown to be equally conservative as, or more conservative than, designs developed using the criteria contained in this set of guidelines. The breach-analysis procedures described in Chapter 8 do not depend on which design method is used, and more exact full breach-analysis procedures can always be used in lieu of the conservative simplified procedures.

1.1 Regulatory Authority

These guidelines supplement the Texas Administrative Code, Title 30, Part 1, Chapter 299.

1.2 Professional Responsibility and Duty

These guidelines assume that anyone using or referencing them is a licensed professional engineer or is working under the guidance of a professional engineer. Users should also have appropriate knowledge of the processes and methodologies referenced, and be able to use standard software common in the engineering profession that is appropriate to the analysis.

The hydrologic and hydraulic analyses associated with the design or evaluation of dams, or their rehabilitation, in Texas is considered the practice of Engineering and, as such, subject to the Texas Engineering Practice Act, as amended.

1.3 Copies

Copies of the guidelines may be viewed online at <www.tceq.state.tx.us/goto/damhhguidelines>.

1.4 Feedback

Direct any questions or comments on the content of these guidelines to the coordinator of the Dam Safety Program, Texas Commission on Environmental Quality.

1.5 Applicability

The guidelines described in this document apply to all dams and all design floods determined for dams under the jurisdiction of the TCEQ Dam Safety Program. Some dams may also need to meet the requirements of other agencies such as the NRCS or the U.S. Army Corps of Engineers. Design floods developed to meet requirements of these agencies will be accepted by TCEQ as long as their results are shown to be at least as conservative as would be required by this document.

1.6 Definitions

Many of the words and terms used throughout these guidelines are defined in the Glossary.

1.7 Acknowledgments

These guidelines were prepared by Freese and Nichols, Inc., Austin, under the direction of Warren D. Samuelson, P.E., coordinator of the Dam Safety Program, Texas Commission on Environmental Quality, and Jack Kayser, Ph.D., P.E., senior water resources engineer, Dam Safety Program, TCEQ.

These guidelines have drawn liberally upon the work of many agencies and individuals who have greatly contributed to the state of the art in hydrologic and hydraulic designs of dams in the United States. Acknowledgments of the contributions of these agencies and individuals appear throughout the text of these guidelines.

Appreciation is expressed to the following organizations and firms that supplied data and gave input or reviewed these guidelines:

Federal Agencies

U.S. Army Corps of Engineers
Natural Resources Conservation Service
U.S. Bureau of Reclamation
Interagency Committee on Dam Safety
Federal Energy Regulatory Commission
National Weather Service
U.S. Geologic Survey

State Agencies

Texas Department of Transportation
North Central Texas Council of Governments

Professional Associations

Association of State Dam Safety Officials
American Society of Civil Engineers

Project Consultants

Freese and Nichols, Inc., Austin
Raymond Chan and Associates, Austin

CHAPTER 2

Submitting Reports

2.0 Introduction

All hydrologic and hydraulic analysis reports investigating one or more dams in Texas are to be prepared by, or under the direct supervision of, a professional engineer with direct responsibility for the analysis of the dam. Reports submitted to the TCEQ must document the technical basis for the analysis sufficiently for a thorough review by TCEQ personnel, including methods used, key assumptions, the results and conclusions of the analysis, and any recommendations. Such reports must also include all pertinent and significant data utilized in the analysis and necessary for the TCEQ to perform their desired review of the analysis. The engineer should supply the required information regardless of whether the analysis is a standalone review of an existing dam or supports the design of a new dam or the rehabilitation of an existing one.

The TCEQ's requirements as to detailed preparation of plans, specifications, and designs are not part of these guidelines.

2.1 Minimum Requirements for Submission

For hydrologic and hydraulic studies that are either individual or part of a design project, include their bases and results in a report. Fill in all appropriate Dam Information Forms (Appendix B) and submit them with the report. Tabulate the following data in the report, if applicable:

Rainfall and Runoff Information

- characteristics for the entire watershed and all subbasins, as applicable to calculation methods
- data used to develop parameters describing the watershed characteristics, including any available calibration data
- design-flood inflow and discharge hydrographs
- reservoir routing data and parameters
- discharge-frequency relationships
- determinations of hydraulic roughness
- water-surface profiles

Dam and Spillway Information

- spillway stage–discharge relationships
- maximum height and reservoir storage values
- elevation-area-storage relationship
- key operational elevations for the dam and spillway
- pertinent spillway dimensions
- energy-dissipating facility features
- results of hydraulic model tests when the hydraulic design is based on a model study
- details of low-flow release structures

Breach-Analysis information

- breach parameters
- profile of peak flood levels
- profile of warning time versus distance downstream
- delineation on the best available mapping base of the extent of inundation for the normal pool and design-flood breach events for the project
- identification of any potential loss of public services and of critical facilities
- assessment of hazard-potential classification

CHAPTER 3

Dam Classification

3.0 Introduction

Dams more than 6 ft high fall under TCEQ jurisdiction and are to comply with TCEQ regulations on dam safety regardless of whether the TCEQ requires a water right for the impoundment.

The TCEQ regulations and these guidelines do not apply to:

- dams designed by, constructed under the supervision of, and owned and maintained by federal agencies such as the Corps of Engineers and the Bureau of Reclamation;
- embankments used for roads, highways, and railroads, including low-water crossings, that may temporarily impound floodwater;
- dikes or levees designed to prevent inundation by floodwater; and
- off-channel impoundments authorized by the TCEQ under Texas Water Code Chapter 26.

3.1 Dam Size Classification

The classification for size based on the maximum height of the dam or maximum reservoir storage capacity shall be in accordance with Chapter 299 of the Texas Administrative Code (TAC).

3.2 Design-Flood Criteria

Existing and proposed dams must safely pass the design-flood hydrograph, expressed as a percentage of the probable maximum flood. The design flood is determined based upon the size (previous section) and hazard-potential classification (Chapter 9) of the dam. TAC Chapter 299 describes the required design flood for the various combinations of size and hazard classification. Safely passing a flood for an existing dam means discharging the flood without a failure of the dam or one of its critical elements. A failure would be considered an unintended release of impounded water due to the loss of all or a portion of the dam or affiliated structure. For dams without a structural design that allows for safe overtopping, any overtopping of an earthen embankment would be considered not safely passing the flood.

Design-flood criteria established by other public agencies, if shown to be more conservative, will generally be acceptable. Those that may produce a less conservative result, such as the FEMA Inflow design-flood methodology, if based on a properly prepared incremental risk analysis, may be acceptable, but will require a thorough review of the risk analysis as well as the hydrologic and hydraulic analyses.

3.3 Minimum Freeboard

No freeboard for wave action is required for existing dams above the peak design-flood level, either for determination of existing conditions or for the design of an upgrade or modification.

New dams should have appropriate freeboard. As part of the freeboard calculations for a proposed new dam, consider an appropriate wave run-up. Overtopping from wave action due to design wind loads, as described below, is generally not allowable. It may, however, be acceptable if the design engineer can show reasonable cause—as in the case of a new concrete dam or a dam with other appropriate slope protection on the downstream side. Freeboard between the effective crest of the dam and the various water surface elevations that may be associated with the reservoir is to be based on suitable assumed wind speeds and related wave heights.

The longer that a reservoir is shown to be at or above a certain level, the higher the potential wind speeds that should be considered. In addition, the timing of the peak lake level with respect to the storm event that generated it is also a factor. For example, the freeboard above the maximum normal operating level should be greater than or equal to the maximum wave height, including run-up, caused by the maximum wind potential along the maximum fetch of the reservoir.

Freeboard above higher flood levels in the reservoir, such as the top of any dedicated flood pool, should consider wave height and run-up for lesser winds consistent with the potential risks associated with wind-driven waves overtopping or eroding the embankment and potential flood durations at those levels. Freeboard above the maximum reservoir level resulting from the design flood does not need to reflect significant wave height from unusual wind conditions, if it can be shown that the peak reservoir level occurs after the intense portions of the storm that generated the design flood. Multiple storm events do not need to be considered.

The freeboard should include the expected wind effects that could occur during the design-flood event if the peak reservoir level occurs within the critical portion of the storm event itself. This critical portion would generally be considered the portion of the critical duration prior to the break point, if the temporal distribution described in Chapter 4 is employed. An acceptable rule of thumb would be to use 50 percent of the maximum wind speed if the peak occurs before the break point, 33 percent of the maximum if the peak is after the break point but before the onset of the critical storm, and 20 percent of the maximum wind speed if the peak occurs after the end of the assumed rainfall event.

These are general guidelines and the engineer should provide reasonable explanation of assumed winds for freeboard determination. Appropriate determinations will be needed if a different temporal distribution is used.

All freeboard calculations should include the expected future settlement and consolidation of the embankment after construction in addition to wave run-up.

Determining the Design Flood—Precipitation

CHAPTER 4

4.0 Introduction

The design flood hydrograph for existing and proposed dams shall be derived from the appropriate percentage of the probable maximum flood (PMF), which is, in turn, derived from the estimated runoff resulting from the probable maximum precipitation (PMP). The PMP varies depending on the size and shape of the dam's contributing drainage area. The intent of the precipitation analysis is to find the critical storm size, location, orientation, and duration that would produce the most critical loading on the dam. PMP values in Texas are generally derived from HMR-51 (Schreiner and Riedel 1978) and HMR-52 (Hansen, Schreiner, and Miller 1982) for most of the state and HMR-55A (Hansen et al.1988) for parts of extreme west Texas. (*HMR* = 'hydrometeorological report.') These would apply unless an approved site-specific PMP study is performed. All references to "PMP" are to one of these sources of derivation.

4.1 Watershed Delineation

Many of the dams in Texas can be modeled appropriately with a single basin. However, many will need to be divided into multiple subbasins. The size and delineation of the subbasins is dependent on the rainfall-runoff method used and various hydrologic factors. Subdivision should also be considered if there are portions of the drainage basin that:

- possess hydrologic characteristics obviously different from the average characteristics of the total basin,
- may contribute to delays in flood passage, such as upstream lakes,
- are controlled by large constrictions that can act as hydraulic control structure by restricting, cross-sectional areas and attenuating water flow, as may occur at some bridges,
- have a total drainage area that is too large for averaging a single storm distribution, or
- have stream gauges or observed data that may be used for calibration.

Watersheds should be delineated and their characteristics determined in accordance with the standards of the following references:

- *National Engineering Handbook,* Part 630 (Hydrology) (NRCS 1997)
- EM 1110-2-1417 (U.S. Army Corps of Engineers, 1994)
- Federal Energy Regulatory Commission (2001)

4.2 Minimum PMP Duration

The PMP depths for a particular storm size and range of storm durations are used to determine the critical storm duration for a dam. The intent is to review multiple potential durations of storm events in order to determine a critical event, namely, that which produces the maximum reservoir level. Possible durations would include 1, 2, 3, 6, 12, 24, 48, and 72 hours. The minimum design-storm duration is based on the total contributing drainage area for the dam, as shown in Table 4.1.

Table 4.1. Minimum PMP Duration

Contributing Drainage Area (DA) (sq mi)	Minimum Storm Duration (hr)
DA < 25	1
25 ≤ DA < 100	3
100 ≤ DA < 1,000	6
1,000 ≤ DA < 10,000	24
DA ≥ 10,000	72

The PMP depths should first be determined for the minimum storm duration listed in Table 4.1. Then each possible duration up to 72 hours should be reviewed in order to determine the critical duration. For example, for a reservoir with a drainage area of 80 sq mi, the minimum duration is 3 hr. First, the peak reservoir level from a 3-hour PMP is determined, then that of a 6-hour and a 12-hour PMP event. This continues until the peak reservoir level from a longer duration event is lower than the previous one, thus bounding the critical duration. The duration that produces the maximum reservoir level then becomes the critical duration and that duration event is used for the PMF. If the 72-hour PMP produces the maximum reservoir level, then a 72-hour PMF is utilized. No durations longer than 72 hours need to be reviewed.

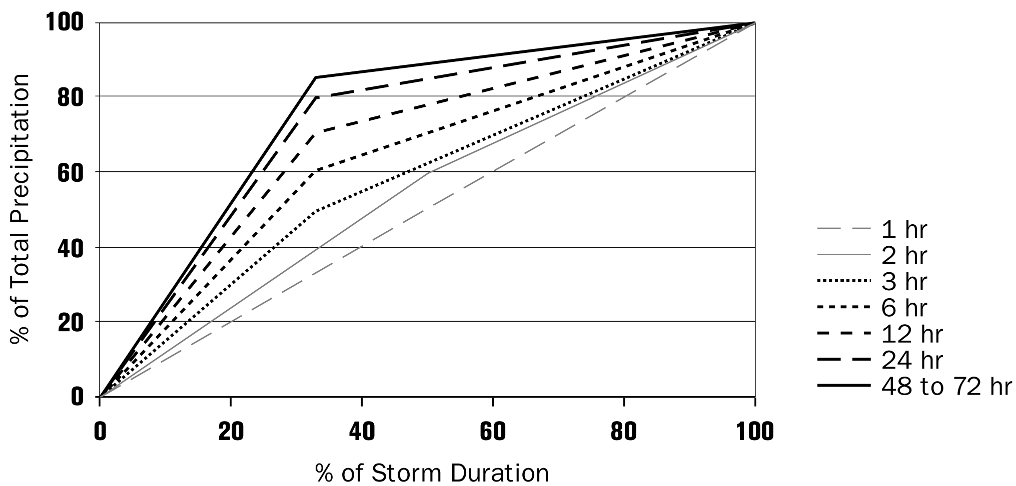
4.3 Temporal Distribution of Design-Storm Precipitation

Distribute the total depth of the PMP, for both the entire basin as well as for each subbasin, as appropriate, temporally in accordance with the dimensionless parameters of Figure 4.1 and Equation 4.1. **Since the new temporal distributions are different for each duration, a dam needs to be evaluated for all of the durations required by Section 4.2 and the peak elevation for each duration must be estimated in order to determine the critical duration for that structure.** This critical peak lake level may then be compared, if desired, to the peak lake level determined by other methods in order to determine which method is more conservative.

The new temporal distributions will tend to reduce the conservatism of the PMP on the flood routings by reducing the intensity of the peak portion of the rainfall event. The result will be tend to include flatter inflow hydrographs, significantly lower peak inflow rates, and slightly lower peak lake levels.

The development of the guideline's temporal distributions is based on observed evidence that near-PMP values for significantly different durations have not occurred in the same event. In other words, though previous methods assumed the PMP value for the peak one-hour event occurred within the same event as the peak PMP value for 24 hours and also for 72 hours, such storms have never actually been observed. Historical data has also shown that the most extreme near-PMP events tend to be front loaded, with most of the rainfall occurring early in the event. The guidelines attempt to provide a reasonably conservative temporal distribution for the given set of durations. It is important to note that only the distribution of the rainfall has changed; the total rainfall amounts for any given duration are unchanged from previous methods. More

Figure 4.1. Temporal Distribution of Total Depth of PMP for All Durations of PMPs



This distribution can be estimated within calculations and spreadsheets as:

Eq. 4.1 For $T \le x$: $P = (T / x) \cdot y$

For $T > x$: $P = y + ((100-y) / (100-x)) \cdot (T-x)$

Where:

P = percentage of total precipitation

T = percentage of storm duration

x, y = coordinates of breakpoint

The breakpoint will vary depending on the duration storm being analyzed (Table 4.2). For a one-hour event, a breakpoint with coordinates at 50 percent, 50 percent is listed for consistency, though that represents a linear distribution of rainfall over the hour.

conservative distributions can be used, such as the HMR-51 or the NRCS distributions.

Table 4.2. Breakpoints for PMP Temporal Distributions

Duration (hr)	x (%)	y (%)
1	50	50
2	50	60
3	33	50
6	33	60
12	33	70
24	33	80
48 to 72	33	85

4.4 Storm Location and Spatial Distribution of PMP

Drainage Areas ≤ 10 Square Miles: Apply the total depth of the PMP, estimated as the point values delineated in HMR-51 and HMR-52, over the entire drainage area for all storm durations.

Drainage Areas > 10 Square Miles: Distribute the total depth of the PMP for all storm durations spatially over the drainage area using the single-centered concentric ellipse pattern and methodology specified in HMR-52. For single basins, the center of the storm should generally be at the centroid of the basin and a basin-average total depth of design storm precipitation calculated for the specified duration. For larger basins, when the watershed is divided into multiple subbasins, the center of the PMP storm isohyets must be moved to multiple locations away from the geometric centroid of the overall drainage area to verify the critical design storm location and orientation that produces the maximum corresponding PMF level in the reservoir. This will generally be the same storm center that produces the maximum basin-average total PMP depth. However, in very large basins (greater than 10,000 sq mi), the location of the storm center producing the maximum rainfall depth and the storm center producing the maximum basin discharge may not be the same, depending on the size and orientation of the various tributaries, so the full flood routing through the reservoir should include iterative trials to determine the critical storm location.

CHAPTER 5

Determining the Design Flood—Runoff Calculations

5.0 Introduction

Precipitation hyetographs, developed as described in Chapter 4, that are calculated for the watershed above a reservoir are used to develop estimates of runoff hydrographs in two basic steps. First, the excess precipitation is generated by deducting estimated losses from the total precipitation. This excess precipitation will generate the full volume of runoff from the storm event. In the second step, the excess rainfall is applied to a suitable unit hydrograph for the basin or subbasins to produce a runoff hydrograph.

5.1 Antecedent Moisture Conditions

Superimpose the PMP upon watershed soils assumed to be saturated. This will equate to losses at the beginning of the design storm equal to zero or Natural Resources Conservation Service Antecedent Runoff Conditions III (ARC III), or some other equivalent and approved assumptions. In Texas, there is no need to analyze snowmelt contributions to runoff or frozen ground conditions for infiltration for design-flood calculations.

5.2 Infiltration Losses—Excess Precipitation

Determination of excess rates of precipitation and infiltration losses can be determined by one of several precipitation loss methods. The two most common are:

- Initial and Constant-Rate Loss Method
- NRCS Curve Number Loss Method

Other usable methods include:

- Green and Ampt Loss Method
- Holton Loss Rate
- Exponential Loss Rate

These methods are described in most hydrology textbooks and in user manuals for modeling software.

For certain areas—paved areas, buildings, and open water—it may be appropriate to assume a certain percentage of the basin or subbasin has no infiltration at all. Such areas are typically designated by an impervious-area percentage in the description of the basin characteristics. A large area—such as the reservoir area itself, if it is a significant portion of the drainage area—can be modeled as a unique subbasin with zero infiltration losses.

The methodologies for the first two methods and their associated input parameters are described below.

Initial and Uniform

This simple method is widely used and consists of establishing an initial loss amount and a uniform loss rate. The initial assumption is that all rainfall is lost to infiltration up to the initial loss amount. After that, the uniform rate is adjusted to the calculation time step and then subtracted from each rainfall amount for that time step. The remaining precipitation is the excess rainfall.

For all design-flood calculations, the initial loss amount should be zero, equivalent to saturated conditions. The uniform rate is estimated based on soil types. The values will typically range from 0.05 in/hr for clays to as high as 0.4 in/hr for sandy soils.

NRCS Curve Number Loss Method

The NRCS has standardized detailed procedures for developing estimates of infiltration rates based on soil types and land use characteristics. The process is summed up in the derivation of the curve number (CN), from which estimates for soil moisture deficit, initial abstraction, and the resulting excess rainfall are derived. Multiple NRCS publications are available that provide guidelines for estimating the CN based on soil classifications and land use parameters. Soil classifications are most readily obtained from NRCS County soil maps. Many of these were published when the agency was known as the Soil Conservation Service (SCS). All of the soil classifications listed in the County Survey reports are classified in one of the four hydrologic soil groups, A, B, C, and D. These four groups range from the most pervious, A, to the most impervious, D. Generally, multiple groups will be represented within a basin or subbasin and the representative values can be averaged over the basin, weighted by representative area. Either calculate the average to develop a basin average hydrologic group and then assign the entire basin a CN, or assign each of the various soil types within the basin a hydrologic group and then a CN, and average all the CNs, weighted by area.

Most of the available tables indicating CN assume an ARC (formerly referred to as *antecedent moisture condition*, or *AMC*) II antecedent condition. This needs to be adjusted to reflect ARC III conditions.

Infiltration Loss Methods

For comparison, Table 5.1 shows a general relationship between the NRCS soil classification (described in the next section) and uniform loss rates.

5.3 Land-Use Assummptions

For developing the design storm runoff hydrograph for design and risk assessment of proposed dams or modifications to existing dams, assume land uses expected to exist at the completion of the modification or construction project. Dam owners will be held responsible for the safety of the dam throughout its entire life; therefore, they should attend to the build-out conditions that are reasonably expected to occur within the entire drainage area during the operational life of the dam.

5.4 Unit Hydrograph Method

The PMP needs to be transformed into the PMF runoff hydrograph for each basin or subbasin using an acceptable unit hydrograph method. The two most commonly used methods for hydrologic and hydraulic studies associated with dams are:

- Snyder Unit Hydrograph Method
- NRCS Dimensionless Unit Hydrograph Method

Other possible methods that are used include:

- Clark Unit Hydrograph Method
- Kinematic Wave Method

Table 5.1. NRCS Hydrologic Soil Groups and Uniform Infiltration (Loss) Rates

Soil Group	Description	Range of Uniform Loss Rates (in/hr)
A	Deep sand, deep loess, aggregated silts	0.30–0.40
B	Shallow loess, sandy loam	0.15–0.30
C	Clay loams, shallow sandy loam, soils low in organic content, soils usually high in clay	0.05–0.15
D	Soils that swell significantly when wet, heavy plastic clays, certain saline solutions	0.00–0.05

References: NRCS 1997, Skaggs and Khaleel 1982

These two most widely used methods, the Initial and Constant Rate method and the NRCS Curve Number method, are contrasted in Table 5.2.

Table 5.2. Infiltration-Loss Methods Compared

Method	Pros	Cons	References
Initial and Constant Rate	■ Simple and easy to use. ■ Easy to calibrate. ■ Apply to all storm durations.	Since it does not reflect varying loss rates, it can misestimate losses within the event, particularly those of very short duration.	■ EM 1110-2-1417 (U.S. Army Corps of Engineers 1994) ■ *Hydrology Handbook* (ASCE 1996)
NRCS Curve Number	■ Simple and easy to use. ■ Easy to calibrate. ■ Good availability of material to estimate parameters for ungauged areas.	Infiltration rate will be asymptotic to zero and losses tend to be understated for storms longer than 24 hours in duration.	■ EM 1110-2-1417 ■ *Hydrology Handbook* (ASCE 1996) ■ Win TR-55 (NRCS 1998) ■ *National Engineering Handbook* (NRCS 1997)

These methods are described in most hydrology textbooks and in user manuals for modeling software.

Procedures for the first two methods and their associated input parameters are described below.

Snyder Unit Hydrograph

The Snyder method estimates a peak discharge and a time to the peak of the unit hydrograph. It also estimates shape parameters. Rainfall runoff models, such as HEC-1, will typically complete the unit hydrograph based on assumed parameters and relationships. Typically, two parameters are needed to develop a Snyder Unit Hydrograph:

- T_L, lag time
- C_p, shape factor, also commonly expressed as C_p640.

The lag time has historically been calculated in multiple ways. The following equation is best suited to regional parameters:

$$T_L = C_T(L \cdot L_{CA}/S^{0.5})^{0.38}$$

T_L = Lag Time (hr)
C_T = coefficient
L = hydraulic length of watershed along the longest flow path (mi)
L_{CA} = hydraulic length along the longest water course from the point under consideration to a point opposite the centroid of the drainage basin (mi)
S = weighted slope of the basin (ft/mi), measured from the 85% to the 10% points along the longest stream path in the basin (EM 1110-2-1405)

The value C_T is a dimensionless parameter that is typically assumed to be consistent for various areas of the state. For instance, it could be estimated from neighboring areas or calibrated for the whole or portions of the basin, and then applied to multiple subbasins within the watershed.

Note that there are multiple forms of the Snyder equation for T_L. Some use ft/ft for the slope and some do not include the slope at all. If a regional value for C_T is used, verify that the same equation was used in the study within which it was developed. Values generally range from about 0.7 up to about 3.0, though values outside that range have been calibrated.

The shape factor, C_p640, reflects the sharpness of the hydrograph. High values, up to about 500, reflect a rapidly responding basin with a sharp peaked hydrograph. Low values, such as 250, generally reflect a flatter, slow responding basin with a longer, flatter hydrograph. These values are generally divided by 640 and entered into HEC-1, if that model is used, as the C_p value, ranging from about 0.4 to 0.8. Generally, smaller C_T times are associated with higher C_p640 values, though many exceptions exist.

NRCS Dimensionless Unit Hydrograph

Only one parameter is used in the models that use the NRCS Unit Hydrograph method—the lag time, T_L, typically estimated as 0.6 times the time of concentration, T_C, which is estimated through a procedure of several steps based on parameters reflecting the basin. The factor 0.6 for conversion from T_C to T_L has been shown to vary with certain urban characteristics, but, without detailed information, 0.6 is generally considered acceptable for most situations. The time of concentration, T_C, is the time it takes for water to flow from the most hydraulically remote point of the drainage area to the outlet of the drainage basin. There are multiple methods to determine the time of concentration, each generally associated with a particular unit hydrograph estimation procedure.

One of the more common methods for estimating T_C is to sum three runoff time components: overland sheet flow, shallow concentrated flow, and open channel. *Sheet flow* reflects the uppermost end of the basin and consists of flow traveling over the open planar surfaces and not in formed channels. Its length is generally estimated from maps, but should be no greater than 100 ft. The primary factors for estimating the time for sheet flow are length, slope, and roughness. *Shallow concentrated flow* reflects the flow as more concentrated, but still not in a fully formed channel. It may be in minor ditches and swales and is also affected primarily by the length, slope, and roughness. *Open channel flow* is the flow in distinct, well formed channels, within which flow can be readily depicted using Manning's equation. More than one channel type, with separate time calculations for each, may be added to obtain the overall time of concentration. However, there cannot be more than one component for sheet flow or shallow concentrated flow.

The intention of these estimates is to sum the estimated travel time of flow across each component. The factors listed above are estimated in order to determine a flow velocity which is assumed constant over the defined length. Times for each of the three components are estimated, totaled, and the sum used as the time of concentration for the basin, which is then adjusted to estimate the lag time, T_L. These two widely used unit hydrograph methods, Snyder's and the NRCS method, are contrasted in Table 5.3.

5.5 Calibration

The calibration of hydraulic and hydrologic runoff parameters is strongly preferable in most cases. For breach analyses, a recently prepared PMF analysis usually suffices. If the previously prepared

Table 5.3. Unit Hydrograph Methods Compared

Method	Pros	Cons	References
Snyder's Unit Hydrograph	■ Simple and easy to use. ■ Easy to calibrate. ■ Applies to wide range of area.	Parameters cannot be estimated from field observations. Values must be calibrated or estimated from similar areas.	■ EM 1110-2-1417 (U.S. Army Corps of Engineers 1994) ■ *Hydrology Handbook* (ASCE 1996)
NRCS Dimensionless Unit Hydrograph	■ Simple and easy to use. ■ Easy to calibrate. ■ Good availability of material to estimate parameters for ungauged areas.	Not well-suited to large drainage areas. Should not be used for subbasins larger than about 20 sq mi.	■ EM 1110-2-1417 ■ *Hydrology Handbook* ■ TR-55 (NRCS 1998) ■ *National Engineering Handbook* (NRCS 1997)

PMF was not based on calibrated runoff values, a new calibration is not needed, except for large, high-hazard dams. For PMF determinations associated with new dams or the upgrade of existing dams, use calibrated values for all intermediate and large high-hazard dams and large significant-hazard dams. Exceptions may be allowed if the values chosen can be demonstrated to be conservative or if insufficient data are available.

The following suggestions should be considered during calibration:

- The process compares calculated runoff hydrographs to observed hydrographs from gauges or calculated from lake levels. Inflow rates estimated from reservoir levels will generally need some smoothing, as small errors in lake-level measurements typically represent a large error in inflow. However, these will tend to be self-correcting in subsequent time steps.
- There is error in all observed data, so multiple storms should be used for calibration. Three or four events are preferable. If an event suggests values that are inconsistent with others, consider not using that event.
- Most rain gauges use daily values and their temporal distribution will need to be estimated based on adjacent hourly gauges. Some lag time between the gauges is often appropriate.
- The Theissen Polygon method is a simple and suitable way to distribute rainfall values across basins. However, multiple tools that make use of GIS technology are also available and well-suited to the task.
- Distributing observed rainfall values across large areas will tend to exacerbate the errors inherent in the rainfall. Inevitably, the calibration of infiltration rates tends to be more of a correction of rainfall errors than a determination of true infiltration. Unless a strong, repetitive pattern is noted, determine loss rates from analysis of the soil types with little emphasis on the calibrated infiltration rates.
- When calibrating a model, first adjust infiltration losses to obtain a matching runoff volume. Then, adjust the parameters that reflect the timing to match the time of the peak flow. Finally, adjust the hydrograph shape, including the magnitude of the peak flow. Some iterations will typically be necessary.
- Minimize the number of variables to be calculated. For example, rather than attempting to adjust the lag time for each subbasin within the Snyder methodology, assume one C_T and one C_p640 value for the entire group of subbasins above the observed hydrograph, with each basin's lag time calculated based on the dimensions of its own basin. If variations in the C_p640 values are deemed appropriate, then it helps to keep the ratios between them constant so that all are effectively calibrated together.

CHAPTER 6

Determining the Design Flood—Routing Methodologies

6.0 Introduction

The last of the three primary steps in determining the design flood is the routing of hydrographs through the reservoir. Watersheds modeled as a single basin need no streamflow routing, and the design-flood runoff hydrograph is the reservoir-inflow hydrograph. Watersheds modeled with multiple subbasins need streamflow routing unless all subbasins drain directly into the reservoir.

6.1 Methods for Hydrologic and Hydraulic Streamflow Routing

In models that require routing of the design-storm runoff hydrographs through a stream channel, either a hydrologic or hydraulic routing method may be used. The hydrologic methods are either empirical or semi-empirical. Approved hydrologic routing methods include:

- Muskingum
- Muskingum-Cunge
- Average-Lag
- Successive Average-Lag (Tatum)
- Modified Puls
- Working R and D

Hydraulic routing methods are more complicated and difficult to use as they are based on theoretical hydraulic equations, but they more accurately reflect flood-routing conditions. Hydraulic methods are typically used under the following conditions:

- Very flat channel slopes, less than 5 ft/mi
- Wide floodplains with significant storage effects on the hydrograph
- High flows from a tributary
- Unusual backwater effects from structures

Acceptable hydraulic modeling methods include:

- Kinematic Wave
- Dynamic Wave
- Diffusion Wave

6.2 Base Flow

Estimates of base flow conditions to which calculated runoff hydrographs are added are sometimes appropriate on larger rivers, particularly for frequency flood–level events, such as 10-year or 100-year floods. However, in Texas, they are rarely a significant component of the design-storm peak flows, generally much less than 1 percent. For any river with a dependable flow that could be counted as a base flow, the drainage area is usually quite large and the resulting design-flood runoff will still dwarf the base flow. For these reasons, base flow is not required within design-flood calculations but may be employed if the analyst deems appropriate; it will typically be of the same order of magnitude as median flows or estimated as the receding limb of an antecedent event.

6.3 Hydraulic Input Parameters

All of the streamflow routing methods use various input parameters, generally measured or estimated from the physical characteristics of the channel. All will include a parameter that measures the length of the channel directly or reflects it in an estimate of travel time. For these, use the full length of the channel without assuming shortening due to overbank flooding. Even in very high flows, the predominant conveyance is usually within the channel itself. If the overbank flows are thought to be a dominant factor in the flood conveyance, then employ a routing method that takes differing floodplain flow characteristics into account, possibly including hydraulic models.

The Muskingum routing method, instead of physically measured values, incorporates an estimate of storage effects though the storage coefficient, x. This value ranges from 0.0 to 0.5, where 0.0 reflects a straight translation of a hydrograph and 0.5 reflects a storage-controlled routing process. The former would indicate of a steep narrow channel with little attenuation of the peak expected. The latter would reflect a broader, flatter channel with significant attenuation of the peak from overbank storage effects. This parameter cannot be measured from physical characteristics of the channel, but can be calibrated or estimated with experience.

Most methods will also typically use some description of a typical cross-section or multiple cross-sections. These should simply be measured from available mapping and should represent average conditions. For hydraulic models, where the entire length of the channel is modeled with cross-sections, reasonable mapping will be necessary, though rarely are surveyed cross-sections justified. For unsteady-flow hydraulic models, such as the NWS models and HEC-RAS, it is important that the cross-sections reflect with reasonable accuracy the existing floodplain storage. In steady-state models, there is a tendency to place cross-sections at constrictions and other features that control the flow rate. However, in unsteady models, the full flood storage of the floodplain tends to control the downstream movement of the hydrograph. Therefore, cross-sections should reflect both the constricted sections and the wider sections, as well as the storage that occurs in tributaries and adjacent draws. Off-channel storage, or ineffective flow area—often ignored in steady-state models—can be significant in unsteady models.

Manning's Roughness Coefficients

Except for the purely empirical hydrologic equations, each of the streamflow routing models will use the most widely recognized flow relationship, Manning's equation. Input parameters to Manning's equation consist simply of descriptions of the topography through the use of cross-sections, either in detail or simplified, and the roughness coefficient.

Numerous sources exist that describe the use of the equation and provide means to estimate Manning's coefficient (n), both for channel flow and overbank flow. The primary criterion is the size of the roughness particle, usually vegetation or the exposed channel surface, relative to the overall flow area. For example, the same grass-lined channel will have a smaller value for n if the channel area is large than if it is small. For that reason, roughness values will decrease with increasing flow for a consistent channel. However, this trend is often countered by the fact that roughness in the form of vegetation tends to increase quickly when significant portions of the flow are in the overbank areas. Single n values for a channel regardless of flow apply only in simplified estimates or in a narrow range of flows.

For design-flood calculations, where there is likely to be a large component of flow area in the floodplain, some variation in the value of n will need to taken into account. The simplest way to do this is to use three n values, one for the channel and one for each overbank. This is a very common technique, used in some routing methods in HEC-1 and often in the hydraulic model, HEC-RAS. The NWS models require input roughness values that are related to each top-width elevation or to flow, so weighted values need to be determined. These are typically best weighted by conveyance rather than by area as that will proportion the effective roughness relative to the portion of the flow it affects.

Table 6.1 compares some of the more common hydrologic and hydraulic streamflow routing methods and gives some of the pros and cons of each.

6.4 Calibration

The calibration of hydraulic and hydrologic streamflow routing parameters, similar to runoff parameters, is strongly preferable in most cases when sufficient data is available. For breach analyses, a recently prepared PMF analysis can usually be employed. If the previously prepared PMF was not based on calibrated runoff values, a new calibration is not needed, except for large high-hazard dams when sufficient data are available. For PMF determinations associated with new dams or the upgrade of existing dams, use calibrated values for all intermediate and large high-hazard dams and large significant-hazard dams. Exceptions may be allowable if the values used can be demonstrated to be conservative or insufficient data are available. During calibration, consider the following:

- The process compares calculated routed hydrographs to observed hydrographs from gauges or calculated from lake levels. Inflow rates estimated from reservoir levels will generally need some smoothing, as small errors in lake-level measurements typically represent a large error in inflow. However, these will tend to be self-correcting in subsequent time steps.
- There is error in all observed data, so multiple storms should be used for calibration. Three or four events are preferable. If an event suggests values that are inconsistent with others, consider not using that event.
- When calibrating a model, first calibrate runoff components, if they can be isolated, in order to match the flow volume. Then, adjust the streamflow parameters to match the translation time of the peak flow. Finally, adjust the hydrograph shape, including the magnitude of the peak flow. Some iterations will typically be necessary.
- Isolate the variables to be calculated and minimize their number as much as possible. For example, if two stream gauges exist, use the observed upstream hydrograph with lateral inflows estimated and calibrated separately, if possible, and estimate the resulting downstream hydrograph. This will isolate the routing parameters of that reach. As another example, rather than attempting to calibrate a wide range of roughness coefficients, relating all the appropriate values of n to standard channel and overbank values will allow the calibration of only two or three values. This can be carried out with overbank roughness values representing clear and

wooded areas, for instance, that are averaged for different reaches based on field conditions.

6.5 Modeling Through Reservoirs

Hydrologic Routing

For hydrologic routing, the peak design-flood reservoir elevation is determined by routing the estimated inflow hydrograph through the reservoir using one of the various hydrologic models. The methods most typically used for routing a hydrograph through a reservoir in hydrologic models are:

- Level Pool Routing
- Modified Puls

Take the following into consideration:

- Assume that reservoir losses equal zero.
- For gated dams, route the design-flood inflow hydrograph through the reservoir and through the dam's hydraulic control structures using the planned flood operational rules for the spillway or in a manner that takes into consideration downstream flood risks.
- The antecedent reservoir elevation in the reservoir should be the maximum normal operating pool (MNOP) level, which is the highest water surface elevation within the range of planned operating levels for the reservoir, above which floodwaters would be released. No antecedent storm is required. This applies to all upstream reservoirs in the drainage area.
    - ▼ For detention ponds that are dry or do not have significant permanent storage, consider the MNOP to be at the level of the primary outlet, above which water is always released.

Table 6.1. Streamflow Routing Methods Compared

Method	Pros	Cons	References
Muskingum	■ Simple and easy to use. ■ Easy to calibrate. ■ Applies to wide range of channel types.	■ Parameters cannot be estimated from field observations. Values must be calibrated or estimated from similar areas. ■ Does not account for backwater effects of structures or tributaries.	■ EM 1110-2-1417 (U.S. Army Corps of Engineers 1994) ■ HEC-1 user's manual ■ *Hydrology Handbook* (ASCE 1996)
Muskingum-Cunge	■ Simple and easy to use. ■ Parameters can be estimated from field observations.	■ Does not account for overbank or other storage effects well in broad, flat channels.	■ EM 1110-2-1417 ■ HEC-1 user's manual ■ *Hydrology Handbook* (ASCE 1996)
Kinematic Wave	■ Simple and easy to use. ■ Parameters can be estimated from field observations.	■ Does not account for overbank or other storage effects well in broad, flat channels.	■ EM 1110-2-1417 ■ HEC-1 user's manual ■ *Hydrology Handbook* (ASCE, 1996)
Modified Puls Storage	■ Simple and easy to use. ■ Applies to storage routing for reservoirs.Well suited to flat, broad channels with significant overbank storage.	■ Can be difficult to apply to streams if no relationship between flow and storage is available.	■ EM 1110-2-1417 ■ HEC-1 user's manual ■ *Hydrology Handbook* (ASCE 1996)
Dynamic Wave	■ Most accurate, particularly in broad, flat channels, with slopes less than about 5 ft/mi. ■ Best method for translating values calibrated from actual events up to design flood–level event. ■ Good availability of material to estimate parameters for ungauged streams. ■ Only method that can accurately describe the impact of major tributary flow that creates backwater or even reverse flow on the main stem.	■ Difficult to use, requires a large quantity of data.	■ HEC-RAS user's manual ■ NWS user's manuals (DWOPER, NETWORK, DAMBRK)

- ▼ For recharge reservoirs that are normally dry but have no release capabilities, the MNOP would be an empty reservoir provided that it can be shown that the lake has historically been, or will typically be, dry within a week of a major storm event. If not, the design engineer should show a statistically based justification for an appropriate starting water level.
- ▼ For existing storage reservoirs that have not filled up to their MNOP within the last 10 years, use starting levels at both the MNOP and the maximum level of the lake within the last decade. If the dam can safely pass its appropriate design flood at the lower historical level, but not at the MNOP, then modifications to enable the dam to pass the design flood will still be required. However, these modifications do not need to be initiated until such time as the reservoir reaches a water level starting at which it cannot safely pass the design flood. Determine this “trigger starting elevation”—at which the dam is overtopped by the design flood—and report it along with the rest of the analysis.

- Do not assume the reservoir to be drawn down below the maximum normal operating level in advance of the design storm.
- For new structures, no long-term effects of sedimentation on flood storage capacity need to be assessed for flood routing. For modification or rehabilitation of existing structures, a revised state-storage curve, accounting for sedimentation, should be developed from a field survey.

Hydraulic Routing

For hydraulic routing of hydrographs through reservoirs, there is no distinction between streamflow and reservoir routing. The reservoir is simply modeled with cross-sections as part of the stream with the spillway modeled either directly in the computer model or as an internal boundary based on the spillway rating curve. Though the modeling methodology is the same, certain issues should be considered.

- Through all but the shallowest portions of the reservoir, water levels are not sensitive to roughness coefficients.
- Adjust cross-sections and the intervening lengths to match the overall reservoir elevation-storage relationship.
- For gated dams, route the design-storm inflow hydrograph through the reservoir and through the dam’s hydraulic control structures using the planned flood-operation rules for the spillway or in a manner that takes into consideration downstream flood risks.
- Do not assume the reservoir to be drawn down below the maximum normal operating level in advance of the design storm.

Each method will require hydraulic data about the reservoir and spillway, as described in Chapter 7.

6.6 Design-Flood Hydrographs

After estimating the full PMF hydrograph, determine the design-flood level, based on the size and hazard classification as described in Section 3.2. For hydrologic models that produce a full reservoir-inflow hydrograph, determine the design flood by multiplying each ordinate of the PMF inflow hydrograph by the required percentage. For example, if the design flood is 75 percent of the PMF, then multiply the 100 percent PMF-inflow hydrograph ordinates by 0.75. Make no adjustments to the precipitation data. This design flood–inflow hydrograph is then routed through the reservoir appropriately to determine the peak design-flood level.

For models using hydraulic flood routing methods for the streamflow components and the reservoir, similarly adjust each runoff hydrograph that represents a boundary or lateral inflow hydrograph, multiplying each ordinate of the hydrograph by the specified percentage. Then route these adjusted hydrographs would through the hydraulic model to determine the design-flood level.

6.7 Computer Models

Acceptable computer models for hydrologic and some hydraulic modeling methods include:

- HEC-HMS (U.S. Army Corps of Engineers)
- HEC-1 (U.S. Army Corps of Engineers)
- SITES (National Resource Conservation Service)
- WIN TR20 (National Resource Conservation Service)
- WIN TR55 (National Resource Conservation Service)

Acceptable computer models for hydraulic modeling include:

- HEC-RAS, unsteady flow (U.S. Army Corps of Engineers)
- NWS Dynamic Wave Models (DAMBRK, DWOPER, FLDWAV) (National Weather Service)

Other models may be acceptable upon written approval of the coordinator of the Dam Safety Program.

CHAPTER 7

Hydraulic Design Criteria

7.0 Introduction

Important components of any design flood determination are the accurate representation of the elevation-area-capacity (EAC) relationship for the reservoir and the dam's spillway rating curve. Each is an essential component of the process of routing hydrographs through the reservoir.

7.1 Elevation-Area-Capacity

Determine the EAC from the best mapping available, commonly by simply measuring the area within the reservoir at all available contours. GIS techniques often suffice, giving due importance to properly accounting for islands within the overall area. In many areas of Texas, U.S. Geological Survey 1:24,000 mapping is the best available with 10-foot contour intervals. If no other updated information is available, these are generally sufficiently accurate for flood routing. In such situations, measure and plot the areas at each contour interval from the bottom of the reservoir to the first contour above the top of the dam. Then use the curve generated through these points to pull off areas at individual one-foot increments and tabulate a summation of the average area for each one-foot increment. This process will generate appropriate volumes of storage at each elevation needed for flood routing.

For existing reservoirs for which no mapping below the water surface is available, this process can be performed starting with an assumed storage at the maximum normal operating level, which will be used as the starting water surface elevation for the flood routing. Incremental storage amounts below the starting water surface do not impact hydrologic flood routing procedures.

For hydraulic routing procedures, do not use an EAC directly, as the reservoir is to be modeled using cross-sections. As described in the previous chapter, adjust the cross-sections or the distances between them so that the volumes calculated within the model are reasonably close to the actual EAC.

7.2 Spillway Rating Curves

The spillway rating curve needs to be determined for both existing and proposed reservoirs and for each component of the dam that will be used to pass flood flows during the design flood event. For many dams, this will be a combination of a principal spillway that is used for all flood events and an emergency spillway that is only used in larger, rarer events. Numerous sources describe appropriate methodologies for determining rating curves. Widely used references—for both existing and proposed spillways—include *Design of Small Dams* (U.S. Department of the Interior, Bureau of Reclamation, 1987) and NRCS methodologies.

Principal Spillways

Rating-curve development needs to reflect the unique characteristics of the individual spillway. Ungated spillways will generally be shaped as a weir—either ogee, sharp crested, or broad crested—or as a drop-inlet, or morning-glory, spillway. The weirs would utilize the simple weir equation, $Q = CLH^{3/2}$, with C set by the shape and dimensions of the structure. In planning stages, C can be assumed to be constant, but in design-level analyses C will also vary with the height of water over the crest. Gated spillways will typically require the same procedure for determining total capacity, assuming the gates are opened sufficiently so as to not affect the flow. For discharges through partial gate openings, use orifice-flow assumptions. Standard drawdown profiles are necessary, assuming unimpeded flows, in order to determine the size of the gate opening needed to switch back to weir control for the rating curve.

Rating curves for drop-inlet, or morning-glory, spillways are generally calculated assuming two different types of hydraulic control. For low flows, the circumference of the inlet operates as a weir, with discharge estimated using the weir equation. For higher flows, the inlet will work as an orifice at the narrowest portion, or throat, of the vertical column. If the spillway conduit through the dam is designed for pressure flow, then the hydraulic control may rest with the sum of energy losses acting through the closed system as a whole. In these cases, sum entrance, bend, friction, and exit loss coefficients, along with other losses that may apply, and determine the rating curve determined for a closed, pressure-flow system. For these spillways of these types, make calculations assuming each type of flow patterns and use the lowest discharge as the controlling situation.

Points on the final rating curve developed for reservoir routing reflect the total discharge, typically more points at the lower end of the curve where the rates change more rapidly, with points plotted for key break points such as the crest of the emergency spillway or where changes in the gate operating procedures occur.

For proposed dams and new spillways, keep in mind the following concepts in order to determine the most appropriate principal spillway type:

- Generally use drop-inlet, or morning-glory, spillways when there is plenty of available flood-storage volume. The flow capacity of the spillway does not significantly increase once the reservoir reaches a level at which the spillway "plugs" or operates under pressure or orifice control. Morning-glory spillways are well suited to flood control.
- In morning-glory spillways, it is preferable that the conduit through the dam be designed to have open channel flow with depths no more than 75 percent of the height of the conduit. This will generally require a hydraulically steep slope carrying supercritical flow and a diameter greater than that of the throat of the spillway riser. If that is not practical, then a conduit significantly smaller than the riser that forces pressure flow through the conduit quickly will be preferable. Both of these concepts attempt to minimize the large pressure fluctuations that typically occur with flow transitioning from open channel to pressure flow.
- Larger morning-glory inlets will need anti-vortex devices to break up naturally occurring vortices in the entering flow.
- Gated spillways require considerable additional cost for the operating system, operating personnel, and maintenance. In addition, it is generally perceived that an owner takes on significant additional potential liability with a gated spillway.

Emergency Spillways

Emergency spillways are generally cut into an abutment and have little or no erosion protection from flows discharging through them. For this reason, only for the largest and rarest of floods are they an economical way to pass large quantities of flow. It is often accepted that erosion damage will occur should the emergency spillway operate, but that the effective cost of the very infrequent repairs is much lower than the upfront capital costs of the means to prevent the erosion. Most emergency spillways are built to prevent passage of flows for less than about the 50- or 100-year flood.

Generally, determine rating curves for emergency spillways using a backwater analysis with a steady-state water- surface profile model, such as HEC-RAS. Perform several runs with varying discharges, relating each to a reservoir water-surface elevation. Enough sections are needed such that the most upstream section has minimal approach velocity; ignore any energy losses upstream from that point. Then find the rating curve by assuming that the energy level, not the water surface elevation, at the most upstream section equates to the reservoir level for the specified discharge. Then plot these values and determine the discharges for set elevations from the curve. A standard equation for broad-crested weirs should be used only for rough planning. For such an equation to be accurate, the slope downstream from the crest would need to be steep enough to create supercritical flow down the slope, which has the consequence of causing much more damage than would occur under critical flow.

For proposed dams and new emergency spillways, consider the following in order to determine the most appropriate configuration and location:

- Locate emergency spillways such that any flows that discharge through them will not strike or flow against the dam embankment.
- Configure the channel such that critical flow will occur as far downstream as reasonably possible so as to maximize the length of any erosion path back to the reservoir. The crest can be centered, but the slope downstream from the crest should be set to effect subcritical flow—generally be a slope of about 0.25 percent or less, depending on the vegetation. An alternative would be to address the potential erosion from supercritical flow, either with the provision of an erosion-resistant surface or a determination that the final configuration will not erode sufficiently to cause a significant release of water from the reservoir.
- The crest of the spillway should be set above the 50- or 100-year flood level to minimize its frequency of operation. In general, the less frequently an emergency spillway operates, the more erosion will be acceptable.
- The roughness coefficients used in the analysis should reflect ultimate vegetative conditions of the emergency spillway, not newly constructed conditions.

CHAPTER 8

Dam-Breach Analyses

8.0 Introduction

Breach analyses for existing dams can be performed in one of two manners, *simplified* and *full*.

The simplified method is for proposed and existing small dams, and existing intermediate dams. The method is empirical and conservatively approximates the assumptions for downstream flow and extent of flooding. It will significantly reduce the preparation time and cost of inundation mapping. A full breach analysis may be used for a dam of any size if the owner wishes either to demonstrate a lower hazard classification than that determined using the simplified method or simply to estimate inundation more accurately.

For large dams, use the full breach analysis, whether evaluating a proposed or an existing dam. Also use a full breach analysis for proposed intermediate-sized dams.

8.1 Hydrologic Conditions

Perform full breach analyses for the following hydrologic conditions at a minimum:

- *Sunny-day breach:* Reservoir at its maximum normal operating pool level.
- *Barely overtopping breach:* Inflow design flood set to the percentage of the PMF or design flood that equals the top of the dam. If the dam passes 100 percent of the design flood without overtopping, this scenario does not need to be run.
- *Design-flood breach:* Inflow hydrograph equal to the full design flood.

Compare barely overtopping and design-flood breach runs to runs for the same event assuming that the dam does not fail. The simplified breach method for existing dams only reviews the impacts from a breach occurring with the reservoir at the effective crest of the dam.

If the design-flood breach overtops the dam, the analysis will need to either assume flow over the top of the dam or not. The former adds complexity to the model as the length of the dam that is overtopped is reduced by the breach width, but it also provides a more exact and less conservative determination of breach discharge for existing conditions. For dams for which upgrades to the dam are being considered, assume no flow over the crest, as if the dam were raised to contain the design flood. This is simpler and more conservative.

Initiate the breach at the peak reservoir level under each scenario.

8.2 Downstream Conditions

Under the barely overtopping and design-flood conditions, estimate inflows from downstream tributaries by extending the design-flood ellipses under the HMR-52 methodology over the associated areas. Use the size, location, and orientation of the ellipses used for determining the critical design flood without adjustment. Adjust downstream-runoff hydrographs to the same degree that the barely overtopping flood is upstream of the reservoir. Assume that runoff parameters from the dam's watershed apply, or extrapolate them appropriately to the adjacent basins. No additional calibration downstream is warranted. If the stream on which the breach hydrograph is being routed opens into a much larger river, then multiple considerations are necessary:

- DAMBRK cannot model a dendritic system and can only be used iteratively, with the initial and receiving stream each modeled separately. The stage hydrograph on the receiving stream will serve as a downstream boundary of the initial stream and the discharge hydrograph of the upstream river will serve as a lateral inflow hydrograph to the downstream river. Iterations continue until agreement is reached. HEC-RAS and FLDWAV can model a dendritic system and are usually more appropriate.
- A large volume of water discharging into the receiving stream will tend to distribute flows in both the upstream and downstream directions. Depending on the initial flow on the receiving stream, this can be seen as a reduction in flow rate or even in negative

flows traveling upstream. Therefore, sufficient cross-sections on the receiving stream need to be included upstream of the junction to allow for this phenomenon, if appropriate.

- If the drainage area of the receiving stream is too large for it to be effectively covered by the ellipses representing the design flood, an assumption on initial flows is needed. It can be assumed that significant flows will occur coincidental to the design flood on the tributary; however, their timing and magnitude will be virtually unrelated and indeterminate. Therefore a constant flow hydrograph can be assumed. As an example, this could be equal to the 10-, 50-, or 100-year flood, depending on the relative sizes of the two streams. The larger the ratio of the drainage areas (the receiving stream drainage area divided by the dam watershed area), the smaller the assumed flow level should be in the receiving stream. A reasonable level that puts the receiving stream at or slightly above flood stage before the breach flows arrive will usually produce the critical incremental impact due to the breach. This should be considered in the decision.

8.3 Breach Parameters

Breach Location

Perform the breach analysis on the component of the dam for which failure would create the worst impact downstream, regardless of the relative likelihood of failure. Analyze this component of the structure for the hydrologic conditions listed above. Review each major component of the dam to determine the maximum discharge. This review will not take into account the likelihood of failure of any component, but should look at the most likely configuration of a breach, should one occur. For each structure component, the breach section should be at the maximum portion of the structure that can contain the full bottom width of the breach. For example, if the breach width for an embankment is 200 feet wide, the location should be planned for the lowest 200 ft section of the dam above natural ground at the toe. If the channel downstream is only 50 feet wide, then it would not be in the original channel or at the maximum height of the dam. However, the 200-foot-wide breach at a higher level should be compared to a 50-foot-wide breach at the maximum section over the river channel. Use whichever has the higher peak discharge.

Breach Configuration (Embankments)

Assume breach configurations in an embankment, regardless of the failure mechanism to have, at a minimum, a width of three times the depth of water impounded under each hydrologic condition described above, with vertical side slopes. Any configuration that will produce a larger peak breach discharge will be acceptable. This configuration represents a minimum; larger values may be more appropriate in certain situations, based on the engineer's judgment.

Breach Configuration (Structural Components)

Determine breach configurations in a structural component of the dam, such as the spillway or gravity section, case by case. The minimum breach width will generally match individual elements of the structure, such as one buttress in a slab and buttress dam or one monolith in a gravity non-overflow section. Review multiple adjacent components with varying failure times in order to determine the critical configuration.

Time of Failure (Embankments)

Assume that breach configurations in an embankment—regardless of the failure mechanism—form at a minimum rate of three feet of depth of water impounded per minute under each hydrologic condition described above. Any time of failure that will produce a larger peak breach discharge is acceptable. This failure time represents a maximum. Smaller values may be more appropriate in certain situations, based on the engineer's judgment. Longer times to failure may be used in the modeling process if needed for computational stability and if it can be shown that the peak breach discharge is not sensitive to the time to failure. This is often the case for dams with large storage volumes for which the lake level does not change significantly during the elapsed time of the breach formation.

Time of Failure (Structural Components)

Determine time of failure for a breach configuration in a structural component of the dam case by case. When assuming an individual element of the structure, such as one buttress in a slab and buttress dam or one monolith in a gravity non-overflow section, the time should be instantaneous. Also review more than one adjacent component with varying failure times in order to determine the critical configuration. Base the incremental failure time of adjacent structures on the estimated time for the component to fail due to erosion of the foundation. Present justification in all cases. However, the amount of failure time per adjacent component needs not exceed 30 minutes per component in an alluvial foundation and one hour in a rock foundation. The analyst should also consider the likelihood that this failure mechanism of adjacent structural components will occur in both directions outward from the original component.

8.4 Dam-Breach Models

When using a full breach analysis method, choose appropriate models that can properly determine and route full breach flood waves. For large high-hazard dams, choose an unsteady, dynamic wave model—such as HEC-RAS, DAMBRK, or FLDWAV—to determine the downstream impacts and inundation limits of a breach. For existing small or intermediate-size dams, regardless of hazard rating, a simpler (but less accurate) hydrologic model, such as HEC 1 (HMS), is acceptable for modeling the breach flood wave and to determine inundation limits. However, a dam owner who is performing the breach analysis in order to justify a reduction from a high hazard classification should use one of the dynamic wave models.

8.5 Breach Inundation Lengths

Extend models of breach flood waves far enough downstream to allow analysis of the area that is likely to be significantly affected by a breach of the dam and to provide sufficient information for proper development and execution of an EAP. Though judgment is needed on the part of the engineer performing the analysis, the following guidelines will generally be suitable:

- *Sunny-day breach*—The floodwave should be modeled for a length downstream, beyond which approximately 75% of the flow is within the channel and no structures are threatened.
- Barely overtopping and design-flood breaches—These floodwaves should be modeled for a length downstream, beyond which the increase, due to the breach, in the peak flood level over the non-breach condition is insignificant and with no adverse effects—generally 1 ft in developed areas. In undeveloped areas, a higher differential may be acceptable.

8.6 Dams in Sequence

For design-flood PMF analyses, generally assume that upstream reservoirs remain intact, unless the design flood for the dam under consideration would overtop the upstream dam. If so, further analysis of the upstream dam may be warranted. For an upstream dam assumed to breach, assume it to breach in both the breach and non-breach runs for the downstream dam under consideration. Multiple combinations of breach and non-breach conditions for multiple dams are not necessary.

Assume that downstream dams breach if overtopped by either the breach or non-breach condition and that they do not breach if not overtopped. Assumptions possibly could differ if specific information indicates otherwise. However, as with upstream dams, multiple combinations of breach and non-breach conditions for multiple dams are not necessary. It is possible that the design flood for the upstream dam does not overtop the downstream dam, but does so with a failure of the upstream dam. In that case, assume that the downstream dam fails in the breach scenario and does not in the non-breach scenario. The failure of the upstream dams will contribute to the inflow hydrograph, and have an impact on whether the downstream dam overtops and breaches.

8.7 Inundation Mapping

As described above, perform full breach analyses for a sunny-day breach, a barely overtopping breach (if needed), and a design-flood breach. In a breach study, the barely overtopping breach flood is compared to a barely overtopping flood which does not have a breach. A similar comparison would be done using the design-flood breach, and the design flood with no breach. However, inundation mapping, which is to be used in an Emergency Action Plan, should only include the outline of the water levels reached in the sunny-day and the design-flood breach runs. Do not show non-breach runs in EAP inundation maps. The report should compare breach and non-breach runs in tabular form. As described in the following section, for studies using the simplified breach method, only one inundation condition, equivalent to a flood level at the top of the dam, is to be shown.

Base inundation maps on the best available mapping and present them as sequenced 11" × 17" maps for ease in inclusion in EAPs. Choose scales that allow for clear depiction of structures and major infrastructure, yet that do not generate a large number of sequenced maps that would be difficult to interpret during an emergency by non-technical personnel. USGS 1:24,000 maps are generally suitable, though often outdated with respect to showing structures and major infrastructure. Aerial photos, if available with reasonable clarity and scale, can also be used as a background for inundation maps.

Inundation maps should also indicate times to flood, or the time from the breach to the time that critical structures are flooded. Label these times directly on the map at occasional intervals or at critical structures.

8.8 Simplified Breach Method

For small and intermediate-size dams, the following approximation of peak discharge and inundation limits can be applied. The peak discharge from a breach, using the assumed breach criteria for the dam as described above, can be estimated by the following equation:

Eq. 8.1 $Q_B = 3.1 \cdot B \cdot H^{3/2}$, where

Q_B = peak total discharge from the breach, in cfs
B = bottom width of breach, assumed to be $3 \cdot H$ for embankments or $\frac{1}{2}$ the width of a structural spillway or concrete structure, in ft
H = maximum height of the dam, in ft

The total release discharge (Q_T) would then be:

Eq. 8.2 $Q_T = Q_B + Qs$, where

Q_S = peak discharge capacity from the spillway(s) with the reservoir at the top of the dam, in cfs

Estimate the inundation at selected locations downstream using normal flow calculations and an appropriate representation of the cross-section from available mapping. Manning's equation should be used for the normal flow calculations. Within this equation, roughness coefficients estimates should be increased by 25% to account for increased turbulence and energy losses typically associated with breach floodwaves. The peak discharge should be assumed to attenuate at a linear rate from its peak at the downstream toe of the dam, $Q_{T,}$ down to Q_S over the "inundation length" of the stream downstream. This inundation length, L_U, is to be determined by the following equation:

Eq. 8.3 $L_U = 0.012 \cdot K_S \cdot \sqrt{2 \cdot C \cdot H}$, where

L_U = inundation length in miles.
K_S = Correction factor for spillway size
$K_S = Q_B/Qs$; Maximum value = 2.0
Minimum value = 0.5
C = Total capacity of the reservoir at the top of the dam, in acre-feet
H = Maximum height of the dam, in ft

If the inundation length extends past the point where the stream on which the dam is located flows into a larger stream, continue the length on the larger stream either for the full inundation length or to a point where the normal flow estimates show approximately 75 percent of the flow within the channel and no structures threatened.

For each location of interest, estimate the inundation limits by normal flow calculations, as described above, for sufficient points within the inundation length to map an approximate inundation boundary. Also estimate the limits at all identified structures—residences and infrastructure elements alike. At locations where the stream on which the dam is located flows into a larger stream, the first elevation determined on the larger receiving stream shall be used as the elevation of all backwater inundation on that larger stream upstream of where the tributary joined. The downstream surface calculations can also be performed, if desired, using a standard water-surface-profile model, such as HEC-RAS in steady-state mode, using the interpolated discharges along the inundation length.

Then use this entire approximate inundation limit for impact evaluation, hazard classification, and EAP development. Since time is not considered in this simplified method, consider all structures that may be affected as having no warning time for evaluations of hazard classifications. No times to flood need be estimated or shown on inundation maps developed using the simplified method.

CHAPTER 9

Risk Assessment and Classification of Hazard Potential

9.0 Introduction

Based upon its hazard potential, a dam is classified into one of three risk categories: low, significant, and high. Determine each hazard-potential classification, as described in the following sections, based on the consequences and losses caused by a dam breach under the most critical assumptions for the three hydrologic scenarios described in Chapter 8: sunny-day, barely overtopping (if applicable), and design-flood breaches. In hazard-potential classification, give consideration to potential adverse consequences including deaths and the loss of major infrastructure elements—infrastructure whose loss may indirectly place such a burden on a community that lives would be at risk as a result. Examples include major roads and highways, hospitals, water supply reservoirs, cooling reservoirs, and the like. Hazard assessment is also based on the potential economic risk associated with the flooding of industry, businesses, and infrastructure. This risk and the related regulatory issues are also handled within the confines of local flood protection regulations, such as those enforcing FEMA's 100-year-floodplain regulations. A hazard-potential classification does not reflect any estimate of the likelihood that a dam may fail, but only reviews the consequences of the assumed failure.

In many cases, the hazard potential classification of a dam is visually apparent from field reconnaissance and, with TCEQ approval, a description of the observations will be sufficient to support a determination. These will typically be for dams that are clearly either low or high hazard. In other cases, classifications must be based on the recommendations—in which conservatism is expected—of a licensed professional engineer knowledgeable in the field. To determine the potential of lowering the hazard classification from a conservative field evaluation, detailed studies including dam-breach analyses are to be performed for various hydrologic conditions to evaluate the effects of a failure of a dam, as described below. For this study, use either the full breach analysis or the conservative, simplified breach-inundation estimation method, as described in Chapter 8, to identify the areas at risk.

9.1 Multiple Dams

If failure of an upstream dam will not cause failure of another dam downstream, then the hazard-potential classification of the upstream dam must be determined independently from that of the downstream dam. If the failure of an upstream dam will likely cause the failure of a downstream dam, then the hazard-potential classification of the upstream dam needs to take into account the potential failure of the downstream dam.

9.2 Individual Components of a Dam

Separate components of a dam may not be assigned separate hazard classifications. Determine a single hazard classification for the entire dam. If the harmless failure of an isolated dike or levee reduces the likely failure of the dam, then the failure of the isolated component should be properly incorporated into the design and operation of the dam, as in the example of a fuse-plug spillway. However, a breach analysis to demonstrate the different incremental impacts of various components may be a useful means of allocating limited resources for repair and maintenance.

9.3 Hazard-Potential Classification

Upon completing the mapping of a breach inundation area and the identification of the population and infrastructure at risk, determine the hazard-potential classification. These guidelines do not provide any set numerical markers or definitive equations for defining the hazard classification. The evaluation is to be a conservative judgment based on the available information. General guidelines and descriptions appear in the Texas Administrative Code, Title 30, Part 1, Chapter 299.

9.4 Alternative Means of Assessing Risk and Hazard

Hazard classifications are subjective, but conservative, evaluations reflecting the quantity of structures that exist within the breach inundation area, based on the assumption that structures reflect lives at risk. Though purposefully simple and adaptable to many situations, the procedure may not prove capable of discerning clear distinctions in marginal cases. More precise assessments of risk can be made through a variety of procedures. These alternative methods can be used to determine the population at risk within the inundation zone or the incremental value at risk from a breach relative to the costs of implementing modifications to reduce that risk. Such alternative methods are typically based on evaluating the probability of people being home when the flood passes by, or on a more exact analysis of the flood depth and velocity effects on the population exposed to the flood. Statistical economic methods can also be applied. Submission of alternative analyses such as these must include documentation of a theoretical explanation for the method, verifiable calculations, and adequate references for justifying the assumptions and parameters used. The TCEQ must approve all alternative methods.

Appendix
Submittal Forms

Information Sheet: Existing Dam
Form TCEQ-20344

Information Sheet: Proposed New Construction Modification, Repair, Alteration, or Removal of a Dam
Form TCEQ-20345

Hydrologic and Hydraulic (H&H) Evaluation Summary
Form TCEQ-20346

Engineer's Notification of Completion
Form TCEQ-20347

Texas Dam Safety Program, MC 174
Field Operations Support Division, Office of Compliance and Enforcement
Texas Commission on Environmental Quality
P.O. Box 13087
Austin, TX 78711

INFORMATION SHEET: EXISTING DAM

(PLEASE PRINT OR TYPE)

Reference 30 Texas Administrative Code, Chapter 299, Dams and Reservoirs

SECTION 1: OWNER INFORMATION

Owner's Name ______________________ Title ______________________

Organization ______________________

______________________ ______________________

(Signature of Owner) *(Date)*

Owner's Address ______________________

City ______________ State ______________ Zip Code ______________

Phone Number (   ) ______________ Emergency Contact Phone (   ) ______________

Fax Number (   ) ______________ E-mail ______________

Owner Code *(Please check one)*: ❑ Federal (F) ❑ Local Government (L) ❑ Utility (U) ❑ Private (P) ❑ State (S)
❑ Other (O) please specify: ______________

Year Built ______________ Year Modified ______________

Dam and Reservoir Use *(Please check one)*: ❑ Augmentation ❑ Diversion ❑ Domestic ❑ Erosion Control
❑ Evaporation ❑ Flood Control ❑ Fire Control ❑ Fish ❑ Hydroelectric ❑ Industrial
❑ Irrigation ❑ Mining ❑ Municipal ❑ Pollution Control ❑ Recreation ❑ Stock Water
❑ Settling Ponds ❑ Tailings ❑ Waste Disposal ❑ Other, please specify: ______________

Engineering Firm ______________________

Project Engineer ______________________ Texas P.E. License Number ______________

Engineering Firm Address ______________________

City ______________ State ______________ Zip Code ______________

Phone (   ) ______________ Fax (   ) ______________

E-mail ______________________

SECTION 2: GENERAL INFORMATION

Name of Dam ______________________

Other Name(s) of Dam ______________________

Reservoir Name ______________________

Location ______________ Latitude ______________ Longitude ______________

County ______________ Stream Name ______________

River Basin ______________ Topographic Map No. ______________

Distance & Direction from Nearest City or Town ______________

Last Inspection Date ______________ Inspected by (name of company or agency) ______________

TX Number ______________ Water Rights Number ______________

Date of Emergency Action Plan (EAP), if one exists ______________

Describe the current operating condition of dam ______________

____

If you have questions on how to fill out this form or about the Dam Safety Program, please contact us at 512-239-5195.
Individuals are entitled to request and review their personal information that the agency gathers on its forms. They may also have any errors in their information corrected. To review such information, contact us at 512-239-3282.

SECTION 3: INFORMATION ON DAM

Classification

Size Classification: ❑ Large ❑ Medium ❑ Small

Hazard Classification: ❑ High ❑ Significant ❑ Low

Number of People at Risk ________________ Study Year ____________________

Type of Dam: ❑ Concrete ❑ Gravity ❑ Earthfill ❑ Rockfill ❑ Masonry ❑ Other (specify) __________________

Dam Structure **(dimensions to nearest tenth of foot, volume to nearest acre-foot or cubic yard, areas to nearest acre):**

Spillway Height ________________ ft *(natural surface of ground to bottom of emergency spillway at longitudinal centerline)*

Embankment Height ____________ ft *(natural surface of ground to crest of dam at centerline)*

Structural Height _______________ ft *(bottom of cutoff trench to crest of dam at centerline)*

Length of Dam _________________ ft

Crest Width ____________________________________ ft

Normal Pool Elevation ______________________ ft-MSL

Principal Spillway Elevation ___________________ ft-MSL

Emergency Spillway Elevation _________________ ft-MSL

Top of Dam Elevation _______________________ ft-MSL

Embankment Volume ______________________________ cu yd

Maximum Impoundment Capacity ___________________ ac-ft *(at top of dam)*

Normal Reservoir Capacity ___________________________ ac-ft *(at normal or conservation pool)*

Reservoir Surface Area ______________________________ acres *(at normal or conservation pool)*

Outlet

Outlet Diameter: ___________________ ❑ in ❑ ft *(check one)*

Type: ____________________________

Principal Spillway

Type: ❑ Natural ❑ Riprap ❑ Concrete ❑ CMP ❑ RCP ❑ Other

Width (Diam.): ____________________ ft Capacity: _____________________ cfs

Emergency Spillway

Type: ❑ Natural ❑ Riprap ❑ Concrete ❑ CMP ❑ RCP ❑ Other

Width (Diam.): ____________________ ft Capacity: _____________________ cfs

Total Spillway Capacity: ____ cfs (crest of the dam)

SECTION 4: HYDROLOGIC INFORMATION

Required Hydrologic Criteria (% PMF) _____________ % PMF Passing ________________________________

PMF Study Year ________________________________

Drainage Area: _________________________________ acres, or __________________________________ sq mi

Curve Number (AMC III condition) _______________

Time of Concentration ___________________________ hr

Peak Discharge _________________________________ cfs

Peak Stage ____________________________________ ft-MSL

Storm Duration Causing Peak Stage _______________ hr

Texas Dam Safety Program, MC 174
Field Operations Support Division, Office of Compliance and Enforcement
Texas Commission on Environmental Quality
P.O. Box 13087
Austin, TX 78711

INFORMATION SHEET: PROPOSED NEW CONSTRUCTION, MODIFICATION, REPAIR, ALTERATION, OR REMOVAL OF A DAM

(PLEASE PRINT OR TYPE)

Reference 30 Texas Administrative Code, Chapter 299, Dams and Reservoirs

PLEASE CHECK ONE: ❑ New ❑ Modification ❑ Repair ❑ Removal ❑ Alteration

SECTION 1: OWNER INFORMATION

Owner's Name ______________________ Title ______________________

Organization ______________________

I have authorized the submittal of the final construction plans and specifications to the TCEQ Dam Safety Program according to 30 TAC Chapter 299.

______________________ ______________________

(Signature of Owner) *(Date)*

Owner's Address ______________________

City ______________ State ______________ Zip Code ______________

Phone Number ( ) ______________ Emergency Contact Phone ( ) ______________

Fax Number ( ) ______________ E-mail ______________

Owner Code *(Please check one)*: ❑ Federal (F) ❑ Local Government (L) ❑ Utility (U) ❑ Private (P) ❑ State (S)
❑ Other (O) please specify: ______________

Dam and Reservoir Use *(Please check one)*: ❑ Augmentation ❑ Diversion ❑ Domestic ❑ Erosion Control
❑ Evaporation ❑ Flood Control ❑ Fire Control ❑ Fish ❑ Hydroelectric ❑ Industrial
❑ Irrigation ❑ Mining ❑ Municipal ❑ Pollution Control ❑ Recreation ❑ Stock Water
❑ Settling Ponds ❑ Tailings ❑ Waste Disposal ❑ Other, please specify: ______________

Engineering Firm ______________________

Project Engineer ______________ Texas P.E. License Number ______________

Engineering Firm Address ______________________

City ______________ State ______________ Zip Code ______________

Phone ( ) ______________ Fax ( ) ______________

E-mail ______________________

SECTION 2: GENERAL INFORMATION

Name of Dam ______________________

Other Name(s) of Dam ______________________

Reservoir Name ______________________

Location ______________ Latitude ______________ Longitude ______________

County ______________ Stream Name ______________

River Basin ______________ Topographic Map No. ______________

Distance and Direction from Nearest City or Town ______________

TX Number ______________ Water Rights Number ______________

If you have questions on how to fill out this form or about the Dam Safety Program, please contact us at 512-239-5195.
Individuals are entitled to request and review their personal information that the agency gathers on its forms. They may also have any errors in their information corrected. To review such information, contact us at 512-239-3282.

SECTION 3: INFORMATION ON DAM

Classification

Size Classification: ❑ Large ❑ Medium ❑ Small

Hazard Classification: ❑ High ❑ Significant ❑ Low

Number of People at Risk ________________ Study Year ____________________

Type of Dam: ❑ Concrete ❑ Gravity ❑ Earthfill ❑ Rockfill ❑ Masonry ❑ Other (specify) __________________

Dam Structure (dimensions to nearest tenth of foot, volume to nearest acre-foot or cubic yard, areas to nearest acre):

Spillway Height ________________ ft *(natural surface of ground to bottom of emergency spillway at longitudinal centerline)*

Embankment Height ____________ ft *(natural surface of ground to crest of dam at centerline)*

Structural Height _______________ ft *(bottom of cutoff trench to crest of dam at centerline)*

Length of Dam _________________ ft

Crest Width ____________________________________ ft

Normal Pool Elevation ______________________ ft-MSL

Principal Spillway Elevation ___________________ ft-MSL

Emergency Spillway Elevation _________________ ft-MSL

Top of Dam Elevation _______________________ ft-MSL

Embankment Volume _______________________________ cu yd

Maximum Impoundment Capacity ____________________ ac-ft *(at top of dam)*

Normal Reservoir Capacity ___________________________ ac-ft *(at normal or conservation pool)*

Reservoir Surface Area _______________________________ acres *(at normal or conservation pool)*

Outlet

Outlet Diameter: ___________________ ❑ in ❑ ft *(check one)*

Type: _____________________________

Principal Spillway

Type: ❑ Natural ❑ Riprap ❑ Concrete ❑ CMP ❑ RCP ❑ Other

Width (Diam.): ____________________ ft Capacity: ______________________ cfs

Emergency Spillway

Type: ❑ Natural ❑ Riprap ❑ Concrete ❑ CMP ❑ RCP ❑ Other

Width (Diam.): ____________________ ft Capacity: ______________________ cfs

Total Spillway Capacity: ____ cfs (crest of the dam)

SECTION 4: HYDROLOGIC INFORMATION

Required Hydrologic Criteria (% PMF) ______________ % PMF Passing ________________________________

PMF Study Year ________________________________

Drainage Area: _________________________________ acres, or __________________________________ sq mi

Curve Number (AMC III condition) ________________

Time of Concentration ___________________________ hr

Peak Discharge _________________________________ cfs

Peak Stage ____________________________________ ft-MSL

Storm Duration Causing Peak Stage ________________ hr

Texas Dam Safety Program, MC 174
Field Operations Support Division, Office of Compliance and Enforcement
Texas Commission on Environmental Quality
P.O. Box 13087
Austin, TX 78711

HYDROLOGIC AND HYDRAULIC (H&H) EVALUATION SUMMARY

(Please complete **all** sections, unless otherwise specified)

Name of Dam: ______________________________

TCEQ Dam Safety Project No.: ______________________________

County: ______________________________

Year to Build: ______________________________

Maximum Record Precipitation (in): ______________________________

Record Area (county or city): ______________________________

Duration (hr): ______________________________

Date of Record (MM/DD/YY): ______________________________

Source Ref. (FEMA, National Weather Service, etc.): ______________________________

Downstream Dam Toe ____________ (ft-MSL)

Normal Pool ____________ (ft-MSL)

Principal Spillway ____________ (ft-MSL)

Emergency Spillway ____________ (ft-MSL)

Top of Dam ____________ (ft-MSL)

Normal Reservoir Capacity ____________ (ac-ft)

Maximum Reservoir Capacity ____________ (ac-ft)

Reservoir Surface Area ____________ (ac)

Drainage Area ____________ (ac)

Outlet Diameter or Cross-Section ____________ (in)

Storm Duration	Peak Inflow (cfs)	Peak Outflow (cfs)	Peak Stage (ft-MSL)	% PMF Passing	Comments (if needed)
1 hr					
2 hr					
3 hr					
6 hr					
12 hr					
24 hr					
48 hr					
72 hr					

To the best of my knowledge, I certify the above data are correct. I will supply the hydrologic and hydraulic reports to the Texas Commission on Environmental Quality upon request.

____
(Signature)

(P. E. Seal)

____
(Date)

Texas Dam Safety Program, MC 174
Field Operations Support Division, Office of Compliance and Enforcement
Texas Commission on Environmental Quality
P.O. Box 13087
Austin, TX 78711

ENGINEER'S NOTIFICATION OF COMPLETION

(PLEASE PRINT OR TYPE)

PLEASE CHECK ONE: ❑ New ❑ Modification ❑ Repair ❑ Removal ❑ Alteration

TX Number ______________________ County ______________________

Adjudication Number ______________________ Permit Number ______________________

Name of Dam/Project ____

Owner:

Name ____

Address ____

City ______________________ State ____________ Zip Code ____________

Phone (   ) ______________________ Emergency Contact Phone (   ) ____________

Fax (   ) ______________________ E-mail ______________________

Engineering Firm:

Firm Name ____

Project Engineer ______________________ TX P.E. License No ____________

Firm Address ____

City ______________________ State ____________ Zip Code ____________

Phone (   ) ______________________ Fax (   ) ______________________

E-mail ____

The project was completed on ____________, 20_____. To the best of my knowledge, the project was constructed in substantial conformance with plans, specifications, and change orders filed with and approved by the Texas Commission on Environmental Quality.

____
(Signature)

(P. E. Seal)

____
(Date)

If you have questions on how to fill out this form or about the Dam Safety Program, please contact us at 512-239-5195.
Individuals are entitled to request and review their personal information that the agency gathers on its forms. They may also have any errors in their information corrected. To review such information, contact us at 512-239-3282.

Glossary

breach—An excavation through a dam or spillway that is capable of draining the entire reservoir so the structure—no longer considered a dam—will no longer impound water.

breach analysis—The determination of the most likely uncontrolled release of water from a dam (magnitude, duration, and location), using accepted engineering practice, to evaluate the inundation downstream.

breach area—An area that would be flooded as a result of a dam failure.

dam—Any barrier or barriers, with any appurtenant structures, constructed for the purpose of impounding water.

design flood—The flood used in the design and evaluation of a dam and appurtenant structures, particularly for determining the size of spillways, outlet works, and the effective crest of the dam.

effective crest—The elevation of the lowest point on the crest (top) of the dam, excluding spillways.

emergency action plan (EAP)—A written document prepared by the owner or the owner's professional engineer describing a detailed plan to prevent or lessen the effects of a potential failure of the dam or appurtenant structures.

emergency spillway—A secondary spillway designed to pass a large, but infrequent, volume of flood flows.

fetch—The straight-line distance across a reservoir subject to wind forces.

fuse-plug spillway—An auxiliary spillway that is intentionally blocked by an erodible berm. A higher discharge elevation is maintained during normal floods, while during extreme flooding the discharge elevation is lowered by erosion.

hazard classification—A categorization of the potential for loss of life or property damage in the area downstream of the dam in the event of a failure or malfunction of the dam or appurtenant structures. Does not represent the condition of the dam.

height of dam—The difference in elevation between the natural bed of the watercourse or the lowest point on the toe of the dam, whichever is lower, and the effective crest of the dam.

isohyet—An elliptical area representing the size, shape, and rainfall intensity of a PMP event.

inundation map—Map delineating the area that would be newly covered by water in a particular flood event.

maximum normal operating level—The highest water-surface elevation within the range of planned operating levels for the reservoir, above which floodwaters would be released.

maximum storage capacity—The volume, in acre-feet, of the impoundment created by the dam at its effective crest. Only water that can be stored above natural ground level or that could be released by a failure of the dam is considered in assessing the storage volume.

minimum freeboard—The difference in elevation between the effective crest of the dam and the maximum water surface elevation resulting from routing the design flood appropriate for the dam.

normal storage capacity—The volume, in acre-feet, of the impoundment created by the dam at the lowest uncontrolled spillway crest elevation, or at the maximum elevation of the reservoir under normal operating conditions.

population at risk—The number of people present in an area that would be flooded by a particular flood event.

principal spillway—The primary or initial spillway, designed to pass normal flows, that is engaged during a rainfall-runoff event.

probable maximum flood (PMF)—The flood magnitude that may be expected from the most critical combination of meteorological and hydrologic conditions that are reasonably possible for a given watershed.

probable maximum precipitation (PMP)—The theoretically greatest depth of precipitation for a given duration that is physically possible over a given storm area at a particular geographic location at a certain time of the year.

professional engineer—An individual licensed by the Texas Board of Professional Engineers to practice engineering in Texas, with expertise in the investigation, design, construction, repair, and maintenance of dams.

proposed dam—Any dam not yet under construction.

spillway—An appurtenant structure that conducts overflow from a reservoir.

top width elevation—The elevation of the water surface of a flood, associated with the top width of the flood cross-section.

References

American Society of Civil Engineers. 1996. *Hydrology Handbook*. 2nd ed. New York: McGraw-Hill.

Hansen, E. M., et al. 1988. *Hydrometeorological Report No. 55A: Probable Maximum Precipitation Estimates—United States Between the Continental Divide and the 103rd Meridian*. Washington: National Weather Service, Office of Hydrology.

Hansen, E. M., L. C. Schreiner, and J. F. Miller. 1982. *Hydrometeorological Report No. 52: Application of Probable Maximum Precipitation Estimates—United States East of the 105th Meridian.* Washington: National Weather Service, Office of Hydrology.

Federal Energy Regulatory Commission. 2001. *Engineering Guidelines for the Evaluation of Hydropower Projects.* Chapter 8, Determination of the Probable Maximum Flood. Washington. Available online at: <www.ferc.gov/industries/hydropower/safety/guidelines/eng-guide/chap8.pdf>.

Natural Resources Conservation Service. 1997. *National Engineering Handbook.* Part 630: Hydrology. Washington: U.S. Department of Agriculture.

———. 2005. *TR-60—Earth Dams and Reservoirs*. Washington: U.S. Department of Agriculture.

———. 1986. *TR-55—Urban Hydrology for Small Watersheds*. Washington: U.S. Department of Agriculture.

Reynolds, Dale. 1983. *Regional Relationships for Unit Hydrograph Coefficients—Natural and Urbanized Streams*. Tulsa, OK: U.S. Army Corps of Engineers, Tulsa District.

Rutledge, John. 2006, *Simplified Dam Breach Method—Technical Memorandum*. Austin: Freese and Nichols.

——— and Michelle Evans. 2006. *PMF Temporal Distribution—Technical Memorandum*. Austin: Freese and Nichols.

Schreiner, Louis C., and Riedel, John T. 1978. *Hydrometeorological Report No. 51, Probable Maximum Precipitation Estimates, Unites States East of the 105th Meridian.* Washington: National Weather Service, Office of Hydrology.

Skaggs, R. W., and R. Khaleel. 1982, *Infiltration in Hydrologic Modeling of Small Watersheds, American Society of Agricultural Engineers.* Monograph No. 5. American Society of Agricultural and Biological Engineers. St. Joseph, MI.

U.S. Army Corps of Engineers. 1982. *Hydrograph Analysis and Computation*. EM 1110-2-1405. Washington.

———. 1994. *Engineering and Design—Flood Runoff Analysis*. EM 1110-2-1417. Washington.

U.S. Department of the Interior, Bureau of Reclamation. 1987. *Design of Small Dams.* 3rd ed. Denver.

Texas Commission on Environmental Quality

Subchapter B : Design and Evaluation of Dams

§299.11. Classification of Dams.

All dams will be classified or reclassified as necessary to assure appropriate safety considerations. The three size classifications (small, intermediate and large), based on height of dam or impoundment capacity, and the three hazard classifications (low, significant and high), are combined to indicate a dam's downstream hazard potential. Thus, the classification assignment reflects the hazard potential associated with assumed failure of the dam. For example, dams located such that resulting failure could be catastrophic are classified so as to require a higher degree of design consideration than would be required for similar dams located in remote areas. Classification does not indicate the physical condition of a dam.

§299.12. Size Classification Criteria.

The classification for size based on the height of the dam or maximum reservoir storage capacity, shall be in accordance with Table 1 of this subsection. The appropriate size is the largest category determined for either storage or height.

TABLE 1
SIZE CLASSIFICATION

	Impoundment	
Category	Storage (Ac-Ft)	Height (Ft.)
Small	Less than 1000	Less than 40
Intermediate	Equal to or Greater than 1000 & less than 50,000	Equal to or Greater than 40 & less than 100
Large	Equal to or Greater than 50,000 than 100	Equal to or Greater

§299.13. Hazard Classification Criteria.

The hazard potential classification shall be in accordance with Table 2 of this subsection. Hazard classification pertains to potential loss of human life and/or property damage within either existing or potential developments in the area downstream of the dam in event of failure or malfunction of the dam or appurtenant facilities. Hazard classification does not indicate any condition of the dam itself. Dams in the low hazard potential category are normally those in rural areas where failure may damage farm buildings, limited agricultural improvements and county roads. Significant hazard potential category dams are usually those in predominantly rural areas where failure would not be expected to cause loss of human life, but may cause damage to isolated homes, secondary highways, minor railroads, or cause interruption of service or use (including the design purpose of the facility) of relatively important public utilities. Dams in the high hazard potential category are usually those in or near urban areas where failure would be expected to cause loss of human life, extensive damage to agricultural, industrial or commercial facilities, important public utilities (including the design purpose of the facility), main highways or railroads.

TABLE 2
HAZARD POTENTIAL CLASSIFICATION

Category	Loss of Human Life	Economic Loss
Low	None expected (No permanent structures for human agricultural improvements)	Minimal (Undeveloped to occasional structures or habitation)
Significant	Possible, but not expected (A small number of inhabitable structures)	Appreciable (Notable agricultural, industrial or commercial development)
High	Expected (Urban development or large number of inhabitable structures)	Excessive (Extensive public, industrial, commercial or agricultural development)

§299.14. Hydrologic Criteria for Dams.

(a) The hydrologic criteria contained in Table 3 are the minimum acceptable spillway design flood (SDF) for proposed dams as defined in §299.1 of this title (relating to Definitions), including those to be constructed in accordance with Texas Water Code, §11.142.

(b) Exemptions to Minimum Hydrologic Criteria - Proposed low hazard dams exempt under Texas Water Code, §11.142 are exempt from the minimum criteria. Any other proposed structure may be exempt from the minimum criteria if properly prepared dam breach analyses show that existing downstream improvements or known or planned future improvements will not be adversely affected. A properly prepared breach analysis should include at least three events, the normal storage capacity non-flood event, the barely overtopping event and the PMF event. Data on additional flood magnitudes may be provided as necessary to document other conditions or conclusions. Downstream flooding differentials of one-foot or less between breach and non-breach simulations are not considered to be adverse.

TABLE 3
HYDROLOGIC CRITERIA FOR DAMS

Classification		
Hazard	Size	Minimum Flood Hydrograph
Low (No. 3)	Small	¼ PMF
	Intermediate	¼ PMF to ½PMF
	Large	PMF
Significant (No. 2)	Small	¼ PMF to ½ PMF
	Intermediate	½ PMF to PMF
	Large	PMF
High (No. 1)	Small	PMF
	Intermediate	PMF
	Large	PMF

NOTE: The flood hydrograph in this table is the minimum required flood for a given project, i.e., the project will be required to safely pass this hydrograph. Where a range is given, the minimum flood hydrograph will be determined by straight line interpolation within the given range. Interpolation shall be based on either hydraulic height or impoundment size (§299.12, Table 1 of this title (relating to Size Classification Criteria)), whichever is greater. The minimum flood hydrograph is computed as a percentage of the PMF hydrograph.

§299.15. Evaluation of Existing Dams.

(a) Existing dams, as defined in §299.1 of this title (relating to Definitions), are subject from time to time to reevaluation in consideration of continuing downstream development. Hydrologic criteria contained in §299.14, Table 3 of this title (relating to Hydrologic Criteria for Dams) are the minimum acceptable spillway evaluation flood (SEF) for reevaluating dam and spillway capacity for existing dams to determine whether upgrading is required. Dams not meeting minimum criteria are considered to be below acceptable limits and are subject to action as necessary under §299.2 of this title (relating to General).

(b) Exemptions from Minimum Hydrologic Criteria - Existing low hazard dams are exempt from the minimum hydrologic criteria as given in Table 3 and any other existing structure may be exempt from the minimum hydrologic criteria if properly prepared dam breach analyses show that existing downstream improvements or known or planned future improvements will not be adversely affected. A properly prepared breach analysis should include at least three events, the normal storage capacity non-flood event, the barely overtopping event and the PMF event. Data on additional flood magnitudes may be provided as necessary to document other conditions or conclusions. Downstream flooding differentials of one-foot or less between breach and non-breach simulations are not considered to be adverse.

(c) Structural Evaluation - Evaluating the structural condition of an existing dam includes, but is not limited to, visual inspections and evaluations of potential problems such as seepage, cracks, slides, conduit and control malfunctions and other structural and maintenance deficiencies which could lead to failure of a structure. An active and progressive deteriorating condition is sufficient for a finding that an existing dam is structurally inadequate.

§299.16. Interim Alternatives.

At the time the commission considers the permanent upgrading or removal of an inadequate dam, the dam owner may request the commission to consider interim alternatives including but not limited to temporary repairs, reservoir dewatering, insurance coverage, and/or downstream warning and evacuation plans. Consideration shall be given to the time required to overcome economic, physical and legal restraints to upgrading, the prospect of permanent repair, current use of the facility, degree of risk and public welfare.

§299.17. Emergency Management.

As required for emergency management planning, the executive director may request, and/or the commission may order a dam owner to provide sufficient data to plan for potential effects of failure or malfunction of a dam and/or associated appurtenant facilities.

§299.18. Variance.

The owner of an existing dam that does not meet the hydrologic criteria of §299.14, Table 3 of this title (relating to Hydrologic Criteria for Dams) may request the commission to consider a variance from this criteria, based upon but not limited to the owner's evaluation of the consequences of potential dam failure, proposals to reduce potential hazard, and/or the economic and physical limitations to upgrading.

Appendix C

Project Photo Log

Project Name: Coal Combustion Waste Impoundments Study	Owner's Project No. (If applicable):
Project Owner: San Miguel Electric Plan	**Regulatory Agency Project No. (If applicable)**
HDR Project No.: 240666	**Photo Log Date:** October 9, 2014

Picture	Photo No.	Description
	1	Looking South from south Ash Water Transport Impoundment towards the Caballos Creek floodplain
	2	Looking at Ash Water Impoundments pumps and pressure pipes that bring water from pond to plant

Project Photo Log

Project Name: Coal Combustion Waste Impoundments Study	Owner's Project No. (If applicable):
Project Owner: San Miguel Electric Plan	**Regulatory Agency Project No. (If applicable)**
HDR Project No.: 240666	**Photo Log Date:** October 9, 2014

Picture	Photo No.	Description
	3	Looking north west at north Ash Water transport impoundment and electric plant
	4	Looking at spillway between north and south ash water impoundments. Normal pool at ponds show 18" of freeboard

Project Name: Coal Combustion Waste Impoundments Study	Owner's Project No. (If applicable):
Project Owner: San Miguel Electric Plan	**Regulatory Agency Project No. (If applicable)**
HDR Project No.: 240666	**Photo Log Date:** October 9, 2014

Picture	Photo No.	Description
	5	Looking southeast at small area draining towards equalization impoundment.
	6	Looking southeast at equalization impoundment, access road and embankment

Appendix D