Hydrologic and Hydraulic Guidelines for Dams in Texas

Dam Safety Program
Texas Commission on Environmental Quality

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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.

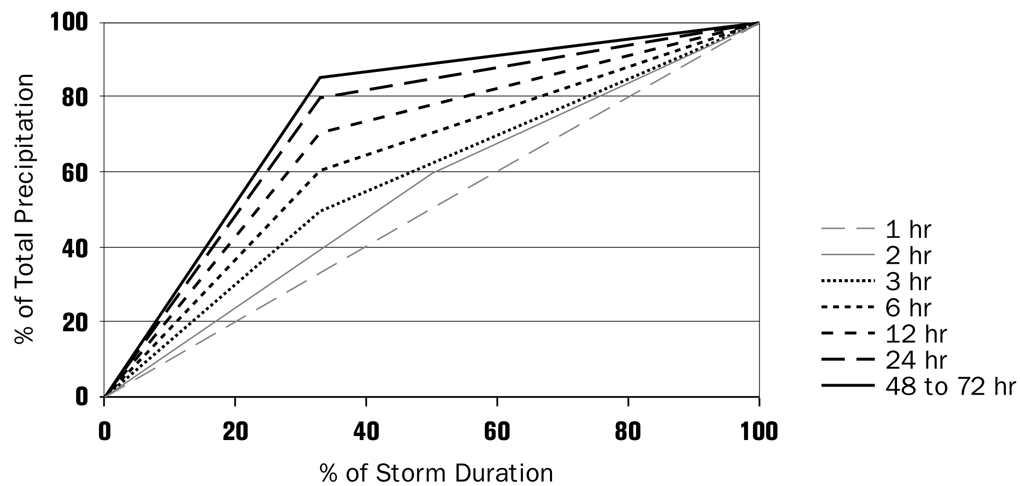
4.3 Temporal Distribution of Design-Storm Precipitation

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 ½ 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 ______________

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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)

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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.

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Texas Commission on Environmental Quality