# CHAPTER 10

## HYDRAULIC ANALYSIS

# SECTION 1

## DETAILED METHOD

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### 1.1 INTRODUCTION

Detailed flood hazard area information including floodplain and floodway limits, flood water surface elevations, flow velocities, etc. can be determined based on the detailed hydraulic analysis methods and guidelines outlined in this section. The detailed hydraulic analysis approach should be used for the following general cases:

- To determine new detailed floodplain and/or floodway boundaries for streams that are located adjacent to existing and/or planned developments.
- To revise existing detailed floodplain/floodway delineations to reflect changes in topography or hydrology caused by natural or manmade activities.
- To determine potential impacts or benefits of proposed improvements within the delineated floodplains.
- To delineate detailed floodplain/floodway boundaries for streams that have been previously studied and delineated using approximate methods.
- To check the flow conveyance capacity of designed or newly constructed drainage facilities.

Detailed analyses generally consider flooding from the 10-, 50-, 100-, and 500-year and sometimes defined floodways. The information presented in this chapter is the current information available at the time of preparation of this Manual and should be updated as better analysis and modeling techniques become available in the future.

### 1.2 PREVIOUS STUDIES

Before proceeding with a detailed hydraulic study, the project engineer should evaluate the applicability of all available hydrologic and hydraulic studies for the subject stream. The previously approved studies should be used whenever practical, unless the watershed/stream conditions have changed substantially and/or the original analysis methodology was determined inappropriate or inadequate.

Where applicable, a comparison of the calculated 100-year water surface elevations (WSEL) at the study limits with the previously approved WSELs for the stream should be provided. Except where clearly identified changes in flooding characteristics or error in the existing water surface profile can be shown, the proposed 100-year flood elevations at the study limits should agree with those of other contiguous studies on the same stream. The 100-year water surface elevations should match within +/- 0.5 foot of the existing valid elevations. Where elevations cannot be reconciled to within +/- 0.5 foot because of changed flooding conditions or an error in the previous analysis, a full explanation and justification for the difference should be provided.

## 1.3 HYDROLOGIC ANALYSIS

Hydrologic analysis should be performed based on the criteria outlined in Chapter 9 of this manual. Peak flow rates should be computed using statistical analysis, rainfall-runoff models, or regional regression methods.

## 1.4 TOPOGRAPHIC MAPPING

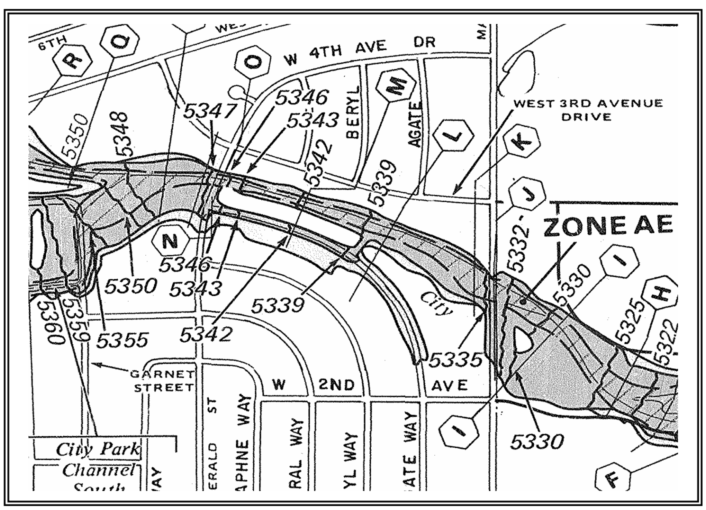
For discussions and specifications on the topographic mapping standards for detailed floodplain delineation studies, please refer to Chapter 8 of this Manual.

### 1.4.1 CROSS SECTIONS

The riverine cross-section data for detailed hydraulic modeling purposes should be obtained based on the following methods:

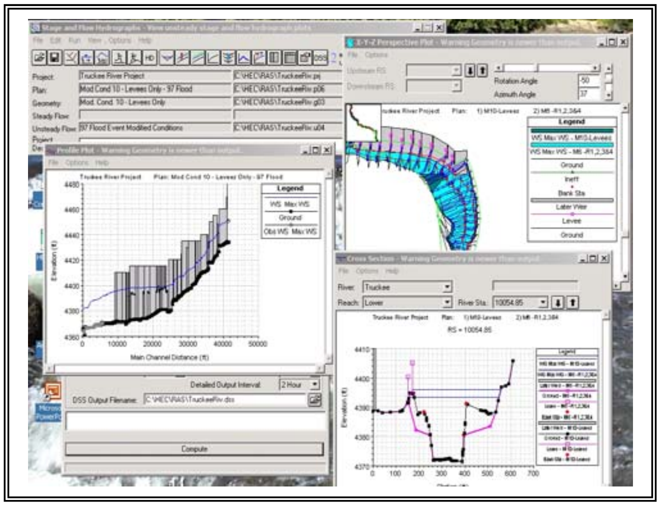
- Photogrammetric methods at the time of map compilation
- From DTM, DEM, or TIN models
- From the map contours and spot elevations
- Through field surveys

All field-surveyed cross section points should be within ±0.5 foot of the true elevations.




In general, cross sections should be aligned perpendicular to the direction of flow and spaced to adequately represent the stream. Additional cross sections should be placed at appreciable changes in flow area, roughness, or stream gradient, bridges and culverts, the head and tail of levees, confluences with tributaries, and all flow control structures.

## 1.5 HYDRAULIC ANALYSIS APPROACHES



The open channel/floodplain hydraulics can be very complex, encompassing many different flow conditions from steady-state uniform flows to unsteady, rapidly varying flows. The calculations for uniform and gradually varying flows are relatively straightforward, however, rapidly varying flow computations can be very complex and the solutions are generally empirical in nature.

Flow hydraulics is three-dimensional in actuality. However, flow hydraulics for most streams can be adequately modeled by using one of the following three modeling approaches:

- One-dimensional Steady Flow Analysis
- One-dimensional Unsteady Flow Analysis
- Two dimensional Steady/unsteady Flow Analysis

There are limitations on all of the three modeling approaches. Therefore, the hydraulic properties of the study stream should be carefully evaluated and compared to the modeling limitations before selecting the appropriate modeling approach. The modeling engineer should coordinate with the CWCB, local jurisdictions, and other study sponsors to select the most appropriate modeling approach and specific model for the stream being studied.

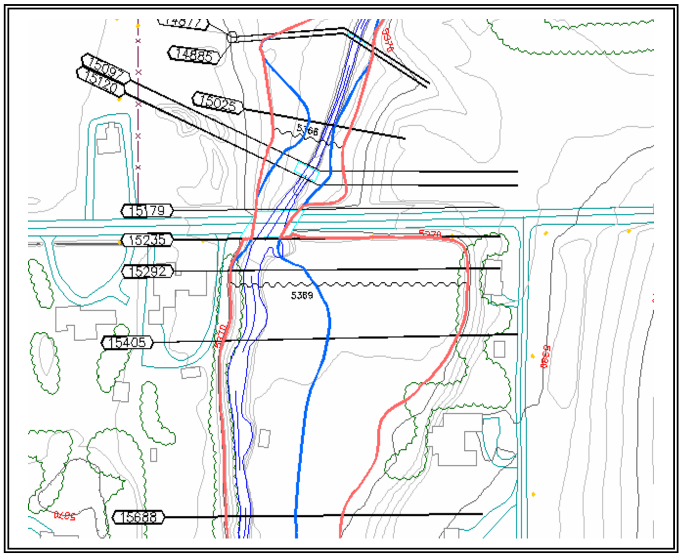
### 1.5.1 ONE-DIMENSIONAL STEADY FLOW

The one-dimensional steady flow analysis is the most commonly used modeling approach due to its simplicity. This approach is widely accepted for modeling of streams with steady and gradually varying flow conditions. The most common occurrence of gradually varying flow is the backwater created by culverts and channel constrictions. For these conditions, the flow depth will be greater than normal depth in the channel, and the water surface profile should be computed using backwater techniques.

Following limitations generally apply to one-dimensional steady flow modeling techniques and programs:

- Flow condition is steady and gradually varied.
- Only the velocity in the direction of flow can be accounted for
- Flow rate is constant for a given channel reach through out the duration of a flood event (only peak flow rates can be used, not hydrographs)
- Channel slope is relatively flat, less than 1 percent.
- Cannot model effects of flow attenuation due to storage

Flood water surface profiles may be calculated using the standard step backwater method employing the Bernoulli energy equation with energy losses due to friction evaluated with the Manning equation. Many computer programs are available for computation of backwater curves. The most general and widely used programs are US Army Corps of Engineers' HEC-2 and HEC-RAS. Both HEC-2 and HEC-RAS programs can be used to model one-dimensional subcritical and supercritical flow conditions. In addition, HEC-RAS can be used to model mixed flow conditions.




Natural riverine flood water surface profiles for the purpose of floodplain delineations should be determined using subcritical flow regime calculations. Critical depths should be used for the natural stream reach where supercritical flow depths occur. Supercritical flow modeling may be used for man-made channels designed to handle supercritical flows.

### 1.5.2 ONE DIMENSIONAL UNSTEADY FLOW

The main difference between the one-dimensional steady and unsteady flow models is that unsteady flow models can compute the effects of flow attenuation due to channel and floodplain storages. Instead of using single point peak flow rates, users can input entire flow hydrographs and route the hydrographs through the channel/floodplain system to compute water surface profiles and routed resulting hydrographs. The US Army Corps of Engineers' HEC-RAS computer program is recommended for 1-D unsteady flow modeling of riverine hydraulics.

While this modeling approach is superior to 1-D steady flow modeling techniques, the following limitations still apply:

- Flow condition is steady and gradually varied.
- Only the velocity in the direction of flow can be accounted for
- Channel slope is relatively flat, less than 1 percent.

Developing 1-D unsteady flow models can be complex and costly. Therefore, this approach has not been used as frequently as the 1-D steady modeling approach. However, drainage systems with significant storage components should be modeled using the 1-D unsteady modeling techniques.

### 1.5.3 TWO-DIMENSIONAL FLOW

While most of the riverine hydraulic conditions can be adequately modeled using either 1-D steady or 1-D unsteady flow modeling techniques, some flooding conditions (i.e., alluvial fans, shallow flooding) may require the use of two-dimensional modeling techniques in order to correctly model and delineate the flood hazard areas. Two-dimensional hydraulic computer programs can be used to model flood flows in two horizontal directions.

For these cases, project engineer should coordinate with the local agencies and CWCB in selecting the appropriate modeling program for the drainage system being studied. The most commonly used two-dimensional hydraulic computer programs are MIKE FLOOD and FLO-2D. For detailed discussions on the modeling of alluvial fans, readers are referred to Chapter 12 of this manual.

## 1.6 STARTING WATER SURFACE ELEVATION

One of the model boundary conditions that need to be defined by the modeling engineer is the starting water surface elevation (WSEL). For a subcritical model run, starting WSEL for the most downstream cross section should be defined, and for a supercritical run, starting WSEL for the most upstream cross section should be defined.

For a riverine reach not affected by backwater, the starting water surface elevation may be estimated based on normal depth calculations, unless a known water surface elevation for the starting cross section can be obtained from an existing model or previous recorded flood events. If normal depth calculation is used to compute the starting water surface elevation, several cross-sections (minimum of 2) should be placed outside of the study limit (upstream or downstream depending on the model flow regime) to improve the accuracy of the computed water surface elevation at the study limit.

## 1.7 ROUGHNESS VALUES

Recommended Manning's "n" values for various channel and floodplain conditions can be found in Table CH13-T102 or other published documents from the USGS and other common documents used in the industry. Manning's roughness coefficients should be estimated considering the following factors:

- Channel bed materials
- Type, density, and height of existing vegetations
- Existing structures in the overbanks
- Roughness variations with different flow depths
- Channel maintenance operations
- Past flood data

Past flood data, if available, should be used to calibrate roughness coefficients, taking into consideration any alteration in the channel subsequent to the floods. The calibrated roughness coefficients should closely match the observed channel and floodplain conditions. Photographs should be taken of the study reaches of the stream channel and floodplain to support roughness coefficients used for hydraulic computations.

## 1.8 SPLIT FLOW ANALYSIS

Spilt flows occur when streams overflow the channel banks and take different flow paths away from the main floodplains. Undersized channels and crossing structures (i.e., culverts, bridges) are the most common reasons for the flow splits.

Flows that split away from the main floodplain may return back to the stream at a downstream location or may divert away to an adjacent stream. The amount of flow splits should be calculated and the downstream flow rates/hydrographs for the drainageway being studied should be adjusted accordingly. Flow splits can be estimated using the built-in split flow computational options in HEC-2 or HEC-RAS programs. It is important that the modeling engineer review the computed results to determine the accuracy of the results. Also, the flood hazard areas resulting from the split flows should be studied and delineated depending on the following factors:

- Purpose of the study
- Amount of flow splits
- Existing and proposed land uses within and adjacent to the flow path (i.e. residential vs. agricultural)

## 1.9 CROSSING STRUCTURE

Numerous roadways have been constructed across streams, and commonly, culverts or bridges are provided at these locations to convey flows beneath roadways. These drainage-crossing structures usually cause increase in water surface elevations by creating additional energy losses. The additional hydraulic energy losses, including contraction and expansion losses, at the crossing structures should be accounted for in the hydraulic analysis to compute water surface elevations.

Culvert and bridge hydraulics can be modeled using HEC-2 or HEC-RAS programs. Readers are referred to the previously referenced program users manuals for detailed discussions on the subject. The following hydraulic elements should be considered in the analysis:

- Size, type, and material of the structure
- Invert elevations (including length and slope) of the structure
- Location of the representative cross sections on both sides of the structure
- Roadway profile for the weir flow computation
- Ineffective flow areas
- Debris and sediment blockage

### 1.9.1 BLOCKAGE

All culverts and bridges should be considered for the potential to become blocked by floating debris and sediment loads. In determining the potential for blockage, and subsequent reduction in the flow conveyance capacity, the following factors should be considered:

- Old photographs
- History of maintenance during high flows
- Ongoing maintenance operations
- Watershed characteristics such as erodibility of channel banks
- Amount and type of vegetation along the stream
- Size and characteristics of the waterway

Blockage may be accounted for in computer runs by increasing width of piers, raising streambed elevation or reducing waterway opening by a percentage. Debris removal activities during flood events (i.e. snagging) should not be considered.

## 1.10 INEFFECTIVE FLOW AREA

Ineffective flow areas may store water during flood events but the velocity in the direction of flow is zero or negligible. Therefore, these areas should be blocked out for the flow conveyance hydraulic analysis. However, the flow storage benefit of the ineffective areas should be modeled when using either one-dimensional unsteady or two-dimensional flow analysis approach. Ineffective flow areas commonly exist at both ends of culverts and bridges. Ponds, local depression areas, and backwater pools may, not always, also act as ineffective flow areas.

## 1.11 MODEL CALIBRATION

Hydraulic models should be calibrated to match the reliable flood data from previous flood events, if available, within 0.5-foot +/- accuracy. When calibrating the models, only the hydraulic parameters that were estimated should be adjusted (i.e., Manning's "n" values, contraction and expansion coefficients, etc.). However, the adjusted values should still closely represent the observed stream conditions.

## 1.12 AREAS PROTECTED BY LEVEES

In order for a levee system to be recognized as providing flood protections, the levee should be structurally sound and adequately maintained. Certification from a federal or state agency that the levee meets the minimum freeboard criteria and that it appears, on visual inspection, to be structurally sound and adequately maintained will be required. Levees that have obvious structural defects, or that are obviously lacking in proper maintenance, should not be modeled as providing flood protections in the hydraulic analysis.

*In order for a levee system to be recognized as providing flood protections, the levee should be structurally sound and adequately maintained. Levees that have obvious structural defects, or that are obviously lacking in proper maintenance, should not be considered in the hydraulic analysis.*

Detailed discussions on the levee freeboard, ownership, design, operations and maintenance, and certification requirements are provided in Chapter 13, Section 4.

### 1.12.1 FLOODPLAIN ANALYSIS

The natural floodplain areas protected from a 100-year event by a levee system can be designated as 100-year Shallow Floodplain with 1-foot average depth (FEMA Zone X). However, the areas inundated by the interior drainage behind the levees should be defined, and the 100-year water surface elevations, flooding limits and depths, flood hazard zone designations should be clearly identified.

If levees protecting the subject area do not meet the necessary requirements, the 100-year flood elevations of the protected area should be computed as if the levees did not exist. For the unprotected areas between the levee and the source of flooding, the 100-year flood elevations should be obtained from either the flood profile computed with the levees in place or the profile computed as if the levees did not exist, whichever is higher. This procedure recognizes the increase in flood elevation in the unprotected area caused by the levees. This procedure may result in the 100-year flood elevations being shown as several feet higher on one side of the levee than on the other. Both profiles should be shown in the final delineation with a line drawn along the levee centerline separating the areas with different BFEs.

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If levees exist on both sides of a drainage-way, several levee failure scenarios should be considered including simultaneous levee failure, left levee only failure, and right levee only failure scenarios.

Where flood protection credit will be given to levees providing 100-year protection, the adequacy of interior drainage systems should be evaluated. Areas subject to flooding from inadequate interior drainage behind levees will be mapped using standard floodplain mapping procedures.

## 1.13 AREAS PROTECTED BY DAMS

### 1.13.1 FLOOD CONTROL DAMS

If a publicly owned, operated and maintained dam or a publicly controlled privately owned dam is specifically designed and operated, either in whole or in part, for flood control purposes, then its effects should be taken into consideration when delineating the floodplains below such a dam. Full credit should be given to the diminution of peak flood discharges, which would result from normal dam operating procedures.

Flood control dams that are not owned and maintained by public agencies should not be considered in the floodplain analysis.

### 1.13.2 NON-FLOOD CONTROL DAMS

If a dam is not specifically designed and operated, either in whole or in part, for flood control purposes, then its effects, even if it provides inadvertent flood routing capabilities which reduce the flooding downstream, should not be taken into account. The delineation of the floodplains below such a dam should be based upon the floods that would occur absent of the dam. However, if adequate assurances have been obtained to preserve the flood routing capabilities of such a dam, then the flood attenuation effects may, but need not, be taken into consideration when delineating the floodplains below such a dam. The project engineer should coordinate with appropriate government agencies and CWCB in determining whether a non-flood control dam should be included in the analysis or not.

If existing dams are not included in the hydrologic analysis, discussions should be provided in the report describing the dams and reasons why they were not considered in the analysis.

## 1.14 ALLUVIAL FAN FLOODING

Alluvial fan flooding is quite different than a riverine flooding, and consequently, the alluvial fan floodplains should be studied and delineated based on a different set of criteria. Alluvial fan flooding can be characterized by unpredictable flow paths, mud-flows, high flow velocity, and erosion and sediment deposition. Alluvial fans typically do not have a well-defined channel capable of conveying a 100-year flows, although, it is not unusual to have smaller defined channel(s). Typically, flood flows do not spread over the entire alluvial fan surface, but are conveyed down from the apex to the toe of the fan by a network of old and new flow paths/channels.

More detailed discussions on the floodplain analysis of active or semi-active alluvial fans are provided in Section 1, Chapter 12.

## 1.15 FLOODWAY ANALYSIS

The floodway represents the community's regulatory limit of encroachment into the 100-year floodplain for those watercourses with the established floodway boundaries. Communities may choose to delineate floodways based on FEMA's 1-foot rise criteria or based on stricter criteria by allowing a lesser amount of rise above the base flood elevations (BFE).

### 1.15.1 EQUAL CONVEYANCE REDUCTION METHOD

Floodway limits should be determined based on the "equal conveyance reduction" method. This method reduces an equal amount of flow conveyance from both overbanks, allowing potential development areas on both sides of the waterway. This floodway analysis method is available in both HEC-2 and HEC-RAS programs.

### 1.15.2 SPECIAL CONDITIONS

Floodways can be delineated for most streams with channel and overbank flooding conditions. However, It is not practical to designate floodways for all flooding conditions. Floodways should not be delineated for the following general cases:

- Split flow areas
- Sheet flow area (divided flow areas)
- Alluvial fans

If the above condition(s) exists for only a small portion of the stream, the floodplain may be set equal to floodway for that portion, and floodway limits for the remaining parts of the stream may be defined using the equal conveyance reduction method.

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## LIMITED DETAILED METHOD

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## LIMITED DETAILED METHOD

### 2.1 INTRODUCTION

Limited Detailed flood hazard area information including floodplain limits, flood water surface elevations, flow velocities, etc. can be determined based on the detailed hydraulic analysis methods and guidelines described in Section 1 of Chapter 10 of this Manual. The Limited Detailed hydraulic analysis approach should be used for the following general cases:

- To determine new Limited Detailed floodplain boundaries for streams that are located adjacent to existing and/or planned developments.
- To revise existing Limited Detailed floodplain delineations to reflect changes in topography or hydrology caused by natural or manmade activities.
- To determine potential impacts or benefits or proposed improvements within the delineated floodplains.
- To delineate Limited Detailed floodplain boundaries for streams that have been previously studied and delineated using approximate methods.
- To check the flow conveyance capacity of designed or newly constructed drainage facilities.

Limited Detailed flood hazard area information is identical in nature and level of detail as the Detailed Method described in Section 1 of Chapter 10 of this Manual except that Limited Detailed studies need only analyze the 100-year flood event. In addition, floodway analyses are optional for Limited Detailed studies.

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### 3.1 INTRODUCTION

The Approximate Method results in the delineation of approximate 100-year floodplain boundaries without base flood elevations (BFEs). The primary advantage of using Approximate Method is that approximate 100-year floodplain limits can be determined with minimal analysis efforts and costs. This analysis approach may be used to delineate approximate 100-year floodplain boundaries for the following general cases:

- To update the currently designated 100-year approximate floodplain boundaries based on changes in the watershed or more current data and methodologies.
- To delineate new approximate floodplain boundaries for streams that do not already have them. New approximate floodplains are typically done on areas that do not have existing development, or in areas that may potentially be developed in the long-term future.

If new developments are proposed or expected within or adjacent to a previously delineated approximate 100-year floodplain, the previously delineated approximate floodplain limits should be restudied by using the detailed method as outlined in Section 1, Chapter 10.

### 3.2 HYDROLOGIC ANALYSIS

Hydrologic analysis should be prepared based on the criteria outlined in Chapter 9 of this Manual. Hydrologic calculations for the approximate method should provide for 100-year peak flow rates as a minimum.

The peak flow rates for approximate 100-year floodplain boundaries can be computed using statistical analysis, rainfall-runoff models, or regional regression equations.

### 3.3 TOPOGRAPHIC INFORMATION

The best available topographic base map should be used to develop approximate floodplain information. Such base map should, at a minimum, meet the requirements outlined in Chapter 8 of this Manual.

Field surveyed channel and floodplain cross-sections can be obtained to supplement the available topographic data as needed.

## 3.4 HYDRAULIC ANALYSIS

The following hydraulic analysis methods or a more detailed method should be used for delineation of approximate 100-year floodplain boundaries:

- Software packages that work together with GIS and digital base mapping to form automated routines for delineating approximate 100-year floodplain boundaries.
- Simplified HEC-RAS or similar computer models that reasonably represent hydraulic conditions of the stream of interest using cross-section spacing and other factors that are less rigorous than for detailed analyses.
- Calculate 100-year water depths for the cross sections that are representative of the stream reach being studied using normal-depth calculations (Manning's Equation). It should be noted that the normal depth approximations do not incorporate backwater effects that may occur due to roadways, dams, etc.
- Published culvert rating charts (i.e., FHWA, 1985) may be used to compute headwater depths at culvert inlets. Weir equations may be used to compute approximate flow depths over roadways.
- Simple computer hydraulics programs may be used to compute normal depths for the selected representative cross sections (i.e., FEMA Quick-2, Heastad Flowmaster, etc.)

A sufficient amount of cross-sections should be used to adequately represent and analyze the physical features (narrows, culverts, bridges, etc.) of the stream.

### 3.4.1 NORMAL DEPTH COMPUTATION

Normal depth occurs for a stream section when the flow is uniform, steady, and not effected by downstream channel features (i.e., drop structures, weirs, etc.). Open-channel flow is said to be uniform if the depth of flow is the same at every section of the channel. For a given channel geometry, roughness, discharge, and slope, the only possible depth for maintaining uniform flow is the normal depth.

The computation of normal depth at a cross section can be performed using the Manning's formula as follows:

$$Q = (\frac{1.49}{n})\, A\, R^{2/3} S^{1/2} \qquad \text{(Eq. CH10-201)}$$

Where Q = Flow rate (cubic feet per second, cfs)
n = Roughness coefficient
A = Area (square feet, sf)
P = Wetted perimeter (feet)
R = A/P = Hydraulic radius (feet)
S = Slope of the energy grade line (feet/feet)

For prismatic channels, the energy gradeline (EGL) slope, hydraulic gradeline (HGL) slope, and the channel bottom slope are assumed to be the same for uniform, normal depth flow conditions. Therefore, the channel bottom slope may be used for the normal depth calculation. The channel and overbank Manning's "n" values may be estimated from field observations and using Table CH13-T102 or other published methods. Area, A, is the flow conveyance area below the water surface elevation. Wetted Perimeter, P, is the length of the channel/overbank along the ground surface of the cross section, below the water surface elevation.

### 3.4.2 WEIR FLOW DEPTH COMPUTATION

There are two main types of weirs: sharp-crested and broad-crested. A sharp-crested weir has a sharp upstream edge. A broad-crested weir has a horizontal or nearly horizontal crest sufficiently long in the direction of flow so that the overflowing sheet of water will be supported and hydrostatic pressures will be fully developed for at least a short distance.

For most of roadways, the flow overtopping depths can be estimated using the following broad crested weir equation (Brater and King, 1976):

For horizontal crested weirs (broad-crested and sharp-crested):

$$Q = CLH^{3/2} \qquad \text{(Eq. CH10-202)}$$

Where,

Q = Flow (cubic feet per second)
C = Weir coefficient
L = Effective horizontal length of weir (feet)
H = Head (feet)

Weir coefficient, C, varies from approximately 2.5 to 3.1 for most broad crested weirs, and the weir coefficients for various weir sizes are summarized in Table CH10-T301. Weir coefficient of 3.0 is reasonable for weir flow over paved roadways.

Effective horizontal length, L, is the effective width of the weir cross section, perpendicular to the direction of flow. Weir head, H, is the difference between the upstream energy grade and road crest elevations. For approximation purposes, weir head can be assumed to be equal to the difference between the upstream water surface and road crest elevations.

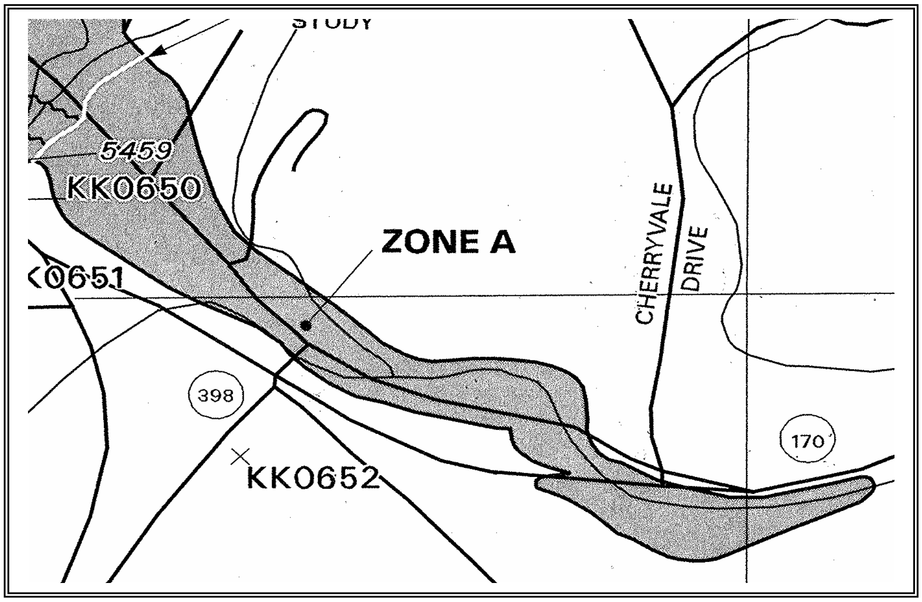
### 3.4.3 SURVEYED HIGH WATER MARKS

If surveyed high water marks from a previous flood event, close to a 100-year event, are available for the drainageway, the high water mark elevations may be used to supplement the computed 100-year water surface elevations/depths.

## 3.5 APPROXIMATE FLOODPLAIN DELINEATION

*The limited method results in the delineation of approximate 100-year floodplain boundaries.*

For detailed discussions on the delineation of approximate 100-year floodplain boundaries and floodplain delineation map requirements, readers are referred to Chapter 11.



Values of C in the Formula Q=CLH^(3/2) for Broad-Crested Weirs

| Measured head in feet, H | Breadth of crest of weir in feet | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0.5 | 0.75 | 1 | 1.5 | 2 | 2.5 | 3 | 4 | 5 | 10 | 15 |
| 0.2 | 2.80 | 2.75 | 2.69 | 2.62 | 2.54 | 2.48 | 2.44 | 2.38 | 2.34 | 2.49 | 2.68 |
| 0.4 | 2.92 | 2.80 | 2.72 | 2.64 | 2.61 | 2.60 | 2.58 | 2.54 | 2.50 | 2.56 | 2.70 |
| 0.6 | 3.08 | 2.89 | 2.75 | 2.64 | 2.61 | 2.60 | 2.68 | 2.69 | 2.70 | 2.70 | 2.70 |
| 0.8 | 3.30 | 3.04 | 2.85 | 2.68 | 2.60 | 2.60 | 2.67 | 2.68 | 2.68 | 2.69 | 2.64 |
| 1.0 | 3.32 | 3.14 | 2.98 | 2.75 | 2.66 | 2.64 | 2.65 | 2.67 | 2.68 | 2.68 | 2.63 |
| 1.2 | 3.32 | 3.20 | 3.08 | 2.86 | 2.70 | 2.65 | 2.64 | 2.67 | 2.66 | 2.69 | 2.64 |
| 1.4 | 3.32 | 3.26 | 3.20 | 2.92 | 2.77 | 2.68 | 2.64 | 2.65 | 2.65 | 2.67 | 2.64 |
| 1.6 | 3.32 | 3.29 | 3.28 | 3.07 | 2.89 | 2.75 | 2.68 | 2.66 | 2.65 | 2.64 | 2.63 |
| 1.8 | 3.32 | 3.32 | 3.31 | 3.07 | 2.88 | 2.74 | 2.68 | 2.66 | 2.65 | 2.64 | 2.63 |
| 2.0 | 3.32 | 3.31 | 3.30 | 3.03 | 2.85 | 2.76 | 2.72 | 2.68 | 2.65 | 2.64 | 2.63 |
| 2.5 | 3.32 | 3.32 | 3.31 | 3.28 | 3.07 | 2.89 | 2.81 | 2.72 | 2.67 | 2.64 | 2.63 |
| 3.0 | 3.32 | 3.32 | 3.32 | 3.32 | 3.20 | 3.05 | 2.92 | 2.73 | 2.66 | 2.64 | 2.63 |
| 3.5 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.19 | 2.97 | 2.76 | 2.68 | 2.64 | 2.63 |
| 4.0 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.07 | 2.79 | 2.70 | 2.64 | 2.63 |
| 4.5 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 2.88 | 2.74 | 2.64 | 2.63 |
| 5.0 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.07 | 2.79 | 2.64 | 2.63 |
| 5.5 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 2.88 | 2.64 | 2.63 |