TOWN OF MARANA DRAINAGE MANUAL

SEPTEMBER 2024

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1. Introduction and Purpose

1.1 Introduction

The Town of Marana (Town) has prepared this manual for the civil engineering and floodplain management community that is designing improvements within the Town limits. This shall apply to public and private improvements. Historically, the Town has relied on regional standards and methods in the hydrology/hydraulic discipline that mainly originated in the City of Tucson and Pima County. The Town prepared its own Subdivision Street Standards Manual for the first time in 2004 and has since updated that manual a total of two (2) times. This experience of having standards specific to the Town, compiled and referenced all in one location, has been very helpful to not only Town Staff but to the end users of the manuals: planners and engineers.

The purpose of this manual is to compile the Town's policy by consolidating the historically used regional standards and methods but modifying standards to be more reflective of Marana's topography and culture. New methods and requirements for specific situations that are applicable within the Town Limits are also included. The standards in this manual are guidelines, which will be criteria for approval of submittals. However, it is recognized that there are situations for which these standards may not be applicable. In these cases, and in cases of conflict or contradiction, sound engineering judgment consistent with accepted practice shall be used and in some cases approval in writing by the Town Engineer will be necessary.

Those portions of these standards prepared by the Pima County Regional Flood Control District (District) or the City of Tucson have been included herein as public domain information. It should be noted that this manual is adopting those standards from other agencies as they were at the time of publishing of this manual. Future changes to policy and methodology are not automatically incorporated into this manual.

1.2 Purpose

The Town has grown substantially in area and population since first incorporating in 1977 and the growth has been subject to standards and policies through development processes that have been implemented and adapted as the town has grown. The Town is developing this drainage manual to manage the future growth of the Town with a single point of reference for drainage policy. This manual seeks to accomplish the following:

- Compile drainage policies that currently exist in multiple references.
- Establish policies that are appropriate to the Town's topography and culture.
- Provide consistency for the development community, the public, and the Town.
- Provide clarity to the development and engineering community of what to expect during the review and approval process.
- A more consistent/efficient review and approval process.
- A better final design that aims to keep the public safe from flooding for an established set of criteria that is achievable and cost-effective.
- A design that acknowledges the maintenance costs of improvements and attempts to minimize undue maintenance for both publicly and privately owned infrastructure.

1.3 Applicability

These standards should be applied to development, both commercial and residential, and public works improvements when no such overriding standards, agreements and conditions apply. In the future, there could be overlapping of these standards with the Marana Town Code, Chapter 17-15 Floodplain and Erosion Hazard Management Code (Ordinance) and in those cases, the more restrictive of the two will take priority.

2. Documentation Requirements

Drainage submittals supporting improvements and developments will need to meet minimum content standards in accordance with the size and complexity of the project. The types of drainage support documents and specific requirements are detailed in this section.

2.1 Drainage Statement

A drainage statement is an appropriate supporting document when the project is five (5) acres or less and meets all of the following conditions:

- Detention/Retention is not required.
- No hydrologic or hydraulic modeling is required.

Prior to pursuing or scoping a Drainage Statement as the supporting documentation for a project, consult with the Town to determine if this is acceptable. A Drainage Statement may be appropriate for developments covered by an approved Master Drainage Report under the condition that Detention/Retention was handled regionally and within the spine infrastructure of the development. There are other scenarios where a drainage statement may be appropriate when the density of development and subsequent drainage complexity does not require a report. See Appendix A for Drainage Statement Format.

2.2 Drainage Report

A drainage report is an appropriate supporting document for public infrastructure projects that may affect drainage and those private development projects that exceed the thresholds for a drainage statement. Drainage reports are appropriate for the following:

- Subdivisions with single family residential land-use.
- Commercial, Industrial and Institutional Developments that are single owner – non-phased improvements.
- Public infrastructure projects.
- Private utility projects.

See Appendix A for Drainage Report Format.

2.3 Master Drainage Report

A master drainage report (MDR) is an appropriate supporting document for master planned developments, phased developments, and any platted subdivisions (residential or commercial) with Blocks that are created for future development. The MDR is approved in perpetuity by virtue of the development review process. MDRs may become obsolete, non-conforming, or not in-step with the owner's/Town's vision for the area. At such time the MDR must be revised and entitled the "Revised" MDR and make reference to former MDR approval. Reasons for revising the MDR include, but are not limited to the following:

- Change in federal flood hazard zone designation
- Change in land-use of the property

- Change in hydrology methodology (runoff method, rainfall, etc.)
- Change in drainage concept

2.3.1 Master Drainage Concept
The master drainage concept (MDC) must address the spine infrastructure drainage, regional detention, and block to block drainage concept. This drainage concept addresses the potential runoff of the entire block and its ultimate destination in the MDC. Blocks that do not drain into spine infrastructure must establish flow types, amounts across block boundaries. Flow types across block boundaries can be divided into three (3) categories:

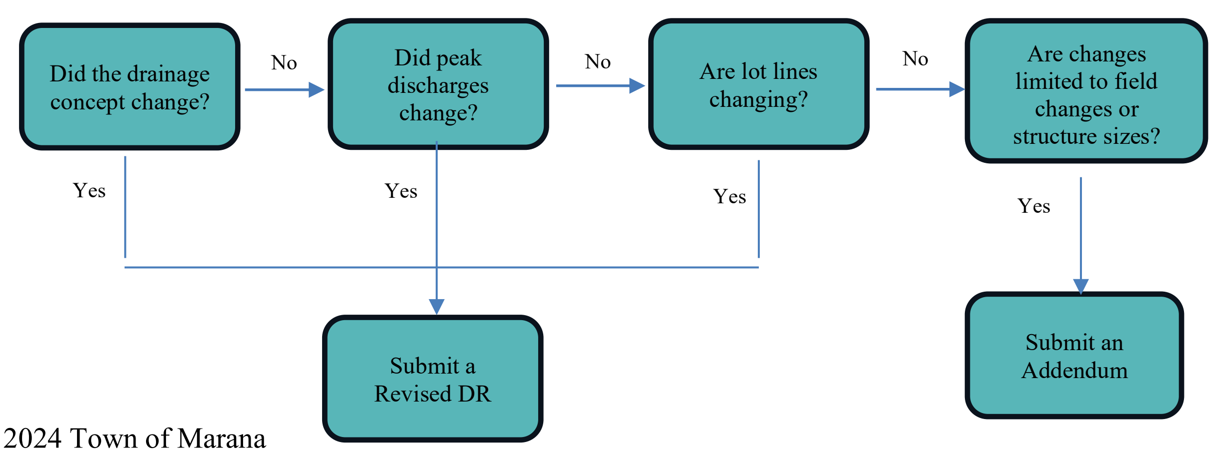
- Sheet flow – downstream block must account for future sheet flow from upstream developed block
- Dispersed flow – downstream block must account for at least 2 or more locations of flow from the upstream block
- Concentrated flow – downstream block need only account for one concentrated inflow from the upstream block

Blocks that develop first shall establish the location of flow acceptance and discharge and post-developed blocks shall adhere and accept those locations. The concept of individual block detention/retention over regional detention must be approved in writing by the Town Engineer.

See Appendix A for Master Drainage Report Format

2.4 Technical Data Support Notebook
The Technical Data Support Notebook (SDTSDN) is a nationally accepted report format for documenting large scale hydrologic and hydraulic analyses with the intent to establish flood risk inundation. This format is required for supporting a Conditional Letter of Map Revision (CLOMR) and Letter of Map Revision (LOMR). A regionally accepted format for the DDTSDN has been published by Arizona Department of Water Resources as State Standard SS1. Attention should be given to projects that overlap municipal boundaries that multiple community concurrence forms may be required. See MT-2 form instructions.

2.5 Revisions
Revisions are for the purpose of revising or adding to approved reports. A report that is not yet approved by the town cannot have a revision or addendum to it. Revisions or refinements to a drainage concept may need to be documented in a re-submittal during the development review process. The decision matrix for submitting a revision (Revised Drainage Report) versus an addendum is laid out in the flow chart below:



3. Regulatory Requirements

3.1 Floodplain and Erosion Hazard Management Code

Chapter 17-15 of Marana Town Code is entitled Floodplain and Erosion Hazard Management Code and was approved by Ordinance 2015.021. This section of the Town Code shall hereafter in this document be referred to as the "Ordinance." In the future, there could be overlapping of these standards with the Ordinance and in those cases, the more restrictive of the two will take priority.

3.2 Northwest Policy

The Northwest Policy is hereby incorporated into the Drainage Manual and will now be the home for future modifications and updates. The Northwest Policy was first introduced in December of 2000 for the purpose of addressing the development of the area of the Town Limits which are largely devoid of drainage conveyance mechanisms either natural or improved. The boundary in which the Northwest Policy shall be administered is shown on Figure 1, NW Policy Area Map. Accordingly, the Town adopted the following requirements for this region:

3.2.1 Sheet Flow

Any offsite sheet flow intercepted by a project shall be passed through the project, onsite, or shall be accommodated via an approved alternative facility. Simply elevating a project out of a sheet-flow area without regard to the displaced floodwaters will not suffice. The Town may require flow corridors to be dedicated to provide conveyance of offsite flow in sheet flow areas.

3.2.2 Insufficient Receiving Waters

Projects lacking sufficient improved or natural receiving waters into which stormwater runoff may be discharged shall be required to retain 100% of all on-site runoff volume generated during the design storm. The design storm shall be a 100-year, 1-hr storm, followed by a 10-year, 1-hr storm. These storms shall be applied assuming a lag of 24 hours between (This is important for calculating drain time). Drain time must be 36 hours or less and the clock begins at the end of the 10-year storm. The volume of runoff to be retained shall be computed using the applicable equation excerpted below. An emergency overflow weir, which may discharge to an existing public street, shall be provided as an element of any onsite retention facility.

$$Equation\ 3.1 \qquad V_{post-rp} = \frac{C_{w-rp}\, P_{1-rp}\, A}{12}$$

Where:

A	=	Watershed area
C_{w-rp}	=	Weighted runoff coefficient for the return period, dimensionless
P_{1-rp}	=	1-hour rainfall depth for the return period, inches
$V_{post-rp}$	=	Runoff volume for the return period, acre feet

3.2.3 Ponding Restrictions
Ponded water in basins shall be in conformance with Section 6.3 of this manual, "Basin Depth and Perimeter Safety Requirements."

Drywells are the most common disposal method used to meet drain time requirements. Drywell and Infiltration Trench requirements are discussed in Sections 6.1.1 and 6.1.2 respectively.

3.3 Excavations next to Flood Control Infrastructure
Recognizing the inherent hazard of potentially unstable slopes from deep excavations upon public infrastructure due to failure mechanisms such as seepage, rilling, and headcutting, the Town has generated a standard for safe setback to public infrastructure and neighboring property for all new sand and gravel operations that are permitted within the town limits.
Without additional geotechnical analysis supporting stability of excavations, the following setbacks from property lines shall be adhered to under the following conditions:

3.31 Upstream of Avra Valley Road

1. Maximum excavation slope shall be 1:1
2. Top of slope (as first permitted) shall be setback:
    a. No closer than 100 feet for 25-50-foot depth excavation
    b. No closer than 200 feet for 50-100-foot depth excavation
3. Excavations deeper than 75 feet shall first install piezometers in the area between effluent flow and excavation to evaluate presence of lateral groundwater flow. A slope failure analysis due to saturation will be required at time of submittal.
4. Final reclaimed slopes shall be no steeper than 3:1

3.32 Downstream of Avra Valley Road

1. Maximum excavation slope shall be 1:1
2. Top of slope (as first permitted) shall be setback:
    a. No closer than 150 feet for 25-75-foot depth excavation
    b. No closer than 300 feet for 75-150-foot depth excavation
3. Excavations deeper than 125 feet shall first install piezometers in the area between effluent flow and excavation to evaluate presence of lateral groundwater flow. A slope failure analysis due to saturation will be required at time of submittal.
4. Final reclaimed slopes shall be no steeper than 3:1

3.33 Additional Geotechnical Analysis

Minimum setback to property line/public infrastructure shall be 100-feet when supported by additional geotechnical analysis, including:

- Appropriate characterization of soils on slope and in between property line/public infrastructure to a depth greater than ultimate excavation depth
- When setbacks are proposed which do not provide for sufficient distance to achieve a reclaimed 3:1 slope, a statement that slopes are stable and will not create a hazard to adjacent personal property or public infrastructure must be made under the seal of a registered professional engineer in the State of Arizona.
- A slope failure analysis will be required for any setbacks proposed that are less than standard minimums published above.

Additional restrictions on excavations outlined in the Ordinance shall remain and be full in force.

4. Hydrology

The Town uses a combination of rainfall runoff methodology to quantify peak discharges within the town limits that depends on purpose of the discharge quantity, the location of the project and the size of the watershed. One distinction in terms of location within Town Limits is the Tortolita Fan which comprises a good portion of the Marana Town Limits. Because the Tortolita Fan area is unique geomorphologically and is subject to unconsolidated sheet flow, distributary flow, and active alluvial channel formation, the method for rainfall runoff requires a rain on grid analysis using finite difference methodology. The remainder of the Town Limits should utilize the Pima County Hydrology Methodology. Both methodologies are discussed in detail below.

4.1 Tortolita Alluvial Fan Study (TAFS) Area

Figure 2, TAFS Area is provided to show the region that is covered by the Town's TAFS model. The TAFS model uses two software programs: HEC-1 and FLO-2D. This modeling effort was approved by FEMA for flood hazard mapping use. The study area has two model scenarios to address rainfall aerial reduction: Regional and Non-regional. Refer to References to download the models.

Regional: Rainfall aerial reduction factor was based on the watershed size of the entire TAFS area (±165 square miles). The regional model was approved by FEMA for floodplain mapping in the TAFS area.
Non-regional: Rainfall aerial reduction factors were based on the watershed size of each major watershed (total 10 major watersheds, with varying watershed size in the range of 7.8 ~ 24.8 square miles (measured by extending major watersheds to Interstate 10). The non-regional model is intended to be used by the Town for development projects

The size of the TAFS area necessitates aerial reduction for accurate flood risk; without aerial reduction the regional discharges would be exaggerated. However, a scenario was executed and entitled "non-regional" which applies aerial reduction at the sub-basin level instead of at the watershed scale. This non-regional scenario was approved for determining design discharges for areas within the fan.

When preparing a Letter of Map Change (LOMC) through FEMA to address changes to flood boundaries within the TAFS area, the revision should utilize hydrology from the regional model. When preparing a development submittal (commercial or residential) and the purpose is to support the plats, development plans and improvement plans for drainage design and accommodation of offsite flows, the Drainage Report should utilize non-regional hydrologic results for offsite hydrology. Non-regional results can come from a non-regional FLO-2D run. The consultant needs to use sound engineering judgement in selecting the offsite hydrology method. Pima County methodology, specifically use of PC-Hydro software, is very conservative and is an accepted method for design hydrology on the fan (See Section 4.2).

Exceptions within the TAFS Area:

- Onsite hydrology within TAFS shall be prepared using PC-Hydro software and methodology.
- Regional FLO-2D model results are not permitted for design discharges.
- Locations on the fan where the FLO-2D does not show flow accumulation and clearly has contributing watershed upstream of planned development must be accounted for with a PC-Hydro runoff calculation.

Regional FLO-2D model results shall only be used for FEMA purposes. Design discharges shall be either the Non-regional FLO-2D model value or other design discharge specified by the Town. Both regional and non-regional FLO-2D models will be provided by the Town with a public information request.

4.2 Areas outside of TAFS

Projects located outside of the TAFS boundary should use Pima County Hydrology Methodology as published in TECH-015. One exception to TECH-015 is that Marana will accept use of PC-Hydro on watersheds up to 10 square miles or times of concentration up to 180 minutes for design purposes only. Rainfall shall use NOAA Atlas 14 (or latest published Atlas) mean point precipitation frequency estimates. If PC-Hydro is used it defaults to Upper 90% point precipitation estimates. The Town will accept manual input of the mean rainfall estimate into the PC-Hydro web application under the following conditions:

- The report shall clearly state that the decision was made to use mean rainfall values.
- The plotted PC-Hydro results sheets shall be clearly marked that they were conducted with mean rainfall data and not Upper 90%.
- A plot of NOAA Atlas 14 rainfall depths showing mean rainfall depths shall be provided.

FEMA remapping projects outside of TAFS should use a rainfall runoff method acceptable to FEMA that is as accurate as possible and verified to published frequency estimates and/or within the confidence intervals of regional regression estimates.

4.3 Regulatory Discharges

The Town has published regulatory and design discharges for certain watercourses. The discharges can be found in the Ordinance Section 17-15-11 Appendix 1.

4.4 Previously Approved Discharges

In addition to Section 4.3 published discharges, the town has authority to direct the use of other design discharges that may have been approved with other projects. Consult with the Town about other discharges that may be affecting the project in question.

4.5 Accepted Software Programs

The Town will accept the following software programs for hydrology in accordance with Section 4.2:

- FLO-2D Pro versions. In the TAFS area, only FLO-2D Pro [Build No. 17.08.17] will be allowed due to other FLO-2D Pro versions having issues with the combined Green-Ampt and SCS curve number method, which was utilized in the TAFS models.
- PC-Hydro 7.4
- HEC-1 allowed for generational projects that have been prepared using HEC-1.
- HEC-HMS for new projects

4.6 Hydrologic Parameters

For projects that use Pima County Hydrology Methodology, parameter selection is defined in the PC-Hydro 7.4 Users Manual embedded in the web application. Soils are not populated automatically, so when choosing soil coverage use PimaMaps-SDCP. Toggle on the Geology Layer and then the Hydrologic Soils Group – NRCS layer.

For projects that use FLO-2D user shall utilize:

- TAFS modeling parameters for projects located in the TAFS area.
- Outside the TAFS area, Pima County's Criteria for Two-Dimensional Modeling Technical Policy (TECH-033) shall be used.

5. Hydraulics

Hydraulic methodology originates from multiple sources that have been accepted by the Town. The most robust source on hydraulics is the Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona (City Manual) (1998) prepared by Simon, Li and Associates, Inc. for the City of Tucson. The City Manual is widely accepted in Southern Arizona and adjacent jurisdictions for drainage design methodology. The following sections make any clarifications or specific departures from the City Manual standards.

5.1 Open Channel Design

It is the preference of the Town to preserve existing natural channels with a natural bottom. If the channel bottom cannot be maintained as natural, then a justification must be provided within the report. Open Channel design methodology is discussed in Chapter 8 of the City Manual (See Appendix B). What follows are some clarifications and/or supplemental information to those standards:

- Sheet flow areas, specifically in the Northwest Area, may require some corridors for offsite generated drainage conveyance. Do not assume that upstream development will retain pass-through flow.
- Sheet flow in areas upstream of residential developments or habitable structures shall be collected following the Collector Channels standards in Chapter 8 of the City Manual. Sheet flow in areas upstream of other projects such as linear roadways may be collected in collector channels designed per alternative capacity and freeboard criteria as approved on a project-specific basis by the Town Engineer. Collector channels should be fully-lined with a material that is resistant to erosion and provides a means to remove sediment to restore constructed grades.
- Constructed channel slopes in the Northwest Marana area (only) may be as flat as 0.0015 ft/ft
- When a project runs adjacent to one of the future Marana Drainage Master Plan channel alignments on Barnett Rd, Marana Rd, or CMID 10.5 canal the Town may require construction or dedication of right-of-way for the future channel.
- CMID Canals shall not be considered as providing flood conveyance capacity.
- Open Channels shall be designed with non-regional discharges.

Acceptable methods for calculating water surface elevations/capacity of channels includes normal depth calculations for those channels/watercourses that do not have the presence of structures/ineffective flow areas that would create backwater. If the watercourse is regulatory (>50 cfs) and has culvert crossings, then it needs to be evaluated with a steady flow direct step backwater model that also has internal drainage structure calculations rather than simple normal depth calculations.

Acceptable software includes Federal Highway's Hydraulic Toolbox and other proprietary software for calculating normal depth. For backwater applications, it is recommended to use the latest available version of HEC-RAS.

5.2 Cross-Drainage

The goal of cross-drainage design will be to achieve all-weather access. On-site roadways shall be designed to convey the 100-year discharge under the roadway. Off-site public roadways will continue to be a challenge until the Town's Master Drainage Plan is implemented. Because of sheet-flow from the TAFS area and large expanses of the Town limits inundated by the Santa Cruz River, there will need to be exceptions until regional flood control improvements go in. As a matter of interpretation and consistency with other adjacent jurisdictions, a development provides all-weather access when it connects to an existing public collector or arterial roadway despite whether the Town or County maintained collector roadway provides all-weather access along its entire length. An on-site all-weather at-grade crossing may be considered if it can be demonstrated that a dry-crossing is not feasible, that it conforms to Section 9.3 of the *Marana Streets and Subdivision Standards* and that is has attained prior approval by the Town before it is included in a development package.

5.2.1 Culverts

Culvert analysis shall follow FHWA's *Hydraulic Design Series (HDS) No. 5 (2012)*. Software based on HDS No. 5 methodology is acceptable for calculating hydraulic results.

Culvert headwater shall be designed to be a minimum of six inches below the hinge point of roadway for newly designed on-site roadways.

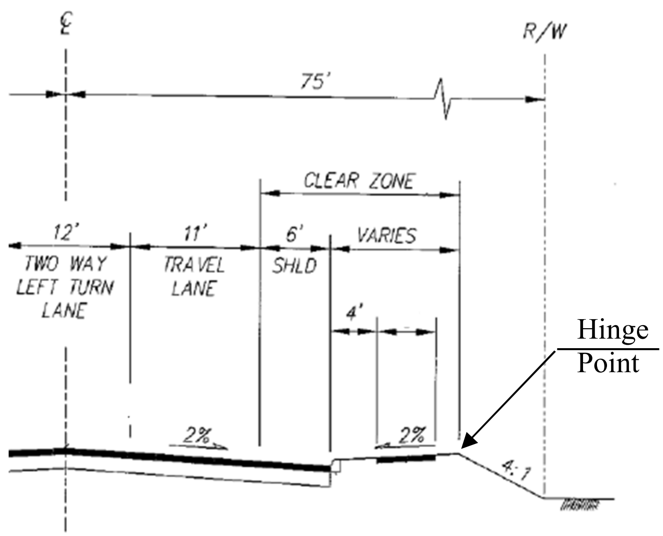


Figure 1 - Partial Roadway Cross Section

Off-site public roadway improvements cross-drainage design criteria shall be determined by the Town on a case-by-case basis.

Inlet and outlet treatment for culverts shall be required and be appropriate for the design of the road. Pipe ends projecting from slope are acceptable in local rural road applications only. Mitered culvert ends are acceptable only on slopes 2:1 and steeper; surrounding slopes must be armored with erosion protection.

Additional provisions for inlet and outlet treatment may be required based on *Roadside Design Guide* (AASHTO). Barricade railing shall be provided at the top of culvert headwalls when distance between established walkway and back of headwall is less than 5-feet.

Drop inlets used to accommodate cover over a cross-culvert must be evaluated in both inlet control and weir control and use the highest calculated headwater. Drop inlets must also provide a minimum of 7-feet from the face of the culvert to the toe of the drop slope to accommodate maintenance equipment. One side of the drop inlet shall be traversable to a piece of tracked equipment. Traversable is defined as a 3:1 slope that can support the weight of a 950G loader or equivalent.

5.2.2 Bridges

Bridges shall be analyzed using a direct step backwater model in a sub-critical regime with the goal to get the most conservative water surface elevation (WSEL) on the upstream side of the bridge face in a 1-dimensional environment. Refer to HEC-RAS *Hydraulic Reference Manual* Section 7.1 for bridge modeling considerations.

Freeboard at bridges shall conform to the guidance in *Guidelines for Establishing Scour and Freeboard for Bridges in Pima County (2012)*. Bridge Scour is addressed in Section 5.5. If the bridge falls in a 2-Dimensional flow environment, it is recommended to schedule a meeting with the Town to discuss the design approach. There could be occasions where a low-flow bridge is acceptable, and those occasions will be reviewed on a case-by-case basis.

5.3 Storm Drain Design

This section is included for describing policy and methodology of hydraulics in storm drain trunklines and laterals. Inlet design capacity will be discussed in Section 5.4, Pavement Drainage. Hydraulic grade line (HGL) calculations should be performed on the trunkline and associated junctions and manholes along the trunkline beginning at the downstream terminus and calculating in the upstream direction. See City Manual Section 10.8 for instructions on HGL calculations. StormCAD and other proprietary software may be appropriate for performing these calculations.

5.3.1 Trunklines

5.3.1.1 Private Systems

Storm drain systems on developed sites that are not in public right-of-way will be maintained by the owner or property owners association. Sites that have a sediment yield shall follow design criteria for public systems. Sites that are determined to be clear-water conditions may follow the guidelines below:

- Minimum trunkline slope may be 0.1% for smooth interior pipes.
- Pipe materials other than reinforced concrete pipe may be used provided the manufacturers recommendations for cover are followed and documented in the drainage report.
- Pipe segments may match at the invert although preference would be to match soffits.
- Pipe diameter or aggregate cross-sectional flow area must increase in the downstream direction.
- Design event can be less than 100-year event if it is demonstrated that the combination of surface and sub-surface flow for the 100-year event is conveyed safely to downstream destination and adjacent habitable structures are elevated appropriately.
- If pipe system is used for supplemental storage volume, then pipes must be designed to be water tight. Furthermore, pipes must also meet basin drain time requirements.

5.3.1.2 Public Systems

Storm drain systems within public right-of-way are maintained by the Town and therefore must meet the following requirements:

- Pipe material shall be rubber gasket reinforced concrete pipe (RGRCP).
- Minimum trunkline slope may be 0.25% if velocity conditions are met.
- Velocity in pipe shall not be less than 3 fps.
- Velocity in trunkline must be maintained or increased in the downstream direction. Decreased velocity will subject the pipe to deposition of sediment.
- Minimum pipe diameter is 24-inches for smooth interior pipes.
- Pipes segments shall match at the inside top of pipe in the downstream direction.
- HGL shall be designed to stay below finished grade and 6-inches below inlets and manholes.

The Town Engineer may allow manhole rims to be bolted down in rare cases when the 6-inch separation cannot be achieved.
- Trunklines shall be designed for the 100-year event.

5.3.2 Laterals

The following requirements shall apply to laterals of both public and private systems:
- Laterals slope may not exceed 10% and should not be less than 0.5%
- Laterals shall connect to trunkline at no greater than 90-degrees.
- Laterals on public systems shall not be smaller than 18-inches.
- HGL at upstream terminus of lateral shall be at least 6-inches below gutter grade or grate elevation for the 100-year event.
- Cover over pipe shall be in accordance with manufacturer's recommendations.

5.4 Pavement Drainage

Subdivision and Commercial Developments shall follow the guidelines in Section 9.0 of the Subdivision Streets Standards. Additional stipulations have been added below:
- Parking may be proposed with ponding areas (including within flood hazard zones) provided that the flooding depth does not exceed 1-foot during the 100-year event.
- Parking areas subject to flooding from a regulatory watercourse must be signed to warn vehicle owners of such risk.
- Arterial Streets and multi-laned curbed roadways shall provide at least one dry lane (10-feet) in each direction during a 10-year storm event.
- Cross-slopes may flatten to zero on local roads to accommodate transverse grates which extend from curb to curb. Cross-slopes must immediately transition back to the design cross-slope(s) in accordance with AASHTO guidance on transition lengths on either side of the transverse grate.
- Curbs may transition from 4-inch roll/5-inch wedge curb to 6-inch vertical for curb inlets and scuppers. Transition must happen abruptly and not impact driveway access.
- Maximum depth in roadway shall be 1-foot at any point, provided it is contained in the right-of-way or is discharged through a controlled structure into a facility with capacity to receive the peak flow. Depth shall be measured at the gutter/base of curb going upward irrespective of inlet depression.
- Inlet capacity calculations should follow procedures in the City Manual Section 10.6.
- Inlet clogging factors can be found in the City Manual Section 10.6.9.
- When sidewalk is present, a curb inlet shall be provided to take the 10-year flow under the sidewalk.
- Overtopping of sidewalks during the 100-year may be allowed provided that the entire length of susceptible overtopping area is protected from erosion.
- Depressed curb, where no sidewalk is provided, may be longer than 10-feet provided post barricades are provided at 5-foot (center to center) but in no case shall be longer than 20-feet.

With the prevalence of transverse grate inlets that extend across the entire road, the City Manual may be appropriate for inlet capacity calculations; however, when curved vane grates are utilized it is recommended to use the capacity calculations within the FHWA's Hydraulic Toolbox, which is a publicly available software. Other proprietary inlet performance charts may be accepted on a case-by-case basis by the Town.

Offsite public roadways pavement drainage design shall follow guidelines in the 4th Edition of the *Pima County Roadway Design Manual (2013)*.

It is the position of the Town that discharge from the street via scuppers to the surface is more desirable than the addition of storm-drain to the Town's infrastructure inventory. Use of public storm drain must be accompanied with a demonstration that it is the best alternative for conveyance of stormwater.

5.5 Scour

There are three scenarios that need scour analysis: 1) Bridge Design; 2) Outfall Scour; and 3) Riverine Scour. Bridge design shall follow the *Guidelines for Establishing Scour and Freeboard for Bridges in Pima County (2012)*.

The latter two scenarios should follow methodology in the City Manual, Chapter 6. When determining a design scour depth on the Santa Cruz River an additional 5-feet shall be added to the design toe-down to account for continuous presence of effluent and the entrenching created by the long-term presence of clear water.

A headcut on the Santa Cruz River continues to migrate towards Marana from Pinal County. For future regional flood control improvements or new bridges over the Santa Cruz River, the propagation of the headcut upstream should be considered in design of features with design life greater than 25-years.

5.6 Erosion Mitigation

For practical purposes most natural undisturbed washes can be considered to be at equilibrium. Once washes are modified or graded, they are susceptible to changes in equilibrium and must be treated as unstable. Grade control shall be provided on earthen channels/bottoms that have design slopes steeper than the equilibrium slope. See Section 6.9 of the City Manual for the procedure on channel grade control.

Erosion potential is also required to be evaluated and mitigated at outlets of drainage structures. There is guidance in Section VI, *Drainage and Channel Design Standards for Local Drainage (1984)* for scour protection at culvert outlets.

Sheet flow areas are not typically environments for erosion; however, newly placed fill and earthen embankment are usually susceptible to erosion even during sheet flooding. To mitigate erosion of fill pads, follow the guidance in *Pima County Technical Policy, TECH-006 Erosion Protection of Fill Pads in Regulatory Floodplains*.

The test for susceptibility to erosion on a fluvial watercourse is to determine what the lateral migration limits are. Erosion Hazard Setbacks (EHS) are calculated for potential migration areas and any improvements within those limits are susceptible. The Ordinance restricts uses within the EHS and if a project needs to mitigate the erosion hazard to reclaim the use of that property, erosion mitigation in the form of bank protection should be installed to eliminate the potential of lateral migration. Bank protection design should follow the guidance in Section 8.5 of the City Manual.

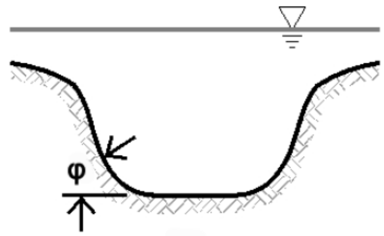
Rock rip-rap is one alternative for bank protection. The effectiveness of dumped rock rip-rap is dependent upon a satisfactory gradation that allows the rock to be laid without significant voids. Voids in rip-rap layers due to homogenous sized rock can induce undermining, sloughing and failure as fine sediments are washed out from behind the layer because of too much porosity. There are two steps to appropriately size rip-rap for use as bank protection: 1) Calculate the D_{50} (the rock diameter that at which 50% passes in a sieve analysis); 2) Calculate the gradation of rip-rap.

5.6.1 Calculating Rip-rap D_{50}

The rip-rap d_{50} can be calculated using the Isbash equation for a straight reach of channel.

The equation (as adapted from Maricopa County) is as follows:

$$d_{50} = \frac{0.0191\,V_a^2}{\cos\varphi}\left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right)$$



With:

d_{50} = the median diameter (ft),
V_a = average velocity (ft/s)
γ_s = specific weight of stone (lb/ft3) = assumed 165 lb/ft^3
γ_w = specific weight of water (lb/ft3) = 62.4 lb/ft^3
φ = bank angle (degrees), see figure above

Minimum D_{50} in all applications is 6-inches. Rip-rap sizing methodology other than the Ishbash equation may be proposed within the drainage report. Refer to the Maricopa County manual for rip-rap sizing guidance in situations where curved reaches are present.

5.6.2 Calculating the Rip-rap Gradation

Rip-rap gradation limits should be specified in drainage reports and on construction documents where rip-rap is to be installed. Limits are meant to be for visual inspection of a mock-up prior to installation of rip-rap. The three-point gradation (adapted from HEC-11) below should be satisfactory in most applications for bank protection:

Stone Size Range (ft)	Stone Weight Range (lb)	Percent of Gradation Smaller Than
1.5 D_{50} to 1.7 D_{50}	3.0 W_{50} to 5.0 W_{50}	100
1.0 D_{50} to 1.15 D_{50}	1.0 W_{50} to 1.5 W_{50}	50
0.4 D_{50} to 0.6 D_{50}	0.1 W_{50} to 0.2 W_{50}	15

5.7 Sedimentation

The Town's concern with sedimentation exists where infrastructure can get plugged with sediment due to abrupt changes in sediment carrying capacity. The areas where this is most likely to happen is at cross-drainage structures of public roadways and storm drain systems. To prevent this, cross-drainage culverts should be designed to transport the sediment delivered to them. The City Manual Section 11.5 has a metric which gives indication of a sedimentation problem, referred to as the Sediment-transport ratio. The ratio compares sediment transport in the approach channel to the sediment transport capacity of the culvert.

It is not desirable to collect off-site, sediment laden flow into a closed storm-drain system. Storm drain systems should be limited to collect clear water from developed areas that do not have a sediment yield. In the event a closed system must convey flow from a watershed with sediment yield, a sediment trap should be sized and installed at the inlet.

Section 5.3.1.2 of this manual provides criteria for storm-drains to ensure they don't create an environment for deposition.

6. Detention/Retention

6.1 Design Standards for Stormwater Detention and Retention (2015)

The Town has adopted the *Design Standards for Stormwater Detention and Retention (2015)* (DSSDR). The standards and procedures within shall apply to those areas outside of the Northwest Policy Area. See Figure 1. Areas within the Northwest Policy Area should reference Section 3.2 of this manual.

6.1.1 Dry Well Design Requirements and Procedures

The Northwest Drainage policy requires the total retention of the 100 year, 1-hour storm followed by a 10 year, 1-hour storm. Although the policy does not specify, there is an expected lag of 24 hours between the storms. The retention basins associated with this storage are required to drain within 36 hours after the end of the 10-year storm. Drywells are the most common method of draining these basins. The calculations of basin size needs and well-injection rates must conform to these requirements. More conservative designs are encouraged. These requirements are based upon the DSSDR, the Northwest Policy, Arizona Department of Environmental Quality Drywell Guidance Manual (ADEQ), and empirical experience from existing projects in Northwest Marana:

1. The maximum drywell well-injection rate to be used for design is nominally 0.5 cubic feet per second (cfs). With specific geotechnical recommendations, the well-injection rate may be increased up to 0.6 cfs. The well-injection rate must be multiplied by a 0.5 de-rating factor for determining the number of drywells needed.
2. The basin must at a minimum totally contain the 100-year, 1 hour storm. Since these basins rely on infiltration as their means of draining, no outflow rate may be considered during the volume design of the basin.
3. Prior to introduction of the 10-year, 1 hour storm, the Town of Marana allows a credit to reduce the amount of stored volume in the basin from the original 100-year storm. This credit is half the aged drywell drain rate over the 24-hour lag period.

   Ex: 0.25 cfs/well * 0.5 * 3600 sec/hr* 24 hours/ 43560 = 0.25 acre-ft/well

4. After drywell credit is applied, the 10-year, 1 hour storm volume is added to the basin. The basin must contain this new volume plus the prior volume less the credit. This final basin volume cannot be lower than the volume calculated in number 2 above (total containment of the 100-year, 1 hour storm).

$$100\,\text{yr} - \text{credit} + 10\text{ year} \geq 100\text{ year for volume requirement}$$

5. The de-rated well-injection rate, while not allowed to be considered for volume requirements, can be used immediately in the calculations for determining if the basin will drain in 36 hours after the final ten-year storm.

6. Other key factors to be considered for basin design:
    a. Ponding depth maximum is 3.0 feet (Section 6.3)
    b. Backwater is allowed into drainage channels and parking areas but not onto streets.
    c. Basin floors may not be designed totally flat. They must be designed with positive drainage to the drywells – 0.5% minimum.
    d. Multiple Drywells should be spaced a minimum of 100 feet apart from center to center. (ADEQ)
    e. Drywell grates should be 24-inch cast iron with raised letters "STORMWATER ONLY". (ADEQ)
    f. Drywell grates should be a minimum of 6 inches above the bottom of landscaped retention basins (ADEQ).
7. During construction, a minimum of 1 well per basin must be tested for verification of flow rate at or above the original design. If the wells do not flow as designed, redesign is necessary, and more wells may be needed.
8. Drywells must be protected from silt infiltration during construction. Any and every drywell known to have been compromised during construction will have to be cleaned and tested prior to closeout of permitting.
9. Drywells must be registered with ADEQ prior to closeout of permitting.

6.1.2 Infiltration Trenches

Alternative methods for disposing of stormwater into the subsurface will be evaluated on a case-by-case basis. Drain time requirements will continue to drive the viability of subsurface disposal and a separate study prepared by a registered geotechnical engineer or hydrogeologist must accompany the construction plans and include at a minimum:

- Design percolation rate of stormwater into subsurface
- Native percolation rates of soils beneath the alternative structure
- Potential for fine sediments to clog the porous material.
- Annual inspection checklist
- Means to maintain the alternative structure.
- Certified statement by a registered professional engineer in the State of Arizona that the structure complies with ADEQ regulations.

6.2 Town of Marana Exceptions to the DSSDR

- Redevelopment of a property shall assume an existing impervious percentage equivalent to its last use prior to demolition of the site.
- First Flush Retention is not required in the Northwest Policy Area.
- The Town has issued its own Balanced and Critical Basin Map (Section 6.4) and will update that from time to time. All updates to that map will be automatically incorporated into the drainage manual.
- The minimum freeboard on detention basins (between maximum water surface and top of impoundment) in all cases will be 6-inches. This freeboard shall apply to Northwest Policy area as well.

6.3 Basin Depth and Perimeter Safety Requirements

Recognizing that ponded water under the right conditions can be a hazard and an environment for insect breeding, to deter the public and protect the pedestrian from potential hazards associated with ponded water, precautions should be taken when certain physical criteria are met.
Specific safety requirements are dependent upon land use and the size of development as follows:

6.3.1 Single Subdivision

1. Maximum ponding depth of 3 feet.
2. Barricade railing must be provided when both of the following occur:
    a. Ponding depth exceeds 2 feet
    b. Side slope is steeper than 4:1 (H:V)
3. If a minimum of 5 feet of refuge does not exist between the top of slope and back of pedestrian way and condition 2(b) is exceeded, then barricade railing must be provided.
4. Railing, at least 42 inches high, must extend along all portions of the basin accessible to pedestrians via a constructed walkway, the railing should terminate in a manner that protects the public from the hazard.

6.3.2 Master-Planned Subdivisions (with regional detention/retention) and Commercial Developments

1. In no case will ponding exceed 6 feet, unless approved by the Town Engineer (3 feet when a full retention system is employed).
2. Barricade railing must be provided when both of the following occur:
    a. Ponding depth exceeds 2 feet
    b. Side slope is steeper than 4:1 (H:V)
3. If a minimum of 5 feet of refuge does not exist between top of slope and back of pedestrian way, then barricade railing must be provided.
4. Railing, at least 42 inches high, must extend along all portions of the basin accessible to pedestrians via a constructed walkway, the railing should terminate in a manner that protects the public from the hazard.
    a. For commercial applications the railing should be provided along the basin edges that have the potential for employee foot traffic if the 5 feet refuge is not provided.
5. When ponding depth exceeds 3 feet and the side slope is steeper than 4:1, the entire basin perimeter must be secured with non-penetrable fencing.

This standard applies to all areas of the Town of Marana. Master-planned subdivisions that do not provide regional detention must follow the Single Subdivision requirements.

6.4 Critical Basin Map

All watersheds within the Town Boundary are designated Balanced Basins unless it has been determined to be a Critical Basin. See Figure 3, Balanced and Critical Basins Map. Future revisions to the Balanced and Critical Basins Map will be automatically incorporated into the Drainage Manual.

6.5 In-lieu Fee Process

Any request to waive the detention requirements shall come in writing in the form of a waiver request letter to the Town Engineer. The process for requesting a waiver should follow the guidance in Section 9 of the DSSDR.

Upon approval of the waiver request, an in-lieu fee calculation sheet will be provided to the applicant. The in-lieu fee calculation will require the following:

- Total required peak detention volume to meet the appropriate balanced or critical designation.
- Total required first flush retention volume.
- The area of land required with volume distributed over 3-feet in depth.
- A sample In-lieu calculation can be downloaded here: Sample Calculation and Excel File.

7. Dedication of Drainage Right of Way

Constructed channels that are dedicated to the public must be labeled as public drainageway on a subdivision plat or via separate instrument. Minimum 16-foot maintenance access shall be provided on each side of the channel. Projects adjacent to regional watercourses shall dedicate 50-feet measured from the front of bank protection. Consult the Town prior to proposing dedication to the public.

8. Maintenance

Improved Channels with side slopes steeper than 6:1 must be equipped with maintenance ramps for both private common areas and public rights of way. Access for maintenance on fully lined channels shall be designed to support the loads of the following equipment:

- 10 CY dump truck
- 950G Loader or equivalent

Ramps shall be a minimum of 10 feet wide and fall in the downstream direction. Ramps shall be cut-out of the bank to prevent loss of conveyance. When roadways bisect the channels and the culverts structures are not big enough for access, ramps shall be designed on the downstream side of each crossing.

Detention Basins must be inspected on an annual basis. A maintenance checklist shall be submitted within the Drainage Report. A sample channel/basin maintenance plan is provided in Appendix A

9. Storm Water Requirements

Storm water regulations exist for the purpose of monitoring water quality and are governed by the Arizona Department of Water Quality under the Arizona Pollution Discharge Elimination System (AZPDES) program. Marana is a Small MS4 General Permit area and must comply with requirements of the AZPDES program. Requirements to comply with the Town of Marana Stormwater division are separate and unique and can be found in the Marana Town Code Chapter 17-16, Stormwater Management.

10. References

TAFS model [(Click to access Models)]

Town of Marana. *Subdivision Street Standards Manual*. 2020

Town of Marana. *Chapter 17-15 Floodplain and Erosion Hazard Management Code (Ordinance)*. 2015

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Pima County. *Drainage and Channel Design Standards for Local Drainage*. 1984

Pima County. *Pima County Roadway Design Manual* (4th Edition). 2013

Pima County Regional Flood Control District. *PC-Hydro 7.4* (Web Application).

Pima County Regional Flood Control District. *Design Standards for Stormwater Detention and Retention for Pima County*. 2015

Pima County Regional Flood Control District. *TECH-006: Erosion Protection for Fill Pads in Regulatory Floodplains*. 2021

Pima County Regional Flood Control District. *TECH-015: Acceptable Methods for Determining Peak Discharges*. 2021

Pima County Regional Flood Control District. *TECH-033: Criteria for Two-Dimensional Modeling*. 2021

FLO-2D Software, Inc. *FLO-2D Pro software*. 2023

Arizona Department of Water Resources. *State Standard 1 Instructions for Organizing and Submitting Technical Support Data Notebooks (TSDN) for Flood Studies*. 2012

Flood Control District of Maricopa County. *Drainage Design Manual*. 2018

Federal Highway Administration. *Design of Riprap Revetment* (Hydraulic Engineering Circular No. 11). 1989

U.S. Army Corps of Engineers. Hydrologic Engineering Center. HEC-RAS (River Analysis System). *Hydraulic Reference Manual*.

11. Glossary

AASHTO: Abbreviation for the American Association of State Highway and Transportation Officials.

AERIAL REDUCTION FACTOR: The ratio of mean precipitation depth over a watershed resulting from a storm to the maximum point depth of the storm.

ALL WEATHER ACCESS: Access considered traversable by normal passenger vehicles, defined as a permanent, durable material with adequate protection against scour and erosion and having a depth of water no more than 12 inches above the roadway surface during a Base Flood. Asphalt, Concrete, and traffic rated pavers are considered durable surfaces. All other surfaces must be approved by the Town Engineer. See also the Floodplain and Erosion Hazard Management Code, Chapter 17-15 of the Town Code.

ALLUVIAL CHANNEL: Water channels made up of loose sediments. The loose sedimentary materials are known as alluvium. The banks of the channel are subjected to erosion, or wearing away, by fast running water.

ADEQ: Abbreviation for Arizona Department of Environmental Quality.

ADWR: Abbreviation for Arizona Department of Water Resources

AZPDES: Abbreviation for Arizona Pollution Discharge Elimination System

AT-GRADE CROSSING: A depression or vertical sag in the roadway designed to allow drainage to cross "at-grade" without using culverts.

BACKWATER: The effect tailwater has upon upstream flow. Backwater can also refer to the calculations that are performed to compute water-surface profiles in an open channel.

CMID CANALS: Cortaro- Marana Irrigation District Canals that provide irrigation water to more than 10,000 acres of farmland.

CONDITIONAL LETTER OF MAP REVISION (CLOMR): Federal Emergency Management Agency's (FEMA's) comment on a proposed project that would affect the hydrologic or hydraulic characteristics of a flooding source. A CLOMR does not revise the current National Flood Hazard Layer (NFHL).

CURB INLET: An inlet which captures street drainage and discharges through the curb. This can consist of a sidewalk scupper or catch basin.

DEPRESSED CURB: This drainage mechanism occurs when the curb is transitioned from full height curb to flowline of the road or parking area and can also be referred to as a curb cut for the purpose of taking flow off pavement without the use of a curb inlet.

DRY WELL: An engineered hole with a grated inlet designed to dispose of floodwaters through a process of passive infiltration of floodwaters into the vadose zone (ie., the unsaturated sediments commonly found above the water table).

DSSRR: Abbreviation for Design Standards for Stormwater Detention and Retention (2015)

EROSION HAZARD SETBACK (EHS): The minimum horizontal distance from the top of bank or the floodplain limit, whichever is closest to the centerline of the primary channel or outside channels in a multiple channel watercourse.

FEDERAL HIGHWAY ASSOCIATION (FHWA) HYDRAULIC TOOLBOX: A stand-alone suite of calculators that perform routine hydrologic and hydraulic analysis and design computations.

FEMA: Abbreviation for Federal Emergency Management Agency

FLO-2D: A two-dimensional flood routing model that can simulate rainfall-runoff.

HEC-1: A watershed computer program designed to simulate the surface runoff response of a river basin to precipitation by representing the basin as an interconnected system of hydrologic and hydraulic components.

HEC-HMS: Hydrologic modeling system designed to simulate the complete hydrologic processes of dendritic watershed systems.

HEC-RAS: One- and two-dimensional simulation software used in computation fluid dynamics to model the hydraulics of water flow through natural rivers and other channels.

HYDRAULIC DESIGN SERIES (HDS) NO. 5 (2012): Combines culvert design information previously contained in Hydraulic Engineering Circulars (HEC) No. 5, No. 10, and No. 13 with hydrologic, storage routing, and special culvert design information.

HYDRAULIC GRADE LINE (HGL): A line which represents the static head plus pressure head of flowing water.

INFILTRATION TRENCH: Rock-filled trench designed for the purpose of temporarily storing runoff, and then subsequently disposing of runoff within the sub-surface through the process of infiltration.

LETTER OF MAP CHANGE (LOMC): A general term used to refer to the several types of revisions and amendments to FEMA maps that can be accomplished by letter.

LETTER OF MAP REVISION (LOMR): FEMA's official modification to an effective Flood Insurance Rate Map (FIRM). LOMRs can result in a physical change to the existing regulatory floodway, the effective Base Flood Elevations (BFEs), or the Special Flood Hazard Area (SFHA).

LOW FLOW BRIDGE: a bridge which does not convey the standard 1% chance flow event with freeboard and typically has overtopping during the less frequent events. Low Flow Bridges may not meet the all-weather access requirement.

MS4 GENERAL PERMIT: Authorizes the discharge of storm water from small Municipal

Separate Storm Sewer Systems provided that the permittee complies with the requirements set forth in the permit.

NATURAL: Undeveloped; existed, undisturbed, prior to development of the site.

NORTHWEST POLICY: Town policy that describes the requirements for total site retention and allowances for drywells within the Northwest Policy area shown on Figure 1.

PC-HYDRO: The official modeling method for hydrology to be submitted to Pima County Regional Flood Control District and accepted by the Town of Marana.

RECLAIMED SLOPE: Restored slope after excavation.

RETENTION BASIN: A facility which stores surface runoff but is not provided with a positive outlet. No flow is discharged directly into a downstream watercourse from a retention basin but may be drained into the subsurface by infiltration.

RILLING: A pattern of narrow, vertical troughs formed in relatively steep earthen embankments by floodwaters cascading down the embankment.

SETBACK: The minimum horizontal distance between a structure and a channel, stream, wash, watercourse, or detention basin. A channel setback is measured from the top edge of the highest channel bank or from the edge of the 100-year water-surface elevation, whichever is closer to the channel centerline.

SINGLE SUBDIVISION: A subdivision which is comprised of lots that is platted without a master plan and can be built without regional/spine infrastructure.

TAFS: Abbreviation for the Tortolita Alluvial Fan Study

TECH-015: Pima County Regional Flood Control District Technical Policy describing acceptable methods for determining peak discharges.

TECH-033: Pima County Regional Flood Control District Technical Policy describing the criteria for two-dimensional modeling.

TOWN: Town of Marana, a political subdivision of the State of Arizona.

UNCONSOLIDATED SHEET FLOW: Shallow, unconcentrated and irregular flow along a plane.

FIGURES

MARANA AZ

Town of Marana
11555 West Civic Center Drive
Marana, Arizona 85653
(520) 382-1999

N
W E
S

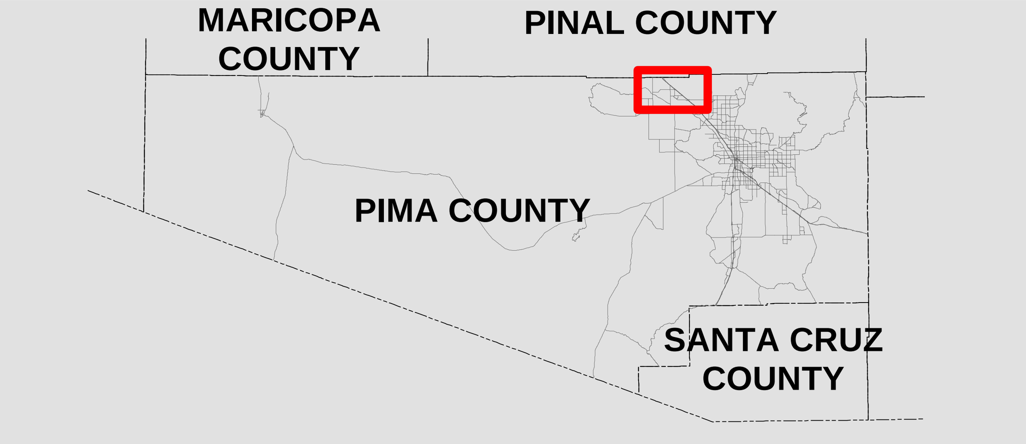
AUGUST 2024

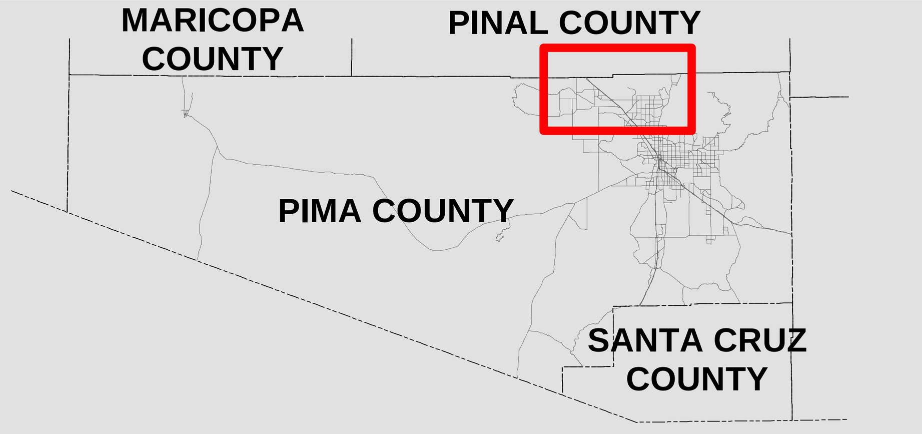
FIGURE 1, NW POLICY AREA MAP

TOWN OF MARANA DRAINAGE MANUAL

0 0.5 1 2 3 Miles

1 Inch = 0.5 Mile





MARANA AZ

Town of Marana
11555 West Civic Center Drive
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(520) 382-1999

N
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August 2024

FIGURE 2, TAFS AREA

TOWN OF MARANA DRAINAGE MANUAL

0 1 2 4 6 Miles

1 Inch = 1 Mile

MARANA AZ

Town of Marana
11555 West Civic Center Drive
Marana, Arizona 85653
(520) 382-1999

N
W E
S

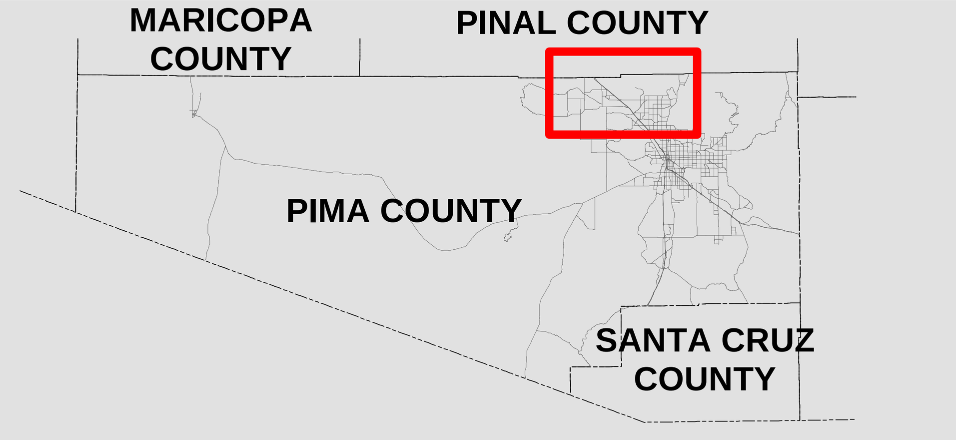
August 2024

FIGURE 3, BALANCED & CRITICAL BASINS MAP

TOWN OF MARANA DRAINAGE MANUAL

0 1 2 4 6 Miles

1 Inch = 1 Mile



APPENDIX A

Report Formats/Maintenance Checklist

Drainage Statement Outline

- Project Description
- Project Location
- Prior Studies
- Floodplain Status
    - Local Regulatory
    - Federal
- Methodology/Parameter Selection
- Hydrology & Hydraulics
    - Offsite Drainage
    - Onsite Existing Conditions
    - Onsite Proposed Conditions
- Erosion Mitigation
- Sediment Deposition Mitigation
- Clean Water Act Compliance
- Conclusion

Drainage Report Outline

- Cover
- Table of Contents
- Project Description
- Project Location
- Prior Studies
- Floodplain Status
  - Local Regulatory
  - Federal
- Methodology/Parameter Selection
- Hydrology & Hydraulics
  - Offsite Drainage
  - Onsite Existing Conditions
  - Onsite Proposed Conditions
- Detention/Retention
- Erosion Mitigation
- Sediment Deposition Mitigation
- Clean Water Act Compliance
- Conclusion

Master Drainage Report Outline

- Cover
- Table of Contents
- Master Plan Description
- Master Plan Location
- Prior Studies
- Floodplain Status
  - Local Regulatory
  - Federal
- Methodology/Parameter Selection
- Hydrology & Hydraulics
  - Offsite Drainage
  - Onsite Existing Conditions
- Master Drainage Concept
  - Block Drainage Plan
  - Regional Detention/Retention
  - Spine Drainage Infrastructure
  - Erosion Mitigation
  - Sediment Deposition Mitigation
- Phasing/Interim Drainage Requirements
- Clean Water Act Compliance
- Conclusion

Private Drainage Improvement Inspection and Maintenance Checklist

Date:		Project Name/Location:
Inspector:	Title:	Affiliation:
Type of Inspection: ☐ Annual ☐ After a Significant Storm Event		

General Requirements

- Improvements shall be maintained to perform as designed for the life of the project and shall not be converted to a different use without a Floodplain Use Permit. A Floodplain Use Permit is not required for maintenance activities.
- Improvements shall be inspected annually and after significant storm events.
- The purpose of the inspection is to evaluate whether as-built characteristics are maintained.

Component	Inspection Item	Requires Maintenance	If maintenance is required, describe corrective action
Inlet	As-built grades and elevations	☐	
	Presence of obstructions	☐	
	Evidence of material damage	☐	
Outlet	As-built grades and elevations	☐	
	Presence of obstructions	☐	
	Evidence of material damage	☐	
Earthen Slopes	As-built grades and elevations	☐	
	Invasive non-native plants	☐	
	Slope treatment	☐	
Retaining walls	As-built grades and elevations	☐	
	Presence of damage or instability	☐	
	Drainage function	☐	
Depth	As-built grades and elevations	☐	
	Sediment accumulation >10% of design volume	☐	
Basin/Channel Floor	As-built grades and elevations	☐	
	Presence of ponding	☐	
	Evidence of oil, grease, chemicals or trash	☐	
	Presence of invasive non-native plants	☐	

Private Drainage Improvement Inspection and Maintenance Checklist (Continued)

Date:	Project Name/Location:

Component	Inspection Item	Requires Maintenance	If maintenance is required, describe corrective action
Perimeter Wall	As-built grades and elevations	☐	
	Presence of damage or instability	☐	
	Drainage function	☐	
Security Barrier	Presence of damage or instability	☐	
Access	Presence of obstruction	☐	
Landscaping	Presence of overgrown vegetation	☐	
	Presence of invasive non-native plants	☐	
	Damage to basin due to landscape elements	☐	
Pump	Alarm System	☐	
	Presence of obstruction	☐	
	As-built specifications	☐	
Dry/Rock Well	Presence of sediment and/or standing water for more than 36-hours	☐	
Armored Slopes	Cracking	☐	
	Sloughing	☐	
	As-built grades and elevations	☐	
Toe Down	Exposed	☐	
	Buried/not visible	☐	
Sediment Deposition	As-built grades and elevations	☐	
	Aggradation ongoing	☐	
	Only at specific locations	☐	

APPENDIX B

Excerpts from Other Manuals

On watersheds larger than one square mile, the guidelines cited above may result in overdesign. The design of sediment basins on these watersheds is a more complicated procedure, involving total watershed sediment yield and channel sediment-transport capacity over a range of discharges. Total watershed sediment yield can be estimated by such methods as the Modified Universal Soil Loss Equation (Williams, 1975; and Williams and Berndt, 1977), the Pacific Southwest Inter-Agency Committee (PSIAC) Method (Pacific Southwest Inter-Agency Committee, 1968), the Flaxman Method (Flaxman, 1972), the SCS Method (U.S. Soil Conservation Service, 1971), the Dendy/Bolton Method (Dendy and Bolton, 1976), and the Renard Method (Renard, 1972). A publication by Renard and Stone (1981) contains a detailed discussion and comparison of some of these methods.

The equations for watershed sediment yield which are listed above do not readily distinguish between sediment production that would be classified as wash load and sediment production that would be classified as bed load. Wash load particles are so small that they would generally remain in suspension as the water passes through the detention basin. Therefore, the wash load is not generally to be considered in sediment basin design. An estimate of wash load, as compared to bed load estimated from equations for total watershed sediment yield, can be made by taking samples of the topsoil throughout the watershed.

Total watershed sediment production may not be an entirely accurate estimate of the amount of sediment that would be delivered to a certain point, because there is sediment storage within the watershed system. Sediment-volume estimates must therefore also consider the sediment-transport capacity of the channel. A detailed discussion of this type of analysis will not be presented here. However, the reader is referred to publications by the U.S. Army Corps of Engineers (1977), Simons, Li & Associates (1982, 1985), the American Society of Civil Engineers (1977), Simons and Senturk (1977), and Zeller and Fullerton (1983) for more detailed information about performing such analyses.

6.9 Equilibrium Slopes within Constructed Channels

Given a fixed size distribution of sediments, the sediment-transport capacity of a stream is dependent primarily upon flow velocity and depth. Within the City of Tucson, transport of all particle sizes of bed material increases, as flow velocity increases, at a rate proportional to approximately the third to fifth power of the velocity. Correspondingly, transport of sediment particles composed of bed material generally decreases as depth increases, while transport increases with decreased depth. However, flow velocity is by far the more important variable.

For purposes of analysis and design, most natural, undisturbed channels in the Tucson area can be assumed to be at or near a state of dynamic equilibrium with regard to sediment transport. This means that, for a given reach of the channel, the sediment-transport capacity of the channel, over the long term, is more or less equal to the sediment supply. The channel bed slope is therefore "stable."

When channelization occurs, the channel top width is often narrowed, and channel roughness is normally decreased. The result is an increase in velocity and depth, with a corresponding increase in sediment-transport capacity. Sediment-transport capacity then exceeds the sediment supply; and, if the bed is composed of sediment that can be transported, the deficiency will be made up from bed material--causing the channel to degrade. Another factor that contributes to this degradation is upstream urbanization. Urbanization increases flood peaks, which also lead to higher flow velocities and depths. Urbanization also reduces the watershed sediment supply, and increases the frequency of runoff. The result of all these occurrences is that channel bed degradation will occur until the channel slope is flat enough to cause the sediment-transport rate to be equal to the incoming sediment supply. This slope then becomes the new, "stable," equilibrium slope. Streambed degradation can threaten underground improvements, bank-protection toe-downs, culverts, and other hydraulic structures that are within and/or that cross the channel. Grade-control structures, or lining of the channel bed, are usually required in order to prevent damage caused by streambed degradation.

The equilibrium slope for a channel which has an upstream sediment supply that is considered to be essentially zero (e.g., a channel located within a highly urbanized watershed) can be computed from:

$$S_{eq} = \left(\frac{1.45n}{q^{0.11}} \right)^2 \qquad \textbf{(6.25)}$$

Where:

S_{eq} = Equilibrium slope after urbanization, in feet per foot;
n = Manning's roughness coefficient; and,
q = Channel unit discharge, in cubic feet per second per foot.

For use with Equation 6.25, channel unit discharge is defined as the channel discharge divided by the channel bottom width. Use of this equation will produce the flattest slope that can be reasonably expected to transport sediment within channels located in the Tucson area. The discharge associated with a 10-year flood is normally chosen when computing the unit discharge for use in Equation 6.25.

For lesser degrees of urbanization, the equilibrium slope is computed from Equation 6.26, which is a generalization of the theoretically derived sediment-transport relationships for sandbed channels developed by Zeller and Fullerton (1983):

$$S_{eq} = \left\{ \left[\frac{n_u}{n_n} \right]^2 \left[\frac{Q_{u,10}}{Q_{n,10}} \right]^{-1.1} \left[\frac{b_u}{b_n} \right]^{0.4} (1-R_s)^{0.7} \right\} S_n \qquad \textbf{(6.26)}$$

Where:

n_u = Manning's roughness coefficient for an urban channel;
n_n = Manning's roughness coefficient for a natural or existing channel;
$Q_{u,10}$ = Ten-year discharge, under urbanized conditions, in cubic feet per second;
$Q_{n,10}$ = Ten-year or bank-full discharge (whichever is less), under natural conditions, in cubic feet per second;
b_u = Bottom width of channel, under urbanized conditions, in feet;
b_n = Bottom width of channel, under natural conditions, in feet;
R_s = Reduction factor for sediment supply. This factor is usually assumed to be equal to the ratio of the impervious area to the total area of the upstream watershed (i.e., $0.0 \leq R_s \leq 1.0$); and,
S_n = Natural or existing channel slope, in feet per foot.

The roughness coefficients for natural and urbanized channel beds are often very nearly the same, so the term in which these coefficients appear in Equation 6.26 can usually be assumed equal to the value 1.0. However, from time to time exceptions to this assumption may occur. For instance, when the existing channel is a wide, flat, sheetflow watercourse; and the proposed channel is a narrow, sand-bed channel, n_u will ordinarily not be equal to n_n.

For moderately urbanized to highly urbanized watersheds, the equilibrium slope should be computed by using both Equation 6.25 and Equation 6.26. The steeper of the two computed slopes should then be used for design. The reason for this is that Equation 6.26 can sometimes produce slope values that are too flat to generate reasonable sediment-transport rates for maintenance of channel stability, when impervious cover within a watershed is very high.

Equation 6.26 should be used with caution within the City of Tucson. An underlying assumption of this equation is that the existing or natural channel is itself in equilibrium. This is not always true in the City, because most channels have undergone alteration. If there is any question as to whether or not the existing channel is in equilibrium, it is best to try and determine through old (pre-development) aerial photographs and topography what the channel characteristics were in its original, undisturbed (i.e., natural) state. In the absence of historical information about the original channel, an examination may be made of existing stable channels in the area to help estimate what the channel in question may have looked like before urbanization.

Equation 6.26 can be used for more than merely the quantification of streambed degradation. It can also be used to determine whether aggradation will occur when a channel is widened beyond existing or natural conditions. Another application would be to use it to design a stable channel cross-section in lieu of installing grade-control structures to otherwise control degradation of the channel bed.

6.10 Spacing and Depth of Grade-Control Structures

If the equilibrium slope of a channel, as determined by use of either Equation 6.25 or Equation 6.26, is flatter than the design slope, grade-control structures may be needed to limit degradation from exceeding a certain depth at any point along the channel. Grade-control structures, sometimes called "cut-off walls" or "check dams," are non-erodible vertical barriers in the channel that prevent the channel bed from degrading at a point located immediately upstream of where they are located. After the channel bed has reached equilibrium, the bed elevation immediately upstream of the grade-control structure is at the design elevation. Downstream of the grade-control structure, the bed is at an "equilibrium" elevation that is lower than the design elevation. For most channels, the design of grade-control structures is an iterative process, involving drop height, reach length, and depth of scour downstream of the drop.

Once a drop height is chosen, the reach length, or spacing, between adjacent structures can be computed from:

$$L_r = \frac{h}{S_{ib} - S_{eq}} \tag{6.27}$$

Where:

L_r = Reach length, or spacing, between adjacent grade-control structures, in feet;
h = Drop height downstream of the grade-control structure, in feet;
S_{ib} = Initial channel bed slope, in feet per foot; and,
S_{eq} = Channelized equilibrium bed slope, in feet per foot.

If the initial and final bed slopes are approximately the same, the distance between grade-control structures will be very large. Under these circumstances, such structures may not be required.

Normally, the drop height downstream of a grade-control structure which consists of poured concrete without reinforcements shall not exceed two feet; and preferably should be only one foot, where feasible. For economical and technical reasons, grade-control structures should be spaced no closer together than twelve times the local scour depth below the grade-control structures, as computed by the use of either Equation 6.13 or Equation 6.14.

The total height of a cut-off wall or a grade-control structure (D_{cw}), from top to toe, shall not be less than the drop height plus the computed depth of scour below the wall or structure (see Figure 6.6). The depth of scour below grade-control structures should be computed according to the guidelines presented in Section 6.6.6 of this Manual. For a one-foot-wide, unreinforced concrete cut-off wall, if structural calculations support same, the maximum allowable height of a cut-off wall, from top to toe, can be six feet. If the depth of scour plus the drop height is greater than six feet, the drop shall be considered to be too great for unreinforced concrete cut-off walls, unless a structural analysis can demonstrate otherwise, and the spacing between the cut-off walls must be reduced. The example which follows (i.e., Example 6.1), illustrates the recommended procedure for cut-off wall design.

There will be many design situations, especially when unit discharges are high, where a cut-off wall with a hegith of six feet, from top to toe, is not sufficient. In such cases, a reinforced concrete cut-off wall that has a height greater than six feet, from top to toe, may be used, provided that a structural analysis is submitted showing that the proposed cut-off wall will be structurally stable. If a structural analysis is submitted and approved, the maximum drop height of two feet will no longer apply.

Grade-control structures for large discharges need not necessarily be vertical on the downstream side. For structural stability, a triangular or wedge-shaped soil-cement grade-control structure is recommended for use on regional watercourses. However, for hydraulic reasons, the use of any grade-control structure with a face flatter than 1:1 on the downstream side shall not be permitted without prior written approval from the City Engineer.

EXAMPLE 6.1: SPACING AND DEPTH OF GRADE-CONTROL STRUCTURES

A channel in a highly urbanized watershed is to be built to contain the 100-year-flood discharge. The sides of the channel are to be of shotcrete, the bottom of earth.

Channel characteristics are as follows:

Bottom Width	= 20 feet
Design Slope	= 0.006 feet/foot
Side Slopes	= 1:1
Manning's "*n*"	= 0.022

Hydraulic characteristics are as follows:

Q_{100} = 700 cfs	Q_{10} = 350 cfs
Y_{100} = 3.1 feet	Y_{10} = 2.1 feet
V_{100} = 9.7 fps	V_{10} = 7.7 fps
q_{100} = 35.0 cfs/foot	q_{10} = 17.5 cfs/foot

Because the watershed is highly urbanized, Equation 6.25 will be used to compute the equilibrium slope. Therefore:

$$S_{eq} = \left[\frac{1.45\ (0.022)}{(17.5)^{0.11}} \right]^2 = 0.0005 \text{ feet/foot.}$$

Assume a two-foot drop height. From Equation 6.27, the spacing between grade-control structures should be:

$$L_r = \frac{2.0}{(0.006)-(0.0005)} = 364 \text{ feet.}$$

The grade-control structure will be submerged. Using Equation 6.14 yields:

$$Z_{lss} = 0.581(q_{100})^{0.667}(h/Y_{100})^{0.411}[1-(h/Y_{100})]^{-0.118};$$

$$q_{100} = 35.0 \text{ cfs;}$$

$$Z_{lss} = 0.581\ (35.0)^{0.667}\ (0.645)^{0.411}\ (0.355)^{-0.118}; \text{ so,}$$

$$Z_{lss} = 5.9 \text{ feet.}$$

Therefore, the total height of the grade-control structure, from top to toe, should be 5.9 feet *plus* the two-foot drop height; or, 7.9 feet (round to 8.0 feet).

However, it is desirable to keep the total vertical dimension of the grade-control structure, from top to toe, equal to or less than six feet. Therefore, a smaller drop height should be used.

Using a drop height of one foot yields:

$$L_r = \frac{1.0}{(0.006)-(0.0005)} = 182 \text{ feet.}$$

$$Z_{lss} = 0.581\ (35.0)^{0.667}\ (0.323)^{0.411}(0.677)^{-0.118}; \text{ so,}$$

$$Z_{lss} = 4.10 \text{ feet (round to 4.0 ft).}$$

Since, in this example, the ultimate drop height at the downstream side of a grade-control structure will be set at one foot, cut-off walls with a height of five feet, from top to toe, could be placed at approximately 180-foot intervals along the bottom of the channel to serve as grade-control structures in order to limit long-term bed degradation to a maximum of one foot anywhere along the subject channel.

CHAPTER VI: EROSION AND SEDIMENTATION

6.1 Introduction

The hydrology and hydraulics of floodwaters are not the only concern of floodplain-management administrators and/or drainage design engineers who work in arid or semi-arid environments which contain alluvial rivers such as those that exist both within and around the City of Tucson, Arizona. The transport of sediment by floodwaters is also a major concern because of the potential for rapid bank erosion and changes in channel bed elevations. Bank erosion can often be so severe that it causes much more damage than inundation by floodwaters. Aggradation or degradation of the channel bed can rapidly change flood limits, or cause bank protection and other channel improvements to fail over a very short period of time.

The study of fluvial geomorphology and the analysis of sediment transport are usually undertaken in an attempt to quantify the broad effects of erosion and sedimentation and the impacts of sediment-transport capacity upon channel morphology. Sediment-transport analysis is a relatively specialized field of study. Predictions based upon its application are often expensive to produce, and can be highly variable in nature. Therefore, as an aid to the user, this chapter of the Manual presents some design and predictive guidelines that can be used within the City of Tucson in the absence of a more detailed sediment-transport analysis.

6.2 Purpose

The purpose of this chapter is to provide guidelines for the estimation of erosion, sedimentation, and channel bed scour when designing drainage channels and hydraulic structures which are to be located within the City of Tucson. These design guidelines and procedures are to be used when normal design situations are encountered. Deviations from these guidelines may occur, provided that the user has experience in sediment-transport technology; and provided that the deviation is technically justified, through detailed sediment-transport analysis, to the satisfaction of the City Engineer.

6.3 Fluvial Geomorphology

The study of fluvial geomorphology normally involves analyses which encompass entire drainage systems. This is so because the response of an individual channel to change within a watershed can often have an effect upon the entire drainage system. Conversely, the fluvial system, as a whole, will ultimately dictate the response of an individual channel to overall change within a watershed. Rarely is it possible to understand the fluvial processes which occur within even a short reach of an alluvial channel in isolation from its upstream and downstream system controls.

The fluvial system is generally divided into three zones (Schumm, 1977). Zone 1 is characterized as the drainage basin, watershed, or source area for sediment. This is the area from which water and sediment are derived. Storage of sediment is not significant in this zone. Zone 2 is characterized as the transport zone; where, for a stable channel, sediment input can equal sediment output. For those reaches where the sediment-transport capacity exceeds the upstream supply, it can be assumed that the sediment deficit will be made up out of the channel bed or banks. Channel bed degradation or erosion of channel banks will be the result. Zone 3 is characterized as the sediment sink or area of deposition.

Obviously, the division between these three zones is indiscrete. Each zone has characteristics of the other two, which are subordinate to the primary characteristic of the zone. Zone 2 is of major concern to the hydraulic and river-control engineer, and to geomorphologists concerned primarily with river-channel morphology. It is this zone with which this chapter deals.

6.3.1 Channel Morphology

Sediment and water moving through alluvial channels are the independent variables that determine the size, shape, and pattern of the channel. Numerous empirical relations have been developed that relate channel morphology to water and sediment discharge.

6.3.1.1 Hydraulic Geometry of Alluvial Channels

As a general rule, the greater the quantity of water that moves through a channel, the larger is the cross-sectional area of that channel. Preceded by numerous studies of canal morphology and stability, Leliavsky (1955) and Leopold and Maddock (1953) demonstrated that, for most rivers, the water surface width, T, and depth, Y, increase with mean-annual discharge, Q_a, in a downstream direction such that:

$$T = k_1 Q_a^b \tag{6.1}$$

and,

$$Y = k_2 Q_a^c \tag{6.2}$$

Both the coefficients and exponents of Equations 6.1 and 6.2 (i.e., the "k_1," "k_2," "b," and "c" values) are different for each river and, when data from a number of rivers are plotted against discharge, the scatter covers an entire log cycle. For a given discharge, there is an order of magnitude range of width and depth. Therefore, other variables apparently influence the hydraulic geometry of channels as well.

6.3.1.2 Influence of Sediment Load

A primary variable which significantly controls river morphology is sediment load. Bed-material load is defined as that part of the stream's sediment load that consists of sediment sizes comprising a significant part of the streambed. The other component of total sediment load is wash load, which is part of the total load not significantly represented in the bed. In and around the Tucson area, wash load is generally composed of sediments smaller than sand (i.e., smaller than about 0.06 mm to 0.07 mm). Wash load is held in suspension by the turbulence of the flowing water, and therefore is transported at the same velocity as the water. Bed-material load is composed of sands and larger sediments, and therefore is generally transported at an average velocity less than the velocity of flowing water.

From an analysis of data from regime canals, Lacey (1930) concluded that the wetted perimeter of a channel is directly dependent upon discharge; but that channel shape reflects sediment size. It is also generally recognized that coarse sediment produces channels with high width/depth ratios, while fine sediment produces channels with narrow and deep cross sections.

In addition to the size of the transported sediment, relative amounts of bed-material load and wash load significantly influence the morphology of sand-bed streams. Large bed-material loads are associated with wide channels, and large wash loads are associated with narrow widths.

The type of sediment load is considered to be a more important control on stable channel shape than the total quantity of sediment transported through a channel. For example, in one channel a certain quantity of bed-material load may exert the dominant control if it is the total load, whereas in another channel the same amount of bed-material load may exert much less influence on channel shape because it is only a small part of the total sediment load (i.e., wash load and bed-material load). Therefore, when load and discharge are constant, an increase in the quantity of bed-material load will cause an increase in channel width, and a corresponding increase in the width/depth ratio. This phenomenon is probably related to the high gradient and velocity of flow generally associated with large bed-material loads.

In summary, for alluvial channels which occur in the Tucson area, the type and amount of sediment load exerts a major control on their shape. Therefore, for a single channel under the ideal assumption of a constant discharge and a fixed amount of wash load, a change in bed-material load would be reflected by a change in both the shape and gradient of the channel.

6.4 Sediment-Transport Theory

Sediment particles are transported by flowing water in one or more of the following ways: (1) surface creep, (2) saltation, and (3) suspension. Surface creep is the rolling or sliding of particles along the bed. Saltation (jumping) is the cycle of motion above the bed, with resting periods on the bed. Suspension involves the sediment particle being supported by the water during its entire motion. Sediments transported by surface creep, sliding, rolling, and saltation are referred to as bed load, and those transported by suspension are called suspended load. The suspended load consists of sands, silts, and clays. Total sediment load is defined as the sum of the bed load and suspended load. Generally, the amount of bed load transported by a large river is on the order of five to twenty-five percent of the suspended load. Although the amount of bed load may be relatively small compared with total sediment load, it is important because it shapes the bed, influences channel stability, determines the form of bed roughness, and affects various other hydraulic factors as well.

As presented earlier, the total sediment load in a channel can be more simply defined as the sum of bed-material load and wash load; where the bed-material load is the sum of bed load and suspended bed-material load, representing that part of the total sediment discharge which is composed of grain sizes found in the bed; and the wash load is that part of the sediment discharge which is composed of particle sizes finer than those found in appreciable quantities in the bed (Simons and Senturk, 1977). The presence of wash load can increase bank stability, reduce seepage, and increase bed-material transport. Wash load can be easily transported in large quantities by the stream, but is usually limited by availability from the watershed. The bed-material load is more difficult for the stream to move, and is normally limited in quantity by the transport capacity of the channel. Figure 6.1 summarizes the various definitions of the components of sediment load, and their contribution to total sediment load.

There is no clear size distinction between wash load and bed-material load. As a rule of thumb for the Tucson area, it should be assumed that the size of bed-material particles is equal to or larger than 0.0625 mm, which is the division point between sand and silt. The sediment load consisting of grains smaller than this size is then considered as wash load. It is generally assumed that most of the wash load is transported through the system by stream flow, and that little wash load is deposited on or in the stream bed. Wash load deposited with coarse material is usually only a very small fraction of the total bed material within the channel.

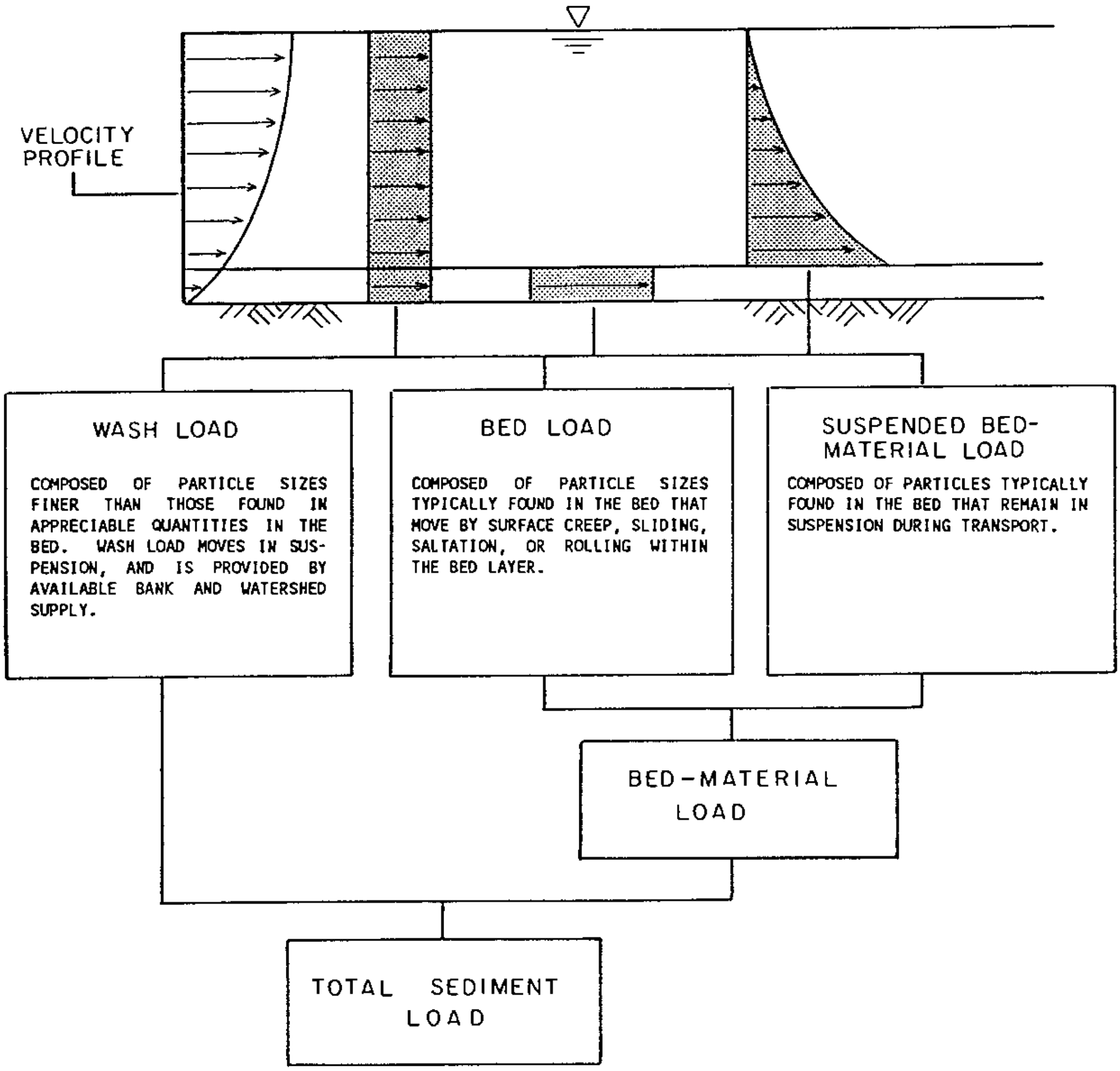
The amount of material transported, eroded, or deposited in an alluvial channel is a function of both the sediment supply and the sediment-transport capacity of the channel. Sediment supply includes the quality and quantity of sediment brought to a given reach. Sediment-transport capacity is a function of the size of bed material, flow rate, and geometric and hydraulic properties of the channel. Generally, the single most important factor determining sediment-transport capacity is flow velocity. Additionally, since sediment-transport capacity is generally proportional to the third to fifth power of the velocity, small changes in velocity can cause large changes in sediment-transport capacity (Simons, Li & Associates, 1982, 1985). Either the sediment supply or sediment-transport capacity may limit the actual sediment-transport rate in a given reach.

6.5 Sediment Routing

Supported by qualitative and quantitative analysis, a detailed evaluation of the fluvial-system response can be made based upon mathematical-modeling concepts. A mathematical model is simply a quantitative expression of the physical processes. The mathematical processes governing watershed and river responses are complicated. Computer programs can provide a means of assessing the many parameters of these complex processes within a fluvial system. There are several computer models available which are applicable to this region. For information on where to obtain these models, the user should contact the City Engineer.

6.5.1 Simplified Sediment Modeling

After evaluating the hydraulic conditions of the river by water-routing programs such as the U.S. Army Corps of Engineers HEC-2 program, the sediment-transporting capacity can be established. Sediment-transport equations are used to determine the sediment-transport capacity for a specific set of flow conditions. Different transport capacities can be expected for different sediment sizes. For each sediment size, the transport rate includes the transport rate of the bed load and the transport rate of the suspended bed-material load.



NOTE:

THE TERM "SUSPENDED LOAD" IS USED WHEN REFERRING TO THE SUM OF THE "WASH LOAD" AND "SUSPENDED BED-MATERIAL LOAD" COMPONENTS. THEREFORE, AN ALTERNATE DEFINITION OF TOTAL SEDIMENT LOAD IS THE SUM OF THE SUSPENDED LOAD AND BED LOAD.

FIGURE 6.1
DEFINITION OF SEDIMENT-LOAD COMPONENTS

One modeling method uses hydraulic conditions from a rigid-boundary model such as HEC-2, or an equivalent program, and computes sediment transport based upon the Meyer-Peter, Muller bed-load equation and the Einstein suspended-load procedure for each sediment size found in the bed. The data required are the same as for HEC-2 (channel geometry, resistance, bridge constriction, etc.). Also needed are the size distribution of the bed-material and the upstream sediment supply. Using the generated hydraulic conditions, the transport capacity for each sediment size at each cross section is then determined.

Actual transport rates depend upon transport capacities as well as supply rates. The change in transport capacity between two cross sections can be used to estimate aggradation or degradation, based upon availability. For example, if sediment is in ample supply to meet the transport capacity at an upstream cross section but at the next cross section downstream the transport capacity is only one-half as much, then the other one-half of the sediment passing the upstream cross section must be deposited between the upper and lower cross sections. This comparison of transport capacities continues reach by reach and size fraction by size fraction through the entire stream segment. The drawback to this simplified approach is that the hydraulic conditions are not readjusted, due to aggradation or degradation, at frequent time increments during the passage of the flood hydrograph. However, this technique does provide "trends" in bed-elevation changes without using more complex techniques.

6.5.2 Quasi-Dynamic Sediment Modeling

The sediment-routing model previously discussed is based upon a gradually-varied-flow backwater program which assumes a rigid-boundary system. This methodology can be extended to account for unsteady flow and alluvial-channel boundaries without going to a fully unsteady water and sediment-routing model.

The quasi-dynamic sediment model uses the same gradually-varied-flow backwater program for hydraulic computations. However, the flow is assumed constant for a given time increment Δt. A flow event, either short-term or long-term, can be broken into a number of time increments, each with a different flow rate, but during each increment the flow is considered steady.

To account for a non-rigid or alluvial boundary, when a predetermined volume of sediment is either deposited on or eroded from the streambed, the cross section is recomputed in the following manner.

Sediment aggradation or degradation within a reach for a given time period is ΔV_s = (sediment supply - sediment transport) x BF, where ΔV_s is the change in sediment volume in the reach and BF is a bulking factor. The change in sediment volume is assumed to be uniformly distributed throughout the reach. Change in area for each cross section is determined by a weighting factor based upon the conveyance in adjacent segments of the cross sections. The changes in elevation are used to generate a new HEC-2 data file for the next time period. Therefore, during any given time period the channel boundary is assumed to be rigid and the HEC-2 analysis is assumed to be valid. After evaluating the hydraulic conditions and the sediment-transport capacity, the channel boundary is modified to reflect the aggradation/

degradation changes occurring throughout the river, and to establish the new channel configuration for the next time step.

This methodology has been successfully applied to a number of practical engineering problems. It provides a feasible and relatively cost-effective approach to design problems in alluvial rivers.

6.5.3 Dynamic Mathematical Modeling

Dynamic mathematical modeling of water and sediment routing is the next level of sophistication and complexity in determining alluvial-channel changes. It involves unsteady, non-uniform flow routing for determining the hydraulic conditions to be used to calculate sediment transport, aggradation, and degradation.

Unsteady, non-uniform flow routing solves equations governing the motion of water in open channels. These equations are mathematical descriptions of the physical phenomena. The two basic principles for water routing are continuity and momentum. Continuity states that water coming into a reach is either stored in the reach or passes downstream without gaining or losing water.

The momentum principle balances the forces and accelerations acting on flowing water. Generally, the continuity and momentum equations, along with a resistance to flow equation involving Manning's n or Chezy's C, are solved numerically in finite-difference form. The results are the hydraulic variables of velocity, depth, and width for unsteady, non-uniform flow. These are then used to route sediment. Sediment movement is controlled by the shear forces acting on the bed, transport capacity of the flow, and both availability and supply. Equations used in these calculations are described in most sedimentation textbooks. To compute aggradation and degradation, the sediment-continuity equation is used.

While dynamic mathematical modeling can give excellent results, it is very complex. Fortunately, it is not often required to solve many of the more straightforward, practical problems that designers will usually encounter within the Tucson area. In fact, most aggradation and degradation problems can be solved to an acceptable degree of accuracy by the several methods previously described within this chapter of the Manual.

6.6 Depth of Scour

Scour, or lowering of a channel bed (excluding long-term aggradation/degradation), can be caused by discontinuity in the sediment-transport capacity of the flow during a runoff event (general scour); the formation of anti-dunes in the channel bed during a runoff event; transverse currents within the flow through a bend (bend scour) during a runoff event; local disturbances, such as abutments or bridge piers, during a runoff event; and the formation of a low-flow channel thalweg. The design depth of scour (*excluding* long-term aggradation/degradation, which must be added for toe-down design) is the sum of all these individual scour components, and can be expressed by:

$$Z_t = 1.3\ (Z_{gs} + 1/2Z_a + Z_{ls} + Z_{bs} + Z_{lft}) \tag{6.3}$$

Where:

Z_t	=	Design scour depth, *excluding* long-term aggradation/degradation, in feet;
Z_{gs}	=	General scour depth, in feet;
Z_a	=	Anti-dune trough depth, in feet;
Z_{ls}	=	Local scour depth, in feet;
Z_{bs}	=	Bend scour depth, in feet;
Z_{lft}	=	Low-flow thalweg depth, in feet; and,
1.3	=	Factor of safety to account for nonuniform flow distribution.

The various equations for depth of scour which are to follow were developed strictly for use in conjunction with sand-bed channels in which the bed material is erodible to the depth specified by the applicable equations. However, this situation does not always exist in channels located within the City of Tucson. In some areas of the city, the channel has degraded to a point where the exposed bed is no longer composed of strictly unconsolidated alluvial material, but rather of consolidated hard-pan or caliche. Channel beds composed of this type of material are not freely erodible, and thus the scour equations which follow may not strictly apply. Should such conditions be encountered, a geotechnical investigation should be submitted by an Arizona Registered Professional Civil Engineer to justify the use of a lesser scour depth than would be determined from the use of Equation 6.3.

6.6.1 General Scour

As previously discussed in Section 6.5 of this Manual, the depth of general scour is best estimated by performing a detailed sediment-transport analysis using the bed grain-size distribution, hydraulic conditions, sediment-transport capacity at different stages throughout the flow event, changes in bed levels throughout the event, and the sediment supply into the reach being studied. An analysis to this level of detail is beyond the scope of this Manual. However, there are several computer models commercially available to aid in making an estimate of general scour. Unfortunately, these models are very sensitive to input, and the results are best interpreted by someone with extensive experience in the field of sediment transport. A detailed discussion of sediment-transport analysis for computing general scour can be found in "Engineering Analysis of Fluvial Systems" (Simons, Li & Associates, 1982), and "Arizona Department of Water Resources Design Manual for Engineering Analysis of Fluvial Systems" (Simons, Li & Associates, 1985).

General scour on regional watercourses should be estimated by undertaking a detailed sediment-transport study, as described above, when and where it is feasible to do so. However, such a study is not usually practical on smaller watercourses. Therefore, as an alternative to the above, on watercourses other than regional watercourses, the following equation (Zeller, 1981) should be used to predict general scour:

$$Z_{gs} = Y_{max} \left[\frac{0.0685 V_m^{0.8}}{Y_h^{0.4} S_e^{0.3}} - 1 \right] \tag{6.4}$$

Where:

Z_{gs} = General scour depth, in feet;
V_m = Average velocity of flow, in feet per second;
Y_{max} = Maximum depth of flow, in feet;
Y_h = Hydraulic depth of flow, in feet; and,
S_e = Energy slope (or bed slope for uniform-flow conditions), in feet per foot.

NOTE: Should Z_{gs} become negative, assume that the general-scour component is equal to zero (i.e., $Z_{gs} = 0$).

6.6.2 *Anti-Dune Trough Depth*

Anti-dunes are bed forms, in the shape of dunes, which move in an upstream rather than a downstream direction within the channel; hence the term "anti-dunes." They form as trains of waves that build up from a plane bed and a plane water surface. Anti-dunes can form either during transitional flow, between subcritical and supercritical flow, or during supercritical flow. The wave length is proportional to the velocity of flow. The corresponding surface waves, which are in phase with the anti-dunes, tend to break like surf when the waves reach a height approximately equal to 0.14 times the wave length. A relationship between average channel velocity, V_m, and anti-dune trough depth, Z_a, can therefore be developed (Simons, Li & Associates, 1982). This relationship is:

$$Z_a = \frac{1}{2} (0.14) \frac{2\pi V_m^2}{g} = 0.0137 V_m^2 \tag{6.5}$$

A restriction on the above equation is that the anti-dune trough depth can never exceed one-half the depth of flow. Therefore, if the computed depth of Z_a obtained by using Equation 6.5 exceeds one-half of the depth of flow, the anti-dune trough depth should then be taken as equal to one-half the depth of flow. Figure 6.2 shows a definition sketch for anti-dune trough depth.

6.6.3 *Low-Flow Thalweg*

A low-flow thalweg is a small channel which forms within the bed of the main channel, and in which low discharges are carried. Low-flow thalwegs form when the width/depth ratio of the main channel is large. Rather than flow in a very wide, shallow state, low flows will develop a low-flow channel thalweg below the average channel bed elevation in order to provide more efficient conveyance of these discharges.

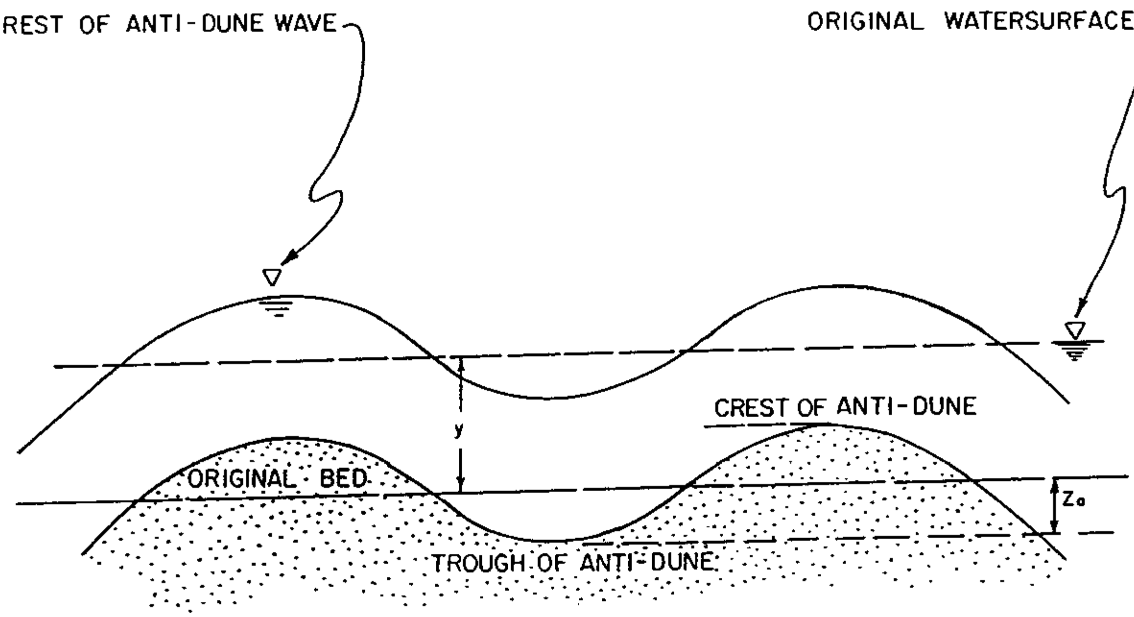


FIGURE 6.2
DEFINITION SKETCH FOR ANTI-DUNE TROUGH DEPTH

When the ratio of the flow width to the flow depth of a channel is greater than 1.15 times the average velocity of flow for the 100-year discharge, a low-flow thalweg must be included in all scour calculations. When the flow width or flow depth exceeds the top width and bank heights of the channel, use the top width and flow depth at bank-full conditions, instead of the actual flow width and flow depth. Presently, there is no known methodology for predicting low-flow thalweg depth. However, observation of channels in the Tucson area has revealed that low-flow thalwegs are normally one to two feet deep. Therefore, if a low-flow thalweg is predicted to be present, it should be assumed to be at least two feet deep within regional watercourses, and at least one foot deep within all other watercourses, unless field observations dictate otherwise.

6.6.4 Bend Scour

Bend scour normally occurs along the outside of bends, and is caused by spiral, transverse currents which form within the flow as the water moves around the bend. Presently, there is no single procedure which will consistently and accurately predict bend scour over a wide range of hydraulic conditions. However, the following relationship has been developed by Zeller (1981) for estimating bend scour in sand-bed channels based upon the assumption of the maintenance of constant stream power within the channel bend:

$$Z_{bs} = \frac{0.0685\, Y_{max} V_m^{0.8}}{Y_h^{0.4}\, S_e^{0.3}} \left\{ 2.1 \left[\frac{\sin^2(\alpha/2)}{\cos\alpha} \right]^{0.2} - 1 \right\} \qquad \textbf{(6.6)}$$

Where:

Z_{bs} = Bend-scour component of total scour depth, in feet;
= 0 when $r_c/T \geq 10.0$, or $\alpha \leq 17.8°$
= computed value when $0.5 < r_c/T < 10.0$, or $17.8° < \alpha < 60°$
= computed value at $\alpha = 60°$ when $r_c/T \leq 0.5$, or $\alpha \geq 60°$

V_m = Average velocity of flow immediately upstream of bend, in feet per second;

Y_{max} = *Maximum* depth of flow immediately upstream of bend, in feet;

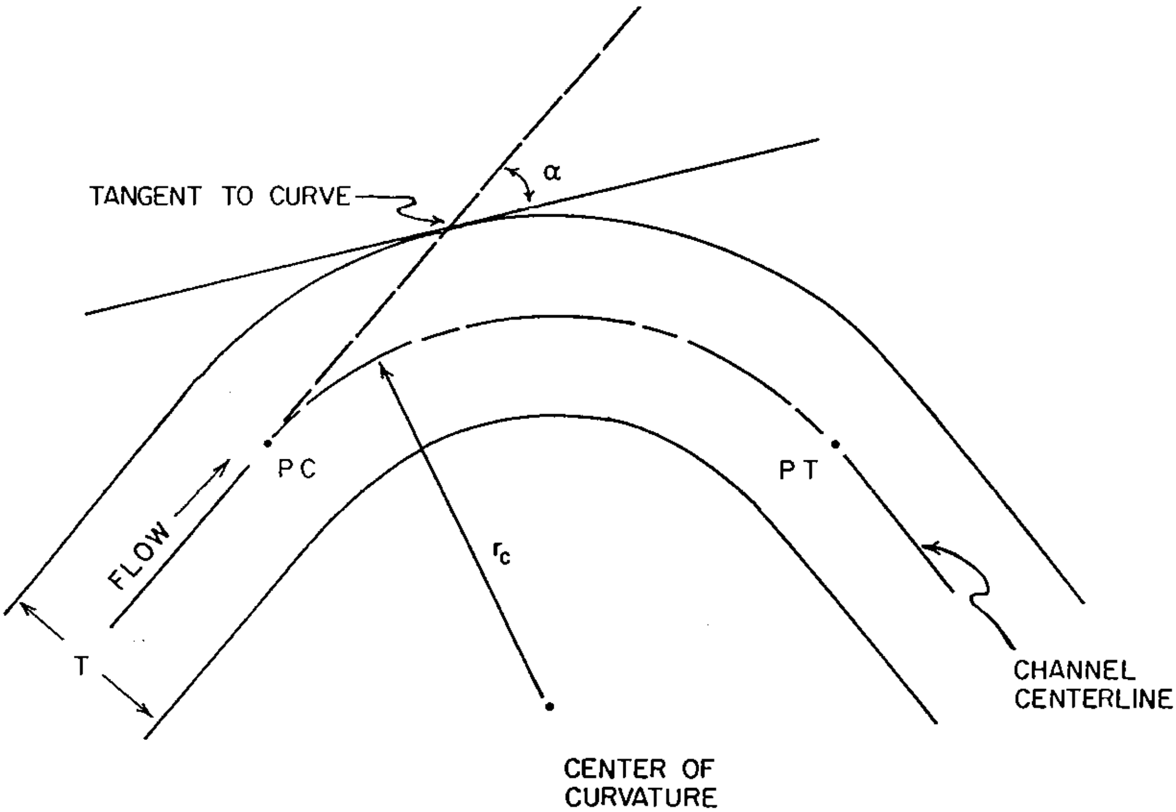
Y_h = Hydraulic depth of flow immediately upstream of bend, in feet;

S_e = Energy slope immediately upstream of bend (or bed slope for uniform-flow conditions), in feet per foot; and,

α = Angle formed by the projection of the channel centerline from the point of curvature to a point which meets a line tangent to the outer bank of the channel, in degrees (see Figure 6.3).

NOTE: Mathematically, it can be shown that, for a simple circular curve, the following relationship exists between α and the ratio of the centerline radius of curvature, r_c, to channel top width, T.

$$\frac{r_c}{T} = \frac{\cos\alpha}{4\sin^2(\alpha/2)} \qquad \textbf{(6.7)}$$



PT = Downstream point of tangency to the centerline radius of curvature.
PC = Upstream point of curvature at the centerline radius of curvature.

FIGURE 6.3
ILLUSTRATION OF TERMINOLOGY FOR BEND-SCOUR CALCULATIONS

Where:

r_c = Radius of curvature along centerline of channel, in feet; and,
T = Channel top width, in feet.

If the bend deviates significantly from a simple circular curve, the curve should be divided into a series of circular curves, and the bend scour computed for each segment should be based upon the angle α applicable to that segment.

Equation 6.6 can be applied to obtain an approximation of the scour depth that can be expected in a bend during a specific water discharge. The impact that other simultaneously occurring phenomena such as sand waves, local scour, long-term degradation, etc., might have upon bend scour is not known for certain, given the present state of the art. Therefore, in order that the maximum scour in a bend not be underestimated, it is recommended that bend scour be considered as an independent channel adjustment that should be added to those adjustments computed for long-term degradation, general scour, and sand-wave troughs.

The longitudinal extent of the bend-scour component is as difficult to quantify as the vertical extent. Rozovskii (1961) developed an expression for predicting the distance from the end of a bend at which the secondary currents will have decayed to a negligible magnitude. This relationship, in a simplified form, can be expressed as:

$$x = \frac{0.6}{n} Y^{1.17} \qquad \textbf{(6.8)}$$

Where:

x = Distance from the end of channel curvature (point of tangency, PT) to the downstream point at which secondary currents have dissipated, in feet;
n = Manning's roughness coefficient;
g = Acceleration due to gravity, 32.2 ft/sec^2; and,
Y = Depth of flow (to be conservative, use maximum depth of flow, exclusive of scour, within the bend), in feet.

Equation 6.8 should be used for determining the distance downstream of a curve that secondary currents will continue to be effective in producing bend scour. As a conservative estimate of the longitudinal extent of bend scour, both through and downstream of the curve, it would be advisable to consider bend scour as commencing at the upstream point of curvature (PC), and extending a distance x (computed with Equation 6.8) beyond the downstream point of tangency (PT).

6.6.5 Local Scour

Local scour occurs whenever there is an abrupt change in the direction of flow. Abrupt changes in flow direction can be caused by obstructions to flow, such as bridge piers or abrupt contractions at bridge abutments.

The depth of scour at bridge piers is highly dependent upon the shape of the pier. Figure 6.4 gives several common pier shapes. A square-nosed pier causes the deepest scour. The depth of scour caused by a square-nosed pier is computed from (Richardson et al., 1975):

$$Z_{lsp} = 2.2\, Y \left(\frac{b_p}{Y} \right)^{0.65} F_u^{0.43} \tag{6.9}$$

Where:

Z_{lsp} = Local scour depth due to pier, in feet;
Y = Flow depth, in feet;
b_p = Pier width normal to the flow direction, in feet; and,
F_u = Upstream Froude number.

Table 6.1 can be used for computing the reduction in the depth of pier scour for the various types of piers shown in Figure 6.4.

TABLE 6.1: REDUCTION FACTORS TO BE USED WHEN APPLYING SCOUR FORMULAS FOR SQUARE-NOSED PIERS TO OTHER SHAPES

(assuming equally projected widths of piers)

Type of Pier	Reduction Factor
Square Nose	1.0
Cylinder	0.9
Round Nose	0.9
Sharp Nose	0.8
Group of Cylinders	0.9

Scour is reduced if the pier is streamlined in the direction of flow. However, many watercourses transport significant amounts of debris during large floods. Such debris can become impaled upon bridge piers, leading to an increase in the pier-width component, b_p, found in Equation 6.9. Therefore, in instances where significant debris

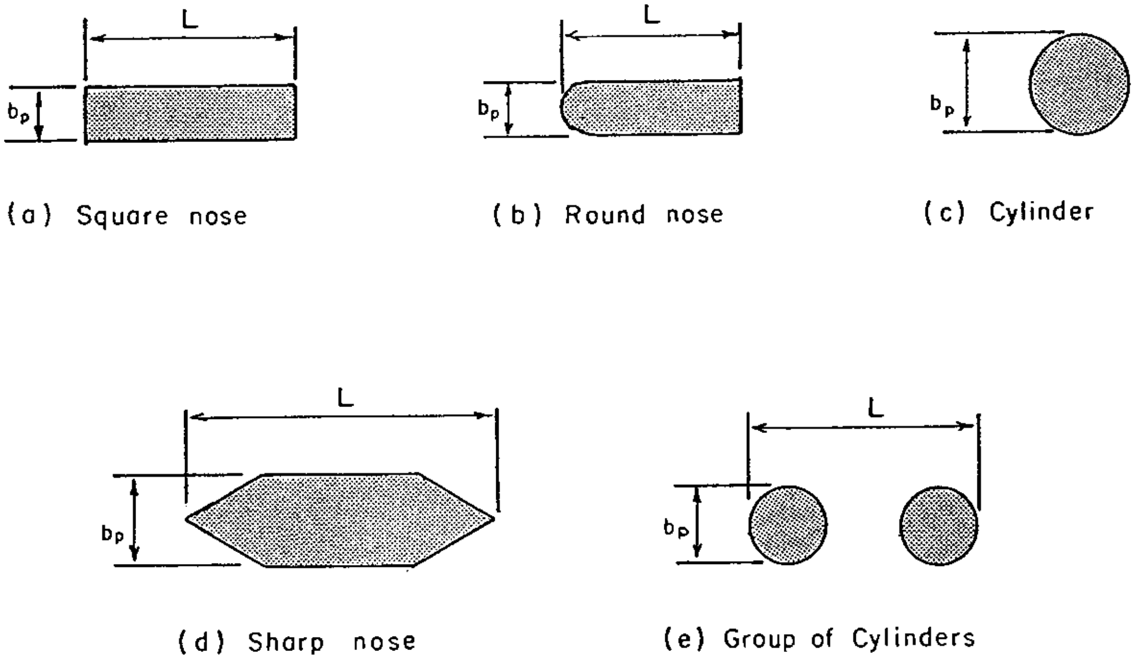


FIGURE 6.4
COMMON PIER SHAPES

transport is anticipated (e.g., within regional watercourses), b_p should be assumed equal to a width of five feet or 1.5 b_p, whichever value is greater. Additionally, pier scour will increase significantly as the direction of flow at the pier becomes more and more skewed in relationship to the pier wall. In such instances, an effective pier width, b_{pe}, can be calculated from the following equation and substituted into Equation 6.9 in lieu of b_p.

$$b_{pe} = L\sin\phi_p + b_p\cos\phi_p \qquad \textbf{(6.10)}$$

Where:

- b_{pe} = Effective pier width, in feet;
- L = Length of pier wall, in feet;
- ϕ_p = Angle of approach flow in relationship to pier wall, in degrees (ϕ_p = 0° for cylindrical piers); and,
- b_p = As defined in Equation 6.9.

In Equation 6.10, b_p should incorporate any width increase due to debris, where applicable.

Local scour caused by embankments projecting into the flow, such as at bridge abutments, fill projections, and overbank levees, can be computed from the following equation:

$$Z_{lse} = 2.15 \sin(\theta_a)\, Y \left(\frac{a_e}{Y}\right)^{0.4} F_u^{0.33} \qquad \textbf{(6.11)}$$

Where:

- Z_{lse} = Local scour depth due to embankment, in feet;
- θ_a = Slope angle of abutment face, measured from the horizontal, in degrees;
- Y = Upstream normal flow depth, in feet;
- a_e = Embankment or encroachment length, measured normal to the edge of the floodplain or channel bank, in feet (see Figure 6.5); and,
- F_u = Upstream Froude number.

For embankments where the quantity a_e/Y is exceedingly large, such that $Z_{lse}/YF_u^{0.33} \geq 4.0$, the following equation (Richardson et al., 1975) should be used in lieu of Equation 6.11:

$$Z_{lse} = 4YF_u^{0.33} \qquad \textbf{(6.12)}$$

Equations 6.11 and 6.12 are based upon relationships developed from both empirical observations and experiments in laboratory flumes. As can be seen from the formulas, the scour depth can be significantly affected by embankment length. In

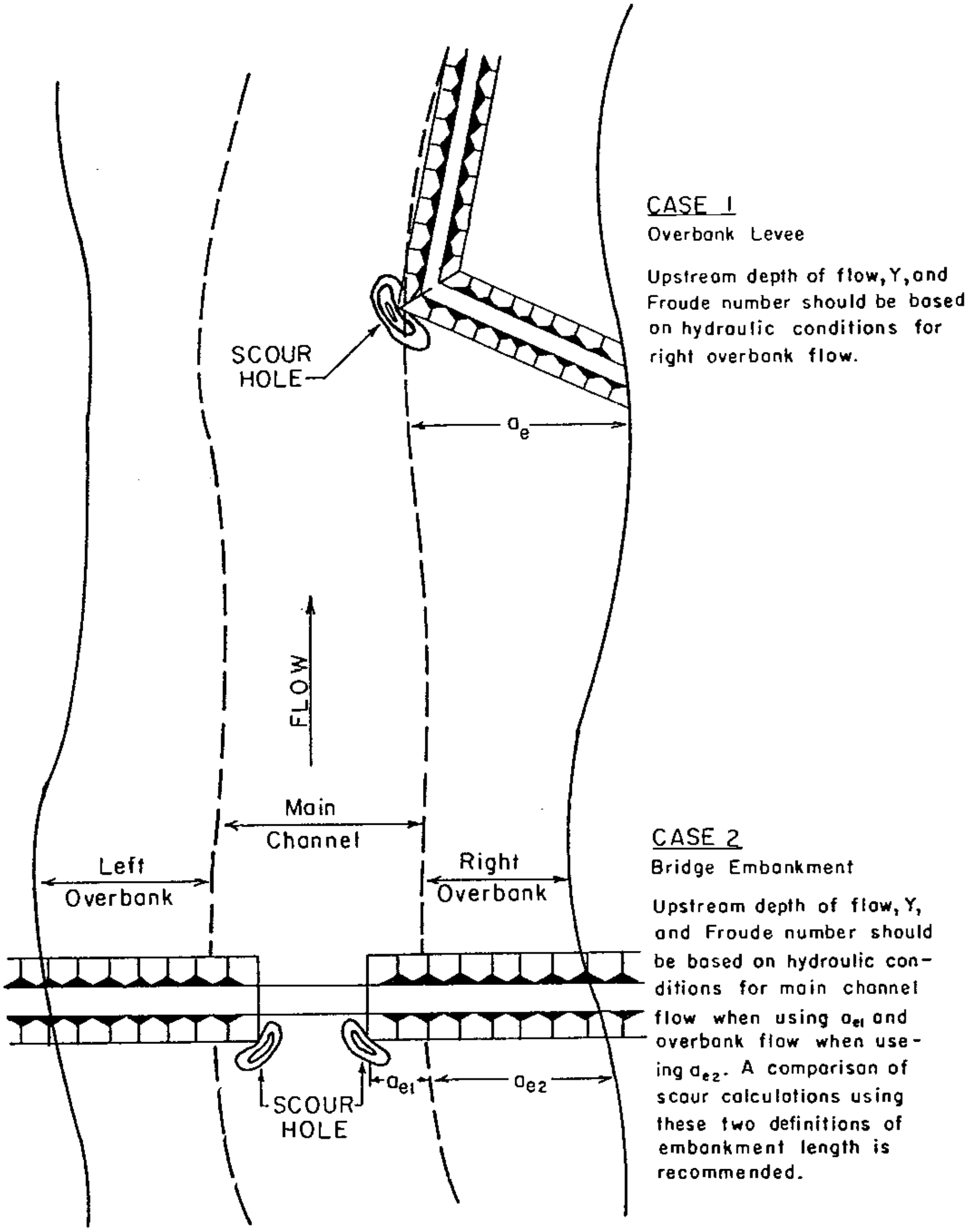


FIGURE 6.5

DEFINITION SKETCH OF EMBANKMENT LENGTH "a_e"

practical situations, the embankment may span a wide floodplain overbank and extend partially into the main channel itself. Due to the normally large differences which exist between channel and overbank hydraulics, caution must be exercised in defining the embankment length. Figure 6.5 shows a recommended embankment length definition for different cases that might be encountered. In the situation where the embankment crosses the entire overbank and extends into the main channel, it is recommended that the scour be computed by utilizing the overbank hydraulics in combination with the embankment length a_{e2}, and that this depth of scour then be compared to the scour depth computed by utilizing the main-channel hydraulics in combination with the embankment length a_{e1}. The larger of the two values should then be used for design purposes.

6.6.6 *Scour Below Channel Drops*

Scour below channel drops, such as grade-control structures, is a special case of local scour. Where the drop consists of a free, unsubmerged overfall, the depth of scour below the drop (U.S. Bureau of Reclamation, 1977) shall be computed from:

$$Z_{lsf} = 1.32\, q^{0.54} H_t^{0.225} - TW \tag{6.13}$$

Where:

- Z_{lsf} = Depth of local scour due to a free-overfall drop, in feet, measured below the streambed surface downstream of the drop;
- q = Discharge per unit width of the channel bottom, in cubic feet per second per foot;
- H_t = Total drop in head, measured from the upstream energy grade line to the downstream energy grade line, in feet; and
- TW = Tailwater elevation (downstream water-surface elevation), in feet.

Figure 6.6 shows the relationship of the parameters in Equation 6.13.

Where the drop is submerged, as will be the case for most instances involving grade-control structures placed along watercourses located within the City of Tucson, the depth of scour below the drop (Simons, Li & Associates, 1986) shall be computed from:

$$Z_{lss} = 0.581 q^{0.667} (h/Y)^{0.411} [1-(h/Y)]^{-0.118} \tag{6.14}$$

Where:

- $h/Y \leq$ 0.99; and,
- Z_{lss} = Depth of local scour due to a submerged drop, in feet, measured below the streambed surface downstream of the drop;
- q = Discharge per unit width of the channel bottom, in cubic feet per second per foot;
- h = Drop height, in feet; and,
- Y = Downstream depth of flow, in feet.

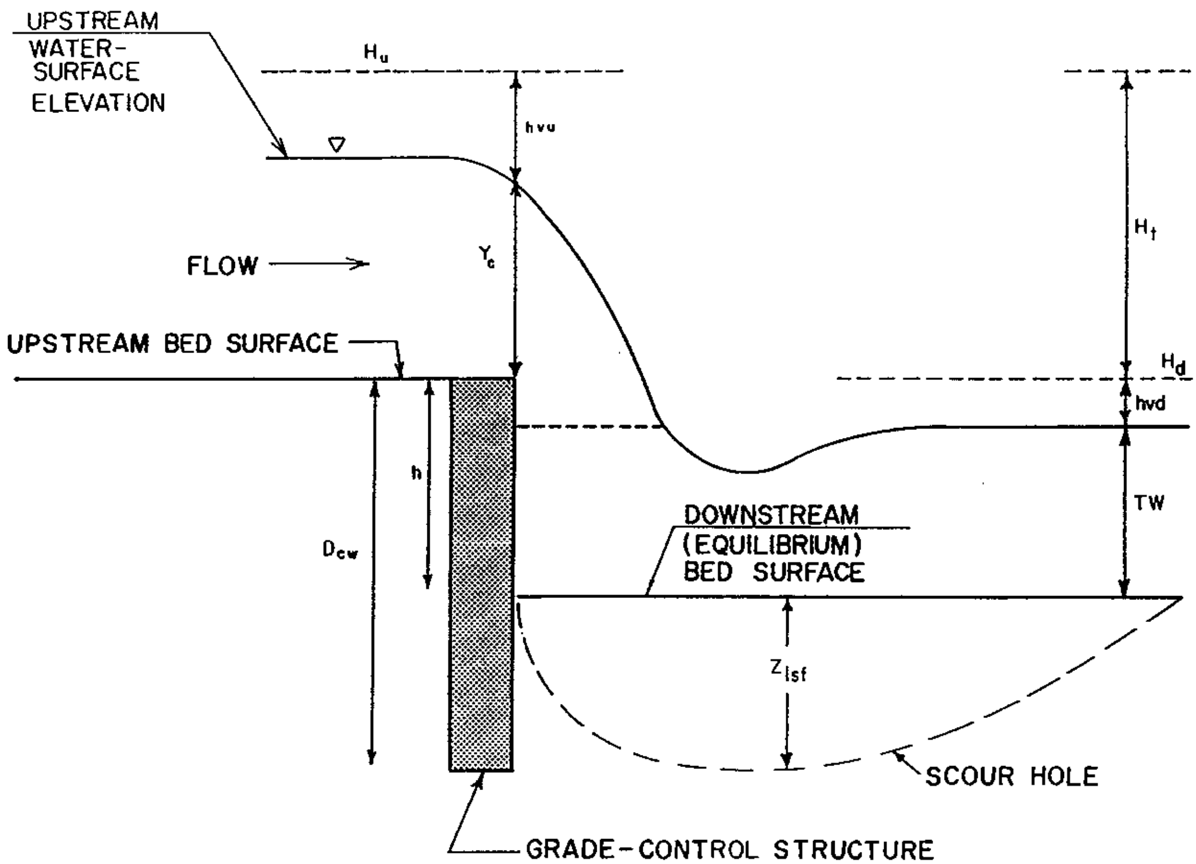


FIGURE 6.6
DEPTH OF SCOUR BELOW A FREE OVERFALL

NOTE: If $h/Y > 0.85$, the predicted scour below a channel drop should also be computed using Equation 6.13. The smaller of the two values thus computed should then be used for design purposes.

Figure 6.7 gives the relationship of the parameters in Equation 6.14.

The longitudinal extent of a scour hole created by either a free or submerged overfall is represented by the distances x_{sce} and L_s, as depicted in Figure 6.7. These dimensions are given by the equations:

$$x_{sce} = 6.0\ Z_{lsf}, \text{ or } 6.0\ Z_{lss} \tag{6.15}$$

$$L_s = 12.0\ Z_{lsf}, \text{ or } 12.0\ Z_{lss} \tag{6.16}$$

Bank protection toe-downs downstream of a grade-control structure shall extend to the computed depth of scour for a distance equal to x_{sce} beyond of the grade-control structure, as computed by Equation 6.15. They shall then taper back to the normal toe-down depth within a total distance downstream of the grade-control structure equal to L_s, as computed by Equation 6.16. Note that L_s includes x_{sce}.

In the absence of bridge piers and/or abutments, the depth of scour below grade-control structures is not added to the other scour components. Rather, the depth of scour caused by the grade-control structure is compared to the depth of scour computed by Equation 6.3, and the larger of the two values is then used for toe-down design.

6.7 Scour-Hole Geometry at Culvert Outlets

Culverts normally have less cross-sectional area available for the conveyance of flow than do the natural channels they replace. Consequently, flow velocities are increased and a potential for erosion is created at the culvert outlet. Often there is a drop at the culvert outlet, either under design conditions or as a result of outlet scour, and this further increases the possibility of outlet scour. The scour hole created at the outlet of a culvert can become large enough to threaten the culvert, the roadway, adjacent property, or other nearby improvements.

For non-cohesive soils, the dimensions of a scour hole downstream of a culvert outlet *where no drop exists* can be computed by:

$$DSG = \alpha \left[\frac{Q_r}{g^{1/2}\, D^{5/2}} \right]^{\beta} (0.09)^{\theta} \tag{6.17}$$

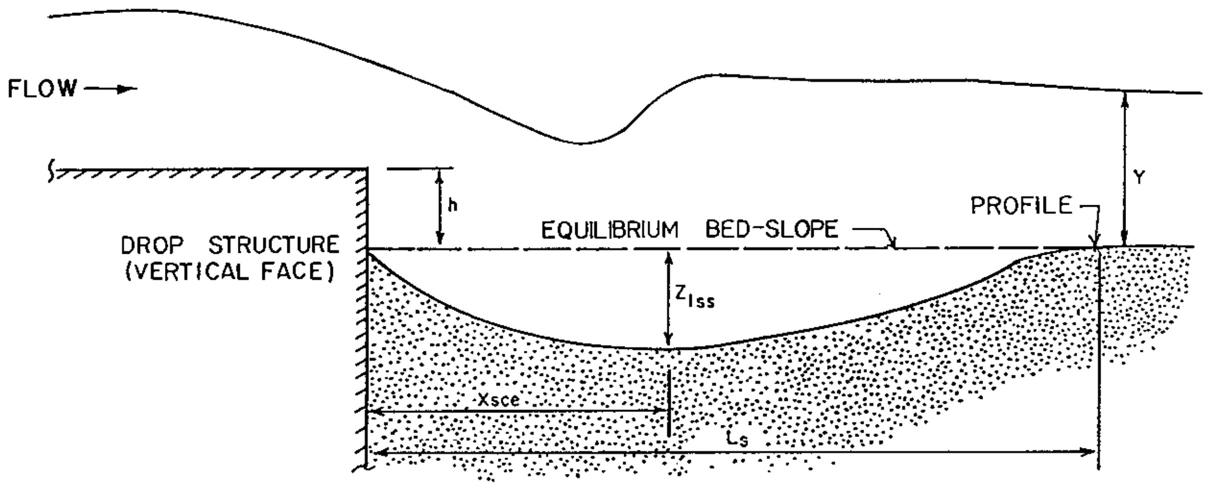


FIGURE 6.7

RELATIONSHIP OF VARIABLES IN EQUATION FOR SCOUR BELOW A SUBMERGED GRADE-CONTROL STRUCTURE

Where:

DSG = Dimensionless scour geometry = $\frac{Z_{lsc}}{D}$, $\frac{W_{sc}}{D}$, $\frac{L_{sc}}{D}$ or $\frac{V_{sc}}{D}$; and,

Z_{lsc} = Depth of scour hole below culvert, in feet;
W_{sc} = Width of scour hole below culvert, in feet;
L_{sc} = Length of scour hole below culvert, in feet;
V_{sc} = Volume of scour hole below culvert, in cubic feet;
D = Culvert diameter, in feet;
Q_r = Representative discharge, in cubic feet per second;
g = Acceleration due to gravity (32.2 ft/sec^2); and,
$\alpha, \alpha_e, \beta, \theta$ = Empirically derived coefficients (see Table 6.2).

The representative discharge is the average maximum discharge that can be expected to occur within a thirty-minute time period during the storm runoff event which is selected for design. In the City of Tucson, the design discharge is the 100-year flood. The representative discharge is calculated by:

$$Q_r = \frac{Q_{100}}{2}\left[1 + \frac{T_r - 10}{T_r}\right] \tag{6.18}$$

Where:
Q_r = Representative discharge, in cubic feet per second;
Q_{100} = 100-year peak discharge, in cubic feet per second; and,
T_r = Hydrograph rise time, in minutes (see Chapter IV, Section 4.5, of this Manual).

For either non-circular or partially-full culverts, the culvert diameter, D, should be replaced in Equation 6.17 by an equivalent depth, Y_e, where Y_e is defined as:

$$Y_e = \left(\frac{A}{2}\right)^{0.5} \tag{6.19}$$

Where:
A = Cross-sectional area of flow, in square feet.

Equation 6.18 is then modified to the following form:

TABLE 6.2A: EXPERIMENTAL COEFFICIENTS FOR SCOUR DEPTH, Z_{lsc}, AT CULVERT OUTLETS

MATERIAL	NOMINAL GRAIN SIZE D_{50} (mm)	SCOUR EQUATION	COEFFICIENTS			
			α	β	θ	α_e
Uniform Sand	0.20	6.17 or 6.20	2.72	0.375	0.10	2.79
Uniform Sand	2.0	6.17 or 6.20	1.86	0.45	0.09	1.76
Graded Sand	2.0	6.17 or 6.20	1.22	0.85	0.07	0.75
Uniform Gravel	8.0	6.17 or 6.20	1.78	0.45	0.04	1.68
Graded Gravel	8.0	6.17 or 6.20	1.49	0.50	0.03	1.33
Cohesive Sandy Clay:						
60% Sand, PI 15	0.15	6.17 or 6.20	1.86	0.57	0.10	1.53
Clay, PI 5-16	Varies	6.22 or 6.23	0.86	0.18	0.10	1.37

TABLE 6.2B: EXPERIMENTAL COEFFICIENTS FOR SCOUR WIDTH, W_{sc}, AT CULVERT OUTLETS

MATERIAL	NOMINAL GRAIN SIZE D_{50} (mm)	SCOUR EQUATION	COEFFICIENTS			
			α	β	θ	α_e
Uniform Sand	0.20	6.17 or 6.20	11.73	0.92	0.15	6.44
Uniform Sand	2.0	6.17 or 6.20	8.44	0.57	0.06	6.94
Graded Sand	2.0	6.17 or 6.20	7.25	0.76	0.06	4.78
Uniform Gravel	8.0	6.17 or 6.20	9.13	0.62	0.08	7.08
Graded Gravel	8.0	6.17 or 6.20	8.76	0.89	0.10	4.97
Cohesive Sandy Clay:						
60% Sand, PI 15	0.15	6.17 or 6.20	8.63	0.35	0.07	9.14
Clay, PI 5-16	Varies	6.22 or 6.23	3.55	0.17	0.07	5.63

TABLE 6.2C: EXPERIMENTAL COEFFICIENTS FOR SCOUR LENGTH, L_{sc}, AT CULVERT OUTLETS

MATERIAL	NOMINAL GRAIN SIZE D_{50} (mm)	SCOUR EQUATION	COEFFICIENTS			
			α	β	θ	α_e
Uniform Sand	0.20	6.17 or 6.20	16.82	0.71	0.125	11.75
Uniform Sand	2.0	6.17 or 6.20	18.28	0.51	0.17	16.10
Graded Sand	2.0	6.17 or 6.20	12.77	0.41	0.04	12.62
Uniform Gravel	8.0	6.17 or 6.20	14.36	0.95	0.12	7.61
Graded Gravel	8.0	6.17 or 6.20	13.09	0.62	0.07	10.15
Cohesive Sandy Clay:						
60% Sand, PI 15	0.15	6.17 or 6.20	15.30	0.43	0.09	14.78
Clay, PI 5-16	Varies	6.22 or 6.23	2.82	0.33	0.09	4.48

TABLE 6.2D: EXPERIMENTAL COEFFICIENTS FOR SCOUR VOLUME, V_{sc}, AT CULVERT OUTLETS

MATERIAL	NOMINAL GRAIN SIZE D_{50} (mm)	SCOUR EQUATION	COEFFICIENTS			
			α	β	θ	α_e
Uniform Sand	0.20	6.17 or 6.20	203.36	2.0	0.375	80.71
Uniform Sand	2.0	6.17 or 6.20	101.48	1.41	0.34	79.62
Graded Sand	2.0	6.17 or 6.20	36.17	2.09	0.19	12.94
Uniform Gravel	8.0	6.17 or 6.20	65.91	1.86	0.19	12.15
Graded Gravel	8.0	6.17 or 6.20	42.31	2.28	0.17	32.82
Cohesive Sandy Clay:						
60% Sand, PI 15	0.15	6.17 or 6.20	79.73	1.42	0.23	61.84
Clay, PI 5-16	Varies	6.22 or 6.23	0.62	0.93	0.23	2.48

$$DSG = \alpha_e \left[\frac{Q_r}{g^{1/2} \, Y_e^{5/2}} \right]^{\beta} (0.09)^{\theta} \tag{6.20}$$

The coefficient α_e can also be found in Table 6.2.

Bed materials are classified in Table 6.2 as being either uniform or graded. Uniform materials are classified as those for which the standard deviation (σ) of the grain-size distribution is less than or equal to 1.5. The material is classified as graded if the standard deviation of the grain-size distribution is greater than 1.5. A simple formula often used for computing the standard deviation is:

$$\sigma = \left[\frac{D_{84}}{D_{16}} \right]^{0.5} \tag{6.21}$$

Where:

D_{84} = The grain-size diameter for which 84% of the bed material consists of smaller particles; and

D_{16} = The grain-size diameter for which 16% of the bed material consists of smaller particles.

The grain-size distribution can be determined by a sieve analysis of the bed material. For planning purposes, or in the absence of a sieve analysis, bed material in the City of Tucson should be classified as graded sand, with a median diameter, D_{50}, equal to one millimeter and $\sigma = 4.0$.

If the soil at the culvert outlet is a sandy clay with a mean grain size in the range of 0.10 to 0.20 mm and a plasticity index, *PI*, of approximately 15, either Equation 6.17 or 6.20 may be used; where the coefficients for such a soil type are also given in Table 6.2.

Equations 6.17 and 6.20 are not applicable to cohesive soils, which have very different properties than the soil types described above. The potential for scour in cohesive soils is related to the critical shear stress of the soils, and is reflected by Equations 6.22 and 6.23. These equations have a wider range of applicability than do the above expressions. These equations are:

$$DSG = \alpha \left[\frac{\rho \, V^2}{\tau_c} \right]^{\beta} (0.09)^{\theta} \tag{6.22}$$

For circular culverts, and

$$DSG = \alpha_e \left(\frac{\rho V^2}{\tau_c} \right)^{\beta} (0.09)^{\theta} \tag{6.23}$$

For culverts with other shapes.

Where:

$\frac{\rho V^2}{\tau_c}$	=	Modified shear number;
V	=	Average velocity at outlet, in feet per second;
τ_c	=	Critical tractive shear stress, in pounds per square foot; and,
ρ	=	Fluid density, in slugs per cubic foot.

All other terms are as previously defined.

The critical tractive shear stress is defined as:

$$\tau_c = 0.0001 (S_v + 180) \tan (30 + 1.73\ PI) \tag{6.24}$$

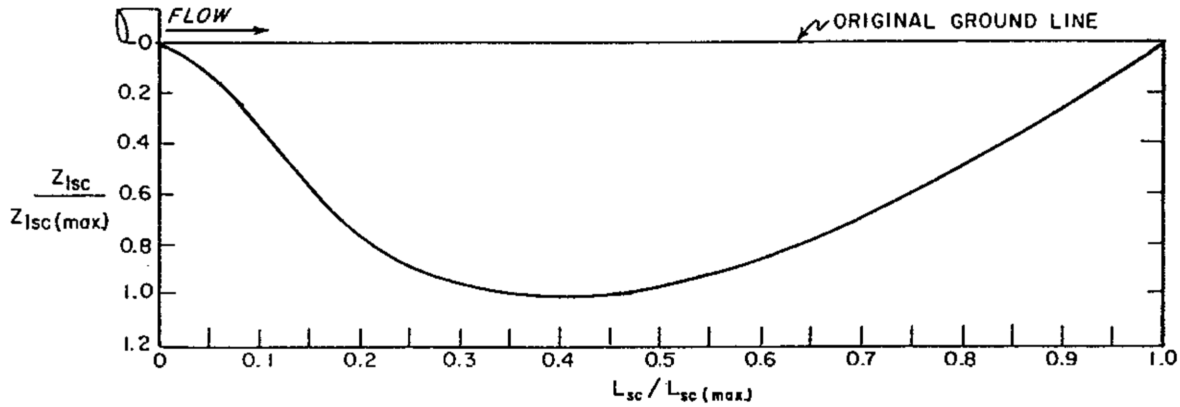
Where:

S_v	=	Saturated shear strength, in pounds per square inch; and,
PI	=	The plasticity index (limits 5-16).

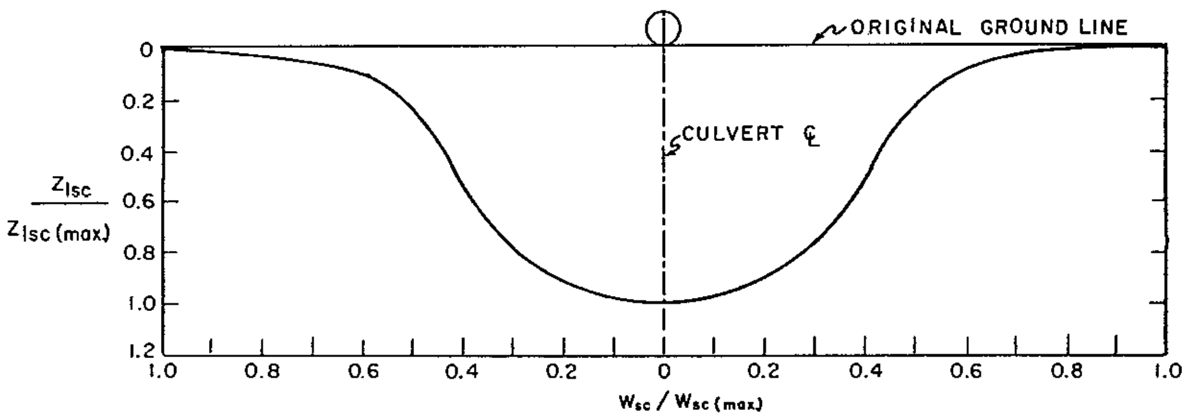
Equations 6.17 to 6.24 can therefore be used to estimate the dimensions of the scour hole that would form at the outlet of a culvert for varying types of soils. Figures 6.8 and 6.9 should be used to determine the shape of the scour hole. If the scour hole is large enough to threaten nearby improvements, adjacent property, or the culvert itself, outlet protection will be required to contain and/or prevent erosion. The user is referred to a publication by the Federal Highway Administration (1983) for further information regarding the design of culvert outlet protection.

6.8 Design of Sediment Basins

On watercourses with a potential for high sediment discharge, sediment basins may be necessary to protect detention basins, culverts, or storm drains from being filled with sediment. If it is felt that sedimentation could pose a problem for a proposed structure, basins should be built to collect and hold sediment for later removal by maintenance personnel. The design of these basins on watercourses where the upstream watershed area is one square mile, or less, shall be in accordance with the guidelines as presented within Section 3.4 of the Pima County and City of Tucson Stormwater Detention/Retention Manual (1987).



DIMENSIONLESS CENTER-LINE PROFILE



DIMENSIONLESS CROSS SECTION AT 0.4 $L_{sc(max.)}$

FIGURE 6.8
DIMENSIONLESS SCOUR-HOLE GEOMETRY
FOR MAXIMUM TAILWATER

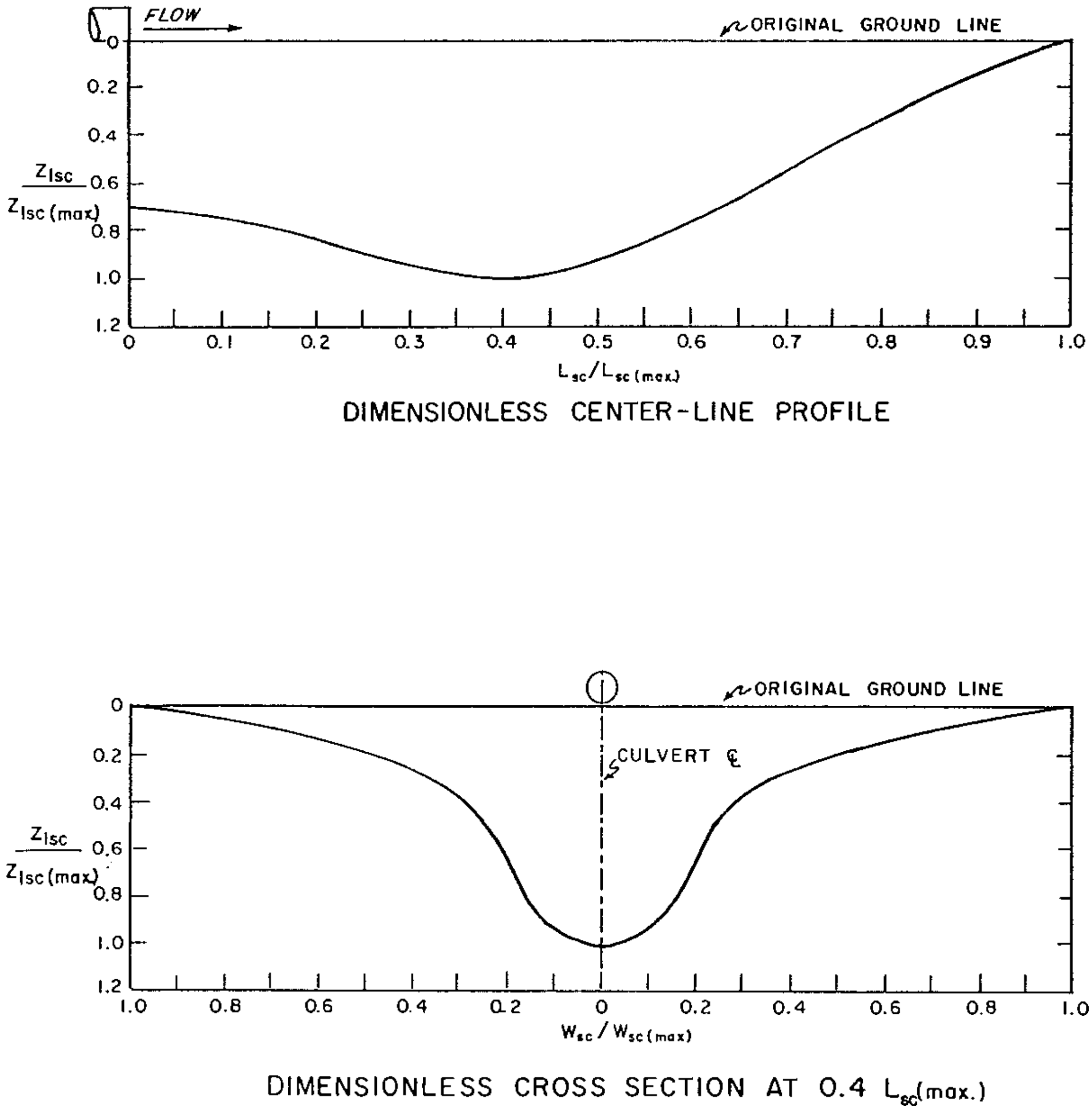


FIGURE 6.9
DIMENSIONLESS SCOUR-HOLE GEOMETRY
FOR MINIMUM TAILWATER

On watersheds larger than one square mile, the guidelines cited above may result in overdesign. The design of sediment basins on these watersheds is a more complicated procedure, involving total watershed sediment yield and channel sediment-transport capacity over a range of discharges. Total watershed sediment yield can be estimated by such methods as the Modified Universal Soil Loss Equation (Williams, 1975; and Williams and Berndt, 1977), the Pacific Southwest Inter-Agency Committee (PSIAC) Method (Pacific Southwest Inter-Agency Committee, 1968), the Flaxman Method (Flaxman, 1972), the SCS Method (U.S. Soil Conservation Service, 1971), the Dendy/Bolton Method (Dendy and Bolton, 1976), and the Renard Method (Renard, 1972). A publication by Renard and Stone (1981) contains a detailed discussion and comparison of some of these methods.

The equations for watershed sediment yield which are listed above do not readily distinguish between sediment production that would be classified as wash load and sediment production that would be classified as bed load. Wash load particles are so small that they would generally remain in suspension as the water passes through the detention basin. Therefore, the wash load is not generally to be considered in sediment basin design. An estimate of wash load, as compared to bed load estimated from equations for total watershed sediment yield, can be made by taking samples of the topsoil throughout the watershed.

Total watershed sediment production may not be an entirely accurate estimate of the amount of sediment that would be delivered to a certain point, because there is sediment storage within the watershed system. Sediment-volume estimates must therefore also consider the sediment-transport capacity of the channel. A detailed discussion of this type of analysis will not be presented here. However, the reader is referred to publications by the U.S. Army Corps of Engineers (1977), Simons, Li & Associates (1982, 1985), the American Society of Civil Engineers (1977), Simons and Senturk (1977), and Zeller and Fullerton (1983) for more detailed information about performing such analyses.

6.9 Equilibrium Slopes within Constructed Channels

Given a fixed size distribution of sediments, the sediment-transport capacity of a stream is dependent primarily upon flow velocity and depth. Within the City of Tucson, transport of all particle sizes of bed material increases, as flow velocity increases, at a rate proportional to approximately the third to fifth power of the velocity. Correspondingly, transport of sediment particles composed of bed material generally decreases as depth increases, while transport increases with decreased depth. However, flow velocity is by far the more important variable.

For purposes of analysis and design, most natural, undisturbed channels in the Tucson area can be assumed to be at or near a state of dynamic equilibrium with regard to sediment transport. This means that, for a given reach of the channel, the sediment-transport capacity of the channel, over the long term, is more or less equal to the sediment supply. The channel bed slope is therefore "stable."

When channelization occurs, the channel top width is often narrowed, and channel roughness is normally decreased. The result is an increase in velocity and depth, with a corresponding increase in sediment-transport capacity. Sediment-transport capacity then exceeds the sediment supply; and, if the bed is composed of sediment that can be transported, the deficiency will be made up from bed material--causing the channel to degrade. Another factor that contributes to this degradation is upstream urbanization. Urbanization increases flood peaks, which also lead to higher flow velocities and depths. Urbanization also reduces the watershed sediment supply, and increases the frequency of runoff. The result of all these occurrences is that channel bed degradation will occur until the channel slope is flat enough to cause the sediment-transport rate to be equal to the incoming sediment supply. This slope then becomes the new, "stable," equilibrium slope. Streambed degradation can threaten underground improvements, bank-protection toe-downs, culverts, and other hydraulic structures that are within and/or that cross the channel. Grade-control structures, or lining of the channel bed, are usually required in order to prevent damage caused by streambed degradation.

The equilibrium slope for a channel which has an upstream sediment supply that is considered to be essentially zero (e.g., a channel located within a highly urbanized watershed) can be computed from:

$$S_{eq} = \left(\frac{1.45n}{q^{0.11}}\right)^2 \tag{6.25}$$

Where:

S_{eq} = Equilibrium slope after urbanization, in feet per foot;
n = Manning's roughness coefficient; and,
q = Channel unit discharge, in cubic feet per second per foot.

For use with Equation 6.25, channel unit discharge is defined as the channel discharge divided by the channel bottom width. Use of this equation will produce the flattest slope that can be reasonably expected to transport sediment within channels located in the Tucson area. The discharge associated with a 10-year flood is normally chosen when computing the unit discharge for use in Equation 6.25.

For lesser degrees of urbanization, the equilibrium slope is computed from Equation 6.26, which is a generalization of the theoretically derived sediment-transport relationships for sandbed channels developed by Zeller and Fullerton (1983):

$$S_{eq} = \left\{ \left[\frac{n_u}{n_n}\right]^2 \left[\frac{Q_{u,10}}{Q_{n,10}}\right]^{-1.1} \left[\frac{b_u}{b_n}\right]^{0.4} (1-R_s)^{0.7} \right\} S_n \tag{6.26}$$

Where:

n_u = Manning's roughness coefficient for an urban channel;
n_n = Manning's roughness coefficient for a natural or existing channel;
$Q_{u,10}$ = Ten-year discharge, under urbanized conditions, in cubic feet per second;
$Q_{n,10}$ = Ten-year or bank-full discharge (whichever is less), under natural conditions, in cubic feet per second;
b_u = Bottom width of channel, under urbanized conditions, in feet;
b_n = Bottom width of channel, under natural conditions, in feet;
R_s = Reduction factor for sediment supply. This factor is usually assumed to be equal to the ratio of the impervious area to the total area of the upstream watershed (i.e., $0.0 \le R_s \le 1.0$); and,
S_n = Natural or existing channel slope, in feet per foot.

The roughness coefficients for natural and urbanized channel beds are often very nearly the same, so the term in which these coefficients appear in Equation 6.26 can usually be assumed equal to the value 1.0. However, from time to time exceptions to this assumption may occur. For instance, when the existing channel is a wide, flat, sheetflow watercourse; and the proposed channel is a narrow, sand-bed channel, n_u will ordinarily not be equal to n_n.

For moderately urbanized to highly urbanized watersheds, the equilibrium slope should be computed by using both Equation 6.25 and Equation 6.26. The steeper of the two computed slopes should then be used for design. The reason for this is that Equation 6.26 can sometimes produce slope values that are too flat to generate reasonable sediment-transport rates for maintenance of channel stability, when impervious cover within a watershed is very high.

Equation 6.26 should be used with caution within the City of Tucson. An underlying assumption of this equation is that the existing or natural channel is itself in equilibrium. This is not always true in the City, because most channels have undergone alteration. If there is any question as to whether or not the existing channel is in equilibrium, it is best to try and determine through old (pre-development) aerial photographs and topography what the channel characteristics were in its original, undisturbed (i.e., natural) state. In the absence of historical information about the original channel, an examination may be made of existing stable channels in the area to help estimate what the channel in question may have looked like before urbanization.

Equation 6.26 can be used for more than merely the quantification of streambed degradation. It can also be used to determine whether aggradation will occur when a channel is widened beyond existing or natural conditions. Another application would be to use it to design a stable channel cross-section in lieu of installing grade-control structures to otherwise control degradation of the channel bed.

6.10 Spacing and Depth of Grade-Control Structures

If the equilibrium slope of a channel, as determined by use of either Equation 6.25 or Equation 6.26, is flatter than the design slope, grade-control structures may be needed to limit degradation from exceeding a certain depth at any point along the channel. Grade-control structures, sometimes called "cut-off walls" or "check dams," are non-erodible vertical barriers in the channel that prevent the channel bed from degrading at a point located immediately upstream of where they are located. After the channel bed has reached equilibrium, the bed elevation immediately upstream of the grade-control structure is at the design elevation. Downstream of the grade-control structure, the bed is at an "equilibrium" elevation that is lower than the design elevation. For most channels, the design of grade-control structures is an iterative process, involving drop height, reach length, and depth of scour downstream of the drop.

Once a drop height is chosen, the reach length, or spacing, between adjacent structures can be computed from:

$$L_r = \frac{h}{S_{ib} - S_{eq}} \tag{6.27}$$

Where:

L_r = Reach length, or spacing, between adjacent grade-control structures, in feet;
h = Drop height downstream of the grade-control structure, in feet;
S_{ib} = Initial channel bed slope, in feet per foot; and,
S_{eq} = Channelized equilibrium bed slope, in feet per foot.

If the initial and final bed slopes are approximately the same, the distance between grade-control structures will be very large. Under these circumstances, such structures may not be required.

Normally, the drop height downstream of a grade-control structure which consists of poured concrete without reinforcements shall not exceed two feet; and preferably should be only one foot, where feasible. For economical and technical reasons, grade-control structures should be spaced no closer together than twelve times the local scour depth below the grade-control structures, as computed by the use of either Equation 6.13 or Equation 6.14.

The total height of a cut-off wall or a grade-control structure (D_{cw}), from top to toe, shall not be less than the drop height plus the computed depth of scour below the wall or structure (see Figure 6.6). The depth of scour below grade-control structures should be computed according to the guidelines presented in Section 6.6.6 of this Manual. For a one-foot-wide, unreinforced concrete cut-off wall, if structural calculations support same, the maximum allowable height of a cut-off wall, from top to toe, can be six feet. If the depth of scour plus the drop height is greater than six feet, the drop shall be considered to be too great for unreinforced concrete cut-off walls, unless a structural analysis can demonstrate otherwise, and the spacing between the cut-off walls must be reduced. The example which follows (i.e., Example 6.1), illustrates the recommended procedure for cut-off wall design.

There will be many design situations, especially when unit discharges are high, where a cut-off wall with a hegith of six feet, from top to toe, is not sufficient. In such cases, a reinforced concrete cut-off wall that has a height greater than six feet, from top to toe, may be used, provided that a structural analysis is submitted showing that the proposed cut-off wall will be structurally stable. If a structural analysis is submitted and approved, the maximum drop height of two feet will no longer apply.

Grade-control structures for large discharges need not necessarily be vertical on the downstream side. For structural stability, a triangular or wedge-shaped soil-cement grade-control structure is recommended for use on regional watercourses. However, for hydraulic reasons, the use of any grade-control structure with a face flatter than 1:1 on the downstream side shall not be permitted without prior written approval from the City Engineer.

EXAMPLE 6.1: SPACING AND DEPTH OF GRADE-CONTROL STRUCTURES

A channel in a highly urbanized watershed is to be built to contain the 100-year-flood discharge. The sides of the channel are to be of shotcrete, the bottom of earth.

Channel characteristics are as follows:

Bottom Width	= 20 feet
Design Slope	= 0.006 feet/foot
Side Slopes	= 1:1
Manning's "*n*"	= 0.022

Hydraulic characteristics are as follows:

Q_{100} = 700 cfs	Q_{10} = 350 cfs
Y_{100} = 3.1 feet	Y_{10} = 2.1 feet
V_{100} = 9.7 fps	V_{10} = 7.7 fps
q_{100} = 35.0 cfs/foot	q_{10} = 17.5 cfs/foot

Because the watershed is highly urbanized, Equation 6.25 will be used to compute the equilibrium slope. Therefore:

$$S_{eq} = \left[\frac{1.45\ (0.022)}{(17.5)^{0.11}} \right]^2 = 0.0005 \text{ feet/foot.}$$

Assume a two-foot drop height. From Equation 6.27, the spacing between grade-control structures should be:

$$L_r = \frac{2.0}{(0.006)-(0.0005)} = 364 \text{ feet.}$$

The grade-control structure will be submerged. Using Equation 6.14 yields:

$$Z_{lss} = 0.581(q_{100})^{0.667}(h/Y_{100})^{0.411}[1-(h/Y_{100})]^{-0.118};$$

$$q_{100} = 35.0 \text{ cfs};$$

$$Z_{lss} = 0.581\ (35.0)^{0.667}\ (0.645)^{0.411}\ (0.355)^{-0.118}; \text{ so,}$$

$$Z_{lss} = 5.9 \text{ feet.}$$

Therefore, the total height of the grade-control structure, from top to toe, should be 5.9 feet *plus* the two-foot drop height; or, 7.9 feet (round to 8.0 feet).

However, it is desirable to keep the total vertical dimension of the grade-control structure, from top to toe, equal to or less than six feet. Therefore, a smaller drop height should be used.

Using a drop height of one foot yields:

$$L_r = \frac{1.0}{(0.006)-(0.0005)} = 182 \text{ feet.}$$

$$Z_{lss} = 0.581\ (35.0)^{0.667}\ (0.323)^{0.411}(0.677)^{-0.118}; \text{ so,}$$

$$Z_{lss} = 4.10 \text{ feet (round to 4.0 ft).}$$

Since, in this example, the ultimate drop height at the downstream side of a grade-control structure will be set at one foot, cut-off walls with a height of five feet, from top to toe, could be placed at approximately 180-foot intervals along the bottom of the channel to serve as grade-control structures in order to limit long-term bed degradation to a maximum of one foot anywhere along the subject channel.

CHAPTER VIII: OPEN-CHANNEL DESIGN

8.1 Purpose

The purpose of this chapter is to (1) provide the minimum requirements for the hydraulic design of all open channels which fall within the jurisdiction of the City of Tucson (both public and private); (2) provide the additional requirements which must be met before the city will accept a channel for maintenance (public channels); and, (3) provide the design requirements for those new channels which will either be constructed near or discharge directly into natural channels. Because erosion, sedimentation, and channel-stabilization components are also an integral part of any channel design, these topics are discussed in much greater detail in Chapters VI, VII, and IX, of this Manual, respectively.

8.2 Introduction

The hydraulic design of drainage channels is not a simple procedure. For a relatively long, straight, and uniform channel, normal-depth (i.e., uniform-flow) calculations can be used to determine the discharge capacity at varying depths for a constant cross-sectional area. However, practicing engineers working in an urban environment will rarely encounter either existing conditions or design conditions where uniform-flow calculations are adequate to totally define the flow conditions associated with a given discharge. Transition sections, channel junctions or confluences, channel bends, and hydraulic structures (e.g., culverts and bridges) can create deviations from uniform-flow conditions. Therefore, the engineer must consider these deviations when designing or analyzing drainage channels.

The procedures outlined in this chapter, although not exhaustive, are sufficient for most situations that will be encountered by design engineers. The basic principles behind these design procedures are found in standard textbooks and manuals which deal exclusively with open-channel hydraulics. The design engineer is encouraged to consult the references for this chapter cited at the end of this Manual for a more complete understanding of these principles. Many of the procedures presented herein are particularly similar to those included within the referenced documents prepared by the Los Angeles Flood Control District (1973) and the U.S. Army Corps of Engineers (1970). However, where appropriate, they have been modified to account for local requirements and regulations. As with the other chapters in this Manual, the procedures outlined herein shall be adhered to unless otherwise stated in the Manual, or unless prior approval to deviate from same is obtained, in writing, from the office of the City Engineer.

8.3 Requirements for Natural Channels

Washes which traverse land designated for a proposed development may be left in their natural state provided that doing so would not be in conflict with an approved master drainage plan for the area, if one exists; and provided that the development is adequately protected from flooding and erosion. One method of developing in the vicinity of a natural wash is to keep all structures out of its 100-year floodplain, as well as its attendant erosion-hazard areas. Floodplain delineations and erosion-setback distances are discussed in Chapters V and VII of this Manual. Another possible method of developing in the vicinity of natural washes is to utilize part of the floodplain for development, while leaving the channel in its natural state. However, this approach would involve demonstrating that (1) the encroachment would not adversely affect adjacent properties; that (2) the development would be located outside of any erosion-hazard areas which border the natural wash; and that (3) in certain key areas, as identified by the City and through the 404-permit process, the disturbance to existing riparian vegetation and habitat is minimized.

8.4 Floodplain Encroachments

Encroachments into the floodplain of a natural wash are to be analyzed according to the procedures outlined in Chapter V. The City of Tucson "Floodplain Regulations" state that the maximum allowable rise in water-surface elevation for the 100-year discharge shall be one-tenth of a foot. However, if the natural wash is small enough that the entire width of the floodplain is owned or controlled by a single entity or corporation, and there are no existing structures in the floodplain, it is possible that an exception to this rule might be granted by the City. Under these circumstances, the maximum rise in the water-surface elevation would be limited to one foot, as per Federal Emergency Management Agency guidelines. However, as with all floodplain encroachments, the development must be adequately protected from flooding and erosion, and must not violate restrictions imposed by area plans, basin-management plans, or Mayor and Council policies. At no time may an encroachment adversely affect the river's stability or adversely alter flooding conditions on other properties. Although the limit of encroachment under these circumstances is more flexible, it is still subject to review and approval by the City Floodplain Engineer. When encroachment is proposed within the floodplain of a watercourse, the City Floodplain Engineer may, at his discretion, request that a detailed study be performed to determine if a reduction in overbank flood storage will significantly affect downstream flood peaks.

When fill material is placed in an encroachment area for the purpose of creating a building pad or pads, each pad must be adequately protected against erosion. In cases where these building pads will be placed outside the limits of an erosion-hazard area, as defined in Chapter VII of this Manual, erosion protection shall be designed using the hydraulic parameters associated with the overbank flow. If the building pads will be located inside an erosion-hazard area, erosion protection shall be designed to reduce the erosion-hazard area by using the hydraulic parameters associated with the main channel. See Section 8.5.5 and Chapter VI of this Manual for information on bank-protection toe-down design.

In some cases, the City will require that the existing riparian vegetation be preserved or enhanced. Therefore, it may not be possible to alter a wash or to provide certain types of bank protection, because doing so would result in the loss of significant riparian vegetation. However if, as with most small washes, the riparian vegetation exists only along the banks of the wash, it may be possible to construct erosion protection of some type outside of this vegetation zone. The width of this zone shall be determined on a case-by-case basis, as reviewed and approved by the City Engineer.

Individual building sites may encroach into a floodplain under circumstances where the sites would be completely surrounded by floodwaters during a regulatory flood provided that (1) the general requirements of the floodplain ordinance are met; (2) the fill slopes for any building pad or pads are protected from erosion; and (3) all-weather access is provided to all building sites.

Erosion protection for the building pads shall be designed using the post-development hydraulic conditions of the overbank floodwaters in the immediate vicinity of the building site. No building shall be built within the erosion-hazard setback limit associated with the main channel, unless adequate bank protection (running the entire length of the development) is first installed to prevent lateral migration of the main channel in the direction of the development. Fill material used to elevate individual building sites must extend at least twenty-five feet away from the building in all directions, unless a study or analysis prepared by an Arizona Registered Professional Civil Engineer demonstrates that a lesser distance is acceptable or that the fill is protected from erosion. In addition, the elevation of the building pad must not be lower than the 100-year water-surface elevation. In all cases, the pad or structure must not worsen flooding on other property.

All-weather access in wide floodplains must be along an obvious, commonly used route that can be easily found by drivers of emergency vehicles who may be unfamiliar with the area. Thus, all-weather-access criteria shall apply to the entire all-weather-access route.

8.5 Constructed Channels

In many cases, the proposed density of a development will require the use of constructed channels. When such a use is permitted, constructed channels can minimize floodplain widths, thereby maximizing the developable area. However, the increased flow velocities generally associated with constructed channels often mandate that constructed channels be stabilized in order to prevent bed and/or bank erosion. Channelization and lining allows the channel alignment to be modified, to a certain degree, in order to accommodate urban development. Therefore, in most cases in the past, engineers and planners have found it easier, and more economical, to restructure a given parcel using constructed channels than to plan the development around natural channels. However, this policy of channelization has resulted in a significant reduction of riparian vegetation and habitat, as well as other adverse effects such as increased downstream flood peaks and channel erosion.

The following discussion provides the basic design criteria for the design of constructed channels. More specific and detailed information can be obtained in the published material cited in the "References and Selected Bibliographies" section found at the end of this Manual.

8.5.1 Channel Geometry

Open drainage channels shall be designed using either trapezoidal, rectangular, or compound cross sections, unless the prior approval of an alternate design is granted, in writing, by the City of Tucson Floodplain Engineer.

8.5.1.1 Side-Slopes

Side-slopes for constructed earthen or riprap channels shall be no steeper than 3:1, unless an approved soils analysis demonstrates that steeper side-slopes are stable. Side-slopes for lined channels may be steeper, depending upon the structural stability of the lining. Reinforced concrete lining may have vertical side-slopes, provided that the design is adequate to prevent failure from hydrostatic or earth pressures. Shotcrete may be placed on side-slopes as steep as 1:1, if these side-slopes are not significantly steeper than the natural angle of repose of the soil. A soil-cement lining may be placed on 1:1 side-slopes, provided it is of sufficient thickness to be structurally stable. The minimum thickness of soil cement on a 1:1 side-slope should be four feet, measured normal to its face. Where soil cement is used as slope paving, with a thickness no greater than one foot, the maximum allowable side-slope should be 4:1. Actually, for ease of construction, even flatter side-slopes (e.g., 6:1) are desirable under such circumstances.

8.5.1.2 Width

Ordinarily, the minimum bottom width of a channel must be ten feet before it will be accepted for maintenance by the City of Tucson. Occasionally, bottom widths as narrow as eight feet may be allowable in certain cases, with prior approval from the City of Tucson Floodplain Engineer. Privately maintained channels have no mandatory, minimum bottom width, except as dictated by hydraulic and/or sediment-transport considerations, as described in subsequent sections of this chapter.

The bottom width of constructed channels which lack bed and/or bank protection should not vary by more than fifteen percent between control points, such as at culverts, junctions, changes in slope, or abrupt contractions or expansions, except at the confluence of a major tributary. The purpose of this constraint is to prevent severe aggradation, degradation, or bank erosion from occurring due to sudden changes in sediment-transport rates. In addition, when channelizing a natural wash, the bottom width should be constructed so that the discharge per unit width within the engineered channel is approximately equal to the discharge per unit width of the natural channel of the wash. Typical ways to mitigate this latter constraint are (1) to line both the bottom and sides of the engineered channel, or (2) to line just the channel sides and install grade-control structures.

The bottom widths of constructed channels which have earthen bottoms should be designed to prevent the formation of an incised, meandering, low-flow channel. Theoretically, a relatively wide channel, designed to convey the 100-year discharge, would convey the more-frequent, low-flow discharges at very shallow depths, were there an equal flow distribution across the entire flow cross section. However, by the laws of nature, such an occurrence is not the case within an alluvial channel. Under such circumstances, the channel will develop a narrow, incised, low-flow channel for more efficient conveyance of these flows. This low-flow channel will often meander within the main channel, and is capable of eroding earthen banks and/or undermining bank protection along engineered channels. In order to avoid this occurrence, the channel either should be stabilized, in order to prevent the formation of an incised low-flow channel, or should be designed so that the following equation is satisfied:

$$\frac{b}{V_{p100}Y_{p100}} \leq 1.15 \qquad \textbf{(8.1)}$$

Where:

b	=	Channel bottom width, in feet;
V_{p100}	=	Average velocity of flow at the peak of a 100-year flood, in feet per second; and,
Y_{p100}	=	Maximum depth of flow at the peak of a 100-year flood, in feet.

8.5.1.3 Depth

The depth of flow in channels, where relatively steady, uniform-flow conditions exist, can be computed by an iterative solution of Manning's equation:

$$Q = \frac{1.486}{n} R^{2/3} S_f^{1/2} A \qquad \textbf{(8.2)}$$

Where:

Q	=	Discharge, in cubic feet per second;
n	=	Manning's roughness coefficient (see Table 8.1);
R	=	Hydraulic radius of flow (= A/P), in feet;
A	=	Cross-sectional area of flow, in square feet;
P	=	Wetted perimeter of flow, in feet; and,
S_f	=	Friction slope, in feet per foot.

The depth of flow in Equation 8.2 is implicit within the terms A and R. To solve for the depth of flow, given a known discharge, the normal procedure is to make an estimate of the depth of flow; compute A, P, and R from the channel cross-section characteristics; then solve for Q using Manning's equation. If the computed discharge is not equal to the known discharge, the depth of flow is adjusted accordingly, and the process is repeated until the computed and known discharges are sufficiently close.

Under steady, uniform-flow conditions, the friction slope is assumed equal to the channel slope. Therefore, channel slope can be used for the friction slope, when channels are designed utilizing Manning's Equation.

Uniform flow does *not* exist under most design conditions, due to disturbances caused by changes in the channel width, discharge, or slope. In addition, the presence of channel bends, transitions, junctions, or obstructions such as culverts can create conditions which lead to non-uniform flow. The effect of such disturbances can propagate far upstream, or downstream, depending upon whether or not the flow is subcritical or supercritical. Whenever there is any reason to suspect that uniform-flow conditions do not exist, the depth of flow shall be determined from backwater computations.

TABLE 8.1: MANNING'S ROUGHNESS COEFFICIENTS*

CHANNEL MATERIAL	ROUGHNESS COEFFICIENT (n)		
	Minimum	Normal	Maximum
Corrugated metal	0.021	0.025	0.030
Concrete			
1) Trowel finish	0.011	0.013	0.015
2) Float finish	0.013	0.015	0.016
3) Unfinished	0.014	0.017	0.020
4) Shotcrete, good section	0.016	0.019	0.023
5) Shotcrete, wavy section	0.018	0.022	0.025
Asphalt (use maximum value when cars are present)	0.013	0.016	0.020
Soil Cement	0.018	0.020	0.025
Riprap (bottom and sides)	--	** $n = 0.04D_{50}^{0.167}$	--
Constructed channels with earth or sand bottom, sides of			
1) Clean earth; straight	0.018	0.022	0.025
2) Earth with grass and weeds	0.020	0.025	0.030
3) Earth with trees and shrubs	0.024	0.032	0.040
4) Shotcrete	0.018	0.022	0.025
5) Soil cement	0.022	0.025	0.028
6) Concrete	0.017	0.020	0.024
7) Dry rubble or riprap	0.023	0.033	0.036
Natural channels with sand bottom, sides of			
1) Trees and shrubs	0.025	0.035	0.045
2) Rock	0.024	0.032	0.040
Natural channel with rock bottom	0.040	0.060	0.090
Overbank Floodplains			
1) Desert brush, normal density	0.040	0.060	0.080
2) Dense vegetation	0.070	0.100	0.160

* Adapted from Chow (1959) and Aldridge and Garrett (1973).
** D_{50} in feet.

Backwater computations proceed upstream for subcritical flow and downstream for supercritical flow. A control section must be established for computations to begin. A control section is a section at a place of known water-surface elevation. Control sections can be at such places as channel confluences, culvert inlets, or at where the flow goes through critical depth. Critical depth occurs when the Froude number (F) is equal to one.

The Froude number is calculated from:

$$F = \frac{V}{(gY_h)^{1/2}} \qquad \textbf{(8.3)}$$

Where:

V	=	Average velocity of flow, in feet per second;
Y_h	=	Hydraulic depth of flow (area/top width), in feet; and,
g	=	Acceleration due to gravity = 32.2 ft/sec^2.

Equation 8.3 should be used with care whenever there is overbank flooding or variations across the cross section which cause the flow velocity to vary within the cross section. In such cases, critical depth should be estimated by the graphical method described in Section 4-4 of Chow (1959).

The hydraulic flow depth, Y_h, used in the Froude-number calculation represents the actual flow depth for a rectangular section, but represents the cross-sectional area of flow divided by the top width of flow for either trapezoidal sections or natural channel sections.

Critical depth can occur at locations where a subcritical channel slope changes to a supercritical slope, and at locations where there is an abrupt drop in the elevation of the channel bed, when subcritical flow exists upstream. Backwater calculations should proceed both upstream and downstream from critical depth at locations where a subcritical slope changes to a supercritical slope.

Backwater calculations in trapezoidal channels of uniform cross section are generally performed by the Direct Step Method. This method is easily adaptable to the computer or hand-held calculator. For those who are interested in doing these calculations manually, a very good discussion and description of the Direct Step Method can be found on page 262 of Chow (1959).

8.5.1.4 Freeboard

Freeboard is the additional depth required in a channel beyond the depth which is calculated for conveyance of the design discharge. The purpose of freeboard is to protect against hydraulic disturbances such as waves, unforseen obstructions of flow, debris, or sediment accumulation. In addition, freeboard provides a margin of safety against (1) the uncertainties which exist in the methods used to predict design discharges; and (2) floods that are larger than the design flood. The amount of freeboard required depends upon whether the flow is supercritical or subcritical, the flow velocity, the design discharge, the consequences of overtopping, and the magnitude of flow disturbances at locations such as junctions and culverts.

The freeboard requirement for channels shall be computed from Equation 8.4, with a minimum freeboard of one foot for channels with design depths of three feet or more.

$$FB = 1/6 \left(Y_{max} + \frac{V^2}{2g} \right) \tag{8.4}$$

Where:

FB = Freeboard, in feet;
Y_{max} = Maximum depth of flow, in feet;
V = Average velocity of flow, in feet per second; and,
g = Acceleration due to gravity = 32.2 ft/sec^2.

The freeboard requirements described above are for uniform channel reaches where no unusual flow disturbances are anticipated. Additional freeboard is required at channel bends and junctions, where backwater effects may occur; and at locations where hydraulic jumps may occur. The additional freeboard required at channel bends and junctions is described in Sections 8.5.10 and 8.5.12 of this Manual. At those locations where a hydraulic jump could form, additional freeboard shall be provided to contain the jump according to the guidelines provided within Section 8.5.9 of this Manual.

Freeboard in regional watercourses, such as the Santa Cruz River, Rillito Creek, Tanque Verde Creek, Pantano Wash, and the Cañada del Oro Wash, shall be determined on a case-by-case basis, following a detailed river-mechanics study.

The lining of protected channels shall extend to an elevation necessary to include the freeboard requirement, unless approval to the contrary is granted, in writing, by the City of Tucson Floodplain Engineer.

8.5.2 Safety Considerations

Deep channels with steep side-slopes and high flow velocities can be a hazard to the health, safety, and welfare of the general public. Therefore, the design engineer must always consider the safety aspects of any design. The design of hazardous channels should be avoided, if possible. All channels greater than five feet deep which have side-slopes steeper than 2:1 shall have emergency escape ladders consisting of a series of iron rungs every 600 feet. Other site-specific safety measures shall be installed as deemed necessary by either the design engineer or the City Engineer.

8.5.3 Right-of-Way

All channels that are to be maintained by the City of Tucson must be dedicated to the City. Dedication may be either in fee title or in the form of an easement. The width of the dedication shall be the width of the channel, including key-ins, plus the width of a maintenance access lane or lanes. The minimum maintenance access width for regional watercourses is thirty feet on each side of the channel. More right-of-way may be required, if a linear park is planned along the watercourse. For major watercourses greater than 2000 cfs, the required width for maintenance access is sixteen feet on each side of the channel. However, one of these access lanes may be omitted, at the discretion of the City Engineer, provided that the channel bottom equals or exceeds twenty feet in width, and is drivable utilizing maintenance vehicles. Maintenance access lanes on minor watercourses are variable, and will be established on a case-by-case basis. Generally, a 16-foot maintenance access lane on one side will be required, as a minimum. In all cases, the right-of-way must be sufficient to allow maintenance vehicles to operate freely.

In areas where basin-management plans have recommended particular channel alignments, or an alignment for a watercourse has been established by a regulatory agency, dedication shall be in accordance with same. The width of dedication in these areas shall be as recommended in the basin-management plan, or as established by the agency, unless a more recent study shows that an alternative alignment and/or width is adequate. Studies of this type must clearly demonstrate that there are no conflicts or adverse effects with existing upstream and/or downstream improvements.

8.5.4 Bank-Protection Key-Ins and Minor Side Drainage

Bank-protection key-ins refer to the additional material provided beneath the surface of the ground at the top of the bank protection. Key-ins are normally provided for concrete and shotcrete bank protection; for thin, soil-cement bank protection; and for riprap bank protection. Their purpose is (1) to prevent fractures along the upper edge of the bank protection; (2) to provide added structural stability for the bank protection; and (3) to help prevent minor side inflow from undermining and damaging the bank protection from the top. Typical key-ins are shown in Figure 8.1. The minimum key-in depth on major channels (excluding regional watercourses) shall be eighteen inches. On minor watercourses, the key-in depth shall also be eighteen inches, unless a lesser key-in is justifiable. Key-ins for soil-cement bank protection along regional watercourses are generally not required because of the thickness of the bank protection. However, if key-ins are required, the design shall be determined by a site-specific engineering analysis acceptable to the City Engineer.

When minor tributary or surface flows enter an unlined channel over its side, rill erosion can create headcuts that will travel away from the channel in the opposite direction of the tributary inflow. If the channel is lined, the side drainage can erode the soil from behind the bank protection and create hydrostatic pressures and seepage problems that can cause failure of the bank protection. Therefore, side drainage must be confined to selected entry points that are adequately protected, or the key-in associated with the lining must be deep enough to prevent, or lessen, the buildup of hydrostatic pressure and seepage behind the bank protection. Under such circumstances, and in the absence of a detailed soils analysis and a knowledge of subsurface flow patterns, the key-in shall extend to a depth that equals the depth of the channel along the tributary inflow area.

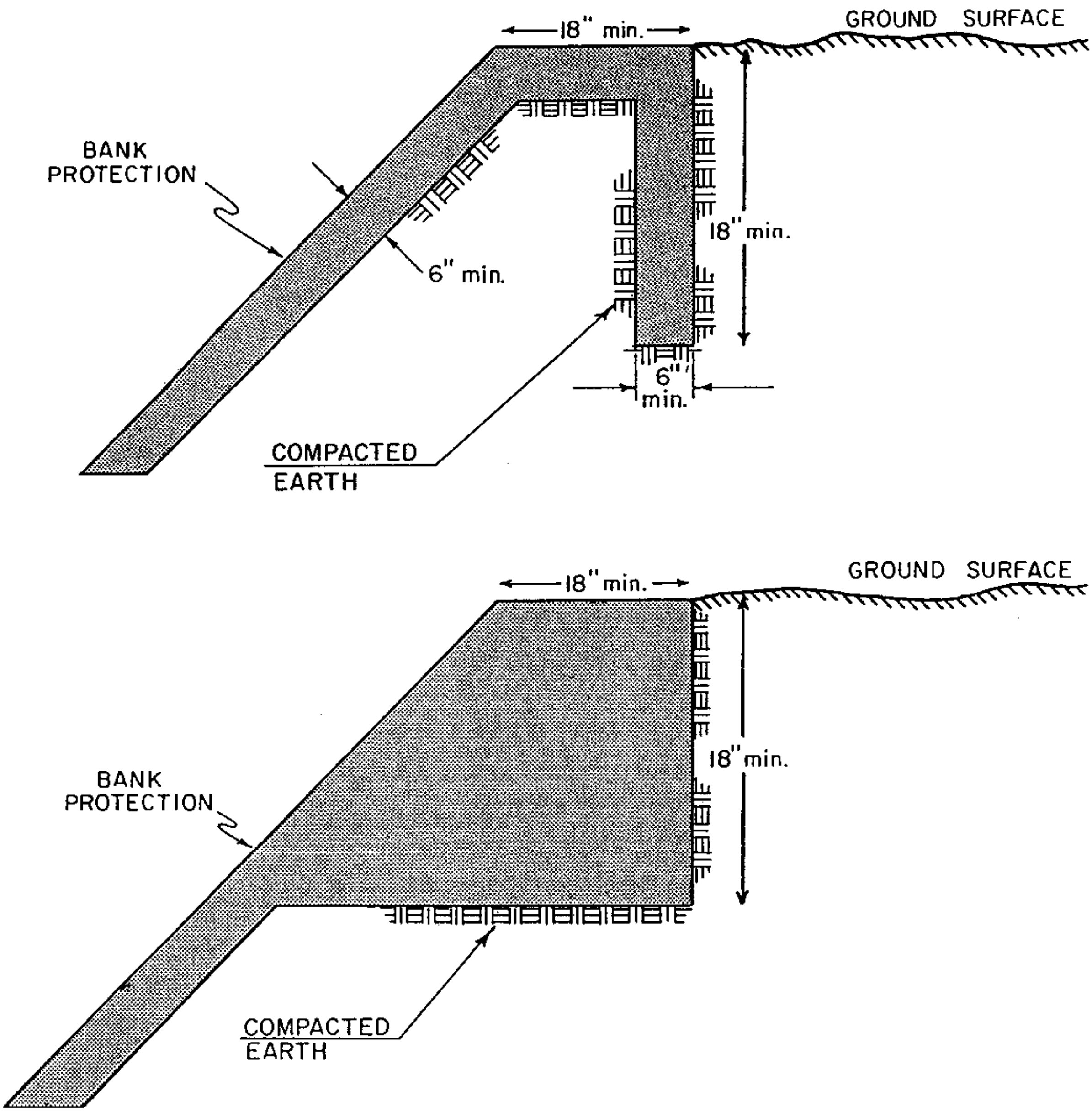


FIGURE 8.1
TYPICAL BANK-PROTECTION KEY-INs
(NOT TO SCALE)

8.5.5 Bank-Protection Toe-Downs

Bank-protection toe-downs refer to the extension of bank protection below the channel bed. Although shallow (i.e., ≤ 6.0 feet) toe-downs are normally vertical, they sometimes are extended below the channel bottom along the same side-slope as the bank itself. The purpose of a toe-down is to prevent failure of the bank protection due to scour or long-term degradation of the channel bed.

Bank-protection toe-downs shall extend to the combined depth associated with general scour, bend scour, local scour, low-flow incisement, sand-wave troughs, and long-term degradation predicted to occur within the channel. The procedures used in calculating these depths are presented in Chapter VI of this Manual. Below grade-control structures, the toe-down shall conform to the geometry of the scour hole, as determined by the methodology also presented in Chapter VI of this Manual.

The soil beneath the channel bed may contain erosion-resistant material, such as caliche. The scour depth calculated using the methodologies outlined in Chapter VI of this Manual may then become unrealistic. A geotechnical report which demonstrates that the bed is composed of erosion-resistant material may be submitted by a soils engineer to justify a reduction in the toe-down depth. However, the toe-down depth along major washes shall never be less than four feet, nor shall toe-downs along minor washes be less than one-half the depth of flow, unless bedrock is encountered.

8.5.6 Low-Flow and Compound Channels

8.5.6.1 Low-Flow Channels

Frequently, the design of a drainage channel that conveys the 100-year discharge leads to a situation in which the bottom of the channel cross section is too wide to efficiently convey the low-flow discharges. As a consequence, these more frequent discharges will create an incised low-flow channel that may meander back and forth across the bed of the channel, instead of allowing flow to spread uniformly across the entire channel width. This meandering process can cause frequent and unnecessary scouring at the toe of the primary banks; and, if left unchecked, can ultimately threaten both the horizontal and vertical stability of the channel. This meander action might even have the capability to destabilize totally lined channels by attacking the lining at the joint between the toe of the bank and the channel bottom. To avoid this meandering process, it is recommended that consideration be given to constructing a small low-flow channel within any larger channel in order to restrict the low flows to a designated area within the primary channel. This low-flow channel should be designed, where practicable, in a manner such that the unit discharge associated with the 2-year event is the same as that which exists under natural conditions. However, practical considerations may require that the low-flow channel, if installed, be somewhat smaller.

8.5.6.2 Compound Channels

A variation upon the concept of a constructed low-flow channel is the compound channel. A compound channel contains a significant portion of the design discharge in a stabilized lower channel. A terrace on each side of the stabilization contains the remainder of the design discharge at a level above the low-flow channel. This terrace may or may not be stabilized. Compound channels are normally constructed in order to satisfy a multi-use concept (e.g., flood-control channels combined with linear parks). The Appendix to this Manual contains more information on the construction of compound channels.

When a compound channel is to be constructed within the corporate limits of the City of Tucson, the normal design discharge to be used in the low-flow portion of such a channel should be the 2-year to 10-year discharge. Because of the potential for erosion of a compound channel terrace during a large discharge event, bank protection which consists of a thin shell, or "veneer," over the supporting embankment is not recommended for these channels. However, observations made during major flood events in the Tucson area indicate that 9-foot-thick soil cement will remain in place following extensive removal of the bank material behind it. Therefore, this "massive" type of bank protection is recommended for the banks of a low-flow channel constructed within a compound channel, unless technical evidence can be provided to the City Engineer which clearly demonstrates that an alternate approach will function effectively within such a channel during a large discharge event. Because hydraulic roughness varies over the cross section of a compound channel, the hydraulic roughness must be "weighted" to develop a composite roughness coefficient for determining the correct depth/discharge relationship. Equation 6-18 in *Open-Channel Hydraulics* (Chow, 1959) is recommended for use in "weighting" roughness coefficients for compound channels.

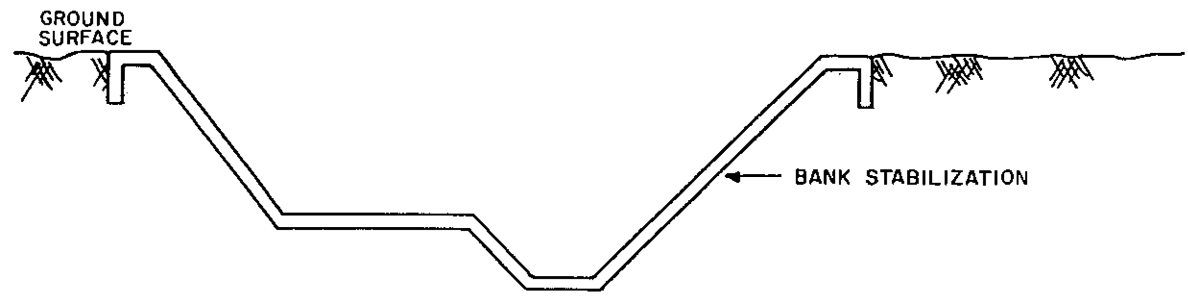
Since compound channels are normally maintenance intensive, they may not be accepted for maintenance by the City of Tucson. The City Engineer will evaluate the acceptability of these channels on a case-by-case basis. The City Engineer may also increase building setbacks from compound channels over those normally associated with completely lined channels, should the erosion potential of the affected watercourse warrant an increased setback. Figure 8.2 illustrates typical cross sections for low-flow and compound channels.

8.5.7 Upstream and Downstream Controls

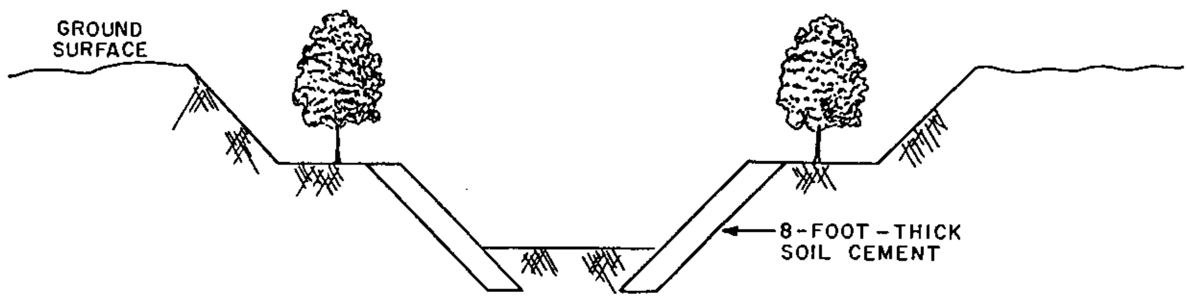
The upstream end of constructed channels must be designed to collect the entire design discharge without raising water-surface elevations on adjacent properties. This may be accomplished by providing wide entrance transitions, or collector channels, at the upstream end. See Section 8.5.11.1 of this Manual for information on entrance transitions.

The downstream end must also be designed to minimize adverse impacts upon adjacent properties. Adverse impacts could result from increased discharge, velocity, or concentration of flow. Mitigation measures to reduce or eliminate these impacts can be achieved by (1) providing expansions at the downstream end of the channels; (2) providing energy-dissipation structures; or (3) building box culverts at street crossings. See Section 8.5.11.2 of this Manual for information on exit transitions.

Drainage must be collected and delivered in the same manner and to the same concentration points that existed prior to channelization, unless a drainage master plan for the area dictates otherwise; or unless an agreement acceptable to the City Engineer



CHANNEL WITH LOW-FLOW CHANNEL (NTS)



COMPOUND CHANNEL (NTS)

FIGURE 8.2

TYPICAL LOW-FLOW AND COMPOUND CHANNELS

is obtained from all affected property owners. If a drainage master plan is available, dedication of all necessary rights-of-way shall be required, as specified within the master plan.

8.5.8 Channel Slope

The slope for a proposed channel is, to a great extent, dependent on the natural topography. However, variations can be achieved by altering the channel alignment within a development, and by adjusting the elevation of inflow and outflow points.

In general, channels with unlined bottoms should not be designed with a slope less than 0.3% in order to prevent vegetation and bed irregularities from creating stagnant pools of water after flows subside. Channels with a concrete bottom may be flatter. Where the natural fall of the land is less than 0.5%, the channel alignment producing the steepest possible slope should be chosen to avoid sediment buildup.

Abrupt changes in slope should be avoided, except where necessary to achieve a specific purpose (e.g., such as to induce a hydraulic jump). For example, if an abrupt change in slope might result in the formation of a hydraulic jump that is not desired, an analysis should be performed to determine whether a jump will occur, and where it will be located. When abrupt slope changes are unavoidable, the slope changes should not cause the channel top width to vary by more than fifteen percent.

Whenever possible, channels should be designed to convey the incoming sediment supply without causing aggradation or degradation. Refer to Chapter VI of this Manual, which addresses erosion and sedimentation, for more detailed information.

Channels with design Froude numbers between 0.86 and 1.16 should be avoided, if at all possible, because of the instability associated with critical flow.

Most channels with earthen beds are constructed on slopes that are steeper than their equilibrium slopes. In such cases, grade-control structures are required. Refer to Chapter VI for grade-control design guidelines.

8.5.9 Hydraulic Jump

A hydraulic jump occurs when flow changes rapidly from low-stage supercritical flow to high-stage subcritical flow. Hydraulic jumps can occur (1) when the slope of the channel abruptly changes from steep to mild; (2) at sudden expansions or contractions in the channel section; (3) at locations where a barrier, such as a culvert or bridge, occurs in a channel of steep slope; (4) at the downstream side of dip crossings or culverts; (5) where channels of steep slope discharge into other channels; and (6) at sharp bends.

Hydraulic jumps are useful in dissipating energy, and consequently they are often purposely forced to occur at drainageway outlet structures in order to minimize the erosive potential of floodwaters. However, because of the large amount of energy dissipated in hydraulic jumps, it is not advisable to allow them to occur except under controlled circumstances. Therefore, if during the design of a channel, it appears that a hydraulic jump might occur at an undesirable location, computations should be made to determine the height, length, and characteristics of the jump. In addition, steps should be taken to either eliminate the jump or contain it, in order to prevent damage to the channel or surrounding property.

The type of hydraulic jump that forms, and the amount of energy it dissipates, is dependent upon the upstream Froude number, F_1.

The various types of hydraulic jumps that can occur are listed in Table 8.2.

TABLE 8.2: TYPES OF HYDRAULIC JUMPS

UPSTREAM FROUDE NUMBER	TYPE OF JUMP	ENERGY LOSS (%)
$1 < F_1 \leq 1.7$	Undular Jump	0-5
$1.7 < F_1 \leq 2.5$	Weak Jump	5-18
$2.5 < F_1 \leq 4.5$	Oscillating Jump	18-44
$4.5 < F_1 \leq 9$	Steady Jump	44-70
$F_1 > 9$	Strong Jump	70-85

8.5.9.1 Height of a Hydraulic Jump

The depth of flow immediately downstream of a hydraulic jump is referred to as the sequent depth. The sequent depth in rectangular channels can be computed by use of the following equation:

$$Y_2 = 1/2\ Y_1\ ([1 + 8F_1^2]^{1/2} - 1) \tag{8.5}$$

Where:
Y_1 = Initial (upstream) flow depth, in feet;
Y_2 = Sequent (downstream) flow depth, in feet; and,
F_1 = Froude number upstream of the jump = $V_1/(gY_1)^{0.5}$, where V_1 = initial (upstream) flow velocity, in feet per second.

The solution for sequent depth in trapezoidal channels can be obtained from a trial-and-error solution of Equation 8.6. Equation 8.6 is derived from momentum equations (see Morris and Wiggert, 1972). It is also acceptable, for design purposes, to determine the sequent depth in trapezoidal channels from Equation 8.5. Equation 8.5 is much simpler to solve, and produces only slightly greater values for sequent depth for trapezoidal channels than does Equation 8.6.

$$\frac{ZY_1^3}{3} + \frac{bY_1^2}{2} + \frac{Q^2}{gA_1} = \frac{ZY_2^3}{3} + \frac{bY_2^2}{2} + \frac{Q^2}{gA_2} \qquad \textbf{(8.6)}$$

Where:

Y_1 & Y_2 are as defined in Equation 8.5; and,
b = Channel bottom width, in feet;
Z = Channel side-slope (horizontal/vertical), in feet per foot;
Q = Channel discharge, in cubic feet per second; and,
A_1 & A_2 = Cross-sectional areas of flow upstream and downstream, respectively, of the hydraulic jump, in square feet.

Figures 8.3 and 8.4 can also be used to determine the height of a hydraulic jump.

8.5.9.2 Length of a Hydraulic Jump

The length of a hydraulic jump, L, is defined as the distance from the front face of the jump to a point immediately downstream of the roller. Jump length can be determined from Figures 8.5 and 8.6.

8.5.9.3 Surface Profile of a Hydraulic Jump

The surface profile of a hydraulic jump may be needed to design the profile of extra bank protection, or training walls, required to contain the jump. The surface profile can be determined from Figure 8.7.

8.5.9.4 Location of a Hydraulic Jump

In most cases, a hydraulic jump will occur at the location in a channel where the initial and sequent depths and upstream Froude number satisfy Equation 8.5. This location can be found by performing direct-step calculations in either direction toward the suspected jump location, until the terms of the equation are satisfied. Refer to Section 15.7 of Chow (1959) for detailed information and an example on locating hydraulic jumps.

8.5.9.5 Undular Hydraulic Jumps

An undular hydraulic jump is the type of jump which occurs where the upstream Froude number is between 1.0 and 1.7. This type of jump is characterized by a series of undular waves which form on the downstream side of the jump. Experiments have shown that the first wave of an undular jump is higher than the height given by Equation 8.5. Therefore, the height of this wave should be determined as follows:

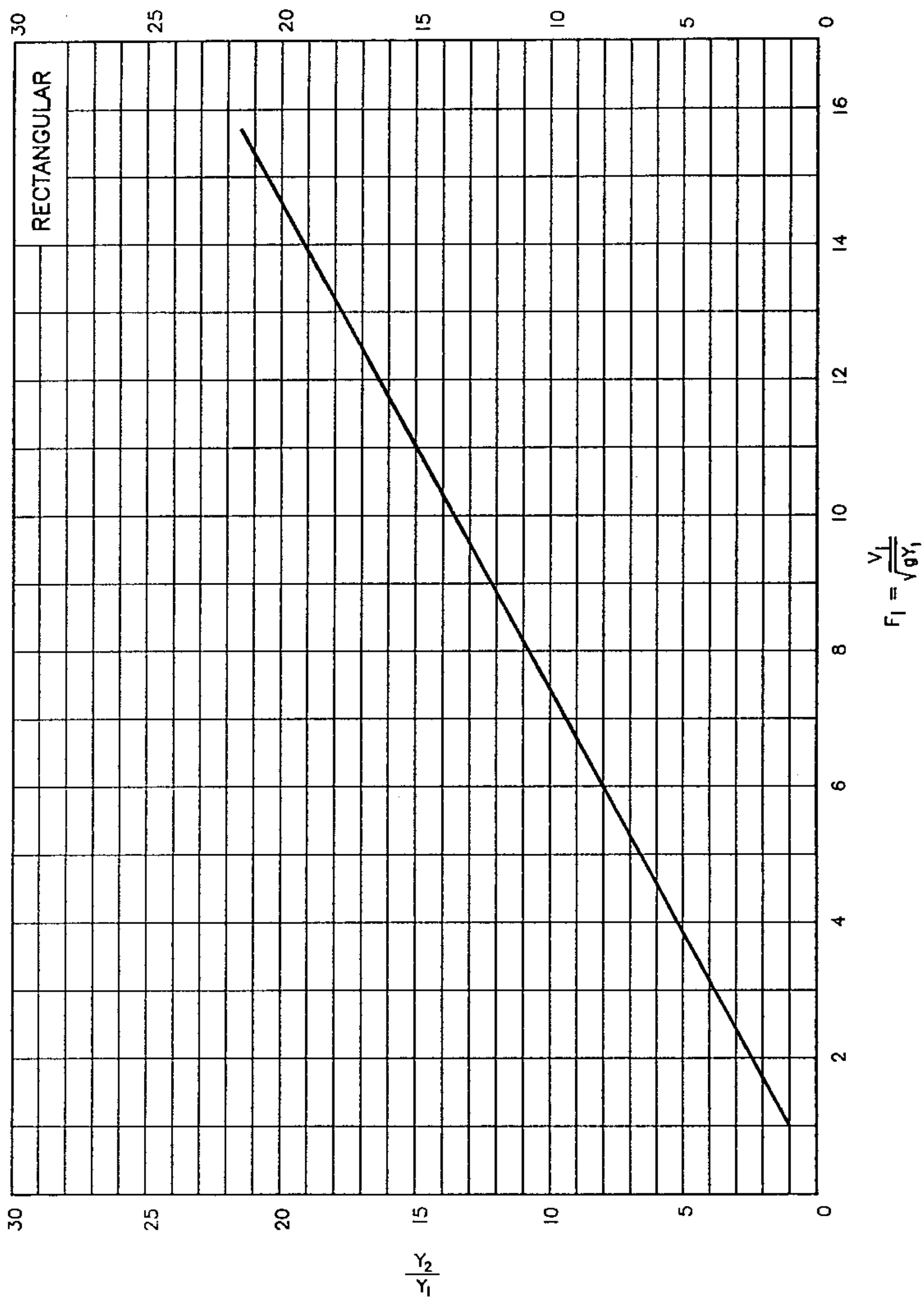


FIGURE 8.3
HEIGHT OF A HYDRAULIC JUMP FOR A HORIZONTAL, RECTANGULAR CHANNEL

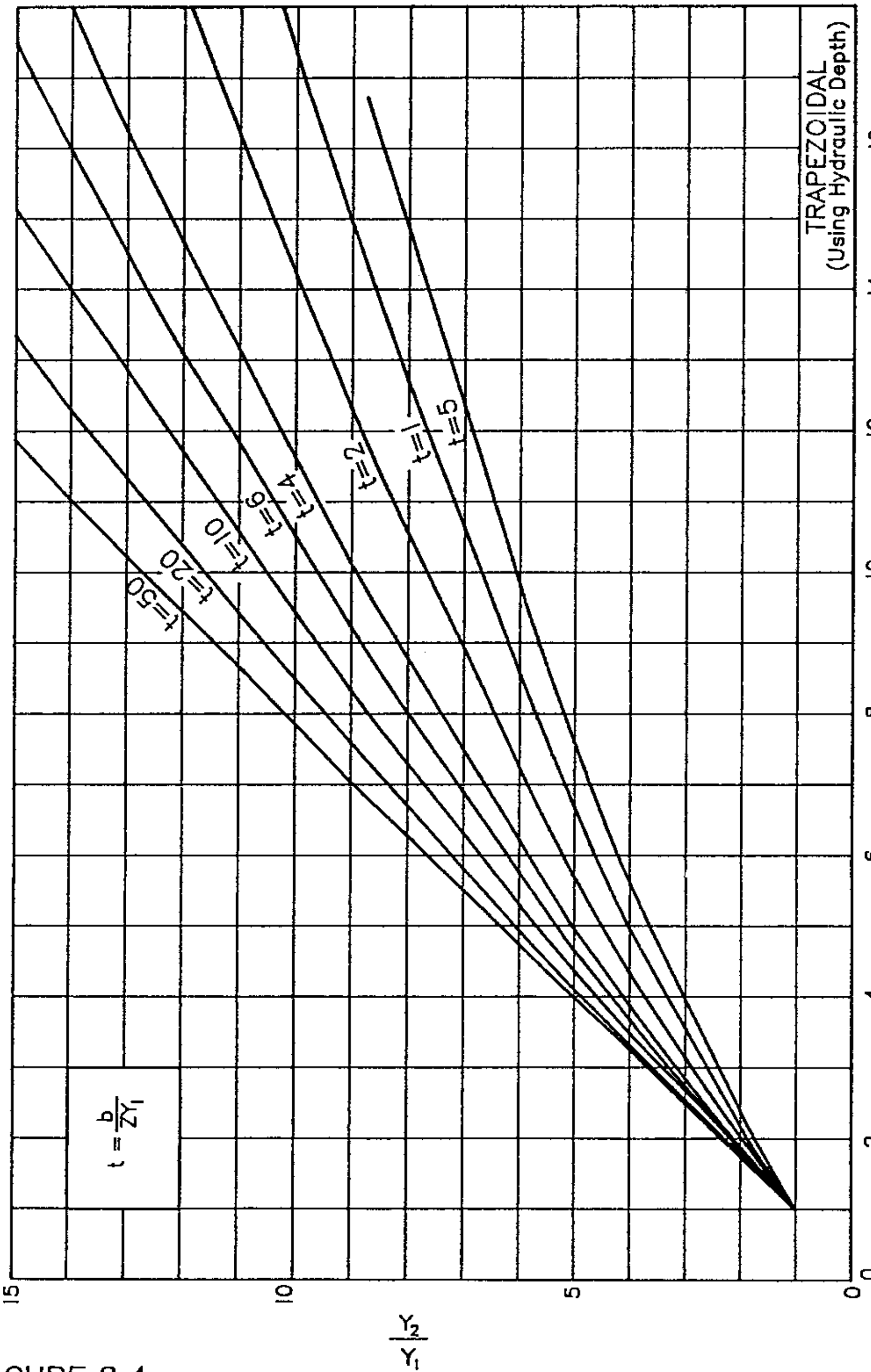


FIGURE 8.4
HEIGHT OF A HYDRAULIC JUMP FOR A HORIZONTAL, TRAPEZOIDAL CHANNEL (USING HYDRAULIC DEPTH)

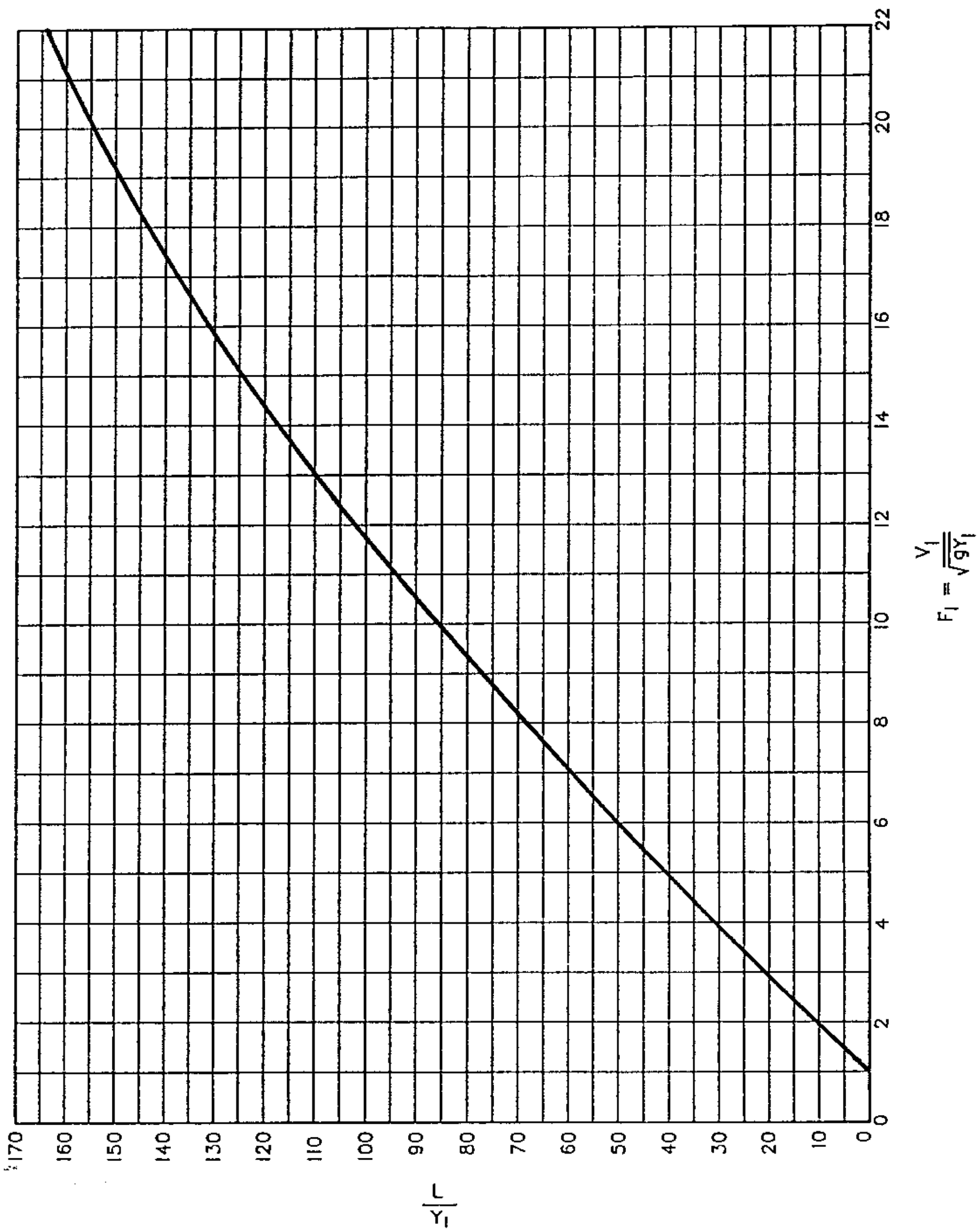


FIGURE 8.5
LENGTH OF A HYDRAULIC JUMP FOR A RECTANGULAR CHANNEL

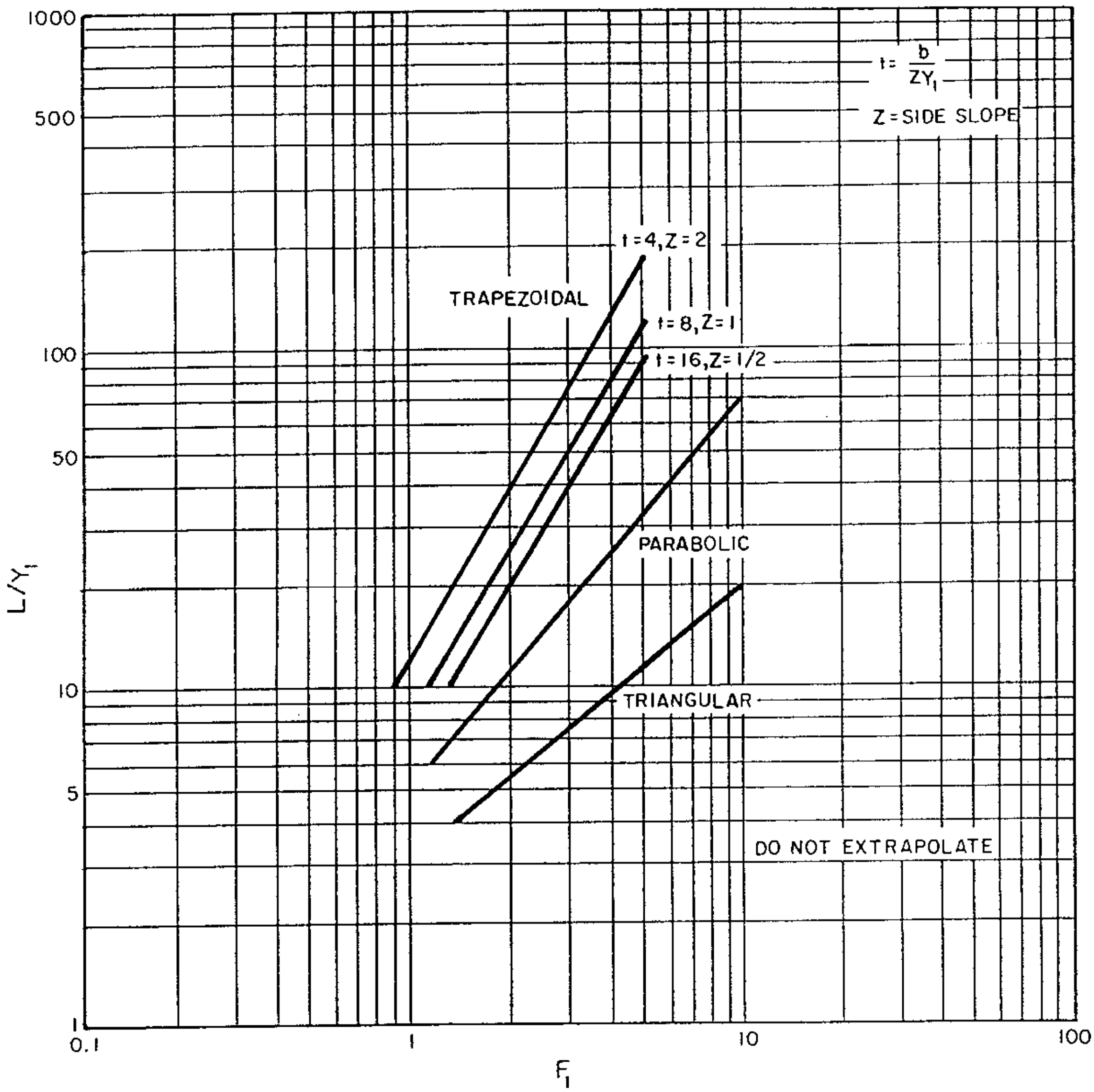


FIGURE 8.6
LENGTH OF A HYDRAULIC JUMP FOR NONRECTANGULAR CHANNELS

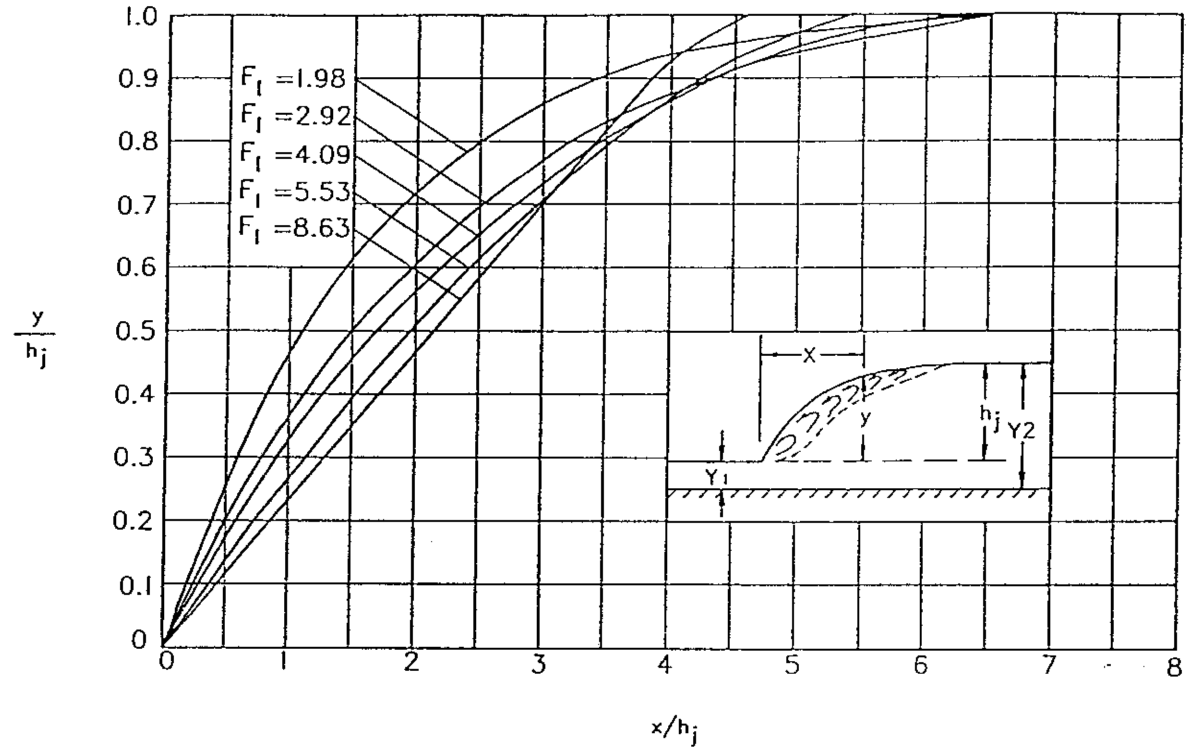


FIGURE 8.7
SURFACE PROFILE OF A HYDRAULIC JUMP

$$\frac{Y_2 - Y_1}{Y_1} = F_1^2 - 1 \tag{8.7}$$

Where all terms are as previously described.

See U.S. Army Corps of Engineers (1970) for the source of this equation.

8.5.10 Flow in a Curved Channel

Flow in a curved channel will create centrifugal forces which will cause a rise in the water surface along the outside of a bend. At the same time, a corresponding depression will be created in the water surface along the inside of the bend. In addition, spiral secondary currents tend to form within the bends. These currents can cause scour to occur along the outside of a bend, and deposition along the inside of a bend. Cross waves that propagate downstream will also form, if the flow around the bend is supercritical.

Although curves are inevitable in the design of most open channels, they should be minimized in order to avoid the special problems associated with their design. The design of channel bends must include consideration for superelevation, limiting curvature, bend scour, and special design curves.

8.5.10.1 Superelevation

Superelevation is the rise in the water-surface elevation around the outside of a channel bend, with an accompanying lowering of the water surface along the inside of the bend. This outside rise in the water surface is generally measured with respect to the mean depth of flow in an equivalent straight reach. Additional freeboard is required along the outside of a channel bend to account for this rise (see Figure 8.8). Superelevation is computed as follows:

$$\Delta Y = \frac{1.5 C V^2 T}{g r_c} \tag{8.8}$$

Where:

ΔY = Rise in water-surface elevation (superelevation) around the outside of a channel bend, in feet;
C = A coefficient (see Table 8.3);
V = Average velocity of flow, in feet per second;
T = Channel width at elevation of water surface, in feet;
g = Acceleration due to gravity (= 32.2 ft/sec^2);
r_c = Radius of curvature of channel centerline, in feet; and,
1.5 = Factor of safety to account for alluvial channel conditions.

See U.S. Army Corps of Engineers (1970) for the source of this equation.

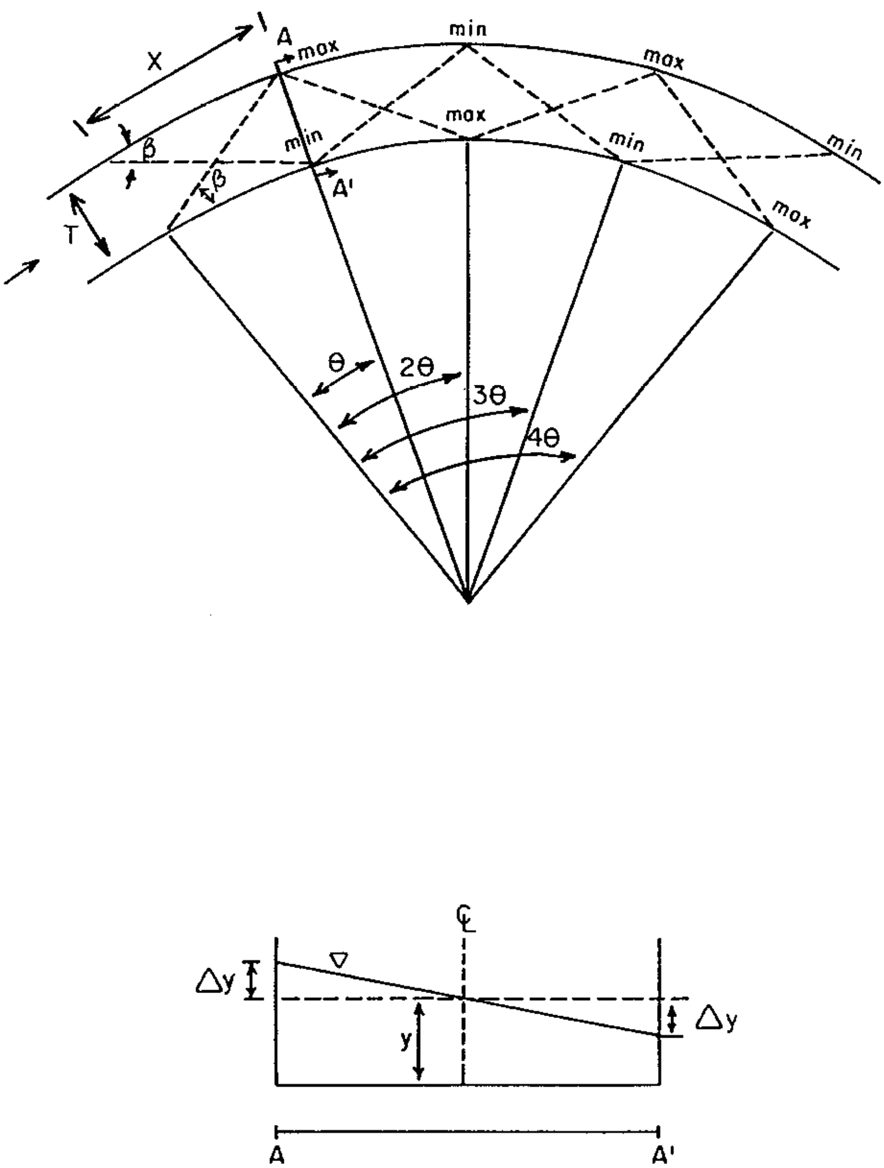


FIGURE 8.8

CROSS-WAVE PATTERN FOR SUPERCRITICAL FLOW IN A CURVED CHANNEL

The coefficient C in Equation 8.8 takes into account the rise due to cross waves and centrifugal forces.

TABLE 8.3: COEFFICIENTS FOR USE IN SUPER-ELEVATION FORMULA

FLOW TYPE	CROSS SECTION	TYPE OF CURVE	VALUE OF C
Tranquil	Rectangular	Simple Circular	0.5
Tranquil	Trapezoidal	Simple Circular	0.5
Rapid	Rectangular	Simple Circular	1.0
Rapid	Trapezoidal	Simple Circular	1.0
Rapid	Rectangular	Easement Transition	0.5
Rapid	Trapezoidal	Easement Transition	1.0

For subcritical flow, the upstream and downstream limits of additional freeboard shall correspond to the beginning and ending points of curvature according to the guidelines in U.S. Army Corps of Engineers (1970). The normal channel freeboard is expected to be adequate to contain any backwater effects of the superelevation upstream of the curve.

For supercritical flow, the disturbances caused by bends (cross waves) can propagate far downstream of the bend. Therefore, special treatment is required to eliminate or minimize these disturbances. Figure 8.8 shows a typical cross-wave pattern. The central angle of the cross-wave pattern, θ, is computed by use of the following equation:

$$\theta = \tan^{-1}\left[\frac{2b}{(2r_c+b)\tan\beta}\right] \qquad \textbf{(8.9)}$$

Where:

θ	=	Central angle of the cross-wave pattern, in degrees;
b	=	Channel bottom width, in feet;
r_c	=	Radius of curvature of channel centerline, in feet;
β	=	Wave front angle = $\sin^{-1}(1/F)$, in degrees; and,
F	=	Froude number.

See Rouse (1950) for the source of this equation.

Freeboard to account for superelevation in channels with supercritical flow shall begin at the upstream point of curvature, and continue at that level to a point downstream of the end of the curve a distance computed by Equation 8.10.

$$L' = \frac{3\,T}{\tan\beta} \qquad \textbf{(8.10)}$$

Where:

L'	=	Distance of *maximum* superelevation downstream of a curve in a channel with supercritical flow, in feet.

All other terms are as defined previously.

Beyond this point, freeboard to account for superelevation shall taper downward to the normal bank-protection height over an *additional* distance equal to $0.67L'$.

8.5.10.2 Easement Curves

Easement curves can be used to reduce cross waves in bends with supercritical flow (see Table 8.3). Easement curves are placed at both ends of the curve proper, and may be either spiral or circular in order to produce the same hydraulic effect. Circular easement curves are recommended, and must have a radius equal to twice the radius of the main curve. The length of the easement curve, L_e, is computed by:

$$L_e = \frac{0.32\,TV}{Y^{1/2}} \qquad \textbf{(8.11)}$$

Where all terms are as previously described.

8.5.10.3 Banking

Banking is an alternative to providing additional freeboard in order to contain superelevated flows around a channel bend. Banking is a modification of the cross slope of the channel bed such that the inside of the bend is lower than the outside of the bend. When banking a channel, the difference in elevation between the inside of a bend (lowest point) and the outside of a bend (highest point) should be equal to the quantity V^2T/gr_c, in feet, where all terms are as previously defined. Hydraulically, this method is preferable to providing additional freeboard, but banking is difficult to construct. Therefore, *banking should only be used in conjunction with the design of totally lined channels.*

8.5.10.4 Limiting Curvature

For flow with a design Froude number less than 0.86, the minimum radius of curvature along the center line of the channel shall be three times the channel top width. For flow with a Froude number greater than or equal to 0.86, the minimum radius of curvature shall be computed as follows:

$$r_c = \frac{4V^2T}{gY_h} \qquad \textbf{(8.12)}$$

Where:

r_c = Radius of curvature of channel centerline, in feet;
V = Average velocity of flow, in feet per second;
T = Channel top width at the water surface, in feet; and,
Y_h = Hydraulic depth of flow, in feet.

See U.S. Army Corps of Engineers (1970) for the source of this equation.

The radius of curvature for channels with design Froude numbers greater than or equal to 0.86 shall not be less than $4T$.

8.5.11 Transitions

Transition sections designed to collect and/or discharge flow between the natural floodplain and constructed channels can be located at either the upstream or downstream ends of the constructed channels. They can also be located along a segment, or segments, of a constructed channel itself. In either case, it is necessary to design the flow transition to minimize the disturbance to flow. In the case where flow in a constructed channel is being transitioned back to the natural floodplain, sufficient distance must be allowed for the flow to adequately expand to the original width of the natural floodplain.

8.5.11.1 Entrance Transitions

When the upstream width of flow in a natural channel exceeds the width of proposed channel, a transition section must be provided. For subcritical flow, the angle of convergence, θ, between the center line of the proposed channel and the transitioning levee, or bank, is computed by use of the following equation:

$$\theta = \tan^{-1}\left(\frac{1}{3.375\ F_u}\right) \qquad \textbf{(8.13)}$$

Where:

θ = Transition angle, in degrees (see Figure 8.9); and,

F_u = Upstream Froude number.

See Pima County Department of Transportation and Flood Control District (1984) for the source of this equation.

The length, L, of the transition is computed by use of the following equation:

$$L = \frac{\Delta T}{2\tan\theta} \tag{8.14}$$

Where ΔT is the change in top width, in feet.

See U.S. Army Corps of Engineers (1970) for the source of this equation.

The maximum allowable transition angle is thirty degrees, unless supplemental engineering calculations demonstrate to the satisfaction of the City Engineer that an angle greater than thirty degrees can be used.

In addition to the design calculations associated with the transition section, a backwater analysis must be performed to determine what effect, if any, the transition will have upon upstream water levels.

The transition losses, h_t, to be used in the backwater analysis are to be computed by use of the following equation:

$$h_{tc} = C_c \Delta h_v \tag{8.15a}$$

or

$$h_{te} = C_e \Delta h_v \tag{8.15b}$$

Where:

h_{tc}, h_{te} = Transition losses in contracting and expanding reaches, respectively, in feet;

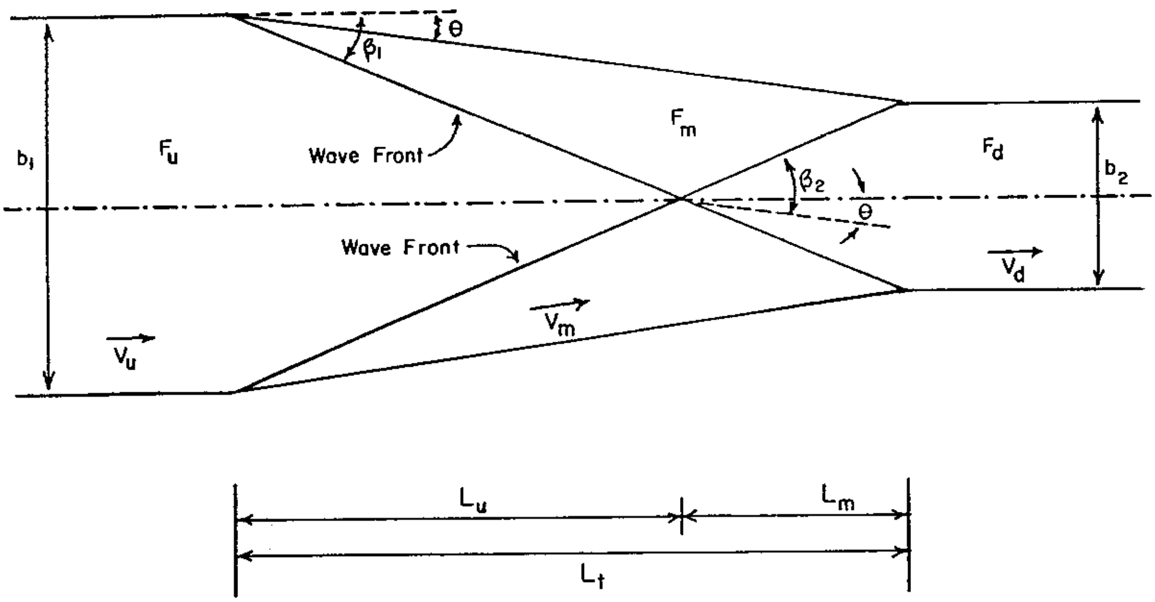
C_c = Coefficient of contraction;

C_e = Coefficient of expansion; and,

Δh_v = Difference in velocity head between the upstream and downstream end of the transition, in feet.

See U.S. Army Corps of Engineers (1970) for the source of this equation.

The head-loss coefficients of expansion and contraction, C_c and C_e, are obtained from the following table:



NOTE: The subscripts u, m, and d represent upstream, midpoint, and downstream flow conditions, respectively.

FIGURE 8.9

TRANSITION FOR CHANNEL CONTRACTIONS IN SUPERCRITICAL FLOW

TABLE 8.4: HEAD-LOSS COEFFICIENTS FOR USE WITH TRANSITIONS IN OPEN CHANNELS

TRANSITION TYPE	C_c	C_e
Warped	0.10	0.20
Cylindrical quadrant	0.15	0.25
Wedge	0.30	0.50
Straight Line	0.30	0.50
Square End	0.30	0.75

For supercritical flow, entrance transitions must be designed to prevent flow disturbances which could propagate downstream. The convergence angle, θ (Figure 8.9), must be chosen to minimize cross-wave action. To accomplish this, the following three equations must also be satisfied:

$$(1) \quad L_1 = \frac{b_1}{2\mathrm{Tan}\beta_1}; \qquad \textbf{(8.16)}$$

and,

$$(2) \quad L_2 = \frac{b_2}{2\mathrm{Tan}(\beta_2-\theta)}; \qquad \textbf{(8.17)}$$

and,

$$(3) \quad L = L_1 + L_2 \qquad \textbf{(8.18)}$$

$$(4) \quad L = \frac{b_1 - b_2}{2\tan\theta} \qquad \textbf{(8.19)}$$

Where all terms are as defined in Figure 8.9.

See U.S. Army Corps of Engineers (1970) for the source of these equations.

The procedure for design of a supercritical transition is as follows:

1. Using the upstream Froude number, F_u, compute the wave-front angle, β_1, from the formula:

$$\beta = \mathrm{Sin}^{-1}(1/F) \tag{8.20}$$

   Where:

   $\beta = \beta_1$ and $F = F_u$ (see Figure 8.9).

2. Compute the distance L_1 from Equation 8.16.
3. Choose a trial transition length, L, where $L > L_1$.
4. Determine the trial transition angle, θ, from L, b_1, and b_2.
5. Determine the transition Froude number, F_t, from the hydraulic conditions at the distance L_1.
6. From F_t, compute a new wave-front angle, β_2, using Equation 8.20.
7. Compute L_2 according to Equation 8.17.
8. Repeat steps 3 through 7 until Equations 8.18 and 8.19 are both satisfied.

The table below is provided as an additional guide to aid in designing entrance transitions under supercritical flow conditions.

TABLE 8.5: RECOMMENDED CONVERGENCE RATES FOR ENTRANCE TRANSITIONS WITH SUPERCRITICAL FLOW

MEAN CHANNEL VELOCITY (FPS)	WALLFLARE (HORIZONTAL TO LONGITUDINAL)	θ DEGREES
10 - 15	1:10	5.71
15 - 30	1:15	3.81
30 - 40	1:20	2.86

8.5.11.2 Exit Transitions

The length of the exiting transition section, L_{TR}, where flow from the proposed channel is expanded to match the width of the natural floodplain, shall be computed by use of the following equation:

$$L_{TR} = 6.5\ (X_2 - 0.7X_1) \qquad \textbf{(8.21)}$$

for subcritical flow ($F_u \leq 1$); and

$$L_{TR} = 6.5\ F_u\ (X_2 - 0.7X_1) \qquad \textbf{(8.22)}$$

for supercritical flow ($F_u > 1$).

Where the terms for both equations are as described in Figure 8.10 (F_u = upstream Froude number).

Equations 8.21 and 8.22 are modified from equations found on Plate 24 of "Hydraulic Design of Flood Control Channels," U.S. Army Corps of Engineers (1970).

Exit transition sections are necessary to prevent adverse downstream impacts caused by increased flow velocities and depths. Acceptable transitions are required in all cases unless (1) an agreement, satisfactory to the City Engineer, can be made with all affected downstream property owners; or (2) a drainage master plan has been developed for the wash, which specifies a particular outlet configuration.

8.5.11.3 Internal Channel Transitions

Internal channel transitions must be gradual to minimize flow disturbances. The same formulas presented in the previous sections for entrance and exit transitions shall be used for contractions and expansions of flow within the channel. For transitions which constrict flow under subcritical conditions, use Equation 8.13 to determine the convergence angle. The maximum transition angle shall be thirty degrees. The length of the transition is computed by using Equation 8.14.

Contractions under supercritical flow conditions are computed by using Equations 8.16 through 8.20. The required length for internal expansions under supercritical flow conditions is computed by using Equation 8.22. Should a shorter transition be desired, it must be justified by computations that document the expected wave heights in accordance with procedures contained in standard hydraulics textbooks, such as Chow (1959) and Morris and Wiggert (1972). Additional freeboard, and possibly additional reinforcement of the channel lining, will be required to account for the destructive effects associated with wave formation.

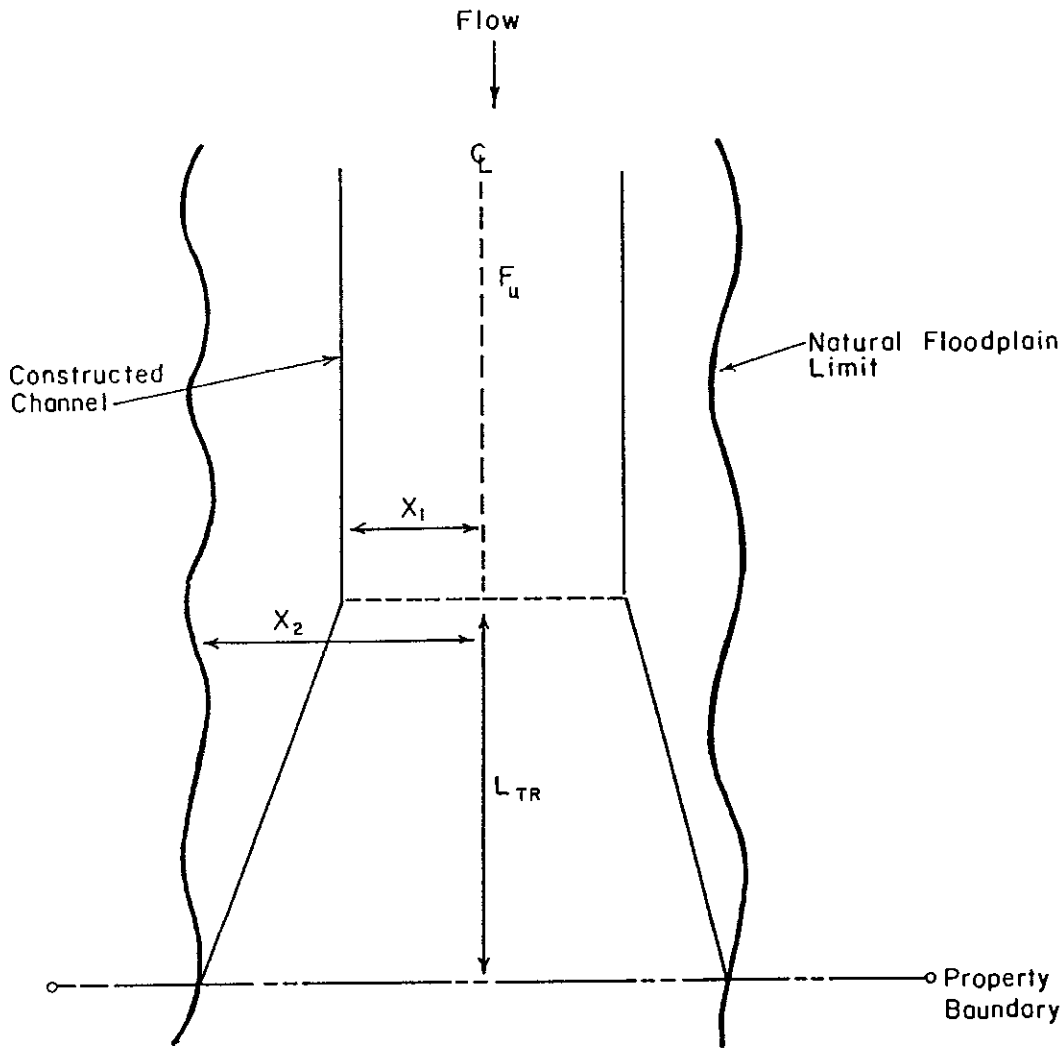


FIGURE 8.10
TRANSITION DISTANCE REQUIRED TO ALLOW FLOW TO RETURN TO NATURAL CONDITIONS

Where flow is to be transitioned from a supercritical state to a subcritical state, a hydraulic jump will develop. The jump must be contained within the transition structure. Additional freeboard will be required, as needed, to contain the jump (refer to Section 8.5.12 of this Manual for information on hydraulic jumps). Additional reinforcement of the channel lining may also be required. One method of ensuring that a hydraulic jump is contained within the designated area is to build an energy dissipator or stilling basin that is designed to contain the jump within a specified reach length. Refer to Chapter IX of this Manual for more detailed information concerning energy dissipators and/or stilling basins.

8.5.12 Channel Confluences

The design of a channel junction or a channel confluence is a very complex procedure due to the many variables involved (e.g., the angle of intersection, discharges, channel and junction shape, and the number of adjoining channels and type of flow encountered). Junctions under subcritical flow conditions must be designed to allow water to merge without creating a backwater condition that can result in the overtopping of one or more of the converging channels. The maximum wave height is generally located on the side-channel wall opposite the junction point, and on the main-channel wall downstream of the junction.

8.5.12.1 General Design Guidelines

General design guidelines for junctions are as follows:

1. Tapered training walls should be constructed between adjoining flows.

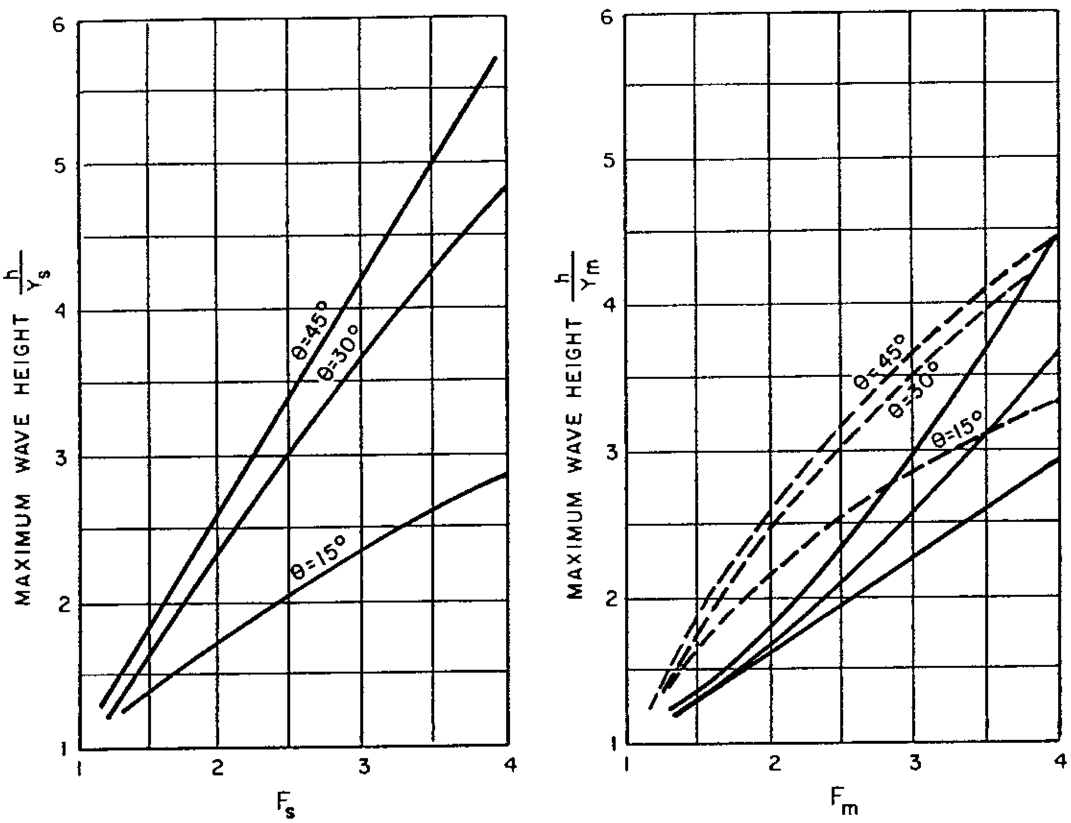
2. The side-channel wave originating at the junction apex should impinge upon the main-channel wall downstream of the enlargement (see Section 8.5.12.3 of this Manual).

3. Junction angles, θ, should be no greater than twelve degrees for subcritical flow, and no greater than six degrees for supercritical flow. Angles greater than these are acceptable, but only if extra bank protection is provided to heights equal to or greater than the maximum wave heights given by Figure 8.11. In addition, if the tributary flow is greater than ten percent of the main channel flow, the maximum angle of the confluence should not be allowed to exceed forty-five degrees. The extra height of bank protection required at a junction should extend downstream of same a distance, L, which is computed from the following equation:

$$L = \frac{3b_2V_2}{V_3 sin\theta} \tag{8.23}$$

Where:

b_2 = Bottom width of the main channel downstream of the junction, in feet;



NOTE: The subscripts m and s represent the main channel and side channel flow conditions, respectively.

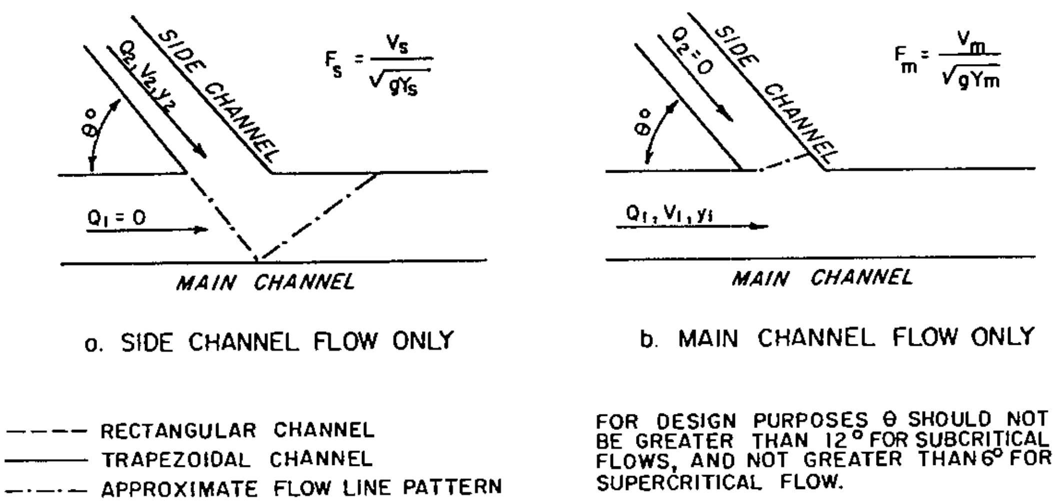


FIGURE 8.11
MAXIMUM WAVE HEIGHT AT A CHANNEL JUNCTION

V_2 = Flow velocity in the main channel downstream of the junction, in feet per second;
V_3 = Flow velocity in the tributary or side channel, in feet per second; and,
θ = Junction angle, in degrees.

Tributary flows that are less than ten percent of the main channel flow may enter at angles up to ninety degrees, but only if extra bank protection is provided to a height that equals the elevation of the energy grade line of the tributary flow. If the angle of confluence is greater than forty-five degrees, the extra bank protection must extend upstream of the junction at least for a distance equal to the bottom width of the tributary channel.

4. Critical flow conditions at junctions should be avoided, if at all possible. Froude numbers should either be below the value 0.86, or greater than the value 1.13.

5. Transition sections should be avoided in the immediate vicinity of junctions.

8.5.12.2 Momentum Equation

Open-channel flow at a junction is best analyzed using the principle of conservation of momentum. There are many momentum-balance equations available that make simplifying assumptions about the flow and confluence configuration. These equations should be used with caution, because many design situations will not adequately meet the assumptions implicit in these equations.

A series of equations developed by the Los Angeles Flood Control District (1973) are of sufficient detail to be applicable for most junctions. These equations shall be used for designing projects to be located within the City of Tucson, unless the engineer can justify using other equations. The general form of the momentum equation is:

$$P_{h2} + M_2 = P_{h1} + M_1 + M_3\cos\theta + P_{hi} + P_{hw} - P_{hf} \tag{8.24}$$

Where:
P_{h1} = Hydrostatic pressure on Section 1;
P_{h2} = Hydrostatic pressure on Section 2;
P_{hi} = Horizontal component of hydrostatic pressure on the channel invert;
P_{hw} = Axial component of hydrostatic pressure on the channel walls;
P_{hf} = Retardation force of friction;
M_1 = Momentum of the moving mass of water entering the junction at Section 1;

M_2	=	Momentum of moving mass of water leaving the junction at Section 2; and,
$M_3\text{Cos}\theta$	=	Axial component of momentum of the moving mass of water entering the junction at Section 3.

Figures 8.12 and 8.13 show the relationship between the main channel and the tributary channel with respect to the preceding equation. For a trapezoidal channel, the following equations represent the variables comprising Equation 8.24:

$$M_1 = \frac{Q_1^2}{g(b_1+Z_1Y_1)Y_1} = \frac{Q_1^2}{gA_1} \tag{8.25}$$

$$M_2 = \frac{Q_2^2}{g(b_2+Z_2Y_2)Y_2} = \frac{Q_2^2}{gA_2} \tag{8.26}$$

$$M_3 = \frac{Q_3^2}{g(b_3+Z_3Y_3)Y_3} = \frac{Q_3^2}{gA_3} \tag{8.27}$$

$$P_{h1} = \frac{Y_1^2}{6}(3b_1 + 2Z_1Y_1) \tag{8.28}$$

$$P_{h2} = \frac{Y_2^2}{6}(3b_2 + 2Z_2Y_2) \tag{8.29}$$

$$P_{hi} = \frac{b_1+b_2}{2}h_d\left[Y_1 + \frac{(Y_2-Y_1)(b_1+2b_2)}{3(b_1+b_2)}\right] \tag{8.30}$$

$$P_{hw} = \frac{Y_1+Y_2}{4}\left[\frac{b_1+b_2}{2}(Y_1-Y_2)+h'(Z_1Y_1+Z_2Y_2)+(b_2+Z_2Y_2)Y_2-(b_1+Z_1Y_1)Y_1\right] \tag{8.31}$$

$$P_{hf} = \frac{L(S_1+S_2)}{4}\left[(b_1 + Z_1Y_1)Y_1 + (b_2+Z_2Y_2)Y_2\right] \tag{8.32}$$

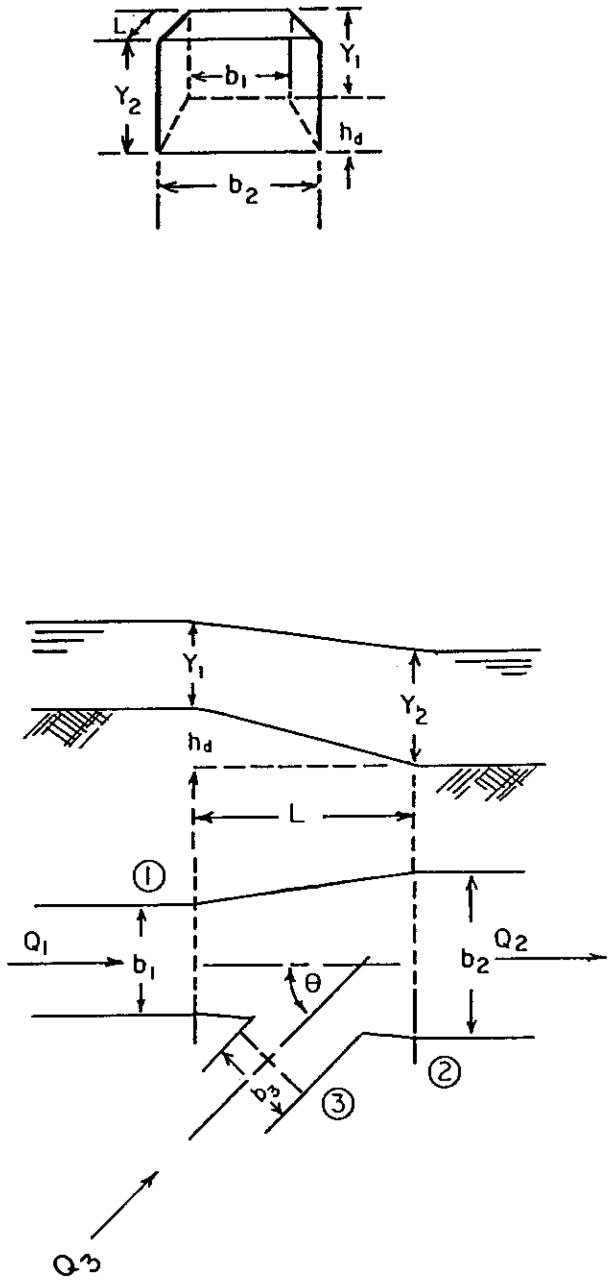


FIGURE 8.12
RECTANGULAR CHANNEL JUNCTION

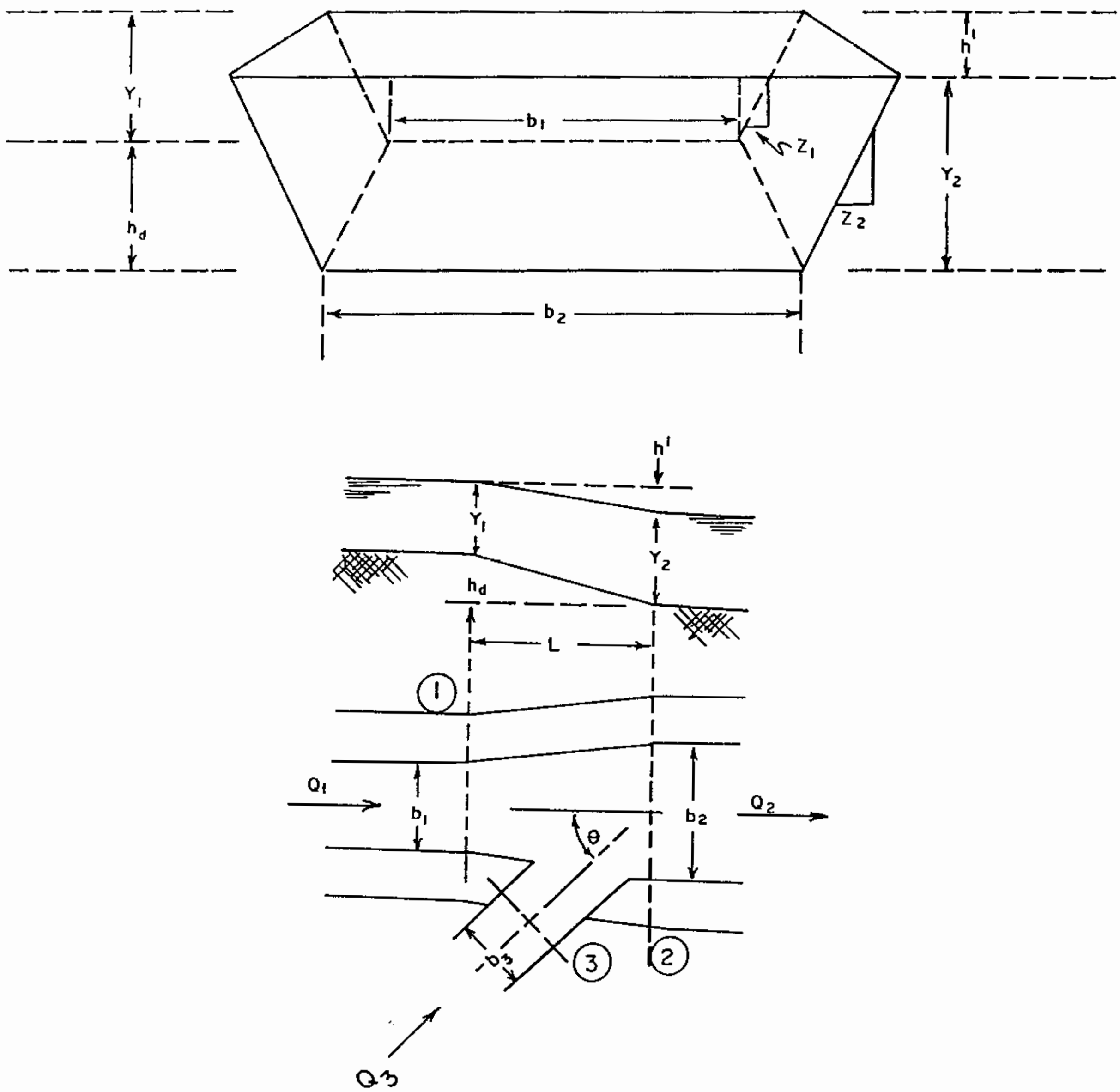


FIGURE 8.13
TRAPEZOIDAL CHANNEL JUNCTION

For a rectangular channel, the equations which represent the variables comprising Equation 8.24 are:

$$M_1 = \frac{Q_1^2}{g b_1 Y_1} \qquad (8.33)$$

$$M_2 = \frac{Q_2^2}{g b_2 Y_2} \qquad (8.34)$$

$$M_3 = \frac{Q_3^2}{g b_3 Y_3} \qquad (8.35)$$

$$P_{h1} = \frac{b_1 Y_1^2}{2} \qquad (8.36)$$

$$P_{h2} = \frac{b_2 Y_2^2}{2} \qquad (8.37)$$

$$P_{hi} = h_d \left(\frac{b_1+b_2}{2}\right) \left(Y_1 + \frac{(Y_2-Y_1)(b_1+2b_2)}{3(b_1+b_2)}\right) \qquad (8.38)$$

$$P_{hw} = \frac{Y_1+Y_2}{4}(b_2-b_1)\left[Y_1 + \frac{(Y_2-Y_1)(Y_1+2Y_2)}{3(Y_1+Y_2)}\right] \qquad (8.39)$$

$$P_{hf} = \frac{L(S_1+S_2)}{4}(b_1 Y_1 + b_2 Y_2) \qquad (8.40)$$

Where:

b_1, b_2, b_3	=	Bottom widths of channels 1, 2, and 3, respectively, in feet;
Y_1, Y_2, Y_3	=	Flow depths in channels 1, 2, and 3, respectively, in feet;
Q_1, Q_2, Q_3	=	Discharges in channels 1, 2, and 3, respectively, in cubic feet per second;
Z_1, Z_2, Z_3	=	Side-slopes of channels 1, 2, and 3, respectively (horizontal/vertical), in feet per foot;
S_1, S_2	=	Friction slopes of channels 1 and 2, respectively.
L	=	Length of channel junction, in feet.

h_d = Vertical drop in channel bottom through the junction, in feet; and,

h' = Vertical drop in water surface through the junction, in feet.

8.5.12.3 Design Procedure: Supercritical Flow

The design of junctions under supercritical flow conditions involves an iterative procedure in which different curve layouts are checked against the momentum equation until one is found that is acceptable. The upstream channel widths and hydraulic conditions are known, while the downstream channel width and depth of flow are the unknown parameters. The procedure is as follows:

1. Assume a downstream width of the channel bottom based upon the total discharge, the approximate channel shape, the selected roughness, and the slope. It is suggested that the first estimate of the width be the combined width of the two upstream channels (b_1+b_3).

   (In the following discussion b_1 refers to the upstream width of the main channel, b_2 is the width of the main channel at the downstream end of the junction, b_3 is the width of the secondary [tributary] channel, and b_4 is the width of the main channel downstream and beyond the influence of the junction).

2. Prepare the confluence layout assuming that the main-channel walls are parallel to the channel center line, as shown in Figure 8.14. If the difference (Δb_1) in widths between b_1 and b_2 is less than b_3, a centerline offset, as shown in Figure 8.14 A, is recommended. If Δb_1 is greater than b_3, an offset with respect to the right bank, as shown in Figure 8.14 B, is recommended to ensure that the horizontal distance between the parallel alignment of the left banks of the main channels b_1 and b_2 is equal to or less than b_3.

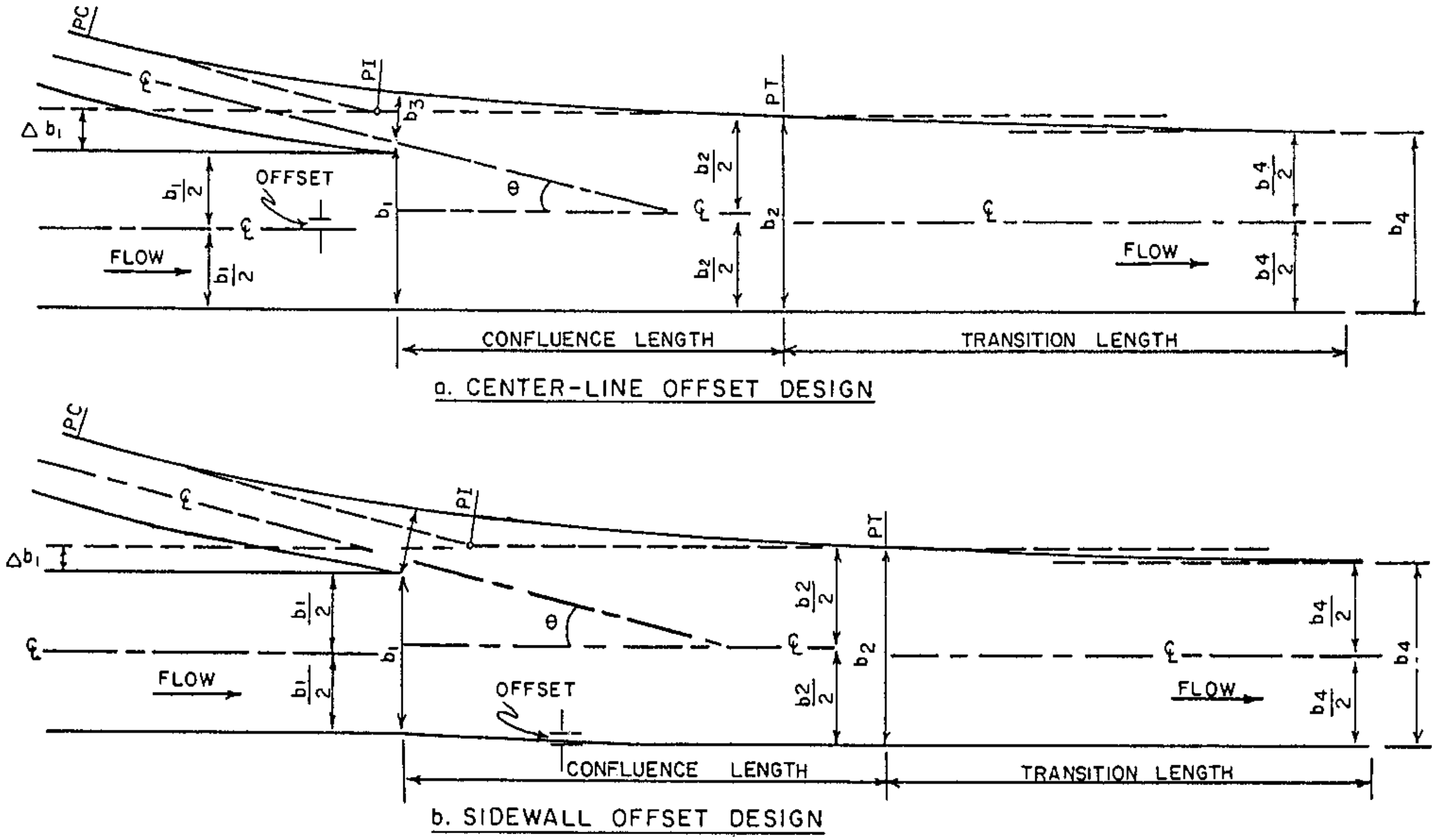
3. Using the point of intersection, *PI*, of the channel walls, draw a circular curve which is determined by the apex angle, θ (Figure 8.14). The radius of curvature of the curve is determined by use of the following equation:

$$r_c = \frac{4V_3^2 b_3}{gY_3} + 400 \qquad \textbf{(8.41)}$$

   Where all terms are as previously defined.

   See U.S. Army Corps of Engineers (1970) for the source of this equation.

   This curve will connect the intersecting, straight channel walls, as shown in Figure 8.14, and will represent the revised edge of the bottom of the channel through the confluence area.



*FOR DESIGN PURPOSES, θ SHOULD BE NOT GREATER THAN 12 DEGREES FOR SUBCRITICAL FLOW AND NOT GREATER THAN 6 DEGREES FOR SUPERCRITICAL FLOW.

FIGURE 8.14
TYPICAL CONFLUENCE LAYOUTS

4. Make the inside bank of the tributary channel bottom concentric with the circular curve, and locate the apex of the junction at the point where this edge of the tributary channel meets the main channel. The distance, L_o, between the Point of Tangency, *PT*, and the junction apex is computed by use of the following equation:

$$L_o = (r_c + b_3)\,\mathrm{Sin}\left\{\mathrm{Cos}^{-1}\left[1 - \frac{b_3 - \Delta b_1}{r_c + b_3}\right]\right\} \tag{8.42}$$

The total length of the curve, L_{ct}, from *PC* to *PT*, can be computed from:

$$L_{ct} = \frac{2\pi r\theta}{360} \tag{8.43}$$

The location of the point of curvature, *PC*, can be computed from:

$$PC = PI - r\,\mathrm{Tan}\,\frac{\theta}{2} \tag{8.44}$$

The location of the point of Tangency, *PT*, can be computed from:

$$PT = PC + L_{ct} \tag{8.45}$$

Figure 8.14 shows the relationship of these parameters.

5. Compute the confluence length, L_c, by using the following equations (Los Angeles County Flood Control District, 1973):

$$L_{c1} = \frac{b_3}{\mathrm{Sin}\theta}\,; \tag{8.46}$$

and,

$$L_{c2} = 5(b_2 - b_1) \tag{8.47}$$

and compare these lengths to the distance L_o (Equation 8.42) from the apex to the *PT* point. The longest of the three is the confluence length, L_c.

6. Using the confluence length, L_c, and the momentum equation (i.e., Equation 8.24), determine the depth of flow and hydraulic conditions at b_2. If either the depth of flow or water-surface elevation is significantly different than its value in the upstream main channel, select a new b_2 and repeat the procedure.

7. Once satisfactory hydraulic conditions at b_2 have been established, determine the transition distance to b_4 by the procedure outlined in Section 8.5.11 of this Manual. (Note: b_2 in Section 10.5.11 of this Manual is equivalent to b_4 in this section of the Manual.)

   An example of this procedure is provided at the end of this chapter.

When designing junctions, consideration should be given to the waves that will occur along the opposite channel wall when only one of the converging channels is discharging into the composite channel. Due to the sporadic nature of thunderstorms in the Tucson Area, it is possible to have flow in one channel and not in the other. Under supercritical flow conditions, experiments have shown that waves can be quite high, particularly if the angle of confluence is excessive. Fortunately, if the angle of confluence is equal to or less than twelve degrees (preferably six degrees), and the design procedure described above is followed, these type of waves should not be a problem. However, should a greater angle of confluence be dictated by site conditions, extra freeboard will be required according to the procedure described in Section 8.5.12.1(3) of this Manual in order to contain waves created by flow impinging onto the opposite bank.

8.5.13 Collector Channels

Collector channels are generally designed to collect unconsolidated sheet flow, or wide, shallow, braided flow for the purpose of removing the downstream property from the floodplain. Collector channels generally do not follow the existing drainage pattern. Therefore, they have more stringent design requirements than do most other channels.

8.5.13.1 Cross Section and Slope

Collector channels provide the best hydraulic performance if the width/depth ratio is as low as possible. Cross sections with wide bottoms and low depths should be avoided, if topography permits. Channel slopes should be as steep as reasonably possible to help accelerate the water and prevent sediment buildup.

8.5.13.2 Depth

The discharge in collector channels increases with distance along the channel. Collector channel flows are subject to head losses associated with the impact and turbulence created by flow entering the channel over its bank, in addition to the normal losses created by friction. Therefore, normal-depth procedures and step-backwater calculations are *not* applicable. The correct procedure for analyzing spatially-varied flow of the type that occurs in collector channels is given in many hydraulics textbooks under the heading "Side-Channel Spillways" (e.g., see Chow, 1959, page 329).

The minimum depth of a collector channel in the City of Tucson shall be twice the critical depth of the design flood for channels with supercritical slopes, and twice the normal depth of the design flood for channels with subcritical slopes. This depth will vary along the length of the channel as the discharge increases. The transition from the collector channel to the main channel shall be designed using standard backwater procedures. Backwater computations should begin at the point where inflow over the side of the collector channel ceases, and end at a point where normal depth is encountered, or where flow is no longer affected by the collector channel.

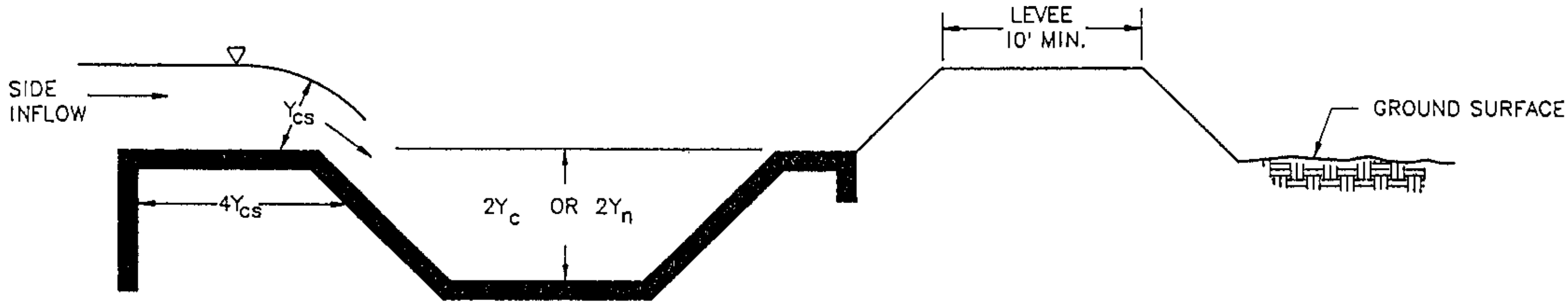
When unusual circumstances exist, such as the presence of a definite control point at or near the end of a collector channel, the "Method of Numerical Integration," as outlined in Chow (1959) shall be used to design same. This method may also be used if there is reason to believe that the guidelines presented above result in an overdesign of a collector channel.

8.5.13.3 Erosion Protection

Erosion protection for a collector channel requires special consideration because of inflow from the side. Hydrostatic pressure in the soil and seepage behind the bank protection can cause the bank protection to fail. Another problem is scour caused by side inflow.

To prevent failure of the bank protection along a collector channel due to side inflow, seepage, and/or hydrostatic pressure, a horizontal concrete apron is normally required along the top of the upstream (inflow) side of the collector channel. This concrete apron shall be connected to the bank protection, and have a width, measured perpendicular to the bank, which is at least four times the critical depth of side inflow during the design flood. A key-in at the upstream edge of the concrete apron should extend to a depth equal to the depth of the collector channel. However, the apron and key-in are not required if the channel bank is constructed of 9-foot-thick soil cement.

The bottom of the collector channel shall be lined, unless the toe-down protection for the bank is deep enough to protect against the scour caused by side inflow. The procedures given in Chapter VI of this Manual shall be used to compute side-inflow scour depth. Normal-depth shall be used as the tailwater depth in the channel for this equation. If the width of the channel is less than five times the computed scour depth, extra toe-down protection to the full depth of scour is needed on *both* banks. For channel bottom widths at least ten times the depth of scour, no extra toe-down is needed on the opposite bank. For widths between five and ten times the depth of scour, the toe-down on the opposite bank should be computed via a linear interpolation between the side-flow scour depth and the normal toe-down depth. A typical collector channel is shown in Figure 8.15.



Y_{cs} = CRITICAL DEPTH OF SIDE FLOW

Y_c = CRITICAL DEPTH OF CHANNEL FLOW

Y_n = NORMAL DEPTH OF CHANNEL FLOW

NOTE:
A PROPERLY DESIGNED LEVEE OF HEIGHT Y_{cc} OR Y_{nc} CAN BE CONSTRUCTED ALONG THE BANK OPPOSITE THE SIDE INFLOW IN LIEU OF THE $2Y_{cc}$ OR $2Y_{nc}$ REQUIREMENT.

FIGURE 8.15
TYPICAL COLLECTOR–CHANNEL CROSS SECTION

8.5.13.4 Sediment

Depending upon the amount of sediment supply, and upon sediment-transport capacity, a collector channel could either aggrade or degrade, if not properly designed. The reader is referred to Chapter VI of this Manual for those procedures that consider the effects of deposition and/or scour of alluvial sediments upon open-channel design.

8.5.13.5 Additional Design Considerations

Material removed by excavation to form the collector channel could be used to construct a levee along the side opposite the lateral inflow. Such a levee, if properly designed, would then be able to serve as a substitute for the depth requirement otherwise imposed upon the design of a collector channel (i.e., two times the appropriate flow depth), and would ensure that all lateral inflow is captured by the collector channel. The minimum height of such a levee should be equal to the normal depth of flow at the peak of the design flood for subcritical conditions, and equal to the critical depth of flow at the peak of the design flood for supercritical conditions.

The lowest floor of the first tier of buildings along the downstream side of a collector channel should be at least one foot above the 100-year water-surface elevation in the collector channel in order to safeguard against possible failure of the collector-channel embankment. This water-surface elevation shall be determined either by the Method of Numerical Integration or by assuming an elevation equal to either (1) two times the normal depth at the peak of the design flood for subcritical flow, or (2) two times the critical depth at the peak of the design flood for supercritical flow, whichever is greater.

EXAMPLE 8.1: SEQUENT DEPTH IN A TRAPEZOIDAL CHANNEL

A hydraulic jump is to be formed in a trapezoidal channel through the use of baffle blocks and an abrupt change in slope from steep to mild. Hydraulic conditions *upstream* of the jump are:

Discharge (Q)	=	500 cfs
Channel Slope (S)	=	0.015
Bottom Width (b)	=	10 ft
Side Slopes (Z)	=	1:1
Roughness (n)	=	0.015
Depth (Y)	=	2.3 ft
Froude Number (F_u)	=	2.2
Hydraulic Depth (Y_h)	=	1.9 ft

Equation 8.6 will be used. Normal depth *upstream* of the jump is 2.3 feet; so, as an initial estimate, a sequent depth of four feet will be chosen. From Equation 8.6:

$$\frac{1(2.3)^3}{3} + \frac{10(2.3)^2}{2} + \frac{500^2}{g(28.3)} = \frac{1(4)^3}{3} + \frac{10(4)^2}{2} + \frac{500^2}{g(56.0)}$$

$$304.9 = 240.0$$

Momentum does not balance, so a new sequent depth is chosen. By trial and error, the sequent depth is found to be 5.6 ft:

$$\frac{1(2.3)^3}{3} + \frac{10(2.3)^2}{2} + \frac{500^2}{g(28.3)} = \frac{1(5.6)^3}{3} + \frac{10(5.6)^2}{2} + \frac{500^2}{g(87.4)}$$

$$304.9 = 304.2 \qquad \text{(close enough)}$$

The engineer should exercise care in using Equation 8.6, especially with calculator or computer-program "root solvers," because there are two other roots besides the correct one for sequent depth. One obvious solution is $Y_2 = Y_1$. The third root is usually negative. In this case, the value -13.1 also satisfies the equation.

Figure 8.4 can also be used to solve for sequent depth in this example. To do this, first compute $t = 10/[1(2.3)] = 4.3$. From Figure 8.4, using $F_u = 2.2$ and $t = 4.3$, $Y_2/Y_1 = 2.4$. Y_2 is then: 2.4 (2.3 ft) = 5.5 ft.

EXAMPLE 8.2: THE DESIGN OF AN OPEN-CHANNEL JUNCTION UNDER SUPER-CRITICAL FLOW CONDITIONS

In this example, a main-channel flow, Q_1, of 2000 cubic feet per second (cfs) is to be joined by a side-channel flow, Q_3, of 775 cfs. The confluence angle, θ, is six degrees. The slope and bottom-width of the side channel have been established to ensure that the depth of flow at the junction is the same as the depth of flow in the main channel. It is desired that this depth of flow be maintained throughout the junction.

Hydraulic conditions in the section located upstream of the channel junction are as follows:

Main Channel			Side Channel		
Q_1	=	2000 cfs	Q_3	=	775 cfs
n	=	0.015 (concrete)	n	=	0.015 (concrete)
b_1	=	20.0 ft	b_3	=	8.0 ft
Y_1	=	4.0 ft	Y_3	=	4.0 ft
Z	=	1:1	Z	=	1:1
S_1	=	0.01 ft/ft	S_3	=	0.008 ft/ft
F_1	=	2.0	F_3	=	1.70
A_1	=	95.8 ft^2	A_3	=	47.8 ft^2
V_1	=	20.9 fps	V_3	=	16.2 fps

Hydraulic conditions in the composite channel section located downstream of the junction are as follows:

Q_4	=	2775 cfs
n	=	0.015 (concrete)
b_4	=	28.0 ft
Y_4	=	4.0 ft
Z	=	1:1
S	=	0.01 ft/ft
F	=	2.0
A_4	=	127.7 ft^2
V_4	=	21.7 fps

STEP 1: Assume $b_2 = b_1 + b_3 = 20+8 = \underline{\underline{28\ feet}}$.

Use centerline offset (Figure 8.14 A).

$\Delta b_1 = 8\ ft$

$$r_c = \frac{4V_3^2 b_3}{gY_3} + 400 = \frac{4(16.2)^2(8)}{g(4)} + 400 = \underline{\underline{465.2\ feet}}$$

Assume station $PI = \underline{\underline{100+00}}$

Station $PC = 100+00 - r_c \text{ Tan } \frac{\theta}{2}$

Station $PC = 100+00 - 465.2 \text{ Tan } 6/2 = \underline{\underline{99+75.62}}$

Curve length, L_{ct}, $= r_c\ (\theta) \frac{2\pi}{360}$

$$L = 465.2(6) \frac{2\pi}{360} = \underline{\underline{48.72\ feet}}$$

Station $PT = 99+75.62 + 48.72 = \underline{\underline{100+24.34}}$

Because $b_3 = \Delta b_1$, the distance from the apex to PT is 0.

The confluence length, L_c, is:

$$L_c = \frac{(8)}{Sin\ (6°)} = \underline{\underline{76.5\ feet}}$$

or

$$L_c = \frac{(28-20)10}{2} = \underline{\underline{40.0\ feet}}$$

Using the largest of these values yields:

$$L_c = \underline{\underline{76.5\ feet}}$$

Assume the depth of flow at b_2 = *4 feet.*

From Equations 8.25 to 8.32:

$$M_1 = \frac{(2000^2)}{[20+1(4)]g(4)} = \underline{\underline{1294.0}}$$

$$M_2 = \frac{(2775)^2}{[28+1(4)]g(4)} = \underline{\underline{1868.4}}$$

$$M_3 = \frac{(775)^2}{(47.8)g} = \underline{\underline{390.2}}$$

$$P_{h1} = \frac{(4)^2}{6}[3(20) + 2(1)4] = \underline{\underline{181.3}}$$

$$P_{h2} = \frac{(4)^2}{6}[3(28) + 2(1)4] = \underline{\underline{245.3}}$$

$$P_{hi} = \left(\frac{20+28}{2}\right)0.77\left(4 + \frac{(4-4)[20+2(28)]}{3(20+28)}\right)$$

$$= \underline{\underline{73.9}}$$

$$P_{hw} = \frac{4+4}{4}\left(\frac{20+28}{2} \quad (4-4)+0.77\ [1(4)+1(4)]+[28+1(4)]4-4[20+1(4)]\right)$$

$$= \underline{\underline{76.3}}$$

$$P_{hf} = \frac{76.5\ (0.01+0.01)}{4}\left([20+1(4)]4+[28+1(4)]4\right)$$

$$= \underline{\underline{85.7}}$$

Using Equation 8.24:

$$245.3+1868.4 = 181.3+1294.0+390.2\text{Cos}6°+73.9+76.3-85.7$$

$$2113.7 = \underline{\underline{1927.9}}$$

Since forces do not balance, another depth should be tried using the same width.

By trial and error, obtain $D_2 = \underline{\underline{4.5 \text{ feet}}}$.

$$M_2 = \frac{(2775)^2}{[28+1(4.5)]g(4.5)} = \underline{\underline{1635.2}}$$

$$P_{h2} = \frac{(4.5)^2}{6}[3(28) + 2(1)4.5] = \underline{\underline{313.9}}$$

$$P_{hi} = \left[\frac{20+28}{2}\right] 0.77 \left[4 + \frac{(4.5-4.0)[20+2(28)]}{3(20+28)}\right]$$

$$= \underline{\underline{78.8}}$$

$$P_{hw} = \frac{4.0+4.5}{4}\left[\frac{20+28}{2}(4.0-4.5)+0.27[1(4)+1(4.5)]+[28+1(4.5)]4.5-[20+1(4)]4\right]$$

$$= \underline{\underline{86.2}}$$

$$P_{hf} = \frac{76.5(0.01+0.01)}{4}\left[[20+1(4)]4 + [28+1(4.5)]4.5\right]$$

$$= \underline{\underline{92.7}}$$

By Equation 8.23:

$$313.9+1635.2 = 181.3+1294.0+390.2\cos 6°+78.8+86.2-92.7$$

$$1949.1 = \underline{\underline{1935.7}} \qquad \text{(close enough)}$$

The momentum balance at this point is close enough to cease further iterations. Therefore, the hydraulic conditions at the end of the junction are as follows:

Q = 2775 cfs

b_3 = 28.0 ft

Y_3 = 4.5 ft

Z = 1:1

F_3 = 1.7

V_3 = 19.0 fps

A_3 = 146.3 ft^2

Additional bank-protection height will be needed to accommodate this depth. A step-backwater computation may be used to compute the distance from the end of the junction to the point at which normal depth occurs.

10.6.9 Clogging

The following guidelines should be followed to provide an appropriate factor of safety against clogging at pavement inlets:

GRATES AND SLOTTED DRAINS:

1. Sump Conditions:

   a. Orifice Flow: required area = 2.0 x calculated area.

   b. Weir Flow: required perimeter = 2.0 x calculated perimeter.

2. Continuous-grade conditions:

   a. Required length of opening = 2.0 x calculated length.

CURB INLETS:

1. Sump Conditions:

   a. Required length of opening = 1.50 x calculated length.

2. Continuous-grade conditions:

   a. Required length of opening = 1.25 x calculated length.

COMBINATION GRATE AND CURB INLET:

1. Sump Conditions:

   a. Orifice Flow: required area = 2.0 x calculated area for grate; required length = 1.25 x calculated length for curb inlet.

   b. Weir flow: required perimeter = 1.0 x calculated perimeter for grate; required length = 1.25 x calculated length for curb inlet.

2. Continuous-grade conditions:

   a. Required length of opening = 1.0 x calculated length for grate; required length = 1.25 x calculated length for curb inlet.

ADOT STANDARD TYPE-3 CATCH BASINS:

1. Continuous-Grade Conditions:

   a. Required curb-inlet length upstream from catch basin = 1.25 x calculated length.

   b. Required length of grate = 1.0 x calculated length.

These general guidelines should be used unless more-detailed information about clogging for a specific grate type is available. A publication by the American Society of Civil Engineers and Water Pollution Control Federation (1987) gives relative rankings for debris-handling efficiencies of several types of grates. Figure 10.5 can also be used to obtain an estimate of the ability of a grate to handle debris. Grates that are longer than necessary to intercept 100 percent of frontal flow will have greater debris-handling efficiencies than will shorter grates.

10.7 Inlet Design Procedure

Inlet Locations:

1. Using the plan-and-profile information developed for the proposed roadway, locate all points where 100-percent interception of runoff will be required. These will be located at sumps, street intersections, and at other locations where it is felt that anything less than 100-percent interception would be unacceptably hazardous.

2. Choose a proposed street-and-gutter cross section. The maximum allowable cross-slope for a street is two percent. Depressed concrete gutters with a width of twenty-one inches and a cross-slope of 0.048 may be used to increase gutter capacity. Using the proposed cross section and slopes, determine the maximum discharge that the street will carry according to the design limitations.

3. Locate drainage area (D.A.) concentration points and determine discharges for all offsite runoff affecting the project. Offsite inlets will be needed for all offsite drainage exceeding the design capacity of the street.

4. The remaining drainage area should consist of the street itself, and possibly some offsite sheet flow. The watershed should be long, and more or less of uniform width. Using (1) an assumed time of concentration of five minutes; (2) the maximum discharge capacity computed in Step Two; and (3) an appropriate runoff coefficient, apply the City of Tucson hydrology method in order to determine the area of watershed required to produce the maximum allowable street discharge. When this area is divided by the width of the watershed, it will give the length of the watershed from its approximate upstream end to the first storm-drain inlet. Check the watershed hydrology to ensure that the assumed five-minute time of concentration is correct.

To determine total gutter flow, the cross section is divided into segments of equal width, and the discharge for each segment is computed by Manning's equation. The parabola can be approximated very closely by two-foot-wide segments. The total discharge is the sum of the discharges in all segments. This procedure is illustrated by Example 10.13.1, found at the end of this chapter.

Some streets within the City of Tucson have inverted crowns (i.e., the lowest point is at the center of the street, instead of at the curb). Discharge for this type of street cross section can be estimated using the following procedures.

For a parabolic cross section, use Manning's equation, along with the following relationships:

Area (A) = $2/3TY$, in square feet; **(10.6)**

Wetted perimeter (P) = $T + 8/3\left[\frac{Y^2}{T}\right]$, in feet; **(10.7)**

Top Width (T) = $3/2\left[\frac{A}{Y}\right]$, in feet; and, **(10.8)**

Hydraulic Depth (Y_h) = $2/3\ Y$, in feet. **(10.9)**

Where:
Y = Maximum Depth, in feet.

However, it should be noted that, within the City of Tucson, streets with inverted crowns are normally built using a triangular cross section. For flow in a triangular inverted-crown section, use either Equation 10.3 or the nomograph shown in Figure 10.1.

10.6 Pavement Inlets

The capability of pavement inlets to quickly remove water from the street and into a storm drain depends upon their inlet geometry and upon the flow characteristics in the street and gutter. Pavement inlets are normally divided into the following three general types, with each having many variations:

1. Grate inlets: These inlets consist either of an opening in the gutter, covered by one or more grates, or an opening which spans the entire width of pavement (i.e., a "street grate").

2. Curb inlets: These inlets consist of a vertical opening in the curb, through which the gutter flow passes.

3. Combination inlets: These inlets consist of a curb inlet and a grate inlet acting as a single unit.

Grate inlets are most effective where clogging due to debris is not a problem. Excluding the effect of debris, the inlet capacity of grates in a sag condition depends mainly upon the open area of the grate and upon the depth of ponding. Capacity of grate inlets on a continuous grade depends primarily upon the discharge flowing directly over the grate, and upon the length and type of grate.

Grate inlets become more effective in relation to curb inlets as the grade of the roadway increases. *On grades of over three percent, grate inlets should be used instead of curb inlets.* Grates are also useful where cross-slopes for depressed gutters at curb inlets are not desirable, from a traffic standpoint, and at locations other than the edge of curb. For instance, grates are commonly used to collect flow at the middle of an inverted street.

The most efficient types of grates on a continuous grade are those which have all bars parallel to the direction of flow. Unfortunately, these grates typically are not safe for bicyclists; and therefore are *not permitted* to be used on City streets. However, there are many varieties of "bicycle-safe" grates which can be used on City streets (the interested reader should refer to a publication by the American Society of Civil Engineers and the Water Pollution Control Federation, 1987).

Curb inlets have few clogging problems; and they are most effective on relatively flat grades, where the depth of flow is sufficient for the inlet to perform efficiently. The interception capacity of curb inlets is largely dependent upon flow or ponding depth at the curb, and upon the length and height of the curb inlet. The flow-interception capacity is increased by a gutter depression at the curb inlet, or a depressed (composite) gutter to increase the proportion of the total flow adjacent to the curb. Top-slab supports can decrease the capacity of an inlet, if placed flush with the opening. Supports should be recessed several inches from the curb line.

One advantage to curb inlets is that they pose little threat to bicyclists. A disadvantage is that the openings are relatively wide, and could pose a danger to children. Therefore, it is recommended that curb inlets with a height of six inches or more be fitted with cross bars. Another disadvantage of curb inlets is that the depression adjacent to them could be hazardous to traffic at some locations.

Combination inlets can be very effective if the grate is placed at the downstream end of the structure--thereby allowing the curb inlet to collect the debris before it can clog the grate. The design capacity of these structures is the sum of the individual design capacities. If the curb inlet and grate are placed adjacent to each other, the total design capacity is only that of the grate alone.

Capacity charts for grate and curb inlets are widely available. However, due to the variety of configurations on the market, it is considered more useful here to merely present the basic relationships under which they operate.

10.6.1 Capacity of a Grate Inlet in a Sag

At low-water depths, a grate inlet in a sag operates as a weir, with a crest length equal to the outside perimeter of the grate along which the flow enters. Weir operation continues to a depth of about 0.4 foot above the top of grate, and the discharge intercepted by the grate is:

$$Q_i = 3.0\ P_g Y^{3/2} \tag{10.10}$$

Where:
- Q_i = Rate of discharge into the grate opening, in cubic feet per second;
- P_g = Perimeter of grate opening, in feet, disregarding bars and neglecting the side against the curb, if present; and,
- Y = Depth of water at the grate, in feet.

When the depth at the grate exceeds about 1.4 feet, the grate begins to operate as an orifice, and the discharge intercepted by the grate is:

$$Q_i = 5.35\ A Y^{1/2} \tag{10.11}$$

Where:
- Q_i = Rate of discharge into the grate opening, in cubic feet per second;
- A = Clear-opening area of the grate, in square feet; and,
- Y = Depth of ponded water above the top of grate, in feet.

For depths over the grate between about 0.4 feet and about 1.4 feet, the operation of the grate inlet is indefinite. In this case, the depth of flow should be computed by both equations. The equation which yields the higher of the two values for depth should then be used for design purposes.

If the grate is sloped such that the side away from the curb is considerably higher than the curb side, the side inflow and end inflow should be computed separately. Inflow over the end of a grate, when it is operating as a weir, should be computed from:

$$Q_e = 2/5 \left(\frac{CL}{Y_2 - Y_1} \right) \left(Y_2^{5/2} - Y_1^{5/2} \right) \tag{10.12}$$

Where:

Q_e = Rate of discharge over the end of the grate opening, in cubic feet per second;
Y_1 = Depth of flow at the shallow side of the grate, in feet;
Y_2 = Depth of flow at the deep side of the grate, in feet;
L = Distance from Y_1 to Y_2, in feet; and,
C = Weir coefficient = 3.0.

Total interception of the flow is then computed by summing the flows calculated at each end of the grate opening, using Equation 10.12, with the flow calculated on each side of the grate opening, using Equation 10.10.

When a sloped grate is operating under conditions of orifice flow, the following equation should be used to compute its interception capacity:

$$Q_i = 3.60\left(\frac{A}{Y_2 - Y_1}\right)\left(Y_2^{3/2} - Y_1^{3/2}\right) \tag{10.13}$$

Where all terms are as previously defined within Equation 10.11 and Equation 10.12.

10.6.2 Capacity of a Curb Inlet in a Sag

A curb inlet in a sag operates as a weir to depths up to the height of the curb inlet, and as an orifice at depths greater than 1.4 times the opening height. Between those depths, flow is in a transition stage.

The equation for computing the interception capacity of a curb inlet *without a depression* which operates as a weir is:

$$Q_i = 2.3\ LY_i^{3/2} \tag{10.14}$$

Where:

L = Length of curb inlet, in feet; and,
Y_i = Depth at lip of curb inlet, in feet (i.e., $Y_i = TS_x$).

The equation for computing the interception capacity of a *depressed* curb inlet which operates as a weir is:

$$Q_i = 2.3\ (L + 1.8W)Y_i^{3/2} \tag{10.15}$$

Where:

W = Lateral width of depression, in feet; and all other terms are as previously described.

Equation 10.15 is applicable to depths at the curb which are approximately equal to the height of the opening, plus the depth of the depression.

Curb inlets operate as orifices at depths greater than $1.4(h)$ (see Figure 10.3). The equation for interception capacity is then:

$$Q_i = 5.35A(Y_i - h/2)^{1/2} \qquad \textbf{(10.16a)}$$

or

$$Q_i = 5.35\ AY_o^{1/2} \qquad \textbf{(10.16b)}$$

Where:

Y_o = Effective head on the center of the orifice throat, in feet;
A = Clear area of opening, in feet;
Y_i = Depth at lip of curb inlet, in feet;
h = Height of curb-inlet orifice, in feet; and,
L = Length of curb inlet, in feet.

Figure 10.3 gives the relationship between the variables for horizontal-throat, inclined-throat, and vertical-throat inlets.

Curb-inlet capacity in the transition stage, when ponding depth is 1.0 to 1.4 times the opening height, should be computed using both the weir equation and the orifice equation. The equation which yields the lesser discharge at equal head should then be used for design purposes.

10.6.3 Capacity of a Combination Inlet in a Sag

When weir-flow applies, the interception capacity of a combination inlet in a sag, consisting of a grate and a curb inlet, is essentially equal to the capacity of the grate only, unless the grate becomes clogged. In orifice flow, the capacity is equal to the capacity of the grate, plus the capacity of the curb inlet.

10.6.4 Capacity of a Slotted Inlet in a Sag

A slotted inlet in a sag normally operates as a weir to depths of about 0.2 feet. At depths greater than about 0.4 feet, it performs as an orifice. Between these depths, the more conservative of the two equations (i.e., the one which predicts the greatest depth) should be used for design purposes. The interception capacity, Q_i, of a slotted inlet operating as an orifice should be computed from:

$$Q_i = 6.42\ LWY^{1/2} \qquad \textbf{(10.17)}$$

$Q = 5.35 h L Y_o^{1/2}$

L = Length of Opening

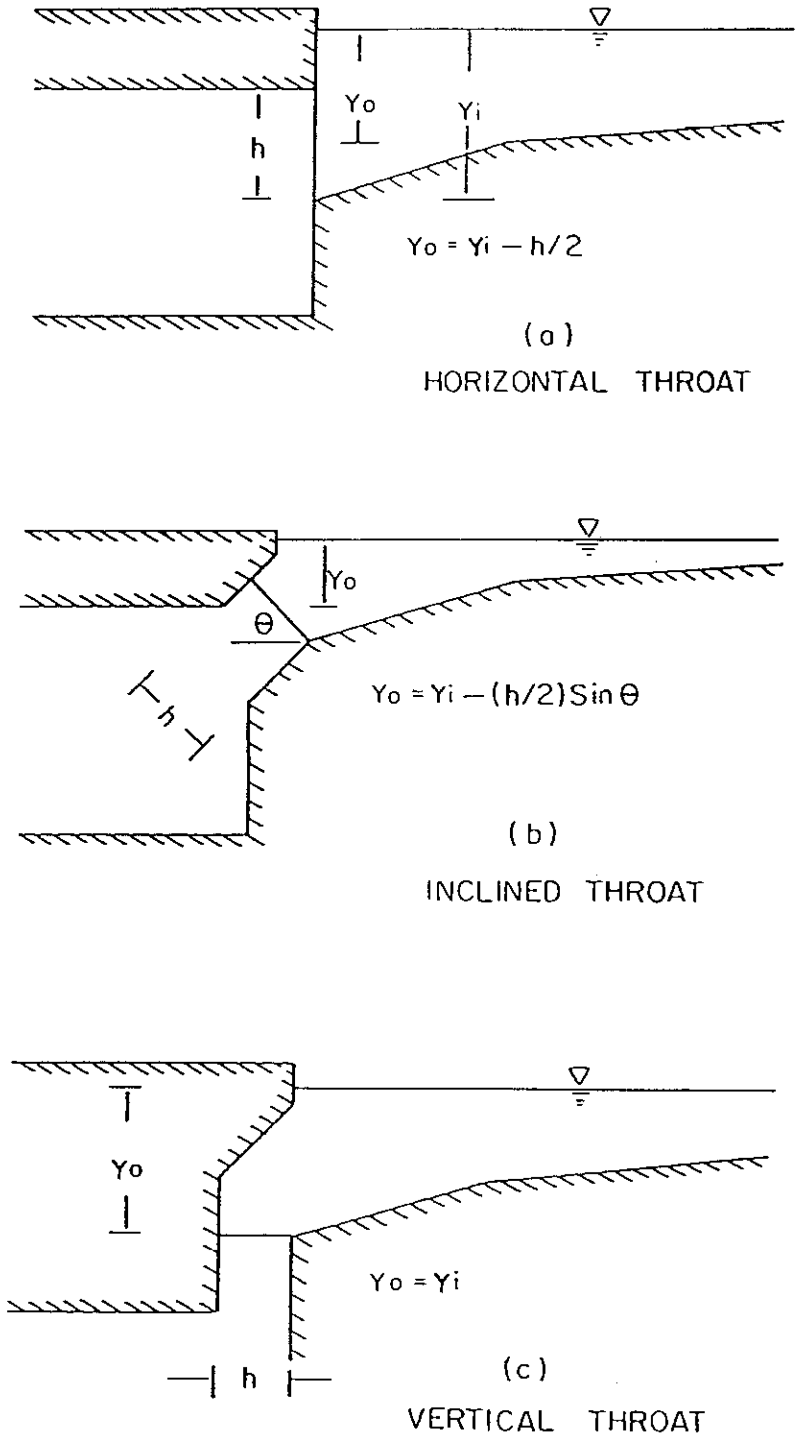


FIGURE 10.3
CURB-OPENING INLETS

Where:

L = Length of slot, in feet;
W = Width of slot, in feet; and,
Y = Depth of water at slot, in feet.

10.6.5 Capacity of a Grate Inlet on a Continuous Grade

A grate inlet on a continuous grade will intercept all of the frontal flow passing over the grate, unless the grate becomes clogged or splash-over occurs. Splash-over will occur, and only a portion of the frontal flow will be intercepted, if the velocity is high or the grate is short. Normally, a small part of the flow along the side of the grate will also be intercepted. Therefore, the total capacity of a grate is the sum of the frontal flow and the side flow, minus the splash-over flow.

The amount of frontal flow, Q_f, should be computed with the following equation:

$$\frac{Q_f}{Q_T} = E_o = 1 - (1 - W/T)^{8/3}$$

Where:

Q_f = Frontal flow at width W, in cubic feet per second;
Q_T = Total gutter flow, in cubic feet per second;
W = Width of grate, in feet;
T = Total spread of water at the gutter, in feet; and,
E_o = Ratio of frontal flow to total gutter flow.

Figure 10.4 provides a graphical solution of the frontal-flow equation.

The ratio, R_f, of frontal flow intercepted, Q_{fi}, to total frontal flow, Q_f, is expressed by:

$$\frac{Q_{fi}}{Q_f} = R_f = 1 - 0.09\,(V - V_o) \qquad \textbf{(10.19)}$$

Where:

V = Velocity of flow in the gutter, in feet per second; and,
V_o = Gutter velocity at which splash-over first occurs, in feet per second.

V_o is different for different grates, and must be determined experimentally. Figure 10.5 gives splash-over velocities for several common grate types and sizes described in a publication by the American Society of Civil Engineers and Water Pollution Control Federation (1987). Figure 10.5 also provides a graphical solution to the ratio of frontal flow captured to total frontal flow.

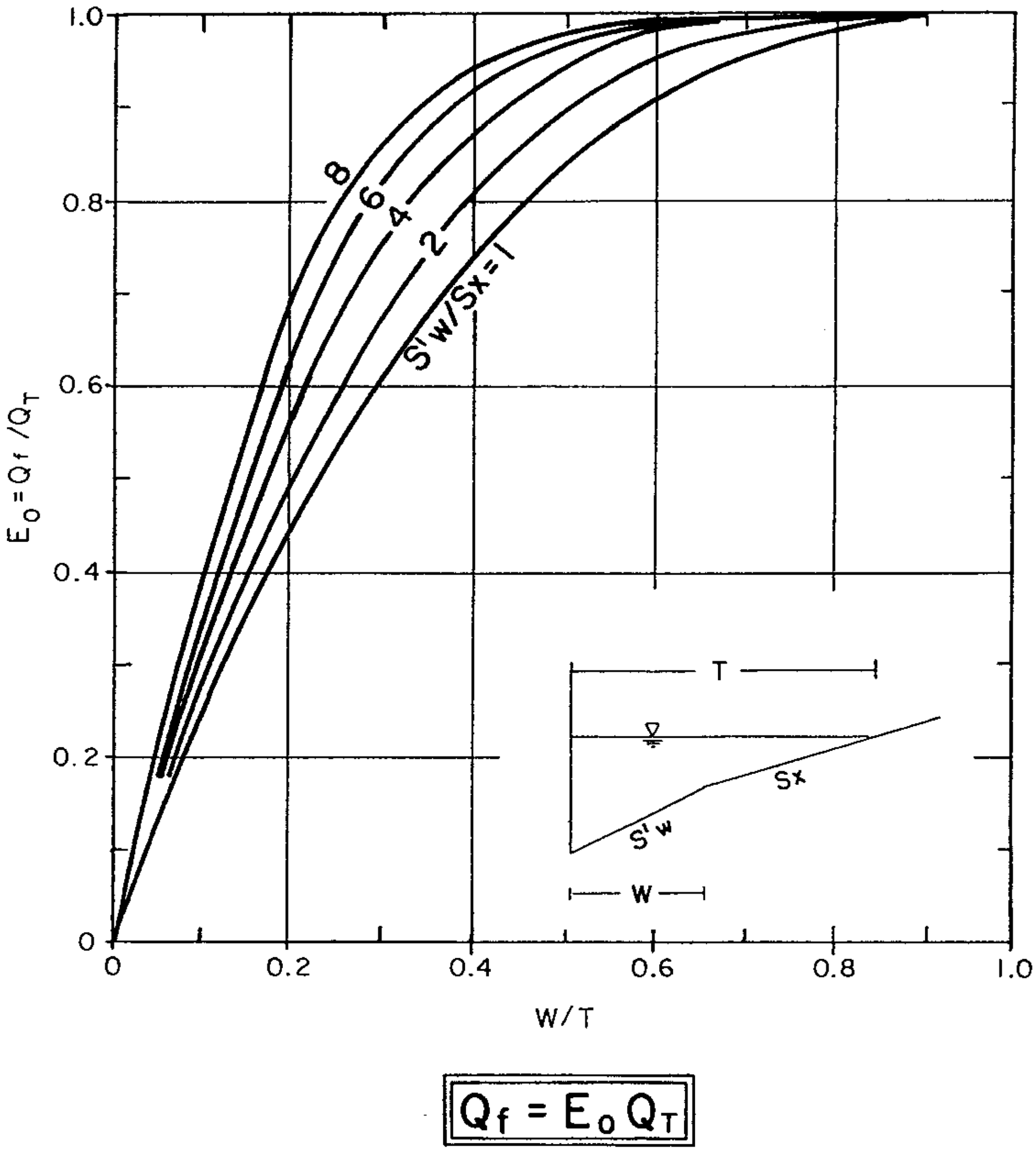
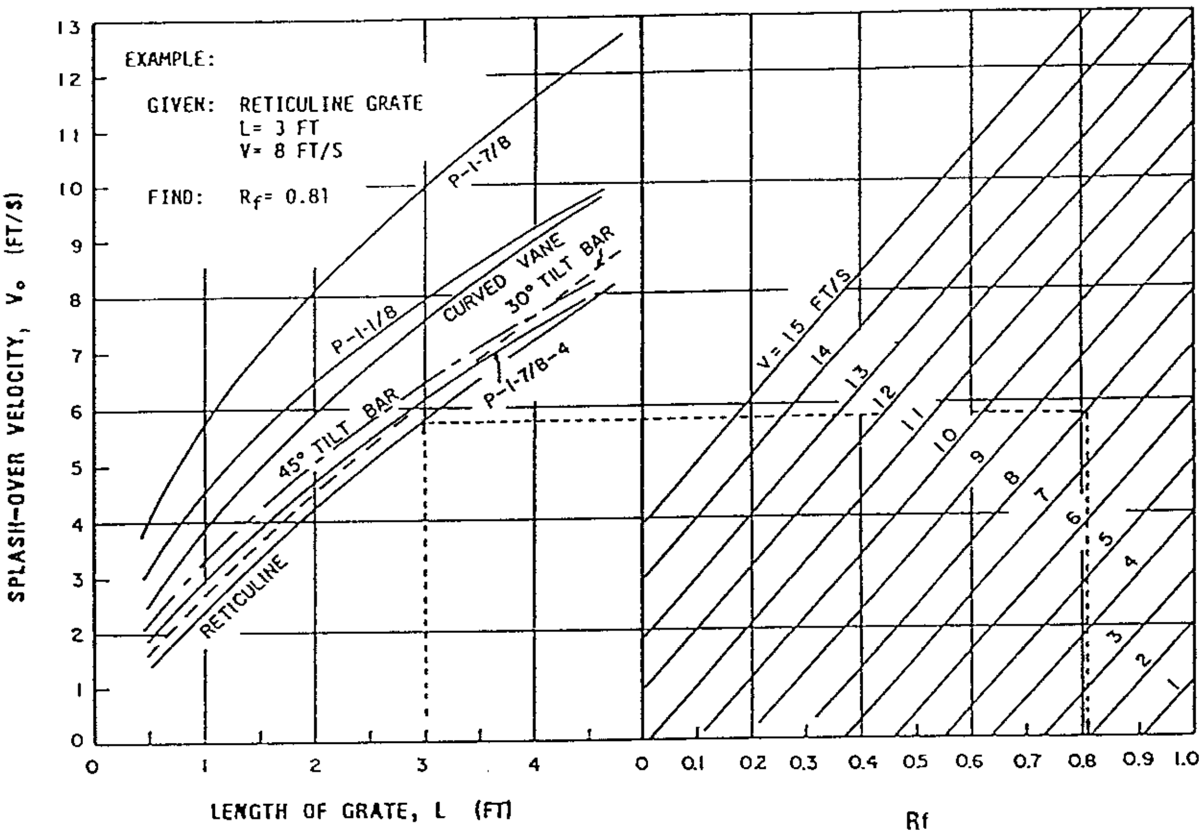


FIGURE 10.4
RATIO OF FRONTAL FLOW TO TOTAL GUTTER FLOW



$$\boxed{Q_{fi} = R_f \, Q_f}$$

FIGURE 10.5
FRONTAL-FLOW INTERCEPTION EFFICIENCY FOR GRATE INLETS

The amount of side flow, Q_s, is equal to the total flow minus the frontal flow (i.e., $Q_s = Q_T - Q_f$).

The ratio, R_{sf}, of side flow intercepted, Q_{si}, to total side flow, Q_s, is given by:

$$\frac{Q_{si}}{Q_s} = R_{sf} = \left[1 + \frac{0.15\, V^{1.8}}{S_x\, L^{2.3}} \right]^{-1.0} \tag{10.20}$$

Where:
- L = Length of the grate, in feet, and the other terms are as previously defined.

Note the negative exponent in this equation. Figure 10.6 provides a graphical solution to this equation.

The total interception capacity (Q_i) of a grate inlet on a continuous grade is therefore equal to:

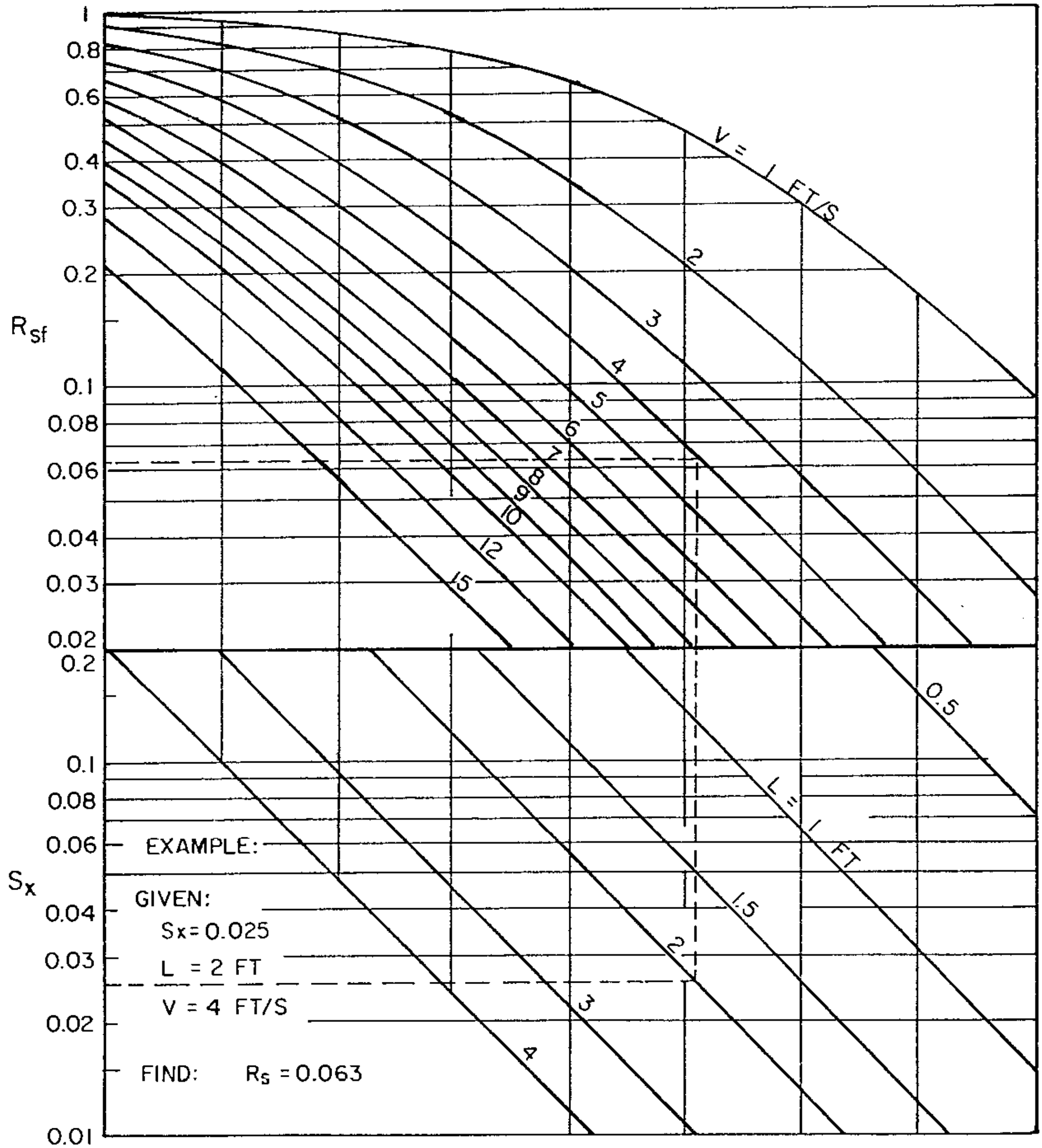
$$Q_i = R_f Q_f + R_{sf} Q_s \tag{10.21}$$

10.6.6 Capacity of a Curb Inlet on a Continuous Grade

The length of a curb inlet required for total interception of gutter flow on a pavement section with a straight cross-slope (i.e., no gutter depression) is expressed by:

$$L_t = 0.6 \left[Q_T^{0.42} S_o^{\,0.3} \right] \left[\frac{1}{n\, S_x} \right]^{0.6} \tag{10.22}$$

Where:
- L_t = Curb-inlet length required to intercept 100 percent of the gutter flow, in feet;
- S_x = Pavement cross-slope, in feet per foot;
- S_o = Longitudinal slope of gutter, in feet per foot; and,
- n = Manning's roughness coefficient.



$$\boxed{Q_{si} = R_{sf} Q_s}$$

FIGURE 10.6
SIDE-FLOW INTERCEPTION EFFICIENCY FOR GRATE INLETS

The efficiency of curb inlets shorter than the length required for total interception is expressed by:

$$E_i = 1 - (1 - L_i/L_t)^{1.8} \tag{10.23}$$

Where:

E_i = Ratio of discharge intercepted by the curb inlet to total discharge (i.e., the "efficiency" of the curb inlet);
L_i = Curb-inlet length, in feet; and,
L_t = As defined in Equation 10.22

Figure 10.7 is a nomograph for the solution of Equation 10.22, and Figure 10.8 provides a solution of Equation 10.23.

The length of inlet required for total interception by depressed curb inlets, or curb inlets in depressed gutter sections, can be found by the use of an equivalent cross slope, S_e, in place of S_x in Equation 10.22, as determined by the following equation:

$$S_e = S_x + S'_w E_o \tag{10.24}$$

Where:

$S'_w = \dfrac{a}{12W}$ = Cross-slope of the gutter, measured from the cross-slope of pavement, S_x, in feet per foot.

And where:

a = Gutter depression, in inches, at the curb inlet (measured as the vertical distance between the low point of the gutter and the point where the cross slope of the pavement intersects the curb. For a standard twenty-one-inch gutter width, with a one-inch drop from one side to the other and a two-percent street cross-slope, "a" is equal to six-tenths of an inch);
W = Width of depressed gutter, in feet; and,
E_o = Ratio of flow in the depressed section to total gutter flow.

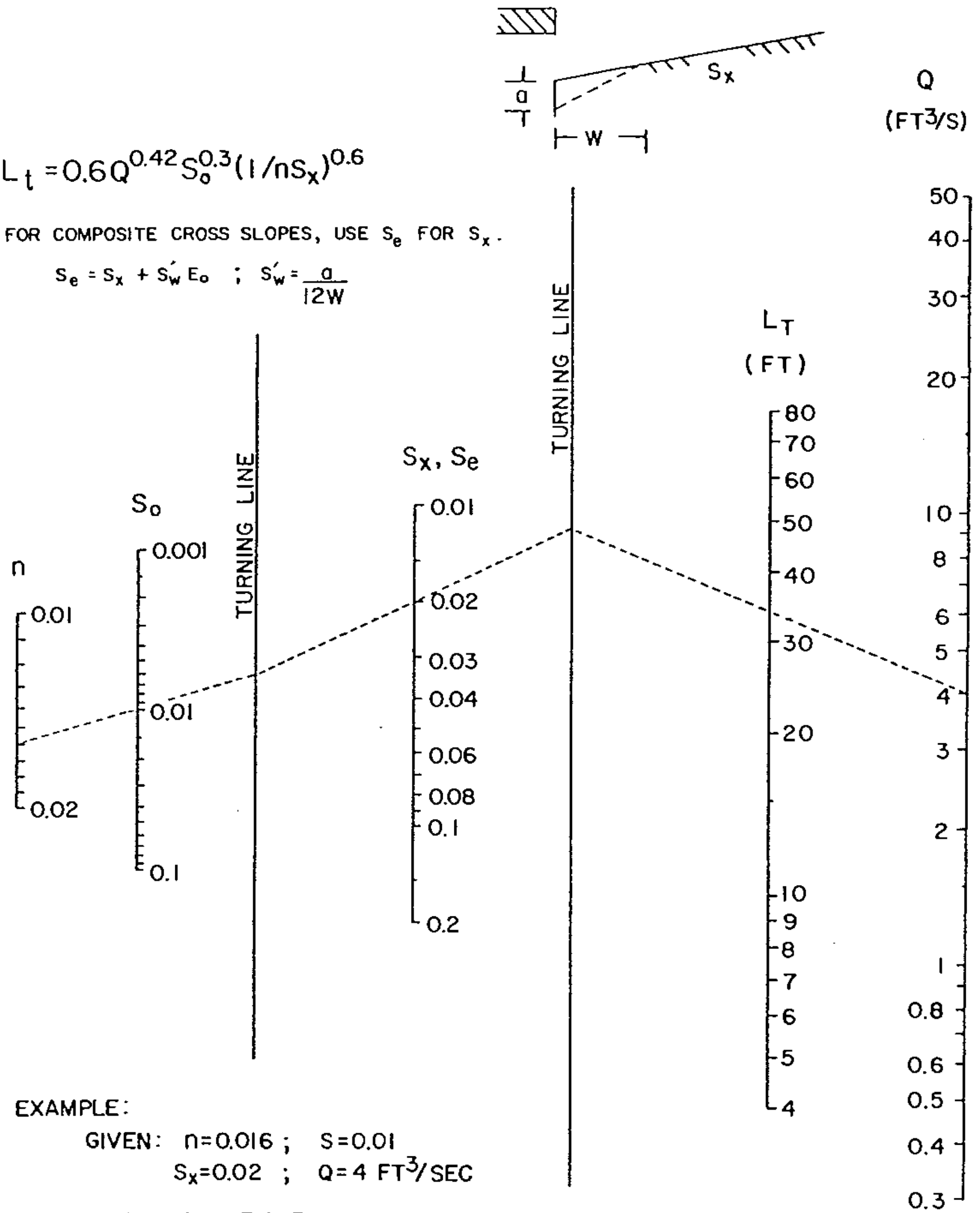
NOTE: E_o is the same ratio as that used to compute the frontal flow interception of a grate inlet.

Equations 10.22 and 10.23 can be combined to directly compute the length of the curb inlet required to intercept a certain percentage of the total discharge. This expression is:

$$L_t = 0.6 Q^{0.42} S_o^{0.3} (1/nS_x)^{0.6}$$

FOR COMPOSITE CROSS SLOPES, USE S_e FOR S_x.

$$S_e = S_x + S'_w E_o \quad ; \quad S'_w = \frac{a}{12W}$$



EXAMPLE:

GIVEN: $n=0.016$; $S=0.01$

$S_x=0.02$; $Q=4$ FT^3/SEC

FINO: $L_t = 34$ FT

FIGURE 10.7

INLET LENGTH FOR TOTAL INTERCEPTION BY CURB OPENINGS AND SLOTTED DRAINS

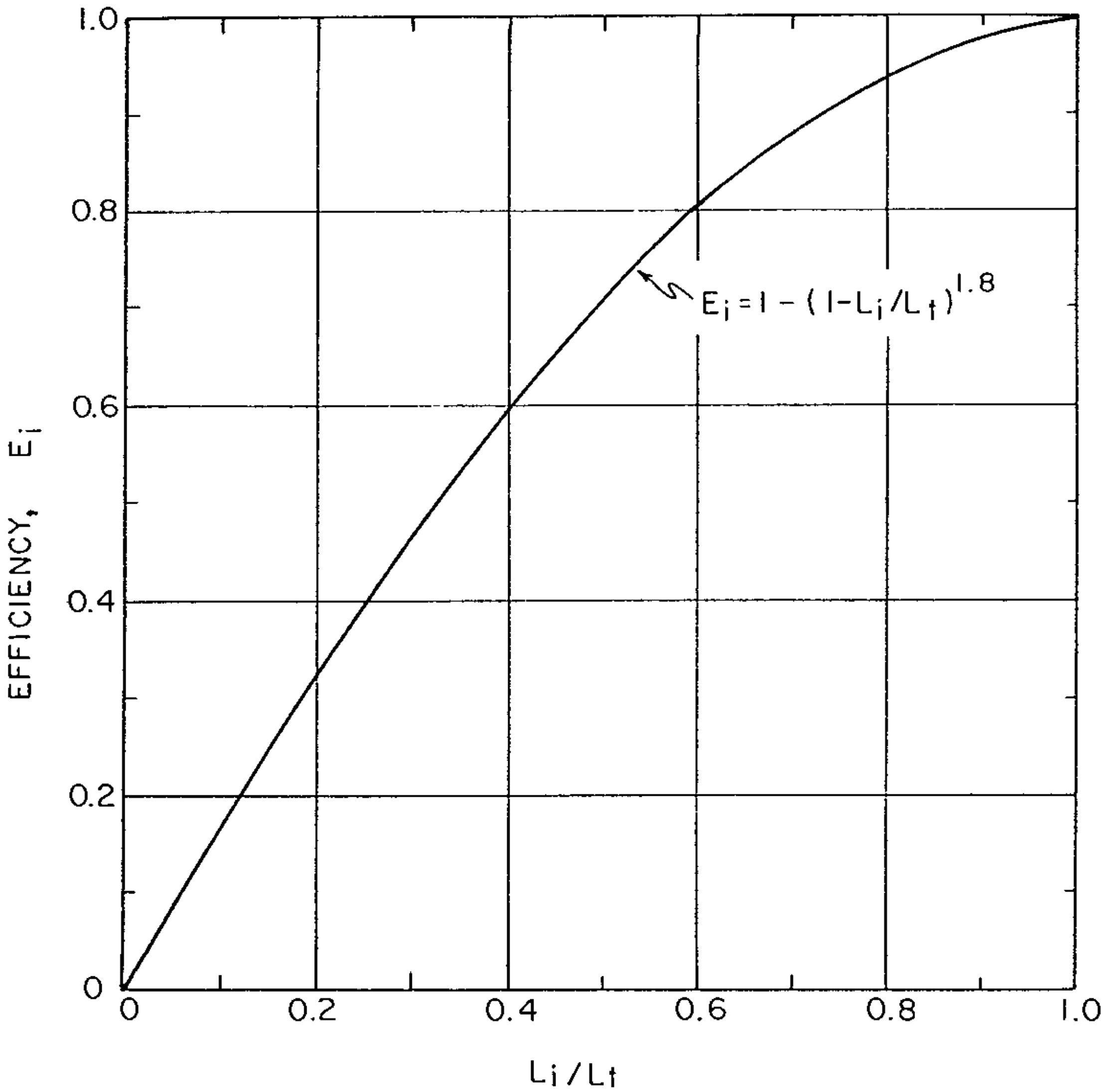


FIGURE 10.8

INLET INTERCEPTION EFFICIENCY FOR CURB OPENINGS AND SLOTTED DRAINS

$$L_i = 0.6 \left(\frac{Q^{0.42}\, S_o^{0.3}}{n^{0.6}\, S_x^{\;0.6}} \right) \left(1 - [1-E_i]^{0.56} \right) \tag{10.25}$$

Where all terms are as previously defined.

As with Equation 10.22, the S_x term is replaced by an equivalent cross slope, S_e, for a compound gutter section (see Figure 10.9). The equivalent cross slope can then be computed by combining Equations 10.4 and 10.24 to form the expression:

$$S_e = S_x + 0.0467 \left(\frac{a\, S_o^{1/2}}{Q\, n} \right) \left(\frac{Y_{cf}^{\;8/3} - Y_{gb}^{\;8/3}}{Y_{cf} - Y_{gb}} \right) \tag{10.26}$$

Where all terms are as previously defined.

NOTE: In Equation 10.24, the "Y_{cf}" and "Y_{gb}" terms represent the depth of flow at the curb face and the depth of flow at the gutter edge, in the gutter approaching the curb inlet, respectively.

As a rule of thumb, for preliminary sizing of curb-inlet lengths with compound gutter sections, it can be assumed that the curb-inlet capacity is 0.75 cfs/foot, if the pavement spread is over two lanes, and 0.40 cfs/foot, if the pavement spread is over only one lane. This assumes a two-inch depressed gutter at the curb inlet; a 75-percent inlet efficiency; and *no consideration for clogging due to debris.*

10.6.7 Capacity of a Combination Inlet on a Continuous Grade

A combination inlet on a continuous grade, where the curb inlet and grate are placed side-by-side, does not have much greater capacity than the grate alone. This type of inlet should not be used on a continuous grade. However, combination inlets with the curb inlet located upstream of the grate are useful, because the curb inlet intercepts normal debris loads which could otherwise clog the grate on a frequent basis. The capacity of these inlets is the sum of the capacities of the curb inlet and the grate. However, the discharge over the grate must be reduced by an amount equal to the interception capacity of the curb inlet.

10.6.8 Capacity of a Slotted Inlet on a Continuous Grade

The capacity of a slotted inlet on a continuous grade can be computed using the same formulas and charts that are used for computing curb-inlet capacities. The advantage of using slotted inlets is their versatility. They can be used on both curbed and uncurbed streets to collect a wide variety of flow patterns.

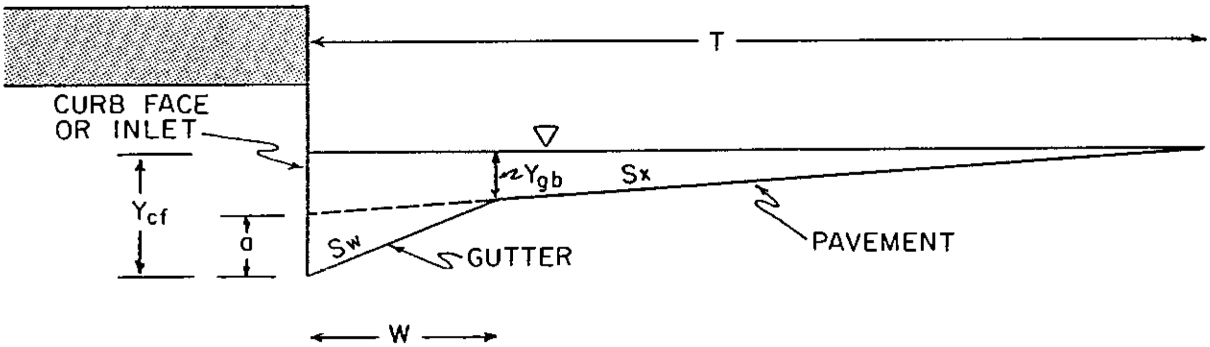


FIGURE 10.9
COMPOUND GUTTER SECTION

10.6.9 Clogging

The following guidelines should be followed to provide an appropriate factor of safety against clogging at pavement inlets:

GRATES AND SLOTTED DRAINS:

1. Sump Conditions:

   a. Orifice Flow: required area = 2.0 x calculated area.

   b. Weir Flow: required perimeter = 2.0 x calculated perimeter.

2. Continuous-grade conditions:

   a. Required length of opening = 2.0 x calculated length.

CURB INLETS:

1. Sump Conditions:

   a. Required length of opening = 1.50 x calculated length.

2. Continuous-grade conditions:

   a. Required length of opening = 1.25 x calculated length.

COMBINATION GRATE AND CURB INLET:

1. Sump Conditions:

   a. Orifice Flow: required area = 2.0 x calculated area for grate; required length = 1.25 x calculated length for curb inlet.

   b. Weir flow: required perimeter = 1.0 x calculated perimeter for grate; required length = 1.25 x calculated length for curb inlet.

2. Continuous-grade conditions:

   a. Required length of opening = 1.0 x calculated length for grate; required length = 1.25 x calculated length for curb inlet.

ADOT STANDARD TYPE-3 CATCH BASINS:

1. Continuous-Grade Conditions:

   a. Required curb-inlet length upstream from catch basin = 1.25 x calculated length.

   b. Required length of grate = 1.0 x calculated length.

These general guidelines should be used unless more-detailed information about clogging for a specific grate type is available. A publication by the American Society of Civil Engineers and Water Pollution Control Federation (1987) gives relative rankings for debris-handling efficiencies of several types of grates. Figure 10.5 can also be used to obtain an estimate of the ability of a grate to handle debris. Grates that are longer than necessary to intercept 100 percent of frontal flow will have greater debris-handling efficiencies than will shorter grates.

10.7 Inlet Design Procedure

Inlet Locations:

1. Using the plan-and-profile information developed for the proposed roadway, locate all points where 100-percent interception of runoff will be required. These will be located at sumps, street intersections, and at other locations where it is felt that anything less than 100-percent interception would be unacceptably hazardous.

2. Choose a proposed street-and-gutter cross section. The maximum allowable cross-slope for a street is two percent. Depressed concrete gutters with a width of twenty-one inches and a cross-slope of 0.048 may be used to increase gutter capacity. Using the proposed cross section and slopes, determine the maximum discharge that the street will carry according to the design limitations.

3. Locate drainage area (D.A.) concentration points and determine discharges for all offsite runoff affecting the project. Offsite inlets will be needed for all offsite drainage exceeding the design capacity of the street.

4. The remaining drainage area should consist of the street itself, and possibly some offsite sheet flow. The watershed should be long, and more or less of uniform width. Using (1) an assumed time of concentration of five minutes; (2) the maximum discharge capacity computed in Step Two; and (3) an appropriate runoff coefficient, apply the City of Tucson hydrology method in order to determine the area of watershed required to produce the maximum allowable street discharge. When this area is divided by the width of the watershed, it will give the length of the watershed from its approximate upstream end to the first storm-drain inlet. Check the watershed hydrology to ensure that the assumed five-minute time of concentration is correct.

For design discharges less than the 100-year flood, use appropriate ratios and procedures as outlined in Chapter IV of this Manual.

5. Choose a type of inlet that is appropriate for the location; and, using the appropriate procedures as described herein, develop a preliminary inlet design. Approximately 75 percent of the flow should be intercepted for maximum design efficiency.

6. Repeat Step Four to determine the distance to the next downstream inlet. Although not strictly accurate, the carry-over flow, Q_{co}, is added directly to the discharge produced in the intervening watershed between the two inlets. In reality, there should be a lag in peaks, and the amount to be accepted by a downstream inlet should be determined by adding hydrographs. However, this procedure would soon become very tedious. In view of the fact that the times of concentration are generally small, and that the inlets are spaced close together, direct adding of peaks is acceptable, and provides a measure of safety to the final design of the inlets.

7. Steps Five and Six are repeated, as necessary, until all drainage is accounted for within the system. At this time, needed revisions may become apparent for practical or economic reasons. Revisions should be made, and standard designs chosen, for all inlets. If the standard designs differ from the preliminary designs, the procedure should be repeated with the standard designs in order to ensure that the system works properly.

Work sheets for this procedure are presented in Figure 10.10, and an example is provided at the end of this chapter.

10.8 Storm-Drain Calculations

The two simplest methods of hydraulic analysis for use in the design of storm drains are (1) the "normal-flow method", and (2) the "pressure-flow method". The "normal-flow method" is much simpler to utilize, but it is often inaccurate. Its use often results in undersized pipes--especially if there are manholes, bends, junctions, and transitions that create energy (head) losses in the storm drain. On the other hand, the "normal-flow method" could also result in the design of storm drains that are larger and more expensive than necessary--particularly if there is sufficient head to create higher than normal flow velocities.

The pipe slope and the friction slope of storm drains designed for normal flow are assumed to be equal. It is therefore not necessary to calculate a hydraulic grade line for these storm drains if the soffits of connecting pipes of unequal size are set at the same elevation, and if the so-called "minor" head losses along the storm drain are minimal.

A hydraulic grade line for pressure flow will need to be computed whenever there is a high tailwater; or when it is desired to determine the effects which occur when a larger than design-frequency storm occurs; or whenever minor losses or pipe alignment may induce pressure flow; or when it is desired to check to see if a smaller pipe size

Sheet______ Of______

PAVEMENT-DRAINAGE WORKSHEET

LOCATION DATA:

Location: ____________________

Project No: ____________________

DESIGN DATA:

Frequency____________________

T_{all} =

Curb Height =

n = ______

RUNOFF CALCULATIONS:

From Station	To Station	L ft.	W ft.	D.A. acre,	C_w	T_c min.	I in/hr.	Q_{100} cfs	Q_{design} cfs	Q_{co} cfs	Q_T cfs	S_o ft/ft	S_x ft/ft	D_{ft}	T_{ft}

INLET CALCULATIONS

Station	Inlet Type	Y ft.	Q_i cfs	$Q_{co} = Q_T - Q_i$

Computed by:____________________ Checked by:____________________

Figure 10.10: Pavement Drainage Worksheet

could be used under conditions of pressure flow. It will generally be a requirement to compute the design hydraulic grade line for any proposed storm drain.

10.8.1 Normal-Depth Calculations

Normal-depth calculations are accomplished by using Manning's equation:

$$Q = \frac{1.486}{n} \left[\frac{A}{P} \right]^{2/3} S_o^{1/2} A \tag{10.27}$$

Where:

Q = Discharge, in cubic feet per second;
A = Flow area within the pipe, in feet;
n = Manning's roughness coefficient;
P = Wetted perimeter of flow, in feet; and,
S_o = Pipe slope, in feet per foot.

Figure 10.11 shows the relationship of these parameters for a circular conduit.

10.8.2 Pressure-Flow Calculations: Computation of Hydraulic Grade Line

Hydraulic grade-line computations for pressure flow are based on the Bernoulli equation. This equation is as follows:

$$\frac{V_1^2}{2g} + D_{hg1} + S_o\,L = \frac{V_2^2}{2g} + D_{hg2} + S_f L + H_m \tag{10.28}$$

Where:

H_m = "Minor" head losses, in feet, and all other terms are as defined by Figure 10.12.

The hydraulic grade line is computed by starting with the control tailwater elevation at the drain outlet, and subsequently performing a hydraulic grade-line calculation in the upstream direction. Friction and minor losses are computed for each segment of the storm drain. These energy losses are added to the total energy elevation at the downstream endpoint of the storm-drain segment in order to obtain the total energy elevation at the upstream endpoint of the segment. The hydraulic grade line is equal to the total energy grade line, minus velocity head at any point along the storm drain.

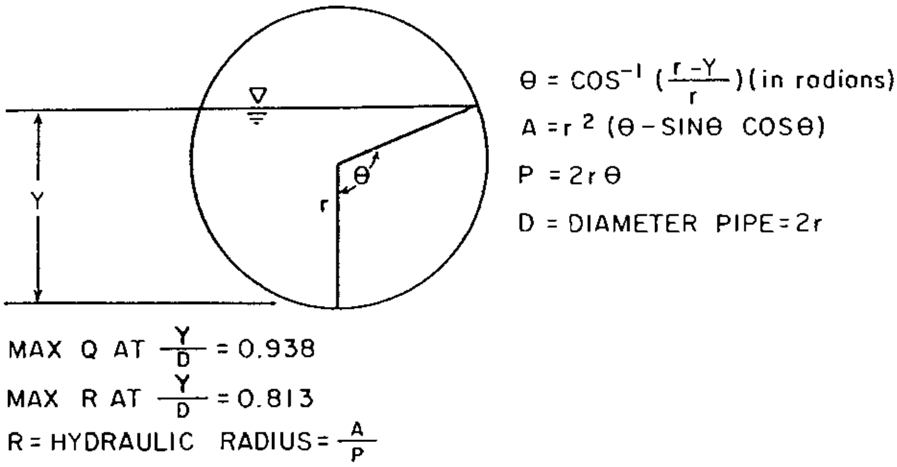


FIGURE 10.11
HYDRAULIC PARAMETERS OF A CIRCULAR CROSS SECTION

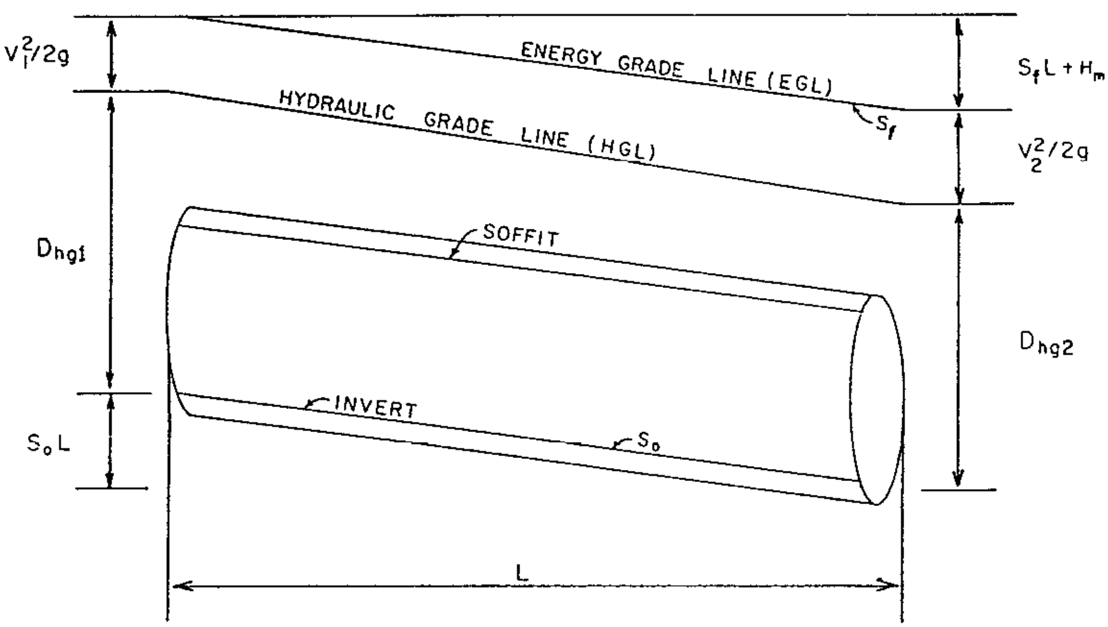


FIGURE 10.12
HEAD-LOSS DIAGRAM FOR PIPES

10.8.3 Friction Losses

Friction losses, h_f, are computed by Manning's equation for an assumed or given discharge. The form of Manning's equation used is:

$$S_f = \frac{29.2n^2}{R^{1.33}} \left[\frac{V^2}{2g} \right] \tag{10.29}$$

Where:

R = Hydraulic radius (i.e., the cross-sectional area of flow divided by the wetted perimeter of flow), in feet.

All other terms are as previously defined.

The friction loss for a storm-drain segment is then computed by the following equation:

$$h_f = S_f\, L = \text{Friction loss} \tag{10.30}$$

10.8.4 Minor Losses

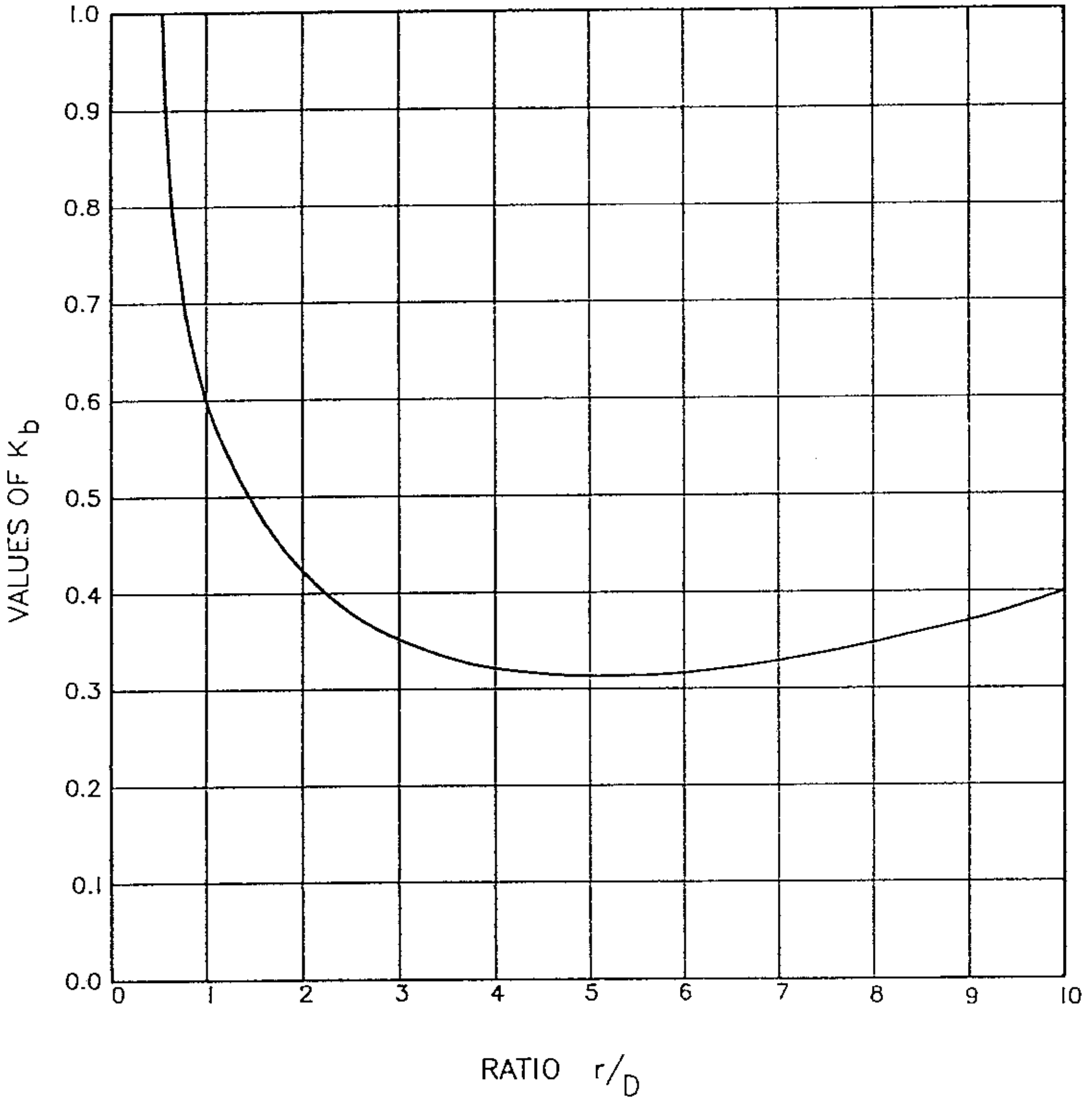
"Minor" losses in a storm drain are those that are associated with the energy necessary for the passage of water through areas such as junctions, manholes, and transitions. The total head loss is the sum of friction losses and minor losses. Minor losses, H_m, are normally represented as a factor K of velocity head:

$$H_m = K \left[\frac{V^2}{2g} \right] \tag{10.31}$$

The factor K varies widely, depending on the type of loss (e.g., bend, entrance, junction, manhole, etc.) and the configuration of the particular structure creating the head loss. A publication by the Denver Regional Council of Governments (1969) gives detailed information on minor losses, as do many hydraulics text books. It is important to note that these so-called "minor" losses can sometimes exceed friction losses within a storm-drain system, and therefore should always be evaluated at some point during the design process. Some of the more common minor losses encountered in storm-drain design are covered in the following sections.

10.8.5 Bend Losses

Head-loss coefficients for pipe bends with a deflection angle of 90 degrees, K_{b90}, can be determined from Figure 10.13.



r = radius of ℄ of bend; D = diameter of circular section or side of square section

FIGURE 10.13
HEAD–LOSS COEFFICIENT FOR 90° PIPE BEND

K_{b90} for 90-degree, square elbows, where there is no rounding of corners of the intersecting conduits, ranges from 1.25 to 1.50. In cases of bends where the deflection is less than 90 degrees, determine the head-loss coefficients for bends as follows:

K_b (For bend < 90°) =

$$\left\{ 1 - \left[\frac{90 - \text{deflection in degrees}}{90} \right]^2 \right\} K_{b90} \tag{10.32}$$

Bend head loss, h_b, is then:

$$h_b = K_b \left(\frac{V^2}{2g} \right) \tag{10.33}$$

10.8.6 Junction Losses

Junction losses, h_j, where the diameter of the main pipe does not change, shall be computed by:

$$h_j = \frac{V_2^2}{2g} - \frac{V_1^2}{2g} - \left(\frac{A_3 V_3^2}{A_2 g} \right) Cos\theta \tag{10.34}$$

Figure 10.14A illustrates this type of junction.

In the case where $D_1 \neq D_2$, junction loss shall be calculated by the Thompson equation:

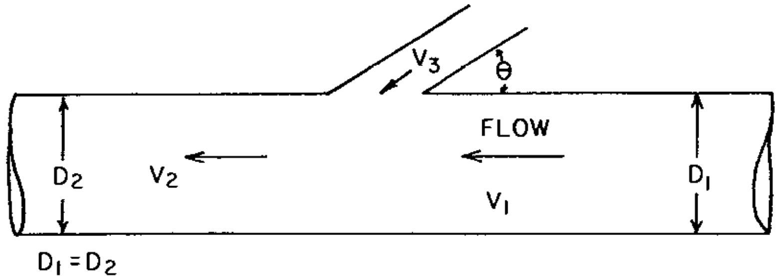
$$\Delta HG = \left(\frac{2}{A_1 + A_2} \right) \left(\frac{Q_2 V_2 - Q_1 V_1 - Q_3 V_3 Cos\theta}{g} \right) \tag{10.35}$$

Figure 10.14B illustrates this type of junction.

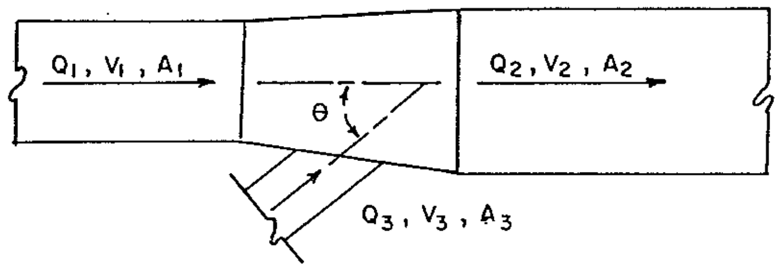
Where:

ΔHG = Difference in hydraulic gradient for the two ends of the junction, in feet;

A JUNCTION



B JUNCTION



C TRANSITION

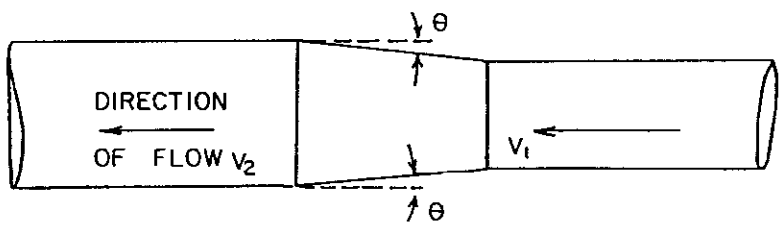


FIGURE 10.14
JUNCTION AND TRANSITION CONFIGURATIONS

A_1	=	Flow area of mainline pipe upstream of the junction, in square feet;
A_2	=	Area of mainline pipe downstream of the junction, in square feet;
A_3	=	Area of tributary pipe, in square feet;
Q_1	=	Discharge of mainline pipe upstream of the junction, in cubic feet per second;
Q_2	=	Discharge of mainline pipe downstream of the junction, in cubic feet per second;
Q_3	=	Discharge of tributary pipe, in cubic feet per second;
V_1	=	Flow velocity in mainline pipe upstream of the junction, in feet per second;
V_2	=	Flow velocity in mainline pipe downstream of the junction, in feet per second;
V_3	=	Flow velocity in tributary pipe, in feet per second; and,
θ	=	The angle formed by the junction between the tributary pipe and the mainline pipe, in degrees.

It is very important to note that ΔHG in this equation is the difference in hydraulic grade-line elevation, not the energy grade line. The total energy loss at the junction, h_j, is represented by:

$$h_j = \Delta HG + \frac{V_1^2}{2g} - \frac{V_2^2}{2g} \tag{10.36}$$

Junction loss should always be applied at the upstream side of the junction.

At junctions where there is more than one tributary inflow, the computation of head loss becomes more complicated. In most simple cases, Equation 10.35 can be used by subtracting $Q_n V_n \text{Cos}\theta$ terms in the numerator for each junction pipe. A publication by the Denver Regional Council of Governments (1969) gives junction losses for many detailed examples found in storm-drain design.

10.8.7 Transition Losses

Transition losses, h_t, for velocities which increase in the direction of flow (i.e., a contraction) are to be calculated using the following formula:

$$h_{tc} = 0.1\left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right) \tag{10.37}$$

Where velocities decrease in the direction of flow (i.e., an expansion), the formula to be used is:

$$h_{te} = 0.2\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) \tag{10.38}$$

See Figure 10.14C for a diagram which illustrates how to calculate transition losses using Equation 10.36.

10.8.8 Manhole Losses

For manholes with no change in pipe size or discharge, and where the flow is straight through, manhole losses, h_{mh}, shall be computed by:

$$h_{mh} = K_{mh}\left(\frac{V^2}{2g}\right) \tag{10.39}$$

Where $K_{mh} = 0.05$.

Head loss for manholes where flow changes direction, but where there is no change in discharge or pipe size, should be determined from Figure 10.15.

For manholes which contain junctions, or that have changes in pipe size, the head loss associated with these elements should be computed according to the guidelines for junction and transition losses, as presented within this chapter. This head loss should then be added to the head loss computed by use of either Equation 10.39 or Figure 10.15, in order to obtain the total head loss through these types of manholes.

10.8.9 Entrance and Outlet Losses

Entrance losses, h_e, are calculated by the following equation:

$$h_e = K_e\left(\frac{V^2}{2g}\right) \tag{10.40}$$

Values for K_e are given by:

TYPE OF INLET	VALUE OF K_e
Inward Projecting	0.78
Sharp Cornered	0.50
Bell Mouth (Beveled)	0.04

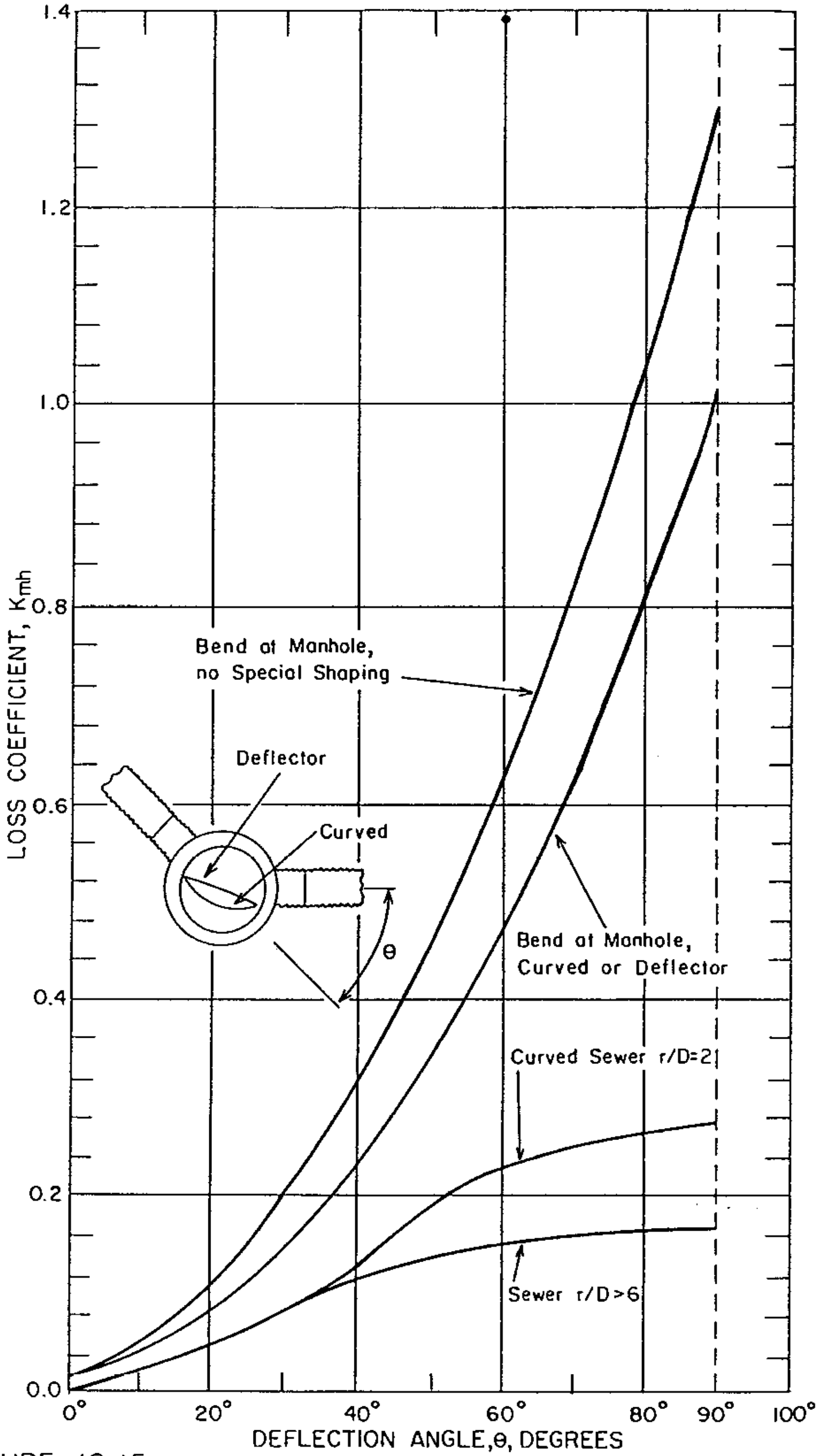


FIGURE 10.15
MANHOLE HEAD LOSS

CULVERT OUTLET VELOCITY	SUGGESTED OUTLET PROTECTION
Less than 4 fps	No protection required
More than 4 fps and less than 10 fps	Dumped rock riprap
More than 10 fps	Wire-tied riprap

If the velocity is greater than 10 fps, consider using a concrete energy dissipator, or increasing culvert size.

2. Structurally-designed downstream cut-off walls should be installed whenever the equilibrium channel slope is less than the existing channel slope. Refer to Chapter VI of this Manual for the sizing and spacing of cut-off walls.

3. Downstream embankment stabilization should be provided whenever the 100-year design flood overtops the roadway for a continuous period of time exceeding 10 minutes in duration (Pima County Department of Transportation and Flood Control District, 1984, P. VI-8).

11.5 Culvert vs. Bridge Crossings

Sedimentation at culvert crossings may be a problem when the culvert cannot transport all of the sediment being delivered by the approach channel. In general, pipe culverts will transport less sediment than box culverts, and smooth pipes (e.g., concrete) will transport more sediment than corrugated metal pipes. However, the most effective method of eliminating sedimentation problems is to utilize a bridge structure which minimizes changes to the hydraulics or geometry of the approach channel. Equation 11.9 is provided as an aid to the engineer in determining if a particular culvert crossing may experience sediment deposition either within the culvert or at its entrance.

$$\Re_s = \frac{Q_{ac}}{Q_p}\left(\frac{S_{ac}}{S_p}\right)^{1.66}\left(\frac{n_{ac}}{n_p}\right)^{-1.55}\left(\frac{R_{ac}}{R_p}\right)^{0.91} \tag{11.9}$$

Where:

$\Re_s$ = Sediment-transport ratio (channel to culvert);
Q_{ac} = Discharge in approach channel, in cubic feet per second;
Q_p = Total culvert discharge, in cubic feet per second;
S_{ac} = Longitudinal slope of approach channel, in feet/foot;
S_p = Longitudinal slope of culvert, in feet/foot;
n_{ac} = Manning's roughness coefficient for the approach channel;

DEFAULT TO CRITICAL SLOPE ONLY IF PIPE IS FLOWING UNDER PRESSURE FLOW - PER ZELLER'S ADDENDUM

n_p = Manning's roughness coefficient for the culvert;
R_{ac} = Hydraulic radius of flow in approach channel, in feet; and,
R_p = Hydraulic radius of flow within the culvert, in feet.

If the value $\Re_s$ in Equation 11.9 is less than 1.0, the culvert will most likely be able to transport the sediment being delivered by the approach channel. If the value of $\Re_s$ is greater than 1.0, sedimentation may occur, and an alternate culvert or a bridge structure should be considered. The value of S_p in Equation 11.9 should never exceed the critical slope of the culvert for the discharge involved. The culvert itself may be placed on a slope greater than critical, but critical slope should always be used in Equation 11.9 under such circumstances. Additionally, if tailwater exceeds the soffit of the culvert, then a hydraulic grade line should be calculated, and the friction slope of the culvert should be used in Equation 11.9.

11.6 At-Grade (Dip) Crossings

Crossings of watercourses which are designed to allow drainage to flow across roadways at-grade are commonly referred to as either *at-grade* or *dip* crossings. These "structures" are often used where strict all-weather-access criteria do not need to be met. Nevertheless, when flows pass over *at-grade* crossings, hazardous conditions may be created both during and immediately after such flows because of downstream erosion and/or sediment and debris buildup within the crossing itself.

In order to minimize these hazardous conditions during and immediately after a flow event, the *at-grade* crossing should be built with a minimum four-percent cross slope, unless horizontal and vertical controls for traffic safety dictate otherwise, in order to reduce the potential for sedimentation within the crossing. The cross-slope should be accomplished by providing the vertical rise on the upstream side of the crossing, with the downstream side meeting existing grade (Pima County Department of Transportation and Flood Control District, 1984). At a minimum, a two-foot-deep cutoff wall should be placed along the upstream side of the *at-grade* crossing in order to protect the pavement edge from general scour. In addition, an adequately deep cutoff wall (i.e., based upon criteria contained within this Manual, but in no case less than three feet in depth), should be placed along the downstream side of the pavement in order to prevent erosion damage, due to local scour and channel degradation, from occurring immediately downstream of the *at-grade* crossing.